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Geotechnical Planning for Slope Stability in Limestone Mining at PT Rahmad Makmur Abadi, Punggulrejo Village, Rengel District, Tuban Regency

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Abstract
This study focuses on optimal slope-geometry planning in the IUP area of Rahmad Makmur Abadi Ltd, Punggulrejo Village, Rengel District, Tuban Regency, to create a safe working environment and maximize material recovery. Slope stability is evaluated by analyzing the Factor of Safety (FoS ≥ 1.2) using the finite element method based on the Hoek–Brown failure criterion, as well as the Probability of Slope Failure (PF ≤ 10%) using Rosenblueth's Point Estimate Method under the weakest slope conditions (saturated, with minimum UCS value), in accordance with KEPMEN ESDM No. 1827/K/30/MEM/2018. Through iterative geometric modeling (trial and error), the study produced recommendations for a single slope geometry (bench height of 20 m, width of 6 m, at a 90° inclination). It divided the overall slope into two blocks: Block 1 (bench height of 8 m, width of 4 m, at an 85° inclination with 9 benches, with the lowest bench at 7 m) and Block 2 (bench height of 8 m, width of 5 m, at an 85° inclination with 8 benches, with the lowest bench at 6 m). Rock mass quality analysis using Rock Mass Rating (RMR = 78) and Geological Strength Index (GSI = 73), along with Uniaxial Compressive Strength (UCS > 40 MPa) tests, supports the recommended mining excavation methods—drill & blast for optimizing productivity or the use of a breaker for cost savings.

1 Introduction

In the IUP area of PT Rahmad Makmur Abadi (RMA), where mining has not yet commenced, slope geometry modeling is required based on optimal Factor of Safety (FoS) and Landslide Probability values, with reference to KEPMEN ESDM No. 1827/K/30/MEM/2018. The slope geometry is modeled iteratively (trial and error) until the desired configuration is achieved. Landslides may still occur on slopes with a sufficient FoS, while some slopes remain stable despite having an FoS that is considered low. In response, landslide probability analysis is necessary to help ensure a high level of confidence in the safety of the slope [1].

Analysis of rock mass quality is essential for classifying and characterizing rock properties. One method developed for this purpose is the Rock Mass Rating (RMR) system by Bieniawski in 1989. RMR can be used to evaluate the stability of rock slopes and assist in planning mitigation measures. It is also useful for determining the mining equipment required based on the rock's strength. By conducting an RMR analysis, mining planning can be carried out more accurately. Based on the results of the RMR analysis, the Geological Strength Index (GSI) can be calculated, supported by uniaxial compressive strength (UCS) laboratory test results, to determine the recommended mining excavation method.

2 Methodology

The factor of safety (FoS) analysis method is the finite element method (FEM) using the Hoek-Brown failure criterion, whereas the probability of failure (PF) analysis method uses Rosenblueth's Point Estimate Method (PEM). The analysis of rock mass quality is conducted using the "Rock Mass Rating" (RMR) classification by Bieniawski (1989), thereby enabling calculation of the geological strength index (GSI). Additionally, the analysis of slope stability is supported by laboratory tests on rock samples from the research area, including tests of physical properties and mechanical properties to assist in the determination of intact rock strength parameters. Slope stability analysis assisted with Excel, AutoCAD, RocLab, and Phase 2 software.

2.1 Geological Condition

Based on the Lembar Babat Regional Geological Map, the research site, located in Punggulrejo Village, Rengel District, Tuban Regency, falls within the Paciran Formation. This formation's rock composition includes coral/reef limestone and calcarenite (see Figure 1). The field observations were conducted by exploring rock outcrops within the PT Rahmad Makmur Abadi Mining Permit (IUP) to determine the rock type and physical condition. Based on these observations, the limestone in the research area is classified as reef limestone with minor clay intercalations.

2.2 Factor of Safety (FoS) – Finite Element Method (FEM)

The Finite Element Method (FEM) is an approach for analyzing material stresses and displacements by dividing a structure (e.g., a slope) into small elements that represent the material's response under various conditions (fig. 1, right). This analysis reveals the displacement at each node and the stress in each element [2]. The elements are connected through nodes, where displacements are calculated and interpolated to obtain values at any point. The strain of each element is determined from the nodal displacements and subsequently used to calculate the stress [3]. FEM analysis generally adopts two main approaches: shear strength reduction and gravity loading.

The shear strength reduction method gradually reduces the material's shear strength until a slope failure mechanism develops [4]. The reductions in cohesion (c) and internal friction angle (∅) are expressed as follows:

$$Cf = \frac{c}{SRF} \quad (2.1)$$

$$\emptyset f = \tan 1 \left(\frac{\tan \emptyset}{SRF}\right) \quad (2.2)$$

Where:
SRF = shear strength reduction factor

Figure 1. Limestone Outcrops at the Research Site

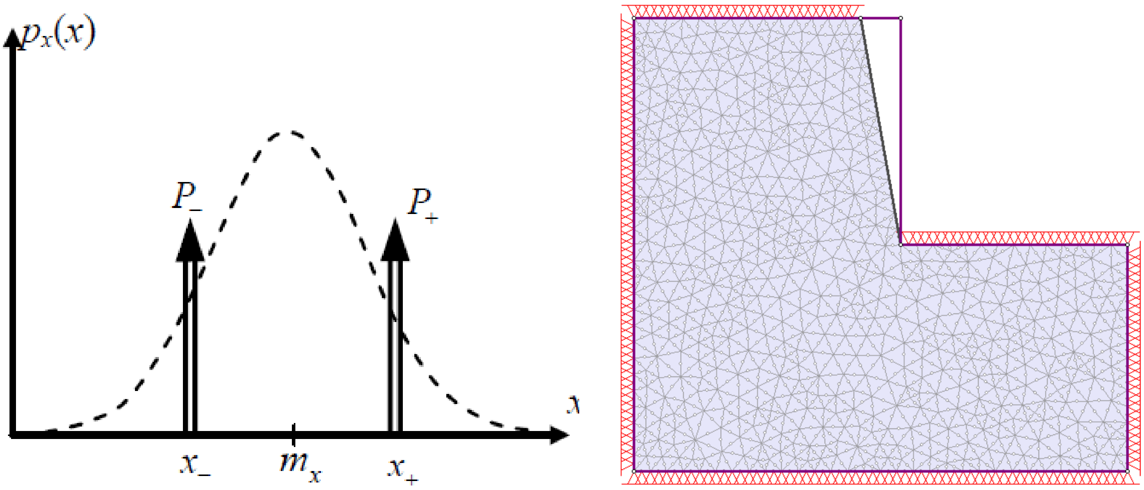

Figure 2. Distribution Function and Probability Density (left), Finite Element Method (right)

Gravity loading method gradually increases gravity until a slope failure mechanism is identified, at which point the factor of safety (FoS) is calculated as follows:

$$(FS)_{gi} = \frac{g\ limit}{g\ actual} \qquad (2.3)$$

Where:
g actual = gravitational constant (9.81 kN/m^3),
g limit = the value of gravity that exactly causes a collapse of the slope.

2.3 Probability of Failure (PF) – Rosenblueth's Point Estimate Method (PEM)

The Rosenblueth Point Estimate Method (PEM) is used in probabilistic analysis to estimate failure probability influenced by various factors (e.g., physical and mechanical properties of soil/rock, loads)[5]. It is widely applied in geotechnical engineering, particularly for slope design with random variables [6]. PEM employs Gaussian quadrature to compute the probabilistic moments of a random function and can be readily applied to implicit response functions or existing deterministic outputs. However, its limitation for multiple variables is that it requires calculations at 2n points, which significantly increases computational time[7]. To address this, Rosenblueth (1975) proposed reducing the number of calculation points to 2n+1 for uncorrelated variables with negligible skewness by approximating continuous random variables with discrete ones having N pulses (Figure 2, left).

$$p_x = \sum_{i=1}^{N} \delta(x - x_i) p(x_i) \qquad (2.4)$$

Where:
$\delta(x - x_i)$ = the Dirac delta,
$p(x_i)$ = the probability assigned to point x_i, for a fixed value of the random variable.

A statistical distribution of input parameters is required to assess mine slope failure probability[8]. Most rock types typically follow a normal distribution for simplified analysis. However, mine rock properties often follow specific distributions (normal, lognormal, gamma) [9]. The method offers two main advantages: it only requires the first two moments of the input variables (without needing their full distribution), and it involves fewer function evaluations compared to Latin Hypercube or Monte Carlo sampling.

2.4 Rock Strength Parameter

2.4.1 Hoek – Brown Criterion

The Hoek-Brown failure criterion is an empirical method to describe the non-linear behavior of isotropic rock by comparing its strength under increasing confining stress. The generalized criterion is formulated as follows [3].

$$\sigma'_i = \sigma'_3 + \sigma_{ci}\left(mb\frac{\sigma'_3}{\sigma'_{ci}} + S\right)^a \quad (2.5)$$

Where:
σ'_i = Maximum effective stress at rock failure
σ'_3 = Minimum effective stress at rock failure
σ_{ci} = UCS (*intact rock,*)
mb = Hoek-Brown constant value for rock masses
$s\ and\ a$ = a constant that depends on the characteristics of the rock mass

In the Hoek-Brown failure criterion, the constant mb replaces the Phi constant (internal friction angle), while the constant s substitutes for the cohesion constant. Hoek – Brown Criterion calculated with phase2 software.

2.4.2 Uniaxial Compressive Strength (UCS)

The uniaxial compressive strength (UCS) test determines rock strength using cylindrical samples loaded uniaxially[10]. It assesses intact rock properties, yielding data on stress–strain curves, elastic modulus, UCS, fracture energy, specific fracture energy, and Poisson's ratio [3]. The test follows ISRM (1981) standards using a compression machine and samples with an L/D ratio of L=2D[11].

2.4.3 Rock Quality Designation (RQD)

RQD is a percentage measure from core samples that indirectly indicates the extent of discontinuities (weak zones) and soft sections in a rock mass[12]. It is calculated by summing the lengths of intact cores longer than 100 mm (4 inches) and dividing by the total drill length [13].

$$RQD = \frac{Total\ core\ drill\ length \geq 0{,}10\ m}{Total\ drill\ length\ (m)} \times 100\% \quad (2.6)$$

If core samples are unavailable, RQD can be indirectly estimated by measuring the orientation and spacing of discontinuities in rock outcrops. The following formula can be used to determine RQD from scanline data.

$$RQD = 100e^{-0{,}1\lambda}(0{,}1\lambda + 1) \quad (2.7)$$

Where:
$\lambda =$ frequency of discontinuities per meter

2.4.4 Rock Mass Rating (RMR)

Rock Mass Rating (RMR) is a geotechnical assessment system used to evaluate rock quality in construction projects. Developed by Professor Z.T. Bieniawski in 1989, it is vital for analyzing the stability of slopes, tunnels, and underground structures. RMR values range from 0 to 100 (higher values indicate better geological quality) and guide risk estimation and design decisions. The classification is based on parameters such as intact rock strength, Rock Quality Designation (RQD), condition of discontinuity, and groundwater conditions.

2.4.5 Geological Strength Index (GSI)

The Geological Strength Index (GSI) estimates the mechanical properties of rock masses. Developed by E. Hoek and E.T. Brown in 1997 [14], it's widely used in rock engineering and geotechnical applications to aid in the design and analysis of structures such as tunnels, slopes, and foundations.

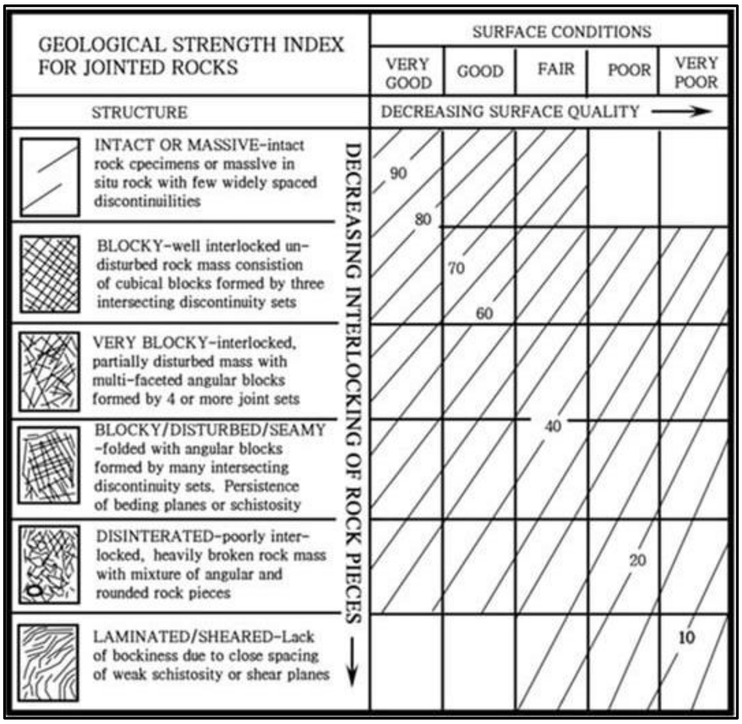

Figure 2. Chart for Determination of Geological Strength Index (GSI)

GSI is typically determined through field observation and visual analysis. This involves assessing the rock mass's structure and surface roughness using a GSI table or diagram, which categorizes rocks into quality classes based on two main components. The formula for calculating GSI is as follows:

$GSI = RMR - 5$ ....(2.8)

2.4.6 Rippability

Rippability is a parameter that determines how easily mining material can be ripped—whether it is easily, moderately, or not rippable—which in turn affects the chosen excavation method. These methods are influenced by the material's physical and mechanical properties, especially rock strength. The rippability guidelines based on KEPMEN ESDM No. 1827/K/30/MEM/2018 (Table 1).

Table 1. Material Rippability

UCS (Mpa)	GSI	Kategori
<1,5	<50	Free Digging
1,5 – 40	50 – 70	Rakes
>40	>70	Drill and Blast

2.5 FK – SRF Correlation

Factor of Safety (FoS) measures a structure's safety (e.g., a slope) against failure. Slope stress can be observed through discontinuities like joints, which reduce cohesion and internal friction, lowering slope stability[5]. The Strength Reduction Factor (SRF) is a numerical approach (particularly within FEM) that gradually reduces soil shear strength until failure occurs. In SRF-based analysis, FoS is determined by comparing the critical SRF at failure with the soil's initial strength. A high FoS (and thus a high SRF) indicates that the soil can endure significant strength reduction before failing, while a low FoS suggests vulnerability. Essentially, SRF reflects how much the soil strength can be reduced before failure, and FoS represents the overall safety margin[15].

$FK = \frac{c+\sigma_n \tan\phi}{\frac{c}{SRF}+\sigma_n \tan\left(\tan^{-1}\left(\frac{\tan\phi}{SRF}\right)\right)}$ ....(2.9)

$$FK = \frac{c+\sigma_n \tan\phi}{\left(\frac{c+\sigma_n \tan\phi}{SRF}\right)} \quad (2.10)$$

$$FK = \frac{c+\sigma_n \tan\phi}{1} + \frac{SRF}{c+\sigma_n \tan\phi} \quad (2.11)$$

Therefore, $FK = SRF$
Phase2 Software uses the Finite Element Method (FEM), producing results in the form of the Strength Reduction Factor (SRF). While the commonly used Limit Equilibrium Method (LEM) yields a Factor of Safety (FoS), in this software, the FoS is equivalent to the critical SRF. The Factor of Safety (FoS) in this study follows KEPMEN ESDM No. 1827/K/30/MEM/2018[16] (table 2).

Table 2. Factor of Safety (min) Acceptable Criterion

Slope Type	Landslide Severity	Acceptable Criterion		
		Static Factor of Safety (FoS) (min)	**Dynamic Factor of Safety (FoS) (min)**	**Probability of Failure (max) (FoS ≤ 1)**
Single Slope	Low – High	1.1	Tidak ada	25 – 50%
Inter – ramp	Low	1.15 – 1.2	1	25%
	Moderate	1.2 – 1.3	1	20%
	High	1.2 – 1.3	1.1	10%
Overall Slope	Low	1.2 – 1.3	1	15 – 20%
	Moderate	1.3	1.05	10%
	High	1.3 – 1.5	1.1	5%

3 Results and DiscussionRESULT AND DISCUSSION

3.1 Result

3.1.1 Slope Geometry Planning

PT Rahmad Makmur Abadi's work area is divided into two mining blocks. Block 1 (171 – 105 masl), at the lower section, remains unmined and has no slopes, while Block 2 (105 – 49 masl), at the upper section, was mined around 1990, leaving degraded slopes. Mining in both blocks starts at the lowest elevation, with slopes created gradually. The focus, however, is to reach the highest elevation as quickly as possible, then mine downward while optimizing slope geometry. The mine road to the highest elevation is designed in an "S" shape relative to the slope for haulage safety. Based on field observations and X-Ray Fluorescence (XRF) test results, the lithology of the study area is reef limestone, non-clastic, and clastic massive limestone. Of the three types of limestones, the limestones are considered to be 1 domain, namely limestones. Slope stability analysis uses several rock strength parameters, as shown in Table 3.

3.1.2 Data Analysis

The results of laboratory tests on the physical and mechanical properties of rocks, which are used in slope stability analysis, are as follows (Table 3). Hoek – Brown criterion (mb, s and a) are calculated using RocLab software, with GSI, Mi value, and disturbance factor (D). GSI value is calculated using formula 2.8 (73); Mi and D values are selected from the software (Limestone, Mi = 12. Small scale blasting, poor condition, D = 1). Probability of Failure parameter calculated using Excel and RocLab software. Result shown in Table 4.

Table 3. Rock Parameter for Slope Stability Analysis

Test Parameter	Unit	Sample 1	Sample 2	Sample 3	Sample 4	Average
Uniaxial Compressive Strength (UCS)	kPa	42040	47410	26660	-	38703
STD. Dev UCS						10.77
Natural Density (Wn)	kN/m^3	19.83	23.46	23.57		22.59
Young Modulus	kPa	9787687	11045637	4654637	-	8495988
Poisson Ratio						

Table 4. Hoek – Brown Criterion Analysis

	Parameter	Value
Hoek-Brown Criterion	mb	1.74427
	s	0.011109
	a	0.501071

3.1.3 Slope Stability Analysis

After data processing, slope stability was analyzed using Phase2 software, with the slope geometry modeled using AutoCAD. Since this study employs FEM, the FoS is replaced by the SRF. The Hoek-Brown criterion was selected for limestone, while landslide probability was assessed with the Rosenblueth Point Estimate Method (PEM). These methods were chosen because the rock is heterogeneous, anisotropic, and exhibits non-linear behavior (e.g., cracking under overload). FEM supports non-linear and plastic modeling, enabling detailed stress and deformation analysis by dividing the slope into small elements.

Table 5. Slope Modeling Result

No	Slope Geometry			Mean Critical SRF	Std. Dev. Critical SRF	Probability of Failure (PF) (%)
	Bench Height	Bench Width	Bench Face Angle			
Single Slope (Figure 3)						
1	20	5.5	90	1.66	0.6647	16.04
2	20	6	90	1.20	0.0071	0
Overall Slope (Figure 4)						
Blok 1 – 1	8	4	85	1.20	0	0
Blok 1 – 2	8	5	85	1.20	0	0
Blok 2 – 1	8	5	70	1.20	0	0
Blok 2 – 2	8	5	75	1.10	0.0071	0

The slope was modeled with a height of 20 m, a width of 5.5–6 m, and a 90° angle, assuming fully saturated groundwater and the minimum UCS value. Apart from that, the slope width design is 5.5 - 6 meters according to the type of transport used, namely the Hino 500, which has a width of 2.45 meters. For Block 1, the slope was modeled at 8 m height, 4–5 m width, and 85° angle; for Block 2, at 8 m height, 5 m width, and a 70°–75° angle. The slope modeling results are presented in Table 5.

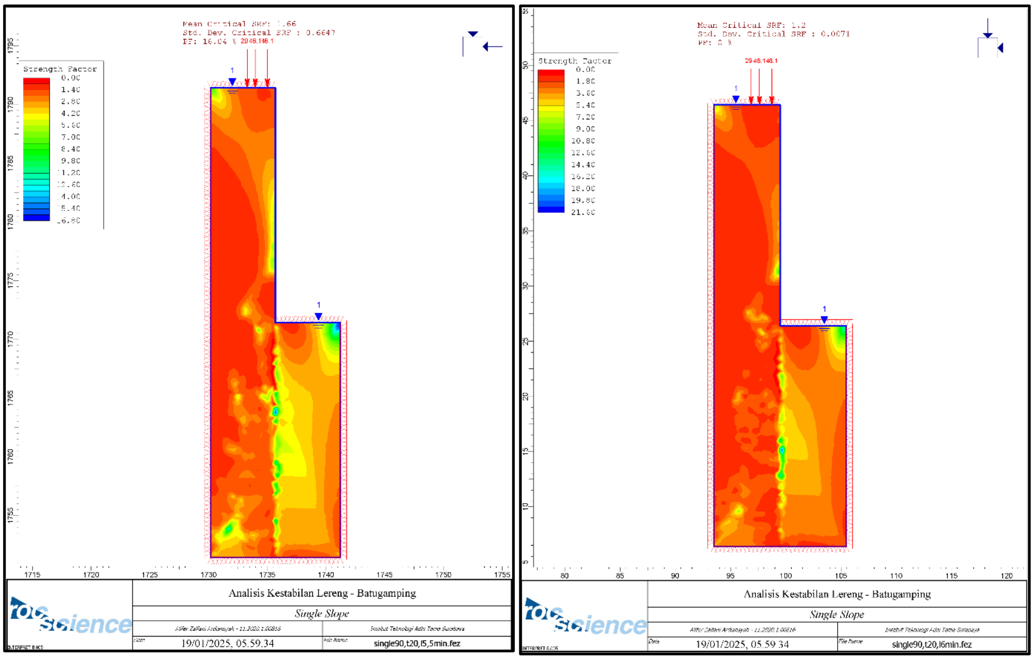

Figure 3. Single Slope – No. 1 (left), Single Slope – No. 2 (right)

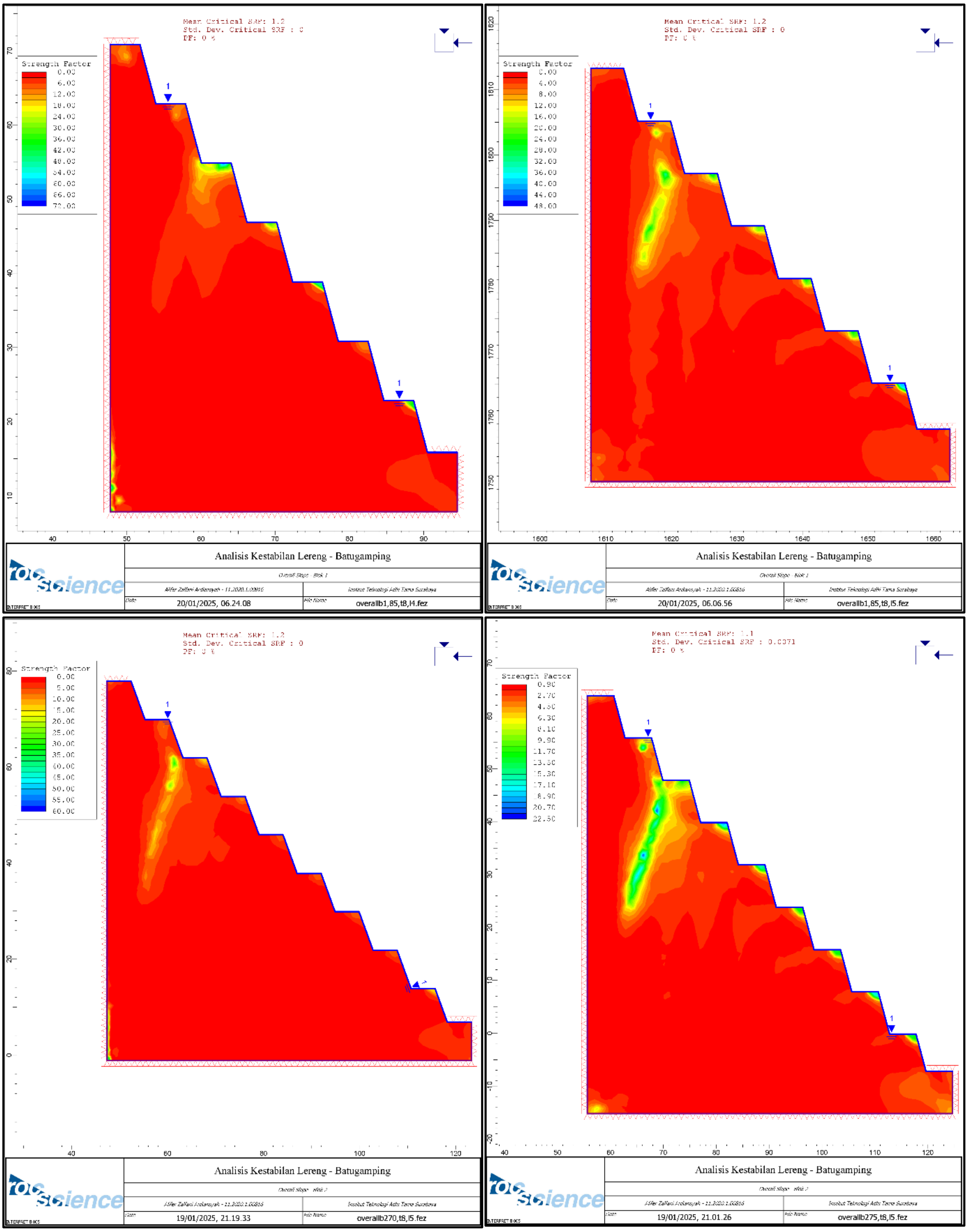

Figure 4. Overall Slope – Block 1, No. 1 (top left), Overall Slope – Block 1, No. 2 (top right), Overall Slope – Block 2, No. 1 (bottom left), Overall Slope – Block 2, No. 2 (bottom right)

3.2 Discussion

The analysis yielded the following results: A single slope with a height of 20 m, 5.5 m width, and 90° angle produced a mean critical SRF of 1.66 with a landslide probability (PK) of 16.04% (suboptimal), while a similar slope with a 6 m width yielded a mean critical SRF of 1.2 and 0% PK (optimal). For overall slopes in Block 1 (8 m height, 4 m and 5 m widths, 85° angle), both cases were optimal with a mean critical SRF of 1.2 and 0% PK. In Block 2, a slope with 8 m height, 5 m width, and a 70° angle was optimal (SRF 1.2, PK 0%), but with a 75° angle, the mean critical SRF dropped to 1.1 (PK 0%), rendering it less optimal.

Additionally, the RMR value of 78, GSI value of 73, and the fact that most UCS tests exceeded 40 indicate a generally good-quality limestone rock mass, but the UCS range of 26.66-47.41 MPa shows meaningful intact-rock variability. Drill-and-blast is therefore appropriate for the stronger and more competent zones, while a hydraulic breaker may be practical in locally weaker material where productivity requirements permit. The recommendations apply to the analyzed static and fully saturated conditions. Implementation should be supported by bench-scale discontinuity mapping, groundwater monitoring with piezometers, and deformation monitoring using survey prisms or slope-stability radar. The design should be reviewed if actual groundwater, structural, or operational conditions differ from the model assumptions.

Additionally, the RMR analysis yielded a value of 78, corresponding to a GSI of 73, and since most UCS tests exceeded 40, drilling and blasting are recommended for excavation.

4 Conclusion

The recommended FoS (equivalent to SRF in this study) follows KEPMEN ESDM No. 1827/K/30/MEM/2018. The optimal single slope is 20 m high, 6 m wide, and 90° steep. For overall slopes, Block 1 should be 8 m high, 5 m wide, and 85° steep, while Block 2 is best at 8 m high, 5 m wide, and 70° steep. Regular monitoring during the rainy season is advised to anticipate any strength loss or slope movement. Monitoring can be seen from the height of the groundwater level as assessed by the packer test. Groundwater level data is linked to slope movement data obtained from the slope stability radar (SSR) to assess whether the type of slope movement is progressive or regressive.

Rock quality was assessed using Bieniawski's (1989) Rock Mass Rating, yielding an RMR of 78. The breakdown includes: UCS of 38.703 MPa (rating 4); RQD of 99.995% (rating 20); discontinuity spacing over 2 m (rating 20); discontinuity condition with slightly rough surfaces and joint openings under 1 mm (rating 19); dry conditions (rating 15); and a strike/dip orientation of 45°–90° (rating 0).

Material rippability classification, based on KEPMEN ESDM No. 1827/K/30/MEM/2018, determines the excavation method. Laboratory tests show limestone UCS values ranging from 26 to 47 MPa and a GSI of 73, classifying the material under the "Drilling and Blasting" category. Although using a breaker can save costs, drilling and blasting are recommended for optimal productivity.

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