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Ground support challenges in arctic mining conditions

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Abstract

The Meliadine and Amaruq mines, inaugurated by Agnico Eagle in 2019, are located in the Canadian Arctic, in areas of deep and continuous permafrost. Excavation of the exploration ramp at Meliadine started in 2013 and at Amaruq in 2017. These underground mines are being developed in conditions that vary from perennially frozen rock, through a transition zone where the rock temperature is below 0°C but where groundwater may be present in a liquid state depending on its salinity, to rock masses with a temperature perennially above 0°C. Brine is required as service water and for all drilling activities in permafrost but may be replaced by naturally saline groundwater below the permafrost. Ambient air conditions underground vary seasonally with depth and following ventilation patterns. As such, ground support elements and materials need to be adapted to the temperature and environmental conditions specific to each stage of mine development and operation. During the initial stages of ramp development, the freezing air and rock temperatures encountered throughout most of the year make the use of resin-grouted rebar difficult. The primary ground support within the permafrost areas therefore consists of inflatable bolts and friction bolts with mesh. As the mine progresses deeper and equipment becomes available, resin-grouted rebar and cement-grouted cable bolts are introduced. The integration and implementation of each type of ground support element require testing and the development of quality assurance and/or quality control protocols. Heating mine air in an arctic environment is costly and was a matter of debate during the project evaluation and development phase. The experience at Meliadine has shown that very cold air in the presence of groundwater can lead to the rapid formation of ice in joints, causing the premature failure of ground support elements. It is concluded that the combined presence of groundwater and freezing temperatures is problematic and that mine air heating is required in such conditions.

Keywords: *ground support, corrosion, ice formation, groundwater, permafrost, risk management*

1 Introduction

The underground Tiriganiaq mine of the Meliadine project is located in the Canadian Arctic, in an area of deep and continuous permafrost. The underground mine is developed and operated under conditions that include perennially frozen rock, a transition zone known as the basal cryopeg, and rock masses with a temperature perennially above 0°C. Within the basal cryopeg, the rock temperature is below 0°C, but groundwater in a liquid state may be present due to its high salinity. In addition to the cold rock temperatures, arctic-specific environmental conditions must be taken into account. Brine is used for all drilling activities and service water while in permafrost, and natural groundwater is expected to be saline when mining below the permafrost. The salinity of the groundwater is site-specific, depending on geological history and connectivity to fresh surface water. Ground support designs and materials therefore need to be adapted to the temperature and environmental conditions. The remoteness of the sites presents an additional challenge in terms of procurement and the need for forward planning, considerations that are not encountered in southern operations and that influence the selection of ground support elements at various stages of the projects.

2 Project description

The Meliadine project is located in the Kivalliq region of Nunavut near the western shore of Hudson Bay, in Northern Canada (Figure 1). The project is accessed by road from Rankin Inlet.

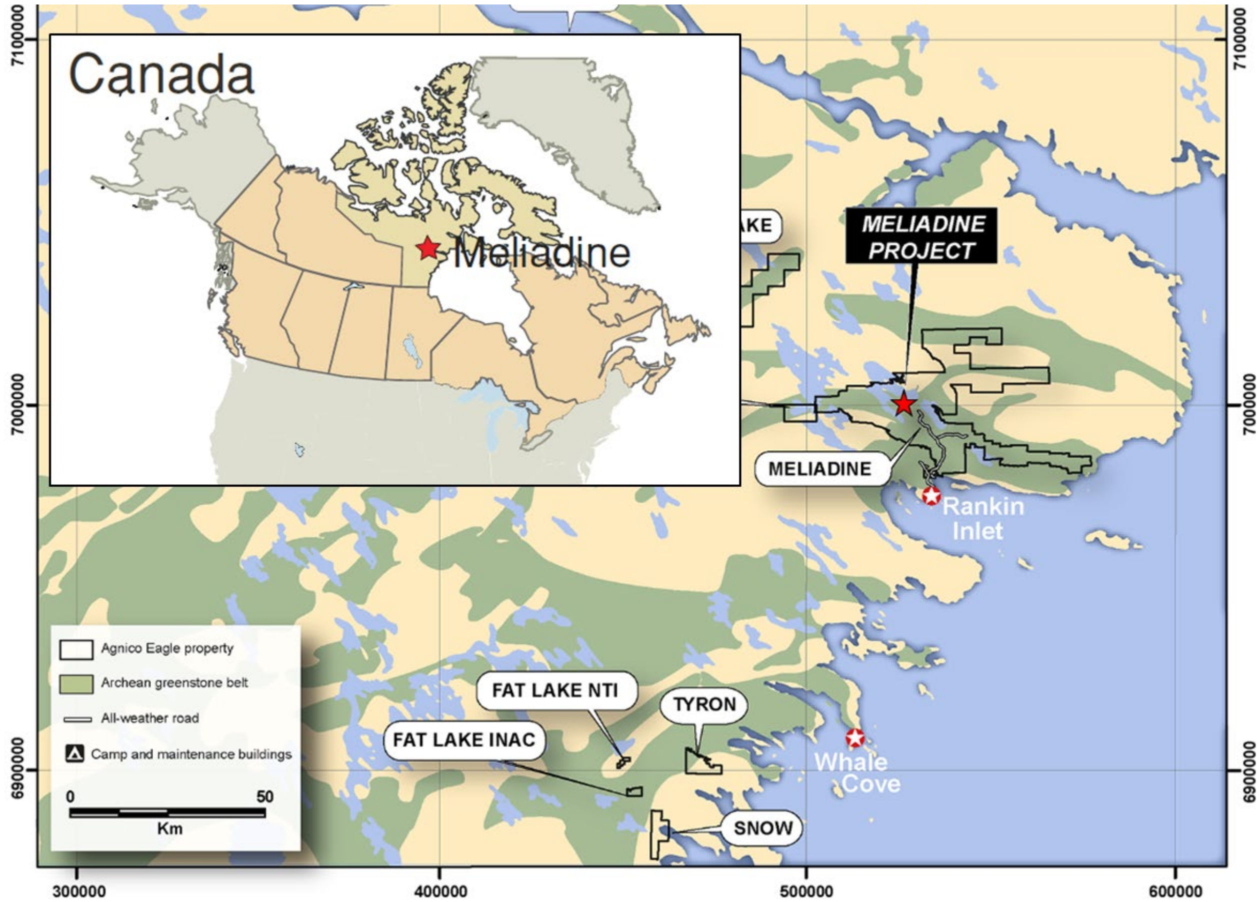


Figure 1 Meliadine project location

The property is about 60 m above sea level in low-lying topography with numerous lakes. Surface waters are usually frozen by early October, and they remain frozen until early June. Equipment, fuel and dry goods are transported on the annual warm-weather sealift by barge to Rankin Inlet via Hudson Bay. Ocean-going barges can access the community from late June to early October.

3 Property description

3.1 Geology

The Meliadine gold project is located in the northern portion of the Archean Rankin Inlet greenstone belt, within the northwestern Hearne subdomain of the Churchill Structural Province that forms part of the northern Canadian Shield. The Meliadine trend is defined by northwest-trending rock units marked by the interpreted major fault zone known as the Pyke Fault. The Pyke Fault is a several-kilometre-wide high-strain zone characterised by multiple foliations and regionally important shear zones. Most of the gold deposits within the Meliadine gold district are spatially associated with the Pyke Fault. The Tiriganiaq orebody is underlain by a sequence of greywacke and siltstones, iron formation and mafic volcanics or volcaniclastics. Gabbros intruded as sills are structurally parallel to the turbidite, volcaniclastic and iron formation sequences. All have undergone multiple deformation events related to southwest and northeast vergence thrusts. At Tiriganiaq, the gold is hosted in interbedded layers of banded chert and oxide iron formation and in quartz veins. Tiriganiaq is the largest of the known deposits within the Meliadine gold district and occurs along the east–west trending and steeply north-dipping Tiriganiaq Lower Fault zone, a splay of the regional Pyke Fault. The stratigraphic sequence hosting the deposit strikes east–west and dips to the north at an average of 60°. Clastic turbidites of the Sam formation are the oldest (northernmost) rocks. Beneath, is the Upper Oxide Formation, a diverse package of iron-rich rocks that includes beds of magnetite, chert, chloritic mudstone and greywacke and that hosts the majority of the mineralisation at Tiriganiaq.

Further south is the Tiriganiaq Formation composed of laminated siltstones. At the base of the Tiriganiaq Formation, there is sporadic black graphitic argillite that commonly underlies the 1,000 lode mineralisation. The Lower Fault likely formed along this obvious stratigraphic weakness. This fault defines the contact between the Tiriganiaq Formation and the underlying Wesmeg Formation composed of chlorite-rich massive basalts with rare gabbro dykes, interflow sediments, ultramafic volcanic rocks and iron formation bands. The Lower Fault is the locus of intense shearing. Associated faults emanating from the Lower Fault have resulted in repetition of the stratigraphy in the Tiriganiaq gold deposit.

3.2 Geomechanical domains

Geomechanical domains are largely based on lithology and consist of the Footwall Volcanics (Wesmeg Formation), the Hanging Wall Sediments (Sam Formation), the Tiriganiaq Formation and the Mineralised Zone (Upper Oxide Formation). The Footwall Volcanics are separated from the Mineralised Zone by the Lower Fault, a fault zone of variable thickness with local graphitic gouge and joint infilling.

Large-scale structural features at Tiriganiaq include the east–west trending Lower Fault and east-northeast trending Lower Fault splays within the Upper Oxide mineralised package. Other faulting orientations have been encountered in the mine development, namely the east-northeast trending undulating RM175 Fault and the west-northwest trending VT-125 Fault. Both these faults contained gouge and ice (Figure 2).

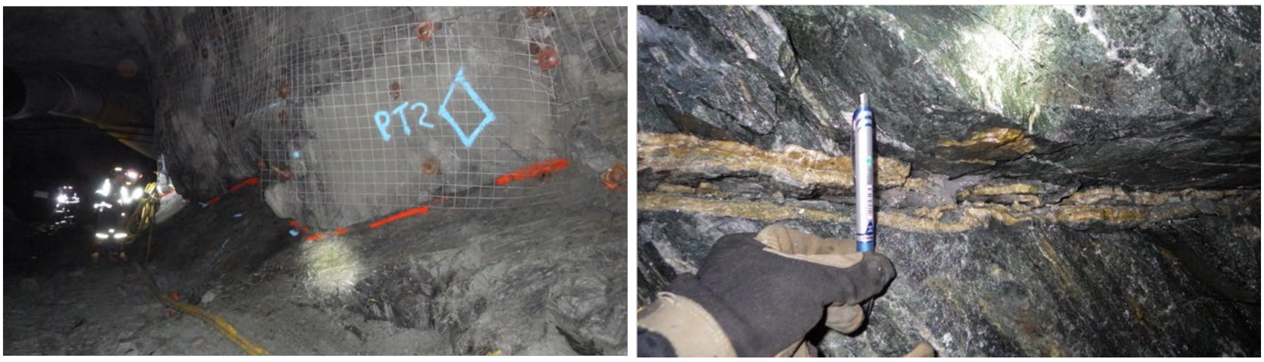
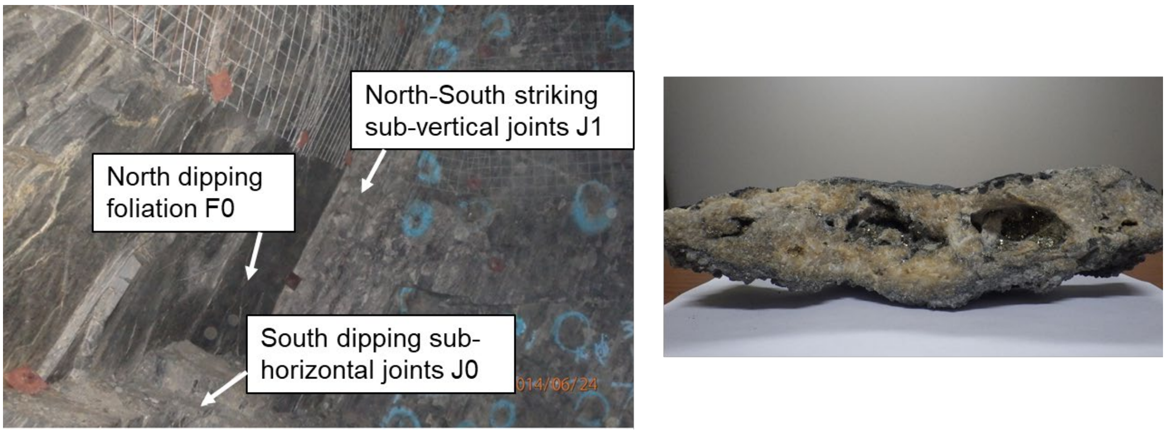


Figure 2 RM175 Fault with gouge and ice encountered during ramp development

Rock mass fabric is relatively homogeneous throughout the Tiriganiaq orebody (Figure 3). The main structural trend is the foliation and/or bedding, which is subparallel to the Lower Fault, trending approximately east–west and dipping 60° to 70° to the north. Average spacing of the foliation is 0.4 to 0.6 m. South-dipping, east–west striking sub-horizontal joints are encountered throughout the deposit. These joints are water pathways as evidenced by the presence of carbonate crystals and local vugs (Figure 3). North–south trending sub-vertical joints are present but are generally more widely spaced.



(a) (b)

Figure 3 (a) Rock mass fabric; (b) Example of crystals and vugs in the sub-horizontal joints

3.3 Permafrost and hydrogeology

The underground Tiriganiaq mine is located in the Canadian Arctic in an area of deep and continuous permafrost. Figure 4 presents a conceptual model of the deep groundwater flow regime as the underground mine extends through the basal cryopeg and into the sub-permafrost.

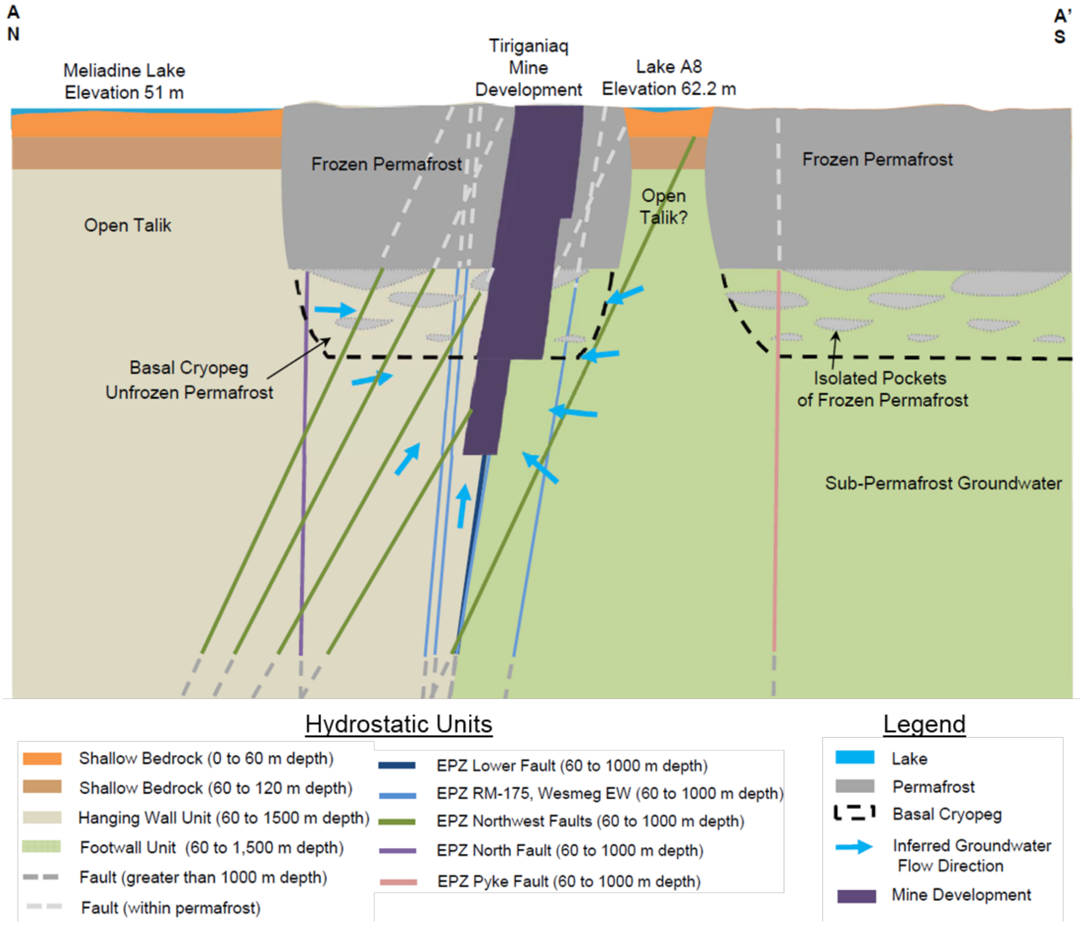


Figure 4 Conceptual model of the deep groundwater flow regime in the underground mine (Chorley et al. 2016)

The mine is developed and operated under conditions featuring perennially frozen rock, a transition zone known as the basal cryopeg, and rock masses with a temperature perennially above 0°C. As mining extends below the permafrost, groundwater from the deep groundwater regime will be induced to flow into the underground excavations. In the Canadian Shield in which the project is located, concentrations of total dissolved solids (TDS) in groundwater increase with depth, primarily in response to upward diffusion of deep-seated brines. The chemicals that contribute to TDS in shield brines are typically chloride and calcium and, to a lesser degree, sodium. By comparison, sea water is mostly composed of chloride and sodium.

Meliadine sub-permafrost groundwater is old, highly saline and of ocean-water origin mixed with autochthonous Canadian Shield brines. Groundwater salinity is 1.8 times that of sea water; it is largely sodium-chloride based with low trace-metal content. The groundwater has a similar chemical composition to Hudson Bay water, albeit more concentrated in major ions (Bertrand & Skeries 2016). In the area of the underground workings, the groundwater is not connected to surface waters (Chorley et al. 2016). A freezing-point depression of approximately 3.4°C was derived, forecasting an estimated depth to the basal cryopeg of between 280 m and 300 m below ground surface. The hydrogeological site investigations performed in 2015 and 2016 confirmed the relatively low hydraulic conductivities of the rock masses at Tiriganiaq. Groundwater flow is therefore strongly related to enhanced permeability zones that may be fault zones, fracture zones or joints.

4 Ground support in an arctic environment

Inflatable (Swellex™ type) bolts and split sets are the most practical types of rock reinforcement for advancing underground mine development in a permafrost environment. The low rock and air temperatures make the use of resin- or cement-grouted rebar or cable bolts extremely difficult and unreliable, as the storage infrastructure on surface and underground is limited during the early stages of project development. The limitations of the inflatable bolts include vulnerability to corrosion and low shear resistance; relying on this type of tendon for primary support requires weighing the risks against the benefits and implementing control measures to mitigate the risks. As the development advances through the basal cryopeg and below the permafrost, the environmental conditions change significantly and the ground support systems must evolve.

4.1 Corrosion studies

Initially patented by Epiroc (Atlas Copco) as Swellex™, inflatable bolts are now manufactured in several countries and are sold under a variety of trade names. Multiple product sources translate into a competitive market and cost advantages for the mining operations as long as the product quality and performance can be ascertained. An in-house project was initiated in 2013 to compare inflatable bolts from various manufacturers. This project, which was designed to compare the vulnerability to corrosion and the mechanical properties of the inflatable bolts, both in a controlled laboratory environment and in situ, consisted of the following steps:

1. Phase 1:
   a. Desktop review:

      Review specifications of each supplier’s rockbolt, assess and compare their expected corrosion resistance and mechanical properties based on their composition, forming process and any heat treatment. Inflatable bolts are different from other types of bolts in that in addition to their chemical composition (type of steel), the forming and manufacturing process and hardness can potentially have an effect on the bolt performance and longevity (Behnood 2015a).

   b. Laboratory testing:
      - Strength testing under controlled laboratory conditions.
      - Accelerated corrosion testing of inflated and non-inflated samples.
      - Visual examination, stereomicroscopy, photography and metallographic examination of samples that had been corrosion tested.
2. Phase 2:
   a. Field-testing to monitor the bolts in situ:
      - Periodic capacity (destructive) testing of bolts installed specifically for this purpose.
      - Bolts from each manufacturer were stored uninflated within the rock mass underground; samples were retrieved on a periodic basis for visual and metallographic examination.
      - Periodic proof testing (nondestructive) of bolts throughout the mine.

4.1.1 Phase 1 results

4.1.1.1 Laboratory testing

Four types of bolts were subjected to chemical analysis and strength testing. All bolts were essentially made of low-carbon steel except for one, which was made of carbon-manganese steel with relatively high C and Mn content. Aside from the C-Mn steel, the chemical analyses of the different bolts did not reveal any significant differences. The manufacturers were reluctant to share details of their manufacturing process; therefore, no further conclusion could be drawn from the information available regarding the impact of cold work or heat treatment on the bolt capacity or longevity.

Sample bolts were sent to NRCAN-CanmetMINING's laboratory at Bells Corner for strength testing. The bolts were inflated within specially fabricated, 38-mm diameter steel sleeves and then subjected to pull testing. All tests were completed in compliance with ASTM F432-13 (ASTM 2013) and ISO 6892-1 (ISO 2016) Standards (Voyzelle & Wu 2015). Table 1 presents the results of the laboratory tensile strength testing.

Table 1 Summary of tensile strength testing and comparison to manufacturers' specifications

Samples tested	Average measured maximum force (kN)	Supplier's specified minimum breaking load (kN)	Measured maximum proof force (kN)	Supplier's minimum yielding load (kN)	Measured total % elonga-tion	Supplier's specified minimum % elonga-tion	Measured tensile strength (MPa)
Bolt 1	126	120	100	NA	28.6	10	523
Bolt 2	144	100	131	90	10.25	6	584
Bolt 3	133	110	129	90	10.40	10	546
Bolt 4	137	120	125	NA	11.40	25	533

Two types of accelerated corrosion tests were conducted in the laboratory: (a) immersion tests in a 12% $CaCl_2$ solution and (b) electrochemical tests. All coupons were subjected to exactly the same testing and cleaning process (Behnood 2015a). Testing was done on both inflated and non-inflated samples. The results of the accelerated corrosion tests on the inflated samples are summarised in Figure 5.

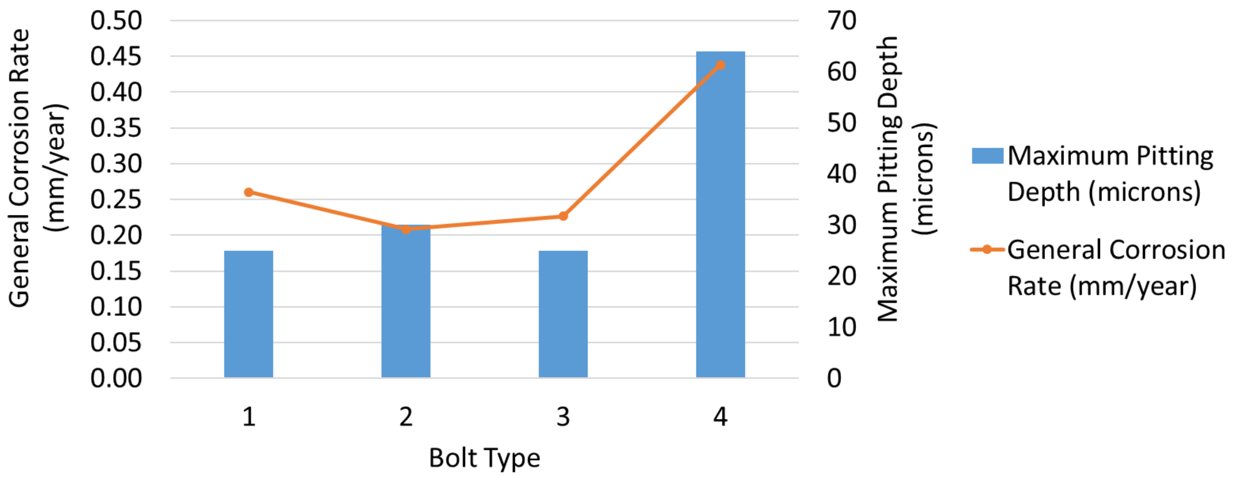


Figure 5 Generalised corrosion rates (line) and maximum pitting depth (bars) during laboratory testing

The results of the two testing methods for general corrosion were consistent. Bolts 1, 2 and 3 presented generally similar general corrosion rates, while Bolt 4 seemed to be more susceptible to generalised corrosion. Bolts 2 and 3 came directly from the factory whereas bolts 1 and 4 came from the local supplier and/or mine and hence had been exposed to a short period of storage prior to testing, which may have influenced the results. Localised corrosion (pitting) has a high growth rate that can result in perforation and local weakening of the bolt. Localised corrosion was apparent on all four bolt groups after the corrosion test in brine. The extent of pitting was observed through metallographic examination under the optical microscope. It was observed that the extent of pitting was more pronounced in Bolt 4. Bolt 2 samples showed a number of crack-type features on the outside surface. This indicates potential susceptibility to stress-corrosion cracking under certain circumstances.

Based on the results of the laboratory testing, cost comparisons and experience within the company and other mining operations (Germain, pers. comm., 2 September 2015), Bolt 3 was selected to initiate the underground exploration ramp development at Meliadine.

4.1.1.2 Field-testing

The field-test setup was located in access VAE-135 off the main exploration ramp, at a depth of approximately 135 mbgs in the permafrost (Figure 6). The holes were drilled using a 37.6-mm bit, and the 2.4-m long inflatable bolts were installed in early May 2015 as per specification (i.e. 300 bar inflation pressure). Eleven Bolt 3-type bolts, nine Bolt 2-type bolts and eight Bolt 4-type bolts were installed. Three bolts of each type were pulled to destruction between 4 May and 12 May 2015 (Time = 0). Destructive testing was repeated in November 2015 (time = 6 months), October 2016 (time = 17 months), and April 2017 (time = 23.4 months) to monitor the loss of capacity over time. The destructive testing results presented in Figure 7 show that all bolts installed in VAE-135 exceeded their nominal capacity up to two years after installation. Bolts 2 and 4 lost between 14% and 16% load capacity over 1.5 years, whereas Bolt 3 lost approximately 8% load capacity after 6 months, and more or less stabilised thereafter.

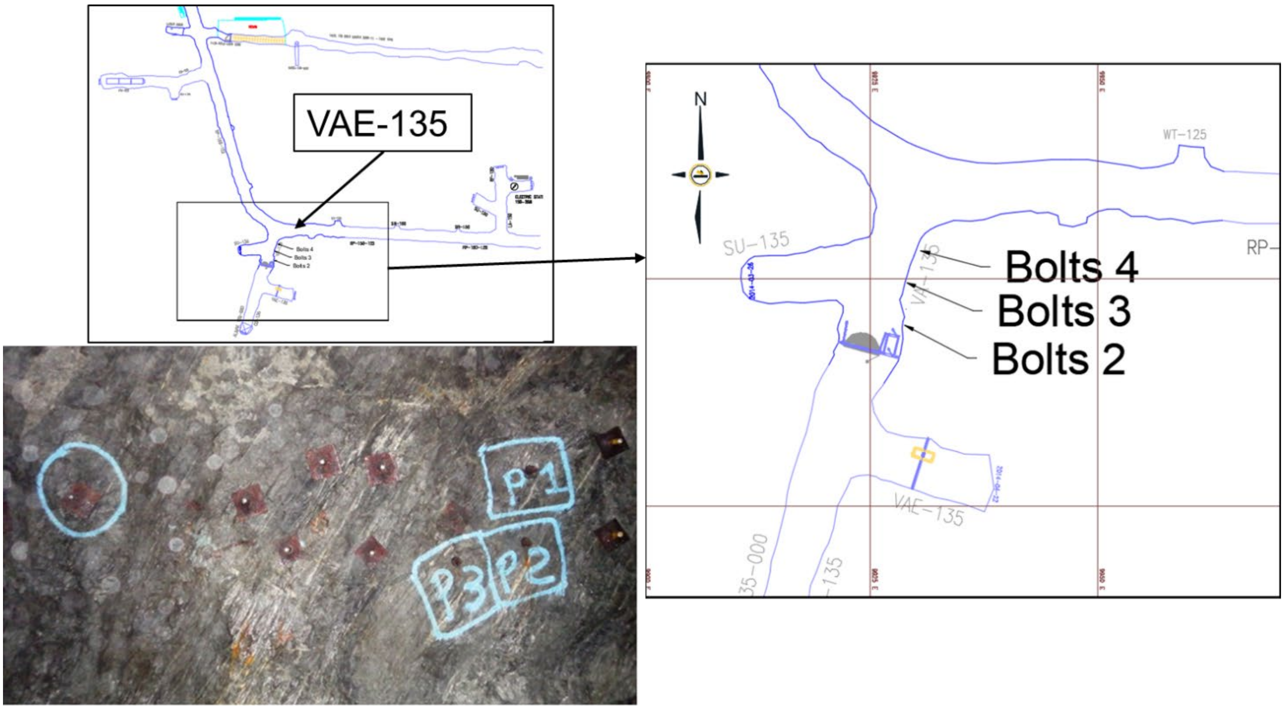


Figure 6 Field-test site in VAE-135 access; the three Bolt 3s tested in May 2015 (Time = 0) are highlighted with squares in the photo; the circled Bolt 3 was inserted uninflated into the drilled hole

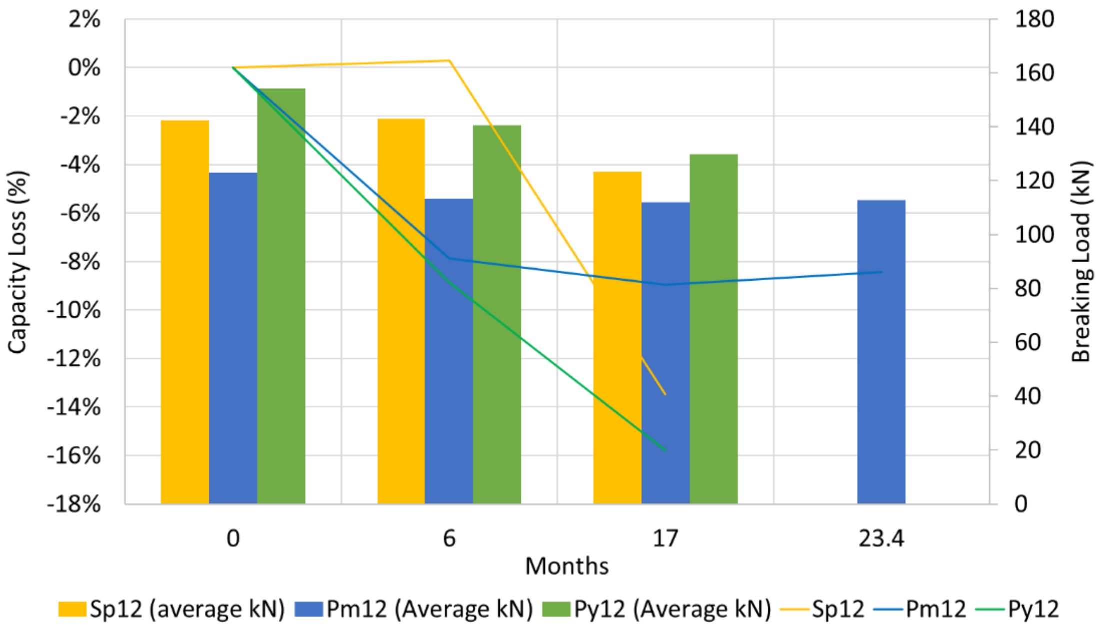


Figure 7 **Results of field destructive testing of Bolts 2, 3 and 4 subjected to similar environmental and ambient conditions. The results are presented as average breaking load and percentage capacity loss**

In addition to the destructive testing, units of the three types of uninflated 2.4-m long bolts were inserted into open holes of similar diameter with a faceplate isolating the bolts from the atmosphere. The ambient air temperature in the area varied from −5°C to 16°C, and the rock temperature within 3 m of the wall surface varied from −8°C to 8°C with seasonal variations. Samples were cut from these bolts and sent for metallographic analysis in June 2015, November 2015, January 2017 and November 2018.

The depth of general corrosion and the maximum depth of pitting were measured on the samples retrieved from storage to monitor the effect of in situ corrosion over time (Behnood 2015b, 2016, 2017, 2018). The test setup is unconventional compared with other studies (Dorion 2013), but the metallographic analysis provides a direct measurement of how the different types of bolts react inside the cold rock mass over time. Figure 8 illustrates the progression of pitting depth with time for the three bolt types stored in the boreholes. Bolt 3 has the lowest pitting and general corrosion rates at about 0.051 mm/year and 0.040 mm/year under the test conditions.

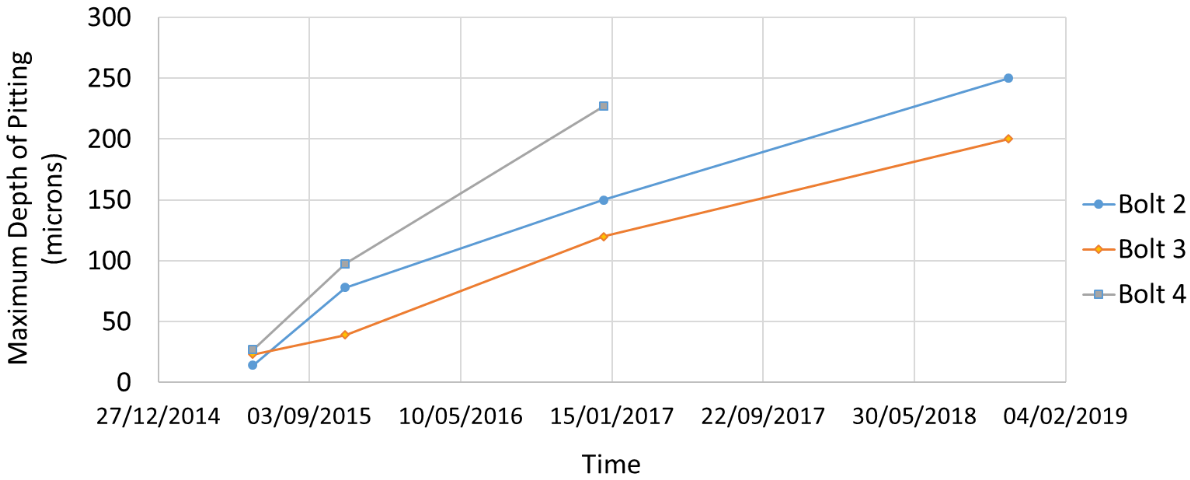


Figure 8 **Results of metallographic analysis of cut samples stored within the rock mass (Behnood 2015b, 2016, 2017, 2018)**

The laboratory and in situ testing provide reassurance that the corrosion levels are low in the permafrost and that Bolt 3 should perform adequately for the period of time required, but the tests cannot represent the true conditions once the bolts are inflated and under load. Regular proof testing of the ground support is part of the Ground Control Management Plan. Bolts installed in the back of excavations are pulled to a maximum of 90 kN. In case of noncompliance, the bolts are immediately replaced. Testing conducted in 2015 to 2017 indicated that three bolts out of 135 tested were noncompliant.

4.2 Ice jacking failure mechanism

A fall of ground occurred in the mine development on 17 March 2017 at a depth of about 375 m below surface. There were no injuries, but the event was classified as a dangerous occurrence and thoroughly analysed. The block, weighing approximately 8 tonnes (2.4 m long × 1.2 m wide × 1 m thick) and well defined by the known rock mass fabric, fell from the roof of the excavation (Figure 9).

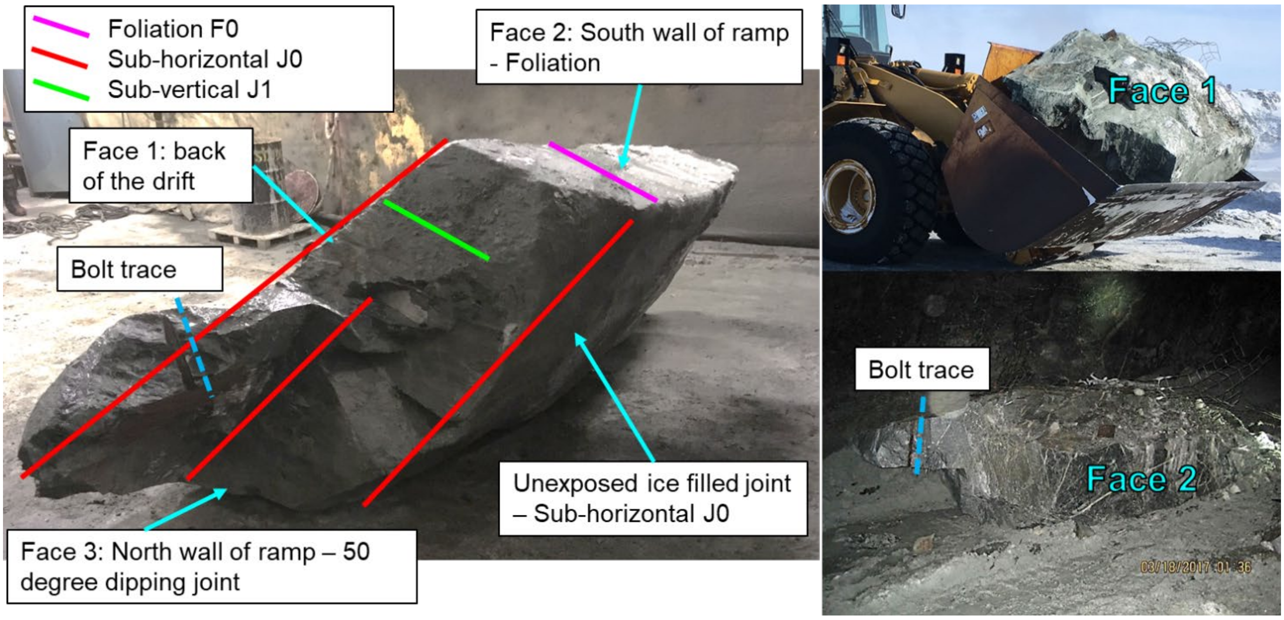


Figure 9 A block weighing approximately 8 tonnes fell from the roof of the excavation

The standard primary ground support consisted of Pm12™ inflatable bolts in the back, and split set bolts in the walls; 60-cm long rockbolts were used to pin the mesh. The ground support was eight months old and had been installed in 33-mm diameter holes, which were smaller than the optimal size (37 mm) recommended at Meliadine, but subsequent testing of surrounding bolts installed at the same time exceeded the rated capacity. Two 2.4-m long Pm12™ bolts were found broken within the block. Considering the rated 110-kN capacity for each bolt, the support capacity should have been sufficient to hold the block in place, even assuming failure or inadequate installation of one bolt.

The broken bolts were recovered from the block and sent to an independent laboratory for analysis. Tensile failure was confirmed by the laboratory and testing of the remaining bolts' tensile capacity showed no strength degradation (Côté & Roussy 2017). In the underground, bolts around the groundfall area were subjected to destructive pull testing and visually assessed for corrosion using a camera (Leung 2017). The results of the laboratory investigation and field-testing showed that capacity loss due to corrosion was not a factor in the fall of ground and that the bolts had failed in tensile overload (Figure 10). The typical causes of vulnerability in inflatable bolts (corrosion and shear) were therefore not responsible for the fall of ground. Another failure mechanism was responsible for the event. Upon inspection of the failure site at the time of the failure (Vanessa Smith, pers. comm., 18 March 2017), ice build-up was noted along the sub-horizontal joint J0 that formed the back surface of the block (Figure 11).

Leung, 2017 (Atlas Copco)

Bolt Type and Length	Date Installed	#	Bit Diameter (mm)	Max Load at rupture (kN)	Max Load at rupture (metric tons)	Displacement Recorded at rupture (mm)	Level of corrosion inside bolt per camera inspection (see Appendix C)
Pm12 2.4m	26th – 30th July 2016	1	33	118	12.0	Did not rupture	Did not test the inside
		2		125	12.7	28.7	Condensation. C3 near collar, C2 near end
		3		127	13.0	30.0	C3 near collar, C2 near end
		4		128	13.1	27.6	C3 throughout

Côté & Roussy, 2017 (Exova)

CONCLUSION

- The bolts failed in tensile overload at the junction between the block and the rock wall.
- The overload cannot have been induced by the weight of the block alone. An additional force has been exerted on the bolts that caused them to break.
- General corrosion was observed on the bolt components but no pits were observed. The corrosion did not induce the failure of bolts #1 and #3.
- The tensile resistance of the bolts meet the 110kN minimum requirement.

Figure 10 Independent reports of in situ field-testing (Leung 2017) and laboratory investigations on broken bolts (Côté & Roussy 2017)

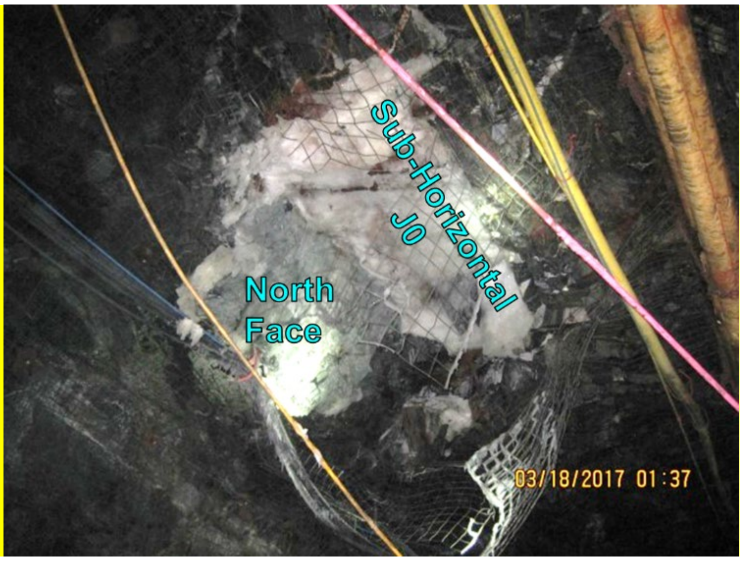
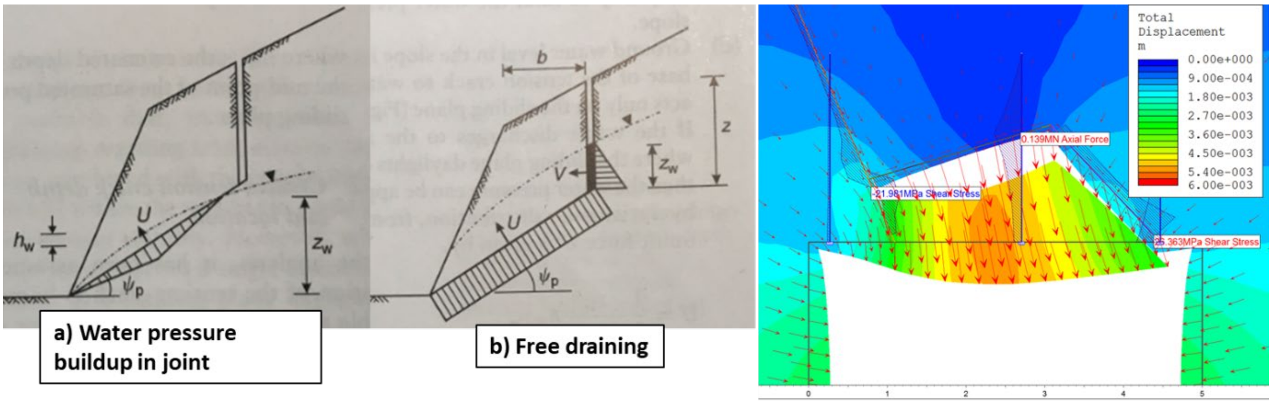


Figure 11 Ice formation along J0 behind the fallen block. The ice was visually estimated to be around 50 mm thick. Thinner ice (grey colour) was also noted along the 50° dipping joint surface on the north face of the block

As the mine development advanced through the permafrost into the basal cryopeg, conditions evolved from dry to progressively wetter as the excavations crossed or approached water-bearing faults and fracture zones. The high TDS groundwater (64,000 mg/L) transforms from solid to liquid state at a temperature of −3.4°C; therefore, the mine excavations started to encounter liquid groundwater from a depth of about 280 m below surface. During the winter of 2017, the formation of ice was noted underground, and ice accumulation was observed on drift walls and in ventilation raises. In March 2017, changes to the ventilation network resulted in lowering the air temperature in the area of the ramp where the fall of ground occurred. Water inflows had been recorded and grouted in this area during the ramp development in July 2016.

The cold air temperatures caused freezing to progress from the surface of the excavation, blocking the flow of groundwater along the sub-horizontal joint behind the surface in the back of the ramp. As ice forms within a narrow space, it expands in volume between 2% and 9% and causes localised strain on the bolts. Figure 12 describes the principles of the failure process and provides a simplified 2D numerical model to illustrate it. It was concluded that although they have higher capacity, rebar would not have withstood the pressure and localised straining generated by the ice formation.



Drop in air temperature in the ramp:

- Blocks outlet of water at drift surface
- Changes groundwater pressure conditions from b) to a)
- Water pressure in joint above the drift more than doubles when the water outlet is blocked
- For low water flow and short duration of outlet blockage, the additional pressure is probably not sufficient to exceed the tensile capacity of the 2 bolts
- Prolonged and very cold temperatures provide additional mechanism for bolt failure when ice fills the joint

Figure 12 Ice jacking failure process and simplified 2D numerical model showing block displacement and bolt failure resulting from applying a 15-MPa pressure in the joint

The ice build-up due to water infiltration and cold temperature affected the same ramp downdip from the previous fall of ground. In this case, the ice build-up affected the walls of the excavation, causing dilation of the foliation (Figure 13). As the ice melted, the walls became destabilised and required significant rehabilitation. The ramp was closed for 10 weeks while ground support was installed and a wall buttress built to support the estimated 110- to 200-tonne wedge in the wall.

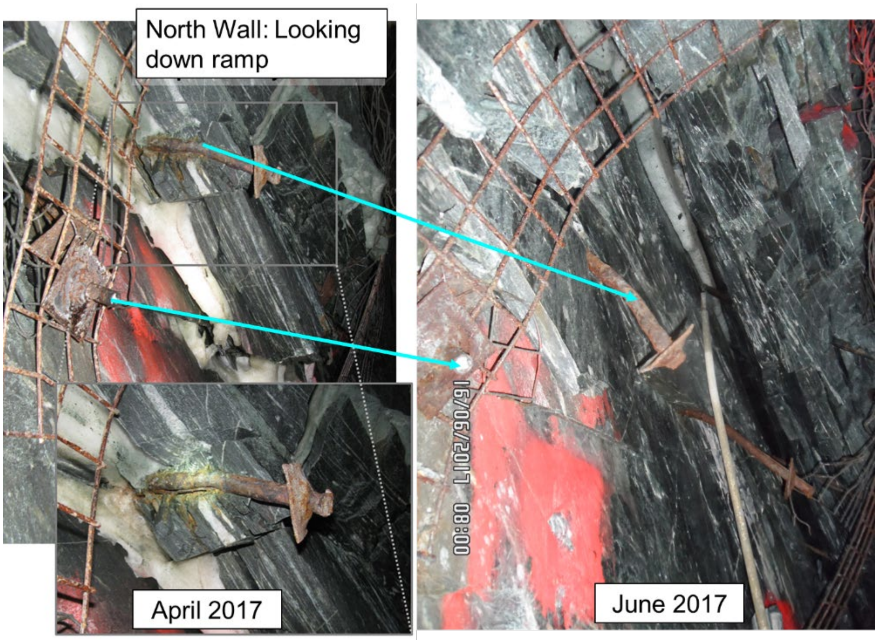


Figure 13 Ice build-up along the foliation (thick white bands) causing dilation and degradation of the wall

These incidents demonstrate that although it is possible and cost-efficient to operate a mine within permafrost at low air temperatures without heating the air, this approach is not desirable in the presence of groundwater. Mine air heating was therefore accelerated and implemented in December 2017 to avoid further seasonal freezing and ice jacking of the discontinuities within the rock mass. With the implementation of mine air heating, the rock temperatures in the ramp at Level 200 have stabilised as shown from the thermistor data in Figure 14. It can be observed that prior to the heating of the mine, the winter seasonal low was less accentuated in winter 2017, mainly due to the presence of more equipment underground.

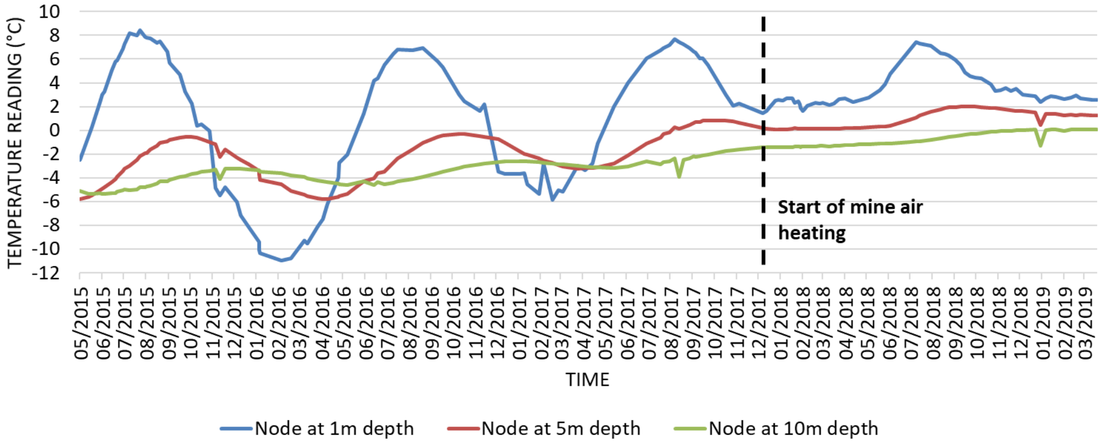


Figure 14 Temperature readings of Level 200 ramp thermistor (nodes at 1-, 5- and 10-m depths in the rock mass) showing seasonal temperature variations. Heating of the air started in December 2017

A procedure was developed for unexpected interruption of the main intake burners heating the air to the mine. An alarm is triggered when the air temperature at the intake falls below −4°C, setting in motion a series of actions including an increased frequency of instrumentation readings (thermistors and extensometers) and the inspection of the ramps and main intake drifts.

4.3 Rebar and cable bolt installation in arctic environment

After the mine air heating system was installed, implementation of resin-grouted rebar and cement-grouted cable bolts was initiated. The procurement and handling of resin cartridges and selection of the cement product for cable bolts present additional challenges in arctic mining conditions compared with southern operations.

4.3.1 Resin-grouted rebars

The procurement process in arctic operations presents many logistical challenges. Most of the material is brought onsite once a year during the sealift (open water) season between mid July and mid October. Suppliers deliver the material to the port of Bécancour in southern Quebec. The material is loaded onto deep-sea vessels (barges), which travel for one week to Rankin Inlet (Nunavut), whereupon the material is unloaded onto the temporary storage laydown at Rankin Inlet. Thereafter, the material is transported by truck over a distance of 7 km on a community road and then on the 25-km long all-weather private access road built by Agnico Eagle Mines Ltd. (AEM). Given that resin cartridges have a maximum shelf life of 12 months according to the manufacturer's recommendation and material can be ordered only once a year to cover an 18-month period, the number of cartridges to be ordered must be estimated as accurately as possible. In case of emergency, airlift is possible, but the cost is prohibitive for such heavy bulk material.

To work effectively, resin must be at a temperature between 12°C and 17°C before being inserted into the hole. The average annual ambient temperature is around −10°C and varies between −30°C and −40°C in winter; underground, the air temperature is on average between −2°C and 5°C. Many logistical steps and a joint effort are required between the warehouse, surface site services and underground departments in order to insert the resin cartridges into the holes at the designated temperature.

Once at the mine site, the resin pallets are left in sea cans and stored outside, where they remain frozen for about eight months per year. Before being sent underground, the resin pallets are transferred to the heated warehouse where they are left to thaw for at least 72 hours. Once underground, the resin pallets are stored in specifically designed heated sea-can storage. Custom resin heater boxes have been designed and installed on all the bolting machines to keep the resin at the right temperature before the installation of the rebar. Two different oven models have been designed to fit onto the two different types of bolting machines used onsite (Figure 15).

(a) (b)

Figure 15 Resin heater boxes installed on bolting machines: (a) drawer model, (b) floor model

The installation procedure was also modified from accepted practice at other AEM operations to account for low air and rock temperature. Two fast-setting FASLOC® cartridges are inserted at the toe of the hole and two slow-setting cartridges at the collar of the hole. The resin is mixed for 20 seconds, and then a 30 second gel time is required before torquing the bolt. Task observations of rebar installation and pull tests are regularly performed by the site ground control staff as part of the Ground Control Management Plan's annual quality assurance and/or quality control (QA/QC) program.

Implementation of the resin-grouted rebar took place over a six-month period. A similar undertaking could be completed within two weeks in a southern Quebec operation. The implementation of new processes takes longer in remote locations due to the delays in obtaining material and specialised staff onsite. In arctic conditions, even more logistical and operational challenges must be met. Longer implementation times must be taken into account during the study and planning phases of such projects.

As temperature control is key to rebar installation, the procedure in case of unexpected interruption of the main intake burners specifies that if the air intake temperature drops below −15°C for more than one shift, installation of resin rebar is suspended and the underground water piping system is drained.

4.3.2 Cement-grouted cable bolts

Cable bolts are currently installed mainly with a Sandvik DS422i cable bolter using Sika (King) Nordic Cable TC grout within the permafrost (above Level 300), and GU Type cement in the cryopeg and below (Level 300 and below). Cables are also installed manually, if needed, with a grout pump using the grout tube method. The cable grout is mixed with fresh water.

The first cable bolt installation trials using GU grout and the breather tube method were inconclusive. The water-to-cement ratio was high due to limited pumping capacity onsite at the time, and installations were carried out in rock masses in permafrost with GU Type cement. It resulted in the cement grout freezing in the holes early on during the hydration process, thus yielding inadequate grout strengths.

Following the initial unsuccessful trial, different products were tested: HE Type cement, modified HE Type cement, GU Type cement, MS cable grout, and Nordic Cable TC grout. The tests were performed with water-to-cement ratios between 0.25 and 0.35, in permafrost and below the permafrost. Relatively low water-to-cement ratios are needed for the installation with the cable bolter, as the grout has to stick by itself in the vertical holes. The manual installation method was changed from the breather tube method to the grout tube method, to be able to use the same products and similar grout mixes for manual installation.

Figure 16 shows the compressive strength development curves for tests with Nordic Cable TC (Cruz 2018) and GU Type cement grouts. The grout cylinders cast for the uniaxial compressive strength tests were cured in the drift at the test locations. For series of tests with both products, thermocouples were installed along the cable bolts to monitor the temperature in the holes during the hydration process of the grout. This instrumentation provided evidence that the hydration process was well engaged and that sufficient curing time was achieved before the grout temperature dropped below 0°C. As the grout cylinders cured at higher temperatures in the drift compared with the grout in the holes, the results presented at Figure 16 are likely higher than the grout in the holes. However, the thermocouple readings indicate that the grout is reaching strengths aligned with the cylinder strengths.

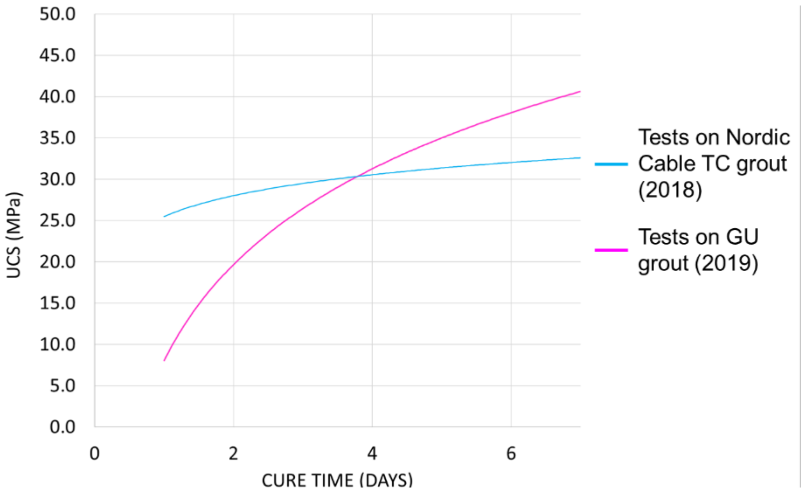


Figure 16 Summary strength curves of uniaxial compressive strength tests performed on cable grout cylinders for Nordic TC grout and GU grout

Some trials were also completed with the Nordic Cable TC grout using the sump-system brine water as mixing water for the grout. The results were highly variable due to the variable salinity and contamination of the sump water. The use of sump water had noticeable effects on the grout performance: the hydration peak was delayed by seven to nine hours and the compressive strengths were 15% to 60% lower than mixes with fresh water.

One of the challenges encountered in arctic operations is the variability of the conditions (temperature and water) with depth and ventilation patterns, at least until the operation is well below the permafrost limit where generally stable conditions are expected. Because of this variability, more trials are needed during and after the implementation processes, and ongoing QA/QC efforts are necessary to confirm the quality and effectiveness of the installations.

5 Conclusion

The development and operation of underground mines in the Arctic involves a number of challenges associated with the harsh climatic ambient conditions, cold rock temperatures, groundwater characteristics and remoteness. The high cost and seasonality of transportation of materials and equipment to the sites are specific to the Arctic mines and add another layer of complexity to ground support implementation and ground control management.

As the mine development advances through the permafrost into the basal cryopeg and eventually into perennially unfrozen rock, the ambient and rock temperatures evolve and affect the selection, durability and installation procedures of the ground support systems. Adequate characterisation of the rock mass, permafrost and hydrogeological conditions is required early in the project development to support high-cost-impact decisions, such as the requirement for mine air heating and groundwater management, and the selection of ground support systems. The specificity of the project’s geotechnical characteristics must be taken into account to evaluate and mitigate risks. This requires a commitment to conducting the necessary site investigations and technical studies in conjunction with advanced exploration and economic evaluation.

In the early stages of underground development in permafrost, inflatable Swellex-type bolts are used as primary ground support elements. As the project matures and the development advances below the continuous permafrost, ground support standards are modified to include resin-grouted rebar as primary reinforcement and cement-grouted cable bolts for long support of intersections and stopes. Laboratory and field-testing were used to evaluate the vulnerability to corrosion of different inflatable bolts in an arctic environment and to monitor the load capacity evolution of the ground support in situ as part of a risk management program. Ongoing monitoring of the ground support is integrated within the Ground Control Management Plan through regular proof testing of the installed bolts. Underground operations operate successfully in dry permafrost environments at below-freezing ambient temperatures. However, experience at Meliadine has demonstrated that the combination of groundwater inflows and very cold temperatures can lead to premature failure of ground support elements due to ice jacking. Mine air heating is therefore required when groundwater inflows are expected in the underground and procedures are implemented in case of system failure.

The introduction and implementation of resin-grouted rebar and cement-grouted cable bolting require significant investments in time and resources that are typically not available in the early stages of project development. Underground mines operating in arctic conditions have to contend with the variability of the physical conditions (air and rock temperature, groundwater quantity and quality) with depth and ventilation patterns, until deep below the permafrost limit, where stable conditions are expected. Because of this variability, more trials are required during the implementation processes, and ongoing QA/QC efforts are necessary to confirm the quality and effectiveness of the installations as conditions evolve.

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