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Impact of the typical errors in geotechnical core logging for geomechanical design in large caving mines

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Abstract

Rock mass classification systems are widely used by geologists and geotechnical engineers for the classification, empirical design, and numerical modelling, especially during the first stages of a mining project, such as for Scoping and Pre-Feasibility studies.

The most used systems in the mining industry are represented by Laubscher rock mass rating, Bieniawski rock mass rating, Barton Q, Laubscher and Jakubec in situ rock mass rating system, and Hoek–Brown geological strength index, consisting of empirical methods that characterise, in a simple and fast way, the rock mass quality while offering engineering applications to the geotechnical design, such as ground support design, pillar strength estimation, fragmentation, rock mass strength, caveability, among others.

The authors have been involved in all the engineering stages of numerous large mining projects, from the scoping to the construction, passing through due diligence and peer review. When reviewing the geotechnical database of these projects, common and frequent errors seem to repeat, related to the collection of the basic geotechnical parameters.

The most frequent errors are associated to a mistaken distinction among natural and mechanical discontinuities, erroneous joint counts and consequently FF/m calculation and a wrong assessment of the rock quality designation. Other typical errors have been detected in the assessment of the joint condition, most of them referred to the characterisation of the joint alteration (J_a) Barton parameter.

The aim of this paper is to show the impact of the geotechnical errors and mistakes over the geomechanical design, quantifying the variation in terms of rock mass strength, caveability, pillar strength, Factor of Safety, etc., that will produce a negative impact on both the capex, opex and high risk for a mining project and/or operation.

Keywords: *geotechnical core logging, geotechnical error, rock mass characterisation, geomechanical design*

1 Introduction

During the early stages of a mining project, most and in many cases the only available geotechnical information is obtained from core logging. Currently, the most used rock mass classification systems for core logging by geologists and mining geotechnical engineers are Q (Barton et al. 1974; Grimstad & Barton 1993), rock mass rating (RMR) (Bieniawski 1989), mining rock mass rating (MRMR) (Laubscher 1990), in situ rock mass rating (IRMR) (Laubscher & Jakubec 2001) and geological strength index (GSI) (Hoek et al. 2013; Hoek & Brown 2019). These systems have in common basic parameters that are used to characterise a rock mass: the spacing of discontinuities and block size, the condition of discontinuities, and the strength of intact rock.

At the beginning of engineering stages of large mining projects, from the scoping to the construction, one of the first tasks that is often required is the review and validation of the geotechnical database; this activity points out common and frequent errors that are related to the collection of the geotechnical parameters. The most frequent errors are related to the assessment of the fracture frequency or rock quality designation

(RQD), the description of the joint condition and the estimation of the intact rock strength for highly veined rock mass.

The aim of this paper is to show how an incorrect geotechnical rock mass assessment can affect the geotechnical and geomechanical design that will produce a negative impact on both the capex, opex and high risk for a mining project and/or operation.

2 Typical errors in geotechnical characterisation and suggested remedies

This section shall present some of the common errors detected during the review of the geotechnical data bases from different mining projects. Some examples will be presented related to the assessment of FF/m, RQD, joint condition and the estimation of the intact rock strength from laboratory testings.

2.1 Fracture frequency assessment

The detection of the quantity of natural joints that are present in a geotechnical interval and their differentiation from mechanical breaks are at the base of the FF/m assessment for a rock mass. Below is a summary of the most frequent errors detected regarding joint counts and FF/m calculation:

- Confusion when differentiating between natural joints and mechanical breaks (Figure 1). Natural discontinuities can be recognised by the presence of oxides or other mineral infills on their surfaces, while mechanical breaks are characterised by a fresh cut and on the discontinuity surface, only the rock matrix can be observed.
- Incorrect assessment regarding the amount of rubble material, as shown in Figure 2. Natural rubble zones are characterised by irregular rock pieces with different size and shape (the cylinder shape is lost) with one or more surface showing oxides or other mineral infill.
- In some projects it has been observed that the presence of rubble, gouge, or highly weathered material it is not taken in account for the joint count. If this kind of material is observed in the interval, four joints every 10 cm of rubble etc. should be added to the joint count, in order to reach 40 joints in 1 m of rubble or highly weathered material.
- Incorrect joint count as observed in Figure 2. The number of joints counted it is not present in the interval. The geotechnical quality of both intervals was clearly underestimated.
- Usually, joints are counted as the total number observed in the interval and are not counted by set, hence FF/m is calculated as the ratio between the total joints count and the total core recovery (TCR). Joints in drill cores should be counted by sets, defined by the α angle formed by the joint and the core axis. The spacing of a joint set is defined as the distance measured perpendicular to a joint surface (Bieniawski 1989; Hudson & Priest 1979; International Society for Rock Mechanics (ISRM) 1978; Laubscher 1990; Palmstrom 1982, 2001, 2005), therefore the FF/m of each set must be calculated according to the real spacing of the set. The FF/m of the rock mass is calculated by the sum of the individual FF/m of each set, divided by the factor two for drill cores (Laubscher 1990).

Figure 1 Examples of natural discontinuities incorrectly classified as mechanical breaks

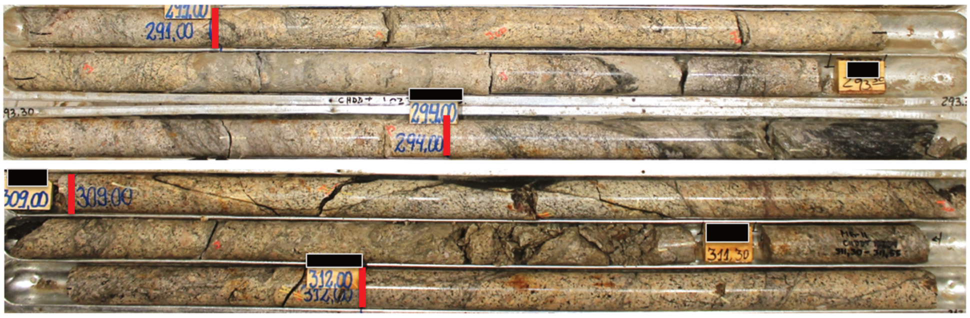


Figure 2 Incorrect classification of amount of rubble. The upper geotechnical interval, delimited by red lines, was assessed as follows: rubble = 0.55 m; FF/m = 10.67; GSI = 20-25; RMR_{B89} = 48; RMR_{L90} = 26 and Q′ = 3.36. The lower interval, delimited by red lines, was assessed as follows: rubble = 1.5 m; FF/m = 22.67; GSI = 15-20; RMR_{B89} = 39; RMR_{L90} = 22 and Q′ = 0.42

2.2 RQD assessment

Deere et al. (1967) proposed the RQD index, defined as a simple classification system to estimate the rock mass geotechnical quality from drillcore logs, is today what is mostly used as an input parameter for more complexes classification systems, as Bieniawski's RMR, Barton's Q and Laubscher's RMR. RQD is defined as the percentage of sound rock pieces longer than 100 mm (4 inches) in the total length of a core run.

During the review of geotechnical databases, errors related to the calculation of the RQD have often been detected, such as:

- Core pieces or core intervals with a degree of weathering/alteration ≥ IV (highly weathered/altered) have been included for RQD calculation. These pieces should be disregarded (Bieniawski 1974; Deere & Deere 1988).
- Core pieces or core intervals with an intact rock strength (IRS) ≤ R1, or friable cores, have been included in the RQD calculation. These pieces should be disregarded.
- Mechanical breaks used to measure the core pieces, as if they were natural.
- Natural joints classified as mechanical, hence, disregarded for RQD calculation.

Figure 3 shows two examples of geotechnical intervals where an RQD = 100% was assigned. The first example shows a weak and friable rock that should receive RQD = 0%; the second example shows an RQD = 100% assigned, instead of a valid value of RQD = 20%.

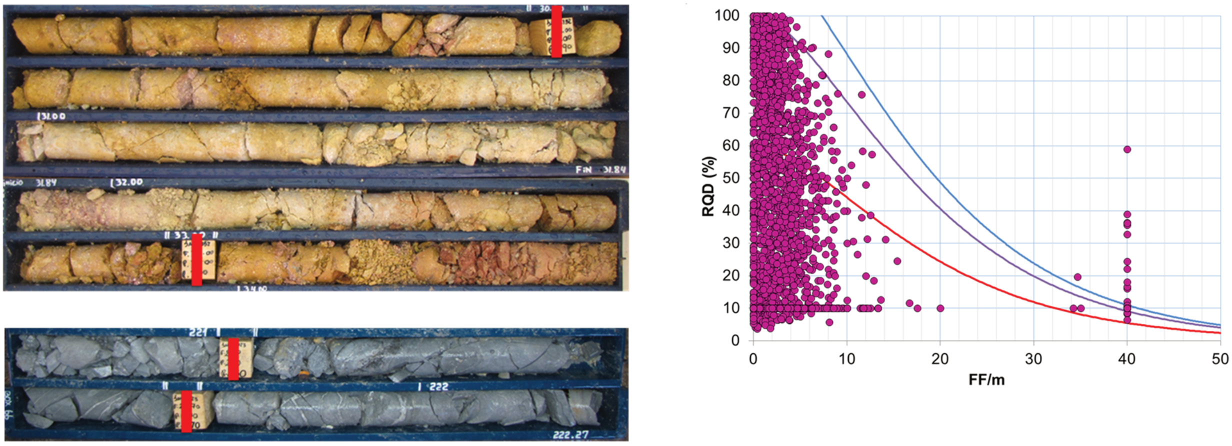


Figure 3 Examples of erroneous RQD estimation. Both intervals were assessed with RQD = 100%. On the right side an example of an incorrect correlation among RQD and FF/m

2.3 Joint condition assessment

The assessment of the joint condition considers several characteristics related with the shape of the joint surface, its alteration, and the type of mineral infill. The review of many geotechnical databases pointed out errors related to the aperture, joint wall alteration and the roughness.

2.3.1 Aperture

The review of many databases pointed out that frequently, the thickness of the mineral infill is recorded as the aperture. According to the ISRM (1978) the aperture is the perpendicular distance between adjacent rock walls of a discontinuity, in which the intervening space is air or water-filled, hence the aperture is the measure of the empty space between adjacent rock walls or between mineral infill and rock wall.

2.3.2 Joint wall alteration

Different types of errors have been detected according to the used classification system. According to Bieniawski (1989), the rating of the joint wall alteration is assigned taking into consideration if the discontinuity walls are fresh or how much the alteration extends into the rock wall. It has been observed that this parameter, often, is underestimated during the core logging, therefore it is common that a worst condition to the joint wall alteration is assigned, as shown in Figure 4.

The Q-system (Barton et al. 1974) evaluates the type of mineral infill and the joint wall alteration through the joint alteration number (J_a), which defines three types of contacts of the joint walls. Type a) is related to joints with rock walls contact; type b) is referred to joints with rock wall contact facing 10 cm of shear; and type c) refers to joints without contact between the rock walls.

The review of the geotechnical data bases shows that often the J_a parameter is underestimated by the geotechnical loggers, driven to an under estimation of the Q-value. It has been observed that the J_a parameter tends to be underestimated more than other classification systems. Figure 5 shows an example of a good quality geotechnical interval where J_a = 15 was assigned, corresponding to c-type rock wall contact (No rock wall contact when sheared - thick mineral fillings) and to type P infill, corresponding to 'thick, continuous zones or bands with clay. Swelling clay. J_a depends on a percentage of clay-size particles swelling'. The correct assessment would be an a-type rock wall contact (rock wall contact - no mineral fillings, only coatings) and a-type B infill, corresponding to 'unaltered joint walls, surface staining only', with a value of J_a = 1.

Figure 4 Example of erroneous joint wall alteration assessment. In the upper interval the joint wall alteration was assessed as 'completely weathered', whereas, in the lower interval it was assessed as 'highly weathered'

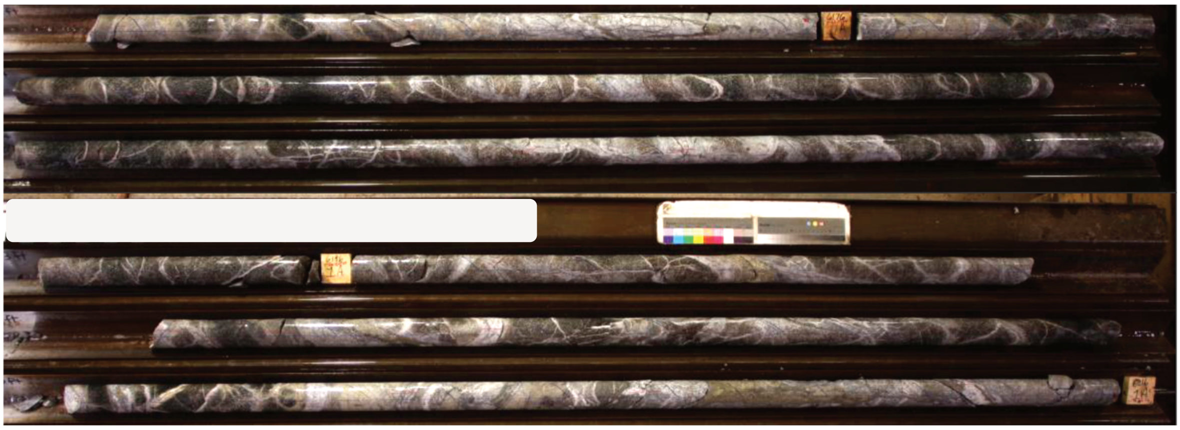


Figure 5 Example of an incorrect J_a assessment. A J_a = 15 was logged instead of a correct J_a = 1

According to the MRMR system (Laubscher 1990; Laubscher & Jakubec 2001), the joint condition will be adjusted for the joint wall alteration only if the joint wall alteration is weaker than the mineral infill and weaker than the rock wall. According to that, the joint condition will be adjusted only if the joint wall is the weakest part of the mineral infill-joint wall–rock wall system.

In has been observed in some projects, that different adjustments have been assigned associated to the degree of weathering defined by the RMR system (Bieniawski 1989). Table 1 shows the adjustment factors used to assess the joint wall alteration for the MRMR system. Clearly, the ratings used in Table 1 do not belong to the MRMR system (Laubscher 1990; Laubscher & Jakubec 2001), therefore implementing these generates an incorrect assessment.

Table 1 Incorrect adjustment factors used to assess the joint wall alteration for MRMR quantification

Joint wall alteration (Bieniawski 1989)	Modified ratings used to assess MRMR
Fresh	95–100%
Slightly weathered	89–94%
Moderately weathered	81–88%
Highly weathered	76–80%
Completely weathered	75%

2.3.3 Joint roughness

The assessment of the roughness of a joint surface has to consider three aspects: the planarity, that is referred to the undulating of the surface of a joint, measured at meter scale; the roughness, that measures the asperity of a joint surface at a centimetric to decimetric scale and, finally, the smoothness, that measures the asperity of a joint at millimetric scale (Palmstrom 2001).

Below, the most frequent errors detected related with joint roughness assessment are detailed:

- Types of planarity have been used to assess the roughness of major structures from a core logging. As mentioned before, planarity is a feature that is characterised at metric scale, hence it would be incorrect to use planarity categories to assess roughness at the centimetric scale of a drillcore.
- A geotechnical core logging that only records smoothness categories: very rough, rough, slightly rough, smooth and slickenside, according to Bieniawski (1989), then the smoothness was used to assign the roughness ratings according to Barton et al. (1974), Laubscher (1990) and Laubscher & Jakubec (2001). This procedure is clearly incorrect because the smoothness measures the joint surface asperity but does not consider the undulation of the joint surface. Both the smoothness and the roughness of the joint sets are required to be collected.
- In many projects and mining operations, the joint roughness and smoothness are assessed from the Joint Roughness Coefficient (JRC). The JRC profiles have been designed at the 10 cm scale, hence, it should be taken in account that the joint roughness is a function of the amplitude of the asperities and the observation scale (Palmstrom 2001), it means that a joint surface, to be classified as strongly undulating (JRC = 15) requires an amplitude of 1.5 mm at a 50 mm observation scale (a typical diameter of a drillcore), whereas, at 10 cm (Barton scale) it requires an amplitude of 3 mm, and at 20 cm (Laubscher scale) it requires an amplitude of 6 mm. That means that if the JRC profiles are used for a typical HQ_3 or NQ_3 core logging, the joint roughness will be underestimated if the mapping scale is not considered.

2.4 Intact rock strength assessment

Porphyry copper deposits are characterised by a stockwork of mineralised veins and veinlets (micro defects) filled with minerals having different shear strength that are controlling the failure mode of the tested samples. Russo & Hormazabal (2016) described different types of failures mode that have been recognised from tested samples: rock, mixed, multiple veinlets and veinlet failure (Figure 6), of these, only the failure mode along a pre-existing veinlet is considered not representative of the intact rock strength, hence, the strength values obtained from these samples are disregarded for the intact rock assessment. In some deposits, it has been observed that some geotechnical units show a large variability in terms of micro defects density, as shown in Figure 7. The analysis of valid uniaxial compressive strength results for these units highlight a wide range of UCS values, hence the characterisation of these units by a simple average value could not be representative of the whole unit.

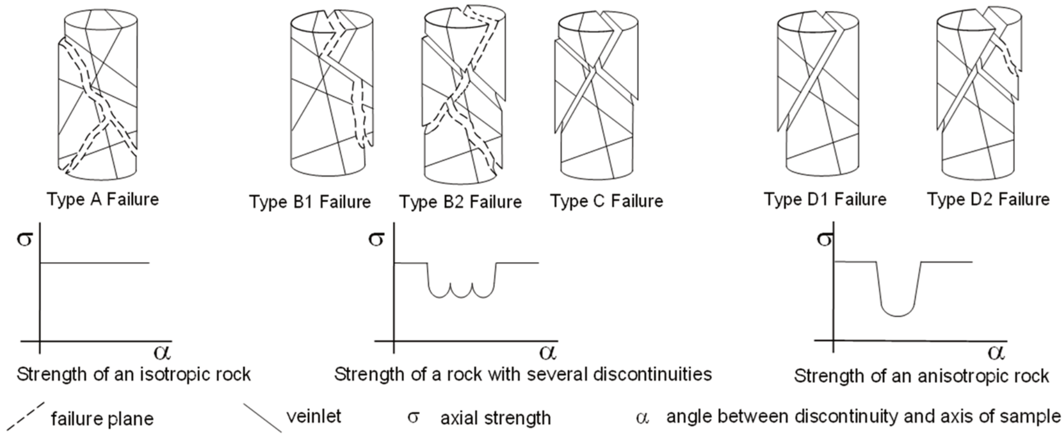


Figure 6 Types of failure observed in tested samples and type of anisotropy strength (Russo & Hormazabal 2016)

(a) (b)

Figure 7 Example of (a) high vein density and (b) low vein density units

The authors faced this challenge in a mining project and in order to reduce the tests results variability, a new concept for a better assessment of the intact rock strength was proposed. Samples tested for UCS and triaxial strength in a rock mechanics laboratory were characterised based on the number of defects or veins with infill up to five in the Mohs scale and the thickness of the mineral infill, defining the concept of weak veins density in the rock sample (D_{wv}). This parameter is calculated by the following equation:

$$D_{wv}[mm] = \sum n_i t_{i,H\leq 5} \tag{1}$$

where:

n_i = count of defects or veins having an infill type with Moh's hardness < 5.0.

$t_{i,H\leq 5}$ = thickness of defects or veins, that accomplish Mohs' hardness < 5.0.

Hence, was assigned a D_{wv} value to each tested sample and then were estimated the intact rock properties, for both groups of rock mass (high veined and low veined), using different D_{wv} threshold values. The comparison of results from all performed analysis defined a threshold value of D_{wv} = 10 mm that allowed to define two Hoek & Brown (2019) failure envelopes (Table 2), one for a low weak veins Density (D_{wv} ≤ 10 mm) and another for a high weak veins density (D_{wv} > 10 mm).

In order to subdivide each geotechnical unit in two sub-units, defined by the weak veins density, the D_{wv} values determined in each tested samples were correlated with the microdefects frequency in the core logging database obtaining a threshold value of 15 microdefetcs/m. Subsequently, two sub-units can be defined, a high weak veins density unit, defined by a D_{wv} > 10 mm or > 15 microdefects/m, and a low veins density unit, defined by a D_{wv} < 10 mm or < 15 microdefects/m. The estimation of different UCS for each sub-unit will not produce a significative variation on the classification systems, the most important contribution is related to a better estimation of Hoek & Brown parameters, like σ_{ci} and m_i, that will be used for 2D and 3D numerical modelling. The intact rock parameters estimated on such way will be showing different behaviour for each sub-unit, instead of averaged results, allowing a better analysis of the mine layout.

Table 2 Estimation of intact rock properties based on the veins' density

Geotechnical unit	UCS (MPa)	σ_{ci} (MPa)	m_i (–)
GU1a low vein density	93.7	93.6	20.3
GU1b high vein density	60.8	68.0	22.2
GU2a low vein density	85.5	88.9	19.1
GU2b high vein density	71.2	74.9	27.9

3 Impact of pitfalls on the geotechnical design

3.1 Geotechnical rock mass assessment

The abovementioned pitfalls have been observed in many mining projects and usually are related to an underestimation of the rock mass quality. In order to quantify how the pitfalls can downgrade the estimated rock mass, the authors reviewed the geotechnical database of a mining project and proceeded to relog the available drill cores in order to fix the detected errors (Figure 8) and achieve a geotechnical characterisation more consistent with the geotechnical quality observed on site. Table 3 shows a comparison among the classification systems calculated using the original and the fixed database. Here, it is possible to see how the geotechnical quality of the different geotechnical units (GU) have been consistently underestimated, i.e. the fracture frequency decreased between 2.5 and 8.7 fracture per meter and GSI increased between 18 and 26 points, for GU1 and GU 4 respectively. The increase of GSI values is significative and will have a big impact on the rock mass behaviour in the 2D and 3D numerical modelling, as will be shown in the next section.

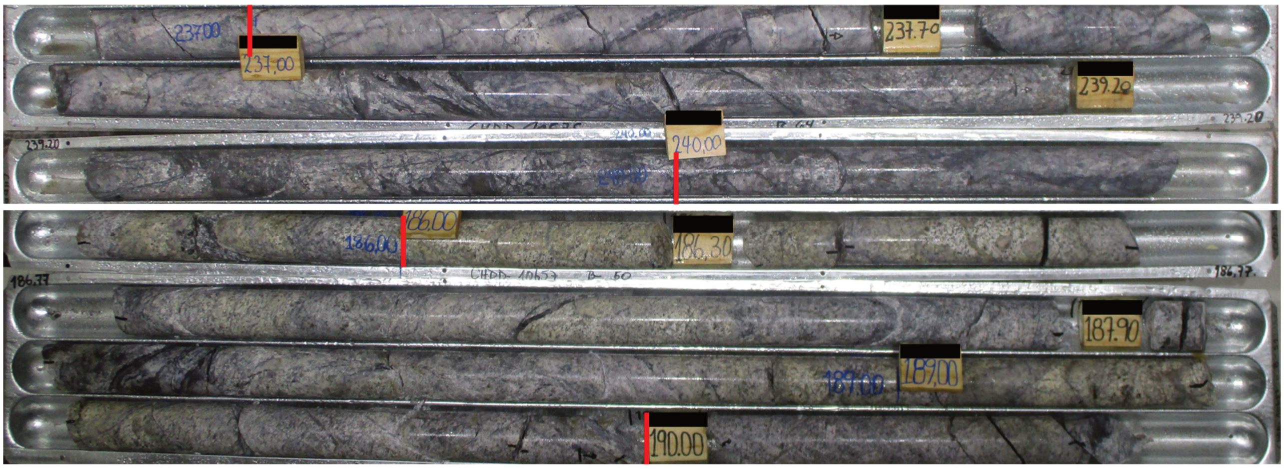


Figure 8 Example of detected errors: upper interval, rubble = 57 cm, FF/m = 14.6; lower interval, rubble = 1.23 m, FF/m = 18.7

Table 3 Comparison between geotechnical parameters and classification systems calculated using original and reviewed data for each geotechnical unit (GU)

Geotechnical characterisation		GU 1	GU 2	GU 3	GU 4
RQD	Original data	81	86	94	75
	Reviewed data	88	94	96	92
FF/m	Original data	9.5	8.5	4.5	11.4
	Reviewed data	2.5	2.2	2.0	2.7
RMR (Bieniawski 1989)	Original data	60	60	63	54
	Reviewed data	75	73	74	69
RMR (Laubscher 1990)	Original data	42	42	47	36
	Reviewed data	60	58	58	53
GSI (Hoek et al. 2013)	Original data	49	51	50	44
	Reviewed data	67	71	73	70

3.2 Fragmentation

In order to quantify the impact of the geotechnical errors on the fragmentation assessment, the fragmentation curves, for each GU, were estimated using the software BCF 3.05, considering a rock mass with three joint sets and for each set was considered the original and the reviewed FF/m. Table 4 summarises the values of the percent passing at 1 m^3 and 2 m^3 obtained, as BCF output, from the estimated fragmentation curves for each GU. The comparison pointed out a significative impact on the estimation of the fragmentation curves, as for example for the geotechnical unit 1 (GU 1) that changed from 98% of the fragmentation curve having a block size less than 1 m^3 to a 55%, as well as the other geotechnical units.

Table 4 Percent passing at 1 m^3 and 2 m^3. Values obtained from the fragmentation curves for each geotechnical unit using the software BCF 3.05

Geotechnical unit		% < 1 m^3	% < 2 m^3
GU 1	Original data	98%	100%
	Reviewed data	55%	90%
GU 2	Original data	98%	100%
	Reviewed data	79%	97%
GU 3	Original data	100%	100%
	Reviewed data	65%	92%
GU 4	Original data	99%	100%
	Reviewed data	86%	98%

3.2 Caveability

The empirical assessment of caveability is estimated according to the Laubscher MRMR. The RMR (Laubscher 1990) calculated using the original and the reviewed data were used to evaluate how the geotechnical error would impact on the estimation of the hydraulic radius and on the area that is required to undercut to start the caving. Table 5 summaries the required hydraulic radius (HR) and area to be undercut to start the caving for each evaluated GU. The required area increased 3,300 m^2, for GU 3, up to 5,600 m^2 for GU 1.

Table 5 Caveability assessment according to the original and the reviewed databases

Geotechnical unit		Hydraulic radius	Area to undercut (m^2)
GU 1	Original data	15	3,600
	Reviewed data	24	9,200
GU 2	Original data	15	3,600
	Reviewed data	23	8,500
GU 3	Original data	18	5,200
	Reviewed data	23	8,500
GU 4	Original data	13	2,700
	Reviewed data	21	7,000

3.3 Pillar stability

In order to verify how the recalculated GSI would impact on the analysis of the pillar stability, a three-dimensional local numerical model was generated in FLAC3D for the GU 4, representing the lowest quality GU, as shown in Table 3. The intact rock properties of the GU 4 that have been used as input in the 3D numerical modelling are summarised in Table 6. This local model consists in a representative sub-model that evaluates the behaviour of the production drifts and draw bells excavated in GU 4. The in situ and induced stresses, based on the available in situ stress measurements, have been extracted from a larger model, at mine scale, and applied to the local model (Figure 9) to evaluate in detail the pillar behaviour during the construction and extraction stages. The uncertainty related to in situ stress measurements has not been evaluated because, the aim of this paper it is not to discuss about their influence on the numerical modelling results, but how the errors in GSI estimation translate to numerical modelling results. The pillar stability is evaluated by the calculation of the Factor of Safety and the convergence response to the maximum abutment stress condition. The convergence was evaluated placing history points in strategical zones and the convergence confinement method (Carranza-Torres & Fairhurst 2000) was included with internal pressure for the relaxation response (Figure 10).

Table 6 Caveability assessment according to the original and the reviewed databases

Density (kg/m^3)	σ_{ci} (MPa)	m_i	Young (GPa)	Poisson (–)	GSI original value	GSI reviewed value
2.70	69.3	24	18.2	0.21	44	70

Figure 11 shows the results obtained from the numerical modelling using the Hoek & Brown Failure Criterion (Hoek & Brown 2019), the original and the reviewed GSI values for the geotechnical unit 4 (GU 4 in Table 3) and considering stress conditions related to the drifts excavation. Figure 11a shows the plastic shear strain contours as indicative of damage in the rock mass (Hamdi et al. 2017), where 5% represents the zone of critical damage. The graph shows the strain evolution that the roof and walls can experiment during the production drift excavation, as well as its relaxation. Using the original value of GSI = 44, it can be assumed that the pillar collapse during excavation. Hence, the pillar is not stable while equilibrium could not exist even considering a heavy support installation. Meanwhile, using the reviewed value of GSI = 70, the pillars show a stable behaviour with a maximum strain observed at the perimetral tunnel roof reaching 3.4% (Figure 11b).

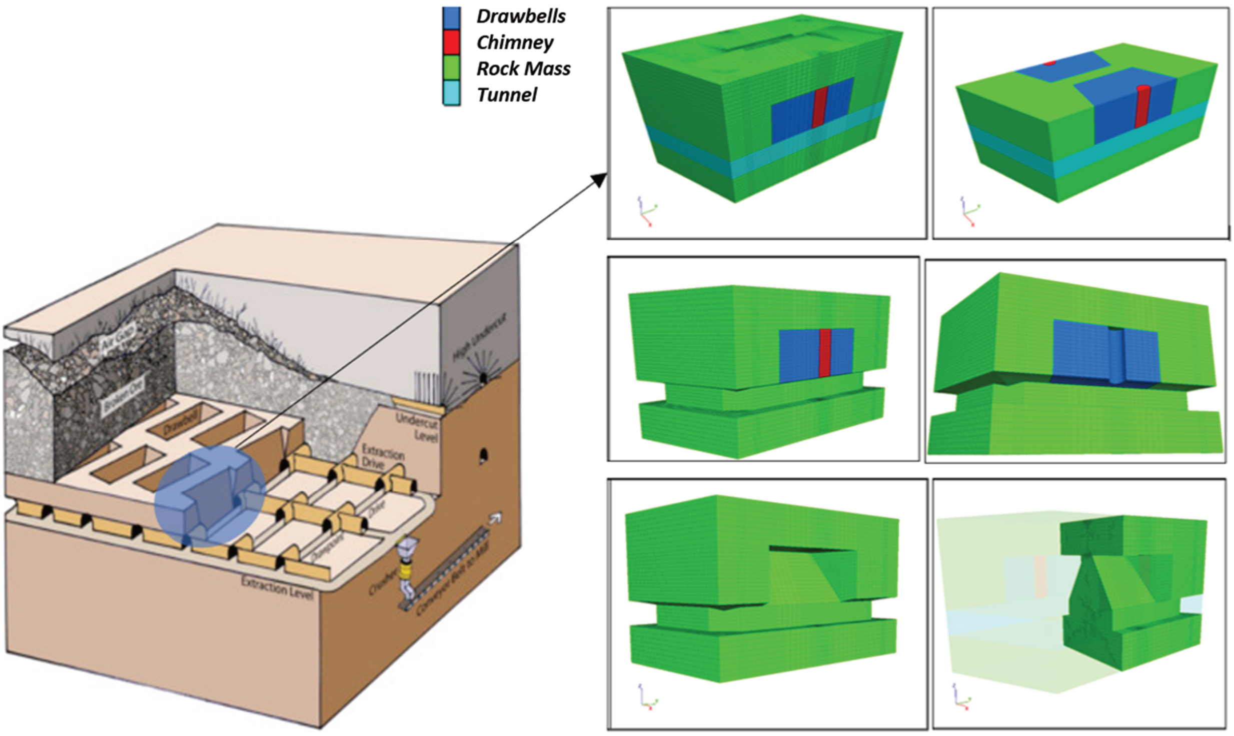


Figure 9 **Example of the tunnel-pillar-tunnel unit for a 3D local model in macroblock (modified from Flores & Catalan 2019)**

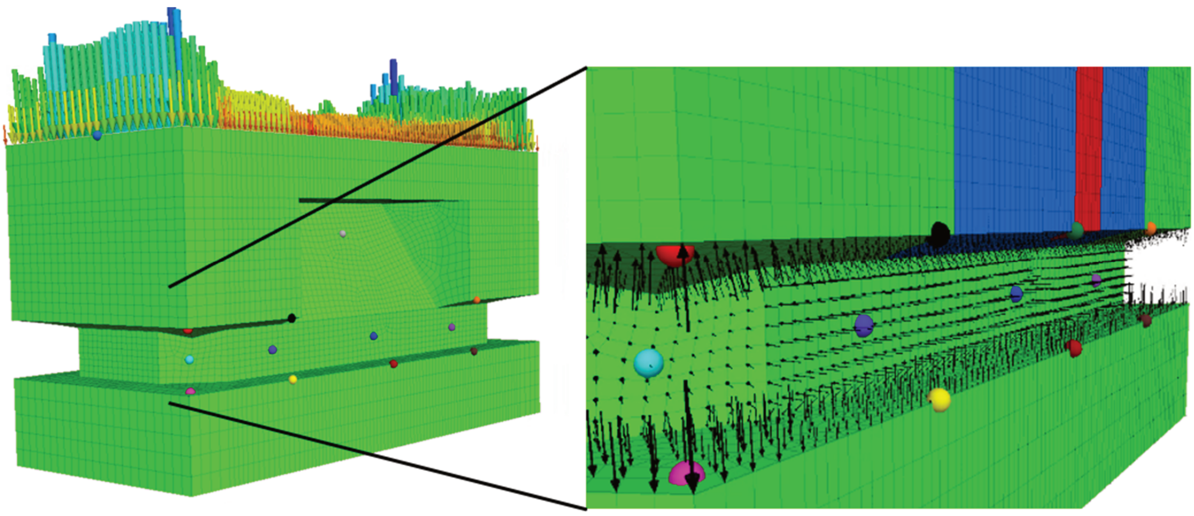
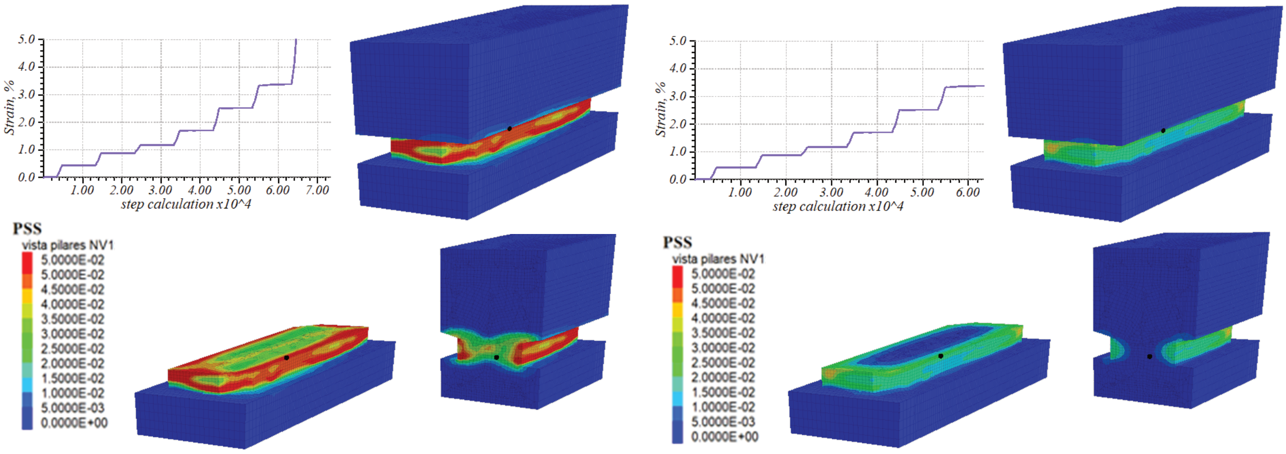


Figure 10 **Local model and load for abutment stress. (a) Internal pressure for convergence confinement analysis; (b) Arrows indicate forces with different magnitude**

Figure 12 shows the results of the pillar stability analysis, for GU 4, obtained using the reviewed GSI = 70 and considering an abutment stress of 1.4 times the in situ stress.

The factors of safety calculated by using the shear strength reduction technique, associated to the analysis using the reviewed GSI = 70 are shown in Figure 13. This figure indicates stable conditions during the construction stage with a FoS = 1.88 and a damaged pillar due to the abutment stress, but still able to keep the drift serviceability, with a FoS = 1.25.

The comparative analysis of the pillar stability hosted by the GU 4, developed using the same intact rock properties and stress conditions, shows the impact on the numerical modelling results, highlighting the negative impact that would have the original database (in this case a GSI = 44) on the mine design and geotechnical condition. If original database is not corrected, wider pillar needs to be designed and heavy support need to be considered, this will affect reserves, draw rates, draw interaction and productivity among others.



(a) (b)

Figure 11 **3D local model response comparison. (a) Original case: GSI = 44, pillar fails during drifts excavation; (b) Reviewed case: GSI = 70, pillar stable during drifts excavations. PSS: plastic shear strain**

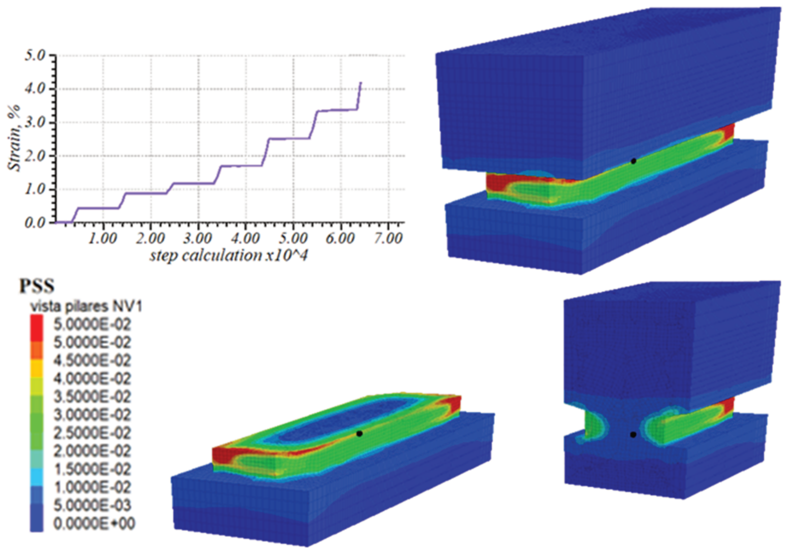
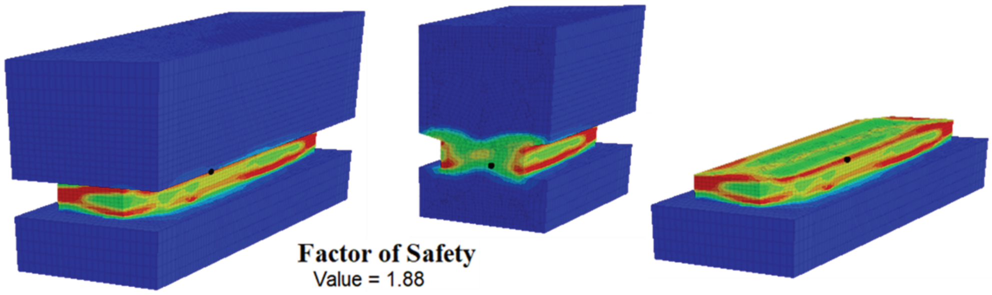
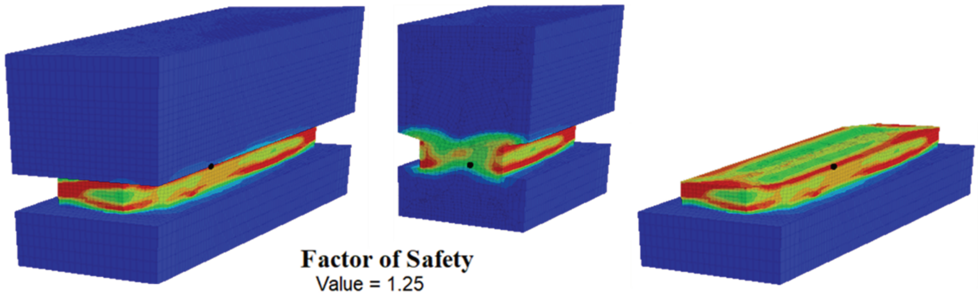


Figure 12 **Reviewed case: GSI = 70. Local damage localised on the pillar contour at the abutment stress condition, the inner part is still unaffected and stable. PSS: plastic shear strain**



(a)



(b)

Figure 13 3D local model (reviewed case: GSI = 70). (a) Pillar Factor of Safety during drifts construction (FoS = 1.88); (b) Pillar Factor of Safety under abutment stress condition (FoS = 1.25)

3.4 Support estimation

This section presents an approach to assess the impact for the design of tunnel support with rockbolts. An example of this type of analysis is shown in Figure 14 in which an 8 m span tunnel is excavated. In this analysis a pattern of 4 m long, 25 mm diameter un-tensioned grouted rockbolts and face plates have been installed 3 m behind the face. The procedure for sequencing the installation of reinforcement and support in a two-dimensional numerical model is described by Hoek et al. (2008).

There is no method for calculating the Factor of Safety of a tunnel, with a combination of reinforcement and support, such as that shown in previous chapter. However, the extent of the failure and plastic zone can be controlled by the installation of reinforcement or support. The aim of the designer should be to retain the tunnel profile as far as possible and to prevent or minimise small rockfalls from the surface (Hoek & Marinos 2009).

In order to achieve this goal it may be necessary to install a combination of reinforcement and support and to vary the rockbolt length, spacing and inclination to capture specific instability zones. In Figure 14 two cases were analysed considering only the variation of the GSI and same intact rock properties showed in Table 6. This figure show that for the same intact rock properties and in situ stresses varying only the GSI from 44 to 70, each rockbolt length can be optimised or reduced by 1 m; this would reduce by 25–35% the rockbolts costs (Russo & Herrera 2011). This percentage do not consider rockbolts installation, pattern reduction and cycle times so costs saving would be higher.

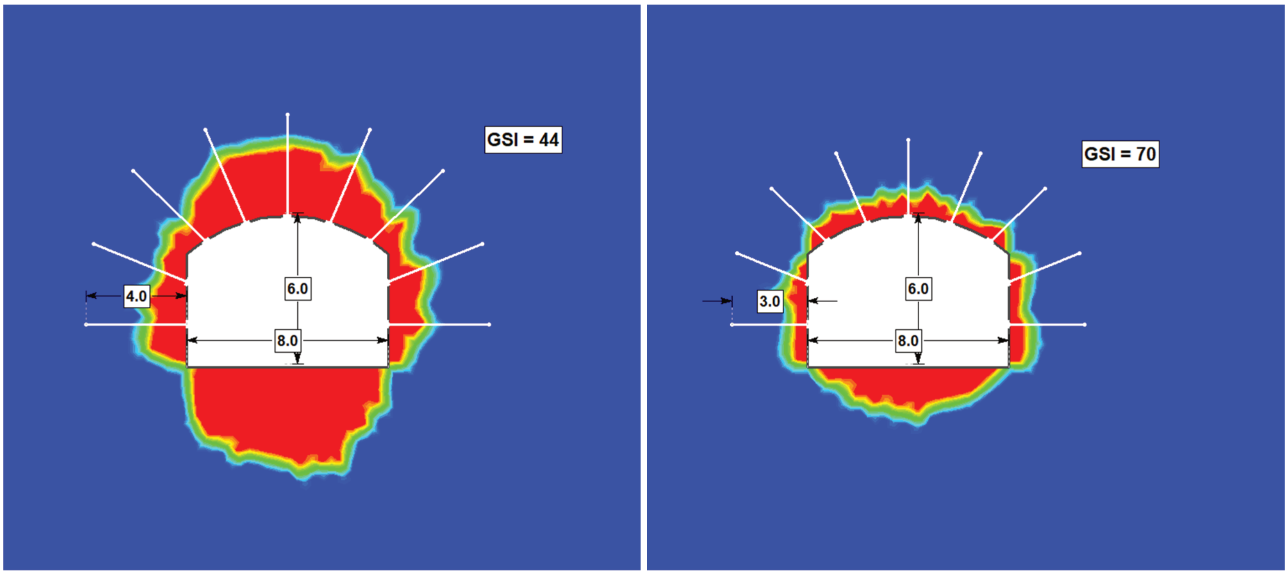


Figure 14 Finite element models of an 8 m span tunnel with typical support considering only the GSI variation

4 Conclusion

The review of geotechnical core logging database in numerous mining projects, from the scoping to the construction engineering stages, pointed out common and frequent errors that are related to the collection of the geotechnical parameters. The most frequent errors are related to the assessment of the fracture frequency or RQD, the description of the joint condition and the estimation of the intact rock strength for highly veined rock mass.

The detected errors are related to different causes, such as an insufficient or incorrect core logging procedure, low skilled geotechnical geologists or technicians in charge of the core logging. The errors observed in databases are random and usually tend to be conservative, underestimating the quality of the evaluated parameters; the result is a downgraded geotechnical quality of the rock mass.

Once the errors have been fixed in databases, it has been observed that classification systems, i.e. MRMR or GSI, can increase up to 15 or 20 points in the final rating. This paper analysed how the incorrect geotechnical characterisation leads to conservative analysis for pillars stability and ground support design and, on the other hand, to optimistic analysis related to caveability and fragmentation assessment. Finally, geotechnical pitfalls will have a negative impact on capex and opex of mining projects or operations that can be very significant. This issue represents a great opportunity for mining companies to reduce costs improving their core logging procedure, hiring skilled geotechnical geologists/engineers and receiving support from high skilled consulting companies.

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