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Quantifying Rock Slope Stability with Kinematic and Limit Equilibrium Methods for KM29 of Karak Highway, Malaysia

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Abstract

The stability of rock slopes has been of great interest to engineering geology studies in ensuring a safe and functional cut slope along highways. Kinematic analysis is widely used as an assessment tool for rock slope stability in Malaysia. This method uses a stereograph plot to identify potential failure modes based on geological discontinuities. However, it does not quantify any forces that could influence the potential failure. To address this limitation, the Limit Equilibrium Method (LEM) is employed to calculate the slope's factor of safety, providing a more comprehensive stability assessment. In this study, both kinematic analysis and LEM were applied to evaluate the stability of a rock slope located at KM29 near the Gombak Toll Plaza, along the Karak Highway, Malaysia. Parameters such as discontinuities and mechanical properties were used to analyse the slope. The Schmidt rebound hammer was employed to evaluate the surface hardness of the rock. The average rebound values for slope sections G1, G2, and G3 were 62, 60, and 54, respectively. These values were then correlated with uniaxial compressive strength (UCS), yielding estimated strengths of 163.97 MPa for G1, 150.65 MPa for G2, and 113.98 MPa for G3. The shear strength test indicated an average cohesion value of 20.56 kPa and a friction angle of 56.79°, derived from four rock samples. Kinematic analysis, conducted using Rocscience Dips software, revealed that slope sections G1, G2, and G3 were susceptible to wedge and planar failures. In contrast, the factor of safety (FOS) determined by LEM, simulated using Slope/W, confirmed that all slope sections are stable, with FOS values exceeding 1.5. The integration of kinematic analysis and LEM should be considered essential for evaluating rock slope stability and reinforcing the final decision-making process.

1. Introduction

Over the last few decades, Malaysia has dedicated significant funding to developing a highway network to connect the urban, suburban, and rural areas throughout the country. The challenges frequently encountered are dealing with rugged and hilly terrain. This has frequently resulted in design choices, including cutting rock slopes with varied dip angles, necessitating special considerations to prevent instability and potential rock slope failure [1]. In mountainous or hilly regions, various types of rock slope failures have been observed, including rock, earth, and debris flows. These failures can occur suddenly or gradually. The main trigger for rock slope failure is rainfall; however, earthquakes, other natural causes, and human activities can also contribute to these events. This includes alteration of slope geometry, excessive surcharge load, and improper terrain cutting procedures, to name a few [2],[3].

Many researchers have used various approaches for assessing rock slope stability [4]. Among them are kinematic and non-kinematic methods. Non-kinematic approaches compute the factor of safety (FOS) using the Limit Equilibrium Method (LEM), which takes force-induced instabilities into account. Kinematic analysis, on the other hand, depends on stereographic projection and does not measure the forces that could cause failure.

Kinematic analysis has been commonly utilised since the 1990s to assess potential rock slope failure. It uses discontinuities and joint orientation data collected at the site (such as the dip angle and dip direction) to determine the failure mechanism, i.e., planar, wedge, and toppling [5]. Discontinuities are a critical parameter in rock slope stability, as they typically possess significantly lower strength compared to intact rock. The fundamental principle of the analysis involves examining the angular relationships between discontinuity planes to predict possible failure types in jointed rock masses, without accounting for the forces that might trigger movement. To analyse and visualise structural discontinuity, the Dips 7.0 software is commonly utilised. However, kinematic analysis requires massive data collection (i.e., discontinuity orientation, drill core structural logging data) to represent actual slope conditions at the site [6].

On the other hand, LEM assesses slope stability by calculating the factor of safety, which is the proportion of forces that resist movement to forces that cause movement on the failure plane [7]. This method divides rock mass into a finite number of vertical slices and calculates the forces acting on the potential failure plane, taking into account properties like cohesion, friction angle, unit weight, and others. Several material models are available to represent rock mass, such as Mohr-Coulomb, Shear Normal Function, Barton Brandis, and Hoek Brown. Due to their simplicity and precision, LEM has been the most popular approach for resolving slope stability. However, LEM is unsuitable for modelling slopes with complex geometry, non-linear problems, and heterogeneous materials.

This study utilised both the kinematic and Limit Equilibrium Method (LEM) to investigate the rock slope near the Gombak toll area on Karak Highway, specifically at KM29. The kinematic method was used to identify the failure modes of rock slopes, with the Limit Equilibrium Method (LEM) being used to assess the stability of the slope in terms of Factor of Safety (FOS).

1.1 Research Significance

Performing a rock slope stability assessment within the study region is crucial in evaluating the threat of rock falls and providing possible remedial measures. Suggestions can be made by installing rock anchors, wire mesh, dowels and a slope monitoring system to protect the risky areas [8]. The resulting outcome will be significantly useful for future development planning for highway concessionaires to ensure the safety and effective operation of highway networks. Until now, there has not been any assessment conducted in the study area to determine the stability of rock slopes. Thus, the aim of this study is to evaluate the potential impact of discontinuities on rock slope failure through the integration of the kinematic method and LEM.

1.2 Geological Setting of the Study Area

Karak Highway serves as a main road connecting Kuala Lumpur to the East Coast states in Peninsular Malaysia. The expressway starts in Gombak, Selangor and ends in Lanchang, Pahang. The road was built on the flat to hilly landscape, passing through three distinct formations (i.e., the Kuala Lumpur Granite, the Genting Sempah complex and the Bentong-Raub Suture zone), as illustrated in Fig. 1. These formations can be found in the Main Range region, also known as Western Belt Granites [9]. KM29 falls between coordinates 3°18'55.0"N 101°44'13.0"E, near Gombak, Selangor. The study area has a tropical rainforest climate with average monthly temperatures ranging from 33.88°C to 24.63°C. The area receives approximately 77.89 mm of rainfall intensity for approximately 212 days, or 58.13% of the year.

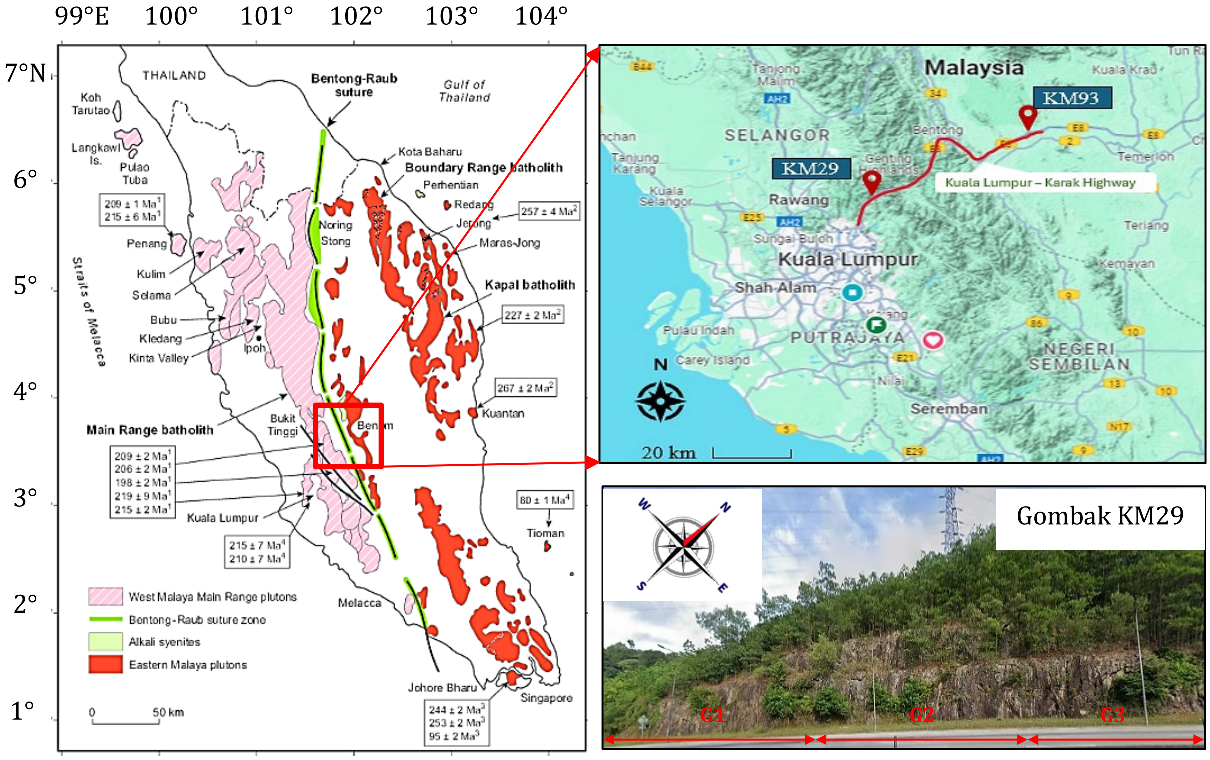


Fig. 1 *Location of study area (modified from [10])*

2. Methodology

The methodology of this study consists of three stages. The first stage involved site visits, field testing and rock sample collection. The next step was to perform laboratory tests and assess the stability of rock slopes using kinematic and limit equilibrium methods (LEM).

2.1 Site Visits, Field Testing and Rock Samples Collection

Site visits were carried out to collect comprehensive data on the slope, including its dimensions, discontinuity orientations, results from field tests, and rock samples for laboratory analysis. The slope section at Kilometre 29 is divided into three segments–G1, G2, and G3–with lengths of 550 m, 355 m, and 345 m, respectively. The total slope length is approximately 1,250 m, with a maximum height of 15 m occurring at section G2. The slope angle for each slope section is determined at the site at approximately 88°, 78°, and 80°. A total of 950 dip angle (DA) and dip direction (DD) measurements were collected using a GeoCompass application, with 400, 350, and 200 data points recorded for sections G1, G2, and G3, respectively.

Research conducted by Zangerl et al. [11] suggested that between 150 and 350 measurements are sufficient to describe a uniform region of rock mass in each segment. The data collected were verified on-site using a geological compass. Rock samples were collected using a geological hammer for shear strength testing, while 600 surface hardness measurements were obtained using a Schmidt rebound hammer.

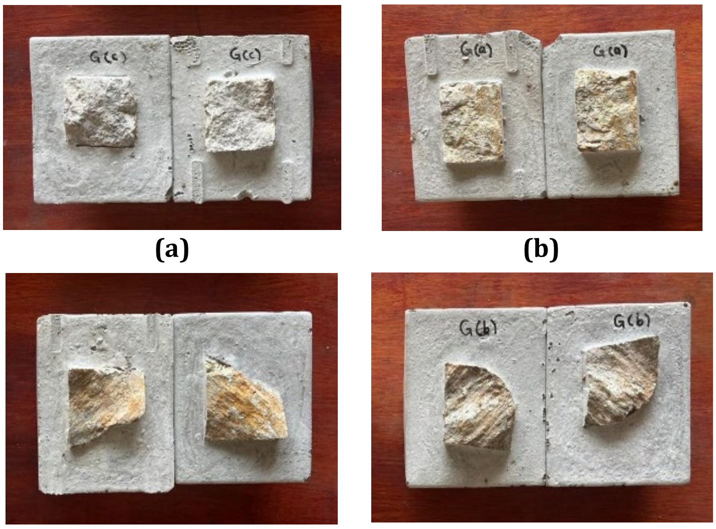
2.2 Laboratory Testing

2.2.1 Shear Strength Test

The shear strength test was conducted using a portable rock shear box apparatus on four rock samples (S1, S2, S3, and S4) collected randomly from G1, G2, and G3 slope sections. The procedure was performed in accordance with ASTM D5607-16 [12]. Four blocks of granite containing joint surfaces were prepared by cutting them into rectangular shapes with a size of 115 x 125 mm. The lower half and upper half samples were encapsulated in separate moulds, and then cement mortar was poured into the shear box mould. The prepared samples for the shear strength test are shown in Fig. 2.

Before the test, the encapsulated lower half and upper half of the rectangular shape sample were aligned carefully so that the plane to be sheared was in a horizontal position. A 1kN force was exerted on the specimen, and the shear force was raised until the displacement gauge showed readings of 2 mm, 4 mm, 6 mm, and 8 mm.

The shear load reading was recorded for each displacement increment. These processes were repeated for another three samples, with increment normal load values of 2 kN, 3 kN, and 4 kN.



(a) (b) (c) (d)

Fig. 2 *Encapsulated rock samples for shear test: (a) Sample 1 (S1); (b) Sample 2 (S2); (c) Sample 3 (S3); and (d) Sample 4 (S4)*

2.2.2 Surface Hardness Test

The Schmidt rebound hammer test is employed in this study to ascertain the uniaxial compressive strength (UCS) of rock due to its simplicity, rapid execution, and portability. To conduct the test, the hammer is placed perpendicularly on the rock surface, and the plunger is pressed against the rock, as illustrated in Fig. 3. The rebound value, indicating surface hardness and serving as an indirect measure of compressive strength, is recorded to the nearest whole number for each impact [13]. These readings are then averaged to obtain a representative rebound number. Finally, the average rebound value is correlated with the UCS using Eq. (1) [14], where Hr denotes the Schmidt hammer rebound value.

$$UCS = \frac{6222}{88.15 - Hr} - 70.38 \tag{1}$$

The Schmidt hammer is preferred over UCS testing for quick, non-destructive, and cost-effective preliminary field assessments, especially in large-scale surveys where sample preservation is crucial. It is important to note that there are limitations and potential sources of error associated with this correlation. Factors such as rock type, anisotropy, moisture content, and surface condition can influence the relationship between rebound values and UCS. Variations in mineral composition and grain size result in differing hardness levels among rock types, which in turn affect rebound values. Moreover, surface conditions such as weathering or the presence of micro-cracks can impact the accuracy of the measurements. To minimise errors, this study accounted for proper calibration and ensured consistency in the application of impact force. Table 1 provides a classification of rock strength based on UCS values.

Fig. 3 *Collecting rebound data using the Schmidt hammer*

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Table 1 *Rock strength is classified into different categories based on UCS value*

Strength Classification	UCS Range (MPa)	Typical Rock Types
Very Weak	10 - 20	Weathered and weakly compacted sedimentary rocks
Weak	20 - 40	Weakly cemented sedimentary rocks, schists
Medium	40 - 80	Competent sedimentary rocks, some low-density coarse-grained igneous rocks
Strong	80 - 160	Competent igneous rocks, some metamorphic rocks, and fine-grained sandstones
Very Strong	>160	-

3. Results and Discussion

The mean cohesion and friction angle values obtained from the analysis of four rock samples were 20.56 kPa and 56.79°, respectively. The high friction angle is attributed to the interlocking texture of mineral grains in granite, which is predominantly composed of quartz–a hard, weathering-resistant mineral [15]. In comparison, the shear strength parameters of biotite granite saprock at KM31 along the Karak Highway were reported as 14.5 kPa for cohesion and 34.3° for the friction angle [16], which are notably lower than those at KM29. The difference is due to weathering that has changed the granite saprock at KM31 into the soil, thus reducing its strength.

The UCS values are tabulated in Table 2, based on classification in Table 1. The rock slope sections G2 and G3 are both classified as strong rock, while slope section G1 is classified as very strong rock [17]. Strengths differ depending on various factors, including weather, the presence of fractures or gaps, and in-situ stress. Granite, in its pristine, unweathered state and with minimal fractures, boasts the highest strength. In contrast, weathered granite with unfavourably oriented fractures tends to be weaker and more susceptible to failure.

Table 2 *Schmidt hammer rebound value and corresponding UCS (MPa)*

Location	Symbol	Schmidt hammer rebound value	Average of the five highest values	UCS (MPa)
Gombak KM29	G1	64,63,62,60,59	62.00	163.97
	G2	62,61,60,59,58	60.00	150.65
	G3	58,55,54,53,52	54.00	113.98

3.1 Stability Analysis

3.1.1 Kinematic Analysis

The kinematic analysis was performed through Rocscience Dips 7.0 software to evaluate possible types of failure mechanisms, i.e., planar, wedge and toppling [18]. Section G1 has a slope orientation of 88°/125°, G2 is oriented at 78°/127°, and G3 at 80°/154°. Planar failure was identified in all three sections, as shown in Fig. 4(a), Fig. 4(d), and Fig. 4(g). The planar failure was identified by determining the pole of the joint set that fell within a critical zone (highlighted in red). In section G1, the pole of joint set F3 fell within the critical zone, indicating that F3 caused the planar failure, as illustrated in Fig. 4(a). Similarly, the joint pole F2 located in section G2 was situated in a critical zone, indicating that it was the cause of the failure (see Fig. 4(d)). In section G3 (see Fig. 4(g)), planar failure was associated with joint set F2, which is situated in the critical zone as well.

Wedge failures were identified in slope sections G1, G2, and G3 due to the intersection of joint sets within the critical zone. In section G1, as depicted in Fig. 4(b), the intersection of joint sets F3 and F2 resulted in wedge failure, with the potential for the wedge to slide toward 88°. In section G2, shown in Fig. 4(e), wedge failure was caused by the intersection of joint sets F1, F3, and F4 with F2, as well as F3 intersecting with F4, all occurring within the critical zone, leading to a probable failure direction toward 78°. In section G3 (see Fig. 4(h)), the discontinuity of F5 intersects with F2 and F3, while F3 intersects with F4, all within the critical zone, suggesting a potential slide direction toward 80°. However, no toppling failures were observed in slope sections G1, G2, and G3. The summary of kinematic analysis was tabulated in Table 3.

The analysis indicates that wedge failures are more likely to occur than planar or toppling failures. This is primarily due to the presence of intersecting planes of weakness that define the wedge geometry, providing multiple potential failure directions. Unlike planar or toppling failures, which typically involve a single discontinuity or axis, wedge failures are structurally more complex and can mobilise larger rock volumes, increasing the driving forces. Furthermore, the alignment of the wedge with the slope face often facilitates sliding along the line of intersection. These factors collectively contribute to a higher likelihood of wedge failures in rock slopes.

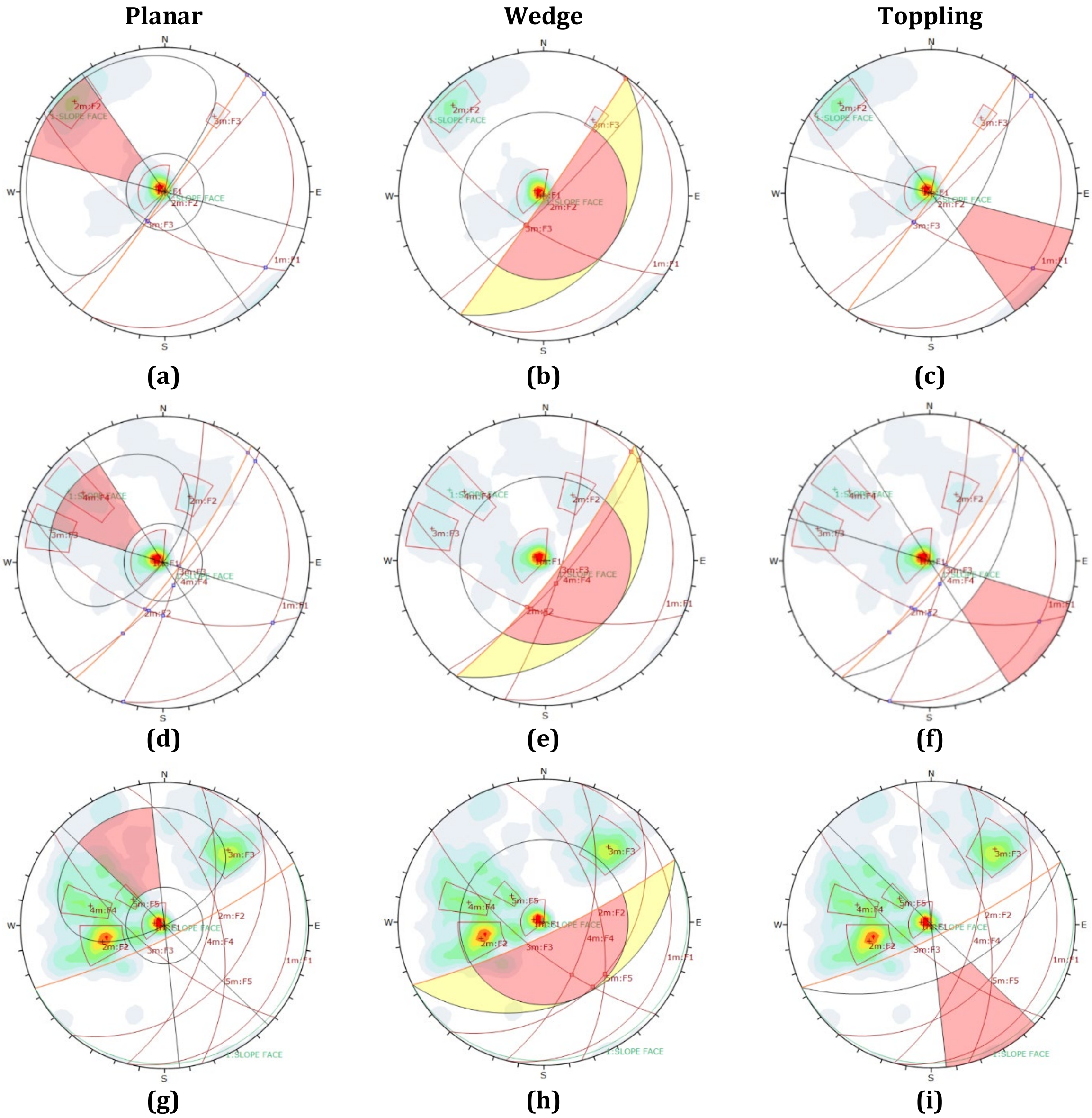


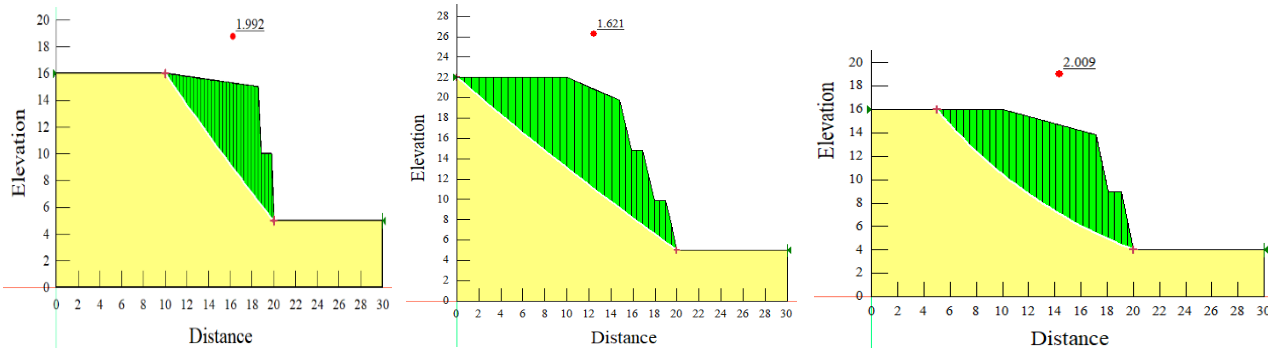
Fig. 4 *Kinematic results using Rocscience Dips software for slope section: (a) to (c) G1, (d) to (f) G2, and (g) to (i) G3 for planar, wedge, and toppling mode of failures*

Table 3 *Summary of kinematic analysis at KM29*

Section	Modes of failure	Slope face orientation (DA/DD)	Percentage of failure (%)
G1	Planar	88°/125°	20
	Wedge		50
	Toppling		0
G2	Planar	78°/127°	14.3
	Wedge		60
	Toppling		0
G3	Planar	80°/154°	8
	Wedge		20
	Toppling		0

3.1.2 Non-Kinematic Analysis

The stability analysis of all slope sections was performed using the LEM approach, simulated using Geostudio 2023 through Slope/W software. The rock material was modelled using the Shear-Normal function. The input parameters included the unit weight, cohesion, and friction angle. The unit weight of granite is obtained from previous study, with values of 18 kN/m^3 [19]. The LEM results indicated that the factor of safety (FOS) for all slope sections exceeded 1.5, as shown in Fig. 5(a) to Fig. 5(c), indicating that the slope is considered stable [20].



(a) (b) (c)

Fig. 5 *Slope stability analysis using LEM for rock slope section (a) G1, (b) G2 and (c) G3*

Kinematics and LEM revealed a contradictory result. The main limitation of LEM is that it overlooks the condition of the rock mass. As a result, a highly fractured rock mass may be incorrectly represented as stable and posing no risk of failure.

Slopes that exhibit kinematic instability are susceptible to planar, wedge, or toppling failures, especially when the discontinuities are poorly oriented. However, it should be noted that kinematic instability alone does not guarantee slope failure. This is because the limit equilibrium factor of safety may still exceed 1.0. Conversely, slopes with a limit equilibrium factor of safety greater than 1.0 are regarded as stable, even if there are potential failure modes identified through kinematic analysis. The limit equilibrium factor of safety serves as an indicator of the safety margin and the probability of failure [21]. Kinematic analysis identifies potential failure modes but does not quantify stability, while limit equilibrium calculates the safety factor against failure. A slope may be kinematically unstable but still have an adequate limit equilibrium FOS. Therefore, incorporating both kinematic and Limit Equilibrium Method (LEM) analyses is essential for a comprehensive evaluation of rock slope stability and for guiding subsequent engineering decisions.

4. Conclusions

This study considered both kinematic and LEM analyses to assess rock slope stability at KM29 of the Karak Highway. Parameters such as discontinuities and mechanical properties were used to analyze the slope. The surface rock hardness was measured using the Schmidt rebound hammer, yielding an average reading of 62, 60, and 54 for slope sections G1, G2, and G3, respectively. The results were correlated to the UCS and the equivalent rock strength of approximately 163.97, 150.65, and 113.98 MPa. Experiments conducted on four rock specimens revealed an average cohesion value of 20.56 kPa and a friction angle of 56.79°. Kinematic analysis using Rocscience Dips revealed that slope sections G1, G2, and G3 are susceptible to wedge and planar failures. In contrast, the Limit Equilibrium Method (LEM), modeled using Slope/W, indicated that all slope sections are stable, with factors of safety exceeding 1.5. These contrasting results highlight the importance of integrating both kinematic and LEM analyses to achieve a more comprehensive and reliable assessment of rock slope stability and to support well-informed engineering decisions.

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Conflict of Interest

The authors declare that there is no conflict of interest regarding the publication of the paper.

Author Contribution

The authors confirm contribution to the paper as follows: ***Study conception and design****: Nurul Afiffah Khairulazman, Jestin Jelani & Zuliziana Suif;* ***Data collection****: Nurul Afiffah Khairulazman & Jestin Jelani;* ***Analysis and interpretation of results****: Nurul Afiffah Khairulazman, Jestin Jelani, Zainuddin Md Yusoff & Mohd Rozi Umor;* ***Draft manuscript preparation****: Nurul Afiffah Khairulazman, Jestin Jelan & Suriyadi Sujipto. All authors reviewed the results and approved the final version of the manuscript.*

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