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Assessment of Rock Slope Stability in Khanuqa Anticline, Northern Iraq

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ABSTRACT

Rock failures are a very frequent phenomenon, especially on slopes that cut off roads in mountainous areas. The road leading to the village of Al-Naml, which is located north of Tikrit City, is considered the most important road that connects this village to the neighboring areas and cities. This road sometimes experiences several rock failures that cause difficulty in transportation for people, especially in winter and spring. Therefore, evaluating the stability of rocky slopes on such a road is very necessary. Four slope stations are selected along this road to evaluate the stability of rocky slopes using different techniques. The selection of slope stations is based on the difference in the pattern of discontinuities, the change in the morphology of the slope, and the difference in the type of failures. The failure that occurred in the area is a rockfall, while the expected one is direct toppling. The information taken from the field has been analyzed to determine the degree of stability probability through engineering analysis (Kinematic analysis) using the DIPS-v6.008 program, as well as through the system of giving values to the slope mass rating (SMR) using the SMRTool-V205 program. The engineering analysis (kinematic analysis) showed that wedge sliding is possible at all stations (1,2,3,4), while plane sliding, flexural, toppling, and direct toppling are not likely to occur. The values of SMR for both Discrete-SMR and Continuous-SMR for the slopes of all stations range from (37-1) and (37-5), respectively. It is observed that the values at stations (1,2,3) are within a completely unstable range with a probability of failures of about (0.9), while the value at station (4) is within the unstable range with a probability of failure of about (0.6).

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تقييم إستقرارية المنحدرات الصخرية في طية خانوكة المحدبة، شمال العراق

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الملخص

تعتبر الانهيارات الصخرية من الظواهر التي تحدث بصورة متكررة جدا خصوصاً في المنحدرات التي تقطع الطرق في المناطق الجبلية. يعتبر الطريق المؤدي الى قرية النمل الذي يقع شمالي مدينة تكريت الطريق الاهم الذي يربط هذه القرية بالمناطق والمدن المجاورة لها. تقع في هذا الطريق أحيانا عدد من الأنهيارات الصخرية التي تسبب صعوبة في النقل والمواصلات للناس، وخصوصا في فصلي الشتاء والربيع. لذلك ان تقييم استقرارية المنحدرات الصخرية في مثل هذا الطريق يكون ضرورياً جداً. تم اختيار أربع محطات انحدار على امتداد هذا الطريق وذلك لتقييم استقرارية المنحدرات الصخرية بتقنيات مختلفة. يكون اختيار محطات الانحدار على أساس الاختلاف في نمط الانقطاعات والتغير في مورفولوجيا المنحدر والاختلاف في نوع الانهيار. إنَّ الانهيار الحاصل في المنطقة هو السقوط الصخري، اما المتوقع حدوثه فهو الانقلاب المباشر، وقد تم تحليل المعلومات المأخوذة من الحقل لمعرفة درجة احتمالية الإستقرارية من خلال التحليل الهندسي (التحليل الحركي) باستخدام برنامج DIPS-v6.008 وكذلك من خلال نظام اعطاء القيم لكتلة الانحدار Slope Mass Rating (SMR) (باستخدام برنامجSMRTool-V205) الذي يستخدم لتحديد استقرارية المنحدر. اظهر التحليل الهندسي (التحليل الحركي) بأن الانزلاق الاسفيني هو الممكن حدوثه في جميع المحطات (1,2,3,4)، اما الانزلاق المستوي والانقلاب الانثنائي والمباشر فلا توجد احتمالية حدوثها. أن قيم كتلة الانحدار لكل من Discrete-SMR و46 Continuous-SMR لمنحدرات كل المحطات تتراوح مابين (1-37) و (5-37) على التوالي. وقد لوحظ بأن القيم في المحطات (1,2,3) تقع ضمن نطاق غير مستقر تماماً مع احتمالية حدوث الأنهيار بحدود (0.9)، أما قيمة في المحطة (4) فتقع ضمن النطاق غير المستقر مع احتمالية الأنهيار بحدود (0.6).

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Introduction

The stability of rocky inclines is regarded as a crucial subject in the realm of engineering geology due to its significance in assessing and understanding the stability of mountainous regions that host a multitude of engineering structures including highways, rock excavations for tunnels, mining operations, agricultural terraces, and various forms of construction; along with its material relevance and its influence on human existence (Qader and Syan, 2021). Moreover, the stability of rock slopes stands as one of the primary hazards and challenges confronting geologists, who complement the efforts of civil engineers, as these challenges manifest as landslides on slopes supporting railways and roads, as well as essential public infrastructure, and during the road construction process for vehicle transit (Aboud et al., 2017). Landslides represent one of the most perilous varieties of environmental calamities that individuals living in mountainous regions face, occurring when the conditions for them are met. The failure may transpire gradually or abruptly. Numerous factors contribute to landslides, with gravity being the most significant (Bromhead, 1992). When the resistance of the rock masses is comprising, i.e., the slope matches or exceeds the gravitational force, it indicates a state of equilibrium, meaning the rock mass is stable. However, any disruption of this balance results in instability within the rock masses, leading to a collapse of the slope through various mechanisms (Al-Jawadi, 2024). The landslides represent the movement of land masses in the form of rocks, debris, or other earth materials towards the bottom of the slope. So, we can define slope failure as the movement of land masses under the influence of their weight towards the bottom of the slope. Instability of these slopes may result from changes in the conditions of the rock masses that form these slopes, as well as external factors such as slope water and seismic activities. The most important factor, which is considered the basic factor in the failure of rock slopes, is the presence of joints, faults, and surfaces that are applied within the surfaces of the discontinuities of the rock masses. Rock failures are classified into four types (plane failure, wedge slide, overturning, and rotational failure) according to Wyllie and Mah (2005).

The objective of this study is a comprehensive work of the engineering geological survey of rock slopes beyond a traditional engineering geological survey that focuses specifically on the slopes adjacent to the road within the designated study area, intending to identify and analyze thoroughly the various factors that significantly influence the stability of these slopes. While also pinpointing the precise locations where collapses have previously occurred, as well as those areas that may be at risk of future collapses, and assessing their potential impacts through the application of a systematic slope classification methodology known as the slope mass rating (SMR) system. All of these will be executed using the advanced SMRTool-v205 program; furthermore, we will employ the kinematic analysis technique to anticipate probable failures using the DIPS-v6.008 program.

Location of the Study Area

The research location resides in the Salahaldin Governorate in northern Iraq, positioned approximately 120 kilometers north of Tikrit City, specifically within the Khanuqa anticline, adjacent to the roadway leading to the village of Al-Naml situated between the northing (3904000 - 3907000) and easting (350000 - 348000) in the UTM coordinate system, within the Khanuqa anticline as illustrated in Figure (1).

Tectonic and Structural Geology

Tectonically, the research area is located in northern Iraq within the Low Folded Zone of the Unstable Shelf (Buday and Jassim, 1984). The Khanuqa anticline runs in a northwest-southeast orientation and is identified as a semi-symmetric double-plunging anticline. It is flanked by the adjacent Makhoul anticline by a syncline known as Wadi Al-Jafr, which is broad at its center and narrow at its plunge (Fouad, 2002). According to Darwish (2010), the Khanuqa anticline is designated as an open fold (gentle fold) following the classification of Fleuty (1964). This fold, akin to other formations in the vicinity, emerged during the second phase of the Alpine orogeny in the Pliocene epoch (Dewey et al, 1973). The area exhibits fractures induced by tectonic activities, including joints and fissures with sets of fractures present within the gypsum-holding formation (the Fatha Formation), filled with clay materials. The fracture sets are also present within the sandstone strata of the Injana Formation. The region showcases numerous surface and subsurface structures resulting from the aforementioned structural dynamics, with the formation of ground elevation intricately linked to the mountain-building processes occurring in the Zagros Highlands (Al-Lahebi, 2023).

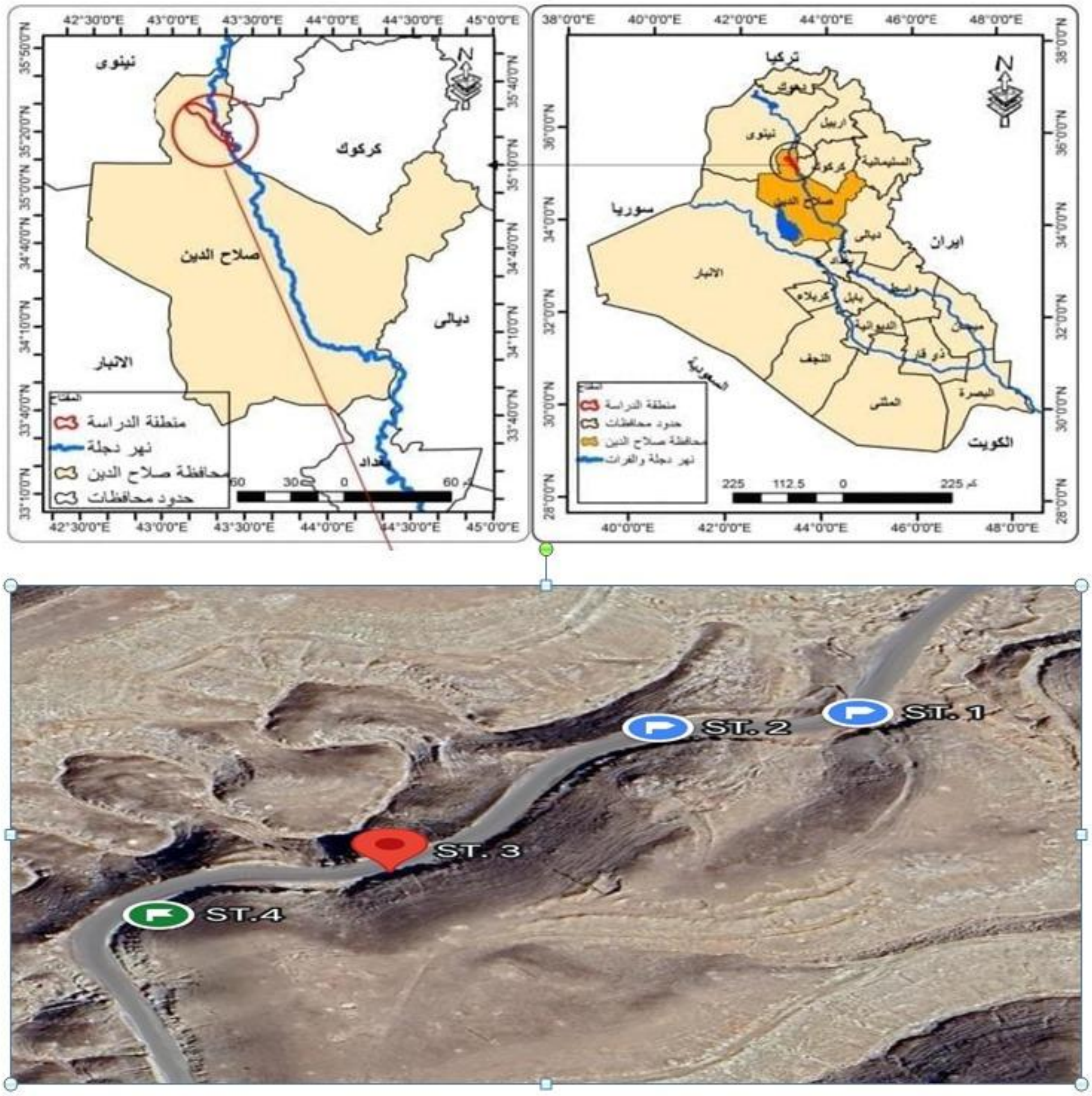

Fig. 1. Location of the study area and selected locations.

Stratigraphy of the Study Area

The geological field of the designated region focused solely on the strata outcrops in the study area, revealing the Fatha Formation (Middle Miocene), alongside the Injana Formation (Late Miocene), as well as Quaternary deposits.

Fatha Formation:

(Middle Miocene) manifests as a plethora of rocky outcrops across the study area, comprising alternating sedimentary sequences where substantial layers of gypsum intersperse with marl, marly limestone, limestone, and additional marl strata. This formation is dated to the Middle Miocene (Bellen et al., 1959). Many scholars, including Buday (1980), concur that this formation accumulated in shallow, highly saline coastal basins, resulting in a notable lack of fossils, primarily represented by micro foraminifera and ostracodes found within the limestone.

Injana Formation:

(Upper Miocene) is initially described in the Fars region of southwestern Iran by Busk and Mayo (1918), as noted by Buday and Jassim (1987), this formation dates back to the

Upper Miocene (Bellen et al., 1959), previously referred to as Upper Fars (Adeeb, 2006), but later renamed to Injana Formation after selecting a prime section in the Injana area within the southern Hamrin anticline south of Tuz (Al-Rawi, 1981) corresponding to the Upper Miocene period. This formation is prevalent as deposits at the onset of the Alpine molasses and is widespread across northern Iraq, while in the southern regions, these deposits vanish, giving way to the Dibdiba Formation. It consists of exemplary sequences of sandstone, alluvial, and clay deposits (Buday and Jassim, 1987), with the exposures in the study area comprising sequences of sandstone and clay.

Quaternary Sediments:

include floodplain sediments, alluvial fans, and river terraces. Flood sediments are a blend of coarse-grained materials (gravel and sand) and fine-grained constituents (clay and silt) along the banks of perennial streams, where the grain size diminishes as moving away from the river channel, as these sediments emerge during flood events (Thornburt, 1969; Ritter, 1986).

Geomorphology of the Study Area

The study area is characterized by the presence of some geomorphic phenomena related to its structure and geological conditions, as well as the occurrence of erosional and weathering processes (Al-Ani, 1997; Al-Talib, 2021). Several factors contribute to the appearance of these phenomena, including erosional phenomena caused by rock erosion processes, rock erosion and ground heights, or fluvial phenomena, namely valleys formed by fluvial sedimentary processes extending along the river channel, and valleys formed by geomorphic erosional processes, or structural phenomena represented by geomorphic factors that cause the movement of rock masses (Al-Jabouri, 2005). Another characteristic of the study area is the presence of strike valleys, whose extension is parallel to the strike of the folds (northwest-southeast) and are characterized by their relative decline resulting from the erosion of weak layers in the formation of the opening. There are also transverse valleys, i.e., perpendicular to the strike of the fold.

Materials and Methods

As seen below, the work techniques comprised many stages, ranging from data collection to the outcomes stage.

Data Collecting:

This stage includes collecting information about the study area by reviewing and reading many reports, theses, and research to evaluate the slopes related to the study area and benefit from them during field work. Maps and pictures related to the study area are also collected to obtain information about the geology of the area.

Fieldwork:

This stage lasted three days (July 29-31, 2024). First, the study area was surveyed (4 stations), where failures could occur, identified, as well as studying the structure, stratigraphy, and geomorphology of the area. Second, the necessary information was collected for each station, including the coordinates and attitude of slopes, layers, and joints, as follows:

1- The coordinates of each station were determined using GPS equipment and UTM units.
2- The attitude of slopes, layers, and joints was determined using a geological compass.
3- The width and height of the slope were measured.
4- The thickness of the layer was measured at each station.
5- The conditions of discontinuities (dip direction and dip amount), spacing, frequency, openings, and filling materials of discontinuities were studied in detail.
6- Measuring the width of the street, the distance between the street and the slope, the number of cars passing per hour, and their speed.

7- Identifying the types of existing and probable failures, as well as the types of slopes.
8- Collecting rock samples representing each station to conduct laboratory tests.
9- Taking photographs to document the sampling process.

Laboratory Work:

This stage includes preparing and setting up the samples of the four stations to test the uniaxial compressive strength according to Brook (1985) and ISRM (1985). This is achieved by point load testing, one of the most important and widely used tests due to its ease of implementation and the ease of preparing and setting up the sample. The sample is equipped with dimensions (D, W, L), where (D) represents the thickness of the sample between the two tips of the device, (W) represents the width of the sample, and (L) represents the length. The distance from the top of the sample to the point of the device must be less than half the height (L < 0.5 D). Afterward, the sample is loaded in compression until it reaches failure, and the uniaxial compressive strength (σc) is calculated (Table 1) to represent the results of this test for the studied stations and the type of rocks according to Anon (1977).

Table 1. Test results of studied stations and the type of rocks according to Anon (1977).

Station No.	Type of rock	D(mm)	L(mm)	W(mm)	F(KN)	$A(mm)^2=D\times W$	$De=\sqrt{\frac{4A}{\pi}}mm$	$De^2=\frac{4A}{\pi}mm^2$	$Is=F/De^2$	$F=(\frac{De}{50})^{0.45}$	$Is_{(50)}=Is\times f$ (Mpa)	$\sigma c=21\times Is_{(50)}$ (Mpa)	Classification of rock according to Anon,1977
1	Gypsum	70	50	90	4.18	6300	89.58	8025.4	0.520	1.300	0.676	14.2	moderately strong
2	Gypsum	60	40	70	3.62	4200	73.14	5350.3	0.676	1.293	0.874	18.35	moderately strong
3	Gypsum	70	50	100	3.54	7000	94.43	8917.19	0.396	1.331	0.527	11.1	Moderately weak
4	Gypsum	80	60	90	2.76	7200	95.77	9171.9	0.300	1.339	0.401	8.421	Moderately weak

Office Work:

This stage includes the presentation of the data obtained in the field and the results of the laboratory tests, as follows: The coordinates of each station in the study area were projected onto a location map using a program (GIS). The program (SMRTool-v205) was also used to determine the degree of stability of each landslide type. The program (Dips v6.008) was also used to determine the probable failure. This study focuses on the assessment of the stability of rock slopes through dynamic analysis, the Slope Mass Rating system (SMR), and the continuous slope Mass Rating (CSMR). Kinematic analysis is the simplest analysis of a failure, given the data such as the attitude of joint sets, layer attitude, slope, and internal friction angle measured by the field slope method. (Bruce et al., 1989) These collected data were presented using the program DIPS v6.008. Kinematic analysis is a simple method to determine the possible failure modes (plane, wedge, and toppling) and failure directions in a rock mass based on the angular relationship between the discontinuity and the slope surface (Hoek and Bray, 1981). The Slope Mass Rating (SMR) proposed by Romana is a system for determining the stability of rock slopes. The method is based on the rock mass classification (RMR) system introduced by Bieniawski in 1989. The RMR system is based on field and office studies and includes the unconfined compressive strength (UCS) of the intact rock, the rock quality designation (RQD), the discontinuity spacing, the discontinuity condition, and the groundwater conditions. The classifications are then added to obtain the RMRb value (basic RMR). RMR plays an essential role in calculating SMR and CSMR. The average spacing between all discontinuities was calculated from the inverse relationship with the average frequency of all discontinuities (Bieniawski, 2011). Remote sensing analysis and geographical information systems GIS are used in many studies of rock mass sliding by identifying lineaments that mostly represent discontinuity density in the rock masses (Bety et al., 2022). The rock quality designation (RQD) was calculated according to Palmstrom (2005) using the number of volumetric discontinuities (Jv) (the number of discontinuities per unit volume), and thus the RQD is equal to 110-2.5Jv. (Table 2) shows the different stability classes and experimentally discovered limit values of SMR associated with different failure modes proposed by Romana (1985). Thomas (2007) also developed a continuous SMR (CSMR), which is a modification of Romana's discrete SMR technique. CSMR provides a unique value for each adjustment factor, unlike the range in discrete SMR. SMR is calculated using RMRb with some adjustment factors proposed by Romana (1985) as shown in the following equation:

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

Table 2: Description of slope mass rating (SMR) (Romana, M. 1985)

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

Results and Discussion

This study includes an engineering geological survey of four stations represented by rock slopes in the Khanuqa anticline within the Fatha Formation. The attitude of these stations in terms of slope attitude, layer, joints, and friction angle is shown in Table 3, which consists of sequences of clay layers followed by a gypsum layer, then a marl layer, then a limestone layer, and then a clay layer. When the clay layer is exposed to weathering, it makes the gypsum layers overhang, which leads to the possibility of toppling when the center of gravity exceeds the support base, when the cohesion across the discontinuities is zero (Al-Jawadi, 2021). At the same time, the resulting failure is of the rockfall type due to the presence of vertical and overhanging slopes, where the rock masses fall due to their weight when the cohesion becomes zero across the discontinuities. The results of the kinetic analysis using the DIPS-v6.008 program showed that the failure probable to occur at the four stations is of the (wedge-sliding) type due to the intersection point of the joint sets J1andJ2 occurring in the area of potential sliding, and its direction for the four stations is as follows, respectively (153, 158, 147, and 128), and there is no possibility of plane sliding, flexural toppling, or direct toppling as in (Figures, 2, 3, 4, and 5), and the letters (a, b, c and d) represent plane sliding, wedge sliding, flexural toppling, and direct toppling, respectively, and SF represents the slope face, So represents the bedding plane, J1 the first set of joints, and J2 the second set of joints.

Table 3: Attitude of slope face, bedding planes, and joints in the stations of road-cut slopes and friction angle.

Station no. (Slope site)	**Slope Dip direction / Dip angle (Average)**	**Bedding plane Dip direction/ Dip angle (Average)**	**Join set (J1) Dip direction/ Dip angle (Average)**	**Joint set (J2) Dip direction/ Dip angle (Average)**	**friction Angle (ϕ)**
1	320/90-OH	070/06	253/87	341/75	39
2	332/90-OH	230/11	01/63	285/70	32
3	335/90-OH	219/06	020/67	297/59	43
4	284/90-OH	195/03	258/70	312/60	43

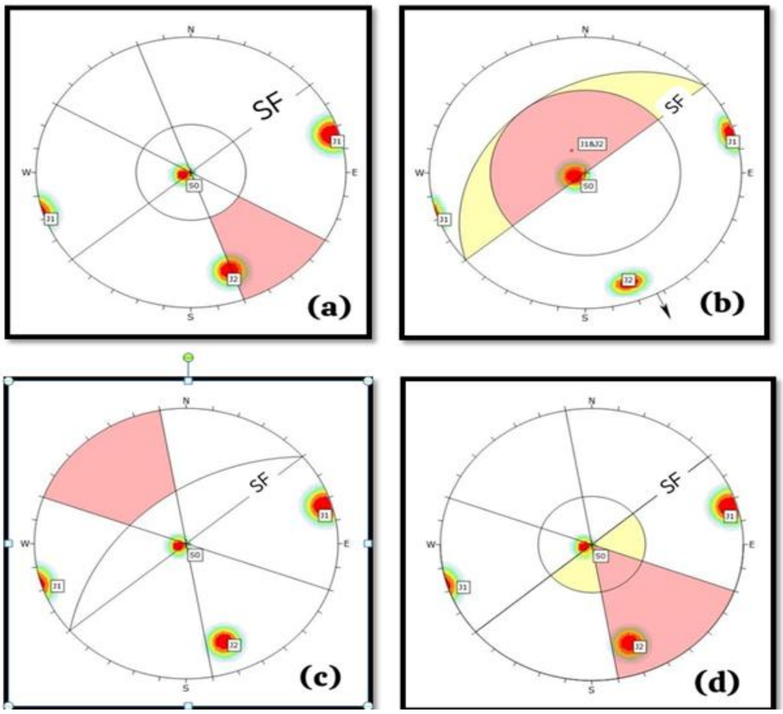

Fig. 2. Results of kinematic analysis of station No. (1), (a) no plane sliding, (b) wedge sliding towards 153, (c) no flexural toppling, (d) no direct toppling.

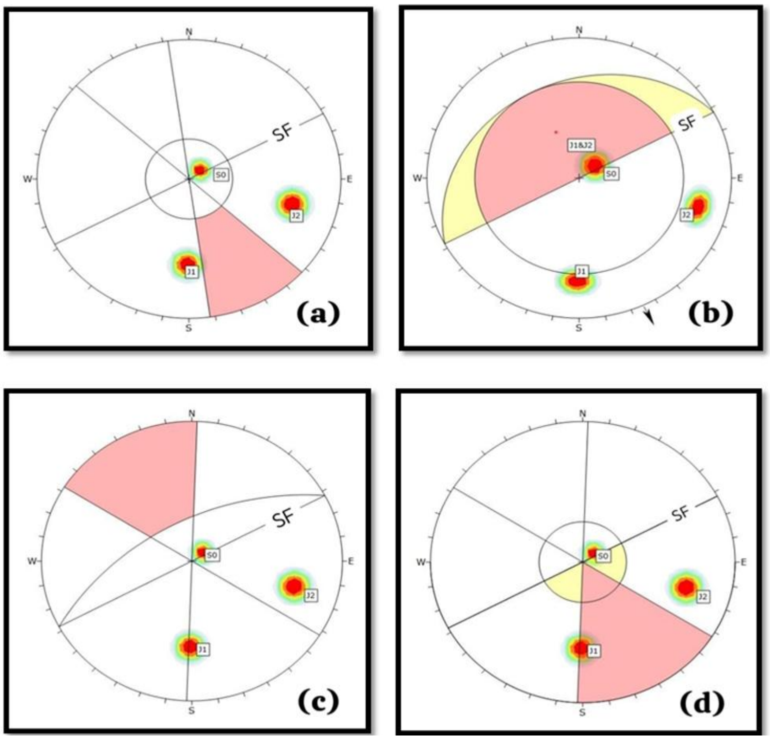

Fig. 3. Results of kinematic analysis of station No. (2), (a) no plane sliding, (b) wedge sliding in the direction of 158, (c) no flexural toppling, (d) no direct toppling.

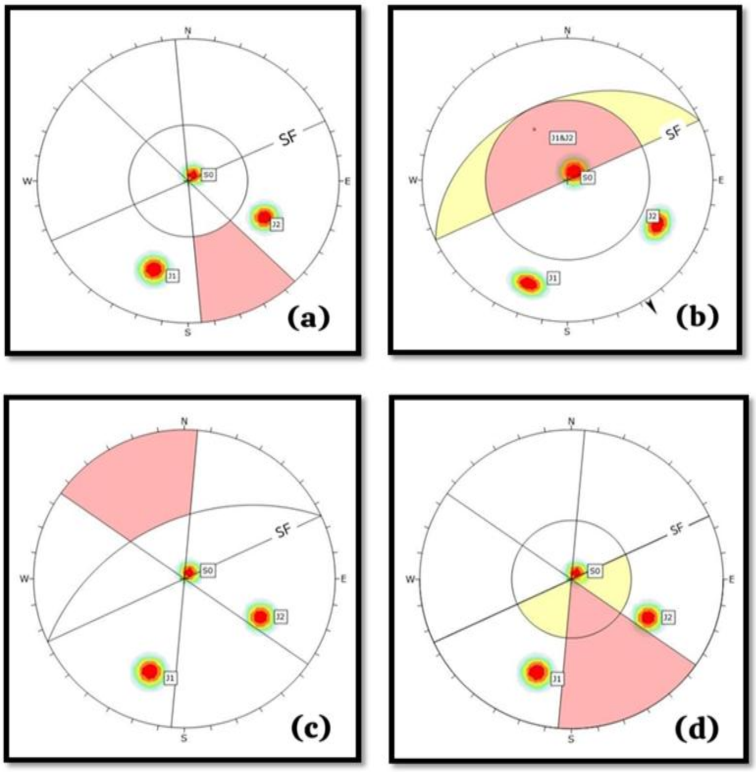

Fig. 4. Results of kinematic analysis of station No. (3), (a) no plane sliding, (b) wedge sliding towards 147, (c) no flexural toppling, (d) no direct toppling.

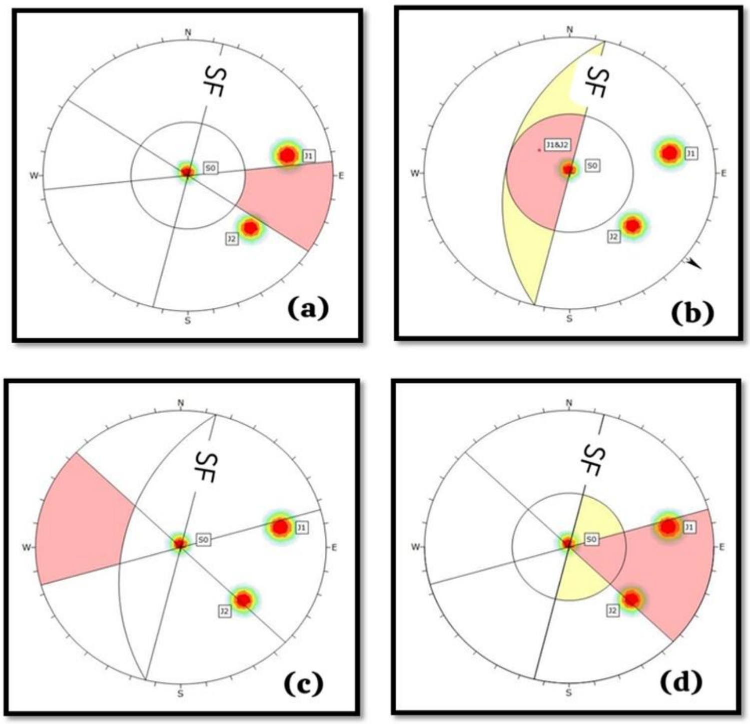

Fig. 5. Results of kinematic analysis of station No. (4), (a) no plane sliding, (b) wedge sliding in the direction of 128, (c) no flexural toppling, (d) no direct toppling.

The results of the SMRTool-v205 program identified one type of failure using the kinematic analysis of structurally controlled rock masses, which is wedge sliding, and its results were used in the slope classification system. The RMRb was calculated according to (Bieniawski, 1989). The unconfined compressive strength UCS was obtained indirectly from the point load test, which was performed according to (After Brook, 1985; ISRM, 1985), where the UCS value ranges from 8.421 MPa to 18.35 MPa, i.e., moderately weak -

moderately strong, as shown in Table 1. The rock quality designation RQD was obtained from the relationship between RQD and the number of volumetric joints (Jv) (RQD=110-2.5 Jv), Palmstrom, 2005 (Table 4), which has a value of 99.70, and the average spacing between all discontinuities was obtained from the inverse of the average frequency of all discontinuities (Bieniawski, 2011), which ranges from 697.6mm to 736.7mm, as shown in (Table. 5).

Table 4: Volumetric joint count (Jv), rock quality designation (RQD), and average spacing of all discontinuities measurements from joint sets observed in the Gypsum rocks at station no. (1).

Discontinuities Bedding plane and Joints	Set spacing and frequency				Average spacing(m)	Average frequency*
	Spacing (m) Min	Spacing (m) Max	Max. frequency	Min. frequency		
(So)	0.3	1.3	3.33	0.769	0.8	1.25
(J1)	0.4	0.9	2.5	1.11	0.65	1.538
(J2)	0.5	1	2	1	0.75	1.33
random joints**						
Jv=Frequencies (joints/m3)						4.118
RQD = 110 – 2.5 Jv		99.70				
Average frequency of all discontinuities = Jv / 3						1.3726
Average spacing of all discontinuities (m) = (1 / average frequency) = 3 / Jv					0.7285m=728.5mm	

*Average frequency=1/Average spacing Palmstrom, A., 2005
**For random joints, a spacing of 5m for each random joint is used in the Jv calculation.
-RQD = 110 -2.5 Jv Palmstrom, A., 2005
-Average frequency of all discontinuities=Jv/3 Bieniawski, Z.T., 2011
-Average spacing of all discontinuities (m)=1/average frequency Bieniawski, Z.T., 2011
Were So: Bedding plane, J1: Joint set 1, J2: Joint set 2, Jv: Volumetric joint count, RQD: Rock Quality Designation.

Finally, by Rock mass characterization in the rock slopes of stations 1, 2, 3 and 4 (Table. 6) the RMRb (1989) values of the rock mass were determined at all slope stations, RMRb (1989) is basic rock mass Rating, without adjustment factor for discontinuity direction, (Table. 7), which shows that the RMRb value at all stations is equal to (61). This data is represented in the SMRTool-v205 program, (Fig. 6), where the SMRTool-v205 program showed that the discrete SMR and continuous SMR values are at their worst (i.e. the lowest SMR value) for the wedge slide at stations (1, 2, and 3) where they range from (1-19), which indicates that the rock mass is within the fifth category (V), i.e. it has very low stability and its condition is very unstable. As for the fourth station, the SMR value is (37), which indicates that the rock mass is within the fourth category (IV), i.e. it has low stability and its condition is unstable (Table 8) and (Table 9).

Table 5: Rock quality designation RQD, volumetric joint (Jv), and average spacing of all discontinuities measurements from joint sets observed in study stations.

Station no.	Jv (Joints/m^3)	RQD	Average spicing of all discontinuities (mm)
1	4.118	99.70≈100	728.5
2	4.235	99.41≈99	708.38
3	4.30	99.25≈99	697.6
4	4.072	99.82≈100	736.7

Table 6: Rock mass characterization in the rock slopes of study stations.

					Remarks
Stability station	1	2	3	4	
Elevation (a.s.l) (m)	179	194	210	212	
Rock type	Gypsum	Gypsum	Gypsum	Gypsum	From Field
Strength of intact rock material UCS (50) (MPa)	14.196	18.35	11.067	8.421	
RQD (%)	100	99	99	100	
Average spacing (mm)	728.5	708.38	697.6	736.7	
Surface condition of discontinuities	moderately weathered, Rough moderately, fine filling < 5mm, Aperture:5cm, persistence:>20m	moderately weathered, Rough moderately, fine filling < 5mm, Aperture:5cm, persistence:>20m	moderately weathered, Rough moderately, fine filling < 5mm, Aperture:5cm,persistence>20m	moderately weathered, Rough moderately, fine filling < 5mm,Aperture:6cm, persistence>20m	From field description
Ground water	Dry	Dry	Dry	Dry	From field

condition	(Summer)	(Summer)	(Summer)	(Summer)	description

Table 7: Rating values of RMRb (1989) and RMR- parameters for the rock masses in the rock slopes of study stations.

Slope station		**1**	**2**	**3**	**4**
Elevation above sea level (m)		179	194	210	212
Rating of parameters	**Strength of intact rock (UCS)**	2	2	2	2
	RQD	20	20	20	20
	The average spacing of all discontinuities	15	15	15	15
	Condition of discontinuities	9	9	9	9
	Ground water condition	15	15	15	15
RMRb (1989)		61	61	61	61

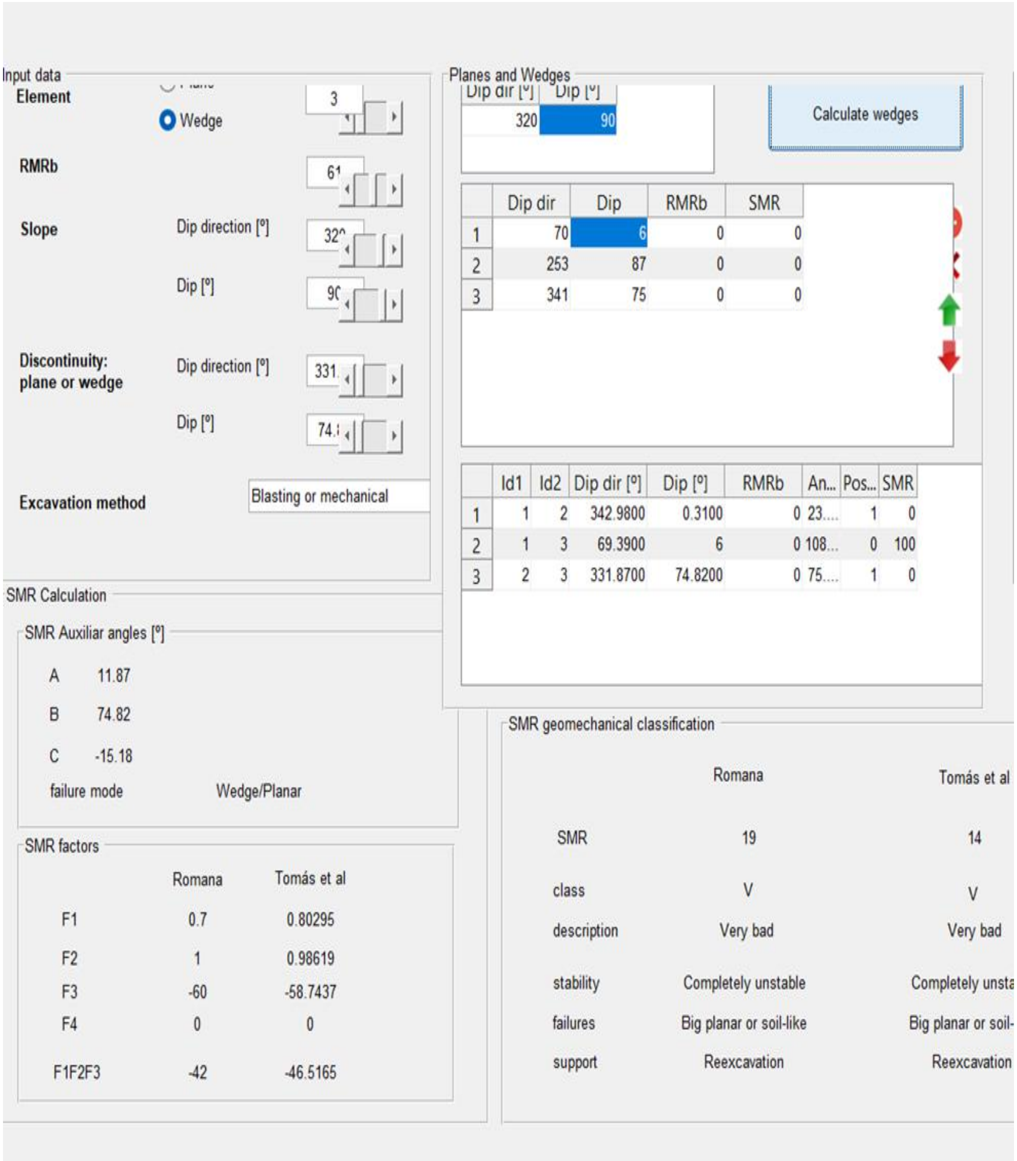

Fig. 6. Evaluation of the stability of the rock slopes of station No. (1) showing wedge sliding using the SMRTool-v205 program.

Table 8: SMR separate slope mass classification results using SMRTool-v205.

Station No.	**Attitude of slope**	**Value RMRb**	**Type of Failure**	**Direct of Failure**	**F1**	**F2**	**F3**	**F4**	**F1.F2.F3**	**value SMR**	**Classification of stability according to Romana, M. 1985**
1	320/90	61	b)WS	153	0.7	1	-60	0	-42	19	V/Unsta.
2	332/90	61	b)WS	158	1	1	-60	0	-60	1	V/Unsta.
3	335/90	61	b)WS	147	0.85	1	-60	0	-51	10	V/Unsta.
4	284/90	61	b)WS	128	0.4	1	-60	0	-24	37	IV/Unsta.

Table 9: Continuous (CSMR) value results using (SMR Tool).

Where: WS: represents wedge sliding; (F1, F1, F2, F3 and and F4): represents

Station No.	Attitude of slope	Value RMRb	Type of Failure	Direct of Failure	F1	F2	F3	F4	F1. F2. F3	value SMR	Classification of stability according to Romana, M. 1985
1	320/90	61	b)WS	153	0.80295	0.98619	-58.7437	0	-46.5165	14	V/Co.Unsta.
2	332/90	61	b)WS	158	0.96631	0.96902	-59.3537	0	-55.5773	5	V/Co.Unsta.
3	335/90	61	b)WS	147	0.90119	0.95792	-59.4528	0	-51.3235	9	V/Co.Unsta.
4	284/90	61	b)WS	128	0.40846	0.96816	-59.3643	0	-23.476	37	IV/Unsta.

correction coefficients for SMR; Unsta.: means unstable; Co.Unsta.: means very unstable., the letter (b) represents failure from wedge sliding.

Conclusion

This study reached the following conclusions: the failure occurring in the field is a rock fall, while the expected one in the field is a direct toppling. The kinetic analysis using the DIPS-v6.008 program revealed that the failure that may occur is a wedge sliding, which is expected to occur at all stations (1, 2, 3, and 4). It also showed that in the worst conditions, the values of the discrete SMR and continuous CSMR range from (1-37) and (5-37), respectively. This indicates that the rock mass at stations (1, 2, and 3) falls within the fifth category (V), meaning that it has a very bad slope and its condition is completely unstable. As for station (4), the rock mass falls within the fourth category, meaning that it has a bad slope and its condition is unstable. When comparing the results of the discrete SMR and continuous SMR, it is noted that the slope stability classification at stations (1, 2, and 3) differs between the discrete and continuous SMR, as the results of the discrete SMR show that these stations are unstable, while the results of the continuous SMR show that these stations are completely unstable, i.e. the CSMR model has given a more accurate assessment than the discrete SMR model in terms of slope stability degrees in terms of quantitative numbers. By comparing the SMR results with the slope mass classification description table, we find that the probability of failure at stations (1, 2, and 3) is (0.9), while at the station (4) the probability is (0.6).

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