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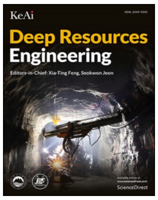

Notch formation in vertical excavations in a deep hard rock mine and rock stabilization methodologies

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ABSTRACT

This paper presents details of the early to mid-stage deterioration in the form of notch growth of two 3 m diameter bored raises that were excavated and slashed to 7.8 m to serve as an internal shaft (winze). In addition, the long-term deterioration of a series of ore/waste passes that were excavated with the bottom-up Alimak methodology is investigated. These excavations were completed at depths between 1200 and 1915 m below the surface in a hard rock mine in Sudbury, Ontario, Canada. The ability to see a cross-section of fracturing around the bored raises while slashing into them and observing the progressive growth of fractures was critical for understanding the factors leading to deterioration. The Alimak-driven ore/waste passes were in an advanced state of deterioration, which provided a late-stage notch growth profile that could be compared with the notch growth observed in the bored raises. The methods used to support and stabilize these excavations are detailed; in addition, methodologies to prevent some of the deterioration from occurring are also presented. The findings of this work are useful for understanding the mechanisms driving brittle rock failure in deep mining and subsequently for minimizing or preventing the deterioration from occurring.

1. Introduction

The Sudbury Basin in Ontario, Canada is one of the largest base metal mining districts in the world. Mining has been ongoing in the region since the late 1800s [1]. Easily exploitable near-surface deposits were mined first; however, as these shallow deposits were exhausted, mining has progressed to significant depths. Many mines currently in operation in the Sudbury Basin are at depths greater than 1500 m. At these depths, stress-induced rock fracturing is unavoidable and, in some cases, can lead to violent rock failure known as rockburst.

Vertical excavations, also known as raises, are a key component of underground mine infrastructure [2]. There are several uses for raises, such as ventilation, material movement, and egress. Historically, raises were driven with the Alimak methodology, whereby the excavation is blasted vertically from bottom to top and supported in-cycle. However, due to an increasing number of incidents, especially with increasing depth, the Alimak methodology has been slowly replaced by the raise boring methodology (mechanical boring) to eliminate exposure of rock failure to operators. In addition to bored raises having a fundamentally different rock mass response compared with a drill-and-blast excavation [3–7], bored raises are pulled over long distances and are left unsupported for long durations compared with the more incremental blast/support Alimak methodology. The Sudbury Basin has high horizontal stresses, which makes vertical excavations more susceptible to stress-induced damage than lateral development orientated with their axis sub-parallel to the maximum horizontal stress. Therefore, at increased depths, bored raises can be unstable as a result of high stresses, which can be exacerbated by the presence of other adverse conditions such as geological structures.

Knowledge of brittle rock failure in the laboratory setting is important for understanding rock failure in the field. Numerous studies have been conducted to understand the mechanism of brittle rock failure in a laboratory setting [8–10]. Several stress thresholds have been identified for brittle rocks under compression. At a stress level of 0.3–0.5 σ_c (σ_c – uniaxial compressive strength (UCS)), microcracks will initiate [11–14]. Fractures propagate parallel with the orientation of the maximum compressive stress. Crack interaction and coalescence occur at a stress level above 0.7–0.8 σ_c [15]. In addition to laboratory study, direct field observation of excavations in deep underground mines provides a useful approach to studying brittle rock failure. The field observations are

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essential to understand the large-scale rock mass behavior which cannot be represented in the laboratory.

Brittle rock failure in bored raises is a common occurrence in deep mines and has been studied extensively. Stacy and Harte [16] stated the main problem associated with raise stability was unfavorable geological structure and stress-induced rock fracturing. McCracken and Stacey [17] identified general raise instability attributed to wedge failure, overbreak, unravelling, squeezing, swelling, stress-induced spalling, and time-dependent deterioration. They proposed a geotechnical risk assessment method based on the Q-system. Coombes et al. [18] and Peck et al. [19] completed a review of raises in Australia and found that the presence of blocky rock mass structure affected raise stability greatly. Edelbro et al. [20] completed a review on raise boring in difficult ground conditions and found that for weaker rock masses it can be difficult to keep the excavation stable due to the time lag from excavation to being able to install rock reinforcement [21].

In addition to ground conditions leading to damage growth, raises that are used for material flow, such as ore and waste passes are subjected to impact damage from material falling and hitting the walls, which accelerates growth in vertical excavations. Hadjigeorgiou et al. [21] described how the integrity of a raise is governed by ground conditions, pass configuration, wear from falling material, operational considerations, and ground support. The work provided an estimate of pass longevity based on several passes that experienced deterioration. That work was extended to examine passes of various shapes, lengths, dips, sizes, and tonnage throughput in a number of Canadian mines and summarize the stability of these vertical excavations as well as methodologies to monitor damage growth [22]. Esmaieli et al. [23] detailed how damage around finger raises of material passes is greater than in other areas and investigated various configurations of passes to mitigate wall damage. They also investigated raises at Brunswick mine and concluded that structural, stress, and flow-induced failure mechanisms exist but the impact-damage failure mechanism is more dominant [24]. Sjoberg et al. [25] detailed that failure in passes in Kiirunavaara mine increases in frequency with the increase of depth; several controlling mechanisms were identified, leading to the establishment of a model that could be used to determine the longevity of a given ore pass.

Due to the high capital costs associated with raises, especially bored raises, and the vulnerability of the raises to deterioration, it is of utmost importance to better understand and predict the rock mass response when excavating a raise and factors such as scouring or the presence of structure that can lead to deterioration. Proactive measures can be taken, such as drilling holes ahead of the excavation to provide cores, which can be used to examine lithological units, structure, and stress anomalies (core disking). Utilizing a televiewer in the hole can provide an orientation to the structure and breakout information and orientation, which can further define the path of the raisebore. At increasing depths, the interpretation of borehole breakouts must consider both stress and rock strength. No breakout may occur in a significantly stronger rock mass, which can increase the potential for strainbursting to occur.

During the construction of an underground shaft, also known as a winze, at the Craig Mine Onaping Depth Project located in Sudbury, Ontario, Canada, severe raise wall deterioration in the form of notch growth occurred. The work of Hall et al. [26] describes the various mechanisms of deterioration that caused wall failure and notch growth in the first Onaping Depth bored raise. That paper was published before slashing had reached the bottom of the first raise; therefore, there were still lessons to be learned from an execution point of view, valuable observations that were made, and additional examples of the deterioration mechanisms. This paper builds on the previous work, detailing the controls and methodology used to safely proceed to the bottom of both raises. The tactical controls used to protect the operators working in the shaft at various stages of raise growth are presented and methodologies to prevent large-scale failure from occurring are discussed. These include the formation of a large void at the bottom of the first raise and the successful prevention of the void forming at the bottom of the second raise, which resulted in a significant reduction in operational risk.

By examining the evolution of growth in the raises, the early to late-stage phases of deterioration could be examined and measured. This work allowed the authors to develop a strategy for rehabbing a separate series of waste passes that exhibited long-term deterioration from millions of tonnes of rocks passing through. Based on this work, a better understanding of the potential failure that can occur in a raise driven in brittle rock is outlined and successful tactical controls and means of limiting growth can be gained.

The rest of the paper is organized as follows: the site where data collection was completed is introduced in Section 2. In Section 3, the mechanisms of deterioration observed in the vertical excavations, which led to notch growth, are discussed. Several methodologies to stabilize vertical excavations are presented in Section 4, and the conclusions are presented in Section 5.

2. Stability expectation, early observations, and data collection

The mine site where the data were collected, is Glencore's Onaping Depth Project within Craig mine. In 2017, lateral development began from the existing Craig mine infrastructure to the location of the new internal winze, which was used to access the Onaping Depth deposit. The winze is approximately 1.2 km away from the main Craig mine shaft. A long section of the site is presented in Fig. 1. A significant amount of time is required to build the infrastructure for an underground headframe and associated material handling system, which includes bins and a rail system.

The time to complete the Onaping Depth headframe, hoist rooms, and the material handling system was 1.5 years. Two different methodologies were used to excavate the winze: raise slashing and conventional blind sinking. To expedite the shaft sink and perform concurrent work, a 3 m diameter raise was pulled from 1915 to 1455 m and another raise was pulled 15 m above from 1440 to 1150 m (all level names refer to depth in meters below surface). By pulling bored raises, the shaft could advance by slashing around the bored raise to the required diameter (7.8 m) and dropping the broken rocks down the hole, where they could be collected at the bottom and hauled up using a system of existing ramps and removed via the existing Craig mine material handling system. The addition of ground support and a concrete liner made a final winze diameter of 7.2 m.

This was a significant gain for the schedule because a conventional shaft sink methodology could not occur until all infrastructure was in place to remove the broken rocks after blasting. The 15 m sill between the two raises, located at 1440 m, allowed for a decoupling of the mucking horizons. A significant amount of excavation was completed on the bottom 1915 level while slashing was occurring in the top raise, and the separation allowed the excavations on the lower level to proceed without interruption from frequent blasting in the shaft. Once the shaft slash reached the 1915 level, a conventional blind sink methodology was used until the bottom of the shaft was reached at 2650 m.

As part of this project, a series of ore/waste passes (shown in Fig. 1), which were excavated in 1992 within the Craig mine infrastructure and were in an advanced state of deterioration, were rehabbed in 2021–2022 to serve as ore/waste passes for the Onaping Depth Project for a planned 15-year life of mine. Those passes were excavated 3 m in diameter using the Alimak mining method (rounds were blasted vertically) and were supported as the excavation progressed. There were two sets of three in-line passes from 1200 to 1435 m below the surface. Since 1992, approximately 15 million tonnes of material were dumped down these raises, causing significant notch growth throughout. The notches had grown beyond the length of the primary ground support in many areas but had stabilized as the notch tips narrowed. Similar to the winze/raisebore, the main direction of damage growth is perpendicular to the direction of the in-situ principal stress.

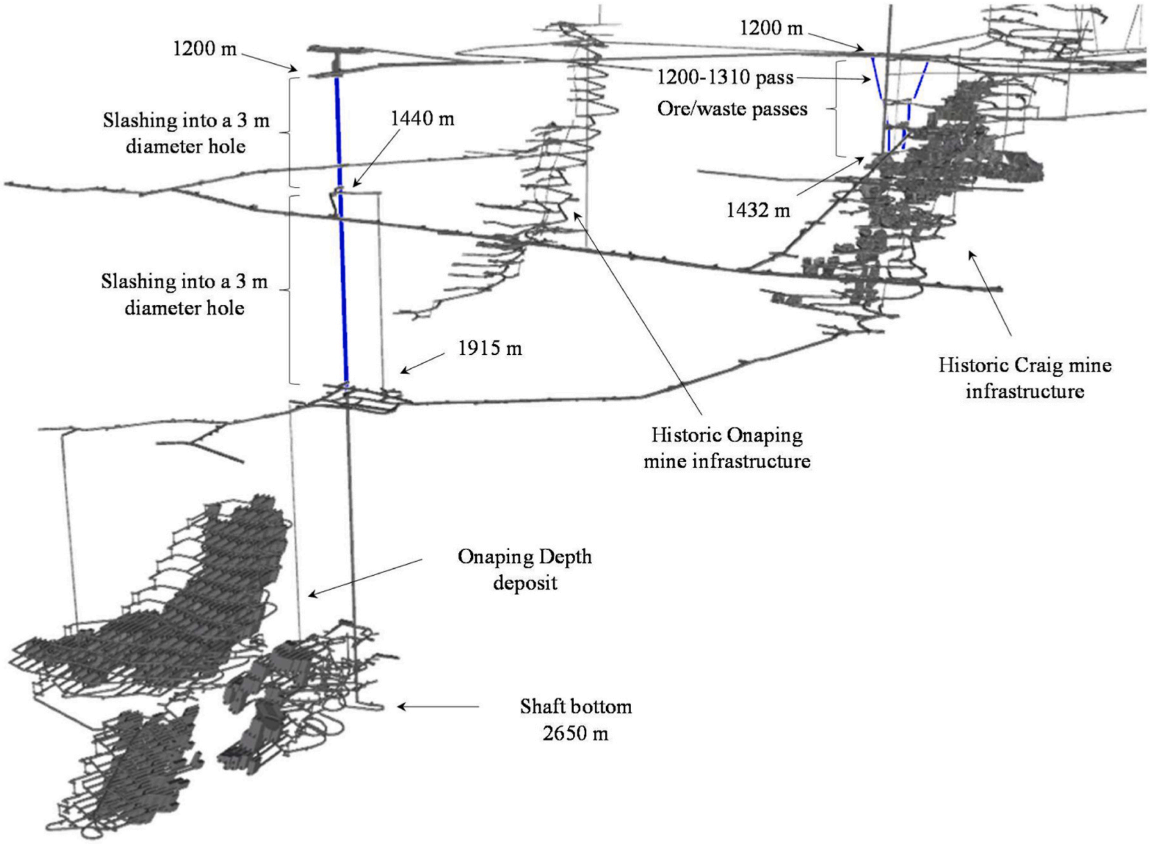


Fig. 1. Long section of Craig mine with the Onaping Depth deposit. Blue excavations are the focus of this paper.

The expectation for the raises was that there would be minimal growth of excavation geometry and they would be stable at these depths for the duration from excavation until the shaft slash was complete and a monolithic concrete liner was poured. Based on the in-situ stress condition at the site (vertical stress $\sigma_3 = \sigma_z = 0.027\,H$ (MPa), $\sigma_1 = 1.8\,\sigma_z$, $\sigma_2 = 1.1\,\sigma_z$, where H is the overburden depth in m), the maximum tangential stress ($\sigma_{max} = 3\sigma_1 - \sigma_2$) on the raise wall ranges from 133.5 to 222.3 MPa at the 1150 and 1915 levels, respectively.

Based on an empirical relation ($\frac{d_f}{a} = 1.25\frac{\sigma_{max}}{UCS} - 0.51(\pm 0.1)$), where $a = 1.5$ m for the radius of the raise, and d_f is the depth of failure) [27] and using the UCS of 225 MPa for Norite, the largest notch depths are 0.35, 0.63, and 1.09 m at the 1150, 1440 and 1915 levels, respectively, and the difference of the notch depths between the top and bottom sections of the raise is approximately 0.8 m.

High-precision LiDAR scans (accuracy ± 10 mm) were completed for both raises after the raise bore was complete and before any slashing occurred. The notch growth observed in the 1150–1440 scan was in line with the expectations stated above; however, the empirical assessment overestimated the depth of failure in the 1440–1915 raise, as can be seen in Fig. 2. There was slightly more notch growth than expected at the bottom of both raises. The immediate area above the 1440 level had grown to 1.25 m (5.5 m wide notch tip to tip) and the immediate area above the 1915 level had grown to 2.1 m (7.2 m wide notch tip to tip), which was isolated to the bottom 15–25 m of the raise. On average, the growth measured in the raises was 0.41 m if the data at the bottom of the raises and the data around a fault in the 1440–1915 section are excluded. This indicates that the stress assumption for the site is overestimating the field stresses at increasing depths.

As the raises were pulled, there was no large magnitude seismicity recorded on the microseismic system, but a significant amount of low magnitude (−2.0 to −0.5 moment magnitude) seismicity was recorded. Due to material falling and impacting the raise walls, it was difficult to differentiate between what was seismicity associated with stress-induced damage and what was a wall impact event. Spatially, events occurring within 10 m of the cutting head were assumed to be from stress spalling and events located down the raise were assumed to be rock impact noises. After the raises were complete, seismicity stopped and no material fell or was observed at the bottom of the raises, indicating that the raises were stable. The 1440–1915 section of the raise experienced no seismicity or spalling for two years, which indicated that the structure was stable. All notch growth in the raise occurred during the raise bore pull and time-dependent rock deformation was not a significant factor for notch growth.

After slashing down 100 m, both of the raises grew rapidly, and another LiDAR scan was taken. Fig. 3(a) presents a scan of the 1150–1440 raise before any slashing occurred; Fig. 3(b) presents a scan of the raise after slashing down 100 m; Fig. 3(c) presents three cross sections of the initial scan; Fig. 3(d) presents three cross sections of the scan after slashing down 100 m. The results of the second scan revealed that the notch growth was significantly higher than that inferred from the above empirical analysis, including a void at the bottom of the raise that was 14 m wide. This indicates that there are other mechanisms that govern deterioration in a bored raise aside from initial boring and subsequent spalling.

In 1992, two sets of three in-line ore/waste passes (see Fig. 1 for locations) were excavated with the Alimak methodology at an incline of 70°. The 3 m diameter ore passes were drilled and blasted and supported with weld mesh and 1.8 m long rebars fully encapsulated with resin, covered with 50 mm plain shotcrete. Throughout their life, approximately 15 million tonnes of rock were dumped down the ore/waste passes. Notch growth was not quantified in these passes until 2018 when LiDAR scans of the ore/waste passes were completed. These passes were also excavated in Norite with the same characteristics and strength as the bored raises discussed above.

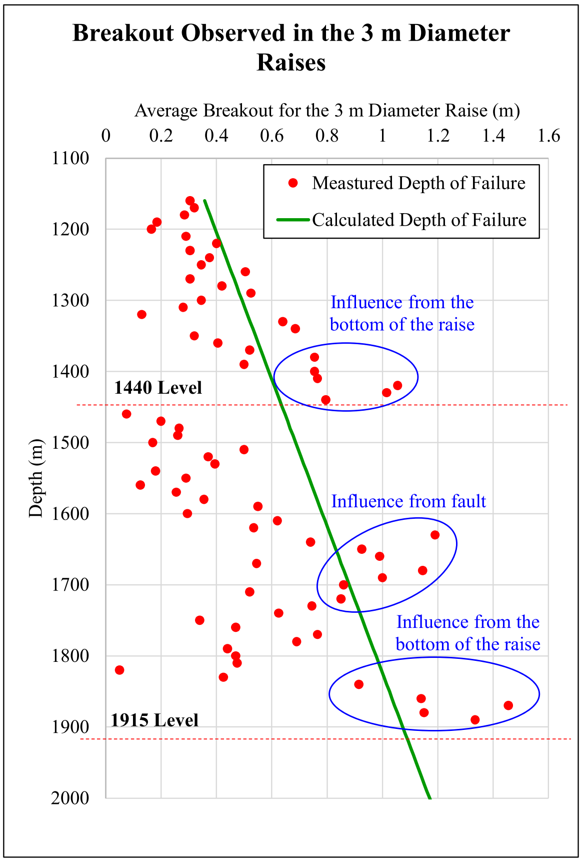


Fig. 2. Average breakout measured at 10 m intervals from the LiDAR scans of both raises with the empirical breakout estimate plotted at corresponding depths.

An image of the 1200–1310 ore pass scan with several cross sections is presented in Fig. 4. This was typical of the notch growth observed in the other ore (and waste) passes. The bottom of the ore pass was not picked up in the scan because it was full of muck for ventilation control. In the North and South walls of the raises the notch growth had exceeded the length of the rock support installed. However, the excavations stabilized as the notches narrowed. For this study, the 1200–1310 ore pass will be examined in detail because other raises exhibited comparable notch growth.

3. Mechanisms of notch growth in vertical mine excavations

3.1. Stress change as the raise was pulled

The bored raises and subsequence slashing provided an excellent opportunity to examine progressive deterioration in the form of notch growth from an early stage. Fig. 5(a) presents a LiDAR scan taken from the bench while slashing and Fig. 5(b) shows a schematic of fracturing around the raise that was visible in the bench. Notch growth in the raise occurred on the North and South walls (approximately 15° off North) because the orientation of the maximum in-situ stress is in the East–West direction. While slashing into the bored raises, a cross-section of the raise was visible in the floor of the slash, which allowed for examination of stress fracturing sustained to the walls of the raise from the original raise pull and the subsequent growth that occurred. Examination of the raise cross-section while slashing over the entire length of both raises showed that up to a maximum of 10–20 cm of heavily tensile fractured rock extending into the North and South walls ($<$ 5 cm spacing between tensile fractures), followed by low to moderate tensile fracturing extending up to 0.5 m into the North and South walls ($>$ 5 cm spacing behind tensile fractures). As the notches developed and narrowed, there was a reduced depth of fracturing visible compared with that when the raises were more intact. The fractures ran approximately parallel to the raise excavation, while the moderately fractured zone had more tangential fractures converging to the narrow apexes on the North and South walls. Minimal to no fracturing was observed on the East and West walls. Fig. 6 presents various views of the raise throughout the shaft slash.

Once the boring was complete, there was limited notch growth (average of less than 0.6 m) visible in the LiDAR scan for both of the raises. Near the top 50 m of the raises, the depth of notch failure was less than 0.2 m while the rest of the raise, aside from the bottom and the area around the fault between 1140 and 1915, exhibited up to 0.41 m. The lower 1440–1915 raise was idle for two years and during this time there was no growth observed in the raise and no fallen rock was visible at the bottom of the raise.

The stress-fractured rock played a significant role in the deterioration of the raises. For notch formation, stress needed to be elevated to cause fracturing of the rock. The stress fractures were the base point for deterioration to occur. However, these fractures also changed the stress field and prevented additional notch growth from occurring. As the rock mass fractured, there was a reduction in strength as the rock went from peak to post-peak strength. The reduction in strength was associated with fracturing, bulking, and spalling. As this occurred, there was a loss of cohesion and tensile strength in the rock (in the notched region), forming the stress-fractured rock. However, with high confinement provided by the notch geometry and the intact rock behind, there was a rapid strengthening due to frictional mobilization as the rock dilated [28,29]. As a result, the tensile fractured rock in the notch provided confinement to the rock behind it and prevented further stress fracturing. Therefore, the development of deep notches in the raises required other external mechanisms to promote failure.

3.2. Scouring deterioration

As discussed in the previous section, the notch growth observed in the raises could not be explained solely by stress-induced failure as the raise bore cut the rock. Other external factors were present, which resulted in significant notch growth in the raises. In addition, these external factors were not isolated to the slashing activity. The large void created above the 1440 station was identified when the slashing was 175 m above the level. It was evident that the external factors occurred rapidly and throughout the entire length of the raises – not exclusively at the mining face where slashing was occurring.

Scouring is the deterioration process by which material falling from higher up the raise impacts the walls leading to notch growth of the raise. Based on observations made in both raises, it is seen that scouring is the most significant factor for deterioration and affects the entire length of the raise below the point where broken rocks are being dumped. During the raisebore pull and subsequent slashing, it was impossible to differentiate between seismicity associated with stress-induced damage and material impacting the walls of the raise because their waveforms looked similar. ESG Consulting were engaged to examine the source mechanism wave polarity of the events in the raise, however the results were inconclusive. This could mean that the impact of material falling at a certain location causes effectively the same amount of damage as that associated with a seismic event of the same magnitude at that same location on the surface of the excavation. Fig. 7(a) presents the seismicity that occurred in the raise for one hour after a blast in the shaft, while Fig. 7(b) presents the seismicity for 24 hours after the same blast. During the 24-hour timeframe, operators mucked broken rocks into the raise (Fig. 7(c)) and then began bolting and scaling additional loose material, which fell down the raise. The material falling down the raise impacted the walls, causing seismic events to register throughout the entire length.

When slashing began, damage growth occurred rapidly throughout

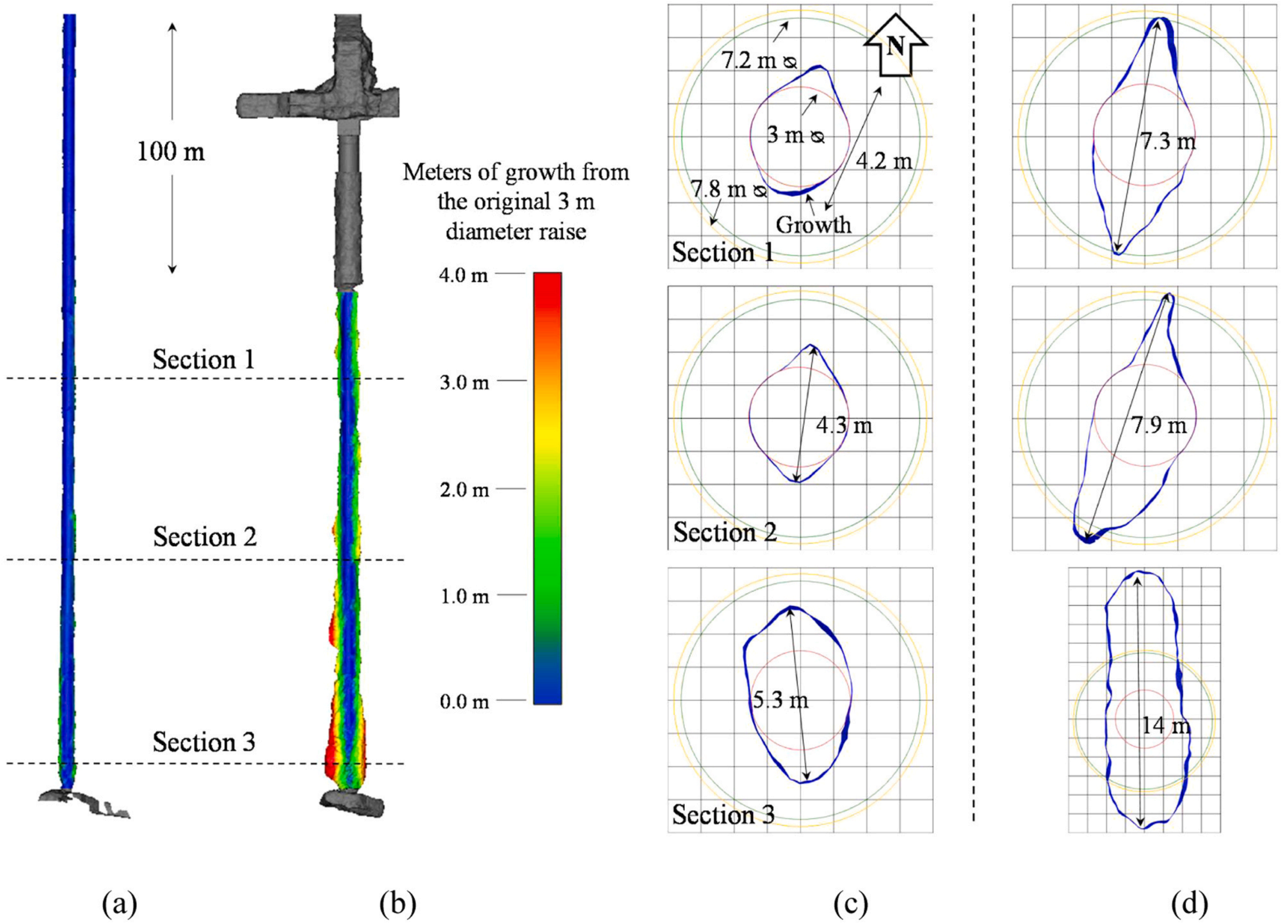


Fig. 3. Comparison of scans before (a) & (c) and after (b) & (d) slashing down 100 m with associated sections in the 1150–1440 bored raise. Note: the squares are 1 m × 1 m and the sections were clipped to 0.25 m.

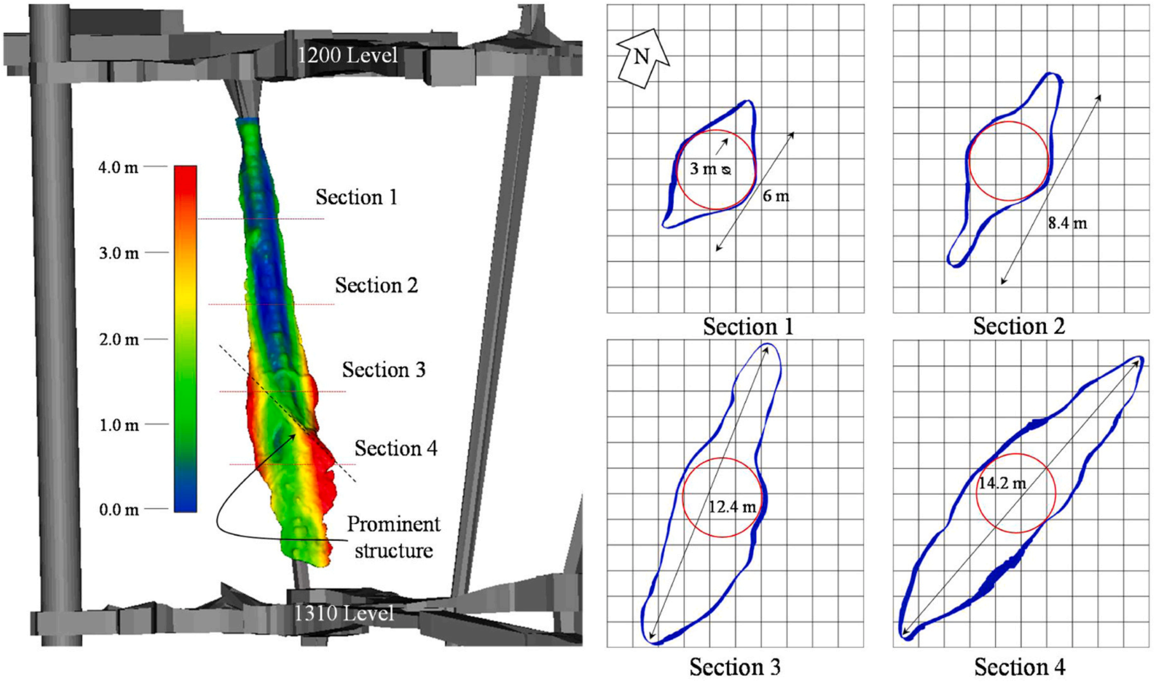


Fig. 4. Scan of the 1200–1310 pass – typical of growth seen in the other passes. Note: the squares in the sections are 1 m × 1 m.

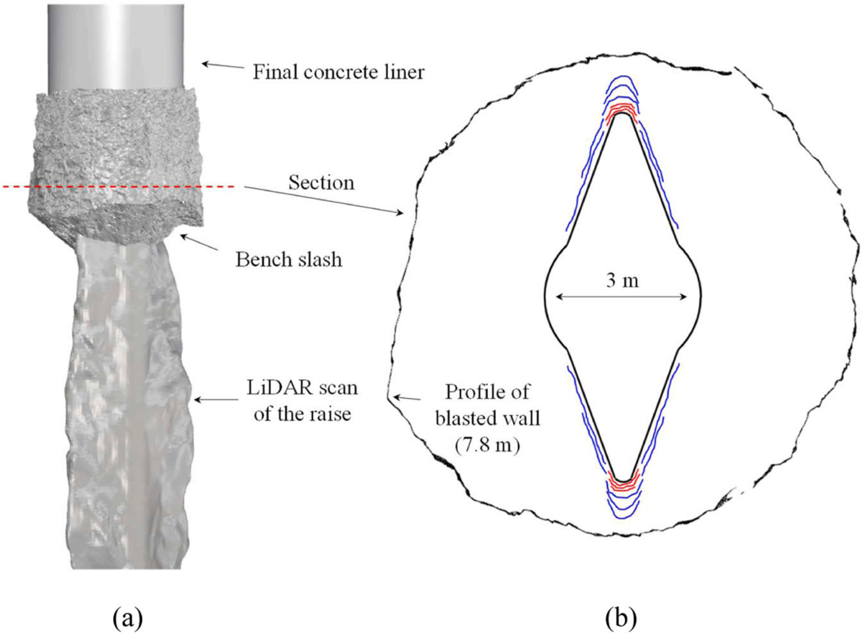


Fig. 5. (a) LiDAR scan of the raise and slashed shaft and (b) plan view schematic of fracturing in the wall of the 3 m diameter raise after notch growth had occurred. Note: red lines represent tightly spaced fractures and blue lines represent widely spaced fractures, visible up to a meter into the wall.

the entire length of the raise. Notch growth continued to favor the North–South orientations as these areas of the raise have the stress conditions necessary for spalling and bursting. As discussed in the previous section, the critically stressed North and South walls had stress-fractured rocks in the notches, which provided confinement to the notch tips and promoted stability. Material falling and impacting the walls at a high velocity removed the stress-fractured rocks. When this material was removed, there was a loss in confinement in the notch tips, which resulted in additional tensile stress fractures forming around the notch. As more material was removed, the process repeated, leading to additional notch growth of the raise. The relatively un-damaged East–West walls, which are parallel to the minimum principal in-situ stress direction, had only minor impact damage. Therefore, it is concluded that the impact forces from falling rock impacting the walls were not sufficient to fracture the intact Norite rock (with a UCS of 225 MPa) but were sufficient to scour previously stress-damaged rocks.

In both raises, there was rapid notch growth initially, but this slowed as the notch apex narrowed and eventually arrested as the notch tips narrowed. It was concluded that in the early stage of deterioration of the raise when the raise was more circular, there was a larger surface area of stress-fractured rocks exposed that could be impacted by falling rocks. However, as the notch tips narrowed there was less probability of large falling rocks getting into the notches and generating damage. In addition, as the notches narrowed, the high-stress conditions were pushed into a narrower area, which inherently had higher confinement and increased stability.

To illustrate the rapid growth followed by slowing and eventual arrest of growth when the notches narrowed, a set of LiDAR scans of the bottom of the 1150–1440 raise are presented in Fig. 8. While slashing down the first 100 m, rapid notch growth occurred throughout; however, after 100 m there was minimal growth observed for the remainder of the slashing. Fig. 8(a) presents a scan of the bottom of the 1150–1440 raise after slashing down 100 m, (b) presents another scan taken 4 months later after slashing down another 140 m, and (c) presents an overlay of both scans. The scans show minimal change in the volume of the raise at the bottom after the initial rapid notch growth had occurred; therefore, it can be concluded that the majority of damage was sustained early in the shaft slash.

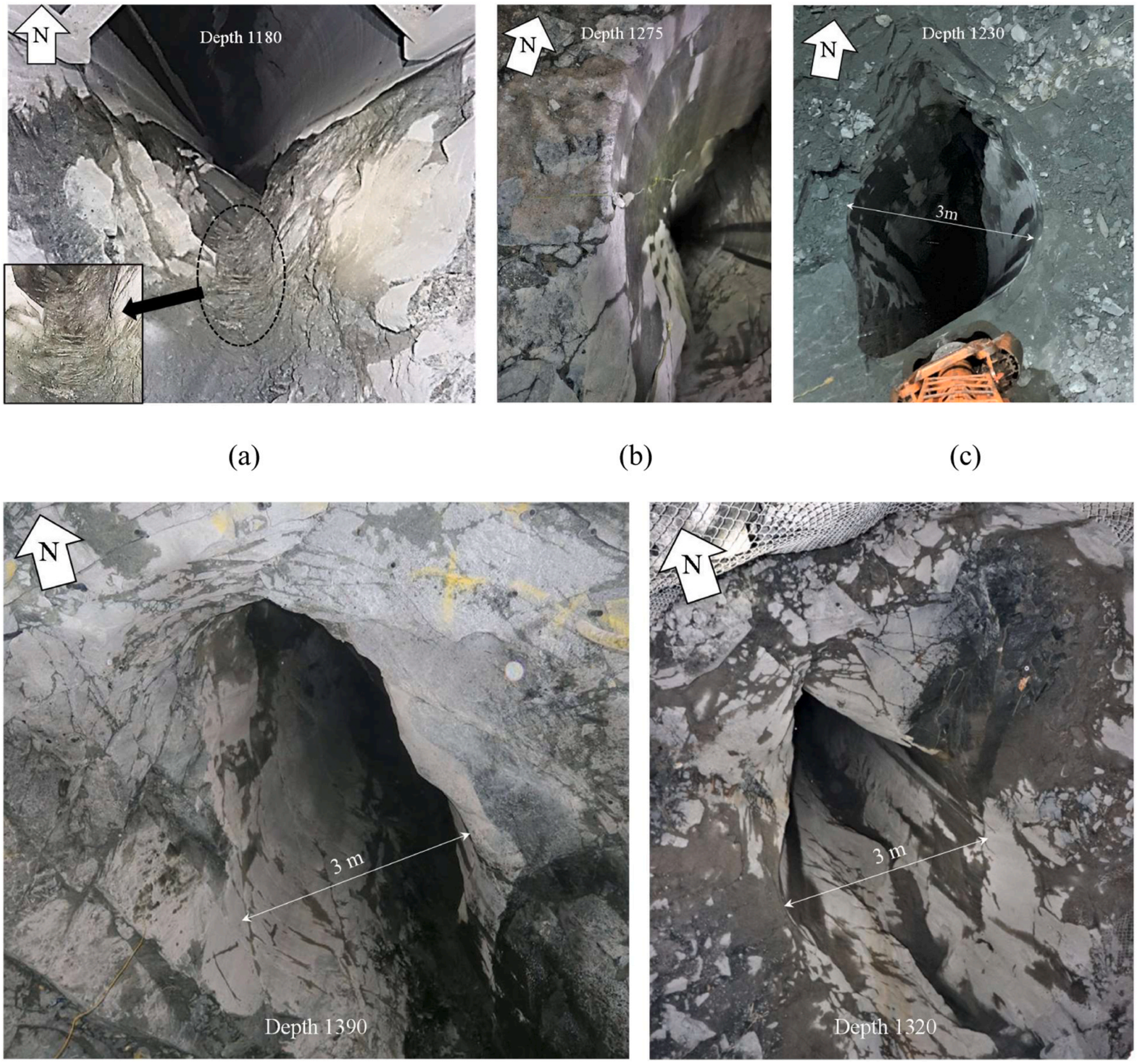
Fig. 9 presents various views of the ore and waste passes. The passes also exhibited relatively smooth walls on the East and West sides and extensive stress-fractured rocks on the North and South sides; therefore, it can be concluded that the same scouring deterioration mechanism occurred in the passes and exploited the stress-fractured rocks on the critically stressed walls. It is important to note that even though the ore and waste passes had millions of tonnes of throughput, the notches were not significantly deeper than the average depth observed in the newer bored raises. This is because that once the notch tips have narrowed, the high confinement at the notch tips promotes stability and the narrow geometry of the notches reduces the probability of broken rocks reaching the notch tips. As long as the low-stress East–West walls remain stable, the notch growth will stabilize once the narrow notch geometry is created.

For comparison, the empirical depth failure indicated potential growth between 0.4 and 1.2 m beyond the excavation in the bored raises at the 1150 and 1915 m levels, respectively, and between 0.5 and 0.7 m in the passes at the 1200 and 1432 m levels, respectively. On average, the growth in the bored raises was 2.2 m beyond the excavation (7.4 m notch tip to tip) after slashing down 100 m and the growth in the passes was 2.6 m beyond the excavation (8.2 m notch tip to notch tip). The maximum growth observed in both the bored raises and the passes was 5.5 m (14 m notch tip to notch tip). It should be noted that the empirical notch size values will underestimate growth in raises when the scouring deterioration mechanism is present because the empirical equation was developed based on case histories without scouring-induced deterioration.

3.3. Geological structure related deterioration

The presence of geological structures can have a significant effect on notch depth and orientation in the shaft. Specifically, the presence of sub-vertical joints, which pass through the critically stressed North or South walls of the shaft, will have the largest impact. Stress fracturing and scouring can exacerbate the effect of structure and, while slashing, these joints can be the culprit for strainbursting.

While slashing into the bored raises from 1150 to 1915 m, six strainbursts resulted in between 15 to 30 tonnes of rock to bulk into the ground support, which caused extensive rehab and delays to the shaft construction schedule. All the bursts had a structural mechanism of



(a) (b) (c)

(d) (e)

Fig. 6. Various views of the raise while slashing: (a) heavy tensile fracturing on the South wall, (b) no tensile fracturing observed on the West wall, (c) moderate to low tensile fracturing extending into the North wall, (d) tight stress fracturing in the apex of the notch dissipating with increased depth, and (e) overall fracturing around the raise.

failure related to long persistent sub-vertical joints. Two of the bursts had joint sets converging into a deeper notch, while the other four bursts were related to a single sub-vertical joint that was oriented approximately 5° – 30° off North. Of the six strainbursts, there were no early warning signs for five of the events because the joints were outside of the 3 m diameter raise barrel and were encountered only when slashing the larger barrel that exposed them. Given the vertical dip of the joints, diamond drill holes, also drilled vertically, were not able to delineate the vertical joint sets adequately.

Due to the planar geometry and the smooth characteristics of the joint surfaces in Norite, a joint can act as a release plane for broken rocks to slide along depending on the orientation of the structure. This is especially true if the joint is located on the North or South walls of the raise/shaft, where the walls are critically stressed and bulked stress-fractured rocks are located. As described in Sections 3.1 and 3.2, the stress-fractured rock is an assembly of thin slabs with random interlocking and endpoints. The intact rock behind the stress-fractured rock in the notches is confined by the fracture zone, thereby strengthened by confinement. However, it was observed that if a vertical joint runs through the stress-fractured rock, all of the fractured rocks can terminate on the joint surface instead of having random interlocking. This results in a greater chance of instability because a single termination on the joint acts as a plane of weakness along which the bulked material can slide.

Fig. 10 presents three examples of structural presence on the North and South walls of the shaft/raise. As the stress increases at the apex of the notch from slashing, the bulked material can slide along the joint surface manifesting as a violent burst. Typically, the bursts occurred with the blast or near after (< 5 min) due to proactive blasting practices, which will be discussed in more detail below.

A long planar sub-vertical joint was exposed in the 1200–1310 waste pass. Although initial notch growth along this structure was not historically documented, the same mechanism of failure occurred to generate notch growth in the passes as well. In the case of the passes, with significantly longer exposure to scouring, a more long-term, stabilized profile of the damage could be examined, which was consistent with the end state observed in the bored raises.

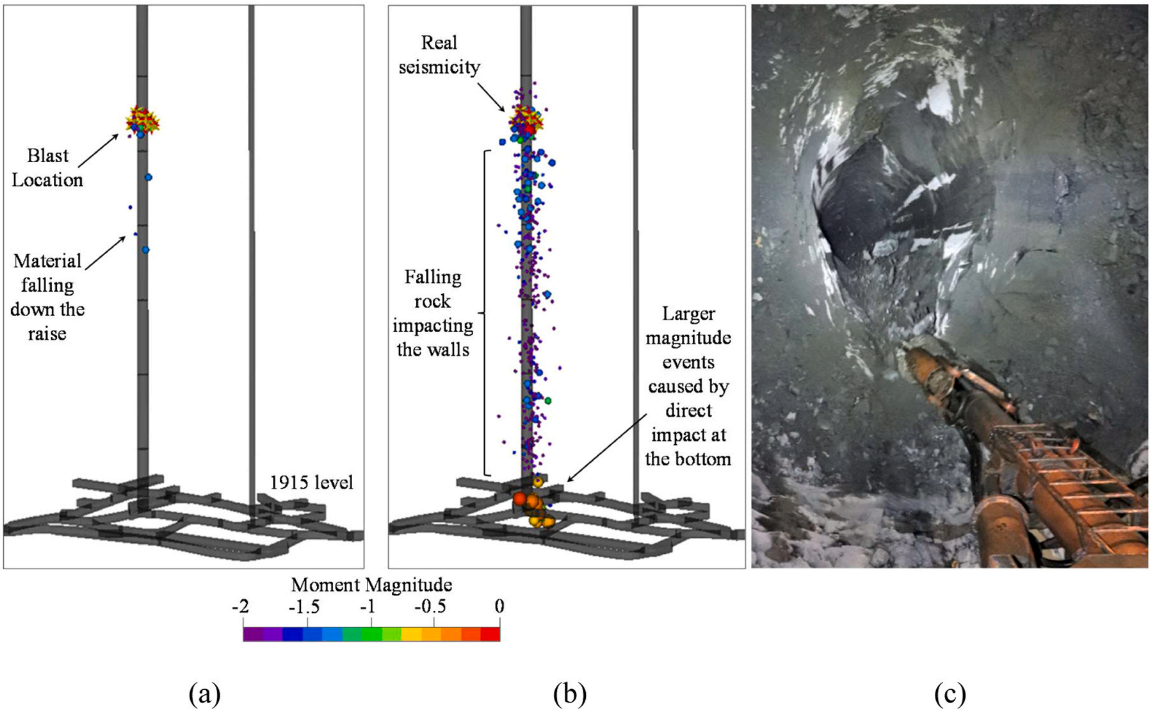


Fig. 7. (a) Seismicity for a 1 hour period after a blast, (b) seismicity for 24 hours after the blast in the shaft, and (c) broken rocks mucked into raise.

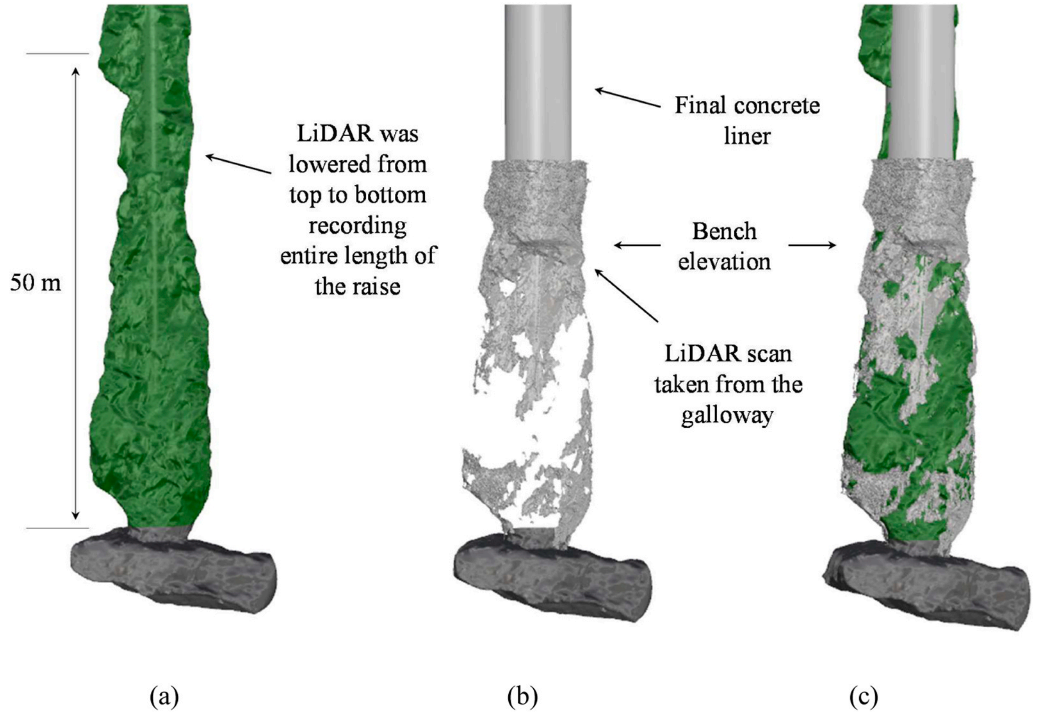


Fig. 8. A comparison of scans completed at the bottom of the 1150–1440 raise taken 4 months apart (looking East). (a) First scan after slashing down 100 m; (b) second scan 4 months later after slashing down another 140 m (Note: the holes in the scan are due to shadow effect as the scan took place from above on the galloway); (c) overlay of both scans.

3.4. Mining-induced deterioration from slashing

Stress-induced damage after slashing also occurred in the bored raises. Blasting while slashing or advancing the Alimak resulted in stress redistribution and stress fracturing around the entire perimeter of the excavation because of the damaging nature of the explosive – the damage did not arrest at the planned break line. While slashing into the bored raises, there was a larger depth of failure on the critically stressed North and South walls of the shaft, which could be examined through holes drilled for ground support in the bored raises that were being slashed. From the borehole camera surveys, there was 5–10 cm of fractured ground on average on the East and West walls in the slash, and

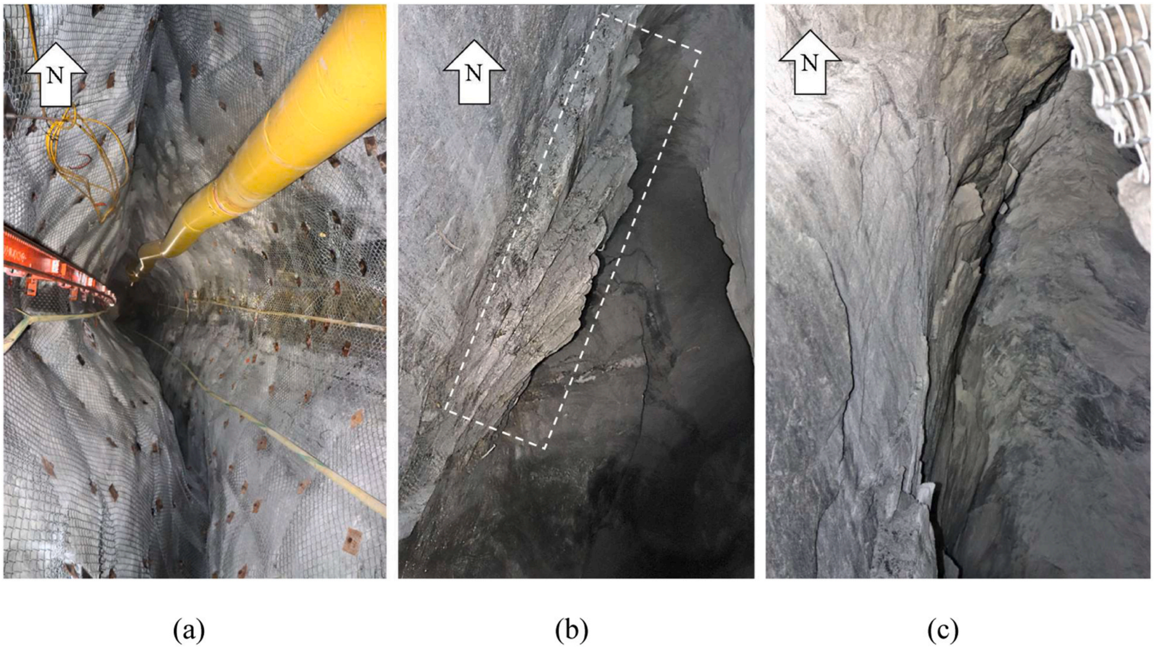


Fig. 9. Stress fracturing visible in the notches of the ore passes shows the same mechanisms of failure that occurred in the bored raises. (a) Looking down into the ore pass, (b) a package of stress-fractured ground that was not scoured away, and (c) stress fractures visible in the apex of the notch.

there was 0.5–1.2 m of damage on average visible on the North and South walls. This depth of fracturing remained relatively constant from 1200 to 1915 m – there was no noticeable increase of fracturing on the East and West walls but there was a slight increase on the North and South of approximately 10 cm with increasing depth. Fig. 11 presents images showing higher intensity fracturing on the North and South walls of the shaft.

This mechanism of deterioration will not be covered in as much detail in this paper because it has a similar rock mass response to the bored raise. However, it should be noted that there were higher levels of seismicity recorded in the raises as notch growth occurred but a significant reduction in seismicity when the raises had grown to their mature state. In addition, it should be noted that there has been no noticeable seismicity recorded in the ore and waste passes throughout the entire duration of the project (since 2017). More detail on this deterioration mechanism is provided in Hall et al. [26].

4. Stabilization methodologies for vertical excavations

4.1. Ground support strategy

The orientation of notch growth in the bored raises and the ore/waste passes is consistent with the orientation of the minimum principal field stress component. At this mine site, the orientation of the maximum principal field stress was roughly East–West, which resulted in the North–South walls being critically stressed. This is important information to consider when developing a support strategy because the critically stressed walls will have an increased depth of failure, experience more notch growth, and are more likely to have spalling and strainbursts. Thus, ground support can be strategically designed to target these high stress areas.

Typically, the ground support used in shallow shafts in Canada consists of 1.6 m long Split sets and chain-link mesh. The use of Split sets is advantageous because it is lightweight, quick to install, and cost-effective compared with other types of rockbolts. The capacity of a 1.6 m long Split set is about 5 tonnes in tension or 1 tonne per 30 cm, which is relatively low compared with a 20 mm diameter rebar, which has a capacity of 25 tonnes. The justification for using Split sets, with a lower capacity, is that it is considered to be temporary ground support because the minimum 30 cm thick concrete liner is kept within 20 m of the working face and the concrete has a much higher support capacity than the Split sets.

Based on the depth of failure observations made in the bored raises, the high stress areas of the shaft (North and South walls) were supported with 2.4 m long Versa-bolts and chain-link mesh. This combination is a dynamic support system that can dissipate energy released from potential strainbursts, while also achieving increased depth of embedment on the walls where the depth of failure in the rock is the largest. The East and West walls were supported with 1.6 m long Split sets and chain-link mesh due to the low-stress conditions on these walls and minimal depth of failure.

Several large seismic events (0.5 to 1.2 moment magnitude) occurred on the North or South walls while slashing into the raise. In all cases, the dynamic support system performed well; in one strainburst, about 30 tonnes of rock bulked and was contained by the support system on the North wall. During the rehab of the bulked area, the depth of failure was measured to be 1.7 m, which was beyond the length of a 1.6 m long Split set. This bolting strategy was carried to 1915 m and it reduced rockburst risk to the operators.

In the Alimak-driven ore/waste passes, each 2 m round advance was originally supported with 1.8 m long rebar fully encapsulated with resin and chain-link mesh, then covered with 50 mm of shotcrete for support protection. The process of dumping broken rock down the raise and scouring the walls wore away the shotcrete and exposed the mesh and bolts. After the screen had worn away, the process of scouring the stress-fractured rock occurred regardless of whether a rebar was installed in the area or not and would continue to grow the notches. The rebar ground support installed on the North and South walls would likely have further exacerbated the growth of the notches because falling rock hitting these bars would cause vibration to attenuate down the bar and deeper into the rock, which would loosen the stress-fractured rock further.

It was observed, while slashing into the bored raises, that once the notches had reached the elongated end growth geometry state, seismicity was significantly reduced compared with that before any growth had occurred. This was a result of the raises reaching a stable point

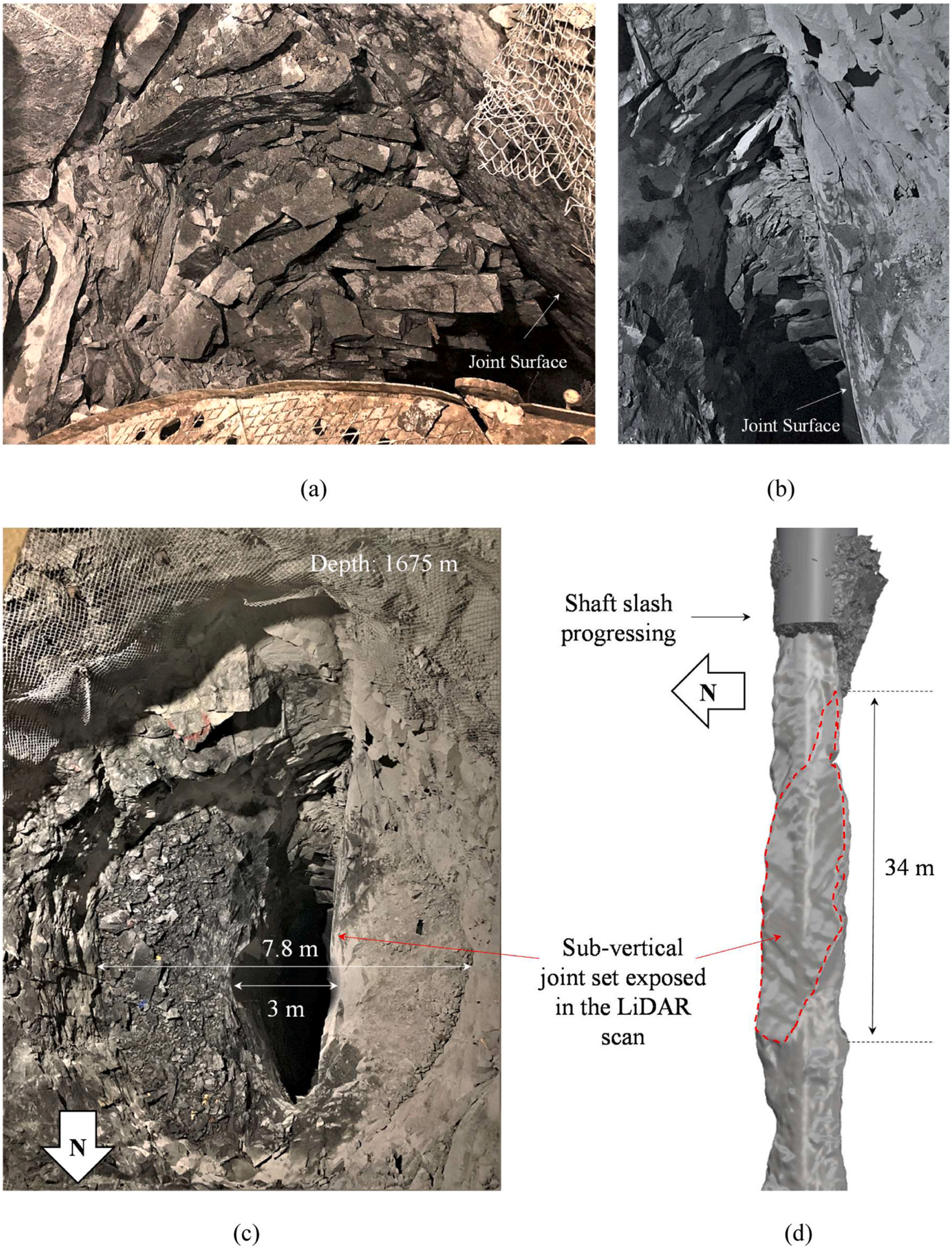


Fig. 10. Examples of structure presence on the critically stressed North and South walls of the shaft/raise: (a) image of tensile fractured rocks between two joint sets, (b) tensile fractures terminating on a single joint, (c) full exposure of a joint trending North–South, and (d) the scan showing the joint set exposed in (c).

where there was a narrow notch geometry, which inherently has higher levels of confinement [26]. It was concluded that the advanced deterioration of notch growth in the ore/waste passes would reduce the potential for strainbursting. Therefore, 1.6 m long Split sets with chain-link mesh, followed by a minimum of 202 mm shotcrete (152 mm fiber-reinforced shotcrete (dyed red) followed by 50 mm of a high impact resistant shotcrete (white)), were used to rehab the ore/waste passes. The added benefit of using Split sets is that there is only a small part of the bolt that protrudes from the rock wall (approximately 25 mm) compared with up to 165 mm for rebar, which will prolong the effectiveness of the shotcrete because the falling / scouring rock will have to wear away more shotcrete before reaching the ground support. If a falling rock does come into contact with the Split set, the bolt will shear itself off more readily than a rebar, which reduces the potential for

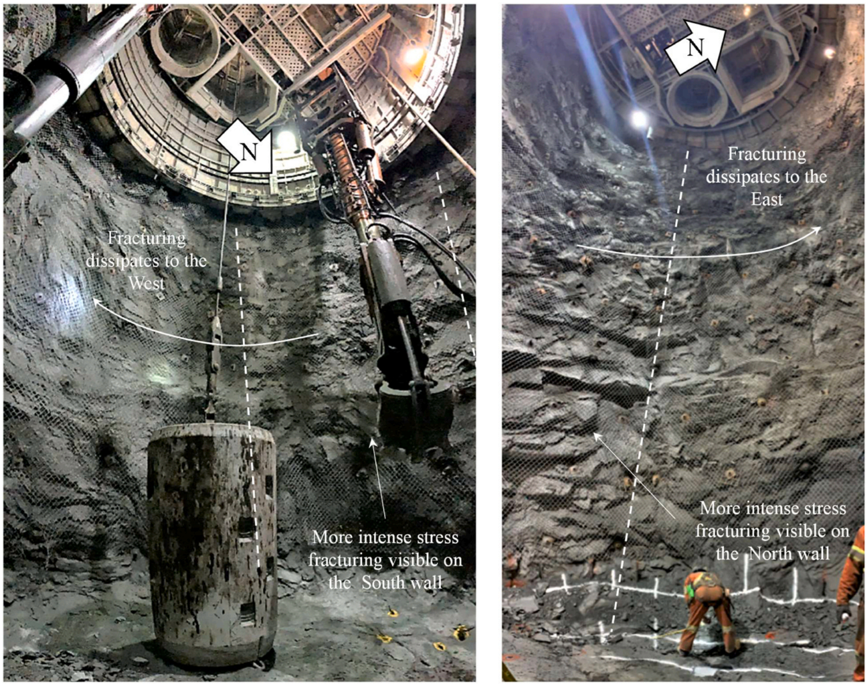


Fig. 11. Images showing more intense stress fracturing on the North and South walls of the shaft.

damage extending deeper into the rock. Due to the geometry of the raise with slender notches, installing bolts perpendicular to the rock surface was impossible (Fig. 9) – the support had to be installed at a shallow angle, which in some cases resulted in critical embedment of 0.5 m. Due to the shallow angle of installation, if loaded, these bolts would be more in shear rather than tension. Therefore, the effectiveness of the bolts in these areas was not as high as if they had been installed perpendicular to the rock surface.

The rehabbed bolt-mesh supported passes were covered by two layers of shotcrete of different colors. The purpose of dying the first shotcrete layer red (Fig. 12) was to monitor for wear points from scouring and proactively recoat these areas as they deteriorated. It was also concluded that due to the fragility of the stress-fractured rock in the apex of the notch and the importance of this fractured rock for maintaining the stability of the excavation, it must be protected as best as possible to service the remainder of the life of mine. Therefore, additional shotcrete was added to this area to protect the notches as well as provide confinement in the apexes. At any location where the notch depth was less than 0.5 m, shotcrete was built out to fill the entire area behind. The increased amount of shotcrete would take longer to wear away from scouring and therefore prolong rehabbed work.

Rock support as part of the ore pass rehabilitation was conducted with an Alimak beginning at the top of the pass. The passes were supported with bolts and screen from top to bottom and then shotcreted from bottom to top as the Alimak rail was removed. The Alimak methodology was advantageous because the rails that the support platform

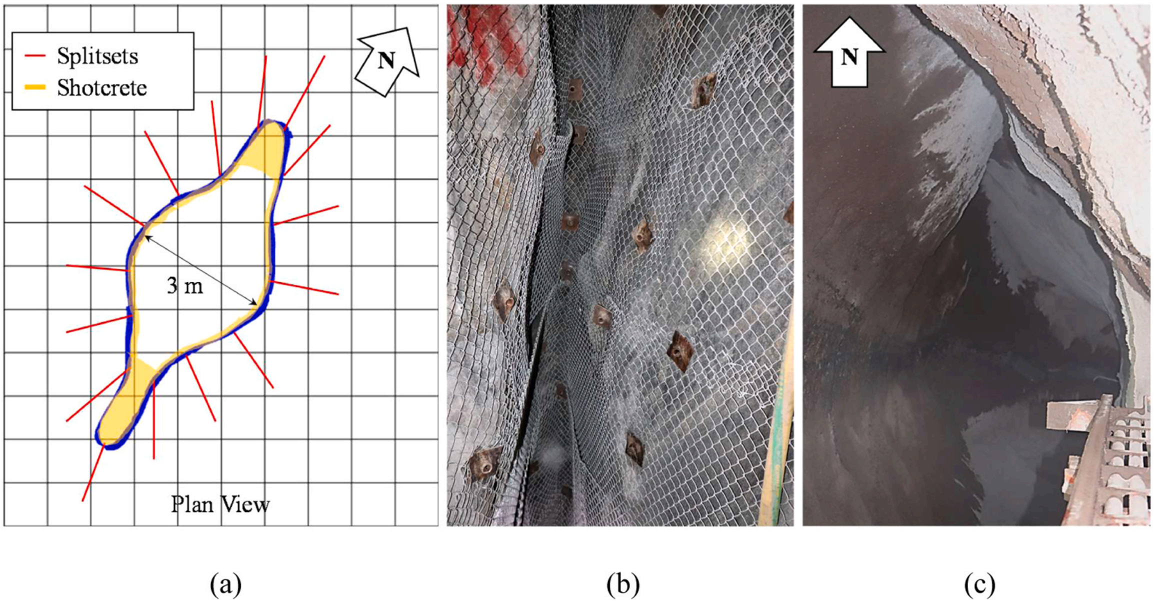


Fig. 12. Supporting the 1200–1310 ore pass in narrow notch areas. (a) A cross-section of the ore pass with the support and shotcrete layout; (b) bolts and mesh installed in a narrow notch area; (c) the ore pass sprayed with red shotcrete.

traveled on could conform to the undulations in the hangingwall and the 70° dip of the passes. In wider areas, the platform was extended and supported with lifting lugs and chains from above. Fig. 13 presents various views of the Alimak setup used to support the passes.

It should be noted that in deep mines with brittle rock, newly bored raises serving as ore passes should not be supported with Split sets because of the strainburst risk while supporting. They can be supported with dynamic bolts; however, the long tails of the bolts protruding through the shotcrete will reduce the longevity of the shotcrete and ultimately lead to growth in the pass. Given the risk to operators having to support the newly bored raise and the limitations associated with conventional ground support, it is recommended to install high-strength, impact-resistant steel cans, which can be lowered and grouted in place. The system will provide confinement to the newly bored raise, prevent scouring, and eliminate the risk to operators having to support the raises conventionally at a great depth where there is increased potential for rockbursting.

4.2. Size and geometry of excavation

The size of the excavation is an important factor to consider when designing a raise or a pass. In the case of the bored raises, 3 m diameter was selected because it was thought that this would reduce the potential for material hang-up while slashing into the raise, allow for adequate ventilation flow, and be stable based on empirical calculations. Any notch growth, based on empirical assessment, would be manageable from the galloway (sinking platform, which was 7 m in diameter). Based on the LiDAR scans, the raise had grown from 3 m to an average of 7.4 m once slashing occurred (notch tip to tip), which was much larger than planned but still manageable with the galloway sinking setup.

The maximum notch depth in the raise is highly dependent on the initial size of the excavation. A larger raise would provide more surface areas of stress-fractured rocks that could be scoured away, which would result in a deeper notch. If the average depth of failure observed in the 3 m diameter raises was 2.2 m (beyond the 3 m diameter raise, leading to 7.4 m from tip to tip of the notches), a 4 m diameter raise could have a notch depth up to 2.75 m (beyond the 3 m diameter raise, leading to 8.5 m from tip to tip of the notches) based on linear scaling, which would not be manageable with the galloway sinking setup. However, if the raise had been 2 m in diameter, the maximum notch depth on average could have been 1.35 m (leading to 4.7 m from tip to tip), again based on linear scaling.

By default, a raisebore is constructed as a circular excavation. Based on the notch formation observed in the vertical excavations at the mine site, it is seen that an ideal ore pass in highly stressed ground can be excavated in an oval shape to reduce the scouring growth. However, in the context of deep burst-prone grounds, the non-entry excavation method using a boring machine is more important than the ideal oval shape. In extreme cases, oval shafts can be slashed into a circular raise or blind sunk with the drill-and-blast method [30], [31]; however, there is a tradeoff of operational advantages versus ideal geometry for stress management.

4.3. Creation of a large void and sinking through the void

While slashing the 1150–1440 raise, a large void formed at the bottom of the raise. The void was 14 m wide in the North–South orientation and 4 m wide in the East–West orientation (Fig. 14 (a)). This was also a result of stress fracturing and scouring. However, the void area above the station was acutely worse than any other area due to the ground support that was installed vertically in the back of the 1440 shaft station before pulling the raise bore. The rock support effectively pinned the back of the station, which suppressed notch growth at the bottom 3–4 m of the raise. This resulted in a ledge or a restriction point forming at the bottom of the raise. As rocks fell and impacted the ledge/restriction point at high velocity, shrapnel rock fragments were deflected laterally around the bottom of the raise. This material was still traveling at a high enough velocity to damage the stress-fractured walls and cause deeper scouring localized to the bottom of the raise. As the bottom of the raise grew, it undercut the volume of rock above, which in turn resulted in sloughing and ultimately a tapered profile from the wide area at the bottom to the more average notch size above.

The void was identified by the LiDAR scan taken after slashing 100 m down the raise, which means that it formed rapidly. However, as the mining face approached the void and additional scans were taken, it was found that the raise had not grown beyond the previous scan. The dimensions of the void at its largest point were 14.5 m in the North–South orientation and 4.5 m in the East–West orientation. This large size was well beyond the limits of the galloway where operators could complete ground support and set up concrete forms safely.

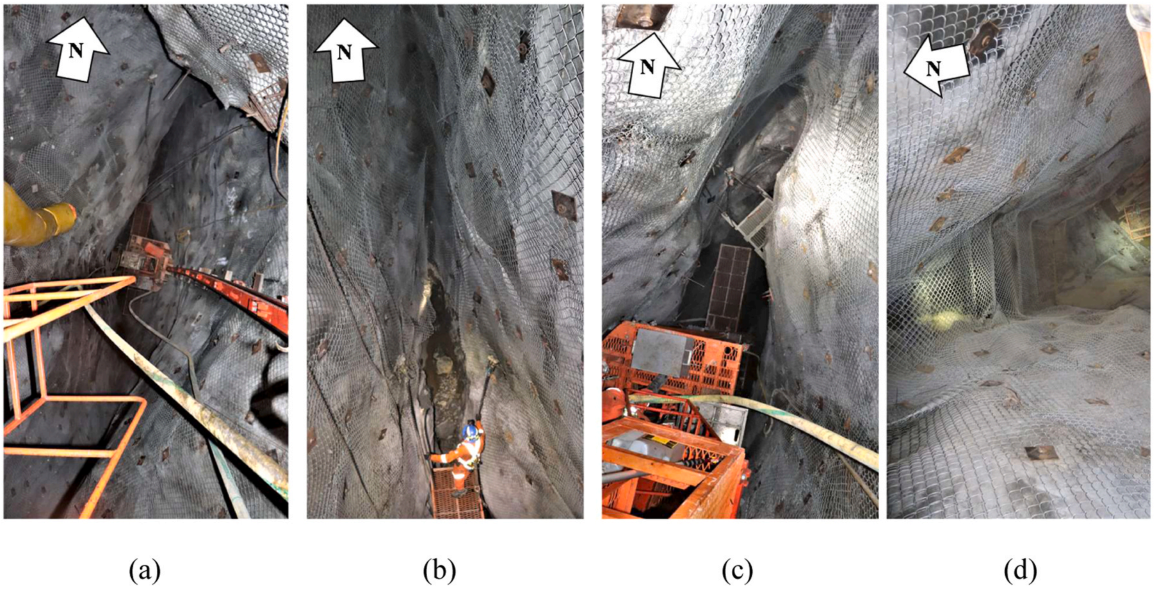


Fig. 13. (a) Looking down at the Alimak bolting platform in the 14 m wide area of the 1200–1310 pass (where a prominent structure is noted in Fig. 4); (b) operators extending the platform to reach wider areas; (c) additional platforms supported with lugs and chains to reach deep notches; (d) area where a deep notch will be filled with shotcrete.

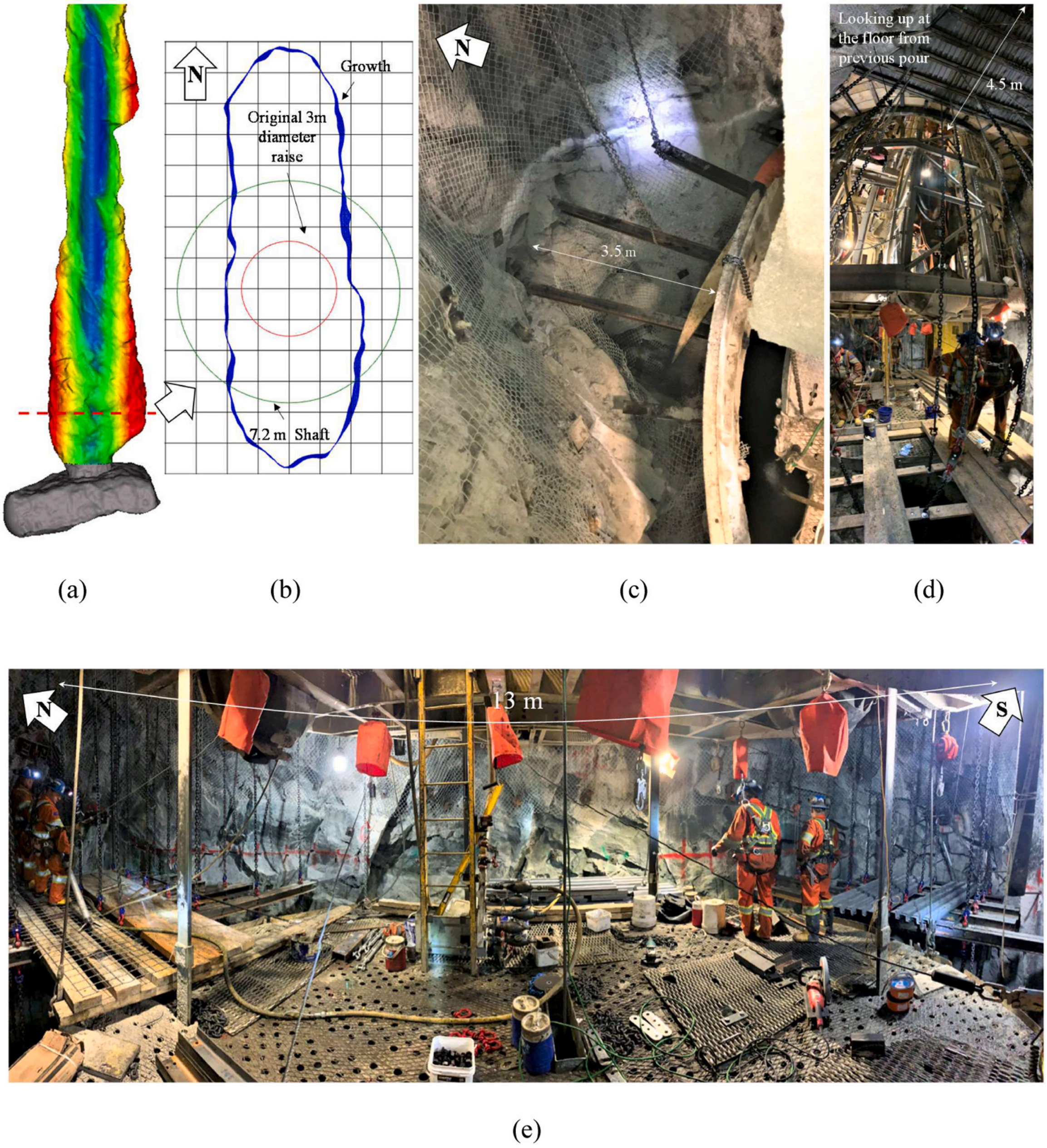


Fig. 14. Images of supporting the large void at the bottom of the 1150–1440 raise. (a) LiDAR scan completed at the bottom of the 1150–1440 raise; (b) cross-section at 1430 m; (c) chains supporting scribing pins so that a floor could be built for concrete to be poured (the ring on the right is the concrete form); (d) chains hanging down from the previous concrete pour supporting the floor; (e) wide-angle image looking East, which shows an N-S cross-section of the floor extending away from the galloway on the North and South sides to support the walls of the deep notches.

Never facing this challenge before, we had to develop a practical engineering solution. Multiple 20-tonne lugs (supported with 8 rebars each) were installed in the narrow areas above the void amongst the other support and heavy gauge chains were hung down from the lugs (Fig. 14 (c)-(d)). From these chains, a suspended floor was built such that bolting activities could occur (Fig. 14 (e)). The chains provided additional support for the base of the large concrete pours, which were required to fill the void. After each round was taken, the chains were buried in concrete and additional lugs and chains were added.

Although seismicity was negligible while sinking through the void, the geometry of the void resulted in increased risk to the operators while installing ground support because they had to support beyond the reach of the galloway. To fill the void to the final 7.2 m diameter of the shaft, there was a 118% increase in concrete required in the bottom 50 m of the raise. The void also caused a 45-day delay to the schedule. Despite the delay and cost, the construction of this segment of the shaft was completed safely.

Based on the observations of the void forming above the 1440 station, it was concluded that a similar void would form at the bottom of the 1440–1915 raise. While the 1150–1440 raise was being slashed, there was no access to the top of the 1440–1915 raise. Therefore, a drone was flown into the bottom of the raise to complete a LiDAR scan. Fig. 15 presents cross-sections of the bottom of both the 1150–1440 raise and the 1440–1915 raise before any slashing had occurred in either. The 1150–1440 raise, which eventually grew to 14 m, had initially grown to 4.5 m, while the 1440–1915 raise was initially 7 m. Therefore, there was potential for the lower 1440–1915 raise to form a much larger void than the 1150–1440 raise if no engineering measures were taken. The method

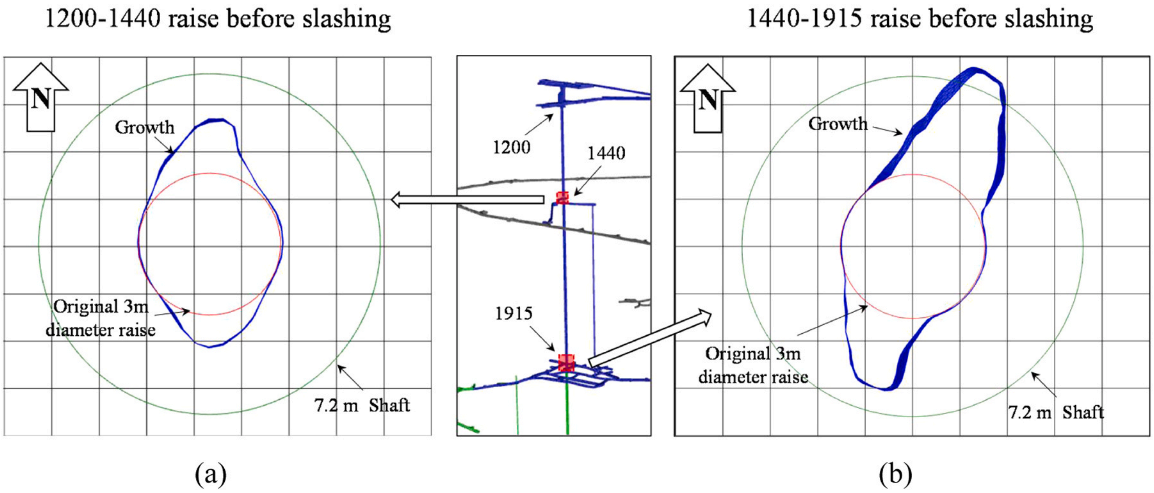


Fig. 15. Cross-sections of LiDAR scans completed at the bottoms of two sections of the 3 m diameter bored raises before any slashing had occurred: (a) Section 1200–1440; (b) Section 1440–1915.

we employed to limit the notch growth at the bottom of the 1440–1915 raise is presented in the next subsection.

4.4. Preventing a large void from forming at the bottom of the 1440-1915 raise

To prevent a similar large void formed at the bottom of the 1150–1440 raise from forming above the 1915 station, a concrete plug was installed at the bottom of the 1440–1915 raise before any slashing occurred so that access could be safely gained below the open hole. Construction of the plug at the bottom of the 475 m long open raise was completed by pushing loose muck tight to the bottom of the raise. A hole was drilled to break into the raise 6 m above the back of the station and sand was blown into the hole to fill the bottom 1 m of the raise to act as a seal for the concrete. Then concrete was pumped through the hole to create a 2 m thick plug. Once complete, the muck was removed and the work was able to be completed below. Fig. 16 presents a schematic of the plug that was used to safely complete work in the 1915 station with the open raise above.

Holes were drilled up into and around the plug and were blasted 5 m deep to a diameter of 7.8 m. This blast effectively removed the final round at the bottom of the 1440–1915 raise as well as the ground support installed in the back of the 1915 station, which prevented a ledge from forming (similar to what occurred at the 1440 station). Therefore, instead of material falling, impacting the ledge, and sending damaging shrapnel around the bottom of the raise, the falling rock was able to fall out of the raise and impact the floor of the level where it would do minimal damage.

Fig. 17 presents a series of scans at the bottom of the 1150–1440 and 1440–1915 raises before any slashing had occurred and after the slashing had advanced 100 m down in each of the raises. There was minor additional notch growth at the bottom of the 1440–1915 raise after blasting the inverse round at the bottom of the raise and sinking 100 m. However, the overall growth at the bottom of the 1440–1915 raise was far less than what was experienced above the 1440 station. As an additional measure, it was made sure that the bottom of the 1440–1915 raise was mucked entirely after each blast to prevent material from building up to the back and acting as a ledge, which could produce the same damage described above. By blasting the inverse round at the bottom of the raise and removing the support installed in the back of the station, no large void formed above the 1915 station. This maneuver significantly reduced the risk for the operators working in the shaft and saved the project both on schedule and costs. We believe that this is the first time that this type of deterioration attributed to ground support has been identified and a methodology to prevent it from occurring has been documented.

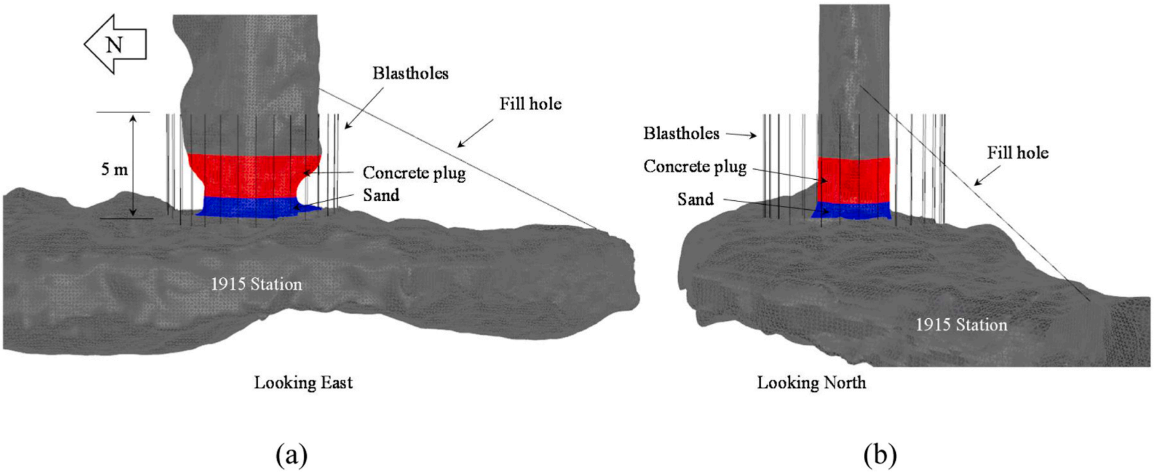


Fig. 16. Schematic of the plug installed at the bottom of the 1440–1915 raise so that work could be performed in the 1915 station: (a) looking East; (b) looking North.

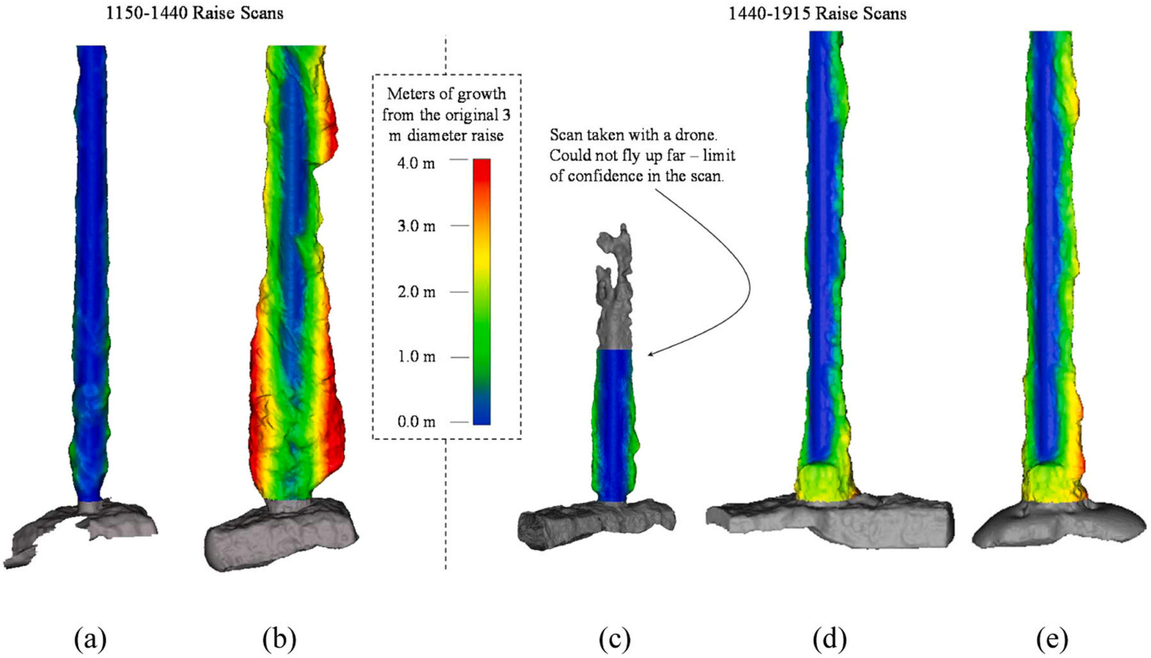


Fig. 17. Images showing the two raises before any slashing and after slashing down 100 m: (a) 1440 level before slashing; (b) 1440 level after slashing down 100 m; (c) 1915 level before slashing; (d) 1915 level after blasting the inverse raise 5 m up and no slashing had occurred; (e) 1915 level after slashing down 100 m.

4.5. Blasting practices

Perimeter control blasting uses fewer explosives on the perimeter to reduce the overbreak of the final excavation and limit damage to the final wall [32]. Overbreak results in additional concrete requirements, which is costly and adds more time to the cycle.

While the perimeter control blasting was practiced at the site, due to the elevated stress conditions experienced in the Onaping Depth shaft, the decision was eventually made to forego perimeter control blasting in favor of releasing more energy into the final wall. This technique was successful in reducing wall-related rockbursts during the sinking of the Creighton No. 9 shaft [33]. More explosive energy generates a larger damage zone around the excavation, which acts as a buffer zone to high stress because the fractured rock has a limited ability to carry high stress. The other benefit of releasing more energy with the blast is that the majority of seismicity is induced with the blast as opposed to occurring randomly while the operators are present [34].

When higher stress conditions were evident, additional perimeter blasting holes were added on the North and South walls of the shaft to fracture more ground and push stress away from the excavation boundary. Normally the perimeter consisted of 36 holes drilled at 0.7 m spacing. When conditions dictated, four additional holes were drilled on the North and South walls (eight total) between the 0.7 m perimeter pattern. The additional eight holes increased the explosive amount by 22.2% on the perimeters of the North and South walls. This proved successful in reducing seismic activities in the walls.

It should be noted that from a comparison of the LiDAR scanned volumes of individual rounds blasted using perimeter control and those where perimeter control was not used, the difference in overbreak was negligible due to the spalling and scaling requirements that were required when perimeter control was used. In addition, the time required for scaling the rounds that used perimeter control increased the overall cycle time; therefore, the impact of forgoing perimeter control was negligible.

Precondition blasting is a technique used to manage high-stress conditions ahead of excavations and prevent strainbursting from occurring. The goal of precondition blasting is to introduce fractures, ahead of the drift or shaft face, that will self-propagate with the high stresses, thereby softening the rock mass around the immediate perimeter of the excavation and pushing stress away [35–37]. Based on the geometry of the notches, which covered a large area of the shaft slash, it was difficult to position precondition blastholes in the bench to condition the bench face properly. The high stress areas were around the tips of the notches on the North and South walls. However, the deep notches promoted stability due to the high confinement resulting from the narrow notch geometry. The East and West walls of the bench had low-stress conditions and therefore did not require preconditioning. It should also be noted that it is of utmost importance to maintain the East–West span of the raise. Any failure of the East–West walls would increase the depth of the notches in the North–South orientation.

Precondition blasting has been used to target the walls of a shaft excavation previously [38,39]; however, in the scenario of slashing into the raise, any attempt to blast behind the apexes of the notches (deeper in the walls) could reduce confinement and cause additional notch growth or instability at the apexes of the notches, which could increase risk for the operators. Furthermore, wall holes introduce more risk to operators due to the potential for drilling into bootlegs (undetonated explosive) while drilling ground support in the blasted round [40]. Wall precondition holes are designed to be drilled such that the explosive is placed at a location 15–20 cm beyond the length of the ground support that will be installed. When designing precondition holes in walls, many factors should be considered such as deviation, over drilling of ground support holes, overbreak of the blasted round, and inability to confirm if wall precondition holes have been detonated or not. Due to the potential cumulative error in the position of the wall precondition holes, it was determined that the risk was too high for this practice to be used in the Onaping Depth shaft slash. Foregoing perimeter control was deemed to be as effective as, if not more effective than wall preconditioning for stress management.

4.6. Discussions

While rehabbing several waste/ore passes at the site, which had been in use since 1992 with notches that had grown excessively, similarities in the mechanisms for notch formation between the passes and the more recent bored raises became clear. The passes provided a late-stage snapshot of deterioration, which closely matched the evolution of deterioration that was observed in the bored raises after slashing into them. Based on the observations made in the raises, with respect to notch growth and deterioration, rehab work in the historically used waste/ore passes could be modified to protect the critical areas of these passes such that they can survive the required tonnage throughput during the life of mine and also be monitored for wear.

Several engineering efforts that are detailed in this work, such as ground support, blasting, and managing a large void, provided varying degrees of stabilization to the vertical excavations. Overall, the stabilization efforts were successful, which reduced the risk to the operators working in the field. There were also significant gains in productivity and schedule associated with several of these methodologies. As a result of limited published documentation related to the challenges associated with vertical excavations at depth, we hope that the details provided herein are valuable for other deep mining projects. It is important to note that when planning for the execution of a vertical excavation at any site, as much site-specific information must be collected related to the rock mass and stress conditions to make better informed decisions.

5. Conclusions

The work presented in this paper describes the main mechanisms of notch formation observed while pulling two 3 m diameter raises and subsequent slashing into these raises to construct an underground winze to access the Onaping Depth deposit within the Craig mine complex. These include stress change as the excavation is made, local geological structure, slashing-induced stress changes, and scouring of the raise as it was blasted and as broken rocks were dumped down the raise. It was also found that these mechanisms are often working simultaneously, leading to overbreak beyond the original bored dimension. The understanding of these mechanisms was made possible using data from LiDAR scans, microseismic monitoring, frequent direct observations during the pulling of the raisebore, and subsequent slashing to the final diameter of the shaft. In particular, the ability to observe a cross-section of the raise as slashing occurred provided an excellent opportunity to monitor the progressive failure occurring in the raise. This effectively gave snapshots in time of the initial state of the raise and the final state after it had developed narrow notch tips and eventually stabilized; most importantly, it allowed for damage sustained deeper into the rock mass to be examined.

An understanding of the mechanisms of deterioration allowed for more suitable support to be selected to prevent further deterioration from occurring and stabilize the excavations. For example, in the shaft slash around the 3 m raise, it was important to understand the maximum principal stress orientation so that dynamic support could be installed on the critically stressed walls. In the more advanced state of deteriorated ore passes, where the manifestation of stress related issues such as strainbursting was less likely, Split sets were used to support the walls and then covered with a thick coat of shotcrete to protect the ground support and provide confinement to the notches.

It should be noted that newly bored raises, which will serve as waste/ore passes in deep mines should not be supported with conventional dynamic ground support due to the limitations of these rock support systems. Instead, it is recommended to use forms to protect the raises, such as installing high-strength impact-resistant steel cans, which are lowered into the raises and grouted in place. These cans will provide confinement to the newly bored raises and eliminate the risk to operators having to support the raises conventionally at great depth.

Several methodologies to prevent the deterioration of vertical excavations were also examined and, in some cases, implemented with success. These included careful blasting practices that were used in the shaft slash as a means of managing stress and strainburst risk, and support strategies to prevent a void from forming at the bottom of a raise. In addition, two methodologies to support and stabilize excavations that had experienced significant notch growth resulting in a challenging geometry are detailed. The findings of this work are useful for understanding the mechanisms driving brittle rock failure in deep mining and subsequently for minimizing or preventing the deterioration from occurring, which benefits operator safety.

Declaration of Competing Interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

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References

[1] J.R. Boldt, P. Queneau, The Winning of Nickel: Its Geology, Mining, and Extractive Metallurgy, Methuen,, 1967 [Online]. Available: https://books.google.ca/books?id=3mLaoAEACAAJ.
[2] W.A. Hustrulid, W.A. Hustrulid, R.C. Bullock, and R.L. Bullock, *Underground mining methods: Engineering fundamentals and international case studies*. SME, 2001.
[3] G. Bäckblom, C.D. Martin, Recent experiments in hard rocks to study the excavation response: Implications for the performance of a nuclear waste geological repository, Tunn. Undergr. Space Technol. vol. 14 (3) (. 1999) 377–394, https://doi.org/10.1016/S0886-7798(99)00053-X.
[4] P.K. Kaiser, D.R. McCreath, Rock mechanics considerations for drilled or bored excavations in hard rock, Tunn. Undergr. Space Technol. vol. 9 (4) (1994) 425–437, https://doi.org/10.1016/0886-7798(94)90101-5.
[5] K. Thuro and R.J. Plinninger, “Hard Rock Tunnel Boring, Cutting, Drilling And Blasting: Rock Parameters For Excavatability,” presented at the 10th ISRM Congress, Sep. 2003, p. ISRM-10CONGRESS-2003–212.
[6] Y. Fan, W. Lu, Y. Zhou, P. Yan, Z. Leng, M. Chen, Influence of tunneling methods on the strainburst characteristics during the excavation of deep rock masses, Eng. Geol. vol. 201 (2016) 85–95, https://doi.org/10.1016/j.enggeo.2015.12.015.
[7] S. Emsley, O. Olsson, L. Stenberg, H.-J. Alheid, and S. Falls, “ZEDEX-A study of damage and disturbance from tunnel excavation by blasting and tunnel boring,” 1997.
[8] C.D. Martin, “Brittle Rock Strength Failure: Laboratory And In Situ,” presented at the 8th ISRM Congress, Sep. 1995, p. ISRM-8CONGRESS-1995–195.
[9] W.R. Wawersik, C. Fairhurst, A study of brittle rock fracture in laboratory compression experiments, Int. J. Rock Mech. Min. Sci. Geomech. Abstr. vol. 7 (5) (1970) 561–575, https://doi.org/10.1016/0148-9062(70)90007-0.
[10] E. Hoek, Brittle fracture of rock, Rock Mech. Eng. Pract. vol. 130 (1968) 9–124.
[11] W.F. Brace, B.W. Paulding Jr, C.H. Scholz, Dilatancy in the fracture of crystalline rocks, J. Geophys. Res. vol. 71 (16) (1966) 3939–3953.
[12] Z.T. Bieniawski, “Mechanism of brittle fracture of rock: part I—theory of the fracture process,”. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, Elsevier,, 1967, pp. 395–406.
[13] C.D. Martin, “The strength of massive Lac du Bonnet granite around underground openings,” 1993.
[14] M. Cai, P.K. Kaiser, Y. Tasaka, T. Maejima, H. Morioka, M. Minami, Generalized crack initiation and crack damage stress thresholds of brittle rock masses near underground excavations, Int. J. Rock Mech. Min. Sci. vol. 41 (5) (2004) 833–847.
[15] M. Cai, Influence of intermediate principal stress on rock fracturing and strength near excavation boundaries—insight from numerical modeling, Int. J. Rock Mech. Min. Sci. vol. 45 (5) (2008) 763–772.
[16] T.R. Stacey, N.D. Harte, “Deep level raise boring-Prediction of rock problems. ISRM International Symposium, International Society for Rock Mechanics and Rock Engineering,, 1989.
[17] A. McCracken and T.R. Stacey, “Geotechnical risk assessment for large-diameter raise-bored shafts,” in *Shaft engineering conference*, 1989, pp. 309–316.
[18] B.M. Coombes, M.F. Lee, and W.A. Peck, “Improving raisebore stability assessments and risk,” in *Proceedings of the 2011 Canadian Institute of Mining, Metallurgy and Petroleum Conference and Exhibition*, 2011.
[19] W.A. Peck, B. Coombes, and M.F. Lee, “Fine-tuning raisebore stability assessments and risk,” in *Proceedings of the 11th AusIMM Underground Operators' Conference*, 2011, pp. 215–226.
[20] C. Edelbro, R. Brummer, M. Pierce, D. Sandstrom, J. Sjoberg, Raiseboring in difficult rock conditions. Proceedings of the Ninth International Symposium on Ground Support in Mining and Underground Construction, Australian Centre for Geomechanics,, 2019, pp. 185–198.
[21] J. Hadjigeorgiou and F. Mercier-Langevin, “Estimating ore pass longevity in hard rock mines,” presented at the The 42nd U.S. Rock Mechanics Symposium (USRMS), Jun. 2008, p. ARMA-08–165.

[22] J.F. Hadjigeorgiou, Ore pass practice in Canadian mines, J. Southern Afr. Inst. Mining Metall. vol. 105 (11) (2005) 809–816.

[23] K. Esmaieli, J. Hadjigeorgiou, Selecting Ore Pass-Finger Raise Configurations in Underground Mines, Rock Mech. Rock Eng. vol. 44 (3) (2011) 291–303, https://doi.org/10.1007/s00603-010-0128-z.

[24] K. Esmaieli, J. Hadjigeorgiou, M. Grenon, Stability Analysis of the 19A Ore Pass at Brunswick Mine Using a Two-Stage Numerical Modeling Approach, Rock Mech. Rock Eng. vol. 46 (6) (. 2013) 1323–1338, https://doi.org/10.1007/s00603-013-0371-1.

[25] J. Sjoberg, P. Lundman, E. Nordlund, and C. Quinteiro, "Stability Analysis of Ore Passes In the Kiirunavaara Mine," presented at the 10th ISRM Congress, Sep. 2003, p. ISRM-10CONGRESS-2003–189.

[26] A. Hall, B. Simser, M. Cai, Mechanisms of deterioration in a bored raise in brittle rock, Int. J. Rock Mech. Min. Sci. vol. 139 (2021) 104666.

[27] C.D. Martin, P.K. Kaiser, D.R. McCreath, Hoek-Brown parameters for predicting the depth of brittle failure around tunnels, Can. Geotech. J. vol. 36 (1) (1999) 136–151.

[28] P. Kaiser, F. Amann, and R. Bewick, "Overcoming Challenges of Rock Mass Characterization for Underground Construction in Deep Mines," 2015.

[29] V. Hajiabdolmajid, P. Kaiser, C.D. Martin, Mobilised strength components in brittle failure of rock, Geotechnique vol. 53 (3) (2003) 327–336.

[30] G. Sturgis, D. Berberick, M. Board, W. Strickland, M. Swanson, in: J. Wesseloo, J. Wesseloo (Eds.), Elliptical Shaft Excavation and Furnishing in Response to Depth Induced Ground Pressure, Australian Centre for Geomechanics, Perth, 2017, pp. 883–897 [Online]. Available: https://papers.acg.uwa.edu.au/p/1704_61_Sturgis/.

[31] G. Walton, E. Kim, S. Sinha, G. Sturgis, D. Berberick, Investigation of shaft stability and anisotropic deformation in a deep shaft in Idaho, United States, Int. J. Rock Mech. Min. Sci. vol. 105 (2018) 160–171, https://doi.org/10.1016/j.ijrmms.2018.03.017.

[32] E.M. Van Eeckhout, "Controlled blasting and its implications for the NNWSI project exploratory shaft," Sep. 1987, doi: 10.2172/60107.

[33] J.S. Redpath, "Creighton No. 9 shaft, 7137 feet sunk in one lift," in *Society of Mining Engineers of the American Institute of Mining, Metallurgical, and Petroleum Engineers*, in Volume 2. Chicago, Illinois, 1972.

[34] M.G. Salamon, Rockburst hazard and the fight for its alleviation in South African gold mines, *Rockbursts: Prediction and Control. Symposium* (1983) 11–36.

[35] M.R. Saharan, H. Mitri, Destress Blasting as a Mines Safety Tool: Some Fundamental Challenges for Successful Applications, ISMSSE2011 vol. 26 (2011) 37–47, https://doi.org/10.1016/j.proeng.2011.11.2137.

[36] A. Saadatmand Hashemi, P. Katsabanis, Tunnel face preconditioning using destress blasting in deep underground excavations, Tunn. Undergr. Space Technol. vol. 117 (2021) 104126, https://doi.org/10.1016/j.tust.2021.104126.

[37] A.Z. Toper, K.K. Kabongo, R.D. Stewart, A. Daehnke, The mechanism, optimization and effects of preconditioning, J. -SOUTH Afr. Inst. Min. Metall. vol. 100 (1) (2000) 7–16.

[38] W. Blake, D.G.F. Hedley, Rockbursts: Case Studies from North American Hard-rock Mines, Society for Mining, Metallurgy, and Exploration,, 2003 [Online]. Available: https://books.google.ca/books?id=qJN6ZPkOgH8C.

[39] M.H. Dickout, "Ground control at Creighton mine of the International Nickel Company of Canada Limited. Proceedings of the 1st Canadian Rock Mechanics Symposium, McGill University,, Montreal, 1962, pp. 121–139.

[40] C. Drover, E. Villaescusa, I. Onederra, Face destressing blast design for hard rock tunnelling at great depth, Tunn. Undergr. Space Technol. vol. 80 (2018) 257–268, https://doi.org/10.1016/j.tust.2018.06.021.

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