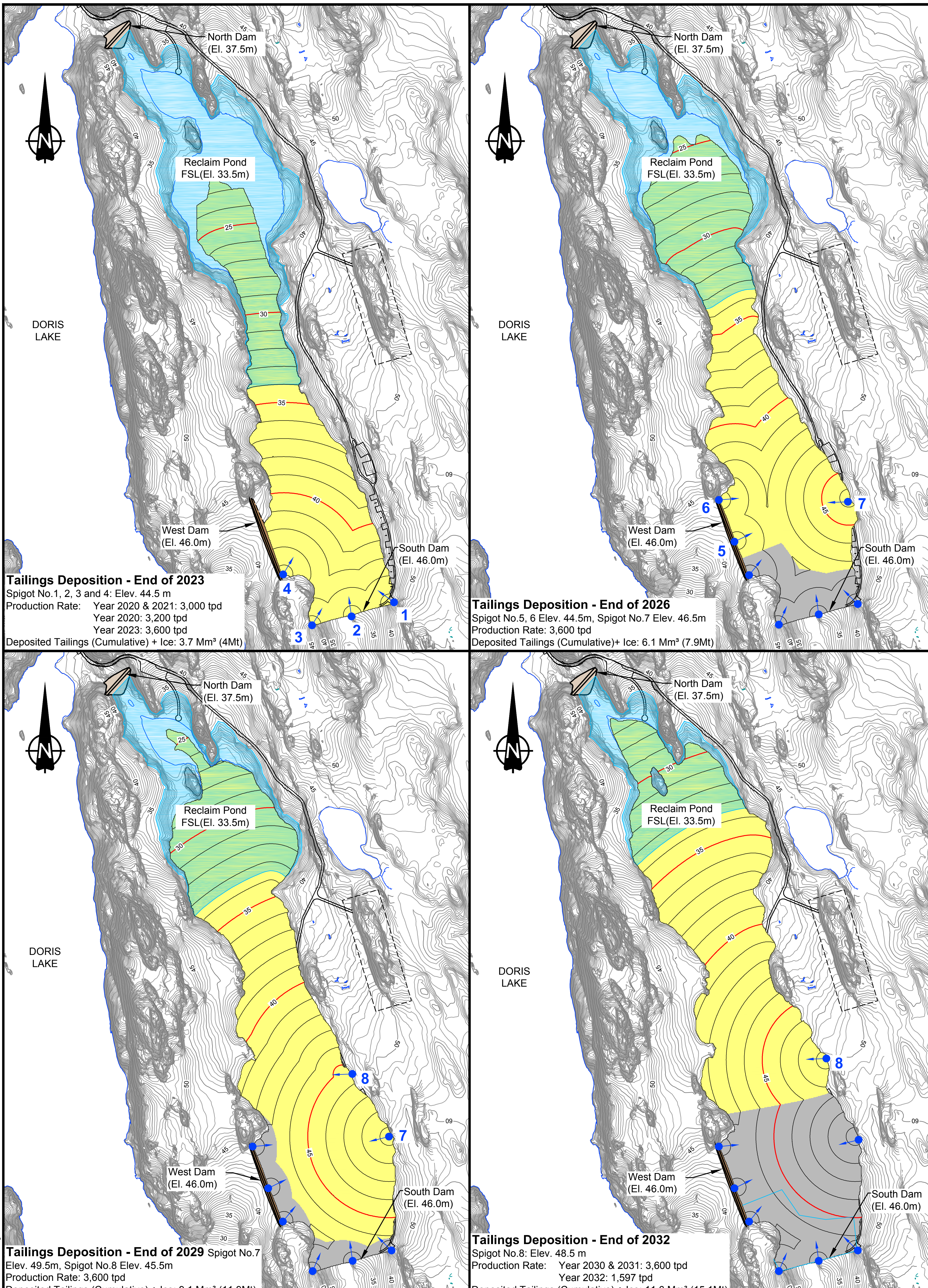

Tailings Deposition - End of 2023
Spigot No.1, 2, 3 and 4: Elev. 44.5 m
Production Rate: Year 2020 & 2021: 3,000 tpd
Year 2020: 3,200 tpd
Year 2023: 3,600 tpd
Deposited Tailings (Cumulative) + Ice: 3.7 Mm³ (4Mt)

Tailings Deposition - End of 2026
Spigot No.5, 6 Elev. 44.5m, Spigot No.7 Elev. 46.5m
Production Rate: 3,600 tpd
Deposited Tailings (Cumulative)+ Ice: 6.1 Mm³ (7.9Mt)

Tailings Deposition - End of 2029 Spigot No.7 Elev. 49.5m, Spigot No.8 Elev. 45.5m
Production Rate: 3,600 tpd
Deposited Tailings (Cumulative) + Ice: 9.1 Mm³ (11.9Mt)

Tailings Deposition - End of 2032
Spigot No.8: Elev. 48.5 m
Production Rate: Year 2030 & 2031: 3,600 tpd
Year 2032: 1,597 tpd
Deposited Tailings (Cumulative) + Ice: 11.6 Mm³ (15.1Mt)

General Notes:
1. Deposition durations are approximate and were based the Mine Plan average production rates provided by TMAC.
2. Assumed an average deposited tailings beach slope of 1.0%.
3. A deposited tailings dry density of 1.3t/m3 was used.
4. All tailings volumes presented include ice entrainment at 20% of production.
5. Dam and dike elevations shown were assumed constant throughout deposition.
6. Total storage requirement includes Madrid and Boston Tailings (Tailings 11.6Mm³ + Ice entrainment 2.3Mm³)

LEGEND

- Spigot Location
- Deposited Tailings
- Dam/Dike
- Previously Deposited Tailings
- Proposed Tailings Topography

0m 200 400 600 800

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SRK JOB NO.: 1CT022.004.600.010
FILE NAME: HB PH2 DEPOSITION FIGURE 9.dwg

TMAC RESOURCES

HOPE BAY PROJECT

PHASE 2 TAILINGS DEPOSITION PLAN

DEPOSITION OPTION 5
STAGING PLAN
(TOTAL TIA STORAGE 17.6 Mt)

DATE:	APPROVED:	FIGURE:
2016/10/27	TPP	09

P:\01_SITES\Hope Bay\!ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 9.dwg

Appendix B – Hope Bay Project: Phase 2 Doris Tailings Impoundment Area Seepage and Stability Analyses

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Memo

To:	John Roberts, PEng, Vice President Environment	**Client:**	TMAC Resources Inc.
From:	Sam Amiralaei, PEng	**Project No:**	1CT022.004.610
Reviewed:	Arcesio Lizcano, PhD Maritz Rykaart, PhD, PEng	**Date:**	December 8, 2016
Subject:	Hope Bay Project — Doris Tailings Impoundment Area Phase 2 Seepage and Stability Analyses		

1 Introduction

1.1 General

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization, plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

The existing Doris tailings impoundment area (TIA) will be expanded to accommodate the Project tailings, continuing with sub-aerial hydraulic deposition of a thickened slurry. Doris TIA is located approximately 1 km east of Doris Camp and accessed via the existing secondary road. To ensure environmental containment, the TIA would be impounded with three dams: North Dam, South Dam, and West Dam. The North Dam was constructed during the winter months of 2011 and 2012 (SRK, 2012), and will not be modified in any way. The South Dam will be raised 8 m to a new crest elevation of 46.0 m. The new West Dam will have a maximum height of 5 m.

The North Dam will continue to function as a water retaining dam, while the South and West dams will have tailings deposited against them, effectively functioning as tailings solids retaining structures. Over time, tailings freeze back is expected (SRK, 2016c). Tailings will be deposited sub-aerially from eight spigot points along the eastern perimeter of the TIA and from the crest of the South Dam creating a landscape that drains towards the North Dam at an average slope of about 1%.

At closure, once water quality discharge criteria are met within the Reclaim Pond, the North Dam would be breached, returning the pond water level to the pre-mining elevation of 28.3 m.

1.2 Objectives

This memo documents methods, assumptions, and results of the stability analyses completed for the South and West dams. Specifically for each structure, the following analysis were completed:

- Seepage analysis to determine the expected leakage rates, and to establish a phreatic level for use in a coupled stability analysis;
- Overall upstream slope stability and interface stability along the geosynthetic clay liner (GCL) contact zone, under static and pseudo-static conditions; and
- Overall downstream slope stability under static and pseudo-static conditions.

2 Design Criteria

2.1 Minimum Factors of Safety

A factor of safety (FOS) is defined as the ratio of the forces tending to resist failure (i.e. the material's shear strength) over the forces tending to cause failure (i.e. the shear stresses) along a given surface. The selection of a design FOS must consider the level of confidence in the factors that will control stability, i.e. material properties, analysis methods, and consequences of failure.

Design FOSs are generally defined through various industry best practice standards and guidelines, and for dams, including tailings dams, the most notable guideline is the Canadian Dam Association (CDA) Guidelines (CDA, 2014). Table 1 summarizes the recommended minimum design FOS in accordance with CDA (2014).

Table 1: Minimum Factors of Safety Used for Slope Stability Analysis

Loading Condition	Minimum Factor of Safety	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
Pseudo-static	1.0	Downstream
Post-earthquake	1.2	Downstream

2.2 Seismic Design Parameters

CDA (2014) provides recommended minimum seismic design criteria, based on the hazard classification assigned to the structure. Assuming a hazard classification of HIGH, the CDA (2014) specifies the design seismic event must be the 1:2,475-year event. For long-term scenarios, i.e. post-closure, the design seismic event must be increased to halfway between the 1:2,475 and 1:10,000-year event.

The Project is located in the lowest category seismic hazard zone of Canada in accordance with the 2015 National Building Code of Canada seismic hazard maps (NRCC, 2015). The seismic hazard is described by spectral-acceleration (Sa) values at different periods, as well as the peak ground acceleration (PGA) and peak ground velocity (PGV). Spectral acceleration is a measure of ground motion that takes into account the sustained shaking energy at a particular period; however, PGA is the parameter considered for foundation design.

These ground motions need to be adjusted for site specific ground type, prior to being used in design. SRK completed a site specific seismic assessment for determining horizontal and vertical seismic parameters to be used in pseudo static slope stability analysis modeling on the Project site (SRK, 2016a). This analysis determines the horizontal seismic coefficient by reducing the site-adjusted PGA based on slope height and allowable deformation. The method assumes an allowable deformation of 1 to 2 inches (25 to 51 mm) for a seismic FOS of 1.1. While a larger allowable deformation is unlikely to affect the stability of the facility, this criteria was thought to be appropriately conservative. The horizontal seismic coefficients for the South and West dams during the operational period was determined to be 0.021 g and 0.025 g, resulting from a 1:2,475 year return period earthquake. Post-closure, since the South Dam and West Dam will remain in perpetuity, the design seismic coefficient was 0.036 g and 0.043 g, respectively.

3 Analysis Method

3.1 Modeling Tools

Seepage and stability analyses were carried out using the commercial Seep/W and Slope/W software developed by GEO-SLOPE International Ltd. (GEO-SLOPE, 2012). Seepage values were generated using saturated/unsaturated steady-state limit equilibrium analysis, while stability analysis were evaluated using the Morgenstern-Price method for circular failure modes. Seepage values and stability analysis were coupled.

3.2 Modeling Method

3.2.1 Seepage Model

Porewater pressure boundary conditions for the stability analysis were applied based on the results of the seepage analysis at the TIA full supply level (FSL). The tailings were conservatively assumed to be fully thawed and saturated.

Seepage volumes were calculated by obtaining the unit seepage rate (per unit width of the dam) at the downstream toe of the dams, and multiplying this value by the total length of the dam. An average dam height was assumed in this calculation, where the average dam height was determined by averaging the height of the dam cross sections created at one meter intervals. Table 2 summarizes the dam dimensions used in this analysis.

Table 2: South Dam and West Dam Geometry used in Seepage Analysis

Location	Total Dam Longitudinal Cross Section Area (m^2)	Total Longitudinal Length (m)	Average Height (m)	Maximum Height (m)
South Dam	5,000	520	9.6	15
West Dam	1,740	470	3.7	5

3.2.2 Stability Model

Slope stability was assessed for both static and pseudo-static conditions. To provide confidence in the results, the models were analyzed using three modes of searching for the failure surface:

- Grid and radius;
- Specified entry and exit locations; and
- Fully specified failure surface.

The following loading conditions, as recommended by CDA (2014), were considered:

- Long-term – static drained (i.e. no excess pore pressure);
- During or at the end of construction – static undrained (with excess pore pressure due to rapid loading);
- During earthquake – pseudo-static undrained condition; and
- Post-earthquake – pseudo-static drained condition.

Since the South and West dams in actual fact have frozen foundations, are expected to remain frozen in the long term, and the area has a low seismic coefficient, post-earthquake deformation is not expected to be material. As such, changes in shear strength parameters in the dam fill and in the foundation soils are expected to be negligible. Therefore, a post-earthquake analysis was not completed.

Considering the loading conditions, the assessment objectives, model geometry and foundation conditions, the following analysis were completed:

- Stability of the upstream slope of the South and West dams respectively, along the interface between the GCL and the dam fill at the end of the dam construction, prior to discharge of any Phase 2 tailings. The following two foundation conditions were considered:

  Fully thawed – thawed foundation layer thickness of approximately 7 m; and
  Partially thawed - thawed foundation layer thickness of 1 m.

- Stability of the foundation and downstream slope of the dams, with tailings reaching FSL. Again, the fully thawed and partially thawed foundation conditions were considered.

Partially thawed foundation conditions are supported by thermal analysis (Figures 4 and 5) completed as part of the dam design (SRK, 2016c). Thaw will however be limited to zones near the toe of the dam, while the key trench and the thicker fill zones of the dam will remain frozen. Nonetheless, the partially thawed condition was applied to the full footprint of the dam for added conservatism. For the partially thawed conditions, the -2°C isotherm, as developed from thermal modelling (SRK, 2016c), was adopted to define the extent of thaw (Figure 3).

Fully thawed foundation conditions are not plausible under the expected long term field conditions, and are only used in the analysis as a theoretical absolute limit. Considering an unbounded timeframe, and uncontrolled constant climate change, it could be argued that the complete foundation might thaw (on a millennial scale). However, should that happen, the change would be so slow that pore water pressures would readily dissipate and therefore an assumption of drained conditions is reasonable.

3.3 Model Geometry

The South and West dams are frozen foundation dams with a GCL keyed into the permafrost overburden foundation. The dams will not have ponded water against them as tailings will be beached from the crest of the dams pushing the supernatant pond water, i.e. Reclaim Pond, away from the dams. To accommodate the Phase 2 tailings, the Phase 1 South Dam will be raised, and the West Dam will be a new structure.

The Phase 2 raise of the South Dam will be a downstream raise following the same geometry as the Phase 1 design. The final crest width will be 10 m, the upstream slope 4H:1V, and the downstream slope 2H:1V (Figure 1). The crest elevation is raised 8 m from 38.0 m to 46.0 m, and results in a maximum dam height of 15 m and total dam length of 520 m. The key trench will be about 4 m deep, have a base width of 4 m with 2H:1V, and 1H:1V upstream and downstream slopes respectively. The GCL previously installed as part of the Phase 1 South Dam will be extended vertically to reach an elevation of 45.0 m at a slope of 3H:1V (4H:1V in the raised portion).

The West Dam was designed with the same cross-section as the South Dam (Figure 2). This dam will be constructed in a single stage, and will be about 470 m long with a maximum height of 5 m.

For both the South and West dams, a single critical cross-section was analyzed. This critical section was conservatively assumed to be the zone where the foundation overburden soils was at its maximum thickness and the dam structure was at its maximum height. Figures 1 and 2 present the model cross-sections for the South and West dams respectively.

3.4 Material Properties

Sub-surface investigations downstream of the Phase 1 South Dam, along the footprint of the Phase 2 raise have not been carried out, and a single drill hole has been completed within the footprint of the West Dam. Material properties for the analysis was therefore based on the North Dam (SRK, 2007), and Phase 1 South Dam (SRK, 2015) designs, supplemented with the site wide geotechnical design properties (SRK, 2016b). Table 3 summarizes the properties used in the analysis.

Table 3: South and West Dam Foundation and Material Properties

Parameter		Marine Silt and Clay	Marine Silt	Tailings	Run of Quarry (Dam Fill)	GCL
Moist Unit Weight (kN/m^3)		18 (1)	18(1)	17.5 (2)	20 (1)	10 (4)
Undrained Shear Strength s_u (kPa)		13 (1)	13 (1)	-	-	-
Non-Frozen	Apparent Cohesion c' (kPa)	0 (1)	0 (1)	0 (2)	0 (1)	0 (4)
	Friction Angle, φ ^{0}	30 (1)	32 (1)	40 (2)	40 (1)	15 (4)
Frozen	Apparent Cohesion c' (kPa)	112 (1)	112 (1)	-	5 (1)	-
	Friction Angle, φ ^{0}	26 (1)	26 (1)	-	40 (1)	-
Non-Frozen Hydraulic Conductivity (m/s)		4.6×10^{-10} (1)	4.6×10^{-10} (1)	1.3×10^{-7} (2)	5.0×10^{-3} (3)	5.0×10^{-11} (4)
Frozen Hydraulic Conductivity (m/s)		4.6×10^{-10} (1)	4.6×10^{-10} (1)	n/a	1.0×10^{-7} (3)	n/a

(1) SRK (2016b)
(2) Knight Piésold (2009)
(3) Engineering Judgement
(4) https://www.layfieldgroup.com/Geosynthetics/Geomembranes/Geosynthetic-Clay-Liner.aspx

4 Results

4.1 Seepage Analysis

The resultant seepage through the South and West dams is presented in Table 4. These results are the upper limit of seepage considering the conservative assumptions of a fully thawed foundation, and fully saturated tailings. In reality due to tailings freeze back, these conditions will not occur and seepage is expected to be approaching zero.

Furthermore, post-closure when the North Dam is breached and the system phreatic level returns to the pre-mining value of 28.3 m, the possibility of long-term seepage is all but eliminated because the lowest toe elevation of the South and West dams is 31.0 m at the South Dam.

Table 4: Seepage Analysis Results

Location	Maximum Tailings Elevation (m)	Unit Dimensional Seepage Rate (m^3/sec)	Total Seepage Rate (m^3/day)
South Dam	44.5	1.14×10^{-6}	51.2
West Dam	44.5	8.08×10^{-9}	0.33

4.2 Stability Analysis

The stability of the upstream slope of the South and West dams, at their maximum design height, was analyzed. The results are presented in Table 5.

The computed FOS for the fully thawed foundation conditions under the South Dam exceed the minimum required values; and therefore, the less conservative partially thawed foundation condition was not analyzed (Table 5). The FOS obtained from a fully thawed foundation under the West Dam does not meet the required minimum required values; however, the more realistic partially thawed foundation condition does satisfy the criteria.

Table 5: Stability Analysis Results for South and West Dams (Upstream Slope)

Location	Loading Condition	Foundation Condition	Analysis Type	Minimum FOS (CDA 2014)	Computed FOS
South Dam	Long Term	Fully Thawed (Drained)	Static	1.5	1.6
	During or at End of Construction	Fully Thawed (Undrained)	Static	1.3	1.6
	During Earthquake	Fully Thawed (Undrained)	Pseudo-static	1.0	1.4
West Dam	Long Term	Fully Thawed (Drained)	Static	1.5	2.3
	During or at End of Construction	Fully Thawed (Undrained)	Static	1.3	1.1
	During or at End of Construction	Partially Thawed (Undrained)	Static	1.3	1.8
	During Earthquake	Partially Thawed (Undrained)	Pseudo-static	1.0	1.3
	During Earthquake	Fully Thawed (Undrained)	Pseudo-static	1.0	0.9

Results of the downstream slope analysis are presented in Table 6. A fully thawed foundation results in FOS less than the required minimum; however, when the more realistic boundary condition of a partially thawed foundation is imposed the FOS all exceed the required minimums.

Table 6: Stability Analysis Results for South and West Dams (Downstream Slope)

Location	Loading Condition	Foundation Condition	Analysis Type	Minimum FOS (CDA 2014)	Computed FOS
South Dam	Long Term	Fully Thawed (Drained)	Static	1.5	1.6
	During or at End of Construction	Fully Thawed (Undrained)	Static	1.3	0.5
	During or at End of Construction	Partially Thawed (Undrained)	Static	1.3	1.3
	During Earthquake	Fully Thawed (Undrained)	Pseudo-static	1.0	0.5
	During Earthquake	Partially Thawed (Undrained)	Pseudo-static	1.0	1.2
West Dam	Long Term	Fully Thawed (Drained)	Static	1.5	1.5
	During or at End of Construction	Fully Thawed (Undrained)	Static	1.3	0.9
	During or at End of Construction	Partially Thawed (Undrained)	Static	1.3	1.3
	During Earthquake	Fully Thawed (Undrained)	Pseudo-static	1.0	0.8
	During Earthquake	Partially Thawed (Undrained)	Pseudo-static	1.0	1.1

5 References

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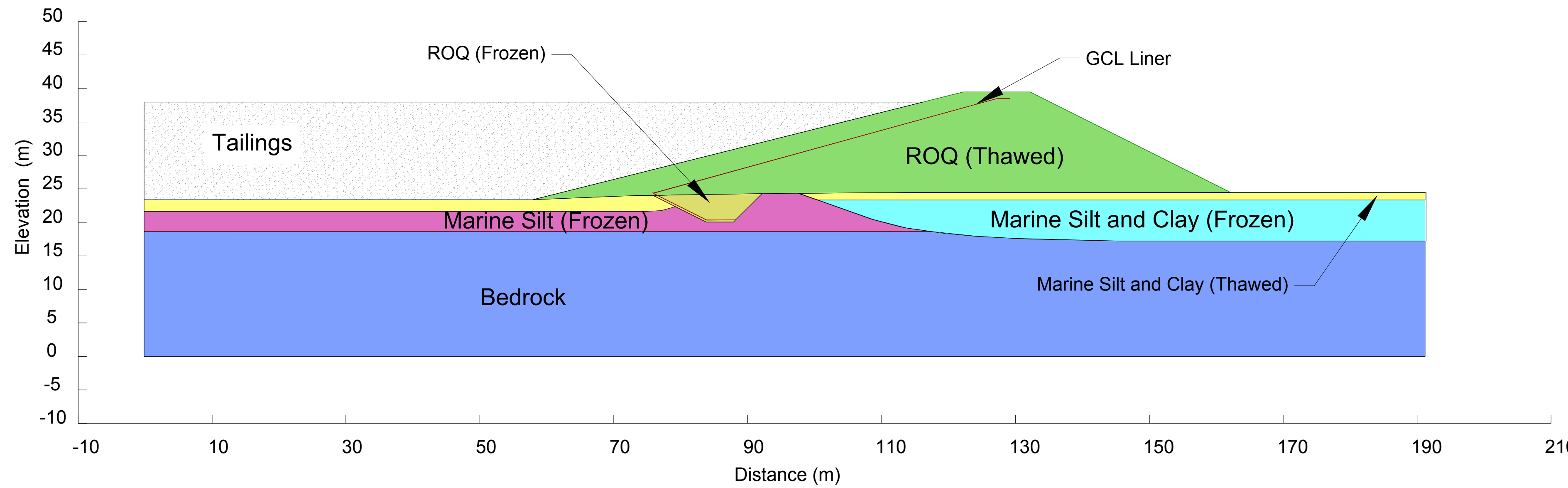
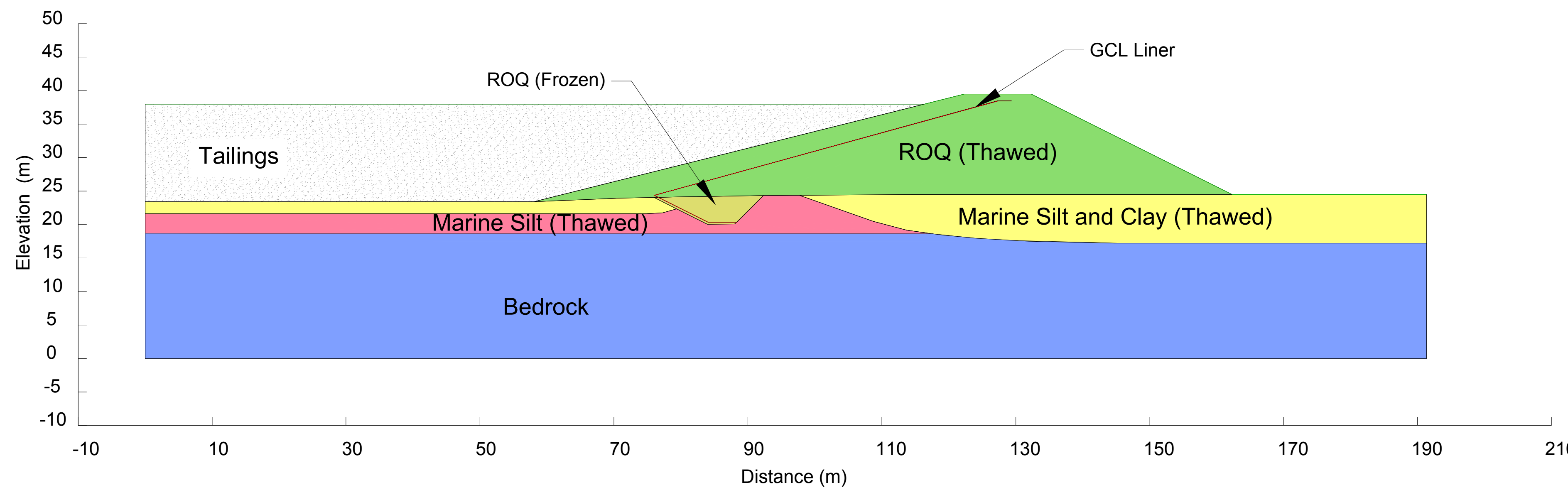
Figures



South Dam
Partially Thawed Foundation Condition

Scale in Metres



South Dam
Fully Thawed Foundation Condition

Scale in Metres

LEGEND

- ROQ (Frozen)
- ROQ (Thawed)
- Marine Silt (Thawed)
- Marine Silt (Frozen)
- Tailings
- Bedrock
- Marine Silt and Clay(Frozen)
- Marine Silt and Clay (Thawed)
- GCL Liner

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MAC RESOURCES

Doris Phase 2 TIA
Seepage and Stability Analysis

South Dam - Analyzed Section

SRK JOB NO.: 1CT022.004.610

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Hope Bay Project

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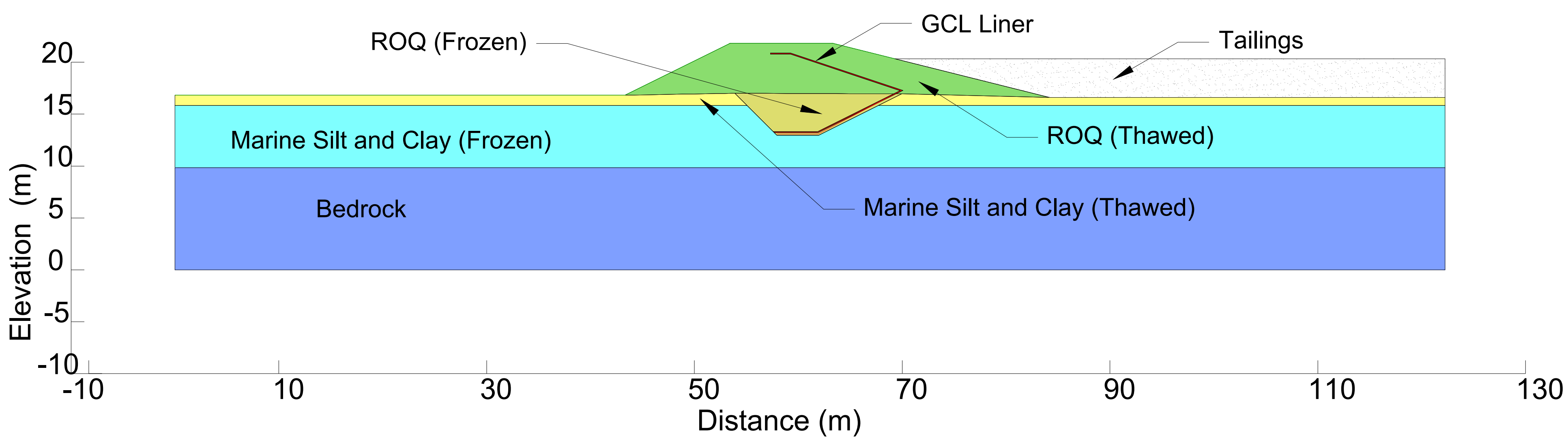
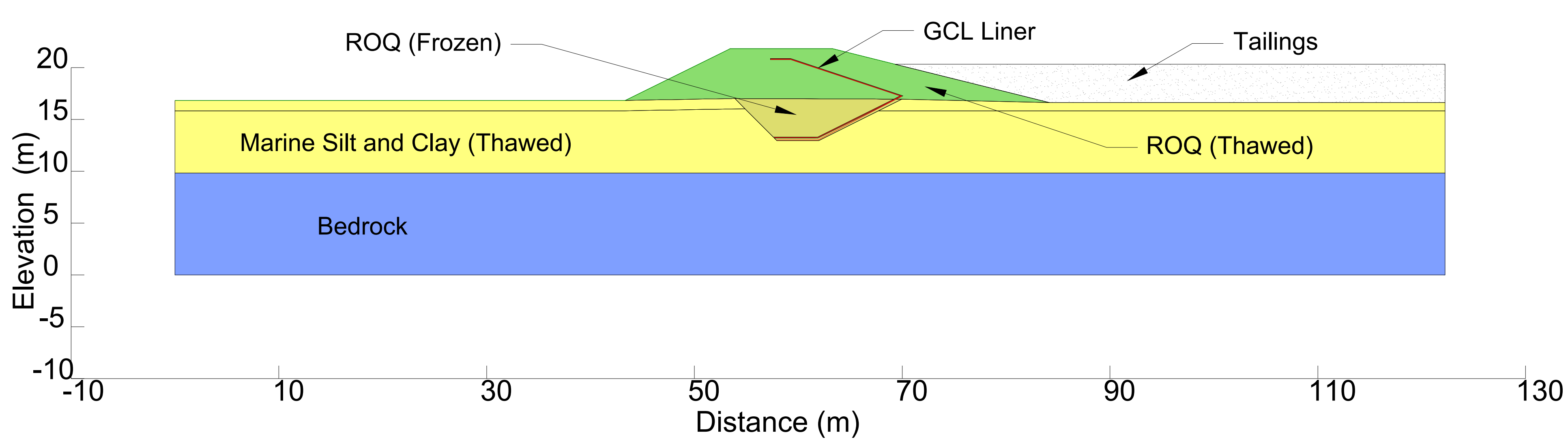


West Dam

Partially Thawed Foundation Condition

0 5 10 15 20 25

Scale in Metres



West Dam

Fully Thawed Foundation Condition

0 5 10 15 20 25

Scale in Metres

LEGEND

- ROQ (Frozen)
- ROQ (Thawed)
- Marine Silt (Thawed)
- Marine Silt (Frozen)
- Tailings
- Bedrock
- Marine Silt and Clay(Frozen)
- Marine Silt and Clay (Thawed)
- GCL Liner

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MAC RESOURCES

Hope Bay Project

Doris Phase 2 TIA
Seepage and Stability Analysis

West Dam - Analyzed Section

SRK JOB NO.: 1CT022.004.610

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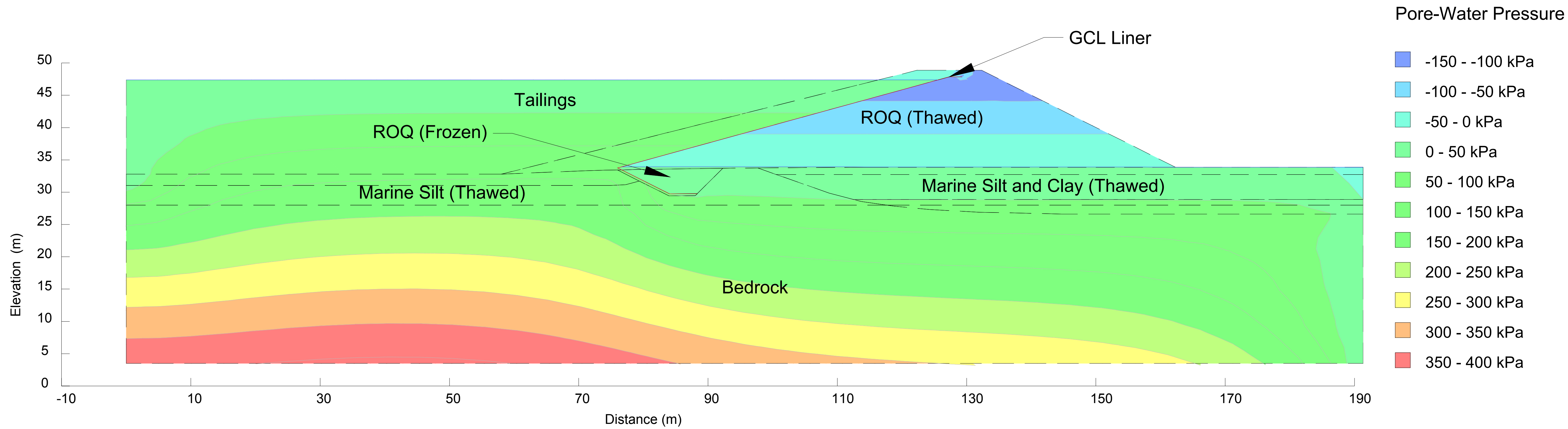
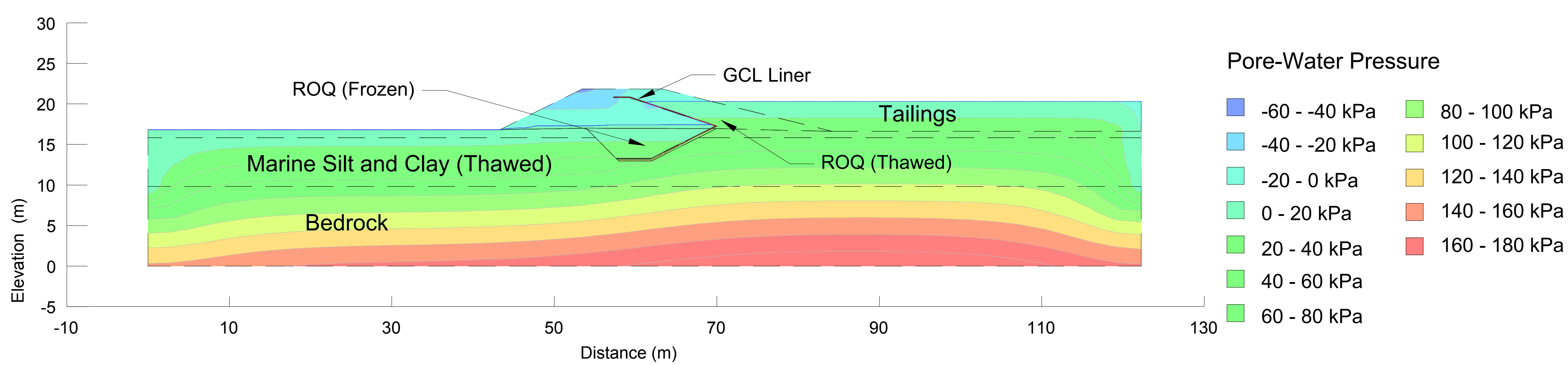


South Dam

Pore - Water Pressure Contours

0 5 10 15 20 25

Scale in Metres



West Dam

Pore - Water Pressure Contours

0 5 10 15 20 25

Scale in Metres

LEGEND

- Phreatic Surface
- Material Boundary
- Contour Line
- GCL Liner

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SRK JOB NO.: 1CT022.004.610

FILE NAME: HopeBay_1CT022.004_SouthWestDams_SlopeStabilityMemo_Figures_Rev03_sa_ms.dwg

MAC RESOURCES

Hope Bay Project

Doris Phase 2 TIA
Seepage and Stability Analysis

Analyzed Seepage Models

DATE:	APPROVED:	FIGURE:
May 2016	IM	3