Sri Lankan Rock Mechanics and Engineering Society

Sociedad Peruana de Geoingeniería-SPEG
Grupo Nacional de la ISRM
International Society for Rock Mechanics and Rock Engineering

PROCEEDINGS

The First ISRM Commission Conference on
Estimation of Rock Mass Strength and Deformability
- an ISRM Specialized Conference

December 6, 2024

Lima, Peru

Proceedings of the 1st ISRM Commission Conference on Estimation of Rock Mass Strength and Deformability

An ISRM Specialized International Conference

1st ISRM Commission Conference on Estimation of Rock Mass Strength and Deformability

[An ISRM Specialized International Conference]

06 December 2024

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Organized by (the ISRM National Groups of)

Sri Lankan Rock Mechanics & Engineering Society [SLRMES]

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Preface

The presence of complicated fracture networks, the inherent statistical nature of their geometrical parameters, and the variabilities and uncertainties involved in the estimation of their geometrical and geo-mechanical properties, in-situ stress, etc. make accurate estimation of rock mass strength and deformability a difficult, and challenging task. On the contrary, understanding the mechanical behaviour of rock masses is crucial in designing safe, economical, and robust engineering structures in or on rock masses.

The strength and deformability of rock masses demonstrate a very significant scale effect and anisotropic behaviour at the three-dimensional (3-D) level mostly due to pre-existing discontinuities. It has been a great challenge for the rock mechanics and rock engineering profession to predict rock mass strength and deformability in 3-D which incorporates the effect of important fracture geometry, relevant intact rock and fracture mechanical properties, and intermediate principal stress and to capture the scale effects and anisotropic properties of jointed rock masses. Various procedures that belong to the following three groups have been suggested in the literature to estimate rock mass strength and deformability: (a) Based on empirical methods that use one or several rock mass classification systems; (b) Based on numerical modelling, and (c) Based on back-calculation methods using field monitored data of rock engineering structures. However, the advantages and disadvantages of the said techniques are not clear to the teachers, practitioners, and researchers who deal with rock mechanics and rock engineering. In addition, neither standardized techniques nor accepted guidelines are available in the literature to estimate rock mass strength and deformability properties with confidence. These facts prompted the establishment of a new ISRM Commission on Estimation of Rock Mass Strength and Deformability.

A 16-member commission which represents all six continents was formed in mid-February 2024 and an application was submitted to form a new ISRM Commission on Estimation of Rock Mass Strength and Deformability. It was approved around April-May 2024. Several online Zoom meetings were held between the commission members and the commission chair. Based on the discussions, it was decided to have the first commission conference in Latin America in 2024. After extensive discussions with Dr. Antonio Samaniego and the ISRM Geo-engineering group in Peru, it was decided to hold the first commission conference in Lima, Peru on December 6th, 2024. The conference has been organized as a collaborative effort between the Sri Lankan Rock Mechanics and Engineering Society (SLRMES) and the Peruvian Society of Geo-engineering (SPEG). Note that both these societies are ISRM National Groups. A website was created in June-July 2024 to organize this conference (https://www.slrmes.org). An application was submitted in June-July to organize this conference as an ISRM-specialized conference. It received prompt approval from the ISRM Board. The conference organization has been primarily conducted using the conference website and through e-mail communications.

The main goals of the conference are to provide a review of the said available methods, and then to take initial steps to provide guidelines for rock mechanics teaching, to suggest future research to improve the available techniques in predicting rock mass strength and deformability properties, and to recommend the best techniques to apply in rock engineering practice to improve the prediction of rock mass mechanical behaviour in field problems associated with mining, civil geotechnical, geological, and petroleum engineering. About 25 extended abstracts were received for possible presentations at the conference. As of November 8, 2024, 4 Session Lead Lectures, and 14 Regular Lectures are expected to be delivered on December 6th at the conference. In addition, critical discussions on different procedures used to estimate rock mass strength and deformability will take place in the final session of the conference.

The valuable contributions made by the authors through their technical documents published in the conference proceedings are very much appreciated. Your registration for the conference and participation at the conference are highly valued because without those it would not have been possible to hold this conference. The tiring and time-consuming efforts of the local organizing committee and the conference dissemination activity performed by the international organizing committee in supporting and executing this conference are very much appreciated.

The Sri Lankan Rock Mechanics and Engineering Society along with the Peruvian Society of Geoengineering invite you to participate in the First ISRM Commission Conference on Estimation of Rock Mass Strength and Deformability to be held in Lima, Peru on December 6, 2024. I hope you will be able to contribute to the subject matter as well as improve your knowledge of the conference topic by participating in the conference.

Conference Chair | 06 December 2024

Prof. Pinnaduwa HSW Kulatilake

PhD, P.E. (Civil Eng.), F. ASCE, Member SME, Member Eng. Geology

President, Sri Lankan Rock Mechanics & Engineering Society
Chair, ISRM Commission on Estimation of Rock Mass Strength and Deformability
Professor Emeritus, Department of Mining & Geological Engineering,
University of Arizona, USA.
Former Distinguished Professor of Rock Mechanics and Rock Engineering,
Jiangxi University of Science & Technology, China.

Message from the Editorial

We are pleased to present the proceedings of the 1st ISRM Commission International Conference dedicated for the Estimation of Rock Mass Strength and Deformability. This ISRM Specialized International Conference hosted at Lima, Peru, is a joint effort by the Peruvian Society of Geo-engineering (SPEG) and the Sri Lankan Rock Mechanics and Engineering Society (SLRMES). We are certain that this conference will provide valuable networking opportunity for insightful and productive discussions in the domain of rock mechanics and engineering.

The recently formed ISRM Commission on Estimation of Rock Mass Strength and Deformability facilitates this inaugural event dedicated for this unique domain of rock mechanics. The pioneering individuals have contributed to this volume of abstracts and their scholarly work has been included within the four technical sessions under the categories of session lead lectures and regular lectures. A total of 25 abstracts were received for the conference where 18 was selected to be presented as 4 session lead lectures and 14 regular lectures. The proceedings contain the detailed program including essential supplementary information in annexes, and additional details are available on the conference website.

I'm grateful to the conference delegates, authors, representatives of ISRM, SPEG, SLRMES, sponsors and well-wishers, for creating a remarkable opportunity in the field of rock mechanics to ignite discussion on a particular area of current interest. While acknowledging the contributions from local and international organizing committees to make this event a success, we invite the delegates comprised of researchers, practitioners, and rock mechanics enthusiasts to utilize this book of proceedings effectively as it provides insights into the latest developments in the domains of rock mass strength and their deformational characteristics.

Thank you for taking part of this momentous event and we hope the outcomes of this conference leads to fruitful collaborative ventures in future.

06 December 2024

Dr Chulantha Jayawardena | Editor, Treasurer SLRMES

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Extended Abstracts of the Session Lead Lectures

A Review on Estimation of Rock Mass Deformability Properties Using Empirical Methods

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Introduction and Motivation

The estimation of rock deformability properties of jointed rock masses is fundamental for the preliminary design of rock slopes, rock foundations, and underground excavations in rock masses. Rock mass property values are also important input parameters in the analysis of rock mass behavior using computational methods including numerical modeling.

Direct methods, i.e., in situ tests on field rock masses or laboratory tests on large rock blocks, are either time-consuming, expensive, and difficult to implement, or inadequate to represent the various joint patterns and joint conditions that exist in field rock masses. In contrast, indirect methods, including back analysis, numerical modeling, and empirical methods, are more implementable, economical, and effective. The former two indirect methods require data, such as monitored field deformations and/or stresses, strength, and deformability of intact rock and joints, information on joint geometry distributions, etc., which are difficult to obtain at the preliminary design stage of a project. Relating deformability properties to rock mass classification parameters and/or other rock properties through empirical methods is relatively easy and popular among practicing engineers. However, studies show limitations of existing empirical equations for the estimation of rock mass deformability. Therefore, the identification of various empirical equations, their applicable conditions, as well as strengths and limitations, is critical to the correct use of this method.

Objectives of the Study

A list of existing empirical equations available in the literature to estimate rock mass deformability will be provided. The strengths and limitations of using empirical methods to estimate rock mass deformability properties, and the applicable conditions of each equation will be discussed. Also, suggestions and comments are given when applying empirical methods to a specific project.

Methods & Procedures

Various empirical equations are compiled from the literature. Furthermore, they are classified based on the factors, i.e., rock mass classification types, to which the rock mass deformation modulus is related.

Results

A large number of empirical equations have been proposed in the literature to estimate rock mass deformation modulus. Some of the studies used simple regression analysis, while others applied more advanced approaches such as neural networks, nero-fuzzy modeling, Bayesian method, support vector regression, etc. Concerning the related factors, rock mass classification systems, i.e., Rock Mass Rating (RMR), Tunneling Quality Index (Q), Geological Strength Index (GSI), rock quality designation (RQD), and Rock Mass index (RMi), were normally utilized since they are the first-hand information available for a project. The existing equations and their applicability are summarized. Also, the strengths and limitations of empirical methods are analyzed. Suggestions are finally provided on the estimation of rock mass deformation modulus using empirical methods.

Conclusions

The empirical method is an alternative and simple way to estimate the rock mass properties. Nevertheless, this method has some limitations. First, since the empirical relations were developed based on in situ data, the volume and reliability of the databases play significant roles. In cases where the data set is reliable, it cannot cover all kinds of rock types or rock mass conditions. As a result, the proposed empirical equations have limited application to specific projects or a certain range of rock mass conditions. Moreover, all empirical methods assume that the rock mass is isotropic and most of them have not taken into account the scale effect. It is also known that rock mass deformation modulus will be affected by confining stress. Although the effect of confining stress or depth on rock mass deformation modulus has been incorporated by some studies, the application of these equations should be verified with more data. Additionally, some single factors (e.g. RQD value) may be inadequate to characterize the rock mass quality, and subjectivity can be involved in determining the values of the rock mass classification system.

Therefore, users should be aware of the limits of existing empirical relations, select appropriate equations by analyzing the rock mass conditions of the site, and use more than one classification system. Once more data are collected, the results estimated from empirical methods should be validated using other sophisticated methods such as field instrumentation, numerical modeling, or back analysis.

Estimation of Rock Mass Strength and Deformability Properties in Three Dimensions Based on a Numerical Modeling Procedure

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Introduction and Motivation

Most naturally occurring discontinuous rock masses comprise intact rock interspaced with different types of discontinuities. Such discontinuities include fissures, fractures, joints, faults, bedding planes, shear zones, and dykes. Discontinuities may be divided into major and minor depending on the feature size. Large features can be considered as major discontinuities and the rest of the small features can be considered as minor discontinuities. For most of the civil and mining engineering projects, at the uppermost level, rock masses contain only a few major discontinuities. For such projects, major discontinuities can be considered as single features and their geometry may be represented deterministically. On the other hand, due to its large number and inherent statistical nature, the geometry of minor discontinuities should be characterized statistically. Henceforth, the minor discontinuities are referred to as either "joints" or "fractures" in this abstract. Rock mass strength and deformability depend on (a) rock mass geology/lithology, (b) the rock discontinuity network, (c) the geomechanical properties of the discontinuities, (d) the geomechanical properties of the intact rock, (e) the in situ stress system, (f) groundwater conditions and (g) the loading/unloading stress paths. A good understanding of rock mass strength and deformability is vital to arrive at safe and economical designs for structures built in and on rock masses. The presence of complicated discontinuity patterns, the inherent statistical nature of their geometrical parameters, and the uncertainties involved in the estimation of their geomechanical and geometrical properties and in-situ stress make accurate prediction of rock mass strength and deformability difficult. Due to the presence of discontinuities, discontinuous rock masses show scale (size) dependent and anisotropic properties.

Currently, the ways to estimate strength and deformability properties of jointed rock masses can be categorized into direct and indirect methods. Direct methods include laboratory and in situ tests. Laboratory results obtained from small-sized specimens that include only micro-joints are very different from the results obtained from large-scale blocks because the laboratory samples cannot accommodate the whole spectrum of different-size discontinuity networks which is present in the field. Many in situ tests have been performed to study the effect of size on rock mass strength and deformability. The results of these investigations show the reduction of rock mass strength and increase of rock mass deformability with increasing size up to a certain size known as the "Representative Elementary Volume" (REV) size, beyond which change becomes insignificant from a practical point of view. It is important to note that the relations developed from in situ tests in the above-stated studies primarily depend on the discontinuity network of the tested rock masses and are highly site-dependent. However, in these early investigations, no attempt had been made to map the discontinuity network before subjecting the rock mass to mechanical behavior testing. In addition, highly simplified rock mass models have been assumed in estimating rock mass deformability parameters from the in-situ test results. Therefore, these results have only qualitative value.

An indirect approach to obtain estimates of the strength and deformability of a jointed rock mass is by empirical correlation. In this approach, rock mass properties are linked to a representative rock mass classification index which reflects rock mass quality. All the available rock mass classification indices are scalars. Therefore, they do not have the capability of capturing the anisotropic, scale-dependent rock mass mechanical properties resulting from the discontinuity network present in the considered rock mass.

The second indirect approach termed as analytical decomposition technique treats jointed rock masses as a combination of two portions-intact rock and joints-and derives the global deformation moduli (or compliances) for rock masses by considering the load-deformation relation for each comprising material and assuming that the global behavior of the jointed rock mass is the summation of each component response. In these derivations, simplified fracture systems have been used with infinite persistence, constant deterministic spacing, and deterministic orientations for joint sets. In reality, joints are of finite size and all the joint geometry parameters are inherently statistical. Also, the interactions between joints are not considered in this method. Therefore, the assumptions made in these models are quite difficult to satisfy for most in situ rock masses.

The third indirect approach available is the numerical decomposition technique. This technique calculates the strength and deformability of rock masses using a numerical method, incorporating the strength and deformability properties of the joints and the intact rock. This technique allows the incorporation of any joint network to the jointed rock mass and also includes interaction between joints and intact rock. A pioneering scale effect research performed using this approach at the three-dimensional (3-D) level using the distinct element method has shown anisotropic, scale-dependent mechanical behavior for jointed rock masses. In the said study, detailed investigations have been performed to investigate the effect of finite-size joint geometry networks on the deformability and strength of jointed rock blocks, REV size, and equivalent continuum behavior at the 3-D level. Also, an incrementally linear elastic, orthotropic constitutive model has been suggested at the 3-D level to represent the pre-failure mechanical behavior of jointed rock blocks. In that model, the effect of the joint geometry network in the rock mass is incorporated in terms of fracture tensor components which captures the effect of all the joint geometry parameters- the number of joint sets, joint density, and statistical distributions for joint orientation and size.

Objectives of the Study

The objective of this presentation is to show how a procedure similar to the aforementioned third indirect method was applied at the 3-D level to study the REV sizes for the fracture network, strength and deformability properties, and equivalent continuum behavior for a rock mass at the Yujian River Dam site at a more refined level. Also, it is shown how an incrementally linear elastic, orthotropic constitutive model is developed to represent the pre-failure mechanical behavior of the jointed rock mass at a refined level.

Methods & Procedures

Based on the procedures developed by the first author of this abstract, fracture data available for limestone rock mass in the dam site of Yujian River Reservoir were used to build and validate a stochastic 3-D fracture network model, and to perform a REV and equivalent continuum study in 3-D. The fracture tensor was used to combine the effect of the number of fracture sets, intensity, and statistical distributions of orientation and size of the fracture sets. By using a 3-D distinct element modeling technique, several relations were developed between the rock mass mechanical parameters and fracture tensor components in 3-D.

Results

The developed fracture network model in 3-D was successfully validated. For all practical purposes, a block size of about 25-35 m and a size of 25 m was found to be suitable to represent the REV behavior for the fracture system and mechanical properties, respectively. The following relations are presented between the rock mass strength/ deformability parameters and fracture tensor components in 3-D: (a) Rock mass strength vs the summation of the fracture tensor components in the two perpendicular directions to the loading direction; (b) Rock mass deformability modulus vs the fracture tensor component in the same direction; (c) Rock mass shear modulus vs the summation of the fracture tensor components on the same two planes; (d) Rock mass bulk modulus vs the first invariant of fracture tensor. The principal parameter values, principal directions, and tensors are developed for rock mass deformability parameters and strength to represent the REV block size properties. An incrementally linear elastic, orthotropic constitutive model is suggested to represent the equivalent continuum pre-failure mechanical behavior of the jointed rock mass by incorporating the effect of the joint geometry network by the fracture tensor components.

Conclusions

The rock mass around the diversion tunnel can be represented through a combination of the aforementioned developed equivalent continuum material having the orthotropic constitutive model and the discrete fractures resulting from the bedding planes (fifth fracture set in the rock mass). Such a rock mass system can be used to evaluate the stability of the diversion tunnel.

A 3-D Strength Criterion Developed for Jointed Coal Masses Based on True Triaxial Tests and 3-DEC Numerical Modeling

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Introduction and Motivation

Most naturally occurring discontinuous rock masses comprise intact rock interspaced with different types of discontinuities. Due to the presence of discontinuities, the geomechanical response of discontinuous rock masses can be highly complicated under complex geology and in-situ stress systems. Therefore, civil and mining engineers face many difficulties in tackling design and construction tasks associated with geotechnical systems that are in or on discontinuous rock masses. To arrive at a safe and economical design, it is very important to have a good understanding of the rock mass strength. The presence of complicated fracture networks, the inherent statistical nature of their geometrical parameters, and the uncertainties involved in the estimation of their geometrical and geo-mechanical properties and in-situ stress make accurate prediction of rock mass strength a very difficult task.

Objectives of the Study

Because of the pre-existing fracture network systems, discontinuous rock masses show a very significant scale effect and anisotropic behavior. This research aims to develop a new 3-D coal mass strength criterion incorporating the influence of pre-existing fracture networks and confining stresses. This criterion should also be capable of capturing the scale effect and anisotropic behavior of coal masses.

Methods & Procedures

The research procedures used in this research can be described as follows:

a) Geomechanical property tests for coal matrix and coal discontinuities were performed in the laboratory to estimate the mean, standard deviation, and coefficient of variation of the geo-mechanical properties of coal matrix and coal discontinuities;

b) CT scanning tests for 110 mm side dimension coal cubes were conducted to detect the pre-existing fracture networks (bedding planes, face cleats, and butt cleats) inside each cubic coal block;

c) The pre-existing fracture network that exists inside each coal cube was constructed by a systematic computational procedure and quantified by the fracture tensor-based methodology;

d) The same cubic coal samples were subjected to true triaxial tests under different confining stress combinations to obtain the jointed coal mass strength (JCMS)L values under laboratory conditions;

e) Numerical models based on the discrete element method were established to simulate the laboratory true triaxial tests;
f) The numerical model with validated parameter values was used to run more numerical calculations for seven coal blocks having different fracture network setups and five other numerically created coal blocks having artificial fracture network setups under different confining stress combinations;
g) The data bank of JCMS built up through the aforementioned calculations was then used to develop a new 3-D coal mass strength criterion by applying multiple regression analysis.

Results

The new 3-D coal mass strength criterion can be expressed through the following equations:

$$\frac{JCMS}{CMS} = exp\left(-\omega\left(F_{xx} + F_{yy}\right)\right) \qquad \text{(Equation 1)}$$

$$\frac{\omega}{\omega_0} = \frac{1}{a(\sigma_x/CMS_u)^b + c(\sigma_y/CMS_u)^d + 1} \qquad \text{(Equation 2)}$$

In equation 1, *CMS* is the intact coal strength under the applied confining stresses σ_x and σ_y, where x and y are two principal perpendicular directions for the fracture system. F_{xx} and F_{yy} are fracture tensor components in the x and y directions. In equation 2, CMS_u is the uniaxial intact coal strength. The following regression coefficients were obtained for equation 2: a = 2.830, b = 0.902, c=1.586 and d=1.476. The obtained R-value of 0.956 in the regression analysis, indicates an excellent fit. The developed new 3-D coal mass strength criterion describes the correlation of the *JCMS* with the fracture tensor components and the confining stresses in the x and y principal directions. Note that the *JCMS* is obtained in the third principal direction of the fracture system, which is perpendicular to both the x and y directions. The new strength criterion includes the important fracture geometry parameters (3-D intensity and the probability distributions of the orientation and size) explicitly in the expression. Since the fracture tensor is a dimensionless parameter that can be estimated for any size of a rock mass, the new coal mass strength criterion can capture the anisotropic strength behavior and scale effect.

Conclusions

The following conclusions can be made from the conducted study:
a) When the summation of the fracture tensor components in the two perpendicular directions to the loading direction is more or less the same, the higher the confining stress system, the higher the cubic coal block strength in the loading direction;
b) When the confining stress system is the same, the higher the summation of the fracture tensor components in the two perpendicular directions to the loading direction, the lower the cubic coal block strength;
c) The new 3-D coal mass strength criterion is capable of capturing the scale effect and anisotropic behaviors of the coal masses. It can be applied to any rock mass by setting up three perpendicular principal directions to capture the influence of the existing fracture network in the rock mass.

Estimation of Rock Mass Properties for Mine Tunnels Based on a 3-D Discontinuum-Equivalent Continuum Back Analysis Method Using Field Deformation Data

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Introduction and Motivation

An imperative task for successful underground mining is to ensure the stability of underground structures. This is more so for deep excavations which may be under significantly high stresses. One of the issues that arise is how to estimate rock mass mechanical properties to investigate the stability of underground rock structures. One possibility is to estimate rock mass mechanical properties based on back analysis procedures incorporating field-monitored deformation data. Such an approach is described in this extended abstract.

Objectives of the Study

The objective of the presentation is to illustrate how a three-dimensional back analysis procedure was applied based on the distinct element method to estimate rock mass mechanical properties for two tunnels, in a deep coal mine in China, using the available field deformation data. The back-analysis procedure also included a pseudo-time-dependent support installation routine which incorporated the effect of time through a stress-relaxation mechanism.

Methods & Procedures

Two deep tunnels, a horseshoe-shaped and an inverted arch-shaped tunnel, in a coal-measure stratum, under a high in-situ stress field, have been modelled using the distinct element method. Available data on in-situ stress, tunnel geometry, joint orientation, joint and intact rock mechanical properties, and support properties have been fully incorporated into the model to back-analyze the rock mass mechanical properties using available field deformation data. Two methods of support installation have been used in the model calibration; one by immediately installing the supports after excavation and another by installing supports after allowing the rock mass around the excavation to undergo some stress relaxation as is the case in the field. The calibrated models have been applied for different supported and unsupported cases to estimate the significance and adequacy of the current supports being used in the mine, and to suggest a possible optimization.

Results

The back-analysis indicates that the rock mass cohesion, tensile strength, uniaxial compressive strength, and elastic modulus values are about 35%-45% of the corresponding intact rock property values. Additionally, the importance of incorporating stress relaxation before support installation has been illustrated through the increased support factor of safety and reduced grout failures. The effects of supports have been demonstrated using deformations and yield zones around the tunnels, and average factors of safety and grout failures of the supports. Since the fractured zones around tunnels were similar in size to the supports used, different models were investigated by installing longer bolts of 3.5 m and 4 m as opposed to the currently used 2.2 m and 2.5 m long bolts. This resulted in improved factors of safety and lower grout failure. Finally, the comparative study performed between the two tunnel shapes using the horizontal and vertical closure strains revealed that the horseshoe tunnel had higher vertical closure strains and comparable horizontal closure strains with the inverted arch tunnel.

Conclusions

Both methods of support point to a rock mass strength of about 35%-45% of the intact rock strength. The two methods of support installation have been observed to yield very similar deformations, but the support factor of safety is higher, and grout failures are lower when the stress relaxation-based method is used. It has been observed that the tunnel is most stable and suffers minimum deformations when supports are installed on the floor as well, in addition to the roof and walls. The use of longer supports and floor bolting has provided greater stability for the rock masses around the tunnels. Finally, a comparison between the two differently shaped tunnels establishes that the inverted arch tunnel may be more efficient in reducing roof sag and floor heave for the existing geo-mining conditions.

Extended Abstracts of the Regular Lectures

Size-dependent Behaviour of Artificially Jointed Hard Rock

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Introduction and Motivation

The design of structures on or within rock masses requires an estimate of the strength of the intact rock blocks and jointed rocks within the mass. These blocks of rock can be many orders of magnitude greater in size than laboratory samples typically tested. The properties obtained from these samples must therefore be 'scaled' to equivalent field values.

Objectives of the Study

The study aimed to have an extensive testing program on jointed rocks with different sizes through a set of laboratory tests on artificially jointed Blanco Mera granite under uniaxial and triaxial conditions.

Methods & Procedures

For the triaxial compressive tests, the maximum confining pressure of 15 MPa was accommodated and for the samples' sizes, three different diameters including 38, 54, and 84 mm were utilised. The resulting stress-strain curves were used to estimate the mechanical properties of tested samples including variable elastic moduli and Poisson's ratio.

Results

The resulting data was used to calibrate the Hoek and Brown rock mass parameters including the Geological Strength Index (GSI). This is the first study of its kind where it proposes a method for GSI estimation using the lab data through an objective method and then scaling up the data to the field setting.

Conclusions

Specimen size does seem to affect strength and deformability results not only in intact rocks but also in jointed specimens. These effects seem to be particularly relevant for peak strength and less relevant for the elastic parameters. The level of jointing affects the stress-strain response of specimens much more significantly than specimen size itself, which at the engineering scale justifies the typical practices of rock mass characterization based on the Hoek-Brown failure criterion and elastic modulus computation using GSI or other rock mass classification systems.

Comparison of Quantitative GSI Methods for Hoek and Brown Failure Criteria: A Case Study of El Teniente Mine, Chile

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Introduction and Motivation

The Geological Strength Index (GSI) system was created to enhance the Hoek and Brown (1980) criterion by addressing factors that impact the mechanical properties of rock masses, particularly the structure and condition of discontinuities. While GSI has been crucial in rock mass characterization, its application by non-geological personnel can lead to errors, highlighting its limitations. This has spurred interest in developing more quantitative GSI estimation methods, though uncertainties remain about their effects on the Hoek-Brown failure criterion and rock mass stability. This research seeks to address these uncertainties by evaluating and comparing prominent GSI methods to assess their impact on stability analysis and their reliability across different geological contexts.

Objectives of the Study

This study aims to compare various methods of GSI quantification. This comparison encompasses both the obtained values and the resulting stability analysis when these methods are applied within the framework of the Hoek-Brown failure criterion. The evaluation is carried out by contrasting the theoretical stability with that observed in a specific case study, allowing for an understanding of the impact of GSI on the stability estimation of rock masses according to this criterion. While this research guides selecting among these quantitative GSI methods, it does not aim to determine which is the best, as the final choice depends on the specific situation and the reader's judgment.

Methods & Procedures

Figure 1 illustrates the study's methodology, using two colors to differentiate the processes: orange indicates prior data used as input, including the creation of Digital Terrain Models (DTM) and the acquisition of structural data through the 2DM Analyst. Cyan represents the specific steps of the study, such as GSI estimation and the construction of failure envelopes with a 0.5 perturbation, allowing for a comparison between empirical stability and that observed in the field.

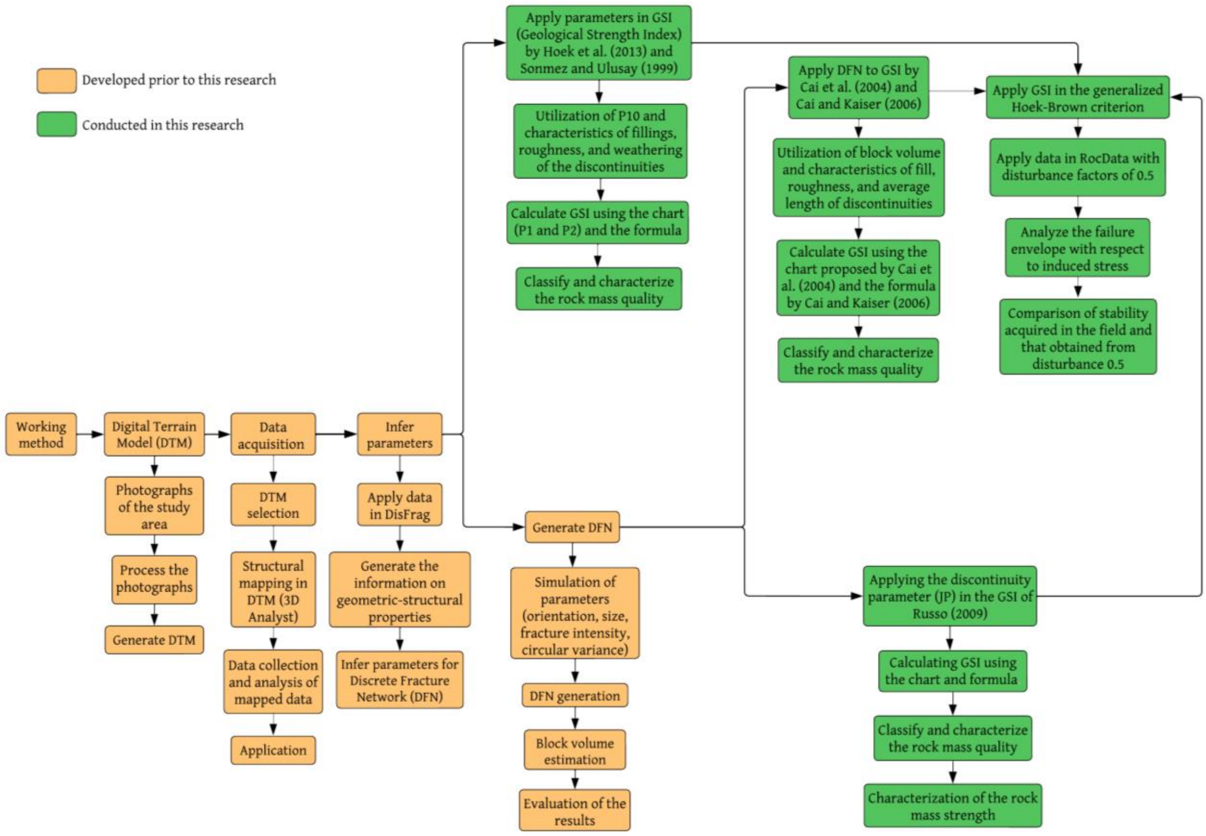

Figure 1: Methodology employed in the research.

Results

The stability analysis using the GSI method of Sonmez and Ulusay (1999) revealed that only 13.46% of the evaluated sections did not match the observed field results, with some unstable sections not assessed due to the lack of a failure envelope. In contrast, when applying the GSI method by Cai and collaborators, it was found that between 35.18% and 50% of the sections matched the observed stability, depending on the formula used. On the other hand, Russo's method showed a 14.8% match in stability prediction, similar to the results obtained using the proposal by Hoek and collaborators, which coincided with 14.82% of the case study's observed stability. These results highlight significant variations among GSI quantification methods and their impact on the evaluation of rock mass stability.

Conclusions

The quantification of the GSI system significantly influences the stability analysis of rock masses according to the Hoek-Brown criterion, which can lead to divergent conclusions among different authors. For instance, while the Sonmez and Ulusay method classifies a section as unstable, other methods consider it stable, as shown in Figure 2. These discrepancies underscore the need to update classification systems to adapt to the current conditions of digitalization and automation in Industry 4.0, ensuring that evaluation approaches remain effective and relevant.

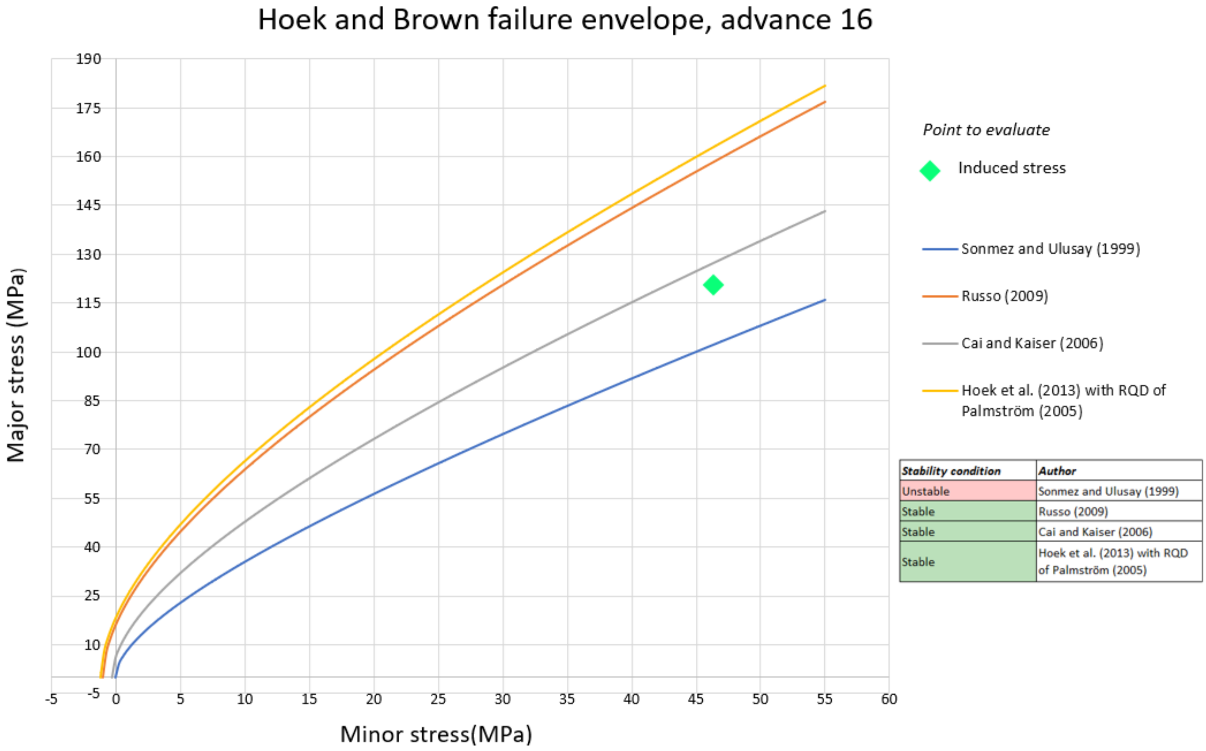

Figure 2: Failure envelopes constructed from GSI values obtained according to the author considered for the analysis of the advance 16.

Validation of Field Investigations of Rock Quality in the RMR89 and RMR76 Systems

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Introduction and Motivation

During the field data collection of rock quality, there exist multiple parameter valuation systems that influence the rock quality value. Such parameters are also multiple and vary in each valuation system, and in turn, each parameter is delimited in several degrees of both qualitative and quantitative characteristics. For example, the RMR76 or RMR89, both consider several parameters such as the resistance of the intact rock, the degree of fracturing, the spacing between discontinuities, the conditions of joints (roughness, opening, persistence, filling, and weathering), and the water content.

Objectives of the Study

Due to all the detailed characteristics to be taken of the rock masses, one comes across inaccuracies or errors in the recording of data by multiple factors. Therefore, it is not possible to arrive at the correct values of the characteristics of the rock mass. This leads to seeking corrections to all possible errors that can arise from different sources during the process of data acquisition, registration, and consolidation, in greater measure guiding us from the logs and mapping protocols used to acquire the data. In these errors, we can find problems in transcription, interpretation, perspective, and calculation. The detail of the transcribed characteristics should then be corroborated by the validation methods detailed below.

Methods & Procedures

The process begins with the consolidation of a database that includes records of parameters collected in the field, either from physical rock samples or geomechanical stations in the rock mass. It is ensured that the evaluation lengths are between 0.30 and 3.10 meters, thus avoiding point values derived from excessively short lengths or inadequate weighting in overly long ones. The investigations will be organized according to lithologies, and a review of the correlation between the RMR and its five parameters will be conducted, along with the percentage distribution by ranges of each of these parameters and the RMR based on each lithology.

An analysis of the correlation between RQD (%) and spacing (mm) will be carried out, recalculating these values through digital logging of boreholes and the collection of new data. Subsequently, the correlation between the RMR and its five parameters will be reviewed again. Inconsistencies in the joint condition parameter will be identified and corrected, ensuring proportionality between the characteristics of the discontinuities and the RMR ratios. Inconsistencies in the resistance parameter will also be addressed by correlating point load tests and uniaxial compressive strength tests to determine the conversion factor K. The obtained data will be used to validate and correct the manual strength indices according to the ISRM standards. Finally, the percentage distribution of the RMR in each lithology and its parameters will be reviewed, delineating subdomains and separating geomechanical units.

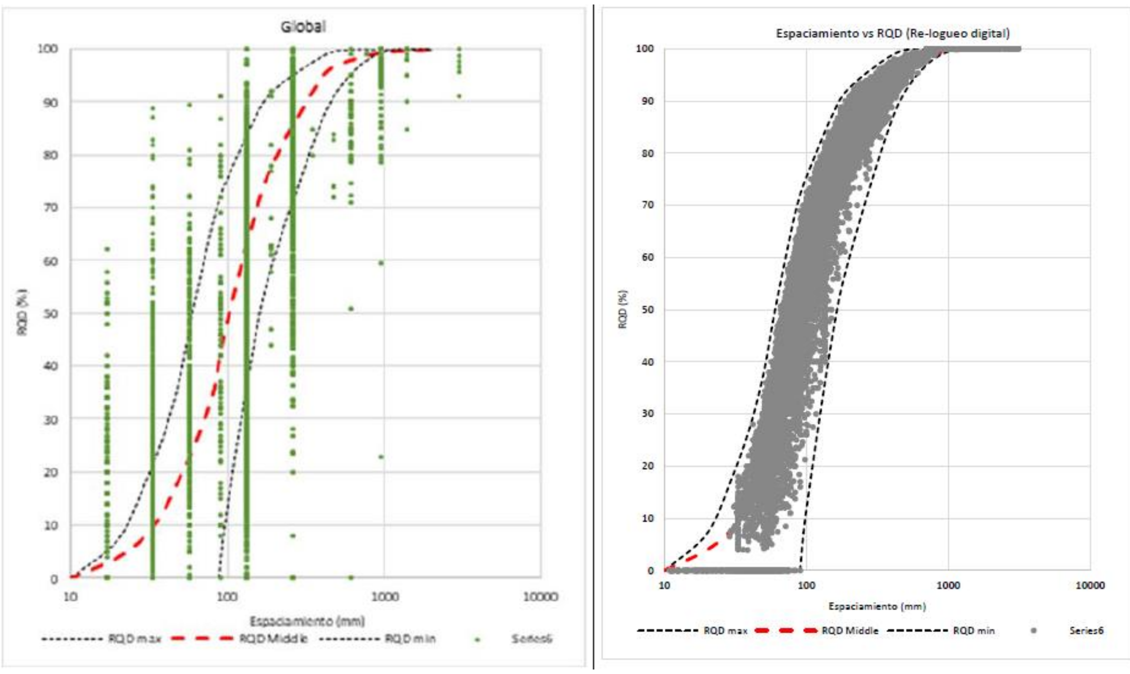

Figure 1: Validation of the RQD Parameter vs. Spacing

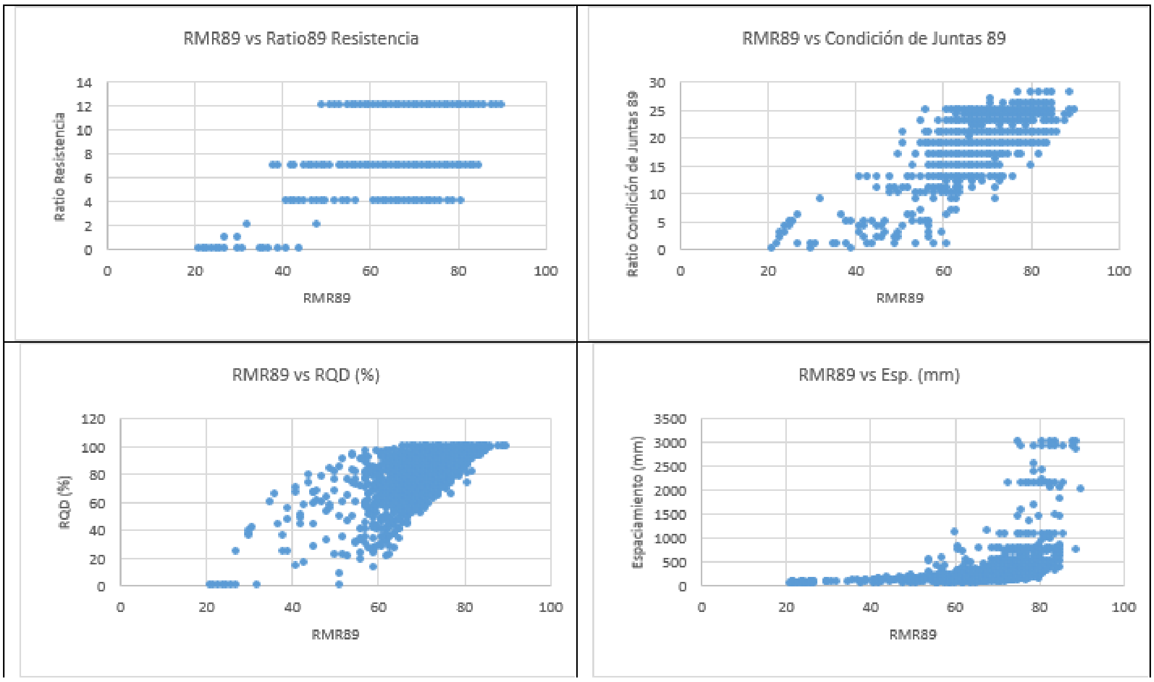

Figure 2: Correlation of RMR76 vs. RMR89 Parameters

Results

The graphs shown in Figures 1 and 2 are obtained and modified, reflecting updated data that will show a correct correlation. Figure 3 presents a summary of the procedure undertaken as part of the process to be followed for the current validation. Ultimately, this will lead to the delineation of subdomains by lithology and the initial separation of geomechanical units.

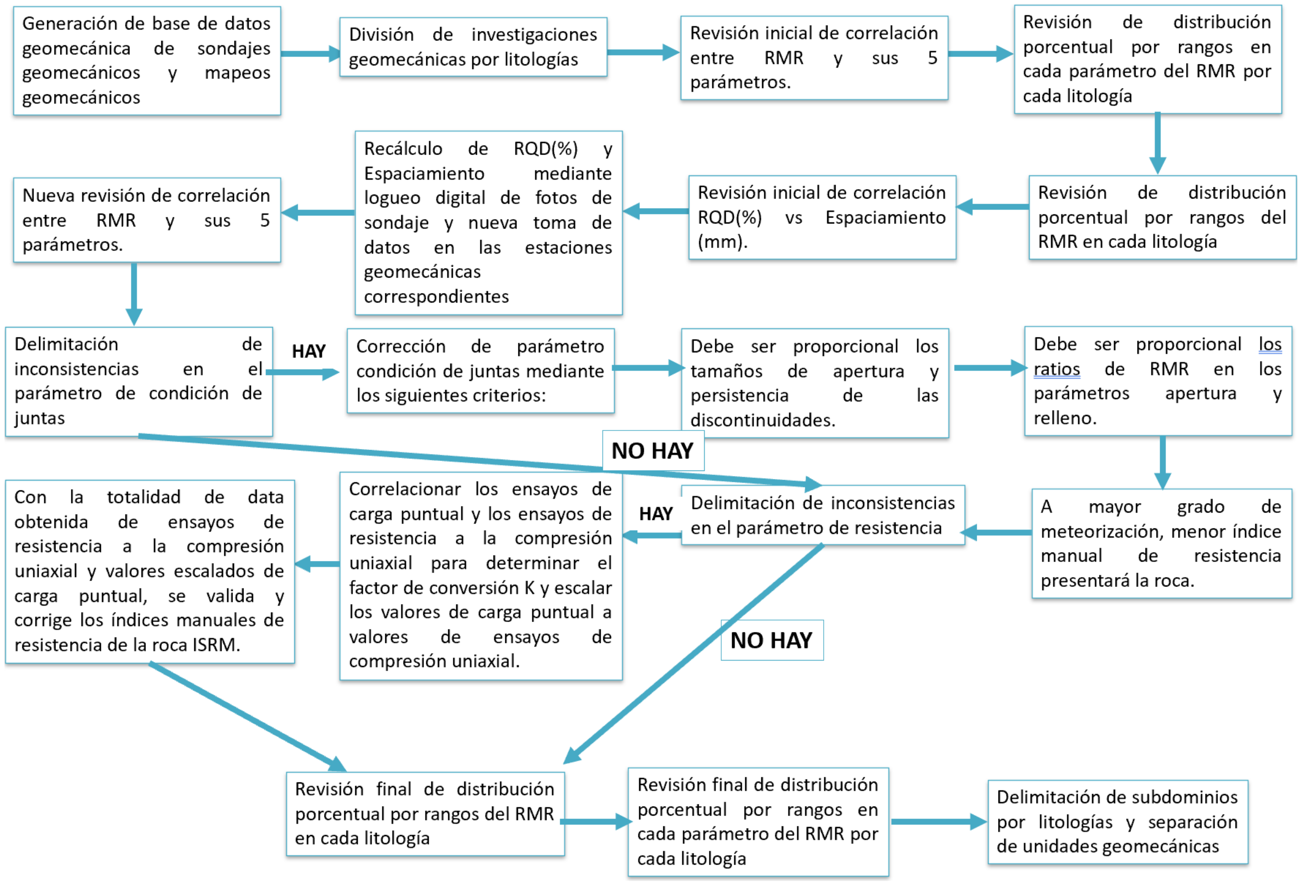

Figure 3: Validation Procedure

Conclusions

- All field investigation information on rock quality must be reviewed and validated through QA/QC processes.
- There are always erroneous data in rock-quality information databases.
- Erroneous data in geomechanical information databases should be minimized to enable more accurate assessments.
- Mapping and logging protocols provide a valuable foundation for data collection and future correction of recorded values.

Correlating Drill Core Morphology Data to Standardized Joint Roughness Conditions

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Introduction and Motivation

Field interpretation of joint roughness conditions (JRC) is usually based on mapping standardized surface roughness profiles to numerical values, which are then used in rock mass rating calculations and for other geotechnical modelling purposes. As this is a qualitative method of joint roughness characterization, results can be subjective and inconsistent between field personnel. This study introduces an automated method for quantifying JRC using LiDAR data to reduce the subjectivity and inconsistency associated with traditional field-based interpretations. The research involved scanning 650 meters of HQ diamond drill core with co-registered high-resolution colour photography, hyperspectral imagery, and 3D LiDAR data. A proprietary algorithm processes the LiDAR point clouds to detect fractures and calculate a best-fit plane, from which deviations are measured to determine joint roughness.

Joint roughness values were calculated for both the joint edge and face across 2,211 fractures. The joint roughness characterized by the joint face analysis is more comprehensive and generally exhibits greater roughness. These automated roughness values were compared to traditional JRC values obtained through visual inspection to validate the approach. Additionally, mineralogical data derived from hyperspectral imagery are integrated, enabling an analysis of the relationships between JRC, mineralogy, and fracture orientation.

Objectives of the Study

To provide joint roughness values with improved consistency and reduced bias for open fractures in the core through the automated processing of LiDAR data.

Methods & Procedures

High-resolution colour photography, hyperspectral imagery, and 3D LiDAR data were captured on 650m of HQ diamond drill core using Corescan's Hyperspectral Core Imaging system, model 4.2 (HCI-4.2). The HCI-4.2 is an advanced, automated scanning system that incorporates imaging sensors, sample handling systems, calibration samples, data acquisition control systems, data processing servers, and all related software. Sensor and data acquisition specifications for the HCI-4.2 system are detailed on the Corescan website: www.corescan.com.au.

Fractures in the core are automatically detected from LiDAR point clouds using a proprietary algorithm calibrated to find fractures as deviations in the surface profile of the core. Fracture surfaces are then represented as dense point clouds with 50 μm point spacing and 15 μm height resolution. A best-fit fracture plane is calculated from the point cloud and is used to determine the orientation of each fracture. The apparent joint roughness is obtained by calculating residuals of fracture coordinates and their calculated planes (Figure 1). This involves measuring the distance from each coordinate to the best-fit fracture plane. Finally, joint roughness is determined by taking the standard deviation of these residuals as shown in Equation 1, where xi is each distance measured between the fracture coordinate and fracture plane, μ is the average distance, and N is the number of distances (fracture coordinates) measured.

$$\text{Joint roughness} = \sqrt{\frac{\sum(x_i-\mu)^2}{N}} \qquad \text{(Equation 1)}$$

Equation 1 provides a numerical measure of joint roughness, in that a smaller standard deviation will indicate a smoother surface, while a larger standard deviation will indicate a rougher surface. Because the fracture face may not be visible for all fractures, joint roughness has been evaluated for both the fracture face and the fracture edge.

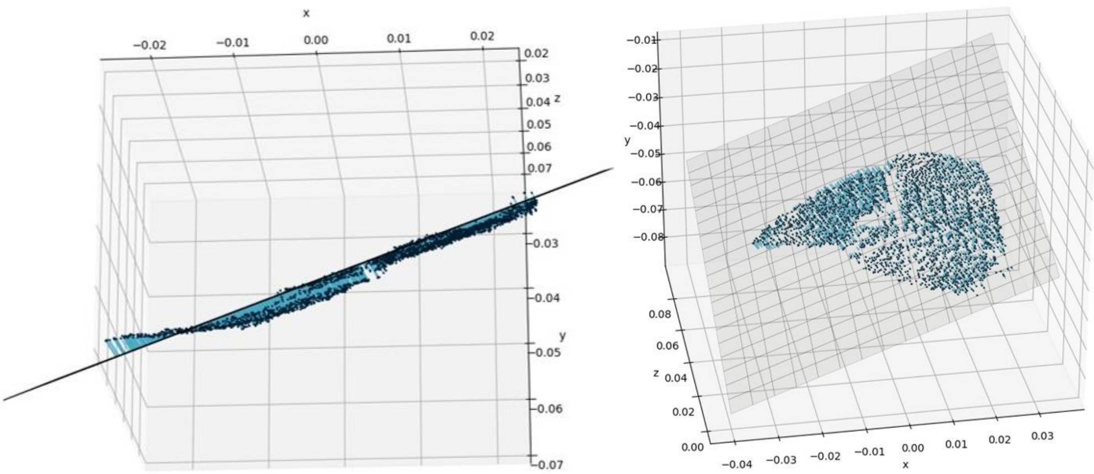

Figure 1: A 3D representation of the fracture coordinates (black points), and their residuals (blue vectors) from the calculated best-fit plane, with views along the fracture (left) and above the fracture (right). Axis units are in meters.

To validate the relationship between calculated joint roughness and JRC, roughness values for each fracture will be compared to JRC values obtained through visual-based, industry-standard methods. A mathematical function will be defined to map roughness values to JRC. The mineralogy of each fracture has been interpreted from the spectra in the hyperspectral data collected, which makes it possible to determine patterns between JRC, joint infill mineralogy, and joint orientation, thereby offering a comprehensive analysis of structures in the core.

Results

Numerical joint roughness values were calculated for both the joint edge and joint face for all 2,211 fractures in a 650m drillhole; Global statistics are shown in Table 1. The roughness magnitude is greater on the joint face than the joint edge, which is to be expected, as there are not only more fracture pixels included in the joint face calculation, but a more complete characterization of the true fracture roughness is possible. Preliminary results show a spread of roughness values through the dataset, indicating the potential for precision interpretation.

Table 1: Global statistics for joint roughness values calculated for all fractures. Values are in meters.

	Roughness (m) – Joint Edge	Roughness (m) – Joint Face
Maximum	0.007750	0.066043
Minimum	0.000042	0.000065
Average	0.000750	0.002141
Standard Deviation	0.000752	0.003260

Conclusions

Joint roughness results show that it is possible to characterize the roughness of joints consistently and efficiently through automated processing of LiDAR data. More research will be completed as outlined in the methods and procedures to relate the calculated roughness to observed JRC values and mineralogy.

Geological Survey of Rock Blocks from Chuquicamata Underground

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Introduction and Motivation

One of the many challenges that every Caving Mine should consider in its initial stage of production is the characterization of the rockmass dismantling process. To achieve this purpose, it is highly recommended the application of a methodology that implements a geological and geotechnical study of the rock fragments that reach the draw points. This work shows the results of a methodology applied in the Chuquicamata Underground Mine for the characterization of rock fragments.

Objectives of the Study

The main objective of this study is to characterize and understand some geological and geotechnical variables that play a role in the rockmass dismantling process exploited through a block-caving method with a macroblock variant.

Methods & Procedures

The procedure carried out consisted of a systematic mapping of the rock fragments extracted from the draw points of the Chuquicamata Underground mine. This process entails the acquisition of some geological and geotechnical parameters from the rock fragments, such as the classification of the basic geotechnical unit ("UGTB"), the measurement of the shape factor (Gy, 1967), a qualitative scale of size used depending on the largest axis, Fine (Axis ≤ 1 m), Moderate (1 < Axis ≤ 2), Coarse (2 < Axis ≤ 3), and Oversize (3 < Axis), and a detailed surveying of the rock's faces, which involves the identification of the type of discontinuity (joint, fault, vein, matrix, or a mixed kind), the roughness, and the infill mineral. Additionally, from each draw point, the column extraction height is registered, which helps to understand the state of evolution that has the Caving process.

Regarding the roughness, the JRC roughness scale from Barton and Choubey (1977) was applied. Concerning the infill mineral, a qualitative scale was assigned depending on the Mohs hardness Scale, these are, hard (5 < Mohs hardness), medium (3 ≤ Mohs ≤ 5), or soft (Mohs < 3). The range for each qualitative range was considered according to the recommendation of Russo, Vela & Hormazabal (2020) who mentioned that for hypogene environments, the minerals up to 5 Mohs hardness have the potential to open in mining activities such as blasting, induced stresses, Caving, etc.

Results

A total of 479 rock fragments were mapped between July 2019 and September 2023.

The different draw points ranged from 0 m to ~ 240 m of extraction column height. It was possible to obtain information from three different UGTBs, named QIS ("Cuarzo Igual Sericita"), PES ("Pórfido Este Sericítico"), and PEK ("Pórfido Este Potásico"), which are the main geotechnical units that contain the mineralized body. Concerning the Shape Factor, infill Minerals, and JRC Figure 1a, Figure 1b, and Figure 1c shows the results, respectively.

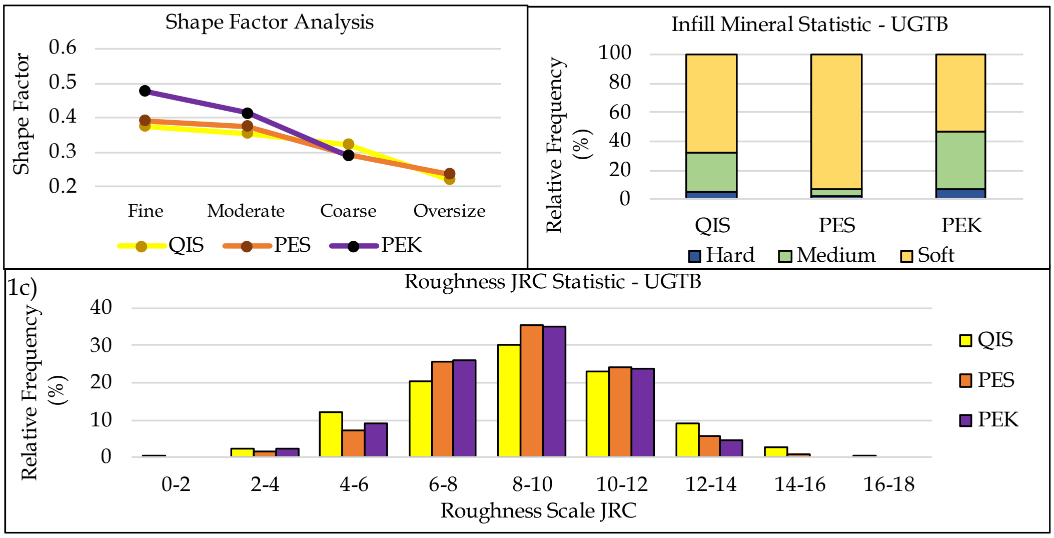
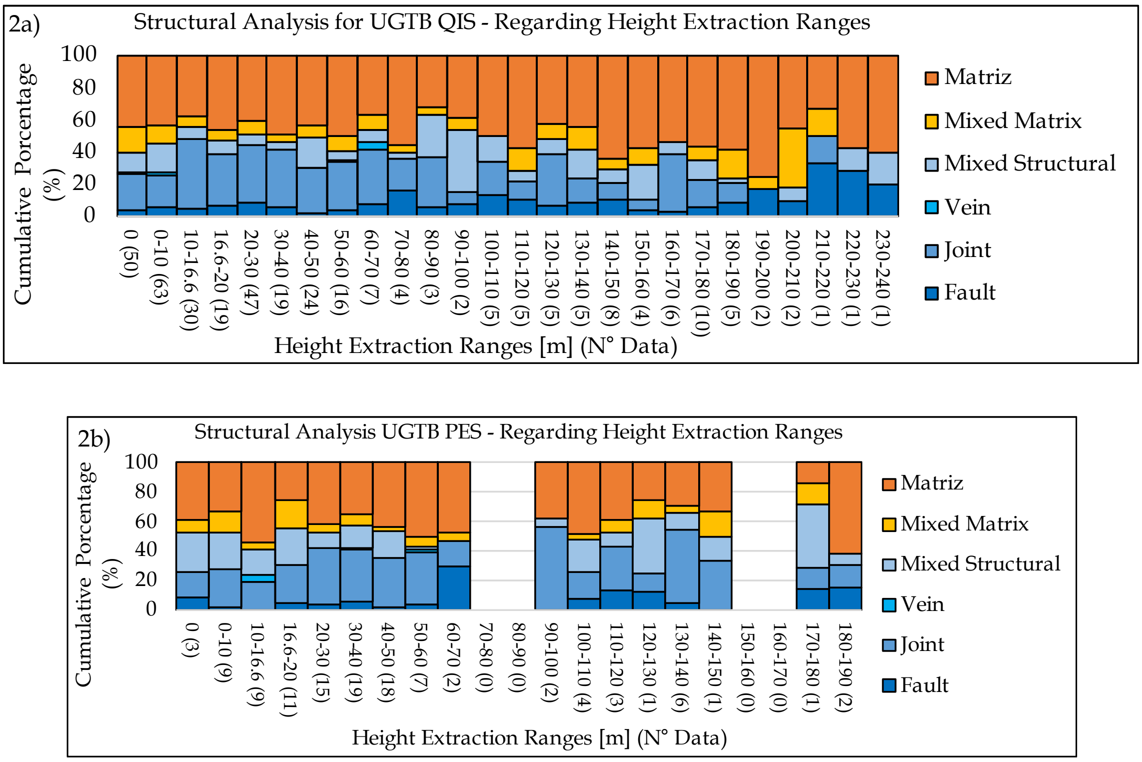


Figure 1: Results for a) Shape Factor, b) Infill Mineral, and c) JRC per UGTB.

Finally, a specific evaluation was conducted for the rock's face discontinuity type. The extraction column height was considered for each rock fragment, and an analysis of the evolution of rupture type through extraction was performed. Figures 2a-c show the relative frequency of the different discontinuities for QIS, PES, and PEK UGTB, respectively.



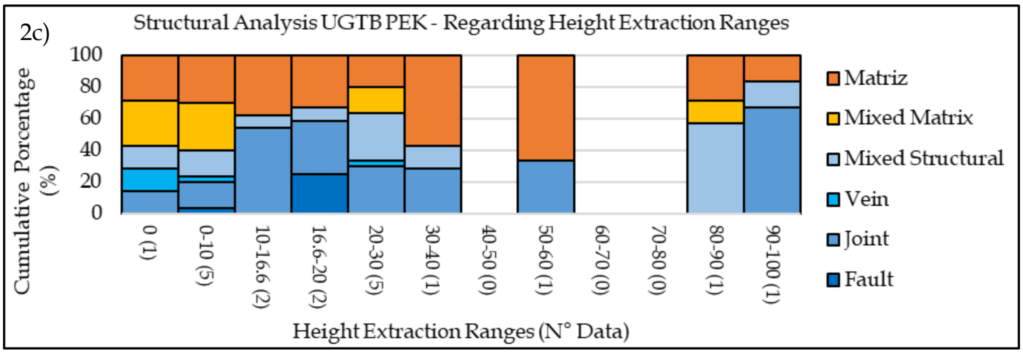


Figure 2: Structural analysis of rock fragments regarding height extraction for UGTB QIS, PES, and PEK.

Conclusions

Regarding the shape factor of fragments, there is a markable trend from sphere shapes for Fine Fragments, to more elongated tabular shapes for oversized boulders. This result could indicate a lower comminution process since the rock fragment separates from the rock mass. Meanwhile, in the analysis of JRC considering the type of discontinuity, there is a clear trend that indicates that surfaces with a structural control tend to be less rough, on the contrary, faces made from the breakage of rock bridges as matrix or mixed surfaces tend to be rougher. The analysis of infill minerals indicates that most of the discontinuities that activate in rock block formation tend to have infills of soft to moderate hardness minerals. However, a few percentages of these surfaces can also break through hard infill minerals.

Finally, regarding the structural analysis of surfaces, for the first meters of extraction, the predominant type of breakage is related to the matrix and mixed types, which can indicate a mechanical behavior related to the blasting process. Meanwhile, for higher extraction a decrease in mixed type and an increase in joint or fault types should be expected; this statement is fulfilled for UGTB PES and PEK, where stress produced for the caving process tends to activate the discontinuities of the rock mass, but for QIS the amount of matrix type stays relatively constant, which could be due to the low strength of this rock, and also, by the comminution process which take more relevance with the extraction process at higher heights. Comparing these results with the El Teniente mine, Chuquicamata showed a higher participation of matrix breakage, which can indicate lower strengths for Chuquicamata rocks. All these results should be considered in numerical modelling when it is performed because they could work as an input for back analysis. It is highly recommended to continue applying this methodology in other macroblocks for a better understanding of rockmass dismantling. Additionally, an analysis of the UCS and TX test for structural and matrix breakage control could complement the analysis of this work.

Cerchar Parameter Estimation as a Function of Equivalent Quartz Content and Properties of Porphyry Copper Igneous Rocks

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Introduction and Motivation

The excavation of future developments in the Undercut level in Macroblock N06 in Chuquicamata Underground mine will be done through a TBM (Tunnel Boring Machine). For this reason, it is essential to provide an abrasiveness estimation of the Basic Geotechnical Units (UGTB) from the sector.

At present to estimate the CERCHAR Abrasiveness Index (CAI) for the excavation project, neither exists a relation to estimate the preliminary CAI in the different rocks from the mine nor exists laboratory evaluations for the CAI. Two previous estimations made in Chuquicamata Underground Mine by Barindelli (2022) and Pereira & Divasto (2024) have been considered conservative due to the use of a model proposed by Plinninger (2003) which has included only two igneous rock samples with predominating samples of metamorphic and sedimentary rocks.

This work recommends a new model following the methodology proposed by Plinninger (2003) but considering a recompilation of data from El Teniente Mine and results from some laboratory tests carried out in Chuquicamata Underground Mine. This model is composed only of rock samples from Porphyry Copper Systems, which could be more representative of future abrasiveness estimations.

Objectives of the Study

The main objective of the study is to provide an estimation model for the CERCHAR Abrasiveness Index for igneous rock from Porphyry Copper Systems.

Methods & Procedures

The first approach for this work involved a review of the technical literature available. This included a recompilation of data from El Teniente Mine which included the abrasiveness test results of igneous rock samples from Porphyry Systems. Additionally, in 2023 a few Abrasiveness CERCHAR tests were made on rocks from Chuquicamata Underground Mine extracted from boreholes near the study area. The tests were done by Derk's Laboratory using the ASTM standard test method (2010) designated as D7625-10. Finally, a model was obtained integrating all background information available for igneous rocks.

Results

A total of 149 data were considered for analysis. The analysis showed a good correlation between Young's Modulus and Quartz Equivalent content versus CAI values. The results are summarized in the next graph, where a linear model proposed in this work (Equation 1) can be seen, along with the model proposed by Plinninger et al., (2003) (Equation 2).

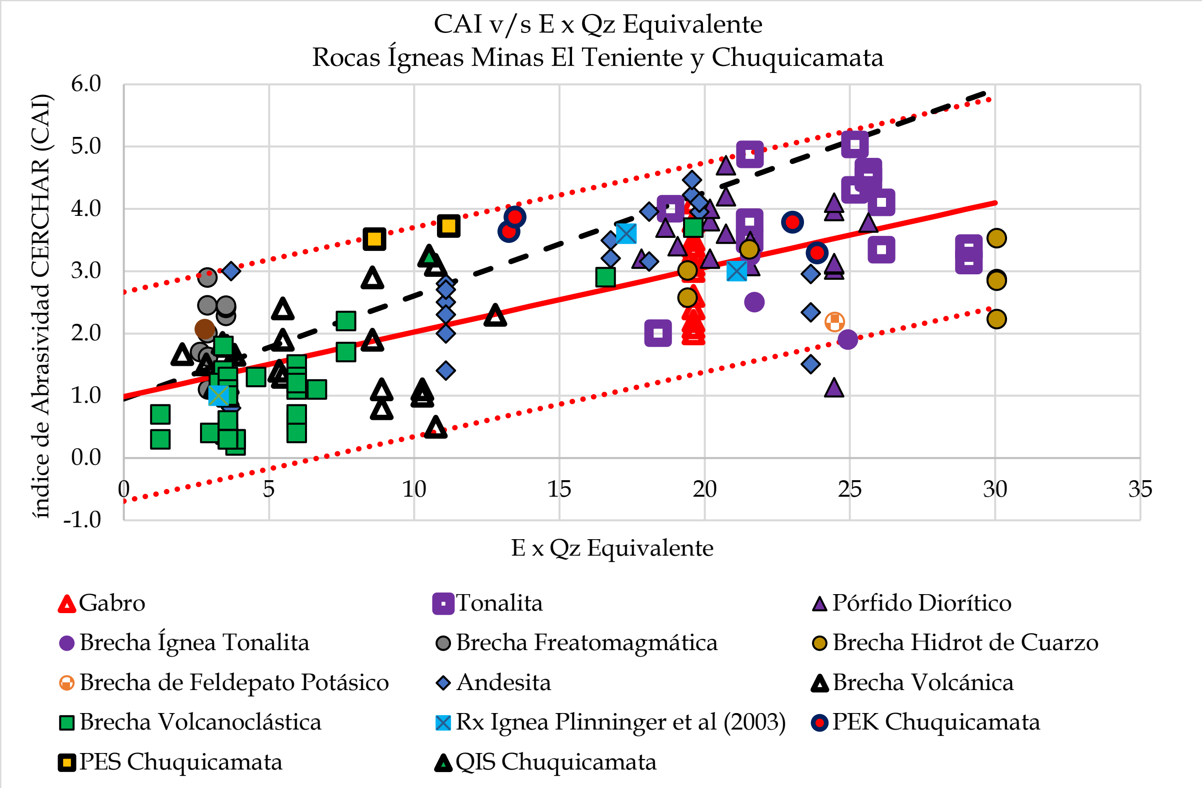

Figure 1: Integrated Graph for correlation between CAI and E modulus and quartz equivalent. The red continuous line indicates linear regression for all data. The red dotted lines indicate a probability interval with a 95% significance. The black dashed line indicates the Plinninger model.

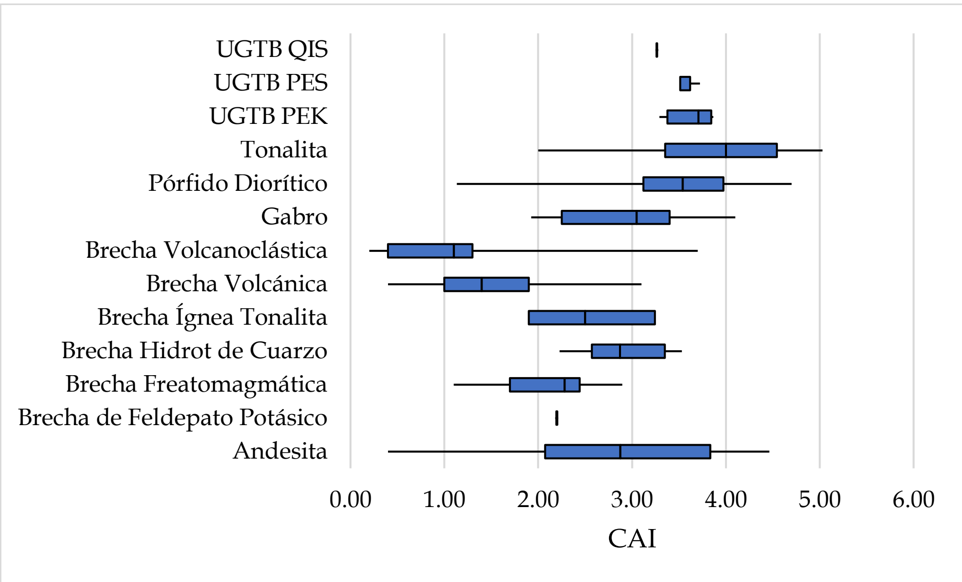

Figure 2: Compilation of typical CAI values for different rock types from Chuquicamata and El Teniente mines.

$$\text{CAI} = 0.1037 * E\ x\ QzEq + 0.986 \text{ (Equation 1)}$$

$$\text{CAI} = 0.16558 * E\ x\ QzEq + 0.95 \text{ (Equation 2)}$$

Conclusions

According to the results obtained, it is inferred that both for the samples from Chuquicamata and El Teniente Mines the Abrasiveness CERCHAR Index would be more influenced by the amount of quartz equivalent than the Young's modulus. The proposed model appears to fit better for igneous rocks than the model proposed by Plinninger et al., (2003) which had sedimentary and metamorphic rocks. From Figure 1 it can be observed that the CAI values measured for Chuquicamata rocks are comparable with the Pórfido Diorítico, Tonalita, and Gabro units from El Teniente Mine. For Chuquicamata Mine, the mean values of quartz equivalent and Young's modulus are 39% to 48% and 21 GPa to 42 GPa, respectively.

Influence of Weathering in the Definition of Geotechnical Units

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Introduction and Motivation
The most accepted approach to define basic geotechnical units, in the mining industry, is the one proposed by Flores and Karzulovic (2003) based on the superposition of lithology, alteration, and mineral zone, where the use of the latter is related to the differentiation between the environment of primary and secondary mineralization. Although this methodology may seem very easy to apply, the implementation in those mining projects where the mineral zones were defined according to geological or metallurgical criteria, that do not necessarily coincide with the geotechnical quality criteria, frequently does not provide a good distinction between the different geotechnical units.

Objectives of the Study
The purpose of this work is to propose a methodology to define the basic geotechnical units based on the combination of lithology, alteration, and the geotechnical zones instead of the mineral zones. In this study, the concept of the geotechnical zones based on the degree of weathering and the geotechnical quality of the rock mass is introduced.

The geotechnical zones defined correspond to (1) the secondary sensu stricto (2ºss) which is located in the shallowest part of the deposit and is recognized by intense leaching or oxidation affecting both the rock matrix and discontinuities, (2) the secondary transition (2ºtr) which is located below the 2ºss zone and where the effects of leaching or oxidation are slight in the rock matrix, concentrating mainly in the discontinuities and finally, (3) the primary geotechnical zone (1º) which is characterized by being an impermeable rock mass, fresh and unaltered from the point of view of weathering.

Methods & Procedures
The proposed methodology has been successfully applied in several mining projects and in this abstract, it is presented as a case study corresponding to a Cu-Mo porphyry copper-type deposit whose geology comprises a series of volcanic rocks and intrusions of mainly granodioritic composition. The predominant hydrothermal alterations in the porphyry system correspond to potassic and sericitic or phyllic quartz alterations, while, regarding mineralization, a primary or hypogene mineral zone is recognized, which underlies the secondary or supergene mineral zone of 80 to 120 m power. In the latter, three main zones are recognized: the leached zone, the oxidized zone, and the secondary enrichment zone.

42,776 m of drill cores were considered for the analysis whose geotechnical logging allowed for the calculation of the RMR index based on the geomechanical classification system

(Bieniawski, 1989), the GSI index (Hoek et al., 2013), RQD (Deere, 1963) and the fracture frequency per meter (FF/m).

The geotechnical characterization of the case study was carried out based on two scenarios. In the first one, the geotechnical units were defined by the superposition of lithology, alteration, and mineral zone, where the secondary mineral zone was defined based on the oxidized zone, leached zone, and secondary enrichment zone. The second scenario considered the superposition of lithology, alteration, and geotechnical zone for the definition of the geotechnical units. The values of each index and classification system correspond to averages weighted by the length of each mapping interval. The intact rock properties were estimated and the results for each geotechnical unit, defined with both methods, were compared.

Results

The comparison of the geotechnical characterization and the intact rock properties pointed out a better distinction of the geotechnical quality between the basic geotechnical units defined using the geotechnical zones. In addition, the use of geotechnical zones allows to distinguish between two different geotechnical units that are homogenized using the mineral zones.

Conclusions

The presented case study shows that the implementation of geotechnical zones, instead of mineral zones, allows for better differentiation of the qualities between primary and secondary rock mass, showing greater differences between the different geotechnical zones than between mineral zones.

The implementation of geotechnical zones for the definition of geotechnical units and subsequent geotechnical characterization has several advantages. Firstly, identification and recording are simple and can be done directly when carrying out geotechnical mapping, avoiding common confusion between geotechnical and mineralogical and/or metallurgical criteria that result when applying mineral zones. It better represents the vertical variability in the geotechnical quality of the rock mass given by the oxidizing conditions near the surface. Additionally, a better differentiation of intact rock properties between the three geotechnical zones has been observed for the same group of lithology and alteration. In particular, it has been observed that the use of the secondary mineral zone would group two types of geotechnical zones, the 2°ss, and the 2°tr, which are characterized by different intact rock properties.

Seismicity and Mining Operations in High-Stress Underground Environments

Tomás Roquer[1,2*], Daniel Melnick[3], Roberto Larregla[1,2], Guillermo Martínez[1,2], Javiera Tillería[1,2], Lucy Bravo[1,2], José Piquer[3], Jorge Torres[3], Luis Felipe Orellana[1,2]

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Introduction and Motivation

High-stress underground mining environments present unique challenges related to the stability and continuity of mining operations, which are significantly influenced by seismicity and topographic changes. This study focuses on the seismic activity within a world-class underground deposit, highlighting the occurrence of seismicity due to mining activities. Understanding these factors is critical for managing risks and ensuring the safety of operations in such high-stress environments.

Objectives of the Study

The objective of this study is to analyze the seismic activity within a high-stress underground mining environment.

Methods & Procedures

By examining seismic records, topographic changes, and the flexural response of the crust due to subsidence, the study aims to identify the factors that control the occurrence and distribution of seismicity and assess their implications for mining safety and efficiency. This study involves a detailed analysis of seismic records in conjunction with observations of topographic changes and the crustal flexural response associated with subsidence.

Results

The results reveal a complex interplay between mining operations and rock mass deformation, manifesting in distinct seismic regimes across different activities and depths:

- Low depth: Operation. The seismic activity is largely driven by shear and compressional fractures resulting from the extensional and contractional deformation fields induced by production mining operations. This seismicity is particularly associated with production and construction activities within the mine.
- Intermediate depth: Construction. Seismic events in this depth range show a prevalence of shear and compressional fractures, with the deformation likely linked to the ongoing development of mining infrastructure. The seismic patterns suggest an increased influence of mining-induced stresses.
- Deeper depth: undisturbed. The seismic regime is dominated by shear fractures that may represent reactivated faults deeper within the rock mass. The exact mechanisms behind

this deep seismicity are under study, but preliminary modeling indicates that isostatic rebound from surface subsidence could be a contributing factor.

Conclusions

This study underscores the importance of incorporating deep seismicity management into mining planning. The findings demonstrate that seismicity in high-stress underground mining environments is closely tied to mining operations. Understanding these relationships is crucial for enhancing operational safety and efficiency in deep mining settings.

Preliminary Analysis of Fault Reactivation in Underground Mining

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Introduction and Motivation

Due to the global increase in ore demand and the exhaustion of superficial resources, the underground mines progress to deeper levels, and therefore, mining infrastructure is subjected to progressively greater in situ stresses. In deep underground excavations, stability challenges are frequently posed by a surrounding zone of fractures. This issue is compounded by the presence of geological structures, such as faults, which can reactivate. Fault reactivation is related to different geological hazards, such as seismic events, subsidence, or rock bursts, which can affect the safety and continuity of underground mining activities.

Objectives of the Study

This study aims to better understand the conditions for fault reactivation in deep underground mines. Specifically, it analyzes parameters for the reactivation of geological faults in an underground world-class deposit using a slip tendency analysis. The study investigates the relationship between in situ stress fields, fault geometry, and rock mass properties to identify critical factors contributing to fault reactivation.

Methods & Procedures

The orientation of the principal stress axes ($\sigma_1 \geq \sigma_2 \geq \sigma_3$) and their relative magnitude ($\phi = (\sigma_2-\sigma_3)/(\sigma_1-\sigma_3)$) were obtained using the Multiple Inverse Method (MIM) with a seismic database. In addition, the absolute magnitudes of the principal stresses were obtained by non-Andersonian equations at distinct depths of interest in the mine. We used such a stress field to calculate fault slip tendency for identified faults using the FractTend software based on Morris's work, which considers the effects of in situ stresses, fault geometry, and rock mass properties, such as the friction coefficient μ. Based on the results obtained in the software, a criterion for fault reactivation was applied based on the distance between Mohr's circle and the fault envelope.

Results

The results obtained from slip tendency analysis reveal that the most stable state corresponds to the value of the differential stress σ_1-σ_3 equal to 1 MPa in all case analyses, where the Mohr's circle is far away from the failure envelope. With high values, near 15 MPa, an unstable state is presented, and Mohr's circle exceeds the fault envelope; even certain geological faults lie above the envelope, making them critical. In addition, in all cases that were analyzed, with values of friction coefficient μ close to 0.4 (lowest value considered in this study), the criticality of failure was higher compared to higher values of μ where failure could reach a much more stable state.

On the other hand, the fault slip tendency for values near 1 MPa is associated with a higher fault slip tendency, leading to a greater number of critical orientations. In contrast, higher differential stress values, around 15 MPa, result in a lower slip tendency, with fewer critical orientations observed.

Conclusions

Preliminary results indicate the criticality of faults due to stress magnitude increases with high differential stress σ_1-σ_3 ($\geq$ 10 MPa) and low values of μ ($\leq$ 0.6). In addition, the fault slip tendency increases with the decrease of differential stress (σ_1-σ_3), increasing critical orientations for fault reactivation. It is important to perform this type of analysis in deep areas, as it allows for anticipating and mitigating the risks associated with geological fault reactivation. These results contribute to a deeper understanding of the relationship between in situ stresses, fault zone geometry, and rock mass mechanical properties, which is key to improving risk management strategies in underground mining.

Impact of Topographic Deformation on Induced Seismicity and Reactivation of Geological Faults in Underground Mining

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Introduction and Motivation

This work focuses on the critical influence of mining production on topographic changes and how these, in turn, impact geomechanical conditions, particularly those favorable for fault reactivation under high-stress conditions. The study aims to explore the intricate interaction between mining production, topographic alterations, and the resulting deformation field changes during underground mining operations.

Objectives of the Study

The primary objective of this study is to employ numerical modeling to analyze the relationship between topographic changes, seismicity, geological faults, and mining productivity. By examining historical and current data from cave mining operations, the research seeks to understand how material extraction influences deformation fields and topographic changes, and how these factors subsequently affect mining structures and operations.

Methods & Procedures

A correlational modeling approach was used, incorporating seismicity studies to evaluate stress distribution in seismic events. These findings were integrated into 2D profiles, capturing both topographic behavior and stress conditions in seismicity. The study employs boundary conditions derived from observed deformation fields to enhance the accuracy of the numerical models. This methodology allows for the simulation of potentially vulnerable areas, including geological faults and lithological contacts, for improved safety assessments and mining planning.

Results

Initial analysis of the topographic data has revealed significant subsidence, with areas reaching up to 450 meters. Notable topographic reliefs are observed in the NE, E, and S sectors relative to the central mine area. Additionally, the deformation field related to seismicity, represented through the distribution of PTB axes, indicates a transition from an extensional regime up to 2013, followed by a notable increase in seismic events uniformly distributed across the entire spectrum of possible occurrences from 1992 to 2023.

Conclusions

The results underscore the significant impact of mining activities on natural behavior patterns, with clear evidence of how mining-induced changes alter topographic and seismic conditions over time. The findings highlight the importance of integrating these effects into long-term planning to ensure safer and more efficient mining operations. The numerical models developed in this study provide valuable insights for predicting future geomechanical behavior under varying stress conditions, contributing to better-informed decision-making in mining management.

Unveiling the Impact of Geological Structures on Seismic Activity in Large Underground Copper Mines

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Introduction and Motivation

Due to the increasing demand for copper, efforts are being made to extend the lifespan of underground mines by exploring new productive sectors located at greater depths compared to previously exploited areas. However, the interaction of geological faults with mining activity in high-stress environments creates potential scenarios for seismic reactivation. These events not only compromise the safety of personnel and infrastructure but also the fulfillment of the mining plan. The faults with the greatest potential to cause damage are characterized by their large dimensions, in length, thickness, and depth. Their reactivation can trigger rock bursts and induce significant seismic activity.

Objectives of the Study

Previous studies have shown that faults have an architecture characterized by a fault core and a surrounding damage zone. In natural environments, the fracture density and number of seismic events decrease from the fault core towards the damage zone, with deformations ranging from microfractures to networks of macroscopic fractures. The main objective of this work is to determine the seismic and structural influence of geological faults in the context of deep high-stress underground mining.

Methods & Procedures

Through the spatial study of fractures and seismic events, the MATLAB script FracPAQ is used to quantify fractures and generate fracture plots in relation to their distance from the fault Core. Simultaneously, relationships between the number of seismic events and the distance to the fault are established.

Results

Preliminary results indicate that some specific faults have a seismic influence range between 60 and 100 meters, being mostly asymmetric with respect to the fault core, i.e., accumulated at a certain distance from it.

Conclusions

These results suggest a potential existence of increased seismicity and fractures towards a fault in an underground mining environment under certain conditions studied. Identifying and quantifying the influence of faults on seismicity is crucial for risk management and safe operation planning in mining.

Estimation of Pillar Strength and Rock Mass Properties using FLAC3D and 3DEC: A Case Study from Australian Mining Operations

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Introduction and Motivation

Estimating rock mass properties is critical for safe and efficient board and pillar underground mining design. This study uses field data from Australian operations to back-calculate Hoek-Brown properties in FLAC3D and bonded block model (BBM) contact properties in 3DEC. Additionally, a novel calibration scheme is proposed to refine the BBM for enhanced pillar stability assessment in discrete element modeling.

Objectives of the Study

This study aims to estimate the rock mass properties of pillars under plane-strain conditions in FLAC3D by back-calculating IMASS constitutive model parameters (GSI, mi, σ_ci) using field data, and to propose an empirical model linking pillar strength to Hoek-Brown properties (mb, a, σ_ci). Additionally, the objective of the 3DEC modeling is to develop a three-dimensional Board and Pillar (3DBP) model as a synthetic rock mass (SRM) and to validate its stability and localized damage performance against field observations. The empirical model proves invaluable for calibrating various lithological units in large-scale 3DEC modeling of mining structures, particularly when field data is limited. This is essential for accurate strata stability assessment and ground support design.

Methods & Procedures

1. **Field Data Collection**: The previous BP operational data including pillar stress, pillar strength, depth, dimension, and safety factors are collected from Australian BP operations. These pillars are recognized as stable without any stability issues. The UNSW pillar design formula is used to calculate the pillar strength.
2. **Numerical Modelling in FLAC3D**: The plane strain models of BP are created in FLAC3D and the IMASS constitutive model properties including the GSI, mi, and σ_{ci} are back-calculated from the comparison between numerical and field (empirical) pillar strength. A new equation, $\sigma^p = {m_b}^a \sigma_{ci}$, is proposed which links the pillar strength (σ^p) to Hoek-Brown parameters (m_b, a, and σ_{ci}). This equation can be used to estimate the pillar strength of other lithological units with similar BP dimensions.
3. **Synthetic Rock Mass (SRM) Modelling in 3DEC**: The BP-SRM models are generated in 3DEC using the BBM in 3DEC. The contact parameters including normal and shear stiffness, cohesion, and tension are then calibrated by comparing 3DEC pillar strength

results with the proposed empirical model and filed data. The fracture response of 3DEC is compared with field observations to ensure a promising calibration procedure.

Results

The FLAC3D back-calculation method effectively characterized pillar rock mass using average field-observed strength (12.7 MPa). It determined GSI and mi, while σ_{ci} was based on laboratory data. An empirical model derived from numerical simulations predicted rock mass strength for lithological units lacking field data (e.g., sandstone, siltstone). This model calibrated the 3DEC SRM, using BBM to simulate 3D pathways initiating representative fracturing responses. During SRM calibration, monitoring contact cohesion ensured peak pillar strength before cohesion loss (40-45%), maintaining structural stability despite localized damage, akin to field observations. Figure 1 shows FLAC3D's prediction of pillar damage at peak strength, alongside 3DEC SRM and field data, confirming accurate rock mass characterization. The 3D SRM model closely replicated observed fracturing, aligning well with field pillar strength predictions (12.7 MPa).

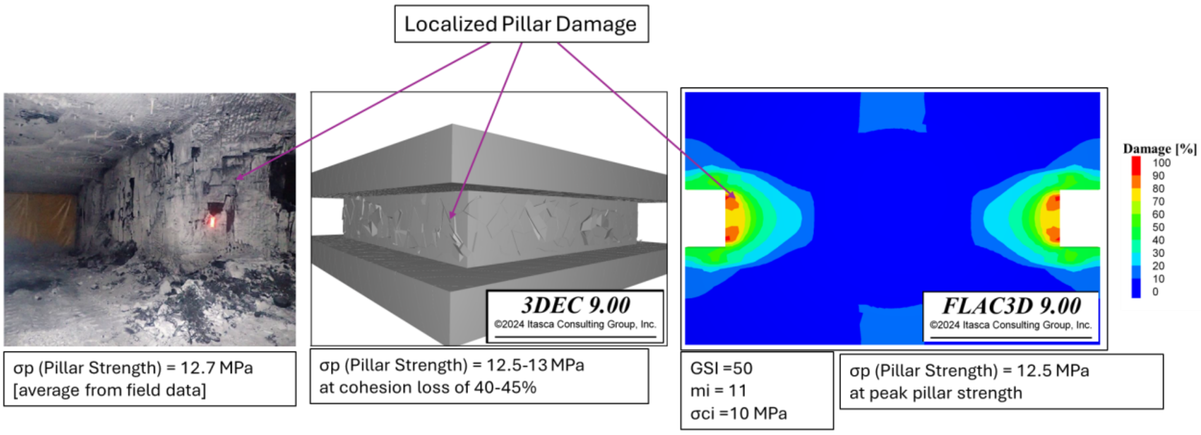

Figure 1: A comparison between field observation and numerical modelling results. The localized pillar damage simulated in 3DEC and FLAC3D is in good agreement with field observation. The average pillar strength of 12.7MPa is very well predicted by the numerical models.

Conclusions

The study demonstrates that the back-calculation approach implemented in FLAC3D is an effective method for rock mass characterization of pillars, closely approximating the average field-observed pillar strength (12.7 MPa). The developed empirical model, based on numerical results, accurately predicts equivalent rock mass strength for lithological units without field data. The calibrated SRM model in 3DEC, using the BBM approach, successfully replicates the fracturing response observed in the field, confirming its reliability for further investigations into pillar stability and design in surface and underground mining projects.

Scale-dependent Behavior of the GSI Index: a Comparative Analysis

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Introduction and Motivation

Recommendations regarding GSI reduction due to scale effects have already been indicated in previous studies, the most outstanding corresponding to Hoek et al. (2013) who stated: "This chart applies to tunnels of about 10 m span and slopes < 20 m high. For larger caverns and slopes consider reducing GSI to account for decreasing block interlocking". Sonmez et al. (2021) indicates that: "When the engineering dimension (slope height or tunnel span) increases, a decrease in GSI should be expected", and "GSI is a scale-dependent parameter". However, the use of these recommendations is generally not considered in slope stability analysis, and it is often done empirically, or simply not done.

Objectives of the Study

Review and compare various approaches available in the technical literature to reduce the GSI based on the scale analysis.

Methods & Procedures

To illustrate the idea, Figure 1 shows schematically the same rock mass in four different scales as a consequence of increasing the slope height (h). In the first case (scale A), the rock mass is observed at the outcrop-level scale, with a recognizable structure and joint condition, and characterized with a GSI-1. Although the rock mass is the same, the GSI-1 mapped at the rock outcrop level is not representative of the other scales (scales B, C, and D). For instance, in scale D a better rock mass description corresponds to a disintegrated or very fractured classification, so it would not be correct to characterize it with GSI-1.

Accordingly, when the slope height increases, a decrease in GSI should be expected:

$$h_1 < h_2 < h_3 < h_4 \qquad \text{(Equation 1)}$$

$$GSI_1 > GSI_2 > GSI_3 > GSI_4 \qquad \text{(Equation 2)}$$

Currently, in the technical literature, there are two formulations to quantify the GSI reduction according to scale effects:

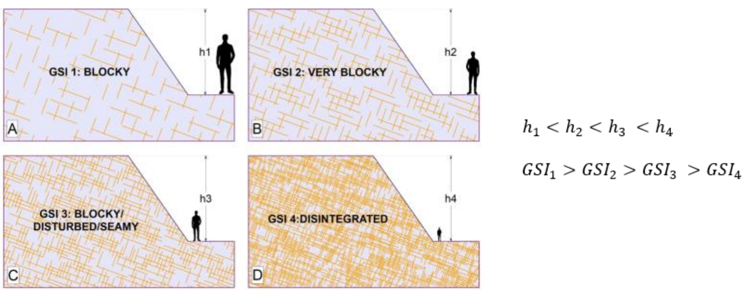

Figure 1: Rock mass slope with the same fracture network seen at different scales (referential scheme)

Sonmez et al. (2024) propose a complementary approach to quantify the basic GSI chart (GSI_{ED}) considering the scale effect on rock structure by applying a scale factor (s_f) calculated based on the "engineering dimension (ED)", where $K=J_v$ or S, and SR is the "Structure Ratio".

$$SR_{sf} = -17.5 ln(s_f K) + 80 \qquad s_f = \frac{ED}{100} \qquad K = J_v \; o \; K = \frac{3.3}{s_{ave}} \; o \; \frac{3.3}{\sqrt{V_b}} \qquad \text{(Equation 3)}$$

$$GSI_{ED} = [0.018(SR_{sf}) + 0.402425] * SCR_S + [0.0016(SR_{sf}^2 + 0.1972(SR_{sf}) + 6.4295] \qquad \text{(Equation 4)}$$

An analytical formulation to quantify the GSI reduction due to scale effects in slope stability analysis was proposed by Pozo (2022). The "Equivalent GSI" or GSI_e is mainly expressed in terms of geomechanical parameters such as orientation, continuity, strength, and joint condition (w_0 a w_5); on the other hand, it depends on geometric factors such as the ratio between the slope height (h) and the average joint spacing (e). As a result, the scale-dependent GSI_e is obtained by multiplying the GSI defined at the rock-outcrop level (GSI_0) by a scale factor k.

$$GSI_e = k \cdot GSI_0 \qquad k = w_0 w_1 w_2 \left(\frac{h}{e}\right)^{-w_3 w_4} - w_5 \qquad \text{(Equation 5)}$$

Results

Both approaches have been applied in four mining slopes located in Turkey and an open pit slope in Mexico. The characterization, description, and analysis of the slopes were presented by Sonmez & Ulusay (1999, 2002) and Pozo (2023).

Table 1: Results of all analyzed cases (scaled GSI)

Parameter	Case 1: Eskihisar Mine	Case 2: Baskoyak Mine	Case 3: Kiskadere Mine	Case 4: Cayeli Quarry	Case 5: Peña Colorada
Slope height (m)	18.5	18.0	78.0	40.0	240.0
GSI_{ED} (Sonmez et al., 2024)	55	25	38	35	44
Equivalent GSI_e (Pozo, 2022)	43	15	22	22	34

Conclusions

The Pozo criterion (2022) provides greater GSI reductions since it considers a decrease in both the rock mass blockiness and the joint condition. On the other hand, the criterion of Sonmez et al. (2024) results in more optimistic reductions, because it only reduces the component related to the rock mass structure.

To fully validate these approaches, it is recommended to extend the evaluation to the case of multi-layer slopes because proposed formulations are mainly applicable to homogeneous rock masses. It is also advisable to increase the number of cases in which both formulations have been applied, preferably in open pits or quarries that have collapsed.

Rock Mass Classification Methods: A Foundation for Back Analysis Processes in Modern Geotechnics

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Introduction and Motivation

The increasing demand for physical and mechanical parameters of rock masses in geotechnical engineering projects in Chile has underscored the significance of existing classification and characterization methods for specialists in the field. As project scope and complexity grow, this need becomes even more pronounced. Often, practitioners find themselves relying solely on the RMR, Q, and GSI indexes—metrics that are typically derived from limited rock mass data due to the challenges associated with conducting large-scale testing.

It is important to note that even large-scale Flat Jack tests have inherent limitations due to their scale factor. In many instances, engineers only have access to values derived from Point Load Tests conducted in the field. Consequently, it becomes challenging to rely on rock mass and intact rock strength values that truly represent the conditions, necessitating a more conservative approach due to the considerable uncertainty involved. This uncertainty has led engineers to adopt methods that have gained prominence in recent years, such as traditional back analysis and numerical techniques (Wittke, Espinoza), which enable probabilistic estimations of the most relevant parameters regarding the stress-strain behavior of rock masses.

Thus, the integration of methods such as Bieniawski, Barton, and Hoek–Brown can serve as a foundational tool for research and design, particularly when complemented by Back Analysis. This combination enables more accurate estimations of the physical and mechanical parameters of rock masses, even in situations where anisotropy requires a nuanced understanding of their stress-strain behavior (Wittke, Sakurai, Gioda, Espinoza).

As a result, it is essential that any rock engineering design project incorporates a robust field campaign alongside comprehensive laboratory testing. This dual approach facilitates proper classification and characterization, yielding results that support a focused and thorough Back Analysis. Such analysis ultimately leads to designs that can be refined through well-planned monitoring of the relevant characteristics.

In this context, this work aims to propose a straightforward methodology that has demonstrated effectiveness in a complex project in Chilean Patagonia.

Objectives of the Study

The primary aim of this work is to present a practical methodology utilized in the initial design of steep slopes within moderately to severely fractured igneous rock masses for a road project in Chilean Patagonia, located along the scenic shores of the Puyuhuapi Fjord (see Figures 1 and 2). In this region, the integrity of the rock mass was significantly compromised due to blasting operations conducted without proper consideration of pre-cutting methods or controlled blasts. As a result, extreme fractures developed in several areas, necessitating intervention to enhance the road segment at Las Pulgas (see Figure 3).

The secondary objective is to illustrate how, through a simplified working method and insightful interpretation of the results, a robust geotechnical model was ultimately developed. This model enabled the calibration of values obtained through back analysis, which were essential in creating a successful design not only for the steep slopes but also for the foundations of two critical structures: a small viaduct and a bridge.

Methods & Procedures

The project utilized data from several studies to estimate the characteristics of rock masses and determine the most effective approach to design the slopes, following the acquisition of the most suitable physical and mechanical parameters pertinent to the issue under investigation. The methodologies employed included:

1. Identification of geotechnical cells to accurately measure discontinuities for each section of the 100-meter stretch.
2. Collection of intact rock samples and execution of one-point load tests, with subsequent verification of results through unconfined compression tests.
3. Estimation of the Rock Quality Designation (RQD) of the rock mass using Palmström's proposed method.
4. Measurement of discontinuities, including multiple assessments of strike and dip, to ascertain the slope's dip and strike characteristics.
5. Generation of polar diagrams corresponding to the measured values for each section, alongside a kinematic analysis to identify the most probable block types.
6. Classification of the rock mass according to the Bieniawski-Romana method, which provided the RMR value and facilitated an approximate estimation of the deformation modulus.
7. Approximate determination of the mechanical parameters of the rock mass utilizing the Hoek-Brown failure criterion.
8. Development of a comprehensive geotechnical model that incorporated all previously acquired data and identified the optimal method for back analysis of the sector.
9. Estimation of the most probable shear strength parameter values, followed by the calibration of the model using the Hoek-Brown criterion.
10. Design of the slopes, complemented by verification of both local and global stability.
11. Formulation of block capture systems for the completed slopes.

Results

Based on the available space, one or more results will be presented to effectively document this work and showcase the significant value of characterization and classification methods for rock masses. Furthermore, this approach highlights the benefits of combining these methods with other modern techniques to develop solutions for designing slopes in moderately to highly fractured rock masses.

Conclusions

The conclusions will provide the reader with insights into the philosophy guiding the work undertaken in Chilean Patagonia, allowing them to apply this understanding to the challenges one would encounter by adapting the methods and procedures to the specific types of rock mass or intact rock encountered in their projects.

Figure 1: Fjord of Puyuhuapi – Location of the project

Figure 2: Fjord of Puyuhuapi – Location of the most difficult slope

Figure 3: Fjord of Puyuhuapi – Highly fractured rock mass

Discussion and Conference Closure

Pinnaduwa H.S.W. Kulatilake[1], Marcos Massao Futai[2]

[1] Conference Chair, President SLRMES; Professor Emeritus, University of Arizona, USA.
[2] President, Comite Brasileiro de Mechanica das Rochas, University of Sao Paulo, Brasil.

(Feel free to use this space for taking notes, etc.)

Annexures

Annexure 1: Conference Program

Annexure 2: Auxiliary Information

FIRST ISRM COMMISSION CONFERENCE ON ESTIMATION OF ROCK MASS STRENGTH AND DEFORMABILITY

LIMA, PERU, DECEMBER 6th, 2024

PROGRAMME

TIME	EVENT
	Hotel: Melia Lima, Peru *Room:* Caral
07:30 – 08:30 AM	**CONFERENCE REGISTRATION**
08:30 – 08:35 AM	**INAUGURAL SESSION** Opening Remarks by Prof. Pinnaduwa H.S.W. Kulatilake, Conference Chair

TIME	**SESSION 1: EMPIRICAL METHODS TO ESTIMATE ROCK MASS PROPERTIES** ***Co-Chairs: Dr. Antonio Samaniego and Mr. Jorge Arriagada***	
08:35 – 09:00 AM	**Session Lead Lecture:** A Review on Estimation of Rock Mass Deformability Properties Using Empirical Methods	***Dr. Yan Xing*** *China University of Mining and Technology* *China*
09:00 – 09:20 AM	Size-dependent Behaviour of Artificially Jointed Hard Rock	***Dr. Hossein Masoumi*** *Monash University, Australia*
09:20 – 09:40 AM	Comparison of Quantitative GSI Methods for Hoek and Brown Failure Criteria: A Case Study of El Teniente Mine, Chile	***Nayadeth E. Cortes*** *Pontificia Universidad Católica de Valparaíso* *Chile*
09:40 – 10:00 AM	Validation of Field Investigations of Rock Quality in the RMR89 and RMR76 Systems	***Sebastian A. Suárez Yallico*** *SRK Consulting, Peru*
10:00 – 10:20 AM	Correlating Drill Core Morphology Data to Standardized Joint Roughness Conditions	***Jonathan Hill*** *Corescan (Epiroc), Canada*
10:20 – 10:50 AM	**Tea/Coffee Break**	

TIME	**SESSION 2: ROCK MASS PROPERTIES** *Co-Chairs: Dr. Mahdi Saadat and Dr. Raul Pozo*	
10:50 - 11:20 AM	**Session Lead Lecture**: Estimation of Rock Mass Strength and Deformability Properties in Three Dimensions Based on a Numerical Modeling Procedure	***Prof. Pinnaduwa H.S.W. Kulatilake*** *University of Arizona* *USA*
11:20 - 11:40 AM	Geological Survey of Rock Blocks from Chuquicamata Underground	***Jorge P. Pereira and Carlo S. Divasto*** *DERK Ingeniería y Geología Ltda.* *Calama, Chile*
11:40 AM - 12:00 Noon	Cerchar Parameter Estimation as a Function of Equivalent Quartz Content and Properties of Porphyry Copper Igneous Rocks	***Jorge P. Pereira and Carlo S. Divasto*** *DERK Ingeniería y Geología Ltda.* *Calama, Chile*
12:00 - 12:20 PM	Influence of Weathering in the Definition of Geotechnical Units	***Dr. Andrea Russo*** *SRK Consulting, Chile*
12:20 – 01:20 PM	**Lunch Break**	

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Grupo Nacional de la ISRM
International Society for Rock Mechanics and Rock Engineering

TIME	**SESSION 3:** **ROCK MASS BEHAVIOUR AND SEISMICITY** ***Co-Chairs: Jonathan Hill and Prof. Nestor Espinoza***	
01:20 - 01:50 PM	**Session Lead Lecture**: A 3-D Strength Criterion Developed for Jointed Coal Masses Based on True Triaxial Tests and 3-DEC Numerical Modeling	***Dr. Peng-fei He*** *China University of Mining and Technology Beijing, China* *and* ***Prof. Pinnaduwa H. S. W. Kulatilake*** *University of Arizona, USA*
01:50 - 02:10 PM	Seismicity and Mining Operations in High-Stress Underground Environments	***Dr. Luis Felipe Orellana and Dr. Tomas Roquer*** *Universidad de Chile, Chile*
02:10 - 02:30 PM	Preliminary Analysis of Fault Reactivation in Underground Mining	***Dr. Luis Felipe Orellana and Lucy Bravo*** *Universidad de Chile, Chile*
02:30 - 02:50 PM	Impact of Topographic Deformation on Induced Seismicity and Reactivation of Geological Faults in Underground Mining	***Guillermo Martinez*** *Universidad de Chile* *Chile*
02:50 – 03:10 pm	Unveiling the Impact of Geological Structures on Seismic Activity in Large Underground Copper Mines	***Javiera Tillería*** *Universidad de Chile* *Chile*
03:10 – 03:40 PM	**Tea/Coffee Break**	

Sri Lankan Rock Mechanics and Engineering Society

Sociedad Peruana de Geoingeniería-SPEG
Grupo Nacional de la ISRM
International Society for Rock Mechanics and Rock Engineering

TIME	SESSION 4: **SCALE EFFECTS AND BACK ANALYSIS** ***Co-Chairs: Dr. Hossein Masoumi and Dr. Luis Orellana***	
03:40 - 04:10 PM	**Session Lead Lecture**: Estimation of Rock Mass Properties for Mine Tunnels Based on a 3-D Discontinuum-Equivalent Continuum Back Analysis Method Using Field Deformation Data	***Prof. Pinnaduwa H.S.W. Kulatilake*** *University of Arizona* *USA*
04:10 - 04:30 PM	Estimation of Pillar Strength and Rock Mass Properties using FLAC3D and 3DEC: A Case Study from Australian Mining Operations	***Dr. Mahdi Saadat*** *Blackrock Mining Solutions* *Australia*
04:30 - 04:50 PM	Scale-dependent Behavior of the GSI Index: a Comparative Analysis	***Dr. Raúl Pozo*** *SRK Consulting (Peru) S.A.,* *Peru*
04:50 - 05:10 PM	Rock Mass Classification Methods: A Foundation for Back Analysis Processes in Modern Geotechnics	***Prof. Néstor R. Espinoza Guillén and*** ***Jorge A. Arriagada Triana*** *Universidad de Valparaíso,* *Chile*
05:10 – 05:20 PM	Short Break	

05:20 – 06:00 PM	SESSION 5: **DISCUSSION AND CONFERENCE CLOSURE** ***Co-Chairs: Prof. Pinnaduwa H.S.W. Kulatilake and Prof. Marcos Massao Futai***	

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Auxiliary Information

1. Sightseeing Tours

Peru is full of tourist attractions, and anyone interested in sightseeing is recommended to directly contact following individuals from; https://kinsatravelcollection.com/en/

Whatsapp (a): Teresa +51-965 263 538

Whatsapp (b): Marita +51-932 586 634

teresanunez@kinsatravelcollection.com

2. Conference Venue & Accommodation

Hotel Melia

Av. Salaverry, 2599, Lima, Peru.

https://www.melia.com/es/hoteles/peru/lima/melia-lima

Tel: +51 1 411 90 00 | Email: melia.lima@melia.com

Distances

0.3 km from Real Plaza Salaverry Mall

5.3 km from Historical center of Lima

13 km from Jorge Chavez International Airport

The conference venue Hotel Melia in Lima, Peru is adjacent to a beach with magnificent views. Many hotels and economical accommodation are available within the range of US$ 40-US$ 125 per night. Please visit https://www.slrmes.org to obtain information on the conference venue, accommodation, visa, and travel to Peru.

An interactive map of Lima Peru is available on the conference website.

Thank you for participating the 1st ISRM Commission Conference on Estimation of Rock Mass Strength and Deformability!

- The Organizing Team | SLRMES & SPEG 2024

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- an ISRM Specialized Conference