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Kinematic and Slope Mass Rating Application for Rock Slope Stability Evaluation Along Shanadar-Goratu Road in the Gali-Ashkafte Valley, Erbil, NE-Iraq

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Abstract

A Study of rock slope stability along the Shanadar-Goratu main road, which is located near Mergasur town, northeast of Erbil city, is very necessary because many many slope failures take place every year, especially in the wet season. It is the main road between Erbil city and many towns and villages in the Mergasur district. For the current research, eight (8) rock-cut slope stations have been selected for 4.5 kms along the road from Shanadar to Goratu, based on differences in discontinuity pattern, slope geometry, and failure types. Field data has been assessed by the kinematic method, through DIPS v7.0 software, and by slope mass rating classification system, through SMRTool - v205 software. Kinematic analysis from DIPS v7.0 software showed that three types of failures (planar, wedge and toppling failures) may occur, and the largest susceptible rock masses to failure are of slope stations no. 1,5 and 8. In the worst conditions, the discrete-SMR and continuous-SMR pointed out that slope stations no. 1, 5 and 7 are completely unstable (class V – very bad slopes) with a failure possibility of 90%, while slope stations no. 2, 3 and 4 are unstable (class IV – bad slope) with failure possibility of 60%. However, slope stations no. 6 and 8 are partially stable (class III – normal slope face) with failure possibility of 40%, with an exception for slope station no.6 which is unstable (class IV – bad slope face) from continuous-SMR.

Keywords: Rock slope stability; Kinematic analysis; SMR, Continuous-SMR; Mergasur district

1. Introduction

Slope failure is among the most common natural catastrophes, and it can lead to significant property damage and loss of the human lives. Assessment of rock slopes in most road cuts, mainly in rugged terrains, is susceptible to instability problems because of variations in rock mass characteristics and environmental variables such as seismic activity and hydrological effects on the slope (Pantelidis, 2009). This route occasionally has several rock collapses, producing risks for people and traffic jams (particularly during the wet seasons). Unfortunately, there is no public property for this, but only private records. The slope height, slope angle, and discontinues orientations play vital effect on the road cut and slope instability problem. Also, the stability vulnerability of rock slopes may be affected by the road curvature, especially in rugged mountainous areas (Hoek and Bray, 1981).

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This research comes from the Zagros mountain range, which is located in the northeastern portion of Iraq, where major and minor landslides are common. In this region, roadways are still the only means of communication. Slope failure may cause traffic disruption due to the random excavating of natural slopes for road constraction and expansion, which makes the slopes vulnerable (Umrao et al., 2011). Several previous geological studies have been reported on the study area (Omer, 2005; Balaki and Omer, 2019). While engineering geological assessments are rarely studied, the most recent paper was published by Qader et al. (2020) on a proposed road in the Lerabire village, which is located in the northwest part of the study area. Furthermore, numerous stability assessments have been conducted by many researchers from around the world to utilize rock mass categorization (Umrao et al., 2011; Sarkar et al., 2012; Siddique et al., 2015; Sardana et al., 2019; Hamasur and Qadir, 2020) and rock fall assessment (Verma et al. 2018).

Rock slope stability problems have motivated rock engineers to suggest a number of approaches for analyzing them, including kinematic analysis (Hoek and Bray, 1981; Wyllie and Mah, 2004) is an easy method for determining the potential failure types (plane, wedge & toppling) and failure direction in jointed rock mass, slope mass rating (SMR) system by Romana (1985), and continuous-SMR by Tomás et al. (2007). SMR is obtained from rock mass rating (RMR) proposed by Bieniawski (1989) using some adjustment factors.

Field studies were done to evaluate the lithological and structural variations in rock slopes along a 4.5 kms stretch of road. Eight stations were chosen on either side of the road depending on the exposed of various lithologies and conditions of the slope. Furthermore, the property of rock mass is also presented in this paper and corresponding support measures are proposed to determine the failure mode in unstable slopes. The purpose of this research is to evaluate the stability of rock slopes along the proposed road by using kinematic analysis to estimate their possible degree of stability and using a variety of rock mass classification systems, including the RMR of Bieniawski (1989), the SMR of Romana (1985), and the Con-SMR of Tomás et al. (2007) to quantitatively evaluate the stability of rock slopes.

The present study incorporates the assessment of road cut slopes along the Shanadar-Goratu main road, which is located within the Iraqi Kurdistan Region about 125 km to the northeast of Erbil city and around 4 km to the northwest of Mergasur town. It begins near Shanadar cave (which is an archaeological site in the Iraqi Kurdistan Region) and ends at Goratu town. The study has been done for slopes along the roadway between latitudes 36 49' 00" to 36 52' 00" North and longitudes 44 12' 30" to 44 15' 30" East, as illustrated in Fig.1. It is one of the major transportation routes between Erbil city and many towns and villages in the Mergasur district, and it also serves as a major road from Barzan town and surrounding villages to Mergasur and then through to Soran city.

2. Geological Setting

Iraq is located at the northeastern margin of the Arabian plate, which has been divided into several parts (Jassim and Goff, 2006). The studied area is a part of an unstable shelf of Arabian plate that is situated within the Zagros Fold-Thrust Belt, exactly in the Imbricated Zone in Northern Iraq. As considered, it is located in the northwestern part of Bardost Anticline and just in its southwestern limb.

The Bradost Anticline is located in the Imbricated zone between the Zagros Thrust and the Foreland Folds Belt (Omer,2005). Balaki and Omer (2019) have given a detailed report about this structure, which is a relatively long and wide, double-plunging asymmetrical anticline that is trending from the NW-SE direction (Fig.2). The SE plunge at Gali-Ali Beg gorge intersects two major structures (Korak and Handrin Anticlines), wherein the NW plunge interferes with the Shirin Anticline in the Barzan area.

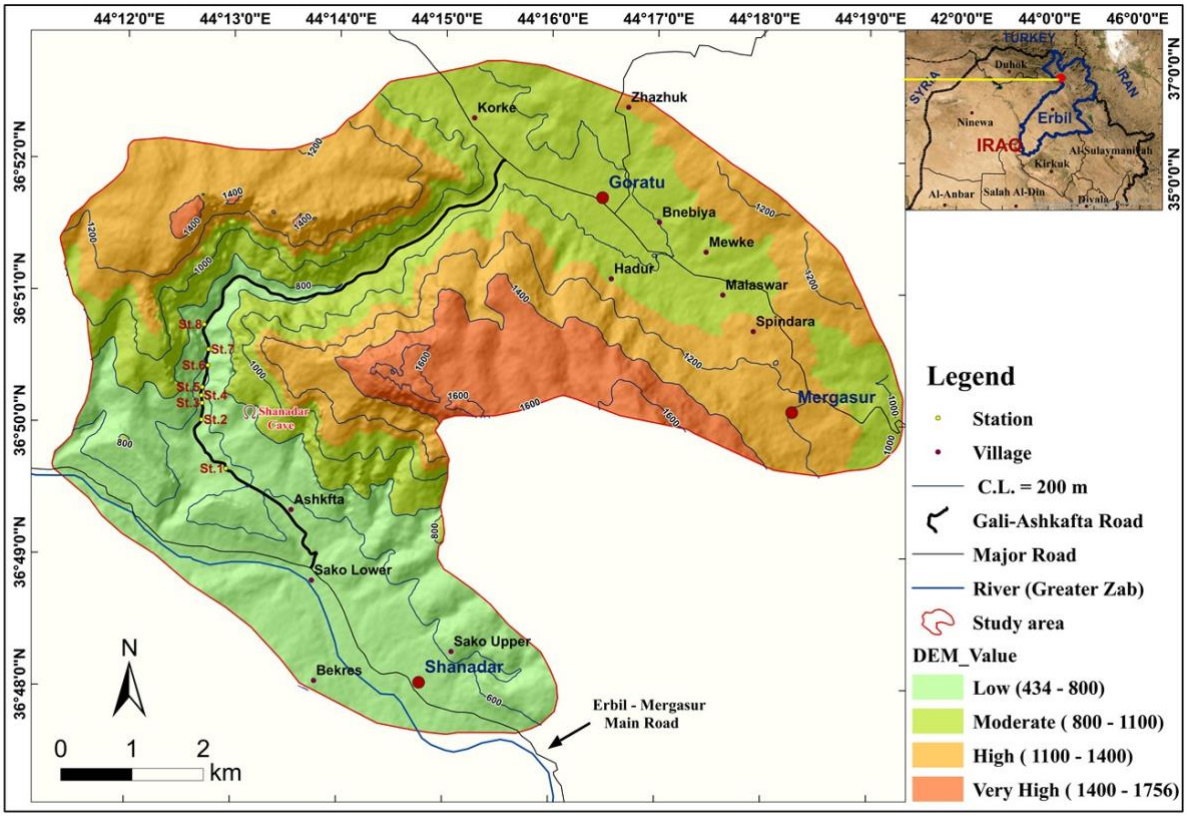


Fig.1. Topographic map shows the location of different rock slope stations in the study area (DEM_12.5m from Alos palsar).

Additionally, the stratigraphic succession that is exposed in the studied area along Gally-Ashkafte main road (in Ashkafta Gorge) deposited from Early Mid Cretaceous (Hauterivian- Barremian) up to Late Cretaceous (Maastrichtian), as in Fig.2. The majority of the anticline is built by Cretaceous Formations, whereas the studied area (SW limb of the Bradost Anticline) is represented by four Formations, which are exposed from the oldest Qamchuqa, Bekhme, Shiranish, and Tanjaro Formations. They are composed of varying rock types: bedded limestone, thick and massive dolomitic limestone (recrystallized limestone), marly limestone and marl (Omer, 2005; Balaki and Omer, 2019).

Seismic activity (earthquake) is also a major factor that induces the stability of rock slopes (Keefer, 1999; Pantelidis, 2009). Al-Sinawi and Al-Qasrani (2003), pointed out that the study area is located in a moderate damage zone, followed by an intensity range of VI-VII, which actually covers the Zagros-Tauros thrust zones. However, the earthquake triggers the failure, but is practically never the cause of the failure. Geological factors, climate, the geometry of a slope, and man-made influences are normally the causes of a failed slope during an earthquake (Hack et al., 2007). The availability of seismic data is very necessary during the study of slope stability, but we could not use it in the study area due to the lack of seismic data caused by the political situation during the last 20 years.

Moreover, the studied area is characterized by rugged topography, which includes steep slopes, narrow gorges, and incised valleys that cut through barren mountains. The general elevation in the study area as showed by Digital Elevation Model (DEM) (Fig.1) ranges between 434 to 1756 meters above mean sea level.

3. Climate of the Investigated Area

The climate is of the subtropical type, marked by dry summers with a rainy season normally starting from autumn (October) to spring (April). According to the Meteorological Organization and Seismology-data center that is related to the Kurdistan Region Government, the average rainfall and

temperature in Mergasur district from 2007 to 2021 varied significantly depending on the season; based on a Mergasur station's data, the average annual rainfall in the area is about 930mm. The average monthly temperature value for this period was 17.64°C, which is the maximum average monthly temperature was 42.89°C in July, 2020, while the minimum was -7.6°C in January, 2008.

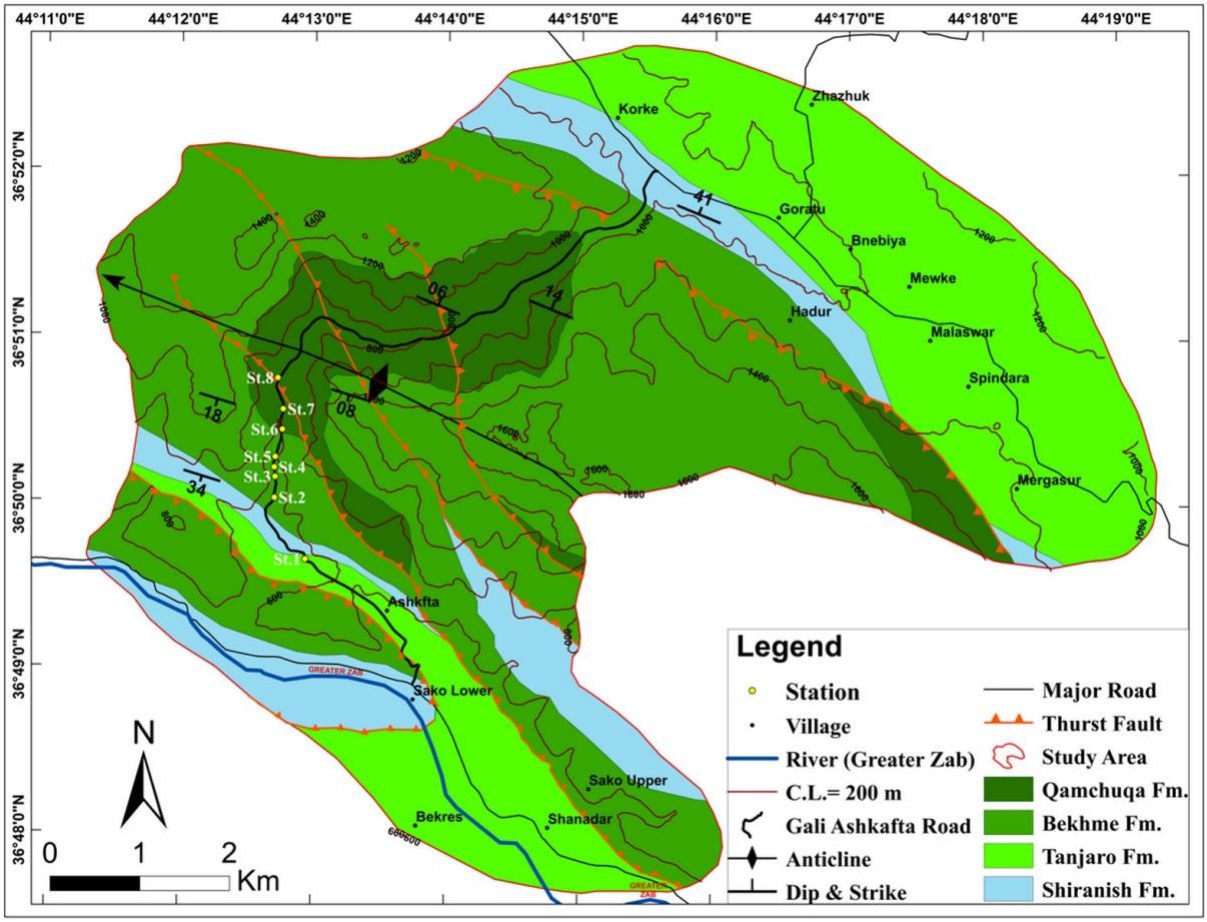


Fig.2. Geological map of the investigated area (After Balaki and Omer (2019)).

4. Material and Methods

Field data were collected from eight critical competent rock cut slope stations on 15th of August to end the September, 2021 along Shanadra - Goratu main road in Gali Ashkafte. As well, the cut slope stations have more resistant and competent, five critical stations (from 2 to 6) are located in Bekhme Formation, and two stations (7 and 8) are in the Qamchuqa Formation and the other one is station no.1, which is lie in the Tanjaro Formation. Field studies have been carried out to determine the attitude of the excavated slope (slope direction/slope angle) and the attitude of discontinuities (bedding planes, faults, joints, foliation, and shear zones, etc.) (dip direction/dip amount), which are taken by Silva compass. After that, various methods have been analyzed to determine the types of failure potential of the rock cut slope by using kinematics analysis, as well as Rock Mass Rating (RMR), Slope Mass Rating (SMR), and continuous-SMR.

Kinematics analysis (Stereographic Projection) by Hoek and Bray (1981) is among the most well-known and simplest methods for determining potential failure types (plane sliding, wedge sliding, and toppling). Kinematics is defined as the motion of bodies without regard for the forces that cause them to move (Goodman, 1983). The potential and mechanisms of failure are determined using angular relations between discontinuities and slope surfaces (Kliche, 1999). Numerous studies have been conducted by various researchers around the world (Markland, 1972; Hocking, 1976; Cruden, 1978;

Hoek and Bray, 1981; Cruden, 1984) and recent study in NE-Iraq also has been done (Hamasur et al., 2020; Qader and Syan, 2021) to determine failure modes by the stereographic projection method. In the current study, data were graphically and statistically evaluted by DIPS v7.0 software (Rocscience Inc, 2016). The software can be used by adding all discontinuities attitude (bedding plane, joint sets, faults, etc.) with the attitude of the slope face and the value of friction angle, then choosing the pole vector mode in equal area and the lower hemisphere of the stereonet option. After that, we can choose the type of failure separately in the kinematics analysis window.

RMR was introduced by Bieniawski (1973) then it was developed several times over the years. In the current study, RMR (1989) is used, which is extensively suitable in various rock engineering projects such as tunneling, dams, and slope stability. The last version of Bieneawski's basic-RMR (RMRb,1989) involves five basic parameters (compressive strength of intact rock, Rock quality designation (RQD), discontinuities spacing, discontinuities condition, and hydrological condition). The ratings of these parameters can be obtained from Table 1A and Figs. 3a, 3b, and 3c. Then the rating values are added to give a value of RMRb (basic RMR).

Table 1: Basic Parameters of Rock Mass Rating (RMR)After Bieniawski (1989).

A. CLASSIFICATION-PARAMETERES AND THEIR RATINGS									
Parameter			**Range of Values**						
1	Strength of intact rock material	Point-load strength index	>10MPa	4 – 10 MPa	2 – 4 MPa	1 – 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 Mpa	100 – 250 MPa	50 – 100 MPa	25 – 50 MPa	5 – 25 MPa	1 – 5 MPa	< 1 MPa
		Rating	**15**	**12**	**7**	**4**	**2**	**1**	**0**
2	Drill core Quality		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		**20**	**17**	**13**	**8**	**3**		
3	Spacing of discontinuities		> 2 m	0.6 – 2 m	200 – 600 m	60 – 200 mm	< 60 mm		
	Rating		**20**	**15**	**10**	**8**	**5**		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweather wall rock	Slightly rough surfaces Separation<1 mm Slightly weathered walls	Slightly rough surfaces Separation<1 mm Highly weathered walls	Slicken sided surfaces Or Gouge <5 mm thick Or Separation 1–5 mm Continuous	Soft Gouge >5 mm thick or Separation		
	Rating		**30**	**25**	**20**	**10**	**0**		
5	Ground water	Inflow per 10m Tunnel length (l/m)	None	< 10	10 -25	25 – 125	> 125		
		(joint water press)/ (Major principal σ)	0	< 0.1	01. – 0.2	0.2 – 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
		Rating	**15**	**10**	**7**	**4**	**0**		
B. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITION									
Discontinuity length (Persistence)			<1 m	1 – 3 m	3 -10 m	10 – 20 m	> 20 m		
Rating			**6**	**4**	**2**	**1**	**0**		
Separation (aperture)			None	< 0.1 mm	0.1 – 1.0 mm	1 – 5 mm	> 5mm		
Rating			**6**	**5**	**4**	**1**	**0**		
Roughness			Very rough	Rough	Slightly rough	Soft filling < 5 mm	Soft filling > 5mm		
Rating			**6**	**5**	**3**	**2**	**0**		
Infilling(gouge)			None	Hard filling < 5mm	Hard filling > 5mm	Soft filling < 5mm	Soft filling > 5 mm		
Rating			**6**	**4**	**2**	**2**	**0**		
Weathering			Unweather	Slightly weathered	Moderately weathered	Highly weathered	Decomposed		
Rating			**6**	**5**	**3**	**1**	**0**		

According to ISRM (1985), UCS is derived from point load testing on prism (block) specimens. RQD is calculated from the relationship between RQD and volumetric joint count (JV) (Palmstrom, 2005). Whereas the rating of the condition of discontinuity (persistence, roughness, aperture widning, weathering of the wall discontinuity, and infilling (gouge) material) has been clearly identified by Bieniawski (1989) in Table 1B.

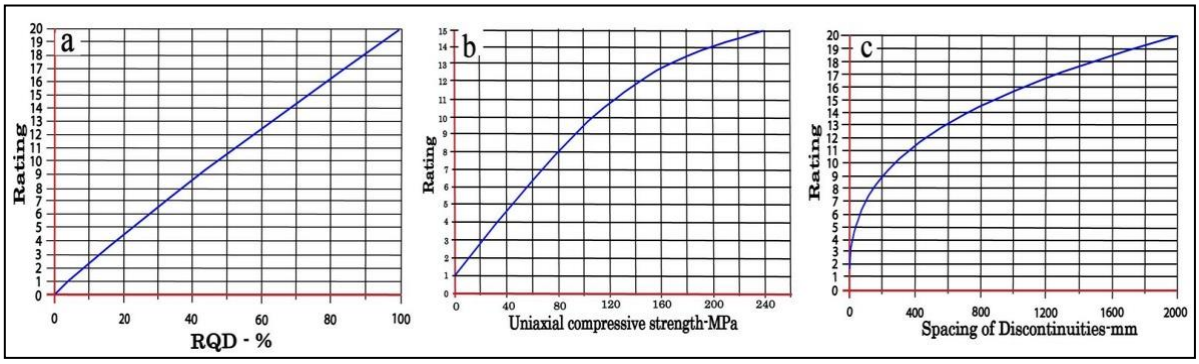


Fig.3. The variation of rating for the RMRb parameters (Bieniawski, 1989); (**a**) RQD; (**b**) Uniaxial compressive strength (UCS) and (**c**) Spacing Discontinuities.

Moreover, ground water plays a significant role in the instability of jointed rock masses, mainly in wet environments. During the field measurements, the studied rock slopes were determined as dry, but for RMR calculation purposes, the studied rock slopes were rated as wet to consider the worst condition, which happens during the rainy seasons and leads to rock failure. Bieniawski (1989) also offered an additional parameter (discontinuity orientation) to estimate the effect of the direction of discontinuity on the stability condition. However, as mentioned by Aksoy (2008), this parameter can be used for stability of tunnel and dam foundations but not for slope. Thus, to analyze the influence of discontinuity direction on slope stability, he suggested using the SMR system, which was proposed by Romana (1985).

SMR is derived from basic RMR (RMRb) by adding some adjustment factors (F1, F2, and F3) based on the joint-slope relationship, while (F4) depending on slope type (method of excavation). This technique has been modified by different researchers (Anbalagan et al., 1992; Romana, 1993; Romana et al. 2015). Initially, this method was only studied for planar and toppling failure. After a while, wedge failure was added to this system by Anbalagan et al. (1992) as shown in Table 2. Furthermore, Romana (1985) provided several classes to define the slope and its stability conditions, which are presented in Table 3. In addition to this, some guidelines for the use of remedial measures based on SMR have been proposed by Romana (1985) to increase the factor of safety, as in Fig.4.

Besides all the above modifications, Tomas et al. (2007) have further developed the SMR system into the continuous-SMR (Con.SMR) by converting the original discrete functions of adjustment factors into continuous rating functions. SMR for both discrete-SMR and continuous-SMR is calculated by the same equation (eq.1) that was suggested by Romana (1985), but the only difference is in the calculation of adjustment factors (F1, F2, F3 and F4).

$$SMR = RMR_b + (F1.F2.F3) + F4 \quad (1)$$

Finally, different software has been utilized in this paper, such as ArcGIS v10.8 to create the most accurate maps (location map and modification of geological map), DIPS v7.0 software has been used in kinematic analysis to determine potential failure type, and SMRTool-v205 software, which was proposed by Riquelme et al. (2014) to estimate both discrete-SMR and con.SMR values in an easy way by adding RMRb values, attitude of slope and discontinuities, and type of the method of excavation. Further description and the result of output data can be clearly seen in Fig. 5, which shows an example of a captured SMRTool-software window.

Table 2. SMR adjustment factors (Modified from Romana (1985) by Anbalagan et al. (1992)

ADJASTMENT FACTORS (F1,F2 AND F3)						
Type of failure			**Very Favorable**	**Favorable**	**Normal**	**Unfavorable**	**Very Unfavorable**
P	**A**	$\lvert\alpha_j - \alpha_s\rvert$					
W		$\lvert\alpha_i - \alpha_s\rvert$	> 30°	30 - 20°	20 - 10°	10 - 5°	< 5°
T		$\lvert\alpha_j - \alpha_s - 180\rvert$					
P/W/T		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			**0.15**	**0.40**	**0.70**	**0.85**	**1**
T		F_2			**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 (F4)			
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; α_j: Joint dip direction; α_s: Slope direction; α_i: Intersection line direction; β_j : Joint dip angle; β_s: Slope angle; β_i : Intersection plunge angle.

Table 3. Description of 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 joins or many wedges	Some blocks	None
Failure Probability	0.9	0.6	0.4	0.2	0

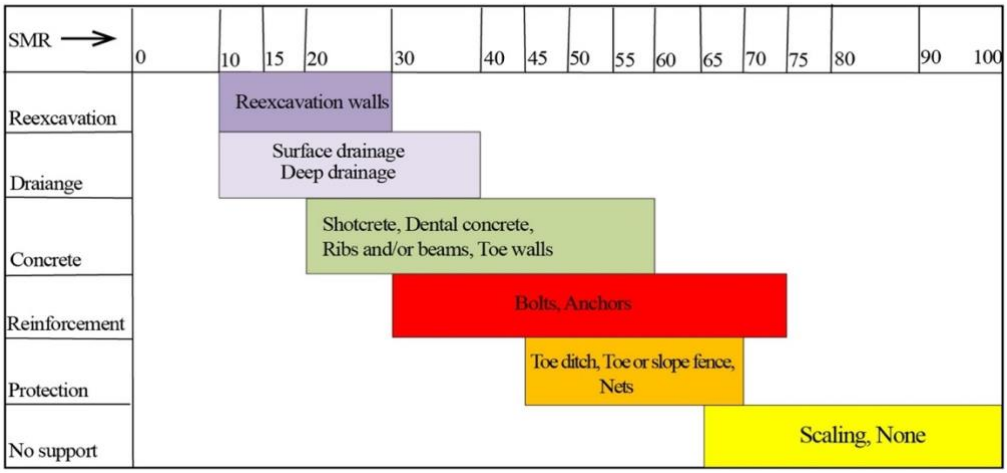


Fig. 4. Slope support guidelines based on SMR (1985) in Romana, et al. (2015).

5. Result and Discussion

Field investigation was carried out for eight rock slope stations with different lithological and geotechnical properties as well as their slope angles have steep to extremely steep slopes with highly jointed rock masses, as detailed field data has been recorded in Table 4.

Furthermore, a lithological description has been done for all slope stations depending on field work and previous studies, which is in the station no.1 is composed of a marly-limestone and nonresistant friable flysch sediment, while stations (2, 3,4,5, and 6) consist mainly of thick to massive hard-bedded limestone and dolomitic limestone with intercalated thin layers of marly-limestone, and the other stations (7 and 8), consist of resistant (cliff-forming) thick and massive beds of dolomitic limestone intercalated by marl. The friction angle of surface discontinuities was determined by performing the tilting method of Bruce et al. (1989). It ranges from 31° to 34° for carbonate rock (limestone).

This study has been conducted for assessing the stability of eight rock slopes sites (slope stations) by kinematic and slope mass rating (SMR) methods.

Table 4. Geological field data of different rock slope stations in the study area.

Station no.	Geological formation	Rock type	Slope side	Slope orientation DD/DA	Orientation of discontinuities (DD/DA)					Internal friction angle
					Bedding plane	J1	J2	J3	R. J	
1	Tanjaro	Marly-Limestone	Main slope	265/80°	081/51°	280/66°	178/76°	----	----	31°
2	Bekhme	Limestone	Main slope L.S slope	076/80° 168/39°	173/34°	060/71°	343/65°	----	----	34°
3	Bekhme	Limestone	Main slope	069/72°	189/33°	055/56°	310/60°	----	280/75°	32°
4	Bekhme	Dolomitic limestone	Main slope	265/70°	193/34°	022/28°	276/69°	332/49°	079/70°	34°
5	Bekhme	Limestone	Main slope	320/80°	186/34°	310/66°	66/74°	015/68°	----	33°
6	Bekhme	Limestone	Main slope	264/78°	189/27°	322/78°	247/74°	----	----	32°
7	Qamchqua	Dolomitic limestone	Main slope	065/82°	169/19°	072/79°	007/68°	----	----	34°
8	Qamchuqa	Dolomitic limestone	Main slope L.S slope	120/90° 200/90°	215/11°	320/77°	045/78°	----	----	34°

Where: DD=dip direction**;** DA**=** dip amount**;** J1, J2 and J3 = Joint set number**;** R. J**=**Random joint**;** L.S**=** Left side

5.1. The Kinematic Analysis

The Kinematic analysis is a technique to determine the probability of different types of rock slope failure. A variety of data has been collected from the field, including the attitude of the joint sets (J1, J2, and J3), bedding plane (So), slope face (SF), and friction angle. As well, these data can be plotted on an equal area and lower hemisphere in stereographic projection by using DIPS 7.0 Software (Rocscience Inc., 2016). The pink area on the stereonet showed the critical zone, when the intersection of planes or the number of poles (perpendicular to the plane) that lie in the critical zone are automatically calculated by software. Based on the kinematic results, different types of failure can be seen in the study area as follows:

- In particular, planar sliding is very common and could occurs in the all rock-cut slope stations, except slope station no.8, as in Figs. 5a, 5b, 6a, 6b, 7a, 7b, 8a, 8b, 9a, 9b, 10a, 10b, 11a, 11b, 12a, and 12b.

- Five out of eight rock slope stations 1,4,5,6 and 7 are potential for occurring the wedge sliding, as illustrated in Figs. 5a, 5c, 9a, 9c, 10a, 10c, 11a, 11c, 12a, and 12c.
- Flexural toppling failure is possible and has occurred in two rock slope stations (1, 8 (main slope) and 8 (left side slope)), as can be seen from Figs**.** 5a, 5d, 13a, 13b, 14a, and 14b.
- Direct toppling failure has also developed at stations 1, 2, 3,5 and 8, as presented in (Figs**.** 5a, 5e, 7a, 7c, 8a, 8c, 10a, 10d, 14a, and 14c.

Further description of wedge sliding, which was reported by Hocking (1976), shows that the wedge failure will occur if the dip directions of two planes lie outside of the angle between direction of the intersection line and slope direction (Fig. 5c), while the wedge failure will not occur if the dip direction of one plane lies within the angle between trend of the intersection line and slope direction, in this case the planar failure will occur on that plane (Fig. 6c).

According to Evans (1981), another type of toppling may occur at station no.8, which is a secondary toppling due to undercutting of the slope's toe. This type of toppling would be induced by water pressure (when the tension crack is filled by water) and the failure by horizontal stress (σ3) direction (perpendicular to the free face) plays a significant role, as it is obvious in Fig.14a by opening the tension crack at the upper part more than at the lower part.

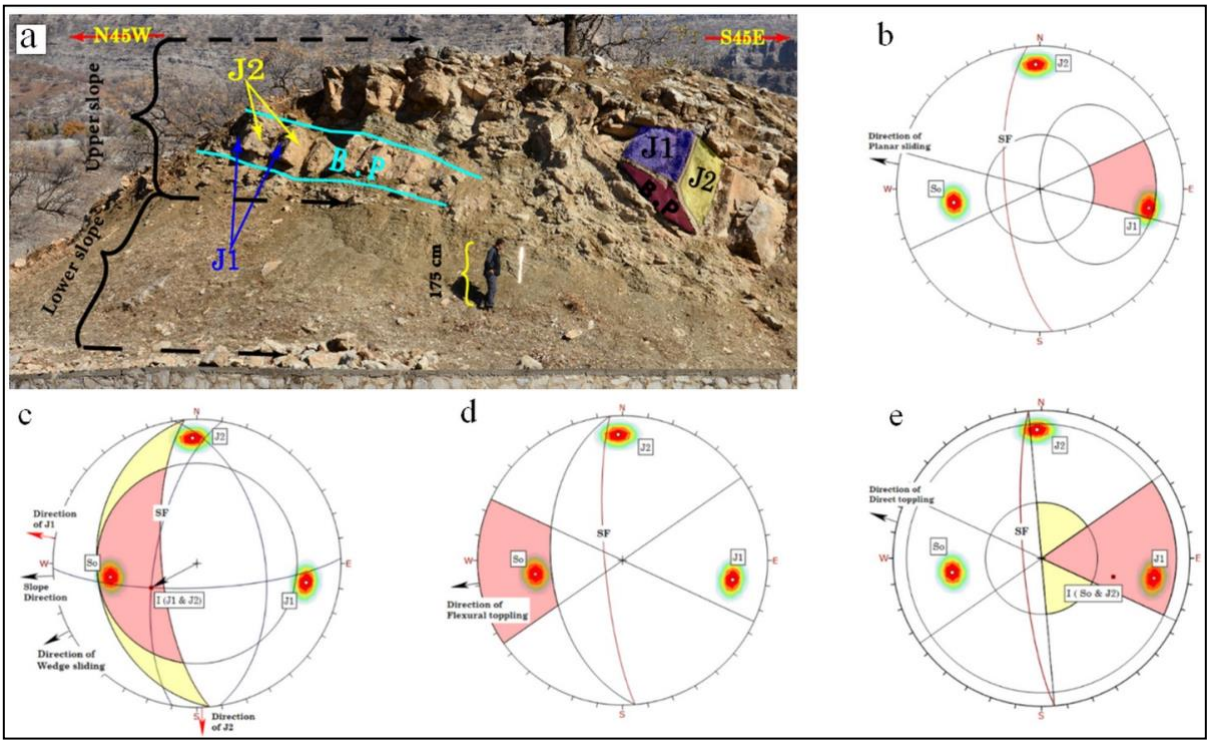


Fig.5. (a) field photograph shows general view of rock cut slope at station No.1 with marked discontinuity sets and stereographic projection showing type of failures: (b) planar sliding could occur on J1; (c) wedge sliding could occur on (J1 and J2); (d) flexural toppling about So; (e) direct toppling via intersected release planes (So & J2). Where: SF=slope face; B.P and So=bedding plane; J1= joint set no.1; J2= joint set no.2; I=intersection between two discontinuity sets.

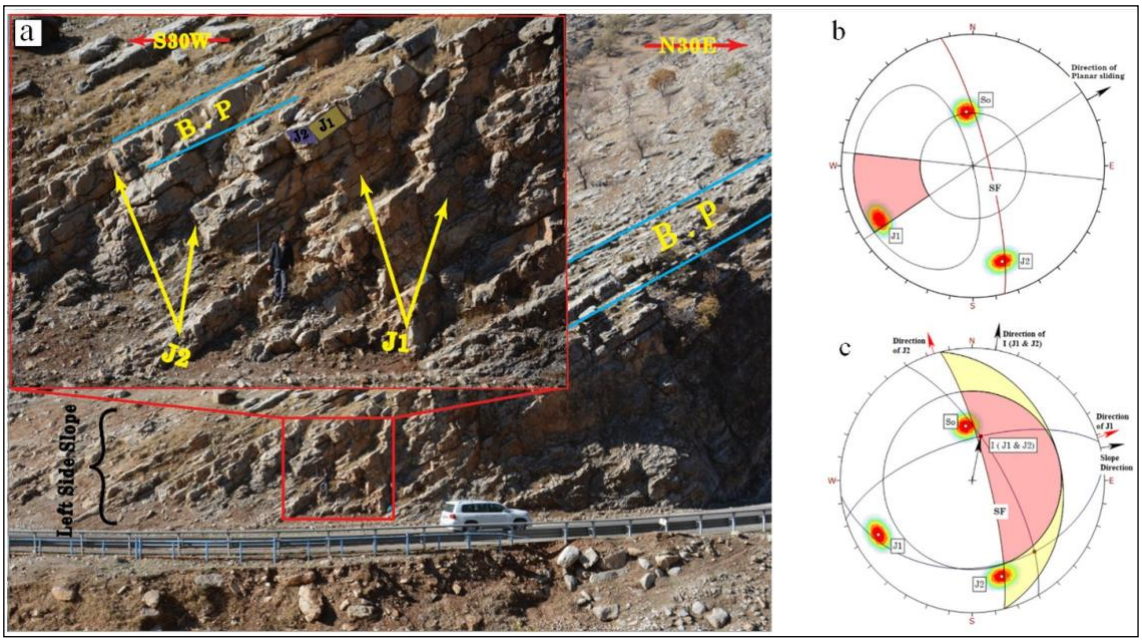


Fig.6. (a) field photograph shows general view of (main slope) rock cut slope at station No.2 with marked discontinuity sets and stereographic projection showing type of failures: (b) planar sliding on J1; (c) no wedge sliding.

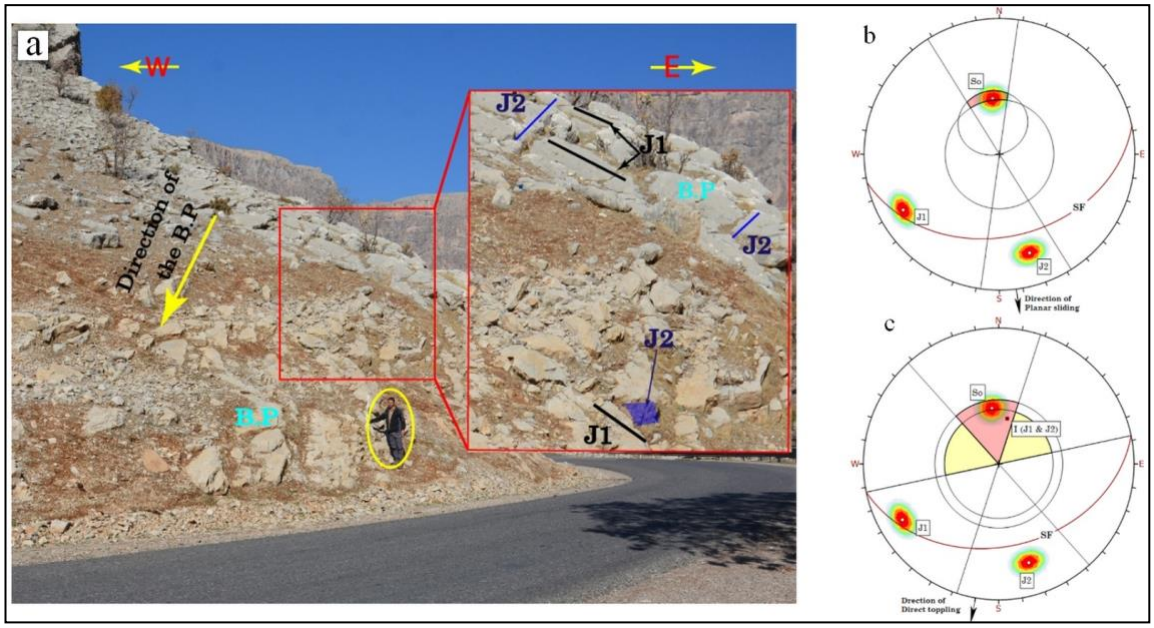


Fig.7. (a) field photograph shows side view of (left side slope) rock cut slope at station No.2 with marked discontinuity sets and stereographic projection showing type of failures: (b) planar sliding on So: (c) direct toppling via intersected release planes (J1 & J2).

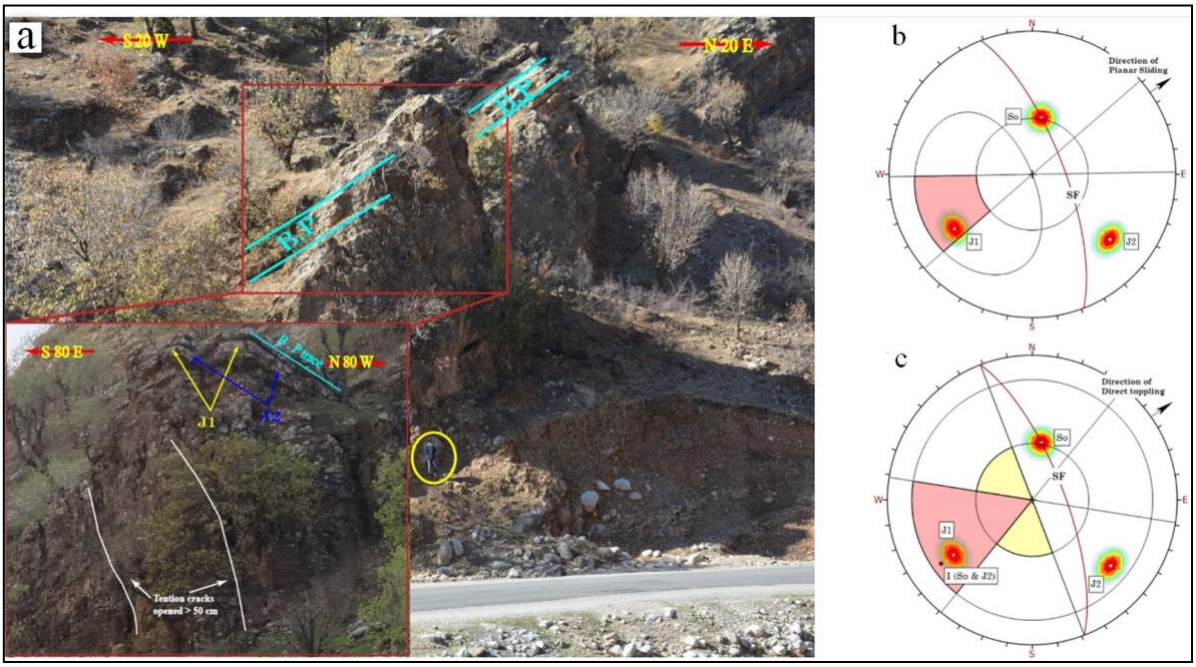


Fig.8. (a) field photograph shows general view of rock cut slope at station No.3 with marked discontinuity sets and stereographic projection showing type of failures: (b) planar sliding on J1; (c) direct toppling via intersected release planes (So & J2).

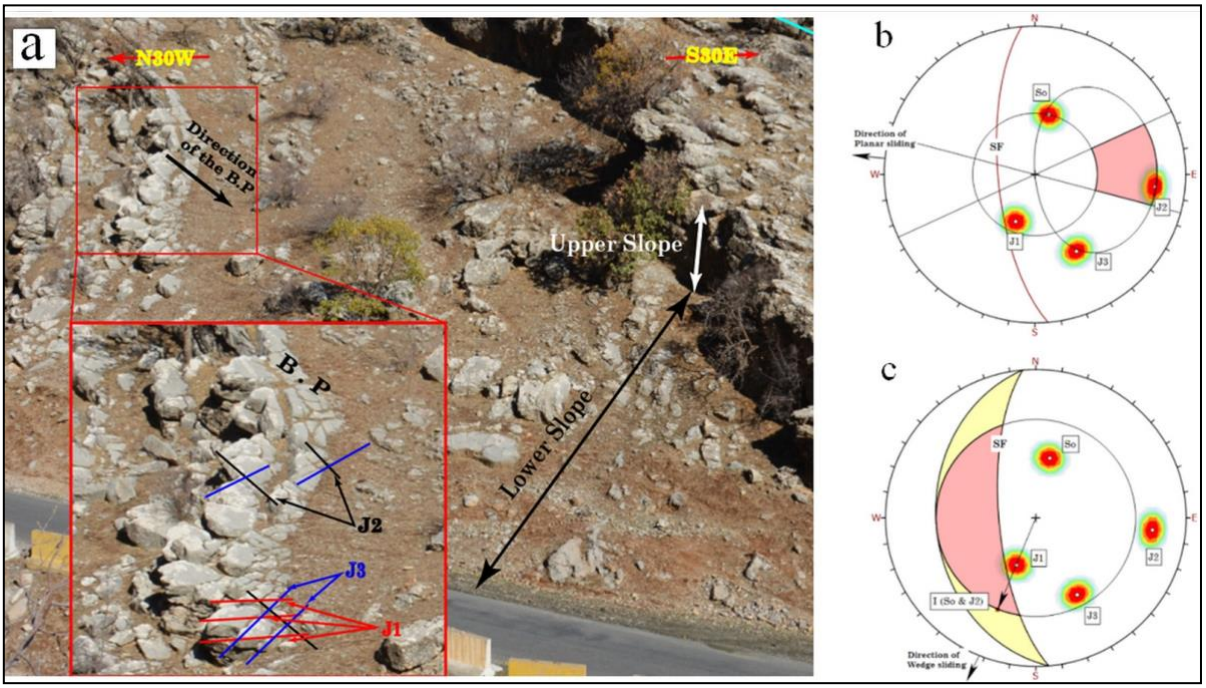


Fig.9. (a) field photograph shows general view of rock cut slope at station No.4 with marked discontinuity sets and stereographic projection showing type of failures: (b) planar sliding on J2;(c) wedge sliding could occur on (So & J2).

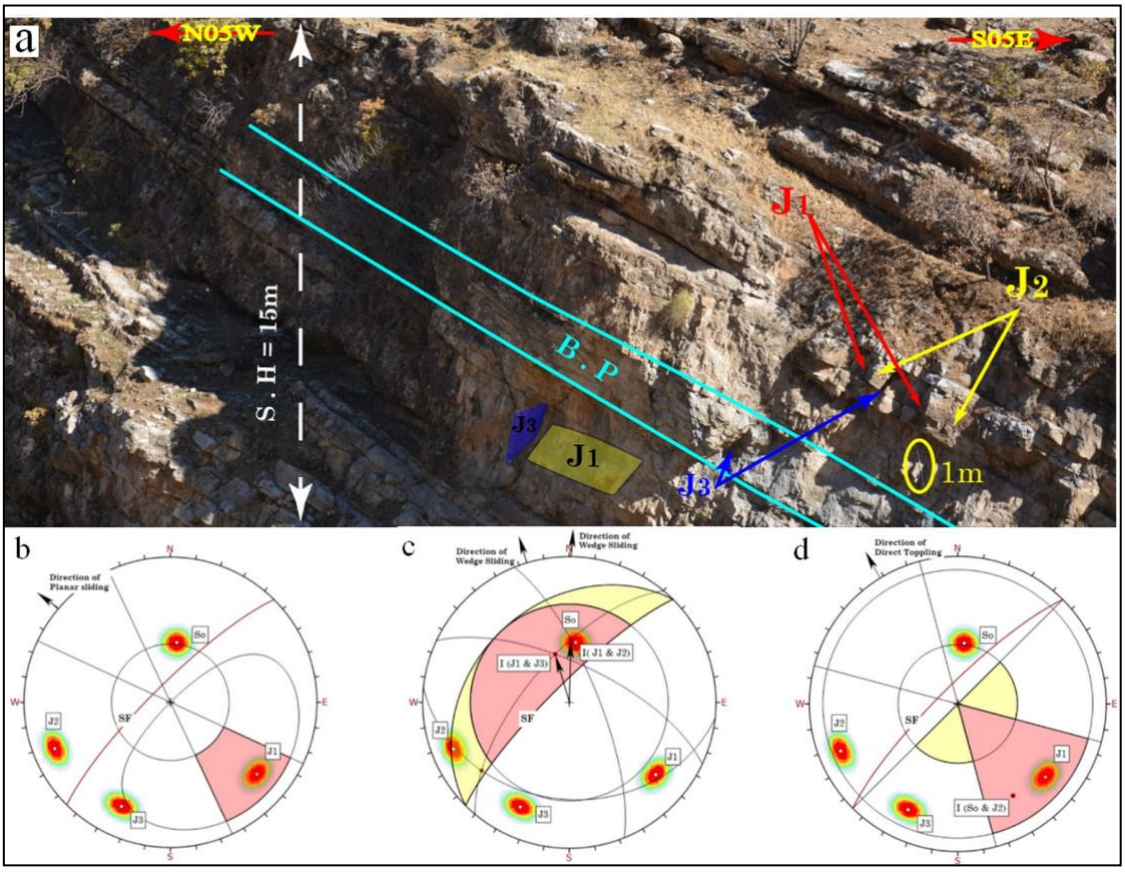


Fig.10. (a) field photograph shows general view of rock cut slope at station No.5 with marked discontinuity sets and stereographic projection showing type of failures: (a) planar sliding on J1;(c) wedge sliding on (J1 & J2) and on (J1 & J3); (d) direct toppling via intersected release planes (So & J2).

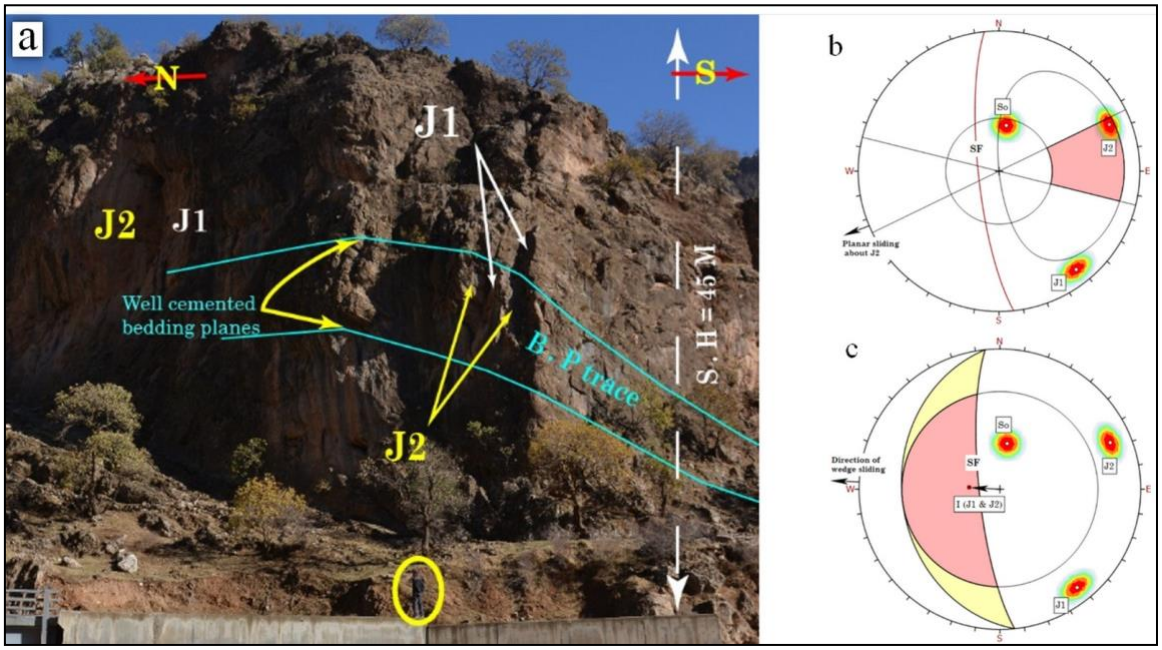


Fig.11. (a) field photograph shows general view of rock cut slope at station No.6 with marked discontinuity sets and stereographic projection showing type of failures: (b) planar sliding on J2; (c). wedge sliding on (J1 & J2)

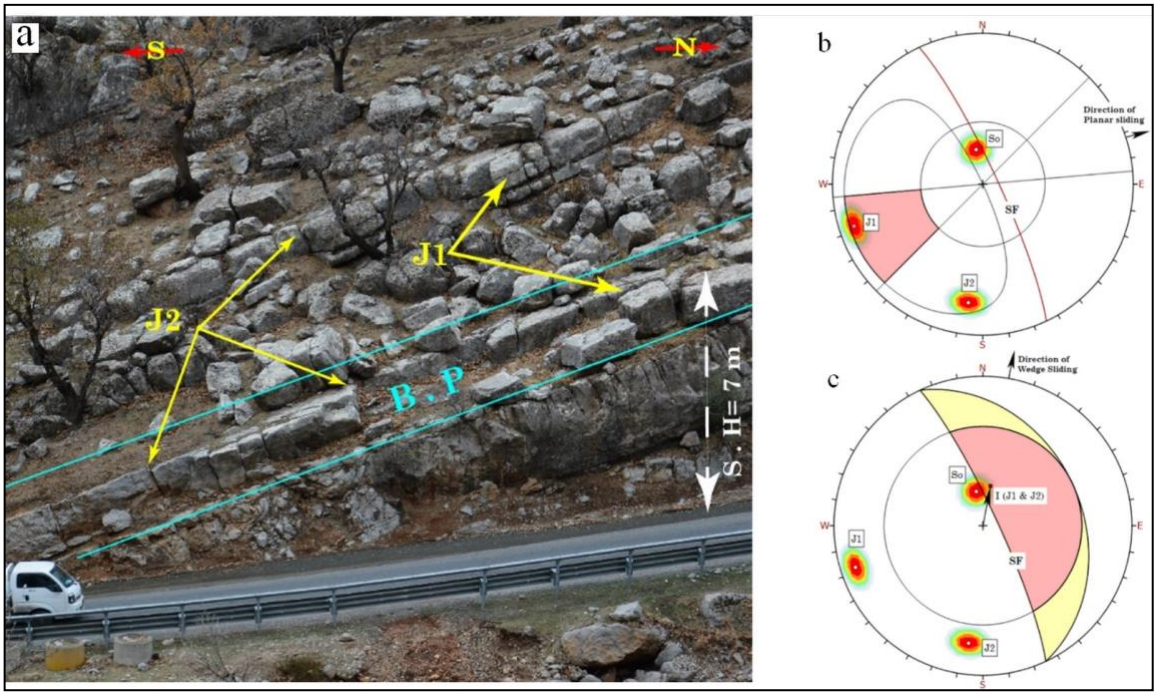


Fig.12. (a) field photograph shows general view of rock cut slope at station No.7 with marked discontinuity sets and stereographic projection showing type of failures: (b) planar sliding on J1; (c) wedge sliding on (J1 & J2).

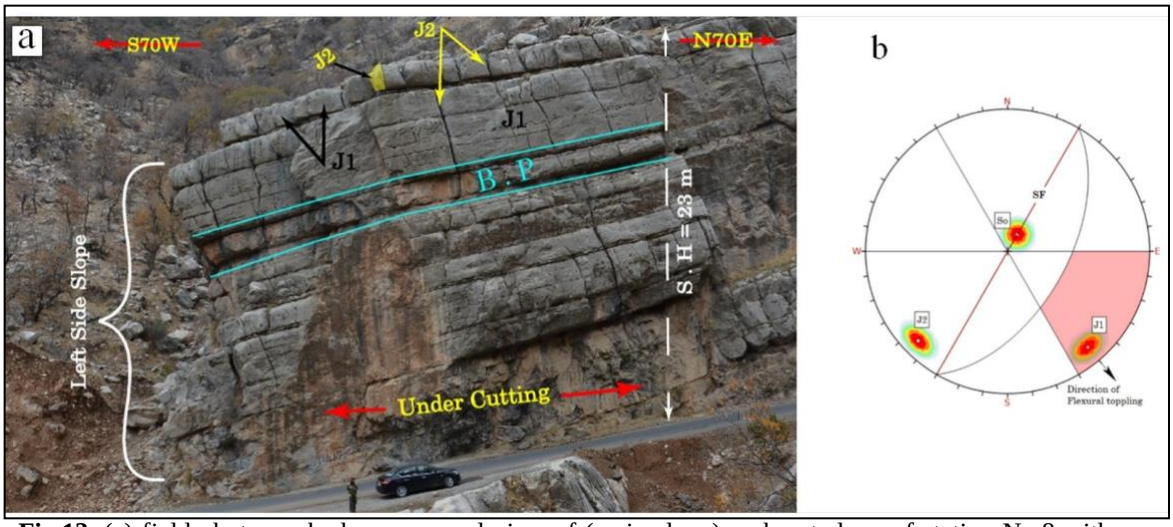


Fig.13. (a) field photograph shows general view of (main slope) rock cut slope of station No.8 with marked discontinuity sets and stereographic projection showing type of failures: (b) flexural toppling about J1.

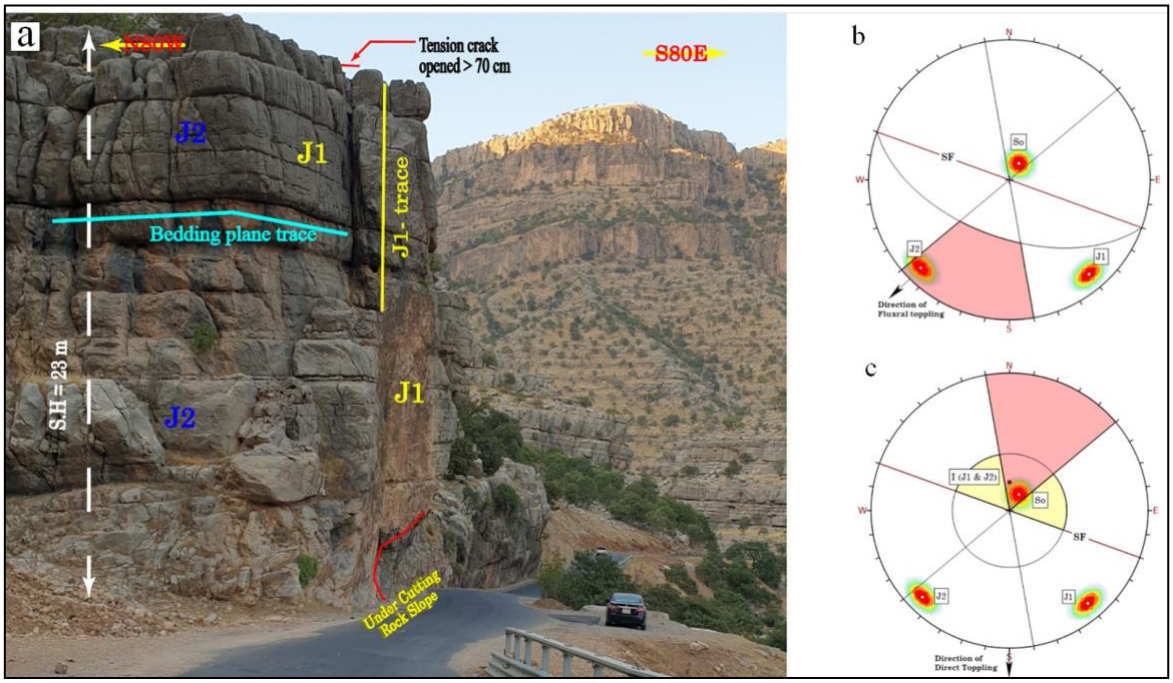


Fig.14. (a) field photograph shows side view of (left side slope) rock cut slope of station No.8 with marked discontinuity sets and Stereographic projection showing type of failure; (b) flexural toppling about J2. (c) direct toppling via intersected release planes (J1 & J2).

Depending on kinematic analysis, rock failure occurs in varying directions. The results are reported in Table 5, which includes the type of failure and its direction, as well as the joint set involved.

Table 5. Kinematic analysis results for each rock slope station were obtained by Dips-Software

Station No. (Slope site)	Planar sliding		Wedge sliding		Flexural toppling		Direct toppling	
	its direction	Joint set involved	its direction	Joint set involved	its direction	Joint set involved	its direction	Joint set involved
1	√ (280°)	J1	√ (242°)	I (J1 & J2)	√ (261°)	So	√ (284°)	I (So & J2)
2	√ (060°)	J1	------	------	------	------	------	------
2 (Left side)	√ (173°)	So	------	------	------	------	√ (190°)	I (J1 & J2)
3	√ (055°)	J1	------	------	------	------	√ (055°)	I (So & J2)
4	√ (276°)	J2	√ (202°)	I (So &J2)	------	------	------	------
5	√ (310°)	J1	√ (001°, 343°)	I (J1 & J2) I (J1 & J3)	------	------	√ (328°)	I (So & J2)
6	√ (247°)	J2	√ (273)	I (J1 & J2)	------	------	------	------
7	√ (072°)	J1	√ (010)	I (J1 & J2)	------	------	------	------
8	------	----	------	------	√ (140°)	J1	------	------
8 (Left side)	------	----	------	------	√ (225°)	J2	√ (180°)	I (J1 & J2)

5.2. Slope Mass Rating (SMR) system

The type of potential failures (planar, wedge, and toppling) were determined for each rock slope station by kinematic technique. Based on this method, it can be seen the direction of an unstable plane (potential of failure plane) with joint set involved, as in Table 5, and then their results have been utilized

by SMR classification system to determine stability condition, which is derived from RMR basic, plus adjustment factors.

RMRb is calculated by five parameters according to the guidelines of Bieniawski (1989). The mentioned parameters of RMRb (1989) were evaluated and compared with rock mass characterization in the basic RMR-table (Table 1), and three charts were suggested by Bieniawski (1989) to determine the accurate rating of UCS, RQD, and discontinuity spacing, as shown in Figs. 1a, 1b, and 1c. UCS was determined for nine samples in the laboratory of the Department of Civil Engineering, College of Engineering, Salahaddin University-Erbil, indirectly from point load tests by using the index-to-strength (Is) and conversion factor (k), which is equal to 22.5 **(**k =22.5**).** This value is suitable for a variety of rock types (Rusnak and Mark, 2000). The UCS value in the study area ranges from 37 MPa to 155 MPa as shown in Table 6.

Further details for station no.1 in the Tanjaro Formation: two samples were taken in the field to determine the strength of the intact rock depending on the variety of lithology, which are composed of limestone and marly limestone, as well as the result of UCS is equal to 74 MPa and 45 MPa, respectively. Later on, the average value will be received for USC, which is equal to 59.5 MPa, as reported in Table 9. RQD ranges from 91 to 100, which is calculated by using equation (eq.2) (Palmstrom, 2005), while the average spacing of discontinuities ranges from 406mm to 1620mm, as shown in Tables 7 and 8. In addition, surface condition of discontinuity and hydrological condition (in the worst condition, rock slopes were rated as wet environment) were discussed in detail as in Table 9 as well as the result of RMRb parameters for all slope stations have reported in Table 9.

$$RQD = 110 - 2.5\ Jv \qquad (2)$$

Finally, the rating of RMRb-parametars and final result of RMRb (1989) were determined for each slope station separately, as shown in Table 10.

Table 6. The UCS results which is determined by point load test for each rock slopes stations.

Station. No	Lithology	D (mm)	W (mm)	F (KN)	F (MN)	Area=D x W (mm^2)	$D_e^2=(4A/\pi)$ m^2	$Is=F/D_e^2$ (MPa)	$f=(D/50)^{0.45}$	$Is_{(50)}=Is*f$	$UCS=22.5*Is_{(50)}$ (MPa)	$UCS=22.5*Is_{(50)}$ (MPa)
1A	Limestone	40	62	11.5	0.0115	2480	0.003156	3.643852	0.904462	3.295725	74.153	≈ 74
1B	Marly Lst.	39	51	5.6	0.0056	1989	0.002531	2.212564	0.894216	1.978510	44.516	≈ 45
2	Limestone	41.5	60	14	0.014	2490	0.003171	4.415011	0.919571	4.059916	91.348	≈ 91
3	Limestone	43.2	50	5.2	0.0052	2160	0.002752	1.889535	0.936335	1.769238	39.808	≈ 40
4	Dolo. Lst.	40	59	19.5	0.0195	2360	0.003006	6.487026	0.904462	5.867269	132.02	≈ 132
5	Limestone	39.5	60.5	23.3	0.0233	2390	0.003044	7.654402	0.899357	6.884041	154.89	≈ 155
6	Limestone	41.5	49	4.6	0.0046	2034	0.002590	1.776061	0.919571	1.633213	36.75	≈ 37
7	Dolo.Lst.	41	63	9.3	0.093	2583	0.003290	2.826748	0.914568	2.585253	58.17	≈ 58
8	Dolo.Lst.	40.5	50	7.7	0.0077	2025	0.002579	2.98565	0.909533	2.71555	61.1	≈ 61

Where: D=Diameter (distance between the two loaded points); W=Width of the specimen; 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 7. The results of RQD, Volumetric joint count (Jv), and average spacing of all discontinuities measurements were observed from joint sets for marly-limestone in the Tanjaro Formation at station no.1.

Discontinuities (Bedding plane and Joints)	**Set spacing and frequency**				**Average spacing(m)**	**Average frequency***
	Spacing (m) Min.	Max.	Max. frequency	Min. Frequency		
Bedding plane (So)	0.1	1.2	10	0.833	0.65	1.538
Joint set 1 (J1)	0.1	1	10	1	0.55	1.818
Joint set 2 (J2)	0.1	1.1	10	0.910	0.6	1.667
Random joint **						
Jv=Σ Frequencies (joints/m3)						5.023
RQD = 110 – 2.5 Jv ……. (RQD=100 for Jv ≤ 4)						97
Average frequency of all discontinuities = Jv / 3						1.674
Average spacing of all discontinuities (m)=(1 / average frequency)= 3 / Jv					0.597m	597mm
*Average frequency=1/Average spacing……. Palmstrom (2005).						
**For random joints, a spacing of 5m for each random joint is used in the Jv calculation.						
-Average frequency of all discontinuities=Jv/3 …….. Bieniawski (2011).						
-Average spacing of all discontinuities (m)=1/average frequency……. Bieniawski (2011).						

Table 8. The results of RQD, Volumetric joint count (Jv), and average spacing of all discontinuities measurements were observed from joint sets for each slope station

Station no.	**Geological Formation**	**Jv (joints /m3)**	**RQD (%)**	**Average spacing of all discontinuities (mm)**
1	Tanjaro	5.023	97	597
2	Bekhme	6.58	93	456
3		3.988	100	752
4		7.397	91	406
5		5.253	97	571
6		2.752	100	1090
7	Qamchuqa	2.416	100	1242
8		3.165	100	948

Table 9. The result of basic rock mass rating (RMRb(1989)) parametares for rock slope station no.1

Slope Station	**Elevation above sea level(m)**	**Rock type**	**Basic Rock Mass Rating (RMRb(1989)) Parameters**				
			Strength of intact rock (UCS)(MPa)	RQD (%)	Average spacing (mm)	Surface condition of discontinuities	Ground water condition
1	565	Marly-Lst.& Limestone	74+45/ 2= 59.5	97	597	Moderately weathered, Slightly Rough, hard filled > 5mm with 1-5mm separated and persistence 1 - 3m.	Wet
Remarks		From field knowledge	Tables (6,7 & 8)			From field description	

Table (9-Continuer). The result of basic rock mass rating (RMRb(1989)) parametares for rock slope stations no. 2,3,4,5,6,7 and 8.

Slope Station	Elevation above sea level(m)	Rock type	Basic Rock Mass Rating (RMRb(1989)) Parameters				
			Strength of intact rock (UCS)(MPa)	RQD (%)	Average spacing (mm)	Surface condition of discontinuities	Ground water condition
2	541	Limestone	91	93	456	Slightly weathered, Rough and undulated surface, hard filling < 5mm with separation 0.1 – 1.0 mm and persistence 1 - 3m.	Wet
3	548	Limestone	40	100	752	Slightly Rough, Slightly weathered, filled fine < 5mm with separation 0.1 – 1.0 mm and persistence 3 - 5m.	Wet
4	552	Dolomitic Limestone	132	91	406	very rough and undulated , slightly weathered, hard filled >5mm, opened aperture > 5mm, , persistence 1 – 3 m	Wet
5	556	Dolomitic Limestone	155	97	571	Rough and undulated , slightly weathered, hard filled < 5 mm, aperture 0.1 – 1.0 mm, , persistence 1 – 3 m	Wet
6	590	Limestone	37	100	1090	Slightly weathered , Slightly rough , Hard filled <5mm with separation < 0.1mm and persistence <5m.	Wet
7	596	Dolomitic Limestone	58	100	1242	Very rough, slightly weathered, hard filling > 5mm with > 5mm separated , persistence >5m.	Wet
8	626	Dolomitic Limestone	61	100	948	very rough and undulated , slightly weathered, filled >5mm opened aperture > 5mm, persistence >5m	Wet
Remarks		From field knowledge	Tables (6,7 & 8)			From field description	

Table 10. The rating of RMRb-parametares and final result of RMRb (1989) for each slope station slope

Slope Station	Geological Formation	Elevation above sea level(m)	Rating of Parameters (RMRb(1989))					RMRb(1989)	
			Strength of intact rock (UCS)(MPa)	RQD (%)	Average spacing (mm)	Surface condition of discontinuities	Ground water condition		
1	Tanjaro	553	5	19.5	13	13	7	57.5	≈ 58
2	Bekhme	522	8.9	18.4	11.8	22	7	67.1	≈ 67
3		545	4.7	20	14.3	18	7	64	64
4		548	11.4	18.1	11.4	17	7	64.9	≈ 65
5		554	12.6	19.4	12.9	22	7	73.9	≈ 74
6		595	4.5	20	16	22	7	69.5	≈ 70
7	Qamchuqa	587	6.1	20	16.9	18	7	67	67
8		631	6.3	20	15.4	15	7	63.7	≈ 64

Where: $RMR_{b(1989)}$ = Basic Rock Mass Rating

The discrete-SMR of Romana (1985) and continuous-SMR (Con.SMR) of Tomas (2007) have been applied in SMRTool-v205 software (Riquelme et al., 2014). SMRTool-Software requires plotting RMRb, discontinuities and slope attitude with selecting a potential failure plane, which was already determined by kinematic analysis, and also selecting the type of excavation method to determine the value of adjustment factor (F4) that was classified by Romana (1985), as shown in Table 2. The value of F4 is equal to zero for most slope stations (six stations) as the excavation method was blasting and mechanical means, but the value of F4 for two rock slope stations (3 and 6) equal to +15 as the rock mass under study is on natural slopes. An example of captured SMRTool-software, which is calculated for planar sliding in a slope station no. 1 as in Fig.15, then all types of failure (plane sliding, wedge sliding, and toppling) are calculated for each slope station by SMRTool-software separately, which is the results of discrete-SMR and Con.SMR are listed in Tables 11 and 12.

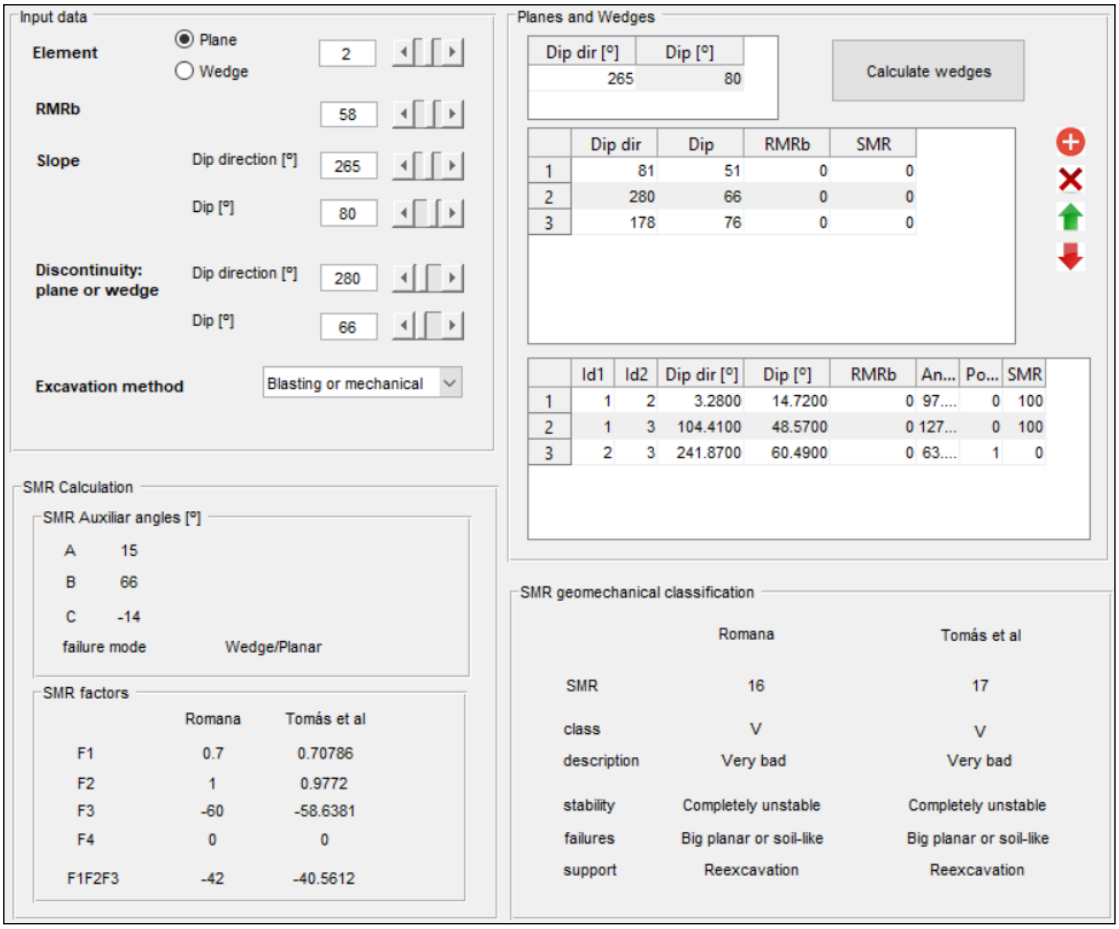


Fig.15. An example of SMRTool-software showing the result of both discrete-SMR and continuous-SMR that is planar sliding may occur on J1 at station no.1.

As depending on SMRTool–Software results for both discrete-SMR and (con.-SMR) values, that they have varies stability condition from class II to IV, which is the SMR classes were suggested by Romana (1985) as shown in Table 3. In the worst conditions, planar sliding at stations no.1, 5, and 7, the SMR-values range from 14 to 18 and they become very bad slopes and completely unstable conditions; they fall into class V (five) with failure probability of 0.9, while for wedge sliding and flexural toppling at stations no.1, and planar sliding at stations (2, 3, 4, and 9), the SMR-values range from 21 to 37 and they fall into class IV (four), they become unstable conditions and bad slopes with

failure probability of 0.6. whereas station no. 8 is partially stable state (Class III (three) with normal slope) SMR range from 50 to 54 with probability of failure is 0.4.

Table 11. Results of discrete-SMR, using SMRTool software for all rock slope stations

Station no.	RMRb	Type of failure	Failure direction	F1	F2	F3	F1. F2. F3	F4	Discrete-SMR Value
1	58	PS	280	0.7	1	-60	-42	0	16
		WS	242	0.4	1	-60	-24		34
		FT	081	1	1	-25	-25		33
		DT	284	0.15	1	0	0		58
2	67	PS	060	0.7	1	-50	-35	0	32
2 (LS)		PS	173	0.85	0.7	-50	-29.75		37
		DT	190	0.4	1	0	0		67
3	64	PS	055	0.7	1	-60	-42	+15	37
		DT	055	0.7	1	0	0		79
4	65	PS	276	0.7	1	-50	-35	0	30
		WS	202	0.15	0.7	-60	-6.3		58
5	74	PS	321	1	1	-60	-60		14
		WS	001	0.15	1	-60	-9	0	65
		WS	343	0.4	1	-60	-24		50
		DT	328	0.85	1	0	0		74
6	70	PS	247	0.7	1	-50	-35	+15	50
		WS	273	0.85	1	-50	-42.5		42
7	67	PS	072	0.85	1	-50	-42.5	0	24
		WS	010	0.15	1	-60	-9		58
8	64	FT	140	0.4	1	-25	-10	0	54
8 (LS)		FT	225	0.4	1	-25	-10		54
		DT	180	0.4	1	-2	-10		54

Where: PS=Planar sliding, WS=Wedge sliding, FT=Flexural toppling, DT=Direct toppling, LS= Left side slope

Table 12. Results of continuous –SMR, using SMRTool software for rock slope station no. 1 and 2.

Station no.	RMRb	Type of failure	Failure direction	F1	F2	F3	F1. F2. F3	F4	Con-SMR Value
1	58	PS	280	0.708	0.977	-58.64	-40.56	0	17
		WS	242	0.451	0.969	-59.02	-25.793		32
		FT	081	0.955	1	-25.12	-23.975		34
		DT	284	0.153	1	-0.466	-0.071		57
2	67	PS	060	0.799	0.983	-57.887	-455.476	0	21
2		PS	173	0.941	0.757	-56.23	-40.069		26
(LS)		DT	190	0.46	1	-0.578	-0.266		66

Where: PS=Planar sliding; WS=Wedge sliding; FT=Flexural topplin; DT=Direct toppling; LS= Left side slope

Table (12-Continuer). Results of continuous –SMR, using SMRTool software for rock slope station no. 3, 4, 5, 6, 7, and 8.

Station no.	RMRb	Type of failure	Failure direction	F1	F2	F3	F1. F2. F3	F4	Con-SMR Value
3	64	PS	055	0.74	0.96	-58.808	-41.791	+15	37
		DT	055	0.743	1	0	0		78
4	65	PS	276	0.826	0.981	-45	-36.443	0	28
		WS	202	0.172	0.749	-59.47	-7.658		57
5	74	PS	321	0.988	0.977	-58.638	-56.612	0	17
		WS	001	0.222	0.966	-59.107	-12.651		61
		WS	343	0.466	0.975	-58.776	-26.683		47
		DT	328	0.904	1	-0.811	-0.733		73
6	70	PS	247	0.64	0.986	-55.321	-34.892	+15	50
		WS	273	0.86	0.984	-56.723	-48.016		36
7	67	PS	072	0.91	0.989	-53.855	-48.486	0	18
		WS	010	0.19	0.98	-58.642	-10.916		56
8	64	FT	140	0.54	1	-25.683	-13.864	0	50
8 (LS)		FT	225	0.408	1	-25.687	-10.481		53
		DT	180	0.537	1	-25.668	-13.791		50

Table 13. Compare of the results between (discrete-SMR) and (Con.-SMR), using SMRTool software.

Station no.	RMRb	Type of failure	Discrete-SMR Value	Con.-SMR Value	Class/Stability (Discrete-SMR Class)	Class/Stability (Con.- SMR Class)
1	58	PS	16	17	V / Completely unstable	V / Completely unstable
		WS	34	32	IV / Unstable	IV / Unstable
		FT	33	34	IV / Unstable	IV / Unstable
		DT	58	57	III / Partially stable	III / Partially stable
2	67	PS	32	21	IV / Unstable	IV / Unstable
2 (LS)		PS	37	26	IV / Unstable	IV / Unstable
		DT	67	66	II / Stable	II / Stable
3	64	PS	37	37	IV / Unstable	IV / Unstable
		DT	79	78	II / Stable	II / Stable
4	67	PS	30	28	IV / Unstable	IV / Unstable
		WS	58	57	III / Partially stable	III / Partially stable
5	71	PS	14	17	V / Completely unstable	V / Completely unstable
		WS	65	61	II / Stable	II / Stable
		WS	50	47	III / Partially stable	III / Partially stable
		DT	74	73	II / Stable	II / Stable
6	70	PS	50	50	III / Partially stable	III / Partially stable
		WS	42	36	III / Partially stable	IV/ Unstable
7	67	PS	24	18	IV / Unstable	V/ Completely unstable
		WS	58	56	III / Partially stable	III / Partially stable
8	64	FT	54	50	III / Partially stable	III / Partially stable
8 (LS)		FT	54	53	III / Partially stable	III / Partially stable
		DT	54	50	III / Partially stable	III / Partially stable

Where: PS=Planar sliding, WS=Wedge sliding, FT=Flexural toppling, DT=Direct toppling, LS= Left side slope

Overall, this research combines and compares the results of both types of SMR methods as presented in Table 13. In the description of stability classification for stations no. 6 and 7, which is the

value of discrete-SMR and continuous-SMR are varying. For wedge sliding, the result of discrete-SMR in station no. 6 is partially stable, while depending on continuous-SMR is unstable. In addition, for planar sliding the result of discrete-SMR at station no. 7 is unstable, whereas dependent on continuous-SMR is completely unstable.

6. Conclusions

- The kinematic analysis and SMR system are very important tools for assessment the stability of rock slopes. Eight rock-cut slopes have been chosen along Shanadar-Goratu main road in Mergasur district from NE-Iraq.
- The Slope kinematic analysis pointed out three types of failure, i.e., Plane sliding, wedge sliding, and toppling (both types) failures. These three mentioned types are best matching with field investigations and observations.
- The SMR study of different stations, indicate that the rock-cut slopes are completely unstable to partially stable with posibility of failure 90% to 40% (class V of very bad slope to class III of a normal slope), which is further confirmed by kinematic analysis. For long stability term and its maintenance, the rock slopes needs immediate taking care to prevent and mitigate chances of failure, so that enhance the rock slope's stability.
- Depending on the SMR-values, stations no.1, 2 and 5 require immediate treatment, such as removing unstable parts and constructing surface drainage and deep drainage pipes into the slopes. Stations no.3, 4, 6, 7 and 8 require concrete support (shotcrete, dental concrete, toe walls) and reinforcement support (bolts, anchors), in addition to the slope station no.8 due to very farovable secondary toppling, it is recommended to remove the unstable rock mass and then use reinforcement support to increase the stability of the rock slope.

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