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Combination of Monte Carlo Simulation and Bishop Technique for the Slope Stability Analysis of the Gol-E-Gohar Iron Open Pit Mine

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Abstract

This study introduces a novel approach, known as Hybrid Probabilistic Slope Stability Analysis (HPSSA), tailored for Mine 4 of the Gol-E-Gohar iron complex in Iran. The mine walls are first divided into 8 separate structural zones, including A-A' to H-H' sections for slope stability analysis. Then, sufficient core specimens are prepared from 22 drilled boreholes and the required parameters for slope design, including cohesion (c), friction angle (φ), and unit weight (γ), are measured. Finally, the HPSSA approach is performed through the combination of Monte Carlo simulation (MCS), Mohr-Coulomb criterion and Bishop's technique. According to the HPSSA results, the normal distribution function is achieved as the best curve fit for c, φ and γ parameters. Also, the obtained values of mean probabilistic safety factor (SF) for defined structural zones vary from 0.93 to 1.86, with the probability of failure (PF) of 0 to 75.6%. Moreover, SF values varied from 0.68 to 1.22 (mean value of 0.93) with a PF of 75% for the A-A' section and from 0.65 to 1.24 (mean value of 0.97) with a PF of 60% for the H-H' section. Hence, it is concluded that the A-A' section and mine's north wall are more prone to instability with PF>60%. On the other hand, SF>1.2 and PF<5% for other mine walls (sections B-B'-G-G') prove that they are highly unlikely to be unstable. Displacement monitoring of the pit walls using installed prisms confirmed that average displacements in structural zones have a similar trend with SF values of the HPSSA. The results show a good agreement between the trend of probabilistic SFs and monitored slope displacements. Lastly, comparative analysis confirmed the validity of the suggested HPSSA approach with relatively higher accuracy than most previous slope stability analysis methods.

1. Introduction

Designing the possible highest slope angle in open pit mines leads to a lower stripping ratio. On the other hand, a high rock slope angle may lead to wall failure and the fall of large volumes of waste rocks into the pit. This can cause equipment damage and loss of mine production over a certain time period. Therefore, determining the optimum slope angle in open pit mining involves critical parameters affecting the mining economy and productivity [1–11]. Usually, the slope stability of an open pit mine is evaluated using the safety factor (SF) index, which is calculated using deterministic methods during the slope stability analysis. Stability analysis of rock slopes is performed based on various approaches such as empirical, theoretical, numerical, artificial intelligence and probabilistic methods [12–28]. Among these approaches, the limit equilibrium method (LEM) has been frequently used for stability analysis of rock slopes due to its accuracy and easy utilization [29–34]. In this technique, SF is defined as the ratio of resistance loads to driving forces on a possible sliding face. In the LEM method, the sliding rocks are considered to be rigid [35–39]. Despite the inherent uncertainties of rock mass properties, uncertainties of geometrical and mechanical properties are commonly considered the invariants in deterministic analysis. Theoretically, a rock

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slope is stable if the computed SF is greater than 1. According to rock anisotropy and heterogeneity caused by the existing discontinuity plans, crystallization, etc., the stability of rock slope is significantly influenced by uncertainties. A deterministic slope simulation with a safety factor ≥ 1 can't provide permanent slope safety. Consequently, a reliable and safe slope design must consider the above-mentioned uncertainties. Hence, probabilistic approaches can be used along with the deterministic methods for slope stability analysis of pit walls, reducing the risk and uncertainties.

1.1. Probabilistic stability analysis

Probabilistic stability analysis of rock slopes is essential and crucial for designing open-pit mines. In situ and experimental tests can help identify the most representative shear strength variables of rock and their statistical characteristics for probability analyses [35]. In the last few decades, uncertainties of geotechnical characteristics have attracted more attention, and several researches have been conducted in the field of open-pit mine slope evaluation based on probability analysis [40]. In practice, many of these probabilistic approaches are utilized to analyze slope stability. These approaches can be categorized into five major groups: estimated methods, MCS, analytical approaches, numerical models, and intelligent techniques. The MCS method is commonly used to study the influence of uncertainties on slope stability. In fact, this method is a useful tool for uncertainty analysis, in which many possible samples can be produced and analyzed [41–43].

1.2. Uncertainties in slope stability analysis

During the geotechnical engineering design and analysis, various types of inherent uncertainties are incorporated due to insufficient data about the issue under study. Geotechnical projects have different types of uncertainties, including local, modeling and data uncertainties. Generally, the source of uncertainties in slope stability analysis is categorized into three types as follows [16, 35, 40]:

1- The strength properties of materials, including the resistance properties of rock such as c and φ, which have a high dispersion, directly affect the accuracy of slope stability analysis. The intact rock properties involve less uncertainty due to the lack of joints and cracks. Conversely, the length and geometry of joints and cracks, and their frictional resistance are very important factors that create uncertainty in rock masses.

2- Slope geometry, bedding characteristics, faults and boundary conditions are other factors that can affect the uncertainties.

3- Pore water pressure distribution is another source of uncertainty in the slope stability analysis.

1.3. Study aim

Mine 4 of the Gol-E-Gohar complex iron deposit is considered a case study in this research. The north wall of this mine consists of different rock mass units, which are tectonized in some parts. Thus, there are large variations of geological and geo-mechanical properties in the mine's north wall. Due to the existence of high tectonic zones on the north wall of the mine, slope failure has occurred in the form of circular failure. Therefore, the variability in material properties of the pit wall makes it impossible to use deterministic methods for slope stability analysis. Using a deterministic approach for this situation based on average rock mass properties will result in a slope design with a low probability of accuracy, leading to pit wall failure and significant economic repercussions on the mine. On the other hand, the overall slope angle significantly impacts the economic design of mine 4, considering the calculated strip ratio. Accordingly, a probabilistic approach was adopted in this study for the slope stability analysis to design the slope in a safe and economical manner. The pit walls stability of mine 4 of the Gol-E-Gohar iron complex is probabilistically analyzed based on the new HPSSA method incorporating MCS and LEM approaches.

2. Methodology

2.1. LEM

LEM is frequently utilized to statically model ambiguous issues requiring certain assumptions about inter-slice shear forces to be statically represented. In slope stability using the LEM, the sliding environment is discretized into different columns with perpendicular interfaces. The definite slice numbers depend on the slope geometrical characteristics and rock outline. Some LEM techniques assume a circular sliding surface, while others assume an unspecified non-circular sliding surface. The LEM techniques, which are based on a circular sliding surface, consider the equilibrium of forces around the center of the circle for the entire body made up of all slices. Conversely, the LEM techniques, which are based on an unspecified shape for the sliding surface, typically consider the equilibrium condition based on individual slices [28, 37, 38]. This study examines slope stability based on the simplified Bishop method using the slide software.

2.2. PSSA

In probabilistic slope stability analysis (PSSA), the estimation of the probability of failure is directly based on the probability density function of the variables and involves multiple integration over the entire failure range. Due to the complexity of the final probability function and the challenges of multiple integrations, determining the exact probability of failure is often difficult and typically requires numerical approximations. However, simulation methods make slip probability calculations easier by leveraging the computational power of modern computers.

The ultimate objective of probabilistic methods is to ascertain the statistical characteristics of the shear strength parameters of rock masses. Probabilistic methods such as MCS, point estimation, and first-order second moment (FOSM) offer a more straightforward interpretation and can be readily applied by users. The MCS method is utilized in this study and will be detailed in the subsequent section.

2.2.1. MCS method

The MCS method is a strong and practical technique to conduct the probabilistic analysis, which can be applied to solving various problems in rock mechanics. Particularly in the slope stability analysis, different components of rock shear strength variables are considered through a distribution scheme, and SF is calculated by using the desirable limit equilibrium methods. Many simulations are needed to obtain the desired output with acceptable accuracy. Ultimately, the probability of failure (PF) is calculated through the ratio of the number of simulations converged with a SF value lower than 1 to the whole number of simulations. The number of simulations depends on the uncertainties of input variables and the desired level of assurance, which constantly increases in the geometrical sequences [1, 35, 40]. Generally, the procedures of this probabilistic method can be summarized as follows:

- Selecting the appropriate deterministic analysis solution method.
- Selection of input parameters for probabilistic analysis and quantification of their variations.
- Random sampling is conducted for each selected parameter from the probability density function or data column related to that parameter.
- Solving the problem through the deterministic analysis method with a set of selected parameters to calculate the performance function.
- The operation continues by repeating the last two steps until a sufficient number of simulations is reached. Using the output values, the distribution of the performance function is defined. At the end of this step, the probability of failure is ultimately determined.

This method provides a solution to investigate deterministic analytical problems using the random sampling method. In this method, a value is randomly selected from each possible domain of the random variable, and after substitution in the target equation, its output value is obtained. By repeating this process, a large number of target function values are obtained, and finally, the target distribution function is acquired. The results of calculations depend on the number of steps. Also, it depends on the number of parameters in the calculations, in which the number of steps to reach a reasonable solution increases with increasing the number of parameters. Usually, a reliable answer can be reached with 1000 iterations. Generating random numbers is also a complicated task, as the numbers must follow a particular distribution. In addition, sometimes the parameters that are generated depend on each other; however, this dependency is not visible when these values are generated using random numbers. For example, C and φ parameters depend on each other in the slope stability analysis, so the dependency of these two parameters can't be ignored, and this correlation should be considered during the generating values. For this purpose, a correlation coefficient is defined between two parameters, which usually change from +1 to -1.

Before entering the data, the minimum number of simulation iterations should be determined. It is obtained based on the number of primary variables and the desired level of confidence according to the following equation [7]:

$$n = \left(\frac{100d}{E}\right)^2 \times \frac{1 - P_f}{P_f} \times m \tag{1}$$

where n is the minimum number of simulation iterations, P_f is the probability of unfavorable performance, E is the comparative percent of error in approximating the probability of P_f, m is the number of accidental parameters, and d is the standard normal deviation which is obtained based on the confidence levels shown in Table 1.

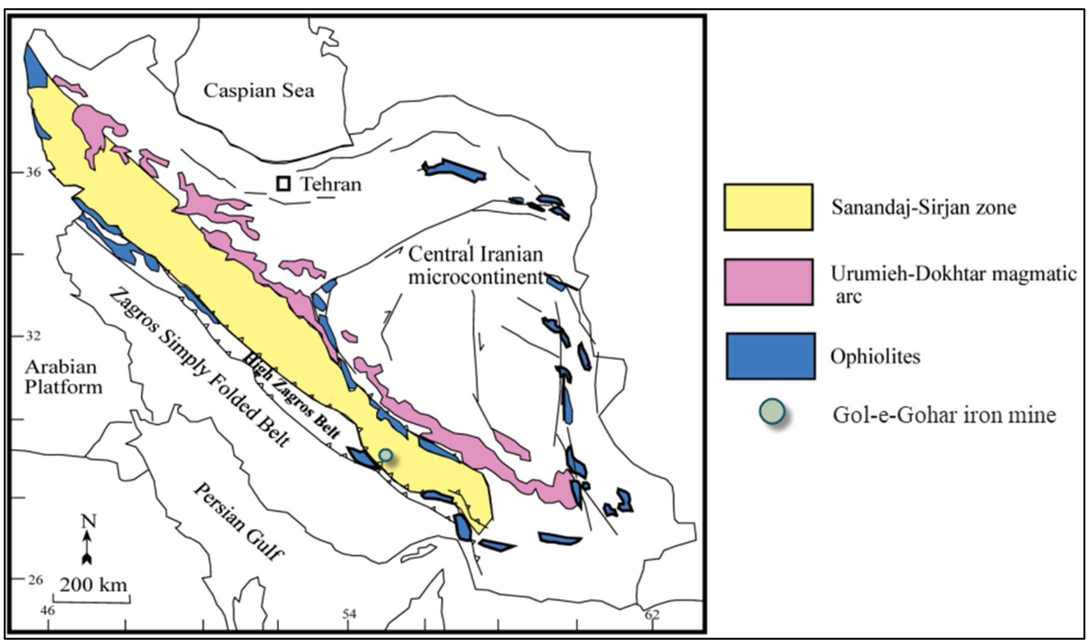
3. Case study

Mine 4 of the Gol-E-Gohar iron complex is considered a case study in this research. The Gol-E-Gohar iron complex includes six main mine areas, which are situated about 60 km southwest of Sirjan town, in the Kerman province of Iran, on the eastern border of the Sanandaj-Sirjan structural zone (SSSZ). This structural zone, along with the Gol-E-Gohar iron complex location, is demonstrated in Figure 1a. The SSSZ is one of Iran's main metallogenic belts. The Gol-E-Gohar iron ore complex contains six main ore bodies, named Mine 1 to Mine 6. This iron complex is one of the most important economic iron resources in Iran, with a proven ore of about 900 million tons of iron ore (Figure 1b). This mining area mainly consists of metamorphic rocks (including mica schist, gneiss, quartz schist and amphibolite in the lower parts, and quartz schist gneiss and in the upper parts), Ker white complex (mainly consisting of schist and marble), Mesozoic and Cenozoic sedimentary rocks (comprising conglomerate, reef limestone and shale) and Quaternary alluviums. The geological section of the case study (Mine 4) is shown in Figure 2. The indicated reserve of Mine 4 was estimated at about 100 million tons [43]. The final pit of Mine 4 is designed with a strip ratio of 8:1, which is represented in Figure 3.

Considering the uncertainties in the slope stability analysis and the significant influence of the mine's final slope angle on mining productivity and economics, 22 geotechnical boreholes were drilled in the mine area (Figure 4) to specify the rock mass characteristics and apply them to mine design and planning. From these drilled boreholes, sufficient cores were collected, and the necessary field information was recorded.

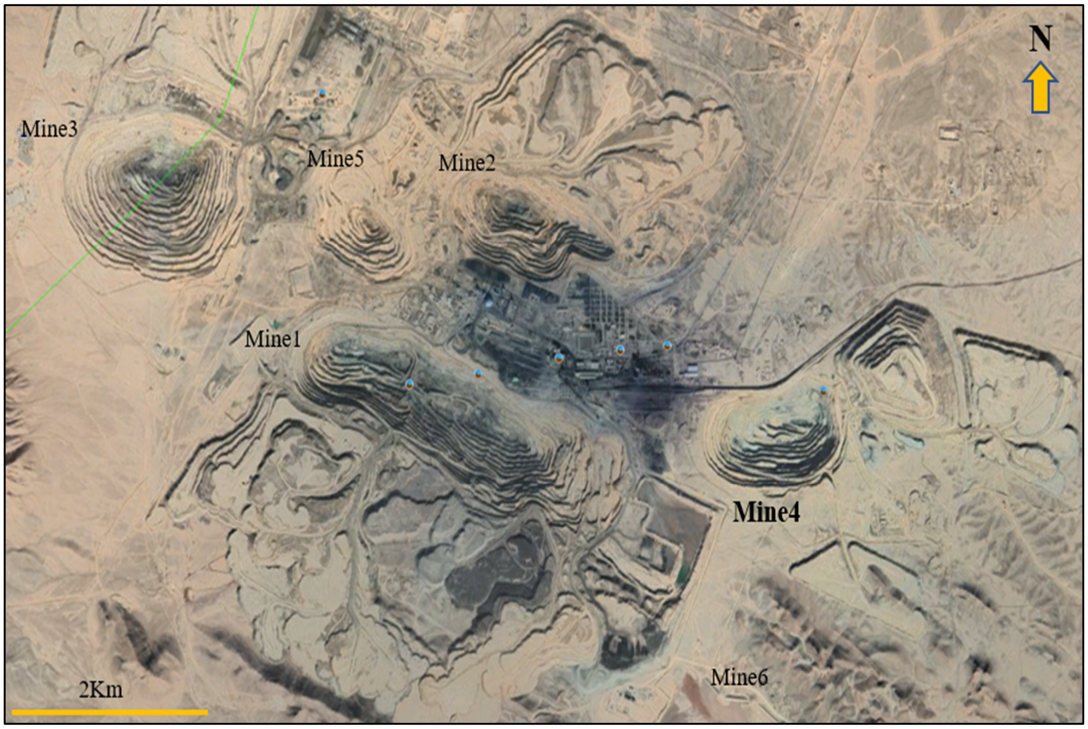
Table 1. Instruction of determining the standard normal deviation.

Standard normal deviation	1.282	1.645	1.960	2.576
Confidence level	80%	90 %	95 %	99 %



(a)

Figure 1. a) Location of Gol-E-Gohar mines collection inside the Sanandaj-Sirjan zone [43], b) Six mine areas of Gol-E-Gohar complex.



(b)

Continues of Figure 1. a) Location of Gol-E-Gohar mines collection inside the Sanandaj-Sirjan zone [43], b) Six mine areas of Gol-E-Gohar complex.

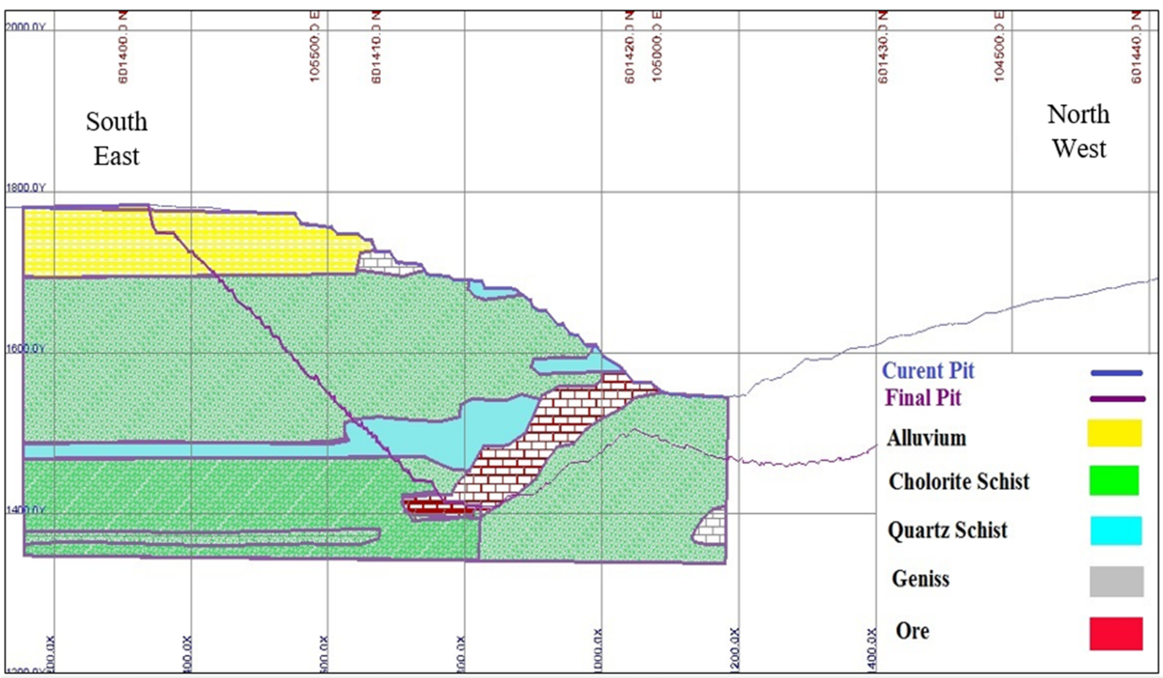


Figure 2. The north west-south east geological section of Mine 4 [43].

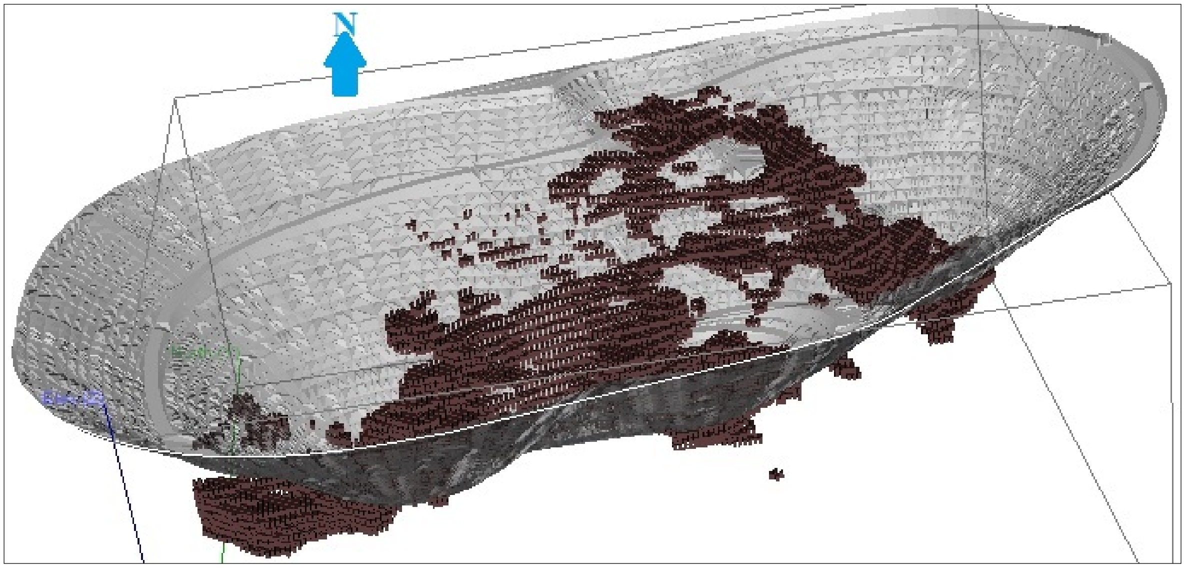


Figure 3. The final pit of Mine 4 [43].

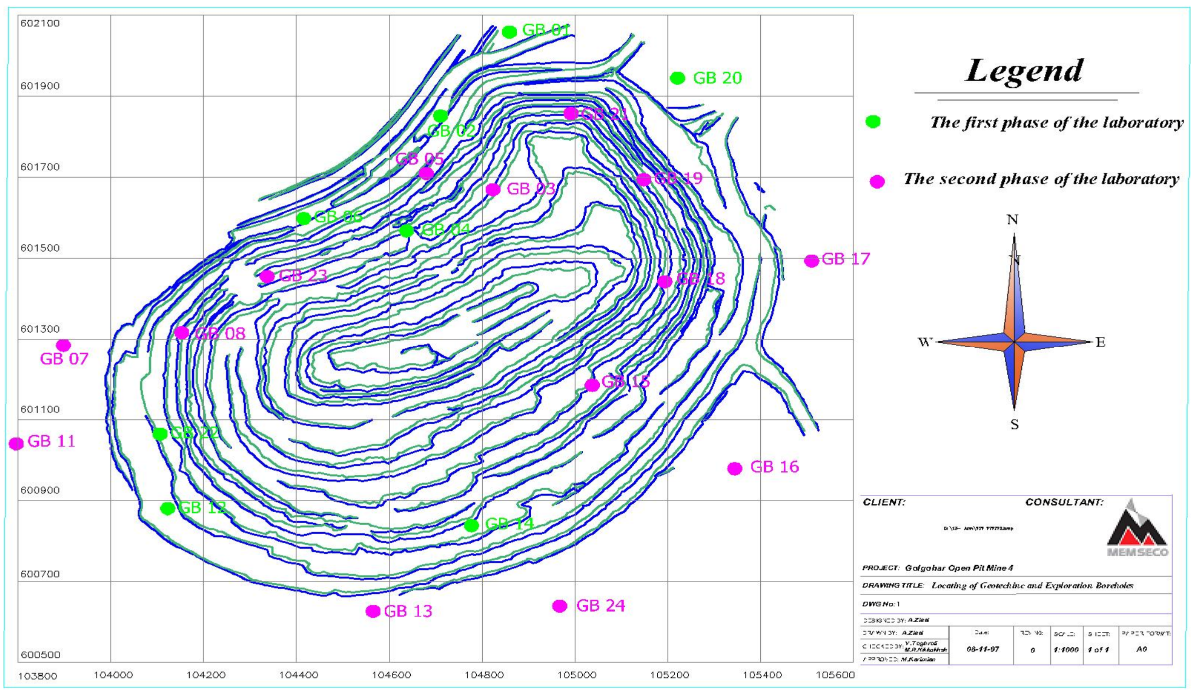


Figure 4. Location of drilled geotechnical boreholes [43].

3. Slope stability analysis of Mine 4

3.1. Structural zones and sections selected for stability analysis

According to the geology map, tectonic structures, hydrogeology situation, and mine geometry, the studied mine area is divided into eight separate geotechnical zones, as shown in Figure 5. To increase the accuracy of slope stability analysis, one section in each zone and a total of eight sections are selected for HPSSA, named A-A' to H-H' (Figure 6).

3.2. Laboratory rock testing

After drilling the geotechnical boreholes, required cores from every available rock type (including alluvium, conglomerate, chlorite schist, quartz schist, mica schist, quartzite, gneiss and ore) were collected and transferred to the laboratory for preparation and testing (Figure 7). In the next step, the physical and mechanical properties of rock specimens, including γ, φ and c, were measured according to the suggested standards by ISRM [44]. Detailed descriptions of the procedures of these tests can be found in the related literature [45–48], and thus, a brief explanation is provided here. Used instruments for triaxial compressive strength (TCS) testing and a failed sample after loading are presented in Figure 8. Also, the number of performed TCS and γ tests on the different rock types for HPSSA is given in Table 2.

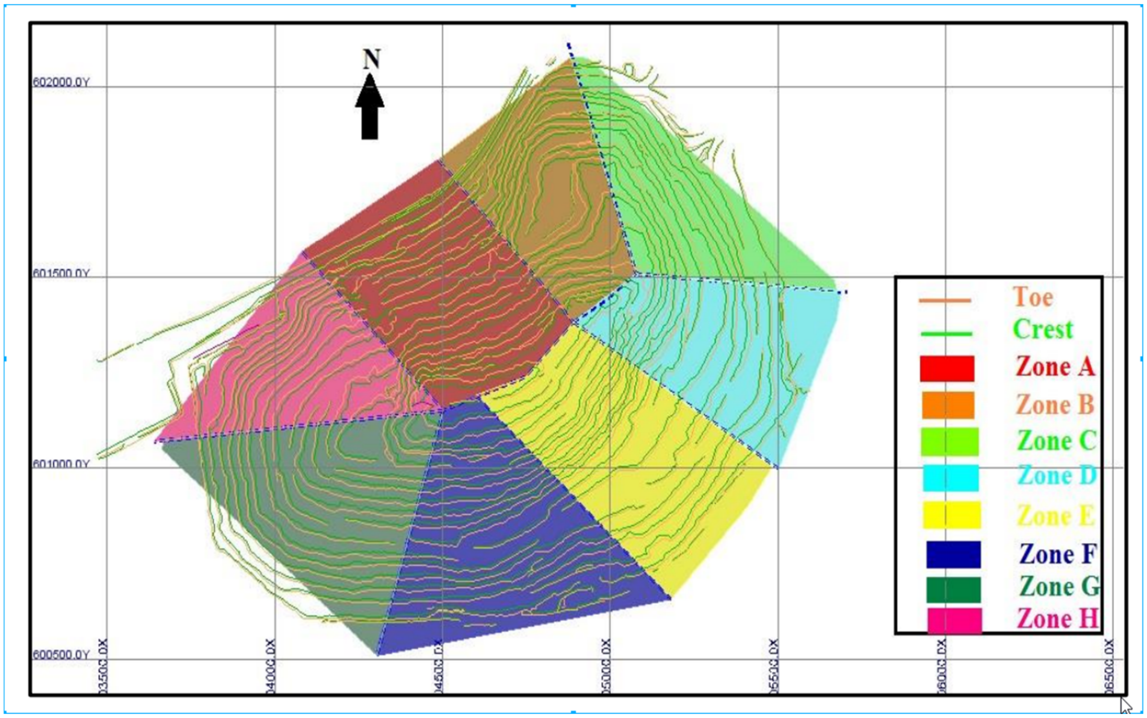


Figure 5. Separate eight geotechnical zones of Mine 4.

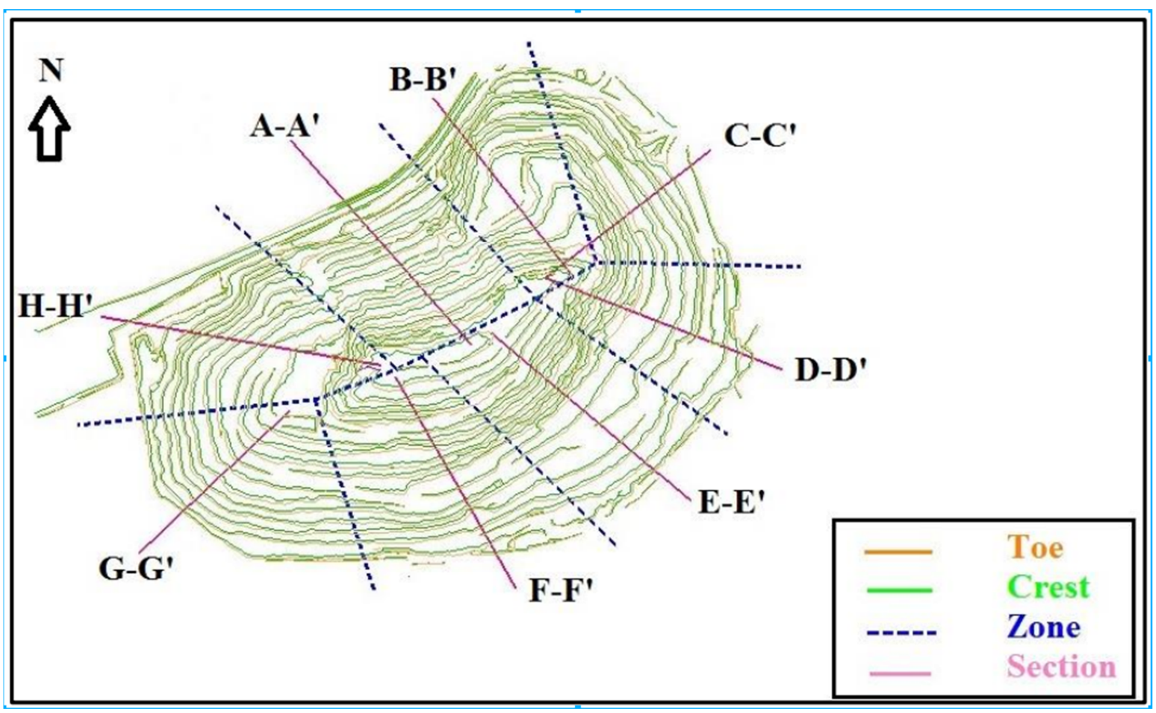


Figure 6. Location of eight separate sections selected for slope stability analysis.

Figure 7. Transferred cores to the laboratory for preparation and testing.

Table 2. Number of performed tests for slope stability analysis.

Test type	Number of conducted tests
γ	370
TCS	300

3.2.1. Unit weight measurement

As mentioned in the previous sub-section, about 370 tests were conducted over the different existing rock types in the areas of Mine 4 to measure the unit weight of various prepared samples. The statistical characteristics of measured γ related to the iron ore and other available rock types are given in Table 3. Also, the distribution of γ for a number of rock samples is drawn in Figure 9. In this study, the unit weight of cylindrical rock samples was measured based on the approach proposed by ISRM [44]. Generally, the unit weight of samples is calculated according to this equation [44]:

$$\gamma = \frac{W}{V} \tag{2}$$

where W and V are the weight and volume of samples.

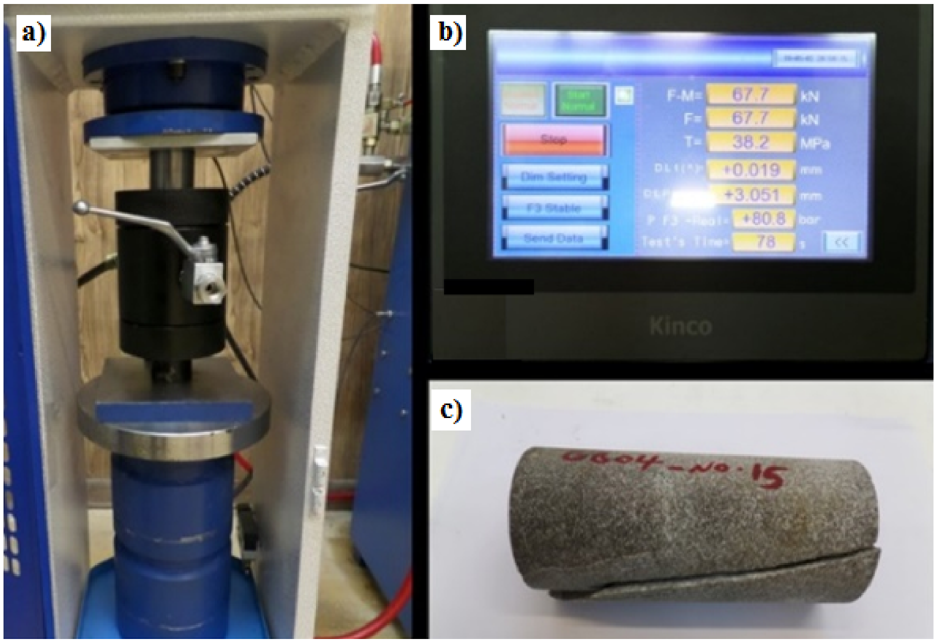


Figure 8. Conducting TCS test: a) Instrument, b) Data monitoring, c) A failed sample.

Table 3. Statistical characteristics of γ (kN/m^3) for various tested samples.

Sample type	Tests number	Minimum	Maximum	Average	Standard deviation
Conglomerate	20	20.11	25.60	23.15	0.16
Chlorite schist	181	24.03	29.14	26.59	0.12
Gneiss	66	25.21	28.65	26.68	0.07
Quartzite	20	25.51	27.96	26.88	0.09
Mica schist	137	22.66	28.06	26.09	0.09
Quartz schist	71	24.92	29.23	26.19	0.09
Ore	36	27.96	43.65	32.96	0.6

3.2.2. TCS measurement

Different TCS tests were performed on the different rock samples. A full automatic servo-control tool was used to perform the TSC test. This test was performed according to the suggested standard by ISRM [44]. After analyzing the test results, statistical characteristics of *c* and φ were calculated for tested samples. The first step of probability analysis is using the Monte Carlo method to determine the best statistical distribution function adapted to the input variable parameters. Based on the obtained information from the geotechnical boreholes and test results analysis, statistical characteristics related to the φ and *c* distributions for the mentioned samples in each structural zone are defined and given in Tables 4 and 5, respectively.

To prepare the experimental results and specify the characteristics of intact samples, the rock masses' properties were determined for different geological units in A-H structural zones using the Hoke and Brown criteria. Figure 10 also shows the histogram of these parameters along with their distribution curves (which tended to follow a normal distribution).

Table 4. Results of the normal distribution of φ (º) for various rock masses and alluvium.

Sample type	Tests number	Minimum	Maximum	Average	Standard deviation
Alluvium	11	-	-	41.6	-
Conglomerate	6	54.1	54.1	54.1	0
Chlorite schist	159	41.7	56.7	49.1	3.03
Gneiss	46	47.2	58.6	54	2.53
Quartzite	4	-	-	53.2	-
Mica schist	60	48.5	53.9	53	2.1
Quartz schist	9	-	-	58.8	-
Ore	5	-	-	56.6	-

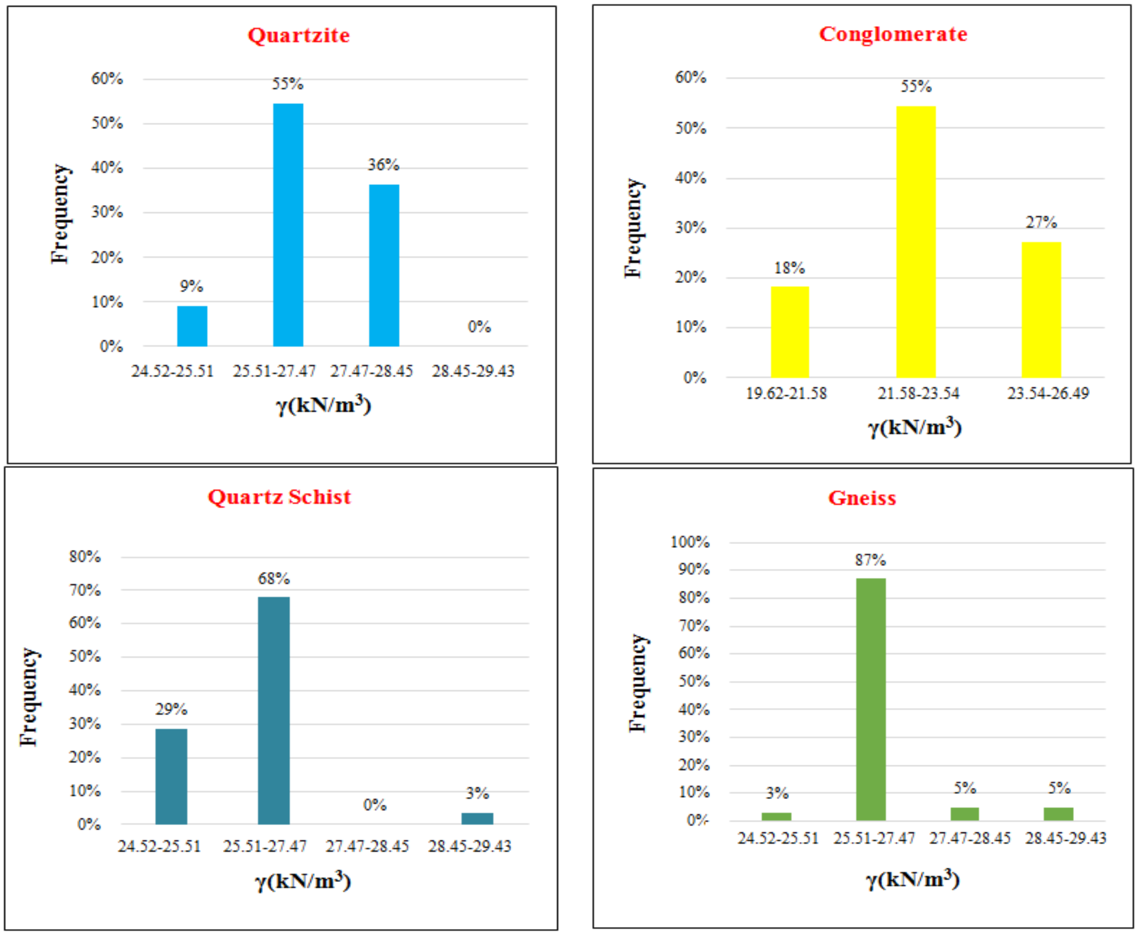


Figure 9. Distribution of γ for quartzite, conglomerate, quartz schist and gneiss rocks.

Table 5. Results of the normal distribution of *c* (kPa) for various rock masses and alluvium.

Sample type	Tests number	Minimum	Maximum	Average	Standard deviation
Alluvium	11	-	-	2.2	-
Conglomerate	6	1.9	1.9	1.9	0
Chlorite schist	159	3.2	6.4	4.5	1.73
Gneiss	46	2.9	7	4.8	2.96
Quartzite	4	-	-	5.4	-
Mica schist	60	1.7	5.8	3.38	0.8
Quartz schist	9	-	-	6.7	-
Ore	5	-	-	4.7	-

3.3. HPSSA of mine 4

3.3.1. Method implementation

The HPSSA approach is conducted based on the Mohr-Coulomb criterion in this study. This behavioral model represents the materials that reach the yield limit only due to the shear factor. This is a conventional model in rock mechanics for describing material behaviour under shear stress conditions. The distribution of parameters that represent the plastic state include *c*, φ and *γ*. These parameters are investigated as effective variables in stability analysis that involve inherent uncertainty. Although the utmost care has been taken in core drilling, sample preparation and conducting the laboratory tests to determine the geo-mechanical properties of the rock mass in this study, the certainty of the test results can't be fully confirmed due to the rock mass complexity, inhomogeneity and anisotropy as well as the human errors in practice. According to the above facts, a comprehensive MCS approach has been used in this study to cover the available inherent uncertainties in all processes of the slope stability analysis. Latin Hypercube sampling method in the MCS is utilized for the probabilistic slope stability analysis due to its proven advantages in the sampling stage, convergence process and gaining reliable results. This approach relies on "stratified" sampling, where selections are made randomly within each category. This method results in a smoother sampling of the probability distributions. Traditionally, employing the Latin Hypercube technique allows for conducting each analysis with a thousand samples, enhancing the reliability of the probabilistic analysis being performed. The PF value is then calculated to provide an objective measure of failure risk and reliability index (RI), which better represents the safety level. The Slide 6.0 software is utilized to analyze the probabilistic slope stability in this study. The distribution function of input variables is normally considered based on the frequency distribution diagram of measured data. Also, the global minimum SF for each failure search is calculated using the simplified Bishop technique.

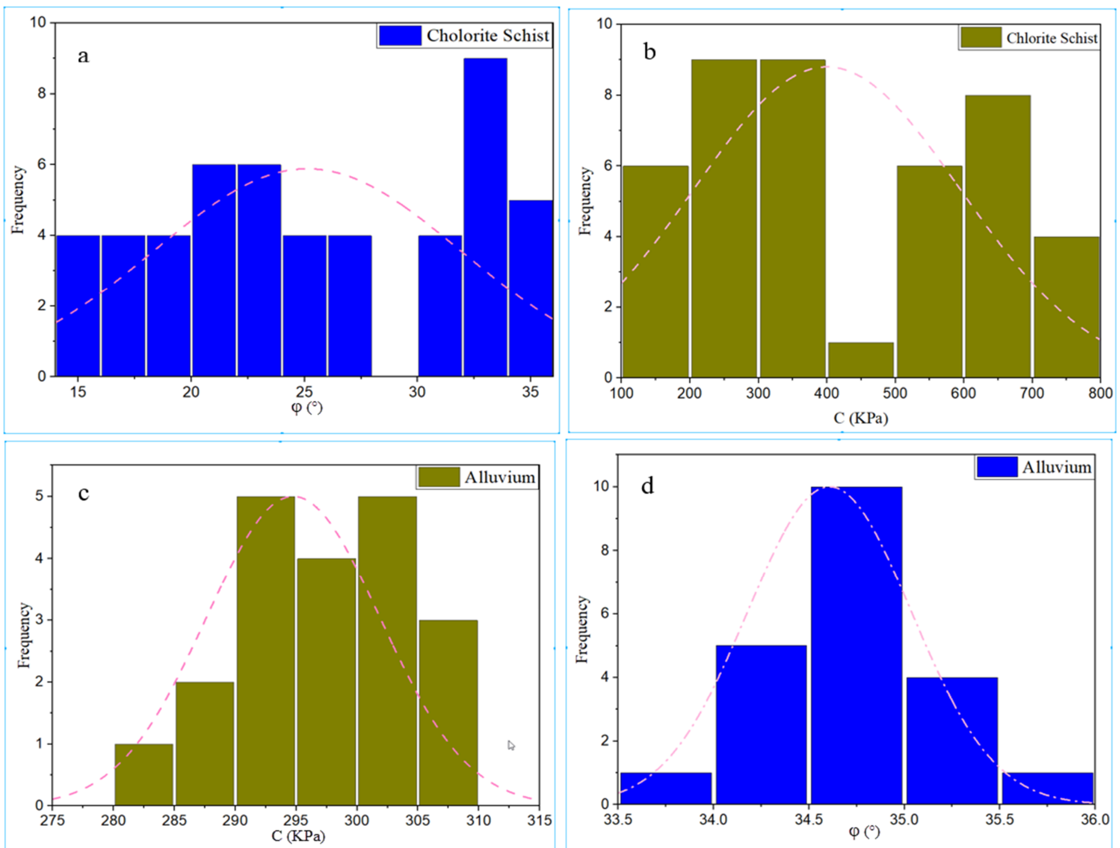


Figure 10. Histogram of measured datasets and related normal distribution functions for intact samples: a) φ of chlorite schist, b) *c* of chlorite schist, c) φ of alluvium, d) *c* of alluvium.

3.3.2. Method results

The results of the conducted HPSSA are demonstrated in Table 6 for circular slip and distribution function of SF and PF indices obtained from the probabilistic analysis of 8 structural sections using Bishop's method. The distribution function of SF is also obtained for different sections in the form of distribution functions named normal, beta and gamma. According to Table 6, the mean probabilistic SF varies from 0.93 to 1.86 for the case of auto grid at the PF of 0 to 75.6%. Also, the results show that the probability of failure in sections A-A' and B-B' (current fall range) is more than 75% and 60%, respectively. The outputs of the LEM probabilistic analysis can also be presented graphically. For example, Figure 11 presents the obtained SF values for different rock units in section A-A' using the circular slip search–Bishop corrected method. This figure represents all global minimum surfaces and critical probabilistic surfaces, which means SF, PF and RI are achieved at 0.939, 75.6% and 0.693, respectively. Also, Figure 12 is a histogram plot that shows the distribution of calculated SF by the probabilistic analysis of section A-A'. The highlighted red bars in Figure 12 show the analyses which achieved a SF value of less than 1.1. This figure represents the PF scheme, which is equivalent to the ratio of the red bars area to the histogram total area or PF=75.6%. As it is observed, a Gama distribution is the best fit in this case. Finally, the histogram of thousand data simulation of C, φ and γ with the normal distribution function for gneiss cohesion and ore internal friction angle is drawn in Figure 13. As can be seen, the histograms of C and φ data follow a normal distribution and are in good agreement with the normal distribution of the slope stability analysis outputs.

Table 6. Probabilistic analysis results for 8 sections of mine 4 using Bishop's method.

Section	Safety Factor	Probability of failure	distribution function	standard deviation	Minimum SF	Maximum SF
A-A'	0.93	75.6%	Gama	0.088	0.68	1.22
B-B'	1.86	0%	Normal	0.24	1.11	2.67
C-C'	1.20	7.4%	Normal	0.13	0.77	1.65
D-D'	1.20	8.5%	Normal	0.14	0.77	1.66
E-E'	1.27	7.2%	Beta	0.18	0.77	1.82
F-F'	1.59	0%	Gama	0.19	1.00	2.2
G-G'	1.61	0.1%	Gama	0.20	0.98	2.34
H-H'	0.97	60.3%	Normal	0.09	0.65	1.24

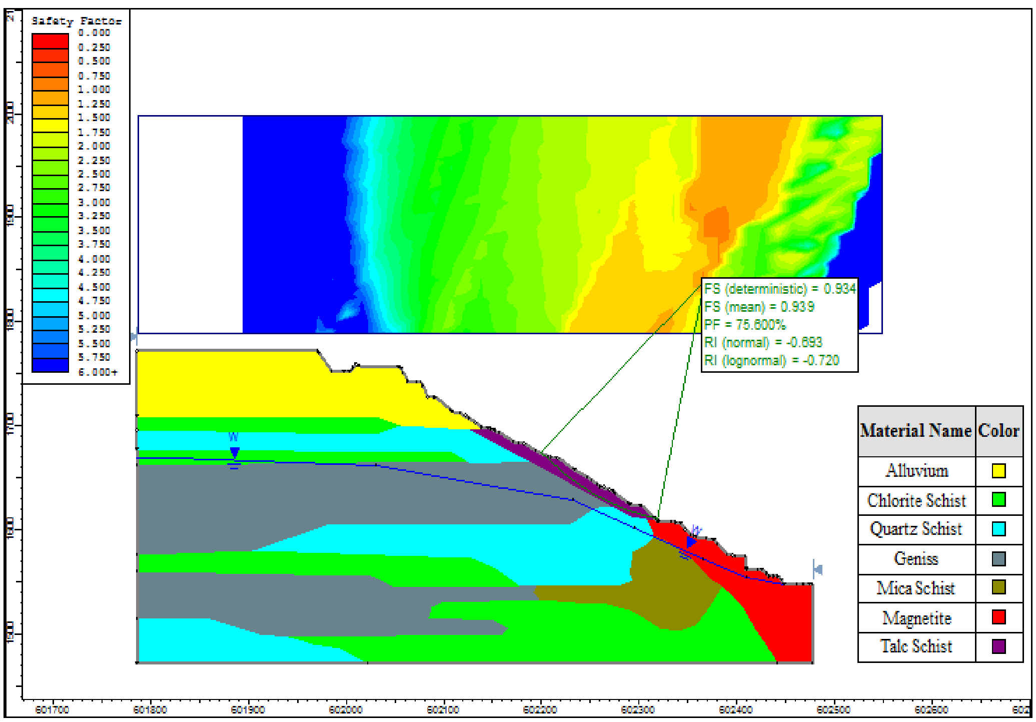


Figure 11. Results of the probabilistic analysis of section A-A'.

4. Results validation

4.1. Slope Monitoring

This section compares the probability failure analysis of sections A-A' to H-H' with the results of displacement monitoring to validate the conducted HPSSA method. In the last 5 years, about 118 prisms were installed inside the rock walls of the mentioned zones. Although some prisms were defective due to the mining operation and landslides, wall displacement was recorded by the usable prisms twice a week. The location of installed reflectors and prisms in the north and east walls of Mine 4, the displacement trend of the T13 reflector and the installed reflector in the south wall during 3 months are depicted in Figure 14.

The average displacement recorded from the reflectors in each zone was calculated and compared with the obtained SFs from the probability failure analysis for sections A-A' to H-H' and given in Table 7. Also, comparison results of probability analysis with the displacement monitoring for sections A-A' to H-H' are represented in Figure 15a in the form of SF-displacement correlation. According to Figure 15b, the average displacement in each zone is consistent and has an acceptable trend with the SF. According to the obtained determination coefficient (R^2=0.99), there is a good agreement between the trend of SF and displacement. The increase in the SF is archived by a decrease in the displacement. For example, the highest displacement (about 10000 mm) is related to the A-A' zone with a SF of 0.93. This comparison confirms the results of the current study and proves its validity.

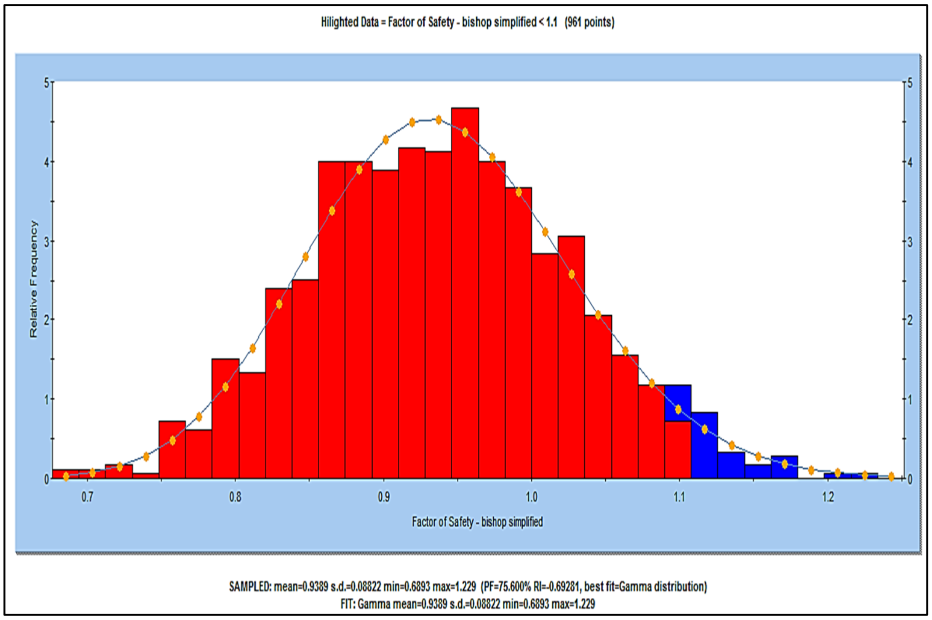


Figure 12. Distribution function of safety factor for section A-A'.

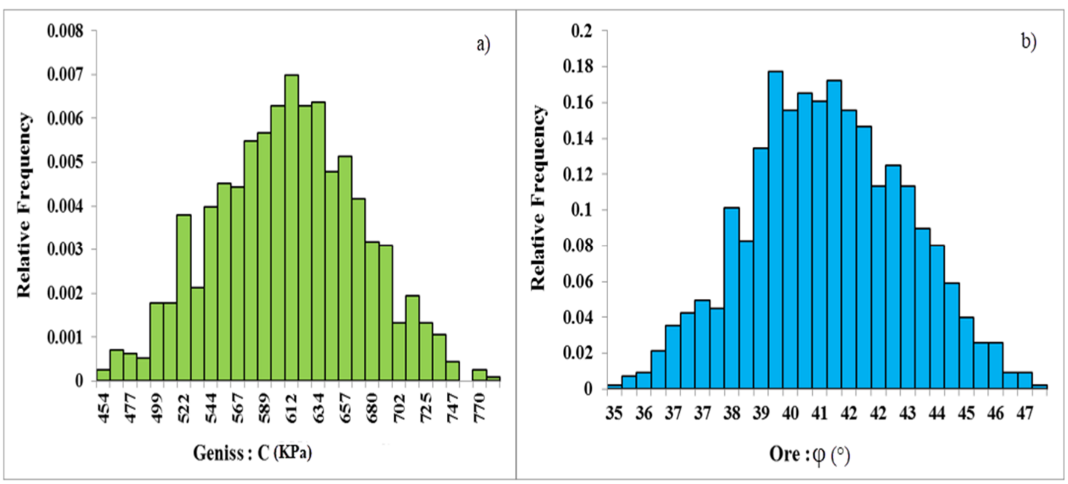


Figure 13. Histogram of distribution function: a) *c* simulated to gneiss b) φ simulated to ore.

Table 7. Comparison results of in-situ displacement monitoring and obtained SF from the probabilistic analysis for 8 structural zones of mine 4.

Section	Average displacement (mm)	SF	Number of reflector
A-A'	9,748	0.93	25
B-B'	115	1.86	22
C-C'	532	1.20	11
D-D'	489	1.20	8
E-E'	352	1.27	8
F-F'	197	1.59	10
G-G'	211	1.61	7
H-H'	6,437	0.97	27

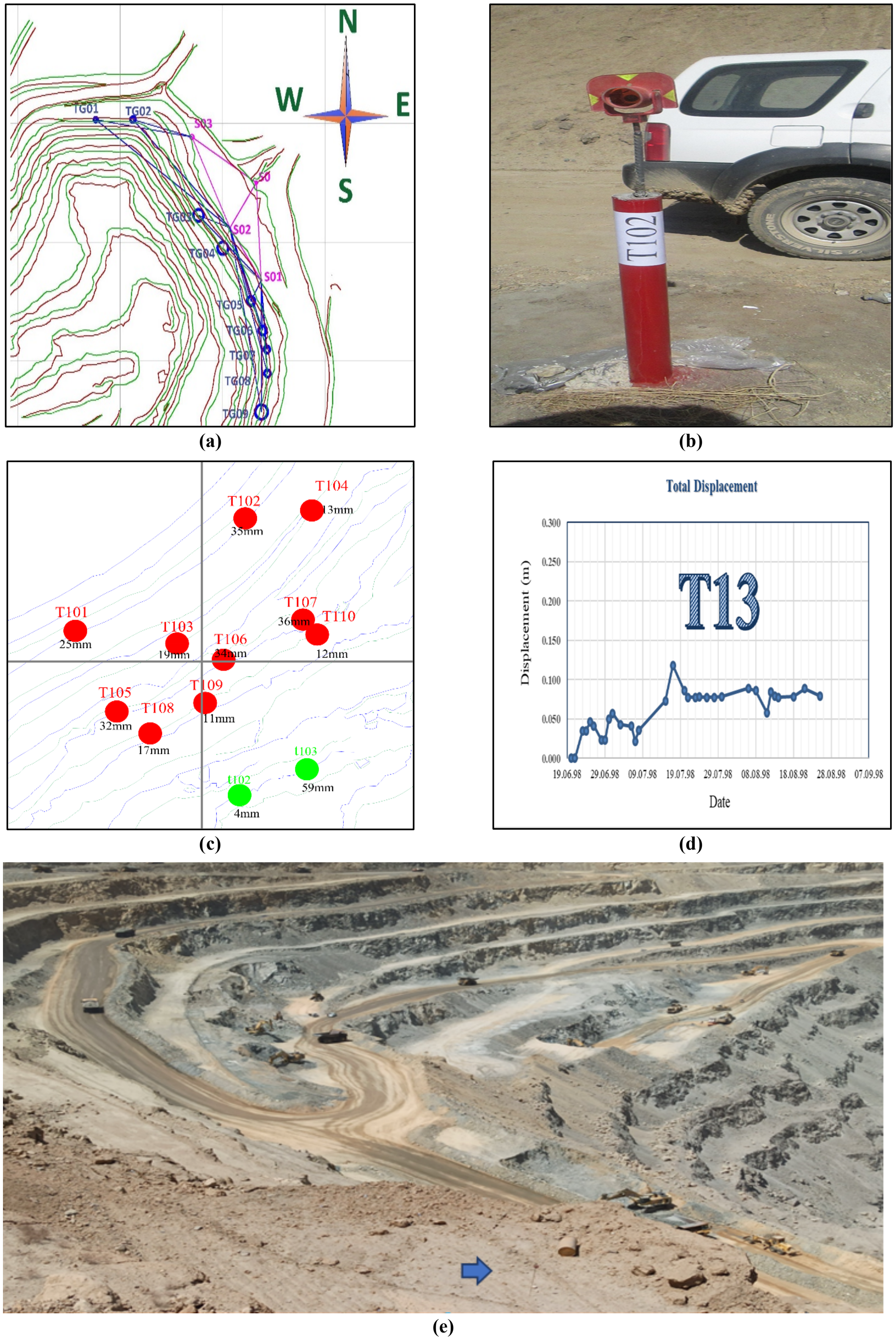
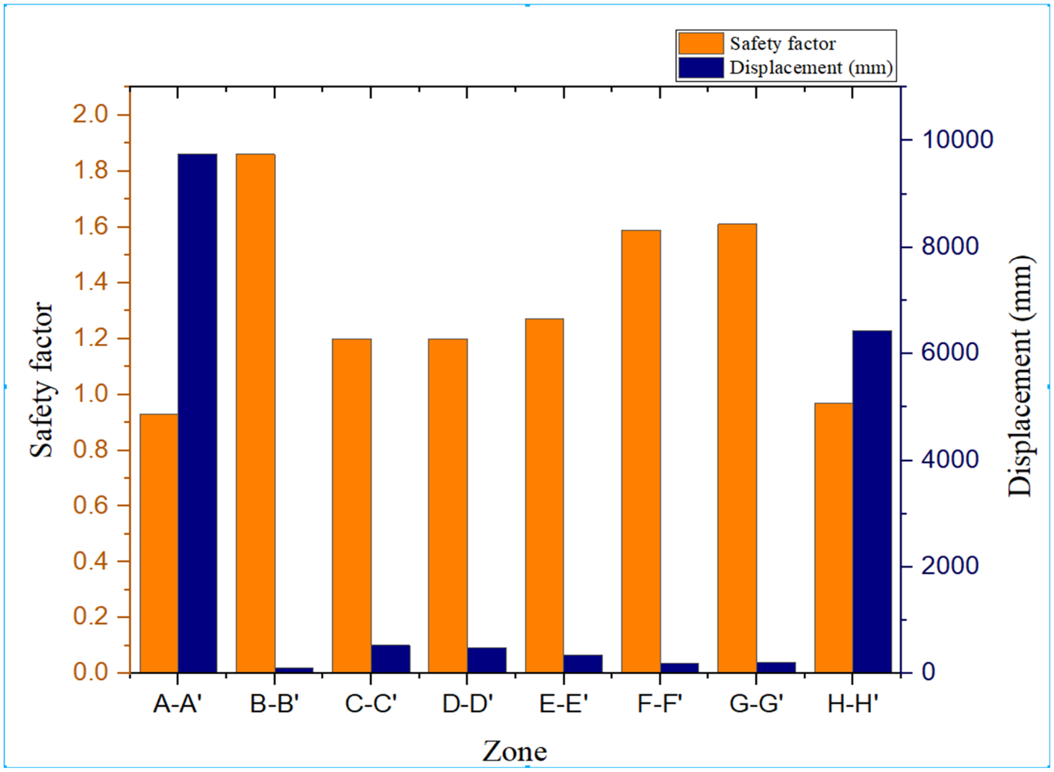
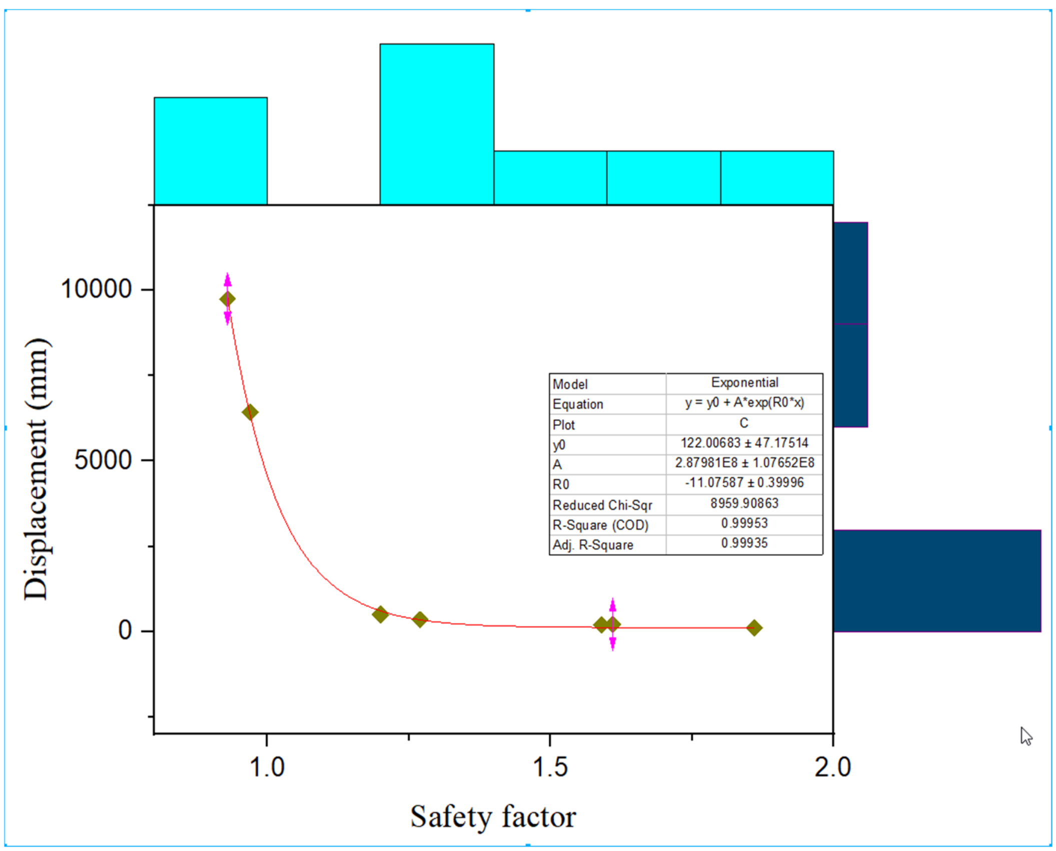


Figure 14. a) Location of installed reflectors in east wall, b) Used prisms, c) Location of installed reflectors in north wall, d) Displacement envelope of T13 reflector, e) Installed reflector in south wall.



(a)



(b)

Figure 15. a) Comparing the obtained SF and displacement monitoring in each structural zone of min 4, b) Correlation of SF with displacement monitoring.

4.2. Validation through comparative analysis

The validation of HPSSA approach with displacement monitoring data proved its accurateness and the reliability of the model. The high correlation between displacement monitoring data and HPSSA results confirms its credibility, suggesting that HPSSA can effectively predict slope behavior. Nonetheless, while the reliability of the methodology was confirmed through monitoring data, it would benefit from a comparison with other slope stability analysis methods. Such comparisons could provide a clearer picture of the HPSSA method's performance relative to existing techniques, further validating its effectiveness and potential for broader applications. Accordingly, output of the HPSSA model is compared with the outputs of some of the previous studies. Results of this comparison are summarized in Table 8. As it can be seen from this table, there is a good agreement between the results of the HPSSA approach and other comparable methods and its better accuracy can be observed than most of the previous slope stability methodologies used for the pit slope stability.

Table 8. Comparing the HPSSA result with the results of some of the previous methods used for the pit slope stability analysis of the mine.

No.	Method	Accuracy (R^2)	Descriptions	Reference
1	Numerical model	0.80	Correlation with in-situ monitoring	[10]
2	Numerical model	0.99	Correlation between simulated and predicted values	[49]
3	SSPC system	0.95	According to maximum probability of discontinuity stability with respect to sliding and toppling	[50]
4	Combination of TLS and UAV	0.80	According to minimum 20% chance for specific slope failure mode	[51]
5	Empirical models	0.815	Maximum acquired correlation	[52]
6	LEM and FEM	0.97	According to minimum probably failure	[53]
7	HPSSA model	0.99	Correlation with in-situ monitoring	Current study

Note: TLS is the terrestrial laser scanner, UAV is the unmanned aerial vehicle, LEM is the limit equilibrium method, FEM is the finite element method and SSPC is the slope stability probability classification.

5. Advantages, limitations, and future studies

It is notable that solely focusing Bishop's technique on circular failure surfaces simplifies the stability analysis, oversimplifies the rock mass geotechnical complexity, and may neglect the main risks from the non-circular failures. However, the combination of this technique with Monte Carlo simulation and Mohr-Coulomb criterion in this study covered its shortcomings in the conducted slope stability analysis. The acceptable results obtained from the HPSSA model proved its validity and showed that the Bishop's technique's weaknesses have been eliminated by its combination with MCS and the Mohr-Coulomb criterion. However, future studies are required to broaden the scope of the analyzed failure geometries in the mine slope stability analysis, which can be an attractive topic for future studies in this field.

On the other hand, the probability of a difference between actual conditions and analysis results is high in deterministic slope stability analysis because material uncertainties play a significant role in the validity of the obtained results. Therefore, the impact of uncertainties is reduced, and the obtained results are presented with more precision and accuracy in the probabilistic stability analysis.

Considering the archived results from this study, it can be concluded that the probabilistic slope stability analysis can lead to better results by taking into account material uncertainties. This is archived by considering the existing uncertainties while evaluating the effect of geo-mechanical properties of rock layers and the influence of geometric parameters on slope stability. Besides the physical, mechanical and geometrical characteristics, the structural characteristics (i.e., discontinuities properties) can cause further uncertainties in determining the physical and resistance properties of rock masses, and thus, more attention to this issue is essential in future studies.

6. Conclusions

In this research, the HPSSA analysis was conducted for the pit slopes of Mine 4 of the Gol-E-Gohar iron complex. For this purpose, the Mine 4 area was first divided into 8 structural zones and 22 boreholes were drilled and logged for geotechnical investigation. Core specimens from the geotechnical holes were obtained and prepared to perform the laboratory tests and provide a suitable database for HPSSA. The dataset of material properties was used as an input for probabilistic slope stability analysis using the combination of MCS, Mohr-Coulomb criterion and simplified Bishop's method in the Slide 6.0 software environment. Using this hybrid method, the PF, RI and SF values were calculated for the 8 structural zones of the pit. Pit wall displacement monitoring data obtained from the installed prisms were used to validate the results of HPSSA. Finally, a comparative analysis was performed to compare the results of HPSSA with the previous slope stability analysis methods conducted at the mine. Accordingly, the main conclusions of this study are summarized as:

1- The normal distribution function was achieved as the best curve fit for C, φ and γ variables in the HPSSA.

2- The mean probabilistic SF of all considered structural zones varied from 0.93 to 1.86, with PF values ranging between 0 and 75.6%.

3- According to the HPSSA results, the SF values of the A-A' section ranged from 0.68 to 1.22, with a mean value of 0.93, which is the most unstable section. So, to avoid the probable collapse, the overall slope angle must be reduced in this section.

4- Based on the slope stability analysis of sections A-A' and H-H', the PF values in the north wall are more than 60% for the H-H' section and 75% for the A-A' section. Accordingly, it can be concluded that the north wall of Mine 4 is more likely to be unstable with a PF of more than 60%. Therefore, deformations in the northern walls must be continuously monitored.

5- The mean SF in other sections (B-B' to G-G') was more than 1.2, and their PF values were less than 5%, which shows their relatively stable conditions.

6- Pit wall displacement monitoring in each structural zone proved that there is a good agreement between the trend of obtained SFs from the HPSSA and measured pit wall displacements (a correlation coefficient of R^2=0.99). This comparison proved the results of the developed HPSSA approach.

7- The comparative analysis indicated that the suggested HPSSA method was more accurate than most of the previous similar studies for the mine's pit wall stability.

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ترکیب شبیه‌سازی مونت کارلو و تکنیک بیشاپ برای تحلیل پایداری شیب معدن آهن روباز گل‌گهر

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چکیده:

در این تحقیق از یک روش جدید تحت عنوان تحلیل پایداری شیب احتمالی ترکیبی (HPSSA) برای معدن ۴ مجموعه سنگ آهن گل گهر در ایران استفاده شده است. به‌منظور تحلیل پایداری شیب، دیوارهای معدن ابتدا به ۸ ناحیه ساختاری مجزا شامل مقاطع 'A-A تا 'H-H تقسیم گردیده است. سپس، نمونه‌های مغزه‌ای کافی از ۲۲ گمانه حفر شده تهیه و پارامترهای لازم برای طراحی شیب شامل چسبندگی (c)، زاویه اصطکاک داخلی (ϕ) و وزن مخصوص (γ) اندازه‌گیری شده است. در نهایت، روش HPSSA با استفاده از ترکیب شبیه‌سازی مونت کارلو (MCS)، معیار موهر-کولمب و تکنیک بیشاپ اجرا شده است. با توجه به نتایج HPSSA، تابع توزیع نرمال به‌عنوان بهترین برازش منحنی برای پارامترهای c، ϕ و γ به‌دست آمده است. همچنین مقادیر به‌دست‌آمده میانگین ضریب ایمنی (SF) احتمالی برای پهنه‌های ساختاری تعریف شده از ۰/۹۳ تا ۱/۸۶ با احتمال خرابی (PF) مختلف از ۰ تا ۷۵/۶٪ متغیر است. بعلاوه، مقادیر SF از ۰/۶۸ تا ۱/۲۲ (مقدار متوسط ۰/۹۳) با PF برابر با ۷۵٪ برای بخش 'A-A و از ۰/۶۵ تا ۱/۲۴ (مقدار متوسط ۰/۹۷) با PF برابر با ۶۰٪ برای بخش 'H-H متغیر است. بنابراین، نتیجه‌گیری می شود که مقطع 'A-A و دیواره شمالی معدن با مقدار PF بزرگتر از ۶۰٪، مستعد ناپایداری بیشتری است. از طرف دیگر، مقادیر SF بزرگتر از ۱/۲ و PF کوچکتر از ۵٪ برای سایر دیوارهای معدن (بخش‌های 'B-B'-G-G) نشان می‌دهد که احتمال ناپایداری آنها بسیار کم است. پایش برجای جابجایی دیواره‌های پیت معدن با استفاده از منشورهای نصب شده نشان داد که میانگین جابجایی‌ها در مناطق ساختاری تعریف شده دارای روند مشابهی با مقادیر SF حاصل از روش HPSSA هستند. همچنین، نتایج حاصله نشان داد که تطابق خوبی بین روند ضرایب ایمنی احتمالی حاصل از روش HPSSA و جابجایی‌های مشاهده شده بر روی شیب دیواره معدن وجود دارد. در نهایت بر اساس تحلیل مقایسه‌ای، اعتبار روش HPSSA پیشنهادی با دقت نسبتاً بالاتری نسبت به اکثر روش‌های قبلی تحلیل پایداری شیب تأیید گردید.

کلمات کلیدی: معدن آهن گل‌گهر، پایداری شیب، شبیه‌سازی مونت کارلو، پایش جابجایی.