International Journal of Basic and Applied Sciences, 14 (2) (2025) 420-428

International Journal of Basic and Applied Sciences

Website: www.sciencepubco.com/index.php/IJBAS
ttps://doi.org/10.14419/g0j8r379
Research paper

Evaluating The Stability of Dyke At 740 Decline Mupane Gold Mine, Botswana

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Received: May 27, 2025, Accepted: June 5, 2025, Published: June 27, 2025

Abstract

This work is focused on evaluating the Stability of Dolerite Dyke at 740 Decline of Mupane Gold Mine. The wire meshes used to support the Dolerite Dyke, proved to insufficient and inadequate to sustain the pressure coming from the supported Dolerite Dyke and they are always at is a risk of unexpected rock falls. The work undertook the investigation of the strength of the Dolerite Dyke, and estimation of the required mean fall out weight evaluating the and extend of failure mechanism. Discontinuity data (mainly joints) obtained through the Scanline Mapping were used to analyze the rock using the RS2 software. The rock mass classification method includes RQD, which is 29.9, RMR is 58, and Q tunneling is 0.2666. These classification methods agree that the Dolerite Dyke is of inferior quality and support is recommended as per given support guidelines. It is found that the Dolerite rock has several instability problems and needs to be addressed. To reconfirm the failure limit, depth was increased to test the stability analyses of the Dolerite Dyke. This showed that as depth was increased, the in situ stress increases, creating deformations that create instability issues. Dips software analyzed the joints sets which were discovered to weaken the rock strength which made the Dolerite Dyke susceptible to failure. From unwedge analysis we the maximum volume of key books is 25cm2. Which individually cannot activate failure. From RS2 the dyke- section experiences deformation even after support installation. Installation of 2.3 m long full column pre-tensioned resin bolts into the hanging wall and sidewalls at an angle of 90 degrees, and application of 25mm thick shotcrete to the hanging wall and sidewalls after the installation are recommended to compensate for blocky ground conditions

Keywords: *Dolerite Dyke, Mupane, Q System, RS2, RMR, RQD*

1. Introduction

Dolerite intrusions in the mines are always a big problem concerning slope stability. [1] have worked on the geotechnical character of some South African Dolerites, especially their strength and durability. Some Dolerites in Natal show a very fast deterioration, probably due to the presence of chlorite and clay minerals present in the Dolerites. [2] has presented a Case study of Zimbabwean Great Dyke Platinum Mining and have suggested a Support Design in Geotechnically Challenging Ground Conditions. [3] have researched ground behaviour analysis, support system design, and construction strategies in deep hard rock mining in Western Australia's mines. Ground behaviour modes and failure mechanisms were identified and assessed. Ground demand for static and dynamic conditions was estimated, and a proper ground support system was selected and evaluated in site-specific conditions according to the proposed method for ground support design at great depth. [4] have emphasized about the geological and geotechnical challenges on the Great Dyke of Zimbabwe and their impact on hard rock pillar design. The Great Dyke's geological complexity includes diverse rock types—dunnites, harzburgites, pyroxenites, and norites—and notable structural features like joints, faults, and shear zones. These factors complicate the stability of underground workings. They advocated the use of Machine Learning and Artificial Intelligence for better risk management.

The Mupane Gold Deposit lies within the Tati Greenstone Belt (TGB), which comprises a NNW striking group of Archaean metavolcanic, metasedimentary, and intrusive igneous rocks trending over 65km strike length and up to 20 km width (Fig.1). [5] Subdivision of the belt into five formations has subsequently been modified by other workers. From youngest to oldest, the succession may be summarized as the Last Hope Formation, Penloghoha Formation, and the Lady Mary Formation.

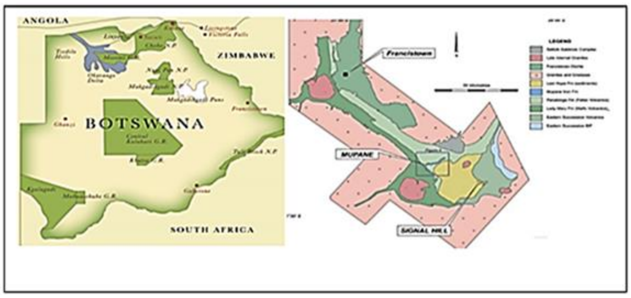

Fig. 1: Location of Mupane mine in Botswana

2. Mupane Tau Pit Geology

'Tau pit' is the largest gold deposit found at Mupane Gold Mines. It consists of two GIF-hosted lenses of gold mineralization, termed the northern and the southern lenses. The deposit has an east-west strike length of 377m and has been defined to a maximum depth of 400 m. The ore zones vary in width between 3 m and 60 m and dip to the south at 45 to 70 degrees. The Tau deposit is hosted within a Quartz-Biotite/Amphibolite-schist, minor Garnet Schists, and a Conglomerate unit. The western end of the mineralization is truncated by a Dolerite dyke, which is a part of the Karoo Dyke Swarm, and may be a remobilization of the dyke along and pre-existing fault/shear. The Grunerite associated with the Amphibolite Schist had weathered to form Nontronite Clays in parts of the oxide/transitional zone. The Tati Greenstone Belt (TGB) hosts numerous small-scale nickel and gold deposits. The gold is principally hosted within pyrite and, to a lesser extent, Pyrrhotite. Gold mineralization conforms to the Archaean Lode Gold or Orogenic Gold style of mineralization. The Tati Greenstone Belt (TGB) hosts numerous small-scale nickel and gold deposits, the gold is principally hosted within Pyrite and to a lesser extent Pyrrhotite. Gold mineralization conforms to the Archaean Lode Gold or Orogenic Gold style of mineralization

2.1 Origin of the Problem

Rock fall has become an ever-present threat in underground mining especially due to the Dolerite dyke. Tau pit underground project has been using wire mesh, nonetheless, the area remains a hazard because the wire mesh is inefficient to support the Dolerite dyke fallout. [6] refer to this approach as a "safety" support system because it eliminates the fall of small rocks; however, this has proved insufficient for holding bigger rock-fragments in Dolerite dyke sections. The chain-link wire mesh is continuously developing leakages (especially at the overlaps) where Dolerite dyke is being supported. This has led the slabs of Dolerite dyke to fall unexpectedly causing injuries to the workforce. The instability is induced by the intersection of three or more joints or fractures discontinuities in Dolerites. These blocks may fall or be ejected into the excavation with little or no warning and may pose a serious risk of equipment damage, injury, or death.

2.2 Objectives

The current work was focused on the assessment of failure mechanisms based on the relative stabilities of the rock mass aggregate in the Tau Pit and recommending suitable safety measures.

3. Methodology

To analyses the stability of Dolerite Dyke at Mupane Gold Mine, numerous activities have been undertaken in order to acquire relevant data and the process.

3.1. Geotechnical Scanline Mapping

The scanline mapping underground was used to find the condition of the Decline. It was done with an aim to assess the characteristics of the discontinuities with major emphasis on joints, such as spacing, frequency, persistence, shape, size, roughness, aperture, etc.

The results obtained from the scanline mapping were also used to calculate the RMR and the Q system. The scanline mapping was conducted by marking the grade line, which was 1.5 m from the ground, and highlighted all the structures, such as faults and joints that intersect the grade line to inspect and find the joint sets appearing, the dip angles, the condition of the discontinuity, and any alterations. Clinoruler was used to measure the dip angle and spray paints to mark visible structure, and tape for measuring the distances between the structures.

3.2. Laboratory Experimentation

3.2.1 UCS Point Load Test

Point load testing was carried out to determine rock strength indexes in geotechnical practice. The point load test apparatus and procedure enable economical testing of core or lump rock samples in either a field or laboratory setting. To estimate uniaxial compressive strength, index-to-strength conversion factors are used.

3.2.2 Determination of RMR and Q Values

The RMR and Q values were determined to express the quality of the ground with respect to tunnel stability. The Rock Quality Designation has been applied to find the quality of the Dykes in question.

3.2.3 Rock Quality Designation

RQD was estimated from the number of discontinuities per unit volume as per the following relationship:

$$RQD = 115 - 3.3Jv \quad (1)$$

here, Jv is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count. It was calculated as per the following formula:

Jv= 1/Si (2)

Where: S is the joint spacing for the actual joint set
And RQD = 100% when Jv ≤4.5
The correlation between the Jv and RQD is shown in Figure.2

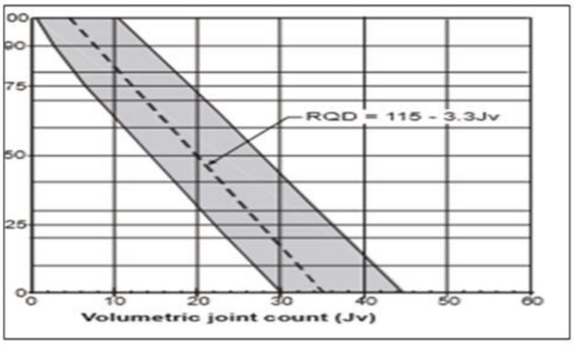

Fig.2: Correlation of JV, RQD with the variation range

[7] Discovered that the use of the volumetric joint count can be quite useful in reducing the directional dependency, as the RQD is a directionally dependent parameter, and its value may change significantly. When using Palmström's relationship for exposure mapping, blast-induced fractures should not be included when estimating Jv.

3.2.4 Rock Mass Rating

Rock mass Rating and came published the details of a rock mass classification called the Aeromechanics Classification or the Rock Mass Rating (RMR) system [8]. The following six parameters are used to classify a rock mass using the RMR system

- Uniaxial compressive strength of rock material.
- Rock Quality Designation (RQD).
- . Spacing of discontinuities.
- Condition of discontinuities.
- Groundwater conditions.
- Orientation of discontinuities.

A guide table for rock mass rating is often used when performing core logging. After accessing a given rock mass, one decides which group the parameter lies in, and then the results will be summed to give the value of the RMR. Having obtained the RMR, a set of guidelines published by [8] was used to select the suitable tunnel support or for estimating the tunnel stand-up time [9].
According to the RMR system, the figure below illustrates the relationship between stand-up time and the span for various rock mass classes according to the RMR system [8].
Through the RMR the standup up time is found the main advantage of the RMR system is that it is easy to use. Common Criticisms are that the system is relatively insensitive to minor variations in rock quality and that the support recommendations appear conservative and have not been revised to reflect the new reinforcement tool.

3.2.5 Q System

The following formula [10] was used to determine the value of Q:
tunneling index expressed as

$$Q = RQD/Jn * Jr/Ja * Jw/SRF \quad (3)$$

- RQD = Rock quality designation
- Jn = the number of joint sets
- Jr= discontinuity or roughness
- Ja = Alteration
- Jw = water condition
- SRF=Stress Reduction Factor

3.2.6 Review of Rock Mass Failure Criteria

When designing an underground excavation, knowledge of the rock mass strength is important. The strength can be estimated using a rock failure criterion. Four main classical failure criteria have been applied to rock

- Coulomb criterion,
- Mohr's envelope,
- Mohr-Coulomb criterion and
- Griffith's crack theory.

3.2.7 Classical Failure Criteria

Coulomb introduced a criterion based on research on the shear failure of glass. He found that the shear strength is dependent on the cohesion of the material and on a constant time the normal stress across the plane. The following is the shear resistance expression for masonry and soils, of the form

$$S = c.a + \frac{1}{n} N \tag{4}$$

Where c is the cohesion per unit area, a is the area of the shear plane, and N is the normal force.
A vast amount of information on the strength of intact rock has been published during the 20th century, and the intact rock strength is well understood. Based on the experience of failure criteria for intact rock, different rock mass failure criteria were developed. The failure criteria for rock masses are based on large-scale and laboratory testing, experience, and/or analysis. The most widely referred rock mass criteria are presented in Table 1.

Table 1: Rock Mass Failure Criteria [11]

FAILURE EQUATION	COMMENTS	AUTHOR/CRITERION/YEAR
$\sigma_1 = \sigma_3 + \sigma_{ci} [\, m_b \, x \, \sigma_{3/} \, \sigma_{ci} + s]^a$	2002 version	Hoek & Brown, 1980
$\sigma_1 = A\sigma_{ci} + B\,\sigma_{ci} [\, \sigma_{3/} \, \sigma_{ci}]^{\alpha}$	A is a dimension parameter and B is a rock material constant, α = 0.65	Yuddhbir et al., 1983
$\sigma_1 = \sigma_{cm} [1+ + \sigma_{3/} \, \sigma_{tm}]^{b}{}_{m}$	Use RMR_{76} value	Sheorey et al., 1989
$\sigma'_1 = \sigma'_3 + \sigma'_3 \, x \, B_j [\sigma'_{cj/} \, \sigma'_3]^{a}{}_{j}$	2001 version	Ramamurthy, 1995

They are all formulated in terms of sigma1 and sigma3 and are like the criteria for intact rock, independent of sigma2. These criteria were derived from triaxial testing of small rock samples.

3.3. Data Analysis through software

The data obtained from scanline mapping and laboratory tests were analyzed using the following software to interpret our results.

3.3.1 Unwedge Software

Unwedge was used to determine the stability of the rock wedges formed by the intersection of structural discontinuities. It calculates the factor of safety and determines the support requirement [12].

3.3.2 DIPS software

Dips were used to analyze structural data and visualize structural data using stereotypes to assess the stability of slopes and evaluate different possible types of slope failures.

3.3.3 RS2 Software

RS2 was used for modelling and conducting detailed analysis of support and tunnel interaction, and to assess the effect of each other, and to determine the effectiveness of the support installed

3. Results And Discussions

The information gathered was simplified into distinct categories for easy calculation of rock mass classification and the use of software when analyzing the data. Results obtained from the scanline are presented in the Table.2 below:

Table 2: Variation of Joint Spacing and Frequency

JOINTING	MIN. SPACING (M)	MAX SPACING (M)	MAX FRQUENCY	MIN FRQUENCY	AV. SPACING (M)	AV FRQUENCY
Joint Set 1	0.11	0.60	9.09	1.67	0.355	5.38
Joint Set 2	0.12	0.55	8.33	1.82	0.355	5.08
Joint Set 3	0.95	0.63	1.05	1.59	0.790	1.32
Joint Set 4	0.30	0.27	3.33	3.70	0.285	3.52
Joint Set 5	0.11	0.43	9.09	2.33	0.270	5.71
Joint Set 6	0.25	0.18	4.00	5.56	0.215	4.78
Volumetric Joint Count J_v			34.90 Max. J_v	16.66 Min J_v	0.380	25.78 Av. J_v

The table above summarizes the data obtained from the scanline mapping, which was used to calculate the Rock Quality designation. The clinoruler was used to measure the spaces between the joints in the joint sets.

4.1 Rock Mass Classification

According to [13], Rock Quality Designation was made to find the Quality of the rock, whether it is favorable for tunneling conditions. RQD is one of the simple classification systems of rock masses, which is used to find the stability of the rock masses. In an underground opening, it is usually possible to get a three-dimensional view of the rock mass. That means that the RQD value is estimated from the number of joints per m and calculated as follows

$$RQD = 115 - 3.3JV \quad (5)$$

(For J between 4 and 44)

$$115 - (3.3 \times 25.78) = 29.926 \quad (6)$$

The results obtained, indicates that the dyke falls in the category of 25- 50 % that classified into a poor rock.

4.2 Rock Mass Rating (RMR)

The rock mass rating is a Geomechanically classification of the rock developed by [8]. Each parameter is assigned a value corresponding to the character of the rock. The parameters were measured and obtained from the scanline mapping data. The results are presented in Table 3 below

Table 3: RMR Parameters

PARAMETERS	VALUES	RATINGS
Uniaxial Compressive strength	100-250	12
RQD	29.926	8
Joint Spacing	300-600	8
Condition of discontinuity	Very rough	25
Groundwater condition	damp	10
Rating adjustment for discontinuity orientation	fair	-5
Total Rating	58	

4.3 Q -System

According to Nick Barton Q system is a classification system for rock masses used during field mapping and underground openings. It uses some parameters in Rock Mass Rating. The value describes the rock mass quality. Q value helps in the rock support design decisions and documenting the rock mass quantity. High Q Value shows good stability, while a low value shows poor stability. Based on the following parameters, Q Value is calculated as follows, and the results can be seen in

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{jw}{SRF} \quad (7)$$

- RQD= Degree of jointing
- Jn= Number of Joint sets
- Jr= Joint roughness number
- Ja= Joint alteration
- Jw = Joint water reduction factor
- SRF=Stress Reduction Factor

Table 4: Q Parameters and Ratings

PARAMETERS	VALUES	Q- CALCULATION	
RQD	29.9	RQD/Jn	1.99
Jn	15		
Jr	2	Jr/ Ja	0.667
Ja	3		
Jw	1	Jw/ SRF	0.2
SRF	5		
		TOTAL Q VALUE	0.266

The Q values are related to diverse types of permanent support, with the schematic support chart visible in the appendix. This makes it easy to find the type and quantity of support that has been applied. From the result presented, Q is 0.266, which corresponds to very poor rock. [10] related Q Value with the support and the stability requirement of underground excavation using equivalent dimension (De), which is given by the ratio.

$$De = \ Excavation\ span\ /\ Excavation\ support\ ratio\ ESR \quad (8)$$

$$De = \frac{10}{16} = \ 6.25 \quad (9)$$

The excavation span is known to be 10, and the excavation ratio is design value depending on the use of the underground excavation. The figure in the appendix shows that the tunnel is a permanent opening to mine working is assigned 1.6, which is used to calculate the equivalent dimensions. The equivalent dimension indicated that the rock type requires support.

4.4 Discontinuity Analyses

Discontinuities were analyzed using the Dips software, which is a program designed for the interactive analysis of orientation-based data. Discontinuities weaken the rock mass by creating planes on which structural instability may occur, making it susceptible to failure. Therefore, the Dip aids in identifying the possible sets. The discontinuities found underground were measured considering the dip and dip direction. 30 readings were analyzed shown in the figure in the appendix, to identify and quantify the joint set prevalence in b 740 decline. The collected structural data is represented in the form of stereographical projections created with the Dip's software. The contours of the stereo net give the concentration of the poles about their orientations.

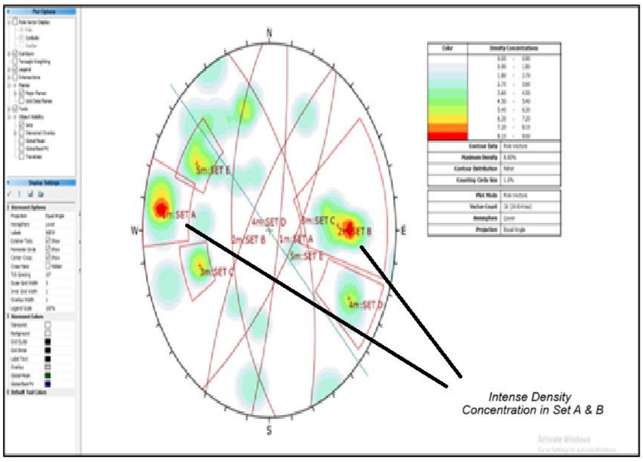

Fig.3: Kinematic Analysis

The above picture shows 5 joint Sets. Set A and SET B have intense density concentration as compared to the other sets. The Critical set was identified to be SET B because of the high-density concentration, but even though SET A has the high concentration, SET B has the joints that are smooth and undulated as compared to SET A, which can easily fail when it is analyzed.

The results obtained from all the Rock mass classification (RQD, Q Tunneling and RMR) all agree that Dolerite Dyke is of inferior quality, and it needs to be supported. The inferior quality is attributed to the joint zones that create planes, making the rock of low strength. According to Yang et al. (2023, the configurations of the rock joints, especially for non-randomly distributed joint sets, affect the mechanical behavior of the rock mass. Even though the rock classification methods are easy to use, they have some limitations that can compromise the accuracy of the results. According to P. Pells, the Q system fails to properly consider joint orientation, Joint continuity, and rock strength. The RQD/ Jn in the Q system does not provide a meaningful measure of relative block size, and the ratio Jw/ SRF does not give a meaningful measure of the stress acting on the rock mass to be supported. In scanline, RQD is directional, but due to its definition, it is more sensitive to the holes or line direction than joint spacing or fracture frequency measurements.

4.5 RS2 Analysis

This analysis has been carried out represents a rectangular-shaped tunnel with approximately a 10-meter span, to be excavated in heavily jointed rock. The rock is described as blocky and of inferior quality and will require support to prevent collapse. Table .9 shows the effects of Support on Excavation

Table 5: Effects of Support on Excavation

PARAMETERS	UNSUPPORTED	SUPPORTED	
		Bolts	Bolts and liners(m)
Total displacement	0.50m	0.410m	0.141m
Strength reduction	<2	<2	<2
Yielding (yielded elements)	630	600	81

In this first step of the Support, the model is analyzed without support. Both elastic and plastic analyses are performed. the most significant areas that are of most interest when performing the analysis are the strength factor, total displacement, and yielding of both the rock bolt and liners, with the rock mass assumed to be homogenous and isotropic.

4.6 Elastic Analysis without Support

Strength factor represents the ratio of available rock mass strength to induced stress at a given point. There is a large zone of overstress surrounding the tunnel. The region with less overstress is at the edges, and all the rock within the contour marked 1 has a strength factor less than 1 (based on the elastic analysis results in the figure below 1 and will probably fail if left unsupported. Though the area under the strength factor of 1 is less than 2 meters. This means that if any support is to be considered, then it must extend beyond the area.

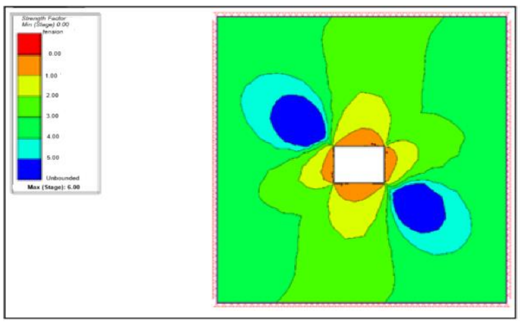

Fig.4: Elastic Analysis Without Support

The excavation model was then interpreted in terms of the overall displacement using deformation vectors. The elastic displacements show an inward displacement of the tunnel walls, as well as a significant floor heave. One of the reasons for such behavior from the tunnel is due to stress unloading, because of the high concentration of stress. The fact that if no rock mass to even out these forces, they tend to move to the area with less resistance, thus meaning this elastic analysis shows that the region of overstress is significant. As a result, support is most likely to be installed, as with time, failure will most likely occur.

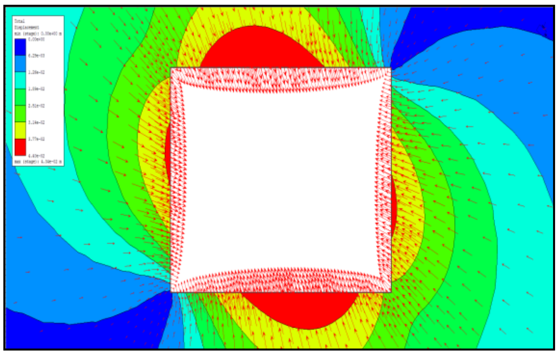

Fig. 5: Elastic Analysis without support showing Deformation Vectors

4.7 Plastic Analysis without Support

Notice the entire region around the excavation has a strength factor of approximately 1. For a plastic analysis, when failure (yielding) occurs, the strength factor is, by definition, equal to one, while in an elastic analysis, the strength factor can go below one as a hypothetical measure of overstress.
The plastic region has a strength factor region that is above 1, with those regions around the excavation proving to be more and needing support. The figure below shows Yielded Elements used to view the failure zone in a plastic model. The number of yielded elements (610 elements).
The zone of plastic yielding is observed (X = shear failure, O = tensile failure) around the excavation. Notice that the yielded zone roughly corresponds with the zone of strength factor < 1 from the elastic analysis, with additional propagation beyond this limit, as expected from plastic analysis. The area around the excavation boundary goes through tension, it will go through tensile failure. There is a bit of shear that goes beyond the yield zone.

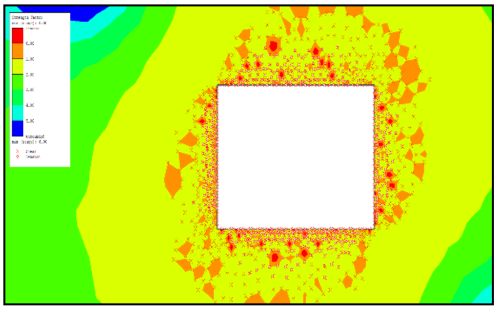

Fig.6: Plastic Analysis without support yielded zone

Notice the entire region around the excavation has a strength factor of approximately 1. For a plastic analysis, when failure (yielding) occurs, the strength factor is, by definition, equal to one, while in an elastic analysis, the strength factor can go below one as a hypothetical measure of overstress.
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4.8 Plastic Analysis with Support

The yielded zone, based on the extent and location of the yielded elements, is not distinguishably different from the unsupported yield zone. However, the number of yielded finite elements decreased from 910 (unsupported) to 723 (bolt support).
Almost all the bolts have yielded, as shown by the bolt sections highlighted in yellow. This indicates tensile failure of a bolt element. though "bolt elements" for fully bonded bolts are defined by the intersections of bolts with the finite elements, all the bolts have yielded, as shown by the bolt sections highlighted in yellow. This indicates tensile failure of a bolt element. "Bolt elements" for fully bonded bolts are defined by the intersections of bolts with the finite elements. From the figure below, the bolts have reached their yield capacity; they still provide support due to the bolt capacity being equal to the peak bolt capacity.
Looking at the effect of the bolts on the displacement, compared to the unsupported excavation, the displacements have been slightly reduced, meaning additional support must be installed, i.e., Shotcrete must be added to provide additional support to the tunnel.

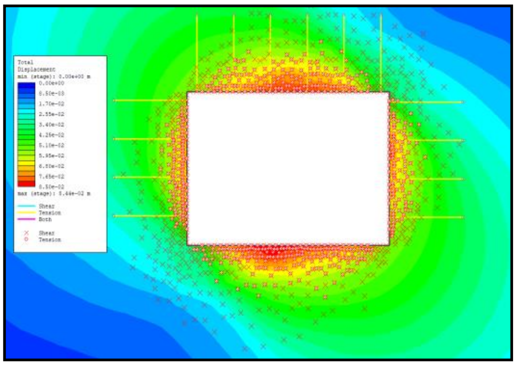

Fig.7: Plastic analysis with support (bolts)

4.9 Plastic Model with Bolts and Liners

The model below shows that in terms of deformation, there is not that much change, even after putting in liners and bolts, deformation still happens. Figure 18 below shows that there is not much change in the deformation rate; the failure zone is in solid red. Slightly changed with the addition of liners. Thus, support is acting on a deformed tunnel, hence reducing the effectiveness of both supports.

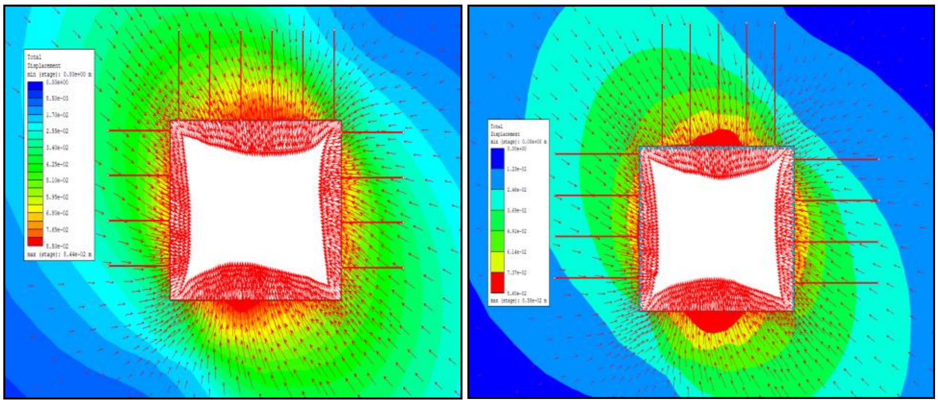

Fig.8: Effects of bolts only and bolts with liners on tunnel deformation

4. Conclusion And Recommendation

The Rock Mass classification schemes all agree that the Dolerite Dyke is a poor rock which needs to be given attention and supported. The conclusion made is that the Dolerite Dyke failure is initiated by the discontinuities around the rock. The mine should do regular analysis of the stress as they mine deeper in the ground. Opening sections should be kept small, as this has contributed to the small stress. Superior quality control is performed daily to ensure that all tendons have been installed and are sufficient in supporting all the wedges supporting the excavation.

Painting support lines before drilling the support holes in (support grid lines) is recommended to ensure more accurate support spacing will be achieved. Installation of 2.3 m long full column pre-tensioned resin bolts into the hanging wall and sidewalls at an angle of 90 degrees, and application of 25mm thick shotcrete to the hanging wall and sidewalls after the installation are recommended to compensate for blocky ground conditions that would otherwise be exposed. The use of Machine Learning and Artificial Intelligence can be an advanced alternative for a better understanding of risks and their management.

Acknowledgements

The authors express their sincere thanks to the Head, Department of Mineral Resources, and the Department of Mining. Botswana International University of Science & Technology. Our sincere thanks are due to the Mupane Gold Mines, Botswana, for allowing us to publish the research findings in the form of a research paper.

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