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RESEARCH PAPER

STABILITY ASSESSMENT OF ROCK SLOPES USING KINEMATIC ANALYSIS AND SLOPE MASS RATING IN SELECTED AREA, MERGASUR TOWN, ERBIL, NE-IRAQ

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A B S T R A C T:
Instability of rock slopes appears to be influenced by the slope height, slope angle, and orientations of the geological discontinuities (bedding planes, joints, faults, etc.). Furthermore, the kinematic technique is the most practicable method for evaluating the rock slope stability that includes joint sets. The stability of eight rock slopes in the research area was assessed using the kinematic approach with DIPS v6.008 software and slope mass rating system (SMR) and continuous- slope mass rating system (Con-SMR) with SMRTool - v205 software. Based on the kinematic result the planar sliding may occur in stations 1–5 and 7, wedge sliding may occur in stations 4, 5, 6, 7 and 8, flexural toppling may occur in stations 1, 2, 7 and 8, and direct toppling may occur in station 3. According to the SMR-Tool software results for discrete-(SMR) revealed that station no.1, 2, 3, 4, 5 and 7 (planar sliding), station 4 (wedge sliding) and 7 (flexural toppling) was shown to be a partially stable slope with failure probability of 0.4. Also, station no.1, 2 and 8 (flexural toppling) and station no. 5, 6, 7 and 8 (wedge sliding) were determined to be a stable slope with a failure probability of 0.2. Then, station no. 3 (direct toppling) was shown to be a completely stable slope with failure probability of 0. But in Con-SMR assessment in a comparison to discrete-SMR, station no. 1 and 7 (planar sliding) and 4 (wedge sliding) is unstable slopes with a failure chance of 0.6. Then station no. 2, 3, 4 and 5 (planar sliding) and station 7 (flexural toppling) are partially stable slopes with failure probability of 0.4. But, station no.1, 2 and 8 (flexural toppling) and station 5, 6, 7 and 8 (wedge sliding) were determined to be a stable slope with a failure probability of 0.2. Nonetheless, station no. 3 (direct toppling) was shown to be a completely stable slope with failure probability of 0.

KEY WORDS: Slope mass rating; Continuous slope mass rating; Kinematic analysis; Mergasur town
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1.INTRODUCTION:

Mostly road cuttings in mountainous areas have rock slopes that are likely to identify instability issues due to changes in rock mass conditions and external factors generated by the environment, such as seismic activity and water in the slope (Pantelidis, 2009). In addition, the features like the rock slope, slope height, slope angle, and rock discontinuity orientations (bedding planes, joints, faults, etc.) seem to contribute to the unstable condition of road-cut slopes (Hoek and Bray, 1981). Because of the impact of rock slope instability on human life and property, road stability assessment is required along and surrounding the road (Qader et al., 2020).

Kinematic, deterministic, and probabilistic approaches were frequently used to assess rock slope stability (Hoek and Bray, 1981; Abramson et al., 2002; Wyllie and Mah, 2004).
In this research, eight stations on the rock slopes were chosen. Eight rock slope stations were evaluated by kinematic analysis using the DIPS v6.008 software (Rocscience, 2015) to assess stability and identify the type of slope failure. By using SMRTool-v205 (Riquelme et al., 2016), the stability of the eight identified rock slopes was also investigated in order to determine failure types, stability circumstances, and failure probability. This allowed for the suggestion of suitable support measures. Field investigations were carried out in July and August of 2021. The

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dip direction/dip angle measurements were performed on the slope face, bedding planes, and joints. Additionally, the slope mass rating (SMR) parameters were recorded using the categorization system established by Romana et al. (2003) and Tomas et al. (2007), and the rock mass characteristics recommended by Bieniawski (1989) were also recorded. The failure types (planar sliding, wedge sliding, and toppling) and likelihood of failure for natural and excavated slopes have been understood using these slope stability analysis techniques (Hamasur et al., 2020; Hostani and Hamasur, 2022). The study area is located in northeast Iraq, 144 kilometres northeast of Erbil. The location is near the village of Lerabire in the Barzan sub-district, between latitudes 36° 54′ 52.18″ and 36° 56′ 7.92″ North, and longitudes 44° 11′ 2.46″ and 44° 09′ 19.74″ East. The rock slope stations in the study area appear in Fig. 1. There have been previous investigations into this topic in a number of geological fields, including structural geology, sedimentology, stratigraphy, hydrology, and geomorphology. The current study aims to prove rock slope stability analysis for selected rock slopes applying (SMR), (Con-SMR) and kinematic analysis.

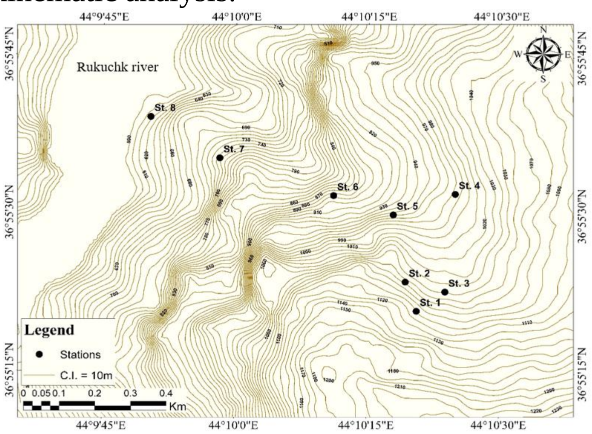

Figure 1: The study area's rock slope stations are indicated on a topographic map

2. GEOLOGICAL SETTING:

The geological setting of the studied area includes tectonics, structure and stratigraphy. The geological setting of the study area is described briefly;

2.1 Tectonics and Structural Geology

Due to the complexity of the structural geology, the study area is located in the imbricated zone (Al-sinawi and Al-Qasrani, 2003). Shirin and Bradost anticlines are distinguished by the presence of numerous large faults of various scales and types dispersed throughout the anticline (Fouad, 2010; Jassim and Buday, 2006; Jassim and Goff, 2006). Omar (2005) explained that some of these faults are reverse types and appear particularly in the south western limb, while others are normal or strike-slip faults. Shirin and Bradost's anticline are an asymmetrical, cylindrical anticlinal fold that extends for more than 60 kilometres and trends NW-SE. Its half wavelength, measured from the middle section, can reach up to 8 km, while its amplitude is roughly 2 km (Balaki and Omar, 2019). The northwest plunge of Bradost anticline is in en-echelon position with Shirin anticline in southeast Barzan town near Dore village (Qader et al., 2020). While the southeast plunge is located in Rawanduz town, it interfered with the Korek anticline and is an en-echelon with the Handrin anticline. The Gali Rukuchk is formed from the cutting (dissection) of the Shirin and Bradost anticlines (Ahmed, 2019). The majority of the research area is situated at the Shirin Anticline's south-eastern plunge.

2.2 Stratigraphy

Two formations which range from the Early to Middle Cretaceous are the exposed geological units in and around the research area. According to Figs. 8 to 11 (rock slope stations no. 5 to 8), the oldest unit of the lower Cretaceous age and the Qamchuqa Formations are classified as the core of the majority of the anticlines in the research area. Bekhme Formation is characterized as an Upper Cretaceous age with carbonate and impure carbonate rocks as Figs. 4-8 respectively (rock slope stations no. 1 – 4) (Qader et al., 2020). These geological units from older to younger are shown in Fig. 2.

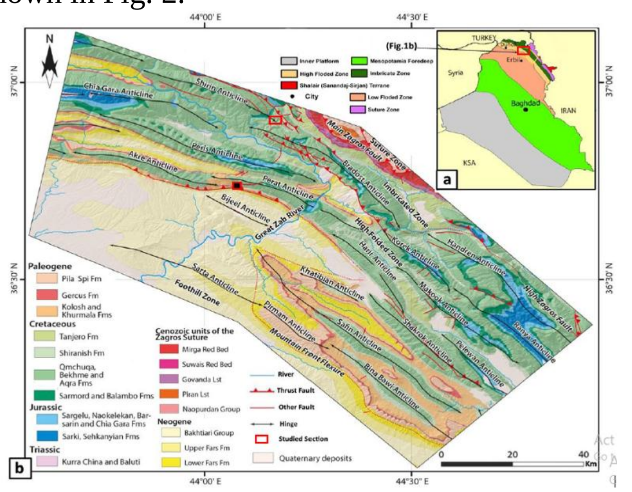

Figure 2: a) Location map of the research area with a map of Iraq's tectonic divides (After Fouad, 2015); b) Geological and structural map of investigated area (After Zebari et al., 2019)

3. MATERIALS AND METHODS:

A geological engineering survey was completed in July and August of 2021 for eight rock slope stations, the first, four stations (station 1, 2, 3 & 4) are located in Bekhma Formation and the other four stations (station 5, 6, 7 & 8) located in Qamchuqa Formation. The field measurements expressed in dip direction/dip angle manner for all discontinuities attitude (bedding planes and joints) and slope attitude. Slope angle and direction, dip and dip direction of discontinuities, spacing, and condition (persistence, roughness, weathering, aperture, filling materials) of discontinuities and groundwater condition are measured and described in the field. The strength of the rock sample was determined using a point load test of Broch and Franklin (1972) as recommended by ISRM (1985).

The basic purpose of the study is to assess the stability of rock slopes using kinematic method, SMR and Con-SMR systems.

As stated by Raghuvansh (2017), the kinematic technique depends on the kinematic principle, which deals with the geometric requirements for the movement of the rock block over the discontinuity plane without considering any factors that may be cause for the failure.

The kinematic approach is a simple practice for identifying potential types of failure (plane, wedge & toppling) using stereographic analysis (Qader and Syan, 2021). Because of the angular correlations between discontinuities and the slope surface, it can also be used to determine the orientation of a jointed rock mass (Hoek and Bray, 1981; Yoon et al., 2002). When a wedge-shaped mass slides along the line where two planes connect, it can cause a wedge failure, Markland (1972) is one of the kinematic analytic techniques for evaluating this risk. Hocking (1976) modified and explained the Markland method.

DIPS v6.008 software was used to graphically evaluate the field data (Rocscience, 2015). The cohesion of a planar and clean (no infilling) discontinuity is zero, and the shear strength is specified primarily by the discontinuity friction angle. In the field, friction angles of 31°-33° were estimated using the Bruce et al. (1989) tilting approach.

Slope Mass Rating (Romana, 1985, 1993) is determined using the first five basic parameters of Bieniawski's (1989) RMR and four adjustment factors. These adjustment factors are determined by the existing relationship between discontinuities intersecting the rock mass and the slope, as well as the method of slope excavation. As shown in Table 1, Anbalagan et al. (1992) modified the SMR approach for corporate wedge failure, plane failure, and toppling failure. Tomas et al. (2007) created Con-SMR, which is a version of Roman's discrete SMR approach. The field guidelines, slope support remediation method and adjustment factors proposed by Romana (1985) and Romana et al. (2015) for SMR system. Con-SMR suggests a unique value for each adjustment variable, as alternative to a range as in discrete SMR.

By compiling the rating values for the following five parameters; uniaxial compressive strength (UCS) of intact rock, rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, and ground water conditions, Bieniawski (1989) estimated the RMRb.

SMR is calculated from expression (1) using RMRb and various adjustment factors given by Romana (1985).

$$\mathrm{SMR} = \mathrm{RMRb} + (\mathrm{F1} \times \mathrm{F2} \times \mathrm{F3}) + \mathrm{F4} \quad \ldots\ldots\ldots (1)$$

By applying RMRb and adjustments parameters to calculate from expression (1), the Con-SMR is obtained, the difference is in computing for the adjustment factors (F1, F2, and F3) proposed by Tomas et al. (2007), as well as the factor F4 depending on the excavation method as shown in Table 1. The SMRTool software was used for computation of both discrete SMR and Con-SMR (Riquelme et al., 2016), and those factors also depend on the discontinuity-slope relationship.

Romana (1985) defined four criteria that collectively determine the adjustment rating for SMR;

- F1 is the rating of the difference between the dip direction of a discontinuity and the slope face or the plunge direction of two discontinuities and the slope face, as indicated in Table 1.
- As shown in Table 1, F2 is the rating of the dip angle or plunge angle of the intersection line of two discontinuities.
- According to Table 1, the rating for the difference between the dip angles of a discontinuity and a slope, or between the plunge angles of a line intersecting two discontinuities and a slope angle, is F3.
- In Table 1, a correction factor called F4 is influenced by the excavation procedure.

Table 2 presents various stability classes, SMR limit values, and failure modes. Romana (1985) provided several remediation methods based on SMR values; additionally, Fig. 3 shows the relationship between SMR value and remediation approach with comprehensive engineering fieldwork (Romana et al., 2015).

Table 1. SMR adjustment factors (Modified from Romana (1985)) by Anbalagan et al. (1992).

TYPE OF FAILURE			VERY FAVORABLE	FAVORABLE	NORMAL	UNFAVORABLE	VERY UNFAVORABLE
P	A	$\lvert\alpha_j-\alpha_s\rvert$	>30°	30-20°	20-10°	10-5°	<5°
T		$\lvert\lvert\alpha_j-\alpha_s\rvert-180\rvert$					
W		$\lvert\alpha_i-\alpha_s\rvert$					
P/T/W	F_1		**0.15**	**0.40**	**0.70**	**0.85**	**1.00**
P/W	B	$\lvert\beta_j\rvert$ or $\lvert\beta_i\rvert$	<20°	20-30°	30-35°	35-45°	>45°
P/W	F_2		**0.15**	**0.40**	**0.70**	**0.85**	**1.00**
T			**1.00**				
P	C	$\beta_j-\beta_s$	>10°	10-0°	0°	0-(-10°)	<(-10°)
W		$\beta_i-\beta_s$					
T		$\beta_j+\beta_s$	<110°	110-120°	>120°	-	-
P/T/W	F_3		**0**	**-6**	**-25**	**-50**	**-60**
EXCAVATION METHOD (F_4)							
Natural slope				+15	Blasting or mechanical		0
Presplitting				+10	Deficient blasting		-8
Smooth blasting				+8			
Where:- P: planar failure; T: toppling failure; W: wedge failure; αj: joint dip direction; αs:slope direction αi:intersection line direction; βj:joint dip angle; βi:intersection plunge angle; β_s:slope angle							

Table 2. Slope mass rating (SMR) classes (Romana, 1985).

Classes →	V	IV	III	II	I
SMR	0-20	21-40	41-60	61-80	81-100
Description	Very bad	Bad	Normal	Good	Very good
Stability	Completely unstable	Unstable	Partially stable	Stable	Completely stable
Failures	Big planar or soil-like	Planar or big wedges	Some joints or many wedges	Some blocks	None
Failure probability	0.9	0.6	0.4	0.2	0

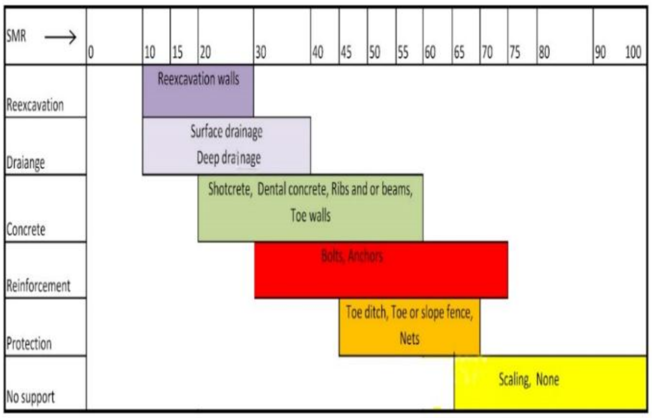

Figure 3: Slope support suggestions are based on SMR (Romana, 1985) in Romana et al. (2015).

4. RESULTS AND DISCUSSION:

To determine the discontinuity pattern, eight rock slopes were chosen in the study area based on structural geological appearance, topographic conditions and lithology characteristics. Because of their geotechnical properties, the rock slopes in the research area have been thoroughly investigated. The rock slopes in Bekhma and Qamchuqa Formation are made of a massive bed of limestone and dolomitic limestone bed.

As shown in Table 3, the rock slopes have a variable slope angle with a developed system of discontinuities. The slope, bedding plane, and joint were measured in the field using a Silva compass, as well as the dip angle and dip direction. The tilting method has been used to determine the friction angle of discontinuity failure surfaces (Bruce et al., 1989), which is 31° and 32° for Limestone and 33° for Dolomitic limestone. Table 4 contains the findings of all kinematic analyses.

Table 3. Field data and measurements in the rock slope stations

Station No.	Rock type	Formation	Dip Direction / Dip Angle				Friction angle
			Slope	Bedding	J1	J2	
1	Limestone	Bekhma	052°/69°	051°/35°	255°/42°	242°/85°	31°
2			053°/62°	060°/35°	256°/48°	248°/73°	31°
3			050°/38°	035°/37°	157°/69°	245°/62°	32°
4			242°/44°	360°/09°	230°/42°	160°/80°	32°
5	Dolomitic limestone	Qamchuqa	285°/53°	030°/65°	200°/27°	266°/50°	33°
6			278°/80°	060°/12°	300°/78°	255°/73°	33°
7			240°/70°	060°/68°	250°/49°	132°/56°	33°
8			292°/73°	091°/61°	220°/52°	320°/59°	33°
J1, J2 and J3 = Joint set numbers							

For conducting kinematic analysis in the examined rock slopes, average attitude of slopes and discontinuities, and sliding friction angle of rock discontinuities were utilized. The DIPS v6.008 software was used to identify probable failure zones; additionally, all rock slopes, discontinuities (bedding planes and joints), and potential failure zones were represented in pink color in stereographic projection were represented as poles (perpendicular to the plane). The following kinematic study of rock slopes was determined:

- Potential for planar sliding at stations no 1 to 5 and 7 as shown in Figs. 4a, 5a, 6a, 7a, 8a and 10a respectively, also 4b, 5b, 6b, 7b, 8b and 10b respectively.
- Potential for wedge sliding at stations no. 4 to 8 as shown in Figs. 7a, 8a, 9a, 10a and 11a respectively, also 7c, 8c, 9b, 10c and 11b respectively.
- Potential for flexural toppling at stations 1, 2, 7 and 8 as shown in Figs. 4a, 5a, 10a and 11a respectively, also 4c, 5c, 10d and 11c respectively.
- Potential for direct toppling at stations 3 as shown in Figs. 6a and 6c.

In kinematic analysis stereographic projection figures the slope face =SF; bedding plane=So; joint set no.1= J1; joint set no.2. =J2; joint set no.3=J3; intersection between two discontinuity sets= I. The failure mode (planar, wedge, flexural toppling and direct toppling) direction is northeast to northwest ranging from 28^0 to 320^0, represented on the stereonet as an arrow also in Table 4.

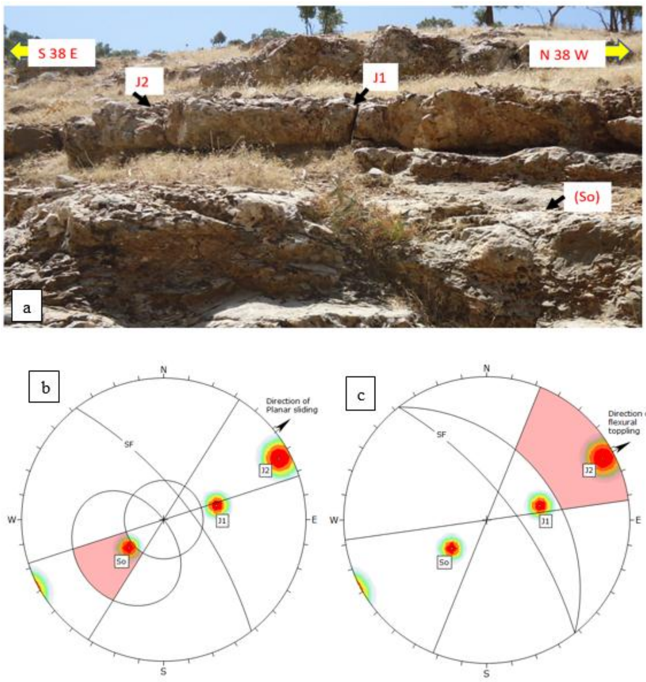

Figure 4: a-Field image with distinct discontinuity sets for station no. 1. b- Planar sliding is shown on the bedding plane (So) in the kinematic analysis for station no. 1. c- Flexural toppling about J2 is visible in the kinematic analysis for station no. 1.

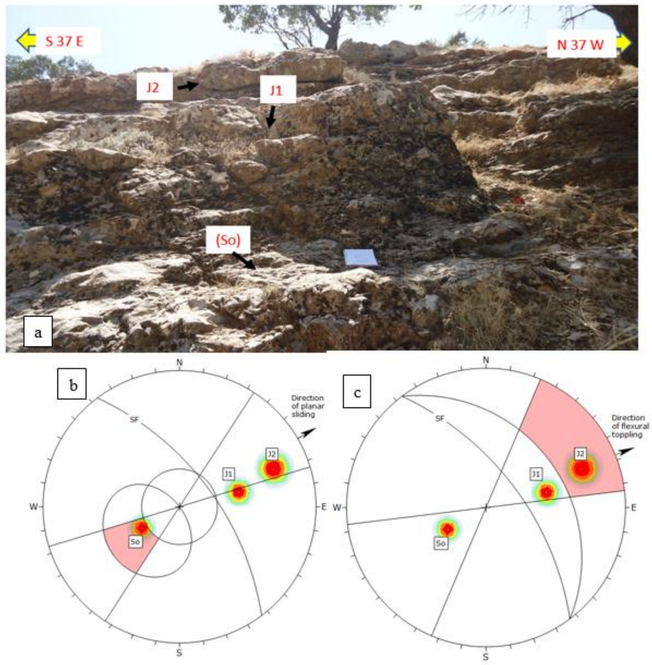

Figure 5: a- Field image with distinct discontinuity sets for station no. 2. b- Planar sliding on the (So) is seen in kinematic analysis for station no. 2. c- shows flexural toppling about J2.

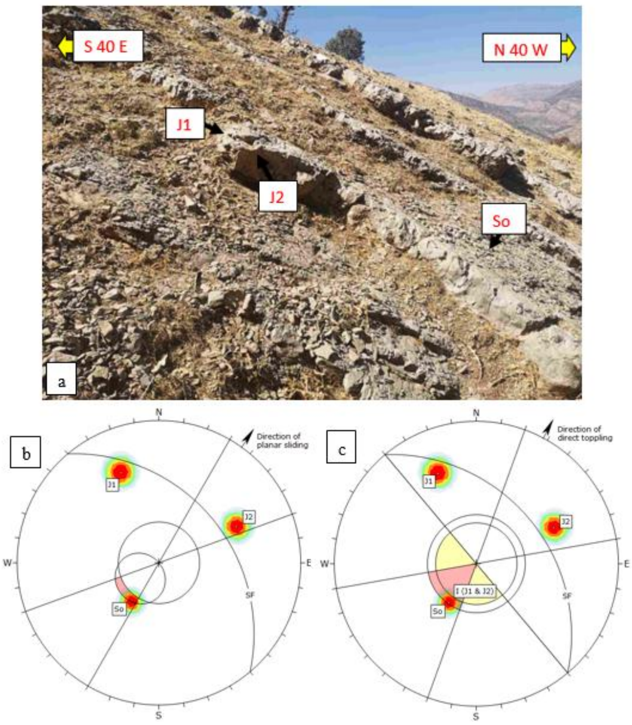

Figure 6: a- Field image with distinct discontinuity sets for station no 3. b- Planar sliding on the (So) is seen in kinematic analysis for station no. 3. c- Direct toppling is seen in intersected release planes J1 and J2 (I (J1 & J2)) in kinematic analysis for station no. 3 with the aid of So as a basal plane.

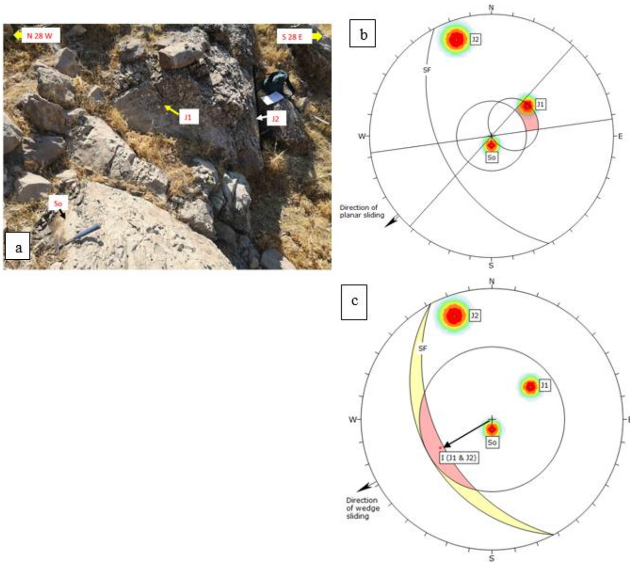

Figure 7: a- Field image with distinct discontinuity sets for station no 4. b- Planar sliding on the (J1) is seen in kinematic analysis for station no 4. c- Wedge sliding on I (J1 and J2) is seen in kinematic analysis for station no. 4.

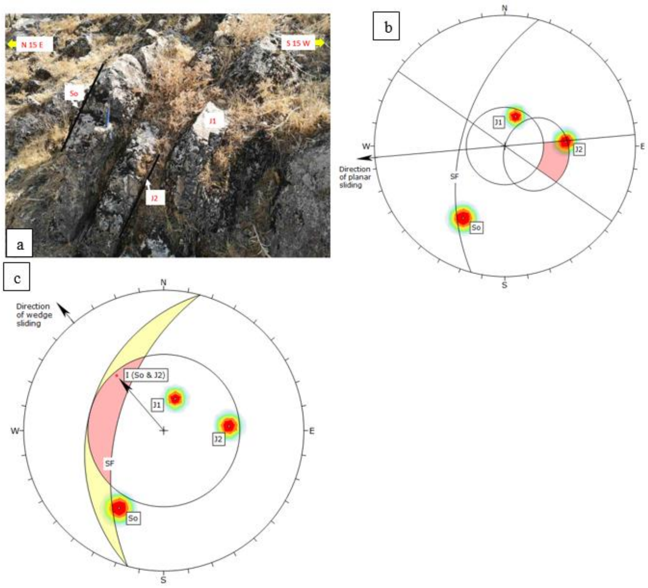

Figure 8: a- Field image with distinct discontinuity sets for station no 5. b- Planar sliding on the (J2) is seen in kinematic analysis for station no 5. c- Wedge sliding on (So & J2) is seen in kinematic analysis for station no 5.

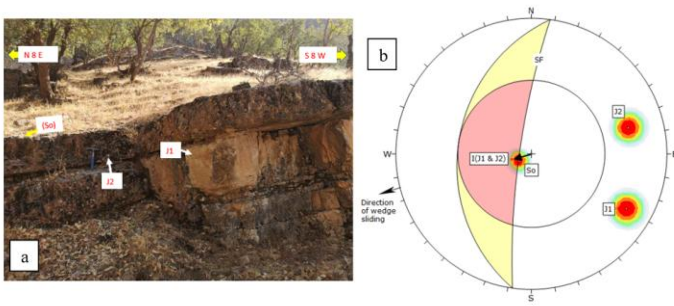

Figure 9: a- Field image with distinct discontinuity sets for station no 6. b- Wedge sliding on J1 and J2 is seen in kinematic analysis for station no 6.

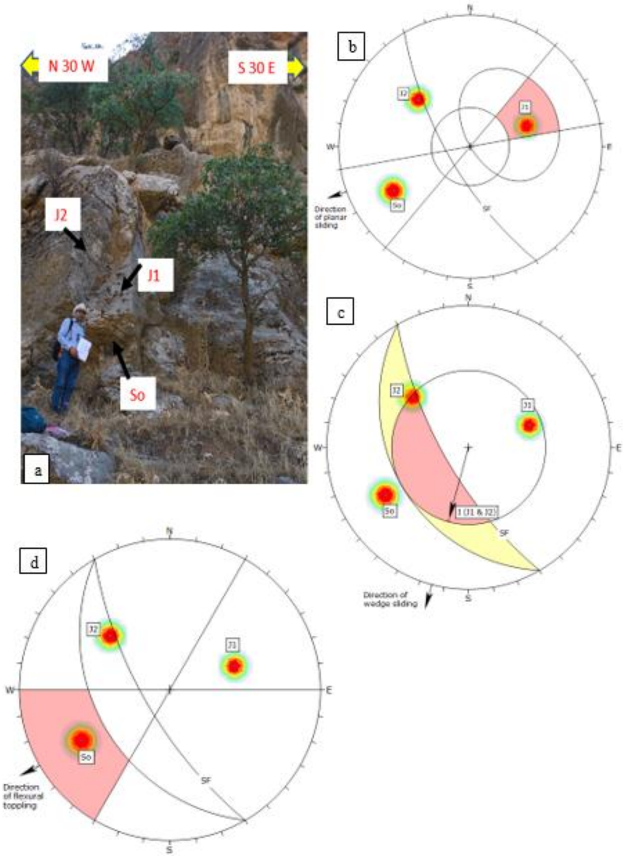

Figure 10: a- Field image with distinct discontinuity sets for station no 7. b- Planar sliding on the J1 is seen in kinematic analysis for station no.7. c- Wedge sliding on (J1 & J2) is seen in kinematic analysis for station no. 7. d- Flexural toppling about So is seen in kinematic analysis for station no. 7.

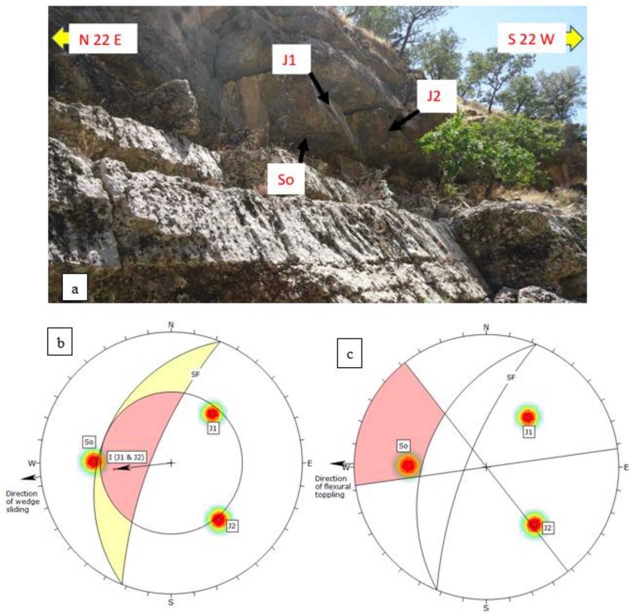

Figure 11: a- Field image with distinct discontinuity sets for station no 8. b- Wedge sliding on (J1 & J2) is seen in kinematic analysis for station no. 8. c- Flexural toppling about So is seen in kinematic analysis for station no. 8.

Based on kinematic study of chosen rock slopes, the major failure mode for structurally controlled rock slopes was found, and the results were applied in the SMR system. Bieniawski's (1989) techniques were used to determine RMRb. UCS was determined indirectly from point load tests performed in accordance with ISRM (1985), with the index-to-strength conversion factor equal to 22.5 (k=22.5), which is applicable for a Limestone and Dolomitic limestone rock types (Bieniawski, 1975).

As demonstrated in Table 5, the UCS ranged from 83 MPa to 132 MPa. The relationship between the volumetric joint count (Jv) and the (RQD) was used to calculate RQD, as shown in equation (2) provided by Palmstrom (2005). The inverse of the average frequency of all discontinuities was used to obtain the average spacing of all discontinuities, as illustrated in Tables 7 (Bieniawski, 2011), with values ranging from 429 mm to 759 mm. Table 6 showing average frequency and spacing of all discontinuities in station no. 1, but Table 7 showing average frequency and spacing of all discontinuities in all stations.

$$\mathrm{RQD} = 110 - 2.5\ \mathrm{Jv} \quad \text{……………………….(2)}$$

When the rock mass characteristics of each rock slope were obtained, as in Table 8, the mandatory parameters of RMRb 1989 were rated using Bieniawski's (1989) general table for discontinuities condition and groundwater, and graphs for UCS, RQD, and discontinuity spacing. Table 9 presents the RMRb values for the assessed rock slopes.

Table 4. Failure types and their directions in rock slope kinematic analyses

Station no.	Formation	Planar sliding direction	Wedge sliding direction	Flexural toppling direction	Direct toppling direction
1	Bekhma	√ (051°)	-	√ (061°)	-
2		√ (060°)	-	√ (068°)	-
3		√ (035°)	-	-	√ (028°)
4		√ (229°)	√ (241°)	-	-
5	Qamchuqa	√ (266°)	√ (320°)	-	-
6		-	√ (255°)	-	-
7		√ (250°)	√ (195°)	√ (240°)	-
8		-	√ (264°)	√ (271°)	-

Table 5. Results of the point load test and the uniaxial compressive strength of complete rock at stations 1 to 8 of the rock slope

Station No.	1	2	3	4	5	6	7	8
Formation	Bekhma				Qamchuqa			
D (mm)	41	39	41	42	43	39	43	43
W (mm)	49	52	62	53	53	51	51	58
F (KN)	15.1	17	15.1	13	15	14.2	11	16.6
F (MN)	0.0151	0.017	0.0151	0.013	0.015	0.014	0.011	0.016
Area mm^2	2009	2028	2542	2226	2279	1989	2193	2494
$De^2=(4DW/\pi)m^2$	0.002559	0.002583	0.003238	0.002836	0.002903	0.002534	0.002794	0.003177
$Is=F/De^2$ (Mpa)	5.900199	6.580375	4.663061	4.584456	5.166740	5.604324	3.937528	5.224940
$Ƒ=(D/50)^{0.45}$	0.914568	0.894216	0.914568	0.924540	0.934382	0.894216	0.934382	0.934382
Is(50)=Is*Ƒ	5.39614	5.88428	4.26469	4.23851	4.82771	5.01148	3.67915	4.88209
UCS=22.5*Is(50) (Mpa)	121.413	132.396	95.955	95.367	108.623	112.758	82.781	109.847
UCS (Mpa)	≈ 121	≈ 132	≈ 96	≈ 95	≈ 109	≈ 113	≈ 83	≈ 110

Where: D=Diameter (distance between the two loaded points), W=Width of the rock block sample, A=W*D (Area of idealized failure plane), F=Force at failure, Is=Point load strength index, f =(size correction factor), UCS=uniaxial compressive strength.

Table 6. Shows the average spacing of all discontinuities measurements from joint sets found in the limestone at station no. 1, the volumetric joint count (Jv) and the rock quality designation (RQD)

Discontinuities (Bedding plane and Joints)	Set spacing and frequency				Average spacing	Average frequency *
	Spacing		Max. frequency	Min. frequency		
	min	max				
Bedding plane (So)	0.18	0.6	5.556	1.667	0.39	2.564
Joint set 1 (J_1)	0.1	1.5	10.000	0.667	0.80	1.250
Joint set 2 (J_2)	0.3	1.5	3.333	0.667	0.90	1.111
Volumetric joint count Jv=Σfrequency (joints/m3)						4.93
RQD = 110-2.5 Jv ……………..(if Jv ≤ 4 so RQD=100)						98
Average frequency of all discontinuities = Jv / 3						1.642
Average spacing of all discontinuities (m)=(1 / average frequency)= 3 / Jv						0.609 m = 609 mm

*Average frequency=1/Average spacing…….(Palmstrom, 2005)
-RQD = 110 -2.5 Jv ………(if Jv ≤ 4 so RQD=100)…………(Palmstrom, 2005)
-Average frequency of all discontinuities=Jv/3 …………………………..(Bieniawski, 2011)
-Average spacing of all discontinuities (m)=1/average frequency……......(Bieniawski, 2011)

Table 7. Average spacing of all discontinuities measures from joint sets detected in the rock slope stations, volumetric joint count (Jv) and rock quality designation (RQD)

Station No.	Formation	Jv (joints/m^3)	RQD	Average spacing of all discontinuities (mm)
1	Bekhma	4.93	98	609
2		3.95	100	759
3		6.99	93	429
4		4.07	100	736
5	Qamchuqa	4.55	99	659
6		4.25	99	707
7		4.28	99	701
8		4.06	100	739

Discrete SMR of Romana (1985) and Con-SMR of Tomas et al. (2007) were both included in SMRTool-v205 of Riquelme et al. (2016), which was used to evaluate the stability of rock slopes at all stations. RMRb, which was derived based on various rock mass parameter ratings, was then utilized to calculate SMR for all rock slope stations. SMRTool-Software calculated the discontinuity-slope relationship for both discrete-SMR and Con-SMR and it is the basis for the adjustment of factors F1, F2, and F3. Because the slope is natural so, the value of F4 is (+15) for all rock slope stations. The attitude of discontinuities and slope, slope excavation method, and RMRb (1989)-value were entered into the SMRTool-v205 program, and the discrete-SMR of Romana - (1985) and Con-SMR of Tomas et al., (2007) were calculated as a consequence.

SMRTool-Software assessed planar sliding on So (bedding plane) in rock slope station no. 1, as shown in Fig. 12, the failure types for all eight rock slope stations, including planar sliding, wedge sliding, and toppling (flexural, direct, and oblique toppling), are listed in Tables 10 and 11.

The SMR-Tool software outcomes for discrete-SMR revealed that station no.1, 2, 3, 4, 5 and 7 (planar sliding), station 4 (wedge sliding) and 7 (flexural toppling) was shown to be a partially stable slope; class III of a normal slope type with failure probability of 0.4. But, station no.1, 2 and 8 (flexural toppling) and station no. 5, 6, 7 and 8 (wedge sliding) were determined to be a stable slope; class II of a good slope type with a failure probability of 0.2.

Table 8. Characterization of the rock mass in the all-rock slope stations.

Station No.	Formation	Rating of parameters				
		Strength of intact rock (UCS) MPa	RQD	Average spacing of all discon-tinuities (mm)	Condition of discontinuities	Ground water condition
1	Bekhma	121	98	609	Persistence 3-10m, No Separation, Rough, Soft filling < 5mm, slightly weathered	Wet (winter)
2	Bekhma	132	100	759	Persistence 3-10m, No Separation, Rough, Soft filling < 5mm, slightly weathered	Wet (winter)
3	Bekhma	96	93	429	Persistence 1-3m, No Separation, Very rough, Soft filling < 5mm, slightly weathered	Wet (winter)
4	Bekhma	95	100	736	Persistence 3-10m, No Separation, Very rough, Soft filling < 5mm, slightly weathered	Wet (winter)
5	Qamchuqa	109	99	659	Persistence 3-10m, No Separation, Very rough, Soft filling < 5mm, slightly weathered	Wet (winter)
6	Qamchuqa	113	99	707	Persistence 3-10m, No Separation, Rough, Soft filling < 5mm, slightly weathered	Wet (winter)
7	Qamchuqa	83	99	701	Persistence 1-3m, No Separation, Very rough, Soft filling < 5mm, slightly weathered	Wet (winter)
8	Qamchuqa	110	100	739	Persistence 1-3m, No Separation, Rough, Soft filling < 5mm, slightly weathered	Wet (winter)
Remarks		From Table 5	From Table 7		From field observation	

Where: RMRb (1989)= Basic Rock Mass Rating, with no adjusting factor for discontinuity orientation

Table 9. For each rock mass in the all rock slope stations, the RMR-parameters are rated and the values of RMRb(1989) are presented

Station No.	Formation	Rating of parameters					RMRb (1989) for SMRTool-software
		Strength of intact rock (UCS)	RQD	Average spacing of all discontinuities	Condition of discontinuities	Ground water condition	
1	Bekhma	11	19.9	13.3	20	7	≈ 71
2	Bekhma	8.9	20	14.3	21	7	≈ 71
3	Bekhma	9.3	18.9	11.9	23	7	≈ 70
4	Bekhma	8.9	20	14.2	21	7	≈ 71
5	Qamchuqa	10.2	20	13.7	21	7	≈ 72
6	Qamchuqa	10.4	20	14	20	7	≈ 71
7	Qamchuqa	6.5	20	13.9	23	7	≈ 70
8	Qamchuqa	10.2	20	14.3	22	7	≈ 74

Where: RMRb (1989)= Basic Rock Mass Rating, with no adjusting factor for discontinuity orientation

Then, station no. 3 (direct toppling) was shown to be a completely stable slope; class I of a very good slope type with failure probability of 0.

In Con-SMR assessment in a comparison to discrete-SMR, station no. 1 and 7 (planar sliding) and 4 (wedge sliding) is unstable slopes; class IV of a bad slope type with a failure chance of 0.6. Then station no. 2, 3, 4 and 5 (planar sliding) and station 7 (flexural toppling) are partially stable slope; class III of a normal slope type with failure probability of 0.4. But, station no.1, 2 and 8 (flexural toppling) and station 5, 6, 7 and 8 (wedge sliding) were determined to be a stable slope; class II of a good slope type with a failure probability of 0.2. Nonetheless, station no. 3 (direct toppling) was shown to be a completely stable slope; class I of a very good slope type with failure probability of 0.

Table 10. Using SMRTool software for all rock slope stations and the results of discrete slope mass rating (SMR) presented

Station no.	RMRb	Failure type	Failure direction	F1	F2	F3	F1.F2.F3	F4	Discrete-SMR value
1	71.2	PS	051º	1	0.7	-60	-42	+ 15	44
		FT	061º	0.7	1	-25	-17.5		68
2	71.2	PS	060º	0.85	0.7	-60	-35.7	+ 15	50
		FT	068º	0.7	1	-25	-17.5		68
3	70.1	PS	035º	0.7	0.85	-50	-29.75	+ 15	55
		DT	028º	0.7	1	0	0		85
4	71.1	PS	229º	0.7	0.85	-50	-29.75	+ 15	56
		WS	241º	1	0.85	-50	-42.5		43
5	71.9	PS	265º	0.7	1	-50	-35	+ 15	52
		WS	319º	0.15	0.85	-60	-7.65		79
6	71.4	WS	255º	0.15	0.85	-60	-7.65	+ 15	78
7	70.4	PS	250º	0.7	1	-60	-42	+ 15	43
		WS	195º	0.15	0.7	-60	-6.3		78
		FT	240º	1	1	-25	-25		60
8	73.5	WS	264º	0.4	0.85	-60	-20.4	+ 15	68
		FT	271º	0.4	1	-25	-10		79

Where: PS=Planar sliding, WS=Wedge sliding, FT=Flexural toppling, DT=Direct toppling, OT=Oblique toppling, F1, F2 & F3 are adjustment factors of SMR, F4=Method of the slope excavation

The outcomes of discrete-SMR and continuous-SMR are compared in this study. Discrete-SMR and Con-SMR's findings are variable in the stability classification for rock slopes at stations 1 and 7 (planar sliding) and station 4 (wedge sliding). The slopes of stations no. 1, 4 and 7 are partially stable in discrete-SMR respectively, but the slopes of stations 1, 4 and 7 are unstable in Con-SMR, as illustrated in Tables 10 and 11. Table 12 shows the comparison between discrete SMR and Con-SMR.

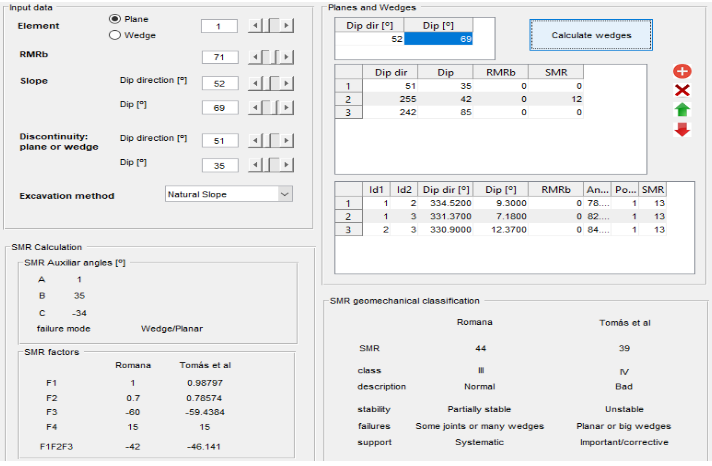

Figure 12: Using the SMRTool program, a rock slope stability assessment at station no. 1 shows planar sliding on So for both discrete-SMR and Con-SMR.

Table 11. Using SMRTool software for all rock slope stations and the results of continuous slope mass rating (Con-SMR) presented

Station no.	RMRb	Failure type	Failure direction	F1	F2	F3	F1.F2.F3	F4	Con-SMR value
1	71.2	PS	051º	0.98797	0.78574	-59.4384	-46.141	+ 15	39
		FT	061º	0.84995	1	-25.6165	-21.7728		64
2	71.2	PS	060º	0.91	0.78574	-59.293	-42.3957	+ 15	43
		FT	068º	0.70786	1	-25.3123	-17.9175		68
3	70.1	PS	035º	0.70786	0.83028	-45	-26.4475	+ 15	58
		DT	028º	0.55546	1	-0.47233	-0.26236		84
4	71.1	PS	229º	0.79939	0.8956	-51.145	-36.6163	+ 15	49
		WS	241º	0.98835	0.89086	-52.8111	-46.4901		39
5	71.9	PS	265º	0.57214	0.94227	-53.825	-29.0337	+ 15	57
		WS	319º	0.27958	0.79638	-58.9148	-13.1173		73
6	71.4	WS	255º	0.17873	0.85701	-59.5385	-9.1196	+ 15	76
7	70.4	PS	250º	0.84955	0.93832	-59.0912	-47.1269	+ 15	37
		WS	195º	0.21926	0.74652	-59.4749	-9.7232		75
		FT	240º	0.99721	1	-25.4029	-25.3319		59
8	73.5	WS	264º	0.3498	0.90208	-58.3687	-18.7337	+ 15	70
		FT	271º	0.50919	1	-25.2735	-12.869		76

Where: PS=Planar sliding, WS=Wedge sliding, FT=Flexural toppling, DT=Direct toppling, OT=Oblique toppling, F1, F2 & F3 are adjustment factors of SMR, F4=Method of the slope excavation

Table 12. Comparison between discrete SMR and continuous SMR (Con-SMR)

Station no.	RMRb	Failure type	Discrete-SMR value	Con-SMR value	Class / Stability (Discrete-SMR	Class / Stability (Con-SMR)
1	71.2	PS	44	39	III / P. stable	IV / Unstable
		FT	68	64	II / Stable	II / Stable
2	71.2	PS	50	43	III / P. stable	III / P. stable
		FT	68	68	II / Stable	II / Stable
3	70.1	PS	55	58	III / P. stable	III / P. stable
		DT	85	84	I / Comp. stable	I / Comp. stable
4	71.1	PS	56	49	III / P. stable	III / P. stable
		WS	43	39	III / P. stable	IV / Unstable
5	71.9	PS	52	57	III / P. stable	III / P. stable
		WS	79	73	II / Stable	II / Stable
6	71.4	WS	78	76	II / Stable	II / Stable
7	70.4	PS	43	37	III / P. stable	IV / Unstable
		WS	78	75	II / Stable	II / Stable
		FT	60	59	III / P. stable	III / P. stable
8	73.5	WS	68	70	II / Stable	II / Stable
		FT	79	76	II / Stable	II / Stable

Where: PS=Planar sliding, WS=Wedge sliding, FT=Flexural toppling, DT=Direct toppling, OT=Oblique toppling, F1, F2 & F3 are adjustment factors of SMR, F4=Method of the slope excavation, P.=Partially, Comp.= Completely

5. CONCLUSIONS:

The best practical method for evaluation the rock slope stability that include joint sets is kinematic approach. The rock slopes of stations 1, 2, 3, 4, 5, and 7 might have planar sliding, while stations 4, 5, 6, 7 and 8 may have wedge sliding.
Stations 1, 2, 7, and 8 may face flexural toppling, however station 3 may have direct toppling. Planar and wedge sliding are the most frequent types of failure in rock slope stations.In comparison, the Con-SMR system is more delicate to slope characteristics and, as a result, provides better value than the discrete-SMR system. In the worst situation, the discrete-SMR values for all rock slopes range from 43 (stations 4 and 7) to 85 (station 3), but the Con-SMR values for all rock slopes range from 37 (station 7) to 84 (station 3), and those established values are characterized of bad to very good slope types with failure probabilities ranging from 0.6 to 0.

In a comparison of discrete-SMR and continuous-SMR (Con-SMR) values of stability classification, rock slope station no. 1 and 7 (Planar sliding) and station 4 (wedge sliding) is the partially stable station in the study area because SMR value equals 44, 43 and 43 respectively, while Con-SMR value equals 39, 37 and 39 respectively which is unstable slope, implying that the Con-SMR has equipped a more accurate evaluation of slope stability ratings. Furthermore, in terms of describing slope stability, the results of discrete-SMR and continuous-SMR diverge, with station no. 1 and 7 with planar sliding and station 4 with wedge sliding changing from class III/ partially stable slope (discrete-SMR) to class IV/unstable stable (Con-SMR).

According to the discrete-SMR results, the rock slope station 1, 4 and 7 requires slope remediation methods like the removal of unstable rock blocks and the construction of a surface or deep pipeline for water drainage. The slope remediation approach like the removing unstable rock blocks, toe ditch, and toe fence is required for rock slope stations 1, 2, 3, 4, and 8.

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Conflict of Interest (1)

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