Goaf and Slope Stability: A Case Study on the Yangla Copper Mine

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Abstract. Considering the wide distribution range of deposits, the complicated geological conditions, and the potential detriment to slope stability during the caving period, underground goaf and slope stability is a critical concern in the Yangla copper mine. In this paper, goaf and slope stability is studied by implementing empirical analysis and numerical simulations with an equivalent continuum model. An overall understanding of underground goaf stability is obtained by the Laubscher graph method, and an approach composed of the geological modeling technology GoCAD and the specialized meshing tool Gridle is proposed to manage the geological database and prepare the numerical model for stability analysis with FLAC$^{3D}$. The Hoek–Brown constitutive model is introduced to simulate the complicated mechanical behavior of rock mass in the process of mining, i.e., the brittleness and confinement effect of rock mass. The key orebodies KT2 and KT5 develop in reverse inclination from slope surface to the inner slope with a maximum depth of 900 m, which means the rock mass bears a high stress induced by mining. The mechanical parameters are calibrated carefully by performing mining simulations that are consistent with the field instability phenomenon and measurement results. In particular, rock-mass behavior is predicted by sensitivity modeling in consideration of the goaf span, mining sequence, and material filling, the aim of which is to optimize the mining design to maintain slope stability during the third mining phase. The calculation results indicate that large deformations at the roof of the goaf and pillar are a critical problem in the Yangla copper mine as they are detrimental to slope stability. Moreover, many goafs collapse during the second and third phases of underground mining, which is also detrimental to slope stability, and the span of the goaf with a deep buried depth needs to be adjusted from 36 to 24 m in the third mining phase. The slope is stable in the second mining phase, but, in the third phase, it enters the large-scale instability state due to the destruction of the rock-mass structure near the slope surface by underground caving. Accordingly, goafs with the potential for collapse should be filled with high-strength materials to maintain slope stability.

Keywords: Yangla Copper Mine, Stability, Geological Modeling, Goaf, Slope.

1. Introduction
The Yangla copper mine is located in the northwest of the Yunnan province, China. Copper exploration began in 1965. By 2003, it was preliminarily proved that the mining area is 8 km long from north to south and 3–5 km wide from east to west, with a total area of 35 km$^2$ and 1.2–1.3 million tons of copper deposit. Yangla copper mine consists of seven mining sections, namely, Beiwu, Nilu,
Linong, Lunong, Jiangbian, Tongjige, and Jiaren from north to south. All mining sections are characterized by strong topographic cutting with high mountains and narrow valleys.

According to the geological conditions and characteristics of the deposit, the Linong section was designated as the first mining area with stope and fill used as the mining method, in which ore bodies are deposited in reverse slope direction as sporadic veins with different scale and locally expose at the topography. KT2 and KT5 are the largest ore bodies, and the mining sequences for them are subdivided into three phases. Fig. 1 summarizes the geology conditions and mining sequences for the Linong section. It should be noted that KT2 and KT5 have a remarkable thickness range of approximately 10–70 m and 6–56 m, respectively, which would be detrimental to the stability of part of goafs formed in.

![Fig. 1. Summary of geology conditions and mining sequence for the Linong section.](image)

The first phase of mining was designed for the shallow part of the ore body in the slope and was implemented before 2012. Unfortunately, on January 19, 2012, the “1.19 landslide accident” took place at the slope of the Linong section due to the first phase of mining, as shown in Fig. 2. This seriously impacted the underground stope, but there were no casualties. As a result, prominent deformation and destruction phenomenon were observed at the site. In light of the considerable size and depth of the area to be mined in the second and third phases, all goafs formed in the first phase of mining were applied targeted treatment by filling with comment material for improving the stability of whole underground goafs and slope in the subsequent mining, then the second stage of mining started at the end of 2012 and was completed by 2018. On the basis of the obtained exploration data and engineering experience, the goaf and slope stability have been forecasted by geological modeling and numerical analysis in this study, and reasonable suggestions are proposed for controlling the potential problems to be encountered during the third phase of mining, such as the choice of treat measure for the goafs formed in the second mining phase and the optimization of the caving design in the third mining phase.
2. Content and Workflow for Stability Study

Fig. 3 shows the research content and workflow for the stability analysis, which was formulated on the basis of the geological conditions and engineering characteristics of the Linong mining area. The topics below are studied in this project.

- **3D geological model**: Because the basic geological conditions and mining structure of the mining area are extremely complex, a 3D model consisting of main geology units and underground stope is constructed using the specialized geological 3D software GoCAD, which provides a fundamental basis for subsequent geological analysis and the quick generation of the FLAC$^{3D}$ numerical calculation model.
- **Empirical stability analysis**: The empirical graph approach for caving stability assessment developed by Laubscher is implemented to obtain an overall understanding of underground goaf stability, which is also the basis of performing further numerical analysis.
- **FLAC$^{3D}$ model**: Taking the geological model created by GoCAD as an input, the professional grid pre-processing tool Griddle is employed to establish a 3D numerical model. Griddle runs in the industrial modeling design software Rhino as a plug-in. By considering the in-situ stress state and rock mechanics in the mesh model, a numerical model for the stability analysis of the Yangla copper mine is obtained.
- **Calibrating mechanical parameters**: Rock-mass properties in the Hoek–Brown constitutive model are calibrated carefully according to the characteristics of rock-mass failure at the slope induced by the first phase of underground mining.
- **Stability analysis**: By neglecting filling goafs with cement material, the rock-mass stability in the second and third mining phases is simulated to obtain an overall understanding of rock-mass stability, especially with respect to potential physical failure characteristics, which is beneficial with respect to identifying an optimization scheme.
- **Optimizing the mining design**: By comparative analysis and engineering practices, the treatment measures are proposed, i.e., filling with cement material and structure parameter optimization for underground goafs, to improve rock-mass stability in the third mining phase. Multiple analyses are performed to determine the strength of the filling cement material and the size of the mine block at great depth.

Fig. 3. Content and workflow for the stability analysis.
It should be noted that, according to the characteristics of the first mining phase and the site failure at the slope, the goafs formed in the first mining phase have already been filled with cement material, which is considered in the model analysis.

2.1. Empirical stability analysis
The graph method proposed by Laubscher has been widely applied to assess underground stope stability [1-5], in which the rock-mass classification index MRMR (Modified Rock Mass Rating) and hydraulic radius of the stope are considered as input parameters. Compared with the original version of RMR (Rock Mass Rating) proposed by Bieniawski [6], in addition to the geological indexes of rock strength (i.e., RQD, joint density, joint condition, and groundwater), MRMR attempts to better summarize the natural conditions that affect stope stability by considering in-situ stress and joint orientation.

According to geological exploration results, the MRMRs of rock mass and hydraulic radius of underground goafs at the Linong mine approximately range in 40–50 and 7–12, respectively. Fig. 4 shows the Laubscher graph, from which it is evident that most goafs are within the stable zone, but some are within the transition zone. Moreover, some of the goafs within the transition zone have a failure risk in the process of mining, and underground goaf and slope stability must be further investigated by numerical model analysis.

![Laubscher empirical diagram for goaf stability evaluation.](image)

2.2. FLAC\(^3\)D model
A complicated FLAC\(^3\)D grid model for performing underground stope and slope stability analysis was built using the geologic modeling technology GoCAD and Griddle, which consist of slope, goafs within the mine orebody KT2 and KT5, as shown in Fig 5.
Fig. 5. FLAC\(^3\)D model for assessing rock-mass stability.

2.3. Rock-mass property estimation

Back analysis based on the field observed failure phenomenon at the slope shown in Fig. 2 induced by underground caving during the first mining phase was performed to obtain the rock-mass properties and ensure reasonable stability prediction for subsequent mining phases.

Fig. 6 shows the depth of orebodies and compares the Mohr–Coulomb and Hoek–Brown constitutive models, from which it is evident that the buried depth of orebodies KT2 and KT5 varies strongly in the slope of the Linong section. Taking KT2 for example, its buried depth ranges from almost 0 m near the slope to more than 800 m. Due to the significant variability of buried depth, the rock-mass strength properties exhibit prominent nonlinear behavior during the caving stage due to confinement effects. Nevertheless, conventional linear constitutive models, such as the Mohr–Coulomb model, cannot represent the nonlinearity of the strength induced by confinement variation. Accordingly, the Hoek–Brown constitutive model (with residual strength embedded in the FLAC\(^3\)D software) was used to simulate the rock-mass mechanical behavior [7-11].

Fig. 6. Determination of constitutive model for rock-mass behavior.

The calibrated rock-mass properties are listed in Table 1, where \(E\) and \(v\) denote Young’s modulus and Poisson’s ratio, respectively; \(GSI_{\text{peak}}, GSI_{\text{residual}}, m_i, D\), and uniaxial compressive strength \(\sigma_c\) obtained in laboratory test are the input parameters for the Hoek–Brown model. Where, \(GSI_{\text{peak}}, GSI_{\text{residual}}\) represents GSI (Geological Strength Index) corresponding to the peak and residual strength of rock mass respectively [12], \(m_i\) is the material constant, and \(D\) is the disturbance factor.

| Property                  | Marble | Sandstone | Ore  |
|---------------------------|--------|-----------|------|
| \(\rho\) (kg/m\(^3\))    | 2,710  | 2,740     | 3,500|
| \(E\)/GPa                | 1.29   | 5.33      | 6.31 |
| \(v\)                     | 0.26   | 0.25      | 0.25 |
| \(GSI_{\text{peak}}\)    | 43     | 48        | 48   |
| \(GSI_{\text{residual}}\)| 45     | 25        | 20   |
| \(m_i\)                   | 9      | 20        | 20   |
| \(D\)                     |        | 0.5       |      |

Table 1. Model properties for Yangla rock mass.
In addition to rock mass, cement materials for filling the goaf and the geological structural plane Fj13 are also involved in the model. The corresponding constitutive model and properties are outlined below.

- **Cement materials:** The Mohr–Coulomb constitutive model was chosen to simulate the mechanical behavior of cement material in the first mining stage. The material properties are consistent with the No. 1 material in Table 2, in which \( c \), \( \phi \), and \( \sigma_t \) denote cohesion strength, internal friction angle, and tensile strength, respectively. The No. 2 material is used for sensitivity analysis to determine the strength of the filling material in subsequent mining stages.

- **Structural plane:** The fault Fj13 composed of cataclastic rock and fragment rock crosses through the orebodies (KT2 and KT5) and the slope in the engineering area, which is simulated using an “interface” element and the Mohr–Coulomb slip model embedded in the FLAC3D software [13]. Since lab tests relating to Fj13 have not been conducted, the mechanical parameters are mainly determined by engineering practices, especially in China’s hydropower industry, as normal stiffness \( j_{kn} = 10 \) GPa, shear stiffness \( j_{ks} = 5 \) GPa, and strength parameters, cohesion strength \( j_{coh} = 0.1 \) MPa, friction strength \( j_{fric} = 30^\circ \), tension strength \( j_{ten} = 0 \) MPa, respectively.

| No. | \( \rho/\text{kg.m}^3 \) | \( E/\text{GPa} \) | \( v \) | \( c/\text{MPa} \) | \( \phi/{}^\circ \) | \( \sigma_t/\text{MPa} \) |
|-----|-----------------|-----------------|-----|-------------|-------|-----------------|
| 1   | 1,150           | 0.40            | 0.40| 0.008       | 15.0  | 0               |
| 2   | 1,850           | 2.61            | 0.16| 1.217       | 28.9  | 0.24            |

### 2.4. Model validation

The model was validated to verify that the obtained properties can reproduce the desired failure phenomenon shown in Fig. 2. Fig. 7 shows the model validation, from which it is evident that the predicted failure is consistent with the “1.19 landslide accident.”

![Fig. 7. Model validation of rock-mass properties.](image)
In slope and underground engineering practices, the potential failure mechanism of rock mass is complicated due to differing geological conditions. In general, the overall stability of rock mass is primarily dependent on rock-mass quality and in-situ stress. Nevertheless, the local stability characteristics are impacted by the development of the geological structure. Usually, the geological structure impacts the local stability characteristics in two ways: it transforms the local in-situ stress state and acts as a discontinuous deformation boundary. On the basis of the model validation results, it can be concluded that the deformation and failure of the slope during the first mining phase in the Linong section can be attributed to the comprehensive effects from in-situ stress, the rock-mass quality, and the local geological structure Fj13, the latter of which played a critical role.

However, the geological structure Fj13 was only speculated by examining a small number of outcrops, and its spatial-distribution range and morphology are uncertain due to limited site exploration data. Due to having insufficient exploration data to describe the slope structure, and since the aim of this study is to investigate the overall stability of slope and underground goafs, the subsequent stability analysis no longer simulates the geological structural surfaces and ignores the impacts on the local stability behavior of the rock mass; rather, it focuses on answering the overall stability properties of the rock masses.

3. Results and Discussion

3.1. Rock-mass failure identification

It is necessary to identify the potential physical failure of rock mass in stability analysis. However, compared with model analysis using the discrete element method [11], it is difficult to identify the potential failure using conventional mechanical indexes obtained by the equivalent continuum model used in FLAC$^{3D}$ due to the complexity of the failure mechanism of rock mass. In this study, a combination of conventional mechanical indexes, such as zone plastic state, zone stress–strength ratio, and velocity embedded in FLAC$^{3D}$, is proposed to identify the potential physical failure of rock mass from a conservative view.

Among these mechanical indexes, velocity is widely used to identify the potential failure of rock mass by investigating its trend of convergence, whereas assessing the plastic zone and stress–strength ratio is usually implemented as an auxiliary means for verification. According to numerical analysis results and engineering practices, this study proposes the evaluation criteria outlined below for identifying potential failure and assessing the stability of underground rock mass.

- **Unstable**: Plastic zones tend to coalesce, the velocity does not converge to zero, or the zone stress–strength ratio is <1.
- **Stable**: Plastic zones do not coalesce, the velocity converges to zero, and the zone stress–strength ratio is equal to or exceeds 1.

However, only the velocity response at the slope is employed to identify the potential instability of the slope. The velocity response during the caving process is monitored at a typical slope location, and the location is considered to be in the stable state when its velocity response converges to a minimum value. Overall, studies have shown that the mining of KT2 can lead to a more serious instability risk compared with KT5. Thus, the subsequent evaluation mainly focuses on the mining process of KT2.

3.2. Stability of goafs in KT2 in subsequent mining phases

To investigate the overall stability of goafs in the second and third mining phases, stability simulations were carried out assuming that a filling cement material was not used during the mining process. Figs.
8 and 9 show the roof and pillar stability, respectively; only the goafs formed in corresponding mining phases are shown in Fig. 8.

![Fig. 8](image_url)

**Fig. 8.** Roof stability at the end of the second and third mining phases.

On the basis of the abovementioned evaluation criterion, it can be seen that a considerable number of goaf roofs are potentially unstable at the end of the second and third mining phases. The pillars generally have good stability at the end of second mining phase, only locally instable zones with the indexes of stress–strength ratio less than 1.0 and velocity unable to be convergent to zero are observed in the model. Whereas a large-scale potential physical failure, which is underlined by the closed curve in pink, is noticed near the slope at the end of third phase of mining.

**Fig. 10** shows the velocity histories at the slope during mining, from which it is evident that the velocities converge to zero, which suggests that the slope is stable during the second mining phase. However, during the third mining phase, the majority of velocities exhibit strong divergent behavior, which means that the slope at these points is unstable. Obviously, this is the result of the abovementioned large-scale pillar collapse.

According to site investigation, no further failure phenomenon was observed at the slope during the second phase of mining. Therefore, the rock-mass stability obtained by simulation analysis is basically consistent with the field performance. However, it is difficult to verify the stability of underground goafs by field performance as they cannot be observed due to the mining method applied. On the basis of model results, measures should be introduced to improve rock-mass stability and meet safety requirements during the third phase of mining.
3.3. Determining the strength properties for the filling cement material

Based on a conservative perspective, all goafs formed in the second mining phase should be filled with cement material. The improvement effect on rock-mass stability by cement material Nos.1 and 2 in Table 2 was comparatively evaluated to provide a basis for selecting proper strength properties.

By filling goafs with cement material, we can significantly improve the rock-mass stability during the third mining phase, especially the large-scale instability of pillars in Fig. 9. Cement material No. 2 is recommended because material No.1 cannot ensure slope stability in some cases. This is because, for material No. 1, the velocity histories at several slope points do not converge to zero.
Due to the large number of mine blocks extracted in the third mining phase, the goaf roof and pillars are at risk of large deformation or stress-induced damage. To improve the rock-mass stability, the structural parameters of the goaf were optimized by performing sensitive model analysis on the goaf span.

**Fig. 12** shows the optimization analysis for the mining block size in the third mining phase, in which the stability of the engineering area with the deepest buried depth (extraction level 3,100–3,125 m) in KT2 under the three proposed span widths of 24, 30, and 36 m is comparatively investigated. The stress–strength ratio in the roofs and pillars increases with an decrease in the goaf span width. In particular, the number of instable roofs with a stress–strength ratio of <1 reduces from four (span width = 36 m) to one (span width = 24 m), which means the rock-mass stability is obviously improved. Therefore, the span width for goafs between an extraction level of 3,100–3,125 m should be reduced from 36 to 24 m.
4. Conclusions

In this paper, the Yangla copper mine was used as a case study to examine goaf and slope stability during the three phases of mining using a combination of geomechanic technologies, including geological modeling, the stability graph method, and numerical modeling.

Generally, the overall rock mass stability in slope or underground engineering practices is primarily dominated by rock mass quality and the in-situ stress condition. However, the local stability of rock mass could be affected by the geological structure at its outcrop. The rock mass properties can be calibrated to avoid failure during the first phase of mining. Based on the model validation, it was found that the geological structure surface Fj13 played a critical role in the previous onsite slope failure.

The goaf and slope stability during the second and third phases were simulated, the results of which indicated that the slope is stable during the second mining phase; however, it is not stable during the third phase, and certain measures should be introduced to with respect to goafs and optimizing the caving block size to improve rock-mass stability. During the third phase of mining, caving will destroy the structure of the rock-mass slope, resulting in large-scale slope instability. Therefore, the unfilled goafs should be filled with cement material No. 2 due to its optimal strength properties. To reduce the risk of roof falling due to subsequent deep mining, the span width should be reduced from 36 to 24 m for goafs between the extraction level 3,100–3,125 m with a maximum buried depth close to 800 m.

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