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Back analysis of inter-ramp slope failure in the Teghout copper mine

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ABSTRACT

Slope stability is one of the most important issues in an open pit mining design. The main purpose of any open pit mine design is to propose an optimal excavation configuration, considering safety, ore recovery and financial return. An accurate pit slope design which accounts for the mine geology, structural geology, rock mass properties, and hydrogeological models of the mine area, efficiently enhances an entire mining operation. The present paper investigates the failure of an inter-ramp slope in the Teghout mine using the limit equilibrium and finite difference approaches. The geotechnical properties of the rock mass were obtained using field investigations and back analysds. In-situ shear strength parameters of the rock mass were back calculated using the limit equilibrium method. Utilizing sensitivity and probabilistic analyses, the internal friction angle and rock mass cohesion values were obtained 33.5 degrees and 13.5 kPa, respectively. On the other hand, the slope failure mechanism and the effect of the slope height and angle on the stability of pit slope were investigated using the finite difference method, and a suitable slope angle was proposed.

Keywords : *Slope stability; Back analysis; Limit equilibrium; Finite difference; Open pit mining*

1. Introduction

Slope failure is one of the most prevalent natural disasters in the rock slope that can take place in any geological situations and slope geometries [1–4]. Slope instabilities can contribute to a major hazard for an open pit mining and may cause significant loss and casualties [5]. Slides usually occur because of the stress field redistribution [6]. Slope stability analysis of an open pit mine usually is carried out to design a stable slope in interaction with minimization of waste to ore ratio ($\frac{W}{O}$) [7]. Every soil or rock slope failure can be assumed as an in-situ shear test, which is performed naturally in a field scale. Back analysis is a calculation process to understand the failure mechanism and to gather the geotechnical characteristics and essential information for a mass failure. In fact, the results of a back analyzed rock mass failure are more reliable than the laboratory or in-situ tests which are influenced by the scale effect. Therefore, this approach is the most robust method to evaluate the geotechnical properties of failed materials. These parameters not only might be utilized for redesigning a failed slope, but also can be used for designation of new working/final pit slopes in the same geotechnical condition. A comprehensive understanding of an instability mechanism for precise back analysis is vital. In general, stability analyses of rock slopes are divided into two major classes: the first approaches can cope with jointed hard rock masses, and the second methods are desirable for heavily jointed and weak rocks such as porphyry rock masses [8]. The failure in the first class is more local and its mechanism is controlled by main structural defects. The main features of these kinds of failures are of planar, wedge and toppling failure mechanisms. In these mechanisms, the failure is governed by the orientation of discontinuities and the shear strength along the defects [9–12]. In this category, estimating the rock mass strength is not straight forward, and back analytical methods can be used as a tool to calculate the rock mass in-situ shear strength parameters of discontinuities along the sliding direction [13, 14]. Taking the complexity of this issue into account, the back analysis of a failed slope can be carried out using the limit equilibrium method (LEM). In fact, LEM is applicable for both of the above-mentioned rock mass classes. On the other hand, in order to utilize LEM for a rock slope in the second class, the discontinuum is assumed as a continuum rock mass [15]. Therefore, the shear strength parameters of the rock mass are back analyzed using the methods originally developed for analyzing the soil slope stability. Some methods are presented here for carrying out a back analysis with LEM [16]:

- Manual trial and error to match the input data with the observed behavior;
- Sensitivity analysis for individual variables;
- Probabilistic analysis for two correlated variables;
- Advanced probabilistic methods for simultaneous analysis of multiple parameters.

A complex distribution of stress, strain and modes of failure in depth can be investigated by numerical or physical modeling techniques [1–4, 17–21]. Advantages and disadvantages of each approach in slope stability analyses have been investigated thoroughly [22]. Back analyses of a failed slope can be considered as an in-situ mechanical test providing an accurate estimation of geotechnical parameters of a porphyry rock mass [2, 19]. On the other hand, some methods have been developed based on optimization and sensitivity analyses for probabilistic back-analysis of slope failures [23]. In addition, a brief overview on the methods of slope stability analysis and on their benefits has been provided in [24]. In porphyry open pit mines, when the behavior of the rock mass is not governed by faults and major joints, the rock mass could be treated as a continuum [7, 9, 13, 19]. The finite difference method (FDM) is a numerical method that is most suitable for problems in the continuum and equivalent continuum rock mass. It has been successfully used in slope stability problems [21, 25].

In this study, parallel studies using the LEM and FDM methods were carried out to investigate the failure of a porphyry rock slope. The geological and geotechnical characteristics of the study area was first investigated. The rock mass geotechnical characteristics of failure were

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collected based on the ISRM suggested method [26]. Based on the RMR and GSI systems, the rock mass was classified and its geomechanical properties were estimated. Accordingly, after conducting LEM-based single-variable and multi-variable back analyses, the shear strength parameters of the rock mass were evaluated. Furthermore, the slope failure mechanism was studied using the FDM, and afterward, slope stability design charts were recommended.

2. Case study

The Teghout mine is a copper and molybdenum open-pit mine in Lori province of Armenia. The location of the Teghout mine is illustrated in Fig. 1.

Fig. 1. Location of the Teghout copper mine

During the mining operation, five benches in the central part of the mine started to fail. The main ramp of the mine has been planned to pass through this area. Therefore, any failure is not acceptable in this part of the mine during the future activities. Therefore, a vast rock mechanics investigation including joint survey, rock mass description, and geotechnical sampling were carried out in the mine area. It is worth mentioning that due to the topography of the study area, the failure domain is completely dry and the groundwater plays no role in the instability.

2.1. Geology of the study area

The Teghout copper mine is a porphyry deposit. The main lithologies of the mine are calcalkaline to alkaline igneous rocks, which are presented as follows:

Volcanic and sub-volcanic rocks: These rocks include porphyritic dacite and tuff units.

Tonalite and quartz diorite: These units are of felsic composition with a phaneritic texture. Field investigations indicate that the alteration processes have extremely affected and weakened these rocks. Actually, they are the weakest units in the mine area in which the failed slope is located.

Porphyritic Quartz dacite: This unit usually appears in the form of dikes, and is a moderately to strongly fractured rock with a low alteration. The degree of alteration increases in the margin of this rock.

2.2. Geotechnical characteristics

In order to investigate the geotechnical properties of the rock mass and discontinuities, rock sampling (in outcrops and boreholes) and discontinuity mapping were conducted using two methods of scanline and window mapping according to the ISRM suggested method [26,27].

In order to investigate the characteristics of the discontinuities in the failure area, linear scanning of rock mass discontinuities was performed (Fig. 2). Then, 70 structural discontinuities were plotted in the Dips software and were analyzed statistically (Fig. 3). The analysis of discontinuities data revealed that the rock mass had two high-dip joint sets, two moderate-dip joint sets, and one low-dip joint set. The geometrical characteristics of the discontinuities of the failure area are presented in Table 1.

Fig. 2. A scanline station in the failure area

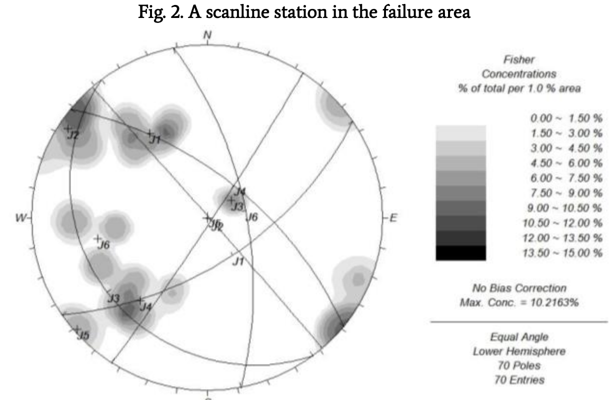


Fig. 3: Main joint sets of the failure area

Table 1. Geometrical characteristics of joint sets in the failure area.

Discontinuity type	Dip (degree)	Dip Direction (degree)
Joint set 1	48	119
Joint set 2	72	109
Joint set 3	11	196
Joint set 4	46	16
Joint set 5	83	39
Joint set 6	47	61

On the other hand, based on engineering geology mapping, the rock mass was classified as a highly jointed and altered rock mass. Besides, rosette diagram of discontinuities has shown that the joint sets are mainly oriented in three directions and the major direction of discontinuities is NE-SW.

Detailed information of rock mass scanline, including persistence, spacing, aperture, joint surface weathering, and joint surface roughness data are given in Fig. 4.

The persistency of rock joints varies from less than 1 m to 20 m, and those of more than 50% of the joints fall into the interval of 1-3 m (Fig. 4a). The joint set spacing differs from 20 mm to 60 mm, and more than 40% of joint spacing values belong to the interval of 20-60 mm (Fig. 4b). Furthermore, the variation of joint set apertures is from 0.1 mm to rarely more than 10 mm, and more than 35% of the discontinuities have an aperture between 0.5-2.5 mm (Fig. 4c). Finally, the joint surface weathering and the roughness of joint sets vary from moderately to completely weathered, and from rough to slickenside, respectively. More than half of discontinuity surfaces are highly weathered and smoothed (Figs. 4d and e).

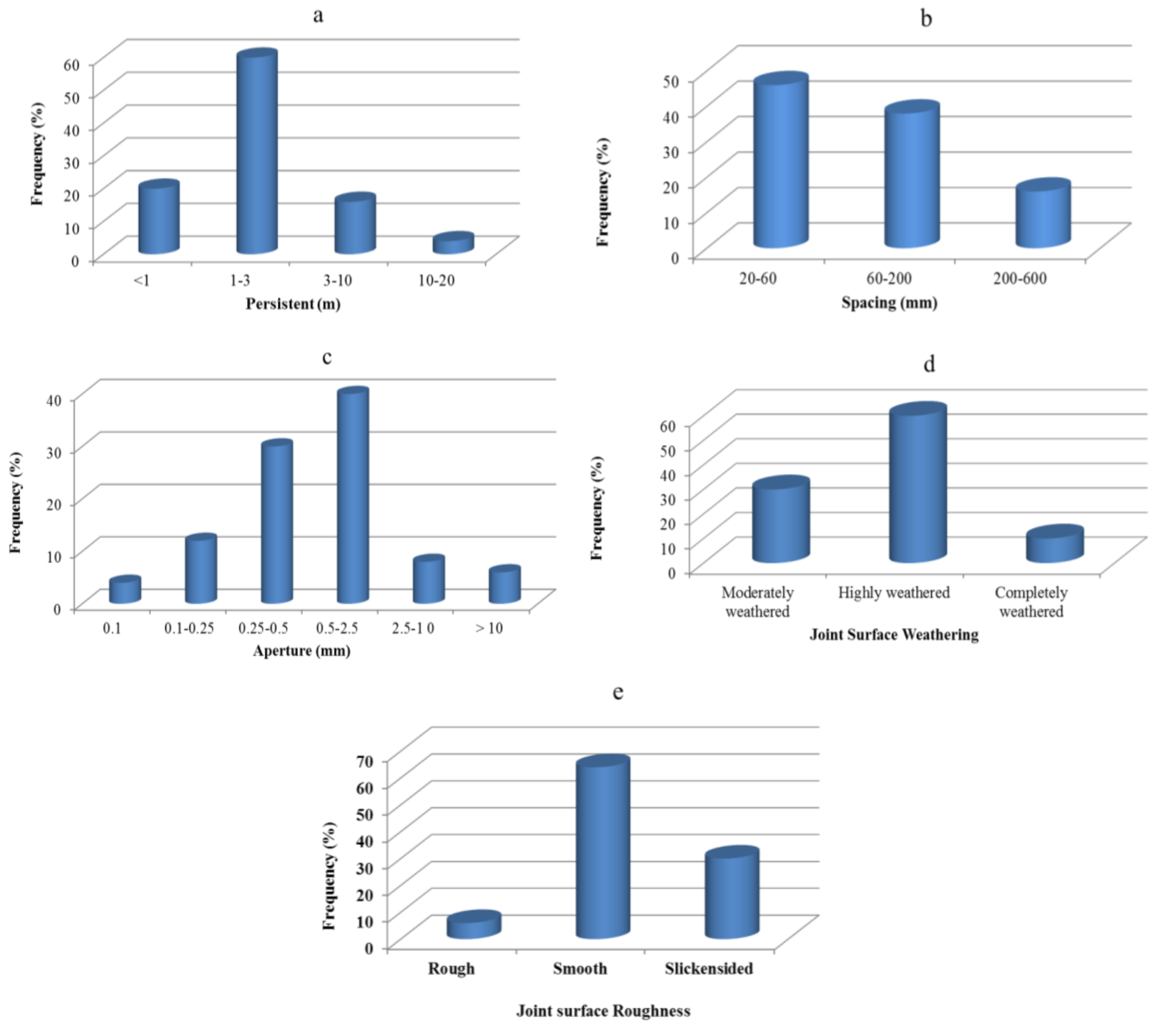


Fig. 4. Bar chart of discontinuity properties, a) Persistent, b) Spacing, c) aperture, d) joint surface weathering, e) Joint surface roughness

Moreover, geotechnical sampling was performed on three boreholes in rock units of the failure area, and laboratory tests such as unconfined compression and Brazilian tests were carried out on twenty samples, according to [28,29]. The results are presented in Table 2.

Table 2 Results of laboratory tests on rock samples

Rock Type	σ_t (MPa)	UCS (MPa)	E (GPa)	ν^*
Oxide Zone	1.7	14.4	6.5	0.25

*Obtained from GSI classification

According to the field investigations and discontinuity analyses, the rock mass was classified in a 'poor rock' category based on RMR and GSI systems (Table 3).

Table 3. Geotechnical Parameters of the oxide zone using rock mass classification systems.

Rock mass classification system	Score	Description	C (kPa)	φ(°)
RMR	30-35	Poor	100-200	15-25
GSI	30-40	Poor	290	35

In order to evaluate the deformation modulus of the rock mass, empirical equations, available in the literature, were used (Table 4). In this way, the average rock mass deformation modulus was estimated as 1.54 GPa.

3. LEM back analysis

In this study, the LEM and FDM methods were used to back analyze the rock mass properties and to investigate the slope failure mechanism. Considering the poor quality of the rock mass that have multiple joint sets with a low spacing, the slope failure could be analyzed using continuum techniques. In the first step, the shear strength parameters of the rock mass were back calculated using the limit equilibrium method of slices, and sensitivity/probabilistic analyses were conducted. Then, incorporating the back analyzed parameters of the rock mass, the slope failure mechanism was numerically studied through an FDM modeling.

As a slope failure happens, the analysis is usually conducted to determine the cause of failure. Having a failure surface, back analysis can evaluate the shear strength of the material, pore pressure or other rock mass parameters. The feedbacks of slope stability analyses can be used for remedial measures or redesigning the plan of slopes. In the Teghout copper mine, a slope failure occurred in a highly jointed and the altered oxide zone. The location of this failure and its photograph are presented in Fig. 5.

Table 4 Evaluation of deformation modulus of the rock mass

Empirical equations	Deformation modulus (GPa)	Reference
$E_m = 10^{(RMR-10)/40}$	3.16 - 4.22	[30]
$E_m = 10^{(RMR-20)/38}$	1.83 – 2.48	[31]
$E_m = 0.1(RMR/10)^3$	2.70 – 4.29	[32]
$E_m = 0.01E_i(0.0028RMR^2 + 0.9e^{\frac{RMR}{22.83}})$	0.38 – 0.49	[33]
$E_m = E_i[0.5(1-(cos(\pi RMR/100)))]$	1.34 -1.77	[34]
$E_m = (1-0.5D)\sqrt{\frac{\sigma_{ci}}{100}}10^{(\frac{GSI-10}{40})}, \sigma_{ci} \le 100\ MPa$	0.6 – 1.07	[35]
$E_m = E_i(S)^{0.25}, s = exp(\frac{GSI-100}{9-3D})$	0.35 – 0.53	[36]
$E_m = E_i(S^a)^{0.4}, s = exp\ exp\left(\frac{GSI-100}{9-3D}\right), a = 0.5 + \frac{1}{6}(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}})$	0.57 – 0.84	[37]
$E_m = E_i(0.02 + \frac{1-0.5D}{1+e^{((60+15D-GSI)/11)}})$	0.15 – 0.26	[36]
$E_m = \sqrt{\frac{\sigma_{ci}}{100}}10^{(\frac{GSI-10}{40})}, \sigma_{ci} \le 100\ MPa$	0.18 – 2.13	[38]
$E_m = tan(\sqrt{1.56 + (ln\ ln\ (GSI))^2})\sqrt[3]{\sigma_{ci}}$	1.27 – 2.28	[39]
Average	1.24 - 1.85	

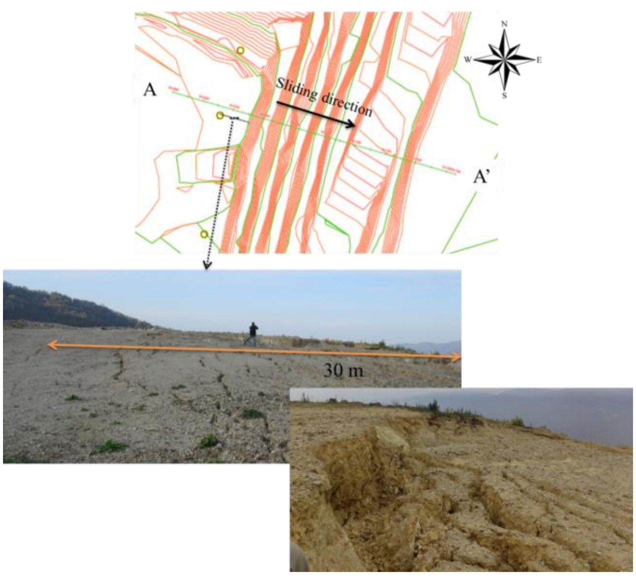


Fig. 5. Plan view of the slope failure location and tension cracks due to the failure (view to the North).

As it is illustrated in Fig. 5, instability has caused a set of tension cracks behind the pit wall. According to extensive field investigations, the approximate final limit of tension cracks and the failure boundary behind the slope was detected 30 m from the slope crest. In order to evaluate the in-situ shear strength parameters of the rock mass, i.e. cohesion and friction angle, the slope failure was back analyzed using the limit equilibrium software Slide. In this paper, the failure was analyzed with both sensitivity and probabilistic analytical methods [16]. According to the rock mass classification, the shear strength parameters of the rock mass that were used in the model for the deterministic and probabilistic analyses are presented in Table 5.

The model was run with the Bishop simplified method. The potential failure surfaces, with a safety factor from 0.99 to 1.01, for the mean values of the cohesion and friction angle are presented in Fig. 6.

The length of the tension cracked area was obtained about 19 m for the assigned mean values of shear strength parameters that it is 10 meter less than the real length on the ground.

3.1. Sensitivity analysis

Sensitivity analysis helps researchers to evaluate the impact of an individual unknown variable, assuming that all other slope parameters are known. In this analysis, one parameter varies and other input parameters are kept constant in their mean values. A sensitivity analysis indicates which input parameter may be critical to the assessment of the slope stability and which parameter has a smaller effect on the instability.

Performing a sensitivity analysis, the cohesion and friction angle of the failure surface were back analyzed. The results are presented in the form of sensitivity graphs in Figs. 7 and 8 in which the vertical axis represents the factor of safety and the horizontal axis represents the cohesion or friction angle. The result indicated that at the verge of failure, i.e. a factor of safety of 1, the cohesion and friction angle values were 13.39 kPa and 33.39°, respectively.

Table 5. Rock mass parameters used for back analyses.

Parameter	Minimum	Maximum	Mean	Distribution Function	Unit Weight (kN/m^3)
Cohesion (kPa)	6.6	15	13.5	Normal	22
Friction Angle (Degree)	29	35	33.5	Normal	

3.2. Probabilistic analysis

In this approach, it is assumed that both the cohesion and the friction angle of the rock mass are unknown. Therefore, a probabilistic analysis can be used to determine the relationship between the pairs of cohesion and friction angle which yielded a factor of safety of 1 for a particular failure surface. Therefore, both the cohesion and the friction angle were defined as random variables with a normal distribution. The statistical characteristics of the variables are presented in Table 5. In Fig. 9, a scatter plot of the all generated cohesion and friction angle values is presented. Fig. 10 shows the fitted relationship between the cohesion and friction angle values of the rock mass which bring the slope to the verge of instability. Any point on this line represents a pair of (c,) values which results in a factor of safety of approximately 1 for the given slope geometry. The results of the probabilistic analysis show that in the failure condition, the cohesion and friction angle values are in the range of 13.1 to 13.7 kPa and 33.3° to 33.55°, respectively.

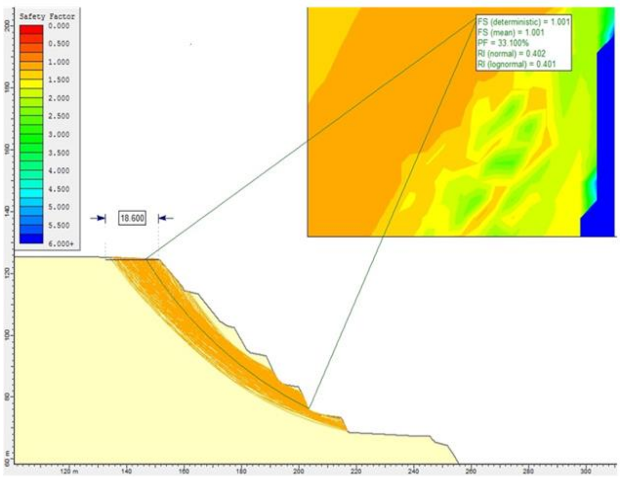


Fig. 6. The potential failure surfaces with safety factor from 0.99 to 1.01 analyzed by Bishop simplified method.

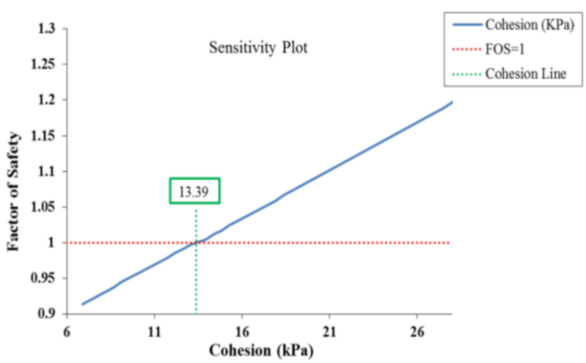


Fig. 7. Sensitivity graph for cohesion

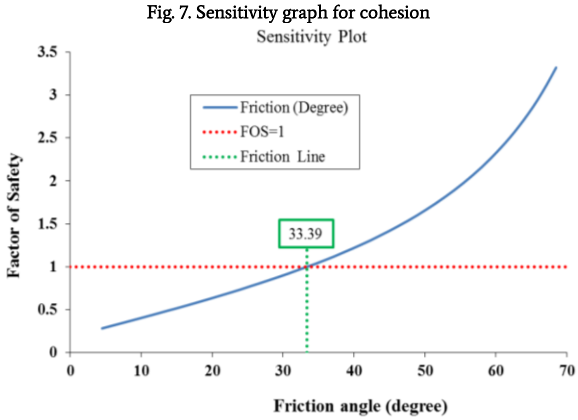


Fig. 8. Sensitivity graph for friction angle

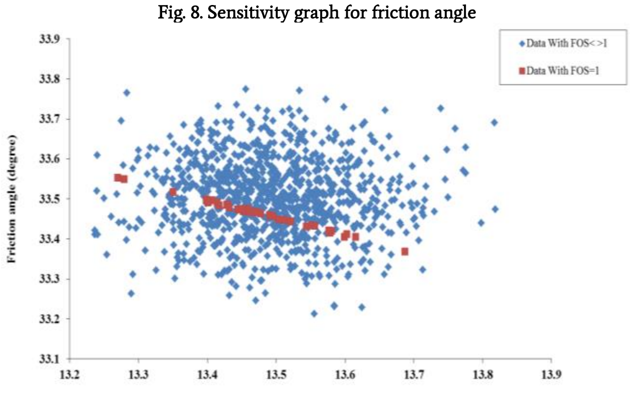


Fig. 9. Scattered plot for cohesion and friction angle values (data points corresponding to an approximate factor of safety of 1 are in red)

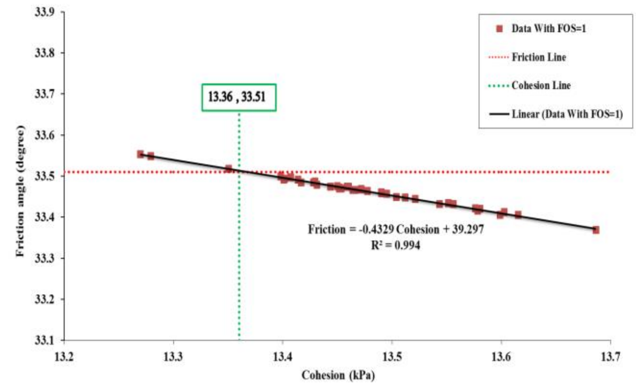


Fig. 10. The relationship between cohesion and friction angle of the rock mass for an approximate factor of safety of 1

4. Numerical modeling

In this section, the FDM method was used to provide a better understanding of the rock mass behavior, the slope failure mechanism and to propose some graphs for a stable slope design. The slope was modeled by finite difference code FLAC^{2D} which uses a two-dimensional explicit solution, allowing simulation of large deformations and instabilities [18]. The behavior of soil, rock or other materials that may experience a plastic flow, when their limits are reached, can be simulated by this software. Geotechnical materials are modelled by elements, or zones, which create a grid that can be modified by a user to fit the conceptual model of the slope. The behavior of an element depends on the adjusted linear or nonlinear stress/strain relation in response to the applied force or the boundary condition [40]. In this study, the Mohr-Coulomb constitutive model was assigned to the rock mass. The mesh generation and boundary conditions of the slope model are illustrated in Fig. 11.

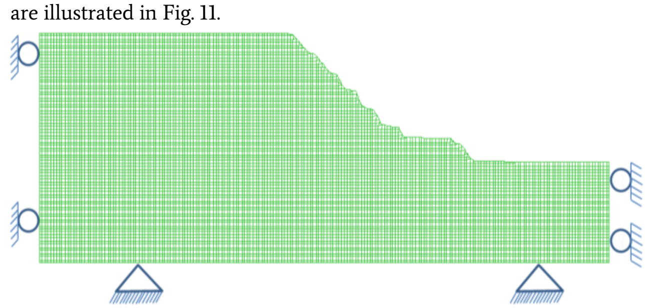


Fig. 11. The mesh generation and boundary conditions of FDM model of failed slope

The values of the geotechnical parameters of the rock slope, i.e. cohesion and friction angle, which were obtained from the limit equilibrium method, were assigned to the model. Geomechanical parameters of the rock mass used in the FDM modeling are given in Table 6.

The calculated factor of safety was equal to one. Fig. 12 illustrates the slip surface and the maximum shear strain-rate variation.

In Fig. 13, plasticity indices indicate that this slope model is failed in shear, while tension cracks are observed on the top of the slope. The FDM model completely complies with the failed slope. The expansion of the tension cracked area behind the slope crest was obtained about 30 m that it was equal to field measurements.

Finally, the graphs for slope design in the failure area were developed (Fig. 14). To do this, a set of FLAC^{2D} models with different slope angles between 20° to 75° in two-degree increments and slope height varying from 10 to 120 meters with 20-m increments were analyzed. As can be observed, significant information was obtained for designing a stable rock slope in the highly jointed and highly altered oxide zone for the future mine planning.

Table 6 Parameters used in FDM model

Parameters	Cohesion (kPa)	Friction angle (°)	E (GPa)	K* (GPa)	G* (GPa)	Unit Weight (kN/m³)
Oxide zone	13.5	33.5	1.54	1.03	0.62	22

* $K=\frac{E}{3(1-2v)}$, $G=\frac{E}{2(1+v)}$

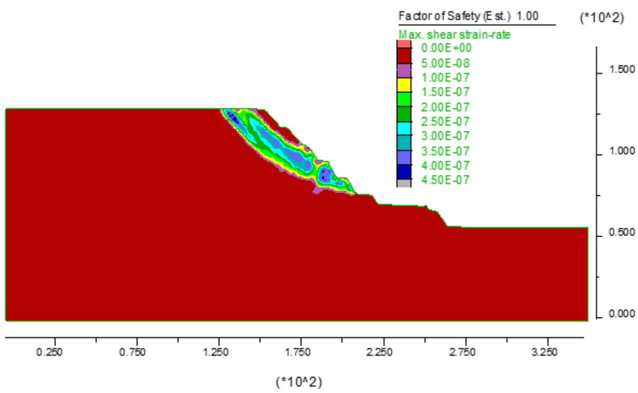
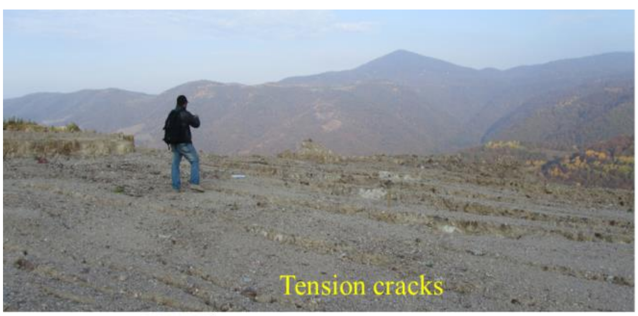


Fig. 12. Maximum shear strain rate in a failure condition analyzed by the FDM (dimensions in meter)



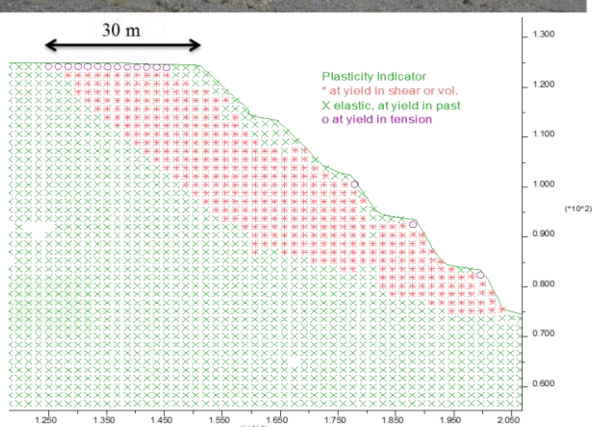


Fig. 13. Yielded zones under tension and shear stress obtained from FDM

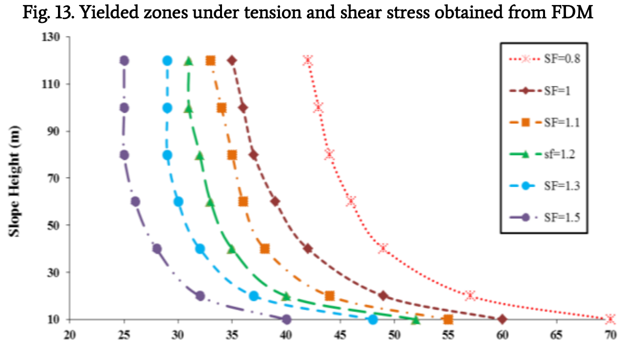


Fig. 14. Graphs for slope design in the failure area

5. Discussion

According to site investigations, it is completely clear that the rock mass is not only highly jointed, but it is also highly altered. Hence, the quality of the rock mass is considered as 'poor'. On the other hand, the occurrence of a circular slope failure suggests that the rock mass behavior is not mainly controlled by discontinuities, and the rock mass can be assumed as an equivalent continuum. A comparison between the values of shear strength parameters of the rock mass obtained from the three different methods, i.e. RMR, GSI and back analysis, are illustrated in Figs. 15 and 16.

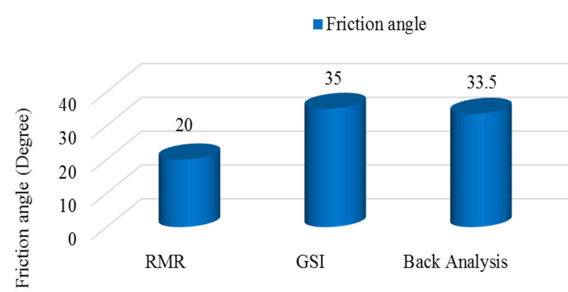


Fig. 15. Rock mass friction angles obtained from empirical methods and back analyses

As it is depicted in Fig. 15, the friction angle proposed by the RMR and GSI methods are the lowest and highest values, respectively, and the friction angle obtained from the back analysis is between these two empirical results. On the other hand, the rock mass cohesion suggested by the back analysis is far lower than those obtained from the empirical methods (Fig. 16). In fact, the GSI and RMR overestimated the cohesion of the porphyry rock mass.

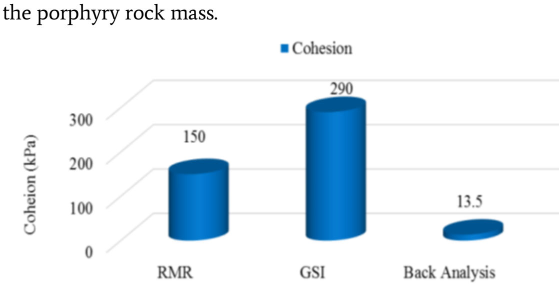


Fig. 16. Rock mass cohesion obtained from empirical methods and back analyses

However, it should be considered that LEM cannot determine the slope failure mechanism and movements. Therefore, the length of the tension cracked area calculated by LEM, i.e. 18 m, is smaller than the real value, i.e. 30 m. On the contrary, the results of slope analysis using the FDM shows an excellent agreement with the real failure geometry.

Furthermore, the numerical simulation shows that in this case, a bench slope with a height of 10 m and an overall slope of 100 m high would be a factor of safety equal to 1.3, provided that the slope angles are 47° and 31°, respectively.

Finally, it is necessary to discuss on restrictions and the fundamental hypothesis of the introduced back analysis. A comprehensive discussion of challenges that exist with the application of back analyses in engineering problems was presented by [41]. The main issue for this method is the quality of data, and hence, the application of back analysis should be considered with caution when no data is available. In addition, it is important to note that the 3D modeling of a slope may have a principle effect on the estimated shear strength parameters of a rock mass.

6. Conclusions

In this paper, a combination of LEM and FDM methods was proposed as an approach for back analyzing a failed slope in weak rocks. A vast joint study program was first carried out in the slope area, revealing that the rock mass is highly jointed and altered. Consequently, the rock mass was assumed as a continuum. Based on field investigations and the available empirical relationships, the deformation modulus of the rock mass was suggested to be about 1.54 GPa. The sensitivity and probabilistic back analyses of the rock mass failure were performed employing the LEM method. The result of the sensitivity analysis indicated that in a failure condition, the cohesion and friction angle values are equal to 13.39 kPa and 33.39°, respectively. On the other hand, the results of the probabilistic analysis showed that in a failure condition, the cohesion and friction angle values are in the range of 13.1-13.7 kPa and 33.3°-33.55°, respectively. The results of both analyses are in an excellent agreement. In the next step, the failed slope was modeled using the FDM method incorporating the back calculated values of the shear strength parameters. The results demonstrated that the shear mechanism governed the failure. However, compared to the LEM model, the calibrated FDM model showed a better agreement with the field observation of failure geometry. Finally, a set of graphs was proposed for slope designing in the highly jointed oxide zone of the Teghout copper mine. The findings showed that the proposed approach is a robust method to investigate the slope stability and to propose suitable stabilization measures.

Acknowledgments

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