Assessment of the influence of the slope stability conditions of an inactive open-pit mine on the design of a nearby highway

Sina Amoushahi & Martin Grenon
Département de génie des mines, de la métallurgie et des matériaux – Université Laval, Québec, Canada
Jacques Locat & Dominique Turmel
Département de géologie et de génie géologique – Université Laval, Québec, Canada

ABSTRACT
This paper presents a review of the literature on the design of public roads in the vicinity of open-pit mines, focusing on the latter's impact on slope stability. It then presents a case study on the design of a major new regional highway along the crest of an abandoned mine in the city of Thetford Mines, Québec, Canada. The first step involved a back analysis of a recent slope failure close to the location of the planned highway in order to derive rock mass properties at the slope scale. Analyses were conducted using the Shear Strength Reduction (SSR) method coupled with finite element (FE) modeling as well as the limit equilibrium analysis (LE) method. Airborne LiDAR surveying results were used in order to calibrate and validate the models. Forward modelling was then performed to assess future slope stability.

RÉSUMÉ
Cet article présente une revue de la littérature sur la conception des routes publiques à proximité de mines à ciel ouvert avec une emphase sur la stabilité des pentes. Il présente ensuite une étude de cas sur la conception d'une nouvelle route régionale située en bordure de la crête d'une mine abandonnée dans la ville de Thetford Mines, Québec, Canada. La première étape a consisté à effectuer une rétro-analyse d'une rupture de pente récente à proximité de l'emplacement de la route prévue pour obtenir les propriétés de la masse rocheuse. Les analyses ont été effectuées en utilisant la méthode de réduction de la résistance au cisaillement (SSR) implémentée dans un code d'éléments finis (FE) ainsi que la méthode de l'analyse à l'équilibre limite (LE). Les résultats de LiDAR aéroportés ont été utilisés pour calibrer et valider les modèles. La modélisation prospective a ensuite été effectuée pour évaluer la stabilité future de la pente.

1 INTRODUCTION

In Québec, there are few examples of open pit mines in the vicinity of highways. There are no existing formal slope design guidelines for such engineering works. In the last few years there has been an increase in the number of open-pit mine projects close to existing public infrastructures.

In 2012, a large slope failure at an existing mine forced the provincial government to close a major highway. This failure ultimately led to the design and construction of a new highway in the vicinity of the same open pit (Caudal et al. 2013, and Caudal et al. 2014).

Such a slope failure can have severe economic and safety consequences for the government and the local population.

This paper presents a brief review of the literature on the design of mining slopes in the vicinity of public roads and other surface infrastructures. It also presents the slope stability assessment for the new highway design in the aforementioned case study using: empirical criteria, the Shear Strength Reduction (SSR) method implemented in Finite Element Modeling (FE-SSR) as well as the Limit Equilibrium (LE) Method.

2 CRITERIA FOR DESIGN OF PUBLIC ROADS IN THE VICINITY OF OPEN-PIT MINES

There is little scientific literature on the design of public roads in the vicinity of open-pit mines. From the available literature, design criteria can be divided into three main categories:

1) Analysis based on empirical criteria such as that proposed by the Australian Department of Industry and Resources, (1997).

2) Analysis based on the factor of safety (FoS) and probability of failure (PoF) (Wyllie & Mah, 2004 and Read & Stacey, 2009).

3) Analysis based on strain analysis (Juncal & Ivars, 2012 and Sjoberg, 2013).

2.1 Empirical Criteria

The only existing formal guidelines for the design of public roads in the vicinity of open-pit mines known to the authors were proposed by the Department of Industry and Resources (1997) in Western Australia. These guidelines are used for long-term slope stability of abandoned open-pit mines in Western Australia. According to these guidelines, the overall angles defining the maximum

potentially unstable rock mass in weathered and unweathered rocks should be 25° and 45° respectively (Figure 1). Furthermore, a bund wall should be constructed at least 10 m outside the area designated as being potentially unstable rock mass. This is considered a conservative design approach aiming at minimising the risk.
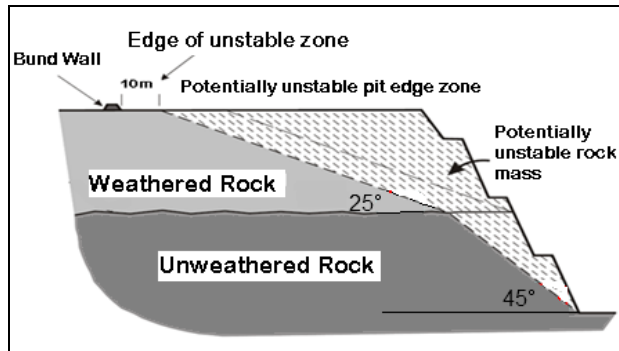

Figure 1. Potentially unstable pit edge zone according to the Western Australian guideline (Department of Industry and Resources, 1997)

2.2   Factor of Safety

Factor of safety (FoS) and probability of failure (PoF) concepts have been used in slope stability analysis of open-pit mines for a long time (Wyllie & Mah, 2004; Read & Stacey, 2009). Accordingly, there is abundant literature on their application for all types of geological conditions. The factor of safety generally used for active open-pit mines is in the range of 1.2 to 1.4 (Wyllie & Mah, 2004). However, higher FoS`s are applied for overall and permanent slopes as shown in Table 1. As can be seen, the most conservative design criterion is based on the guidelines issued by the Department of Minerals and Energy of Western Australia (1999), with a FoS of 2 and PoF of 0.3%. It is also the only criterion specifically aiming at protecting public infrastructures.

Table 1. FoS & PoF used for permanent slopes

Author	FoS	PoF(%)	Application
Department of Minerals and Energy of Western Australia (1999)	2	0.3%	Serious failure consequences; permanent pit walls near public infrastructures.
Hoek (2007)	>1.5	N.A	Permanent slopes with significant risk of damage.
Read & Stacey (2009)	1.3 – 1.5	5%	Overall slope scale; high failure consequences

The Limit Equilibrium Method (LEM) is generally used for slope stability analysis in both soil and rocks to derive FoS and PoF. The basic assumption of the LEM is that a failure criterion is satisfied along an assumed failure surface. Failure surface could be circular or non-circular. The FoS of a slope in LEM is defined as the ratio of rock mass shear strength to shear stress (Wyllie & Mah, 2004). The most important limitation of LEM is that it neglects the stress-strain relationship and presupposes that failure is sudden without real warning signs.

In recent years, numerical modeling has been used frequently for studying slope stability. The shear strength reduction (SSR) approach is one of the most common methods currently used in numerical slope stability analysis. This method is typically used to calculate FoS by progressively reducing or increasing the shear strength of the material by a strength reduction factor (SRF) to bring the slope to the limit equilibrium state.

Application of the SSR method in stability analysis of rock slopes is discussed in details by several authors (Dawson et al. 1999; Hammah et al. 2004 & 2005; Hoek, 2009, and Read & Stacey, 2009). SSR method is well integrated with different numerical methods such as Finite Element (FE), Finite Difference (FD) and Distinct Element (DE) methods (Diederichs et al. 2007).

2.3   Strain Analysis

More recently, a strain criterion originally developed by LKAB mining company (Juncal & Ivars, 2012, and Sjoberg, 2013) was used in the design of public roads and other surface infrastructures in the vicinity of mining sites in Sweden.

This criterion is based on measuring the ground surface deformations between monitoring points, where the differential deformation (strain) limits are not allowed to be greater than 0.3% horizontally and 0.2% vertically (tilt) between monitoring points 50 m or less apart. This means that for a maximum distance of 50 m between monitoring points, the maximum horizontal and vertical (tilt) displacements allowed are 15 and 10 cm, respectively (Figure 2).
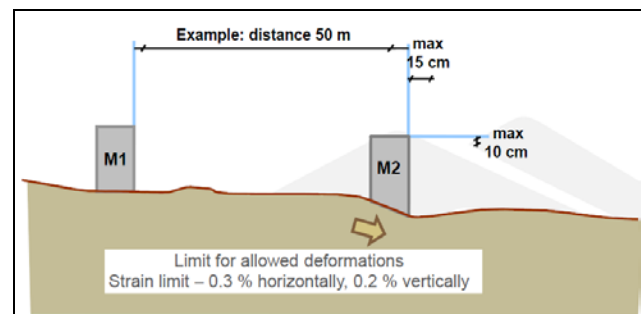

Figure 2. Deformation criterion for maximum allowable influence of mining on the ground surface (Stockel et al. 2012)

Hoek and Karzulovic (2000) also estimated that slopes begin to show signs of instability if the ratio

between the slope crest horizontal displacements and the slope height is greater than 2%.

3 CASE STUDY: LAB CHRYSOTILE MINE

3.1 Introduction

LAB Chrysotile mine is located at Thetford Mines, in the Chaudière-Appalaches region in southern Quebec, Canada (Figure 3). The mining operations began in 1958 (Beauchamp, 1994) and ended in 2011.

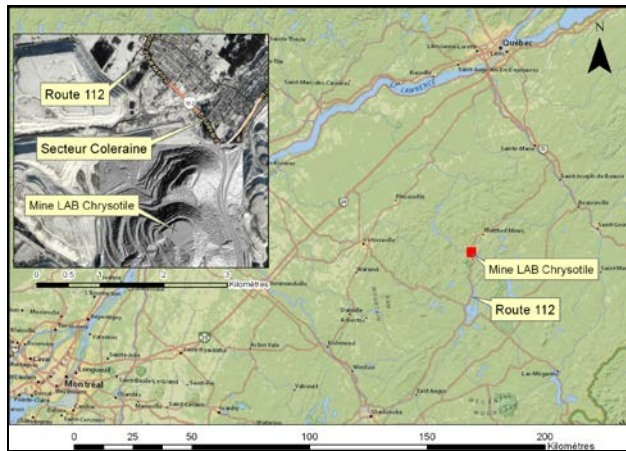The original highway 112 was located along the crest of the East wall of the pit (Figures 4 and 5). In 2012, this road sustained a major failure (Figure 6).

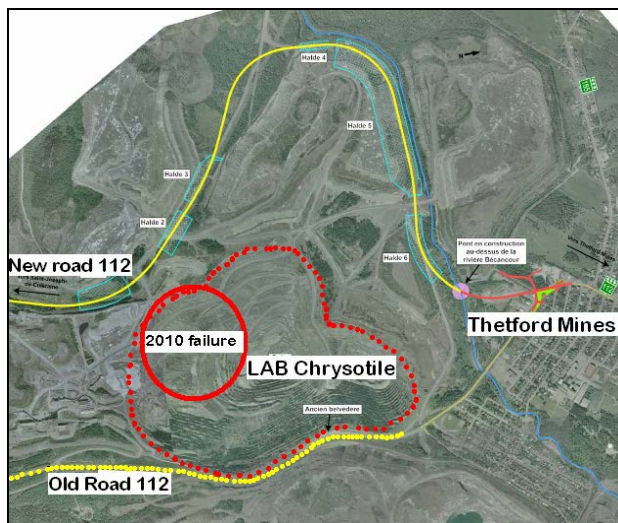Figure 3. Location of LAB Chrysotile mine in Quebec

Figure 4. LAB Chrysotile mine and locations of the old and the new highway 112

A new layout was proposed by the MTQ (*Ministère des Transports du Québec*) for highway 112 in the vicinity

of the West wall of the pit (Figures 4 and 5). The minimal distance between the pit crest and the new highway layout is roughly 260 m. Construction of the new highway started in the summer of 2013.
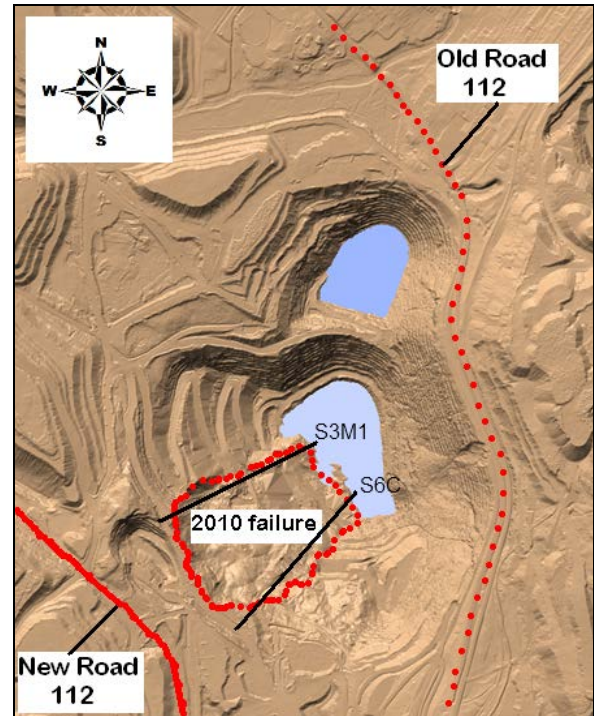

Figure 5. LAB Chrysotile mine and locations of the old and the new highway 112

Figure 6. Failure of old highway 112 in the vicinity of the East wall (2012)

3.2 Geology

The host rock at the mine is peridotite that is subjected to varying degrees of serpentinization. The geology of the

West wall is described in two sections in Figure 7. These sections are also localized in Figure 5. The bedrock mostly consists of massive peridotite. A shear zone divides the massive peridotite and the overlying schistose serpentinite. The abrupt ending of the serpentinite and shear zone was deemed realistic by the engineering department at the mine site based on field work.
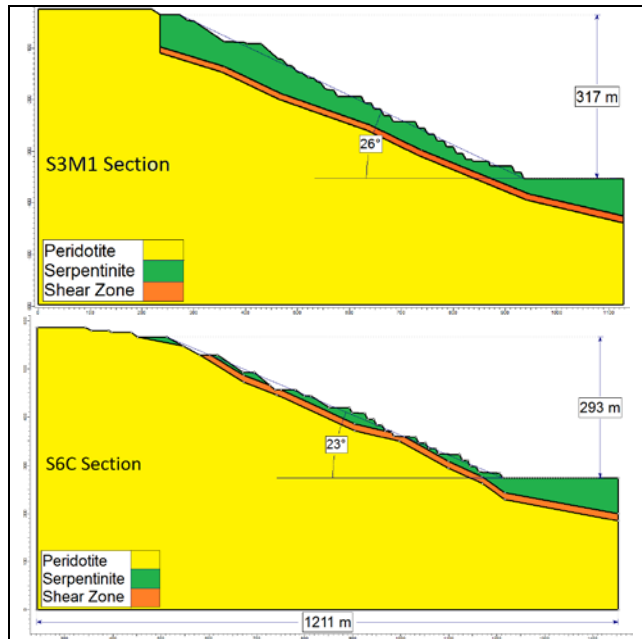

Figure 7. Two sections (S3M1 and S6C) of the West wall of the pit

Table 2. Lower- and upper-bound values for intact and rock mass properties (UCS: Uniaxial Compressive Strength, GSI: Geological Strength Index and mi: material constant for the intact rock) based on the mine geotechnical reports (Beauchamp, 1994)

Lithology	Scenario	UCS (MPa)	GSI	mi	Density (kN/m^3)
Peridotite	Lower	69	45	20	27.5
	Upper	250	55	30	
Serpentinite	Lower	30	27	10	26
	Upper	37	51	14	
Shear Zone	Lower	30	22	8	24
	Upper	37	27	10	
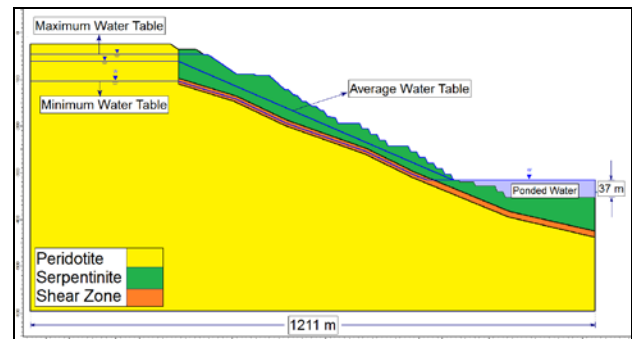
Figure 8. Possible water table geometries investigated in the back- analysis

3.3 Rock Mass Properties

Based on historical data obtained through various mine geotechnical reports, including lab testing and rock mass characterisation, lower- and upper-bound values of rock mass properties for each geological lithology were determined (Table 2). The lower and upper bounds were defined based on the minimum and maximum values reported by Beauchamp (1994).

These historical data were used as a starting point for the back-analysis presented in section 3.5. Additionally, a blast damage factor (D) of 0.85 was established for schistose serpentinte. The D value for the shear zone and massive peridotite was defined as 0.7.

3.4 Water Table

For each section, three possible water table positions were defined (Figure 8). Based on field evidence, it was assumed that the water table was above the shear zone. The minimum water table level was thus considered to be at the elevation of the shear zone. At the maximum water table level, the slope was considered to be fully saturated.

3.5 Back-Analysis of the 2010 Failure

The West wall experienced a certain number of small and medium scale slope instabilities over the last decades. A larger failure occurred on that same wall in 2010 (Figures 5 and 9).

Figure 9. 2010 failure (red dashed line) in the West wall

To perform a complete stability analysis of the mine slope at the vicinity of the new road layout, a back analysis of this large failure was performed to derive rock mass properties at the slope scale.

Figure 7 illustrates two sections (S3M1 and S6C) on the West wall (also located in Figure 5) used for the back-analysis. In this study, the back-analysis is carried out on section S3M1 and then the results are validated using section S6C.

A failure surface for section S3M1 was derived from LiDAR surveying (Figure 10). Given the approximate location of the 2010 failure surface and assuming a FoS of 1 at the time of failure, a back-analysis was performed.
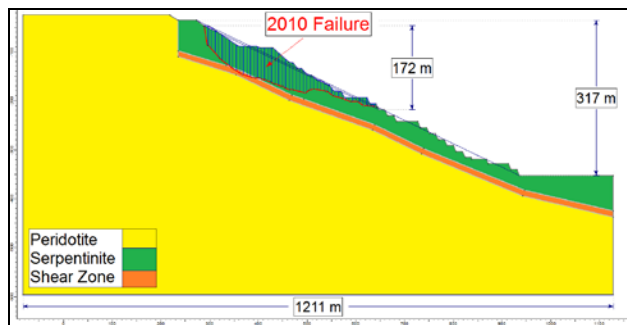

Figure 10. 2010 failure surface for section S3M1 derived from airborne LiDAR surveying

3.5.1 Back analysis using FE-SSR

Back-analysis was first carried out applying an FE-SSR approach using the commercial finite element code Phase2 (Rocscience, 2012). In SSR investigations, slope stability analyses are run for a series of increasing trial factors of safety (SRF) until failure occurs (Hammah et al. 2004 & 2005).

Figure 11 presents the models used in FE-SSR analysis for two water table positions on section S3M1. Ponded water, with a depth of 37 m, is represented by an equivalent distributed load normal to the submerged section of the external boundary of the slope model (Rocscience, 2012).

Rock mass shear strength properties, i.e. cohesion (c) and internal friction angle (Φ), were estimated using the Mohr-Coulomb constitutive model based on the ranges of values presented in Table 2. According to Wyllie & Mah (2004), the average slope height is equal to average "slice" height in the slice methods used in limit equilibrium analysis. This height with the rock mass unit weight together define the average vertical stress on the sliding surface. This average slope height was approximated to 55 m in the present analysis.

The analyses were conducted using a 6-node triangle mesh type based on recommendations of Hammah et al (2005). The mesh is built up of 3000 elements. Gravitational stress distribution was considered throughout the slope.

The SSR search area method was adopted to impose the 2010 failure surface on the slope. As observed, FoS of 1.2 and 0.86 were obtained for average and fully-

saturated water table positions respectively. Due to the uncertainty about the water table level at the time of failure, it can arguably be said that a water table level between the average scenario (FoS = 1.2) and the fully-saturated scenario (F0S = 0.86) can satisfy a FoS of 1 assumed at failure.
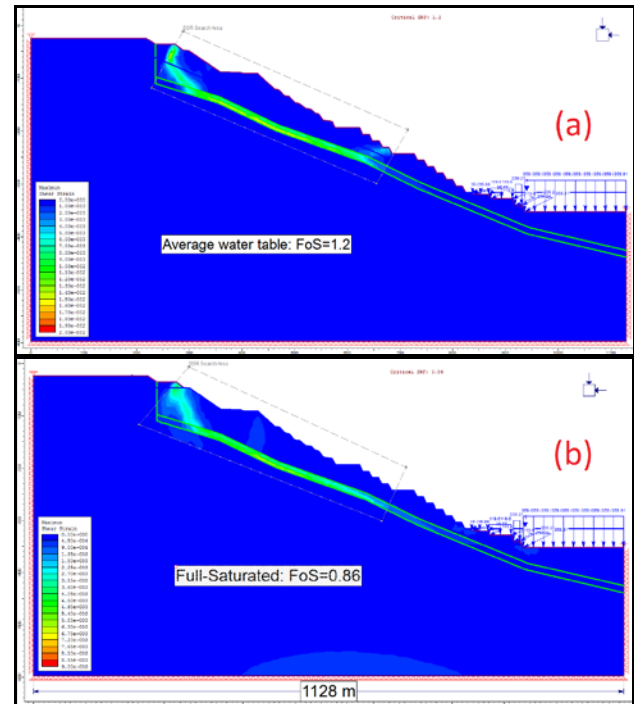

Figure 11. Back-analysis results for section S3M1 by FE-SSR: (a) average water table conditions (FoS = 1.2) and (b) fully-saturated conditions (FoS = 0.86)

The rock mass properties presented in Table 3 do enable replicating the 2010 failure for section S3M1. These calibrated rock mass properties at the slope scale were deemed adequate for the various lithologies. These values correspond to fitting a Mohr-Coulomb model the Hoek-Brown criterion at a depth of 55m (which is the average depth of the failure surface).

Table 3. Calibrated rock mass properties for section S3M1 derived from back-analysis

Lithology	Young's modulus (GPa)	Poisson's ratio	c (kPa)	Φ (º)
Peridotite	6.5	0.3	911	57.8
Serpentinite	1.885	0.27	226	32.4
Shear Zone	0.71	0.25	113	22.6

3.5.2 Back analysis using Limit Equilibrium (LE)

LE analysis was also carried out using Slide (Rocscience, 2012). The LE method is very sensitive to the search method for finding the most critical slip surface. Accordingly, different search methods were investigated and the most critical slip surface was chosen. Non circular slip surfaces were also considered.

LE analysis results are presented in Figure 12. There is relatively a good agreement between the results of FE-SSR and LE methods. Figure 13 presents the superposition of failure surfaces predicted by both methods. The red failure surface belongs to the LE method, and the green one belongs to the FE-SSR method.
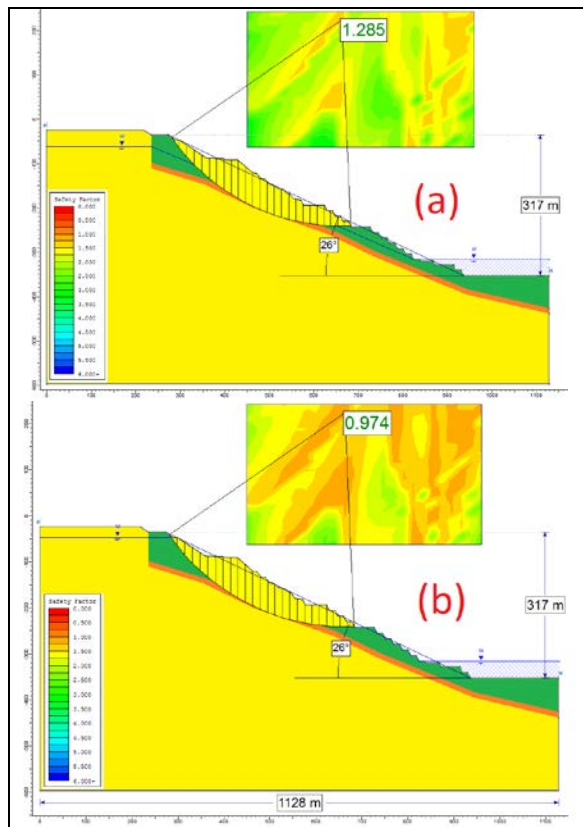

Figure 12. Results for back-analysis using LE (Bishop Simplified method) for section S3M1: (a) average water table (FoS = 1.285) and (b) Fully-saturated conditions (FoS = 0.974)

For the average water table level, LE resulted in FoS of 1.285 and 1.243 for the Bishop simplified and Janbu simplified methods, respectively. The FoS obtained from FE-SSR was 1.2.

In fully-saturated conditions, LEM resulted in FoS of 0.974 and 0.933 for the Bishop simplified and Janbu simplified methods, respectively. The FoS obtained from FE-SSR was 0.86.

It was observed that the failure surfaces predicted by both LE methods are identical, but the Janbu simplified method presents a lower FoS that compares slightly better with FE-SSR FoS.

3.5.3 Validation

Rock mass properties derived from back-analysis of section S3M1 were validated on section S6C. Using the FE-SSR method, a FoS of 1 was obtained for the S6C section in fully-saturated conditions (Figure 14). LE analysis resulted in a FoS of 1.107 and 1.019 for the Bishop simplified and Janbu simplified methods respectively. The results were deemed satisfactory for both sections based on LiDAR field observations.
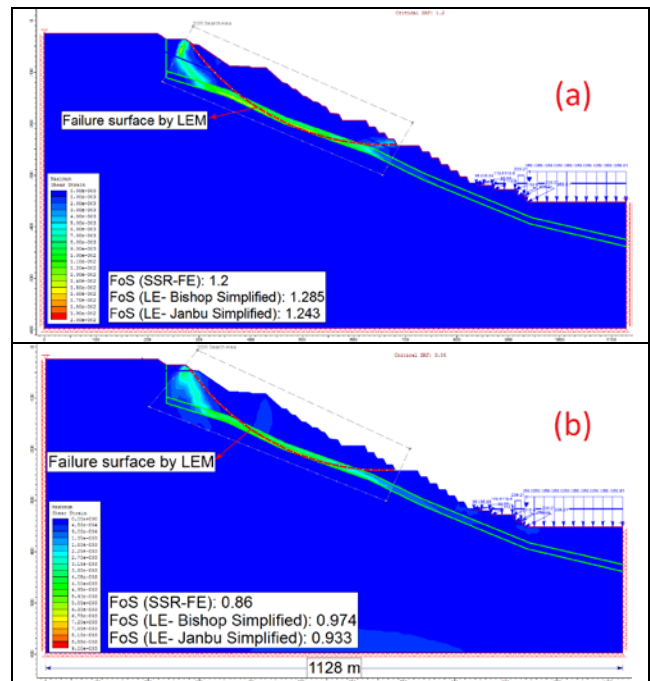

Figure 13. Superposition of failure surfaces predicted by both LE and FE-SSR methods for S3M1 section: (a) average water table (FoS$_{Slide}$=1.285 and 1.243 / FoS$_{Phase2}$= 1.2) and (b) Fully-saturated conditions (FoS$_{Slide}$=0.974 and 0.933 / FoS$_{Phase2}$= 0.86)
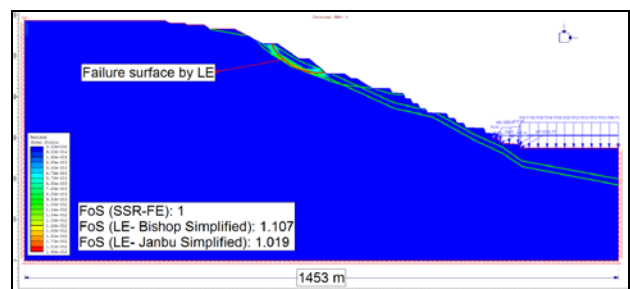

Figure 14. Validation of derived rock mass properties from section S3M1 section on section S6C section (using fully-saturated conditions)

3.6   Forward Analysis

This section presents a forward analysis for the slope stability in the vicinity of the new road layout. The analysis results will be compared with some criterions presented in sections 2.1 and 2.2.

Slope profile S6C obtained from airborne LiDAR surveying (Figure 15) in 2012 -two years after failure- was used in this analysis.
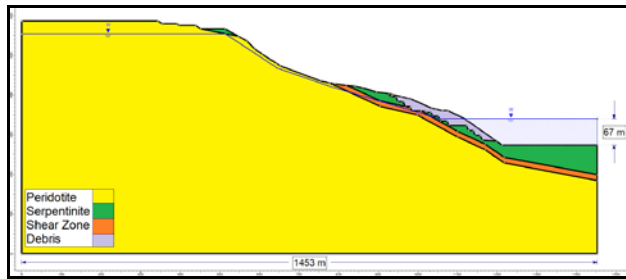

Figure 15. S6C profile for forward-analysis

Rock mass properties for different lithologies were obtained using back-analysis for two sections (S3M1 and S6C). In this section, forward-analysis is conducted on profile S6C using the derived rock mass properties. Forward-analyses were carried out under the most conservative hydrogeological condition, namely fully-saturated condition. The presence of the debris was neglected in the analysis since it only marginally affect the stability of the global slope.

3.6.1   Geometrical analysis

The proposed geometry was compared to the guidelines used in Western Australia and discussed in section 2. Considering that the rock mass is defined as weak rock mass, a slope with an angle of 25° should be applied. This is obviously the case in the proposed design.

3.6.2   Analysis based on FoS and PoF

The results of the numerical FE-SSR analysis are presented in Figure 16. As observed, the FoS in the upper half part of the slope is high (FoS = 4.68) while the lower FoS (1.72) is associated to the lower half of the slope.

The high factor of safety (FoS) obtained by numerical modeling is validated by airborne LiDAR survey results. As seen in Figure 17, there are no significant instabilities in the West wall between 2012 and 2013. The obtained factor of safety for the global slope, are above the values proposed in section 2; the integrated slope and road design is thus acceptable based on these criteria.

Probabilistic analysis was then carried out for section S6C using LE analysis. The Monte Carlo sampling method was used and triangular distributions were selected as a starting point for probability density functions for all parameters. Table 4 summarizes the input parameters for the probabilistic analysis. These

parameters were established based on the back-analyzed properties as well as the possible range for this type of material, based on the available technical literature. The minimum and maximum values are specified as relative values (i.e. distance from the mean value), rather than as absolute values.
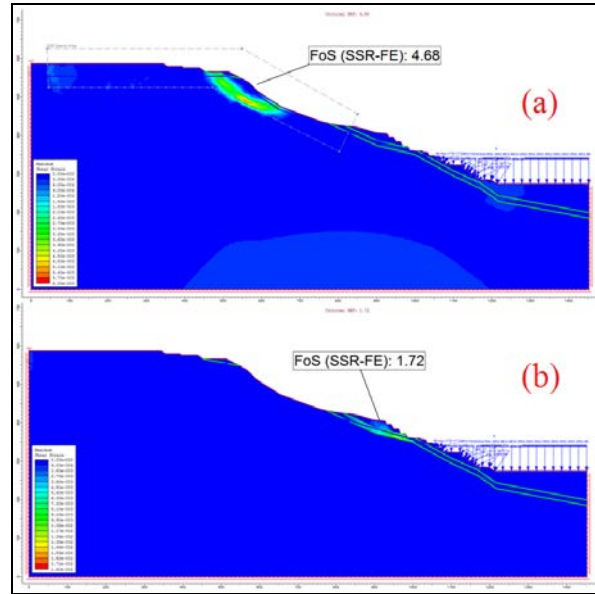

Figure 16. Results for forward-analysis using FE-SSR on S6C section: (a) FoS=4.68 in upper part and (b) FoS=1.72 in the middle part
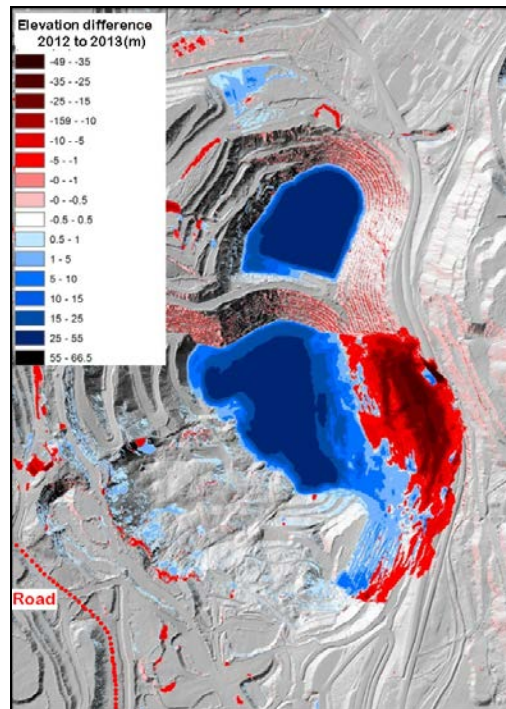

Figure 17. Displacements between 2012 and 2013

Table 4. Input parameters for probabilistic analysis

Material	Property	Mean value	Relative Min	Relative Max
Peridotite	Cohesion (KPa)	911	150	150
	Phi (º)	57.8	10	10
Serpentinite	Cohesion (KPa)	226	100	100
	Phi (º)	32.4	10	10
Shear Zone	Cohesion (KPa)	113	70	70
	Phi (º)	22.6	5	5

The result of probabilistic analysis for section S6C is presented in Figure 18. The obtained PoF of zero satisfies the most conservative criterion, i.e., a PoF of 0.3% suggested by the Department of Minerals and Energy of Western Australia (1999). The mean FoS of 4.66 and the deterministic FoS of 4.62 are determined for section S6C in this analysis. The Reliability Index (RI) is an indication of the number of standard deviations (using normal or lognormal distributions) which separate the mean FoS from the critical one (FoS=1). Considering the FoS and PoF obtained, it can be concluded that the proposed highway layout respects all suggested guide-lines.
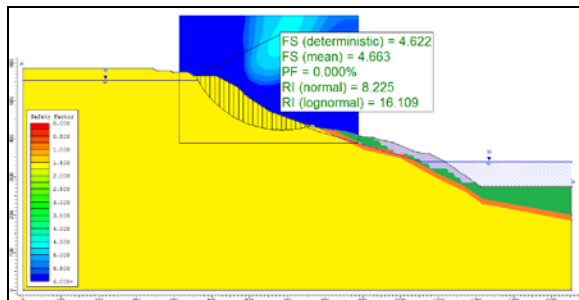

Figure 18. Probabilistic analysis of S6C profile

4 GEOLOGICAL UNCERTAINTY

Due to a certain level of uncertainty concerning the extent of the shear zone and schistose serpentinite beyond the slope crest, two other scenarios were investigated. The first one (Figure 19) used a horizontal extension of the shear zone; the second one (Figure 20) was modelled assuming that the shear zone extends to the ground surface. Fully saturated conditions were also considered.

The results are presented in Figures 19 and 20. A FoS of 1.31 and a PoF of 0.4% were obtained for LE analysis of the horizontal extension. A FoS of 1.32 and a PoF of 0.3% were obtained with an LE analysis for the ground surface extension. Both results clearly indicate that the extension of the shear zone may have an important impact on the stability of the slope; but again, slope failure is limited to a relatively small portion of the entire slope.
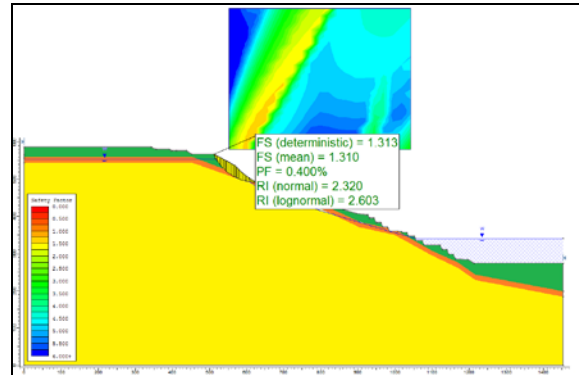

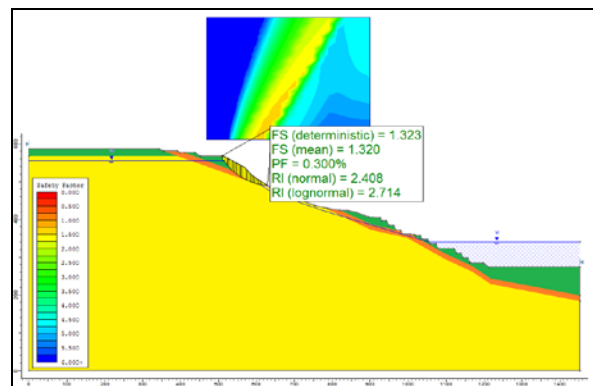Figure 19. Horizontal extension of the shear zone for the S6C profile

Figure 20. Ground surface extension of the shear zone for the S6C profile

Nevertheless, it was proposed that further field investigation be conducted in order to better define the geometry of the shear zone using geotechnical drilling. It was also suggested that two inclinometers and piezometers be installed to survey the slope movement at the crest of the pit in the vicinity of the projected highway. These inclinometers and piezometers could also provide pertinent quantitative information to calibrate a numerical model to perform strain analysis such as the one presented in section 2.3.

5 CONCLUSIONS

In this paper, a brief literature review on the design of public roads in the vicinity of open-pit mines was presented. The guidelines provided in the literature then were used to investigate the design of a new highway in Québec, Canada.

In this case study, rock mass properties at the slope scale, were derived using back-analysis of a recent slope failure (2010). The numerical modelling was verified using airborne LiDAR survey data.

A forward-analysis was then used to evaluate the stability of the actual mining slope in the vicinity of the proposed highway. Both the FE-SSR and LE methods

were used to derive FoS and PoF for the investigated slopes. The proposed design met all studied guidelines.

ACKNOWLEDGEMENTS

REFERENCES

Beauchamp, Y. 1994. *Black Lake Operations*: presentation of the slope stability program on Nov. 10th 1994. McGill University.

Caudal, P., Grenon, M., Locat, J, Turmel D. 2014. Caractérisation et surveillance d'un glissement rocheux actif d'une mine à ciel ouvert en utilisant le LiDAR. Proceedings of GeoHazard6, June 15-18, 2014, Kingston, Canada.

Caudal, P., Amoushahi, S., and Grenon, M. 2013. *Back analysis of the west wall slope failure at LAB Chrysotile mine*, 23rd WMC, Montreal, Canada.

Dawson, E.M., Roth, W.H., and Drescher A. 1999. *Slope stability analysis by strength reduction*. Geotechnique, vol 49, no. 6, pp. 835-840.

Department of Industry and Resources. 1997. *Guideline of safety bund walls around abandoned open pit mines*, Australia.

Department of Minerals and Energy of Western Australia. 1999. *Guideline of geotechnical considerations in open pit mines*, Australia.

Diederichs, M.S., Lato, M., Hammah, R., and Quinn, P. 2007. *Shear strength reduction approach for slope stability analyses*. Proceedings of the 1st Canada-US Rock Mech. Symposium. Vancouver, Canada.

Hammah, R.E., Curran, J.H., Yacoub, T.E., & Corkum, B. 2004. *Stability analysis of rock slopes using the Finite Element Method*. In Proceedings of the ISRM Regional Symposium EUROCK 2004 and the 53rd Geomechanics Colloquy, Salzburg, Austria.

Hammah, R.E., Yacoub, T.E., Corkum, B. & Curran, J.H. 2005. *A comparison of finite element slope stability analysis with conventional limit-equilibrium investigation*. In: Proceedings of the 58th Canadian geotechnical and 6th joint IAH-CNC and CGS groundwater specialty conferences, Sask. Can.

Hoek, E. and Karzulovic, 2000. *Rock properties for surface mines*. Published in Slope Stability in Surface Mining (Edited by W.A Hustrulid, M.K. McCarter and D.J.A van Zyl), Pages 59-70, Littleton, Colorado.

Hoek, E. 2007. *Practical Rock Engineering*. Rocscience, Canada.

Hoek, E. 2009. *Fundamentals of slope design*. Keynote address at Slope Stability 2009, Santiago, Chile.

Juncal, A.S & Ivars, D.M. 2012. *Assessment of Mining Influence on the E4 Highway, Norra Karr Deposit*. Sweden: Itasca Consultants, Sweden.

Read, J. and Stacey, P.F. 2009. *Guidelines for open pit design*, CSIRO Publishing, Melbourne, Australia.

Rocscience, 2004. *A new era in slope stability analysis: shear strength reduction finite element technique*. RocNews.http://www.rocscience.com/library/rocnews/summer2004/StrengthReduction.pdf

Rocscience. Inc. 2013. Phase2. V.8.0- A Finite Element Package.

Rocscience. Inc. 2013. Slide. V.6.0- A Limit Equilibrium Package.

Sjoberg, J. 2013. *Numerical analysis, slope design and in situ stress*. International Symposium on Slope Stability in Open Pit Mining and Civil Engineering, Australian Centre for Geomechanics, Perth, ISBN 978-0-9870937-5-2, Brisbane, Australia.

Stockel, B.M., Sjoberg, J., Makitaavola, K., and Savilahti, T. 2012. *Mining-induced ground deformations in Kiruna and Malmerget*. The Eurock 2012 conference-Rock Engineering & Technology for sustainable Underground Construction, Stockholm, Sweden.

Wyllie, D. and Mah, C.W. 2004. *Rock slope engineering*, Civil and Mining, 4nd ed, New York: Spon Press.