5.4 Minimum Freeboard Estimate

The minimum freeboard for the Back River tailings dam was determined to be 1.3 m for an event equal to the PMF with a wind set-up and wave run-up during a 2-year wind storm.

6 Conclusions

Based on a High dam risk classification an a full supply level of 305 masl, the normal and minimum freeboard limits for the Back River TSF Containment Dam were calculated as follow:

- Normal Freeboard = 1.3 m
- Minimum Freeboard = 1.3 m

Based on these results, the required freeboard for the TSF Containment Dam is 1.3 m.

7 References

[EC] Environment Canada. 2014, Daily records from meteorological stations. Website: http://climate.weather.gc.ca/advanceSearch/searchHistoricData_e.html.

Laio, F., Di Baldassarre, G., & Montanari, A. (2009). Model selection techniques for the frequency analysis of hydrological extremes. *Water Resources Research, 45*.

[SANCOLD] South African National Committee on Large Dams. 1990. Report N°3: Interim guidelines on Freeboard for Dams.

[USACE] US Army Corps of Engineers. 1984. Shore Protection Manual Volume II.

[USACE] US Army Corps of Engineers. 1986. EM 1110-2-2904 Engineering and Design – Design of Breakwaters and Jetties.

[USACE] US Army Corps of Engineers. 1989. EM 1110-2-1414 Engineering and Design – Water Levels and Wave Heights for Coastal Engineering Design.

[USACE] US Army Corps of Engineers. 1997. EM 1110-2-1420 Engineering and Design - Hydrologic Engineering Requirements for Reservoirs.

[USBoR] US Bureau of Reclamation. 1981. Freeboard criteria and guidelines for computing freeboard allowances for storage dams.

[USBoR] US Bureau of Reclamation. 2012. Design Standards N°13 Embankment Dams, Chapter 6: Freeboard.

Appendix E - TSF Containment Dam - Seepage Rate through the Liner

SRK Consulting (Canada) Inc.
2200–1066 West Hastings Street
Vancouver, BC V6E 3X2

T: +1.604.681.4196
F: +1.604.687.5532

vancouver@srk.com
www.srk.com

Memo

To:	Project File	**Client:**	Sabina Gold & Silver Corp.
From:	Sam Amiralaei, EIT Iozsef Miskolczi, PEng	**Project No:**	1CS020.008
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	September 28, 2015
Subject:	TSF Containment Dam - Seepage Rate through the Liner – Final		

1 Introduction

As part of the larger Final Environmental Impact Study (FEIS) for the Back River Project (the Project) in Nunavut, SRK Consulting (Canada) Inc. was retained by Sabina Gold & Silver Corp. to complete the preliminary design of the Tailings Management System (TMS) for the Project and its associated containment dam.

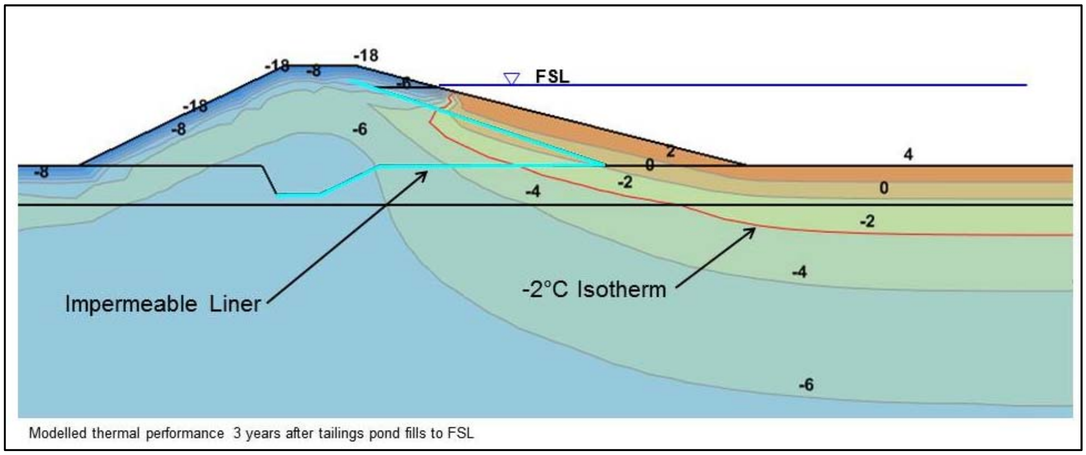
The proposed dam will be located at the Goose Property, about 2 km south of the Goose Main open pit. The facility will be operated as an active TSF for just over two years, after which a portion of it will be used as a normally empty pond for temporary storage of contact water until closure. The dam design encompasses a relatively low profile structure with a maximum crest height of about 14 m, and a length of about 1.7 km constructed primarily of run-of-mine (ROM) rock fill. The dam slopes will be 4H:1V on the upstream side and 2H:1V on the downstream, with a crest width of 10 m. An impermeable liner (Geosynthetic Clay Liner) will be incorporated into the structure of the dam. Tailings beach development will result in the supernatant pond being located directly against the dam structure.

A dam seepage assessment was performed, under the assumption that permafrost will prevent seepage through the foundation and that the impermeable liner will remain keyed into permafrost for the duration of the Project. This memo is documenting the assumptions and the results of the analysis.

2 Seepage Rate Assessment

2.1 General

The TSF dam will be constructed of ROM rock fill, selected waste rock, and crushed gravel materials. These materials have a range of hydraulic conductivities that precludes their normal use for water retention purposes. Therefore, the water retention capacity of the dam relies on the performance of the impermeable liner incorporated within the dam fill, as shown in Figure 1.



Source:\\van-svr0\Projects\01_SITES\Back River\1CS020.008_FEIS\!080_Deliverables\TSF Dam Design Report\030_Appendices\Appendix F - Leakage Rate Through the Liner\Memo\Figure\SeepageMemo_Figures_1CS020.008_rev1_mjm_SA_IM.pptx

Figure 1: Excerpt from Thermal Model (SRK 2015) 3 Years after the TSF was put into Operation

The Project is located in the continuous permafrost region and offers some advantage with respect to water retention, in that the dam fill will be mostly frozen as indicated by the thermal analysis carried out in support of the current design process (SRK 2015). This frozen fill may become a water retention structure as the saturated pore space freezes back. Although the design is not intended as such, the liner bedding and protection layers are analogous to a frozen core.

The seepage assessment; however, ignores the possible frozen fill related aspects and is focused on the liner, adding conservatism to this assessment.

2.2 Assessment Methods

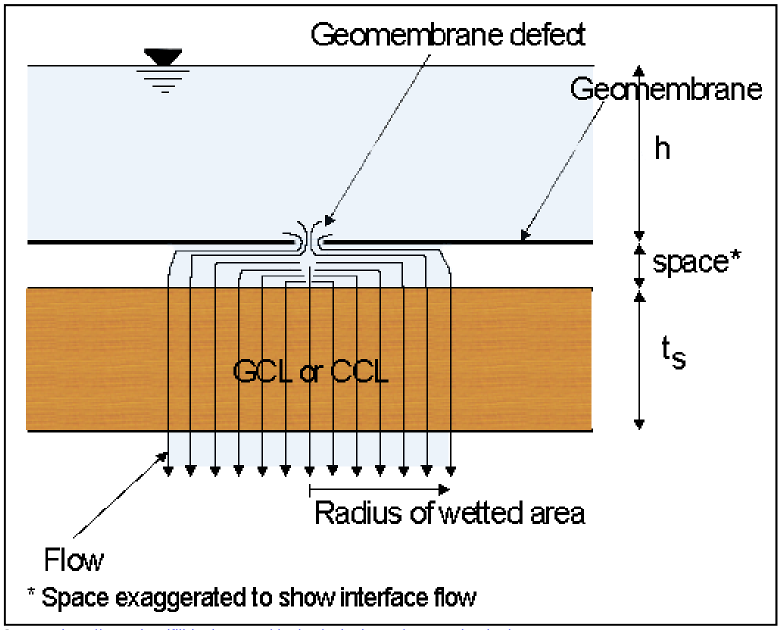
An empirical method (Advanced Geotech Systems 2010) was used to estimate the increase in equivalent infiltration due to liner imperfections. This method assumes that the rate of seepage through a liner due to the liner's permeability is negligible, compared to the rate of seepage through defects in the liner (Giroud and Bonaparte 1989). Therefore, only seepage through defects is considered. The assessment is based on the use of a LLDPE liner, which was deemed to be the most suitable based on a preliminary evaluation of technical and economic considerations.

2.3 Model Setup

The model created by the Advanced Geotech Systems (2010) requires the following input parameters:

- The contact condition between the liner and the bedding material below the liner;
- The installation quality (number of defects per unit surface area);
- The total surface area of the liner;
- The thickness and hydraulic conductivity of the bedding material;
- Total hydraulic head on top of the liner; and
- The shape and size of the defects.

The model also assumes that if there is a defect in the liner, the water first passes through the defect, then it flows laterally some distance between the liner and the bedding layer, and finally it infiltrates into the bedding material (Figure 2).



Source: http://www.landfilldesign.com/design/calculators/composite_leakage.aspx

Figure 2: Conceptual Water Flow Path through a Hole in the Liner

2.4 Input Parameters

The following input values were incorporated into the model for the seepage analysis:

- Good contact condition between the liner and the bedding material was assumed, meaning that the installation would contain few wrinkles.
- Three different installation qualities (excellent, fair, and poor) were analyzed. The frequency of the defects were set at one, seven and fifteen defects per acre (4,000 m^2) for the excellent, fair, and poor installation qualities respectively, as recommended by Giroud *et al*. (1994).
- The total surface area of the liner that was utilized in the model was 54,505 m^2 (the liner surface area above grade, outside of the keytrench).
- The thickness and hydraulic conductivity of the bedding layer that were applied in the model were 0.3 m and 0.003 m/s respectively.
- The average hydraulic head of 4.5 m was determined by calculating the average value of the hydraulic head every 10 m along the centerline of the dam, assuming the pond elevation at full supply level (Elev. 305).
- The imperfections were simulated as having circular or square shapes that could be caused during liner manufacturing (circular pinholes) or during installation (square rips) respectively. It was considered that appropriate quality control will prevent infinitely long linear defects (substandard seams). The defect sizes along with the corresponding defect surface areas are summarized in Table 1.

Table 1: Defect Sizes and Areas

Defect Geometry	Installation Quality Condition	Total Defects (for the Entire Dam)	Defect Dimension (m)	Area of Defect (m^2)
Circular	Excellent	14	0.0005 (diameter)	1.96E-07
	Fair	95		
	Poor	204		
Square	Excellent	14	0.0316 x 0.0316	1.00E-03
	Fair	95		
	Poor	204		

Source: J:\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Leakage Rate Through the Liner\Memo\Table\BackRiver_LinerLeakageVolume_Tables_SA_Rev02

3 Results and Discussion

3.1 Model Results

Only circular and square defects were considered for each model that was analyzed. For each analysis the three quality installations that were previously discussed were considered. Assuming a constant head of 4.5 m, seepage rates were obtained as shown in Table 2.

Table 2: Seepage Rate Estimations

	Excellent Installation Condition		Fair Installation Condition		Poor Installation Condition	
Defect Geometry	**Leakage Rate (m^3/day)**	**Leakage Rate (m^3/year)**	**Leakage Rate (m^3/day)**	**Leakage Rate (m^3/year)**	**Leakage Rate (m^3/day)**	**Leakage Rate (m^3/year)**
Circular	6,409	2,339,382	44,865	16,375,671	96,163	35,099,319
Square	15,051	5,493,507	105,346	38,451,115	225,761	82,402,612
Total Seepage	**21,460**	**7,832,889**	**150,211**	**54,826,786**	**321,924**	**117,501,931**

Source:\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Leakage Rate Through the Liner\Memo\Table\BackRiver_LinerLeakageVolume_Tables_SA_Rev02

3.2 Discussion

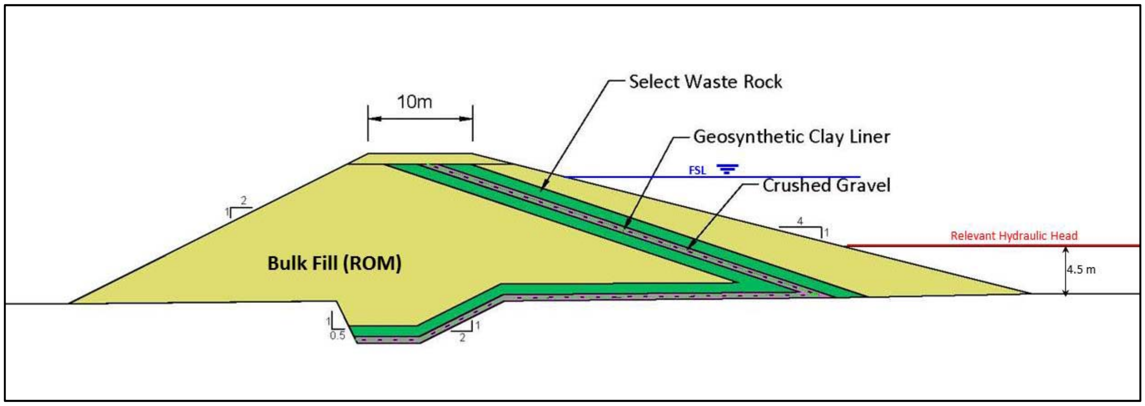
As shown in Table 2, the potential seepage through the liner would render the dam essentially inoperable. This is arguably an extremely conservative assumption and there are numerous examples of LLDPE geomembranes being used successfully in water retaining applications under similar conditions. The theoretical risk however remains, and mitigation strategies are required to reduce the uncertainty.

3.3 Mitigation Alternatives

3.3.1 Replace LLDPE with GCL

One mitigation strategy would be to completely replace the LLDPE geomembrane with a geosynthetic clay liner (GCL). While the permeability of the GCL is somewhat higher than of the LLDPE, the swelling of the bentonite in the liner ensures that, once hydrated, a properly installed liner becomes virtually defect-free.

Based on the GCL properties listed by the manufacturer (Layfield 2015) the flux through the installed and hydrated liner is 1 x 10^{-8} m^3/m^2/sec. Hydraulic conductivity of the GCL equivalent to this flux was applied in a SeepW (Geoslope 2012) model of the dam critical section, as shown in Figure 3.



Source: J:\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Leakage Rate Through the Liner\Memo\Figure\SeepageMemo_Figure3_1CS020.008_rev01

Figure 3: Seepage Flow Modeled in Seep/W

Although the critical section (Figure 3) shows a maximum head of about 12 m, the average head calculated over the entire length of the dam is closer to 4.5 m. Based on this average head of 4.5 m, the SeepW model calculated a total flux of 2.50 x10^{-8} m^3/s through a surface area of 35.5 m^2 of GCL, which represents the surface area of liner per lineal meter of dam at full height. Total seepage of 1,210 m^3/year was calculated by extrapolating this flux to the total area of the liner above grade (outside of the keytrench). This seepage rate is easily manageable by a pump-back system.

From a cost standpoint the GCL is nearly equivalent to the LLDPE liner, as the increased production and transportation costs are offset by the low installation cost and the fact that the non-woven geotextile is no longer required as part of the liner system.

3.3.2 Use GCL as Bedding Layer

The second strategy involves using low permeability blanket (GCL) as a backup. The GCL would replace the non-woven geotextile in the LLDPE liner system, and would be deployed on the underside (downstream) of the geomembrane. An assessment of the seepage through the double-liner system was performed using the same method as for the LLDPE liner alone (Giroud and Bonaparte 1989), with inputs as detailed in Table 1. It was found that the total combined seepage (circular and square defects) expected under the excellent installation scenario is in the order of 204 m^3/year, with detailed values shown in Table 3. While this alternative drastically reduces the estimated seepage, it does not offer the financial offsets presented in the first mitigation alternative.

Table 3: Seepage Rates through the Liner and the GCL

	Excellent Installation Condition		Fair Installation Condition		Poor Installation Condition	
Defect Geometry	**Leakage Rate (m^3/day)**	**Leakage Rate (m^3/year)**	**Leakage Rate (m^3/day)**	**Leakage Rate (m^3/year)**	**Leakage Rate (m^3/day)**	**Leakage Rate (m^3/year)**
Circular	0.17	61	1.16	425	2.49	911
Square	0.39	143	2.73	998	5.86	2,138
Total Seepage	**0.56**	**203**	**3.90**	**1,423**	**8.35**	**3,049**

Source: J:\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Leakage Rate Through the Liner\Memo\Table\BackRiver_LinerLeakageVolume_Tables_SA_Rev02

4 References

Advanced Geotech Systems 2010. "Leakage Rate Through Composite Liner." N.p., 2010. Web. 1 Dec 2012. <http://www.landfilldesign.com/design/calculators/composite_leakage.aspx>.

GEO-SLOPE International, Ltd. GeoStudio. 2012 (Version 8.11.1, Build 7283). Calgary, Alberta.

J.P. Giroud and R. Bonaparte, "Leakage through Liners Constructed with Geomembranes, Part I", Geomembrane Liners, Geotextiles and Geomembranes, 8, 1: 27-67, 1989.SRK Consulting (Canada) Inc. (2015). TSF Containment Dam Thermal Modeling. Technical memorandum prepared for Sabina Gold and Silver Corp., April 14, 2015.

J.P. Giroud, K. Badu-Tweneboah & K.L. Soderman, 1994. Evaluation of Landfill Liners, Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, 5-9 September 1994.

Layfield 2015. Geosynthetic Clay Liner. Product Details. https://www.layfieldgroup.com/ Geosynthetics /Geomembranes/Geosynthetic-Clay-Liner.aspx, accessed April 14, 2015.

SRK Consulting (Canada) Inc. (2015), *TSF Containment Dam Thermal Modeling - DRAFT.* Technical memorandum prepared for Sabina Gold & Silver Corp., Project No. 1CS020.008, September 2015.

Appendix F - TSF Dam Stability Analysis

SRK Consulting (Canada) Inc.
2200–1066 West Hastings Street
Vancouver, BC V6E 3X2

T: +1.604.681.4196
F: +1.604.687.5532

vancouver@srk.com
www.srk.com

Memo

To:	File	**Client:**	Sabina Gold & Silver Corp.
From:	Sam Amiralaei, PEng Iozsef Miskolczi, PEng	**Project No:**	1CS020.008
Reviewed:	Maritz Rykaart, PhD, PEng	**Date:**	October 7, 2015
Subject:	TSF Dam Stability Analysis – Final		

1 Introduction

As part of the Final Environmental Impact Study for the Back River Project (the Project) in Nunavut, Canada, SRK Consulting (Canada) Inc. was retained by Sabina Gold & Silver Corp. to complete the preliminary design of the Tailings Management System (TMS). This included the preliminary design for the containment dam of the Tailings Storage Facility (TSF).

The proposed dam will be located at the Goose Property, about 2 km south of the Goose Main open pit. The facility will be operated as an active TSF for just over two years, after which a portion of it will be used as a normally empty event pond for storage of contact water for an extended period. The dam design encompasses a relatively low profile structure with a maximum crest height of about 14 m, constructed primarily of run-of-mine (ROM) rock fill. The dam slopes will be 4H:1V on the upstream side and 2H:1V on the downstream, with a crest width of 10 m. An impermeable liner (GCL) will be incorporated into the structure of the dam. Tailings beach development will result in the supernatant pond being located directly against the dam structure.

This memo documents the stability analysis completed on the dam, demonstrating that the design adheres to the required stability criteria as defined by the Canadian Dam Safety Guidelines (CDA 2014). Specifically the following scenarios were evaluated:

- Upstream slope and along the interface between the geosynthetic clay liner (GCL) and the dam fill;
- Upstream and downstream slopes of the tailings dam at its full supply level (FSL) under static and pseudo-static conditions; and
- Upstream slope of the dam following a rapid drawdown event.

2 Stability Assessment Parameters

2.1 Dam Hazard Classification

A Dam Hazard Classification for the Back River TSF Dam was completed in accordance with the Canadian Dam Association (CDA) Dam Safety Guidelines (CDA 2014). The designated classification was HIGH.

2.2 Minimum Factors of Safety

The recommended minimum factors of safety (FOS) for the Back River TSF Dam in accordance with the Dam Safety Guidelines (CDA 2014, Tables 3-4 and 3-5) are summarized in Table 1.

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

Factor of Safety	Analysis Method	Application
1.3	Static	End of construction before reservoir filling
1.5	Static	TSF operations
1.2	Static	Rapid drawdown
1.0	Pseudo-Static	All dam configurations, with seismic loading

2.3 Seismic Design Parameters

Minimum seismic design criteria is specified in the Dam Safety Guidelines (CDA 2014). For a dam hazard category of HIGH, a peak ground acceleration (PGA) of the 1:2,475 year recurrence interval is specified.

Site specific seismic parameters for the Project were obtained from the National Building Code of Canada website (NRC 2014) which provides ground accelerations and probability of occurrences. Since the Project is located in an area of Canada not considered to be seismically active, this methodology to obtain seismic parameters is deemed suitable.

The corresponding seismic hazard is described by spectral acceleration (Sa) values at periods of 0.2, 0.5, 1.0 and 2.0 seconds. The PGA value of 0.036, corresponding to the 1:2,475 year recurrence interval event, was used in the stability analysis. These value are all summarized in Table 2.

Table 2: Project Seismic Hazard Values

Spectral Acceleration	Ground Motion (g)
$Sa_{(0.2)}$	0.095
$Sa_{(0.5)}$	0.057
$Sa_{(1.0)}$	0.026
$Sa_{(2.0)}$	0.008
PGA	**0.036**

Source: J:\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Appendix B - Seismicity\ NBCC2010hazard_65.5284_-106.3966__7_eng

2.4 Material Properties

During the winter and early spring of 2015, a detailed geotechnical site investigation was carried out along the proposed centerline of the TSF dam.

The field investigation included 19 boreholes with drill methods that allowed for preservation of permafrost and collection of undisturbed soil samples. Routine indicator and specialized strength and consolidation testing was carried out on collected samples (SRK 2015a).

SRK (2015b) provides a comprehensive summary of site specific geotechnical design parameters for the Project based on geotechnical field investigations carried out elsewhere on the Property. The recent laboratory testing confirms that generic material properties at the TSF are consistent with the site-wide parameter. Properties used for modeling the geosynthetic clay liner (GCL) are based on manufacturer recommended values. Table 3 summarizes the material properties used in the analysis.

Given the limited availability of measured material properties for the stability analysis, the general approach was to select conservative (i.e. low frictional strength) values in all cases. This approach was favoured over a rigorous sensitivity analysis that would normally be performed in similar cases.

Table 3: Material Properties used in Stability Analysis

Geotechnical Domain	Friction Angle (Deg.)	Cohesion (kPa)	Unit Weight (kN/m^3)	Data Source
ROM Waste Rock (Dam Bulk Fill)	38	0	20	SRK past project
Select Waste Rock (Transition Zone)	38	0	20	SRK past project
Crushed Rock (GCL Protection)	35	0	20	SRK past project
Silty Sand (Dam Foundation)	33	0	18	SRK (2015)
GCL	8	0	9	Manufacturer brochure

Source: J:\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Slope Stability Check for Construction\Memo\Table\BackRiver_SlopeStabilityReport_Memo_Tables_SA_Rev00

3 Model Setup

3.1 Numerical Model

The stability analysis was done using the commercial software SLOPE/W, a two-dimensional limit equilibrium slope stability analysis software tool developed by GEO-SLOPE International Ltd. (Geoslope 2007). The model is used to evaluate both circular and non-circular failure modes using many well documented empirical methods. For this study the Morgenstern-Price analysis method with circular failure surfaces was used.

3.2 Model Geometry

The TSF dam will be designed as a water retaining structure since tailings will not be beached from the dam (although through the life of the facility a portion of the dam will have tailings adjacent to it). Water retention is provided by means of freezing in a GCL into the foundation permafrost, creating a frozen foundation dam. The key trench is excavated up to 4 m deep with upstream and downstream slopes of 2H:1V and 0.5H:1V respectively. The key trench gets backfilled with compacted ROM waste rock, and this also forms the bulk of the dam structure. The liner extends from the base of the key trench, along the upstream slope of the key trench excavation, before sweeping back along the upstream face of the dam as slope of 3H:1V, as shown in Figure 1. The liner will be protected by a 0.3 m thick bedding layer and a same thickness cover layer of crushed gravel, with a 1 m thick layer of transition fill between the bedding layers and the ROM waste rock.

For stability assessment, the critical section of the dam was considered to be the tallest section, with the deepest key trench profile, shown on Figure 1.

Two models, A and B, were created and analyzed for stability. The main objectives of each model were:

- Model A: to determine the stability of the upstream slope of the dam along the interface of the GCL; and
- Model B: to determine the stability of the dam at its full supply level (FSL) and during a rapid drawdown event respectively.

The main difference between the two models is that Model A does not take into consideration the effect of the foundation material and the phreatic surface within the dam fill.

3.3 Assessment Method

Pore water pressure conditions were applied through a piezometric line assumed at FSL of the TSF. Further conservatism was introduced by assuming water only, i.e. no tailings, is impounded behind the dam, thus eliminating any “buttressing” effect the tailings may have.

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.

Each analysis mode allowed the slip surface with the lowest FOS to be identified for static and pseudo-static drained (i.e. no excess pore pressure) loading conditions.

3.3.1 Static Analysis

The stability of both models was analyzed under static conditions. As mentioned previously, the main objective for creating and analyzing Model A was to check the upstream slope of the dam along the interface between the rock fill and the geosynthetic liner system. Therefore, only the upstream slope of the dam for Model A was statically analyzed. For Model B, static analyses were carried out for both the upstream and downstream slopes of the dam, with the view of assessing the structural stability with respect to foundation failure.

3.3.2 Pseudo-static Analysis

For pseudo-static analysis, a horizontal seismic load coefficient of 0.036 (equal to the PGA) was applied to the model and the stability of upstream and downstream slopes of Model B were evaluated.

3.3.3 Rapid Drawdown Analysis

Stability analyses for the rapid drawdown scenario were completed for Model B. The simulation consisted of lowering the specified phreatic surface by 10.5 m over a period of 24 hours, simulating a rapid loss of water from the reservoir.

4 Model Results

Results of slope stability analyses are summarized in Table 4.

Table 4: Preliminary Results of Slope Stability Analysis

Model	Analysis Type	Minimum Factor of Safety
Model A (Upstream Slope)	Static	1.5
Model B (Upstream Slope)	Static	1.5
Model B (Upstream Slope)	Pseudo-Static	1.2
Model B (Downstream Slope)	Static	1.6
Model B (Downstream Slope)	Pseudo-Static	1.4
Model B (Upstream Slope)	Static; Drawdown	1.4

Source: J:\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Slope Stability Check for Construction\Memo\Table\ BackRiver_SlopeStabilityReport_Memo_Tables_SA_Rev00

5 Discussion and Recommendations

5.1 General

Determination of the safety of the structure relies on comparing the FOS computed by the model (Table 4) with a set of FOS recommended by the CDA guidelines (Table 3).

As shown in Table 4, the minimum FOS for each of the analyzed conditions exceeds the minimum FOS recommended by the CDA guidelines.

5.2 Dam Stability

5.2.1 Slope Stability

As shown in Table 4, the minimum FOS computed for the upstream and downstream slope exceeds the recommended values. In the case of the upstream slope, a comparison of the results for Model A and Model B shows that the identified critical slip surface lies within the GCL in both cases. This was to be expected, given the fact that the GCL is the material with the lowest friction angle. This result confirms that deploying the liner at a 3H:1V slope is reasonable from a stability standpoint.

5.2.2 Seismic Stability

The FOS computed for the pseudo-static analysis simulating the dam stability during an earthquake event indicates that the dam is safe in the event with a return period of 1:2,475 years.

5.2.3 Rapid Drawdown

During rapid drawdown, the stabilizing effect of the water on the upstream face is lost but the pore water pressures within the embankment can remain high. The dissipation of pore water pressure in the embankment is largely influenced by the permeability of the embankment material. As a result the stability of the upstream face of the dam can be much reduced. Since the permeability of the upstream slope material (ROM) is high, the material can drain quickly following a rapid drawdown event. As shown in Table 4, the model indicates that the upstream slope stability following a rapid drawdown event is actually increasing, compared to the full operation scenario. The explanation lies in the fact that the diminishing pore pressure (as the phreatic surface is lowered) increases the effective stress, thus increasing the overall stability.

5.2.4 Frost Heave

Frost heave is a result of ice segregation in soils, with the formation of ice lenses interbedded with soil layers. Typically fine grained soils (silt) are most susceptible to frost heave, but a wide variety of soils can exhibit heaving (Andersland and Ladanyi 2004).

In the particular case of the TSF Containment Dam the overburden underlying the foundation of the dam is already frozen to its full depth, i.e. to bedrock. The dam construction sequence requires winter construction to maintain the frozen state of the key trench. The dam fill, acting as a thermal blanket protection, will eliminate the possibility of any additional frost heave in the already frozen ground.

With respect to frost heave of the deposited tailings, the potential volume increase and rise of the tailings surface was accounted for by inclusion of an ice entrainment factor representing 20% of the tailings volume. Seasonal frost heave and thaw weakening affecting the strength of the tailings will be limited to the depth of the active layer, and can only occur for a maximum of two seasons. The very shallow depositional slope (1%) of the tailings will preclude any surface failure.

Encapsulating the tailings underneath a waste rock pile will result in permafrost aggradation into the tailings, with implicit permanent strength gain associated with frozen soils.

6 References

Canadian Dam Association (CDA). 2014. Technical Bulletin: *Application of Dam Safety Guidelines to Mining Dams*. 2014.

GEO-SLOPE International, Ltd. 2007. GeoStudio. 2007 (Version 7.17, Build 4921). Calgary, Alberta.

Natural Resources Canada 2014. 2010 National Building Code Seismic Hazard Calculation. Website at http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2010-eng.php Accessed on October 14, 2014.

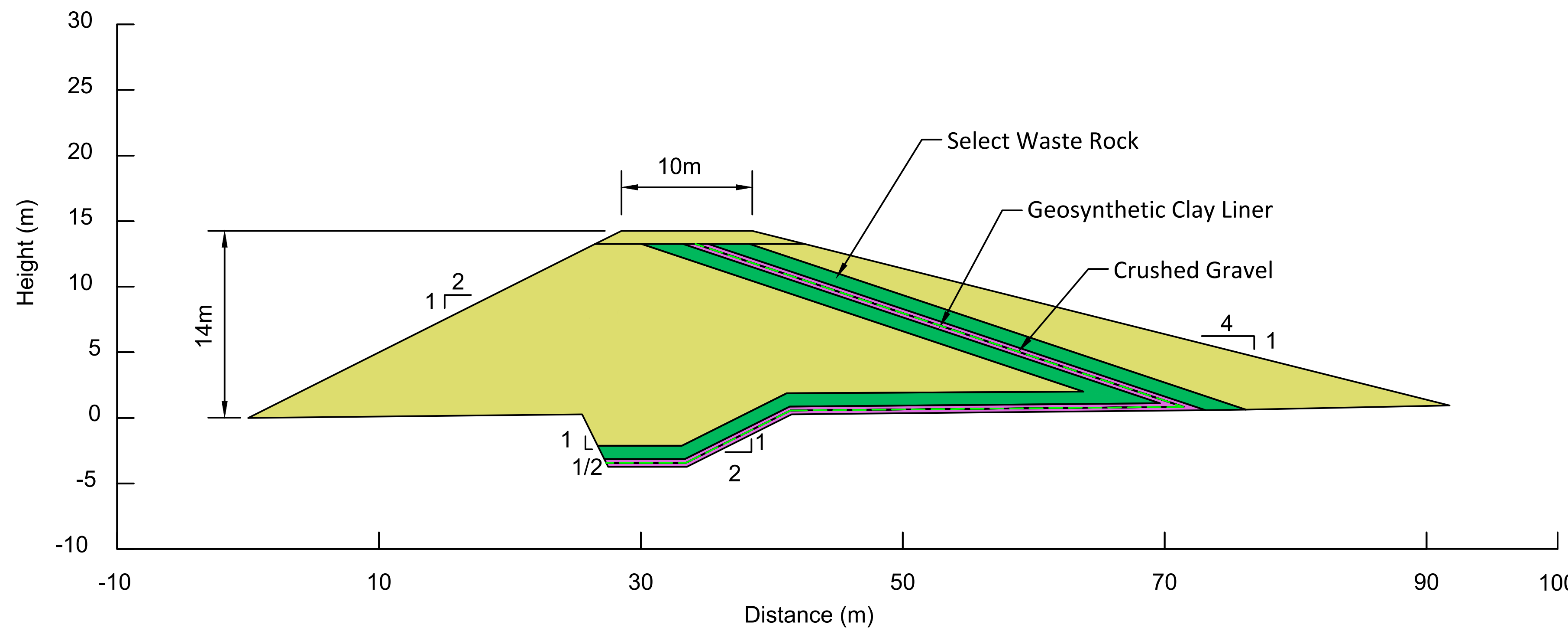
SRK Consulting (Canada) Inc. (2015a), Back River 2015 Overburden Geotechnical Drilling Program Report. Draft Report prepared for Sabina Gold & Silver Corp., Project No. 1CS020.009, October 2015.

SRK Consulting (Canada) Inc. (2015b). *Back River Property Geotechnical Design Parameters*. Report prepared for Sabina Gold & Silver Corp. Project No. 1CS020.006, April 2015.

Rykaart (2014). Personal communication with Maritz Rykaart, November 2014.

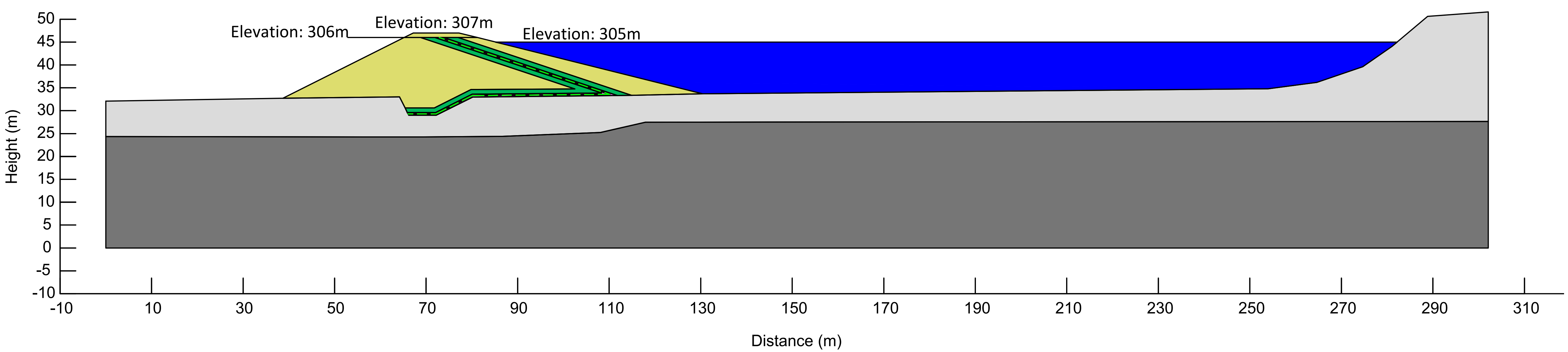
Andersland, O. B. and Ladanyi, B. (2004) Frozen Ground Engineering Second Edition. Wiley & Sons Inc. ISBN 0-471-61549-8

Figures



TSF North Dam
MODEL A

0 5 10 15 20 25
Scale in Metres



TSF North Dam
MODEL B

0 10 20 30 40 50
Scale in Metres

LEGEND

- Bulk Fill (ROM)
- Select Waste Rock
- Crushed Gravel
- Silty Sand
- Bedrock
- Water
- Geosynthetic Clay liner

srk consulting

SRK JOB NO.: 1CS020.008

FILE NAME:BackRiver_SlopeStabilityMemo_Figure1_GCL_SA_Rev01.dwg

Sabina GOLD & SILVER CORP.

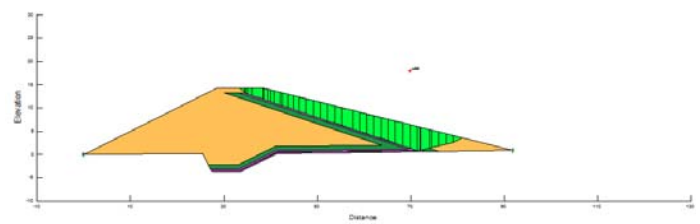
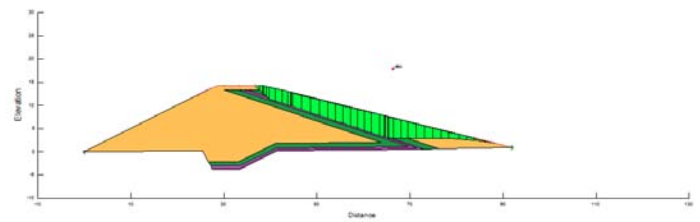
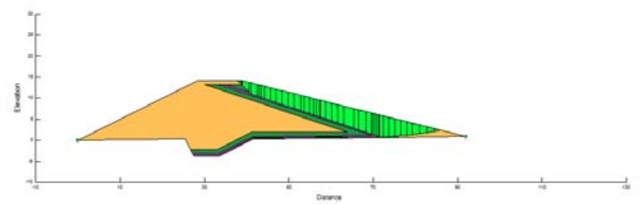
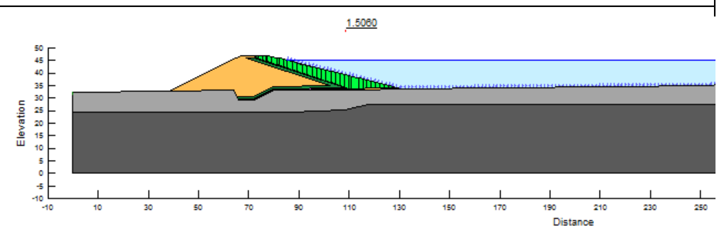
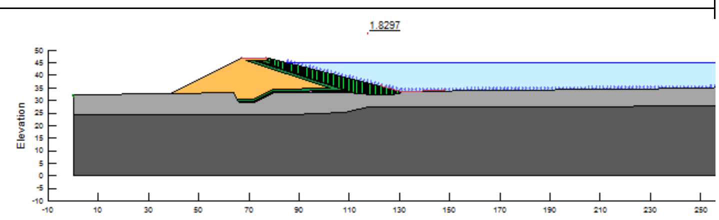
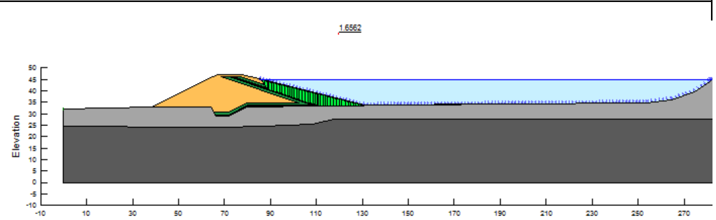
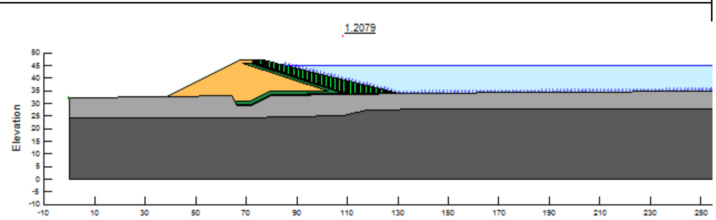
BACK RIVER PROPERTY

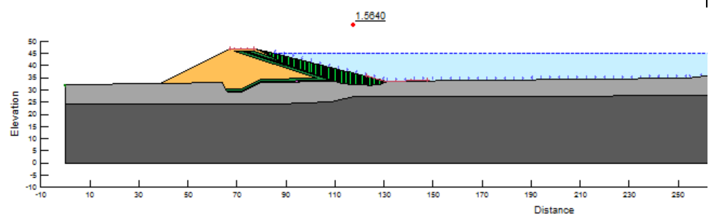
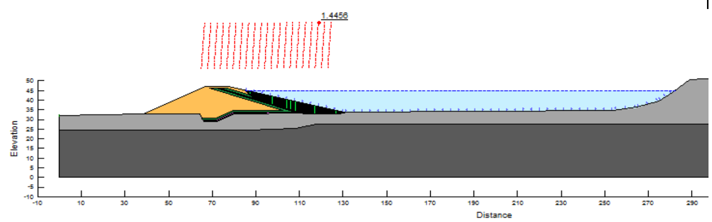
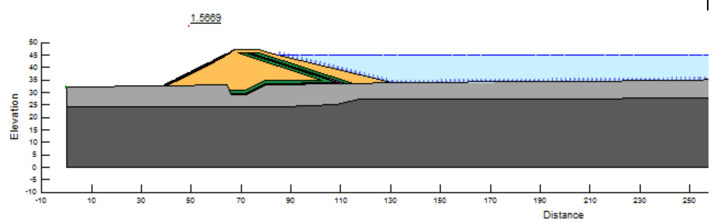
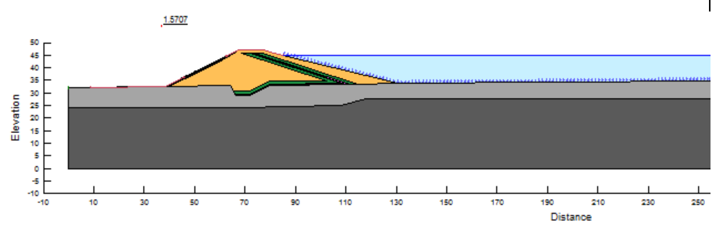
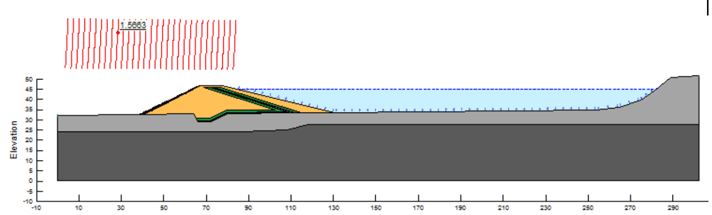
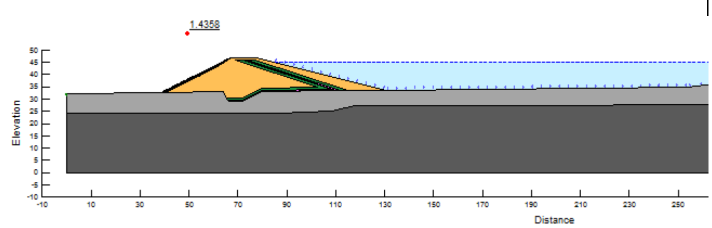
TSF North Dam Stability Analysis

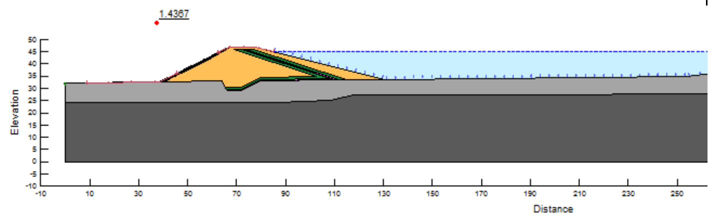
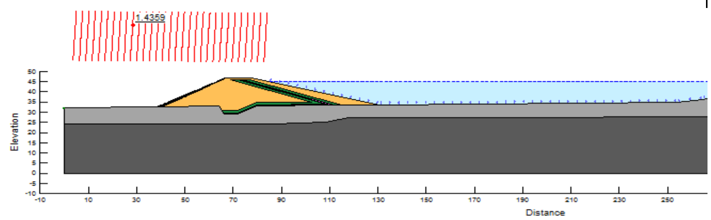
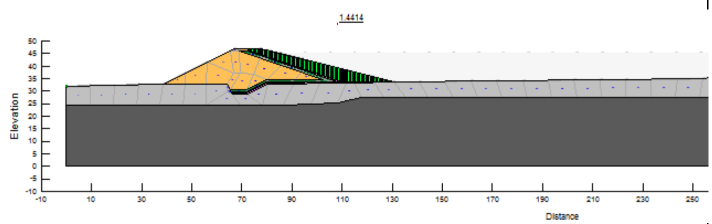
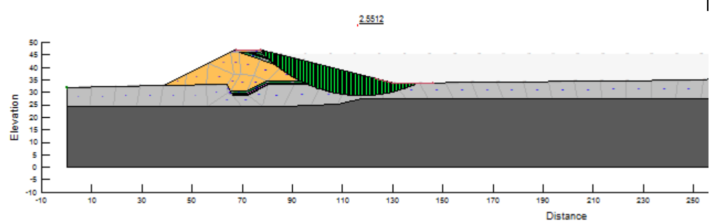
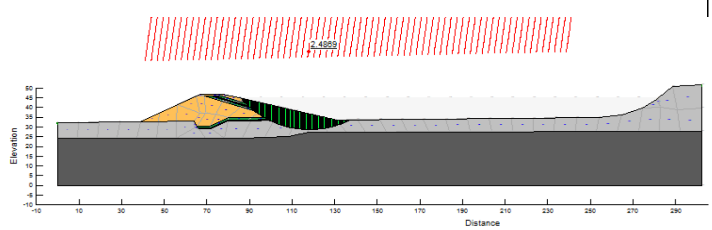
TSF Plan View

DATE:	APPROVED:	FIGURE:
SEP. 2015	IM	1

Attachment A
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model A Upstream Slope (Static)	1.49	Fully Defined	
	1.84	Entry Exit	
	2.10	Grid and Radius	
Model B Upstream Slope (Static)	1.51	Fully Defined	
	1.82	Entry and Exit	
	1.65	Grid and Radius	
Model B Upstream Slope (Pseudo-Static)	1.21	Fully Defined	

Model	Factor of Safety	Slip Surface Option	Figure
	1.56	Entry and Exit	
	1.44	Grid and Radius	
	1.57	Fully Defined	
Model B Downstream Slope (Static)	1.57	Entry and Exit	
	1.57	Grid and Radius	
Model B Downstream Slope (Pseudo-Static)	1.44	Fully Defined	

Model	Factor of Safety	Slip Surface Option	Figure
	1.44	Entry and Exit	
	1.44	Grid and Radius	
Model B Static Rapid Drawdown Upstream Slope	1.44	Fully Defined	
	1.44	Entry and Exit	
	1.44	Grid and Radius	

Appendix G - TSF Containment Dam Thermal Modeling

SRK Consulting (US), Inc.
Suite E-12
4700 Business Park Blvd
Anchorage, Alaska 99503

T: +1 907 677 3520
F: +1 907 677 3620

anchorage@srk.com
www.srk.com

Memo

To:	Project File	**Client:**	Sabina Gold & Silver Corp
From:	Christopher W. Stevens, PhD	**Project No:**	1CS020.008
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	October 8, 2015
Subject:	TSF Containment Dam Thermal Modeling – Final		

1 Introduction

1.1 General

SRK Consulting (Canada) Inc. was retained by Sabina Gold & Silver Corp. to complete a numerical model predicting the thermal performance of the rock fill containment dam (*aka* TSF Containment Dam) associated with the Tailings Storage Facility (TSF) at the Back River Project (the Project) in Nunavut.

The proposed dam will be located at the Goose Property, about 2 km south of the Goose Main open pit. The facility will be operated as an active TSF for three (3) years. Following completion of tailings deposition, a portion of the TSF containment dam (the western flank, aka TSF WRSA Pond) will be used to store contact water up to the end of the active water treatment stage in year 18 (SRK 2015a). Water from the TSF WRSA Pond will be seasonally pumped from the pond to Goose Main TF.

The TSF containment dam will have an active design life of two years as a full water retaining structure and sixteen additional years as a normally empty event pond. At final closure the Containment Dam can be breached and gravity flow to the Goose Main TF will be established (SRK 2015a).

The dam will be constructed primarily of run-of-mine (ROM) waste rock, with a maximum crest height of about 14 m. The dam face will be 4H:1V on the upstream side and 2H:1V on the downstream, with a crest width of 10 m. An impermeable liner (Geosynthetic Clay Liner) will be incorporated into the structure of the dam. Tailings beach development will result in the supernatant pond located directly against the dam structure. The tailings, once drained, will be covered with ROM waste rock following the three years of TSF operation.

The dam will rely on an impermeable liner incorporated in the dam fill and keyed into the permafrost foundation to achieve the required water retention properties. In order to ensure adequate performance of the dam, it is imperative to maintain the frozen state of the key trench.

1.2 Objectives

The objective of the modeling was to determine whether or not the proposed design was suitable given the permafrost foundation conditions. The foundation was considered to be valid if the temperature within the dam key trench remains colder than -2°C, which represents a conservative freezing point depression for the average overburden pore water salinity measured from samples collected at the Goose Property (SRK 2015b; 2015c).

This memo summarizes the assumptions and the results of numerical modeling completed to predict the thermal performance of the TSF Containment Dam. The TSF also has a small South dyke as part of the design. This dam will have tailings beach development and no ponded water against the structure, allowing for seasonal heat loss. As such, thermal modeling was not completed for the South dyke.

2 Methods

2.1 Model Setup

Modeling was completed in a two-dimensional domain by solving for conductive heat movement in the soil, using SoilVision's SVHeat (SoilVision 2011) software package in combination with FlexPDE (FlexPDE 2014). SVHeat version 6 was utilized for the problem setup, while FlexPDE 6.35 solver was used to complete the calculation.

The modeling was based on two critical sections; Section 0+900 and Section 0+300 of the TSF dam (SRK 2015d). Section 0+900 was selected for being the highest section of the dam which will have the most water impounded against the upstream side, acting as a heat source throughout the period of operation. Section 0+300 was selected for being located adjacent to the west pond which will have water impounded against the dam face during operation. After operation of the facility, a seasonal pond will be operated in this location for 16 years. Water accumulating in the pond will be seasonally pumped and the area will be exposed to the atmosphere. Minimal thermal effects from the seasonal water are expected during this period of time.

For Section 0+900, the dam crest was modeled with a 10 m width, 14 m height, upstream slope of 4H:1V, and downstream slope of 2H:1V. The key trench into the in situ overburden material has a 6 m wide base with upstream slope of 2H:1V and downstream slope of 0.5H:1V. The key trench excavation is located near the centre of the dam, so that the upstream toe of the trench is aligned with the centerline of the dam. The model included 5.5 m overburden sand underlain by bedrock. The model geometry for TSF dam Section 0+900 is presented in Figure 1.

For Section 0+300, the dam crest was modeled with a 10 m width, 8 m height, upstream slope of 4H:1V, and downstream slope of 2H:1V. The key trench excavated into overburden material has a 6 m wide base with upstream slope of 2H:1V and downstream slope of 0.5H:1V. The key trench excavation is located near the centre of the dam with 2.2 m of overburden sand underlain by bedrock. The model geometry for TSF dam Section 0+300 from Year 0 to 3 and Year 3 to 25 is presented in Figure 2 and Figure 3, respectively.

2.2 Model Inputs

2.2.1 Material Properties

Three material units were considered: native soil (overburden sand), dam fill (ROM material), and bedrock. No peat or organic layer was considered during the modelling. This omission is reasonable given the minimal thickness of this layer (estimated 10 cm), and the fact that removal of this layer from the model makes the analysis more conservative. If organic layers were to be considered in the model, the rate of thaw would be less due to latent heat required to change phase of ice to water. Table 1 presents a summary of the material properties.

Table 1: Material Thermal Properties

Material	Thermal Conductivity (kJ m^{-1} day^{-1} $°C^{-1}$)		Volumetric Heat Capacity (kJ m^{-3} $°C^{-1}$)	
	Unfrozen	**Frozen**	**Unfrozen**	**Frozen**
ROM Material	104	117	1,697	1,509
ROM Material, Unfrozen Saturation 100%	141	117	2,576	1,509
Overburden Sand (23 ppt pore water salinity)	150	185	2,178	1,801
Bedrock	346	350	2,120	2,110

The thermal properties for ROM material were taken from previous work completed by SRK for granular pad design (SRK 2015e). An additional ROM material was added to consider the change in unfrozen thermal conductivity and heat capacity of the material located on the upstream side of the liner below water (Figure 1). The unfrozen properties for this ROM material (*ROM Material, Unfrozen Saturation 100%*) were estimated for 100% saturation (Table 1).

The thermal properties for overburden sand were based on average soil properties measured from drill samples collected at the TSF (SRK 2015f) and average pore water salinity measured from samples collected at the Goose Property (SRK 2015c). The thermal conductivity was calculated in accordance with Cote and Konrad (2005). The freezing point depression was calculated to be -1.4°C for the sand with an average pore water salinity of 23 parts per thousand (ppt) (SRK 2015b).

2.2.2 Climate Boundary Conditions

Rescan (2014a) presents the available baseline meteorological data collected from the Goose Property. Regional air temperatures were also reviewed and analyzed to establish MAAT for a recent 10-year period (SRK 2015e).

An annual cycle ground response curve was developed based on the 10-year MAAT (2003 to 2012), representing the ground temperature at, and immediately below, ground surface (SRK 2015b). This has been modeled as a sinusoidal function of temperature-time relationship based on Equation 1 and the parameters shown in Table 2.

$$T = \max(nf * \left[MAAT + Amp * Sin\left(\frac{2\pi+(t+230)}{365}\right)\right], \mathrm{nt}\left[MAAT + Amp * Sin\left(\frac{2\pi+(t+230)}{365}\right)\right] \quad \text{Eq. 1}$$

Climate change is considered in Equation 2 using the air climate change factor which allows for a daily increase in air temperature based on the results of SRK (2015g).

$$T = \max(nf * \left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi+(t+230)}{365}\right)\right], \mathrm{nt}\left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi+(t+230)}{365}\right)\right] \quad \text{Eq.2}$$

The air climate change factor applied to Equations 1 was 0.00016°C d^{-1}. This rate is equivalent to an increase in air temperature of +0.58°C per decade.

Where:

- T *is the ground temperature measured in °C*
- nf *is the surface freezing n-factor*
- nt *is the surface thawing n-factor*
- $MAAT$ *is the mean annual air temperature measured in °C*
- Amp *is the air temperature amplitude measured in °C*
- C_A *is the air climate change factor in °C* d^{-1}
- t *is time measured in days*

Table 2 summarizes the model inputs used for the current climate boundary conditions. Surface freezing and thawing n-factors are used in Equation 1 and Equation 2 to account for the thermal offset between the air and the ground surface. Air and ground surface temperature are assumed to equal unity when the n-factor is 1. As the factor decreases, a greater departure in temperature (offset) occurs. A value greater than 1 would result in ground temperatures that exceed the air temperature.

Table 2: Current Climate Boundary Parameters

Model Parameter	Value
Mean annual air temperature *(MAAT)*	-10.7°C
Air temperature amplitude *(Amp)*	21.5°C
ROM Surface, Thawing n-factor *(nt)*	1.0
ROM Surface, Freezing n-factor *(nf)*	0.7
Natural Overburden, Thawing n-factor *(nt)*	0.85
Natural Overburden, Freezing n-factor *(nf)*	0.65
Seasonally Drained Pond, Thawing n-factor (*nt*)	1.5
Seasonally Drained Pond, Freezing n-factor (*nf*)	0.65

The n-factors for exposed ROM surfaces and naturally vegetated surfaces are shown in Table 2. The ROM n-factors were based on published values and engineering judgment. N-factors for natural overburden were based on values calibrated to ground temperature measured at the Goose Property (SRK 2015b) and applied to the area downstream of the TSF Dam (Figures 1 through 3). For TSF dam Section 0+300, separate n-factors were used for the seasonally drained pond present from Year 3 to Year 25. The n-factors for the seasonally drained pond assume similar snow conditions to the natural overburden (0.65) and a conservative thawing n-factor of

1.5 to account for limited vegetation and lower surface albedo immediately following TSF operation.

2.2.3 Initial Conditions

The initial conditions were defined by each material region in the model (Figures 1 through 3). The dam fill (ROM material) was set to an initial temperature of -6°C, since construction of would take place in the winter. The temperature of the ROM material would likely be colder during the winter construction period and therefore represent a conservative condition in the model. The overburden sand and underlying bedrock were set to an initial temperature of -6.5°C which is consistent with ground temperatures measured at the Goose Property (Rescan 2014b). The model assumes continuous permafrost exists beneath the dam alignment prior to construction.

The sides and lower boundary of the model were based on heat flux boundaries presented in previous thermal modelling for the Project (SRK 2015b).

2.2.4 Transient Conditions

TSF Dam Section 0+900

The thermal model for TSF dam Section 0+900 was run for ten (10) years with consideration that the TSF is expected to be operated for three (3) years. Ground temperatures were estimated over this extended period of time to provided additional confidence in the thermal performance of the dam.

A constant temperature of +4°C was applied to the bottom of the TSF and the upstream face of the dam to simulate water impounded at the full storage level of 11 m for ten (10) years. Water at the Full Supply Level (FSL) provides the greatest source of heat acting to thaw the dam foundation. The +4°C temperature at this boundary is consistent with mean annual lake bottom temperatures, and therefore direct modelling of heat transfer through the water column was not necessary. The temperature and FSL are conservative conditions in the model.

The climate boundary was specified for the remaining exposed face of the TSF dam and the area upstream of the dam (Figure 1). The climate boundary prescribed in the model was based on Equation 1 and parameters shown in Table 2. No allowance was made for climate change given the short period of operation of the TSF.

TSF Dam Section 0+300

The thermal model for TSF dam Section 0+300 was run for 25 years. Ground temperatures were estimated over this period of time to estimate the thermal performance of the dam with water at the FSL and operation of the seasonal pond following drainage of the TSF. The seasonal pond was simulated for 25 years for conservatism.

From Year 0 through Year 3, a constant temperature of +4°C was applied along the bottom of the TSF and the upstream face of the dam to simulate water impounded at the full storage level of 6 m (Figure 2). The climate change boundary was prescribed for modeling of TSF dam Section 0+300 given the longer period of time. The climate change boundary was based on Equation 2 and parameters shown in Table 2. The FSL boundary conditions were replaced with the boundary conditions for the seasonally drained pond from the end of Year 3 through Year 25 (Figure 3). The boundary condition assumes draining of the pond throughout the thawing season and no water during winter freeze-up.

3 Results

The model results are shown in Figures 4 and 5 for TSF dam Section 0+900 for Year 3 and Year 10, respectively. The maximum position of the -2°C isotherm, representing the thawing temperature, is shown as a solid red line. The maximum position of the -2°C isotherm during Year 3 is outside of the key trench. Permafrost temperatures at the key trench are estimated to be less than -6°C. The key trench warms to -5°C at the end of Year 10. Similar temperatures are estimated for TSF dam Section 0+300 with the position the -2°C isotherm is outside of the key trench (Figure 6). For this section, permafrost temperatures at the key trench are less than -4°C at the end of three years of TSF operation.

At the end of three (3) years, permafrost thaw beneath the TSF is estimated to be 9.7 m below the original ground surface for the -2°C isotherm. Thaw beneath the TSF increases to 21 m by the end of ten (10) years. Figure 7 shows the estimated temperature beneath the seasonally drained pond for TSF dam Section 0+300 during Year 25. The model shows progressive cooling of the ROM material within the dam and foundation from Year 3 to Year 25.

4 Conclusions

The modeling completed suggests that the thickness of the bulk ROM material of the dam, acting as thermal protection over the key trench is adequate in the proposed TSF dam configuration. Even under the conservative assumption that the operational period of the TSF is extended to ten (10) years, the base of the key trench remains below -2°C for Section 0+900, confirming that the impermeable liner will continue to provide the intended water retaining capacity. Model Section 0+300 also confirms the key trench will remain below -2°C following TSF operation along the western flank where a seasonal pond with be present (aka TSF WRSA Pond).

5 References

Cote J., and Konrad, J-M. 2005. A Generalized Thermal Conductivity Model for Soils and Construction Materials. Canadian Geotechnical Journal. 42: 443-458.

Flex PDE Solutions Inc. 2014. FlexPDE 6. Version 6.36 7/29/2014. http://www.pdesolutions.com /download/flexpde636.pdf, Accessed Dec. 17, 2014.

Rescan 2014a. Back River Project 2006 to 2013 Meteorology Baseline Report. Report submitted to Sabina Gold & Silver Corp. January 2014.

Rescan 2014b. Back River Project Cumulative Permafrost Baseline Data Report (2007 to May 2014). Report submitted to Sabina Gold & Silver Corp. June 2014.

SoilVision Systems Ltd. 2011. SVHeat 1D/2D/3D Geothermal Modelling Software Examples Manual. Soil Vision Systems Ltd. Saskatoon, Saskatchewan, Canada. http://www soilvision .com/downloads/software/svoffice2009/SVHeat_Examples_Manual.pdf, Accessed on Dec. 17, 2014.

SRK Consulting (Canada) Inc. 2015a. Tailings Management System Preliminary Design Report. Report prepared for Sabina Gold & Silver Corp. Project No. 1CS020.008

SRK Consulting Inc. 2015b. Back River Project: Saline Water Pond Containment Dams Thermal Modeling. Memo prepared for Sabina Gold & Silver Corp. Project No. 1CS020.008.

SRK Consulting (Canada) Inc. 2015c. Back River Property Geotechnical Design Parameters. Report prepared for Sabina Gold & Silver Corp. Project No. 1CS020.006.

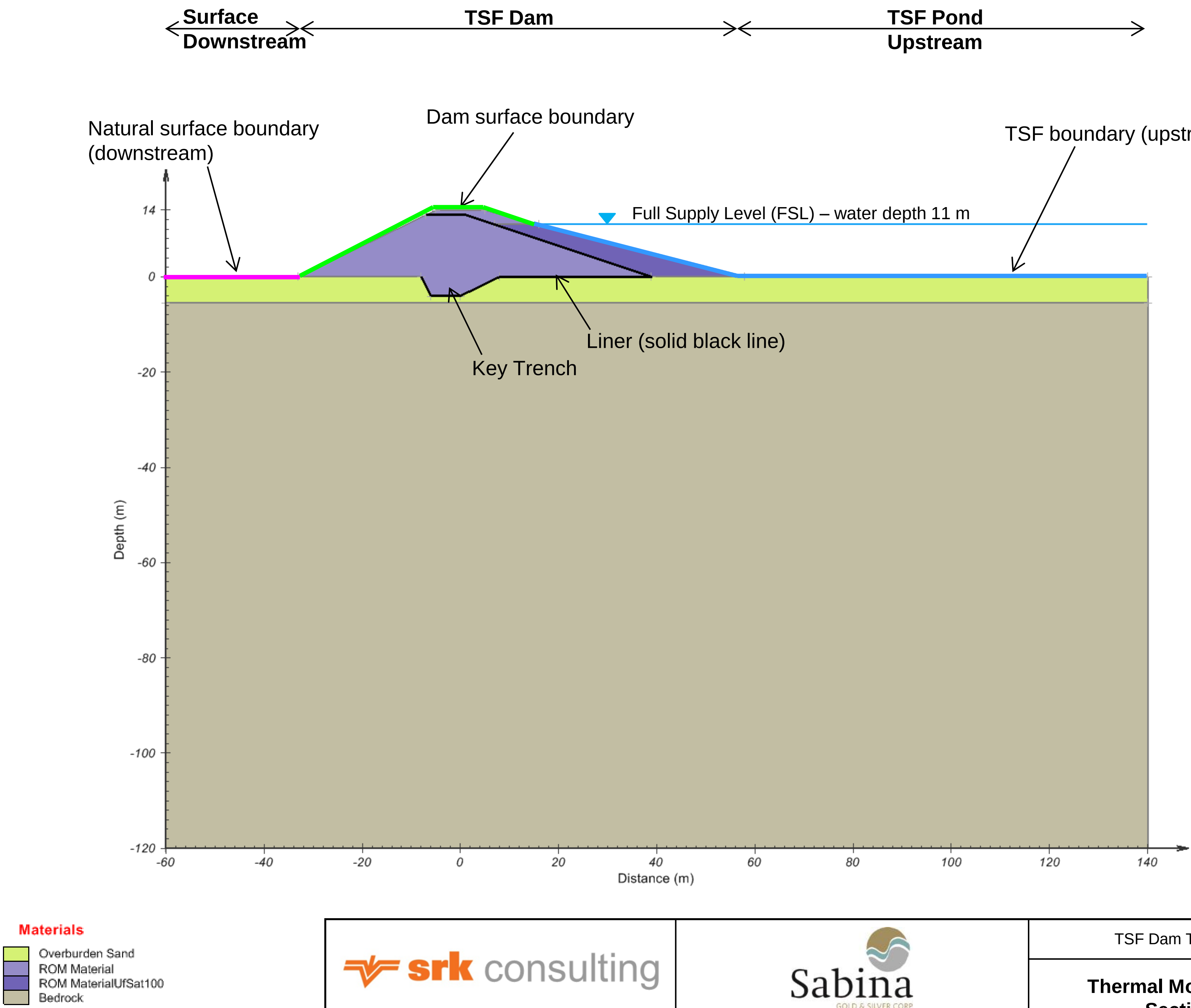
SRK Consulting (Canada) Inc. 2015d. Tailings Management System FEIS Design Report – DRAFT Back River Property, Nunavut, Canada. Report prepared for Sabina Gold & Silver Corp. Project No. 1CS020.008.

SRK Consulting Inc. 2015e. Thermal Modelling to Support Run-of-Quarry Pad Design. Memo prepared for Sabina Gold & Silver Corp. Project No. 1CS020.008.

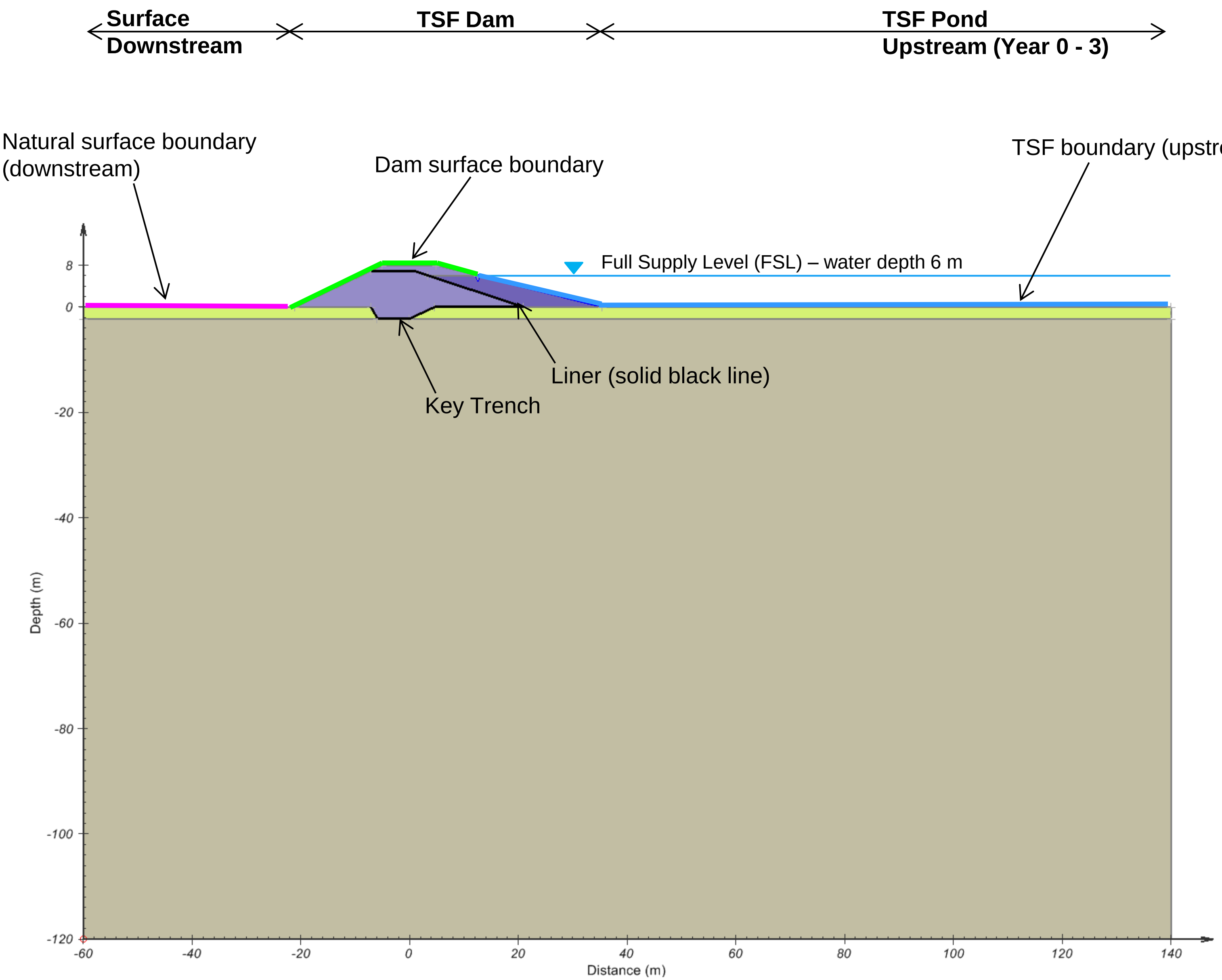
SRK Consulting (Canada) Inc. 2015f. Goose Lake – Back River 2015 Overburden Geotechnical Drill Program Report. Report prepared for Sabina Gold & Silver Corp. Project No. 1CS020.009.

SRK Consulting (Canada) Inc. 2015g. Standardized Procedure for Climate Change Integration into Engineering Design. Memorandum prepared for Sabina Gold & Silver Corp. Project No. 1CS020.008.

Figures



srk consulting	Sabina GOLD & SILVER CORP	TSF Dam Thermal Model
		Thermal Model Geometry - Section 0+900
Job No: 1CS020.008 Filename: TSF_Model_GeometrySection0+900.pptx	BACK RIVER PROJECT	Date: 10/4/2015 | Approved: cws | Figure: 1



Materials

- Overburden Sand
- ROM Material
- ROM MaterialUfSat100
- Bedrock

srk consulting

Sabina
GOLD & SILVER CORP

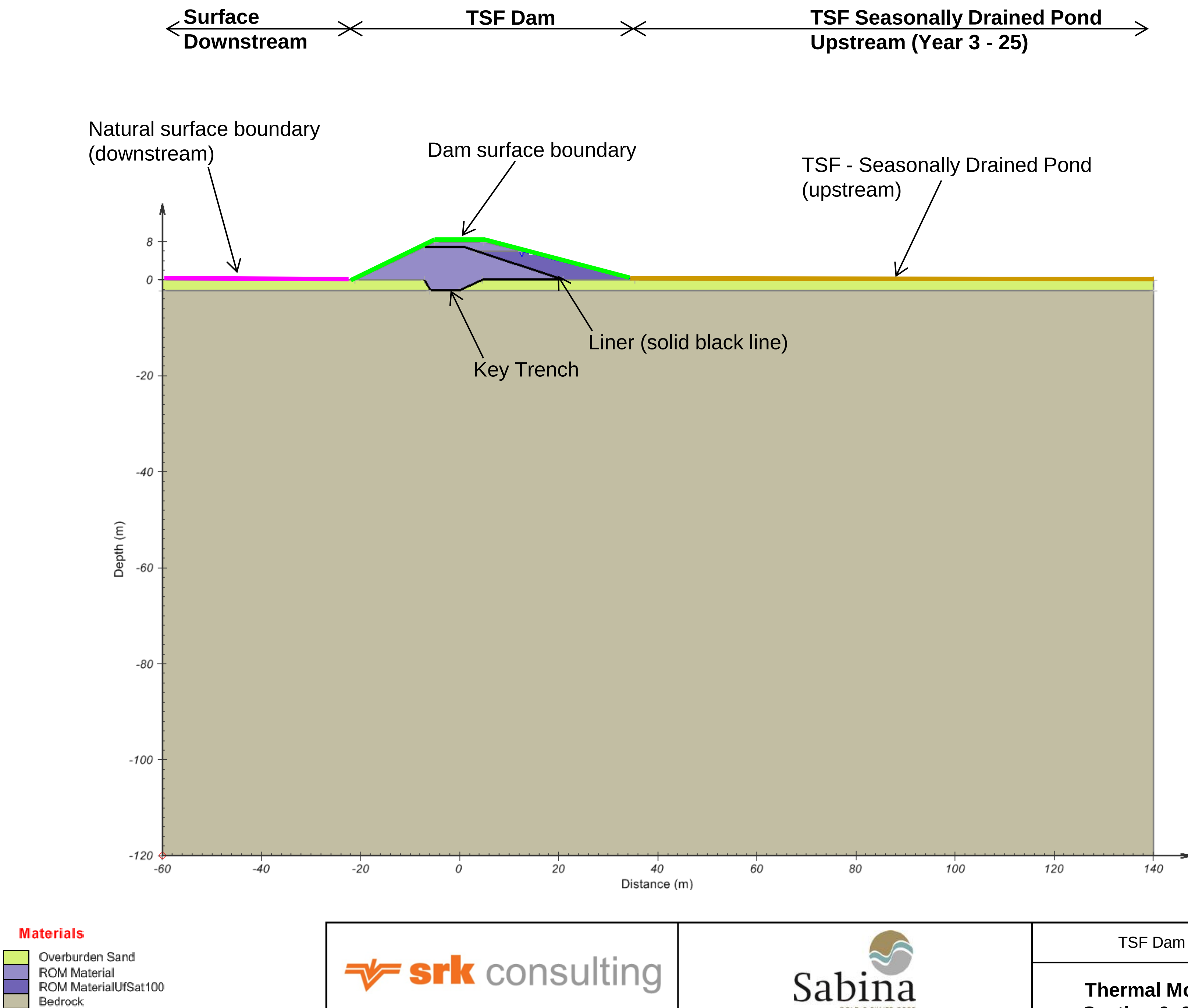
Job No: 1CS020.008
Filename: TSF_Model_GeometrySection0+300.pptx

BACK RIVER PROJECT

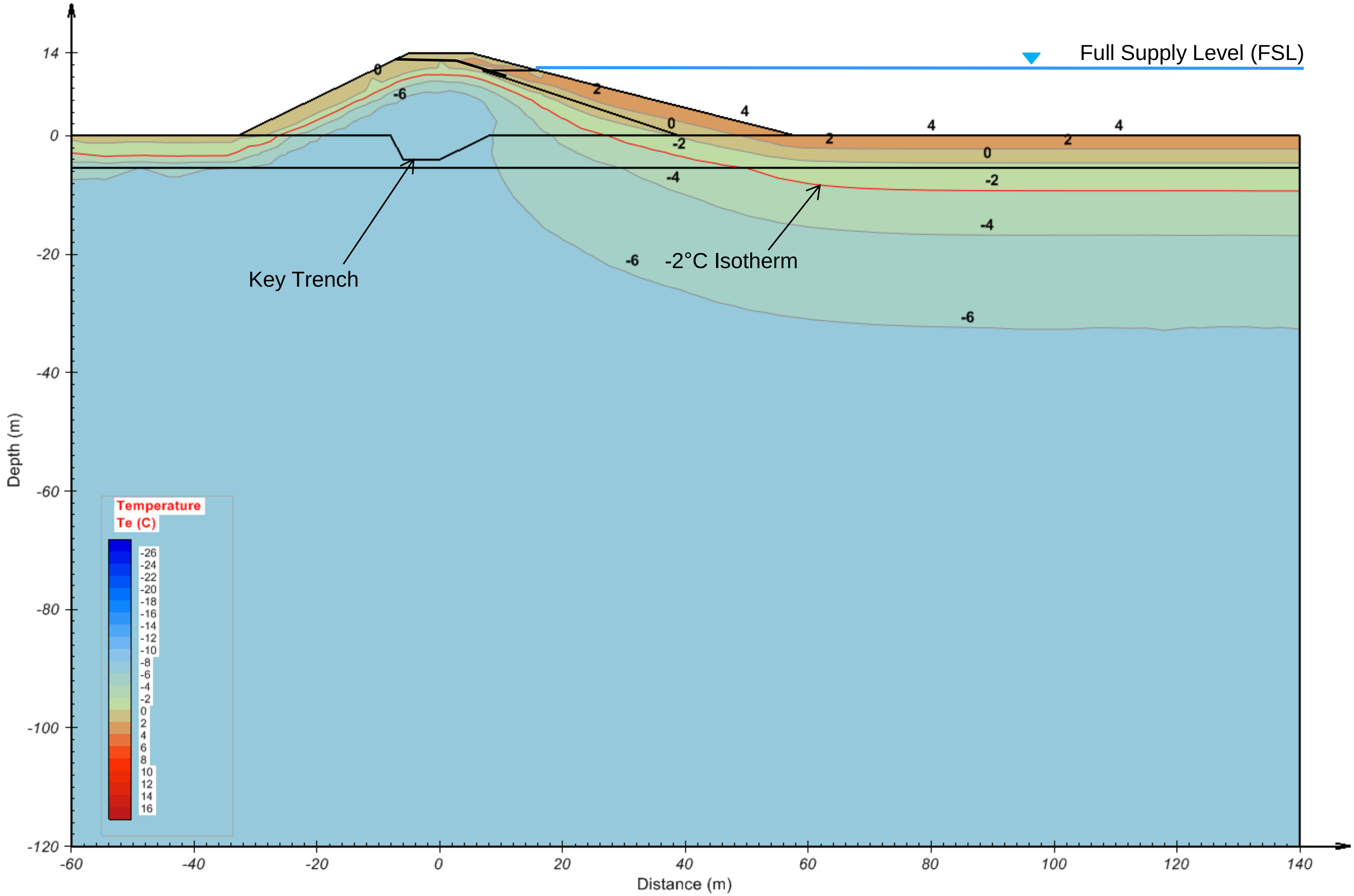
TSF Dam Thermal Model

Thermal Model Geometry - Section 0+300 (Year 0 – 3)

Date:	Approved:	Figure:
10/4/2015	CWS	2



srk consulting	Sabina GOLD & SILVER CORP	TSF Dam Thermal Model
		Thermal Model Geometry - Section 0+300 (Year 3 – 25)
Job No: 1CS020.008 Filename: TSF_Model_GeometrySection0+300_Yr3-25.pptx	BACK RIVER PROJECT	Date: 10/4/2015 · Approved: cws · Figure: 3



Note:

1. Model section represents maximum position of -2°C isotherm during Year 3

srk consulting

Sabina GOLD & SILVER CORP.

TSF Dam Thermal Model

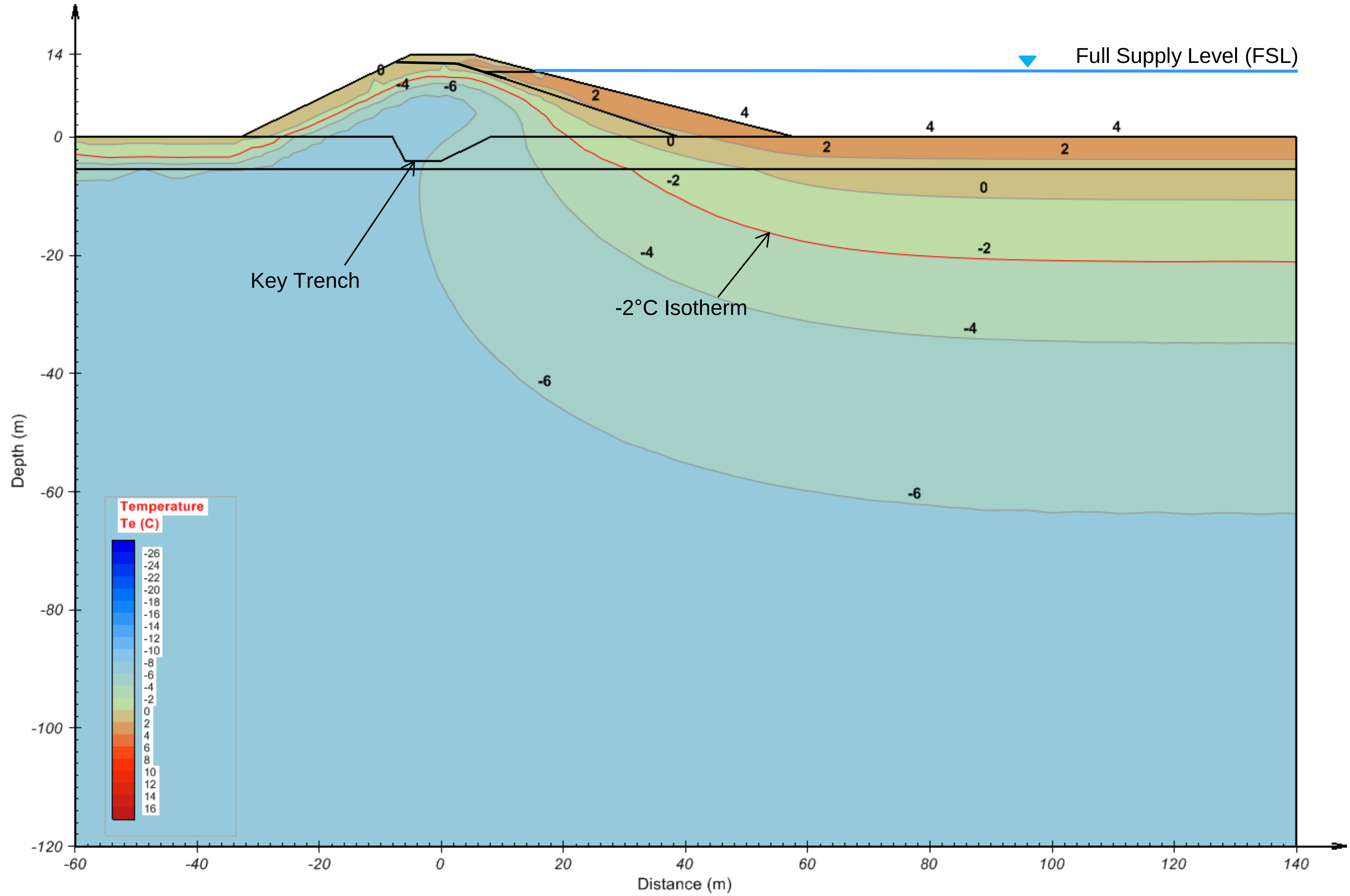
Thermal Model Results – Section 0+900 (Year 3)

Job No: 1CS020.008

Filename: TSF Model ResultsSection0+900Yr3.pptx

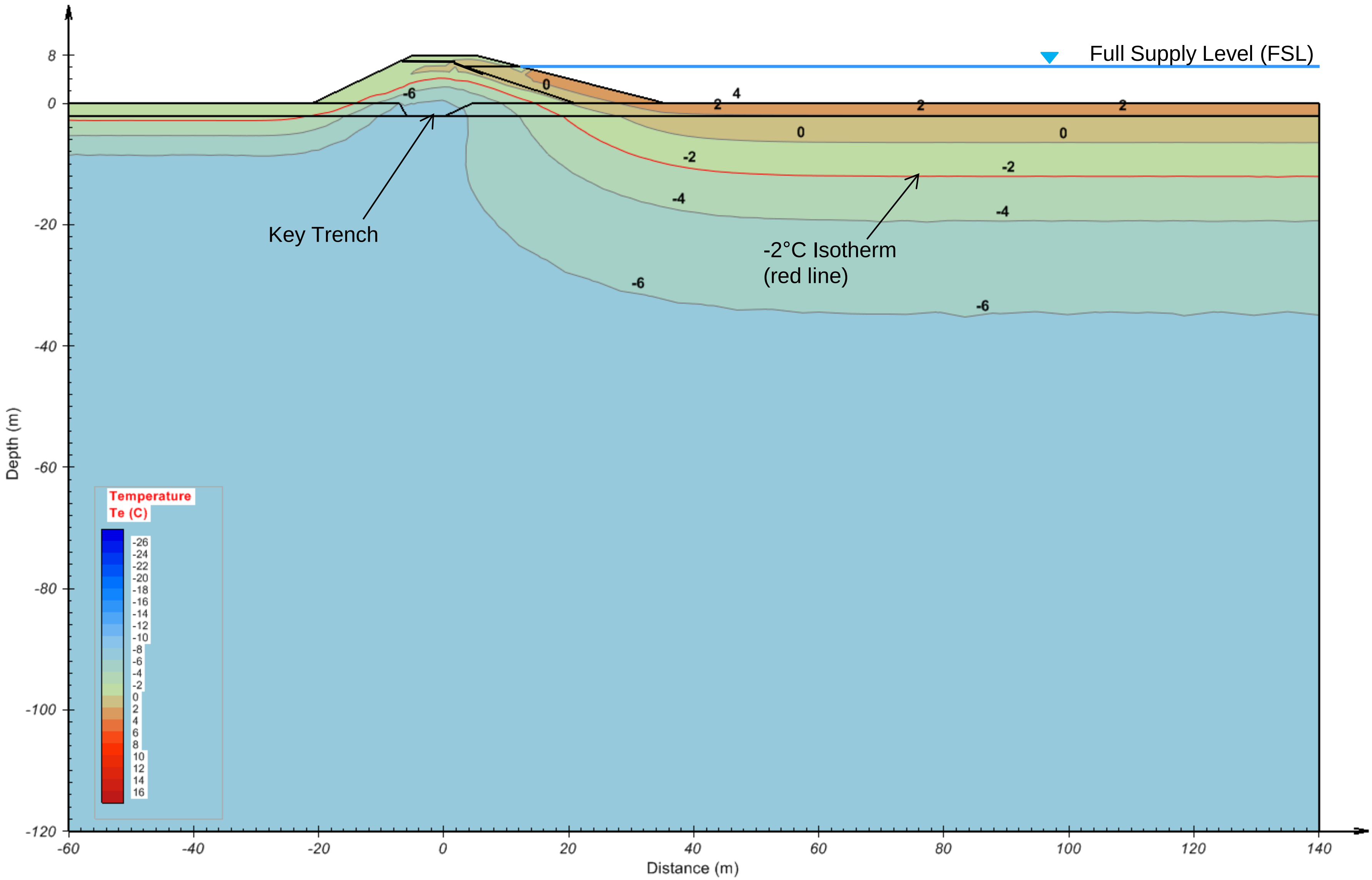
BACK RIVER PROJECT

Date:	Approved:	Figure:
9/17/2015	cws	4



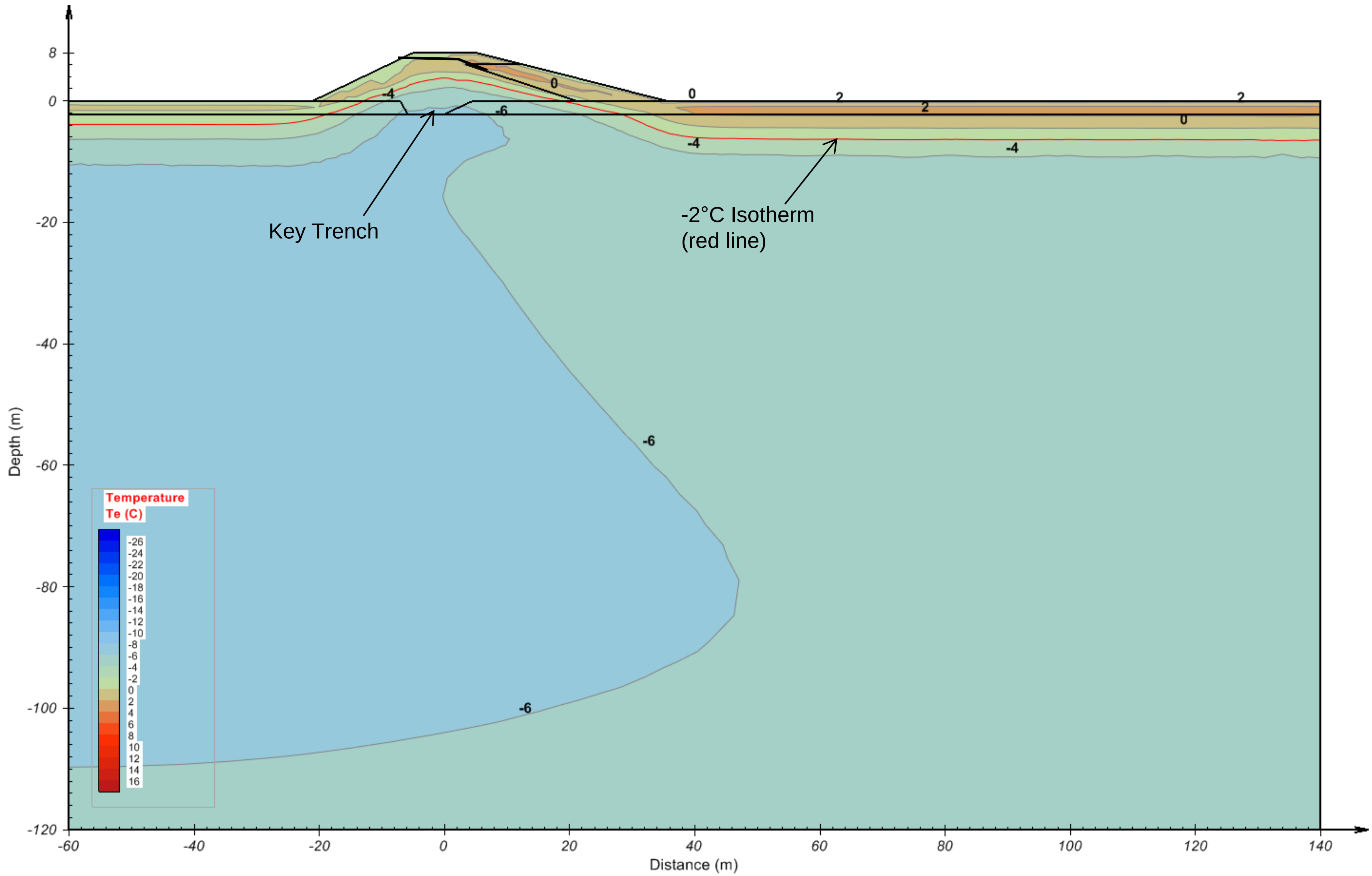
Note:
1. Model section represents maximum position of -2°C isotherm during Year 10

srk consulting	Sabina GOLD & SILVER CORP	TSF Dam Thermal Model
		Thermal Model Results – Section 0+900 (Year 10)
Job No: 1CS020.008 Filename: TSF Model ResultsSection0+900Yr10.pptx	BACK RIVER PROJECT	Date: 10/4/2015 | Approved: cws | Figure: 5



Note:
1. Model section represents maximum position of -2°C isotherm during year 3

srk consulting	Sabina GOLD & SILVER CORP	TSF Dam Thermal Model
		Thermal Model Results – Section 0+300 (Year 3)
Job No: 1CS020.008 Filename: TSF Model Results Section0+300End Yr3.pptx	BACK RIVER PROJECT	Date: 10/4/2015 | Approved: cws | Figure: 6



Note:

1. Model section represents maximum position of -2°C isotherm during Year 25

srk consulting

Sabina GOLD & SILVER CORP

Job No: 1CS020.008

Filename: TSF Model Results Section0+300End Yr25.pptx

BACK RIVER PROJECT

TSF Dam Thermal Model

Thermal Model Results – Section 0+300 (Year 25)

Date:	Approved:	Figure:
10/4/2015	cws	7

Appendix H - Tailings Consolidation Modeling

SRK Consulting (Canada) Inc.
2200–1066 West Hastings Street
Vancouver, BC V6E 3X2

T: +1.604.681.4196
F: +1.604.687.5532

vancouver@srk.com
www.srk.com

Memo

To:	Project File	**Client:**	Sabina Gold & Silver Corp.
From:	Sam Amiralaei, EIT Iozsef Miskolczi, PEng	**Project No:**	1CS020.008
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	September 28, 2015
Subject:	Tailings Consolidation Modeling – Final		

1 Introduction

As part of the larger Final Environmental Impact Study (FEIS) for the Back River Project (the Project) in Nunavut, SRK Consulting (Canada) Inc. was retained by Sabina Gold & Silver Corp. to complete the preliminary design of the Tailings Management System (TMS) for the Project and its associated containment dam.

The current tailings management plan makes provision for disposal of tailings in a purpose-built Tailings Storage Facility (TSF) for the first two years of mill production. At that point tailings disposition will transition to in-pit tailings deposition using the available mined out pits. Umwelt open pit (Umwelt TF) will first be used and once it has reached capacity, deposition will transition to Goose Main open pit (Goose Main TF). The majority of the tailings (over 75%) will be disposed of in these two open pits.

This memo documents the results of the tailings consolidation modelling for the Project. The purpose of completing this modelling was to establish the total tailings consolidation and density profile with respect to depth of the tailings at the end of in-pit tailings deposition.

2 Methods

2.1 Modelling Code

The commercially available code, FS Consol, distributed by GWP Software Inc., was used to construct and run the models. The code solves the material state equations defined by the finite strain consolidation theory (GWP 2007).

The models calculate the void ratio of the deposited tailings based on the effective stresses estimated from the height of the tailings column at each time step. The final output is a series of point values calculated for each time step.

2.2 Conceptual Model

Consolidation modelling was carried out using the High Level Mine Plan prior to finalization of the feasibility study mine plan and associated tailings deposition plan. During this early stage, consideration was given to using Llama, Umwelt and Goose Main open pits for deposition for a combined period of about nine years, with each pit being used on average for a period of about three years.

Table 1 shows the average deposition rate, total deposition duration, maximum deposition depth and model surface area for each of the three pits. The method of determining the deposition depth and deposition area is discussed in the following section.

Table 1: Model Parameters

Location	Average Deposition Rate (TPD)	Deposition Duration (years)	Maximum Depth (m)	Modeled Area (m^2)
Goose Main Pit	1,820	3.26	136.0	52,706
Llama Pit	5,970	2.61	94.0	51,715
Umwelt Pit	6,000	3.13	128.5	49,140

Source: \\van-svr0\Projects\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Tailings Consolidation\Memo\Tables

2.3 Input Parameters

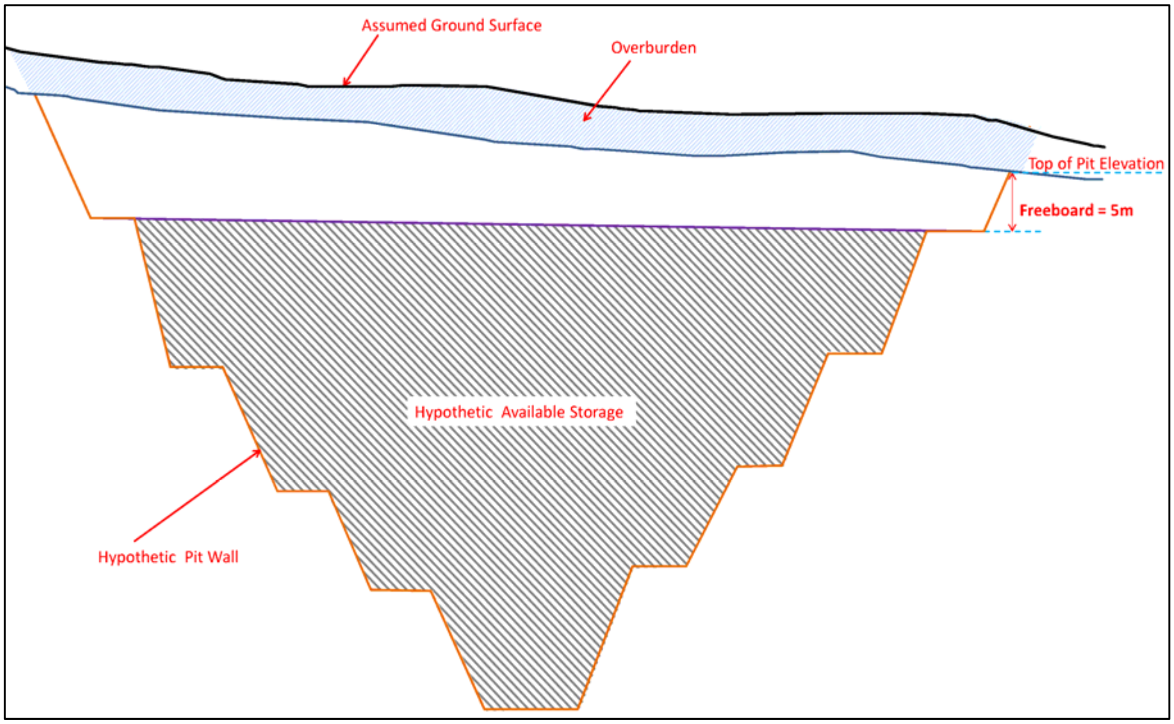
2.3.1 Model Geometry

The total available storage volumes of the open pits were determined by making the following assumptions:

- The top elevation of the pit was considered at the location where the overburden was at its lowest elevation (Figure 1).
- The thickness of the overburden was obtained from the findings presented in the overburden geotechnical investigation report (SRK 2015a).
- 5 m of freeboard was considered for each of the open pits.
- The available storage volumes were calculated based on the High Level Mine Plan pit shell models.

The maximum depth of tailings deposition was calculated by corroborating the lowest elevation in the pit shell and the elevation of the freeboard level for each pit. The geometry of the pits is greatly simplified to consider a cylinder having the height equal to the maximum deposition depth and a footprint that was calculated by dividing the available storage capacity to the maximum deposition depth.

The mass/volume balance of the tailings is calculated based on the volume of the cylinder filled at the specified deposition rates. Being a one-dimensional problem, the model solves the equations assuming a zero-width column in the middle of the deposit, with the elevations determined based on the specified geometry and fill rates.



Source: \\van-svr0\Projects\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Tailings Consolidation\Memo\Figures\BackRiver_TailingsConsolidationMemo_Figure2_SA

Figure 1: Typical Pit Storage Assumptions

2.3.2 Material Properties

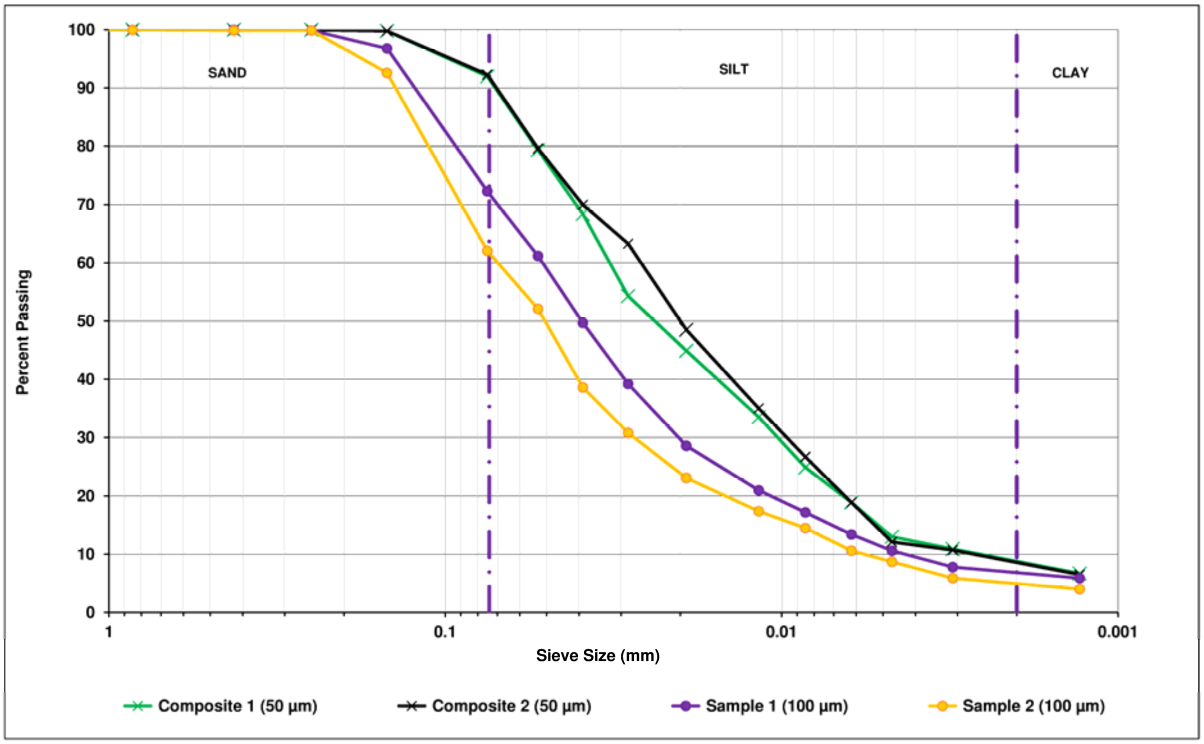
Two sets of tailings properties are available and described in the tailings physical characteristics memo (SRK 2015b). The first set represents extensive testing performed on tailings samples with a grind of 100 µm (Knight Piésold 2013), while the second one is a set of limited testing performed on tailings with 50 µm grind. Although the 50 µm grind is the more representative of the two, the current model is based on the extensive testing results of the 100 µm tailings because the properties required for the model setup were not determined for the 50 µm tailings.

Table 2 provides a summary of select tailings properties while Figure 2 shows a compilation of the particle size distributions for the 50 and 100 µm tailings.

Table 2: Summary of Tailings Geotechnical Properties Obtained from Laboratory Test Results (SRK 2015)

	Sample ID	Atterberg Limits	Solids Content (%)	Specific Gravity	Dry Density (kg/m^3)	Hydraulic Conductivity Range (m/sec)
100 Micron Grind	Test 1	Non-Plastic	48	3.15	700	$4 \times 10^{-7} - 6 \times 10^{-7}$
	Test 2	Non-Plastic	47	3.17	660	$2 \times 10^{-7} - 7 \times 10^{-7}$
50 Micron Grind	Composite 1	Non-Plastic	50	2.87	740	-
	Composite 2	Non-Plastic	50	2.88	740	-

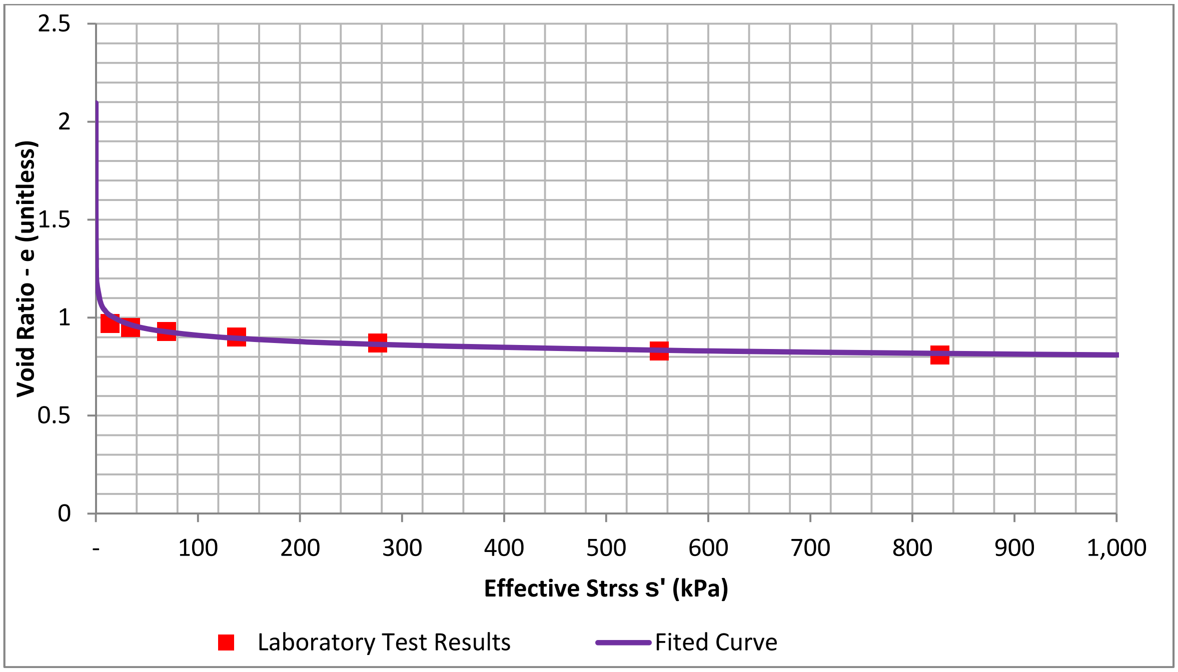
Source: \\van-svr0\Projects\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Tailings Consolidation\Memo\Tables



Source: \\van-svr0\Projects\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\TailingsConsolidation\Memo\Figures\BackRiver_TailingsConsolidationMemo_Figure1_SA

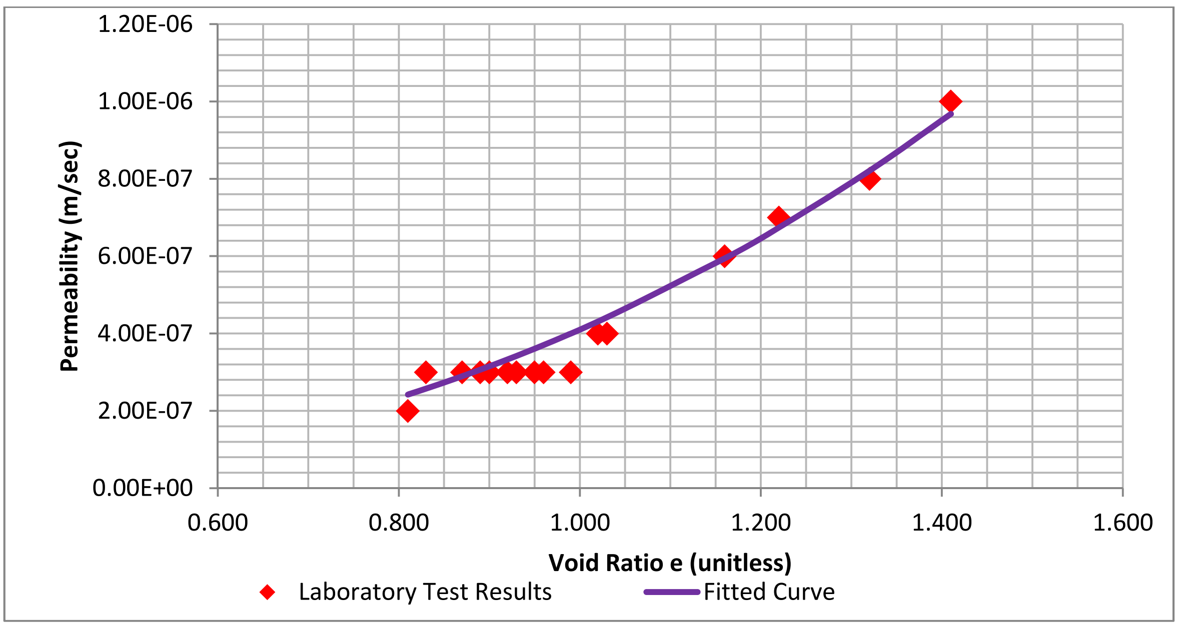
Figure 2: Particle Size Distribution of the Tailings Samples

The material properties required as inputs were extracted from the laboratory test results for the 100 µm tailings, while the fitting parameters for extending the curve past the tested stress range were obtained using the Solver add-in for Microsoft Excel. The variables (A, B, and M for void ratio; C and D for permeability, respectively) were calculated applying the minimum quadratic error between the actual values (laboratory results) and calculated values for the tested range of stresses. The solver in the FS Consol then applies the fitting parameters over the entire range of stresses encountered within the modelled geometry, as illustrated in Figures 3 and 4.



Source: \\van-svr0\Projects\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Tailings Consolidation\Memo\Figures\BackRiver_TailingsConsolidationMemo_Figure3_4SA

Figure 3: Best Fit Void Ratio versus Effective Stress Curve



Source: \\van-svr0\Projects\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Tailings Consolidation\Memo\Figures\BackRiver_TailingsConsolidationMemo_Figure3_4SA

Figure 4: Best Fit Permeability versus Void Ratio

2.4 Initial Condition

A tailings deposit height equal to zero was chosen as the initial condition, to simulate the deposition in a vacant open-pit mine. The initial solids content of the tailings was set at 49% (by mass).

2.5 Boundary Condition

The upper boundary condition was set as a constant water cap with a depth equal to zero, essentially simulating the water table at the surface of the deposited tailings, without a pond. The bottom boundary was set as impermeable, meaning that the no fluid can seep through the bottom of the pit. No side boundaries are specified, the model being one-dimensional.

2.6 Output Parameters

To support the analysis of the model, at each time step the software is tracking the following parameters:

- Solids content;
- Void ratio;
- Effective stress;
- Excess pore pressure ;
- Permeability;
- Maximum depth of deposition; and
- Total settlement.

The output parameters are provided at several discharged tailings depth and time increments. The user can adjust the depth and time increments which indeed will affect the accuracy of the final results. For analyses that were completed in this report, the smallest possible depth and time increments (2.5 m and 15 days, respectively) were used while ensuring the models would not become unstable.

3 Results

Tables 3 and 4 summarize the results of the analysis that were obtained for each model. The output parameters that are outlined in Tables 3 and 4 represent the values for the entire depth of the deposition at the end of deposition period.

Table 3: Summary of the Solids Content and Dry Densities at the End of Deposition Period

Location	Solids Content (%)		Dry Density (kg/m^3)		
	Average	Max	Min	Average	Max
Goose Main Open Pit	76.4	78.4	735	1,611	1,687
Llama Open Pit	77.4	79.2	735	1,653	1,725
Umwelt Open Pit	76.9	79.4	735	1,640	1,734

Source: \\van-svr0\Projects\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Tailings Consolidation\Memo\Tables

The dry density values shown in Table 3 are not an output from the model; they were calculated based on voids ratio using basic soil mechanics relationship equation (Equation 4.1).

$$\gamma_d = \frac{SG \times \rho w}{1+e} \qquad (4.1)$$

Where:

SG = Specific gravity

ρw = Water density

e = Void ratio

Table 4 presents the total maximum depth of tailings deposition based on the average pit surface area that was inputted into the model (Table 1). Table 4 also shows the total time required for the tailings material to reach complete settlement after the end of deposition.

Table 4: Summary of Maximum Depth Deposition and Total Settlement

Location	Maximum Depth of Deposition[1] (m)	Total Settlement (m)	Total Settlement As Percentage of Total Depth	Time to complete Consolidation[2] (Day)
Goose Main Open Pit	25	0.06	0.2 %	47
Llama Open Pit	65	0.43	0.7 %	185
Umwelt Open Pit	83	0.52	0.6 %	298

Source: \\van-svr0\Projects\01_SITES\Back River\1CS020.006_FS_Study\!080_Deliverables\TSF Design Report\030_Appendices\Tailings Consolidation\Memo\Tables

Note:

(1) Maximum depth of deposition at the end of deposition period

(2) After the end of deposition period

Figures 5 to 7 show the profiles of the solids content and dry density versus deposition depth for the Goose Main, Llama, and Umwelt pits. These figures show the profiles at the end of the deposition period, at each of the tailings storage facilities.

Source: \\van-svr0\Projects\01_SITES\BackRiver\1CS020.006_FS_Study\!080_Deliverables\TSFDesign Report\030_Appendices\TailingsConsolidation\Memo\Figures\BackRiver_TailingsConsolidationMemo_Figure5_6SA

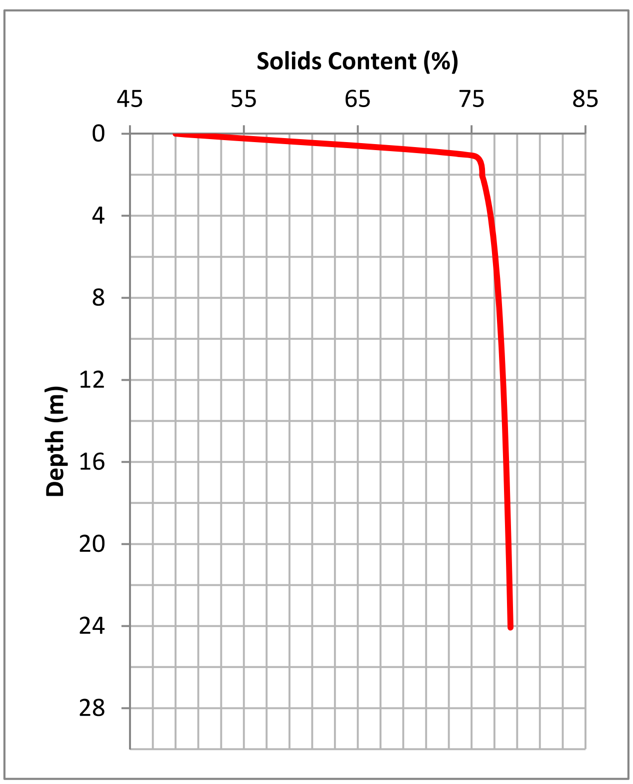


Figure 5a: Goose Main Pit Solids Content with Depth at End of Deposition (3.26 Years)

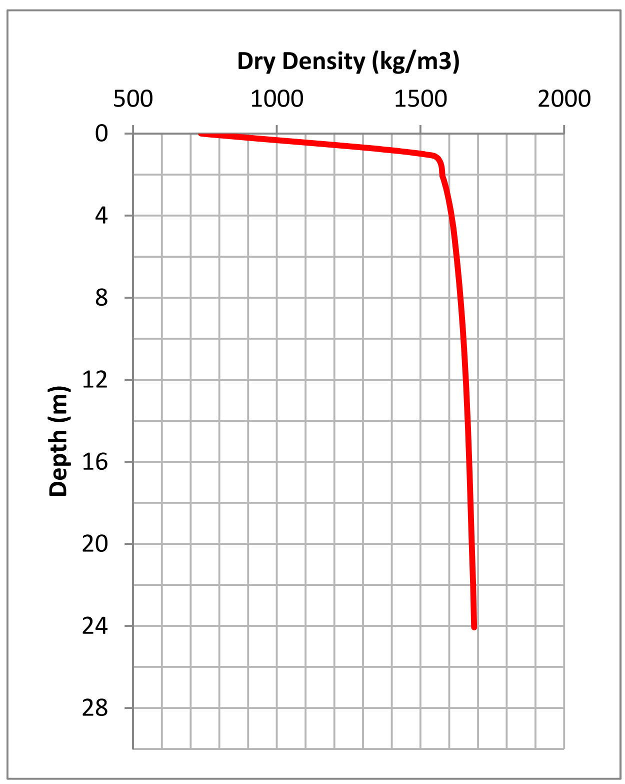


Figure 5b: Goose Main Pit Dry Density with Depth at End of Deposition (3.26 Years)

Source: \\van-svr0\Projects\01_SITES\BackRiver\1CS020.006_FS_Study\!080_Deliverables\TSFDesign Report\030_Appendices\TailingsConsolidation\Memo\Figures\BackRiver_TailingsConsolidationMemo_Figure5_6SA

Source: \\van-svr0\Projects\01_SITES\BackRiver\1CS020.006_FS_Study\!080_Deliverables\TSFDesign Report\030_Appendices\TailingsConsolidationMemo\Figures\BackRiver_TailingsConsolidationMemo_Figure7_8SA

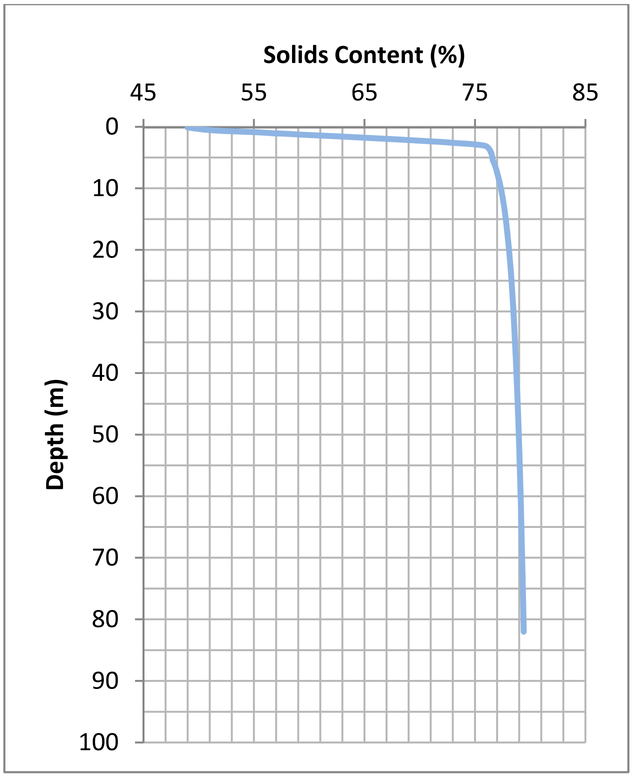


Figure 6a: Llama Pit Solids Content with Depth at End of Deposition (2.61 Years)

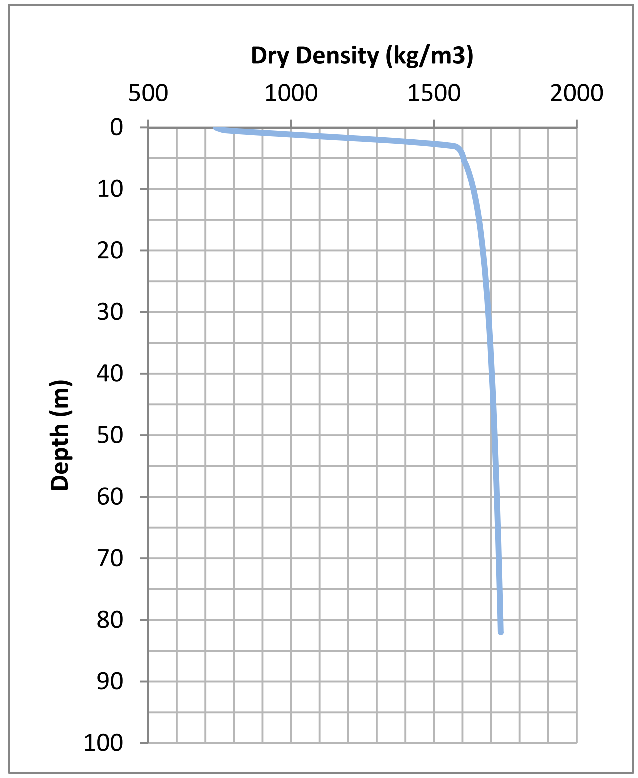


Figure 6b: Llama Pit Dry Density with Depth at End of Deposition (2.61 Years)

Source: \\van-svr0\Projects\01_SITES\BackRiver\1CS020.006_FS_Study\!080_Deliverables\TSFDesign Report\030_Appendices\TailingsConsolidationMemo\Figures\BackRiver_TailingsConsolidationMemo_Figure7_8SA

Source: \\van-svr0\Projects\01_SITES\BackRiver\1CS020.006_FS_Study\!080_Deliverables\TSFDesign Report\030_Appendices\TailingsConsolidationMemo\Figures\BackRiver_TailingsConsolidationMemo_Figure9_10SA

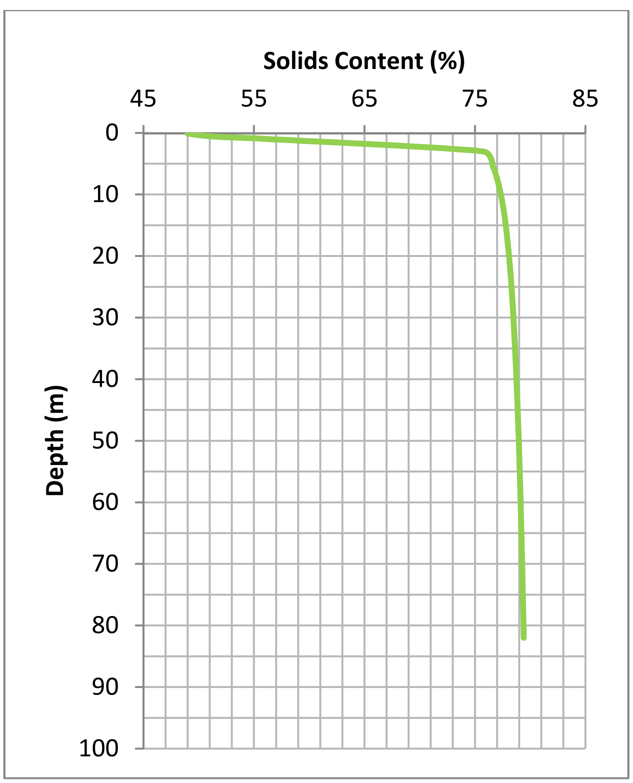


Figure 7a: Umwelt Pit Solids Content with Depth at End of Deposition (3.13 Years)

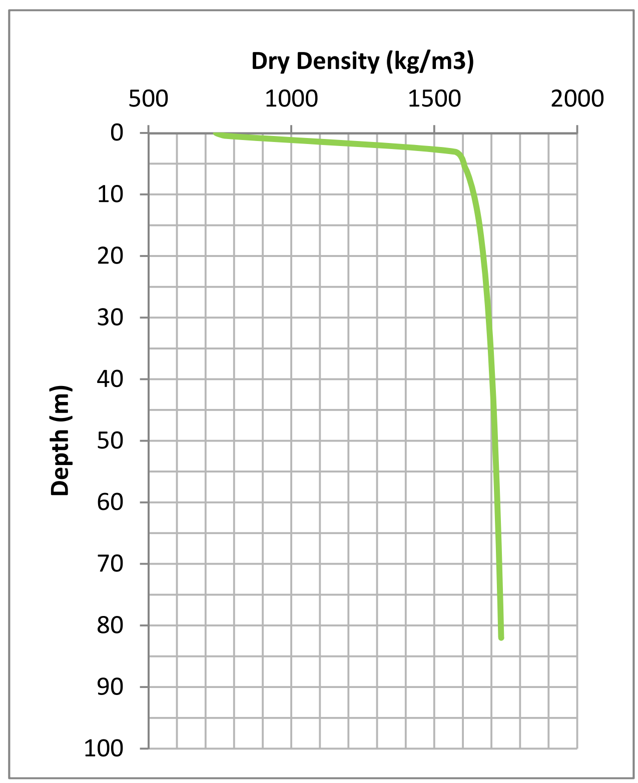


Figure 7b: Umwelt Pit Dry Density with Depth at End of Deposition (3.13 Years)

Source: \\van-svr0\Projects\01_SITES\BackRiver\1CS020.006_FS_Study\!080_Deliverables\TSFDesign Report\030_Appendices\TailingsConsolidationMemo\Figures\BackRiver_TailingsConsolidationMemo_Figure7_8SA

4 Discussion

As shown in Table 3, the final dry density ranges between 1.6 and 1.7 t/m^3 which is in line with the laboratory test results. The maximum solids content (at the bottom of the pits) ranges from 78 to 79 percent. The maximum solids content values are relatively high, but are also in line with the laboratory test results.

As illustrated in Figures 5 through 7, the curve shapes for solids content and density are relatively flat at very shallow depths (top 5 m), but steepen sharply. This indicates that the solids content and dry density are at their lowest values at the top (freshly discharged tailings) and increase with increasing tailings depth, indicating that about 7 m of tailings are required to achieve high solids contents.

As shown in Table 4, the total consolidation of the tailings material is completed in about 50 to 300 days after the end of tailings deposition.

As mentioned previously, due to lack of available laboratory data for 50 µm tailings the variables required to run the model could not be determined at this time. However, as shown on Figure 2, the two materials are in the same range of particle sizes, spanning the fine sand and silt range with little clay size particles. Due to this similarity and nearly identical fines content (D_{10} on PSD curves) the time to achieve complete consolidation and magnitude of consolidation will be similar. It is therefore SRK's opinion that the results presented here for the coarser tailings are a close enough approximation of the behavior of the 50 µm tailings.

5 References

GWP 2007. FS Consol Slurry Consolidation Software User Manual. GWP Software Inc. 2007.

Knight Piésold Consulting. (2013). Back River Project Report on Laboratory Geotechnical Testing of Tailings Materials Report, prepared for Sabina Gold and Silver Corporation, October.

SRK Consulting (Canada) Inc. (2015a). Back River Property Geotechnical Design Parameters. Report prepared for Sabina Gold and Silver Corp., Project No. 1CS020.006, April 2015.

SRK Consulting Canada Inc. (2015b). Tailings Physical Characteristics. Technical memorandum prepared for Sabina Gold and Silver Corp., Project No. 1CS020.006, April 14, 2015.

Appendix I - TSF Containment Dam Geotechnical Monitoring Instrumentation

SRK Consulting (Canada) Inc.
2200–1066 West Hastings Street
Vancouver, BC V6E 3X2

T: +1.604.681.4196
F: +1.604.687.5532

vancouver@srk.com
www.srk.com

Memo

To:	Project File	**Client:**	Sabina Gold & Silver Corp
From:	Emma Helmers, EIT Iozsef Miskolczi, PEng	**Project No:**	1CS020.008
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	September 27, 2015
Subject:	TSF Containment Dam Geotechnical Monitoring Instrumentation - Final		

1 Introduction

As part of the larger Final Environmental Impact Study (FEIS) for the Back River Project (the Project) in Nunavut, SRK Consulting (Canada) Inc. was retained by Sabina Gold & Silver Corp. to complete the preliminary design of the Tailings Management System (TMS) for the Project and its associated containment dam.

The proposed dam will be located at the Goose Property, about 2 km south of the Goose Main open pit. The facility will be operated as an active TSF for just over two years, after which a portion of it will be used for storage of contact water for an extended period. The dam design encompasses a relatively low profile structure with a maximum crest height of about 14 m, constructed primarily of run-of-mine (ROM) rock fill. The dam slopes will be 4H:1V on the upstream side and 2H:1V on the downstream, with a crest width of 10 m. An impermeable liner (Geosynthetic Clay Liner) will be incorporated into the structure of the dam. Tailings beach development will result in the supernatant pond being located directly against the dam structure.

This memo describes the type and location of the geotechnical instrumentation to be installed within the dam to monitor the performance of the structure.

2 Instrumentation

2.1 Concepts

Based on the feasibility level mine plan, the TSF will be operating for the first two years of production, after which a portion of it will be used for storage of contact water for an extended period. Based on this relatively short design life and operational window, thermal modelling confirmed that the dam foundation will not thaw a significant amount (SRK 2015).

The dam will experience deformation through thaw settlement and creep. Due to the short operational life of the structure these deformations are expected to be negligible; however, monitoring of the thermal regime is key in confirming these assumptions. Monitoring of the thermal regime is also paramount in ensuring that the geomembrane and the frozen key trench fill provide an adequate seal against undue foundation seepage.

To assess the magnitude of deformation, accurate survey monitoring is required. The instrumentation plan described in the following sections is illustrated in Figure 1.

2.2 Ground Temperature Cables

Ground temperature cables (thermistor strings) should be installed as detailed in Table 1. Each vertical cable should contain between 12 to 16 beads which must be spaced to allow monitoring within the dam, the permafrost foundation, and the bedrock. Two horizontal cables along the length of the dam will have beads spaced evenly along the length of the dam. All cables should be directed to a central data logger to allow continuous monitoring and automated data collection.

Table 1: Proposed Thermistor Locations

Thermistor	Chainage Along Dam (0+m)	Location Description	Length (m)	Number of Beads	Location of Beads along String (m from top)
Th-001	n/a	Downstream of the middle of the dam	50	16	1, 2, 3, 4, 6, 8, 10, 12, 14, 19, 24, 29, 34, 39, 44, 49
Th-002	0+100	Crest of dam through to bedrock	33	12	2, 6, 9, 11, 13, 15, 17, 19, 22, 25, 30, 33
TH-003	0+390	Crest of dam through to bedrock	35	12	2, 6, 12, 15, 17, 19, 21, 23, 26, 29, 32, 35
Th-004	0+550	Crest of dam through to bedrock	36	12	2, 7, 13, 16, 18, 20, 22, 24, 26, 29, 33, 36
Th-005	0+740	Crest of dam through to bedrock	37	12	2, 8, 14, 17, 19, 21, 23, 25, 27, 30, 34, 37
Th-006	0+840	Crest of dam through to bedrock	39	12	2, 9, 15, 19, 21, 23, 25, 27, 29, 32, 36, 39
Th-007	0+990	Crest of dam through to bedrock	39	12	2, 9, 15, 19, 21, 23, 25, 27, 29, 32, 36, 39
Th-008	1+160	Crest of dam through to bedrock	37	12	2, 8, 14, 17, 19, 21, 23, 25, 27, 30, 34, 37
Th-009	1+400	Crest of dam through to bedrock	36	12	2, 7, 13, 16, 18, 20, 22, 24, 26, 29, 33, 36
Th-010	1+550	Crest of dam through to bedrock	35	12	2, 6, 12, 15, 17, 19, 21, 23, 26, 29, 32, 35
Th-011	n/a	Along dam on upstream toe of key trench	1,700	68	Every 25 m
Th-012	n/a	Along dam on upstream crest of key trench	1,700	68	Every 25 m

Vertical ground temperature cables should be installed into boreholes drilled vertically into the dam fill from the downstream crest of the completed dam. This will ensure that the drilled holes are located downstream of the liner and do not compromise the liner integrity or create preferential flow paths through the dam foundation. Normally the drill holes should be backfilled with sand or similar thermally conductive material. Horizontal cables must be installed in a bedding layer of sand (or gravel).

2.3 Survey Prisms

Deformation of the dam should be directly monitored through regular surveys using survey prisms (or basic settlement pins) permanently installed along the crest and downstream slope of the dam. The proposed installation plan is as follows:

- At least every 25 m along the crest of the dam, on both the upstream and downstream sides of the crest in areas where the foundation conditions are changing (from 0+000 to 0+300 and from 1+575 to 1+775). Prism sets should be on either side of the foundation interfaces;
- Every 50 m along the crest of the dam, on both the upstream and downstream sides of the crest in areas where the foundation conditions are relatively constant (from 0+300 to 1+575); and
- Every 50 m along the downstream slope of the dam, half way down the slope from the crest, in areas where the height of the dam is greater than 10 m (from 0+425 to 1+525).

The survey prisms should be embedded in large boulders which are in turn embedded into the waste rock forming the crest and downstream shell. This will ensure that the prisms themselves are subject to minimal movement so that differential settlement measurements are as accurate as possible.

3 Monitoring

3.1 During Construction

Where practical, instrumentation will be installed as construction occurs and manual readings will need to be taken regularly in order to ensure they are in proper working order.

3.2 Post-Construction

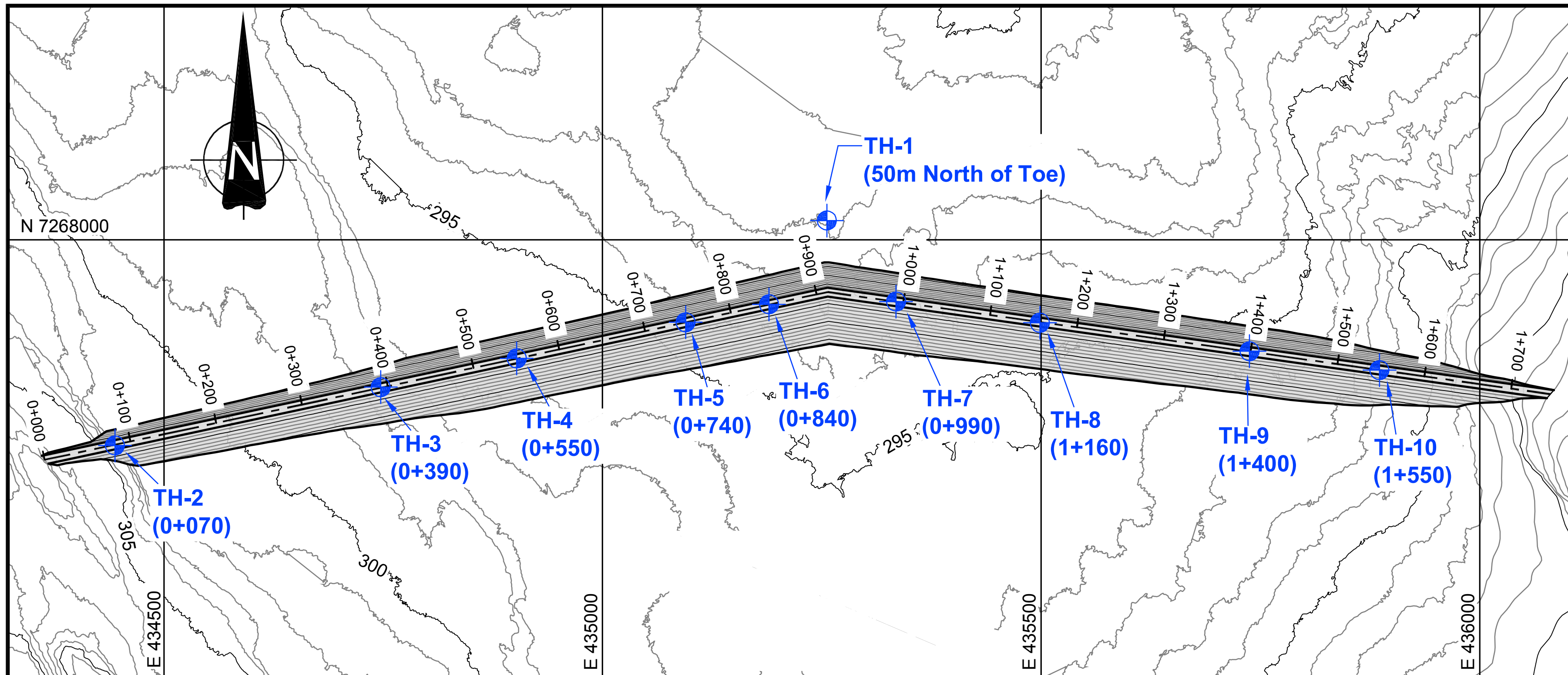
All ground temperature cables installed in the dam will be connected to data loggers in order to enable automatic recordings. The data loggers should be downloaded at regular intervals of one week or less.

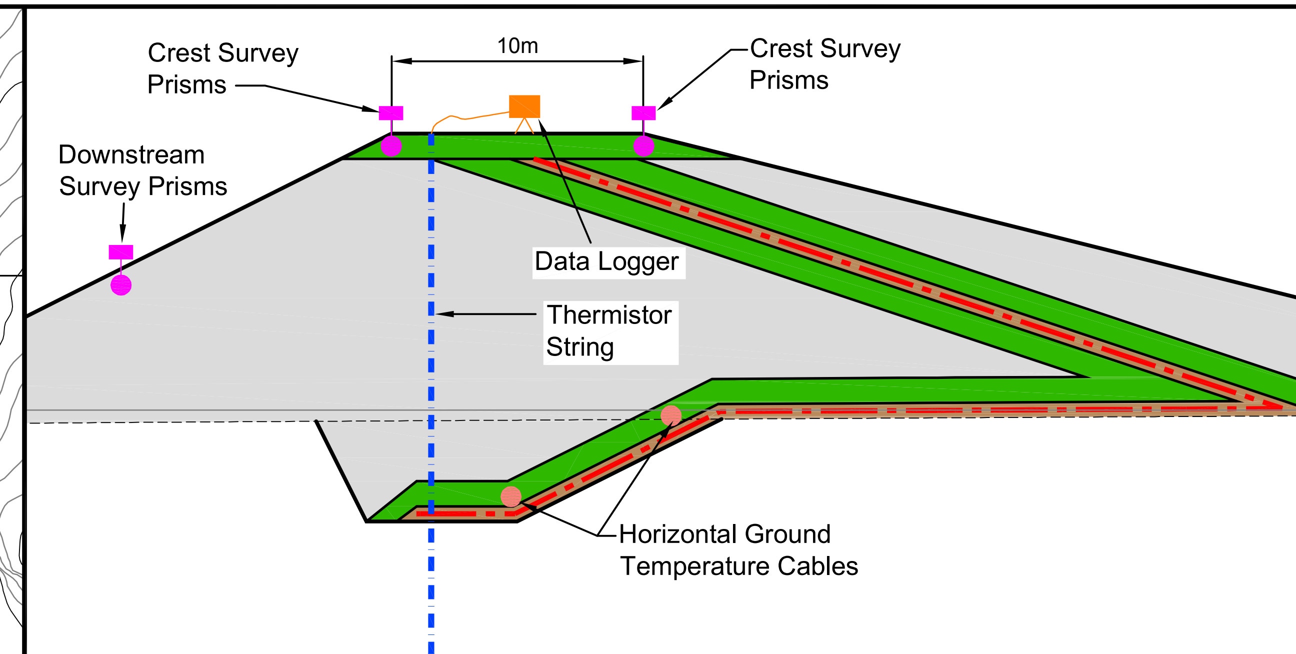
Surveys at regular intervals shall provide a record of the dam performance with respect to deformations. Surveys need to be carried out at monthly intervals until alternate monitoring instructions are provided by the Engineer-of-Record.

4 References

SRK Consulting (Canada) Inc. 2015. TSF Containment Dam Thermal Modeling. Technical memorandum prepared for Sabina Gold and Silver Corp. Project Number 1CS020.006, April 14, 2015.

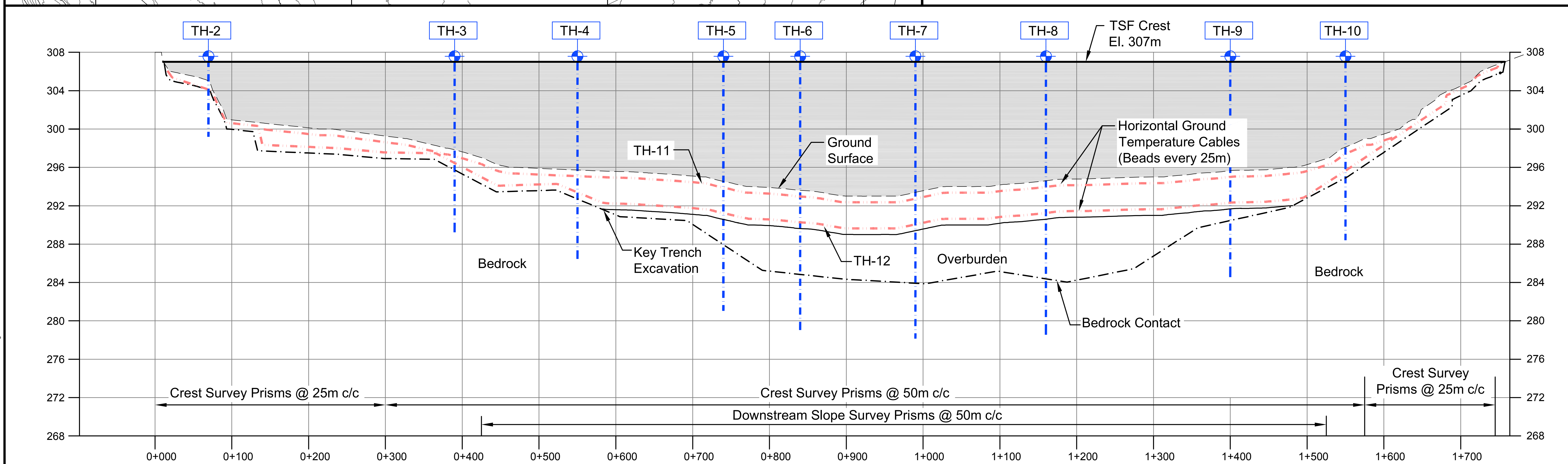
Figures





TYPICAL SECTION GROUND TEMPERATURE CABLE INSTALLATION

NTS



CENTRELINE TSF DAM PROFILE

LEGEND

- Ground Temperature Cable Location
- TSF Dam Fill

0m 50 100 150 200 (Horizontal Scale)

0m 4 8 12 16 (Vertical Scale)

srk consulting

Sabina GOLD & SILVER CORP.

TSF Containment Dam Instrumentation

Geotechnical Instrumentation Plan, Elevation and Section

SRK JOB NO.: 1CS020.008

FILE NAME: BR-TMSD FIGURE 13.dwg

BACK RIVER PROJECT

DATE: SEPT. 2015 | APPROVED: IM | FIGURE: 1

C:\MDDS\NC\CLIENTS\SRK\2014\BACK RIVER TAILINGS\DRAWINGS\FIGURES SEPT 2015\BR-TMSD FIGURE 13.dwg