



Rock Slopes Stability Analysis along the Qaywan-Mokba Road in Sulaymaniyah Governorate, Northeastern Iraq

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Abstract

The current study includes an evaluation of rock slope stability along the Qaywan Mountain (anticline) main road 20 km north of Sulaimani City, which is considered an important task due to the effect of slope instability on human lives. The stability of rock slopes along road cuttings that connect Sulaimaniyah city to the Sharbazher area a beautiful tourist area. Thus, it is crucial to assess a slope's stability in its current state or when it is being used, as well as the procedure for determining why a collapse occurs. Seven stations (slope locations) were chosen, and kinematic analysis and the slope mass rating system were used to research and analyze the potential level of stability of each, using DIPS v6.008 software and SMRTool – v205. Based on the Kinematic analysis, stations 2, 4, 6, and 7 may experience planar sliding and wedge sliding, stations 1 and 2 may experience flexural toppling, and station 2 may experience direct toppling. Slope mass rating and continuous-slope mass rating using SMR-Tool software showed that the worst condition for flexural toppling at stations 1, 2 and 3 ranges from 44 to 58 for slope mass rating and continuous-slope mass rating, also wedge sliding at stations 2, 4, 6 and 7 ranges from 59 to 69 for slope mass rating, and ranges from 59 to 66 for continuous- slope mass rating, so the rock mass under study falls in class III of normal slope type, which they are in partially stable condition with failure probability of 0.4, and worst condition for planar sliding at stations 4, and 7 ranges from 12 to 15 for slope mass rating and 16 for continuous- slope mass rating. Hence, the rock mass falls in class V of the very dangerous slope type, which is in a completely unstable condition with failure probability of 0.9. Also, planar sliding at stations 2 and 6 ranges from 26 to 34 for slope mass rating, and ranges from 21 to 29 for continuous- slope mass rating, so the rock mass falls in class IV of danger slope type, which they are in unstable condition with failure probability of 0.6.

Keywords: Rock-cut slopes; Kinematic Analysis; Discrete-SMR; Continuous-CSMR

1. Introduction

An important field of engineering geology study is rock slope stability. Slope stability is crucial for assessing and understanding the stability of hilly terrain, where many engineering facilities, like buildings, are constructed, roads, cutting rocks in mines, tunnels, and construction in all its forms (Qader and Syan, 2021). Thus, in most locations in the research area, the stability of the rock slopes along road-cut slopes is a severe concern. Landslides are considered one of the most dangerous environmental disasters for the inhabitants of mountainous areas, and they usually occur when the factors causing them are available (Bayram et al., 2021).

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The forces that cause collapses are the forces resisting them, or the Humidity, freezing, and melting of snow, as well as the rock structures that help the collapses, such as joints and cracks and the most important factors causing them are the gravitational force. In general, there are three main reasons for the instability of slopes in mountainous areas, namely: (1) Geological and environmental causes; (2) Morphological and dynamic causes; (3) Causes of human activities, such as excavation of slopes, blasting, and land use (Bayram et al., 2021). Numerous scholars such as Umrao et al. (2011); Sarkar et al. (2012); Siddique et al. (2015); Basahel and Mitri (2017); Saranaathan and Kannan (2017); Sharma et al. (2019); Fattah et al. (2023); Mohammed et al. (2023) examined the stability of the rock slopes near road cuts that link remote areas in valleys or on the hills and mountains' slopes.

The important topic is evaluating the stability of slopes in their current reality or when exploited, as well as explaining the occurrence of collapse. Seven stations (slope locations) were chosen based on the rock exposures and slope parameters. They used kinematic analysis and the Slope Mass Rating (SMR) system to research and analyze the potential level of stability of each through DIPS v6.008 software (Rocscience, 2015) and SMRTTool –v205 (Riquelme et al., 2014). Kinematic analysis is commonly used to predict potential structural failure mechanisms (planar, wedgs, and toppling) and the possible direction of movement failure along the potentially unfavorable joint planes using the stereonet projection technique. Additionally, the Rock Mass Rating (RMR) parameter values were recorded for the slope stability method suggested by Bieniawski (1989), while the (SMR) parameter values were recorded using the classification method suggested by Romana et al. (2003) and Tomas et al. (2007).

The study aims to evaluate the stability of the rock slope along the road of Qaywan Mountain (anticline), located 20 km north of Sulaymaniyah, which is considered a crucial task due to the potential impact of slope instability on human life.

2. Location and Geology of Study Area

The study area is located about 20 km to the north of Sulaimani City at the latitudes $45^{\circ} 25' - 45^{\circ} 27' N$ and longitudes $35^{\circ} 41' - 35^{\circ} 45' E$ along Qaywan Mountain-Mokaba main road (Fig. 1).

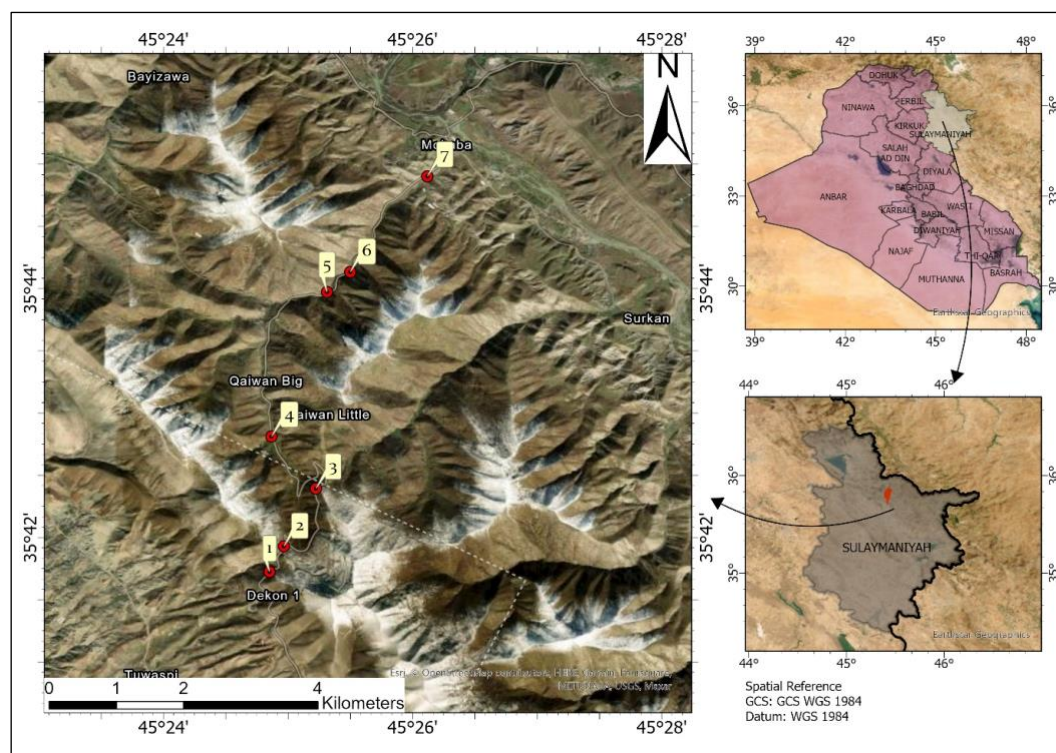


Fig. 1. Location map of the study area

From a structural perspective the study area is situated close to the center of the NW-SE trending Qaywan anticline, which has multiple minor folds. The anticlines extend from the northwest end of the Azmir-Goizha anticline. The Sarmord Formation occupies the anticline's core, around 400 m thick. The Balambo and Kometan formations are exposed along their inner and outer arcs. The last two formations have thicknesses of 300 and 80 meters, respectively.

A structural analysis of the Qaywan region revealed that the axes of the small folds (parasitic folds) diverge shift from the main folds of the Zagros Belt by around 20 degrees northward (Karim, 2020). Tectonically, the region is located as a part of the Imbricate Zone in the Zagros Collisional Belt that resulted from the collision of the Arabian and Iranian plates (Jassim and Goff, 2006; Karim and Ahmad, 2014). Stratigraphically, the study area of the respectively, Balambo Formation is of the (Valanginian - Turonian). The sediments of this formation in the study area consist of fine-grained limestone layers containing chert nodules. Sarmord Formation (Hauterivian–Barremian) consists of grey to dark green marl and light yellow marly limestone. Kometan Formation (Late Turonian-Early Campanian) consists of thick-massive beds of light brown limestone with occasional chert nodules, and Shiranish Formation (Upper Late Campanian – Maastrichtian) consists of bluish grey marl and marly limestone with rare beds of lensoidal limestone beds (Karim, 2020).

3. Material and Methodology

An engineering geological survey was carried out in November 2022 and March 2023 for seven stations; stations 1 and 2 lie within the Balambo Formation. The other stations 3 and 4 are within Sarmord Formation rock-cut slopes, stations 5 and 6 are within Kometan Formation rock-cut slopes, and the last station 7 is within Shiranish Formation. Measurements made in the field included slope direction and slope angle, dip and dip direction of discontinuities, and their spacing and condition (roughness, persistence, aperture, weathering, filling materials), as well as assessing the condition of groundwater. Additionally, laboratory analysis was carried out in compliance with ISRM (1985) suggested method to measure the strength index from the point load test.

In this study, the kinematic approach along with SMR and CSMR systems, are used to analyze the stability of the rock-cut slopes. Kinematic analysis is the most fundamental failure analysis regarding joint sets, bedding plane, cut slope, and sliding friction angle; however, it is only suitable for initial design (Hoek and Bray, 1981). Can be easily kinematic analysis used to determine the possible categories of failures (planar sliding, wedge sliding, toppling), and failure direction in jointed rock masses from the angle between the discontinuities and the slope surface (Hoek and Bray, 1981). A kinematic technique called the Markland test is used to evaluate the likelihood of wedge sliding, in which the wedge-shaped mass slides along the intersection line of two planes (Karim and Ahmad, 2014).

DIPS v6.008 software stereographically portrays the obtained data (Rocscience, 2015). The cohesiveness for a planar discontinuity will be zero, and the discontinuity friction angle will be the only factor determining the shear strength. The tilting method was used to compute friction angles (Bruce et al., 1989). Proposed the Slope Mass Rating (SMR) system to rate rock slope stability (Romana, 1985). The Rock Mass Rating (RMR) system proposed is the foundation for this approach (Bieniawski, 1989). Five parameters from field and lab research are necessary for the RMR method to work: (1) Uniaxial compressive strength; (2) Rock quality designation (RQD); (3) Discontinuity spacing; (4) Discontinuity condition; (5) Groundwater condition. The rating of these parameters can be obtained from Bieniawski (1989) and Figs. 2, 3, and 4, and then the ratings are added to give a value of RMR_{b1989} and basic RMR_{1989} .

Romana, 1985 proposed that SMR is calculated by multiplying RMR_{b1989} and basic RMR_{1989} by three correction factors, one of which is the excavation method factor. Modified SMR somewhat to include wedge failure and plane and topple failures (Anbalagan et al., 1992).

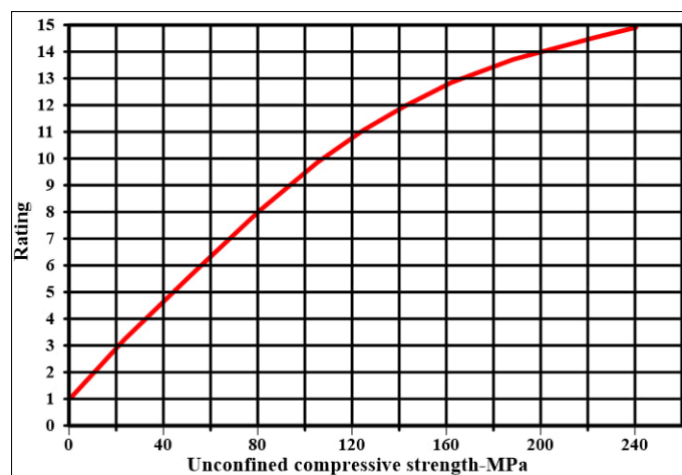


Fig.2. Variation of rating for the unconfined compressive strength (Bieniawski, 1989)

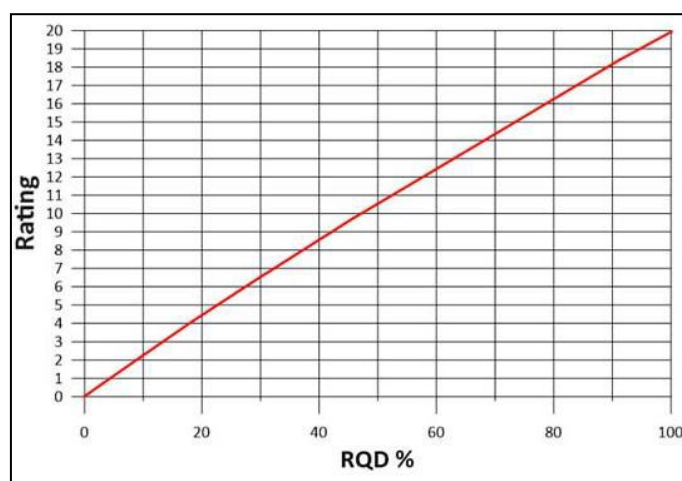


Fig.3. Variation for the (RQD) rating (Bieniawski, 1989)

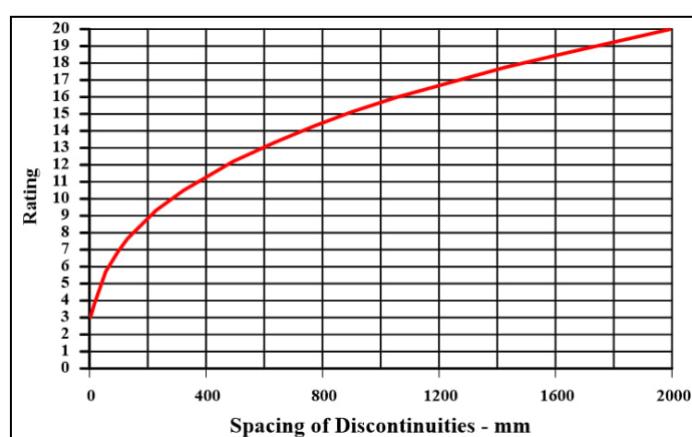


Fig.4. Variation of rating for the discontinuity spacing (Bieniawski, 1989)

The various stability classes and empirically determined SMR limit values correspond to the various failure modes by Romana (1985). Tomas et al. (2007) modified Romana's discrete SMR approach and created Continuous-SMR (CSMR), unlike discrete SMR, which offers a range for each

adjustment factor. The CSMR offers a unique value for each one. Equation 1 illustrates how SMR is calculated using RMRb in addition to a few adjustment factors suggested by Romana (1985).

$$SMR = RMRb + (F1 \cdot F2 \cdot F3) + F4 \quad (1)$$

The adjustment factors F1, F2, and F3 for CSMR are determined using the following formulas put forth by Tomas et al. (2007):

$$F1 = (16/25) - (3/500) \text{Arctan}, [(1/10) (||A|| - 17)] \quad (2)$$

Where: $|A| = |\alpha_j - \alpha_s|$ for plane sliding, $||\alpha_j - \alpha_s| - 180|$ for toppling failure, $|\alpha_i - \alpha_s|$ for wedge sliding, and α_j , α_s , and α_i are dip direction of joint, slope, and plunge direction of intersection line of two joint planes.

$$F2 = (9/16) + (1/195) \text{Arctan}, [(17/100) B - 5] \quad (3)$$

For plane sliding and wedge sliding.

$$F2 = 1 \quad (4)$$

For toppling failure. Where: B equals the dip (β_j) of the joint for planar and toppling failure and the dip of the plunge of the intersection line for wedge sliding.

$$F3 = -30 + (1/3) \text{Arctan} (C) \quad (5)$$

For plane sliding and wedge sliding.

$$F3 = -13 - (1/7) \text{Arctan} (C - 120) \quad (6)$$

For toppling failure. Where: C is an angular difference of dip of joint and slope (β_j , β_s) for planar failure. C is the difference between the dip of the plunge of the intersection line and the dip of slope ($\beta_i - \beta_s$) for a wedge. C is defined as a sum of dip of joint and slope ($\beta_j + \beta_s$) for toppling.

The adjustment factor for the rock slope excavation method F4 has been empirically fixed (Anbalagan et al., 1992). These values are also relevant for the CSMR classification. Using SMRTool-v205 (Riquelme et al., 2014), which uses RMRb values, discontinuities, slope attitude, (SMR) of Romana (1985), and continuous slope mass rating CSMR of Tomas et al. (2007) have been implemented using this type of slope excavation for all rock-cut slope stations.

4. Results and Discussion

In this study, slopes at seven stations rock-cut slopes are surveyed, which differ in structural geological and geomorphological outcrop properties and record patterns of discontinuities. The rock-cut slopes consist of massive fine-grain limestone with chert nodules, as in slope stations 1 and 2. Stations 3 and 4 are composed of grey to dark green marl and light yellow marly limestone. Stations 5 and 6 consist of thick-massive beds of light brown limestone with occasional chert nodules, and the last station 7 consists of bluish-grey marl and marly limestone. The steepness of the rock-cut slopes ranges from mild to extreme, as shown in Table 1.

Table 1. Data discontinuities between stations (rock-cut slope)

Stations	Rock type	Formation	Dip direction / Dip amount					Friction angle
			Slope	Bedding	J1	J2	J3	
1	Limestone	Balambo	330/90°	066/80°	146/85°	174/80°	-----	33°
2	Limestone	Balambo	345/85°	066/78°	334/74°	190/42°	060/40°	30°
3	Limestone and Marly	Sarmord	302/58°	224/58°	350/40°	112/70°	-----	33°
4	Limestone and Marly	Sarmord	070/90°	218/40°	072/60°	350/40°	-----	30°
5	Limestone	Kometan	320/60°	270/40°	315/80°	135/44°	-----	24°
6	Limestone	Kometan	340/54°	345/51°	305/85°	120/80°	-----	32
7	Limestone and Marly	Shiranish	120/66°	032/32°	108/80°	112/52°	-----	31°

4.1. Results and Discussion from the Kinematic Analysis

Kinematic analysis of rock-cut slopes dependent on Markland's test was conducted using internal friction angle of rock discontinuities. DIPS v6.008 software (Rocscience, 2015) uses the average slope and discontinuity orientation to detect any probable structurally controlled failure. In stereographic projection, the possible failure zone is displayed in pink for each of the seven stations (Figs. 5, 7, 9, 11, 13, 15, and 17), and the joints and bedding plane discontinuities were depicted as poles perpendicular to the plane.

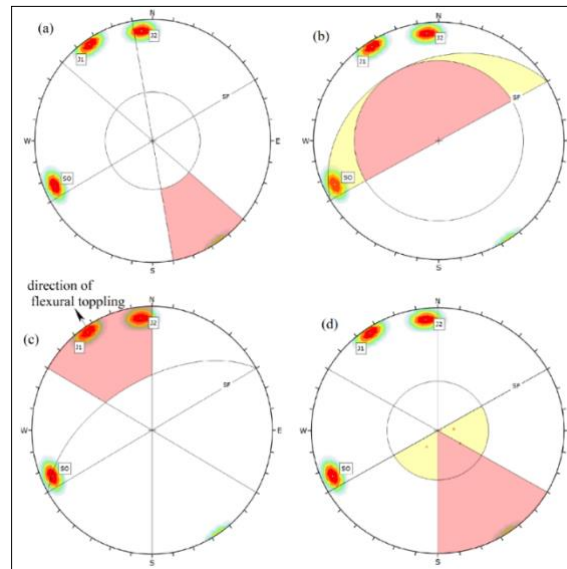


Fig.5. Kinematic evaluation of station 1. a: No possibility of plane sliding; b: No possibility of wedge sliding failure; c: Possibility of flexural toppling about (J1 & J2); d: No possibility of direct toppling

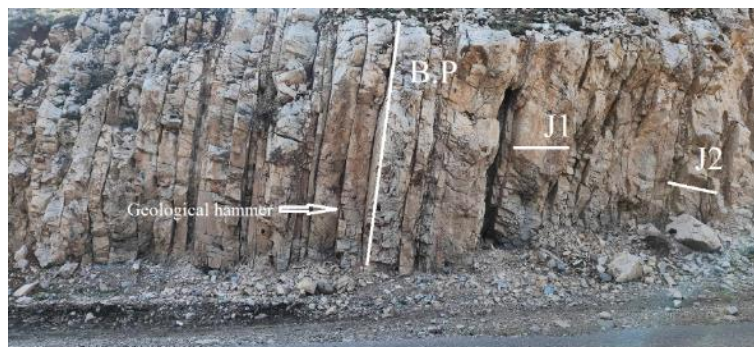


Fig. 6. Bedding plane and discontinuity set for station 1

Rock-cut slope kinematic analysis reveals that there are:

1. Potential for plane failure at stations 2, 4, 6, and 7 in which the plane sliding occurs on J1 at stations 2, 4, station 6 on SO, and station 7 on J2, as displayed in Figs. (7a, 8, 11a, 12, 15a, 16, 17a and 18).
2. Potential for wedge sliding at stations 2, 4, 6 and 7 in which the wedge sliding occurs on (SO & J1) and (J1 & J3) at station 2, and station 4 on (J1 & J2), while station 6 on (SO & J1), and station 7 on (SO & J2), as displayed in Figs. (7b, 8, 11b, 12, 15b, 16, 17b, and 18).
3. Potential flexural toppling about J1 & J2 is in station 1 and stations 2 and 3 about J2 as shown in Figs. (5c, 6, 7c, 8, 9c, and 10).
4. Potential direct toppling is in station 2 about intersection planes (SO & J2), as in Figs. (7d, and 8). The kinematic analysis's complete results are shown in Table 2.

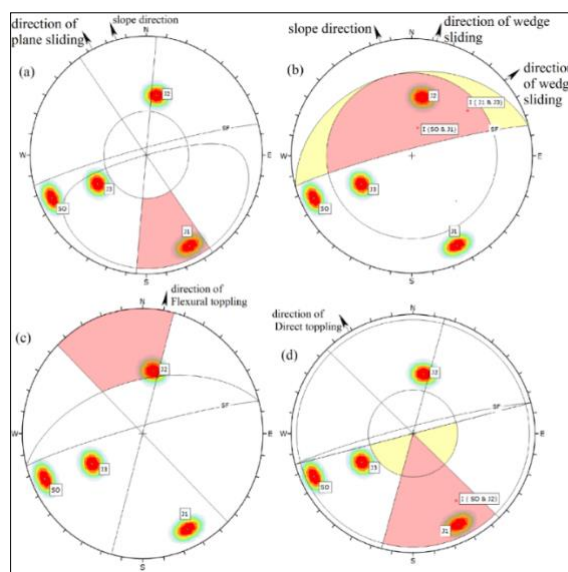


Fig.7. Kinematic evaluation of station 2. a: Possibility of Plane sliding on J1; b: Possibility of Wedge sliding on (so & J1) and (J1 & J3); c: Possibility of Flexural toppling about J2; d: Possibility Direct toppling via release intersected planes (So & J2)



Fig.8. Bedding plane and discontinuity set for station 2

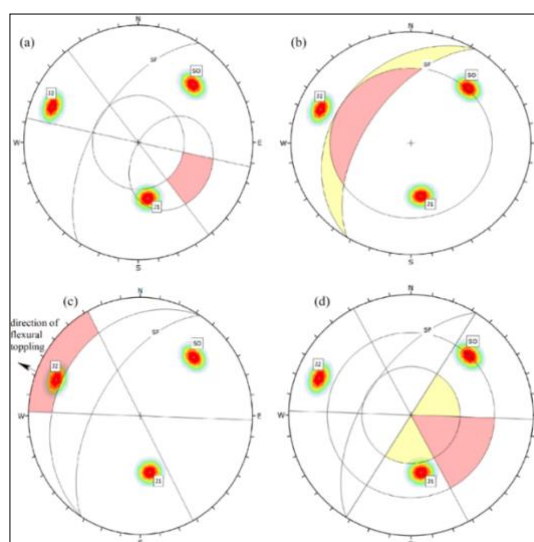


Fig.9. Kinematic evaluation of station 3. a: No possibility of plane sliding; b: No possibility of wedge sliding; c: Possibility of Flexural toppling about J2; d: No possibility of direct toppling



Fig.10. Bedding plane and discontinuity set for station 3

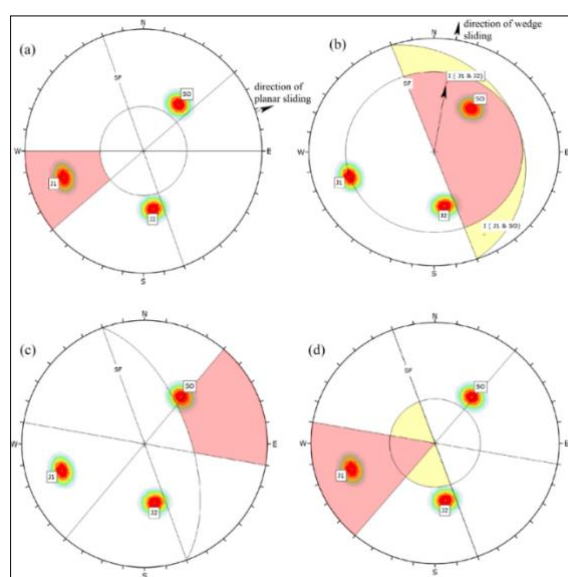


Fig.11. Kinematic evaluation of station 4. a: Possibility plane sliding on J1; b: Possibility wedge sliding on (J1 & J2); c: No possibility of flexural toppling; d: No possibility of direct toppling

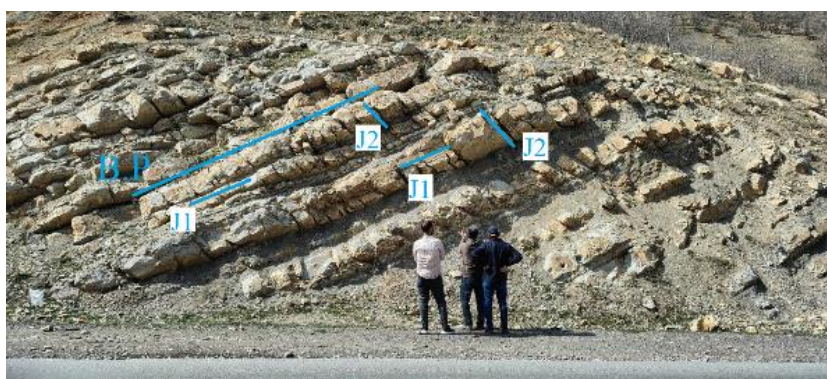


Fig.12. Bedding plane and discontinuity set for station 4

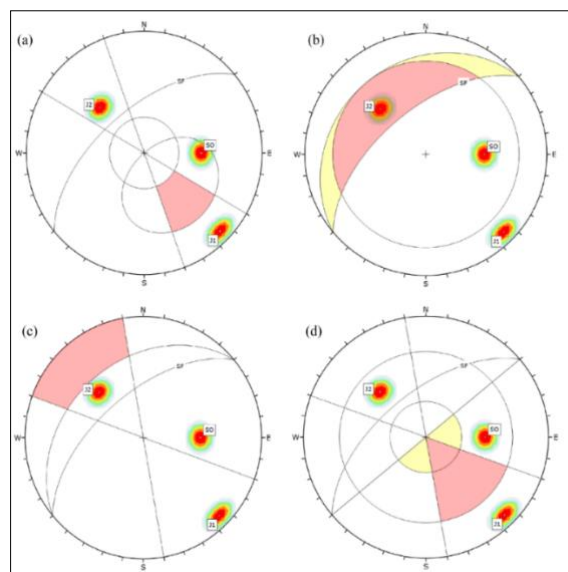


Fig.13. Kinematic evaluation of station 5. a: No possibility of plane sliding; b: No possibility of wedge sliding; c: No possibility of Flexural toppling; d: No possibility of direct toppling

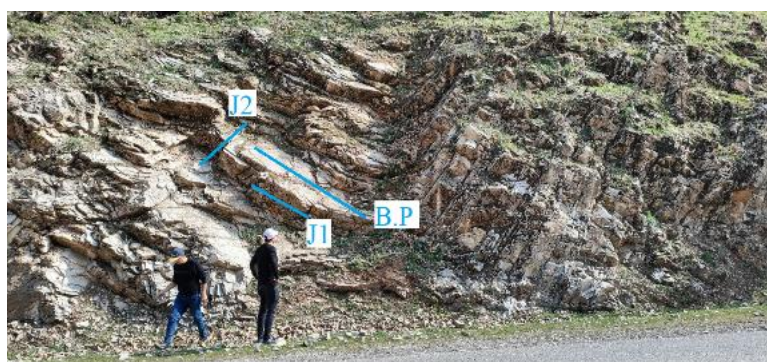


Fig.14. Bedding plane and discontinuity set for station 5

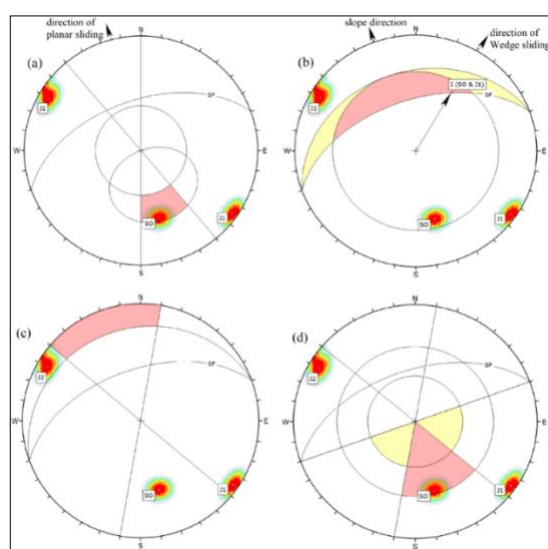


Fig.15. Kinematic evaluation of station 6. a: Possibility plane sliding on SO; b: Possibility wedge sliding on (SO & J1); c: No possibility of flexural toppling; d: No possibility of direct toppling

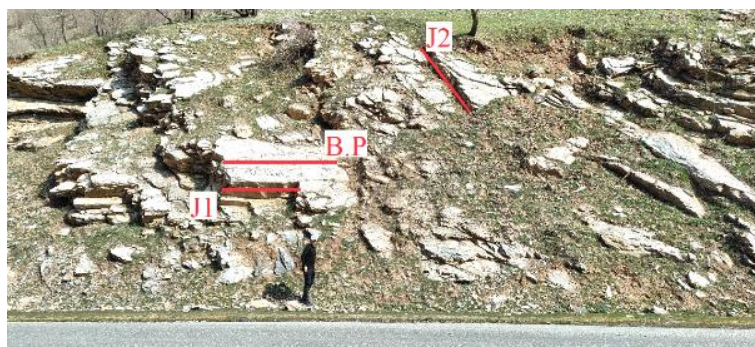


Fig.16. Bedding plane and discontinuity set for station 6

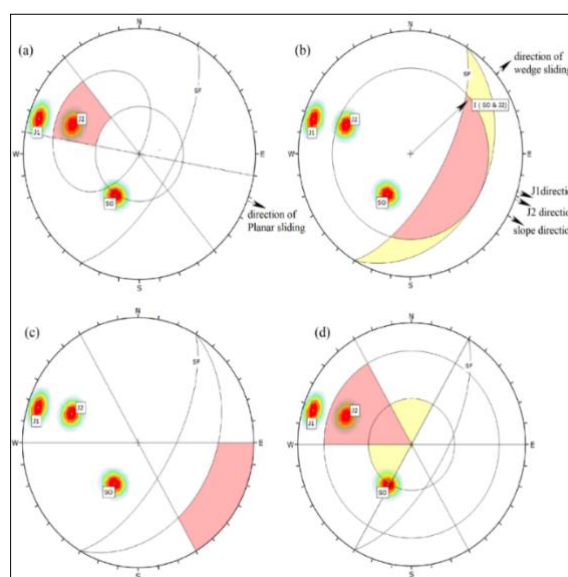


Fig.17. Kinematic evaluation of station 7. a: Possibility plane sliding on J2; b: Possibility wedge sliding on (so & J2); c: No possibility of flexural toppling; d: No possibility of direct toppling



Fig.18. Bedding plane and discontinuity set for station 7.

Table 2. Kinematic analysis results

Formation	Stations	plane sliding, failure probability, and trend	Wedge Sliding, failure probability, and trend	Flexural Toppling, failure probability, and trend	Direct Toppling, failure probability, and trend
Balambo	1	-----	-----	327° 33.33%	-----
	2	333° 25%	12°, 51° 33.33%	10° 25%	328° 25%
	3	-----	-----	293° 33.33%	-----
Sarmord	4	72° 33.33%	10° 66.67%	-----	-----
	5	-----	-----	-----	-----
Kometan	6	345° 33.33%	31° 33.33%	-----	-----
	7	112° 33.33%	50° 33.33%	-----	-----

4.2. Results and discussion from the Slope Mass Rating (SMR) system

Kinematic analysis was used to identify three main failure modes for the structurally controlled rock cuttings (planar, wedge, and toppling (flexural and direct toppling)), and these results were applied to the Slope Mass Rating classification system.

RMRb was determined according to the recommendations of Bieniawski, 1989. UCS was calculated indirectly from point load tests, which were carried out following ISRM's guidelines (ISRM, 1985), where this value is suitable for a variety of rock types (Rusnak and Mark, 2000), Table 3 illustrates the UCS value range, which is 39 MPa to 139 MPa.

The relationship between the number of volumetric joints (J_v) and RQD was used in the equation:

$$RQD = 110 - 2.5J_v \quad (7)$$

To calculate the RQD value (Palmstrom, 2005), which is between 84.35 and 98.80, and the range of 292 mm to 670 mm, which is the average spacing of all discontinuities (Bieniawski, 2011), as indicated in Table (5). Table 6 shows the (RMR) coefficients used to describe the rock mass of the stations in the study area.

Table 3. Uniaxial compressive strength (UCS) results in stations of the study area

Stations	Formation	D mm	W mm	F KN	F MN	A mm ²	D _e ² mm ²	f	IS (50)	UCS* (Mpa)
1	Balambo	46	53	11.1	0.0111	2438	0.00310	1.050	3.752	78.81
2	Balambo	52	68	13.4	0.0134	3536	0.00450	1.141	3.396	71.32
3	Sarmord	42	68	10.1	0.0101	2856	0.00363	1.088	3.020	63.43
4	Sarmord	57	62	13.1	0.0131	3534	0.00450	1.141	3.321	69.75
5	Kometan	58	72	29.2	0.0292	4176	0.00531	1.185	6.505	136.6
6	Kometan	49	69	25.3	0.0253	3381	0.00430	1.130	6.638	139.4
7	Shiranish	51	67	7.1	0.0071	3417	0.00435	1.132	1.847	38.80

* The unconfined compressive strength

Table 4. Number of volumetric join (Jv), (RQD), and the average spacing at station 1.

Discontinuities (Joints and Bedding plane)	Set spacing and frequency				Average spacing (m)	Average frequency
	Spacing (m)		Max. Frequency	Min. Frequency		
	Min.	Max.				
Bedding plane (So)	0.10	1.0	10	1	0.55	1.818
(J1)	0.05	0.40	20	2.5	0.225	4.444
(J2)	0.05	0.20	20	5	0.25	4
Jv= Σ Frequencies' (Joints/m ³)						10.26
RQD=						84.35
Average frequency of all discontinuities =Jv/3						3.42
Average frequency of all discontinuities (m) =(1 / Average frequency)						292 mm

After comparing the three tables proposed by Bieniawski (1989) to determine the precise classification of UCS, and RQD, and discontinuity spacing (Figs. 2, 3 and 4), the rock mass is characterized by the general RMR (Bieniawski, 1989), and then the Obtain RMRb₍₁₉₈₉₎ values for each station, as shown in Table (7)

Table 5. Number of volumetric joins (Jv), (RQD), and the average spacing of all discontinuities for stations in the study area

Formation	Stations	Jv (Joint/m ³)	RQD	Average spacing of all discontinuities (mm)
Balambo	1	10.26	84.35	292
	2	7.471	91.322	401
Sarmord	3	7.738	90.65	387
	4	5.647	95.88	531
Kometan	5	7.751	90.62	387
	6	5.865	95.33	511
Shiranish	7	4.478	98.80	670

SMRTool-v205 (Riquelme et al., 2014), the slope mass rating SMR of Romana (1985), and the mass rating for continuous slopes CSMR of Tomas et al. (2007) were applied to investigate quantitatively in stations of the study area. SMR the six slope stations 1, 2, 3, 4, 6, and 7 were then estimated using RMRb, which was determined based on various rock mass parameter ratings. For both discrete-SMR and CSMR. the modification factors (F1, F2, F3) were determined based on the relationship between discontinuity and slope.

Table 6. Description of the rock mass in stations of the study area

Formation	Stations	Uniaxial compressive strength (UCS)(Mpa)	RQD	Average space in all discontinuities (mm)	Surface condition of discontinuities	Ground water condition
Balambo	1	78.810	84	292	Persistence 4-5m, separation < 5, Rough, no infilling, slightly weathered	Dry
	2	71.321	91	401	Persistence 1.5m, separation > 5mm, Smooth, no infilling, slightly weathered	Dry
Sarmord	3	63.433	90	387	Persistence 1m, separation> 5mm, Slightly rough-rough, no infilling, slightly weathered	Dry
	4	69.754	95	531	Persistence 1.5m, separation> 5mm, Slightly rough-rough, no infilling, slightly weathered	Dry

Kometan	5	136.66	90	387	Persistence 2m, separation > 5mm, Rough, no infilling, slightly weathered	Dry
	6	139.41	95	511	Persistence 2m, separation > 5mm, Slightly rough-rough, no infilling, slightly weathered	Dry
Shiranis	7	38.805	98	670	Persistence 5m, separation > 1mm, Slightly rough, no infilling, slightly weathered	Dry

Table 7. RMR-parameters' rating and RMR_b (1989) values for the rock in stations of the study area

Formation	Stations	Rating of parameters					RMR_b (1989)
		(UCS)	RQD	Average space in all discontinuities	Condition of discontinuities	Ground water	
Balambo	1	8	17	10	19	15	69
	2	8.3	18.2	11.2	16	15	68.7
Sarmord	3	6.5	18.2	11	19	15	69.7
	4	7	19	12.5	19	15	72.5
Kometan	5	11.7	18.1	11	21	15	76.8
	6	11.8	19	12.5	19	15	77.3
Shiranish	7	4.5	19.9	13.6	13	15	66

SMRTool-Software determined F1, F2, and F3 based on the relative orientation of joints with respect to the slope. Blasting and mechanical drilling were used to dig the rock slope ($F4 = 0$). The slope excavation operations involve blasting and drilling.

The results using SMR- Tool software for discrete-SMR and CSMR revealed that the worst condition for flexural toppling at stations 1, 2, and 3 ranges from 44 to 58, so the rock mass under study falls in class III of normal slope type, which they are in partially stable condition with failure probability of 0.4, and worst condition for planar sliding at stations 4, and 7 ranges from 12 to 16, so the rock mass falls in class V of very-danger slope type, which is in completely-unstable condition with failure probability of 0.9.

Also, planar sliding at stations 2 and 6 ranges from 21 to 34, so the rock mass falls in class IV of danger slope type, which is in unstable condition with a failure probability of 0.6 and in the worst condition for wedge sliding at stations 2, 4, 6, and 7 ranges from 59 to 69, so the rock mass under study falls in class III of normal slope type, which they are in partially stable condition with a 0.4 failure probability.

Table 8. The adjustment factors and the values of discrete (SMR)

Stations	Slope attitude	RMR_b	Failure type	Failure Direction	F1	F2	F3	F4	F1.F2.F3	SMR value	SMR class/ Stability
1	330/90°	69	FT	327°	1	1	-25	0	-25	44	III / Pasta
2	345/85°	68.7	PS	333°	0.7	1	-60	0	-42	26	IV / Unsta
			WS	51°	0.15	0.85	-60	0	-7.65	61	II / Sta
			FT	10°	0.4	1	-25	0	-10	58	III / Pasta
			DT	328°	0.7	1	-60	0	-4.2	64	II / Sta
3	302/58°	69.7	FT	293°	0.7	1	-25	0	-17.5	52	III / Pasta

4	070/90°	72.5	PS	72°	1	1	-60	0	-60	12	V / CUnsta
			WS	10°	0.15	0.85	-60	0	-7.65	64	II / Sta
6	340/54°	77.3	PS	345°	0.85	1	-50	0	-42.5	34	IV/ Unsta
			WS	31°	0.15	0.85	-60	0	-7.65	69	II / Sta
7	120/66°	66	PS	112°	0.85	1	-60	0	-51	15	V / CUnsta
			WS	50°	0.15	0.7	-60	0	-6.3	59	III / Pasta

Where: PS=Planar sliding, WS=Wedge sliding, FT=Flexural toppling, DT=Direct toppling, F1, F2& F3 are adjustment factors of SMR, Sta=Stable, Pasta=Partially stable, Unsta=Unstable, Cunsta=Completely unstable.

Table 9. The adjustment factors and the values of continuous (CSMR)

Stations	Slope attitude	RMRb	Failure type	Failure Direction	F1	F2	F3	F4	F1.F2.F3	SMR value	(CSMR) class/ Stability
1	330/90°	69	FT	327°	0.95459	1	-25.70	0	-24.50	44	III / Pasta
2	345/85°	68.7	PS	333°	0.82578	0.9855	58.268	0	-47.49	21	IV /Unsta
			WS	51°	0.17021	0.8700	-59.57	0	-8.824	59	III / Pasta
			FT	10°	0.40804	1	-24.69	0	-10.06	58	III / Pasta
			DT	328°	0.6345	1	-5.571	0	-3.535	65	II / Sta
3	302/58°	69.7	FT	293°	0.84995	1	-24.83	0	-21.112	48	III / Pasta
4	070/90°	72.5	PS	72°	0.97786	0.9682	-59.363	0	-56.20	16	V/CUnsta
			WS	10°	0.1772	0.8536	-59.63	0	-9.019	63	II / Sta
6	340/54°	77.3	PS	345°	0.94117	0.9458	-53.85	0	-47.94	29	IV/Unsta
			WS	31°	0.19925	0.8838	-58.55	0	-10.31	66	II / Sta
7	120/66°	66	PS	112°	0.89192	0.9491	-58.638	0	-49.64	16	V/CUnsta
			WS	50°	0.16377	0.6287	-59.45	0	-6.121	59	III / Pasta

Where: PS=Planar sliding, WS=Wedge sliding, FT=Flexural toppling, DT=Direct toppling, F1,F2&F3 are adjustment factors of SMR, Sta=Stable, Pasta=Partially stable, Unsta=Unstable, Cunsta=Completely unstable.

5. Conclusions

The most prevailing type of failure is planar sliding and wedge sliding. According to kinematic analysis, stations 2, 4, 6, and 7 may experience plane sliding and wedge sliding failure, stations 1, 2, and 3 may experience flexural toppling failure, and station 2 may experience direct toppling failure.

According to SMR- Tool software for Discrete-SMR and Continuous-CSMR, showed that the worst condition for flexural toppling at stations 1, 2, and 3 ranges from 44 to 58 for SMR and CSMR, also wedge sliding at stations 2, 4, 6, and 7 ranges from 59 to 69 for SMR, and ranges from 59 to 66 for CSMR, so the rock mass under study falls in class III of normal slope type, which they are in partially stable condition with failure probability of 0.4, and worst condition for planar sliding at stations 4, and 7 ranges from 12 to 15 for SMR and 16 for CSMR, so the rock mass falls in class V of very-danger slope type, which is in completely-unstable condition with failure probability of 0.9.

Also, planar sliding at stations 2 and 6 ranges from 26 to 34 for SMR, and ranges from 21 to 29 for CSMR, so the rock mass falls in class IV of danger slope type, which is in unstable condition with a failure probability of 0.6. Upon comparing the results of the Discrete-SMR and Continuous-CSMR, it was observed that one case wedge failure exhibited varying SMR-classes and stability conditions, changing from class II/ stable slope discrete-SMR to class III/ partially stable slope (CSMR) in station 2 (Tables 8 and 9).

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