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Assessment of Rock Slope Stability along Sulaimaniyah -Qaradagh Main Road, Near Dararash Village, Sulaimaniyah, NE-Iraq

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Abstract

The road network in the Baranan mountain, near Dararash village, connecting Sulaymaniyah city with Qaradagh town, plays a major role in socio- economic activities of Qaradagh town and its surrounding villages. Any type of slope failure in the area may cause breaking up in traffic, loss of lives, and injuries.

For assessing the stability of rock slopes in the area, seven stations (rock-cut slopes) were selected along the road and evaluated by kinematic analysis, using DIPS v6.008 software and slope mass rating system (SMRTool - v205 software).

The kinematic analysis revealed that planar and wedge sliding may occur in stations no.2, 5, 6, and 7, flexural toppling may occur in station no.1, direct toppling may occur in station no.2, and oblique toppling may occur in station no.3.

SMR- Tool software for discrete-SMR and continuous-SMR (CSMR) revealed that stations no.2, 5, 6 and 7 are unstable slopes (class IV of a bad slope type) with failure probability of 0.6, with an exception for station no.7 which is a partially stable slope (class III of a normal slope type) with failure probability of 0.4. Station no.1 is partially stable slope (class III of a normal slope type) with failure probability of 0.4 and station no.3 is stable slope (class II of a good slope type) with failure probability of 0.2.

Due to the lack of structural and failure surface data (attitude of discontinuities and slumping surface) in station no.4, stability analysis was interpreted by using the general conventional method, depending on the field criterion and vision. The station can be interpreted as a rotational failure, the upper part of which consists of slump motion and the lower part of flow motion.

Keywords: Dararash village, Kinematic analysis, Slope Mass Rating, Slope support

تقيم استقرارية المنحدرات الصخرية بمحاذاة الطريق الرئيس بين السليمانية – قرداغ، قرب قرية داره ره ش، السليمانية، شمال شرق العراق

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الخلاصه

الطريق الرئيس في جبل برانان، قرب قرية داره ره ش الذي يربط مدينة السليمانية بقضاء قرداغ يلعب دورا مهما في النشاطات الاجتماعية والاقتصادية لقضاء قرداغ والقرى المجاوره له. حدوث اي نوع من الانهيارات الصخرية على الطريق تؤدي الى قطع المواصلات وخسائر في الارواح.

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لتقيم استقرارية المنحدرات الصخرية في المنطقة، تم اختيار سبع محطات (منحدرات صخرية) بمحاذاة الطريق المذكور وتم تقيم تلك المحطات من خلال التحليل الهندسي الحركي (Kinematic analysis) باستخدام برنامج (DIPS-v6.00) وكذلك من خلال نظام اعطاء القيم لكتلة الانحدار ((Slope Mass Rating(SMR) باستخدام برنامج (SMRTool-V205).

اظهر التحليل الهندسي ان كلا من الانزلاق المستوي والاسفيني يمكن ان يحدث في المحطات رقم 2 و5 و6و 7. اما الانقلاب الانزلاقي (Flexural toppling) من الممكن حدوثه في المحطة رقم 1. يمكن حدوث الانقلاب المباشر(Direct toppling) في المحطة رقم 2 والانقلاب المائل (Oblique toppling) يمكن حدوثه في المحطة رقم 3.

اظهر برنامج SMRTool لنوعي SMR (المتقطع والمستمر) ان المحطات رقم 2و5و6 و7 عبارة عن منحدرات غير المستقرة (صنف الرابع ومن نوع المنحدر الرديء) مع احتمالية انهيار بحدود 0.6 مع استثناء في المحطة رقم 7, حيث ان المنحدر يكون مستقر جزئيا (الصنف الثالث من نوع المنحدر الاعتيادي) في حالة تقيمه من خلال SMR المستمر، مع احتمالية الانهيار بحدود 0.4 . المحطة الاولى تكون مستقرة جزئيا (الصنف الثالث من نوع المنحدر الاعتيادي) مع احتمالية الانهيار بحدود 0.4, واخيرا المحطة رقم 3 تكون مستقرة (الصنف الثاني ومن نوع المنحدر الجيد) مع احتمالية الانهياربحدود 0.2.

بسبب قلة المعلومات التركيبية ومعلومات عن سطح الانهيار (عدم الحصول على وضعية الأنقطاعات و وضعية سطح الانهيار) في المحطة رقم 4, حيث تم تحليل وتفسير الاستقرارية لمنحدر هذه المحطة من خلال الطريقة التقليدية العامة, وبالاعتماد على الرؤية والدليل الحقلي يمكن تفسيره بان هذا الانهيار هو من نوع الدوراني الذي يتألف جزئه العلوي من الحركة الزحفية والجزء السفلي من الحركة الجريانية للمواد الارضية .

1-Introduction

Rock slopes, in most road cuts in mountainous areas, are liable to detect instability problems due to changing in the rock mass conditions and external factors induced by the environment, such as seismic activities and water in the slope [1]. Also, the material characteristics of the rock slope, the slope height, the slope angle, and rock discontinuity orientations (bedding planes, joints, faults, ….etc.) play an important role in the instability problem of the road cut slopes [2].

Slope failure is known as one of the most repeated natural disasters which can lead to huge loss of property and lives and breaking up in traffic [3].

There are several methods for slope stability assessments. These methods can be basically grouped into four categories, i.e. Kinematic analysis, limit equilibrium, numerical modelling and rock mass classification (empirical methods) [4,5].

Road networks are the only way of communication in mountainous regions. The presently studied road-cut slopes along Dararash village-Qaradagh town in Sulaymaniyah city –NE Iraq failed many times at various locations and have become more liable to different types of failure. The failed materials occasionally blocked the road, as occurred in February, 2019. Figure-1 shows that the slumped materials were removed from the road at station no.4.

In this study, seven stations in the road-cut slopes were selected along the road. The possibility of failure occurrence in the six rock slope stations (stations: 1, 2, 3, 5, 6 & 7) was assessed by Kinematic analysis through DIPS v6.008 software [6] to analyze the stability and determine the slope failure type. Also the stability of the six mentioned rock slopes was assessed using SMRTool –v205 [7] to identify failure types, stability conditions, and failure probability, so that corresponding support measures can be proposed. Station number four (St. no.4) was analyzed by a general conventional method.

Geological surveys were conducted in February and March, 2019, during the rainfall period. The attitude measurements of slope face, bedding planes, joints, and faults were recorded in dip direction /dip amount manner. Also, the values of RMR parameters were recorded for slope stability proposed by Bieniawski [8], while those of SMR were recorded using the classification system proposed by Romana [9] and Tomas [10]. Such methods for slope stability analysis have been applied for understanding the stability types (planar sliding, wedge sliding and toppling), and probability of failure for natural and excavated slopes [3,4,11,12].

Figure 1-General view showing slumped materials to the left of the road and removed part that was blocked the road at station no.4

2- Location of the study area

The study area is located in Sulaymaniyah governorate, Kurdistan region, NE-Iraq, about 25km to the south of Sulaymaniyah city. It lays between latitudes 35º 21′ 45″ and 35º 22′ 20″and longitudes 45º 29′ 00″ and 45º 29′ 45″, as in Figure-2.

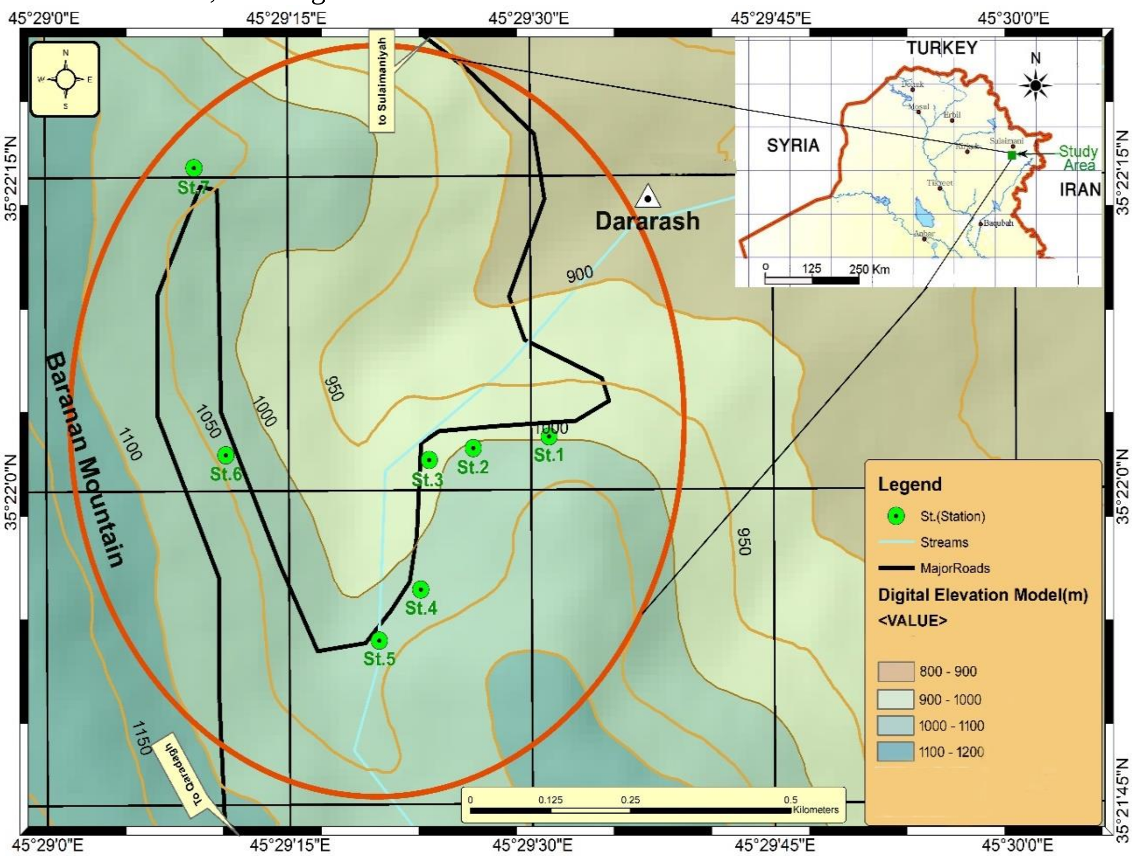

Figure 2-Location map showing stations of rock slopes in the study area

3- Geological setting of the study area

Tectonically, the area is located in the high-folded zone of western Zagros fold-thrust belt. From a structure point of view, it represents a homoclinal structure of intermediate dip that forms striking ridges, due to alternating resistant and nonresistant rocks [13]. The resistant rocks are limestone, dolomitic limestone, and marly limestone, whereas the nonresistant rocks are marl and weak sandstone, as in Figure-3.

Stratigraphically, the study area is composed of the oldest to youngest Kolosh, Sinjar, Khurmala and Gercus Formations (Figure-3). Kolosh Formation is of the Middle Paleocene–Early Eocene, whereas Sinjar, Khurmala and Gercus Formations are of the Late Paleocene–Early Eocene [14].

Formerly, the complete succession of carbonate rocks between Kolosh at the bottom and Gercus at the top was considered as Sinjar Formation. However, Karim [15] described the upper part of carbonate rocks succession as Khurmala Formation which consists of dolomitic limestone, marly limestone, with some thick beds of sandstone and conglomerate. The study emphasized that the lower part is Sinjar Formation which consists of intercalation of thick limestone beds and marl.

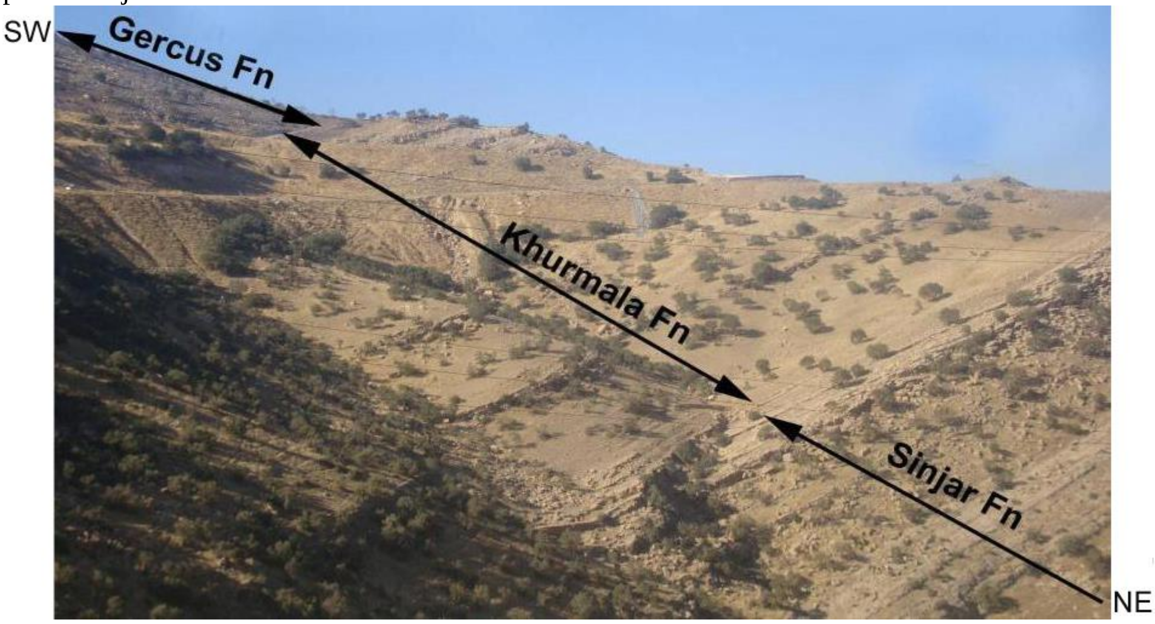

Figure 3-Dararash Stratigraphic section (After Karim [15])

4- Materials and Methods

A detailed engineering geological survey was carried out at seven rock-cut slope stations, three of them (stations no. 1, 2 & 3) are among Sinjar Formation rock-cut slopes, and the other four (stations no. 4, 5, 6 & 7) are within Khurmala Formation rock-cut slopes. All field attitude measurements of discontinuities (bedding planes, joints and faults) and results are in the dip direction / dip amount manner.

This study focuses on the assessment of the stability of the rock-cut slopes by using the kinematic method, as well as SMR, and CSMR systems.

The kinematic method is based on the principle of kinematics which deals with the geometric condition that is required for the movement of the rock block over the discontinuity plane, without considering any forces responsible for the Failure [5, 16, 17, 18].

The kinematic method is an easy practice to use stereographic analysis for the determination of potential failure types (plane, wedge & toppling). It is also used to test the direction in jointed rock mass from angular relationships between discontinuities and slope surface [2, 19, 20]. Markland test [21] is one of the kinematic analysis methods designated to assess the possibility of wedge failure, in which the wedge shaped mass, slides along the line of intersection of two planes. A refinement to Markland's test was discussed by Hocking [22]. The measurements taken in the field are slope angle, dip, and dip direction, spacing, and persistence of discontinuities. The data were graphically analyzed using DIPS v6.008 software [6]. For a planar and clean (no infilling) discontinuity, the cohesion will be zero and the shear strength will be defined solely by the discontinuity friction angle. Friction angles of 32°-36° were calculated by using the tilting method in the field.

SMR was proposed by Romana [23] and it is calculated by adding three adjustment factors, with the excavation method factor, to the $RMR_{b\ (1989)}$ (basic RMR_{1989}). SMR was slightly modified by Anbalagan [24] to incorporate wedge failure along with plane and topple failures, as in Table-1. Tomas [10] developed a continuous-SMR (CSMR), which is a modification of discrete SMR technique of Romana and is based on Bieniawski's RMR technique. The SMR system offers adjustment factors, field guidelines and recommendations on support methods which allow a systematic use of geomechanical classification for slopes [3]. The CSMR offers a unique value of each adjustment factor, unlike a range as in the discrete SMR.

RMRb is computed according to Bieniawski's [8] by adding rating values for five parameters: 1) Uniaxial compressive strength (UCS) of intact rock, 2) Rock quality designation (RQD), 3) Spacing of discontinuities, 4) Condition of discontinuities, and 5) Ground water conditions.

The slope mass rating (SMR) is obtained by subtracting factorial "adjustment factors" (F1, F2, and F3) from RMRb, depending on the discontinuity-slope relationship, and adding a factor depending on the excavation method (F4), as in equation (1):

$$SMR = RMRb + (F1 \cdot F2 \cdot F3) + F4 \qquad \text{...........(1)}$$

The CSMR is also computed using an equation which is similar to discrete SMR, but the difference lies in calculating the adjustment factors (F1, F2 and F3), that they are calculated from SMRTool software, wherein these factors depend on the discontinuity-slope relationship. The factor F4 (Table-1), depending on the excavation method, is the same for discrete SMR as well as CSMR.

The adjustment rating for discrete SMR is the product of three factors proposed by Romana [23] as follows:

i) F1 is the rating of the difference of dip direction between discontinuity and slope face or between the plunge direction of two discontinuities and slope face (Table-1).

ii) F2 is the rating of the dip angle of discontinuity or plunge angle of intersection line of two discontinuities (Table-1).

iii) F3 is the rating of the difference of dip angle between discontinuity and slope dip angles or between the plunge angle of intersection line of two discontinuities and slope angle (Table-1).

Table-2 shows the different stability classes and the empirically found limit values of SMR associated with the different failure modes. Romana [23] also proposed some guidelines for the use of remedial measures based on SMR, as in Figure-4. Although the design of remedial measurements of a slope requires detailed field-work and good engineering sense, these recommendations provide a first approximation during the first preliminary stages of a project [25].

Table 1-Adjustment factors for SMR (Modified from Romana [23] by Anbalagan [24].

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; α_j: joint dip direction; α_s: slope direction α_i: intersection line direction; β_j: joint dip angle; β_i: intersection plunge angle; β_s: slope angle							

Table 2-Description of slope mass rating (SMR) classes [23]

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

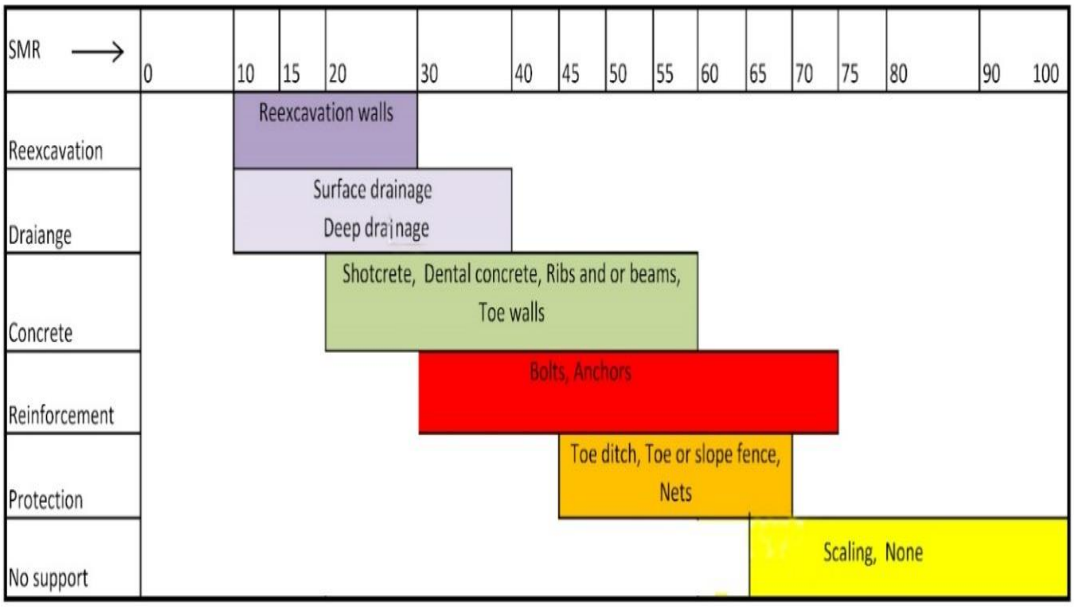

Figure 4-Slope support guidelines based on SMR [23] in Romana [25].

4-Results and Discussion

The road-cut rock slopes constitute the best outcrops for determining the lithological variations, weathering conditions, and structural geological characteristics of the outcrops to record discontinuities patterns. This study is comprised of an investigation of slopes at seven stations (rock slope sites) which have different geotechnical characteristics.

The rock-cut slopes are composed of intermediate-massive beds of limestone and dolomitic limestone that are intercalated with marl beds (stations: 1, 2, 3, 6 & 7). In station no.4, the marl beds are intercalated with marly limestone beds. Station no.5 is composed of thick-massive beds of limestone and dolomitic limestone that is intercalated by thick sandstone beds. The rock-cut slopes have steep to very steep dip angle with a developed system of discontinuities, as in Table-3.

Table 3-Discontinuities data of different stations (rock-cut slope sites)

Station no.	Rock type	Formation	Dip direction / Dip amount					Friction angle
			Slope	Bedding	J1	J2	J3	
1	Limestone	Sinjar	004/70	240/50	185/70	090/58	-	34
2	Limestone		346/77	200/24	274/70	355/75	066/44	32
3	Limestone		255/70	245/24	286/86	034/74		36
5	Limestone	Khurmala	335/85	238/30	318/82	052/80	030/50	34
6	Limestone		060/70	230/20	340/65	060/68	320/50	33
7	Limestone		114/82	228/26	132/80	048/66	058/24	35
Note: J1, J2 & J3 = Joint set number								

Kinematic analysis of rock-cut slopes was conducted for the structurally controlled failure using DIPS v6.008 [6]. The potential failure zone is shown in pink color in the stereographic projection for all the six stations (Figures- 5, 7, 9, 11, 13 & 15). The orientation of discontinuities is with respect to slope forming planar, wedge, and toppling failure type. In a case where the two discontinuities are intersecting and forming wedge in the potential failure zone, the orientation of plunge of intersection line of the two discontinuities was calculated. In the case of direct toppling (block toppling), there must be the intersection of two discontinuities to create detached blocks. In addition, the availability of basal plane facilitates the occurring of direct toppling. The analysis results show that there is the possibility for planar sliding in stations (rock slope sites) 2, 5, 6 and 7, as shown in Figures- (5a, 6, 7a, 8, 9a, 10, 11a & 12). Also, the same four stations show the possibility for wedge sliding, as shown in Figures- (5b, 6, 7b, 8, 9b, 10, 11b & 12). One station (St. no.1) show the possibility for flexural toppling, as shown in Figures- (13c & 14), while another station (St. no.2) show the possibility of direct toppling, as in Figures- (5d & 6). Also, one station (St. no.3) show the possibility of oblique toppling which is a type of direct toppling, as in Figures- (15d & 16).

Due to the lack of structural and failure surface data (discontinuities attitude and attitude of slumping surface) in station no.4, stability analysis was interpreted by using a general conventional method. Depending on the field criterion and vision, it can be interpreted as having a rotational failure, the upper part of which consisting of slump motion and the lower part is of flow motion, as shown in Figure-17. In rotational failure, the surface of rupture is curved concavely upward (spoon shaped), and the slide movement is more or less rotational. A slump is an example of a small rotational landslide. the body of the slump may be further subdivided into discrete blocks, each bounded by smaller individual shear surfaces that dip backward into the slope. Slumps have a characteristic head scarp (which exposes the upper part of the failure surface) and a bulging toe (where material piles up) [26].

All rock slope stations are sites that already failed, as shown in Figures-(6, 8, 10, 12, 14, 16 & 17). The results of all kinematic analyses are listed in Table-4.

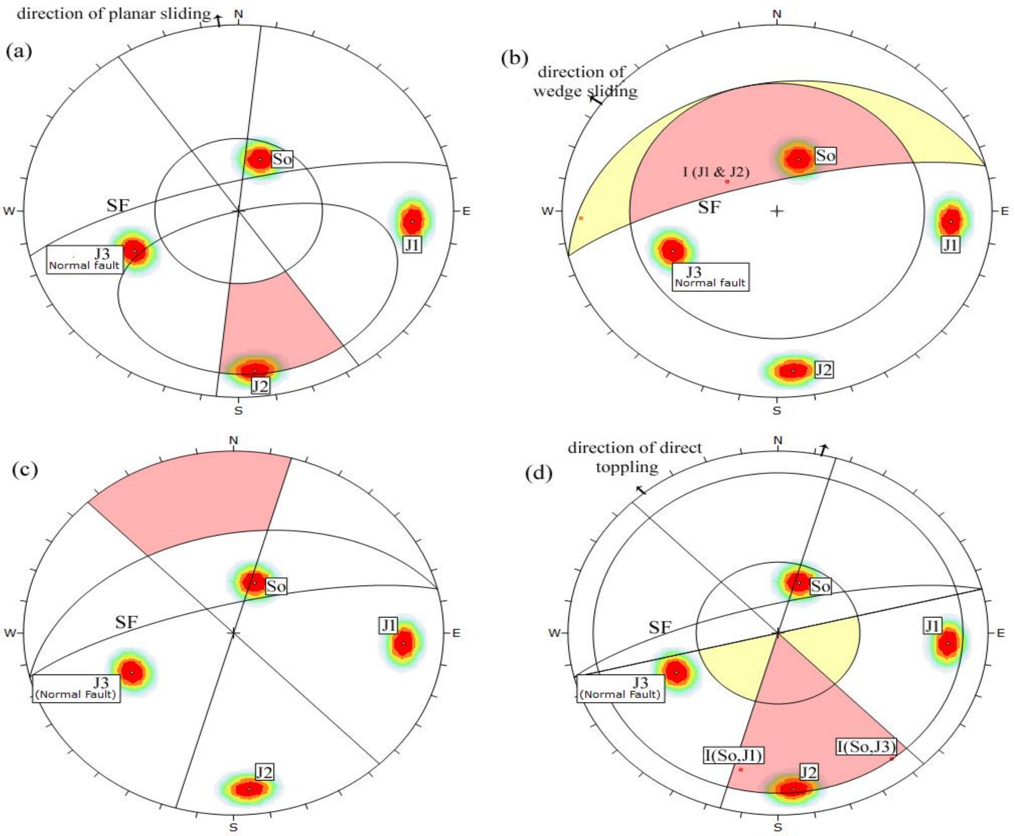

Figure 5-Kinematic analysis for station no.2: (a) plane sliding on J2; (b) wedge sliding on J1 and J2; (c) no flexural toppling; (d) direct toppling about I(So,J1) and I(So,J3) with the aid of J2. Where: SF=slope face; So=bedding plane; J1= joint set no.1; J2= joint set no.2; J3=joint set no.3

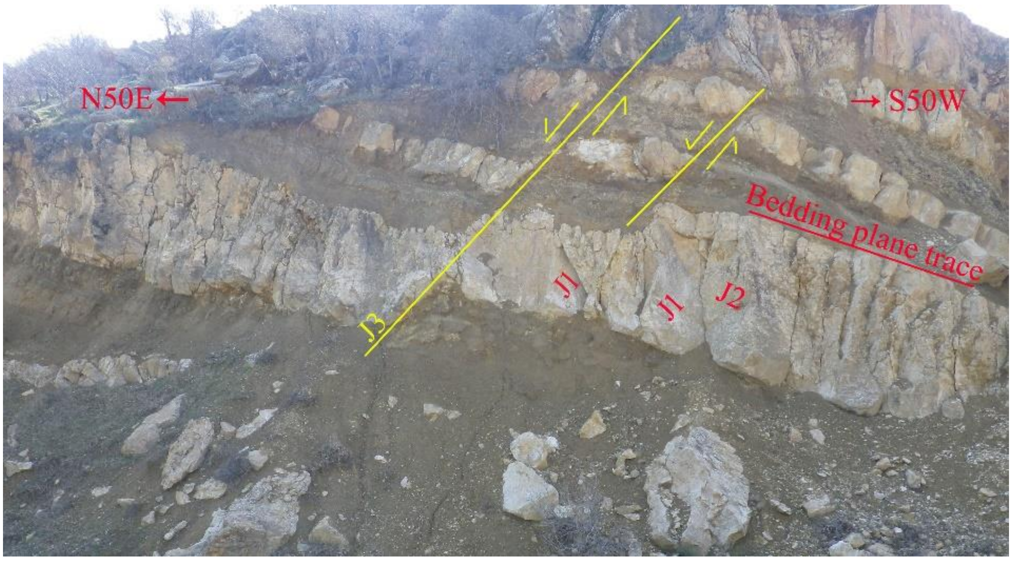

Figure 6-Field view for station no.2 with marked discontinuity sets

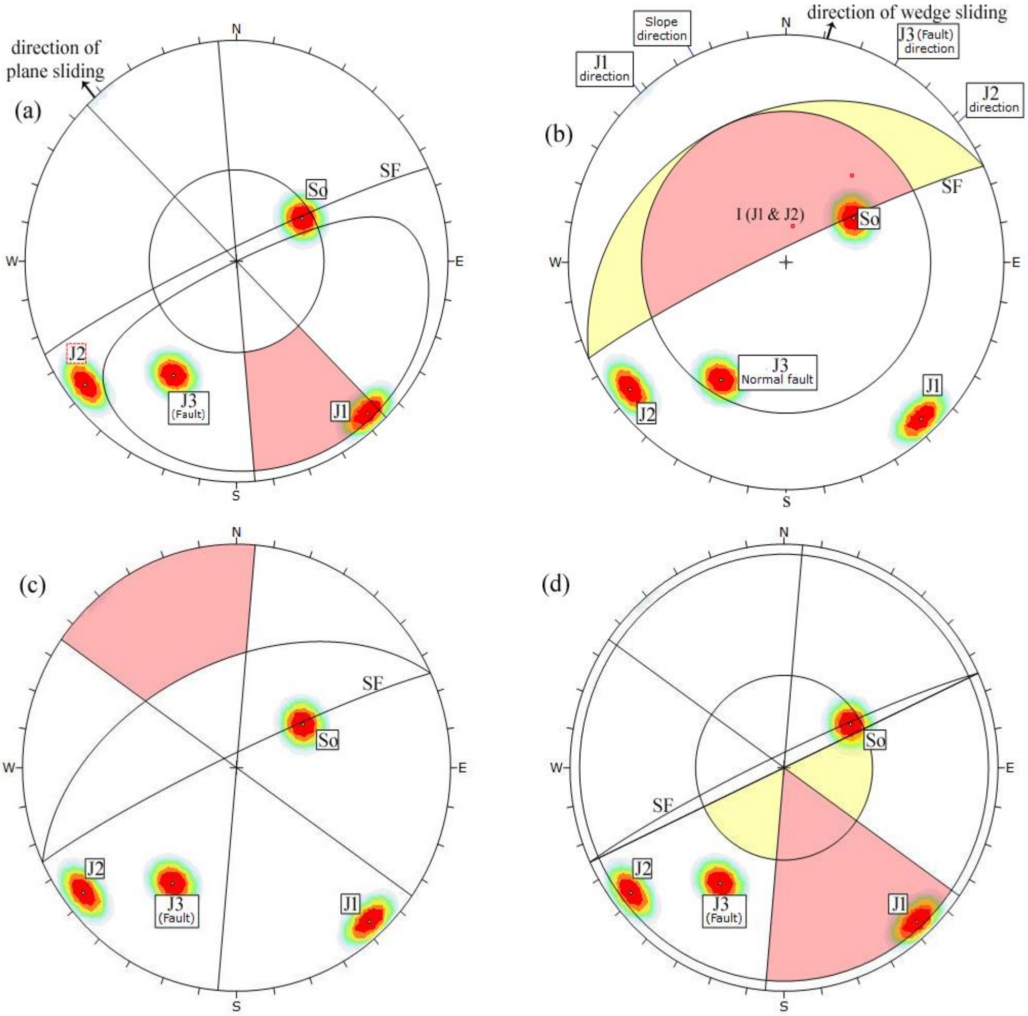

Figure 7-Kinematic analysis of station no.5: (a) plane sliding on J1; (b) wedge sliding on J1 and J2; (c) no flexural toppling; (d) no direct toppling. Where: SF=slope face; So=bedding plane; J1= joint set no.1; J2= joint set no.2; J3=joint set no.3; I=intersection between two discontinuity sets

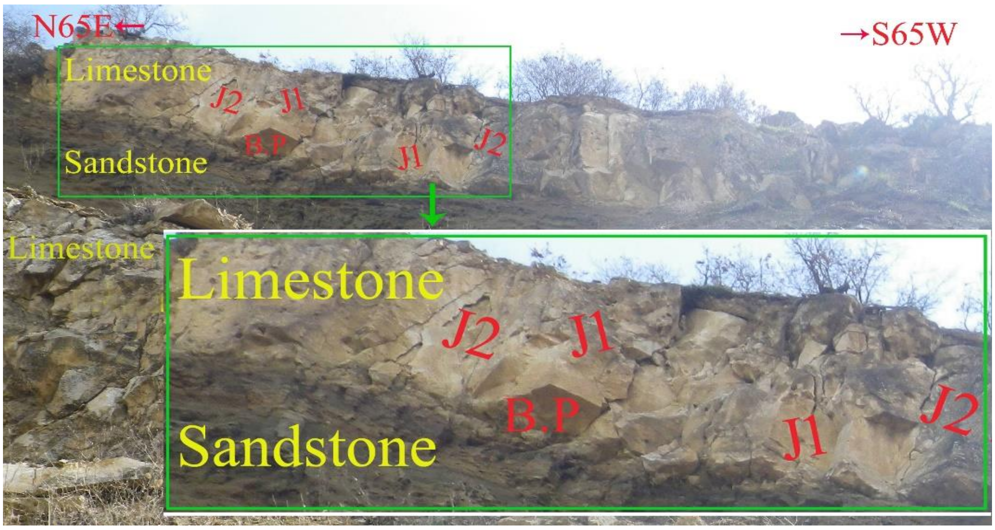

Figure 8-Field view for station no.5 with marked discontinuity sets

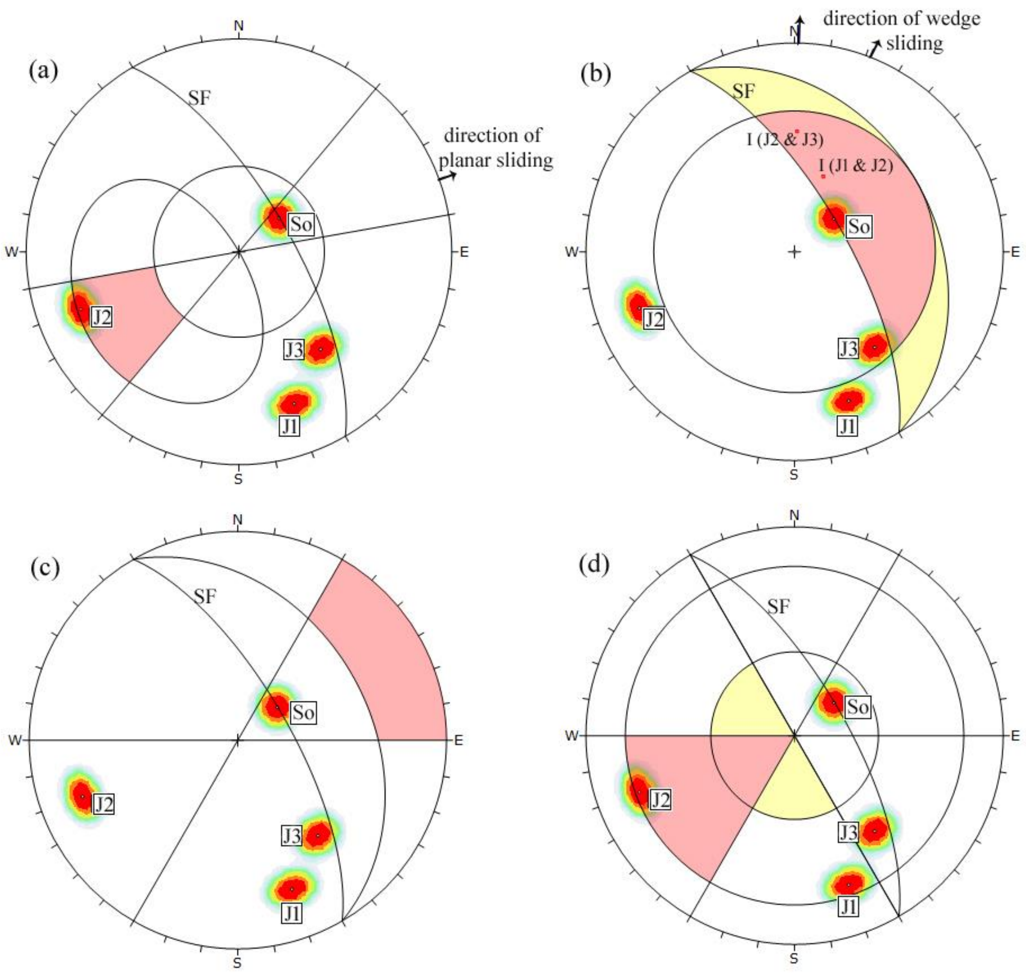

Figure 9-Kinematic analysis of station no.6: (a) plane sliding on J2; (b) wedge sliding on J1 & J2 and J2 & J3; (c) no flexural toppling; (d) no direct toppling. Where: SF=slope face; So=bedding plane; J1= joint set no.1; J2= joint set no.2; J3=joint set no.3; I=intersection between two discontinuity sets

Figure 10-Field view for station no.6 with marked discontinuity sets

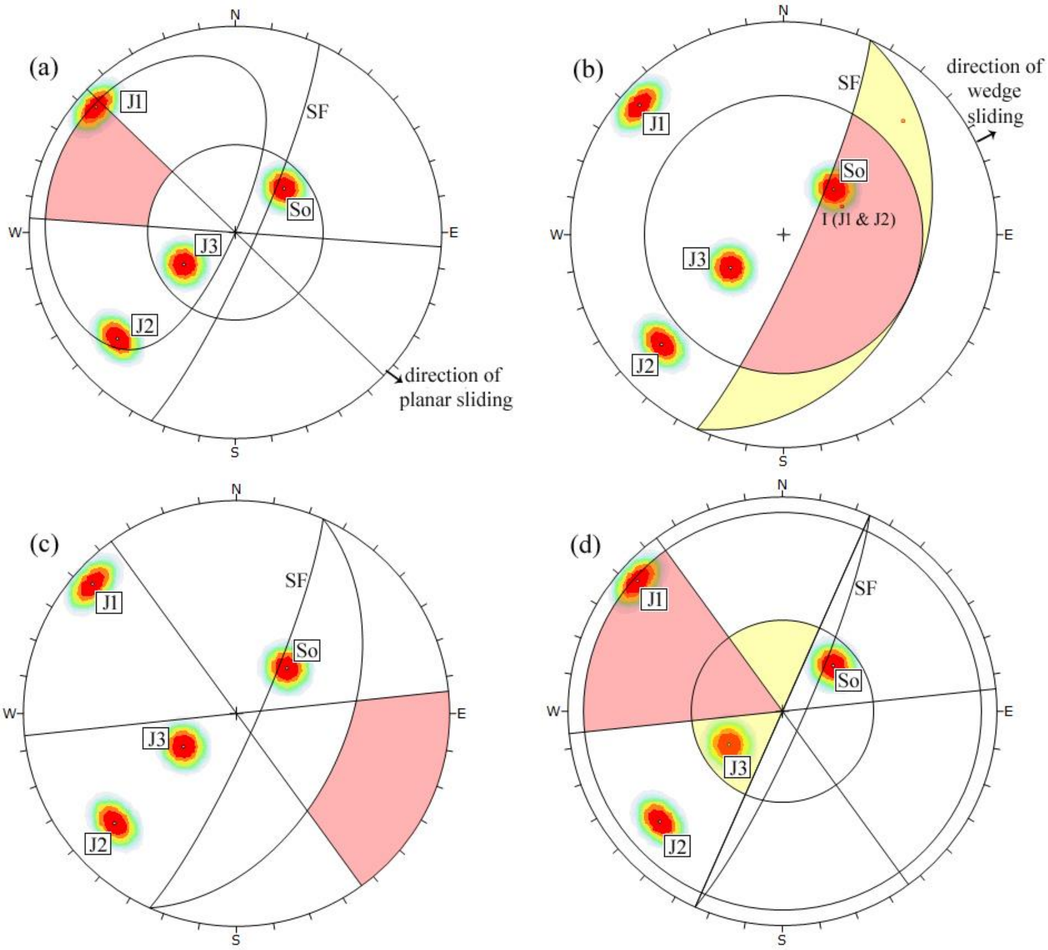

Figure 11-Kinematic analysis of station no.7: (a) plane sliding on J1; (b) wedge sliding on J1 & J2; (c) no flexural toppling; (d) no direct toppling. Where: SF=slope face; So=bedding plane; J1= joint set no.1; J2= joint set no.2; J2=joint set no.3; I=intersection between two discontinuity sets

Figure 12-Field view for station no.7 with marked discontinuity sets

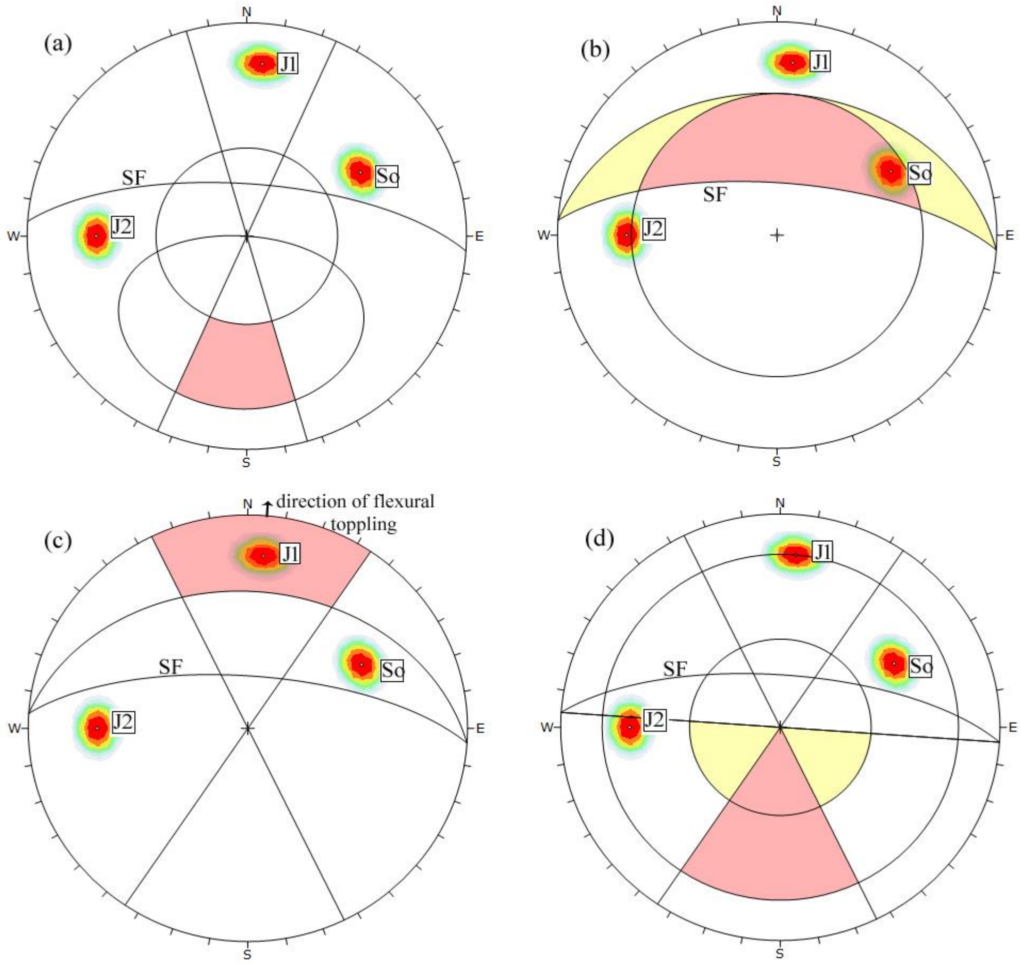

Figure 13-Kinematic analysis for station no.1: (a) no plane sliding; (b) no wedge sliding; (c) flexural toppling about J1; (d) no direct toppling. Where: SF=slope face; So=bedding plane; J1= joint set no.1; J2= joint set no.2; I=intersection between two discontinuity sets

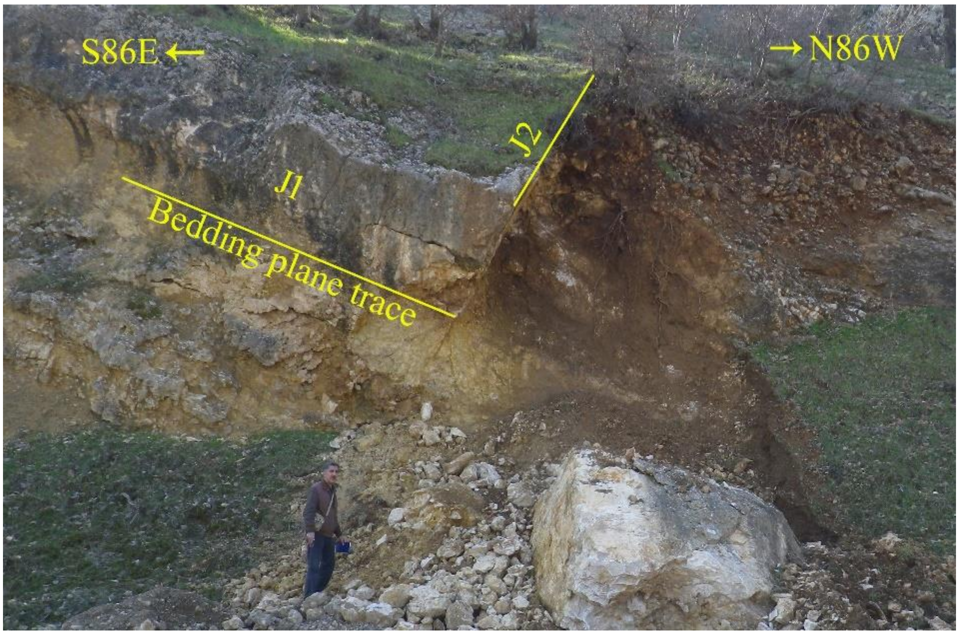

Figure 14-Field view for station no.1 with marked discontinuity sets

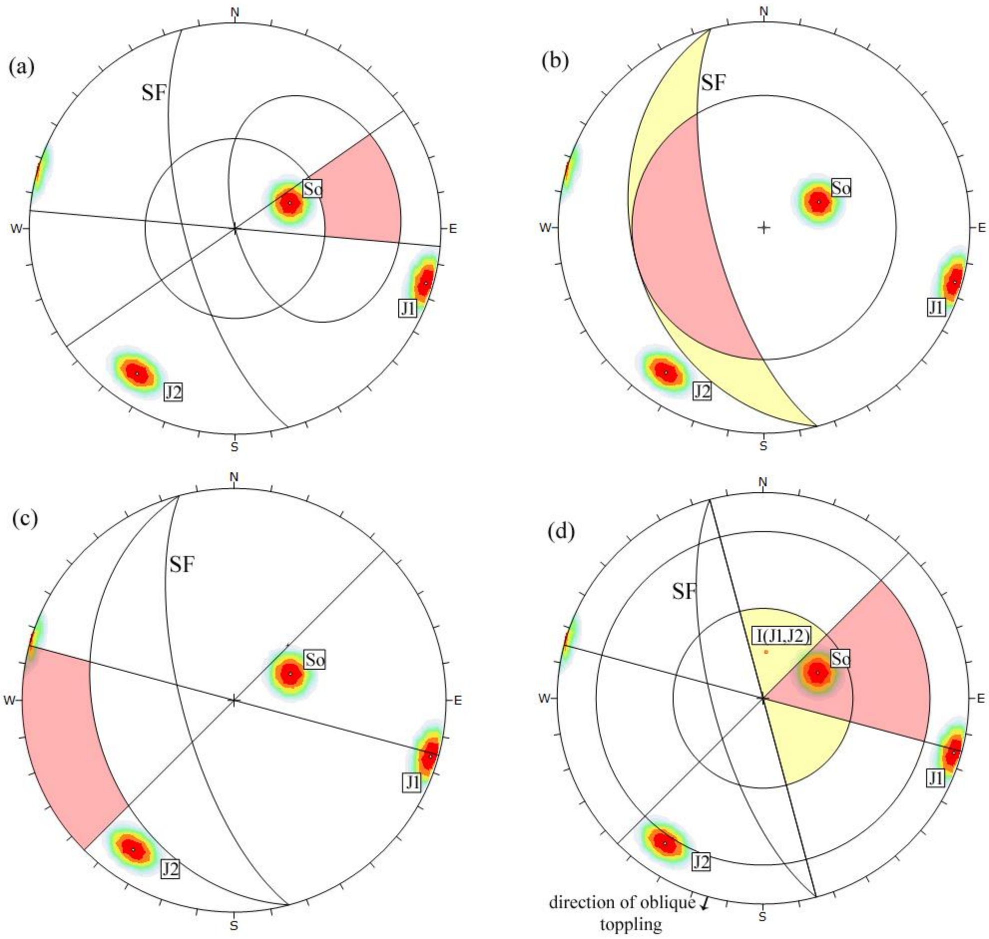

Figure 15-Kinematic analysis for station no.3: (a) no planar sliding; (b) no wedge sliding; (c) no flexural toppling; (d) oblique toppling about I (J1, J2) with the aid of So. Where: SF=slope face; So=bedding plane; J1= joint set no.1; J2= joint set no.2; J3=joint set no.3; I=intersection between two discontinuity sets

Figure 16-Field view for station no.3 with marked discontinuity sets

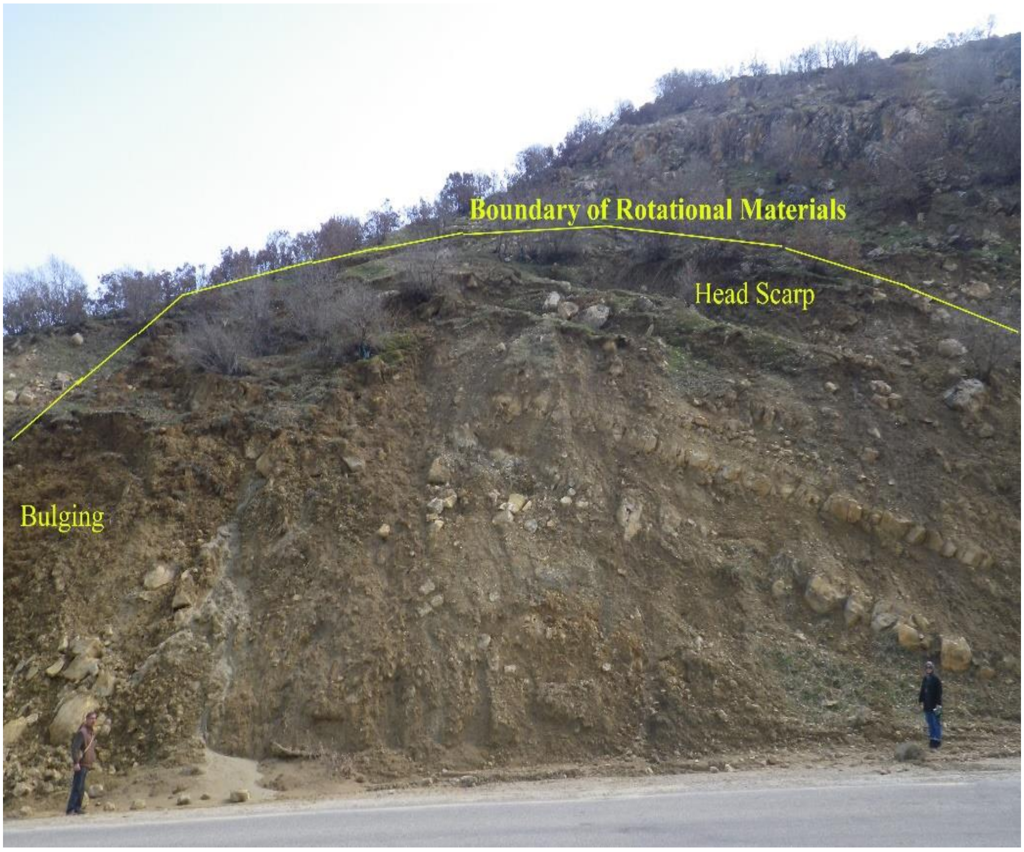

Figure 17-Rotational failure in station no.4, with an upper part of slump motion and lower part of flow motion

Table 4-Results of kinematic analysis of rock slopes using DIPS-Software

Formation	Station No. (Slope site)	Planar sliding, its direction & failure possibility	Wedge sliding, its direction & failure possibility	Flexural toppling, its direction & failure possibility	Direct toppling, its direction & failure possibility	Oblique toppling, its direction & failure possibility
Sinjar	1	-	-	√ (005) 33.33 %	-	-
	2	√ (355°) 25 %	√ (305°) 33.33 %	-	√ (013°&321°) 33.33 %	-
	3	-	-	-	-	√ (196°) 33.33 %
Khurmala	5	√ (318°) 25 %	√ (011°) 33.33 %	-	-	-
	6	√ (070°) 25 %	√ (021°&001°) 33.33 % & 33.33%	-	-	-
	7	√ (132°) 25 %	√ (064°) 33.33 %	-	-	-

RMR values were calculated by the five mentioned parameters. UCS was calculated indirectly from point load testing machine, performed according to the procedure of ISRM [27] using an index-to-strength conversion factor of 21 (k=21), a value reported to work well for a variety of rock types [28]. The results of the point load test are shown in Table-5. RQD was calculated from the relation between RQD and volumetric joint count (Jv), as in equation no. (2). The average spacing of all discontinuities was calculated from the inverse of average frequency of all discontinuities [29], as shown in Tables-6 & 7.

$$RQD = 110 - 2.5\ Jv \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)\ [30]$$

The summary of rock mass characterization for the RMR-parameters in the mentioned stations of rock-cut slopes is shown in Table-8.

After determining the rock mass characteristics in each rock slope station, the required parameters of $RMR_{b\ (1989)}$ were rated according to many tables (a general table and individual tables for each of UCS, RQD and discontinuity spacing) that were proposed by Bieniawski [8]. Then the values of $RMR_{b\ (1989)}$ were determined in each station, as shown in Table-9.

Table 5-Results of point load test and value of uniaxial compressive strength of the intact rock in the rock slopes of stations 1, 2, 3, 5, 6 & 7

Station no.	1	2	3	5	6	7
Formation	Sinjar			Khurmala		
D (mm)	41	40	41	40	40	41
W (mm)	45	62	56	54	64	56
F (KN)	9.90	12.39	12.98	9.10	9.06	9
F (MN)	0.0099	0.01239	0.01298	0.0091	0.00906	0.009
A (mm^2)	1845	2480	2296	2160	2560	2296
$D_e^2=(4DW/\pi)\ m^2$	0.002350	0.003159	0.002925	0.002749	0.003261	0.002922
$Is=F/D_e^2$ (MPa)	4.212765	3.921835	4.437848	3.310294	2.778164	3.080082
$f=(D/50)^{0.45}$	0.914568	0.904462	0.914568	0.904462	0.904462	0.914568
$Is_{(50)}=Is*f$	3.852860	3.547152	4.058716	2.994035	2.512745	2.816944
$UCS=21*Is_{(50)}$ (MPa)	80.910	74.490	85.233	62.874	52.767	59.155
UCS (MPa)	≈ 81	≈ 74	≈ 85	≈ 63	≈ 53	≈ 59
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 6-Volumetric joint count (Jv), Rock Quality Designation (RQD), and average spacing of all discontinuities measurements from joint sets observed in the station no.1

Discontinuities (Bedding plane and Joints)	Set spacing and frequency				Average spacing(m)	Average frequency*
	Spacing (m)		Max. frequency	Min. frequency		
	Min.	Max.				
Bedding plane (S_o)	0.10	1.20	10	0.833	0.65	1.538
Joint set 1 (J_1)	0.30	3	3.333	0.333	1.65	0.606
Joint set 2 (J_2)	0.30	3	3.333	0.333	1.65	0.606
2Random joints(in 1m^2 surface)	-		-	-	-	-
Volumetric joint count Jv=∑Frequencies (joints/m^3)						2.75
RQD = 110 – 2.5 Jv ……………..(if Jv ≤ 4 so RQD=100)				100		
Average frequency of all discontinuities = Jv / 3						0.916
Average spacing of all discontinuities (m)=(1 / average frequency)= 3 / Jv					1.091 m = 1091 mm	
*Average frequency=1/Average spacing…………………………………[29] **For random joints, a spacing of 5m for each random joint is used in the Jv calculation. -RQD = 110 - 2.5 Jv…………...…...(if Jv ≤ 4 so RQD=100)……………[30] -Average frequency of all discontinuities=Jv/3 ………………………… .[29] -Average spacing of all discontinuities (m)=1/average frequency……..[29]						

Table 7-Volumetric joint count (Jv), Rock Quality Designation (RQD) and average spacing of all discontinuities measurements from joint sets observed in the stations

Formation	Station no.	Jv (joints /m^3)	RQD	Average spacing of all discontinuities (mm)
Sinjar	1	2.75	100	1091
	2	3.913	100	766
	3	2.834	100	1059
Khurmala	5	3.43	100	874
	6	5.983	95	501
	7	3.921	100	765

Table 8-Rock mass characterization in the rock slopes of stations no. 1, 2 and 3

Formation name	Sinjar			Remarks
Station No.	1	2	3	
Elevation(a.s.l) (m)	954	967	975	
Rock type	Limestone	Limestone	Limestone	From Field
Strength of intact rock material $UCS_{(50)}$(MPa)	81	74	85	From Table-5
RQD (%)	100	100	100	From Table-7
Average spacing of all discontinuities(mm)	1091	766	1059	
Surface condition of discontinuities	Rough- very rough, slightly weathered, fine filling > 5mm, no separation >5mm, persistence 3-5m	Slightly rough-rough, slightly weathered, fine filling > 5mm, no separation, persistence 1.5 - 2.5m	Rough- very rough, slightly weathered, fine filling > 5mm, several centimeters separation, persistence 2-10m	From field description
Ground water condition	Damp (in winter)	Dry	Damp (in winter)	From field description

Table 8–Continuer Rock mass characterization in the rock slopes of stations no. 5, 6 and 7

Formation name	Khurmala			Remarks
Stability station	5	6	7	
Elevation(a.s.l) (m)	1007	1016	1055	
Rock type	Limestone	Limestone	Limestone	From Field
Strength of intact rock material $UCS_{(50)}$(MPa)	63	53	59	From Table-5
RQD (%)	100	95	100	From Table-7
Average spacing of all discontinuities(mm)	874	501	765	
Surface condition of discontinuities	Slightly rough-very rough, slightly weathered, no filling, separation >5mm, persistence ≈ 5m	Smooth- slightly rough, slightly weathered, fine filling <5mm, separation >5mm, persistence: 1-1.5m	Rough- very rough, slightly weathered, no filling, several centimeters separation, persistence:7-8m	From field description
Ground water condition	Dry	Dry	Dry	From field description

Table 9-Rating of RMR-parameters and values of RMRb(1989) for the rock masses in the rock slopes of stations no. 1, 2, 3, 5, 6 and 7

Formation name		Sinjar			Khurmala		
Station No.		1	2	3	5	6	7
Rating of parameters	Strength of intact rock (UCS)	8	7.7	8.3	6.5	5.5	6.3
	RQD	20	20	20	20	19.2	20
	Average spacing of all discontinuities	16.3	14.2	16	15	12.4	14.2
	Condition of discontinuities	19	19	12	18	13	18.5
	Ground water condition	10	15	10	15	15	15
$RMR_{b\ (1989)}$		73.3	75.9	66.3	74.4	65.1	74
Where: $RMR_{b\ (1989)}$= Basic Rock Mass Rating, with no adjusting factor for discontinuity orientation							

SMR of Romana [23] and CSMR of Tomas [10] were applied using SMRTool-v205 [7] utilizing RMR_b values for all six stations (stations: 1, 2, 3, 5, 6 & 7) from Table-9. The adjustment factors F1, F2 and F3, based on the discontinuity-slope relationship, were calculated for both discrete-SMR and CSMR. The rock slope was excavated using blasting and mechanical drilling method (F4 = 0). The blasting and drilling operation for the excavations of the slope led to the failure of the small and large size blocks (Figures-6, 8, 10, 12, 14 & 16).

For the direction and dip of discontinuities (bedding planes, joints, faults) and slope, the excavation method of slopes and $RMR_{b\ (1989)}$-value was applied using SMRTool-v205 software, which includes both discrete-SMR of Romana and CSMR of Tomas [10]. The SMRTool-Software was used to calculate the flexural toppling in station no.1, as in Figure-17, as well as the planar sliding, wedge sliding, and toppling (flexural, direct and oblique toppling) failure for all the six stations, as shown in Tables- (10 & 11).

The results obtained using SMR- Tool software for discrete-SMR and CSMR revealed that stations no.2, 5, and 6 and 7 are unstable slopes (class IV of a bad slope type) with failure probability of 0.6. However, station no.7 was also shown to be a partially stable slope (class III of a normal slope type), by CSMR assessment, with failure probability of 0.4. Station no.1 is partially stable (class III of a normal slope type) with failure probability of 0.4, whereas station no.3 is stable slope (class II of a good slope type) with failure probability of 0.2.

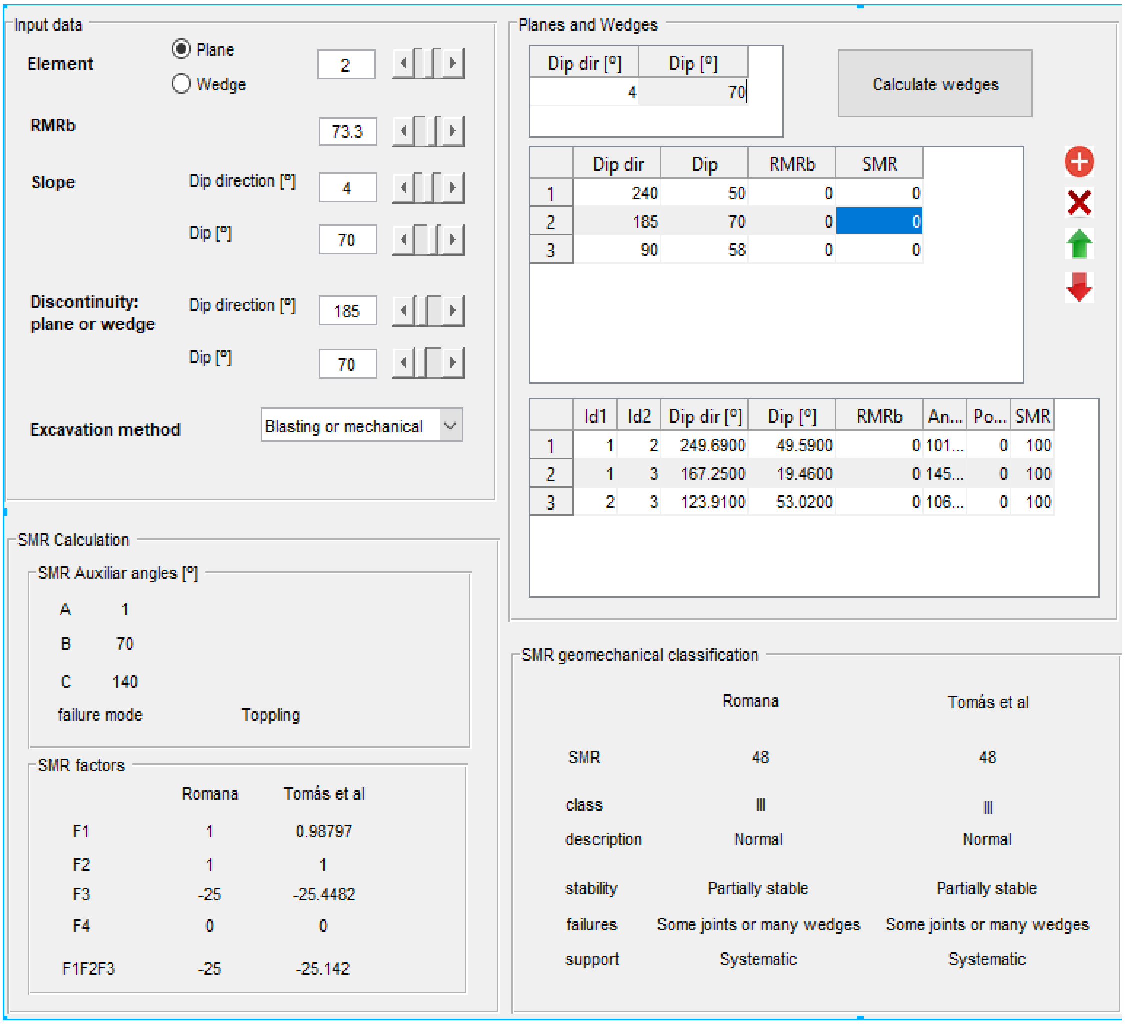

Figure 18-Assessment of rock slope stability at station no1, showing flexural toppling about joint set no.1 for both discrete-SMR and continuous-SMR (CSMR), using SMRTool-software

Table 10-Results of discrete slope mass rating (SMR), using SMRTool software

Station no	1	2	3	5	6	7
RMR_b	73.3	75.9	66.3	74.4	65.1	74
Slope (direction / dip)	004° / 70°	346° / 77°	255° / 70°	335° / 85°	060° / 70°	114° / 82°
Failure type	(a) F.T	(a) P.S (b) W.S (c) D.T (d) D.T	(a) O.T	(a) P.S (b) W.S	(a) P.S (b) W.S (c) W.S	(a) P.S (b) W.S
Failure direction	(a) 005°	(a) 355° (b) 305° (c) 013° (d) 321°	(a) 196°	(a) 318° (b) 011°	(a) 070° (b) 021° (c) 001°	(a) 132° (b) 064°
Plane or (intersection line) of Failure	(a) J1	(a) J2 (b) J1 & J2 (c) J2 (d) J2	(a) So	a) J1 (b) J1 & J2	(a) J2 (b) J1 & J2 (c) J2 & J3	(a) J1 (b) J1 & J2

F1	(a) 1	(a) 0.85 (b) 0.15 (c) 0.4 (d) 0.4	(a) 0.15	(a) 0.7 (b) 0.15	(a) 0.7 (b) 0.15 (c) 0.15	(a) 0.7 (b) 0.15
F2	(a) 1	(a) 1 (b) 1 (c) 1 (d) 1	(a) 1	(a) 1 (b) 1	(a) 1 (b) 1 (c) 0.85	(a) 1 (b) 1
F3	(a) -25	(a) -50 (b) -50 (c) 0 (d) 0	(a) -25	(a) -50 (b) -50	(a) -50 (b) -60 (c) -60	(a) -50 (b) -60
F1. F2. F3	(a) -25	(a) -42.5 (b) -7.5 (c) 0 (d) 0	(a) -3.75	(a) -35 (b) -7.5	(a) -35 (b) -9 (c) -7.65	(a) -35 (b) -9
F4	0	0	0	0	0	0
Discrete-SMR value	(a) 48	(a) 33 (b) 68 (c) 75 (d) 75	(a) 62	(a) 39 (b) 66	(a) 30 (b) 56 (c) 57	(a) 39 (b) 65
Class / Stability	(a)III/ Pa.Sta	(a) IV/Unsta (b) II /Stable (c) II /Stable (d) II /Stable	(a) II /Stable	(a) IV/Unsta (b) II /Stable	(a) IV/Unsta (b)III/ Pa.Sta (c)III/ Pa.Sta	(a) IV/Unsta (b) II /Stable
Where: P.S=Planar sliding, W.S=Wedge sliding, F.T=Flexural toppling, D.T=Direct toppling, O.T=Oblique toppling (Lateral direct toppling), F1,F2&F3 are adjustment factors of SMR, F4=Method of the slope excavation, Pa.Sta=Partially stable, Unsta=Unstable						

Table 11-Results of continuous slope mass rating (CSMR), using SMRTool software

Station no	1	2	3	5	6	7
RMR_b	73.3	75.9	66.3	74.4	65.1	74
Slope (direction / dip)	004° / 70°	346° / 77°	255° / 70°	335° / 85°	060° / 70°	114° / 82°
Failure type	(a) F.T	(a) P.S (b) W.S (c) D.T (d) D.T	(a) O.T	(a) P.S (b) W.S	(a) P.S (b) W.S (c) W.S	(a) P.S (b) W.S
Failure direction	(a) 005°	(a) 355° (b) 305° (c) 013° (d) 321°	(a) 196°	(a) 318° (b) 011°	(a) 070° (b) 021° (c) 001°	(a) 132° (b) 064°
Plane or (intersection line) of Failure	(a) J1	(a) J2 (b) J1 & J2 (c) J2 (d) J2	(a) So	a) J1 (b) J1 & J2	(a) J2 (b) J1 & J2 (c) J2 & J3	(a) J1 (b) J1 & J2
F1	(a) 0.9879	(a) 0.87196 (b) 0.23278 (c) 0.36559	(a) 0.16276	(a) 0.64 (b) 0.26648	(a) 0.84995 (b) 0.24614 (c) 0.18078	(a) 0.60574 (b) 0.20214

		(d) 0.42775				
F2	(a) 1	(a) 0.98633 (b) 0.97857 (c) 1 (d) 1	(a) 1	(a) 0.99131 (b) 0.98779	(a) 0.97959 (b) 0.96508 (c) 0.89442	(a) 0.99003 (b) 0.97605
F3	(a) -25.4482	(a) -51.145 (b) -58.0735 (c) -0.56989 (d) -0.41779	(a) -25.4793	(a) -53.855 (b) -55.6683	(a) -51.145 (b) -58.3674 (c) -59.3213	(a) -51.145 (b) -58.8706
F1. F2. F3	(a) -25.142	(a) -43.9869 (b) -13.2287 (c) -0.20885 (d) -0.17871	(a) -4.1469	(a) -34.1676 (b) -15.1796	(a) -42.5836 (b) -13.8646 (c) -9.592	(a) -30.6714 (b) -11.6153
F4	0	0	0	0	0	0
CSMR value	(a) 48	(a) 31 (b) 62 (c) 75 (d) 75	(a) 62	(a) 40 (b) 59	(a) 22 (b) 51 (c) 55	(a) 43 (b) 62
Class / Stability	(a) III/Pa.Sta	(a) IV/Unsta (b) II /Stable (c) II /Stable (d) II /Stable	(a) II /Stable	(a) IV/Unsta (b) III/Pa.Sta	(a) IV/Unsta (b) III/Pa.Sta (c) III/Pa.Sta	(a) III/Pa.Sta (b) II /Stable
Where: P.S=Planar sliding, W.S=Wedge sliding, F.T=Flexural toppling, D.T=Direct toppling, O.T=Oblique toppling (Lateral direct toppling), F1,F2&F3 are adjustment factors of SMR, F4=Method of the slope excavation, Pa.Sta=Partially stable, Unsta=Unstable						

Conclusions:

The kinematic method and SMR are the most widely used techniques for rock slope assessment.

The kinematic analysis revealed that planar and wedge sliding may occur in stations no.2, 5, 6, and 7, flexural toppling may occur in station no.1, direct toppling may occur in station no.2, and oblique toppling may occur in station no.3. From the results of kinematic analysis, it can be concluded that the most vulnerable slopes to failure are rock slopes of stations no.2 & 6 (each with three failure possibilities), followed by rock slopes of stations no.5 & 7 (each with two failure possibilities), whereas the least vulnerable slopes to failure are rock slopes of stations no.1 & 3.

The Continuous-SMR system was shown to be more sensitive to slope characteristics and provides finer rating values than those obtained by using the discrete-SMR system. In the worst conditions (the least value of SMR), the values of discrete-SMR range from 30 to 62 and those of continuous-SMR range from 22 to 62, wherein these values are the description of bad – good slopes and unstable – stable slopes, with failure probability of 0.6 – 0.2.

According to the value of discrete-SMR and CSMR, the most unstable rock slope is of station no.6 (SMR=30; CSMR=22) and the most stable one is of station no.3 (SMR & CSMR= 62). From the comparison of the results of discrete-SMR and CSMR (tables no. 10 & 11), two cases showed different results in SMR-classes and Stability conditions. The first case is wedge failure, with a change from class II/stable slope (discrete-SMR) to class III/partially stable slope (CSMR) in station no.5. The second case is planar failure, with a change from class IV/unstable slope (discrete-SMR) to class III/partially stable slope (CSMR) in station no.7.

Depending on the SMR-values, stations no.2 & 6 require immediate treatment, such as removing unstable parts and constructing surface drainage and deep drainage pipes into the slopes. Stations no.1, 5 & 7 require concrete support (shotcrete, dental concrete, toe walls) and reinforcement support (bolts, anchors), while station no.3 requires protection support, such as toe ditch and toe fence.

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