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Research Article

Advanced optimisation of ground support systems for enhancing underground tunnel stability in geologically adverse conditions

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Abstract: This research aims to optimize ground support systems for underground tunnels in geologically challenging environments, specifically addressing the reduction of Fall of Ground (FOG) incidents in a gold mine in Mashava, Zimbabwe. The study integrates advanced detection and classification methodologies to enhance tunnel stability and safety. Tunnel Reflection Tomography (TRT) was employed to identify unfavorable geological structures ahead of excavation, while core logging at 20 locations on level 7 provided rock mass quality assessments using three classification systems: Bieniawski's Rock Mass Rating (RMR), Laubscher's Mining Rock Mass Rating (MRMR), and Barton's Q-system. The results consistently indicated poor rock mass quality, informing the design and refinement of a robust ground support system. Fallout height data from past FOG incidents and probabilistic key block analysis using J-Block software further validated the support system's effectiveness. The findings significantly reduce collapse risks and downtime, enhancing operational safety and efficiency. This research contributes to developing practical strategies and tools for improving tunnel stability in complex geological settings, offering valuable insights for future advancements in mining support technologies. The study's necessity stems from the industry's growing demand for innovative solutions to enhance tunnel stability in adverse geological settings, particularly in regions with limited access to advanced technologies or methodologies.

Keywords: Ground support; Reflection Tomography; Tunnel stability; Rock Mass Rating; Empirical support design

1. Introduction

Effective support design and responsible mining practices ensure economically viable and sustainable mining operations. Fall of Ground (FOG) incidents constitute a significant challenge when tunnels pass through unfavorable geological bodies. It is widely acknowledged that mining operations can disrupt the stability of the rock mass, leading to ground instabilities [25]. These instabilities are particularly pronounced when mining encounters unfavorable geological bodies, such as faults, fractures, or weak rock zones. Underground tunnels, crucial for accessing mine areas and transporting ore, are vital for underground mine projects [2]. Ensuring their stability is paramount, as it directly impacts the safety of personnel and equipment and successful ore production. The stability of tunnels is a significant concern in underground mining, particularly in complex and unfavorable geological conditions [34, 35].

Unfavourable geological bodies are characterised by the presence of geological discontinuities structures such as joints, faults, and sympathetic jointing associated with faults. The occurrence of planes of weakness in the hanging wall strata is a major factor for excavation stability in underground mining environments [1, 27]. Effective support systems are critical to preventing and mitigating Fall of Ground (FOG) incidents [28]. FOG incidents are the most significant threat to miner's safety in Zimbabwean mines, accounting for most fatalities and injuries. The fall of ground incidents at the gold mines is mainly bound by geological discontinuities - structures such as joints, faults, potholes, and sympathetic jointing associated with faults. Various support design methodologies are applied in Zimbabwean mines to mitigate rock failure and ensure safe working conditions. The current support design approach used at the gold mine is based on the fallout height method, which utilises statistical analysis of past Fall of Ground incidents to determine the required thickness of hanging wall support, corresponding to the height of 95%of the FOGs. A significant limitation of the approach is that it fails to account for geological discontinuities, including their characteristics, orientations, and exposures in relation to the excavation [28].

In mining, rock fall-related hazards are forever present. According to a Mining Zimbabwe article, among the 33 reported fatalities in 2024, 15, 45%, were attributed to incidents of ground collapse, which is a clear indication that the fall of the ground is a major concern [26]. The fall of the ground management system is an important aspect of mining; hence, a mine must implement a strategy that will aid in combating rock fall-related hazards. This study is conducted to find a robust support system for underground tunnels that pass through unfavorable geological conditions. The study helps improve the current system and combat falls of ground at the mine. The study first focuses on predicting the location of unfavorable geological bodies and classifying the rock mass. Secondly, the study helps to design an empirical support system based on the rock mass classification results and the fall-out height data. The hole-drilling technique is a common method for advanced subsurface detection; however, it is often time-consuming, inefficient, and limited in detection range. The integration of Tunnel Reflection Tomography (TRT) technology addresses these limitations by reducing the number of drilling holes required for advanced detection, provided a skilled technician interprets the data. TRT significantly enhances the precision of evaluating rock mass quality when combined with rock mass classification systems.

As easily accessible resources are depleted and mining operations extend to deeper, more complex geological settings, rock engineering practitioners increasingly focus on safely extracting resources under challenging conditions. This study aims to develop safe methodologies for resource extraction, optimize production rates, and extend the operational lifespan of mines. A key focus of this research is optimizing ground support systems to enhance tunnel stability in geologically adverse conditions a critical issue in the mining industry. Unstable geological environments often pose significant risks to worker safety and disrupt productivity. By exploring advanced techniques for improving tunnel support, this study seeks to provide practical solutions for increasing tunnel stability, reducing risks, and minimizing operational delays. The findings are expected to contribute to safer, more efficient mining practices while paving the way for future advancements in ground support system designs tailored to complex geological conditions. The key innovation of this study lies in its integrated approach to optimizing ground support systems for underground tunnels in geologically challenging environments. Specifically, the study combines advanced detection (Tunnel Reflection Tomography, TRT), multi-method rock mass quality classification (RMR, MRMR, and Q-system), and probabilistic key block analysis using J-Block software. This holistic framework enables a more precise assessment of geological risks and the design of tailored robust ground support systems. The innovative use of fallout height data from past fall-of-ground (FOG) incidents to refine and validate the support design further ensures its practicality and reliability. By integrating predictive and validation tools, the study reduces collapse risks and operational downtime and establishes a repeatable methodology that can be adapted to other mining contexts. This research significantly advances tunnel stability strategies, offering theoretical insights and practical applications for enhancing safety and efficiency in mining operations. Despite advancements in ground support systems, there remains a critical gap in integrating predictive tools, empirical data, and probabilistic modeling to optimize these systems effectively. Traditional methods often fail to adequately predict geological hazards or refine support designs based on site-specific data, leaving room for improvement in safety and cost-effectiveness.

The research addresses this urgent need by developing the comprehensive framework that integrates advanced geological assessment tools, data-driven support design, and probabilistic modelling, ensuring safer and more efficient mining operations. The study's necessity stems from the industry's growing demand for innovative solutions to enhance tunnel stability in adverse geological settings, particularly in regions with limited access to advanced technologies or methodologies.

2. Geological Settings and Mining Layout

The mine lease area consists of three to five North-South striking parallel shear zones across a width of over three kilometers and a strike of 21km. The shear zones primarily occur within mylonitic gneiss in the Mashaba greenstone belt. Quartz veins with significant auriferous content are also located within the greenstone. Their formation is believed to have occurred in areas of tensional stress proximal to significant fault lines. The distribution and the orientations of ore shoots have been significantly disrupted by intense faulting activity. Severe faulting has further complicated the distribution of the iron formation and the pattern of ore shoots [6]. Frequent cross-faults, alternating zones of compression and extension, and deformed buildings all indicate that significant stress is being accommodated elsewhere. The mine is 300 meters deep and has ten levels of operation with a 30-meter distance between them. Winding machinery installed at the shafts serves a dual purpose: transporting workers and hoisting ore during mining operations. The primary mining method is sublevel stoping, complemented by underhand and shrinkage stoping for enhanced efficiency [7]. Stoping is the preferred method of extracting narrow reefs, where the reefs are less than a meter and characterised by a steep dip. The dimension of the stope typically measures 1.8 meters in width and 2 meters in height.

Supporting underground excavations serves three crucial purposes, which are ensuring the safety of the working places, preventing the fall of key blocks that could trigger ground collapse from falling, and controlling the movement of large blocks or fragments near the excavation boundaries. When designing support systems for underground excavations, two key factors are considered: demand and capacity, both measured in kN/m^2. The demand refers to the load the rock mass (hanging wall) imposes that requires support. The capacity represents the ability of the support system to resist that load over a specific area. If the demand exceeds the capacity, then failure occurs. The objective is to prevent hanging wall failure by providing adequate resistance and ensuring stability.

3. Research Approach

The primary goal of this research was to develop a robust support system that would minimize the likelihood of rock fall occurrences and guarantee the stability of underground tunnels that traverse unfavorable geological formations. Tunnel Reflection Tomography was employed to precisely predict the location of unfavorable geological bodies to achieve the first goal of this study. Using core log samples that were gathered from twenty different places, Bieniawski's Rock Mass Rating and Laubscher's Mining Rock Mass Rating were utilized to categorize the rock mass. With the aid of empirical techniques and a combination of fallout data and the results of the rock mass classification, a robust support system was created for the excavation, guaranteeing a stable and dependable design.

3.1. Detecting geotechnically challenging grounds

The first objective of this project is to forecast geotechnically challenging grounds ahead of the working face. This was achieved - using Tunnel Reflection Tomography (TRT). This technique harnesses seismic waves to generate three-dimensional images (3D seismic tomograms) and detect challenging geotechnical grounds ahead of the tunnel excavation. TRT is a seismic processing technique developed by NSA Engineering of Golder Colorado to generate a seismic wave that propagates into the surrounding rocks by hammering the source point. A sensor fitted with a wireless module then picks up these 3D waves (seismic impulses) that are recorded by the sensor and saved on a computer as data files. The seismic signals can now be processed once the sensors have recorded and saved them as data files. Obtaining coordinates, filtering noise signals, separating P and S waves, processing the image, and producing the outcome are the stages of processing the captured data.

3.2. Detecting geotechnically challenging grounds

A comprehensive geotechnical core logging program involved systematically documenting and analyzing core samples. The purpose of core logging was to ascertain the properties of the rock mass, including its quality, discontinuity frequency, and potential influence of groundwater, which is important to characterise the rock mass using -various rock mass rating methodologies, e.g., RMR, Barton's Q system, and MRMR. Additionally, core recovery from different boreholes was determined. The information obtained from core logging was then utilised to apply various rock mass classification systems to classify the rock mass. The following critical parameters were gathered at the core shed to inform the rock mass classification process: core recovery, rock mass Quality, and detailed descriptions of discontinuity surface characteristics.

3.3. Rockmass Classification

Determining the rock mass state was a crucial step in the research process. The nature and quality of the sub-surface ground conditions are derived from data collected from the geotechnical borehole log and laboratory testing results. The analysis of the quality of the subsurface conditions focuses on three rock mass classification systems: MRMR, Rock Mass Rating Classification, and Barton's Q-system. The Q-system, originally developed based on civil engineering principles, offers reliable predictions for support requirements in tunnel boring machine excavations within civil engineering projects. However, its application to mining operations is often considered overly conservative.

3.3.1. Rock Tunnelling Quality Index (Q)

The Q-system comprises three quotients incorporating six parameters describing the rock mass quality. These parameters are interconnected by Equation (1) proposed by Barton et. al [4]:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{1}$$

where RQD is the Rock Quality Designation, J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor, SRF is the stress reduction factor, The Q system classification is based on the following three aspects: Block size (RQD/J_n); Joint shear strength (J_r/J_a); Confining stress (J_w/SRF).

3.3.2. Bieniawski's Rock Mass Rating (RMR)

Rock Mass Rating consists of six parameters that are used to assess the quality of the rock mass which are Uniaxial Compressive Strength of the rock material (UCS), Rock Quality Designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions, and orientation of discontinuities. Each parameter is assigned a weight based on its importance and a maximum rating to ensure a total of 100 [22]. The summation of the rated parameters provides the final RMR value utilised for design purposes.

3.3.3. Laubscher's Mining Rock Mass Rating (MRMR)

Laubscher [24] highlighted that the key difference is that in-situ rock mass ratings (RMR) need to be adjusted based on the mining environment to obtain final ratings (MRMR) for mine design. The MRMR system is well-suited for predicting rock mass behavior around excavations as it accounts for downgrading rock mass parameters. Parameters such as weathering, mining-induced stresses, joint orientation, and blasting effects must be adjusted. The four parameters, rock material strength (UCS), RQD, joint spacing, joint condition, and groundwater, were added to determine the MRMR, as suggested by Hoek and Brown [23]. The justification for using this method of rock mass classification lies in its simplicity and ability to account for both the inherent rock mass rating and the external factors that can affect the rock mass due to mining activities.

3.4. Rockmass Classification

Various support design methodologies are being employed within the Zimbabwean mining industry. The current support design methodology in the gold mines relies on the fall-out height method, which statistically analyses past falls of ground (FOGs) to determine the required hanging wall thickness [11]. However, this method has significant limitations, notably neglecting geological discontinuities and their properties, orientations, and exposures relative to the excavation, as noted by Dong et al. [15]. In this research, a combined approach, utilising empirical design methods, the fall-out height method, and insights from similar operations, was employed to determine ideal ground support requirements for tunnels passing through unfavourable ground conditions. The empirical support requirements were adjusted to account for anticipated mining conditions. Additionally, cases, where tunnel spans exceed planned dimensions, will require individual assessments and adjustments to support specifications.

4. Results and Discussions

Understanding the quality of the rock mass is the crucial step in designing a robust support system. Quantifying the rockmass's quality is critical since the quantitative output is used for further empirical design, such as the support requirements, excavation methods, and sequencing. This section clearly outlines the rock mass classification results from the three rock mass classifications and how the empirical methods were used to design support based on the results.

4.1. Tunnel Reflection Tomography (TRT) Results

This research used the Tunnel Reflection Tomography technique to detect geotechnically challenging grounds ahead of the working face accurately. The seismic signals captured by the Tunnel Reflection Tomography (TRT) sensors were then processed to generate a visualisation of the subsurface structures (Figures 1 to 4). As shown in Figures 1 to 4, the visual results highlight the presence of adverse geological bodies identified by analyzing data collected using the Tunnel Reflection software. Within the mapping range, the blue range indicates that the geological structures of the level 7 section are weak. In contrast, the yellow range indicates that the geological structure of the section is rigid. The location of the working face and receiver points is clearly discernible, with a 10m x 10m mesh providing an effective reference for estimating the distance between the working face and potential geological anomalies. The figures illustrate that adverse geological bodies were detected using TRT, with one body identified 20m ahead of the working face based on the 10m x 10m mesh. Excavation confirmed this geological body to consist of poor ground conditions. While other geological bodies were detected at greater distances from the working face, the detection reliability decreases at such ranges, suggesting room for improvement in detection accuracy over longer distances. Therefore, repeated surveys will be necessary as these bodies are approached to validate the initial findings and ensure accurate characterisation.

Additional surveys were performed at a distance of 60 meters from the initial survey location to validate initial findings. The TRT forecast area was restricted to a designated zone, extending 90 meters from the front and 20 meters in each of the four cardinal directions. It can be observed from TRT detection results (Figure 3) that the geological bodies that are different from the ones at the working face were detected at a distance of 10m ahead of the working face. A significant continuous low-impedance anomaly region has been detected on both sides, directly in front of the center.

TRT detection results at 160m in level 7 (Figure 4) revealed the presence of the geological body, which differs from the one at the working face detected. Still, the bigger portion of the geological body is not centrally aligned. Instead, it is situated 25 meters ahead of the tunnel face's center, 10 meters to the right, and varying in height between 5 and 20 meters. The small portion of the detected geological body, which is centrally aligned with the tunnel's face, is at a distance of 30 to 40 meters from the working face. This variation will be considered as part of the ongoing project and further examination.Using TRT provides a cutting-edge approach to identifying unfavorable geological structures ahead of tunnel excavation. TRT allows for proactive decision-making in high-risk zones by offering real-time imaging of subsurface conditions. However, its effectiveness can be limited by signal attenuation in certain rock types, reducing its resolution in complex geological formations. Furthermore, its application requires specialized equipment and expertise, which may not be readily available in all mining contexts, particularly developing regions. Future research could focus on improving signal processing algorithms to enhance TRT resolution and adaptability in diverse geological environments. Detecting geological anomalies at greater distances using TRT presents several reliability challenges. One significant issue is signal attenuation, where the strength of reflected signals weakens over longer distances, reducing data clarity and quality. This also impacts resolution, making it difficult to accurately determine detected geological bodies' shape, size, or precise location. Additionally, noise interference becomes more pronounced at greater distances, with equipment vibrations, external sources, and geological factors potentially obscuring the true signals. Signal reflections may scatter or overlap in complex geological environments (e.g., fractured rock or layered structures), further complicating detection. The accuracy of long-distance detection also relies heavily on precise calibration and robust geological models; inaccuracies in these can lead to false positives or misinterpretations. Finally, processing data from greater distances requires advanced techniques to address increased noise and weaker signals, which may not always be practical in field conditions or real-time. These challenges highlight the need for ongoing refinement in TRT methodologies and equipment.

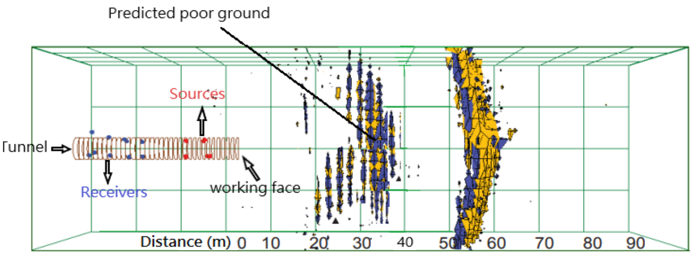

Fig. 1. Top view of TRT detection results at 40m in North direction level 7

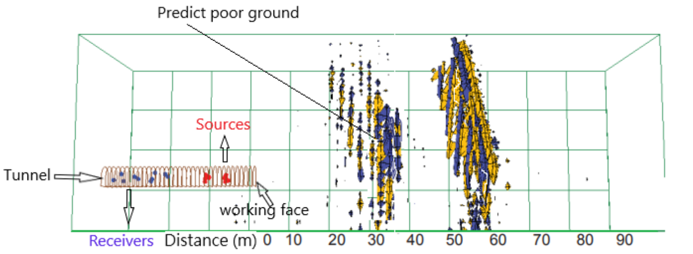

Fig. 2. Side view of TRT detection results at 40m in North direction level 7

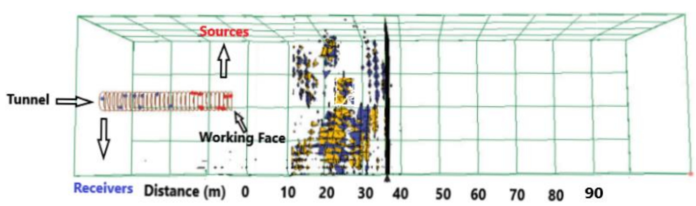

Fig. 3. Front view of TRT detection at 110 in North direction at level 7

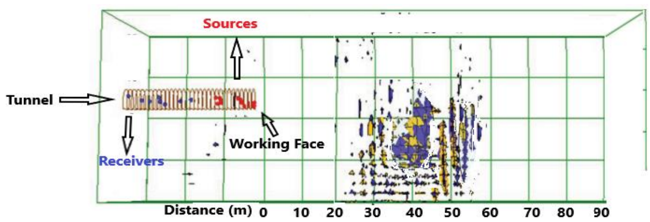

Fig. 4. Front view of TRT detection at 160m in North direction at level 7

4.2. Rock mass classification results

Core drill samples were collected from areas predicted to be poor ground based on Tunnel Reflection Tomography findings, providing valuable data for the rock mass classification exercise and enabling a more reliable assessment of the rock conditions. Laubscher's [24] Mining Rock Mass Rating Classification, Bieniawski's [5] Rock Mass Rating Classification, and Barton et al.'s [3] Norwegian Geotechnical Institute's Q-system were used to classify the rock mass. This study utilized multiple classification systems, selected for their relevance and widespread use in underground mining applications, to analyze twenty borehole core samples obtained from Level 7 of the mine. The samples were collected from three distinct areas, W-01, W-02, and W-03, to capture lateral variations in geological conditions. Boreholes L7-BH-01 to L7-BH-07 correspond to area W-01, L7-BH-08 to L7-BH-13 to area W-02, and L7-BH-14 to L7-BH-20 to area W-03. The spatial distribution of these boreholes is depicted in Figure 5.

4.2.1. Rock Quality Designation Results

The RQD for every geotechnical interval was determined using a volumetric joint count through the application of Palmstrom's [33] equation (2):

$$RQD = 115 - 3.3J_V \tag{2}$$

where: J_V is the number of joints per m^3 given by $J_V = J_h + J_d + J_S$......J_h , J_d and J_s – represent a number of joint sets per unit length in the hanging wall, dip, and strike direction.

RQD values can vary greatly depending on the orientation of the borehole relative to the predominant joint set. This bias arises because RQD is more sensitive to the direction of the borehole or scanline than joint spacing or fracture frequency measurements. To address this issue, the Volumetric Joint Count (Jv) offers a more comprehensive, three-dimensional assessment of jointing by quantifying the number of joints per cubic meter. Jv provides a more accurate representation of rock mass quality by considering all joint orientations, minimizing the directional bias inherent in RQD measurements. Table 1 summarizes the Rock Quality Designation (RQD) values. The joint data analysis reveals a minimum RQD of 42.14, a maximum of 54.86, and a mean value of 47.54. Consistently low RQD values were observed across Level 7 of the mine, highlighting poor rock quality. These results indicate significant geological deficiencies within the rock mass. Deere's [13] classification defines weak rock as having an RQD percentage between 25% and 50%.

4.2.2. Results on Rock Tunneling Quality Index (Q)

For this analysis, a value for Q was calculated for each borehole. The calculated Q values from the study area are listed in Table 2. The Q-system was developed by Barton in 1974 and describes the rock mass quality by combining the six rock mass parameters, which are RQD - the rock quality designation, the joint set number (J_n), the joint roughness number, joint alteration number (J_a), joint water reduction factor (J_w) and stress reduction factor (SRF). Q values varied from 1 to 4 and are classified as poor rock mass. Since the Q system is well suited to determine rock mass quality, it does not consider rock mass strength explicitly, nor does it include joint orientation information. The Q-system analysis yielded extremely low values, indicating very poor rock mass quality and a high risk of failure. This unfavorable classification is primarily attributed to a combination of low RQD values, high joint frequency, and unfavorable joint orientations, all contributing to the rock mass's inherent instability. The RQD values were sourced from Table 1, while the remaining parameters were derived from the data presented in Appendix A.

4.2.3. Rock Mass Rating.

The rock mass rating system developed by Bieniawski in 1989 was applied for this exercise. This version accounts for the impact of groundwater and joint orientation on the rock mass. The following parameters are included in the calculation of RMR: Groundwater conditions, Discontinuity orientation, Discontinuity spacing, Discontinuity condition, and Rock quality designation (RQD). A minimum RMR value of 40 and a maximum value of 47 were recorded from the gathered data. Bieniawski [5] describes the rock mass class as poor to fair rock. There is a slight variation from the Q rating rock mass class description. With RMR values between 40 and 47, the rock mass was unstable and required additional support (Table 3).

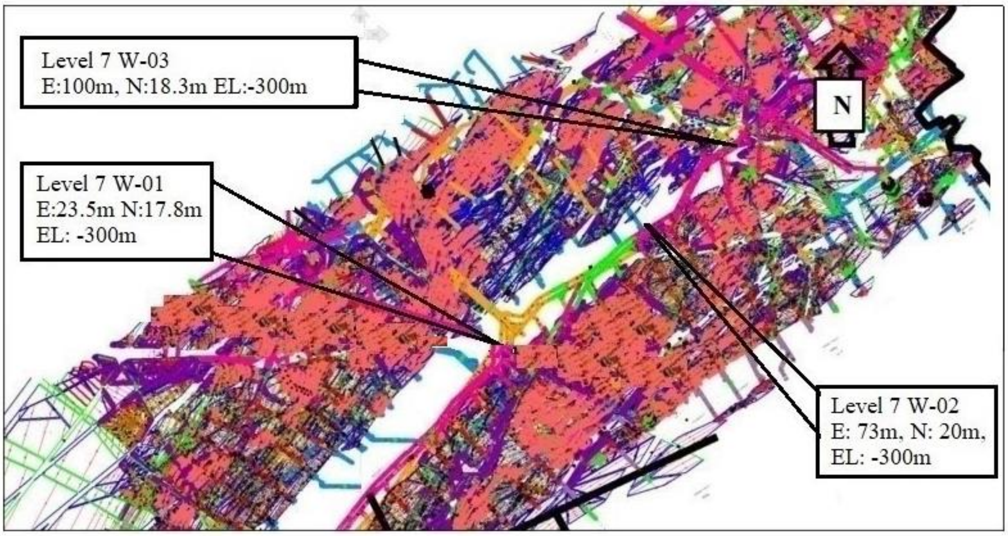

Fig. 5. Three district areas (W-01, W-02, and W-03) indicating level 7 borehole locations (the Gold Mine Geotechnical Database).

Table 1. Rock Quality Designation Results

Location	No. of Joints in Strike direction per unit length			No. of Joints in Dip direction per unit length			No. of Joints in hanging wall direction per unit length			No. of Joints per m^3	Rock Quality Designation
	No.	Distance	J_s	No.	Distance	J_p	No.	Height	J_h	J_v	RQD
L7-BH01	38	14	2.7	28	6	4.7	30	2.1	14.3	21.7	45
L7-BH02	42	13	3.2	32	5	6.4	22	2.1	10.5	20.1	50
L7-BH03	42	10	4.2	22	6.2	3.5	22	2.1	10.5	18.2	55
L7-BH04	46	13	3.5	34	5.5	6.2	30	2.3	13.0	22.8	40
L7-BH05	36	7	5.1	32	6	5.3	26	2.3	11.3	21.8	45
L7-BH06	42	10	4.2	24	6	4.0	24	2	12.0	20.2	50
L7-BH07	48	11	4.4	26	5	5.2	24	2.2	10.9	20.5	50
L7-BH08	48	12	4.0	22	6.2	3.5	24	2	12.0	19.5	50.
L7-BH09	36	8	4.5	32	6	5.3	20	2	10.0	19.8	50
L7-BH10	40	11	3.6	26	6	4.3	28	2.2	12.7	20.7	45
L7-BH11	50	15	3.3	32	6	5.3	26	2.2	11.8	20.5	45
L7-BH12	44	10	4.4	30	6	5.0	22	2.1	10.5	19.9	50
L7-BH13	32	7	4.6	24	6.2	3.9	30	2.2	13.6	22.1	40
L7-BH14	42	12	3.5	28	6	4.7	26	2.2	11.8	20.0	50
L7-BH15	34	7	4.9	24	6	4.0	22	2.2	10.0	18.9	50
L7-BH16	46	12	3.8	32	5	6.4	24	2.3	10.4	20.7	45
L7-BH17	42	10	4.2	34	6	5.7	28	2.3	12.2	22.0	40
L7-BH18	36	6	6.0	24	6	4.0	21	2.2	9.5	19.5	50.5
L7-BH19	42	10	4.2	32	6.2	5.2	23	2.1	11.0	20.3	47.5
L7-BH20	44	12	3.7	30	6.5	4.6	25	2.2	11.4	19.6	50.0

Table 2. Results on Rock Tunneling Quality Index (Q)

Location	RQD	Joint Set Number (J_n)	Joint Roughness (J_r)	Joint Alteration (J_a)	Joint Water (J_w)	Stress Reduction Factor (SRF)	Q Rating
L7-BH01	43.39	6	1.5	2	1	4	1.36
L7-BH02	48.65	6	1.5	3	1	4	1.01
L7-BH03	54.86	9	1.5	2	1	4	1.14
L7-BH04	39.88	6	1.5	2	1	4	1.25
L7-BH05	43.12	6	1.5	2	1	4	1.35
L7-BH06	48.34	6	1.5	3	1	4	1.01
L7-BH07	47.44	6	1.5	3	1	4	0.99
L7-BH08	50.49	6	1.5	3	1	4	1.05
L7-BH09	49.55	6	1.5	3	1	4	1.03
L7-BH10	46.70	6	1.5	2	1	4	1.46
L7-BH11	47.40	6	1.5	2	1	4	1.48
L7-BH12	49.41	6	1.5	3	1	4	1.03
L7-BH13	42.14	6	1.5	2	1	4	1.32
L7-BH14	49.05	6	1.5	3	1	4	1.02
L7-BH15	52.77	6	1.5	3	1	4	1.10
L7-BH16	46.80	6	1.5	2	1	4	1.46
L7-BH17	42.27	6	1.5	2	1	4	1.32
L7-BH18	50.50	6	1.5	3	1	4	1.05
L7-BH19	47.96	6	1.5	3	1	4	1.00
L7-BH20	50.17	6	1.5	3	1	4	1.05

Table 3. Rock Mass Rating Results.

Location	UCS	Rating	RQD	Rating	Spacing (mm)	Rating	Conditions of Discontinuity	Rating	Groundwater	Rating	Adjustment for Orientation	Rating	RMR
L7-BH01	143	14	43.39	6	200	10	1mm-5mm	10	Completely dry	15	Unfavourable	-10	45
L7-BH02	143	14	48.65	8	48	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	42
L7-BH03	143	14	54.86	8	200	10	1mm-5mm	10	Completely dry	15	Unfavourable	-10	47
L7-BH04	143	14	39.88	6	100	8	1mm-5mm	10	Completely dry	15	Unfavourable	-10	43
L7-BH05	143	14	43.12	6	57	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	40
L7-BH06	143	14	48.34	8	99	8	1mm-5mm	10	Completely dry	15	Unfavourable	-10	45
L7-BH07	143	14	47.44	8	201	10	1mm-5mm	10	Completely dry	15	Unfavourable	-10	47
L7-BH08	143	14	50.49	8	56	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	42
L7-BH09	143	14	49.55	8	49	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	42
L7-BH10	143	14	46.70	8	55	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	42
L7-BH11	143	14	47.40	8	43	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	42
L7-BH12	143	14	49.41	8	101	8	1mm-5mm	10	Completely dry	15	Unfavourable	-10	45
L7-BH13	143	14	42.14	6	79	8	1mm-5mm	10	Completely dry	15	Unfavourable	-10	43
L7-BH14	143	14	49.05	8	200	10	1mm-5mm	10	Completely dry	15	Unfavourable	-10	47
L7-BH15	143	14	52.77	8	51	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	42
L7-BH16	143	14	46.80	8	53	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	42
L7-BH17	143	14	42.27	6	56	5	1mm-5mm	10	Completely dry	15	Unfavourable	-10	40
L7-BH18	143	14	50.50	8	200	10	1mm-5mm	10	Completely dry	15	Unfavourable	-10	47
L7-BH19	143	14	47.96	8	103	8	1mm-5mm	10	Completely dry	15	Unfavourable	-10	45
L7-BH20	143	14	50.17	8	200	10	1mm-5mm	10	Completely dry	15	Unfavourable	-10	47

Table 4. Results on Mining Rock Mass Rating

Location	UCS	UCS Rating	RQD	RQD Rating	Joint condition and ground water rating	Joint spacing adjustment factor	Joint spacing rating	MRMR
L7-BH01	143	14	43.39	6	11	0.258	6	37
L7-BH02	143	14	48.65	8	11	0.270	7	40
L7-BH03	143	14	54.86	8	11	0.210	5	38
L7-BH04	143	14	39.88	6	11	0.229	6	37
L7-BH05	143	14	43.12	6	11	0.176	4	35
L7-BH06	143	14	48.34	8	11	0.222	6	39
L7-BH07	143	14	47.44	8	11	0.229	6	39
L7-BH08	143	14	50.49	8	11	0.229	6	39
L7-BH09	143	14	49.55	8	11	0.266	7	40
L7-BH10	143	14	46.70	8	11	0.222	6	39
L7-BH11	143	14	47.40	8	11	0.299	6	39
L7-BH12	143	14	49.41	8	11	0.218	5	38
L7-BH13	143	14	42.14	6	11	0.210	5	36
L7-BH14	143	14	49.05	8	11	0.166	4	37
L7-BH15	143	14	52.77	8	11	0.210	5	38
L7-BH16	143	14	46.80	8	11	0.270	7	40
L7-BH17	143	14	42.27	6	11	0.201	5	36
L7-BH18	143	14	50.50	8	11	0.210	5	38
L7-BH19	143	14	47.96	8	11	0.210	5	38
L7-BH20	143	14	50.17	8	11	0.176	4	37
				Average MRMR				**38**

4.3. Mining Rock Mass Rating (MRMR)

Laubscher adapted Bieniawski's Rock Mass Rating to develop the Mining Rock Mass Rating. According to Laubscher [24], the MRMR is calculated by adding the following four key parameters together: 1) Rock material strength (UCS), RQD, Joint spacing, Joint condition, and groundwater. The calculated values of MRMR for each borehole are presented in Table 4. The minimum and maximum values of MRMR value recorded where 36 and 40 respectively, and the mean value is 38 and will be used to calculate the desired rock mass strength. Applying Bieniawski's Rock Mass Rating (RMR), Laubscher's Mining Rock Mass Rating (MRMR), and Barton's Q-system provided a robust, multi-dimensional assessment of rock mass quality. The high correlation among these systems demonstrated their reliability in evaluating poor-quality rock masses. However, these classification methods rely on empirical parameters that may not account for site-specific geological complexities or the dynamic nature of underground environments. In future studies, incorporating advanced data-driven techniques, such as machine learning, could improve rock mass classification accuracy and site-specific adaptability. MRMR offered a mining-specific modification of the traditional RMR system, adjusting for mining-induced stresses and operational conditions. While this adaptation enhanced the relevance of the classification for the study site, its reliance on predefined adjustment factors may oversimplify complex geological behaviors. Developing dynamic adjustment models considering real-time stress distribution and geological variability could address this limitation.

4.4. Determination of Support Requirements

The design of underground excavations benefits from several well-established, empirical, and semi-empirical rules. These rules enable approximations of the expected mining conditions and support requirements based on a detailed rock mass description. The design recommendations derived from the empirical relationships are adjusted for domain variability, expected excavation performance, and engineering judgment.

4.4.1. Using Barton's Q Chart [3]

This chart is a robust design tool developed based on case histories of tunnels, shafts, and caverns collated over the years and accommodates advances in support technology. The chart provides support recommendations based on various combinations of rock quality (Q values) and Equivalent Span, which is calculated by the ratio of the span and the Excavation Support Ratio (ESR). The ESR is a factor Barton employs to account for varying degrees of permissible instability, considering the excavation's service life and intended usage. Given that the tunnels in the research area are classified as permanent mine openings, a theoretical ESR value of 1.6 is necessarily assigned. The graph in Figure 6 indicates systematic bolting with a bolt length of 1.95m with unreinforced shotcrete between 4-6cm thick.

4.4.2. Using Barton's Equation [3].

Barton et al [3] formulated the equation to determine the required length of rock bolts or cables. According to Chen [8], a crucial aspect of support design is ensuring that the tendon extends far enough into the stable hanging wall to create a secure anchor, forming a stable beam. The formula (in the absence of statistical data reflecting the height of rock requiring support) is as in equation (3):

$$Length\ of\ rock\ bolt = \frac{2+0.15\times B}{ESR} \tag{3}$$

In this equation, ESR is the safety factor that accounts for the excavation's importance, and B is the width of the excavation considered. Barton [4] recommended an ESR value of 1.6 for permanent mine excavation, including tunnels. Thus, for a tunnel with a span of 4m and a mean ESR value of 1.6, the rock bolt length is calculated to be of 1.8 m. By adding the 0.3m maximum allowance for protruding rock bolts and 0.13m for the threaded portion of the rock bolt, the total required length of the rock bolt is 2.23m

4.4.3 Using Stacey and Swart [32] design rules

As a rule of thumb, for reasonable rock conditions, the length of bolts should be one-third the span of the wall height [31]. For very good rock mass conditions, this multiplier could be as low as 0.25; for a poor-quality rock mass, it could be as high as 0.5 (or even higher). The rock mass classification results concluded that the rock mass is of poor quality, so the suitable multiplier is 0.5. The length of a tendon will be this multiplier multiplied by either the Span or height of excavation in where they are to be installed; then it will be 0.5 × 4 m = 2.0 m

4.4.4. Using the Fallout height method

The fallout height, representing the height of 95.0% of the FOGs, was determined by statistically analyzing historical FOG data from the mine's database to estimate the thickness of the hanging wall. Based on this analysis, the fallout height was identified as 2.0 meters, corresponding to the cumulative frequency of 95% (Figure 7), and was calculated using the raw fallout thickness data presented in Table 5.

A key limitation of using FOG data to determine support requirements is the potential for the database to be outdated, leading to inaccuracies. In this case, the FOG data referenced in Table 1 was last updated in 2019. Considering the possibility of changes in geological conditions and mining activities since then, caution is warranted when relying solely on this data. The results should be interpreted with other relevant data and informed expert judgment to ensure accurate and reliable support design.

Stacey and Swart [32] recommended that rock bolts be at least 0.2m longer than the potential fallout thickness to securely anchor and prevent failure along potential weaknesses or splitting planes. Rock bolt holes should be drilled vertically, systematically, and methodically for optimal utilization of the bolt length. A 0.1m protrusion should be added to the bolt length, resulting in a total tendon length that is 0.3m longer than the fallout thickness. Based on the fallout thickness, the minimal tendon length is 2.3 meters.

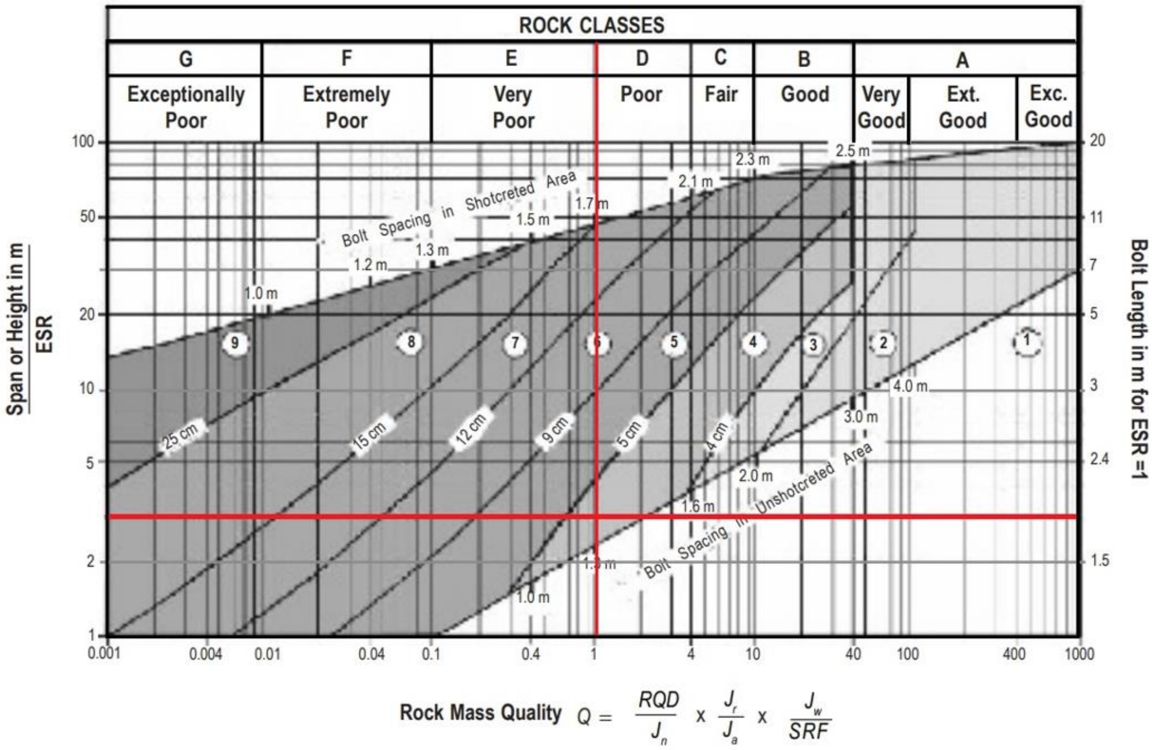

Fig.6. Barton's Q Chart (2002)

Table 5. Fallout thickness raw data

Height (m)	0	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.4	2.6
Occurrence	0	2	2	4	6	8	19	20	12	17	5	3	1	1
Cumulative frequency	0	2	4	8	14	22	41	61	73	90	95	98	99	100

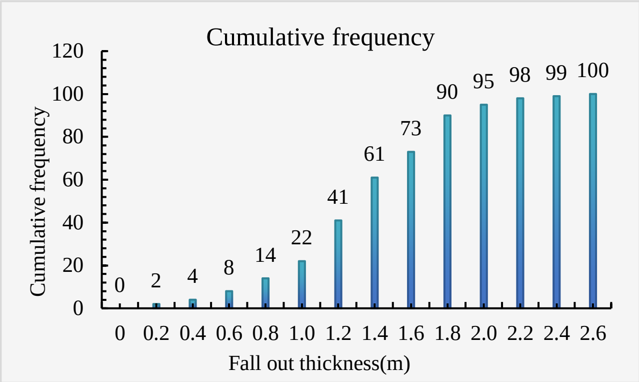

Fig.7. Fallout height determination

From these four methods, a sound support system would have 2.3m long tendons since it is sufficient to effectively manage the average fallout thickness at the mine. This is shown in the following calculations. This research area's hanging wall rock mass density is 2600 $kg.m^{-3}$. The suggested support consists of 16 tonnes of strength tendons spaced 1.3m x 1.3m with a bolt length of 2.3m. The tensile strength will be 156.96 kN with an area of 1.69 m^2. The support demand will be 51.01 kNm^{-2}, and the support resistance will be 92.88 kNm^{-2} with a tensile strength of 137.34 kN. The 2.3m long bolts provide a FOS of 1.82, deemed acceptable. Hoek and Brown [21] noted that a FOS as low as 1.56 could be considered sufficient, making the current design even more conservative. Hoek and Brown [23] suggest that a FOS of 1.3 is suitable for temporary mine openings, while a value of 1.5 to 2.0 is required for permanent excavations.

This research paper recommended closer spacing of tendon support units in the research area due to the presence of geological faults and fractures. The 1.7-meter-long roof bolts currently used at Level 7, initially designed for areas with fair ground conditions, have proven inadequate in maintaining roof stability in zones with poor ground conditions. The frequent occurrence of roof falls in these areas emphasizes the insufficient anchorage capacity of the existing bolts, underscoring the urgent need for a more robust support system to address the risks posed by unstable ground conditions. While increasing support density can help reduce localised Fall of Ground, it is insufficient in this case, as the primary objective is to securely fasten and consolidate the overlaying rock strata. The failure of the current support systems, which utilises shorter roof bolts, necessitates designing a new tendon system to provide adequate support.

4.5 Joints distribution across the mine

The J-Block software has been used to analyze joint data and predict the probability of rock falls. This software evaluates the probability of potential rock falls in mining excavations. Creating a detailed underground map of every individual stress fracture and joint is not entirely feasible. To overcome this challenge, the software generates simulated blocks in the hanging wall based on joint patterns to better understand the rock's behavior. The simulation uses statistical techniques to pinpoint potentially unstable key blocks, helping identify areas at risk of failure. Overall, joint data analysis results retrieved from level 7 are summarized in Table 6. With the summarized data, we can forecast the likelihood of rock falls and assess the effectiveness of support measures, enabling a more comprehensive rock fall risk management strategy.

4.5.1 Key block size distribution

The simulation outputs provide a comprehensive picture of rockfall risks, including the distribution of key block sizes, failure probability distributions, and the dominant failure and stability mechanisms, enabling a more informed approach to rock fall mitigation and management. The dimension and geometry of individual blocks play a crucial role in assessing the potential rock fall thickness (fallout thickness) and the necessary support strength required to prevent or mitigate rock falls [36,37]. J-Block effectively addresses both critical aspects during simulation, yielding outcomes identifying key block sizes that can fall between supports and predicting support failure scenarios.

The simulation results indicate that 65% of key blocks formed are smaller than one cubic meter in size (Figure 8). As the block size increases, the key block size distribution also decreases. The probability of 1m^3 blocks falling between supports is 19% (Figure 8). An inverse relationship exists between block size and key block size distribution, meaning that the distribution of key block sizes decreases as block size increases. Despite the decrease in key block size distribution, the probability of blocks greater than 1m^3 falling due to support failure increases.

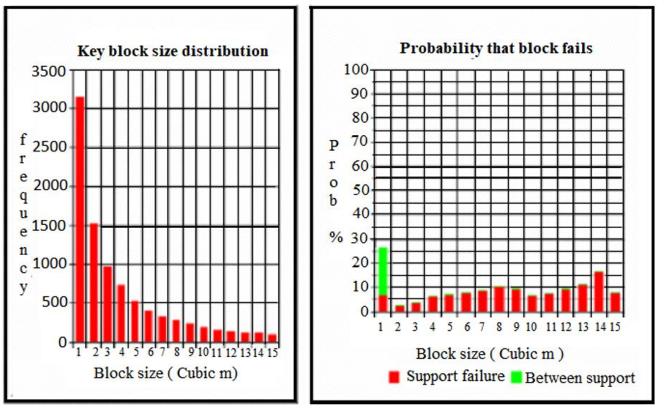

Fig. 8. Key block size distribution and Probability that block fails

4.5.2 Block failure modes and stability modes

The J-Block detects rock failures in four distinct modes: single plane, double plane, drop out, and rotation. There is a 94% likelihood that 1m^3 key blocks will fail due to dropout, primarily because these small blocks can easily fall through the gaps between support units. As shown in Figure 9, the key block size and rotation failure mode increase together. The probability of dropout failure decreases with increasing key block size. The angle for sliding planes for key blocks is relatively steep, falling within 60° and 80° (Figure 10). As block size increases, the probability of rotational failure also rises. This is due to greater exposure to stress concentrations around joints, fractures, and other discontinuities, which makes the blocks more susceptible to such failure. The stability mode results (Figure 11) indicate that friction is crucial in stabilizing all block sizes rather than relying on external support for stability.

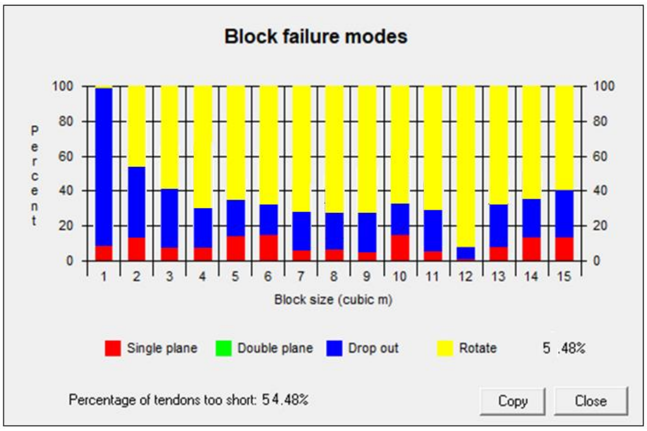

Fig.9. Block failure modes.

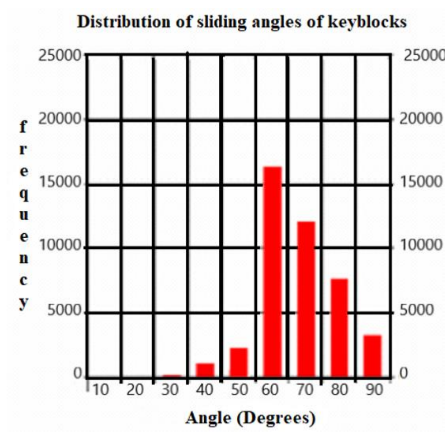

Fig. 10. Distribution of sliding angles of key blocks

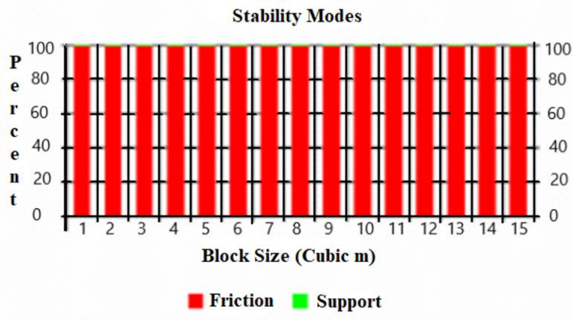

Fig. 11. Stability modes: Percentage versus Block Size 4.6 Support analysis.

4.6 Support analysis.

Excavation stability depends on the support layout and capacity of the support system. Failure happens when the weight of key blocks surpasses the support system's limit. Consequently, the support systems used must be sufficient to ensure the stability of the excavation. According to the analysis, failures are most likely to occur between support units for 1m^3 key blocks. As a result, the support spacing significantly influences the overall stability of the key blocks, affecting the resistance provided by the supports.

The 2.3m long, 150kN non-grouted roof bolts were tested in three different support spacing layouts to assess their performance. The relationship between support spacing and failure probability for a 1m³ key block is presented in Table 7. Decreasing the support spacing decreases the chances of small key blocks falling between support units. Conversely, decreasing the support spacing will require more support, thereby driving up costs. Furthermore, the probability of support failure increases when support units are spaced closely together, as shown in Table 7. Support failure can happen when the capacity of the support is insufficient to meet the necessary resistance demands. Further models were run with varied support lengths and capacities, keeping a fixed 2m x 2m, to mitigate the risk of support units being too short, as detailed in Table 8. Based on these results, it can be concluded that the probability of support failure decreases with increasing support capacity for the same support length and spacing. The findings suggest that, for a given support length and spacing, the likelihood of support failure decreases as the support capacity increases.

J-Block software has validated the application of joint data for predicting rock fall probability. Using the method proposed by [18], the estimated average thickness of fallout in the mine was determined to be 2 meters (Table 7 and Figure 7). Probabilistic analysis can be approached to assess the likelihood of rock falls, informing the design of support measures to improve stability.

Table 6. Overall joint data

Joint Set number	Dip	Dip direction	Range	Spacing (m)			Length (m)		
				Mean	Min	Max	Mean	Min	Max
J_1	50.0	0.0	10.0	2.6	1.5	4.0	10.0	2.0	19.5
J_2	65.0	180.0	10.0	3.0	1.0	6.0	8.0	4.0	18.0
J_3	80.0	315.0	10.0	2.5	0.9	5.0	14.0	6.0	21.0

Table 7. Effect of support spacing on the probability of failure.

Tendon spacing (m)	2.0 x 2.0		1.5 x 1.5		1.0 x 1.0	
Failure type and Block size (m^3)	Probability of support failure (%)	Probability of Fall between support (%)	Probability of support failure (%)	Probability of Fall between support (%)	Probability of support failure (%)	Probability of Fall between support (%)
1	1.5	5.9	1.8	4.5	2.4	3.8
2	6.1	0.3	8.7	0.1	3.1	0.0
3	9.7	0.1	12.8	0.0	5.5	0.0
4	13.3	0.0	13.9	0.0	9.9	0.0
5	9.1	0.0	18.9	0.0	7.1	0.0
6	8.4	0.0	23.5	0.0	19.7	0.0
7	9	0.0	19.8	0.0	18.2	0.0
8	7.6	0.0	23.7	0.0	27	0.0
9	11.9	0.0	18.9	0.0	13.4	0.0
10	11.8	0.0	25.3	0.0	31	0.0
11	17.9	0.0	18.6	0.0	38.4	0.0
12	15	0.0	28.1	0.0	30.3	0.0
13	13.9	0.0	9.1	0.0	22.6	0.0
14	8.9	0.0	23.3	0.0	25.8	0.0
15	11.2	0.0	21.9	0.0	29.1	0.0

Table 8. Effect of support length and support capacity on stability.

Effect of Tendon length and Capacity on 2m x 2m constant spacing									
Capacity (kN)	130			150			200		
Tendon length (m)	2	2.3	2.5	2	2.3	2.5	2	2.3	2.5
Block size (m^3)	Probability of support failure (%)			Probability of support failure (%)			Probability of support failure (%)		
1	1.5	1.2	1.4	1.2	1.3	1.2	1.1	1.3	1.2
2	6.1	6.3	6.5	5.5	5.1	6.0	3.3	4.8	4.9
3	9.7	10.8	9.5	8.8	10.2	9.9	5.7	8.4	7.9
4	13.3	12.4	8.9	10.5	11.0	12.4	7.6	7.4	8.7
5	9.1	11.1	8.9	12	10.1	6.3	11.1	8.2	9.6
6	8.4	9.3	16	9.9	7.5	7.5	10.5	11.1	7.7
7	9.0	14.4	11.9	12	18.4	15.1	10.8	7.7	13
8	7.6	10.6	16.6	12.5	12.9	12.2	12.8	6.1	13.4
9	11.9	11.2	16.8	11.9	11.6	10.5	9.7	13.4	11.0
10	11.8	13.7	23.3	15.5	16.1	18.2	8.8	16.3	7.9
11	17.9	12.1	17.2	8.6	14.4	13.8	13.7	12.3	12.5
12	14.9	7.4	10.7	9.5	4.3	8.9	6.7	7.3	9.9
13	13.9	18.3	13.1	9.7	14.9	11.5	11.8	11.8	22.1
14	8.9	14.2	24.5	9.3	12.0	16.1	14.3	11.5	13.9
15	11.2	12.9	16.9	9.7	10.6	7.9	19.2	7.5	8.8

4.7 Stability Analysis of the excavation with the change in support spacing.

Further simulations were carried out to understand better the effects of support spacing on the excavation stability. The joint sets detected in the field through structural mapping were leveraged to generate the wedge geometries across the excavation. The intersection of three joints creates a three-dimensional tetrahedral shape, commonly called a wedge—the wedge analysis aimed to identify the optimal support spacing for ensuring the long-term stability of the excavation. An analysis entailed the installation of a support system across the excavation with varying support spacing (Figure 12). The simulation results indicate a significant enhancement in the Factor of Safety for the wedges across the excavation as the support spacing decreases (Table 9). With a reduction in support spacing, the number of installed support tendons increases, improving excavation stability. The simulation confirms that using short spacing in the jointed rock mass is likely the most effective approach to achieving long-term stability of the excavation.

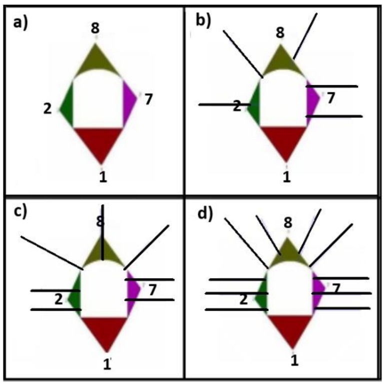

Fig. 12. 2D distribution of support units across the excavation with different support spacing (a) no support unit installed, (b) 2.0m x 2.0m (c) 1.5m x 1.5 (d) 1.0m x 1.0m

Table 9. Simulated Safety Factor, Apex, and failure mode of the wedges across the excavation.

Spacing of bolts	Criteria	Roof Wedge (8)	Lower-Right Wedge (7)	Lower-Left Wedge (2)
	Factor of Safety	1.534	1.150	1.012
No Support	Weight of the wedge (MN)	0.286	0.105	0.098
	Apex Height (m)	1.65	0.82	0.80
	Factor of Safety	1.861	1.235	1.186
1.0 m x 1.0 m	Weight of the wedge (MN)	0.286	0.105	0.098
	Apex Height (m)	1.65	0.82	0.80
	Factor of Safety	1.754	1.130	1.002
1.5 m x 1.5 m	Weight of the wedge (MN)	0.286	0.105	0.098
	Apex Height	1.65	0.82	0.80
	Factor of Safety	1.702	1.110	1.002
2.0 m x 2.0 m	Weight of the wedge (MN)	0.286	0.105	0.098
	Apex Height	2.05	1.08	1.12

It was noted that as the spacing of the support units reduces, the number of support units installed across the wedge increases, and as a result, the Safety Factor of the wedge increases as well as the stability of the excavation. The simulation results suggest that support spacing has a more significant impact than other factors related to the support systems and wedges. Thus, a well-designed support system is essential for mining stability in underground excavations.

This study highlights the advanced integration of multiple methods to create a tailored, site-specific solution for optimizing ground support systems. The methodologies offer a significant leap forward in predictive and design capabilities for tunnel stability in challenging environments. However, limitations related to data collection, model assumptions, and the high technical demands of advanced tools underline the need for further development. Future research should address these limitations by incorporating real-time monitoring technologies, machine learning algorithms, and enhanced modeling techniques to make these advanced methodologies more accessible and adaptable to diverse mining conditions. The framework developed in this study provides a valuable foundation for improving underground tunnel stability. Its innovative combination of advanced tools and traditional methods offers a practical approach to mitigating geotechnical risks, setting the stage for future advancements in mining and geotechnical engineering.

6. Conclusions

Unfavorable geological grounds pose a high risk of fall-of-ground incidents in underground mining. Rock falls and ground failures are the leading causes of fatalities in the mining industry. Fall-of-ground incidents occur when there is no or insufficient ground support in place. The primary objective of this research was to determine the optimal support systems for tunnels passing through unfavorable geological grounds to reduce the risk of fall of ground incidents. Tunnel Reflection Tomography was employed to predict the location of unfavorable geological grounds, which is an advanced detection method.

A rock mass classification exercise was conducted to find the quality of the rock mass. Three rock mass classification systems were used to determine the quality of the rock mass. In addition, the Tunnel Reflection Tomography (TRT) technique was used to predict geotechnically challenging grounds ahead of the working face. The analysis of 20 borehole core samples revealed poor rock mass quality, with a significant correlation among the three systems. The limitations of each system highlighted the need for multiple classification methods. While most joints were assumed dry, potential groundwater infiltration, resulting in cohesion weakening and unstable key blocks, should be considered.

The rock mass classification exercise is critical to all evaluations in this study and should be conducted with precision and accuracy. The analysis of the rock mass classification results has led to the general conclusion drawn in this study. Empirical methods can be used to design support for stability enhancement. Evaluating the support is crucial for ensuring the stability of excavations. Stability analysis was also conducted through factors of safety simulation using UnWedge, and based on the simulation result, a support spacing of 1.0m x 1.0m was found to be ideal. Probabilistic analysis helps predict rock fall probability and optimize support design for stability. Large key blocks require support units with resistance greater than their weight. The mine is advised to introduce probabilistic analysis to improve the FOG management system and design support for highly jointed ground conditions. The findings of this research on optimizing ground support systems in geologically challenging conditions are invaluable for mining professionals, geotechnical engineers, and mining organizations. By improving tunnel stability, enhancing operational safety, and minimizing downtime caused by geological instability, this work contributes significantly to advancing mining practices. It also lays a foundation for future innovations in tunnel support technology, promoting more cost-effective and sustainable solutions within the industry. This study is essential for reducing accident risks, safeguarding workers, and ensuring the long-term success of mining projects—factors critical to the economic and operational stability of the mining sector.

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Conflicts of Interest

The authors affirm that this work was free from any conflicts of interest and that no financial backing or sponsorship influenced the results or conclusions presented in this publication.

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APPENDICES

Appendix A: Classification of individual parameters used in the Tunnelling Quality Index Q

1. JOINT SET NUMBER (J_n)

DESCRIPTION	VALUE
A. Massive, few random joints	1.0
B. One joint set	2.0
C. One joint set plus random joints	3.0
D. Two joint sets	4.0
E. Two joint sets plus random joints	6.0
F. Three joint sets	9.0
G. Three joint sets plus random joints	12.0
H. Four or more joint sets, random heavily jointed	15.0
I. Crushed rock	20.0

2. JOINT ROUGHNESS NUMBER (J_r)

DESCRIPTION	VALUE		
	Discontinuous	Undulating	Planar
A. Rough or irregular	4.0	3.0	1.5
B. Smooth	3.0	2.0	1.0
C. Slickensided	2.0	1.5	0.5
D. +5mm thick gouges	1.5	1.0	1.0

3. ROCK QUALITY DESIGNATION (RQD)

DESCRIPTION	*VALUE*
A. Very poor	0 – 25
B. Poor	25 – 50
C. Fair	50 – 75
D. Good	75 – 90
E. Very good	90 – 100

Note 1: Where RQD<10, use value of 10
and if RQD > 100, use value of 100

4. JOINT ALTERATION NUMBER (J_a)

DESCRIPTION	Fill < 1mm	Fill 1 – 5mm	Fill > 5mm
A. Tightly healed, hard rockwall joints, (Quartz)	0.8	1.0	2.0
B. Unaltered joint walls	1.0	2.0	3.0
C. Non-cohesive mineral (Calcite)	2.0	4.0	6.0
D. Serpentinite/Talc Infill	3.0	6.0	10.0
E. Clay	4.0	8.0	12.0
F. Shattered Zones or crushed rock	5.0	10.0	12.0

5. JOINT WATER (J_w)

DESCRIPTION	VALUE
A. Dry	1.0
B. Wet/Moist	0.8
C. Dripping water (< 5litres/min)	0.5
D. Gushing > 101/min	0.1

6. STRESS REDUCTION FACTOR (SRF)

DESCRIPTION	VALUE
A. No shear, faults,	1.00
B. One shear/fault or major E-W joints with opposing dips.	2.50
C. One shear/fault and blocky ground	4.00
D. Multiple faults or dykes	6.00
E. Curved Low angled joints or domes	7.50
F. Joints sub // to advance direction (same/opposing dips)	8.00
H. Multiple faults/Wide shears mylonite zones	10.00