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Geotechnical Stability Design Consideration for Cemented Rock Fill at Munali Nickel Mine in Zambia

[1,2]Daniel Mabeti and [1]Ruth N. Mutakela

[1]Mabiza Resources Limited, Munali Nickel Mine, mining department

[2]University of Zambia, School of Mines, Mining Engineering Department

Corresponding author(s)
Daniel Mabeti Email: dmabeti@gmail.com

Abstract:
This paper investigates into the essential geotechnical considerations underpinning the innovative mining method being implemented at the Munali nickel mine, specifically using cemented rock fill (CRF). The geotechnical design parameters play a crucial role in shaping the method's economic feasibility and ensuring safe extraction practices, all while considering the unique geological conditions present at the site. These parameters are input into advanced numerical modelling software to enhance understanding and predictive capabilities, specifically, the Phase 2D program developed by RocScience, which facilitates detailed stress analysis. With this, a comprehensive stability assessment is executed using the renowned Norwegian Geological Institute Q (NGI) system). This empirical methodology provides robust insights into ground stability. The design of this mining approach is particularly significant, as it intricately involves various factors such as the dimensions and configurations of both primary and secondary stopes, the natural formation of the orebody, the properties of the backfill material, the compressive strength of the CRF, and the sequencing of the stoping process. Each of these elements plays a vital role in the overall effectiveness and safety of the mining operation highlighting the importance of meticulous planning and analysis in successfully implementing this method.

Keywords: safe extraction, numerical, stability, backfilling, Strength, Stress analysis, Q system

1. Introduction

The Munali nickel mine, nestled 70 kilometers south of Lusaka in Zambia's picturesque Southern Province embodies a remarkable opportunity for nickel production (Figure 1). Since November 2011, this mine has been placed under care and maintenance, grappling with challenges stemming from geological inaccuracies that resulted in less effective mining methods (CNM, 2018). However, a significant restoration milestone was achieved in mid-2018, rejuvenating hopes for operational revival and signaling a new beginning. Ambitious plans were laid to ramp up nickel ore production by the close of 2019, harmonizing with favorable market conditions highlighted by soaring prices on the London Metal Exchange (London Metal Exchange, 2018). In its history, the mine utilized a sub-level caving method (Tetra Tech, 2015), which faced formidable challenges due to crown pillar caving, ultimately leading to the formation of a substantial sinkhole in the landscape. This sinkhole has been meticulously backfilled with waste rock, illustrating resilience and determination in overcoming obstacles. A comprehensive monitoring system has also been established to oversee ongoing developments, ensuring safety and efficiency in the implementation of new mining methods.

To address the intricate geological factors, the Munali mine has adopted the innovative AVOCA mining method, which employs cemented rock fill (CNM, 2018). This cutting-edge approach integrates empirical geotechnical assessments and numerical stability analyses, reflecting a commitment to enhancing safety and operational efficiency tailored to the mine’s unique geological conditions (CNM, 2018).

Extensive research and analysis were conducted during the feasibility study for the reopening of the Munali nickel mine (Mungu, 2017). Among the various mining methods evaluated, it is important to highlight that the AVOCA mining technique emerged as the most advantageous choice at this stage (Mungu, 2017). This method is particularly well-suited for handling the thicker sections of the orebody, ensuring both efficiency and effectiveness in the extraction process (CNM, 2018).

Figure 1: *The Munali Nickel Mine's strategic geographic location (Kabinga, 2023)*

2. Munali Geology

Figure 2 presents the Munali nickel deposit, a major geological resource that includes the prominent Enterprise area alongside two well-explored zones: Voyagers and Intrepid. All of these locations are situated within the Munali Gabbro (Blank et al., 2017). Through detailed mapping and diamond drilling, it has been established that the gabbro spans between 0.5 and 2.5 kilometers in plan and descends to depths of approximately 700 meters (Evans et al., 2017). This geological feature is interpreted as an important sill that intrudes into the limestones and quartzites of Katanga age. The Enterprise deposit is particularly interesting due to the geological complexity it presents (Evans et al., 2017). A notable fault influences this deposit, resulting in a significant drop of around 120 meters on the northwestern side. Within the Enterprise area, an extensive mineralization zone has been identified, measuring approximately 100 meters in width, 100 meters in length, and extending 600 meters in height (Evans, 2005). This mineralization zone is characterized by solid alteration and narrows toward then northwest and at greater depths, demonstrating the dynamic nature of this geological formation (David et al., 2017). The formation of dolomite within the deposit occurs through the assimilation of surrounding limestone sediments, highlighting the intricate geochemical processes at play (Evans, 2005). Additionally, hydrothermal replacement processes contribute to the increasing grain sizes of the sulfides as they approach dolomite, emphasizing the rich mineral potential of this area. The sulfides are found in various forms—such as disseminated blebs, patches, veinlets, or stringers—with average sizes ranging from 45 to 85 microns (Barnes et al., 2017). The primary ore minerals present include pyrrhotite, pyrite, pentlandite, and chalcopyrite, underscoring the economic significance of the deposit. Notably, pyrrhotite accounts for approximately 66% of the sulfides and can transform into violarite near the surface (Evan et al., 2017).

Pyrite exists in two distinct phases: the earlier phase may incorporate cobalt, while the later phase is nickel-bearing and can replace both pentlandite and pyrrhotite. Pentlandite is recognized as the key nickel-bearing mineral, occurring as exsolution phases within pyrrhotite and forming distinctive rims around the grains (CNM, 2016). This multifaceted geology reinforces the importance of the Munali deposit for nickel extraction (Evans et al., 2017).

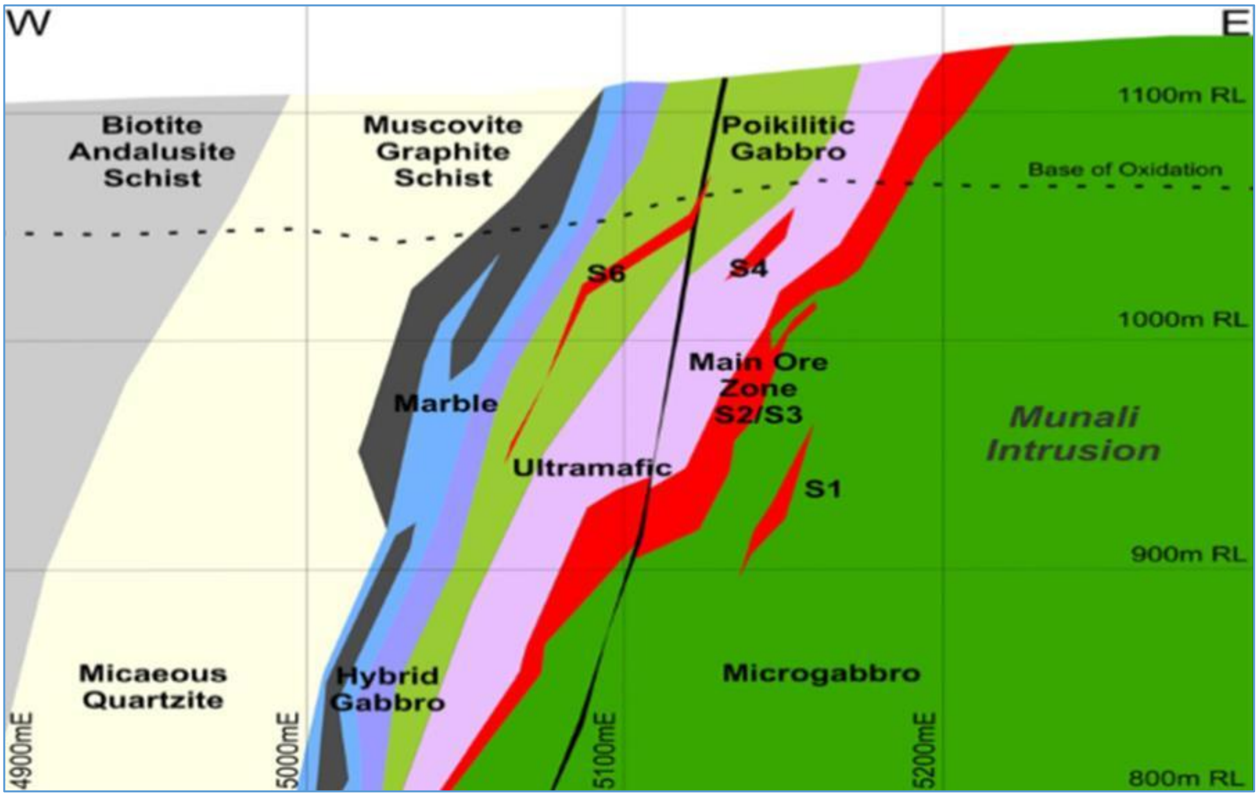

Figure 2: *The Munali Nickel Mine's geological settings (Blank et al., 2017)*

3. Geotechnical Considerations

Geotechnical mapping and core logging at the mine site provided valuable insights into the rock mass characterization of the immediate hanging wall, footwall, and ore formation. Additional geotechnical data collected included information on failure modes and major controlling structures within the orebody. The primary failure modes observed were wedge failures occurring along prominent geological structures, mainly localized in a talc-like weak zone 1.5 to 2 meters thick, with dips ranging from 60° to 80°. The Munali orebody itself has a dip between 60° and 70° (Albidon, 2006). The geological formation consists of massive, heavy rock lightly to moderately jointed toward the footwall contact, classified as fair to good quality. In contrast, the hanging wall exhibits poor and highly jointed characteristics, classified as fair to poor quality. The hangingwall formation primarily comprises massive micro gabbro, whereas the footwall formation is typical of good to excellent-quality gabbro (Evans, 2005). The average uniaxial compressive strength (UCS) for the ore formation ranges from 50 to 100 MPa. The UCS for the hanging wall formation is approximately 50 MPa, while the UCS for the footwall formation exceeds 150 MPa. On average, the thickness of the orebody ranges from 15 to 40 meters (Evans, 2005).

4. Mining Induced Stresses

The Munali nickel mine currently operates at an active level that reaches depths of less than 300 meters below the surface. Although our current knowledge of stress measurements at the mine site is somewhat limited, we can reasonably infer that the stress level is primarily due to overburden pressure from the material above. To deepen our understanding of the geological conditions, we will conduct detailed calculations of mining-induced stresses through comprehensive numerical analysis. This analysis will incorporate the various mining activities that have taken place at the upper levels of the mine, providing valuable insights into how these actions influence conditions

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below. As we implement a new, innovative mining method, we aim to maintain a manageable induced stress of approximately 11 MPa. This will be achieved after successfully operating at the lowest level of the mine while utilizing backfill to support the excavated areas (refer to Figure 3). Our strategy emphasizes the importance of collaboration, ensuring that all stakeholders are actively engaged in the process. This collective effort will not only optimize our mining strategy but also enhance safety and operational efficiency, securing the long-term viability of the Munali nickel mine.

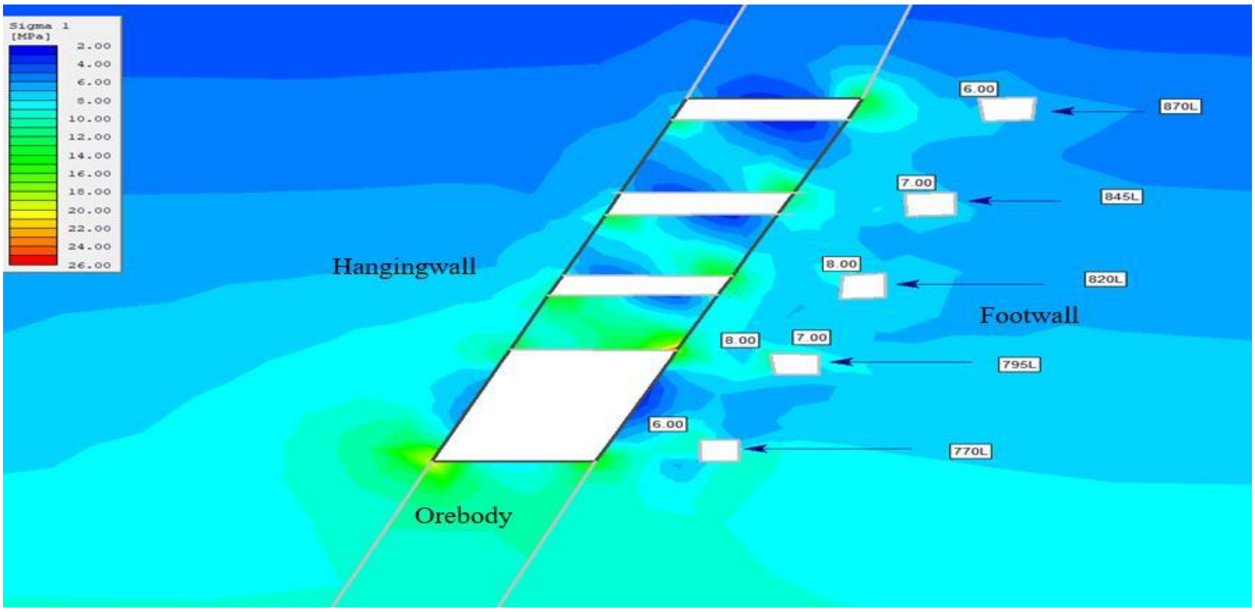

Figure 3: *Mining induced stresses after stopping the bottommost level.*

5. Mine Development

The footwall drive will be mined on both sides of the south and north retreat; it will be at a minimum of 10.0m from the geological footwall contact (GFW) as per the numerical model conducted in Phase 2D (Figure 3) without any stress influence. The orebody drive for primary stopes dimension is 5.0m wide and 5.0m high, mined through up to the assay hangingwall contact or geological hangingwall contact (GHW). The primary stopes will be mined at 14.0m intervals, leaving a pillar later mined as a secondary stope after two adjacent primary stopes are fully backfilled with cemented rock fill, ensuring the safety of the mining operations (Figure 4). The footwall drive will permanently access the working level, tramming route and fresh air intake. The drive will be 5.0 m wide by 5.0 m high to accommodate both Load Haul Dump (LHD) and Dump trucks. Raise crosscuts (4.0 m x 4.0 m) will be mined on the hanging wall at an apparent dip of 70°. The raise is planned to hole through to the upper extraction drive. The upper extraction drive, designed for efficient ventilation management, will serve as the collector of vitiated air from stopping activities and provide backfilling access.

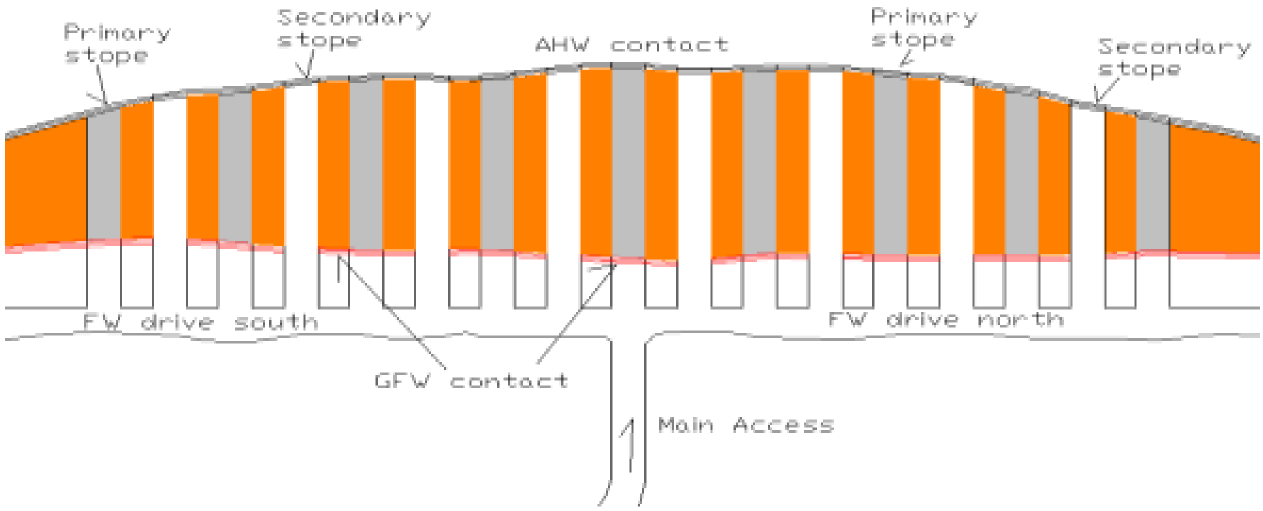

Figure 4: *Schematic plan view of the proposed Primary and Secondary stopes.*

6. Ground Support Requirements

The Q system stands as a vital and practical tool in the quest to evaluate the quality of rock masses surrounding proposed mining operations. Born from the pioneering work of Barton and his colleagues at the Norwegian Geotechnical Institute (NGI) in 1974, this system transcends theoretical limits to provide a clear method for determining the support structures essential in civil engineering tunnels. By systematically classifying rock masses based on block size and joint condition, the Q system reveals insights that empower the design and safety measures for tunneling projects (Barton et al., 1974). Its application inspires engineers to navigate the challenges of underground construction with confidence and clarity. Equation [1] below shows an expression of the Q-system.

$$Q = \left(\frac{RQD}{Jn}\right) x \left(\frac{Jr}{Ja}\right) x \left(\frac{Jw}{SRF}\right) \qquad \text{Equation [1]}$$

Where: -

RQD is the Rock Quality Designation
Jn =number of joint sets,
Jr =Joint roughness number
Ja =Joint alteration or filling
Jw =Water condition
SRF =Stress Reduction Factor

6.1 Geotechnical mapping (Worst case scenario); *Discontinuity condition*

Highly jointed Rock Mass
Damp (since this is underground environment)
50 -100 MPa (Estimated)
2 Joint sets + random.
Joint spacing ranging between 0.4 – 0.5m
Joint condition –slightly weathered.
Slightly rough, irregular, and planar, soft filled joints (less than 5mm thick), persistence (3 to 10m)
RQD = 50% -60%

6.1.1 Q-System ratings

Jn = 6 (2 joint sets + Random)
Jr = 1.5 (Slightly rough, irregular planar)
Ja = 1.0 (Unaltered joint walls, surface staining only)
Jw = 1.0 (dry)

SRF = 2.5 (Low stress near surface, Strength/stress ratio = 100Mpa/11.3Mpa = 8.8

$$Q = \left(\frac{60}{6}\right) x \left(\frac{1.5}{1}\right) x \left(\frac{1}{2.5}\right) = 6$$

6.2 Borehole geotechnical core logging-(borehole); *Discontinuities Condition*
Intact, massive, and unweathered rock
Damp (since this is underground environment)
UCS = 50 - 100MPa (estimated)
3 Joint sets
Joint spacing ranging between 0.1, 0.15, 0.09
Joint condition –slightly weathered. Slightly rough and planar, soft filled joints (1mm -5mm thick), persistence – 3 to 10m and RQD = 60% - 80%

6.2.1 Q-System ratings
Jn = 9 (3 joint sets)
Jr = 1.5 (Slightly rough, irregular planar)
Ja = 1.0 (Unaltered joint walls, surface staining only)
Jw = 1.0 (dry)
SRF = 2.5 (Low stress near surface, Strength/stress ratio = 100Mpa/11.3Mpa = 8.8)

$$Q = \left(\frac{50}{9}\right) x \left(\frac{1.5}{1}\right) x \left(\frac{1}{2.5}\right) = \mathbf{3.3}$$

The Excavation Support Ratio (ESR) indicates safety levels based on the excavation's intended use and lifespan (Grimstad and Barton, 1993). With an Excavation Support Ratio of 1.6, the Equivalent Dimension (D_e) can be calculated as shown in equation [2].

$$\text{De} = \frac{\text{Span,Height in meters}}{\text{ESR}} \qquad \text{Equation [2]}$$

$$\text{De} = \frac{5.0}{1.6} = \mathbf{3.1}$$

Using the equivalent dimension (De) and the calculated 'Q' value, we determine the support design by plotting these values on the graph shown in Figure 5. The graph indicates that the required support falls under category 4, which involves systematic bolting with 40-100 mm of unreinforced shotcrete. This is an efficient solution. Instead of using 40-100 mm of unreinforced shotcrete, we will employ galvanized welded mesh along with split sets as the primary support, further enhancing the efficiency of our design.

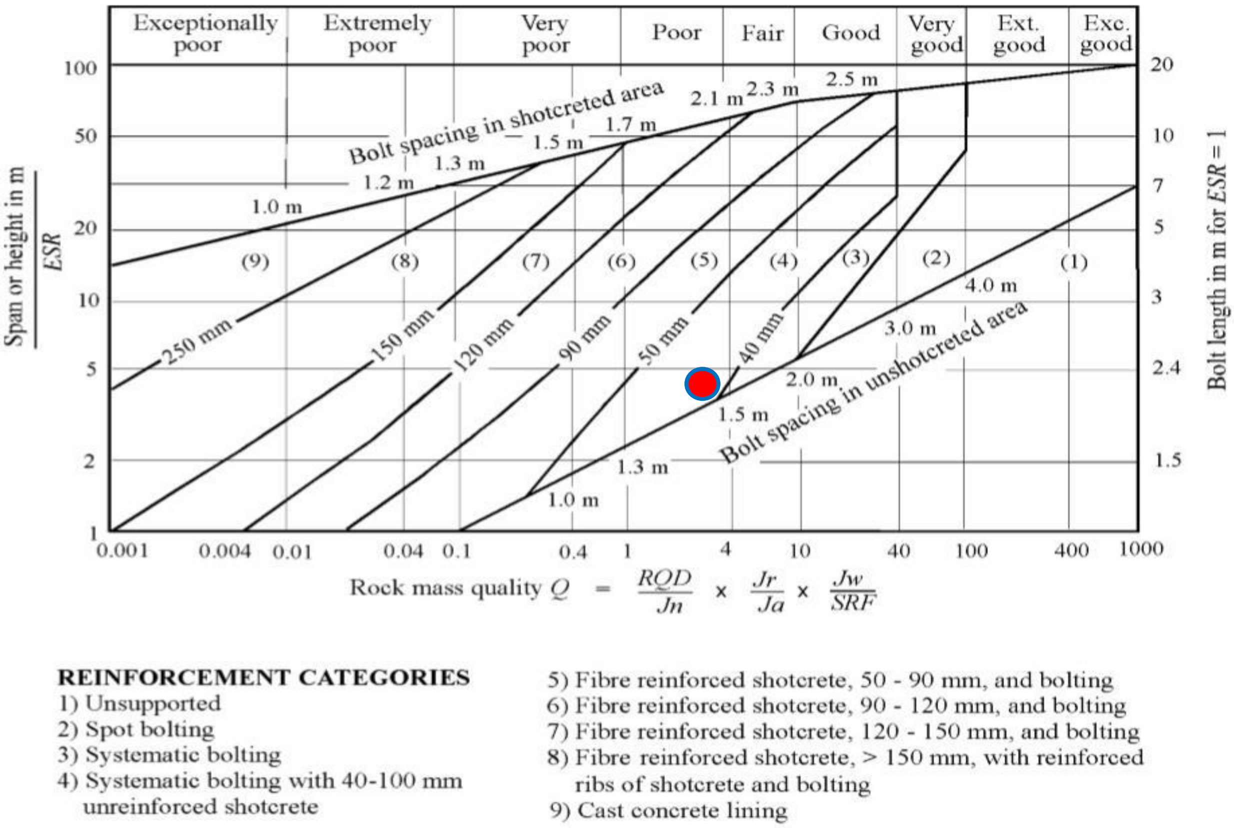

Figure 5: *Estimated support category based on the tunneling quality index Q (After Grimstad and Barton, 1993)*

7. Support Resistance

Maximum fall out height ≤ 1.2m based on the underground damage mapping. Equation [3] below depicts the demand of the support and the dead weight.

$$YZ = \frac{\text{Demand}}{\text{Dead Weight}} \qquad \text{Equation [3]}$$

Dead weight = $0.03MN/m^3 \times 1.2m = 0.036MN/m^2 = 36.0kN/m^2$
Demand = 36.0kN/m

$$\text{Support Resistance (SR)} = \frac{\text{Force}}{\text{Area}} \qquad \text{Equation [4]}$$

$$SR = \frac{100kN}{1.1m \times 1.4m} = 64.93kN/m^2$$

Factor of safety for the ground support design is defined as follows;

$$FoS = \frac{Capacity}{Demand} \qquad \text{Equation [5]}$$

$$FoS = \left(\frac{64.93kN}{m2}\right)/\left(\frac{36kN}{m2}\right) = \underline{\mathbf{1.8}}$$

Therefore, the FOS is **1.8**

8. Cable Bolt Support at Intersection

At intersection, the fall-out depth is taken from the rule of thumb, in which (6.0 m-2.5 m) = **3.5 m.**
Deadweight/Demand as shown in equation [6] = 5.3 m x 5.0 m x 3.5 m x 3.0 tonnes/m^3 = **278.3 tonnes.**

Allowable minimum factor of safety = 1.2
Cable bolt ultimate load bearing capacity = 38 tonnes
No. of cable bolts per ring using equation [6] below

$$\frac{Demand\ x\ FoS}{SR} = \frac{278.3\ x\ 1.2}{38} = 8.8 = 9 \text{ bolts} \qquad \text{Equation [6]}$$

Therefore, No. of cable bolts at the intersection = **9 cable bolts** at the intersection.

Table 1: *Final support design for the footwall and extraction drives*

#	Support Element	Support Standard
1	Friction anchor bolts	2.4 m long at 1.1 m bolt and 1.4 ring spacing following square pattern
2	Welded mesh	Galvanized 4.0 mL x 2.4 mW (overlapped at 300 cm)
3	Cable bolts at Intersection	6.0 m long, 38-ton cable bolts at 1.5 m bolt spacing x 3.0 m ring spacing
4	Shotcrete	50 mm thick shotcrete

9. Hanging wall Stability analysis

The stability of the hanging wall for the 30m roof span in the open stope, before backfilling, can be calculated using the method by Mathews et al. (1981). Below is the summarized geotechnical mapping database for the hanging wall obtained from boreholes.

Intact, massive, and unweathered rock
Damp (since this is underground environment)
UCS = 50 – 100 MPa (estimated)
2 Joint sets plus random
Joint spacing ranging between 0.1, 0.15, 0.09
Joint condition –slightly weathered. Slightly rough and planar, soft filled joints (1mm-5mm thick), persistence – 3 to 10 m and RQD = 60% - 70%

Modified Q' is calculated by the following equation [7] below: -

$$Q' = \left(\frac{RQD}{Jn}\right) x \left(\frac{Jr}{Ja}\right) \qquad \text{Equation [7]}$$

$$Q' = \left(\frac{60}{6}\right) x \left(\frac{1.5}{1}\right) = \underline{\mathbf{15}}$$

Hydraulic Radius or Shape Factor is also calculated by the following equation [8] below:

$$\text{Hydraulic Radius (HR)} = \frac{\text{Area}}{\text{Perimeter}},\ \frac{(25\ x\ 390)}{2(25+30)} = 6 \qquad \text{Equation [8]}$$

The stability of the hanging wall is assessed based on several key parameters, which are depicted in Figure 6 below. These parameters include factor A, which represents the stress factor that influences the structural integrity; factor B, which pertains to the orientation of the design surface and its impact on stability; and factor C, which accounts for the gravitational forces acting on the hanging wall. Each of these factors plays a crucial role in determining the overall stability of the hangingwall structure as shown in equation [9].

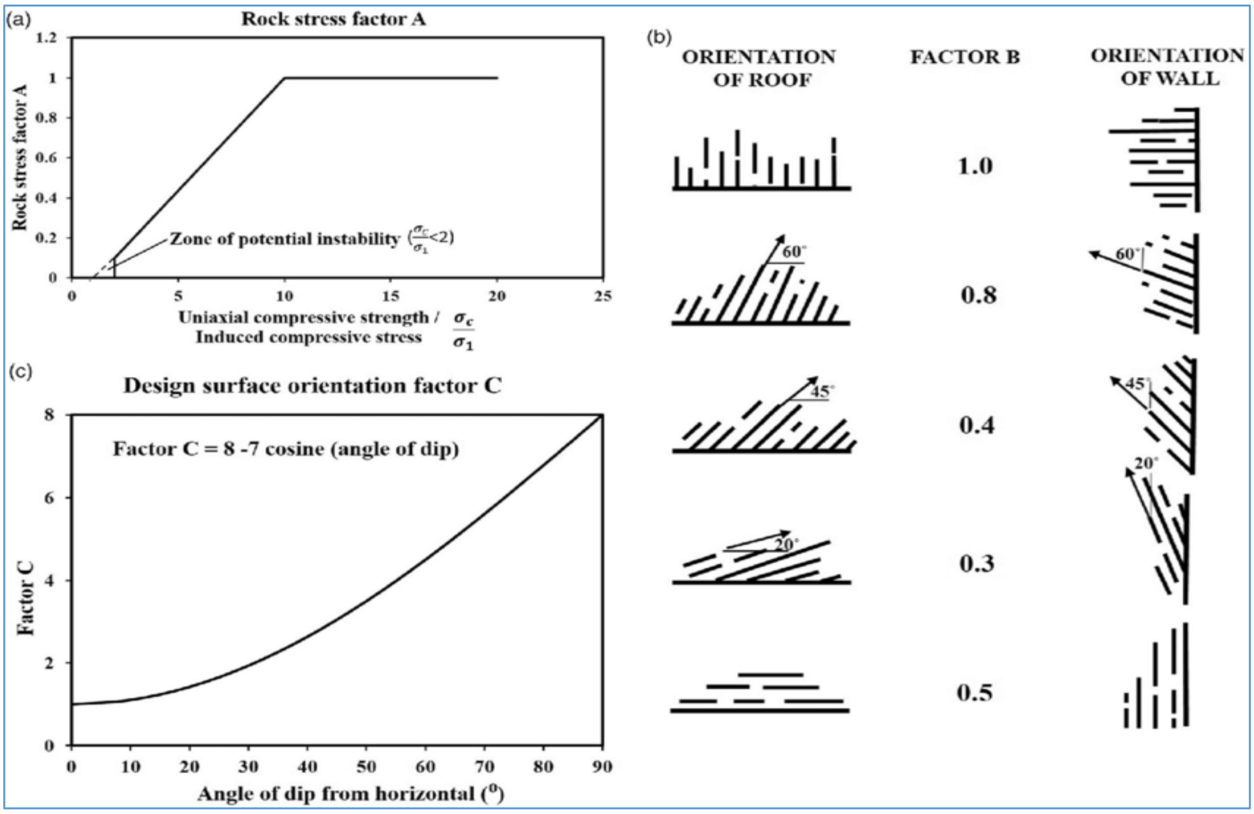

Figure 6: *Original stability graph number factors after Mathews et al. (1981): (A) rock stress factor (B) joint orientation adjustment factor (C) gravity adjustment factor.*

$$N' - \text{Stability Number} = Q' \text{ x A x B x C} \qquad \text{Equation [9]}$$

A – Rock Stress Factor = 0.8
B – Rock Defect Orientation Factor = 0.2
C – Design Surface Orientation Factor = 5.95
Q' – Modified NGI Rock Mass Rating = 15

N' = 15 x 0.8 x 0.3 x 5.95 = **21.42**

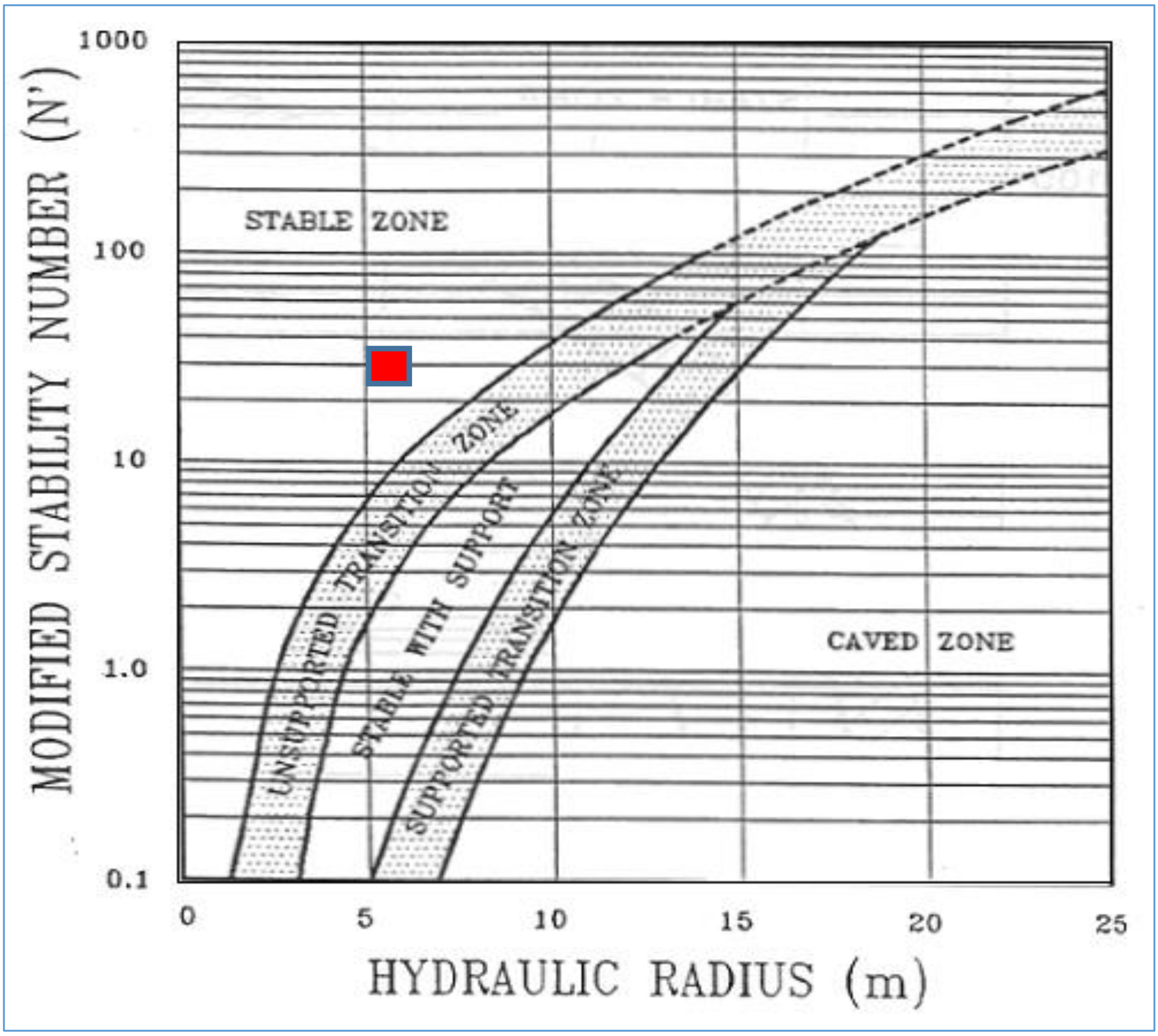

Figure 7: *Showing hangingwall in stable zone (after Mathews et al., 1981)*

The stability plot of the hydraulic number versus the modified stability number by Mathew et al. (1981) reveals a promising stable zone. This confirms that the primary stope will stand strong and resilient, allowing us to move forward without relying on cable bolts (Figure 7).

10. Numerical modelling

The option to mine four primary stopes, each measuring 30.0 m in height and 6.0 m in width, can be implemented without encountering any stability challenges during the mining process. The numerical analysis conducted in Phase 2D indicates that these four stopes can be mined successfully without the need for backfill (Figure 7). The results confirm that it is feasible to mine the four primary stopes before applying backfill to the secondary stopes. The Phase 2D numerical model evaluates the worst-case scenario, assuming no ground support in the form of roof reinforcement. However, since the Munali Nickel Mine is shallow, we do not expect significant mining-induced stresses during primary stoping to impact the stability of the excavations (Figure 8).

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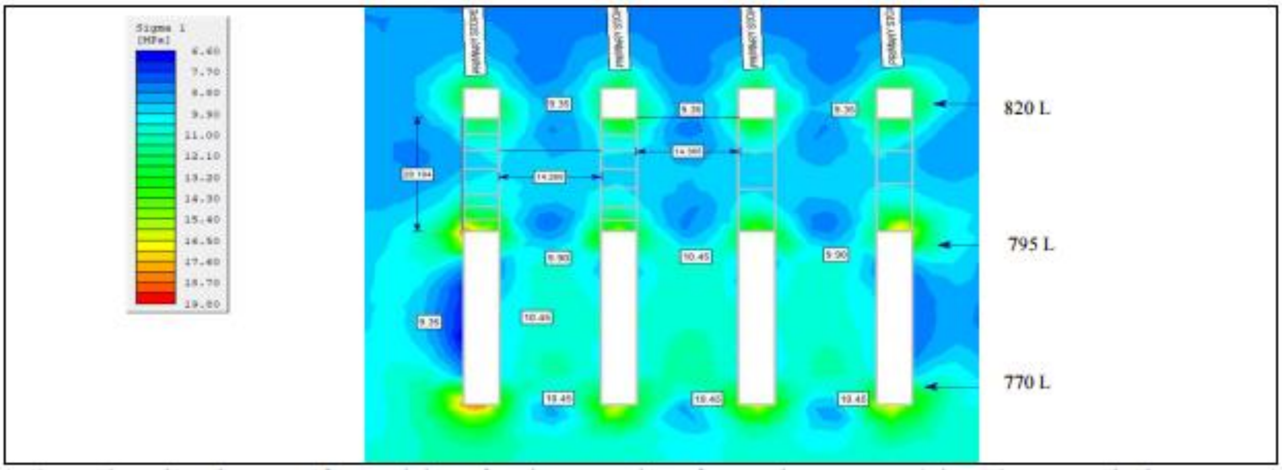

Figure 8: *Induced stresses from mining after the extraction of four primary stopes (Phase 2D Numerical Modeling)*

Phase 2D numerical modeling also took into account the strength factor to assess the stability of the secondary stopes after the primary stopes were backfilled with cemented rock fill. Figure 9 illustrates that the results were not favorable for the pillars designated as secondary stopes without backfill. However, this has a limited impact on mining secondary stopes, as primary stopes will be backfilled prior to the mining of the secondary stopes.

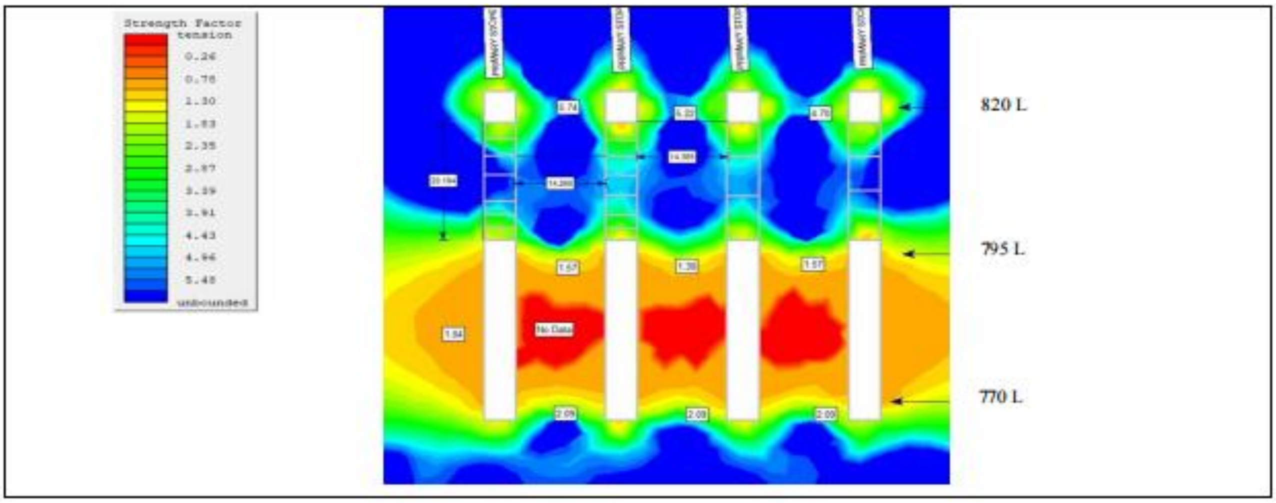

Figure 9: *Strength factor after mining four primary stopes (Phase 2D Numerical model)*

The four primary stopes were successfully backfilled with cemented rock fill (CRF), leading to positive outcomes that support the consideration of the 14.0-meter pillars as secondary stopes. As illustrated in Figure 10, the results clearly indicate that the backfilling with CRF enhances the viability of these pillars for classification as secondary stopes.

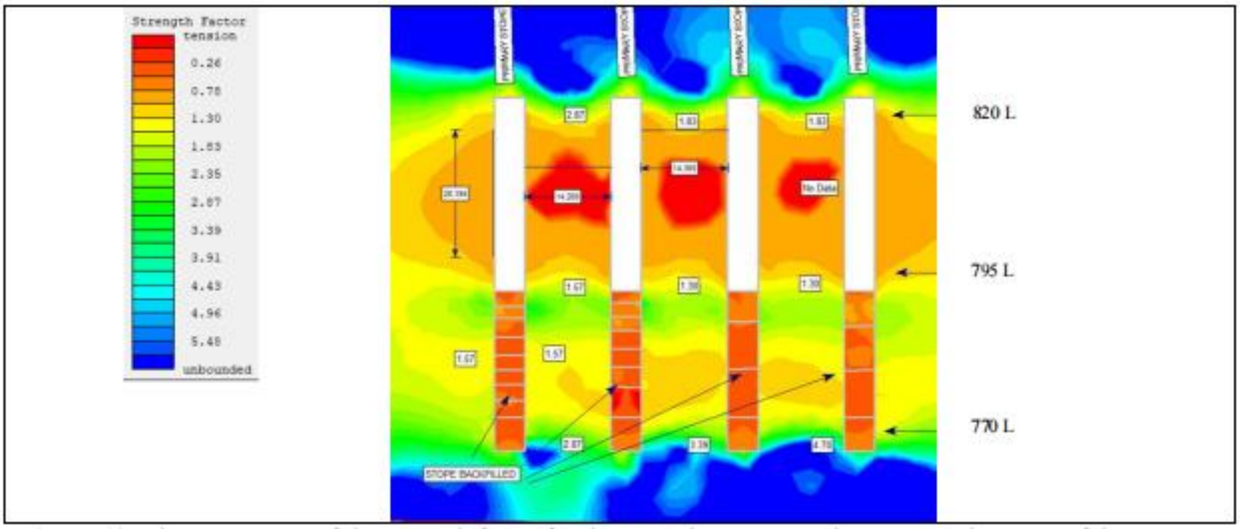

Figure 10: *The assessment of the strength factor for the secondary stopes subsequent to the successful backfilling of the primary stopes, as demonstrated through Phase 2D numerical modeling.*

The process of mining four primary levels and backfilling them with cemented rock fill can be effectively replicated on the upper and lower levels, yielding consistent results even with a low strength factor. To improve the overall strength factor, it is essential to focus on backfilling the primary levels as well.

11. Conclusion

The geotechnical parameters used in this design were obtained from underground mapping and current geological boreholes. For the analysis, we considered the lowest modified stability number fair to poor rock mass and plotted it against the stope hydraulic radius. We applied a similar method to the graph by Grimstad and Barton (1981), using the worst-case scenario, which provided valid results. Even though the plot of the stability number and hydraulic radius remained within the stable zone, it is crucial to consider widely spaced secondary support with 6.0-meter-long anchors. This precaution enhances the safety factor for both equipment and personnel during drilling, charging, blasting, and drawing activities. Additionally, this approach considers the stand-up time of the stope from development to stoping.

During the development of both primary and secondary stope, ensure the installation of primary support in the form of 2.4 m long split sets advancing together with welded mesh. 6.0m cable anchors must be installed at intersections before any stoping activities. The primary stope must be backfilled with cemented rock fill, and quality assurance and controls must be ensured on the backfill. Before any secondary stoping activities, it's crucial to ensure that the backfill achieves a strength of at least 1.2 Mpa after the recommended curing days by the mine. This strength is necessary to achieve a safety factor of more than 1.5, ensuring the cemented rock fill can withstand the dynamic loading from blasting of the secondary stopes.

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