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# Deterministic and probabilistic stability analysis of a mining rock slope in the vicinity of a major public road — case study of the LAB Chrysotile mine in Canada



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## ABSTRACT

In recent years, several large open-pit mines have started operating in the province of Quebec in Canada, and some of the largest planned pits are located close to public infrastructure. Historically, large open-pit mining has seldom been done in many mining regions, such as the Abitibi region, where underground mines are the norm. As an integral part of achieving social acceptability of open-pit mining, the stability of mining slopes must be carefully analyzed during the design process and the presence of public infrastructure near the slopes must be adequately considered. The province of Quebec does not have specific guidelines regarding such design considerations. This paper provides a short overview of the literature on some current practices regarding mining slope design close to public infrastructure. To demonstrate its applicability in the Quebec provincial context, the paper then investigates the stability of the west wall of the LAB Chrysotile open-pit mine in Thetford Mines (Quebec) near the new Road 112. Deterministic and probabilistic analyses were conducted using finite element shear strength reduction and limit equilibrium methods to investigate slope stability. The impact of pit infilling and rapid dewatering as well as long-term stability of the slope were investigated. The results of all analyses reveal that the current mining slopes at LAB Chrysotile are within acceptable design criteria limits.

## KEYWORDS

open-pit mining, slope stability, public infrastructure, deterministic and probabilistic analysis, case study.

## CITATION

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## 1 INTRODUCTION

Routine stability analysis of open-pit mine slopes involves a trade-off between stability and profit maximization. As the slope angle becomes steeper, the stripping ratio (waste to ore ratio) is reduced, and the mining project accordingly becomes more cost-effective. However, increasing the slope angle will generally increase the risk of instability. One of the most important tasks of the design team is finding the optimized angle at which the waste to ore ratio is minimal and stability is within acceptable limits. Generally speaking, in mining contexts, slopes should be stable up to the end of the mine life — controlled (small) failures in certain slopes are acceptable—expected (Read and

Stacey 2009) — while civil slopes are designed to be more permanent. In all slope stability analyses, four parameters should be known in advance (Fleurisson and Cojean 2014).

- Slope scale (bench, inter-ramp, and global) — One of the most important parameters that affects the failure mechanism and therefore the slope design method is the slope scale. Each slope scale has its own failure mechanism and therefore, slope scales as well as the corresponding failure mechanisms should always be taken into consideration. At bench and inter-ramp slope scales, structural instabilities are common. At inter-ramp and global slope scales, rock mass instabilities should be investigated. A good synthesis is provided by Read and Stacey (2009).

- Purpose of slope design — It is important to know the purpose of slope design. For instance, slope stability design methods associated with small- to medium-scale slopes with medium to high tolerance of failure consequences (e.g., during mining) are different from those associated with an overall slope-scale stability with a low to very low tolerance of failure consequence (e.g., slopes with important infrastructure such as public roads in the vicinity). Therefore, it is important to establish a design criterion for each case, depending on the importance of the project and the failure consequences, particularly for third-party risks.

- Type of loading — Different types of loading can be accounted for as being the triggering parameter of slope failures. Loading could be the result of mining activity or it could be due to some hydraulic conditions (e.g., the rising level of ponded water); dynamic loading caused by earthquakes could also occur. Loading can be positive or negative (i.e., the removal of rock mass or water).

- Short- and medium-term versus long-term slope stability — Short- and medium-term slope stability analyses are conducted during the life of the mining operation. At that stage, it is possible to ignore some acceptable rate of deformation or some localized failure, ensuring that these do not adversely affect either the safety of staff or the operation. However, slope stability of the final pit at the end of the mining operation and even after abandonment requires further stability assessments. At that stage, different design criteria should be considered, in addition to different scenarios describing possible changes in geotechnical properties of the materials over time as well as changes in the rock mass hydraulic conditions due, e.g., to the end of pumping.

## 1.1 Problem description

Instability or failure of open-pit mine slopes can have major impacts on nearby public infrastructure such as public highways and railways. However, there is limited scientific literature on this subject. Over the last few years in Quebec (Canada) there has been an important increase in the number of large open-pit mine projects close to existing public infrastructure in the near north of the province. The most important is the Canadian Malartic mine along a stretch of Road 117 in the heart of the town of Malartic in the Abitibi region. During the same period, older large open-pit mines were still active in the southern part of the province.

LAB Chrysotile, an older open-pit asbestos mine located in Thetford Mines in the Chaudière-Appalaches region in southern Quebec, Canada (Fig. 1), was in operation until 2011. The mining operations started in 1958 (Beauchamp 1994). The final pit was approximately 320 m deep, with a diameter at the crest ranging between 1.4 and 1.8 km.

Road 112 is the most important public highway in the region, serving as the major commercial link between the region and northeast USA. This road was located immediately adjacent to the crest of the east wall of the LAB Chrysotile mine (Figs. 1 and 2). In July 2012, a major slope failure occurred on the east wall of the pit, taking with it a large portion of Road 112 (Fig. 2). During this failure, a vertical movement of 70 m was observed. The lateral extent of the failure was 1.1 km. The estimated displaced volume was  $5.0 \times 10^7 \text{ m}^3$ . This failure is documented extensively in Caudal et al. 2016 and Grenon et al. 2016. These references also present a detailed back-analysis of the east wall failure.

A new road was thus necessary to serve the region. The chosen location was along the west wall of the LAB Chrysotile pit (Figs. 1 and 2). An important slope failure occurred on the west wall in 2010

close to the location of the new road. At its closest point, the new road would be 270 m behind the slope crest. The construction of the new road started in summer 2013, and the road was commissioned in late fall 2015.

Currently, Quebec legislation does not contain specific guidelines for mining slope design, but it does state that a stability analysis must be performed and approved by an engineer with well-documented expertise and an adequate knowledge of mining geotechnics (MERN 2016). Likewise, other Canadian provincial legislation requires that an independent expert assess and issue an opinion and recommendations on the geotechnical issues associated with an open-pit project. No specific recommendations are made for pit slopes close to public infrastructure.

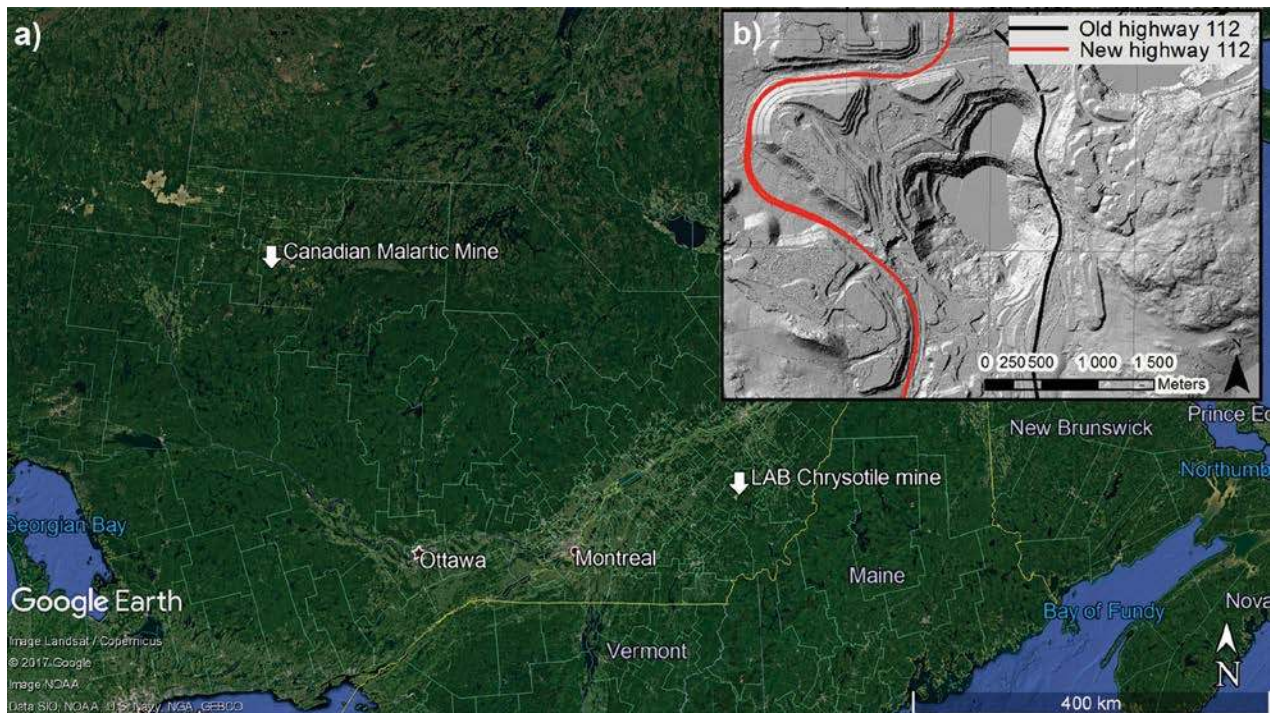


Fig. 1. (a) Location of LAB Chrysotile open-pit mine in Canada. (b) Close-up image shows the hillshade derived from the 2014 LiDAR data. The new Road 112 is indicated in red and the old Road 112 in black. [Colour online.]

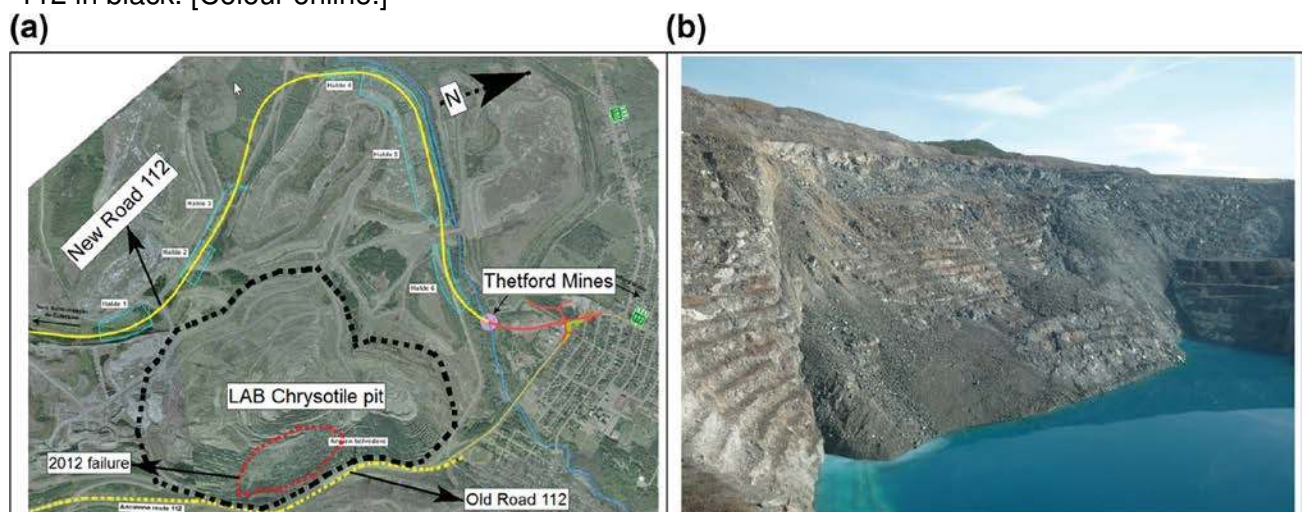


Fig. 2. (a) LAB Chrysotile mine and location of old and new Road 112 (the 2012 failure on the east wall is also illustrated). (b) The 2012 failure on the East wall of LAB Chrysotile mine (looking south-east). [Colour online.]

## 1.2 Objectives

The objectives of this paper can be summarized as follows:

- Present an overview of the existing design guidelines in the legislation from various locations around the world regarding slope stability analysis and design of mining slopes in the vicinity of public infrastructure such as roads or railways.
  - Perform a back-analysis of the 2010 west wall failure.
  - Perform a forward slope stability analysis to investigate the potential for large, deep-seated failure of the west wall using both deterministic (factor of safety, FoS) and probabilistic (probability of failure, PoF) methods.
    - Study the impact of both pit infilling with water and rapid dewatering on the stability of the west wall, considering different scenarios for rock mass properties, and investigate the long-term slope stability of the west wall.
      - Compare the analysis results with existing guidelines.
      - Suggest a best-practice approach for the design of mining slopes in Quebec in the vicinity of public infrastructure.

## 2 EXISTING MINING SLOPE DESIGN GUIDELINES IN THE VICINITY OF PUBLIC INFRASTRUCTURE

There is little scientific literature on the design of public roads in the vicinity of open-pit mines, while for engineered civil rock cuts, the focus is primarily on shallow slopes, as in Andrew et al. (2011), Gomes et al. (2012), and Maerz (2000), amongst others. From the available literature, design criteria can be divided into three main categories: analyses based on empirical criteria, analyses based on FoS and PoF, and analyses based on deformation and strain analysis. This section will present a brief overview of the existing literature relating to the first two categories, arguably the most applicable approaches in greenfield studies or when limited monitoring information is available, a similar situation to the case study presented in this paper. Deformation and strain analyses for slopes are presented in multiple references; namely, Brox and Newcomen (2003), Dight (2006), Hormazabal et al. (2011), Price et al. (2006), Sjöberg (2013), and Stacey et al. (2003).

### 2.1 Application of empirical (geometrical) criteria

Empirical or geometry-based criteria rely on the experience gained through different case studies under different conditions. These criteria are generally conservative and normally are used at early stages of a project when the available geotechnical data are limited. The only existing formal empirical guideline known to the authors for the design of public roads in the vicinity of open-pit mines was proposed by the Department of Industry and Resources in Western Australia (DoIR 1997). The objective of this guideline is to define criteria to establish a “safety zone” around open pits at the end of mining. This guideline is based on field investigations of failures and tension cracks around pit edges, in operating and abandoned open-pit gold mines in Western Australia. It states that upon abandonment, a bund or fence must be constructed around the mine workings to minimize inadvertent public access. This bund should be built at least 10 m outside the area designated as being susceptible to wall collapse or having potentially unstable rock mass. This guideline is mostly used in the absence of any geotechnical investigation. It is used for analyzing long-term slope stability of abandoned open-pit mines in hard rock, aiming at minimizing the risk. According to this

guideline, the overall angles (the angle between the toe of the global slope and the horizontal) defining the maximum potentially unstable rock mass in weathered and unweathered (fresh and slightly weathered) rocks should be 25° and 45°, respectively. The use of this design criterion is based on the assumption that no major unfavorably oriented geological features are present within the pit walls.

## 2.2 Design criteria based on FoS and PoF

FoS and PoF are concepts that have long been used in slope stability analysis of open-pit mines. Accordingly, there is abundant literature on their application for all types of geological conditions (Read and Stacey 2009; Wyllie 2017).

### 2.2.1 FoS

The FoS can be defined as the ratio between the resisting forces (strength) and the driving forces (loading) along a potential failure surface. There are no unique criteria for specifying acceptable FoS. Nonetheless, according to the literature, for static loading conditions the values of 1.2–2.0 (FoS of 1.2 for temporary small-scale slopes and FoS of 2 for permanent large-scale slopes) have been used depending on the slope scale and the consequences of failure (Read and Stacey 2009; Wyllie 2017).

Hoek (2007) proposed an FoS > 1.5 for “permanent” slopes with a significant risk of damage. He also suggested that where displacements are critical, higher FOSs should generally be applied. Sullivan (2006) mentioned that both the original Hoek and Bray (1981) recommendation and arguably, that of Hoek (2007) were proposed based on a conservative choice of strength input parameters.

The FoS approach is based on single values of rock mass properties as input parameters. It can be defined as a deterministic approach. The results of such an approach could be conservative or not. Furthermore, using worst-case scenario inputs can be misleading, as the combined occurrence of worst-case values is expected to be low (Valerio et al. 2013; Dunn 2013) Sensitivity analyses can be used to determine the impact of input parameter variability on the resulting FoS. However, these types of analyses are not well suited to address the probability of occurrence of the various scenarios.

### 2.2.2 PoF

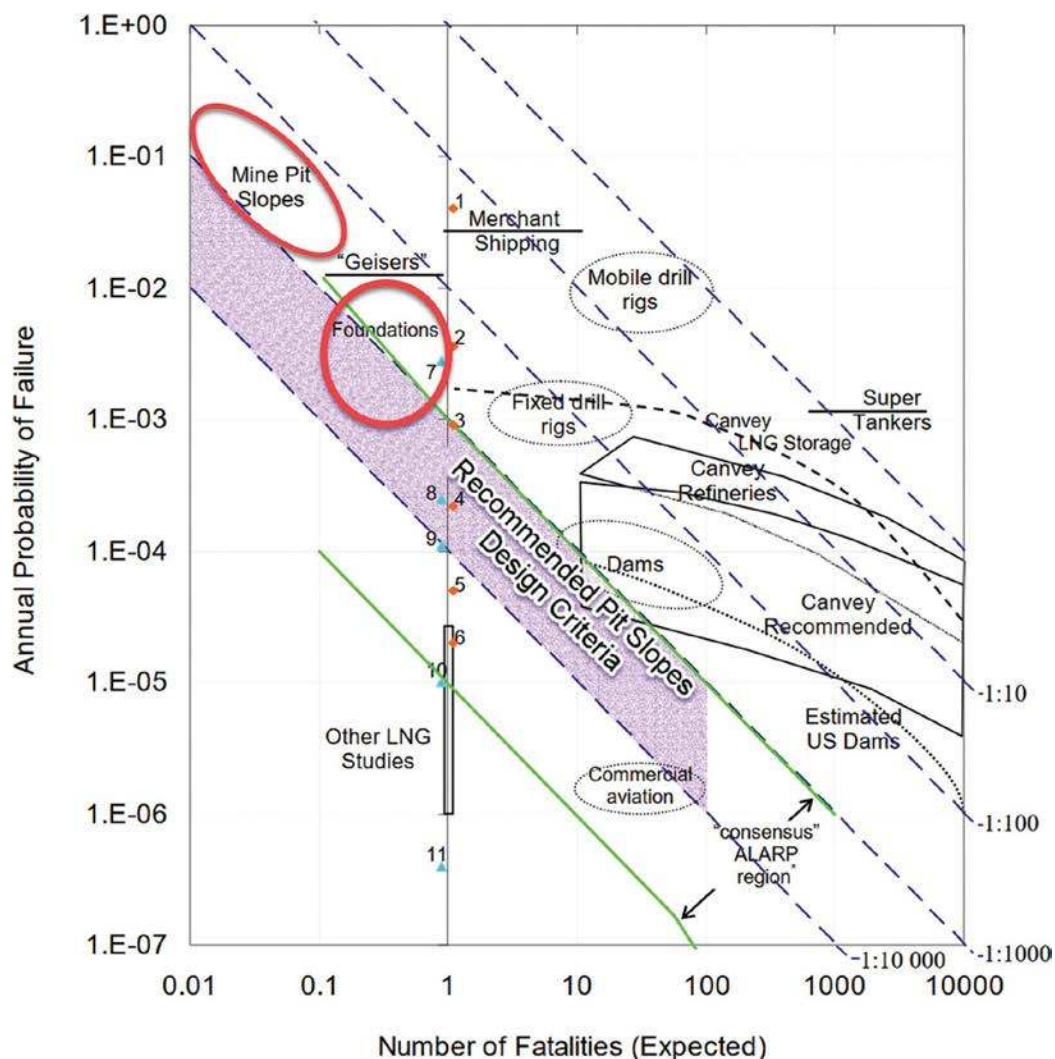
An alternative and (or) complementary approach to stability analysis is to use the concept of PoF, whereby the probability of whether a slope will be stable or not is calculated from FoS distributions. The FoS distributions are derived using random samples from probability density functions of input values. PoF recognizes FoS as a random variable, targeting the probability of it being  $\leq 1$  ( $PoF = P[FoS \leq 1]$ ) (Hoek 1999).

The most widely used design acceptance criteria are still FoS based, although the use of PoF is increasingly popular (Dunn 2013; Duncan and Wright 2005). Steffen et al. (2008) demonstrate the impact of uncertainty on the obtained FoS distribution. These authors show that an FoS distribution defined by a higher mean FoS can also have a higher PoF due to dispersion around the mean FoS value. There is a direct relationship between the PoF and the likelihood of failure, whereas the same is not true for FoS (Steffen et al. 2008). A larger mean FoS does not necessarily represent a safer slope, as the magnitude of the implicit uncertainties is not accounted for.

## 2.3 Existing design (acceptance) criteria based on FoS and PoF

In a mining context (open-pit slopes), it is not uncommon to expect some degree of slope instability during mining operations, but it must be manageable at every slope scale. The acceptability of any failure depends on its consequences. If the failure of a particular slope is deemed to have no impact on safety and production, then there is likely to be minimal concern (Safe Work Australia 2011). The consequence of failure for a mining slope close to public infrastructure is much higher. Figure 3, adapted from Steffen et al. (2006), presents annual PoF versus the expected number of fatalities for various engineering structures. Based on historical data, mining pit slopes are characterized by a low annual PoF and a low consequence of failure. Foundations would be characterized by a 10-times-smaller annual probability of failure and a 10-times-higher consequence of failure. The PoFs for civil slopes are arguably comparable to foundations' PoFs (Wyllie 2017).

In the mining industry, it is not common practice to compute PoF on an annual basis; it is rather computed without a temporal reference. Nevertheless, it should be expected that acceptable PoF design criteria for large mining slopes (in a strictly mining context) should be 10 times higher than those for large mining slopes close to public infrastructure such as roads and railways.



\* region defined by the upper most and lower most ALARP boundaries from: Hong Kong planning department, ANCOLD, U.S. Bureau of reclamation and U.K. HS executive guidelines.

**Individual Fatality statistics U.S.A - Voluntary**

- 1. Space Shuttle program (per flight)
- 2. Cigarette smoking (1 pack per day)
- 3. Average individual voluntary sporting risk
- 4. U.S. Police killed in line of duty (total)
- 5. Frequent flying profession
- 6. Alcohol (light drinking)

**Individual Fatality statistics U.S.A - Involuntary**

- 7. Cancer
- 8. Motor vehicle
- 9. Home accidents
- 10. Air Travel
- 11. Hurricanes / Lightning / Tornadoes

Fig. 3. Comparison of risk acceptability criteria with statistics (adapted from Steffen et al. 2006). ALARP, as low as reasonably practicable; ANCOLD, Australian National Committee on Large Dams; HS, health and safety; LNG, liquified natural gas. [Colour online.]

There are no universal criteria for determining the acceptable FoS and (or) PoF in slope design. Different guidelines are proposed by various authors in the literature, depending on the slope scale (bench or overall scale), and (or) slope design life (temporary or permanent), and (or) slope failure consequences (low, medium or high), and (or) uncertainty about the geotechnical data (data quality) and analyses. This paper focuses mostly on the design criteria applied to deep-seated failure, and to large-scale slopes with high failure consequences such as a mining slope in the vicinity of a public road or other public infrastructure.

One of the earliest design criteria incorporating both FoS and PoF is probably that proposed by Priest and Brown (1983). They developed three slope categories based on the consequence of failure (not serious, moderately serious, and very serious) and suggested design values for FoS ( $>2$ ) and PoF ( $P[\text{FoS} < 1] < 0.3\%$  and  $P[\text{FoS} < 1.5] < 5\%$ ), as presented in Table 1.

Table 1. Design criteria for permanent overall mining slopes in the vicinity of public infrastructure.

Author	FoS	PoF (%)	Slope type	Remarks
Hoek (2007)	$\geq 1.5$	N.A.	Permanent slopes with significant risk of damage	Conservative strength parameters must be chosen for slope design
Priest and Brown (1983)	$\geq 2$	$<0.3$ ; $P[\text{FoS} < 1.5] < 5$	Medium-sized (50–100 m) and high slopes (<150 m) carrying major haulage roads or underlying permanent mine installations with “very serious” failure consequences	All three design criteria should be respected
DME (1999)	$\geq 2$	$<0.3$	Permanent pit walls near public infrastructure with “high” failure consequences	No direct relationship between FoS and PoF; used when lower-bound standard of (acceptable) geotechnical data is available
Hormazabal et al. (2011) (SRK)	$\geq 1.5$	$<3$	Slopes affecting infrastructure or public roads	—
Read and Stacey (2009)	1.3–1.5	$<5$	Overall slopes with high failure consequences	—
Sullivan (2006)	N.A.	$<1$	Slope (overall or inter-ramp) below haul road or important infrastructure	—

Note: N.A. not applicable

According to this design criterion, if one of the three criteria is not met, the slope is deemed to be potentially unstable.

The Department of Minerals and Energy (DME 1999) proposed a design criterion inspired by the Priest and Brown (1983) guideline. This guideline (which is shown in Table 1) is based on data collected in Western Australia and a literature review. According to DME (1999), this approach does not imply any form of direct relationship between FoS and PoF. In other words, according to these authors, the use of one design criterion rather than another (FoS or PoF) will largely depend on the local ground conditions, the potential modes of failure, and the amount of information available. This is the only slope guideline that considers the FoS and PoF as not being mutually exclusive. It is also one of the few that takes into consideration public infrastructure in the vicinity of slopes for design purposes.

Another guideline that takes public infrastructure into consideration is presented by Hormazabal et al. (2011) and uses a mutually exclusive FoS or PoF design criterion for their slope projects. According to this criterion, for slopes affecting infrastructure or public roads, FoS should be  $>1.5$  or PoF should be  $< 3\%$  (Table 1).

Design guidelines were proposed by Read and Stacey (2009), and are actually considered as industry standards for large open pits. According to these guidelines, overall slopes with high failure consequences can have an acceptable FoS between 1.3 and 1.5, and a PoF of  $<5\%$ . Both FoS and



PoF requirements must be satisfied in this approach. These guidelines are not meant to be used when public infrastructure is located near the slope.

From a review of Table 1, it can be concluded that the FoS for slopes close to public infrastructure should be at least 1.5. Nevertheless, each design criterion is based on some assumptions, such as failure consequences, data uncertainties, slope design life, and slope scale. It is unwise to simply compare the numbers without understanding the logic behind them. This paper focuses on overall slopes (scale issue) that should be constructed to be permanent (slope design life). Most importantly, the failure consequence of this slope is considered “high” or “serious”. The only design criteria incorporating slope “failure consequences” are in Priest and Brown (1983), DME (1999), and Read and Stacey (2009). Another important parameter is “data uncertainty,” which is considered only in DME (1999). Both DME (1999) and Priest and Brown (1983) recommend a minimal FoS of 2.0 when data uncertainty is high or when only the lower-bound standard of geotechnical data are available. As mentioned previously, the only PoF-based design criterion that considers “failure consequences” and data uncertainty is DME (1999), with a PoF < 0.3%.

#### 2.4. Long-term and extreme events stability analysis

There is limited scientific literature on long-term stability analysis, due mostly to extensive laboratory tests needed in this regard. However, when it comes to permanent and important structures such as public roads, the importance of long-term stability of nearby slopes should not be overlooked. Long-term stability in abandoned mines can be analyzed according to two topics: aging effects (degradation) and creep (progressive) effects (ISRM 2008). Creep is a type of behavior, while aging is a cause of strength reduction with time. The first one can be a consequence of the second one.

Two types of approaches can be used when performing long-term slope stability analyses. The first approach consists of using degraded rock mass properties as input parameters by applying some reduction factors to critical rock mass properties such as uniaxial compressive strength (UCS) and geological strength index (GSI) and comparing the results with the design guidelines presented in Table 1. Several authors propose a reduction coefficient for these values (ISRM 2008; Woo et al. 2005; Porokhovi 1995; Szczepanik et al. 2003; Chiwaye 2010; Sandøy and Nilson 2012; Franz 2009). The second approach consists of using the current rock mass properties as input parameters and comparing the results with guidelines specifically proposed for long-term stability; e.g., see Table 2 (WMC 1999). According to these guidelines, a FoS between 2 and 3 or higher and a PoF <5% should be used to guarantee slope stability for a period of >100 years.

Lorig (2016) identified two main types of extreme events (earthquakes and rainfall) potentially affecting pit slope stability. He explains why open-pit slopes are, arguably, seemingly more resistant to dynamic loads than natural landforms, which can experience catastrophic landslides. As mentioned in Lorig (2016), the main differences between large open-pit slopes and natural slopes — differences that are believed to be reasons for better performance of large open-pit slopes during earthquakes — are the following:

- Infrequent occurrence of strong earthquakes during the mine’s lifetime.
- Natural slopes exist at a wide range of conditions and wide range of FoS, with some of them being metastable (close to FoS = 1.0) under static conditions.
- Open pits are typically excavated in relatively strong, competent rocks.
- Topographic amplification is greater in natural slopes than in open pits.
- Wave amplification due to heterogeneities is much greater in natural slopes than open pits.

Finally, Lorig (2016) emphasizes that rainfall is more likely to have an extreme-event effect on open-pit slope stability.

Table 2. Australian guideline for long-term stability of rock slopes (adapted from WMC 1999).

Ranking or category	FoS	PoF (%)	Stability	Vulnerability and probability of natural events
1	<1.2	>50	Severe cracking or failing	Critical and $\geq 1$ in slope life
2	1.2–1.5	20–50	Minor cracking	Vulnerable and $\geq 1$ in slope life
3	1.5–2.0	5–20	Apparently stable	Vulnerable and $\geq 1$ in 100 years
4	2–3	0–5	Apparently stable	Vulnerable or $\geq 1$ in 100 years
5	>3	0–5	Stable	Not vulnerable or <1 in 100 years

### 3 LAB CHRYSOTILE MINE, WEST WALL CASE HISTORY

This section presents the geometrical and geomechanical information available at the LAB Chrysotile mine in Quebec, the investigated case study used to demonstrate the applicability of the reviewed guidelines in the context of a stability analysis of a mining slope close to public infrastructure. The actual geometry of the west wall slope is essentially defined by a large failure that occurred in 2010 (Fig. 4). The minimum distance between the pit crest and the new Road 112 is roughly 270 m

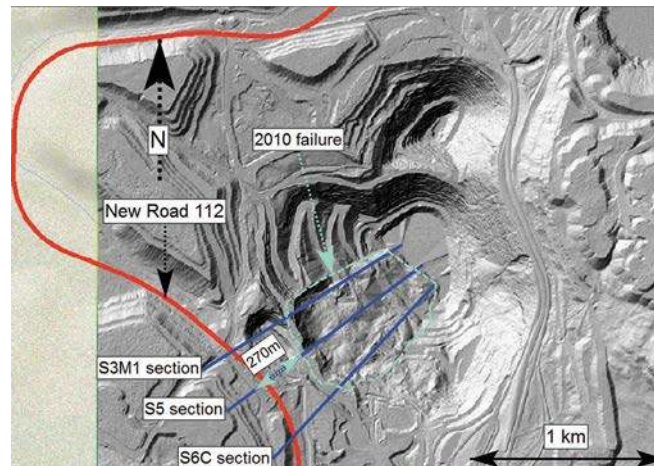


Fig. 4. LAB Chrysotile mine and location of slope sections S3M1, S5 and S6C shown on the hillshade derived from the 2010 LiDAR survey. [Colour online.]

#### 3.1 Slope geometry and lithology

The post-2010 geometry and lithology of the west wall are described in three schematic slope sections: S3M1, S5, and S6C. The location of these sections is shown in Fig. 4, and the lithologies are illustrated in Fig. 5.

The overall slope angle after the 2010 failure varied between  $21^\circ$  and  $26^\circ$ . In addition, the depth of accumulated water (ponded water) in the pit in 2010 was between 32 and 37 m. The slope height varied between 293 and 317 m.

As can be seen, the bedrock mostly consists of massive, unaltered or weakly serpentinized peridotite. A shear zone (talc-carbonate and sheared serpentinite) divides the massive peridotite and the overlying serpentinite. There are also granitoid intrusions in some parts of the west wall including sections S3M1 and S5.

The geometry of the rock units is based on old mine plans provided to the authors by the engineering department of LAB Chrysotile mine. According to these mine plans, the extent of the shear zone was not well defined near the surface. Initial numerical modelling performed by the

authors suggested that the geometry of the shear zone could be a critical factor governing the stability of the west wall (Amoushahi et al. 2014).

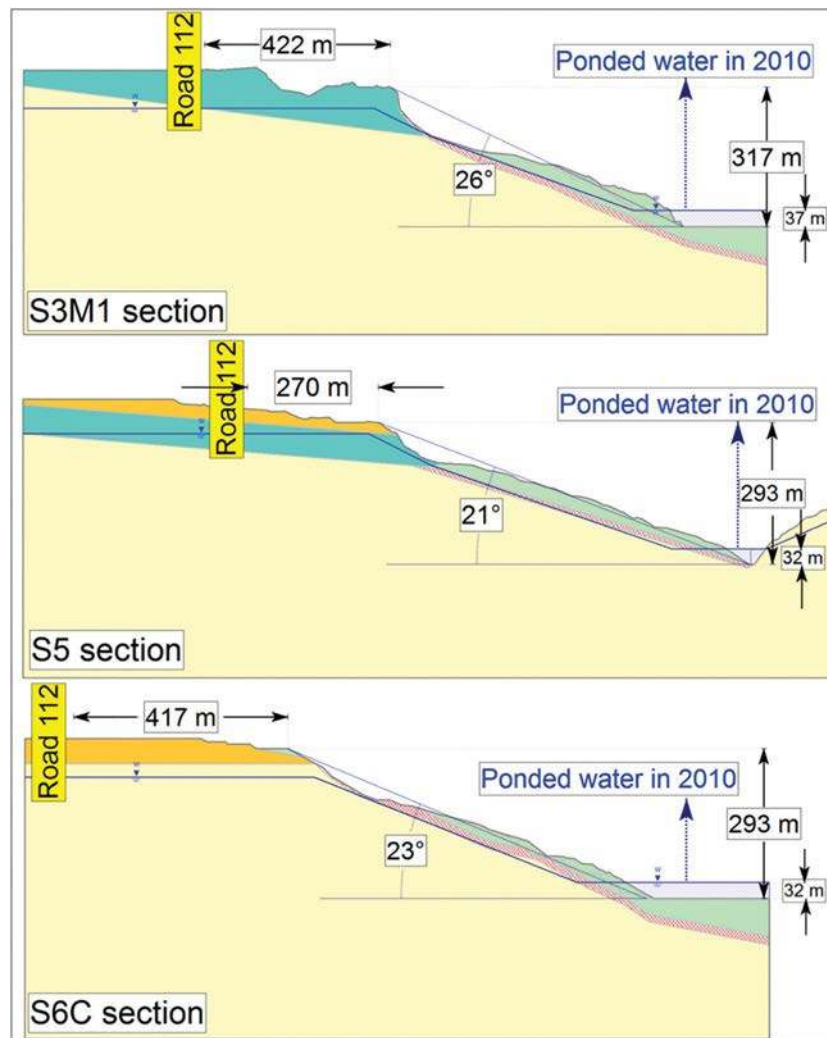


Fig. 5. Schematic view of slope sections S3M1, S5, and S6C after the January 2010 failure (based on November 2010 LiDAR survey). [Colour online.]

A field campaign was conducted to better define the geometrical and geomechanical conditions of the wall. Geotechnical drilling was conducted in April and May 2014 to better define the shear zone geometry and its extent, as well as to better quantify the rock mass characteristics and geometry of other geomechanical units. The 2014 geotechnical drilling plan is illustrated in Fig. 6; it consists of three vertical boreholes measuring approximately 160 m (total length).

Additionally, to further monitor slope movements, two inclinometers and one piezometer were installed at the crest of the pit in the vicinity of the projected road. Six prisms (white circles in Fig. 6) were planned to be installed on both sides of the road to also further monitor slope movement. This instrumentation could additionally provide pertinent quantitative information to calibrate future numerical models. The two inclinometers were installed between the slope crest and the new road in May 2014. The inclinometers readings were taken regularly since their installation and showed no movements.

Four airborne LiDAR surveys have been taken since 2010: in November 2010 (after failure), in July 2012, in August 2013, and in November 2014. LiDAR is a remote sensing technology that

measures distances by illuminating a target with a laser and analyzing the reflected light (Lato 2010; Kemeny 2008).

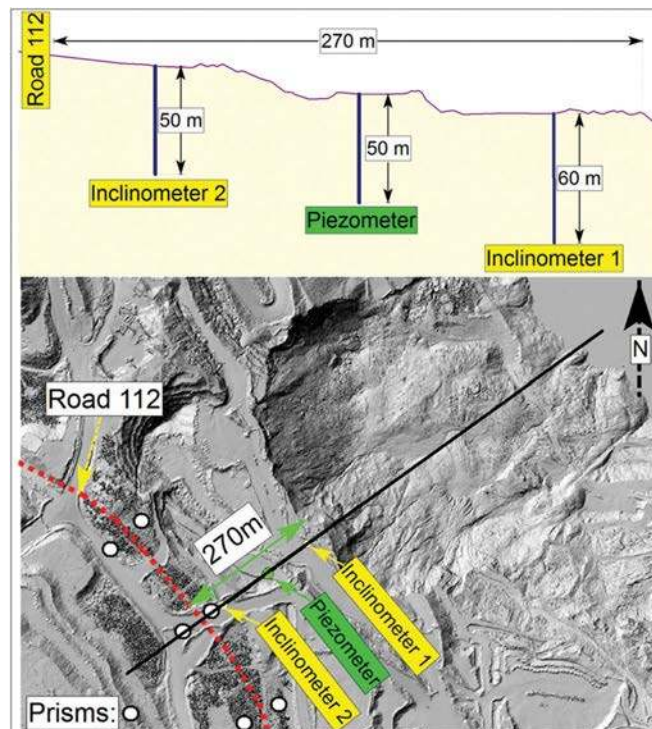


Fig. 6. Geotechnical drilling and instrumentation plan in the vicinity of the pit crest and Road 112; borehole locations and depths are shown. [Colour online.]

Firstly, airborne LiDAR was used to derive slope profiles for different time periods. Secondly, because the 2010 pre-failure slope profiles for sections S6C and S3M1 were available from old mining plans, failure surfaces on these sections were derived by superimposing the 2010 post-failure LiDAR results on these 2010 pre-failure slope profiles. This was made possible because the failure surface was fully visible (outcropping). Thirdly, differential elevations (the difference in Z coordinates) between two LiDAR surveys can be quantified. Differential elevations from 2012–2013 and 2013–2014 were computed. As observed, the west wall experienced no serious differential elevations (or vertical movement) between 2012–2013 and 2013–2014. The LiDAR survey accuracy and change-of-detection threshold assessment method used in this paper are described in Caudal et al. (2016). Based on these surface surveys and inclinometer readings, the west wall appears to be stable since 2012.

The hydrogeological regime assessment is based on existing information regarding hydrogeological conditions. In 2011 and after the mine closure, pumping was stopped and, since then, the pit has been filling up with water. It is expected that the pit will be completely filled with water in 2035 (Caudal et al. 2016).

Water table positions before the 2010 failure were determined based on the piezometer results documented in mine technical reports (Beauchamp 1994). The historical monitoring records indicate a groundwater level below the failure surface for section S6C, and a groundwater level above the failure surface for the S3M1 section. This information is presented in Fig. 5. Current water table positions are provided based on information from the recent piezometer installed in May 2014.

### 3.2. Rock mass properties

Based on historical failures at the mining site, in this paper large-scale failures are assumed to be occurring throughout the highly fractured rock mass. It is thus deemed adequate to use an equivalent continuum approach to represent the rock mass properties.

The major lithological units presented in Fig. 5 are summarized in Table 3. Rock mass properties were derived for each of these main lithologies. This involved compiling data from mine geotechnical reports in which lab testing and rock mass characterization data were reported. Because these reports span several different testing and field characterization campaigns (over a period of more than 50 years), all data were reviewed and evaluated to establish lower-bound, average, and upper-bound values based on values reported (Table 4).

Table 3. Major geomechanical units and summary of their rocks mass characteristics.

Unit	Geological description	Rock mass characteristics
Peridotite	A post-Ordovician intrusive rock consisting mainly of olivine and pyroxene with alteration to serpentinite; the unaltered olivine crystals are about 0.5 mm in diameter. Massive, unaltered to weakly serpentinized. Fractures with calcite and chlorite or asbestos (chrysotile) coatings.	A hard and strong rock with UCS varying from 70 to 237 MPa (average 150 MPa). This unit was described as being moderately to massively fractured (average GSI of 60). RQD = 90%.
Serpentinite	Semi-schistose to schistose; coated by fibrous minerals, with evidence of shearing.	The units are described as being highly fractured (GSI 27–51). RQD < 50%.
Shear zone	Sheared talc–carbonate and (or) serpentinite. Soil behavior materials.	Very weak material (UCS < 20 MPa). The units are described as disintegrated (GSI 10–20).
Granite	Medium to coarse-grained intrusive, felsic, igneous rock. Two types of granite exist at the site: altered granite (67% of total volume) and nonweathered or fresh granite (33% of total volume). Altered granites are more-or-less serpentinized.	Highly variable strength (UCS from 60 MPa in altered granites to 210 MPa in fresh granite) with an average UCS of 112 MPa. The units are described as being highly fractured (GSI 45–55). RQD = 50%–60%.

Note: RQC. rock quality designation

Table 4. Intact rock and rock mass properties for west lithologies.

Lithology	Unit weight (kN/m <sup>3</sup> )	Intact rock								Rock mass											
		UCS (MPa)				mi				GSI				c (kPa)				φ (°)			
		Min.	Avg.	Max.	SD	Min.	Avg.	Max.	SD	Min.	Avg.	Max.	SD	Min.	Avg.	Max.	SD	Min.	Avg.	Max.	SD
Peridotite	27.5	70	150	237	28	20	25	30	2	45	60	75	5	650	1700	4250	600	33	52	61	5
Granite	27.5	60	112	210	25	26	30	35	2	45	50	55	2	450	820	1290	140	38	51	58	3
Serpentinite	26	30	33.5	37	1	10	12	14	1	27	39	51	4	70	125	200	22	21	31	41	3
Shear zone	24	5	12.5	20	3	6	8	10	1	10	15	20	2	14	34	60	8	10	18	26	3

Note: mi, material constant for the intact rock; c, mass cohesion; φ, friction angle; SD, standard deviation.

This work was enhanced by field- and laboratory-based assessments made by the authors during and after fieldwork carried out at the mine site between 2012 and 2014. The rock mass descriptions and rock strengths provided are compiled from several modeling, laboratory testing, and field measurement sources. The rock mass properties were also assessed performing a back-analysis of the east wall failure (Caudal et al. 2016). To capture the variability in the various rock mass properties, the normal (Gaussian) distribution was used. This distribution is the most common type of probability density function, and is generally used for probabilistic studies in geotechnical engineering (Hoek 2007). For a normal distribution, 99.73% of all samples fall within three standard deviations (SD) of the mean value. Therefore, for a normally distributed random variable, the following rule of thumb (the Three Sigma Rule) can be used: SD is the difference between the highest and lowest conceivable values divided by six (Duncan 2000), with the limitation that the mean must be at least three SDs greater than 0, otherwise negative values will occur. This rule was used to assess the SD for the various properties presented in Table 4.

As performed and explained by Woo et al. (2012), from these values, rock mass shear strength properties are estimated to define a Mohr–Coulomb strain-softening constitutive model. These authors also correctly state that most practitioners have more experience and therefore an intuitive

feeling for the physical meanings of cohesion and friction on which the Mohr–Coulomb criterion is based. Accordingly, Mohr–Coulomb rock mass shear strength properties for use in the numerical analysis were derived through empirical procedures based on GSI, rock mass rating (RMR), and the Norwegian Geotechnical Institute (NGI) system (Q). Several empirical procedures exist to derive Mohr–Coulomb rock mass shear strength properties, one of the most commonly used being Hoek et al.’s (2002) conversion, which is achieved by fitting an average linear relationship to the nonlinear Hoek–Brown envelope for a range of minor principal stress values with an upper bound of “ $\sigma_3 \text{ max}$ ”, where  $\sigma_3$  is the minor principal stress. Analytical relationships are provided by these authors (Hoek et al. 2002) for estimating  $\sigma_3 \text{ max}$ .

From a preliminary analysis and the geometry boundaries of the various lithologies, the depth of the critical failure within the rock mass was estimated for all lithologies (peridotite, granite, serpentinite, and shear zone) to be at 150, 80, 30, and 25 m respectively.  $\sigma_3 \text{ max}$  was estimated on these bases. Table 4 reports the corresponding ranges of equivalent rock mass cohesion ( $c$ ) and friction angle ( $\phi$ ) values for the lower-bound, average, and upper bound scenarios. The SD established for each lithological unit represented in the analyses is also presented.

#### 4 BACK-ANALYSIS

Numerical analyses of the 2010 west wall failure were carried out to back-analyze and constrain the material properties to be used for subsequent forward modelling. Two sections (S3M1 and S6C) were used for these analyses. They were the only sections available with pre-failure slope geometry and lithology. The extent of the observed failure was 150 m in height and with a displaced volume of  $2.0 \times 10^6 \text{ m}^3$ .

Two types of analyses were performed: a finite element (FE) analysis (shear strength reduction (SSR) method) using the RS2 software (Rocscience 2015a, version 9.0) and a limit equilibrium (LE) analysis using the Slide software (Rocscience 2015b, version 6.0). Figures 7 and 8 present results for the two sections. The first image (at the left) gives the geometry of the failure as measured by a LiDAR survey; the second and third columns present typical results for FE–SSR and LE analysis, respectively. The failure surface geometry of the numerical model results is in good agreement with LiDAR scanning results.

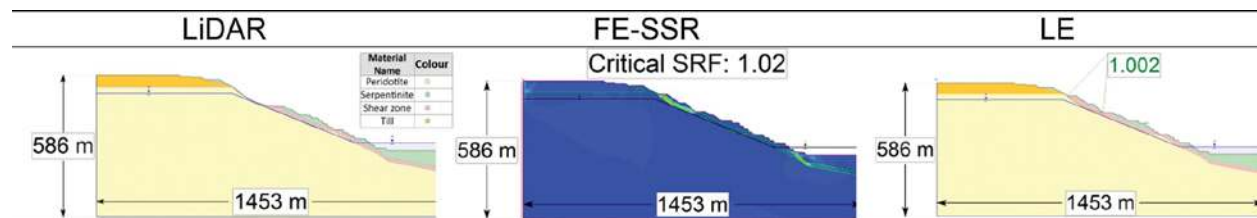


Fig. 7. Section S6C. Back-analysis of the 2010 failure: serpentinite ( $c = 90 \text{ kPa}$  and  $\phi = 23^\circ$ ) and the shear zone ( $c = 60 \text{ kPa}$  and  $\phi = 16^\circ$ ). [Colour online.]

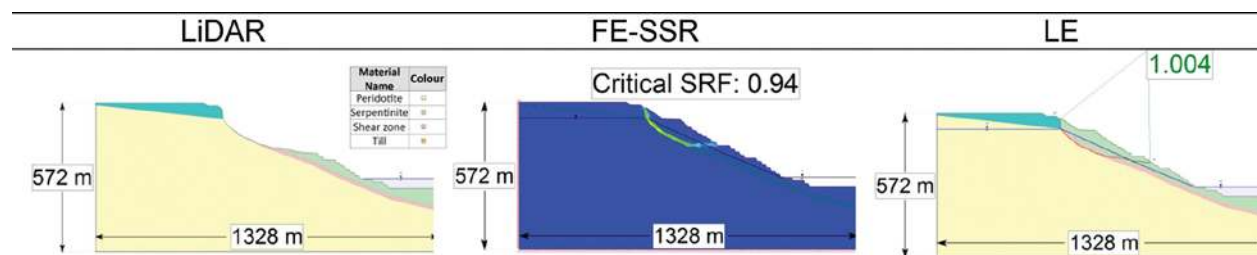


Fig. 8. Section S3M1. Back-analysis of the 2010 failure: serpentinite ( $c = 90 \text{ kPa}$  and  $\phi = 23^\circ$ ) and the shear zone ( $c = 60 \text{ kPa}$  and  $\phi = 16^\circ$ ). SRF, strength reduction factor. [Colour online.]

Rock mass properties of the shear zone and serpentinite were derived by back-analyzing the 2010 failure on these two sections (S6C and S3M1). Rock mass properties were tested from worst-case to best-case scenarios presented in Table 4. The objective was to obtain a FoS close to unity for both analysis methods with a failure geometry that matches LiDAR surveys. The results for rock mass properties are summarized in Table 5. FE–SSR and LE-derived back-analyzed rock mass properties for the shear zone were compatible with the worst to best-case values of historical data. Similarly, back-analyzed serpentinite properties matched well the worst- to the average-case scenario properties of historical data. Based on these results, it was deemed that the historical data presented in Table 4 were a good basis on which to perform forward slope stability modelling for the west wall.

Table 5. Range of rock mass properties obtained by back-analyzing the 2010 failure.

Rock type	Unit weight (kN/m <sup>3</sup> )	$c_{\text{average}}$ (kPa)	$\phi_{\text{average}}$ (°)	Range of $c$ (kPa)	Range of $\phi$ (°)	Remarks
Serpentinite	26	90	23	70–120	21–27	Back-analysis of S3M1 section
Shear zone	24	60	16	15–111	10–22	Back-analysis of S6 C section

## 5 FORWARD ANALYSIS

This section presents both deterministic and probabilistic forward stability analyses of the west wall slope. It investigates the impact of the ponded water levels during pit infilling and dewatering as well as the long-term stability of the wall. Finally, a comparison of the modelling results with the design criteria addressed in Section 2 is presented.

### 5.1 Deterministic approach

This section presents the results of deep-seated deterministic analyses conducted on all three slope sections (S3M1, S5, and S6C). The analyses were undertaken for different ponded water depths, from 0 to the present-day depth of approximately 100 m, based on the latest LiDAR survey available, from November 2014; the analyses continue up to the year 2035 (about 270 m of ponded water), using both FE–SSR and LE methods. The section also presents a deterministic prospective analysis for rapid pit dewatering in the event that mining activity is restarted and that water in the pit could be pumped extremely rapidly, leaving the rock mass saturated. Rapid drawdown analyses are conducted by lowering the water table in different stages (every 50 m) and considering the rock mass above the ponded water level to be fully saturated. The methodology used in rapid drawdown analyses is based on the work of several authors (Cojean and Fleurisson 1990; Lane and Griffiths 2000) and Rocscience recommendations (Rocscience 2015b).

Firstly, deterministic prospective analyses were conducted on slope sections using FE–SSR analysis. The general modelling settings for FE–SSR are presented in Table 6. This table is based on the work of Diederichs et al. (2007) and Hammah et al. (2005). As the horizontal stress distribution in the rock mass was not known, it was deemed reasonable to use hydrostatic-stress field conditions as per the Rocscience recommendation (Rocscience 2015a; Diederichs et al. 2007; Hammah et al. 2005).

A parametric analysis using different alternative input parameters between the worst- and average-case scenarios (Table 4) was performed. A typical result for SSR analysis is presented in Fig. 9 for section S6C. The numerical results were obtained using worst-case input data. The distance from the crest to the critical failure surface is presented.

For section S3M1, all the investigated water levels during filling of the pit lake are presented in Figs. 10a–10f. We see that as the water level rises, the FoS increases and the position of the critical

failure surface moves further behind the slope crest when the water reaches 150 m. The total length of failure surface also increases.

Table 6. General settings for all FE–SSR analyses based on Diederichs et al. (2007) and Hammah et al. (2005).

Parameter	Value or condition
<b>General</b>	
Analysis type	Plain strain
Solver type	Gaussian elimination
<b>Stress analysis</b>	
Maximum number of iterations	500
Tolerance	0.001
Convergence type	Absolute energy
<b>Mesh</b>	
Type	Uniform
Element type	6-noded triangles
Number of mesh elements	3000
<b>Boundary conditions</b>	
Bottom	Restrain y
Left and right	Restrain x
Top (surface)	Free
Upper corners	Restrain x and y
<b>Stresses</b>	
Stress type	Gravity (using actual ground surface)
Total stress ratio (horizontal/vertical)	
In plane	1
Out of plane	1
Locked-in horizontal stresses	0
<b>Material properties</b>	
Initial element loading	Field stress and body force
Elastic type	Isotropic
Failure criterion	Mohr–Coulomb
Material type	Perfectly elastoplastic (peak values = residual)
Young's modulus (MPa)	100
Poisson's ratio	0.3

For this same section (S3M1), similar results were obtained in the case of rapid drawdown of the pit (with a saturated rock mass), shown in Figs. 10g–10l. The FoS results are slightly lower than the one obtained during infilling, due to the level of water within the rock mass in the upper part of the slope (above the ponded water level). In this case and using worst-case properties, the analyses reveal that after reaching a ponded water depth of 100 m, the FoS could be below 2.0.

The SSR results for all sections, for all investigated scenarios, and all parametric studies are presented in Fig. 11. Furthermore, the SSR results for a set of input values situated between worst- and average-case values are also presented. This scenario corresponds to the 5th-percentile value of the input parameters distribution, which could adequately represent a realistically achievable worst-case scenario as suggested by CEN (2004) and Bozorgzadeh et al. (2015).

The results were then compared with the results of LE analyses. A typical result for section S6C is presented in Fig. 9. The FoS results for all sections and for all investigated scenarios are also presented in Fig. 11. It can be seen that when using worst- or average-case scenarios input data, the results were identical for LE and FE–SSR. The numerical results and trends obtained were similar for all sections. The conservative input parameters assumptions (UCS, GSI, material constant for the intact rock ( $m_i$ ), and disturbance factor ( $D$ )) are the accumulation of conservative values. Deterministic approaches considering the worst-case scenario does not take into account



the low probability of the combined occurrence of worst-case values. Probabilistic analysis can provide a more realistic account of the instability potential.

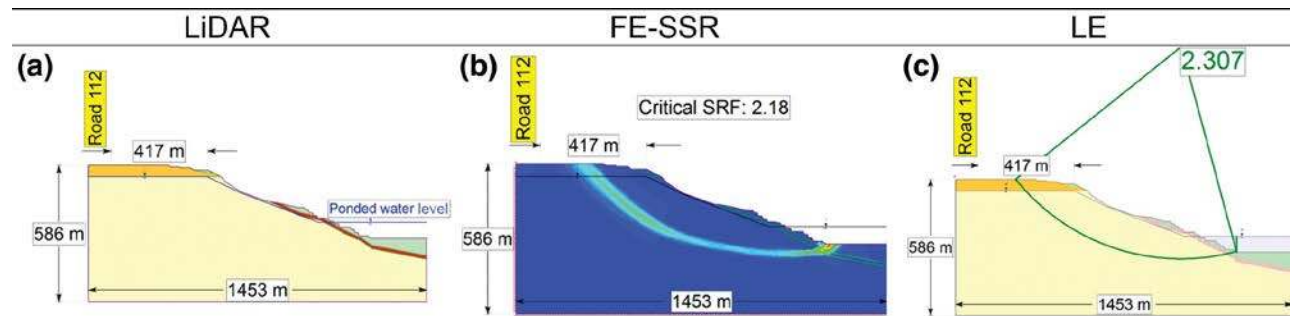


Fig. 9. Deep-seated deterministic analysis carried out on section S6C (ponded water level of 2012 and considering worst-case properties): (a) model geometry; (b) FE-SSR model result; (c) LE model result. [Colour online.]

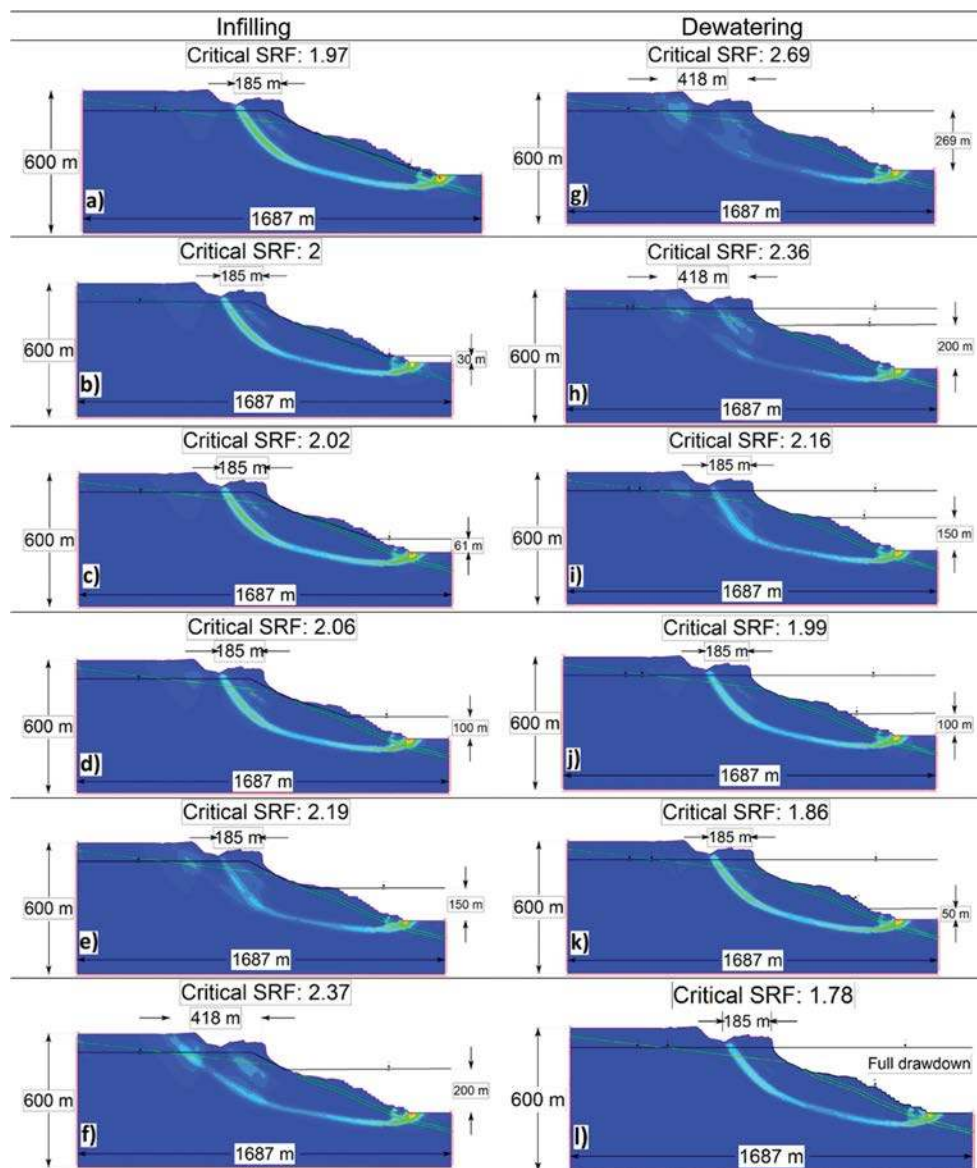


Fig. 10. FE–SSR analysis results for section S3M1: (a–f) pit infilling; (g–i) rapid dewatering. [Colour online.]

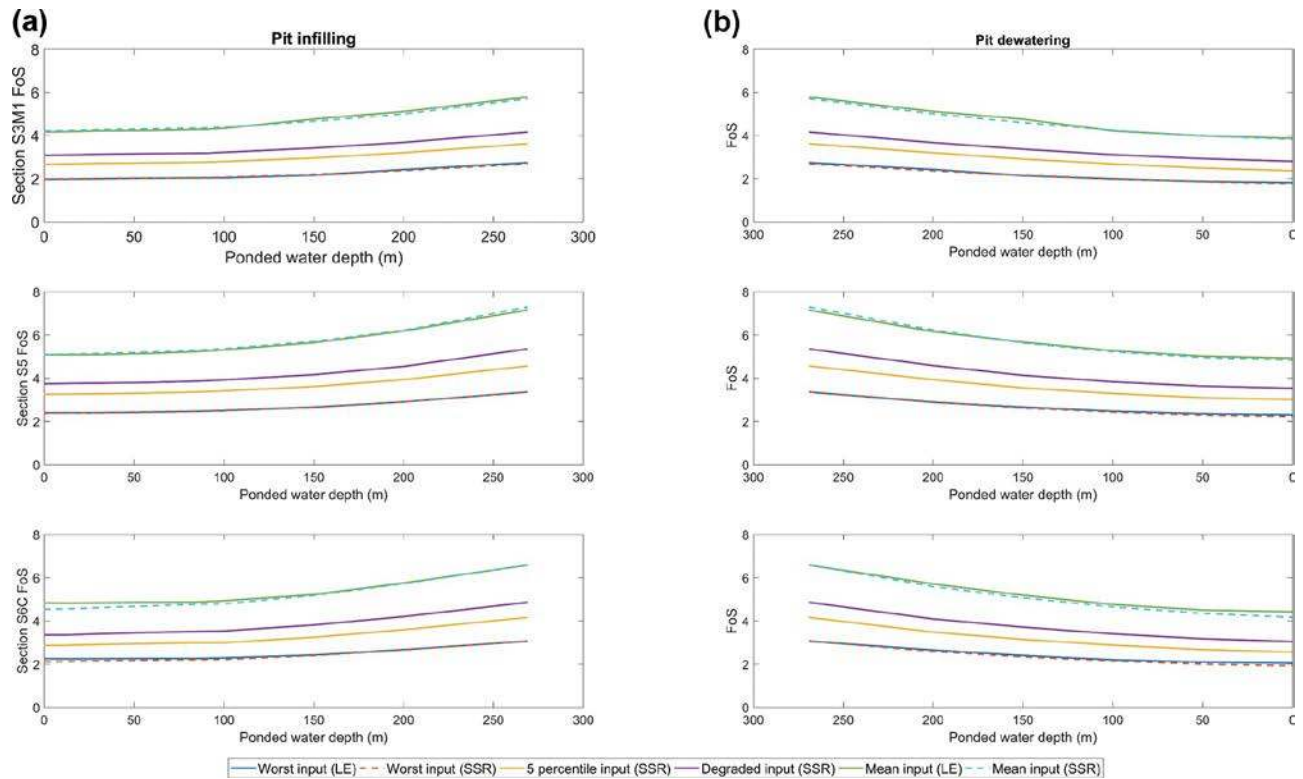


Fig. 11. Deterministic FoS obtained for slope sections S3M1, S5, and S6C considering various water levels: (a) pit infilling; (b) rapid dewatering. [Colour online.]

## 5.2 Probabilistic approach

In this section, probabilistic FE–SSR and FE analyses are undertaken on all sections (S3M1, S5, and S6C), and the results are compared with LE results. Probabilistic analyses for slope stability are not routinely performed in Quebec for slope analysis using LE methods. Monte-Carlo probabilistic FE–SSR were implemented in RS2 (version 9.0, Rocscience, Inc.) in 2015 and thus not performed at mining sites.

FE–SSR probabilistic analyses are time-consuming. Nevertheless, they are useful in assessing the distributive nature of the FoS results. For the investigated slope models, when performing FE–SSR analysis, the computation time ranged from 3 to 5 h per analysis for 100 samples and about 30 h for 1000 samples. For assessment purposes, mean FoS was deemed sufficient to compute 100 samples for all 36 investigated scenarios.

FE–SSR Monte-Carlo simulations were thus performed to compute mean values of the FoS distribution for the investigated slopes. To do so, 100 values for each random variable,  $c$  and  $\phi$ , were generated during FE–SSR analysis for every ponded water level, according to the normal distribution attributed to every random variable (Table 4).

To assess the PoF, especially when the PoF values are low, 100 FE–SSR simulations are not sufficient, and more robust approaches should be used (Huang et al. 2017). One valid approach is to carry out a large number of Monte-Carlo simulations. In practice, 10 000 simulations are typically used. Unfortunately, due to computing time limitations, such a large number of simulations is not practically feasible when using an FE–SSR approach. Alternatively, PoF could be computed without evaluating an FoS and focussing only on the numerical convergence of the simulations. Such an

approach, although conceptually simple, is not always embedded in commercial software such as RS2. A methodology was developed to perform Monte-Carlo analysis with 10 000 samples—using standard FE — and then compute PoF, by investigating the models’ numerical convergence. This methodology was validated by comparing FE PoF results with FE–SSR PoF results for a simple worked example. The FE PoF approach is less computer intensive than FE–SSR. For the 36 scenarios investigated, single simulations—10 000 samples — took on average 24 h to compute, for a total of 36 days of computing time. All PoF obtained using a FEprobabilistic approach (10 000 samples) were evaluated to be  $< 10^{-4}$ . Although arguably data-limited (100 samples), due to computer time, FoS distributions were used to compute the mean of the FoS distribution. The mean FoS results for probabilistic FE–SSR analyses conducted on all slope sections are shown in Fig. 12. The FoS associated with the 5% passing of the FoS distribution is also presented. Although we can see the difference when considering 5% and the mean, the results are still well above a FoS of 2.

The results of LE probabilistic analyses conducted on all three sections are presented in Fig. 12. The same FoS were observed for both LE and FE–SSR analysis at 5% and mean values. The PoF values obtained for LE (10 000 samples) analysis were evaluated to be  $< 10^{-4}$ .

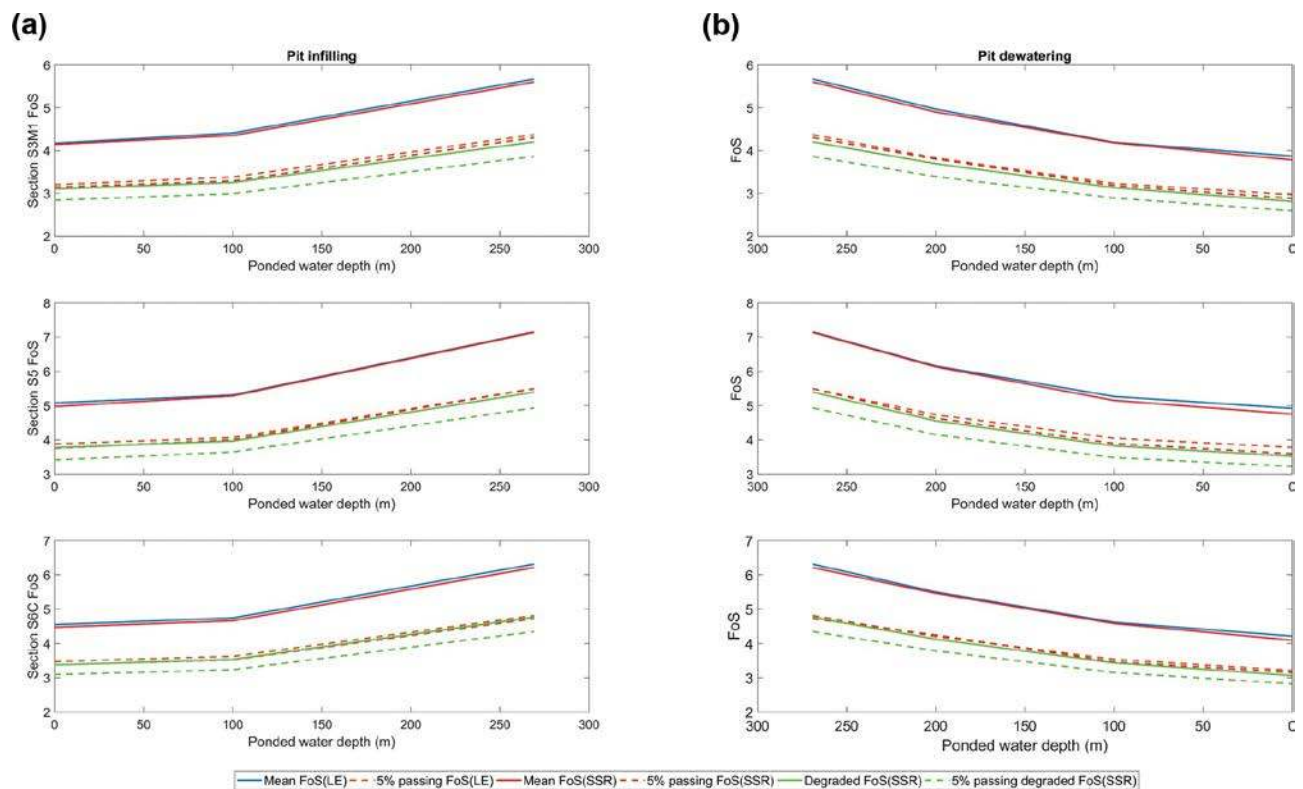


Fig. 12. Probabilistic FoS obtained for slope sections S3M1, S5, and S6C considering various water levels: (a) pit infilling; (b) rapid dewatering. [Colour online.]

### 5.3 Long-term stability

This section presents the probabilistic long-term, deep-seated slope stability of the west wall using three slope sections: S3M1, S5, and S6C. As presented in Section 2, two approaches are used. Firstly, the results of the probabilistic slope stability analysis presented in Section 5.2 are compared with the long-term design criteria presented in Table 2; this will be discussed in Section 5.4. Secondly, a series of probabilistic FE–SSR analyses were conducted on all slope sections assuming that the rock mass properties would be reduced over the years.

Based on the literature presented in Section 2 (ISRM 2008; Woo et al. 2005; Porokhovi 1995; Szczepanik et al. 2003; Chiwaye 2010; Sandøy and Nilson 2012; Franz 2009) it was deemed acceptable to reduce UCS by 20% and GSI by 5 units to reproduce the long-term properties of the rock mass. These reduction coefficients were applied on the UCS and GSI values for worst-, average-, and bestcase scenarios to derive new Hoek–Brown criteria. Accordingly, new ranges of rock mass strength parameters ( $c$  and  $\phi$ ) were established. In Fig. 12, the value of the obtained FoS for a FE–SSR analysis using degraded material are presented. Using the methodology presented in Section 5.2, PoF were all computed and evaluated to be  $<10^{-4}$ . Even when considering the FoS corresponding to the 5% passing of the computed FoS distribution, it is  $> 3$  for all scenarios.

#### 5.4 Comparing with available design criterion

In Section 2, some design criteria were presented for acceptable FoS and PoF for mining slopes in the vicinity of public infrastructure. As mentioned, the conservative design FoS and PoF for such slopes with high failure consequences are, respectively, 2.0 and 0.3%. A FoS between 1.5 and 2.0 and PoF below 1% would be tolerable. It must be kept in mind that for a given slope, the acceptable design upper limit PoF should be reduced by a factor of 10 when comparing a global mining slope with high failure consequences (from the operation-only perspective) and a global mining slope close to a public road or a public railway. This is in direct relationship with the failure consequences as stated by Steffen et al. (2006) and Wyllie (2017).

The results obtained from FE–SSR and LE models indicate that all slopes analyzed would be stable with mean FoS  $> 3.0$  and PoF  $< 10^{-4}$ . In other words, the derived FoS and PoF meet the acceptable design criteria. These conclusions are also valid for degraded material associated with the long-term behavior of the slope. The design guidelines proposed in the literature and tested in this paper are perfectly applicable in the context of open-pit mines in Quebec.

## 6 CONCLUSIONS

This paper aimed at investigating the best design practice for slope design for an open-pit mine close to public infrastructure. This was deemed important in the Quebec context, where several large pits were currently developing, and where no specific design guidelines existed in the province for such engineering work. It was also deemed important to investigate deterministic and probabilistic design methods in that context.

An overview was given of the existing design guidelines, in the literature, and in other mining legislations, for slope stability analysis and design of mining slopes in the vicinity of public infrastructure such as public roads. To validate and test the approach, the paper presented slope stability analysis results for a major public road at the vicinity of an existing open-pit mine that previously experienced major slope failures. Rock mass property ranges and statistical distribution were assessed based on historical data and field and laboratory work. Back-analysis of the 2010 failure was used to constrain rock mass properties for some lithologies, to improve our understanding of the failure mechanisms on the west wall and to improve our confidence in using deterministic and probabilistic slope stability analysis, using both the LE method of slices and the FE–SSR approaches. The limitations of the proposed methodology could not be ignored, especially the sensitivity of the modelling results to the analysis inputs.

Nevertheless, the deterministic (worst-case and mean-case scenarios) and the probabilistic analysis approaches, combined with the design criteria used, were deemed appropriate to adequately account for data uncertainty. Forward modelling was thus performed considering data uncertainty, and using both deterministic and probabilistic approaches. The impact of pit infilling or rapid dewatering was investigated. Long-term stability analysis was also studied. All analysis results suggested that the slope is stable, and that the integrity of the public road should not be affected in either the short or long term.

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