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RESEARCH ARTICLE

IRREGULAR CROWN PILLARS MODELING USING THE HOEK AND BROWN FAILURE CRITERION IN THE FEKOLA UNDERGROUND MINE

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ABSTRACT

This Crown pillars are essential structural elements for the stability and safety of underground mines, particularly when they separate underground excavations from the surface or an adjacent open pit. At the Fekola gold mine, the transition from open-pit to underground mining of a shallowplugingorebodyled to the formation of a crown pillar with an irregularshape and steep inclination, presenting a particular geomechanical challenge. This study aims to assess the stability of thisatypical crown pillar as part of the plannedmining sequence. The methodology is based on three-dimensional numerical modeling using the Hoek-Brown failure criterion within the MAP3D software. The input parameters were determined from an integrated geotechnical database, including deepborehole logs, laboratory tests (UCS, triaxial, Brazilian), as well as rock mass classification (RMR and Q′). Several mining scenarios, including full extraction and the strategicabandonment of certain sites combined with back filling with cemented rock (CRF), were simulated. The main results show that the complete extraction of all plannedstopes leads to localized instabilities, with safety factors below 1.0. Stability is effectively restored by leaving four stopesunmined, which increases the effective thickness of the pillar and allows for a more favorable stress redistribution. This work confirms the relevance of using advanced 3D numerical modeling, coupled with the Hoek-Brown criterion, for the safe design of complex crown pillars in hybridmining contexts.

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INTRODUCTION

Crown pillars are essential structural elements for the safety and long-termstability of underground miningoperations, ensuring both the mechanical stability of the overlying rock and the hydraulic isolation of underground workings from the surface or an adjacent open-pit operation (T. Carter, 2014; Mohanto *et al*., 2024). Their design and evaluation present substantial challenges related to the complexity of potential failure mechanisms, the uncertainty surrounding the geomechanical properties of oftenaltered rock masses, as well as the frequentlack of knowledge about the precise distribution of in-situ stresses and hydrogeological conditions (Bétournay, 1987; Hoek, 1989). Historically, this complexity has led to a largely empirical design approach, based on 'rules of thumb' and limit equilibrium analyses developed for regular geometries (T. G. Carter, 1992; T. G. Carter & Miller, 1995). Faced with the need for more reliable assessments, advanced empirical methods, such as the 'Scaled Span Method' calibrated on large case databases, have been developed (T. Carter, 2014). At the same time, the advent of numerical modeling and the widespread adoption of the empirical Hoek-Brown failure criterion, notable for its ability to incorporate the quality of the rock mass through classification systems such as RMR or GSI, have revolutionized geomechanical analysis, allowing the realistic simulation of the nonlinear behavior of complex structures (Hoek *et al*., 1995; Hoek & Brown, 1980a, 1980b).

In this context, the Fekola gold mine in Mali presents a notable case study of geometric complexity. The sequential transition from open-pit mining to underground mining of a deposit that gently dips to the north (6-8°) generated a crown pillar with unique characteristics: ahighly irregular geometry and a pronounced inclination. This atypical configuration, resulting from the interaction between the pit and the underground workings, deviates significantly from conventional models for which existing empirical and analytical methods have been primarily calibrated. The reliable assessment of the stability of this irregular structure under the plannedmining sequence thus represents a central geomechanical challenge, requiring an adapted methodology capable of accounting for its three-dimensionality and the nonlinear behavior of the rock mass. The aim of this study is to propose and implement an integrated methodology for the evaluation and optimization of the stability of the complex crown pillar at the Fekola mine. This methodology combines a thorough geotechnical characterization of the rock mass with explicit three-dimensional numerical modeling using the Hoek-Brown failure criterion, via the MAP3D software.

Study Area : The Fekola gold mine islocated in the Kédougou–Kéniéba basin in southwestern Mali, in the Kéniéba district on the Menandi permit, near the Senegal-Mali border and about 520 km from Bamako. Itsgeographiccoordinates are 12°35′ N and 11°50′ W, with an averageelevation of 135 meters. This region, belonging to the West African Craton, constitutes one of the main gold corridors in West Africa (Garagan *et al*., 2019). Figure 1shows the location of the

Fekola mine in its regional context. The mining of the deposit combines open-pit and underground methods, organized around a central pillar system thatensures the separation and transition between the open pit and the underground works. This pillaris located at the heart of the mineralized zone and constitutes a strategic element for the overall stability of the mining system and the safety of the excavations. Its positioning and geometry influence the redistribution of stresses and the mechanical behavior of the rock mass, particularly in the context of stability analyses through numerical modeling. The **Error! Reference source not found.** illustrates the location of the central pillar in relation to the open-pit mine and the underground workings.

Figure 1. Location Plan (prepared by B2Gold, Garagan *et al.* (2019))

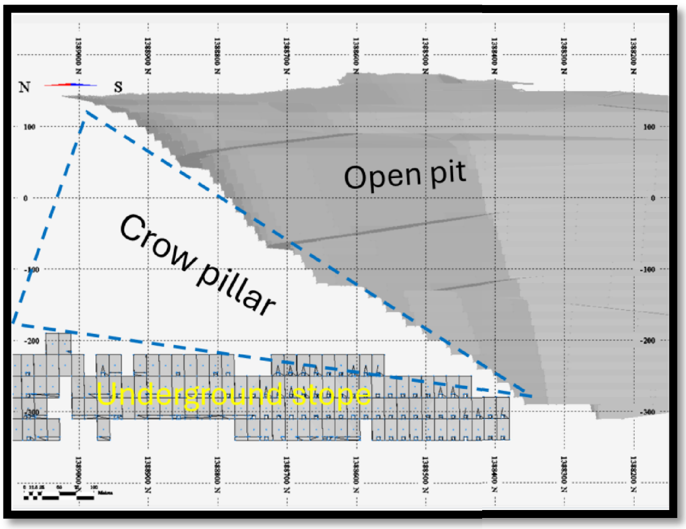

Figure 2 Location of the Fekola crown pillar between the quarry and the mine shafts

The Fekoladeposit is located within the Birimian Paleoproterozoic formations of the Kédougou–Kéniébacratonic block, characterized by a sequence of mafic to intermediatevolcanic rocks, volcano-sedimentarydetrital rocks, and calc-alkalinegranitoid intrusions (Lawrence *et al.*, 2013). These formations make up the main host of the gold mineralization and are representative of the volcano-sedimentary belts of the West African Craton. The litho logical units of the region have under gonepoly phasede formationac companied by low- to medium-grade metamorphism, bothrelated to the Eburneanorogeny. This tectono-metamorphic evolution generated a complex network of structures, including faults, shear zones, and schistosity planes, preferentially oriented in a north–south direction. These structures control the geometry of gold deposits and the distribution of mineralization in the area (Dabo &Aïfa, 2010; Gueye *et al*., 2008; Hirdes & Davis, 2002; Lawrence *et al*., 2013; Ledru *et al*., 1991; Milési *et al*., 1992). At the local scale, the rock mass is affected by structural disturbances such as fractures, discontinuities, and zones of weakness, which can influence the mechanicalbehavior of the pillar and mining structures. The identification and characterization of these geological features are essential for the reliability of numerical simulations and for assessing the stability of the mining system. Figure 2 presents the geologicalmap of the Fekola area, highlighting the main lithologies and structures.

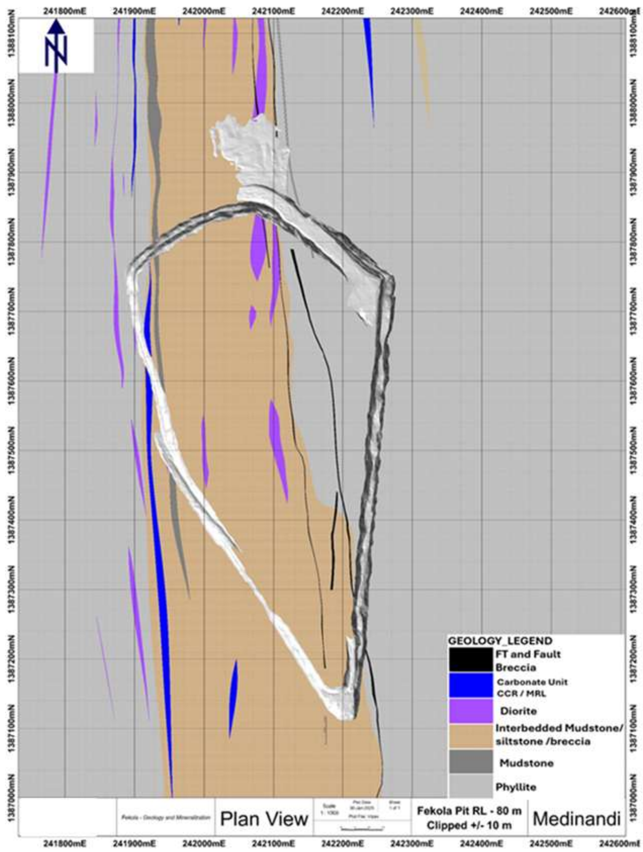

Figure 2. Fékola Geological Map

STUDY METHODOLOGY

Hoek and Brown'scriteria : Hoek et Brown ((Hoek & Brown, 1980a, 1980b) proposed an empirical relationship intended to represent the results of a wide range of triaxial tests conducted on intact rock samples. This relationship is given by the equation (1):

$$\sigma_1 = \sigma_3 + \sigma_{ci}\sqrt{m_i \frac{\sigma_3}{\sigma_{ci}} + 1} \quad (1)$$

Where $\sigma_1$and σ_3 represent the major and minor principal stresses, respectively, σ_{ci} is the uniaxial compressive strength of the intact rock, and m_i is a material characteristic constant.

In order to estimate the strength of rock masses, (Hoek, 1994), then (Hoek *et al*., 1995), have extended this criterion in a generalized form, expressed by the equations(2) à (5):

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \quad (2)$$

The parameters m_b, s and a are constants specific to the rock mass and are expressed as follows:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14}\right) \quad (3)$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \quad (4)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \quad (5)$$

In the case of intact rock, the constants are such that m_b=m_i, with s=1 and a=0,5. The disturbance factor D allows for taking into account the degree of alteration of the rock mass, including damage related to blasting and the effects of stress relaxation. A typical value of D=0.5 is generally used for controlled blasting methods (pre-splitting, smooth blasting), while D=0.7 corresponds to mechanical excavations accompanied by damage due to stress relief. Finally, D=1.0 characterizes a rock mass that is heavily fragmented by production blasting (Hoek & Brown, 2019).

Analytical model of a crown pillarsafetyfactor : In order to obtain an initial analytical assessment of stability, a limit equilibrium model was applied to an idealized crown pillar in the form of a rectangular block, with dimensions x and y in plan. The height of the pillar is denoted z, whilez$_w$ represents the water level measured above the base of the pillar. The pillar is subjected to stresses σ_x andσ_y (Figure 3)(Hoek, 1989). The safety factor F is then expressed by the equation(6):

$$F = \frac{2(\tau_{xz} * xz + \tau_{yz} * yz)}{\gamma_r \ * x * y * z} = \frac{2}{\gamma_r}\left(\frac{\tau_{xz}}{y} + \frac{\tau_{yz}}{x}\right) \quad (6)$$

Where γ_r refers to the density of the rock material, τ_{xz} and τ_{yz} correspond respectively to the shear stresses acting on the xz and yz faces of the pillar. Shearresistances can becharacterized either by the friction angle φ′ and cohesion c′ of the Mohr–Coulomb failure criterion, or be evaluated directly from the nonlinear failure criteria proposed by Barton (1976)or by(Hoek & Brown, 1980b) and Hoek & Brown (1988). In the present work, the Hoek–Brown criterion is favored, as it allows a direct estimation of the shear strength from the quality of the rock mass evaluated in situ. (Hoek, 1989). Thus, the shear resistance on each face (τ_{xz}, τ_{yz}) is calculated using the Hoek-Brown criterion, leading to the equations(7)-(10):

$$\tau_{xz} = (\cot \phi'_{ixz} - \cos \phi'_{ixz}) \frac{m\,\sigma_c}{8} \quad (7)$$

$$\phi'_{ixz} = \arctan\left(\frac{1}{\sqrt{4\,h_{xz}\cos^2\theta_{xz} - 1}}\right) \quad (8)$$

$$\theta_{xz} = \frac{1}{3}\left(90 + \arctan\left(\frac{1}{\sqrt{h^3{}_{xz} - 1}}\right)\right) \quad (9)$$

$$h_{xx} = 1 + \frac{16(m\sigma'_y + s\sigma_c)}{3m^2\sigma_c} \quad (10)$$

In this context, m and s correspond to the constants of the Hoek–Brown failure criterion, while σ_c refers to the uniaxial compressive strength of intact rock (Hoek, 1989).

The expression of τ_{yz} is obtained by simply transposing the parameters associated with the xz plane to those of the yz plane in the equations (7), (8)et (9), as well as by replacing $\sigma_x$byσ_y in equation(10) (Hoek, 1989). Considering that the crown pillar has undergone disturbances related to blasting, groundwater infiltration, and the movements of the rock mass surrounding the chamber, the constants m and s can be evaluated from the value of Bieniawski's RMR, according to the relationships proposed by (Hoek, 1989):

$$m = m_i \ \times \exp\left(\frac{RMR - 100}{14}\right) \quad (11)$$

$$s = \exp\left(\frac{RMR - 100}{6}\right) \quad (12)$$

where m_i represents the value of the constant m corresponding to the intact rock, as defined in the Table . Effective lateral constraints σ_x' andσ_y' can be determined from the geometric dimensions of the crown pillar and the water level present in it, as illustrated in Figure 3(Hoek, 1989):

$$\sigma'_x = \frac{\gamma_r\, z\, k_x}{2} - \frac{\gamma_w\, z_w^2}{2w} \quad (13)$$

$$\sigma'_y = \frac{\gamma_r\, z\, k_y}{2} - \frac{\gamma_w\, z_w^2}{2z} \quad (14)$$

where k_x and ky respectively represent the ratios between the horizontal and vertical stresses along the x and y directions, and γ_w refers to the density of water (Hoek, 1989).

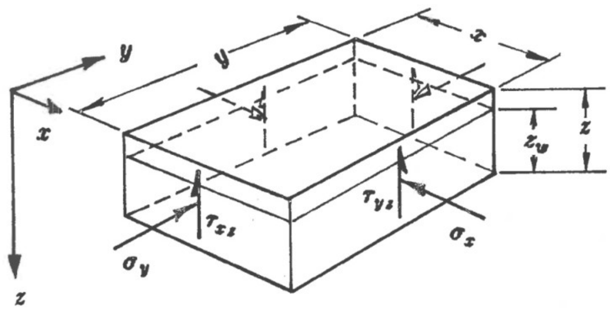

Figure 3. Pillar size and direction of lateral stresses (Hoek, 1989)

Table 1. Approximate values of m_i for different types of rocks (Hoek & Brown, 1988)

Rock types	Examples	m_i
Carbonate rocks withwell-developedcrystalcleavage	Dolomite, limestone, marble	7
Lithifiedclayey rocks	argillite, siltstone, schist, slate (perpendicular to cleavage)	10
Sandstone rocks withresistantcrystals and poorlydevelopedcrystalcleavage	Sandstone, quartzite	15
Fine-grainedpolymineraligneous crystalline rocks	Andesite, dolerite, diabase, rhyolite	17
Coarse-grainedpolymineraligneous and metamorphic crystalline rocks	Amphibolite, gabbro, gneiss, granite, norite, quartz-diorite	25

Analytical model of load transfer in back filled chambers : The design of backfilled chambers requires evaluating the interaction between the backfill and the rock mass. The load transfer along the interface, particularly when the backfill is softer than the surrounding rock, strongly influences the mechanical response of the openings (Aubertin *et al*., 2003; Purwanto *et al*., 2014). This mechanical interaction between the cemented Rock backfills (CRF) and the rock mass is a key factor in overall stability. To quantify the transfer of vertical load to the walls, the analysis is based on Marston's theory adapted to vertical mine chambers (Marston, 1930; McCarthy, 1988). The Figure 4 illustrates a chamber backfilled to a height H and a width B, highlighting a backfill element with a unit section dh at height h. This horizontal layer element is subjected to a lateral compressive force C, a shear force S, as well as vertical forces V et V+dV (Aubertin *et al.,* 2003).

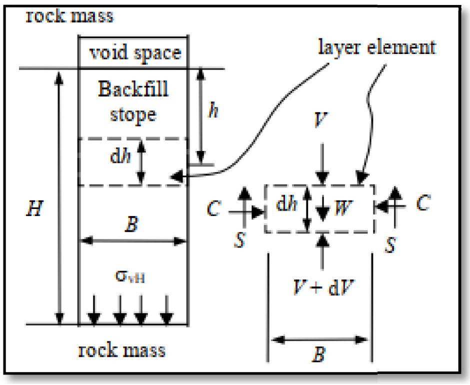

Figure 4A. vertical-sized foundation backfilled with the forces acting on the isolated layer element (of unit thickness), and the vertical stress on the floor (Aubertin *et al.,* 2003)

By considering the balance of vertical forces (weight W, shear forces S) and using the Mohr-Coulomb criterion to define the resistance at the embankment/rock interface (effective friction angle φ'), we arrive at the expression for the vertical stress at the roof of the chamber (σ_{vH}) given by the equation(15). The corresponding horizontal constraint (σ_{hH}) is given by the equation (16). The lateral reaction coefficient K ($K = \sigma_h/\sigma_v$) is a crucial parameter controlling the arching effect in the embankment. These expressions allow estimating the load transmitted by the embankment to the walls and to the underlying crown pillar.

$$\sigma_{vH} = \gamma B \left[\frac{1 - \exp\left(\frac{-2KH \tan\varphi'}{B}\right)}{2K \tan\varphi'} \right] \tag{15}$$

$$\sigma_{hH} = \gamma B \left[\frac{1 - \exp\left(\frac{-2KH \tan\varphi'}{B}\right)}{2 \tan\varphi'} \right] \tag{16}$$

Acquisition and Processing of Geotechnical Data

A detailed geotechnica characterization campaign was carried out to provide the models with parameters representative of real conditions. This approach made it possible to better understand the mechanical behavior of the crown pillar and the surrounding rock units. Six deepgeo technical drillings, totaling 2,484 meters, were carried out to intercept the pillar and the surrounding masses. These drillings made it possible to collect representative samples of the dominant lithologies, namelyphyllite, siltstone, and breccia. The samples were then subjected to various laboratory tests to determine their mechanical properties: uniaxial compressive strength (UCS), Brazilian tensile strength (BTS), triaxial compressive strength (TCM), and base friction angle (BFA). Finally, the field data fromlogging, combinedwith the experimental results, made it possible to calculate the quality indices of the rock masses, no tably the RMR (Bieniawski, 1979) and Barton's Q, with out taking into account water-related or stress reduce factors. The summary results of this characterization are presented in the Table 2.

Table 1. Rock mass properties

Properties	Dominant lithologies	Max	Min	Average	Median
Q de Barton	Phyllite	600.00	0.71	94.68	52.22
	Siltstone	600.00	1.67	106.18	66.83
	Breccia	526.22	0.48	88.33	57.89
RMR79	Phyllite	92.00	38.00	75.56	67.00
	Siltstone	92.00	33.00	68.10	64.00
	Breccia	92.00	35.00	69.74	67.00
UCS (MPa)	Phyllite	271.33	16.50	79.24	66.78
	Siltstone	314.91	22.85	106.15	82.32
	Breccia	200.28	17.91	107.29	108.93
TCM (MPa)	Phyllite	347.14	27.29	171.77	173.89
	Siltstone	549.48	89.78	240.35	210.87
	Breccia	366.73	118.22	235.39	234.97
BTS (MPa)	Phyllite	22.34	3.84	12.22	11.80
	Siltstone	18.07	7.62	13.07	13.27
	Breccia	18.91	3.71	12.01	12.84
BFA (°)	Phyllite	40.00	24.00	34.61	36.00
	Siltstone	42.00	28.00	35.75	36.00
	Breccia	41.00	30.00	36.00	36.00

Pillar modeling with 3D MAP: The pillarstability was analyzed using the MAP3D software. The model accounted for four material types (host rock, ore, cemented rock fill [CRF], and uncemented rock fill [URF]), the in-situ rock stress, and the back fill properties. Analyses were conducted for multiple scenarios (see Table 1 to Table 3). The main and secondary shafts measure 30 m high, 10 m long, and 5 m wide. The main shafts are mined first, back filled with cemented rock fill (CRF), and then serve as pillars during the subsequent mining of the secondaryshafts. Through outt his process, the crown pillar must behave as an elastic plate with a safety factor exceeding 1 to withstand the induced stresses.

Table 2. Material Properties for the Hoek-Brown Model (Host Rock, Ore, CRF, URF)

Property	Host Rock	Ore	CRF	URF
Material ID	1	2	3	4
Material Model	Hoek-Brown	Hoek-Brown	Hoek-Brown	Hoek-Brown
Uniaxial Compressive Strength, σ_ci (MPa)	80	60	60	60
Hoek-Brown Constant, m_i	2	1.86	7	7
Hoek-Brown Constant, s	0.0198	0.0159	1	1
Young'sModulus, E (MPa)	28,000	25,000	1,000	200
Poisson's Ratio, ν	0.25	0.25	0.25	0.25
TensileCutoff (MPa)	-0.8	-0.6	-60	-60
ViscousModulus - Gn (MPa)	0	0	0	0
ViscousModulus - Gs (MPa)	0	0	0	0
Standard Deviation	0	0	0	0
Expansion Coefficient	1	1	1	1
Conductivity	1	1	1	1

Table 3. In-situ stress for host rock and cemented rock fill (CRF) material

Property	Rocker	Bakfill (CRF)
Horizontal Datum(m)	135	0
Stress gradients (MPa/m)		
σ^{Hmax} constant	4.0	0.3
σ^{Hmin} constant	0.0	0.3
σ^{Vert} constante	0	0.6
Stress variation as a function of depth (MPa/m)		
$\Delta\sigma^{Hmax}$ variation	-0.0521	0.0000
$\Delta\sigma^{Hmin}$ variation	-0.0361	0.0000
$\Delta\sigma^{Vert}$ variation	-0.0269	0.000
Orientation of the principal stress (°)		
σ^{Hmax} trend	342	0
σ^{Hmax}plunge	0	0
σ^{Hmin}trend	72	270
σ^{Hmin}plunge	0.000001	0
σ^{Vert}trend	0	0
σ^{Vert}plunge	90	90

RESULTS

The results from three-dimensional numerical modeling with MAP3D software illustrate the progressive evolution of the crown pillar's mechanical behavior across the differen to perational phases. In the initial state, which represents an intact pillarprior to any underground excavation, the stress distribution remainslargely homogeneous. Stress concentrations are limited to the transition zones between the open pit and the undisturbed rock mass. In this configuration, the calculated safety factor remains well above unity through out the pillar, indicating satisfactory stability (Figure 5). However, the introduction of underground shafts significantly alters the stress field within the pillar. In particular, the complete extraction of all the chambers leads to an unfavorable redistribution of stresses, characterized by an increase in deviatoric stresses and a concentration of plasticized zones near the southern boundary of the pillar. This development results in a significant decrease in the safety factor, with the appearance of areas where it drops below 1, indicating a state of potential instability (Figure 6 - Figure 9). These critical areas define a preferred fracture path connecting the underground excavations to the open pit. Furthermore, backfilling the excavated shafts with cemented rock fill (CRF) leads to a significant improvement in the mechanical response of the system. The CRF contributes to a partial load recovery and a more even redistribution of stresses within the pillar. However, despite this improvement, some areas remain close to the limit state, particularly in regions where the effective thickness of the pillar is still insufficient; these critical areas are also highlighted by red circles in the corresponding figures (Figure 10). Finally, the optimized scenario, which includes leaving four wells unexploited and combining this with the backfilling of adjacent chambers, leads to a significant improvement in the overall stability of the crown pillar. Keeping these wells unexploited increases the local structural thickness of the pillar and ensures better mechanical continuity between the open-pit excavation and the surrounding rock mass. This configuration results in a marked reduction of stress concentrations and a noticeable improvement in the stability of areas previously close to the limit state, identified and highlighted in red in the result figures. In this case, the safety factor once again exceeds 1 over almost the entire pillar, indicating stability compatible with continuing excavation under acceptable safety conditions (Figure 11).

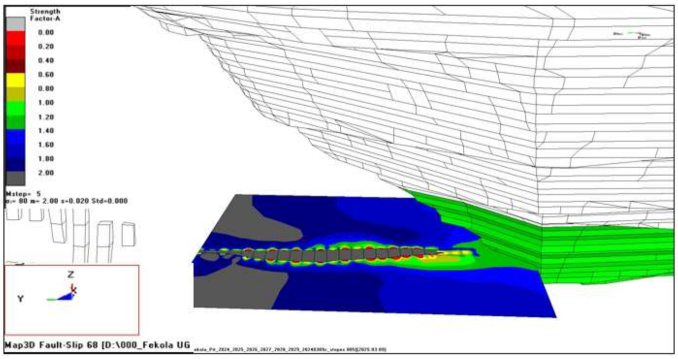

Figure 5. Crown pillar without stopes (plan view Z = -257)

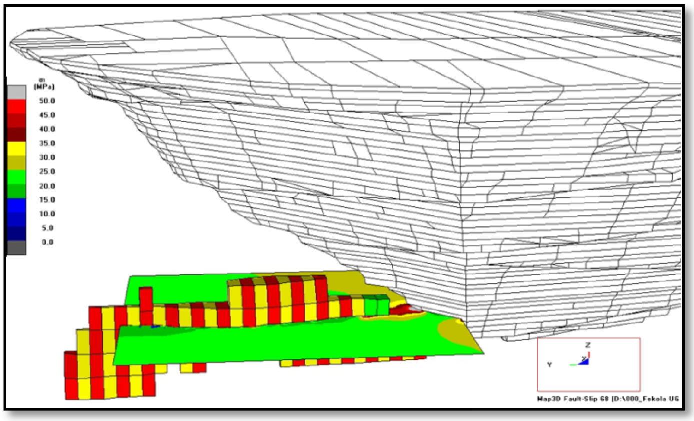

Figure 6. Crown pillar with stopes (plan view Z = -257)

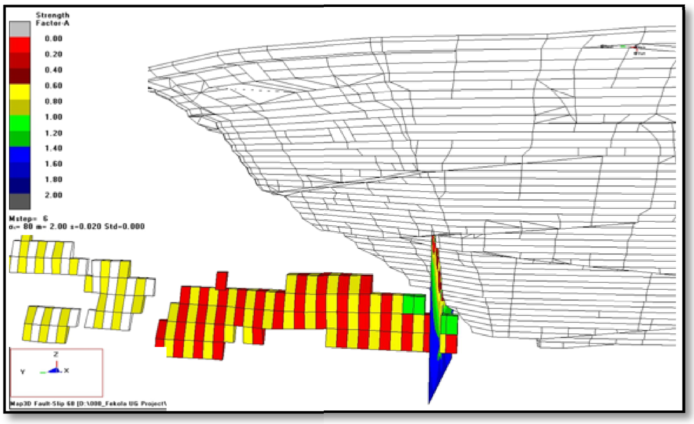

Figure 7. Crown pillar model with stopes (section view Y = 1388407)

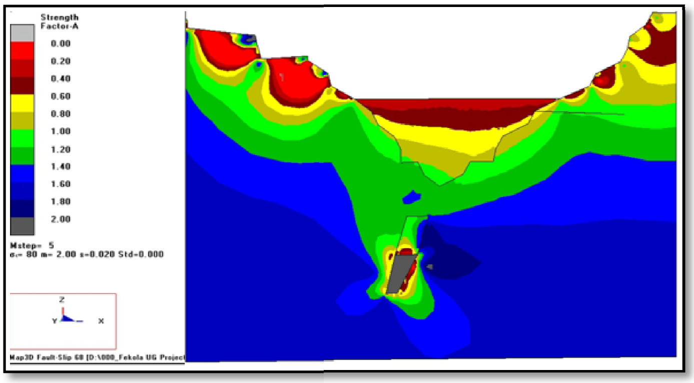

Figure 8. Crown pillar model showing open pit and stopes (section view Y = 1388407)

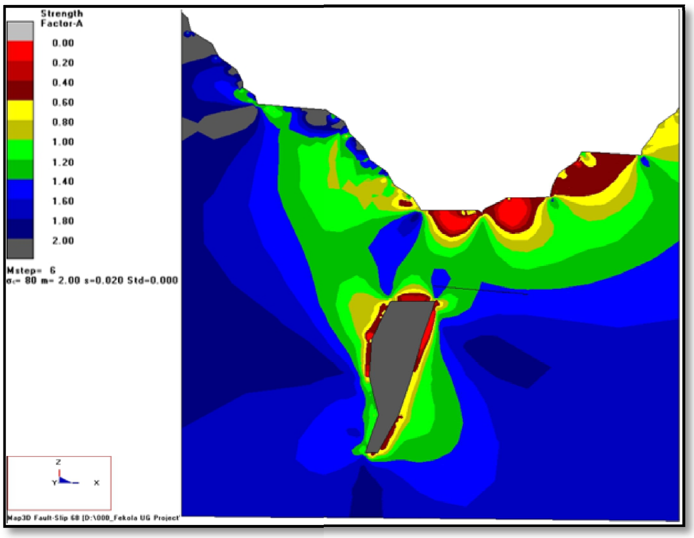

Figure 9. Crown pillar model showing open pit and stopes (section view Y = 1388887)

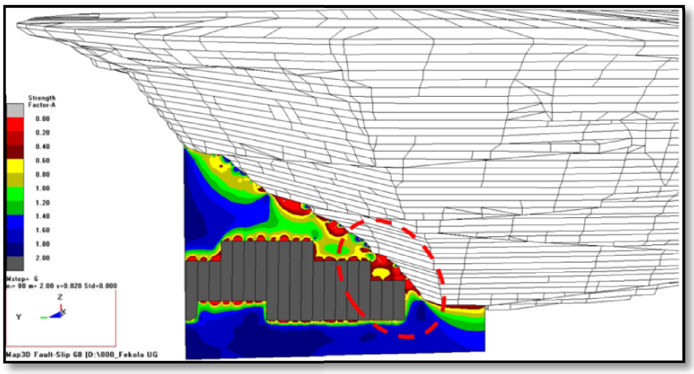

Figure 10. Crown pillar model with stopes backfilled with CRF

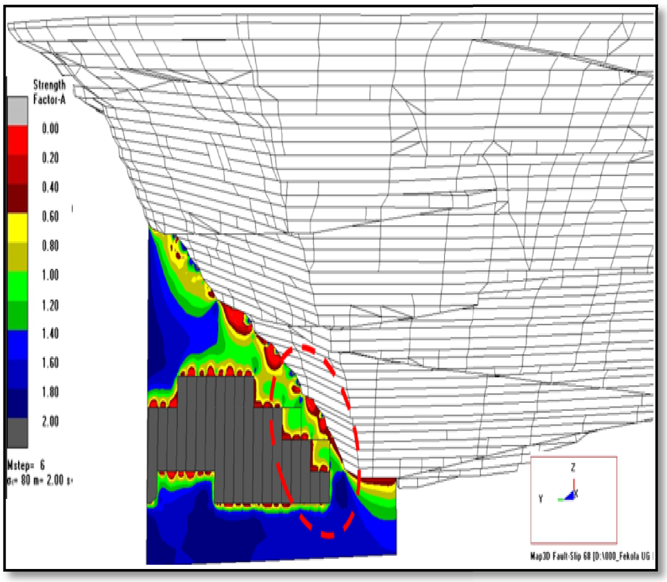

Figure 11. Crown pillar model with stopes backfilled by CRF and 4 stopes left unmined

DISCUSSION

The results highlight the crucial importance of the geometry and continuity of the crown pillar in a hybrid open-pit and underground mining context. Numerical modeling confirms that classical analytical or empirical methods, oftendeveloped for regular geometries, are insufficient to understand the behavior of an irregular and inclined crown pillarsuch as the one at Fekola. The complete extraction of the workings leads to a loss of the pillar's mechanical continuity, promoting the coalescence of plasticized zones between the pit and the underground works. This mechanism is consistent with observations reported in the literature regarding thin or discontinuous crown pillars (T. Carter. 2014; T. G. Carter & Miller. 1995; Hoek. 1989). The contribution of backfillingwith CRF results in an improvement in overall stability, but on its ownis not enough to ensure an acceptable safety factor when the pillar thickness is too small. This result confirms that backfill should beconsidered a supplementary support element and not a substitute for adequate pillar geometry, in line with the work of Aubertin *et al.* (2003) and Purwanto *et al.* (2014). The strategy of leaving certain stopes unexploited appears to be an effective solution for increasing the pillar's structural thickness and restoring a more favorable stress path. This approachillustrates the value of adaptive design based on 3D numerical modeling, allowing for the optimization of the mining sequence while maintaining a satisfactory level of safety.

CONCLUSION

This studye valuated the stability of an irregular crown pillar within the complex transition from open-pit to underground mining at the Fekola mine. An integrated methodology was implemented, combining detailed geotechnical characterization with three-dimensional numerical modeling based on the Hoek–Brown failure criterion. The results demonstrate that the complete extraction of all planned shafts leads to localized instabilities, with safety factors falling below unity, there by compromising the pillar's barrier function. While back filling withcemented rock fill (CRF) improves stability, itremains in sufficient when the pillar's effective thicknessis too low. In contrast, the strategy of maintaining four unexploited panels significantlyincreases the pillar's structural thickness, ensures a more favorable redistribution of stresses, and restores an overallsafety factor above 1. These findings confirm the value of 3D numerical modeling as a criticaldecision-makingtool for designing and optimizing complex crown pillars. The methodology developed in this study is applicable to other mining contexts featuring irregulargeometries and comparable geomechanical conditions.

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