

Integrated assessment of open-pit slope stability considering structural anisotropy, strength parameter uncertainty, and slope geometry

Aibek Mussin¹ , Askar Imashev^{1*} , Nurlan Suleimenov^{1*} , Gulnur Nurshaiykova² ,
Zukhra Tungushbaeva² , Temirlan Malgazdarov² 

¹ Abylkas Saginov Karaganda Technical University, Karaganda, Kazakhstan

² D. Serikbayev East Kazakhstan Technical University, Ust-Kamenogorsk, Kazakhstan

*Corresponding author: e-mail a.imashev@ktu.edu.kz, n.suleymenov@ktu.edu.kz

Abstract

Purpose. To develop and validate an integrated methodology for open-pit slope stability assessment that provides a traceable transition from structural data of a fractured rock mass to Q'-GSI classification, kinematic analysis, strength parameter calibration, deterministic and probabilistic stability assessment, and quantitative evaluation of the influence of slope geometry.

Methods. Structural data from a deposit in the East Kazakhstan Region were processed in Dips using Terzaghi correction for orientation bias and Fisher contour analysis. Q' and GSI were determined from RQD, J_n , J_r , and J_a . Strength parameters were calibrated in RSDData using the Hoek-Brown and Mohr-Coulomb criteria. The simplified Bishop method, the Spencer method, and the General Limit Equilibrium (GLE) method with the Morgenstern-Price interslice force function were applied in Slide2.

Findings. Four dominant discontinuity sets were identified from 436 orientations, and values of Q' = 3.67 and GSI = 55.7 were obtained. The proportion of kinematically feasible elements was 14.83-29.73% for planar sliding and 54.80% for flexural toppling. Four of the six intersection lines between the mean discontinuity sets satisfied the conditions for wedge sliding. For the 51.8 m high profile with an overall slope angle of 44°, the mean FS ranged from 0.9896 to 0.9947, while the probability of failure P_f ranged from 53.6 to 57.8%. The strongest relationship with FS was established for the internal friction angle ($r = 0.9863$). At a slope angle of 40°, increasing the height from 40 to 60 m increased P_f from 23.5 to 47.9%, whereas at $H = 20$ m, increasing the slope angle from 45 to 50° increased P_f from 23.9 to 90.0%.

Originality. A unified assessment sequence is proposed that integrates structural-statistical data processing, Q'-GSI classification, kinematic analysis, local calibration using the Hoek-Brown and Mohr-Coulomb criteria, probabilistic limit equilibrium analysis, and assessment of the influence of slope geometry. It is shown that kinematic susceptibility to structurally controlled failure mechanisms and P_f characterize different levels of risk and therefore require joint interpretation.

Practical implications. The proposed methodology enables the identification of unfavourable spatial orientations and geometric scenarios, determination of the parameters requiring priority refinement, and simultaneous assessment of stability changes in terms of the factor of safety FS and probability of failure P_f .

Keywords: open-pit slope; fractured rock mass; slope stability; Geological Strength Index; kinematic analysis; limit equilibrium method

1. Introduction

Open-pit slope stability is one of the key challenges in engineering geomechanics, as deformation of the rock mass forming the pit walls directly affects personnel safety, the integrity of haulage infrastructure, mineral recovery, and the continuity of mining operations. As open pits become deeper, slope heights and extents increase, the stress state changes, and lithologically and structurally heterogeneous zones become involved in the potential deformation region. Unlike a homogeneous soil slope, the behaviour of a rock slope is controlled by both the strength of the intact rock material and the presence of discrete discontinuities. Consequently, simi-

lar mean strength values may result in different failure mechanisms depending on the orientation, connectivity, aperture, infilling, and roughness of discontinuities [1], [2].

Current approaches to rock slope stability assessment can be broadly divided into four complementary groups: empirical classification systems, kinematic analysis, limit equilibrium methods, and numerical modelling of the stress-strain state [1]. Classification systems provide a rapid integral characterization of rock mass quality; kinematic analysis establishes the geometric feasibility of planar and wedge sliding or toppling; limit equilibrium methods determine the relationship between available shear resistance and mobilized driving forces; and numerical methods allow deformation patterns and the develop-

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ment of plastic zones to be investigated. The most robust design decisions are therefore obtained not by selecting a single “best” method, but by consistently comparing the results obtained at several levels of analysis [3], [4].

A structural model provides the initial basis for such integration. Conventional discontinuity-orientation measurements remain important, although their results depend on the orientation of the survey line and the accessibility of exposed surfaces. Terzaghi correction reduces the systematic underrepresentation of discontinuities oriented at small angles to the survey line [5]. In recent years, field mapping has increasingly been supplemented by terrestrial photogrammetry, laser scanning, digital outcrop models, and automated extraction of discontinuity orientations [6]–[8]. These technologies improve spatial coverage but do not eliminate the need for engineering verification of the identified discontinuity sets, because a statistically distinct cluster does not necessarily represent a mechanically continuous plane of weakness.

Kinematic analysis remains one of the most effective approaches for the preliminary identification of structurally controlled failure mechanisms. Modern software and GIS-based tools automate Markland tests and enable the orientation of slope surfaces to be compared locally with large numbers of measured discontinuities [9], [10]. However, the percentage of poles or intersection lines falling within a critical region represents kinematic susceptibility rather than the probability of actual failure. For a failure mechanism to develop, the discontinuities must have sufficient persistence and connectivity, daylight on the free surface, overcome shear resistance, and be subjected to an appropriate stress state. Confusing these concepts may lead to inappropriate comparison of kinematic percentages with the probability of failure P_f , obtained from slope stability analysis [1], [3].

For an integral characterization of rock mass structural quality RQD, Q, Q', GSI, and the slope-specific Q-slope system are commonly used. Q-slope retains the basic RQD/J_n and J_n/J_a , ratios while supplementing them with orientation-related and external factors [11]–[13]. Recent studies have confirmed its usefulness at the preliminary assessment stage, while also demonstrating that the resulting classification is sensitive to the quality of field documentation and cannot replace analysis of a specific potential failure surface [11]. In the present study, Q' was used because the parameters describing groundwater conditions and the stress-related factor were not available with comparable reliability. This approach reduces the number of additional assumptions at the classification stage, while dry conditions are subsequently considered separately in the stability analysis.

The next methodological issue concerns the transition from intact rock properties to rock mass properties. The generalized Hoek-Brown criterion relates rock mass strength to intact rock strength, the m_i parameter, GSI, and the disturbance factor D [14]. Its nonlinearity is particularly important over a wide range of confining stresses. For LEM applications, the nonlinear envelope is often approximated by a linear Mohr-Coulomb relationship; however, the resulting values of c and ϕ depend on the stress range selected for linearization. Therefore, parameters derived at a low σ_{3max} cannot be transferred directly to failure surfaces characterized by substantially higher confinement. Similarly, the rock mass deformation modulus should account for structural

condition and scale rather than being equated to the modulus of an intact rock specimen [15].

Practical studies of complex-structured ore rock masses also confirm the need to consider rock mass structural conditions together with excavation geometry. In stability assessments of stopes and inter-stope pillars, RQD, Q, and GSI are used in combination with laboratory data and numerical modelling, with both geometric parameters and the condition of the fractured rock mass exerting a substantial influence. Similarly, when assessing blasting effects, the structural properties of the rock mass govern the development of damage zones, whereas in stoping systems the stability of exposed rock surfaces depends on excavation geometry and extraction sequence [16]–[18].

Deterministic analysis based on a single set of c , ϕ , and γ values remains an essential engineering basis, but it does not represent the natural variability and uncertainty of the input data. Abdulai and Sharifzadeh [2] showed that probabilistic methods are increasingly being applied in open-pit slope design because they enable a transition from a single FS value to a distribution of outcomes and a probability of failure. Budetta [3] emphasized the need to consider deterministic and probabilistic approaches jointly: the former is convenient for conventional factor-of-safety assessment, whereas the latter reveals the sensitivity of the results to variability in the input parameters. At the bench scale, studies have shown that accounting for variability in structural and strength characteristics can substantially alter risk estimates, while deterministic assessment based on mean parameter values does not always reflect the probability of unstable block formation [19], [20].

Spatial variability of rock mass properties represents another important issue. Jiang et al. [21][20] reviewed the development of random-field methods and showed that the use of mean values alone may either overestimate or underestimate reliability depending on the scale of fluctuations and the correlation length. Weak layers and their spatial continuity are particularly important for rock slopes. Zhang et al. [22] found that variability in a weak layer may have a greater influence on stability than variability in the properties of the surrounding rock mass, while seismic randomness can further modify P_f . Recent studies of open-pit mining conditions have also demonstrated the influence of weathering, groundwater conditions, and spatial heterogeneity on the location of the critical failure surface [23], [24].

To propagate uncertainty, direct Monte Carlo simulation, Latin Hypercube Sampling (LHS), response surface methods, FORM/SORM, and various surrogate models are commonly used. McKay et al. [25] showed that LHS provides a stratified representation of the range of each variable and more uniform coverage of the parameter space. Helton and Davis [26] further developed this approach for the analysis of complex engineering systems and correlated variables. In slope stability problems, sample adequacy should be confirmed through convergence analysis of the relevant statistical characteristics, because a fixed number of realizations alone does not guarantee the stability of a probabilistic estimate [27], [28].

Slope geometry is one of the principal controllable factors linking safety with the economic efficiency of open-pit mining. Increasing the slope angle reduces the volume of overburden removal, but at the same time increases the downslope component of the rock mass weight and reduces

the stability margin where unfavourably oriented discontinuities are present. Abdellah et al. [29] proposed determining the overall slope angle by considering the probability of instability together with productivity and cost. Comparative studies indicate that combining LEM, probabilistic analysis, and numerical verification provides a more robust basis for design than relying on a single method [30]. Therefore, practical slope design should assess not only an individual critical profile, but also the trends in FS and P_f associated with variations in slope height and angle.

In Kazakhstan, integrated geotechnical monitoring schemes have been developed that combine engineering-geological mapping, instrumental mine surveying observations, terrestrial laser scanning, unmanned aerial vehicles, satellite geodesy, and GIS-based stability mapping. Such approaches are used to identify disturbed zones within the rock mass, monitor deformation of open-pit slopes, and forecast changes in the geomechanical and geodynamic state during mineral extraction [31]–[35].

Despite the substantial body of research, a methodological gap remains in practical applications between structural mapping, rock mass classification, calibration of strength parameters, and probabilistic stability assessment. Parameters may be transferred between software packages without explicit consideration of scale effects and the relevant working stress range; indicators of kinematic susceptibility are sometimes interpreted on the same basis as P_f and the influence of geometry is often examined for only a single design profile. This highlights the need for an integrated methodology providing a traceable sequence of “measurement – interpretation – parameterization – calculation – sensitivity and result-stability analysis”.

The aim of this study is to develop and apply an integrated methodology for assessing open-pit slope stability using data from a deposit located in the East Kazakhstan Region. To achieve this aim, 436 discontinuity orientations were statistically processed and the rock mass was classified using Q' -GSI; kinematic analyses of planar sliding, wedge failure, and toppling were performed; strength parameters were calibrated using the Hoek-Brown and Mohr-Coulomb criteria; deterministic and probabilistic stability assessments were carried out using the Bishop, Spencer, and GLE/Morgenstern-Price methods; LHS analysis was performed with 1000 realizations; sensitivity and convergence were evaluated; and the influence of slope height and angle was analysed.

2. Methods

2.1. Input data and overall research framework

The study was based on actual geological-structural and geomechanical data from a deposit located in the East Kazakhstan Region of the Republic of Kazakhstan. The input information included discontinuity orientations, RQD values, fracture frequency, roughness parameter J_r and alteration/infill parameter J_a , rock mass strength and deformability characteristics, as well as calculation projects prepared in the Dips, RSDData, and Slide2 software packages. The name of the deposit is not disclosed due to confidentiality requirements. The main calculation model was represented by a two-dimensional slope profile 51.8 m high with an overall slope angle of 44°.

The overall research framework was structured as a sequential transition from the initial geological-structural data to stability assessment and engineering interpretation of the results. It included statistical processing of structural data and identification of dominant discontinuity sets; determination of Q' and GSI; kinematic analysis of possible failure mechanisms; calibration of strength parameters using the Hoek-Brown and Mohr-Coulomb criteria; deterministic and probabilistic stability analyses; assessment of sensitivity and convergence of the probabilistic calculations; and analysis of the influence of slope geometry. The key principle was parameter traceability: each numerical value used in the analysis had to be linked either to the original observations or to an explicitly defined modelling assumption.

For clarity, the individual procedures were grouped into five major stages: collection and systematization of input data; rock mass processing and parameterization; kinematic and geomechanical analysis; stability modelling and uncertainty analysis; and interpretation of the results and development of engineering recommendations (Fig. 1).

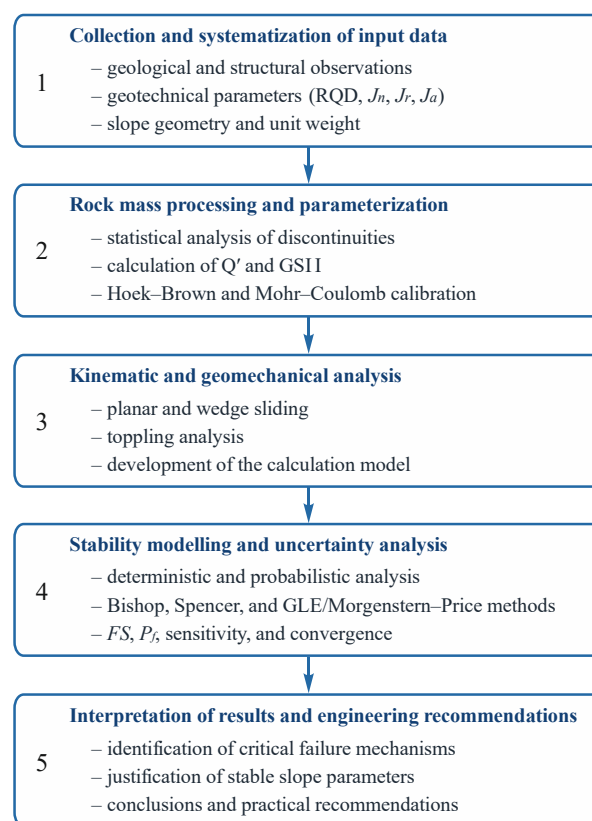


Figure 1. Sequence of the integrated open-pit slope stability assessment

2.2. Processing of structural data and rock classification

The spatial orientations of discontinuities were processed in Dips using a lower-hemisphere equal-area projection. To reduce orientation bias arising during scanline surveys because discontinuities of different orientations have unequal probabilities of being recorded, Terzaghi weighting was applied [5]. After quality control and correction, 436 discontinuity orientations were included in the analysis. Pole density was evaluated using Fisher contouring. The dominant discontinuity sets were identified from statistically distinct clusters, followed by an engineering assessment of their consistency.

Discontinuity spacing was analysed for the identified sets. Sample statistics and distribution characteristics were determined to account for differences in the degree of rock mass fragmentation and the potential size of the blocks formed. RQD, fracture frequency, the joint roughness number J_r , and the joint alteration number J_a were also analysed. These parameters were used as input variables for classification-based assessment of the rock mass condition.

For the integral classification, the modified Q' index was used, incorporating the structural characteristics obtained from mapping data:

$$Q' = \left(\frac{RQD}{J_n} \right) \cdot \left(\frac{J_r}{J_a} \right), \quad (1)$$

where:

RQD – rock quality designation, %;

J_n – joint set number;

J_r – joint roughness number;

J_a – joint alteration number.

For conditions corresponding to four or more discontinuity sets, J_n was taken as 15.

Based on Q' , the Geological Strength Index (GSI) was determined using the empirical relationship:

$$GSI = 9 \ln(Q') + 44. \quad (2)$$

Q' and GSI were used as integral indicators of the average structural condition of the rock mass. However, they were not regarded as substitutes for analysis of the spatial variability of RQD, J_n , and discontinuity characteristics.

2.3. Kinematic analysis

Kinematic analysis was performed in Dips to identify geometrically feasible structurally controlled failure mechanisms (Fig. 2).

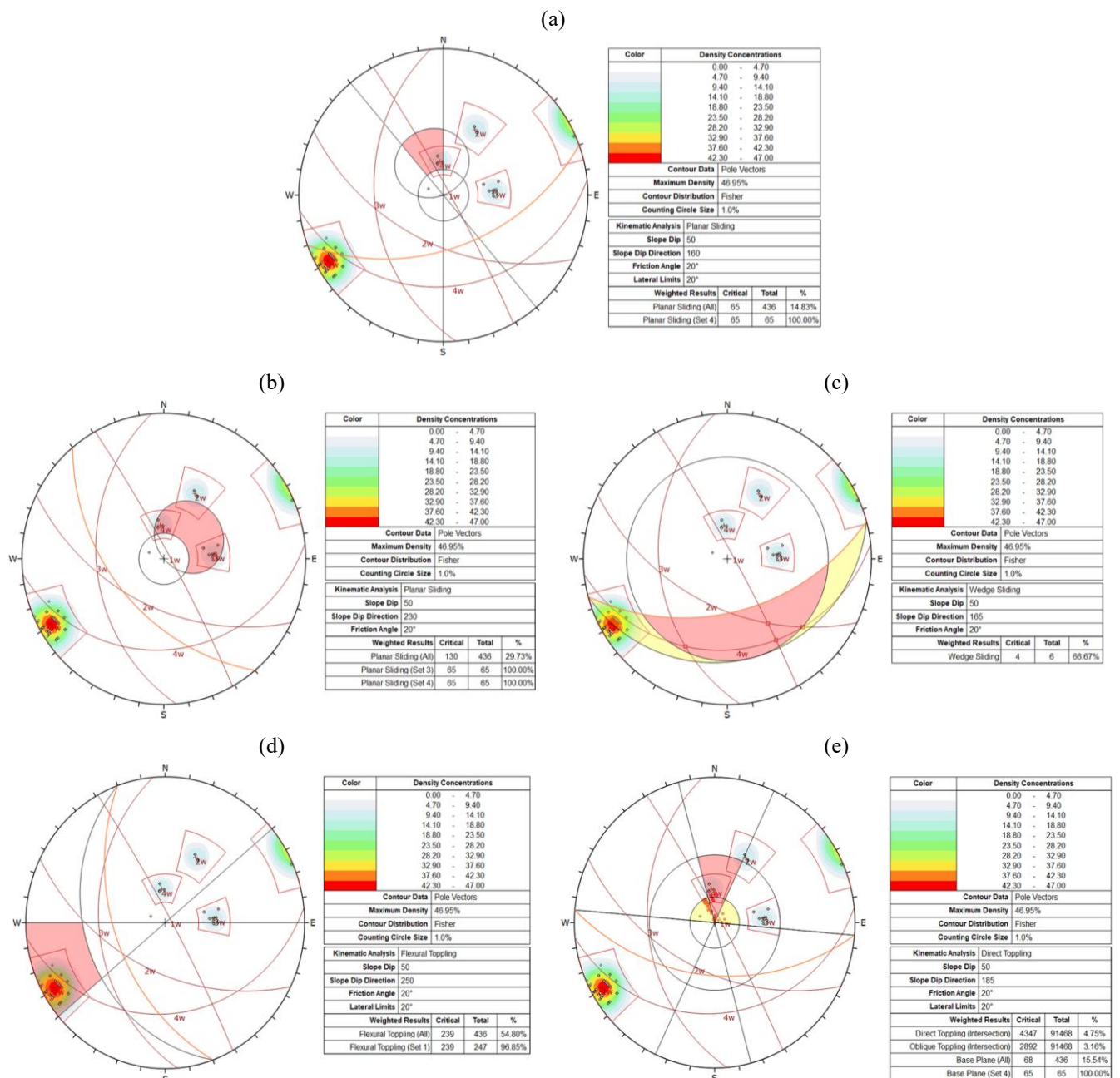


Figure 2. Main kinematic analysis configurations: (a) planar sliding; (b) planar sliding without lateral limits; (c) wedge sliding; (d) flexural toppling; (e) direct and oblique toppling

Planar and wedge sliding, as well as flexural, direct, and oblique toppling, were considered. For the directional analysis, the slope angle was taken as 50° , the friction angle along discontinuity surfaces as 20° , and the lateral limits as $\pm 20^\circ$. The dip direction of the slope was varied over the full range, which made it possible to identify the most unfavorable orientations relative to the recognized discontinuity sets.

Planar sliding was considered kinematically feasible if the plane dipped toward the slope, its dip angle exceeded the friction angle but remained lower than the slope angle, and its dip direction fell within the specified lateral limits. In addition, a No Limits option was considered, in which the lateral constraint was removed to obtain a more conservative assessment of geometric susceptibility to planar sliding.

For the wedge mechanism, the lines of intersection of pairs of mean discontinuity sets were analyzed. Wedge sliding was considered geometrically feasible if the line of intersection daylighted toward the slope and its plunge lay between the friction angle and the slope face angle.

Flexural toppling was evaluated based on the poles of steeply dipping planes oriented opposite to the slope dip direction. Direct and oblique toppling were analyzed with consideration of block geometry and the position of the basal plane. The results of the kinematic assessment were expressed as the proportion of weighted poles or intersection lines falling within the corresponding critical region. Because the number of input elements differed among the various mechanisms, the resulting percentages were neither summed nor interpreted as a single probabilistic scale.

2.4. Calibration of strength parameters

The rock mass strength parameters were calibrated using RSData. The nonlinear relationship between the effective principal stresses at failure was described by the generalized Hoek-Brown criterion [14]:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left(m_b \frac{\sigma'_1}{\sigma_{ci} + s} \right)^a, \quad (3)$$

where:

σ'_1, σ'_3 – effective principal stresses at failure;

σ_{ci} – uniaxial compressive strength of the intact rock;

m_b, s and a – Hoek-Brown parameters for the rock mass.

For the initial model, $\sigma_{ci} = 9.323$ MPa, $m_b = 3.433$, $s = 0.00109371$, and $a = 0.504$ were used. Young's modulus was taken as 2508.925 MPa and Poisson's ratio as 0.30. These parameters served as the initial basis for subsequent comparison with the calibrated model.

To compare the nonlinear Hoek-Brown envelope with the parameters used in limit equilibrium calculations, equivalent linearization was performed using the Mohr-Coulomb criterion to determine cohesion c and the internal friction angle ϕ . In interpreting the equivalent parameters, the principal stress range used for linearization was taken into account, because the values of c and ϕ depend on the adopted $\sigma_{3\max}$.

In the main Slide2 stability model, effective rock mass strength parameters at the relevant scale were taken from the original calculation project. The RSData results were used to analyse the shape of the strength envelope, compare the initial and calibrated models, and assess the admissible range for transferring equivalent Mohr-Coulomb parameters into the stability calculations.

2.5. Deterministic and probabilistic stability analysis

Stability analysis was performed in Slide2 for the main two-dimensional slope model with a height of 51.8 m and an overall slope angle of 44° . The unit weight of the rock mass was taken as 19 kN/m^3 , the effective cohesion as 12 kPa, and the internal friction angle as 35° . A dry static scenario was considered: no groundwater table was specified, the coefficient r_u was taken as 0, and no seismic coefficients were applied. The search for the potential failure surface was performed using the Global Minimum option. Failure was defined by the condition $FS < 1.0$.

For cross-checking the results, the simplified Bishop method [36], the Spencer method [37], and the GLE method with the Morgenstern-Price interslice force function [38] were used. The Bishop method satisfies moment equilibrium and was applied to circular slip surfaces, whereas the Spencer and GLE/Morgenstern-Price methods satisfy both force and moment equilibrium. Comparison of the results obtained by the three methods was used to assess the sensitivity of the stability conclusion to the adopted interslice force formulation.

The probabilistic analysis included three random variables: cohesion c , internal friction angle ϕ , and unit weight γ . Cohesion was assigned a normal distribution with a mean of 12 kPa, a standard deviation of 1 kPa, and limits of 9-15 kPa. The internal friction angle was described by a normal distribution with a mean of 35° , a standard deviation of 3° , and limits of $26-44^\circ$. Unit weight was assigned a normal distribution with a mean of 19 kN/m^3 , a standard deviation of 0.5 kN/m^3 , and limits of $17.5-20.5 \text{ kN/m}^3$. A negative correlation of $\rho = -0.5$ was specified between c and ϕ .

Combinations of random parameters were generated using Latin Hypercube Sampling [24], [26] with realizations. For each realization, the factor of safety FS was determined and the occurrence or non-occurrence of the failure condition was recorded. The resulting probabilistic indicators included the mean FS , standard deviation, minimum and maximum values, probability of failure P_f , and reliability index.

The main slope model and the field of trial slip surfaces are shown in Figure 3. The distribution parameters of the random variables and the conditions adopted for the probabilistic analysis are summarized in Table 1.

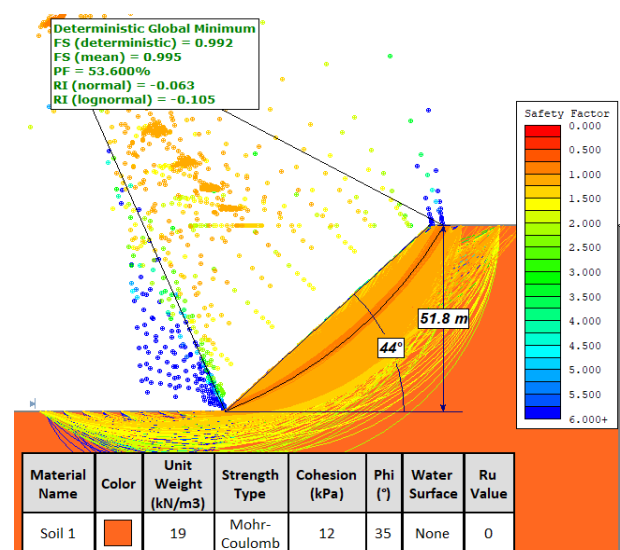


Figure 3. Main calculation model with a height of 51.8 m and an overall slope angle of 44°

Table 1. Parameters of the probabilistic analysis for the main model

Random variable	Cohesion (c), kPa	Friction angle (ϕ), °	Unit weight (γ), kN/m ³
Distribution	Normal		
Mean	12	35	19
Standard deviation	1	3	0.5
Limits	9-15	26-44	17.5-20.5

2.6. Sensitivity, convergence, and slope geometry analysis

The sensitivity of the factor of safety FS to variations in the random input parameters was evaluated using plots of FS versus the corresponding variables. For the strongest relationship, the correlation coefficient and linear regression parameters generated within the Slide2 probabilistic analysis were additionally considered. This approach was used to identify the parameters exerting the greatest influence on the variability of the factor of safety. The actual values of the correlation coefficient and regression parameters are presented later in the Results section.

The statistical stability of the probabilistic analysis was evaluated from the convergence of the probability of failure P_f and the standard deviation of FS as the number of realizations increased. For this purpose, the corresponding convergence trends were analysed as a function of the current Latin Hypercube Sampling sample size. The final number of realizations was $N = 1000$.

To assess the influence of controllable geometric parameters, an additional series of saved Slide2 projects with slope angles of 35, 40, 45, and 50° and heights of 20-60 m was analysed. Only scenarios for which interpretation results were available in the project files were included in the comparative dataset. A common baseline parameterization was used for all scenarios: $c = 7$ kPa, $\phi = 35^\circ$, and $\gamma = 19$ kN/m³. Normal distributions were assigned to the random parameters, with standard deviations of 1 kPa, 3°, and 0.5 kN/m³, respectively; the limits were set at ± 3 standard deviations. The correlation between c and ϕ was $\rho = -0.5$, and the number of realizations was $N = 1000$.

This additional series was considered separately from the main 51.8 m/44° model because a different cohesion value was used. Its purpose was to assess the relative sensitivity of FS and P_f to changes in geometry while keeping the statistical parameters unchanged. For each scenario, the deterministic factor of safety, the mean probabilistic FS , and the probability of failure P_f were compared. The influence of height was analysed at fixed slope angles of 35 and 40°, whereas the influence of slope angle was examined for 20 m high profiles by increasing the angle from 45 to 50°.

3. Results and discussion

3.1. Structural characteristics and rock classification

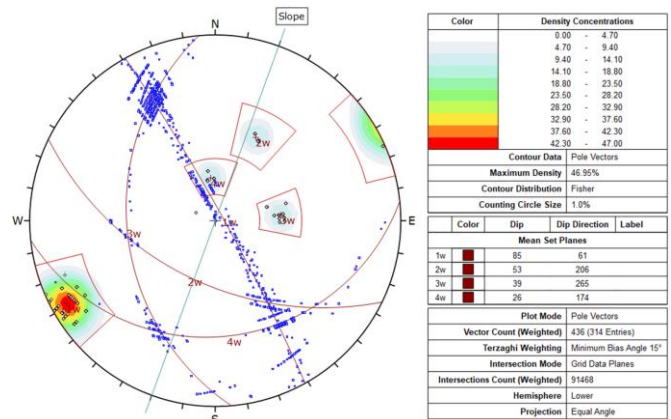
Processing of 436 discontinuity orientations identified four dominant sets with mean orientations of 85/61°, 53/206°, 39/265°, and 26/174° in dip/dip-direction format (Table 2). Set 1 is characterized by an almost vertical orientation, Set 2 by a steep dip, Set 3 by a moderate dip, and Set 4 by a shallow dip. The maximum pole density reached 46.95%, indicating pronounced structural anisotropy of the rock mass.

The stereographic projection shows the spatial separation of the identified clusters and confirms the presence of four persistent discontinuity sets (Fig. 4).

Table 2. Mean orientations of the dominant discontinuity sets

Set	Dip, °	Dip direction, °	Characterization
Set 1	85	61	Almost vertical
Set 2	53	206	Steeply dipping
Set 3	39	265	Moderately dipping
Set 4	26	174	Shallow dipping

Their engineering significance differs: the nearly vertical Set 1 creates conditions favourable for toppling, Sets 2 and 3 may contribute to the formation of wedge blocks, whereas the shallow-dipping Set 4 may be unfavourable for the day-lighting of individual planes on the slope face when the slope is appropriately oriented.

**Figure 4. Stereographic projection of 436 discontinuity orientations showing the four dominant sets**

Analysis of discontinuity spacing also revealed substantial differences among the sets. The mean true spacing was 0.085 m for Set 1, 0.135 m for Set 2, and 0.328 m for Set 3; for Set 4, the mean apparent spacing was 0.410 m. For the first two sets, the spacing distributions were approximated by a lognormal distribution, whereas Sets 3 and 4 exhibited more pronounced asymmetry and a wider range of values. These differences indicate varying degrees of rock mass discontinuity: smaller spacing promotes the formation of a more finely blocky structure, whereas larger spacing may control the size of potentially unstable blocks.

RQD values ranged from 11 to 100%, with a mean of 55.04% and a standard deviation of 21.52%. The mean fracture frequency was 8.97 m⁻¹, with a standard deviation of 2.51 m⁻¹ and a range of 1-12 m⁻¹. This variability indicates pronounced heterogeneity in the structural condition of the rock mass and demonstrates the limitations of describing it using a single mean value. Zones with RQD < 25% may correspond to intensely fractured or disturbed rock, whereas RQD > 75% characterizes relatively more intact zones.

The distributions of J_r and J_a indicated a predominance of moderately rough and slightly altered discontinuity surfaces. At the same time, individual discontinuities were characterized by low J_r values and elevated J_a values, which may substantially reduce shear resistance under unfavourable orientation and sufficient spatial connectivity. Therefore, the mean classification indices were considered together with locally unfavourable combinations of these parameters.

Based on the mean values of RQD = 55.04%, $J_r = 2$, $J_a = 2$, and the adopted $J_n = 15$, a Q' value of 3.67 was obtained. The corresponding Geological Strength Index was

GSI = 55.7. These indices characterize the average structural condition of the rock mass but do not represent the full range of its spatial variability. In particular, the wide range of RQD values and the presence of locally unfavourable J_r and J_a values indicate that Q' and GSI should be treated as integral baseline indicators rather than as evidence of uniformly stable rock mass conditions.

Overall, the structural classification results confirm the need to combine rock mass rating indices with spatial kinematic analysis. Average rock mass quality alone does not determine the failure mechanism, because its development depends strongly on the relative orientation of the discontinuity sets and the slope surface. Accordingly, the structural characteristics were subsequently considered in the context of planar sliding, wedge sliding, and toppling mechanisms.

3.2. Results of kinematic analysis

The kinematic analysis showed that the potential failure mechanism depends strongly on the orientation of the slope relative to the identified discontinuity sets. For planar sliding at a slope dip direction of 160°, 65 of the 436 weighted poles satisfied the kinematic criteria, corresponding to 14.83%. All critical elements in this case belonged to Set 4, confirming its dominant role in the development of planar sliding for this slope orientation.

When the slope dip direction was changed to 230° and the lateral limits were removed in the No Limits analysis, the number of critical elements increased to 130, or 29.73%. In this case, Sets 3 and 4 fell entirely within the critical region. The most unfavourable sectors for planar sliding correspond-

ded approximately to directions of 165-185° and 220-275°, which is consistent with the mean dip directions of these sets.

For wedge sliding, analysis of the six possible intersection lines formed by the mean discontinuity sets showed that four fell within the critical region. Thus, 66.67% of the mean intersections were kinematically feasible. However, this percentage refers only to the limited number of intersection lines between mean discontinuity sets and is not statistically comparable with the proportion of critical poles calculated from the 436 individual orientations. The highest susceptibility to wedge sliding was observed for directions of approximately 140-190°. Increasing the slope angle expanded the region in which wedge daylighting was possible, whereas increasing the friction angle reduced the critical region.

Flexural toppling was the most widespread mechanism in terms of the number of individual orientations. A total of 239 of the 436 poles satisfied the kinematic criteria, corresponding to 54.80%. For Set 1, the proportion of critical poles reached 96.85%, which is associated with its nearly vertical orientation. The highest susceptibility to flexural toppling was observed predominantly within the 230-270° sector, i.e. in directions opposite to the dip of the nearly vertical discontinuity set.

Direct and oblique toppling were considerably less pronounced. The proportions of critical combinations were 4.75% and 3.16%, respectively, whereas the corresponding value for the basal plane was 15.54% (Table 3). These values indicate that these mechanisms are more localized compared with the more widely represented flexural toppling mechanism.

Table 3. Summary results of the kinematic analysis

Mechanism	Input dataset	Critical result	Engineering interpretation
Planar sliding	436 poles	65 (14.83%)	Set 4; direction 160°
Planar sliding, No Limits	436 poles	130 (29.73%)	Sets 3-4; direction 230°
Wedge sliding	6 mean intersections	4 (66.67%)	Unfavourable sector 140-190°
Flexural toppling	436 poles	239 (54.80%)	Set 1: 96.85%
Direct toppling	Block pairs/basal plane	4.75%	Local mechanism
Oblique toppling	Block pairs/basal plane	3.16%	Local mechanism
Basal plane	Block pairs/basal plane	15.54%	Potential base for rotation

The combined interpretation of the results shows that changes in slope orientation can alter the dominant failure mechanism even when the strength properties of the rock mass remain unchanged. For directions of approximately 160-190°, conditions favourable for planar and wedge sliding become more pronounced, whereas within the 230-270° sector the role of flexural toppling increases. Therefore, slope design should account not only for the overall slope angle but also for the spatial orientation of individual slope sectors relative to the dominant discontinuity sets.

3.3. Results of strength parameter calibration

Calibration in RSDData resulted in substantial changes in the parameters describing the strength and deformability of the rock mass. For the initial model, $\sigma_{ci} = 9.323$ MPa, $m_b = 3.433$, $s = 0.00109$, and $a = 0.504$, were used, whereas the calibrated model yielded $\sigma_{ci} = 15.601$ MPa, $m_b = 5.073$, $s = 0.12110$, and $a = 0.501$. Young's modulus increased from 2508.925 to 22678.292 MPa, while Poisson's ratio changed from 0.30 to 0.25. Equivalent linearization using the Mohr-Coulomb criterion also showed a substantial difference between the two models. For the initial model, cohesion was

3.626 MPa at an internal friction angle of 15.448°, whereas the calibrated model yielded $c = 0.812$ MPa and $\varphi = 57.973^\circ$. A comparison of the Mohr circles and the corresponding strength envelopes is presented in Figure 5.

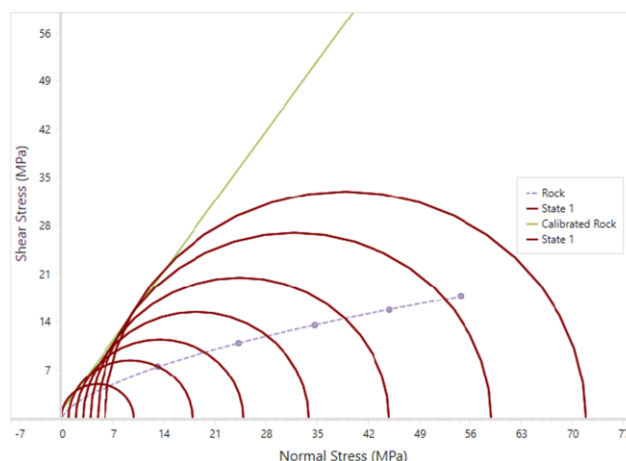


Figure 5. Mohr circles and strength envelopes for the initial and calibrated models

A complete comparison of the parameters of the initial and calibrated models is presented in Table 4.

Table 4. Parameters of the initial and calibrated RSData models

Parameter	Initial model	Calibrated model
σ_{ci} , MPa	9.323	15.601
m_b	3.433	5.073
s	0.00109	0.12110
a	0.504	0.501
c , MPa	3.626	0.812
φ , °	15.448	57.973
Tensile strength, MPa	0.003	0.372
Rock mass UCS, MPa	0.300	5.423
Global strength, MPa	2.284	6.180
E , MPa	2508.925	22678.292
ν	0.30	0.25
σ_{3max} , MPa	40.0	0.035

The resulting differences between the Mohr-Coulomb parameters cannot be interpreted as an unambiguous increase or decrease in rock mass strength. In the initial model, the high cohesion value of $c = 3.626$ MPa was combined with a relatively low internal friction angle of $\varphi = 15.448^\circ$, whereas the calibrated model was characterized by lower cohesion, $c = 0.812$ MPa, and a substantially higher friction angle, $\varphi = 57.973^\circ$. Because the corresponding linear envelopes have different slopes and intercepts on the shear-strength axis, their relative positions depend on the level of normal stress. At low stresses, the initial model may provide higher shear resistance because of its greater cohesion, whereas with increasing normal stress the contribution of the higher friction angle in the calibrated model becomes more significant.

A fundamentally important difference is the range of σ_{3max} used for equivalent linearization. For the initial model, $\sigma_{3max} = 40.0$ MPa, whereas for the calibrated model this parameter is only 0.035 MPa. Consequently, the values $c = 0.812$ MPa and $\varphi = 57.973^\circ$ characterize the equivalent linear envelope specifically within a narrow low-stress range. Their direct application outside this range without repeated linearization may result in an incorrect strength assessment.

The change in the calculated deformation modulus from 2.51 to 22.68 GPa also indicates a substantial difference between the initial and calibrated parameterizations. The modulus E does not directly affect the factor of safety in the LEM formulation used in this study; however, it is important for subsequent deformation assessment and potential FEM verification. Therefore, the RSData results were used not for the direct transfer of all derived parameters into Slide2, but for comparison of the strength envelopes, assessment of the relevant stress range, and evaluation of the applicability of the equivalent Mohr-Coulomb parameters.

3.4. Deterministic and probabilistic stability assessment of the main profile

For the main profile with a height of 51.8 m and an overall slope angle of 44° , the deterministic global minimum obtained using the simplified Bishop method was $FS = 0.992$. Thus, even under the deterministic formulation, the analysed profile is close to the limit equilibrium condition of $FS = 1.0$. The probabilistic analysis confirmed this conclusion: the mean FS values obtained using the different methods ranged from 0.9896 to 0.9947 (Table 5).

Table 5. Probabilistic results for the main model at $N = 1000$

Indicator	Simplified Bishop	Spencer	GLE/Morgenstern-price
Mean FS	0.9947	0.9898	0.9896
Standard deviation	0.08501	0.08522	0.08490
Minimum	0.7693	0.7633	0.7638
Maximum	1.285	1.281	1.281
P_f , %	53.6	56.1	57.8

The differences among the three calculation methods were small. The maximum difference between the mean FS values was 0.0051, while the standard deviations were nearly identical. This indicates that the overall conclusion regarding the limiting state of the analysed profile is only weakly dependent on the adopted interslice force formulation. The Bishop method yielded a slightly higher mean FS , whereas the GLE/Morgenstern-Price method produced the lowest value.

The probability of failure P_f ranged from 53.6 to 57.8%. The Bishop method yielded the lowest P_f , whereas the GLE/Morgenstern-Price method produced the highest. The difference of 4.2 percentage points does not alter the principal conclusion: for all three methods, more than half of the probabilistic realizations satisfy the condition $FS < 1.0$. Unlike a single deterministic value, the probabilistic analysis demonstrates the spread of the factor-of-safety distribution: individual realizations reach approximately 1.28, while a substantial proportion of the sample lies below unity.

For the FS distribution obtained using the simplified Bishop method, the gamma distribution provided the best fit. A mean value of 0.9947 and a standard deviation of 0.08501 correspond to a coefficient of variation of about 8.55%. The negative reliability index is associated with the mean FS lying slightly below the threshold value of 1.0. Therefore, even moderate variability in the input parameters leads to $P_f > 50\%$, because the centre of the distribution is located very close to the failure threshold. Figure 6 shows the distribution of the factor of safety obtained using the simplified Bishop method, with the realizations satisfying the condition $FS < 1.0$ highlighted.

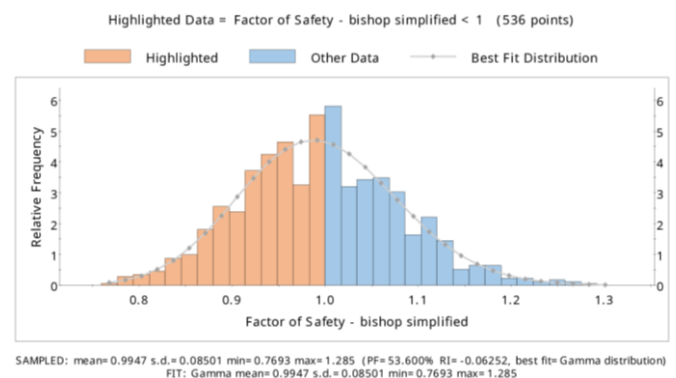


Figure 6. Distribution of FS obtained using the simplified Bishop method (realizations with $FS < 1.0$ are highlighted in orange)

The actual characteristics of the generated cohesion sample were consistent with the adopted probabilistic formulation. The sample mean was 12 kPa, the standard deviation was 0.987 kPa, the minimum value was 9.08 kPa, and the maximum value was 14.84 kPa. Unstable realizations were present throughout the entire range of cohesion values considered, although their proportion increased at lower c values.

Thus, cohesion affects stability, but by itself does not determine the transition to failure. The cohesion distribution and the positions of unstable realizations are shown in Figure 7.

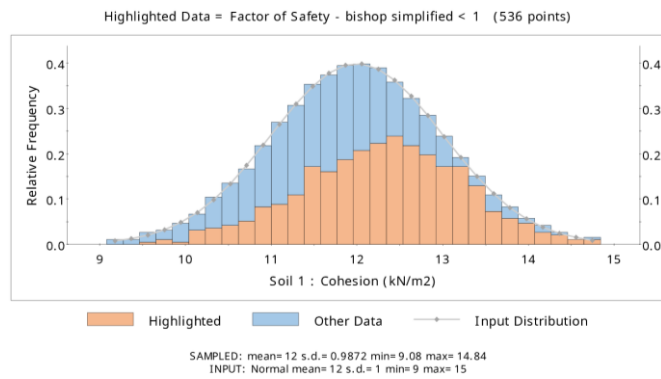


Figure 7. Cohesion distribution and identification of unstable realizations

The strongest relationship was observed between the factor of safety and the internal friction angle. For the $FS-\varphi$ relationship, the correlation coefficient was $r = 0.986294$, while the linear regression coefficient was $\beta = 0.028315$. Within the investigated range, an increase in φ by 1° was associated with an average increase in FS of approximately 0.028-0.03. The $FS = 1.0$ threshold was crossed at φ of approximately $35-36^\circ$; therefore, even a small change in the mean value or variability of the internal friction angle can substantially alter the proportion of realizations falling below the stability threshold. The relationship between FS and the internal friction angle, together with the corresponding regression line, is shown in Figure 8.

The convergence analysis showed that the most pronounced fluctuations in P_f occurred during the initial part of the sampling process, approximately within the first 100-150 realizations. As the number of realizations increased, P_f gradually stabilized near its final value of 53.6%. The standard deviation of FS reached a relatively stable plateau after approximately 300-400 realizations, while beyond 600 realizations changes in both parameters were minor. Thus, the adopted sample size of $N = 1000$ provided sufficient statistical stability of the results for the input parameter distributions specified in this study. However, this convergence characterizes the stability of the numerical sample and does not eliminate uncertainty associated with the assumed probability distributions and their parameters. The convergence curves for the probability of failure and the standard deviation of FS are presented in Figure 9.

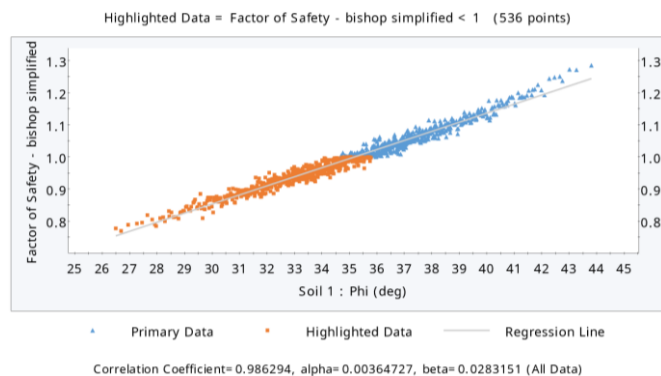


Figure 8. Relationship between FS and the internal friction angle with the corresponding linear regression line

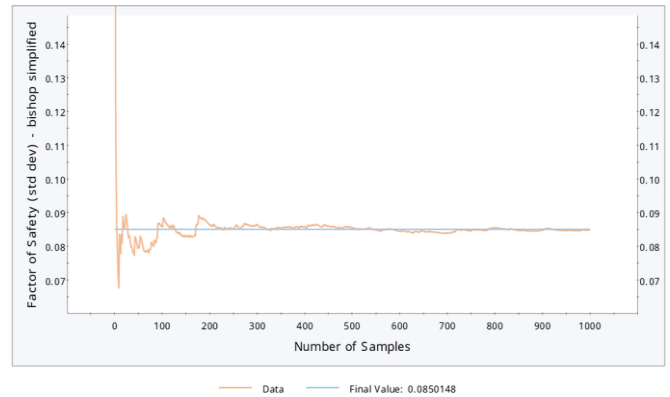
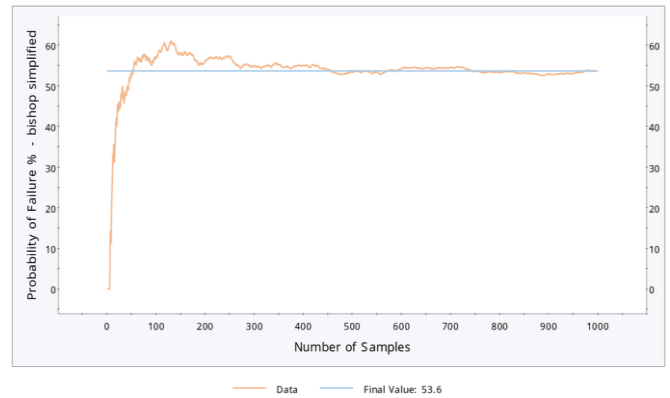


Figure 9. Convergence of the probability of failure and the standard deviation of FS with increasing number of LHS realizations

Taken together, the deterministic and probabilistic analyses indicate that the main 51.8 m/44° profile is close to the limit state. The probabilistic formulation provides a more comprehensive assessment of stability because it shows not only the position of the mean FS relative to the critical value, but also the proportion of possible realizations corresponding to failure. The results also demonstrate that, for the adopted parameterization, uncertainty in the internal friction angle is particularly important, as variations in φ have the strongest effect on the FS distribution and, consequently, on P_f .

3.5. Influence of slope geometry on FS and P_f

The additional series of calculations demonstrated a pronounced influence of slope geometry on both the mean factor of safety FS and the probability of failure P_f . The response of P_f to changes in geometry was strongly nonlinear and depended on the position of the FS distribution relative to the threshold value of 1.0. A summary of the analysed scenarios is presented in Table 6.

At a slope angle of 35° , increasing the height from 30 to 50 m was accompanied by a decrease in the mean FS from 1.287 to 1.211, i.e. by approximately 5.9%. At the same time, P_f increased from 0.3 to 2.5%. Despite the decrease in the factor of safety, the mean FS remained above 1.2 in all three scenarios, and the probability of failure remained low. This indicates a noticeable stability margin for the gentler slope geometry even with increasing slope height.

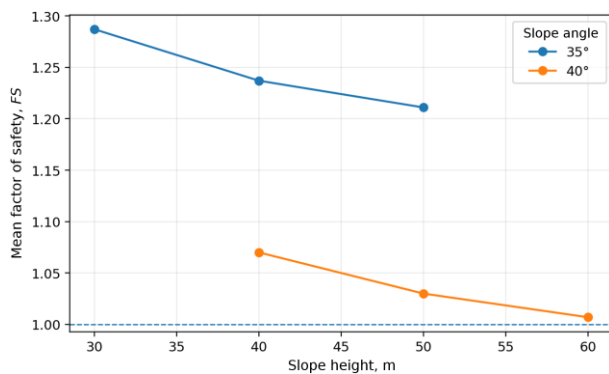
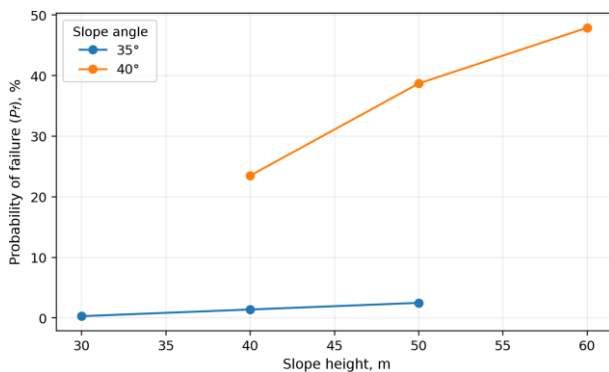
A different pattern was observed at a slope angle of 40° . Increasing the height from 40 to 60 m reduced the mean FS from 1.070 to 1.007, again by approximately 5.9%, whereas P_f increased much more strongly, from 23.5 to 47.9%.

Table 6. Summary results of the additional series of geometric scenarios

Slope angle, °	Height, m	Deterministic FS	Mean FS	P_f , %
35	30	1.282	1.287	0.3
35	40	1.233	1.237	1.4
35	50	1.207	1.211	2.5
40	40	1.066	1.070	23.5
40	50	1.026	1.030	38.7
40	60	1.002	1.007	47.9
45	20	1.058	1.061	23.9
50	20	0.912	0.915	90.0

Thus, nearly the same relative reduction in mean FS at slope angles of 35 and 40° was accompanied by substantially different responses in the probability of failure. This is related to the position of the distribution relative to the $FS = 1.0$ threshold: at 35°, its centre remains sufficiently far from the failure boundary, whereas at 40° it approaches that boundary, so that even a small shift in the distribution sharply increases the area of its left tail below the critical value. Figure 10 shows the change in mean FS with increasing slope height at slope angles of 35 and 40°.

The corresponding change in the probability of failure is shown in Figure 11.

**Figure 10. Change in mean FS with slope height at slope angles of 35 and 40°****Figure 11. Change in P_f with slope height at slope angles of 35 and 40°**

The influence of slope angle was even more pronounced. At a fixed height of $H = 20$ m, increasing the slope angle from 45 to 50° reduced the mean FS from 1.061 to 0.915, i.e. by 13.8%, while P_f increased from 23.9 to 90.0%. Within the analysed series, the change in slope angle therefore had a stronger effect on stability than increasing the slope height by 20 m. However, this conclusion applies specifically to the

adopted parameterization of $c = 7$ kPa, $\varphi = 35^\circ$, and $\gamma = 19$ kN/m³ under the dry calculation scenario; for other strength parameters, the quantitative limits may differ.

Comparison of the deterministic and mean probabilistic values shows that they are close for all analysed scenarios, whereas P_f changes much more strongly and nonlinearly. For example, for the 40°/50 m profile, the deterministic factor of safety is $FS = 1.026$, formally exceeding unity, while the probability of failure reaches 38.7%. Therefore, assessment of slope geometry based on a single deterministic FS value may conceal a substantial level of risk, particularly when the factor of safety is close to unity.

The obtained relationships confirm that geometric parameters should be assessed jointly with probabilistic stability characteristics. Reducing the slope angle can provide a more pronounced increase in stability than a moderate reduction in slope height; however, the practical selection of slope geometry should account not only for FS and P_f but also for technological constraints and the volume of overburden removal. For design comparison, it is advisable to consider a set of “angle-height-berm” alternatives and select the preferred solution using combined deterministic and probabilistic stability criteria. At the same time, geomechanical solutions should be coordinated with the technological scheme of open-pit mining, including the parameters of the mine haulage system and transfer points that directly affect mine productivity [39].

3.6. Integrated interpretation and practical implications

The combined interpretation of the obtained results makes it possible to distinguish three interrelated levels of risk assessment. The first level characterizes structural susceptibility and identifies which failure mechanisms are kinematically feasible for a given slope orientation. The second level reflects the strength capacity of the rock mass, defined by the shape of the strength envelope and the effective parameters within the relevant stress range. The third level characterizes the reliability of the overall slope geometry and shows how variability in strength parameters and geometric conditions is transformed into the FS distribution and the probability of failure P_f . These levels are not interchangeable and should be considered jointly.

For the investigated rock mass, two structurally controlled scenarios are particularly important. The first is associated with toppling along the nearly vertical Set 1, whereas the second involves planar and wedge sliding controlled mainly by Sets 3 and 4. Therefore, stability management should begin with spatial zoning of slope sectors according to the orientation of the dominant discontinuity sets. Where changes in slope orientation are not feasible, local flattening of the slope, increasing berm width, removal of potentially unstable blocks, rock bolting, or mesh retention may be considered depending on the scale and nature of the expected failure mechanism.

The probabilistic assessment shows that the main profile with a height of 51.8 m and an overall slope angle of 44° does not have a pronounced stability margin even under the adopted dry static scenario. The mean FS values are close to unity, while P_f exceeds 50% for all limit equilibrium methods used. Because groundwater effects and seismic loading were not included in this formulation, the obtained result should not be regarded as conservative for the full range of possible operating conditions. Priority directions for model

refinement include determination of the actual slope geometry and groundwater level, further refinement of the c and φ parameters, and monitoring of discontinuity opening and rock mass displacements [40]. When refining the hydrogeological formulation, attention should be paid not only to the groundwater level, but also to lithological heterogeneity and the water-physical properties of the overburden rocks, which control infiltration and seepage conditions [41].

The parametric analysis further shows that changes in slope geometry can substantially affect the probabilistic stability assessment. Within the analysed range, reducing the slope angle produced a more pronounced effect than a moderate reduction in slope height. However, such a decision cannot be evaluated solely from a geomechanical perspective, because changes in slope angle directly affect the volume of overburden removal and the technological configuration of the slope. Design selection should also consider the efficiency of disturbed-land use and subsequent reclamation [42]. Therefore, it is advisable to develop an “angle-height-berm” matrix of alternatives and compare them simultaneously in terms of deterministic FS , P_f , and operational feasibility.

The obtained results have several limitations. The main Slide2 model is homogeneous and two-dimensional and does not reproduce spatial variations in RQD or the orientation of individual discontinuity sets. The strength parameters are represented as random variables without the use of random fields; therefore, correlation length and localization of weak zones are not considered. The analysis was performed only for a dry static scenario, while the kinematic criteria do not account for the finite persistence and connectivity of individual discontinuities. In addition, the RSDData calibration was performed over a limited σ_3 range, which requires caution when transferring the resulting equivalent parameters to other stress levels.

Further development of the model should include the sequential consideration of hydrogeological scenarios with a specified piezometric surface or k_h , pseudostatic loading at different k_h values, and a spatial model in which strength domains and discontinuity orientations are linked to the actual geological structure. Additional verification may be achieved by comparing the LEM results with FEM/SSR analyses and displacement monitoring data [43], [44].

Thus, practical slope stability assessment should jointly consider the spatial orientation of discontinuities, the relevant strength envelope, and the FS distribution rather than relying solely on a single deterministic factor of safety. Further model development should include groundwater conditions, seismic loading, actual bench geometry, and spatial variability of rock mass properties.

4. Conclusions

An integrated methodology for open-pit slope stability assessment was developed and applied, providing a traceable transition from the processing of 436 discontinuity orientations and Q' -GSI rock mass classification to kinematic analysis, Hoek-Brown/Mohr-Coulomb calibration, deterministic and probabilistic LEM stability assessment, and analysis of the influence of slope geometry.

Four dominant discontinuity sets were identified, with mean orientations of 85/61°, 53/206°, 39/265°, and 26/174°. Mean values of RQD = 55.04%, $Q' = 3.67$, and GSI = 55.7 characterize the overall structural condition of the rock mass,

whereas the wide RQD range of 11-100% indicates pronounced heterogeneity.

The kinematic analysis identified 14.83-29.73% critical poles for planar sliding, four of six critical intersection lines of the mean discontinuity sets for the wedge mechanism, and 54.80% critical poles for flexural toppling. The most unfavourable directions depend on the failure mechanism and are concentrated mainly within the 160-190° and 230-270° sectors.

The equivalent Mohr-Coulomb parameters obtained for the calibrated RSDData model were $c = 0.812$ MPa and $\varphi = 57.973^\circ$ at $\sigma_{3\max} = 0.035$ MPa. These values characterize a local linearization within a low-stress range and should not be transferred to other stress levels without repeated linearization.

For the main profile with a height of 51.8 m and an overall slope angle of 44°, the mean FS ranged from 0.9896 to 0.9947, while the probability of failure P_f ranged from 53.6 to 57.8%. The close agreement between the Bishop, Spencer, and GLE/Morgenstern-Price results indicates that the overall conclusion regarding the profile condition is only weakly dependent on the adopted interslice force formulation.

The strongest statistical relationship with FS was established for the internal friction angle, with $r = 0.9863$ and a linear regression coefficient of $\beta \approx 0.0283$ $FS/^\circ$. The convergence analysis showed stabilization of the main probabilistic characteristics as the sample size increased; after approximately 400-600 realizations, further changes became minor. Therefore, $N = 1000$ provided sufficient statistical stability for the adopted input parameter distributions.

The additional series of calculations revealed a nonlinear response of P_f to changes in slope geometry. At a slope angle of 40°, increasing the height from 40 to 60 m increased P_f from 23.5 to 47.9%, whereas at $H = 20$ m increasing the slope angle from 45 to 50° increased P_f from 23.9 to 90.0%. Within the analysed scenarios, changes in slope angle had a stronger influence on stability than changes in slope height.

Author contributions

Conceptualization: NS; Data curation: ZT; Formal analysis: AI, GN; Funding acquisition: AM; Investigation: AM; Methodology: NS, TM; Project administration: AI, ZT; Resources: GN, NM; Software: AM, NS; Supervision: AI; Validation: NS, TM; Visualization: ZT; Writing – original draft: AM, NS, GN; Writing – review & editing: AI, TM. All authors have read and agreed to the published version of the manuscript.

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Conflicts of interest

The authors declare no conflict of interest.

Data availability statement

The original contributions presented in the study are included in the article, further inquiries can be directed to the corresponding author.

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Інтегрована оцінка стійкості борту кар'єру з урахуванням структурної анізотропії, невизначеності міцнісних параметрів і геометрії укосу

А. Мусін, А. Імашев, Н. Сулейменов, Г. Нуршайикова, З. Тунгушбасва, Т. Малгаждаров

Мета. Розробити та апробувати інтегровану методику оцінки стійкості борту кар'єру, що забезпечує простежуваний перехід від структурних даних тріщинуватого масиву до класифікації Q'-GSI, кінематичного аналізу, калібрування міцнісних параметрів, детерміністичної та ймовірнісної оцінки стійкості, а також кількісного оцінювання впливу геометрії укосу.

Методика. Структурні дані родовища Східно-Казахстанської області оброблено у програмному комплексі Dips із корекцією орієнтаційного зміщення за Терцагі та контурним аналізом Фішера. На основі RQD, J_n , J_r і J_a визначено Q' та GSI. Калібрування міцнісних параметрів виконано в RSData за критеріями Хоска-Брауна та Мора-Кулона. У Slide2 застосовано спрощений метод Бішопа, метод Спенсера та узагальнений метод граничної рівноваги (GLE) з функцією міжблокових сил Моргенштерна-Прайса.

Результати. За 436 орієнтаціями виділено чотири домінуючі системи тріщин і отримано значення величин $Q' = 3.67$ і $GSI = 55.7$. Для площинного ковзання частка кінематично допустимих елементів становила 14.83-29.73%, для гнучкого перекидання – 54.80%. Чотири з шести ліній перетину середніх систем задовольняли умовам клинового ковзання. Для профілю висотою 51.8 м і загальним кутом 44° середнє значення FS становило 0.9896-0.9947, а ймовірність втрати стійкості P_f – 53.6-57.8%. Найбільш виражену залежність FS встановлено для кута внутрішнього тертя ($r = 0.9863$). При куті укосу 40° збільшення висоти з 40 до 60 м підвищило P_f з 23.5 до 47.9%, а при $H = 20$ м збільшення кута з 45° до 50° – з 23.9 до 90.0%.

Наукова новизна. Запропоновано єдину послідовність оцінювання, що поєднує структурно-статистичне опрацювання даних, класифікацію Q'-GSI, кінематичний аналіз, локальне калібрування за критеріями Хоска-Брауна та Мора-Кулона, ймовірнісний аналіз методом граничної рівноваги та оцінювання впливу геометрії укосу. Показано, що кінематична схильність до структурно зумовлених механізмів руйнування та P_f характеризують різні рівні ризику й потребують спільної інтерпретації.

Практична значимість. Запропонована методика дає змогу виявляти несприятливі просторові орієнтації та геометричні сценарії, визначати параметри, що потребують першочергового уточнення, і оцінювати зміну стійкості одночасно за коефіцієнтом запасу FS та ймовірністю втрати стійкості P_f .

Ключові слова: борт кар'єру; тріщинуватий масив; стійкість укосу; геологічний індекс міцності; кінематичний аналіз; метод граничної рівноваги

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