Frozen Core Tailings Dam: Part 2, Long-Term Creep Deformation

M. Rykaart, A. Barrero, and A. Lizcano
SRK Consulting (Canada) Inc., Vancouver, British Columbia, Canada

ABSTRACT: Tailings management at the Hope Bay Project in Nunavut, Canada, includes reliance on an innovative frozen core dam. This dam does not have a tailings beach against it, and has been designed as a water retaining dam with a 30-year design life. The dam foundation is subject to significant long-term creep deformation. This paper, which is the second in a two-part series, describe the dam performance six years post construction, and compares modelled creep deformation with field performance data collected from shallow and deep settlement monitors, and inclinometers. The data and modelling confirm that creep deformation is less than originally anticipated during the design stage, and that continued long-term creep deformation of the dam will not result in excessive long-term strains that could impact the structure's performance. The first paper in this series describes the thermal performance of the dam, which is a key parameter driving creep deformation.

1 INTRODUCTION

The tailings management system for the Hope Bay Project, in Nunavut, Canada, includes subaerial tailings deposition into the Doris tailings impoundment area (Doris TIA). The Doris TIA is located in the basin of the former Tail Lake, a shallow lake which has been delisted in accordance with Schedule II of the Metal Mining Effluent Regulations. To provide containment, a frozen core water retaining dam (North Dam) was designed (SRK 2007, 2017a), and constructed over the winter seasons of 2011 and 2012 (SRK 2012). Following six years of monitoring, conclusions regarding dam performance can be made, both with respect to thermal response and creep deformation. This paper, which is Part 2 of a two-part series, describes measured deformation data, and associated updated creep deformation modeling, while Part 1 (Rykaart et al. 2018) describes the thermal performance monitoring and associated modeling. Section 1 through 6 of Part 1 and Part 2 is identical to allow the papers to be read independently.

2 TAILINGS MANAGEMENT SYSTEM

The Doris TIA, when fully developed, will consist of one frozen core water retaining dam (North Dam), and two frozen foundation dams (South Dam and West Dam). Tailings will be discharged from the South and West Dams, as well as select locations around the perimeter of the facility. The North Dam will provide water containment for the Reclaim Pond, and no tailings will be in contact with the dam during its 30-year design life. At closure, the Reclaim Pond will be drained, and the North Dam breached, with no need for a permanent water retaining dam (SRK 2017a).

3 FOUNDATION CONDITIONS

The project is located within the continuous permafrost region of Canada, approximately 140 km north of the Arctic Circle. Site measurements indicate that permafrost in the area is approximately 570 m thick, with an active layer of 0.9 m to 1.7 m, depending on the material type. The ground temperature at the depth of zero annual amplitude is -8°C, and the geothermal gradient is 0.021°C/m (SRK 2017b).

Numerous geotechnical characterization programs have been performed within the alignment of the North Dam, to characterize the foundation conditions. Programs included boreholes, test pits, in-situ hydraulic conductivity testing, installation of ground temperature cables (GTCs), long-term ground temperature monitoring (starting in 2002), percolation testing, geophysics, and both undisturbed and disturbed sample collection and laboratory testing (SRK 2017b).

The North Dam is located in a narrow valley approximately 200 m downstream of the northernmost extent of the former Tail Lake. The stratigraphy under the dam has two distinct zones; the southwest abutment is dominated by ice-saturated sand deposits 10 m to 15 m thick, overlain by up to 3 m of silt and clay, while the northeast abutment is dominated by ice-saturated marine clayey silt with a maximum thickness of 15 m. Excess ground ice, averaging 10 % to 30% by volume, occasionally as high as 50%, is also present on both abutments.

A thin layer of sand and gravel overlies the bedrock surface in the upper portions of the valley, and a peat unit was encountered near the center of the dam, in the area of the lake outflow. The average pore water salinity is 39 parts per thousand (ppt), with a freezing point depression of -2.2°C (SRK 2017c). During construction, an isolated hypersaline zone, with salinity values in excess of 90 ppt, was also encountered within the key trench (SRK 2012). Bedrock is generally competent basalt.

Figure 1 provides a generalized longitudinal section of the North Dam foundation conditions, as understood prior to percolation testing and key trench excavation.

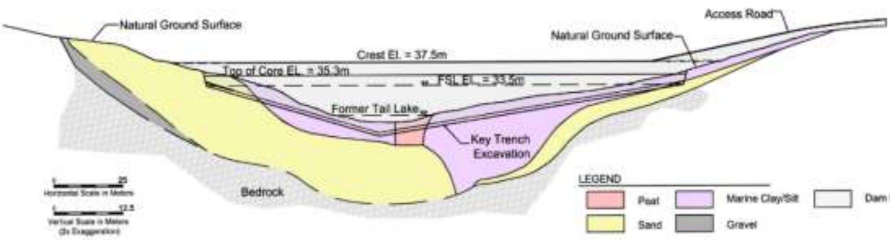

Figure 1. Longitudinal section of the North Dam showing generalized foundation conditions.

4 DAM DESIGN

A conventional unfrozen dam was not suitable for this location due to the thick ice-rich overburden foundation, remote site location, and lack of suitable low-permeability borrow material (Miller et al. 2013). Additionally, the thick overburden meant a frozen core dam founded on bedrock, similar to those constructed at other Canadian mines (Miller et al. 2013), was not feasible. Therefore, an innovative frozen core dam was designed to accommodate these challenging foundation conditions.

The North Dam is approximately 200 m long, with a maximum overall height of 10 m. The frozen ice-saturated core, keyed-in to the frozen foundation are designed to ensure water retention properties and containment. A geosynthetic clay liner (GCL) was installed along the upstream side of the frozen core to provide secondary water-retaining capability in case cracks develop in the core caused by thermal expansion, thermal erosion, differential settlement, or creep deformation. Thermal design to ensure primary containment requires that the frozen core maintains a temperature at or below -2°C, at a width that is at least twice the head of water impounded against the dam. In addition, the saline foundation needs to maintain a temperature at or below -8°C for the same width, extending to bedrock under normal operating conditions (SRK 2007).

The dam was uniquely designed for the saline ice-rich foundation that is particularly susceptible to creep deformation. Therefore, the dam was designed to accommodate long-term shear strains in the core and foundation approaching 2% and 10%, respectively, and maximum shear strain rates at or below 1.0E-05 sec^{-1} (3.2E+02 $year^{-1}$) (SRK 2007).

The dam design included a key trench ranging from 2 m to 5 m deep, to allow complete bonding of the core to the permafrost foundation (SRK 2012). Twelve sloped thermosyphons, six extending from each abutment of the dam, were installed at the base of the key trench to adequately cool the core and foundation over the design life, assuming water permanently impounded against the upstream face to full supply level, and with consideration for climate change. The typical design cross section for the North Dam is illustrated in Figure 2, and the equivalent as-built cross section is illustrated in Figure 3 for comparison.

The core material is comprised of sand-sized crushed basalt, placed in a near saturated state and allowed to freeze during construction. The core is surrounded by a transition layer, consisting of jaw crusher run rock that acts as a filter, should the dam thaw. An outer shell constructed of run-of-quarry rock acts as a thermal protection layer for the frozen core, provides buttressing against creep deformation, and provides ice and wave run-up protection.

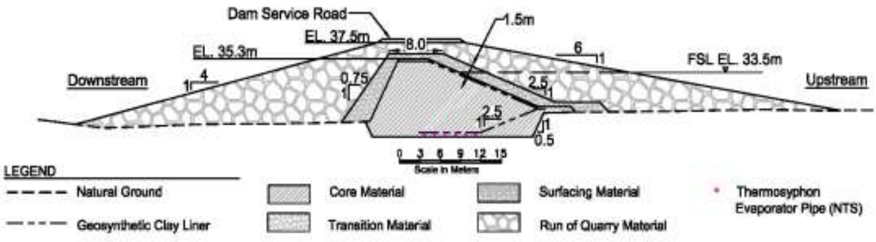

Figure 2. Typical design cross-section of the North Dam.

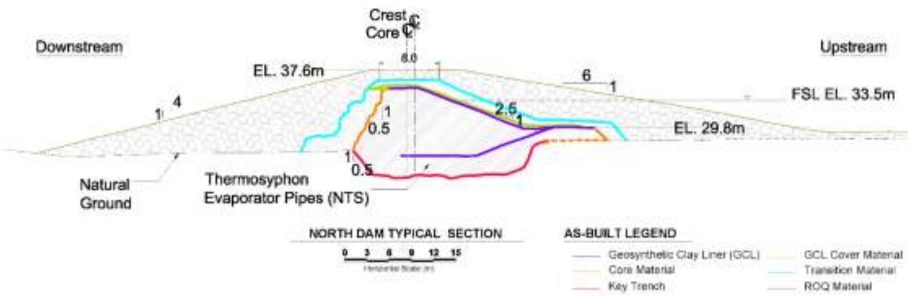

Figure 3. Typical as-built cross section of the North Dam.

5 CONSTRUCTION

The North Dam was designed to be constructed using construction techniques similar to those employed for the frozen core dams constructed at other Canadian mines (Miller et al. 2013). However, the site climate, available construction material, and foundation conditions necessitated adaptations to the construction method (SRK 2012); some of which are summarized in Kurylo et al. (2013). Details on lessons learned from the North Dam construction are presented in Miller et al. (2013).

Drilling and blasting were used to excavate the key trench. Key trench excavation included the removal of all peat from the center portion of the dam, and over-excavation of the hypersaline zone. Following completion of the key trench excavation, the frozen core was constructed by placing and compacting 0.2 m to 0.3 m thick lifts of core material. Core material was produced in a modified asphalt plant by mixing and heating crushed rock and water. The lifts of saturated hot core material were then left to freeze back to target temperature (at or below -2°C) prior to the placement of the next lift.

Core placement required stringent quality control and quality assurance to ensure that saturation, freeze-back, density, and material specification requirements were met. Quality control and quality assurance included continuous material testing (laboratory and field), visual inspections, and constant and open communication with the client and contractor. These procedures ensured freeze-back, saturation (average 85% or greater, with no test below 80%), and compaction requirements (90% or greater of standard Proctor) were met. Between each lift, loose material and snow was cleared. To ensure that the lifts would freeze within a reasonable timeframe, frozen core placement was not attempted when ambient air temperatures were warmer than -10°C.

Due to the larger than expected excavation required to remove the peat and hypersaline soils, and a warm spring, the dam could not be completed in one winter season as originally planned. At the end of the 2011 construction season, the lower GCL and horizontal thermosyphons were installed and covered; however, the frozen core was not completed. To protect the core during the subsequent summer, a temporary 2 m thick run-of-quarry cover was placed over the partially constructed core. Work recommenced in the winter of 2012, and the dam was substantially complete by April 2012. The last remaining instrumentation on the downstream side of the dam was installed in August 2012 (SRK 2012).

6 INSTRUMENTATION

Performance of the North Dam is dependent on the core and foundation maintaining design temperatures, and the long-term strains, and strain rates remaining below target limits (SRK 2012, 2018a). Thermal performance of the dam is monitored with a series of thirteen horizontal GTCs, and eleven vertical GTCs. Figure 4 provides a typical dam cross section, including the location of these GTCs. Three horizontal GTCs are positioned at the top, middle and base of the core; while vertical GTCs are positioned along the foundation on the upstream side, in the center of the dam (key trench) and on the downstream side.

The working condition of each thermosyphon unit is confirmed using a single bead thermistor attached to the radiator riser pipe and compared with ambient air temperature measurements collected over the same period of time.

Deformations are monitored by a series of surficial, shallow, and deep settlement points and six inclinometers. Eighteen surficial settlement points are located on the downstream face of the dam, to monitor deformation of the dam shell in the location of the greatest expected deformation. Three deep settlement points are used to monitor deformation of foundation soils, again at the location of the greatest expected deformation. Fourteen shallow settlement monitoring points were installed on the crest of the dam to monitor crest deformation and differential settlement. The inclinometers are installed along the location of the greatest expected deformation and monitor deformation of both the dam shell and foundation. The instrumentation layout at the area of greatest expected deformation can be seen in Figure 4.

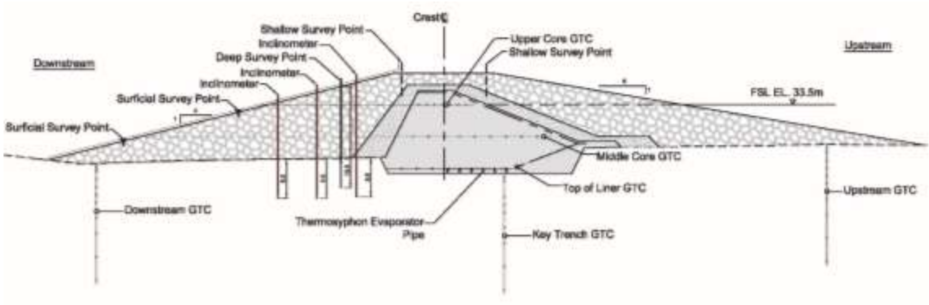

Figure 4. North Dam typical instrumentation.

7 DEFORMATION MONITORING DATA

North Dam monitoring include ground temperature measurements recorded every six hours with data loggers, and monthly manual deformation measurements (i.e. survey settlement and inclinometers). Additionally, ongoing visual inspection of the dam structure by Project staff, and a formal annual geotechnical inspection by a Professional geotechnical engineer, who is also the engineer-of-record, is conducted (SRK 2018a).

Annual inspections (SRK 2018b), and review of monitoring data, suggest the dam is performing in accordance with the design expectations. The North Dam has had impounded water since the first winter of construction in 2011. The operating water level impounded against the upstream face of the dam has averaged 29.8 m, with a maximum level of 30.2 m over the period from May 2011 to June 2018. The original water level of Tail Lake prior to construction of the North Dam was 28.3 m, and the design full supply level is 33.5 m. Core and foundation temperatures have been below the design temperatures of -2°C and -8°C, respectively (Rykaart et al. 2018).

Tables 1 and 2 summarize displacement and deformation data collected between August 2012 and June 2018. Surficial settlement as measured by the downstream dam shell settlement points (Table 1) show the greatest overall displacement. However, much of this occurred soon after construction and this probably largely reflects natural consolidation settlement of the run-of-quarry dam fill material used to construct the dam shell.

Both vertical and horizontal settlement of the dam crest (Table 1) is similar, ranging between 0.02 m and 0.08 m, with the data suggesting no real change over the most recent two years of monitoring. The deep settlement points (Table 1), which provides an indicator of foundation settlement, shows similar overall horizontal deformation as the crest but approximately half of the vertical displacement as compared to the crest.

The inclinometer measurements (Table 2) show similar deformations as measured by the deep- and crest-settlement points (Table 1) and confirm that the most deformation occurred in the upper part of the dam shell. Foundation deformation as determined by the lower sections of the inclinometers show very little movement at this point in time, suggesting minimal foundation creep has occurred.

Table 1. Overall vertical (downward) and horizontal displacements measured with settlement points (August 2012 through September 2017).

Location	No. of stations	Vertical Displacement (m)			Horizontal Displacement (m)		
		Min.	Max.	Avg.	Min.	Max.	Avg.
Deep settlement points	3	0.02	0.04	0.03	0.01	0.08	0.05
Crest settlement points	14	0.03	0.08	0.05	0.02	0.07	0.04
Downstream dam shell settlement points	18	0.03	0.21	0.11	0.01	0.25	0.07

Table 2. Maximum deformation measured with inclinometers (August 2012 through June 2018).

Inclinometer	Overall Maximum Deformation (m)	Depth Below Dam Surface (m)	Height Above Foundation (m)	Maximum Foundation Deformation (m)
070-1	0.023	1.0	8.5	0.002
070-2	0.026	2.5	6.5	0.017
070-3	0.013	3.5	4.0	0.002
120-1	0.011	0.5	6.5	0.009
120-2	0.022	0.5	5.5	0.005
120-3	0.017	0.5	2.7	0.002

8 CREEP DEFORMATION ANALYSIS

8.1 Model Setup

Creep deformations were re-assessed post-construction (SRK 2017d) by plane strain conditions using the two-dimensional nonlinear finite difference code, Fast Lagrangian Analysis of Continua (FLAC 2-D), by Itasca (2012). The analysis was carried out along the thickest as-built cross section of the dam (Figure 2). Thermal modelling was completed for the same cross section (SRK 2017c, Rykaart et al. 2018). Five material regions were considered in the deformation model: shell, transition zone, core, foundation, and bedrock. The GCL liner was not represented in the model as it has no structural significance.

8.2 Assessment Approach

The ice-saturated granular material in the frozen core is a dense material with a void ratio around 0.35 (SRK 2017b). The deviatoric strength (peak or residual) of the frozen core material is expected to be above 1 MPa based on published results from laboratory tests conducted on frozen sand samples under constant temperatures around -5°C, and constant strain rates around 1E-07 sec^{-1} (Bragg and Andersland 1981, Arenson 2002). Considering the height of the North Dam, the level of deviatoric stresses within the frozen core is anticipated to be low relative to the expected deviatoric strength of the frozen material in the core. According to Andersland and Ladanyi (2004), medium to high-density ice-saturated sands under low stress levels exhibit only primary creep (i.e. decreasing strain rates). Therefore, the creep deformation analysis assumed the frozen core will exhibit only primary creep.

Secondary creep (i.e. constant creep strain rate) was assumed for the frozen marine clayey silt in the foundation. These type of soils exhibits a short primary-creep period, and a prolonged secondary-creep phase (Andersland and Ladanyi 2004).

Based on the Bailey-Norton law (Norton 1929, Bailey 1935), creep strain rates ($\dot{\varepsilon}$) of frozen soils due to deviatoric stresses ($\bar{\sigma}$) can be described by the following general equation:

$$\dot{\varepsilon} = (A\bar{\sigma}^n) \cdot mt^{m-1} \quad (1)$$

where A is a creep parameter that depends on soil type and temperature, n and m can be considered temperature independent parameters, and t is the elapsed time after load application.

Secondary creep is commonly described by Equation 1, with $m = 1$. In this case, the equation can be rewritten as:

$$\dot{\varepsilon} = A\bar{\sigma}^n \quad (2)$$

With Equation 2, frozen soils are always predicted to creep for any given deviatoric stresses. Even for very small stresses, frozen soil is predicted to creep. This may lead to overestimating actual long-term displacements. A threshold stress (σ_{th}) for frozen soils likely exists, as for metals (Norton 1929), below which creep cannot be measured and Equation 2 no longer applies. Equation 2, as most constitutive equations for creep, is however formulated without a threshold stress.

In the performed analysis, creep strains were evaluated using a constitutive relation represented by Equation 2 implemented in FLAC, described as "The Two-Components Power Law" (Itasca 2012). For the analysis, a temperature independent threshold stress of 30 kPa was selected for all frozen materials, based on published laboratory testing results (Ladanyi 1971, Nixon and Lem 1984, Wijeweera and Joshi 1991, Arenson 2002) and engineering judgment. No creep strains were predicted ($\dot{\varepsilon} = 0$) for $\bar{\sigma} < \sigma_{th} = 30$ kPa. This stress is considered to be low, relative to the expected peak deviatoric strength. The assumed stress was not a threshold for the deviatoric stresses as introduced by Norton (1929). In this latter case, the deviatoric stresses ($\bar{\sigma}$) in Equation 2 is reduced by σ_{th}, or $\dot{\varepsilon} = (\bar{\sigma} - \sigma_{th})^n$. Likely thresholds for other creep mechanisms in frozen soil (e.g. temperature) were not considered in the analysis. Equation 2 can therefore be written as follows:

$$\frac{\dot{\varepsilon}}{\dot{\varepsilon}_r} = \left(\frac{\bar{\sigma}}{\sigma_r}\right)^n \tag{3}$$

where $\dot{\varepsilon}_r$ and σ_r are reference values for the strain rate and stress. According to Equation 3, the creep parameter A in Equation 2 is:

$$A = \frac{\dot{\varepsilon}_r}{(\sigma_r)^n} \tag{4}$$

Based on the experimental work from Nixon and Lem (1984) on saline fine grained frozen soils, Andersland and Ladanyi (2004) proposed the following empirical expression for σ_r in kPa as a function of temperature and salinity:

$$\sigma_r = 0.323(1 - T)^2 \left(\frac{59.505 - S}{8.425 + S}\right) \tag{5}$$

where T is the temperature in Celsius degrees, and S is the salinity in ppt.

The parameter A ($\text{kPa}^{-n} \cdot \text{year}^{-1}$) can be then calculated with Equation 4 as a function of temperature and salinity, using Equation 5 for σ_r, and a reference strain rate of $\dot{\varepsilon}_r = 10^{-4}$ year^{-1} (Anderson and Ladanyi 2004). For the analysis, the parameter A was determined with Equation 4 at different temperatures for the reported average salinity of 39 ppt.

8.3 Methodology

The creep analysis used the ground thermal conditions predicted through thermal modelling (SRK 2017c, Rykaart et al. 2018). It is expected that the creep behavior of the frozen core and foundation changes as the temperature changes over the dam design life. An accurate prediction of long-term creep deformations therefore requires a thermomechanical coupled constitutive model. However, an efficiently implemented coupled thermo-mechanical model is not available in commercial codes. Hence, long-term creep behavior was evaluated for the ground temperature distribution predicted ten years after dam construction. This time interval is considered as representative for the long-term creep deformation in the North Dam. Subsequently the analysis steps, as described in Table 3, were followed.

Table 3. Creep deformation analysis process.

Step	Procedure
Initial State	The initial stresses of the dam embankment and foundation was obtained in the model by using elastic properties for all materials and turning gravity on.
Elasto-plastic Phase	Dam shell, transition zone, and foundation zone (thawed clayey silt) over the -2°C isotherm was changed from elastic to Mohr-Coulomb materials, and the model was again brought to equilibrium.
Creep Phase	Temperature dependent elastic and creep properties were assigned to the frozen core and foundation, based on the predicted temperature ten years after dam construction. The model was allowed to deform for 30 years.

8.4 *Material Properties*

Elastic and creep material properties from laboratory tests are not available and were estimated based on previous site-specific studies (EBA 2006), published literature, and engineering judgment. Initial state elastic and shear strength properties were taken from EBA (2006). However, the frozen foundation elastic modulus was adjusted. Table 4 summarizes the elastic and shear strength properties used for obtaining the initial state in the model.

Table 4. Elastic and shear strength properties used for obtaining the initial state[1].

Model Region	Material	Unit Weight (kN/m^3)	Elastic Modulus (kPa)	Poisson's Ratio (-)	Cohesion (kN/m^2)	Friction Angle (^{0})
Shell	Run-of-quarry	22	1.0E+05	0.35	-	40
Transition	150 mm minus	21	1.0E+05	0.30	-	35
Core	20 mm minus: 5 mm minus (2:3 blend by volume)	22	1.0E+05	0.25	-	-
Foundation	Thawed clayey silt	17	6.4E+05[2]	0.35	40	-
Bedrock	Basalt	26	1.0E+08	0.25	1000	-

Notes:
1. EBA (2006).
2. Adjusted; EBA (2006) Elastic Modulus was 5 x 10^{-4} kPa.

Table 5 summarizes the parameters used for the creep phase of the analysis. Parameters n, m, and A (Equation 1) for the frozen core were estimated based on the laboratory test results from Ottawa sand (Sayles 1968) and an average temperature of -9˚C in the core ten years after dam construction (Rykaart et al. 2018). For the frozen foundation, n, (Equation 2) was estimated based on published laboratory testing results from saline fine-grained soils (Nixon and Lem 1984, Wijeweera and Joshi 1993). Temperature dependent A values for the frozen foundation were calculated with Equations 3 and 4 for a constant salinity of 39 ppt. For reference, Figure 5 plots Equations 3 and 4 for different temperatures and salinities. The figure includes values from Nixon and Lem (1984) for a salinity of 35 ppt, and those used by EBA (2006) for a salinity of 45 ppt.

Table 5 includes the estimated temperature dependent elastic moduli of the frozen core and foundation required for the elastic strains. Since creep is considered to be a constant volume process, the analysis used a Poisson's ratio of 0.5 for the frozen core and foundation.

Table 5. Creep and elastic properties of the frozen core and foundation[1].

Model Region		b (-)	n (-)	A ($\text{kPa}^{-n}\text{year}^{-b}$)	Elastic Modulus (kPa)
Core (-9^0 C)		0.26	1.32	2.0E-07	3.0E+05
Foundation[2]	-3^0 C	1	3	9.6E-05	1.0E+04
	-4^0 C	1	3	2.5E-05	3.2E+04
	-5^0 C	1	3	8.4E-06	6.6E+04
	-6^0 C	1	3	3.3E-06	1.4E+05
	-7^0 C	1	3	1.5E-06	2.8E+05
	-8^0 C	1	3	7.4E-07	5.7E+05
	-9^0 C	1	3	3.9E-07	1.2E+06
	-10^0 C	1	3	2.2E-07	2.4E+06

Notes:
1. Constant volume deformation; Poisson's ratio $\nu = 0.5$.
2. Salinity 35 ppt.

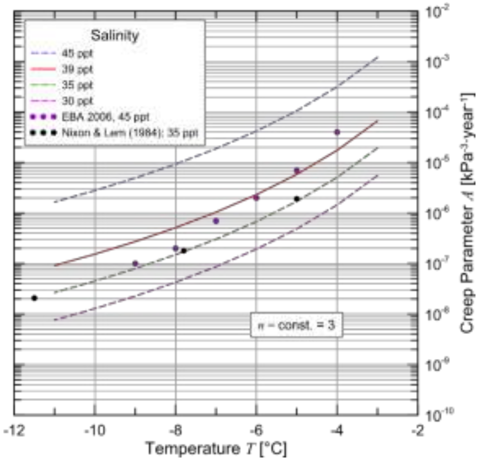

Figure 5: Creep parameter A for the marine clayey silt foundation material.

9 RESULTS

Table 6 summarizes the predicted creep strains and stresses as well as the displacements in the frozen core and underlying frozen foundation, 10 and 30 years post dam construction. Displacements are also listed for the core crest.

Table 6. Creep strains, stresses and displacements in the frozen core and foundation.

Period	Location	Maximum Shear Strain Rate ($year^{-1}$)	Maximum Shear Strain (m/m)	Maximum Deviatoric Stress (kPa)	Shear Stress (kPa)	Displacement (m)	
						Max. Horizontal	Max. Vertical
10 years post construction	Core crest	-	-	100[3]	-	-	0.2
	Core[1]	5.0E-08	5.0E-02	300[4] 400[5]	20 50[5]	0.4	0.6
	Foundation[2]	1.0E-07	~1.0E-01	50	20	0.4	0.6
30 years post construction	Core crest	-	-	50[3]	-	-	1.0
	Core[1]	2.0E-08	1.0E-01	300[4] 450[5]	20 70[5]	0.8	1.0
	Foundation[2]	4.0E-08	~2.0E-02	50	20	0.6	1.0

Notes:
1. Within the frozen core.
2. Underneath the frozen core.
3. At the top of the frozen core.
4. At the bottom of the frozen core.
5. Localized at the upstream lower corner of the core.

9.1 Shear Strain Rates and Shear Strains

The analysis predicts shear strain localization mainly at the downstream side of the dam. The shear localization zone is almost circular and goes along the transition zone and through the saline frozen foundation. This surface can be considered as a likely failure surface in the event that the material strength is mobilized along this surface.

In general, the predicted shear strain rates are very low in all zones of the dam and foundation compared with strain rates usually used in laboratory tests with frozen soils (Sayles 1968, Wijeweera and Joshi 1991, Arenson 2002). The maximum shear strain rates are 3.5E-07 $year^{-1}$ and 1.E-07 $year^{-1}$, 10 and 30 years after dam completion, respectively. The maximum shear strains are 4.0E-01 m/m (40%) and 6.0E-01 m/m (60%) for the same periods of time. Maximum shear strain rates and shear strains are predicted to occur in points within the shear localization zone (i.e. outside the frozen core and underlying foundation).

In the frozen core and underlying foundation (Table 6), the maximum rate of shear strain meets the design criteria for ductile material behavior (Section 4), while the shear strains themselves exceed the criteria. However, for the frozen core, ductile material behavior is expected because the maximum rate of shear strain is predicted to be very low (~ 1E-08 $year^{-1}$). Based on Bragg and Andersland (1981), and Arenson (2002), a brittle mode of failure can be excluded in frozen sands that deform under a shear strain rate below < 1E-05 sec^{-1} (3.2E+02 $year^{-1}$).

9.2 Principal Stresses Difference

Creep strain rates were evaluated as a response to induced deviatoric stresses by the dam weight. Maximum principal stresses differences of around 75 and 300 kPa are predicted to be almost constant at the crest and bottom of the frozen core, respectively, throughout the dam design life (Table 6). In the frozen saline foundation, an almost constant principal stress difference below 50 kPa is predicted over the design life.

The predicted stress differences at the bottom of the frozen core can be considered as intermediate, compared with the expected peak deviatoric stresses. In the remaining areas of the frozen core and underlying foundation, low stresses difference will prevail.

9.3 Shear Stresses

In general, the shear stresses in the frozen core and foundation are predicted to be relatively low over the design life of the dam, compared with the expected shear strengths of these materials.

9.4 Displacements

Based on the thermal modelling, a greater frozen area is predicted in the foundation at the downstream side than at the upstream side (SRK 2017c, Rykaart et al. 2018). Therefore, the maximum creep displacements due to the frozen foundation will be expected at the downstream side of the dam.

Maximum horizontal displacements of 2.8 and 4.2 m are predicted to occur 10 and 30 years after dam construction, respectively, in a small zone of the foundation at the downstream side. Within and underneath the frozen core, the horizontal displacements remain under 0.8 m over the design life of the dam (Table 6).

Figure 6 shows the vertical displacement history of a point located on the frozen core crest determined for a threshold stress of 30 kPa and pore water salinity of 30 ppt. At the end of the planned design life, the core crest is predicted to settle around 1.0 m.

As a reference for the predicted vertical displacements, Figure 6 includes the maximum measured vertical displacements. These displacements were measured at the downstream part of the dam crest in station 1+20 m, over the first four years after dam completion (although data since then shows a consistent trend).

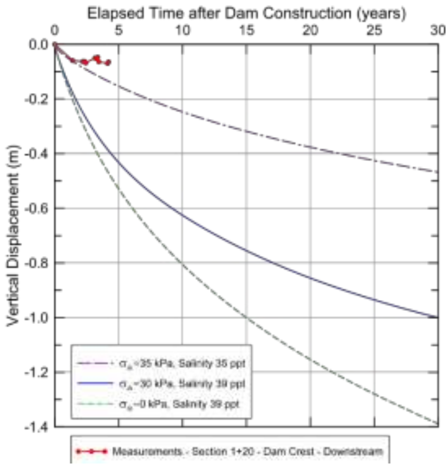

Figure 6: Vertical displacement history of the core crest.

To show the impact of threshold stresses and salinities on the creep deformation analysis, Figure 6 also includes predictions of core crest settlements obtained with thresholds stresses of 0 and 35 kPa, and salinities of 39 ppt and 35 ppt. The lower the threshold stress and higher the salinity, the greater the core crest settlement will be. For a salinity of 39 ppt, and without threshold stress ($\sigma_{th} = 0$ kPa), the core crest is predicted to settle around 1.4 m at the end of the dam design life, i.e. the core crest will settle up to the water full supply level of 33.5 m. However, this result is considered an overestimation of vertical displacements for the reasons outlined in Section 8.2.

10 CONCLUSIONS

The innovative 30-year design life water retaining North Dam constructed in Nunavut, Canada has challenging foundation conditions consisting of thick, saline, ice-rich permafrost, marine clayey silts, and is highly susceptible to creep deformation. Rigorous thermal and deformation monitoring data has been collected to monitor the dam performance since its completion in 2012.

Rykaart et al. (2018) presents the results of six years of thermal performance monitoring data which demonstrates that the dam is meeting its required thermal design criteria, and updated thermal modeling confirms that this is expected to be maintained for the design life of the structure.

This paper similarly presents six years of deformation monitoring data and updated numerical creep deformation analysis that confirms that the measured deformations in the dam are less than predicted, but even the predicted deformations would not subject the dam to stresses and strain rates outside of the deformation design criteria.

Long-term performance of the frozen core is not expected to be compromised throughout the dam design life. Thirty years after dam construction, the total settlement of the core will be around 1.0 m, i.e. 0.5 m above the full supply level (33.5 m).

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