

BINGQUAN WANG¹, SHUAI CAO^{1*}, EROL YILMAZ^{2,3}**FAILURE MECHANISMS AND PRE-REINFORCEMENT DESIGN FOR ORE PASSES UNDER HIGH IN-SITU STRESSES: A CASE STUDY FROM THE ASHELER DEEP MINE**

The current deep shaft environment at Asheler features highly stressed expansive rock with well-developed joints, poor surrounding rock stability, and a distinct foliated structure that softens and mudifies upon contact with water. Under joint impacts of high shaft stress and ore impact, existing ore pass has experienced multiple instances of shaft deformation and collapse. Taking the existing deep ore pass at Asheler as the engineering context, this study investigates the deformation and failure mechanisms of the surrounding rock during downward excavation (raise boring). Based on these mechanisms, a composite pre-reinforcement system consisting of pre-grouting and a steel-concrete lining is proposed. A numerical model of the pre-grouting diffusion range was constructed using COMSOL Multiphysics. The coupled reinforcement analysis incorporating the grouted zone was subsequently performed using FLAC^{3D}. The numerical results indicate that the proposed pre-reinforcement scheme can effectively control the deformation of the surrounding rock and limit the expansion of the plastic zone, thereby significantly enhancing the safety and stability of the ore pass.

Keywords: High in-situ stress; Grouted steel pipe pile; Pre-reinforcement; Ore pass stability; Numerical simulation

1. Introduction

As easily accessible surface deposits are depleted, mining operations are being forced to dig much deeper into the Earth [1-2]. This shift brings greater technical challenges, higher costs, and increased environmental and safety concerns [3-5]. As deep mining becomes standard practice,

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challenges known as “three highs and one disturbance” are becoming more pronounced [6]. These include high temperatures, high stress, high water pressure, and strong mining-induced disturbances [7-9]. Together, they pointedly increase operational difficulty and safety risks [10]. Against this backdrop, ore pass engineering faces multiple challenges, including the risk of surrounding rock collapse under high in-situ stress, impact-induced damage during ore transportation, and deteriorating rock mass conditions at greater depths [11]. These factors pose a risk of ore pass instability and collapse during both construction and operation, severely limiting the safe and efficient extraction of deep-seated orebodies.

Currently, Ashele Copper Mine, mining operations have extended beyond 1100 m in depth, where high in-situ stress becomes increasingly pronounced. Field measurements indicate that the maximum principal stress exceeds 43 MPa, severely constraining mining efficiency. Studies confirm a strong correlation between roadway damage and stress concentration zones, with the high horizontal tectonic stress predominantly causing wall shear failure in ore passes, leading to collapses and deformations that critically impair mine safety and production [12-13]. Intensifying mining depth, rock mechanical problems caused by the highland compressive stress environment in the “three highs and one disturbance” are one of the core factors inducing the deformation and collapse of the ore pass shaft [14]. Therefore, accurately understanding the stress distribution in deep wells, as well as the mechanisms of wellbore collapse and deformation, is increasingly vital. Such knowledge helps predict failures, guide safer designs, and improve the reliability of deep mining operations [15].

Deep shafts constructed as single vertical excavations have reached unprecedented depths in modern mining operations worldwide. The South Deep gold mine in South Africa operates shafts exceeding 3,000 m in depth, the Kidd Creek mine in Canada has reached depths of 3,000 m below surface, and in China, deep shaft construction has exceeded 1,500 m at several mines. At such depths, the intersection zone between the vertical shaft and horizontal access tunnels – i.e., the shaft station – becomes a critical structural region where high stress concentrations develop from two orthogonal directions. These intersection zones require support materials with enhanced durability and resistance to dynamic loading induced by stress relief and mining blasts [16]. The ore pass, as a special form of vertical excavation connecting adjacent mining levels, faces similar but more severe challenges due to the additional impact loads generated by ore transportation.

Extensive research has been conducted on the mechanisms of wellbore collapse and the distribution of stress around underground excavations. After shaft excavation, stress redistribution generates rock pressure, which is the primary cause of deformation, instability, and damage in the surrounding rock and support structures [17]. Under high stress, high water pressure, high rock temperature, and nonlinear dynamic loading, the failure of deep rock masses differs fundamentally from that of shallow rock masses [18-19]. Several failure criteria have been employed for wellbore stability analysis. Burnwal et al. [20] applied the Mogi-Coulomb criterion to optimize mud pressure design in shaley sedimentary reservoirs. Liu et al. [21] developed a wellbore stability model for transversely isotropic formations accounting for poroelastic effects. Zhou et al. [22] and Chen et al. [23] introduced a strain-softening Drucker-Prager model based on triaxial compression tests. More recently, Yang et al. [24] proposed a coupled thermo-mechanical framework incorporating both the Mohr-Coulomb and Mogi-Coulomb criteria for ultra-deep wellbore stability analysis.

Various support techniques have been proposed for ensuring the stability of deep boreholes. Hu et al. [25] validated the feasibility of high-performance steel fiber reinforced concrete for deep freezing shaft linings through field application. Xiang et al. [26] proposed an optimization method for deep shaft grouting reinforcement that balances stability and water inflow control.

Zhang et al. [27] revealed that circular structures minimize deformation through circumferential forces. Sun et al. [28] revealed the strength degradation mechanisms of rock masses under complex deep geological conditions and proposed novel constant-resistance (NPR) anchor bolts for controlling large deformation in deep shaft sinking. He et al. [29] developed a high pre-stress excavation compensation technique that transforms the surrounding rock from a two-dimensional to a three-dimensional stress state, effectively reducing stress concentration and enhancing support efficiency. Wang et al. [30] utilized an equivalent radius model and FLAC^{3D} simulation to optimize the parameters of a double-shell grouting system, effectively controlling deformation in loose and fractured rock.

However, most existing studies on ore pass stability focus on shallow or moderately deep environments under uniform stress conditions. Systematic investigations on pre-reinforcement design for ore passes under non-uniform high in-situ stresses – particularly those accounting for the directional distribution of principal stresses – remain limited. Compared with previous studies on the Asheler mine, which primarily examined in-situ stress measurement [12] and general ground pressure control [13], the present study makes the following original contributions: (1) it systematically investigates the progressive failure mechanisms of the ore pass during downward excavation under high non-uniform stress, revealing the evolution patterns of stress, displacement, and the plastic zone; (2) it proposes a graded pre-reinforcement design strategy that places support piles at variable densities according to the directional distribution of principal stresses – a design principle not previously applied to ore passes in this mine; and (3) it provides a comparative evaluation of four pre-reinforcement schemes, identifying the optimal configuration based on a combination of mechanical performance and engineering feasibility.

2. Engineering background

The Asher copper mine is located in northern Xinjiang, it is the second largest copper mine in China in terms of copper reserves. The Asher copper mine is currently mining to a depth of nearly 1000 m, with a depth of 1200 m being explored. It is a typical kilometer deep shaft expansion rock environment. The maximum principal stress is more than 43 MPa. The Ashele copper deposit occurs in the Ashele Formation (D₂as₂) of the Devonian volcanic-sedimentary sequence, with a nearly N–S trending synclinal structure dominating the mine area. The ore body is stratiform and lens-shaped, striking N–S with a dip of 55°–75°, buried at depths of 18–930 m. Surrounding rocks consist predominantly of tuff (Class IV, poor integrity), copper-sulfur ore, and pyrite (Class III), with well-developed bedding and foliation. Three engineering rock mass types are identified: massive, layered, and fractured structures, the latter being associated with fault zones and exhibiting the poorest stability.

Combined with on-site studies and related reports, nearby rock is mainly tuff with minor pyrite and copper-sulfur ores. Fig. 1 shows the the field ore pass. The on-site study shows that the geological structure is relatively developed, and the surrounding rock shows obvious sheet-like structure and high ground stress. There is a risk of deformation and collapse of the existing ore pass. As a vital transport channel connecting the two middle sections, the ore pass is a serious menace to safer mining and production. Fig. 2 presents a three-view schematic of the ore pass. The ore pass has a diameter of 4 m and connects the two levels over a vertical length of 50 m. All other dimensions depend on the size of the subsequent model. Pre-reinforcement is designed to mitigate the risk of ore pass collapse under high in-situ stress condition.

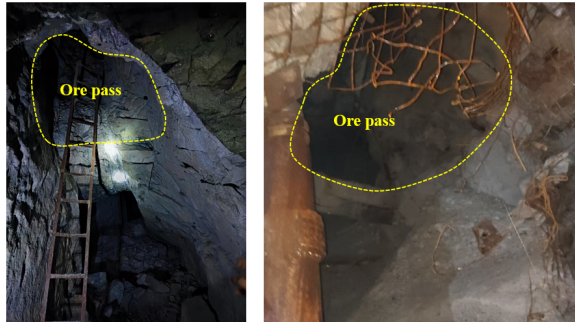


Fig. 1. On-site ore pass

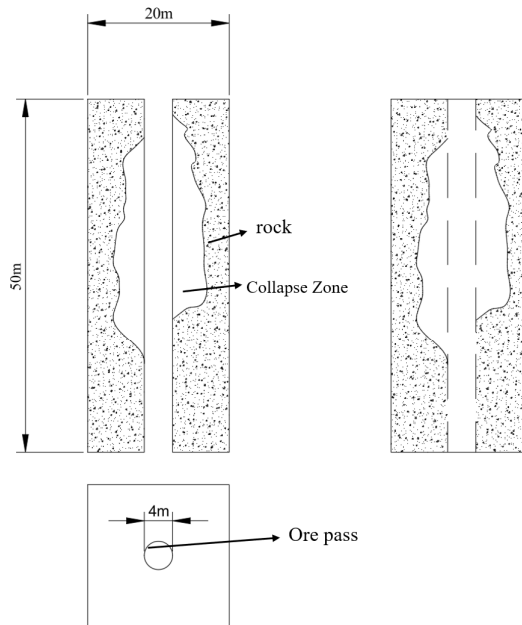


Fig. 2. Simplified three-view drawing of a ore pass collapse

The existing support system failed severely under deep high-stress conditions in swelling ground. In the tuff section, concrete lining cracks reached up to 5 mm in width, and localized collapse occurred. In-situ stress measurements indicated a maximum horizontal principal stress of 42.0 MPa and a lateral stress coefficient of 1.5, implying that horizontal tectonic stress overwhelmingly dominates [13]. Given that tuff readily softens upon water contact, the original support proved inadequate. Hence, the revised scheme joins a composite reinforcement package: C30 concrete lining 500 mm thick (800 mm in wear-prone zones), double-layer rebar meshes, rock bolts ($\Phi 22$ mm, $L = 2.7$ m at 1 m spacing), and U25 steel arches set at 1m vertical intervals. Moreover, 40 mm manganese steel plates and light rails are installed on the side subject to ore impact, forming an integrated system capable of resisting compression and preventing collapse.

3. Ore-pass simulation

3.1. Numerical model

Along with the Aschel copper mine's geological/actual working circumstances, a numerical model was established to explore the impairment development course of nearby rock in ore pass excavation. Via analyzing dislocation zone, stress field and plastic-zone development and distribution features of nearby rock subjected to unsupported mining condition, failing trends of nearby rock of ore pass were revealed.

Before the simulation, the ore pass excavation model was constructed using FLAC^{3D} with an ore pass radius of 2 m. To account for size effects, a three-dimensional numerical model measuring $20 \times 50 \times 20$ m (L $\times$ H $\times$ W) was established using the Mohr-Coulomb failure criterion. The model was discretized using hexahedral (brick) elements with approximately 628409 zones. The top surface was left free, lateral boundaries were constrained in the X and Y directions, and the base was fully fixed. A burial depth of 1,000 m and a lateral pressure coefficient $\lambda = 1.5$ were applied. The vertical stress was measured 27 MPa, and the horizontal stress 40.5 MPa. Since $\sigma_h \neq \sigma_v$, the stress field is anisotropic, with the maximum principal stress acting horizontally. The value $\lambda = 1.5$ was obtained from field measurements using the overcoring method at the 1000 m level of the Asheler mine [12]. The resulting model configuration is illustrated in Fig. 3.

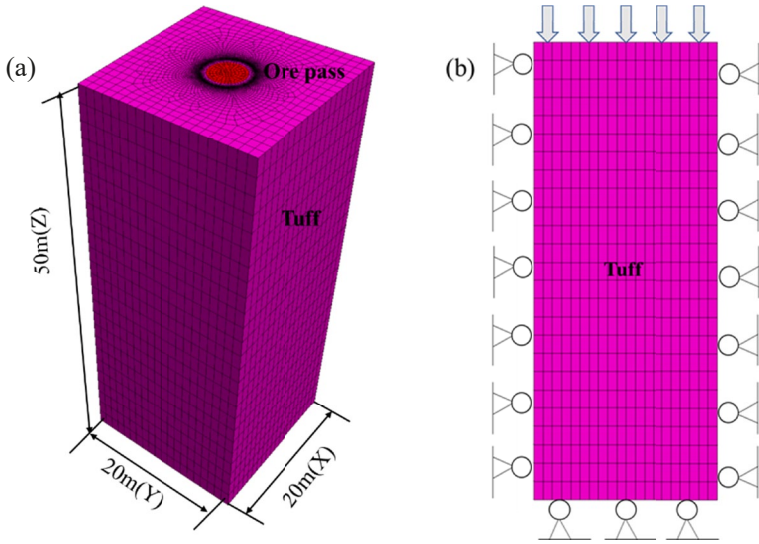


Fig. 3. Simulation scheme: (a) model size and composition, and (b) boundary conditions

The physico-mechanical parameters of intact rock samples (TABLE 1) were determined through laboratory tests. Considering the influence of multiple factors on the in-situ rock mass properties, the improved Hoek-Brown criterion [31] was adopted to estimate the rock mass parameters. Structural discontinuities at the Ashele Copper Mine were surveyed using the scanline method and analyzed with Dips software [32]. The estimated parameters are presented in TABLE 2.

TABLE 1

Physico-mechanical features of rock mass

Lithology	Density (kg·m ⁻³)	Compressive strength (MPa)	Tensile strength (kN)	Modulus of elasticity (GPa)	Poisson's ratio
Tuff	27.24	54.27	6.83	47.51	0.27
Pyrite	31.81	51.41	6.77	33.54	0.31
Copper sulfur ore	46.61	178.44	11.54	78.03	0.20

TABLE 2

Physico-mechanical features of rock mass based on the Hoek–Brown failure criterion

Lithology	Tensile strength (MPa)	Compressive strength (MPa)	Modulus of elasticity (GPa)	Cohesive force (MPa)	Friction angle (°)	Density (kg·m ⁻³)
Tuff	0.07	2.70	11.46	3.01	31.26	27.41
Copper sulfur ore	0.30	12.35	22.45	7.18	40.40	44.66
Pyrite	0.24	9.50	15.78	6.09	38.59	37.22

3.2. Evolution and distribution process of stress in ore pass

Fig. 4 illustrates the development and spatial redistribution of stress in the surrounding rock of the ore pass. After the first excavation stage (5 m), an unloading zone extending 1.5 times the ore pass radius was formed near the excavation surface. Due to the shallow excavation depth, the stress redistribution range was initially limited. As the excavation footage increased, both the length and width of the unloading zone expanded, and the stress absorption zone gradually

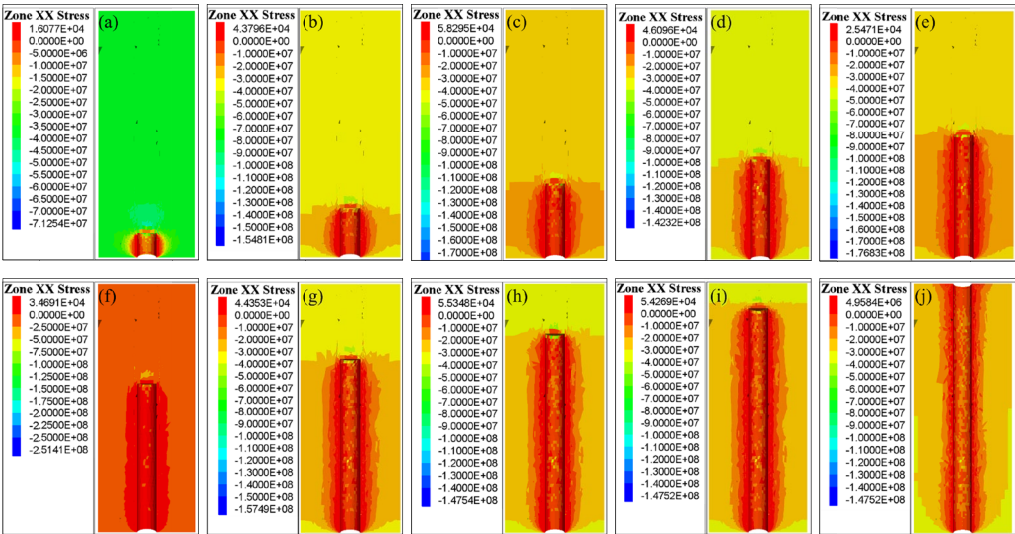


Fig. 4. Unpre-reinforcement excavation XX Stress: (a) 5 m; (b) 10 m; (c) 15 m; (d) 20 m; (e) 25 m; (f) 30 m; (g) 35 m; (h) 40 m; (i) 45 m; (j) 50 m

propagated into the surrounding rock mass. During the later stages of continued excavation, the extent of the stress redistribution zone no longer increased significantly, and the stress values stabilized. After the excavation was completed, the extensive rock damage prevented the surrounding rock from sustaining high stress levels.

3.3. Distribution process characteristics of displacement

Fig. 5 illustrates the evolution of displacement in the surrounding rock of the ore pass. The X-displacement contour map exhibits a gradient distribution symmetric about the ore pass axis. The displacement is maximum near the shaft wall and decreases exponentially with distance, becoming negligible at a distance of approximately twice the ore pass diameter. The contour lines are continuous without abrupt changes. During the initial excavation, stress is released under high in-situ stress conditions. Owing to the inherently poor mechanical properties of the rock mass, the rock is extensively damaged, with a maximum displacement of 17.67 mm, indicating that the rock enters the plastic state immediately after the first excavation. As the excavation depth increases, the zone of maximum displacement migrates downward with the advancing excavation face, and the maximum displacement increases to 25.98 mm.

Upon completion of the ore pass excavation, the entire shaft was fully penetrated. The continuity and stability of the rock mass were severely compromised. At the breakthrough point, the rock mass lost the supporting effect of the adjacent unexcavated rock, resulting in a weakening of the confinement provided by the surrounding rock. The residual Z-axis stress was fully released, and the strain energy was transferred to the surrounding rock around the unexcavated zone. The maximum radial displacement in the X-direction reached 30 mm. In combination with the densely concentrated plastic zone at the final excavation stage, the XX-stress in the shaft wall rock increased, resulting in a high degree of damage.

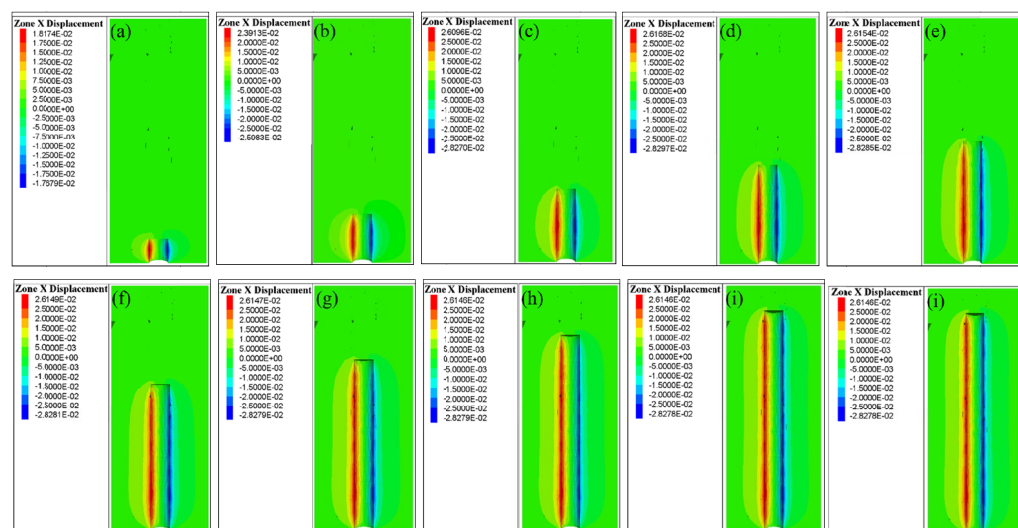


Fig. 5. Unpre-reinforcement excavation X displacement: (a) 5 m; (b) 10 m; (c) 15 m; (d) 20 m; (e) 25 m; (f) 30 m; (g) 35 m; (h) 40 m; (i) 45 m; (j) 50 m

3.4. Distribution characteristics of plastic zone

Fig. 6 illustrates the spatial evolution of the plastic zone in the ore pass under unsupported conditions. The development proceeds as follows: during the initial excavation, the removal of radial confinement induces circumferential stress concentration in the ore pass wall, forming a symmetrical annular plastic zone. As excavation progresses, damage from preceding stages degrades the rock strength, gradually forming a micro-crack network. This leads to stress redistribution and diversification of energy release pathways, causing the plastic zone to transition from a continuous concentrated zone to a widely distributed discrete pattern. By the final excavation stage, in the absence of effective support, energy is suddenly released, forming a new concentrated plastic zone in the shaft wall. This may ultimately result in breakthrough instability.

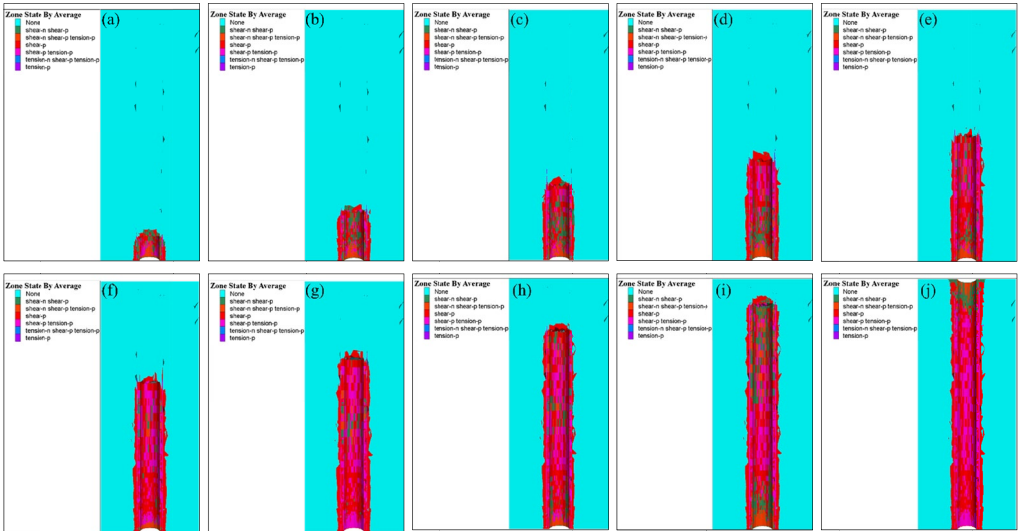


Fig. 6. Unpre-reinforcement excavation plastic zone: (a) 5 m; (b) 10 m; (c) 15 m; (d) 20 m; (e) 25 m; (f) 30 m; (g) 35 m; (h) 40 m; (i) 45 m; (j) 50 m

The evolution of the plastic zone with excavation length and plastic zone volume is summarized in TABLE 3. As the excavation advances, the maximum plastic zone radius increases from 3.49 m at 5 m depth to 4.76 m beyond a depth of 50 m. Shear failure consistently dominates throughout the excavation process. And the plastic zone increase with excavation depth. The corresponding plastic zone volume grows with excavation depth, reaching a total of 1140.9 m³ at full excavation.

Fig. 7 presents a cross-sectional view of the plastic zone in the excavation section. Slices were taken at 5 m intervals to study plastic zone evolution. The consequences signify that plastic-zone predominantly extends about 1.5 m beyond the ore-pass radius. Shear failure is the dominant failure mode in nearby rock, with minor occurrences of tensile failure.

TABLE 3

Evolution of the plastic zone with excavation length under un-reinforced conditions

Excavation length (m)	Max. plastic zone radius (m)	Plastic zone volume
5	3.49	97.3
10	3.92	221.8
15	4.07	343.5
20	4.07	452.3
25	4.07	553.9
30	4.07	657.8
35	4.07	759.7
40	4.07	877.2
45	4.07	1016.7
50	4.76	1140.9

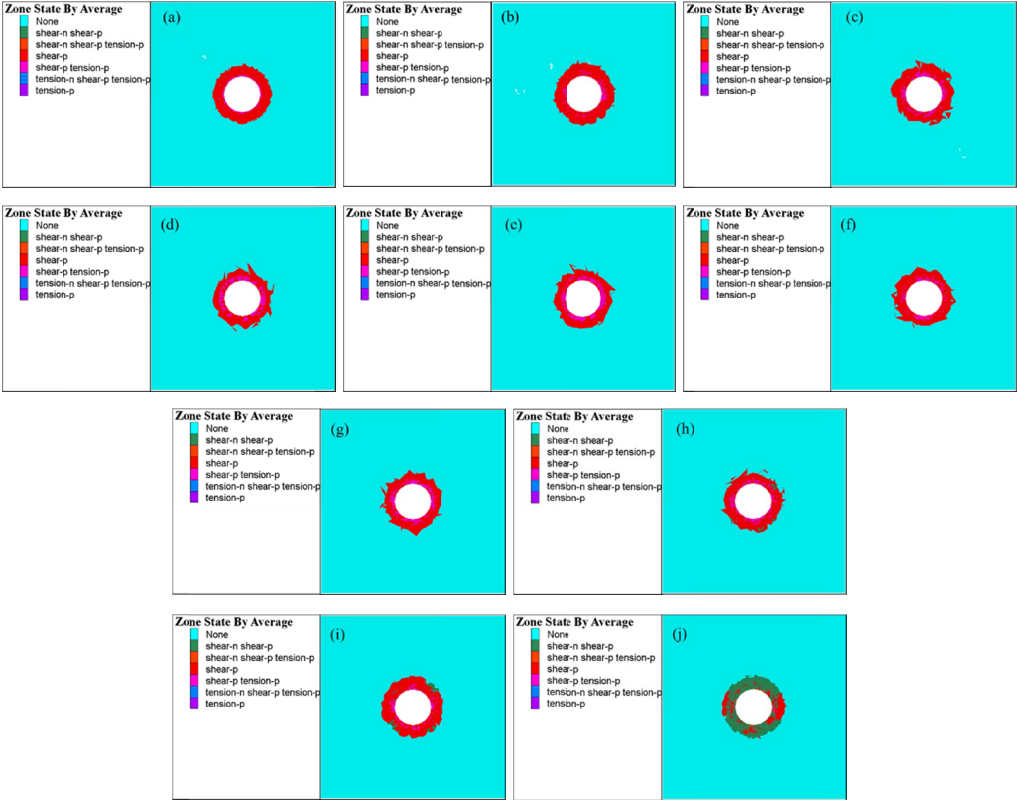


Fig. 7. Unpre-reinforcement excavation plastic zone: (a) 2.5 m; (b) 7.5 m; (c) 12.5 m; (d) 17.5 m; (e) 22.5 m; (f) 27.5 m; (g) 32.5 m; (h) 37.5 m; (i) 42.5 m; (j) 47.5 m

4. Ore-pass's pre-reinforcement and simulation

4.1. Pre-reinforcement design principles

As indicated by the studies reviewed above, relying solely on conventional active and passive support systems cannot achieve the desired results in deep high-stress environments. Fractured rock masses exhibit inherently low mechanical properties. Grouting is a well-established technique that can effectively enhance the strength characteristics and reduce the hydraulic conductivity of fractured rock masses. Pre-grouting of fractured rock has been shown to be more effective than post-excavation grouting [31-36]. Pre-grouting fills rock fractures and improves the shear strength of structural planes. Through pre-grouting, the loose surrounding rock is transformed into a stable structure with enhanced cohesion and friction angle. To address the risk of deformation, collapse, and failure of the original support system in the Asheler ore pass, we propose the pre-reinforcement scheme shown in Fig. 8.

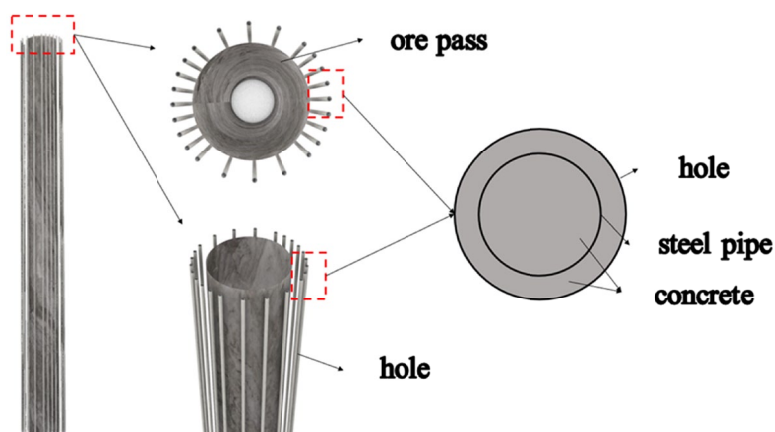


Fig. 8. Pre-reinforcement construction schematic diagram

4.2. Design scheme of pre-reinforcement technology for ore-pass

4.2.1. Pregrouting design

Based on the numerical simulation results and the deep ground pressure profile of the Asheler copper mine, a comprehensive high-strength grouting pre-support scheme was developed. The drilled holes and steel pipe piles used for pre-grouting penetrate and fill fractures in the surrounding rock, reducing its permeability while enhancing the cohesion and friction angle of the rock mass. This improves the coupling between the pre-support system and the surrounding rock, significantly enhancing ore pass stability during excavation. The grouting material was modeled as Portland cement P.O. 42.5. The steel pipes were Q235-grade seamless steel pipe with an outer diameter of 108 mm, wall thickness of 6 mm, yield strength of 235 MPa, and elastic modulus of 200 GPa. The concrete filling between the pipes and the borehole wall was C30-grade concrete with a compressive strength of 30 MPa and an elastic modulus of 30 GPa.

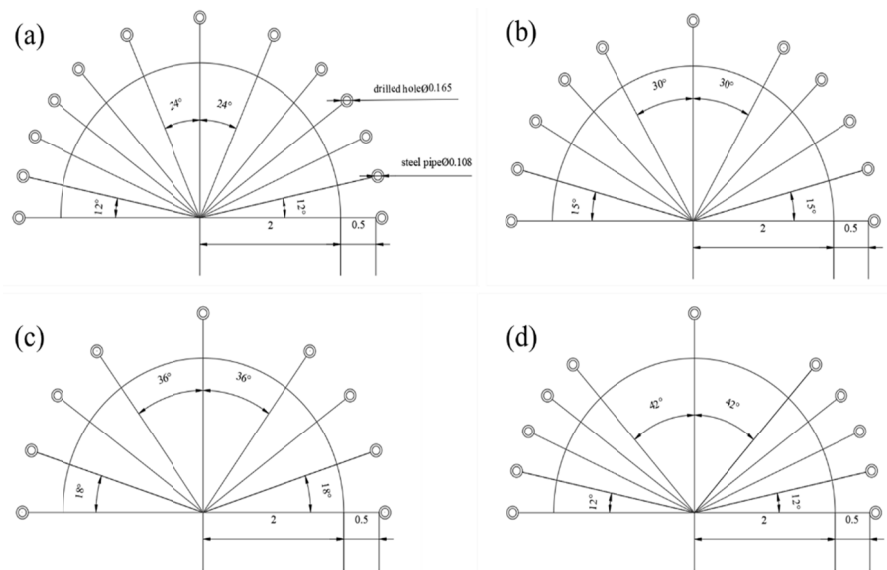


Fig. 9. Diagram of diverse pre-reinforcement structures: (a) I; (b) II; (c) III; and (d) IV

Drilled holes and steel pipe piles used for pre-grouting penetrate and fill fractures in the surrounding rock, reducing its permeability while enhancing cohesion and friction angle. This strengthens the coupling between pre-support system and rock mass, suggestively improving ore-pass stability during excavation. Additionally, the density of pre-support piles is strategically increased along the east-west direction, where ground stress is higher, to better align with the stress distribution and maximize the reinforcement effect. TABLE 4 summarises the detailed parameters for the various pre-support schemes.

TABLE 4

Experimental layout scheme

Scheme	Angle in concentrated areas	Number of macroholes	Number in non-concentrated areas
Scheme I	12	12	3
Scheme II	15	11	3
Scheme III	18	9	3
Scheme IV	12	10	1

Schemes I, II, and III are designed to consider the layout angle of the dense pile area in high ground stress track. Schemes I and IV mainly explore operation of not drilling support piles in the non-dense areas of the piles. The safe, efficient and economical pre-strengthening scheme is selected through simulation experiment results from the stress, displacement and plastic zone after excavation to make comparison. The specific model diagrams for different pre-reinforcement schemes are shown in Fig. 10. Since Schemes I and IV differ only in the number of holes, they share the same model for the considered analyses.

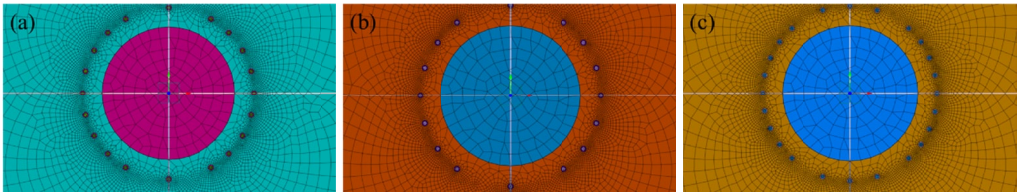


Fig. 10. Schematic diagrams of different pre-reinforcement schemes: (a) I; (b) II; and (c) III

4.3. Simulation

4.3.1. Grout spread in ore-pass

Using COMSOL Multiphysic, the diffusion range of pre-grouting within the weakened zone of the ore pass was calculated, with boundary and initial conditions defined according to the actual engineering context., the diffusion range of pre-grouting within the weakened zone of the ore pass was calculated, with boundary and initial conditions defined according to the actual engineering context. A grouting diffusion model measuring $20 \times 20 \times 60$ m was established, with a slurry concentration of 0.5 mol/m^3 selected as the effective strengthening range. To analyze the distribution, cross-sections were generated every 5 m, resulting in 10 sections in total. Contour plots for each section were created and compiled into a three-dimensional map illustrating the grouting diffusion range Fig. 11. The grout propagation was modeled using a pressure-driven Darcy flow coupled with a solute transport equation (advection-diffusion) in COMSOL Multiphysics. The grout was treated as a non-Newtonian Bingham fluid with a yield stress of a plastic viscosity of $\mu_p = 0.02 \text{ Pa}\cdot\text{s}$. The threshold concentration of 0.5 mol/m^3 was used to delineate the effective strengthening boundary. Within the effective grouting zone ($C \geq 0.5 \text{ mol/m}^3$), the mechanical properties of the rock mass were enhanced as TABLE 5. Since there is no relevant field test data, the exact enhanced parameters was followed with the relevant literatures [37-40]. The specific reinforced parameters were determined by referring to relevant published studies.

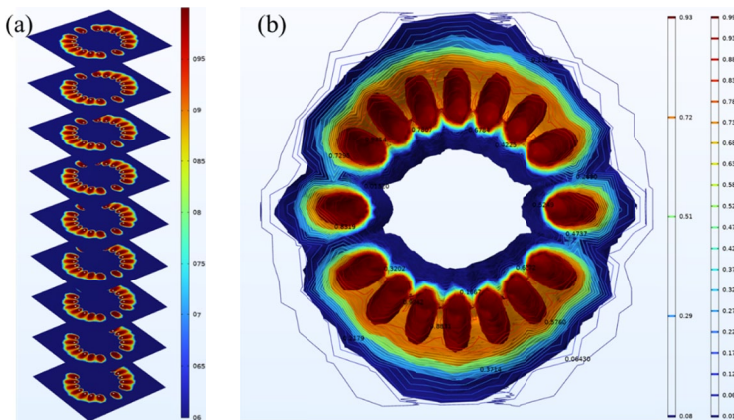


Fig. 11. Chart of diffusion of pregROUTING: (a) pregROUTING diffusion sections; (b) pre-grouting diffusion isosurface

In practical engineering conditions, grout is injected under pressure; adopting 0.5 mol/m^3 as the critical threshold enables more accurate identification of the reinforcement scope.

TABLE 5

The enhanced mass key parameters

Parameter	Un-grouted rock mass	Grouted rock mass
Cohesion (MPa)	3.25	5.42
Friction angle φ ($^\circ$)	31.26	31.8
Tensile strength (MPa)	0.07	1

A 1D plotting scheme was developed to analyze grout concentration variations along the X-axis, identifying both the diffusion extent and attenuation behavior of grout. The effective consolidation range was then determined. Based on this, a corresponding numerical model was constructed in Rhino, and grout diffusion range was implemented in another software using appropriate command sequences. The process concluded with the simulation of shaft excavation. The steel pipe-concrete-rock mass interface was modeled using the perfect bonding (tie constraint) assumption in FLAC^{3D}, justified by the pre-grouting process ensuring sufficient grout penetration and mechanical interlocking at all interfaces. The steel pipes were modeled using shell elements (six degrees of freedom per node), while the concrete and rock were represented by solid elements.

4.3.2. Numerical simulation results

Fig. 12 shows the volume and percentage reduction of the plastic zone in the ore pass under different pre-reinforcement schemes. Enhancing the strength of the surrounding rock through pre-grouting alone significantly limits the growth of the plastic zone. This treatment not only improves the shear resistance of the rock mass but also effectively constrains the propagation of plastic damage. The plastic zone volumes for Schemes I through IV are 611.9, 624.5, 643.0, and 614.6 m^3 respectively. Compared with the un-reinforced case ($1,140.9 \text{ m}^3$), the plastic zone volume was reduced by 46.3%, 45.0%, 43.5%, and 46.1% for Schemes I-IV, respectively. All four pre-grouting schemes effectively limit the plastic zone volume.

TABLE 6 shows the comparison of four schemes. The total plastic zone volume under un-reinforced excavation conditions is $1,140.9 \text{ m}^3$, of which the shear plastic zone accounts for

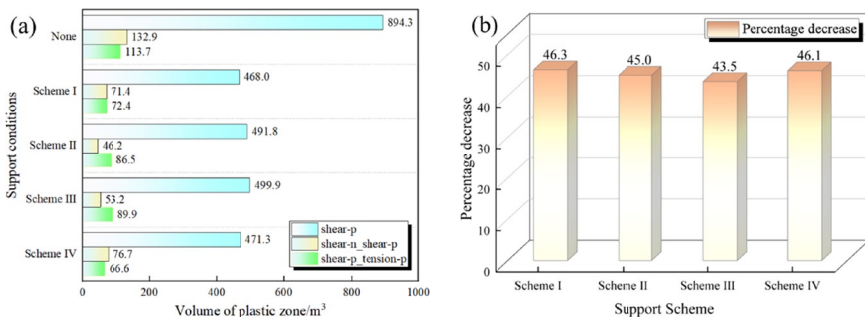


Fig. 12. Size of plastic-zone of nearby rock by diverse pre-reinforcement structures:
(a) Plastic zone's volumetric histogram; (b) Percentage reduction in excavated rock mass's plastic zone

TABLE 6

Summary of plastic zone volumes for different pre-reinforcement schemes

Scheme	PZ volume (m ³)	Reduction (%)
I	611.9	46.3
II	624.5	45.0
III	643.0	43.5
IV	614.6	46.1
None	1,140.9	0

approximately 1,027.2 m³ representing 90.09% of the total volume. This clearly indicates that the dominant failure mode of the ore pass is shear failure.

Fig. 13 shows the displacement evolution under the four pre-grouting schemes. The maximum horizontal displacements of the surrounding rock are 9.90, 10.16, 10.20, and 9.87 mm for Schemes I through IV, respectively. For comparison, the maximum displacement under unreinforced excavation is 30 mm, indicating complete destruction of the rock mass. The difference in maximum displacement between Schemes I and IV is negligible (9.90 mm and 9.87 mm, 0.3%), falling within numerical simulation error. The slightly lower displacement of Scheme IV is attributed to: (i) drilling disturbance in low-stress regions being avoided in Scheme IV, which preserves rock integrity; and (ii) the pre-grouting overprint, where grout strengthening alone in low-stress zones matches the marginal contribution of additional piles. These observations are consistent with findings in deep tunnel support literature.

Equated to unpregrouting condition, displacement of four schemes decreased by 67.0%, 66.13%, 66.0%, 67.1%, respectively. It can be seen that during the construction of prestressed piles, the maximum displacement of different pregrouting schemes are obviously reduced. It is mainly

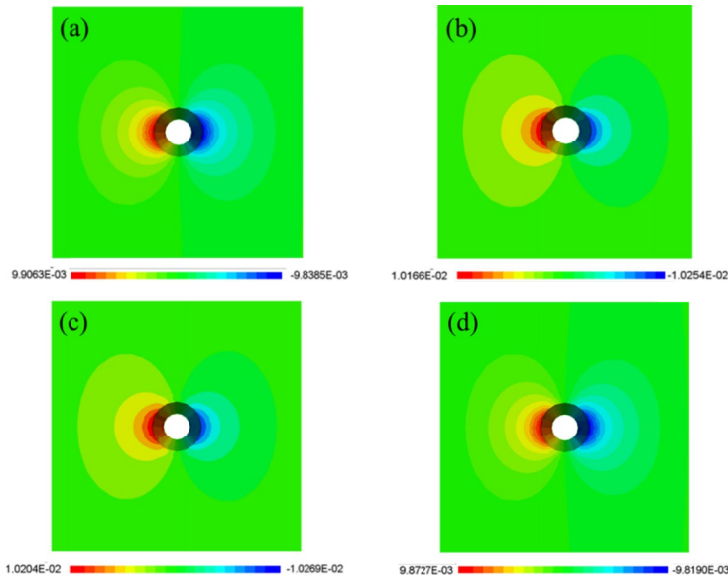


Fig. 13. *X* displacement under diverse pre-reinforcement schemes: (a) I; (b) II; (c) III; and (d) IV

due to mechanical property of rock mass are enhanced by the reinforcement effect of the pre-grouting process, together with the prestressed piles and surrounding rock forming stable support structure, which effectively controls nearby rock dislocation's expansion. This is akin to plastic zone's progress tendency. Analysis of Schemes I-III shows that increasing pre-stress pile density along the maximum principal stress has little effect on wellbore displacement, while Schemes I and IV indicate that reducing piles along the non-maximum stress direction also has minimal impact.

Fig. 14 shows the stress contour plots after excavation for all four pre-reinforcement schemes. The results demonstrate that pre-reinforcement significantly improves strength performance of nearby rock, while the steel-concrete composite system effectively confines stress concentration zones along X and Y directions. From an engineering feasibility perspective, the four proposed schemes differ primarily in the number of drill holes and steel pipe piles required. Based on construction conditions typical of deep metal mines, the relative cost comparison is presented in TABLE 7. Scheme I offers the best overall performance, while Scheme IV achieves an approximately 15% cost reduction with comparable displacement control. For critical ore passes requiring long-term service, the additional cost of Scheme I is justified by its more uniform reinforcement and higher safety margin.

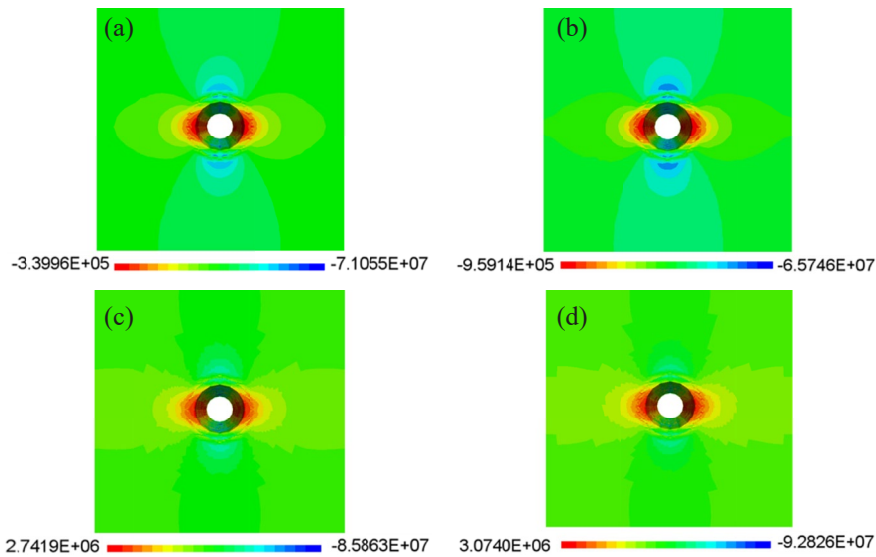


Fig. 14. XX stress under pre-reinforcement structures: (a) I; (b) II; (c) III; and (d) IV

TABLE 7

Cost comparison of pre-reinforcement schemes

Scheme	Relative cost	Displacement reduction (%)	Plastic zone reduction (%)
I	1.00 (baseline)	67.0	46.3
II	0.92	66.1	45.0
III	0.92	66.0	43.5
IV	0.85	67.1	46.1

5. Conclusions

This paper investigates the evolution patterns of rock mass fracturing and the optimization of pre-reinforcement during ore pass excavation in deep high in-situ stress environments. Based on the simulation software and integrated with the actual conditions of the Asher Copper Mine, this study systematically explores the stress field, displacement field evolution features, and deformation failure instruments of nearby rock in ore pass. Key conclusions are below:

- (1) During downward excavation without support, the surrounding rock undergoes progressive damage: stress redistribution, displacement accumulation, and expansion of the plastic zone from localized concentration to widespread distribution. The plastic zone extends approximately 1.5 m beyond the ore pass radius, with a total volume of 1,140.9 m³, of which shear failure accounts for 90%.
- (2) Along with physico-strength features of nearby rock in ore pass, its failure mechanisms, and actual site conditions, an optimized pre-excavation reinforcement scheme combining “pre-grouting with steel pipe piles” was proposed. The results demonstrate that anti-digging pre-reinforcement optimization scheme effectively improves the mechanical properties of nearby rock in ore pass wall. Distortion resistance of nearby rock is markedly enhanced.
- (3) Under un-reinforced conditions, the plastic zone extended approximately 1.5 m beyond the ore pass radius (radius = 2 m), with a total plastic zone volume of 1140.9 m³, of which shear failure accounted for 90.09%. After pre-reinforcement (Scheme I), the plastic zone volume was reduced to 611.9 m³, a decrease of 46.3%, and the maximum surrounding rock displacement was limited to 9.90 mm, representing a 67.0% reduction compared to the un-reinforced case.
- (4) Based on a comprehensive evaluation of plastic zone control, displacement, and cost, no single scheme outperforms in all aspects. Scheme I offers superior plastic zone control (611.9 m³) and uniform stress distribution, while Scheme IV achieves comparable displacement (−67.1%) at approximately 15% lower cost. Scheme I is recommended for critical long-life ore passes requiring high safety margins; Scheme IV is a viable alternative when cost constraints apply.

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