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# Stability Assessment of Block–Flexural Toppling Using Numerical and Deterministic Approaches

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## Abstract

Block–flexural toppling is a complex failure mechanism commonly observed in jointed rock slopes, resulting from the combined effects of block rotation and flexural deformation. This instability is primarily controlled by geological structures, particularly the orientation, persistence, and mechanical properties of discontinuities. This study presents a stability assessment of block–flexural toppling at Cliff 05, Citatah, West Bandung Regency, West Java, Indonesia, where recurrent rock block detachments have been documented. The slope is dominated by a primary joint set that is nearly perpendicular to the slope face and intersected by a secondary joint set, creating conditions favourable for toppling failure.

To investigate the failure mechanism, stability analyses were conducted using both numerical modelling and the Limit Equilibrium Method (LEM). Numerical simulations were performed using the finite element method within an elastic–plastic framework, while LEM was applied to evaluate the factor of safety under varying geometric and mechanical conditions. Rock mass and discontinuity parameters were derived from field observations and rock mass classification systems, including the Rock Mass Rating (RMR) and Geological Strength Index (GSI). Joint shear strength parameters were estimated using the Barton–Bandiscriterion for both joint sets. The joint network was represented using deterministic and statistical approaches.

Results from numerical modelling indicate total displacements ranging from approximately 9.5 to 19 mm, with plastic zones developing around surface joints and joint intersections. LEM analysis shows a reduction in factor of safety consistent with increasing slope inclination and unfavorable joint orientations. Sensitivity analysis identifies internal friction angle, slope inclination, and slope face angle as the most influential parameters governing block–flexural toppling stability. The comparative results highlight the advantages and limitations of each method in assessing toppling-prone rock slopes.

## Keywords

Stability assessment, Block–flexural toppling, Joint fracture network, Numerical Modelling, LEM.

# 1 Introduction

The stability of rock slopes affected by block–flexural toppling is highly dependent on the geometry and connectivity of discontinuities within the rock mass. Previous numerical studies have often represented toppling mechanisms using simplified joint configurations, such as a single persistent joint set dipping opposite to the slope face or two fully persistent joint sets extending continuously throughout the model. While these idealized models are useful for understanding fundamental toppling behaviour, they may not sufficiently capture the complexity of natural discontinuity systems. In the field, discontinuities are commonly variable and discontinuous, influencing block formation, flexural bending, block interaction, and progressive failure development.

This study assesses the block–flexural toppling mechanism at Cliff 05 Citatah using integrated numerical and deterministic approaches. Field-based scanline mapping is used to characterize joint orientation, spacing, and persistence, which are then incorporated into the stability assessment. Finite element modelling is applied to evaluate stress redistribution, deformation behaviour, and progressive failure, while limit equilibrium analysis provides a deterministic estimation of stability based on block geometry, force equilibrium, and joint shear strength. By comparing both approaches, this study aims to evaluate how simplified deterministic analysis represents the observed toppling mechanism and how numerical modelling can improve the interpretation of failure evolution in discontinuous rock masses.The paper compares FEM and LEM analyses on the same limestone rock slope to evaluate the capability, limitations, and consistency of both methods in assessing block–flexural toppling failure.

# 2 Research study location

Cliff 05 is a limestone cliff within a quarry site in Citatah village, Cipatat subdistrict, West Bandung regency, West Java, Indonesia. Geomorphologically, the study site is categorized as steep hills (van Zuidam 1986). Geologically, Cliff 05 consists of calcarenite limestone with sand grain size (1/16-2 mm) (Sudjatmiko 1972).Rock mass and discontinuity data of the study site were collected by undertaking engineering geological mapping (Figure 1).

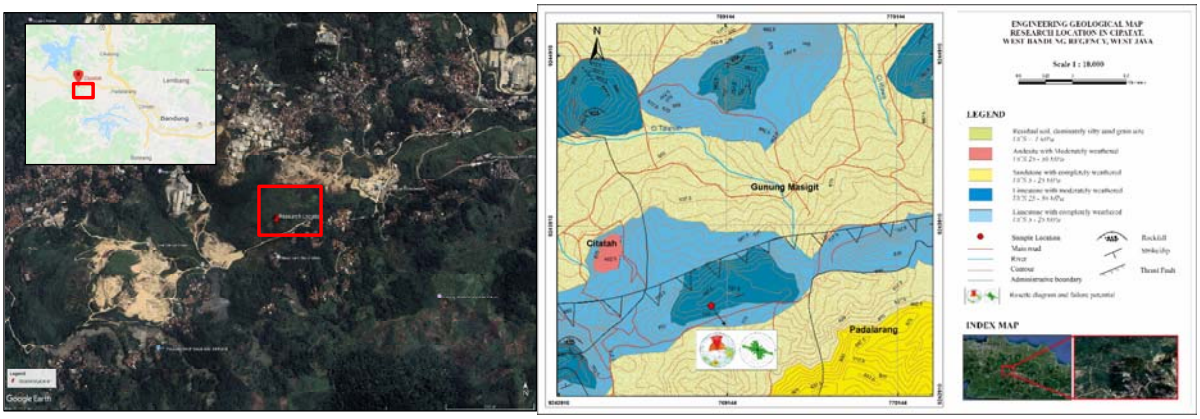



Fig. 1 Engineering geological map of the study location (modified from Atmojo 2021).

The limestone of Cliff 05 Citatah has GSI values ranges between 50 and 60, with a median value of 55 selected for numerical modeling input(Atmojo 2021; Marinos and Hoek 2000).A GSI value of 55 corresponds to a disturbed, interlocked rock mass, consisting of angular blocks formed by the intersection of three discontinuity sets(Hoek and Brown 2018). The surface conditions are rough, slightly weathered, and exhibit oxidation along discontinuity planes (Figure 1 and 2).

The limestone of Cliff 05 is categorized as fair rock mass (Class III) with an RMR value of 60,25 (Atmojo 2021; Bieniawski 1989). From the RMR classification, some material properties can be determined empirically, unit weight of 0.0253 MN/m$^3$, Poisson's ratio of 0.32, Young's modulus of 5340 MPa, tensile strength of 0.1 MPa, cohesion 0.294 MPa, and friction angle of 51.32°.

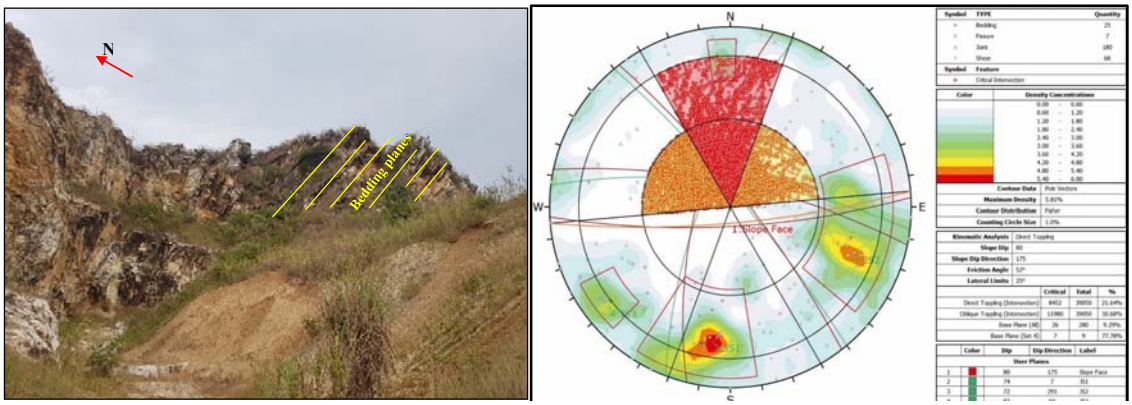



Fig. 2 Outcrop of the cliff shows the discontinuity orientation have dip direction relatively perpendicular to slope face.

The cliff shows a slope geometry with an approximate height of 25 m, a slope length of around 150 m for the area of interest, and a slope face orientation of 80°/N175°E. The discontinuity observed consists of bedding planes, shear fractures, and joints. (Table 1).

Table 1. Discontinuity condition along the slope.

| Discontinuity Type | Dip Direction/Dip | Spacing (m) | Persistence (m) | Aperture (mm) | Surface Characteristics | Weathering/ Infill | Seepage Condition |
|---|---|---|---|---|---|---|---|
| **Bedding planes** | 007°/74° | 1.5 m | >10 | 0.1–10 | Very rough; undulating to stepped | Moderately to highly weathered; mainly sandy clay weathered infill, locally calcite | Generally dry, with minor seepage |
| **Shear fractures** | 050°/82° and 291°/72° | 0.8 - 2 | 1-3 | 0.1–0.5 | Rough; planar to stepped | Slightly weathered; locally sandy weathered infill | Predominantly dry |
| **Joints** | 177°/65° to 300°/75° | 0.45 m | <7 | 0.1–2.5 | Smooth to moderately rough; undulating to stepped | Moderately to highly weathered; sandy weathered infill | Generally moist |

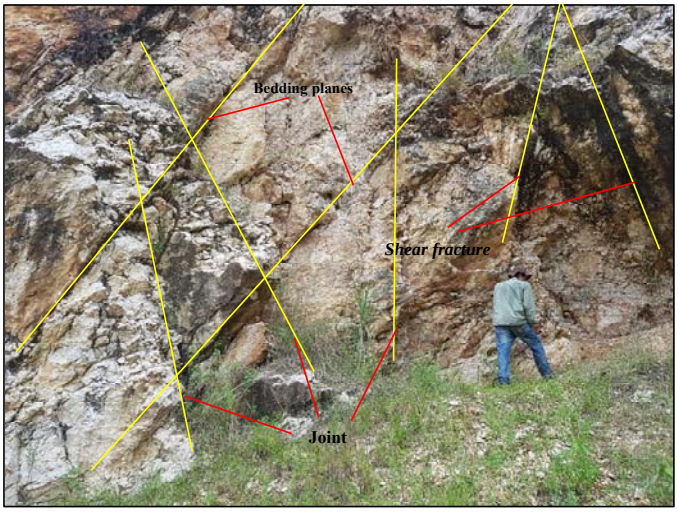



Fig. 3 Discontinuity type in the outcrop contains bedding planes, joint, and shear fracture. The orientation of joint and shear fracture intersects with the bedding planes.

# 3 Methodology

The discontinuity condition and failure mechanism was determined, contributing discontinuities were picked to be further analyzed. The properties of the contributing discontinuities were then calculated using the formula by (Barton and Bandis 1990) (Equation 1).

$$\tau = \sigma_n \tan\left(\varphi_b + JRC \log_{10}\frac{JCS}{\sigma_n}\right) \quad (1)$$

Where $\tau$ Joint shear strength
$\sigma_n$ Normal stress
$\varphi_b$ Basic friction angle
$JRC$ Joint roughness coefficient
$JCS$ Joint wall compressive strength

In this study, numerical modeling is a computer programming with the purpose to represent response mechanism of a rock mass that experiences changes in its conditions (Wyllie and Mah 2004). The numerical method utilized in this study is FEM with an elastic-plastic material model. The material model represents the stress-strain relationship, reflecting its behavior. The elastic-plastic model utilizes a strength parameter to limit the acceptable shear stress. The deterministic modeling in this study was conducted by LEM. The method only uses static equilibrium between forces/moments in a slope, and neglecting stress-strain relationship it has (Azizi, 2014). The reliability of LEM depends on the accuracy of slope geometry and the sliding surface. FEM analysis was performed using *RS2* and LEM using *RocTopple*(Rocscience 2021a and Rocscience 2021b).

# 4 Result and Discussion

## 4.1 Proposed Model of Block Flexural Toppling

The numerical model was developed using a finite element approach in RS2 with explicitly defined discontinuity elements. Therefore, although the overall analysis is based on FEM, the main discontinuity sets observed in the field were incorporated as discrete interfaces to represent the block–flexural toppling mechanism. The proposed toppling model considers both the type and condition of discontinuities. The main triggering discontinuities for toppling failure are those with an orientation relatively opposite to the slope face, commonly referred to as anti-dip discontinuities. However, under actual field conditions, toppling failure is often also influenced by other discontinuity sets that intersect the anti-dip fractures. Therefore, the proposed slope model represents the discontinuity condition using a joint fracture network, allowing both deterministic and statistical fracture parameters to be incorporated into the numerical model. The proposed slope model is presented below.

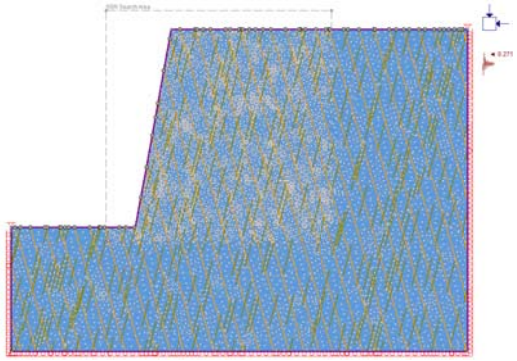


Fig. 4 The slope model of Cliff 05 Citatah. JS1 (yellow), JS2 (green).

In this model, two joint sets are considered, namely Joint Set 1 (JS1) and Joint Set 2 (JS2). JS1 represents the bedding planes, whereas JS2 represents the shear fractures and joints.By using the Barton-Bandis criterion, the properties of JS1 and JS2 were determined. The discontinuity model of JS1 is parallel deterministic, with the average spacing applied, while JS2 is parallel statistical, using a normal distribution of discontinuity. The slope model has a geometry with a slope height of 25 m and a face inclination of 80°.

JS1 has a spacing of 1.5 m and a dip of 74°. Cohesion and friction angle of JS1 are 0.031 MPa and 38.05°. JS2 has a dip of 75, an average spacing of 0.456 m (standard deviation = 0.35), an average length of 4 m (standard deviation = 0.5), and an average persistence of 0.55 (standard deviation = 0.3). Cohesion and friction angle of JS2 are 0.037 MPa and 39.25°. Normal stiffness, shear stiffness, and tensile strength were assumed using the following values for JS1 and JS2, respectively: 100,000 MPa/m, 10,000 MPa/m, and 0.

Groundwater was not incorporated into the FEM model because no groundwater seepage or groundwater level was identified during the field investigation. Therefore, the slope was modelled under dry conditions, and pore-water pressure was assumed to be negligible. The pseudostatic condition in the FEM model was represented by applying a seismic coefficient to simulate earthquake-induced inertial forces acting on the rock slope. The seismic coefficient was defined following SNI 8630:2016. Based on a peak ground acceleration (PGA) of 0.542 g and an amplification factor (FPGA) of 1.0, the horizontal seismic coefficient (Kh) used in the model was 0.271.

## 4.2 Numerical modeling by FEM

The finite element method analysis effectively describes the deformation patterns associated with toppling failure instability. As illustrated in Figure 5, the deformation mesh highlights the toppling failure mechanism occurring at the intersection of Joint 1 and Joint 2. The maximum total displacement is observed at the crown of the slope, ranging from 9.5 to 19 mm, with the toppling movement oriented away from the slope along the discontinuity plane, following the pattern of Joint 2. Although the displacement values remain relatively low, the deformation pattern of toppling failure is distinctly visible.Furthermore, the analysis reveals the yield elements, as shown in Figure 5, indicating the development of a plastic zone concentrated around the slope surface and the joint intersection. This area corresponds to tensile bending stress, which plays a critical role in the progression of toppling failure.

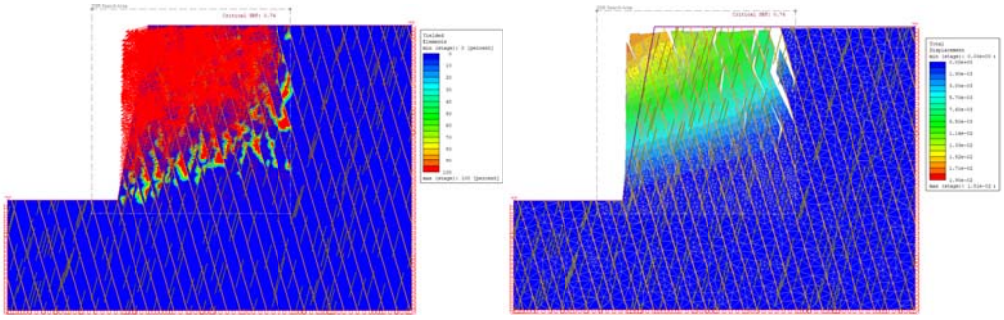


Fig. 5 The deformation model of Cliff 05 Citatah.

## 4.3 Deterministic modeling by LEM

The deterministic LEM analysis was established to represent the block–flexural toppling mechanism controlled by the anti-dipping discontinuity system observed at the study site. The slope geometry was simplified into a 25 m high rock cliff with a slope angle of 80°, while the basal sliding surface was represented by a block base inclination of 16°. The rock mass was divided into several columnar blocks based on the spacing of the main toppling discontinuity, allowing the interaction between individual blocks to be evaluated. This configuration reflects the field condition, where persistent bedding planes form the main anti-dip discontinuity, while intersecting fractures contribute to block separation and potential flexural deformation. The toppling joint has cohesion of 0.031 MPa and a friction angle of 36.86. The parameters input for the rock mass are cohesion of 0.2 MPa, friction angle of 51$^{o}$. Based on those inputs, the FoS of the cliff is 0.746, below the stability criteria of SNI 8460:2017. Figure 6 shows several blocks that potentially topple.

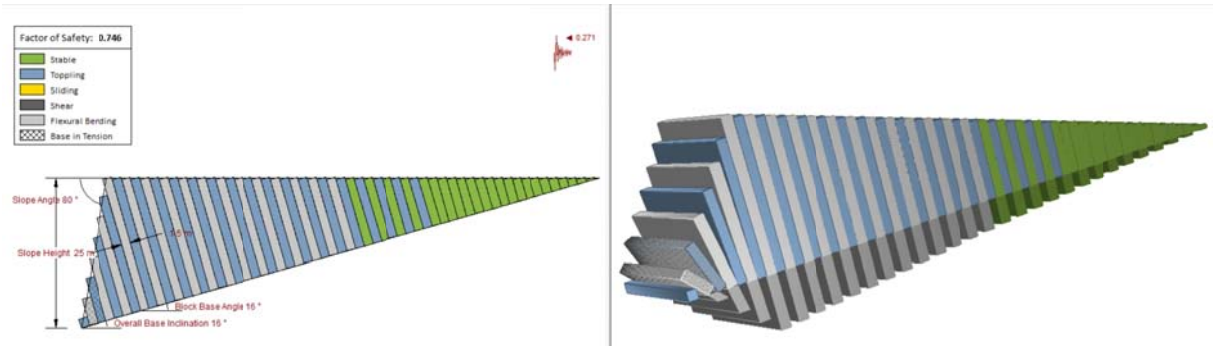



Fig. 6 The block-flexural toppling failure model of the study site.

## 4.4 Sensitivity analysis

A sensitivity analysis was conducted to evaluate the impact of input parameters on the variation of the factor of safety (FoS) in the numerical model. Both slope and toppling joints parameters were included in the analysis. In this case study, the most influential parameters (curves with the highest gradient) in changing the safety factors are slope angle and toppling joint friction angle (Figure 7). The higher the slope angle, the less safe the slope. The higher the toppling joint friction angle, the safer the slope.

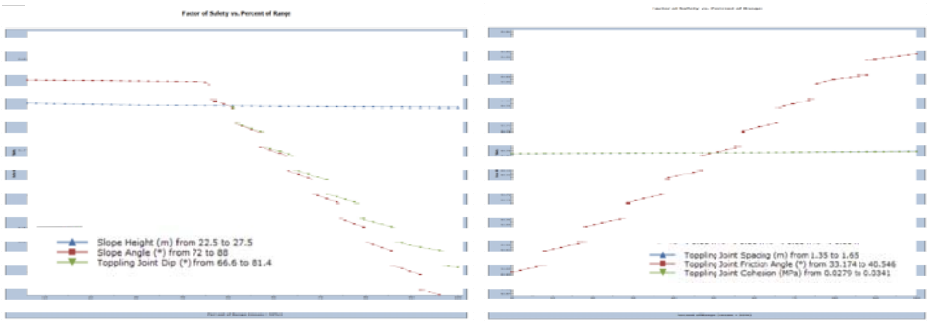



Fig. 7 The sensitivity analysis of the toppling failure parameters.

### 4.5 Discussion

Both FEM and LEM results indicate that Cliff 05 is in the unstable conditions, with FoS values below the stability criterion of 1.1 based on SNI 8460:2017. FEM produced SRF values of 0.74, while LEM gave a lower FoS of 0.746. The comparison of factor of safety (FoS) values indicates that the FEM and LEM analyses produced relatively similar results, with no substantial difference in the estimated slope stability condition. The difference is mainly caused by the different ways each method represents the slope geometry and discontinuity system. The FEM model provides a more realistic representation of the block–flexural toppling mechanism because both joint sets can be included in the model. JS1 represents the bedding planes as the main anti-dip discontinuity, while JS2 represents shear fractures and joints that cut the bedding planes. The deformation pattern and plastic zone around the slope surface and joint intersections show that the instability is controlled by the interaction between these discontinuities. In contrast, LEM provides a simpler deterministic evaluation by dividing the slope into several columnar blocks. This method is useful for estimating the FoS and identifying the general toppling mechanism. However, LEM cannot fully represent the intersecting discontinuities within the rock mass. Overall, FEM is more suitable for interpreting deformation and progressive failure, whereas LEM is useful as a practical preliminary stability assessment. The sensitivity analysis shows that slope angle and toppling joint friction angle are the most influential parameters. Therefore, accurate field characterization of discontinuity orientation and shear strength is essential for reliable block–flexural toppling assessment.

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