Hazard identification, stability assessment and risk mitigation overview of near-surface openings and pillars at the former Mine Principale, in Chibougamau, Quebec

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ABSTRACT
Mine Principale refers to the historic Campbell, Kayrand and Merrill mines near Chibougamau, Quebec. The copper-gold sulphide deposit, hosted in an anorthosite rock mass, was mined via underground and open pit from 1953 to 2005. A closure plan, complying with local regulations, was established based on the results of an exhaustive historical-data review, 3D-model construction of near-surface openings and site investigation campaign to confirm the locations of the openings and rock properties for stability analysis. Remediation work started in 2018 and included collapse-blasting of thin crowns, backfilling of underground stopes either from surface or via 12-inch-diameter boreholes, stabilization of thicker crowns with galvanized anchors, and capping of raises with concrete slabs. This paper presents the site conditions, describes the construction of the 3D model, summarizes the findings of the field program, discusses the results of the near-surface stability analyses, shares the risk mitigation and remediation approaches used, and concludes with ongoing monitoring observations.

RÉSUMÉ
« Mine Principale » est le nom donné aux mines Campbell, Kayrand et Merrill qui ont été exploitées de 1953 à 2005. Le gisement de cuivre-or était encaissé dans de l'anorthosite. Un plan de fermeture a été établi après une revue exhaustive des données historiques, la construction d'un modèle 3D des ouvertures souterraines et la réalisation d'une campagne d'investigation de site visant à confirmer l'emplacement de ces ouvertures et les propriétés du massif rocheux pour fin d'analyse de stabilité. La remise en état a commencé en 2018 et comprenait le dynamitage de piliers de surface, le remblayage des chantiers souterrains à partir de la surface ou via des trous de forage, l'utilisation d'ancrages et le recouvrement de monteries. Cet article présentera les conditions du site, la construction du modèle 3D, les résultats de chantier et d'analyses de stabilité, la gestion des risques, les méthodes de sécurisation utilisées et le programme de surveillance en cours.

1 INTRODUCTION

1.1 Site Conditions

Mine Principale is an amalgamation of three former mines, consisting of two open pits and several underground workings: Mine Campbell (open pit and underground), Mine Merrill (open pit and underground) and Mine Kayrand (underground only). It is located approximately 15 km south-east of Chibougamau, QC, on Merrill Island. Figure 1 shows an aerial photograph of the site (Google, 2019). Note that the Campbell open pit is not visible as it had been backfilled prior to the satellite picture.

Mine Principale exploited copper-gold sulphide ore hosted in anorthosite. Mineralization was concentrated along two northwest striking shear zones where diorite and mafic dyke units intersected: Lens A to the south and Lens B to the north.

Figure 1. Mine Principale site layout

Typical NQ rock core (47.5 mm diameter) recovered from the mineralized crown pillars of a stope at the site can be seen in Figure 2.

Figure 2. Anorthosite and shear zone, Mine Campbell (BH16-MC-C5A, 0 to 25.2 m)

1.2 Project Objectives

Wood Environment & Infrastructure Solutions, a division of Wood Canada Limited (Wood), was retained by WSP and the Ministère de l'Énergie et des Ressources Naturelles (MERN) to secure the Mine Principale near-surface openings for closure.

The collection of additional geomechanical and hydrogeological data was required to meet the Quebec Mining Rehabilitation Guidelines (MERN, 1997). In 2016, a site investigation campaign was undertaken to confirm and supplement 1) rock mass properties and 2) volume of voids (stopes) determined during former site investigations, such as the Mine Principale Securing Study (Golder, 2011).

Joint orientations, intact rock strengths, hydraulic conductivities, void extent and rock mass classifications from the field program were used in subsequent stability analyses of the open stope and crown pillars to support the selection of mitigation measures.

2 3D MINE-MODEL CONSTRUCTION

The scope of work for the 2016 field campaign was developed following an exhaustive background desktop review (Ressources Meston Inc., 1996 & 2001; Roche, 2002; Golder, 2010 & 2011; SMi, 2013 & 2015; Université Laval, 2014) and an update of an existing 3D wireframe model. The geometry of the upper 100 m of the underground workings (stopes and drifts) was modelled in AutoCAD based on historical sections (Resources Campbell/Copper Rand Mine and Campbell Chibougamau Mines Ltd) and level plans (Campbell Chibougamau Mines Ltd). Figure 3 shows an extract from the model, including stopes, drifts and general infrastructure, from the 400 level up to the surface (approximately 400 ft or 120 m).

Level plans from the 125, 150 and 250 levels were also used to gain better insight into stope positions during model construction. Further, a number of the zones had no cross sections but only longitudinal sections, requiring interpretation based on the level plans and other zones.

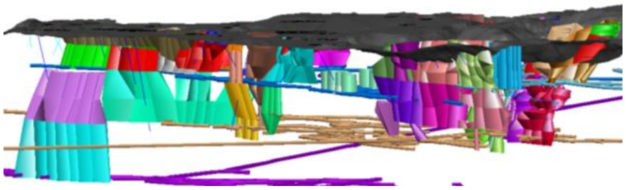

Figure 3. 3D AutoCAD mine model, isometric view looking north

Part of the 3D AutoCAD model was imported into Map3D Fault Slip Software (Map3D) for linear elastic stress modelling of Kayrand's near-surface stope/crown pillar.

The main challenges encountered during the construction of the 3D model related to:

- Accessing historical data;
- Identifying relevant level plans and sections from a huge database, and regrouping information from three distinct mines representing a period greater than 50 years;
- Combining drawings from various sources (production versus exploration) and formats (hard copies versus electronic copies), and with different coordinate systems and units requiring the use of scaling and conversion factors;
- Confirming locations of surface infrastructure, elevations and sizes of near surface stopes, drifts and the Campbell shaft, where different sources contradicted each other; and
- Ensuring final versions of drawings were used when several versions were available.

The most important starting point of any reclamation project like this is the construction of the 3D mine model based on all available information to be as accurate as possible. This sets the basis for investigation and studies going forward.

3 2016 SITE INVESTIGATION

3.1 Scope of Work Details

The main site investigation program took place between July 12 and August 24, 2016. The drilling program consisted of 67 NQ-sized boreholes (>1,900 m of overburden and rock core drilling) to intercept 29 near-surface stopes, a geotechnical berm and a rock barrier between the Campbell pit and the Lac aux Dorées (see Figure 5).

Core from 44 boreholes was oriented using the Reflex ACT III. In addition, acoustic and optical televiewer surveys were performed on 38 of the 67 boreholes, which provided additional oriented structural information. A total of 146 core samples collected from the drilling campaign were sent to Wood's rock-testing laboratory, located in Burlington, Ontario, where 112 test specimens were subjected to unconfined compressive strength, triaxial strength, Brazilian indirect tensile, and open-joint-directshear testing to determine intact rock strength and joint characteristics.

Hydrogeological investigations (packer testing, water-level monitoring) were performed on selected boreholes to assess the permeability of a geotechnical berm and surrounding bedrock at the west end of the Campbell Mine site adjacent to Lac aux Dorées, as well as the overall connectivity of the mine workings with the lake.

The scope of work included stope-void and backfill assessments, integrating drilling performances, fill recovery and plumb bob surveys, to determine which stopes were at risk and would require additional analyses. Four laser surveys were performed using the cavity auto-scanning laser system (C-ALS from Renishaw Canada Limited), targeting stopes 1-15E and 1-26A which had voids above the water table.

3.2 Investigation Findings

While significant variability from stope to stope was observed in the data, a summary of the main trends is provided below.

3.2.1 Jointing

Each lens shows two dominant joint sets: an east-west sub-vertical set striking at approximately 100° and dipping at 80° SW (set 1) and a horizontal set dipping at less than 5° (set 2). A third minor set (set 3), oriented quasi-orthogonally to set 1, with a strike of 200° and an average dip of 50° was also identified. A stereographic projection combining oriented core and acoustic televiewer data is shown in Figure 4.

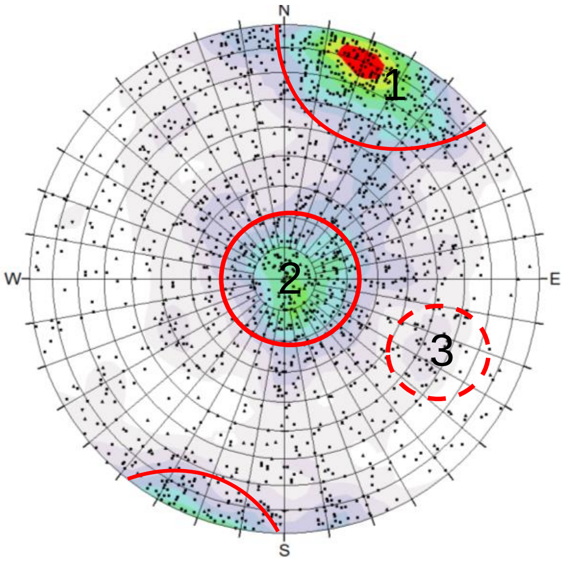

Figure 4. Stereographic projection of oriented core and ATV data, Lens A or South Zone (1704 poles, 0.5%-3.5% concentration, magnetic declination of -15.8° or 15.8°W)

The rock at the two extremities of the pit (east/west), where both lenses tapered off, and at the Kayrand Mine, showed the highest degree of fracturing. These areas had a joint number J_n of 9 (three sets) to 12 (three sets plus random) compared to a J_n of 6 (two sets plus random) elsewhere.

3.2.2 Intact Rock Strength

Most of the rock types tested had a mean UCS of 113 to 166 MPa, indicating very strong rocks (see σ_{ci} in Table 1). Overall, anorthosite was the strongest rock tested, although locally, strengths in diorite (Campbell – Lens B) and gabbro (Merrill – Lens A) were also high. Some medium to strong rocks (UCS of 33 MPa to 48 MPa) were found within Lens A (Merrill, Glory Hole, and Campbell), all within mafic rocks (gabbro, mafic dykes). It must be noted that a high variability in the results was observed, with Cv's greater than 30%.

Anorthosite had consistent Brazilian indirect tensile strength results, ranging from 7 MPa to 14 MPa (see σ_t in Table 1). Overall (including the weaker mafic rocks), the Brazilian tensile strengths varied from 2 MPa to 18 MPa.

Testing on anorthosite showed an average Young's modulus (E) of 79.6 GPa, corresponding to the stiffest rock tested. On the other side, gabbro appears to be the softest rock type tested with an average modulus of 48 GPa. The Poisson's ratio (v) for all rock types tested was relatively constant with an average value comprised between 0.3 and 0.4.

Intact peak strength parameters have been determined by rock type for Hoek-Brown (intact rock material constants m_i, s, a) and Mohr-Coulomb (cohesion C, angle of friction Φ) failure criteria (see Table 1).

Table 1 .Intact peak strength parameters

Rock Type	**# of σ_{ci}**	**RocLab (MPa)**		**Hoek-Brown[1]**	**Mohr Coulomb σ$_3$ Max = 5 MPa**	
		σ_{ci}	**σ_t**	**m_i**	**C (MPa)**	**Φ (°)**
Anorthosite	26	160.5	-11.0	14.5	24.2	55.9
Diorite	5	147.8	-14.1	10.5	25.8	51.5
Gabbro	7	128.6	-8.0	16.0	18.7	56.5
Mafic dyke	5	102.7	-7.1	14.5	15.7	54.7

[1]: s = 1.0; a = 0.5

Anorthosite had the highest intact rock strength, while diorite had the highest intact tensile strength. The mafic dyke showed the lowest intact strengths, with both the lowest intact tensile strength and weakest overall intact rock strength.

3.2.3 Rock Mass Classification

Based on the geotechnical data collected from the drilling program, rock mass classification was performed for each rock type using three standard classification systems: NGI's Q-Rating, Q (Barton *et al*., 1974), Bieniawski's Rock Mass Ratings, RMR '76 and '89 (Bieniawski, 1976, 1989), and Hoek's GSI (Hoek *et al*., 1995). The average Q' and

GSI ratings observed across the site, based on the rock type, are summarized in Table 2.

Table 2: Average rock mass ratings by rock type

Rock Type[1]	# of Runs	Q'		GSI	
		Mean	σ[2]	Mean	σ
Anorthosite	311	18.0	11.9	60	8.8
Gabbro	105	23.3	16.4	61	7.6
Diorite	89	17.2	10.0	60	7.6
Mafic dyke	31	18.2	8.8	61	4.2

[1] For every run, features are assigned the most dominant rock type for that run

[2] Standard deviation

The standard deviation is also shown to give an indication of the variability observed. Note that zone and stope-specific ratings were used for the designs.

The ratings indicated a *Good* quality rock mass across the site, with local zones where the quality was lower, from *Fair* to *Poor*.

3.2.4 Rock Mass Strength

The rock mass failure curves showed similar trends to those for the intact rock; the exception was the gabbro unit, which was stronger than the diorite at a confinement greater than 0.3 MPa.

Local variations in strength were observed within the same rock type (rock mass compressive and tensile strength σ_{cb} and σ_{tb}, rock mass material constants m_b, s). Table 3 shows how the north zone (Lens B) was found to be stronger than the south one (Lens A) for the main rock type (anorthosite).

Table 3. Rock mass strength properties

Rock Type	GSI	RocLab (MPa)		Hoek-Brown		Mohr Coulomb σ_3 Max = 5 MPa	
		σ_{cb}	σ_{tb}	m_b	s	C (MPa)	Φ (°)
Anorthosite North	64	24.8	-0.8	4.0	0.018	3.9	55.0
Anorthosite South	59	14.7	-0.3	4.6	0.011	3.0	54.9
Diorite	60	15.8	-0.7	2.5	0.012	3.0	49.7
Gabbro	61	14.6	-0.4	4.0	0.013	2.9	52.7
Mafic dyke	61	11.6	-0.4	3.6	0.013	2.6	50.2

3.2.5 Hydraulic Conductivity

Based on the packer testing results, rock hydraulic conductivity values ranged from 3.4E-06 m/s to 1.3E-08 m/s. Elevated hydraulic conductivities were found near the bedrock surface and then decreased with depth. The most permeable bedrock observed during the site investigation was located below the geotechnical berm (BH16-MC-H-27), indicating that the barrier and its foundation might not be as impermeable as anticipated. The area to the east of the berm and west of Campbell pit has a lower hydraulic conductivity (BH16-MC-H26 & BH16-MC-H25), with a slight permeability increase to the north (BH16-MC-H25).

4 RISK ASSESSMENT & MITIGATION APPROACH

4.1 Risk Assessment

Part of the site investigation purpose was to assess the structural integrity of the walls of Merrill pit (primarily the high north wall), and to confirm the location of the stopes as defined in the 3D model (geometry, crown pillar thickness). The program was also designed to determine if the stopes had been backfilled prior to closure. If a void was encountered, an estimate of the void height below the stope back was performed; if backfill was encountered, using a semi-qualitative method of drilling observation, the most probable nature of the backfill and its approximate elevation was established.

4.1.1 Slope Stability

Loose blocks and overburden material that could fall into Merrill pit were identified by visual inspection. Recommendations for re-sloping the soil and fill near the pit crest and for rock-wall scaling to remove loose blocks were made, as the kinematic and limit equilibrium analysis of the north wall indicated an overall factor of safety acceptable for closure (FoS ≥1.75).

4.1.2 Stope Void and Backfill Assessment

The extent of backfill and/or voids inside the target stopes was established by dropping a plumb-bob down the drill holes once completed. A steel aircraft cable mounted on a reel and kept taut by a 1.5 kg torpedo weight was lowered into each hole until the weight hit a solid surface at the bottom of the void. The length of cable in the hole when the bottom was encountered was then measured. After subtracting the length of the drill string from the measurement, the length measured corresponded to the vertical height of the void space observed within the stope.

A semi-qualitative method was used when drilling to identify the presence and type of backfill. On approach to the potential location of the stope, drill rotations were slowed, and water return was often lost indicating the proximity. The following guidelines were used to help identify the stope backfill condition:

In the absence of backfill (open stope):

- There should be no drilling resistance after intercepting the stope. The drill string should be able to advance easily when pushed, indicating a void.

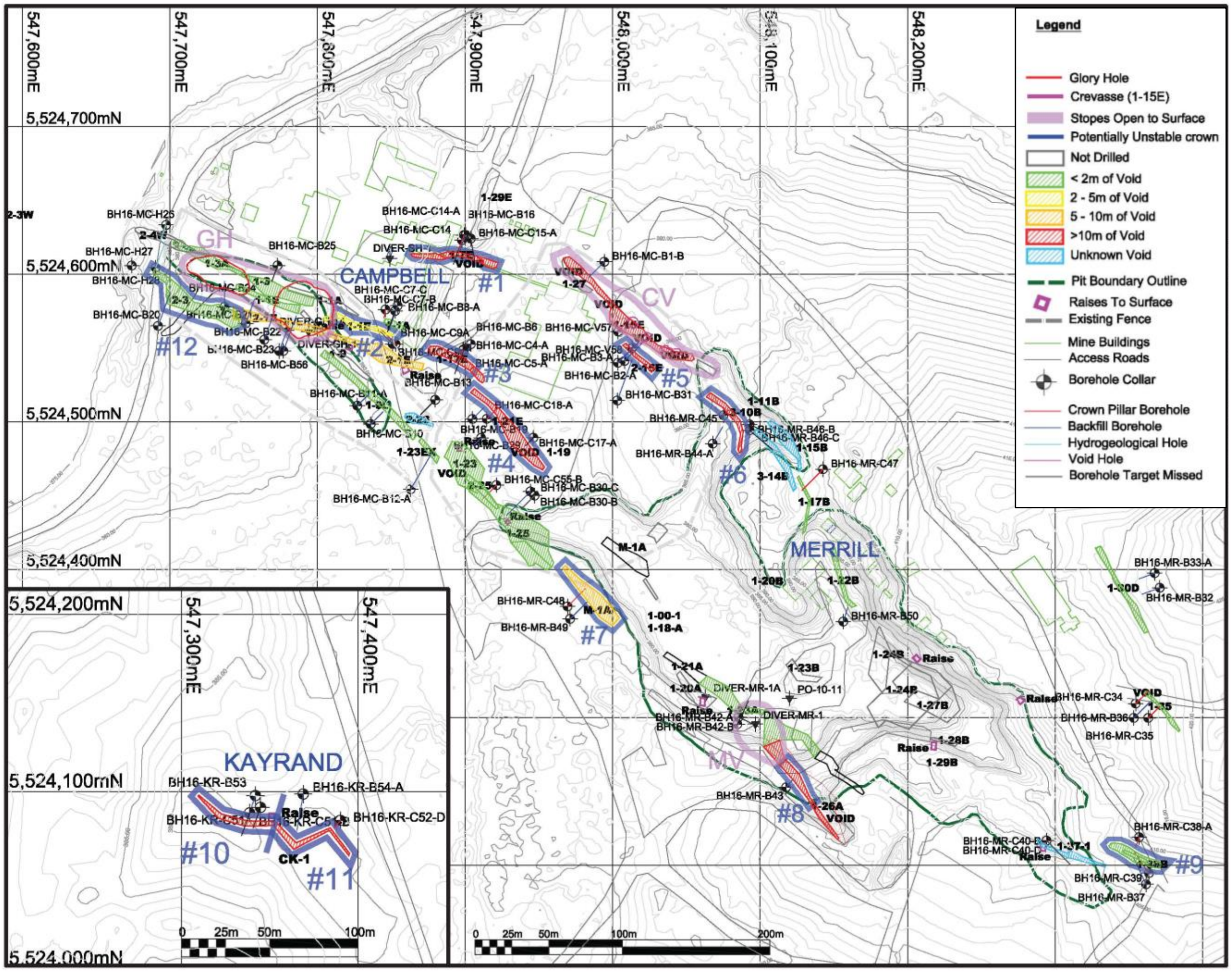

Figure 5: Risk assessment map showing estimated stope void heights and identifying potentially unstable stopes / crown pillar areas for further analysis

In presence of backfill:

- If there is some resistance after intercepting the stope, drilling was changed to triple tube (NQ3) to attempt to sample the backfill:
    - Attempt to advance the drill string without rotation. If advancement is possible with minimal resistance, then uncemented sand or tailings have most likely been used;
    - If the drill string will not advance easily without rotation, start rotation to obtain backfill sample:
        - If the backfill present is difficult to drill, then rock fill or cemented rock fill is probable;
        - If the backfill present is moderately difficult to drill, then cemented tailings or sand fill is probable.

Note: if the backfill is cemented there is typically a good chance of material retrieval.

The determined hazard results obtained are summarized on a heat map (Figure 5), where void ranges are shown by targeted stope. Of the 25 stopes assessed (over the initial 29 targeted stopes), the following conclusions were drawn:

- Nine (9) stopes were found to be of no immediate concern and required no remediation;
- 12 stopes were found to be potentially unstable and required further analysis;
- Two (2) areas (encompassing four stopes) clearly required remediation: the Glory Holes (GH) and the Crevasse area (CV) or stope 1-15E.

The total void height for the stopes was estimated from the 3D model, updated with the data from the plumb-bob surveys; void volumes were estimated by multiplying the average cross-sectional area of the targeted stope by the stope length. The C-ALS survey data was used to provide a more complete void estimate for stopes 1-15E and 1-26A.

Table 4 summarizes the estimated void volumes by stope. The volume for stope CK-1 significantly increased as the initial estimations were modified based on a revision of the historical plans and probable failure of a very thin sill pillar with the realization that the entire stope contains no backfill. It was also concluded that some of the plumb-bob surveys most likely came to rest on the footwall without fully reaching the bottom of the stope.

Table 4: Void assessment by stope

Crown Pillar Nº.	**Stope**	**Void Assessment (m^3)**	
		Preliminary	**Updated**
CV	1-15E	19,945	16,803
3	1-17E	2,735	3,121
4	1-21E	4,874	5,794
7	M-1A	3,553	3,553
10/11	CK-1	5,829	15,670
8	1-26A	4,574	378
GH	Glory Hole	9,623	9,039
	Total	**51,133**	**54,358**

4.2 Risk Mitigation

Following the site investigation, empirical stability analyses were performed to help mitigate the risk of an unstable stope or crown pillar. Analyses were performed on the 12 stopes/crown pillars identified in Figure 5, for which the majority (seven) showed void heights greater than 10 m.

4.2.1 Open Stope Stability Assessment

Stability assessment of potentially unstable stopes identified at the site was performed using the modified stability graph method (after Potvin, 1988, Nickson, 1992 and Hadjigeorgiou *et al*., 1995). This method was applied to the back, hangingwall (HW) and footwall (FW) surfaces of each stope. Based on the rock mass quality and stability numbers obtained, the following stopes were determined to be unstable: CK-1 (HW/FW), 1-15E (HW/FW), 1-17E (HW), 3-10B (HW/FW) and 1-1E (HW) or Glory Hole (GH).

4.2.2 Crown Pillar Stability Assessment

The Scaled Span Method (after Carter *et al.*, 2008) was used to determine the factor of safety (FoS) and probability of failure (PoF) for each crown pillar deemed unstable. For more accuracy and sensitivity, the method was modified from a single crown span, to review several cross sections perpendicular to the stopes' strike lines to better capture variations in stope geometry and identify the unstable portions of the stope with respect to variation in rock mass quality. This methodology allowed the client to remediate only selected portions of a stope instead of the entire stope, significantly reducing remediation costs. Variations in rock mass qualities (higher bound HB and lower bound LB corresponding to +/- one standard deviation, respectively) and of water inflow rates were incorporated into the analyses for additional sensitivity. The FoS for the crown pillars are shown in Table 5.

Note that the cross sections were cut on average every 10 m and labelled with increasing numbers towards the west for reference. The exception was stope 1-1E, where the reference sections increased towards the east.

For closure, a minimum FoS of 1.75 to 2.0 should be considered; however, if stopes are backfilled to within 2 to 5 m of the back, depending on the crown pillar thickness, any failure propagating to surface can typically be retarded assuming chimney failure occurs.

The scaled span crown pillar analyses indicated that the following stopes or portions of stope require remediation: a) the western potion of 1-15E (Sections 98 to 108), b) western portion of 1-1E (Section 5) or GH West, c) stopes 1-17E, d) 1-21E & e) M-1A and, f) eastern portion of CK-1 (Sections 10 to 30). As stope 1-26A was mined to surface and evidence of backfill loss (arching effect) was observed from the open pit and confirmed via the laser survey, it was also considered to require remediation.

Table 5: Scaled span crown pillar Factor of Safety (FoS)

Crown Pillar No.	Stope	Section	LB Q		Avg Q		HB Q	
			Low[1]	Med[2]	Low[1]	Mod[2]	Low[1]	Mod[2]
			Inflow		Inflow		Inflow	
CV	1-15E (west)	108	1.5	1.4	1.6	1.4	1.7	1.5
		98	1.5	1.3	1.5	1.4	1.6	1.4
1	1-7E	All[3]	2.8	2.5	3.0	2.7	3.2	2.8
2	1-1E	5[4]	1.1	1.0	1.3	1.1	1.4	1.2
		25[5]	2.5	2.2	2.9	2.6	3.2	2.9
3	1-17E	30[6]	0.9	0.8	1.5	1.4	1.8	1.6
4	1-21E	70	1.9	1.7	2.1	1.9	2.3	2.0
		60	1.7	1.6	1.9	1.7	2.1	1.9
		50	1.4	1.3	1.6	1.4	1.7	1.5
		40	1.1	1.0	1.2	1.1	1.3	1.2
		30	1.0	0.9	1.1	1.0	1.1	1.0
		20	1.0	0.9	1.2	1.0	1.2	1.1
		10	1.2	1.1	1.4	1.2	1.5	1.3
5	2-15	20[3]	4.8	4.3	5.4	4.9	5.9	5.3
6	3-10B	40[3]	3.3	2.9	3.7	3.3	3.9	3.5
7	M-1A	50	1.9	1.7	2.0	1.8	2.1	1.9
		40	1.3	1.1	1.3	1.2	1.4	1.3
		30	1.0	0.9	1.1	1.0	1.1	1.0
		20	0.9	0.8	1.0	0.9	1.1	0.9
		10	1.4	1.3	1.5	1.3	1.6	1.4
8	1-26A	Mined to surface, partially backfilled						
9	1-38B	40	2.8	2.5	3.1	2.8	3.4	3.1
		30	3.2	2.9	3.7	3.3	4.0	3.6
		20	1.6	1.5	1.9	1.7	2.0	1.8
		10	1.7	1.5	1.9	1.7	2.1	1.9
10/11	CK-1	100	2.9	2.6	3.4	3.1	3.8	3.4
		90	2.5	2.3	3.0	2.7	3.3	2.9
		80	2.1	1.9	2.5	2.3	2.8	2.5
		70	1.6	1.5	1.9	1.7	2.1	1.9
		60	2.0	1.8	2.3	2.1	2.5	2.3
		50	2.1	1.9	2.5	2.2	2.8	2.5
		40	2.3	2.0	2.7	2.4	3.0	2.7
		30	1.7	1.5	2.0	1.8	2.2	1.9
		20	1.4	1.3	1.6	1.5	1.8	1.6
		10	1.1	1.0	1.3	1.2	1.5	1.3
12	2-3	50[3]	1.7	1.5	1.9	1.7	2.1	1.9

[1]: Jw is equal to 1.0 (dry or minor water inflow – flooded and below the water table)

[2]: Jw is equal to 0.66 (moderate water inflow > 5 l/min)

[3]: Lowest Factor of Safety shown. All well above requirements

[4]: Most Western part of the stope corresponding to the West GH

[5]: Section 45 to 15 all above min FoS

[6]: All Sections below min FoS. Lowest FoS shown.

5 REMEDIATION AND MONITORING

Remediation of the near-surface openings consisted of collapse-blasting (where necessary) and surface backfilling, backfilling via borehole, crown pillar reinforcement, pit wall scaling and raise capping with concrete slabs.

5.1 Remediation Plan

Following the integration of the site investigation findings into the 3D model, and a review of the risk assessment and empirical analysis results, a remediation plan was established. It consisted of securing the stopes/openings as follows:

- Stopes 1-26A, 1-15E & 1-21E: filled via blasting/backfilling from the surface;
- Stopes 1-17E and M-1A: backfilled from boreholes;
- East Glory holes: backfilled from the surface;
- Crown pillar of CK-1: anchoring of the crown with galvanized cable bolts;
- Kayrand raises: concrete cap replacements;
- Merrill North pit wall: scaling of walls and re-sloping overburden at crest;
- Raises 1-37-1 and 1-23A: backfilled from surface.

Backfill quantities for the remediation were estimated from the 3D model and then increased, by 10 to 20%, to obtain quotes from contractors. The suggested securing methods for each near-surface opening were reviewed and finalized in discussions with the MERN and the contractor to meet the project objectives and schedule at a reasonable cost.

5.2 Methodology

Prior to remediation, it was necessary to draw the ground water level below a certain elevation (360 m) to consolidate the backfill material already present in the stopes prior to adding more, or to ensure the water table was drawn down below the back of the stopes to be blasted to provide an air void and reduce coupled shock vibrations to the water and other openings.

The water drawdown is shown in Figure 6, which indicates the water level was drawn down over a period of four weeks. The first week of pumping was performed using one 95 HP pump, which was subsequently replaced by two 60 HP pumps after a mechanical breakdown. Water levels were measured at several locations to identify any variation across the site and were compared to the lake level. Data loggers were used along with a water tape for cross-verification.

A safety perimeter was established around each stope to be secured based on potential subsidence. Concrete blocks of 0.6 m x 0.6 m x 1.2 m (height x width x length) were positioned at 2 m spacing (centre to centre), 2 m away from the edge or back projection of the openings on the surface to prevent equipment or personnel from working on or near unstable ground.

Depending on the proximity to the surface (thickness of the crown pillar or depth of the opening) and the strength of the rock mass above the workings, different methods were planned to secure the stopes. The two main remediation techniques included blasting and backfilling, or a combination thereof.

Backfilling was performed in two ways: 1) from the surface, using an excavator and a dozer, and pushing material into the stope when a large enough opening was available (with or without additional blasting) or 2) via 12-inch boreholes, when a thicker layer of overburden or a rock pillar was present above the stope.

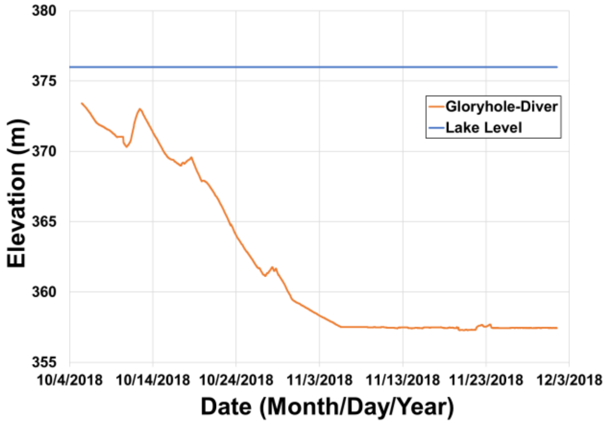

Figure 6: Water level drawn down over time

In some instances, surface openings needed to be enlarged or very narrow crowns between openings needed to be blasted to facilitate the backfilling operations. Sections of the hanging walls were drilled and blasted to enlarge the openings without compromising the stability of the surrounding areas. Additional percussion drilling (using an air track-mounted drill rig) was performed in some locations to confirm the location of the stope back or blasted rock had adequate fragmentation ratios and had collapsed into the stope without bridging.

Non-acid generating mine waste was used to backfill the stopes. The material was composed of two types: coarse (diameter between 2.54 cm and 50 cm) or fine (diameter smaller than 2.54 cm). A combination of coarse and fine materials was used to backfill from the surface, while only fine materials were used in the boreholes to prevent bridging. Backfill material was sourced from the Ile Pilcher stockpile, located less than 3 km north of Mine Principale.

Backfill material had to undergo a process of screening and crushing, temporary stockpiling, surveying for quantity checks, and transportation by trucks prior to being placed in the stopes.

While backfilling via boreholes, in order to avoid obstructions and facilitate the placement of the fill in the stopes, a series of wide-diameter boreholes (12-inch) were drilled using a truck-mounted rotary drill rig with hydraulic heads. The backfill holes were oriented with a dip of 70° to offset the rig from the top of the unstable stopes and were spaced approximately 10 m apart along the strike to maximize material deposition in an estimated 36° deposition cone. A 2-inch-diameter pilot hole was drilled at every backfill hole location with a conventional diamond drill rig to confirm the stope location prior to drilling a large-diameter hole. The position of the backfill hole was adjusted based on the pilot hole and offset by about a metre along the strike to optimize the backfilling operation.

Where necessary, a geocamera was used to assess the stope backs, the height of voids and water levels in the stopes prior to backfilling. During backfilling, a portable crusher was used at the collar along with a conveyor belt to regulate the size and flow of backfill into the stope. Once the openings were backfilled, plumb-bob surveys and surface surveys were performed to confirm the backfill levels in the boreholes or the stopes respectively. Final fill quantities were assessed and crosschecked with the help of an independent surveyor.

5.3 Remediation by Stope / Area

Stope 1-26A is located inside the Merrill pit at its south-east end. This stope was mined to surface and showed signs of backfill deterioration; it was decided to collapse-blast around this stope to improve long-term stability and remove the continuation of the progressive failure.

Stopes 1-15E and 1-21E, which had factors of safety as low as 1.3 and less than 1, respectively, are located close to surface. These stopes had a maximum of 7 m of overburden, composed of gravel and tailings for stope 1-15E and 4 m of mine waste for 1-21E. To secure these stopes, the overburden was excavated and the crowns or side pillars (in between crevasses) were blasted, and the stopes backfilled from the surface, first with coarse material (to within 2 m from the surface) and then capped with fine material (Figure 7).

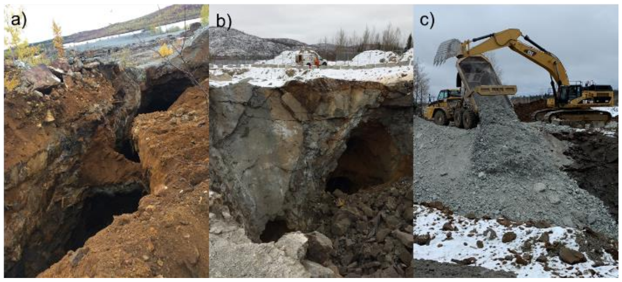

Figure 7: Stope 1-15E a) Initial crevasses b) Opening exposed after blast c) Backfilling

The top soil/fill excavated to access the stope crowns was then replaced on top of the fill to help revegetate the area.

Stopes 1-17E and M-1A had factors of safety less than 1. They were found at depth, approximately 17 m and 21 m (bgs). The stopes were covered with backfill and construction debris from the destruction of either the mill or the office buildings on site, consisting of dry/dangerous material such as wood, metal and possibly asbestos (as this side of the pit had been used as a landfill during demolition). Stope M-1A had uneven rock cover due to the benching on the south pit wall above it. These stopes were backfilled via large-diameter (12-inch) boreholes as described earlier (Figure 8).

The East and West Glory Holes are located inside the former Campbell pit. The East Glory Hole was larger with a depth of 10 m to fill; the West Glory Hole was only 4 m deep. To date, only the East Glory Hole has been secured via backfilling from the surface. The West Glory Hole is still being used as a retention pond and for water quality monitoring.

Kayrand Mine is located approximately 500 m south west of the Campbell Mine. The eastern sections of stope CK-1 have factors of safety as low as 1.

Figure 8: Drilling of backfill hole (12-inch) at stope M-1A

As the stope is in a remote area, with an average 7 m of rock cover (potentially acid generating), galvanized Garford bulge cable bolt anchors have been chosen to reinforce the crown pillar. This work is planned for June 2019. Note that backfilling of this stope was not considered cost-effective due to the large volume of material required. Two raises were encountered at this location; to comply with current legislation (MERN, 1997), they were capped with new concrete slabs.

For the north wall of Merrill Pit, the upper benches and walls were professionally scaled using a combination of specialized backhoe and personnel on ropes to remove loose blocks on the face and benches. The overburden within 3 m of the pit crest was sloped using a small backhoe to prevent material sloughing over the pit crest. Additional measures, such as the construction of a boulder fence, a geotechnical berm with trenches, and posting of signage required to secure the rest of the open pit will be performed at a later date. Two (2) raises that daylighted within the easily accessible areas of the pit (1-37-1 and 1-23A) were also backfilled with fine material.

5.4 Remediation Results

Records were kept of quantities used to backfill the stopes. Table 9 presents a summary of the estimated quantities versus the actual ones used to complete the work.

In almost all cases, the fill quantities were overestimated by a factor of 2 or more when compared to the actual volumes. This is a consequence of constructing the 3D model based on limited plans and sections available but in all cases the model was accurate in confirming stope thicknesses. The estimated fill quantities were necessarily conservative and assumed that the open stope walls may have sloughed (Pakalnis, 2002). In addition, it was noted at the time of the investigation that uncemented sand/tailings backfill could act as a fluid and rise up the intersected boreholes by a few metres. As a result of this it was assumed in the volume calculations a certain amount of consolidation and settlement during and after backfilling. Remediation of more recent mines would benefit from the availability of up-to-date records, including Cavity Monitoring System (CMS) surveys.

Table 9: Summary of backfill material quantities

Stope	Materials[1]	Quantities (m^3)		
		3D Model	Specifications	Actual
1-15E	Overburden	-	2 360	2 310
	Coarse backfill	16 803	13 443	4 235
	Fine backfill		6 721	2 941
1-17E	Fine backfill	3 121	4 213	1 383
1-21E	Overburden	-	1 700	4 525
	Coarse backfill	5 794	6 065	1 344
	Fine backfill		3 457	0
M-1A	Fine backfill	3 553	4 264	3 306
GLORY HOLES	Coarse backfill	9 039	9 039	3 913
	Fine backfill		4 520	1 821

[1]: Coarse material (particles diameter comprised between 2.54 cm and 50 cm) and fine material (particles diameter smaller than 2.54 cm)

5.5 Ongoing Monitoring

Monitoring at site is ongoing to assess backfill settlement and ground water levels.

Three (3) concrete blocks (0.6 m x 0.6 m x 1.2 m) were installed on the fill above each of the 1-15E, 1-21E and the East Glory Hole stopes. A survey pin was installed in each block and will be surveyed about three (3) times per year to monitor any settlement. Additional fine backfill material will be added as necessary prior to placement of the top soil.

Backfill levels in stopes 1-17 and M1-A will be checked using the plumb-bob tool in the large diameter boreholes. If settlement in the stopes occurs, additional fine material will be added to the boreholes.

Finally, two to three groundwater pumping cycles have been recommended for 2019 using the pump in the Campbell Shaft. These pumping cycles are intended to improve backfill consolidation in each stope, thus limiting further settlement over time.

6 CONCLUSIONS

Modern mine closures are performed with the objective of securing a site, in a cost-effective manner, and returning it to an acceptable condition for eventual reuse. For the Mine Principale site, the Quebec government is looking to provide restricted access to the land upon completion of the remediation work, mainly for hunting and fishing purposes only. Future remediation works proposed for the site will include the rehabilitation of the tailing paddocks, relocation of contaminated soils, reduction of seepage through dam reconstruction and securing of the open pit and the Campbell shaft.

This paper illustrates how a complementary site investigation was conducted and used to establish a site mitigation plan based on a detailed review of historical drawings. Success rates for this type of approach will vary from site to site and are linked to the accuracy of the 3D stope models that can be constructed, as well as on the experience of the field staff in understanding and assessing the void heights and backfill levels within the underground workings. It should be noted that 95% of the targeted stopes were intercepted within a few metres of their anticipated locations. The time spent on construction of a representative 3D mine model was essential in the success of this project. Empirical stability-analysis methods, with variable stope geometry, rock strength properties and jointing orientation, were used to refine the mitigation plan. Ultimately, by performing numerous sensitivity analyses, the Ministry was able to limit the extent of the remediation program by reducing the number of near-surface stopes to be secured and by targeting only the portions of the stopes deemed unstable.

Out of the 29 stopes and 12 crown pillars analyzed, 5 stopes (excluding the Glory Holes) were found to be unstable and requiring remediation. Securing works started in the fall of 2018 with the successful blasting and filling of three crown pillars (1-15E, 1-21E and 1-26A), the backfilling of two stopes (M-1A and 1-17) via borehole, and the scaling of the pit face. The Glory Hole West was also backfilled from the surface, and two raises at Merrill were backfilled and two at Kayrand were sealed with concrete caps. Remaining work will resume in June 2019 to reinforce one crown pillar (CK-1) and monitor backfill settlement and ground water levels. This will complete the securing of the near-surface openings deemed unstable at the site. It is estimated that the field work required will be completed in less than four months.

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