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Research article

Back-Analysis and Slope Geometry Stabilization Recommendations for the Lowwall Pit 4 Interburden Area Seam B–C, IUP PT. Dizamatra Powerindo

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Keywords

Back-analysis

Sensitivity Analysis

Slope Stability

Morgenstern-Price

Degraded shear strength

Geometric redesign

Abstract

A lowwall slope failure occurred in the interburden area between Seams B and C of Pit 4 at the PT. Dizamatra Powerindo coal mine during January–February 2026. The failure was associated with high-intensity rainfall and inadequate drainage conditions and was characterized by tension cracks of approximately 100 m in length, ground displacement of 50–100 mm, and surface subsidence of 20–90 cm. This study aims to identify the material parameters controlling the observed slope instability, estimate degraded shear-strength parameters through back-analysis, and evaluate alternative slope geometries using the back-analyzed parameters. Two-dimensional slope stability analyses were conducted using the Morgenstern–Price limit equilibrium method and the Mohr–Coulomb failure criterion. Sensitivity analysis was first performed to identify the controlling material, followed by deterministic and probabilistic back-analysis to estimate the degraded shear-strength parameters under the adopted loading conditions. The initial model yielded a deterministic factor of safety (FoS) of 1.692 and a mean FoS of 1.707. Sensitivity analysis identified sandstone as the material exerting the greatest influence on slope stability. The back-analyzed failure condition resulted in a deterministic FoS of 0.914, a mean FoS of 0.926, and a probability of failure (PoF) of 62.40%. Under this condition, sandstone cohesion decreased from 57.00 to 32.76 kPa, while the internal friction angle decreased from 22.68° to 11.80°, corresponding to reductions of 42.53% and 47.97%, respectively. The back-analyzed parameters were subsequently applied to three alternative slope geometries. The deterministic FoS values for Options 1, 2, and 3 were 1.365, 1.350, and 1.307, respectively, while the corresponding mean FoS values were 1.375, 1.358, and 1.312. All alternatives yielded a PoF of 0.00% and satisfied the adopted minimum FoS criterion of 1.1. Option 3 was selected as the preferred geometry because it provided adequate stability while requiring the smallest cross-sectional cut area. The results demonstrate that sensitivity-guided back-analysis provides a practical basis for estimating degraded material strength and evaluating alternative lowwall geometries following observed slope failure.

1. INTRODUCTION

Slope stability is a critical geotechnical consideration in open-pit mining because slope

instability can affect operational safety, production continuity, and access to active mining areas. The stability of an excavated slope is controlled by the balance between driving forces and the shear resistance of the materials forming the slope. In conventional slope design, the shear strength parameters of the slope materials, particularly cohesion and internal friction angle, are commonly obtained from laboratory testing and subsequently incorporated into numerical or limit-equilibrium analyses. The Morgenstern–Price method combined with the Mohr–Coulomb failure criterion is widely applied to evaluate slope stability and determine the factor of safety (FoS) under specified loading and material conditions [1,2]. However, the use of laboratory-derived strength parameters may not always represent the mechanical condition of a rock mass at the time of field instability. Rock masses in open-pit mines are affected by geological discontinuities, weathering, excavation disturbance, environmental conditions, and variations in material properties. For instance, discontinuities such as fractures, joints, and bedding planes present in rock masses are known to significantly affect the overall strength properties and mechanical behavior of the structure, often necessitating advanced evaluation methods beyond conventional single-approach numerical modeling [3]. Consequently, a slope may exhibit instability even when the calculated FoS based on the original design parameters satisfies the adopted stability criterion. This discrepancy highlights the importance of evaluating the actual or degraded shear strength conditions associated with an observed failure rather than relying solely on the original design parameters.

This discrepancy highlights the importance of evaluating the actual or degraded strength conditions associated with an observed failure rather than relying solely on the original design parameters. Field observations conducted in March 2026 subsequently documented tension cracks extending approximately 100 m, ground displacement of approximately 50–100 mm, and surface subsidence ranging from 20 to 90 cm (Fig. 1).



Fig. 1. Field condition of the lowwall failure at Pit 4 interburden area (Seam B–C) showing tension cracks and surface subsidence.

Back-analysis provides an approach for investigating this discrepancy by using an observed failure condition to estimate the material parameters associated with slope instability. In contrast to forward analysis, which predicts slope performance from assumed material properties and loading conditions, back-analysis adjusts selected input parameters until the numerical model approaches the observed failure state. Previous studies have demonstrated that back-analysis can be used to estimate degraded shear-strength parameters and improve the interpretation of slope failure mechanisms in mining and geotechnical engineering applications [4–6]. The method is particularly useful when the original design parameters produce a stable numerical condition that is inconsistent with the actual field response.

Before conducting back-analysis, sensitivity analysis can be used to identify the material properties that exert the greatest influence on slope stability. Sensitivity analysis reduces the number of parameters requiring adjustment and provides a rational basis for selecting the controlling material and its relevant strength parameters. Variations in cohesion and internal friction angle may produce substantially different stability responses depending on the position and mechanical characteristics of the materials within the potential sliding mass. Therefore, identifying the dominant material through sensitivity analysis can improve the efficiency and interpretability of the subsequent back-analysis [4–6].

The practical application of back-analysis is particularly relevant to post-failure slope assessment, where the objective is not only to identify degraded material strength but also to establish a feasible geometry for subsequent mining operations. Conventional slope redesign commonly focuses on achieving a required FoS through flattening or modification of the slope geometry. However, a geometry that provides the highest FoS may require substantially greater excavation and may therefore be less practical from an operational perspective. For an active open-pit mine, slope redesign should consequently consider both stability and the excavation required to implement the proposed geometry. A comparison of alternative geometries can provide a more practical basis for selecting a design that satisfies the stability criterion while limiting unnecessary excavation.

A case illustrating this issue occurred at the lowwall slope of Pit 4 in the interburden area between Seams B and C at PT. Dizamatra Powerindo, South Sumatra, Indonesia. The slope experienced significant instability during

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January–February 2026. Field observations subsequently documented tension cracks extending approximately 100 m, ground displacement of approximately 50–100 mm, and surface subsidence ranging from 20 to 90 cm. The instability also affected mining access by blocking Jalan Austria, an access road leading toward the bottom of Pit 4. The original slope geometry had an overall angle of approximately 13°, while the initial numerical assessment based on the available material properties produced a deterministic FoS of 1.692 and a mean FoS of 1.707, exceeding the adopted minimum criterion of 1.1 [2]. The difference between the calculated initial stability condition and the observed field failure provides the basis for a post-failure back-analysis of the slope.

In this study, sensitivity analysis is first employed to identify the material controlling the stability response of the failed lowwall slope. The identified material parameters are then varied through deterministic and probabilistic back-analysis to estimate a degraded shear-strength condition associated with the observed failure. The resulting parameters are subsequently applied to three alternative slope geometries. The alternatives are evaluated using FoS and PoF, together with their corresponding cross-sectional cut areas, to compare stability performance and excavation requirements. The study therefore provides an integrated case-based workflow linking field-observed failure, sensitivity-guided parameter identification, back-analysis of degraded strength, and geometric redesign.

Accordingly, the objectives of this study are to: (1) identify the material parameters that most strongly influence the stability of the failed lowwall slope through sensitivity analysis; (2) estimate the degraded shear-strength parameters associated with the observed failure condition through back-analysis; and (3) evaluate alternative slope geometries based on the back-analyzed parameters by considering both slope stability and excavation requirements.

2. METHODOLOGY AND APPROACHES

2.1. Data Collection and Material Properties

The study was conducted using secondary data obtained from PT. Dizamatra Powerindo. The main datasets consisted of the C–C′ cross-sectional slope geometry from January 2026, an End-of-Month (EOM) orthophoto from March 2026, and laboratory-derived material properties for the lithological units forming the Pit 4 lowwall slope. The investigated cross-section represents the interburden area between Seams B and C where the observed slope instability occurred.

The slope materials were represented by seven lithological units: siltstone, carbonaceous claystone, sandstone, soil, claystone, coal, and backfill. The Mohr–Coulomb failure criterion was adopted to describe the shear strength of the materials, with unit weight, cohesion, and internal friction angle used as the principal material parameters. The initial material properties used in the numerical model are presented in Table 1.

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Table 1. Material Properties

No.	Lithology	Material Property	Distribution	Mean	Std. Deviation
1	Siltstone	Unit Weight (kN/m3)	Normal	18.50	1.64
		Cohesion (kPa)	Normal	498.05	358.81
		Phi (°)	Normal	42.15	6.72
2	Carbonaceous Claystone	Unit Weight (kN/m3)	Normal	19.40	1.87
		UCS (Mpa)	Normal	2.73	2.39
		Cohesion (kPa)	Normal	325.51	179.47
		Phi (°)	Normal	43.83	4.11
3	Sandstone	UCS (Mpa)	Normal	1.22	0.63
		Unit Weight (kN/m3)	Normal	20.31	2.32
		Cohesion (kPa)	Normal	57	63
		Phi (°)	Normal	22.68	2.27
4	Soil	UCS (Mpa)	Normal	2.15	2.33
		Unit Weight (kN/m3)	Lognormal	17.94	1.02
		Cohesion (kPa)	Lognormal	28.06	2.54
		Phi (°)	Lognormal	13.00	2.07
5	Claystone	UCS (Mpa)	Normal	0.14	0.10
		Unit Weight (kN/m3)	Normal	19.40	1.87

No.	Lithology	Material Property	Distribution	Mean	Std. Deviation
		Cohesion (kPa)	Normal	307.50	279.55
		Phi (°)	Normal	37.32	13.80
6	Coal	UCS (Mpa)	Normal	1.03	1.18
		Unit Weight (kN/m3)	Normal	13.00	5.00
		Cohesion (kPa)	Normal	84.15	12.99
		Phi (°)	Normal	27.28	2.86
7	Backfill	UCS (Mpa)	Normal	7.83	0.16
		Unit Weight (kN/m3)	Normal	20.01	1.22
		Cohesion (kPa)	Normal	20.80	52.73
		Phi (°)	Normal	37.32	13.80

The initial material parameters were used as the baseline condition for the slope stability model. Subsequent parameter modifications were performed only after the sensitivity analysis identified the material properties that had the greatest influence on the calculated stability.

2.2 Operational Distributed Load

Mining activities around the slope crest introduce surcharge loads from haulage equipment. To represent these operational loads in the two-dimensional limit-equilibrium model, the loads generated by the mining trucks were converted into equivalent distributed pressures applied to the corresponding haul-road surfaces.

For each vehicle, the total load was determined from its gross vehicle weight, while the effective contact area was estimated from the relevant vehicle dimensions. The equivalent distributed load was then calculated as:

$$q = \frac{w}{A} \quad (1)$$

where $q$is the equivalent distributed load (kN/m^2), $W$is the vehicle load (kN), and $A$is the effective contact area (m^2).

The resulting loads were applied to the haul-road sections represented in the C–C′ cross-section. The calculated operational surcharge loads were 114.41 kN/m^2 for Emirates Road and 98.07 kN/m^2 for Austria Road, based on the vehicle configurations considered in the study. The loads were maintained consistently in the subsequent slope stability analyses**.**

2.3. Seismic Load

Seismic effects were incorporated using a pseudo-static approach in accordance with the adopted Indonesian geotechnical design provisions. The pseudo-static method represents seismic effects through equivalent horizontal and vertical inertial forces applied to the potential sliding mass.

The seismic coefficients used in the model were based on the regional seismic conditions and the adopted design provisions. The resulting horizontal and vertical seismic coefficients were incorporated into the Morgenstern–Price limit-equilibrium analysis for evaluating the slope under the specified loading condition.

The pseudo-static approach was used as an equivalent representation of seismic loading rather than as a simulation of the complete dynamic response of the slope.

2.4. Sensitivity Assessment Workflow

Sensitivity analysis was conducted as the first stage of the back-analysis workflow to identify the material parameters that exerted the greatest influence on slope stability. The analysis was performed using Slide V6.0 and the Morgenstern–Price limit-equilibrium method.

The cohesion and internal friction angle of each lithological unit were systematically varied within the parameter ranges adopted from the available laboratory data. The resulting changes in the FoS were compared to determine the relative sensitivity of the slope to variations in the strength parameters.

The material exhibiting the strongest relationship between parameter variation and FoS was subsequently identified as the controlling material for the back-analysis. This procedure reduced the number of parameters requiring adjustment and provided a basis for focusing the degradation analysis on the material most relevant to the observed instability.

2.5. Back-Analysis Methodology

Back-analysis was conducted to estimate a degraded shear-strength condition that was consistent with the observed failure state. The procedure used the pre-failure slope geometry and the field-observed failure condition as the basis for adjusting the material parameters.

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The selection of degraded strength parameters in the back-analysis was guided by individual sensitivity analysis curves. To address the non-uniqueness inherent in back-analysis, parameter values were extracted at the exact intersection on the sensitivity plots where the calculated FoS reached the critical failure threshold (FoS < 1.0). Trial parameter combinations yielding FoS > 1.0 were rejected, as they overpredicted slope stability and failed to represent the observed field failure. Both deterministic and probabilistic analyses were considered. The deterministic analysis provided the FoS associated with the selected degraded parameters, while the probabilistic analysis was used to quantify the corresponding probability of failure (PoF).

The back-analysis results were then compared with the initial laboratory-derived parameters to quantify the reduction in cohesion and internal friction angle. These degraded parameters were subsequently adopted as the material condition for evaluating alternative slope geometries.

2.6. Limit Equilibrium Analysis

Slope stability calculations were performed using the Morgenstern–Price limit-equilibrium method implemented in Slide V6.0. The method considers both force and moment equilibrium of the slices forming the potential sliding mass.

The Mohr–Coulomb failure criterion was used to represent the shear strength of the slope materials according to:

$$\tau = c + \sigma_n \tan \phi \quad (2)$$

where $\tau$is the shear strength, $c$is cohesion, $\sigma_n$is the normal stress, and $\phi$is the internal friction angle. A block search method was used to represent the potential translational/planar failure mechanism observed in the investigated lowwall. The calculated FoS was used as the primary deterministic indicator of slope stability.

For the probabilistic analysis, variations in the relevant material parameters were considered to obtain the distribution of FoS and the resulting PoF. The same general modeling framework was maintained for the initial condition, back-analysis condition, and alternative redesigns to facilitate comparison among the scenarios.

2.7. Geometric Redesign and Evaluation Criteria

Following the back-analysis, three alternative slope geometries were developed using the degraded sandstone strength parameters obtained from the failure-condition analysis. The purpose of the redesign was to identify geometries that could satisfy the adopted stability criterion while remaining practical for excavation.

Each alternative was evaluated using deterministic FoS and PoF. The adopted minimum FoS criterion was 1.1 for the investigated slope condition. In addition to stability performance, the excavation requirement of each alternative was compared using the two-dimensional cross-sectional cut area obtained from the slope geometry. Because the numerical model represents a two-dimensional cross-section, cut area was used as the direct geometric excavation metric. An indicative excavation volume may be estimated by multiplying the cross-sectional cut area by an assumed longitudinal extent; however, such a volume is not a direct output of the two-dimensional model.

The preferred geometry was therefore selected based on a combination of:

1. Compliance with the adopted FoS criterion;
2. Acceptable probabilistic stability represented by PoF; and
3. Comparatively smaller cross-sectional cut area.

This evaluation framework allows the redesign alternatives to be compared not only according to their stability margins but also according to the excavation required to implement each geometry.

3. RESULTS AND CONCLUSIONS

3.1. Operational Distributed Load

Based on the vehicle specifications (summarized in Table 2) and the adopted contact-area approach, the distributed load for each individual vehicle was calculated (Table 3). To represent the maximum operational surcharge, the total distributed load applied to each haul road was determined by combining the typical fleet configurations operating in those specific areas. Consequently, the combined distributed load associated with Emirates Road was 114.41 kN/m^2, whereas Austria Road produced a distributed load of 98.07 kN/m^2. [11–13].

Table 2. Equipment Specifications

Vehicle Unit	Gross Vehicle Weight (kg)	Wheelbase (m)	Track Width (m)
Hino 280 JD	26.000	5.38	2.05
HD 785	163.780	4.95	4.325
SANY SRT100	160.000	4.57	4.215
AROCS	48.000	5.70	2.50

Table 3. Distributed Load

Vehicle Unit	Weight Force (Kn)	Contact Area (m^2)	Distributed Load (kN/m^2)
Hino 280 JD	254.8	11.029	23.10

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HD 785	1605	21.408	74.97
SANY SRT100	1568	19.262	81.40
AROCS	470.4	14.250	33.01

The calculated values indicate that Emirates Road represents the larger operational surcharge among the two investigated road locations. In a slope-stability model, an increase in surcharge near the crest can increase the driving forces acting on a potential sliding mass. Previous studies have similarly demonstrated that surcharge loads associated with mine-road or surface-traffic activities should be considered when evaluating slope stability [13]. Therefore, incorporating the operational loads into the present analysis provides a more representative loading condition than evaluating the slope solely under self-weight and seismic loading.

Nevertheless, the distributed-load calculation involves an important simplification. The actual loads generated by moving mining equipment are dynamic and may vary spatially and temporally, whereas the numerical model represents them as static distributed loads. Consequently, the calculated surcharge should be interpreted as an equivalent static representation of the operational loading condition rather than a direct simulation of vehicle-induced dynamic effects. The original manuscript likewise treated vehicle loading through an equivalent distributed-load approach rather than dynamic vehicle simulation.

From an operational perspective, the results indicate that the spatial relationship between haul roads and the potential failure zone should be considered during post-failure slope redesign. Relocating or controlling heavy traffic near a vulnerable slope crest can reduce the external loading imposed on the slope and may therefore complement geometric modification. This consideration is consistent with mine-slope optimization studies in which slope geometry and operational conditions are evaluated together rather than treating slope geometry as the only design variable [9,15].

3.2 Seismic Load

The seismic loading condition was incorporated using a pseudo-static approach based on the Indonesian National Standard (SNI 8460:2017) [14]. The regional Peak Ground Acceleration (PGA) for the Lahat area is 0.2g. Based on the site classification, the slope is dominated by rock material and categorized as Site Class SB (rock base), which corresponds to a seismic amplification factor (Fga) of 1.0.

$$PGAm = Fga\ x\ PGA = 1.0\ X\ 0.2G = 0.2g \quad (3)$$

The design spectral acceleration (PGAdesign) is then determined as two-thirds of the modified surface acceleration:

$$PGAdesign = \frac{2}{3} x\ PGAm = 0.133g \quad (4)$$

Following the approach recommended by Hynes-Griffin and Franklin [15], the horizontal seismic coefficient (kh) was determined as half of the design spectral acceleration:

$$Kh = 0.5\ x\ PGAdesign = 0.0665\ \approx 0.067\ g \quad (5)$$

while the vertical seismic coefficient (kv) was taken as half of kh:

$$Kv = 0.5\ x\ Kh = 0.03325\ \approx 0.033\ \text{g} \quad (6)$$

The corresponding design spectral acceleration was 0.133g, resulting in horizontal and vertical seismic coefficients of kh = 0.067 and kv = 0.033, respectively. The complete seismic loading parameters are presented in Table 4.

Table 4. Seismic Load

Parameter	Value
Amplification Factor (Fga)	1.0
Regional Peak Ground Acceleration (PGA)	0.2g
Maximum Surface PGA (PGAm)	0.2g
Design Spectral Acceleration	0.133g
Horizontal Acceleration Coefficient (kh)	0.067
Vertical Acceleration Coefficient (kv)	0.033

The pseudo-static approach converts seismic excitation into equivalent inertial forces acting on the potential sliding mass. This approach provides a practical means of incorporating seismic effects into a limit-equilibrium analysis and is consistent with the adopted Indonesian geotechnical design framework [14]. The results should, however, be interpreted within the limitations of the pseudo-static method. The approach does not reproduce the transient nature of earthquake loading, including cyclic deformation, frequency-dependent response, or progressive damage. Instead, earthquake excitation is represented by equivalent static forces. Therefore, the calculated FoS represents the slope response under the adopted pseudo-static loading condition rather than a complete dynamic response of the slope.

For the present study, the seismic loading was retained consistently throughout the back-analysis and redesign stages. This consistency allows differences in calculated FoS among the investigated conditions to be interpreted primarily in relation to changes in material strength and slope geometry rather than changes in the seismic loading assumptions.

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3.3 Initial Slope Stability Analysis

The initial stability analysis was conducted using the original slope geometry (Fig. 2) and the laboratory-derived material properties for the seven lithological units: siltstone, carbonaceous claystone, sandstone, soil, claystone, coal, and backfill. The Morgenstern–Price limit-equilibrium method was used to calculate slope stability under the adopted operational and seismic loading conditions.

The initial model produced a deterministic FoS of 1.692 and a mean FoS of 1.707, while the adopted probabilistic analysis yielded a PoF of 0.00%. The calculated FoS was therefore above the adopted minimum criterion of 1.1 [2]. On the basis of the initial numerical parameters alone, the slope would be classified as stable. The corresponding initial analysis is shown in Fig. 3.

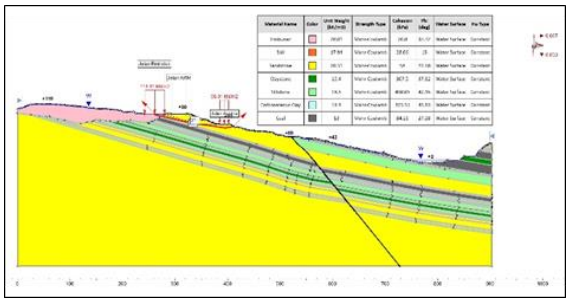

Fig. 2. Cross-Section C-C'

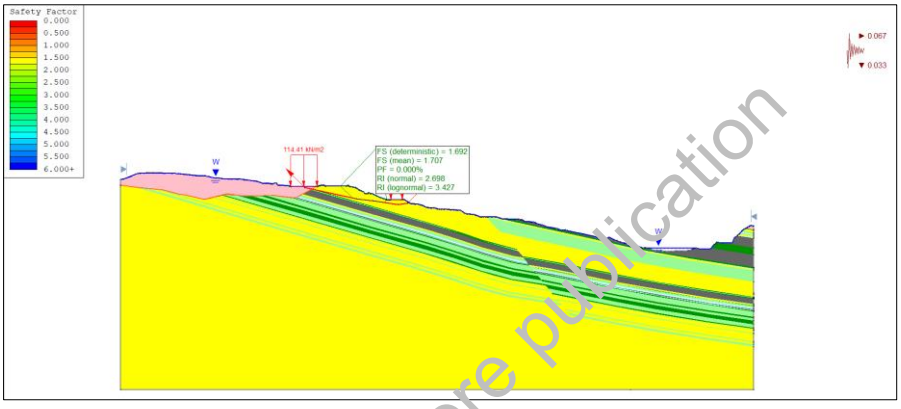

Fig. 3. Slope Stability Analysis Results

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However, this numerical result contrasts with the documented field condition, in which significant slope instability had already occurred. The discrepancy between the calculated stable condition and the observed failure is important because it indicates that the laboratory-derived parameters used in the initial model may not adequately represent the mechanical condition of the slope at the time of failure.

This discrepancy provides the principal basis for the subsequent sensitivity analysis and back-analysis. Rather than modifying all lithological units simultaneously, the analysis first identifies the material whose properties exert the greatest influence on the calculated FoS.

3.4. Sensitivity Analysis

The objective was to determine which material had the greatest influence on the calculated FoS and consequently should be prioritized during back-analysis. The overall material sensitivity curves are presented in Fig. 4.

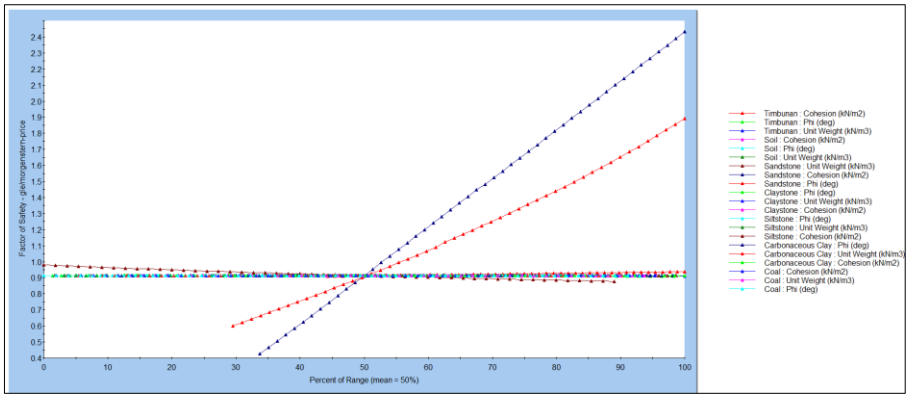

Fig. 4. Material Sensitivity Analysis

The sensitivity curves for backfill, soil, claystone, siltstone, carbonaceous claystone, and coal showed comparatively limited variation in FoS over the investigated parameter ranges. In

contrast, the sandstone curve exhibited a substantially steeper response, with the calculated FoS varying approximately from 0.4 to 2.4. This result identifies sandstone as the material exerting the greatest influence on the modeled stability response. The original manuscript shows this same contrast between the relatively flat responses of the other materials and the strong sandstone response.

The finding is consistent with the purpose of sensitivity analysis in geotechnical slope assessment, namely, to distinguish parameters that have a comparatively strong influence on stability from those whose variations produce smaller changes in the response. Previous research has demonstrated the application of sensitivity analysis to identify influential geotechnical parameters in slope-stability problems [21]. Other mining slope studies have likewise used parameter variation to investigate the role of material properties in the resulting FoS [6]. The individual sensitivity analyses of sandstone cohesion and internal friction angle further indicate that both parameters have a substantial influence on the slope response. This observation is consistent with the Mohr–Coulomb formulation, in which shear strength is governed by cohesion and the frictional component associated with the internal friction angle [10].

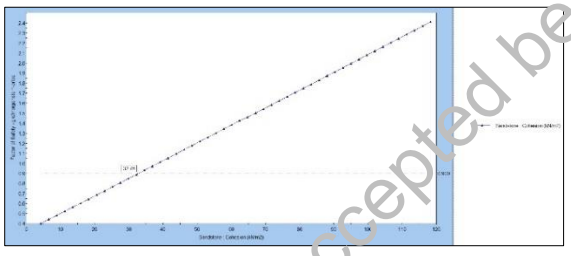

Fig. 5. Sandstone Cohesion Sensitivity Analysis

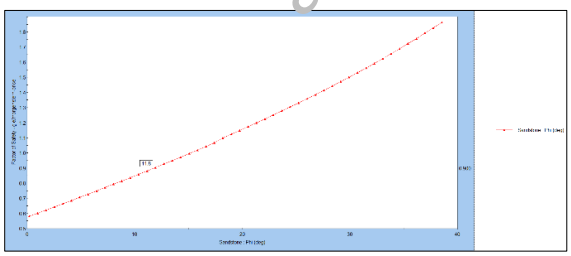

Fig. 6. Sandstone Internal Friction Angle Sensitivity Analysis

The individual sensitivity analyses of sandstone cohesion (Fig. 5) and internal friction angle (Fig. 6) further indicate that both parameters have a substantial influence on the slope response. This observation is consistent with the Mohr–Coulomb formulation, in which shear strength is governed by cohesion and the frictional component associated with the internal friction angle.

This distinction is important because field failure may involve multiple interacting factors, including material degradation, geological discontinuities, loading conditions, and environmental effects. The sensitivity analysis therefore provides a rational basis for narrowing the back-analysis to the sandstone layer instead of modifying all seven lithological units indiscriminately.

3.5. Back Analysis of the Failed Condition

Based on the sensitivity results, the back-analysis was focused on the sandstone layer while the properties of the other lithological units were maintained at their initial values. The sandstone cohesion and internal friction angle were progressively reduced until the calculated stability condition approached the observed failure state.

This procedure follows the general principle of back-analysis, in which observed slope behavior is used as a constraint for estimating mechanical parameters that are more representative of the condition associated with failure [17–20]. Previous mine-slope studies have also demonstrated the use of iterative parameter adjustment to investigate observed instability and support subsequent slope redesign [4–6,20].

To evaluate the failed lowwall condition, the target deterministic criterion for the back-analysis was explicitly set to an unstable failure threshold of FoS < 1.00. The parameter selection was guided by individual sensitivity analysis curves for the sandstone layer. Sensitivity iterations demonstrated high model sensitivity near unity: minor parameter increments (which theoretically indicated an FoS approx 0.99 on sensitivity plots) consistently caused the limit equilibrium search algorithm during full simulation runs to compute an FoS > 1.00, overpredicting slope stability.

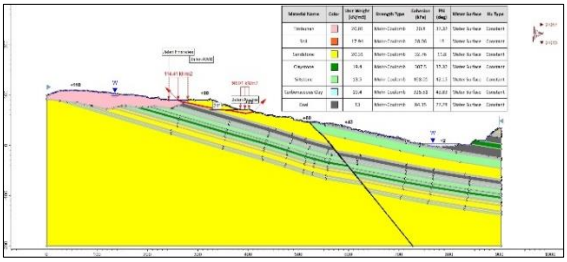

Fig. 7. Back-Analysis Material Properties for Cross-Section C-C'

Consequently, c = 32.76 kPa and phi = 11.80° were identified as the maximum threshold strength parameters that reliably drove the numerical model into the verified post-failure state (Fig. 7). Compared with the initial

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laboratory-derived values of 57.00 kPa and 22.68°, respectively, the estimated reductions were 42.53% in cohesion and 47.97% in internal friction angle, as compared in Table 5.

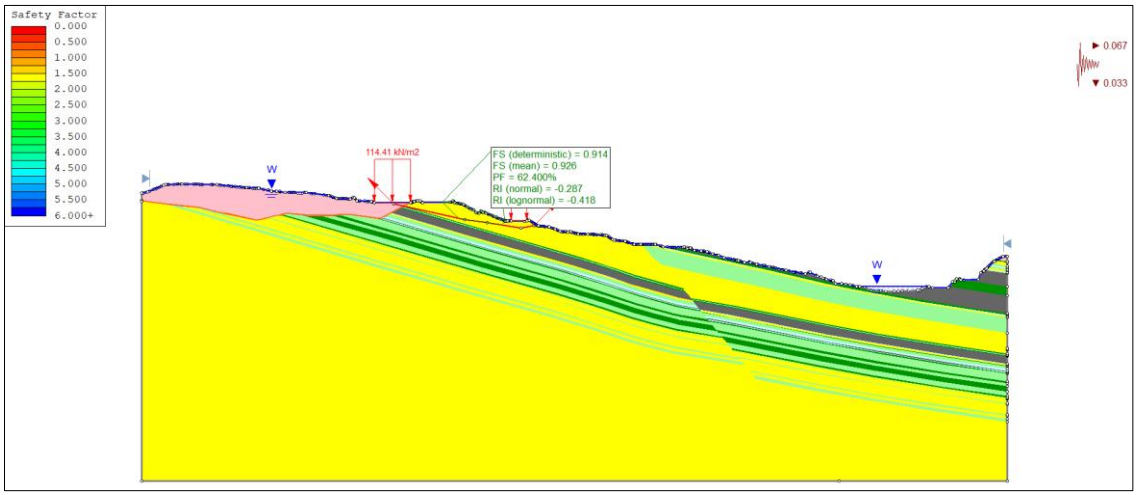

Fig. 8. Back-Analysis Slope Stability Results

Under the back-analyzed parameters, the deterministic analysis yielded an FoS of 0.914, with a mean FoS of 0.926 and a PoF of 62.40% (Fig. 8). The deterministic FoS below 1.0 is consistent with the representation of an unstable condition. The probabilistic result further indicates that a substantial proportion of the simulated parameter combinations produced a FoS below the adopted stability threshold.

To evaluate the numerical influence of the pseudo-static seismic coefficients (kh = 0.067, kv = 0.033) adopted from the baseline geotechnical model of PT. Dizamatra Powerindo, a comparative back-analysis run was conducted with no seismic loading (kh = 0, kv = 0) using the exact same degraded sandstone parameters (c = 32.76 kPa, ϕ = 11.80°). Without seismic loading, the model yielded a deterministic FoS of 1.108, a mean FoS of 1.128, and a PoF of 36.90% (Fig. 9). Although a deterministic FoS of 1.108 numerically overpredicts stability for an observed failure state (FoS < 1.00), the PoF of 36.90% still substantially exceeds the maximum allowable probability of failure threshold of 10% mandated by the Decree of the Minister of Energy and Mineral Resources (Kepmen ESDM) No. 1827 K/30/MEM/2018 for high consequence of failure (CoF) inter-ramp slopes [2]. Retaining the pseudo-static seismic loading is therefore essential to capture the verified post-failure condition (deterministic FoS = 0.914, PoF = 62.40%) while ensuring that all subsequent slope redesign options adhere strictly to company design baselines and regulatory safety standards.

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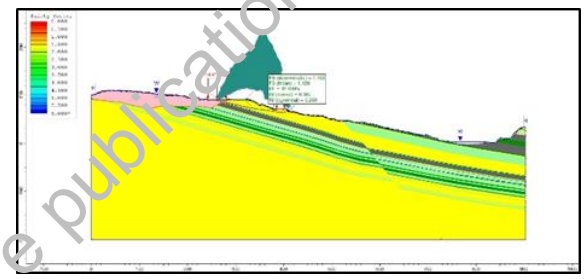

Fig 9. Slope stability back-analysis result under non-seismic loading condition

The field failure occurred following periods of high-intensity rainfall and inadequate surface drainage. The numerical model incorporated a static groundwater surface; however, transient seepage and dynamic rainfall-induced pore-water pressure changes were not explicitly modeled due to the absence of piezometric field monitoring data [7,8]. Consequently, the back-analyzed strength parameters (a 42.53% reduction in cohesion and 47.97% reduction in internal friction angle) represent an equivalent lumped strength reduction capturing the combined influence of material saturation, weathering, and structural disturbance rather than an explicit modeling of rainfall-induced pore-pressure buildup. This limitation is formally acknowledged.

Table 5. Comparison of Cohesion and Internal Friction Angle Before and After Back-Analysis

Material Properties (Sandstone)	Laboratory Data	Back-Analysis	Reduction Percentage
Cohesion (kPa)	57.00	32.76	42.53%
Internal Friction Angle	22.68°	11.80°	47.97%

Accordingly, the present back-analysis treats the degraded sandstone strength as the parameter adjustment required to represent the observed

failure condition rather than attributing the calculated strength reduction exclusively to rainfall infiltration.

3.6. Geometric Redesign

To address the geometric configuration of the proposed slope stabilization, three alternative redesign options (Option 1, Option 2, and Option 3) were developed and evaluated using the back-analyzed sandstone parameters. Each alternative incorporates distinct geometric adjustments to the overall slope angle, bench dimensions, and total number of benches to achieve stability while minimizing excavation requirements. The specific geometric parameters and corresponding stability performance for each option are detailed in Table 6.

Table 6. Geometric Parameters of Alternative Slope Geometries

Redesign Option	Elevation Stage (RL)	Slope Angle (°)	Bench Height (m)	Bench Width (m)	Number of Benches
Option 1	RL +92 to +87	30°	5	30	3
	RL +87 to +80	16°	7	10	
	RL +80 to +70	16°	10	—	
Option 2	RL +92 to +87	30°	5	30	3
	RL +87 to +80	18°	7	10	
	RL +80 to +70	18°	10	—	
Option 3	RL +92 to +87	30°	5	30	3
	RL +87 to +82	20°	5	20	
	RL +82 to +70	20°	12	—	

The back-analyzed sandstone parameters were subsequently applied to three alternative slope geometries. The purpose of the redesign was to determine whether the failed slope could achieve the adopted stability criterion while limiting the amount of material that would need to be excavated. The results are summarized in Table 7.

Table 7. Stability Results of Alternative Slope Geometries

Option	Deterministic FoS	Mean FoS	PoF	Cut Area (m^2)
Option 1	1.365	1.375	0.00%	518.47
Option 2	1.350	1.358	0.00%	566.12
Option 3	1.307	1.312	0.00%	433.40

All three alternatives exceeded the adopted minimum FoS criterion of 1.1 [2]. The results demonstrate that geometric modification can restore the modeled slope to an acceptable stability condition even when the back-analyzed sandstone parameters are retained.

The redesign should be interpreted as a response to the back-analyzed failure condition, rather than as a universal optimum geometry for the mine. The selected geometry is appropriate within the assumptions of the present two-dimensional model, including the adopted material parameters, operational loads, seismic coefficients, and slope cross-section.

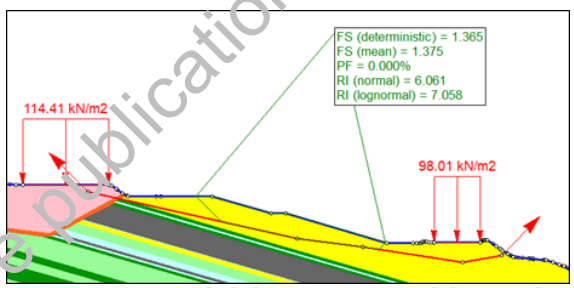

Fig. 10. Recommended Slope Geometry Stability Results

The proposed geometry incorporates a staged cutback and modification of the slope configuration. The redesign includes relocation of the affected haul-road alignment and adjustment of the slope angles and benches. The stability models for Option 1 and the preferred Option 3 are illustrated in Fig. 10 and Fig. 11, respectively, while Option 2 follows a similar staged configuration with an 18° lower slope angle but is not visually depicted here to maintain conciseness.

The results demonstrate that the highest FoS is not necessarily the only criterion for selecting a practical slope geometry. Option 1 produced the highest FoS, followed by Options 2 and 3. However, the differences in FoS among the alternatives were relatively limited, while the excavation requirements differed substantially. This supports the concept of evaluating slope geometry not solely on the basis of maximum stability but also in relation to the amount of excavation required to implement the design [8,14].

3.7. Stability–Excavation Trade-off and Selection of the Preferred Option

Because the analysis was conducted using a two-dimensional cross-section, the directly

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calculated excavation metric is the cross-sectional cut area, rather than a three-dimensional excavation volume.

The calculated cut areas were 518.47 m^2 for Option 1, 566.12 m^2 for Option 2, and 433.40 m^2 for Option 3.

Option 3 therefore requires the smallest cross-sectional excavation area among the three alternatives. Relative to Option 1, the reduction is approximately 16.4%, while relative to Option 2, the reduction is approximately 23.4%.

The comparison reveals an important stability excavation trade-off. Option 1 provides the highest FoS, but it also requires a larger excavation area than Option 3. Option 2 has a slightly lower FoS than Option 1 while requiring the largest excavation area. Option 3 produces the lowest FoS among the alternatives, but its deterministic FoS of 1.307 and mean FoS of 1.312 remain above the adopted criterion of 1.1, while its cut area is the smallest.

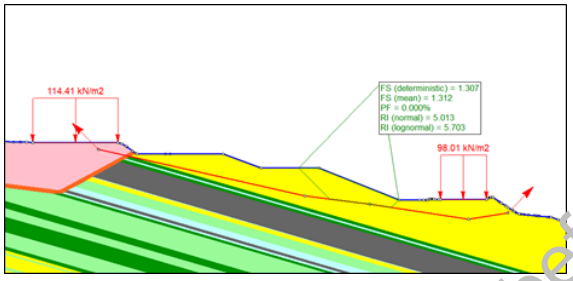

Fig. 11. Recommended Slope Geometry Stability Results.

For the present case, Option 3 is therefore selected as the preferred geometry (Fig. 11). The selection is not based on achieving the maximum FoS, but on achieving an acceptable stability condition while minimizing the required cross-sectional excavation. This approach is consistent with the practical objective of slope optimization, in which stability requirements need to be considered together with geometric and operational constraints [9,16].

If an indicative excavation volume is required for planning purposes, the two-dimensional cut areas can be multiplied by the adopted average observed landslide width of 57.29 m. The resulting indicative volumes are approximately:

1. Option 1: 29,703.15 m^3
2. Option 2: 30,266.31 m^3
3. Option 3: 24,829.49 m^3

These values should be treated only as indicative estimates, because they are derived from a two-dimensional cross-sectional area and an assumed longitudinal width rather than from a three-dimensional excavation model.

The selection of Option 3 consequently reflects a balance between stability adequacy and excavation efficiency. The result demonstrates the practical value of combining sensitivity analysis and back-analysis with geometric redesign: the slope geometry is evaluated using strength parameters derived from the observed failure condition rather than relying solely on the original laboratory-based design condition.

4. CONCLUSION

This study investigated the post-failure stability condition of a lowwall slope in the interburden between Seams B and C at Pit 4 of the PT. Dizamatra Powerindo coal mine using a sensitivity-guided back-analysis approach followed by geometric redesign. Based on the results and discussion, the following conclusions are drawn:

1. The initial slope model produced a deterministic FoS of 1.692, a mean FoS of 1.707, and a PoF of 0.00%, indicating a stable numerical condition under laboratory-derived parameters. This result was inconsistent with the observed field failure, providing the basis for sensitivity analysis and back-analysis.
2. Sensitivity analysis identified sandstone as the material exerting the greatest influence on slope stability. Both sandstone cohesion and internal friction angle were selected as the principal parameters for back-analysis.
3. Back-analysis resulted in degraded sandstone cohesion of 32.76 kPa and an internal friction angle of 11.80° (reductions of 42.53% and 47.97% from laboratory baseline values, respectively). Under these parameters, the slope produced a deterministic FoS of 0.914, a mean FoS of 0.926, and a PoF of 62.40%, matching the observed failure state.
4. The back-analyzed strength parameters represent an equivalent lumped degradation capturing overall material weakening rather than an explicit dynamic modeling of rainfall-induced pore-water pressure buildup. Due to the absence of dedicated monitoring wells or piezometric field data within the company's operational area, dynamic pore-water pressure and transient seepage modeling could not be explicitly performed, and this limitation is formally acknowledged [7,8].
5. Three alternative slope geometries evaluated with the back-analyzed parameters satisfied the stability criteria. Options 1, 2, and 3 produced deterministic FoS values of 1.365, 1.350, and 1.307, respectively, all with a PoF of 0.00%.

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6. Option 3 was selected as the preferred geometry because it achieved an acceptable FoS (1.307) while requiring the smallest cross-sectional cut area (433.40 m^2), reducing excavation by 16.4% relative to Option 1 and 23.4% relative to Option 2.
7. The study demonstrates that sensitivity-guided back-analysis combined with trade-off evaluation between stability and excavation cut area provides a practical workflow for post-failure mine slope assessment and redesign.

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REFERENCES

[1] Arif, I. (2014). *Indonesian Coal*. PT Gramedia Pustaka Utama.

[2] Ministry of Energy and Mineral Resources. (2018). Decree of the Minister of Energy and Mineral Resources of the Republic of Indonesia Number 1827 K/30/MEM/2018 concerning Guidelines for the Implementation of Good Mining Practice.

[3] Khanizadeh Bahabadi, M., Yarahmadi Bafghi, A., Fatehi Marji, M., Shahami, H., & Abdollahipour, A. (2024). A hybridized DDA-DDM for modeling jointed rock masses. *Theoretical and Applied Fracture Mechanics*, 131, 104445. https://doi.org/10.1016/j.tafmec.2024.104445

[4] Lazuardi, F. K., Sujatono, S., & Putra, B. P. (2025). Back analysis of slope stability for the main IPD (in-pit dump) disposal of Seam C in the South Pit B area of PT. Mifa Bersaudara, Aceh. *Mining Journal Exploration, Exploitation Georesource Processing and Mine Environmental*.

[5] Selsabeel, S., Widiarso, D. A., & Trisnawati, D. (2021). Back analysis of mine slope stability and its geotechnical engineering recommendations, case study: Low wall Pit Y area block 4900–5500 strip 3500–4300 PT. Pamapersada Nusantara site PT. Adaro Indonesia. *Jurnal Geosains dan Teknologi*, 4(3). https://doi.org/10.14710/jgt.v4i3.12519

[6] Yanottama, A., Seprizal, M. F., Wiratama, J., Navtalia, Z., & Lestari, L. (2025). Back analysis of highwall slope stability in the Pandu Pit of PT. Putra Muba Coal, Musi Banyuasin Regency, South Sumatra Province. *Jurnal Riset Rumpun Ilmu Teknik*, 4(2), 283–296. https://doi.org/10.55606/jurritek.v4i2.4633

[7] Chen, T., Shu, J., Han, L., Tovele, G. S. V., & Li, B. (2023). Landslide mechanism and stability of an open-pit slope: The Manglai open-pit coal mine. Frontiers in Earth Science, 10, 1038499. https://doi.org/10.3389/feart.2022.1038499

[8] Zhou, T., Zhang, L., Cheng, J., Wang, J., Zhang, X., & Li, M. (2022). Assessing the rainfall infiltration on FOS via a new NSRM for a case study at high rock slope stability. *Scientific Reports*, 12(1). https://doi.org/10.1038/s41598-022-21142-6

[9] Rahman, H. T., Zakaria, Z., & Gunawan, W. (2022). Optimization of the highwall slope in the Sikui project area open-pit coal mine based on the Morgenstern–Price limit equilibrium method. *Padjadjaran Geoscience Journal*, 6(3). https://doi.org/10.24198/pgj.v6i3.39801

[10] Wu, D., Chen, X., Tao, Y., & Meng, X. (2023). Estimating Mohr–Coulomb strength parameters from the Hoek–Brown criterion for rock slopes undergoing earthquake. *Sustainability*, 15(6), 5431. https://doi.org/10.3390/su15065431

[11] Hibbeler, R. C. (2016). *Engineering Mechanics: Statics* (14th ed.). Pearson.

[12] Setiawan, A. R., Syahrudin, S., & Syafrianto, M. K. (2022). Mine road construction planning at the Mine Development and SGA Plant Project of PT. Antam Tbk, bauxite commodity, Mempawah Regency, West Kalimantan Province. Jurnal PWK, Laut, Sipil, Tambang (JELAST), 9(1).

[13] Syahirah, A., Ishak, A., Jelani, J., Wong, S. Y., Suif, Z., Loqman, A., & Mazuki, A. (2024). Stability analysis of a silica sand slope model subjected to surcharge load using LEM and FEM methods. *GEOMATE Journal*, 27(121), 128–135. https://doi.org/10.21660/2024.121.4321

[14] National Standardization Agency. (2017). *SNI 8460:2017 Geotechnical Design Requirements*. BSN.

[15] Hynes-Griffin, M. E., & Franklin, A. G. (1984). *Rationalizing the seismic coefficient method*. US Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS, Miscellaneous Paper GL-84-13.

[16] Zulfahmi, Z. (2022). Geotechnical study for analyzing slope stability between two mining pit boundaries. *Indonesian Mining Journal*, 25(1), 1–11. https://doi.org/10.30556/imj.v25i1.1278

[17] Sakurai, S. (2019). *Back Analysis in Rock Engineering*. CRC Press/Balkema. https://doi.org/10.1201/9781315364261

[18] Sakurai, S. (2017). *Advances in Back Analysis in Rock Engineering*. CRC Press/Balkema.

[19] Sakurai, S., et al. (2009). Back analysis of measured displacements for assessing the stability of rock slopes.

[20] Sugiawati, G. N., Sophian, R. I., Alfadli, M. K., Khoirullah, N., & Putra, M. A. (2025). Geotechnical evaluation of landslide through a back analysis approach on the disposal area slope at Pit "X",

accepted before publication

Tanjung Enim, South Sumatra. *Indonesian Mining Journal*, 28(2), 73–84. https://doi.org/10.30556/imj.v28i2.1724

[21] Lin, H., Li, L., Qiang, Y., Zhang, Y., Liang, S., Xu, X., Li, H., & Hu, S. (2024). Sensitivity analysis of slope stability based on eXtreme gradient boosting and SHapley Additive exPlanations: An exploratory study. Heliyon, 10(16), e35871. https://doi.org/10.1016/j.heliyon.2024.e35871

accepted before publication