Reliability Analysis of Rock Supports in Underground Mine Drifts: A Case Study

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Reliability analysis of rock supports in underground mine drifts: a case study

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Abstract

Support failures in mine drifts represent potential hazards threatening underground mine safety and productivity. The aim of this study is to determine the reliability index associated with the rock supporting elements used in Ridder-Sokolny mine, an underground mine located in East Kazakhstan. Numerical simulations of the drift support and the first order reliability method (FORM) were employed to carry out the analysis. Several support cases were considered including; shotcrete, bolting, concrete, and combined bolting and concrete as well as unsupported drift case. For each support case, the factors of safety (FS), the reliability index (β) and the probability of failure (PF) were determined in accordance with the corresponding rock mass quality and excavation geometry. The results indicated the average FSs varied little for the different support cases (except for shotcrete); while β and PF vary more significantly between 0.62–3.25 and 0.05–27 (×10³ %) factor depending on the rock conditions and support installed. The probability of failure of the rock support increases with a decrease in the rock mass quality. Similar trends were observed with an increase of the width/height ratio of the excavations for the same rock domain. These results illustrated that a single FS value obtained from a deterministic method may not always provide a sufficient indication of safety. This is in agreement with the field observations (many of the supports failed). Hence, on the basis of the reliability index of the supports, the requirement in terms of coefficient of variability of the rock mass quality to meet the target performance level was
proposed. It is concluded that the results of this study could help improving the drift support design in Ridder-Sokolny mine.

**Key words:** First order reliability method, reliability index, rock support, mine drift, factor of safety, numerical modelling.
1. Introduction

In underground mining, drifts are referred to as any type of horizontal or sub-horizontal
development excavations with variable size, shape, and length depending on their purpose
(exploration, ventilation or haulage). Typically a mine may require drift from few hundred
even up to thousands of kilometers. For example, in one of the largest underground mines in
the world, El Teniente copper mine located in Chile, approximately more than 3500 km of
drift have been constructed to be able to extract copper ore (Elgenklöw 2003). With the
deployment of shallow deposit around the world, mines have to construct drift much deeper
with difficult geological settings, and support them. It is clear that the support of these drifts
is important. Usually, several types of rock support and reinforcement may be required for
ground control in the same drift. The main function of any drift support system in
underground mines is to keep mine workers safe and equipment in the event of fall of ground.

Notwithstanding, technological advances in innovative ground stabilization and falls of
ground, still continue to be one of the major causes of accidents, injuries and fatalities in
underground mines around the world (Adoko et al. 2017; Grenon and Hadjigeorgiou 2003).
Consequently, extensive research works were accomplished in connection with designing
rock support in excavations. As a result, several innovative supporting devices and methods
of design of rock supports and have been proposed over the past few years and include
numerical modeling (Boon et al. 2015; Hu et al. 2019; Nie et al. 2018), empirical charts
(Barton et al. 1974; Chan and Einstein 1981; Rehman et al. 2019), analytical methods
including the key block theory (Fu and Ma 2014; Zhang et al. 2020; Zou and Zhang 2019);
probabilistic methods (Lü et al. 2012; Oreste 2005); expert systems (Madhu et al. 1995); lab
and in-situ testing and monitoring (Bjureland et al. 2019; Li 2006; Napa-García et al. 2017;
Wu et al. 2019; Yokota et al. 2020).
In most of these studies, the support design is treated as a deterministic problem where load demand and the capacity of the support is not considered as random, rather it is determined using theories of stability analysis and limit equilibrium methods. Hence, in each of these methods a well-founded understanding of the mechanical response of the rock support to the surround rock mass under each type of load is necessary to be able to determine the factor of safety. Nevertheless, the inherent variability of the rock mass characteristics combined with the difficulties of repeatable laboratory and field testing as well as the uncertainties linked to the ground affect the whole process, which eventually may compromise the reliability of these designs. In practice, to deal with the uncertainty of the design parameter, conservative ground and material parameters are often selected based on the subjective judgment and engineering experience of the designer. This leads to selecting a support system designed to sustain the worst ground loading conditions which is justified by the fact that at least an overly conservative design would guarantee an acceptable factor of safety. Engineering should place emphasis on selecting the optimal design in terms of safety and cost, with a good understanding and modelling of the supported rock mass, instead of a mere simplistic and deterministic approach (Grenon and Hadjigeorgiou 2003).

A way of dealing objectively with uncertainties in rock support design is a reliability analysis of the rock support (Langford and Diederichs 2015). The reliability-based design (RBD) approach provides the estimation of the probability of the failure probability of engineering structures. The RBD has proven to be very a useful tool to appropriately model the factor of safety under uncertain conditions of the optimization problem. This ensures the required minimum safety for a cost-effective design of the structure. As such, the use of RBD represents a more rational approach to design rock support for mine drifts. In particular for
mine drifts, by considering the natural variability present within the rock mass through rock mass characterization tools such as the rock mass rating (RMR) or the rock mass quality (Q); and quantifying the uncertainty for the different types of support along the drift, the probability of failure of the rock support associated with each geotechnical domain (representing various limit states) can be determined. Examples of recent RBD applications include rock supports (Do et al. 2020; Fang et al. 2019; Liu and Low 2017; Lü et al. 2017); underground excavations (Gholampour and Johari 2019; Goh and Zhang 2012; Song et al. 2016; Yang et al. 2018); rock strength estimation (Bozorgzadeh et al. 2018); mine pillar design (Deng et al. 2003; Song and Yang 2018); rock failure modes in tunnels (Lü et al. 2013); and rock slope design (Dadashzadeh et al. 2017; Jimenez-Rodriguez and Sitar 2007).

While reliability analysis is quite popular for tunnel support in general, there have been limited applications of reliability analysis for mine drift support. This could be due to the lifespan of these excavations from a few weeks to a few years depending on their use. On the other hand, the need for a reliable and optimal design of mine drift support has been demonstrated (Potvin 2017; Yakubov and Adoko 2020). However, the application of the RBD concepts to drift support reliability presents unique challenges given the complexity of the interaction support-surrounding rock mass, the mechanism of failures, and the limited information about the uncertainties associated with the surrounding rock mass. Following this line of thinking, in this study, the First Order Reliability Method (FORM) and numerical modelling are implemented to investigate the probability of failure of drift support systems for various geotechnical domains. The drift support systems used in Ridder-Sokolny mine located in Kazakhstan, are used as case studies to evaluate their reliability. Field observations showed that even though the mine has approved guidelines for rock support installation and
maintenance, issues with failures of rock support are frequently encountered (see Fig.1). This justifies a reliability analysis of the rock support.

Fig.1 Examples of collapsed supports showing damaged mesh and rock bolts

2. Methods

2.1. Ridder-Sokolny Mine and geomechanical data description

The Ridder-Sokolny mine is located in the mountainous area of East Kazakhstan, approximately 3 km from the Ridder city. It is owned by Kazzinc Corporation, and with more than 200 years of history, currently produces 1.6 million tons of ore per year with an average gold grade of 2.0 g/ton (Kazzinc 2019). The main products after processing include gold, copper, zinc and lead concentrates. Several mining methods are utilized at Ridder-Sokolny mine depending on ore body morphology and thickness such as; sublevel caving, cut-and-fill stoping, sublevel stoping with partial shrinkage applied, shrinkage stoping, upward horizontal slicing with backfilling, and sublevel caving.

The Ridder-Sokolny polymetallic deposit pertains to the Leninogorsk ore field within the W–E striking regional Semipalatinsk–Leninogorsk fault with a geological structure described as weakly deformed layered strata of the arch of anticlinal structure (Zinoviev 2016). The bulk
of industrial mineralization is concentrated in the rocks of the Krukovka formation. In general, the ore deposits are covered by a layer of lavas, tuffs of andesite-basalt composition, tuffaceous conglomerates and sandstones (100 m thick), above which a bundle of argillite and aleurolite with the lenticular bodies of rhyolite extrusive facies with a thickness of more than 400m is deposited. In particular, the Ridder-Sokolny deposit consists of a series of ore deposits. The vast majority of ore bodies are of small thickness and length, both along strike and dip. The rock mass of the investigated excavations consists essentially of six geotechnical domains with variable mechanical properties from weak to competent ground. The strongest rocks consist of microquartzites, agglomerate tuffites, and the weak ones are sericite-chlorite-quartz rocks and schists. A significant variability of the rock strength is noted even within the same type of rock domain. Tables 1 and 2 provide the geo-mechanical properties of the most representative rock masses. As far as the geological structures are concerned, four main sets of joints occur within the ore deposit areas. The first three sets of joints are almost mutually perpendicular to each other in such a way they create cubic block structure of rocks. The first set is almost horizontal and with inclined bedding joints in volcanic-sedimentary rocks composing the massif. The remaining sets are steeply dipping, almost vertical joints, oriented across the strike of steeply falling ore bodies of the lower level of mineralization. The tensile strength, cohesion and the angle of friction of joints are estimated to be considered as 0.5 MPa, 18.8 MPa and 35°, respectively.

| Rock domains | UCS (MPa) | Material constant (mi) | Young mod. Ei (GPa) | Poisson ratio, ν |
|--------------|-----------|------------------------|---------------------|-----------------|
| Lithology    | Main rock types | | | |
| Nº1          | Tuffite and felsite porphyry | 135 | 20 | 69 | 0.19 |
| Nº2          | Microquartzite and sercite | 158 | 20 | 73 | 0.2 |
Table 2 Properties of the rock domains (rock mass)

| Lithology       | Q   | RQD | GSI | Density (t/m³) | σₖ (MPa) | σₖc (MPa) | Eₘ (GPa) | φ(°) | C (MPa) |
|-----------------|-----|-----|-----|----------------|----------|-----------|----------|------|--------|
| Nº1             | 160 | 74  | 75  | 2.74           | 1        | 56.8      | 56.3     | 44   | 11.9   |
| Nº2             | 20  | 58  | 60  | 2.72           | 0.4      | 48.3      | 39.6     | 40   | 11.3   |
| Nº3             | 8   | 61  | 55  | 2.71           | 0.5      | 18.8      | 28.2     | 29   | 5.6    |
| Nº4             | 2.66| 34  | 45  | 2.72           | 0.2      | 7.9       | 19       | 27   | 2.4    |

The rock supports used in Ridder-Sokolny mine include: shotcrete, rock bolting, combined (concrete rock bolting, shotcrete and metallic mesh), concrete and arch metallic lining. The design of the rock supports was accomplished based on the excavation dimensions, purpose, and the rock mass characteristics. The bolt parameters and the mechanical properties of the rock support elements are provided in Tables 3-4. The Q-system for rock classification was used to classify the surrounding rock mass in terms of the ground stability, namely as very stable (Q> 40); stable (10 <Q<40); moderately stable (4<Q<10) and unstable (Q<4). In very stable ground no support is required; shotcrete of 3-6 cm thickness in accordance with the rock mass strength is applied within the rock mass pertaining to stable ground. Bolting (1.6-2m length) is applied for stable and moderately stable ground. In unstable ground bolting is employed in combination with other support elements. Meanwhile, concrete (20-30 cm of thickness depending of the rock mass strength) is applied within moderately stable and unstable grounds; similar to metallic arch (with a minimum bearing capacity of 300kN/arc).
Table 3 Bolting parameters

| Rock domain | Excavation dimensions (m) | Bolting parameters |
|-------------|---------------------------|--------------------|
|             | Width | Height  | Bolt length, (m) | Bolt density, (bolt/m²) | Bolt spacing (m x m) |
| Nº2-Nº3     | 2.15 – 2.85 | 2.55 – 2.95 | 1.6  | 1.00  | 1.0 x 1.0 |
|             | 2.90 – 3.25 | 2.80 – 3.00 | 1.8  | 1.00  | 1.0 x 1.0 |
|             | 3.30 – 3.75 | 2.70 – 3.10 | 1.8  | 1.24  | 0.9 x 0.9 |
|             | 3.80 – 4.10 | 2.70 – 3.10 | 1.8  | 1.39  | 0.85 x 0.85 |
|             | 4.15 – 4.30 | 2.80 – 3.50 | 1.8  | 1.56  | 0.8 x 0.8 |
| Nº4         | 2.90 – 3.15 | 2.95 – 3.05 | 1.8  | 1.56  | 0.8 x 0.8 |
|             | 3.20 – 3.40 | 2.85 – 3.15 | 1.8  | 1.56  | 0.8 x 0.8 |
|             | 3.50 – 4.10 | 2.95 – 3.35 | 2.0  | 1.78  | 0.75 x 0.75 |

Table 4 Mechanical properties of the rock supporting elements

| Rock bolts (swellex) | Metallic Arch | Mesh |
|----------------------|---------------|------|
| Tributary Area, mm²  | 232.5         |      |
| Bolt Modulus, E, Mpa | 200000        |      |
| Spacing, m           | 0.6           | Spacing, m |
| Section depth, m     | 0.254         | Section Depth, m |
| Bond Shear Stiffness, MN/m/m | 100 |      |
| Area, m²             | 0.00285       | Area, m² |
| Moment of Inertia, m⁴ | 2.87E-05     | Moment of Inertia, m⁴ |
| Concrete             |               |      |
| Young's Modulus, MPa | 200000        | Young's Modulus, MPa |
| Poisson's Ratio      | 0.25          | Poisson's Ratio |
| Comp. Strength, MPa  | 400           | Comp. Strength, MPa |
| Thickness, m         | 0.2           | Tensile Strength, MPa |
| Tensile Strength, MPa| 400           |      |

2.2. The first order reliability method (FORM)

The FORM makes use of a probabilistic approach to evaluate the reliability of a system. Conventional deterministic evaluation of rock support stability for underground excavations requires the use of a factor of safety (FS) which defines the margin between the resistant load R and the acting load (stress) S. The boundary dividing the safe and failure domains, is the limit state surface (boundary) represented by a performance function (the acceptability
criterion of the design) commonly defined as: \( G(x) = R - S \) where \( x \) indicates the vector of the random variables. Situations when \( G(x) > 0 \) would correspond to safe design of the rock support while \( G(x) < 0 \) unsafe design i.e. failed support. Most of the cases, \( G(x) \) is not known always explicitly, and therefore its approximate determination can be implemented in an implicit manner by the mean of numerical procedures such as the finite element method (Goh and Zhang 2012).

Formally, in a reliability analysis, the failure probability of the rock support \( P_f \), can be determined via a multidimensional probability integral defined as follows (Ang and Tang 1975):

\[
P_F \equiv P(G(x) \leq 0) = \int_{G(x)\leq0} p_x(x)dx
\]

(1)

Where \( G(x) \) the performance function defined on a space of \( x \) random variables; and \( p_x(x) \) is the joint probability density function (PDF). Because, \( p_x(x) \) is unknown and \( G(x) \) non-linear and complex most of the time, the explicit solution for Eq.(1) can be very difficult. Therefore, methods to approximate its solution have been proposed. In this study, the algorithm proposed by Low and Tang (2007) is implemented and briefly explained below.

The reliability index \( \beta \) also known as the Hasofer-Lind index representing the state of the system in terms of failure is defined as:

\[
\beta = \min_{x \in F} \sqrt{(x - \mu)^T C^{-1} (x - \mu)}
\]

(2)

In Eq.(2) the parameters \( x, \mu \) and \( C \) denote the random variable vector, the vector of mean values of random variables, and is the covariance matrix, respectively. Meanwhile, the
domain $F$ in Eq.(2) represents the failure domain where the performance function $G(x)$ is less or equal to zero. An equivalent form of Eq.(2) can be obtained via adequate interpretation of the Hasofer-Lind index through an expansion of the ellipsoid concept in the original space of the random variables (See Fig.2). The reliability index, $\beta$ is then expressed as follows:

$$\beta = \min_{x \in F} \sqrt{\left(\frac{x - \mu}{\sigma_k}\right)^T (R)^{-1} \left(\frac{x_k - \mu_k}{\sigma_k}\right)}$$

(3)

Where $R$ stand for the correlation matrix, $\mu_k$ the mean of random variable $x_k$ and $\sigma_k$ and the standard deviation of random variable $x_k$.

Fig.2 Illustration of the reliability index in the ellipsoid perspective (Low and Tang 2007)

The design point i.e. most probable point where failure will occur, is the one expressed by the $x_k$ values, which are obtained by minimizing the square root of the quadratic form represented in Eq.(3). The expansion of a multivariate normal dispersion ellipsoid away from the mean-value point implies that the contours of probability values decrease in accordance with the probability density function of the multivariate normal distribution as follows:
In case of correlated non-normal variables, using the ellipsoid perspective shown in Fig.2 with an appropriate constrained optimization, another version of Eq.(3) can be expressed as follows:

\[
\beta = \min_{\mu, \sigma} \sqrt{\frac{(x_k - \mu_k^N)^T (R)^{-1} (x_k - \mu_k^N)}{(\sigma_k^N)^2}}
\]  

(5)

Where \( \mu_k^N \) and \( \sigma_k^N \) are the equivalent normal mean and standard deviation of random variable \( x_k \), respectively. The values of the equivalent normal mean and standard deviation can be determined by the condition that the cumulative probability and the probability density ordinates of the equivalent normal distribution are equal to those of the corresponding non-normal distribution at \( x \). Therefore, \( \mu_k^N \) and \( \sigma_k^N \) can be obtained as follows:

\[
\sigma^N = \frac{\phi \left[ \Phi^{-1}F(x) \right]}{f(x)}
\]

(6)

\[
\mu^N = x - \sigma^N \Phi^{-1}F(x)
\]

(7)

In Eqs.(6-7), \( x \) stands for the original non-normal variate, \( \Phi^{-1} \) is the inverse cumulative density probability (CDF) of a standard normal distribution, \( F(x) \) represents the original non-normal CDF, \( \phi \) denotes the probability density function (PDF) of the standard normal distribution, and \( f(x) \) represents the original ordinate of the non-normal probability density.
It should be noted that for normal variate, the reliability index $\beta$ can be represented as the axis ratio $(R/r)$ of the ellipse touching with the limit state surface (β-ellipse) and the 1standard-deviation dispersion ellipse (1-σ) as in Fig.2. Based on that, and using geometrical properties of ellipse, Low and Tang (2007) developed an alternate FORM algorithm in excel spreadsheet via varying basic random variable $x$ and Eq.(5) can be rewritten as:

$$\beta = \min_{\mathbf{n} \in \mathbb{F}} \sqrt{\mathbf{n}^T \mathbf{R} \mathbf{n}}$$

(8)

$\mathbf{n}$ representing a column vector of $n_k$ that is expressed as:

$$n_k = \frac{x_k - \mu_k^N}{\sigma_k^N} = \Phi^{-1}\left(F\left(x_k\right)\right)$$

(9)

By considering the variation of $n$ over a specific interval through a constrained optimization algorithm, $x_k$ can be determined as:

$$x_k = F^{-1}\left(\Phi(n_k)\right)$$

(10)

Eqs.(9-11) allows to determine the reliability index, $\beta$. After it has been calculated the failure probability, $P_F$, can be obtained as follows:

$$P_F \approx 1 - \Phi(\beta)$$

(11)

More details about the algorithm itself can be found in (Low and Tang 2007)

3. Results

3.1. Numerical modelling

The purpose of the simulation is to determine the factor of safety for each type of support. The well-known RS2 software was used for the simulation. Although the simulated problem
is 3D by nature, 2D simulation was judged adequate enough for an estimate on the factor of safety (FS) of the excavation. A series of simulations corresponding to four rock support cases (in accordance to the rock mass domain) and different excavation geometries were carried out. Plastic model was used and the excavation depth was 700m. The main input parameters were specified according to Tables 1-4. The excavation widths were 2.8, 3.2, 3.6, 4.0, 4.4, 5.0, 6.0 and 7.0 while the heights were 3, 3.4 and 3.8. The safety factor (the ratio of the capacity to the demand) was defined in two ways. For excavation with no support and with shotcrete (domain 1 and 2), the rock strength divided by differential stress is used. For the remaining cases, the actual load on the element and its yielding capacity were considered instead (obtained from the simulations). The simulated cases were: excavation with no support; bolting; concrete; and the combination of bolting and concrete. Some results of the simulation for drift with width = 2.8 m and height = 3.8 m, are provided in Fig.3 (a-d). It should be noted that steel arc supports were also simulated as reinforced concrete but the results were quite close to those of bolting combined with concrete; therefore the results of steel arc supports are not provided in the study.

Fig.3(a) shows the differential stress contours around an unsupported excavation (W=2.8 m and H= 3.8 m). The Mohr-Coulomb failure criterion was used. There was no yielding of rock in shear or tension around the excavation. According to the stress damage criterion, cracks would initiate when the differential stress reaches 0.3 times the compressive stress. The contour of that stress (18.9 MPa) is shown. This indicates that stress damage is likely to take place in the roof and in the corner of the floor. Besides, the strength factor contours were greater than 1. In Fig.3(b), the simulation results of shotcrete reinforcement of 5cm thickness are provided. As can be seen, the rock yielded in tension and shear along the joints (bottom
left corner and top left). The contour of crack initiation stress 16.1 MPa (0.3 times of the compressive strength) is much larger compared with the previous case (unsupported excavation). Axial force contours (hatched area of the figure) cover adequately the stress damage zone. The axial forces vary between a maximum of 1.77MN (on the top left) and a minimum of -0.05MN (on the side) while the shear forces range between 0.16 MN and -0.19 MN on the bottom left corner); the mobilized forces were strong enough to provide support to the excavation. The simulation showed that the shotcrete elements and the joint sets did not yield, suggesting that the thickness of the shotcrete was adequate.

The rock bolting simulation results are shown in Fig.3(c). The bolt tensile capacity was 0.1 MN; the damage zone almost similar as in the shotcrete case; the bolts provided enough support capacity to sustain the damage stress (15.9 MPa). The maximum mobilized forces and stresses of the bolt were 0.02 MN; and 98.5 MPa (compression) and 23.5 MPa (tension), respectively. Meanwhile, in the last case (Fig.3d) the simulation result of the combined support concrete and rock bolting corresponding to unstable ground is provided. This case showed a yielding area about the size of excavation (both shear and tension); the differential stress was lower (about 10 MPa 0.5 m offset distance outside the excavation compared to 16 MPa for the unsupported excavation) which was expected as the mobilized strength of the rock mass was lower. The maximum stress around the excavation was estimated at 28 MPa while the yielding capacity of the concrete structure was 40 MPa. Loads of 2.89-5.81 MN axial forces and negative 3.6 to 3.73MN of shear forces were shown on the concrete support; and 0.06 MN (axial load) on the bolts which is far less than the minimum yielding load of Swellex rock bolt (90KN). In addition, the simulation showed that the shotcrete elements and the rock bolts did not yield, suggesting that the support system was adequate.
Through the simulation of each case as mentioned previously, 90 data points were obtained. A sample of the dataset is shown in Table 5. Next, these data were used to carry out regression fitting to determine the empirical equations of FS relating Q, W, and H. The curve fitting tool of MATLAB software was used to fit the compiled data and the coefficients were determined as well as the goodness of the fit. The fitting results provided in Table 6 indicate high coefficients of determination ($R^2 > 0.87$) which shows very good correlation. These correlations also indicate that an increase of Q increases FS; an increase W/H decreases FS which is in agreement with practice. Hence, these expressions are used to compute the reliability index in section 3.2. In Table 6, in calculating the FS, the global strength and the maximum differential stress on the excavations obtained from the simulations were used.

Fig. 3(a) Unsupported drift
Fig. 3(b) Supported drift with shotcrete

Fig. 3(c) Supported drift with rock bolt
Table 5 FS values for rock domain Nº1-4

| Rock domain | Factor of safety | Fitting performance |
|-------------|------------------|---------------------|
| Nº1         | $FS_1 = 0.16Q^{0.42} + (W/H)^{-1.03}$ | R2=0.97, RMSE=0.12 |
| Nº2         | $FS_2 = 0.13Q^{0.5} + (W/H)^{-0.6}$  | R2=0.96, RMSE=0.04 |
| Nº3         | $FS_3 = 0.28Q^{0.89} + (W/H)^{-1.27}$ | R2=0.87, RMSE=0.18 |
| Nº4         | $FS_4 = 1.26Q^{0.42} + (W/H)^{-1.05}$ | R2=0.94, RMSE=0.15 |

Table 6 Factor of safety data sample

| Q  | W (m) | H (m) | Rock strength (MPa) | Differential Stress (MPa) | Factor of safety (FS) |
|----|-------|-------|---------------------|--------------------------|-----------------------|
| 453| 4.00  | 3.80  | 100.38              | 33.22                    | 3.02                  |
| 453| 3.60  | 3.00  | 100.38              | 34.10                    | 2.94                  |
| 453| 5.00  | 3.80  | 100.38              | 37.50                    | 2.68                  |
| 160| 5.00  | 3.80  | 80.00               | 37.50                    | 2.13                  |
| 160| 6.00  | 3.80  | 80.00               | 39.00                    | 2.05                  |
| Q    | W    | H    | FS  | R   | M   |
|------|------|------|-----|-----|-----|
| 160  | 7.00 | 3.00 | 80.00 | 44.50 | 1.80 |
| 26.66| 7.00 | 3.80 | 55.90 | 40.00 | 1.40 |
| 26.66| 6.00 | 3.00 | 55.90 | 42.00 | 1.33 |
| 16.00| 2.80 | 3.80 | 50.35 | 28.50 | 1.77 |
| 16.00| 3.20 | 3.80 | 50.35 | 32.00 | 1.57 |
| 16.00| 4.00 | 3.80 | 50.35 | 35.22 | 1.43 |
| 5.33 | 7.00 | 3.80 | 90.00 | 48.00 | 1.88 |
| 5.33 | 6.00 | 3.00 | 90.00 | 49.50 | 1.82 |
| 5.33 | 7.00 | 3.00 | 90.00 | 51.70 | 1.74 |
| 2.66 | 2.80 | 3.80 | 40.00 | 11.00 | 3.64 |
| 2.66 | 3.20 | 3.80 | 40.00 | 12.00 | 3.33 |
| 2.66 | 3.60 | 3.00 | 40.00 | 14.20 | 2.82 |
| 2.66 | 5.00 | 3.80 | 40.00 | 15.80 | 2.53 |
| 0.46 | 5.00 | 3.80 | 40.00 | 24.50 | 1.63 |
| 0.46 | 6.00 | 3.80 | 40.00 | 24.73 | 1.62 |

The estimated FS values given by the regressions (Table 5) were compared with the calculated ones from the simulations and the results were provided in Fig.4 (a-d). As it can be seen, excellent correlations between the original FS values (simulation) and the predicted ones (by regressions) were achieved. Fig.5 shows the how the FS varies depending to the excavation dimension ratio (W/H) and the values of Q i.e. the type of rock support. Lower FS values were obtained for \( Q = 0.46 \) while higher FS values were observed in \( Q = 453 \). In general, the FS decreases non-linearly with an increase of the ratio W/H and with a decrease of Q. It is noted that the FS corresponding to cases where \( Q = 160 \) (unsupported drift) are lower than those of \( Q = 8 \) (bolting). This is due to the mobilized strength of the rock bolt. In addition, the FS values were calculated differently according to whether the excavation was supported or not. Meanwhile in Fig.6, the FS values against Q values are shown. The cases were grouped into supported and unsupported excavations for better illustration. Two zones can be seen separated by a clear boundary but yet with similar general trend (FS increases with an increase of Q). In addition, in Fig.7, a 3D visualization of the FS values is provided.
In that figure a color code for the FS contour is used. For example, for a given rock mass quality and the W/H ratio that satisfies an acceptable FS can be determined. When, W/F =1.5, two areas corresponding to FS =2.5 (blue areas) can be found: a lower (supported excavations) and an upper (unsupported excavations). This 3D graph can be potentially useful for preliminary excavation stability design because when Q is known, the FS can be estimated for each excavation type in terms of their dimension ratios. However, more data points would be needed to improve the graph.

![Graphs of Calculated FS vs Predicted FS for different rock domains](image)

Fig. 4 (a) Calculated FS vs Predicted FS for rock domain Nº1

Fig. 4 (b) Calculated FS vs Predicted FS for rock domain Nº2

Fig. 4 (c) Calculated FS vs Predicted FS for rock domain Nº3

Fig. 4 (d) Calculated FS vs Predicted FS for rock domain Nº4
Fig. 5 Factor of safety against W/H for each rock domain

Fig. 6 Factor of safety chart based on Q values for supported and unsupported cases

Fig. 7 Factor of safety contour plot
3.2. Determining the reliability index of the excavation supports

The reliability index \( \beta \) for each support scenario (with the different dimension ratio and Q values) was determined using the EXCEL spreadsheet environment developed by Low and Tang (2007). In that spreadsheet the performance functions were defined using Eqs.5-7, as \( G(x) = FS - 1 \). The spreadsheet has been tested by other researchers and it appeared to be very useful for reliability analysis in previous research works (Goh and Zhang 2012), hence, also adopted in this study. The parameters used for the reliability analysis include Q, W, and H. These parameters were judged enough for the reliability analysis. It should be noted that the support mechanical parameters (strength and elastic modulus) were not explicitly considered, rather implicitly. Fig.8 shows a screenshot of the spreadsheet for computing the reliability index. Q was assumed lognormally distributed; while W and H were normally distributed. A detailed explanation of the use of the spreadsheet can be found in Low and Tang (2007). In cells A2:A4 of the spreadsheet, the common distribution types of the input variables are specified, including the normal, lognormal, uniform, and exponential distributions. These distributions must be selected based on the distribution of the variables on hand; hence, lognormal distribution was selected in cells A2:A4 (Fig.8) because the data of this study were lognormally distributed. Cells C2:C4 correspond to the mean values while cells D2:D4 correspond to the standard deviations. These parameters can be easily obtained from the dataset. The correlation matrix \( R \) as mentioned in (Eq.2) is represented in cells K2:M4; basically it defines the correlations between Q, W, and H. In this study, the variables are assumed to be non-correlated for simplicity (actually, although W and H somehow are correlated for operational reasons). The \( n_x \) vector in cells N2:N4 is identified according to Eq. (3). Regarding the design point (\( x^* \) values), it was initially assigned the mean values then the SOLVER search algorithm was invoked. Next, iterative numerical derivatives and directional
search for the design point $x^*$ were automatically carried out in the spreadsheet environment and ultimately the reliability index can be computed.

Table 7 summarizes the computed reliability indices and the probability of failure for each rock domain and corresponding rock support. In this table, the mean values of $Q$ for each rock domain were used. It can be seen that the reliability index varies between 0.62 and 3.25. Lower values indicated higher probability of failure. The results also showed that stable ground cases ($10 < Q < 40$) where no support is required and only shotcrete is applied in area of potential rock instabilities, are less reliable than when bolting is used. Table 7 shows that the safety factor values are not necessary directly correlated with the reliability indices. Therefore, a higher factor of safety doesn’t necessarily mean a higher reliability index. This is also illustrated in Fig.9. For example, the average factor of safety for rock domain Nº1 and domain Nº4 are 2.17 and 2.14, respectively while the reliability indices are 3.25 and 0.62, respectively.

Table 7 Summary of the reliability indices and probability of failure for each rock domain and corresponding rock support

| Range of Q   | Q<4  | 4<Q<10 | 10<Q<40 | Q>40          |
|--------------|------|--------|----------|---------------|
| Rock support | Bolting, concrete or steel arch | Bolting or concrete | Shotcrete | No support   |
| Safety factor range | 3.64–1.51 | 3.60–1.15 | 1.96–1.13 | 3.59–1.26    |
| Safety factor (average) | 2.14  | 2.27   | 1.45     | 2.17          |
The reliability index was computed using the Q mean. To illustrate the effect of the variability of Q on the reliability index, the coefficient of variability of Q (CovQ) is used to plot Fig.10. In that figure, the variability of the excavation dimensions was maintained as constant since W and H have a specific dimension. It can be seen that in moderately stable (rock domain №3) and unstable ground conditions (rock domain №4), the reliability index is very sensitive to a variation of the rock mass quality. For example, $\beta$ decreased from 1.5 to 0.5 when Cov(Q) varies from 0.1 to 0.8. Meanwhile, in very stable (rock domain №1) and a stable ground condition (rock domain №2), the reliability index $\beta$ remains almost unchanged. This implies that for the drift supports in rock domain №3 and №4, the higher the Cov(Q) the higher the probability of failure. The probability of failure went from 0.0065 to 0.031% in the case of rock domain №4. From Fig.9 and Fig.10, it can be concluded that that a single FS
value obtained from deterministic method may not always provide a sufficient indication of safety.

![Graph showing Reliability index β versus Cov (Q) illustrating the effect of Q variability on β.](image)

Fig. 10 Reliability index $\beta$ versus $\text{Cov} (Q)$ illustrating the effect of $Q$ variability on $\beta$.

4. Discussion

In this study, empirical equations obtained through regressions showed the correlation between FS and Q, H, and W. They indicate that FS decreases non-linearly with an increase of the ratio W/H and with a decrease of Q. Also, FS is proportional to the reliability index depending whether the drift is supported or not while the coefficient of variation of Q mainly dictates the level of reliability. A single FS value obtained from deterministic method may not always provide a sufficient indication of safety. This implies the existence of a degree of uncertainty involved in the reliable design of the mine drift supports; this uncertainty is associated with the variability of the rock mass parameters which play an important role. This nuance cannot be reflected in the conventional safety factor analysis. Hence, it is suggested that the factor of safety and reliability analyses be used together, as complementary measures of safer design, especially when the variability of the rock mass parameters is high. However, it should be noted there have been a series of assumptions made in this study. For example,
the variability of the support elements was not considered for simplicity. The results of this study were based on the use of a 2D simulation of the supports. The reader should be aware of these limitations of the research and make use of sound engineering judgment when implementing a similar research approach.

The ranges of the reliability indices of the support (for each rock domain) fall within 0.6 and 3.25. In comparison with existing research, these results are within the common range of the reliability index of geotechnical structures (Baravalle and Köhler 2016; Goh and Zhang 2012; Müller 2010; Skrzypczak et al. 2017). The next question is to determine the level of reliability acceptable of each support. Unfortunately, clear recommendations on the acceptable level of reliability of mine drift support are very limited. Hence, the discussion here is based on general reliability indices for geotechnical structures. The reliability indices for most geotechnical structures range between 1.0 and 5.0 (Pf ranging from about 0.16 to 3x10-7) and their expected performance reliability level, as detailed in Table 8.a. In addition, in the reliability-based design, it is common practice for practitioners to make decisions through a risk-based approach where the relative cost of safety measures and the failure consequences are used to define the target reliability indices (Table 8.b). Target values of the reliability index β for geotechnical components (concrete structures) can be 1.0-1.3; 1.5-1.7; and 2.0-2.3 for high, moderate, and low relative cost of safety, respectively (Müller 2010).

The reliability index thresholds in Tables 8(a-b) have been calibrated mostly for structure with long life, and might not reflect short life span structure like a mine drift. A poor target performance (β=2.0) for a reinforced concrete structure designed to last 50 years, is not necessarily the same for a mine drift constructed to last a few years. For illustration, the requirements (in terms of Cov(Q) to satisfy the target performance level of geotechnical
structures according to Table 8.a), were evaluated and the results provided in Table 9 (2.8x3.8 m excavation was used). The FS has been computed for each ground conditions based on the average values of Q and the reliability index is dictated by the coefficient of variation of Q. For example, for unstable ground conditions (rock domain Nº4), the corresponding reliability index is 0.62 (refer to Table 7), which would be classified as hazardous reliability level. However, the numerical modelling results indicated no failure. Hence 0.62 could be one minimum acceptable reliability threshold. According to Table 8.a, in order to satisfy an FS of 1.84 with good target performance (β=2.0), a Cov(Q) of 26% of the rock mass quality Q must exist. This is unlikely to happen because the rock mass properties showed a wide variability. On the other hand, in very good ground conditions (rock domain Nº1), at least an acceptable level of performance is likely to be achieved. In summary, Table 9 can be used to meet the desirable target performance levels for a given ground condition. If the desired reliability level cannot be achieved, then FS must be increased to the higher values (refer to Table 7). In other words, if a higher factor of safety is needed, and in absence of low variability of Q, then the support capacities should be increased. Based on Table 7 and Table 9 it can be suggested that β>3.0 correspond to high performance; β ranging between 2 and 3, good performance; and β below 2.0, poor performance reliability level.

Table 8.a  Range of reliability index for geotechnical structures (Skrzypczak et al. 2017)

| Reliability index | Probability of failure | Expected performance reliability level |
|-------------------|------------------------|---------------------------------------|
| 1.0               | 0.16                   | Hazardous                             |
| 1.5               | 0.067                  | Unsatisfactory                        |
| 2.0               | 0.023                  | Poor                                  |
| 2.5               | 0.0062                 | Below average                          |
Table 8.b Example of target reliability indices(yearly) and associated failure probabilities (Baravalle and Köhler 2016)

| Relative cost of safety measures | Ultimate state limit | Irreversible serviceability limit |
|---------------------------------|----------------------|----------------------------------|
|                                 | Minor failure consequences | Moderate failure consequences | Major failure consequences |
| Large                           | 3.1                  | 3.3                              | 3.7                        | 1.3                       |
| Normal                          | 3.7                  | 4.2                              | 4.4                        | 1.7                       |
| Small                           | 4.2                  | 4.4                              | 4.7                        | 2.3                       |

Table 9 Cov (Q) requirement to satisfy the target performance level and corresponding FS

| Ground conditions | FS | Target performance level |
|-------------------|----|--------------------------|
|                   |    | Poor | Below average | Above average | Good |
| Very stable       | 2.91| -    | -              | -              | <0.05%|
| Stable            | 1.80| -    | -              | -              | 70%   |
| Moderately stable | 2.97| 75%  | 61%            | 51%            | 38%   |
| Unstable          | 1.84| 50%  | 40%            | 35%            | 26%   |

Further study could aim to determine the acceptable reliability level perhaps using a risk-based method, where the consequences and level of failure should be assessed thoroughly as suggested by recent research (Baravalle and Köhler 2016). This approach could be useful for drift support design since the consequences and hazards of support failure can be established and considered explicitly in a calibration. Adequate reliability index thresholds should be calibrated.
5. Conclusions

This study was aimed at performing a reliability analysis of the rock support elements in mine drifts. Several support cases were considered including; shotcrete, bolting, concrete, combined bolting and concrete, as well as unsupported drift case. The factors of safety, reliability index, and the probability of failure were determined through numerical simulation and the first order reliability method. Different excavation dimension ratios were considered. The results showed that FS increases with an increase of Q while decreases with an increase of W/H ratio. The reliability analyses indicated that the probability of failure is significantly influenced by Q (i.e. when the coefficient of variability of Q is considered) and to a lesser extent the ratio W/H. This suggests that one single safety factor value cannot provide the complete indication of the underground mine drift support, since the same safety factor can indicate completely different probabilities of failure for different Q values. It is suggested that the reliability index (and the probability of failure) be used together with the traditional factor of safety for drift support design, especially when the variability of the rock mass has to be considered. The reliability target performance level for mine drift can be lowered in comparison to the thresholds used for long life geotechnical structures.

However, it should be noted that in this study, the results of the reliability index are dependent on the factor of safety which was obtained through numerical modelling. Also, the variability of the RQD and UCS values which affect the Q values were not considered. In addition, for simplicity purposes the mechanical parameters of the supports were not explicit in the calculation of the reliability index. Hence, further study could implement more realistic approaches in the simulations by using a 3D model, and by incorporating the support element properties in a straightforward manner. Clear definitions of the reliability target for drift
support could also be looked at in further research. Nevertheless, the results of the present study have some merit. The developed FS regressions, charts, and the suggested reliability target performance levels could serve as basis for practical design of mine drift supports design, where a high level of the variation of rock mass parameters is often encountered.

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Declarations

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Figures

Figure 1
Examples of collapsed supports showing damaged mesh and rock bolts

Figure 2
Illustration of the reliability index in the ellipsoid perspective (Low and Tang 2007)
Figure 3

(a) Unsupported drift (b) Supported drift with shotcrete (c) Supported drift with rock bolt (d) Supported drift with concrete and bolting
Figure 4

(a) Calculated FS vs Predicted FS for rock domain Nº1 (b) Calculated FS vs Predicted FS for rock domain Nº2 (c) Calculated FS vs Predicted FS for rock domain Nº3 (d) Calculated FS vs Predicted FS for rock domain Nº4
Figure 5
Factor of safety against W/H for each rock domain

Figure 6
Factor of safety chart based on Q values for supported and unsupported cases
Figure 7

Factor of safety contour plot

| Distribution | Para1 | Para2 | Para3 | Para4 | $\chi^2$ | $\mu^N$ | $\sigma^N$ | Correlation matrix | $n_2$ | $g(x)$ | $\beta$ | $P_f$ |
|--------------|-------|-------|-------|-------|----------|---------|-----------|------------------|-------|-------|--------|-------|
| Lognormal    | Q     | 1.35  | 0.94  |       | 0.7832   | 1.0548  | 0.4924    | 1 0 0 -0.552 -6E-07 0.6308 26.409 |
| Normal       | W     | 4.95  | 1.5   |       | 4.5125   | 4.95    | 1.5       | 0 1 0 -0.292 |
| Normal       | H     | 3.5   | 0.37  |       | 3.534    | 3.5     | 0.37      | 0 0 1 0.0919 |

Figure 8

Reliability index $\beta$ value for bolting and concrete (Q<4, rock domain Nº4)
Figure 9

Reliability index $\beta$ and FS versus Q values

Figure 10

Reliability index $\beta$ versus Cov (Q) illustrating the effect of Q variability on $\beta$. 