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Assessing of Engineering Geological Properties for the Rock Mass on Road Cut Slopes within Taurus Terrain, Turkey

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ABSTRACT

The majority of road networks and infrastructures in Turkey pass through high terrain like the Taurus Mountains. Therefore, road opening or even widening in these terrains usually needs to be blasted, so a good understanding of geoengineering conditions should be maintained. This study aims to investigate and evaluate the behavior and failure of rock masses of cut slopes along the Bozkir-Hadim Road, Konya region in the south of Turkey, through integrating experimental and empirical parameters using RocLab software. For this study, a field survey with a scan-line manner suggested by ISRM was conducted on rock mass properties in terms of joint properties, rock quality designation (RQD), and grade of weathering. Furthermore, the geological strength index (GSI) and rock mass rating (RMR) indexes are quantified and attached to the characterization of rock mass for each cut slope. Laboratory tests, including unit weight, uniaxial compressive strength, and shear strength, are performed to identify the engineering geological characteristics of intact rocks. Microscopic analyses are also done to detect the weathering effect on the rock structure. Field and laboratory results are employed as input data into the RocLab software for rock mass analysis. Research investigation results show that rock masses are often characterized as highly jointed, intersected by three to four dominant joint sets with various weathering grades from low to high. As a result, a comparison of the results shows that the experimental findings of analyses with RocLab software match the site conditions of the rock mass. Consequently, this software can be used to characterize rock mass and predict any further failure that could occur in the future in massive rocks for road cut slopes, and thus preserve their long-term stability.

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تقييم الخصائص الجيولوجية الهندسية لكتل الصخور على منحدرات قطع الطرق ضمن تضاريس طوروس، تركيا

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الملخص

تمر غالبية شبكات الطرق والبنية التحتية في تركيا عبر تضاريس عالية مثل جبال طوروس. ولذلك، فإن فتح الطرق أو حتى توسيعها في هذه التضاريس عادة ما يحتاج إلى تفجير وفهم جيد للظروف الهندسية الجيولوجية. يهدف هذا البحث إلى دراسة وتقييم سلوك وهبوط الكتل الصخرية للمنحدرات المقطوعة على طول طريق بوزكير-هاديم في محافظة قونيا جنوبي تركيا، من خلال دمج المعطيات التجريبية والتطبيقية باستخدام برنامج (RocLab.). في هذه الدراسة، تم إجراء مسح ميداني بطريقة خط المسح المقترحة من قبل الجمعية الدولية لميكانيك الصخور (ISRM)على خصائص الكتلة الصخرية من حيث خصائص الفواصل وتصنيف جودة الصخور (RQD) ودرجة التجوية. علاوة على ذلك، يتم قياس مؤشرات القوة الجيولوجية (GSI) وتصنيف كتلة الصخور (RMR) وإرفاقها بتوصيف الكتلة الصخرية لكل منحدر قطع. تم إجراء الاختبارات المعملية بما في ذلك وزن الوحدة، وقوة الضغط أحادية المحور، وقوة القص لتحديد الخصائص الجيولوجية الهندسية للصخور السليمة. كما تم إجراء التحليلات المجهرية للكشف عن تأثير التجوية على البنية الصخرية. تم استخدام النتائج الميدانية والمختبرية كبيانات مدخلة في برنامج RocLab لتحليل الكتلة الصخرية. أظهرت نتائج البحث أن الكتل الصخرية غالبًا ما تتميز بأنها شديدة الفواصل، وتتقاطع مع ثلاث إلى أربع مجموعات فواصل سائدة، وتتنوع في درجة التجوية من الأقل إلى الأعلى ونتيجة لذلك، أظهرت مقارنة النتائج أن النتائج التجريبية للتحليلات باستخدام برنامج RocLab تتطابق مع ظروف موقع الكتلة الصخرية. وبالتالي، يمكن استخدام هذا البرنامج لتوصيف الكتلة الصخرية والتنبؤ بأي فشل آخر يمكن أن يحدث في المستقبل في الصخور الضخمة لمنحدرات قطع الطرق وبالتالي الحفاظ على استقرارها على المدى الطويل.

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Introduction

Engineering geological investigations of rock masses for infrastructures are essential to provide input data not only for design but also to supply helpful documentation for post-implementation (Behnia et al., 2020).

In nature, except for the rare case of a completely unfractured rock unit, the majority of rock masses can be considered assemblages of intact rock blocks delineated in three dimensions by a system of discontinuities. These discontinuities can occur as unique, randomly oriented features or as repeating members of a discontinuity set (Hoek et al., 2002;

Bienaiwski, 1989). These joint sets can significantly impact the stability, rock slope failures, and geo-engineering structures of a project (Zhang, 2019).

From an engineering point of view, a perception of the type and intensity of the rock fractures may be more important than the type of rock itself (Terzaghi, 1946). Because of this, several studies (Mulatu et al., 2010; Jena et al., 2020) have been carried out over the last decade or have shown that the criteria for intact rock can be used to predict the onset of brittle fracture at massive failure in brittle rock excavations based on laboratory and field observations.

In general, predicting rock mass behavior is usually difficult to analyze and needs an exhaustive investigation, but it can be conceptualized using the application of numerical analysis software (Zhang, 2019). One of the major obstacles, which are encountered in the field of numerical modeling for rock mechanics is the lack of data input for rock mass properties. In other words, the advantage of elaborate relevant models and powerful numerical analysis software is still greatly limited, unless the analyst has enough reliable input data for rock mass properties (Hoek and Diederichs, 2006). The process of deterioration results in a progressive degradation of the rock mass strength, which increases the differential condition of instability.

In Turkey, many cities have experienced rapid population growth, which has increased the need for engineering infrastructures such as construction, additional railways, and the widening of existing road networks, which have stimulated geological studies in this region.

Unfortunately, most of the roads in Turkey pass through the Taurus terrain. In such regions, due to tectonic stress of different directions and intensity in rocks and complexity in geomorphology, infrastructure construction, and road excavations are often carried out through blasting processes with no regard to other geological and structural considerations.

In regard to characterization of rock mass, particularly for slope instability analysis, many studies (Sonmez et al., 1998; Aksoy, 2008; Mohamed and Bayram,2020; Al-Talib et al., 2021; Al-Jawadi, 2024) have been conducted over and out the study area.

This research aims to characterize rock masses and stabilities in rock-cut slopes on a road, and then to estimate the engineering geological properties of the rock mass based on the laboratory and field investigations.

In this research, to enable the use of RocLab software, which requires material properties for rock mass stability assessment, many laboratory and geotechnical tests on intact rocks were undertaken. Finally, both field and laboratory parameters are evaluated, and then used as input data into RocLab software to conceptualize properties of rock mass along the Bozkir-Hadim Road, Konya region in the south of Turkey.

Geological setting of the study area

To characterize and assess the rock masses for road-cut slopes, a detailed field survey and laboratory tests have been carried out in parts of the Taurus terrains along the Bozkir-Hadim Road, Konya region in the south of Turkey (Fig.1). This road is important; it's contributing to the main trade, transport, and tourism routes over many cities. It also provides a better connection to the residential area and allows the connection of agricultural centers and primary residential areas.

In general, the investigated road is characterized by narrow valleys with high relief (1000 m to 1400 m), particularly in the southern part of the road. The annual average rainfall in the road region is between 648 and 732 mm (MGM, 2019).

Geologically, the investigated cut slopes along the road segment and around reflect diverse environmental conditions of sediments; so geologically, this road was problematic. In the Central Taurus region, the areas exhibit exposed units with different age ranges and lithologies together with their tectonostratigraphic relationships. According to their current structural positions, Özgül (1997) gave from bottom to top, the Upper Triassic- Korualan Group (Kayabaşı Formation and Başkışla Mixture), the Upper Triassic-Upper Cretaceous Huğlu Group (Dedemli Formation, Mahmut Hill Limestone, Kovanlık Mixture), and the Jurassic-Cretaceous Boyalı Group. It is divided into units in the Tepe Group (Soğucak Limestone/Kuztepe Limestone, Asar Tepe Limestone). Although the dominant lithology in the Mahmut Hill Limestone unit is limestone, it seems more appropriate to call it the Mahmut Hill Formation because it contains shale and siltstone intercalations (Turan, 2010; Takçi, 2015). The Bozkir unit mainly contains diverse rocks ranging in age from Triassic to late Cretaceous (Turan, 1990; Özgül, 1997; Öztürk et al., 2008).

The unit is characterized by various fabric changes in terms of composition, lithology, facies features, and alternations, so in several references, it was described as “Bozkir ophiolitic melange” containing limestone with blocks of different ages, shales, radiolarite, and pebbles of ophiolite (Öztürk et al., 2008; Turan, 2019). The Bozkir unit observed at cut slopes (S4, S5) comprises two formations, Boyalıtepe and Korualan, containing a highly deformed pelagic and neritic limestone. It varies in appearance and strength from one site to another and appears repeatedly in some parts. All geological features in the investigated area were mapped.

The Taurus Mountains within the study area is rather complex, well known for their very active tectonics, and widely affected by folding, uplifting-thrusting processes during the Early Cimmerian Orogenesis (Turan, 1990), which has generated the important NW-SE fault system and created a strictly diverse structural features such as joints and shear zones (Özgül, 1976). The formed structural planes, like joints and the widening of these joints at the top of the slope face climate effects that commonly occurred before the failure of slopes.

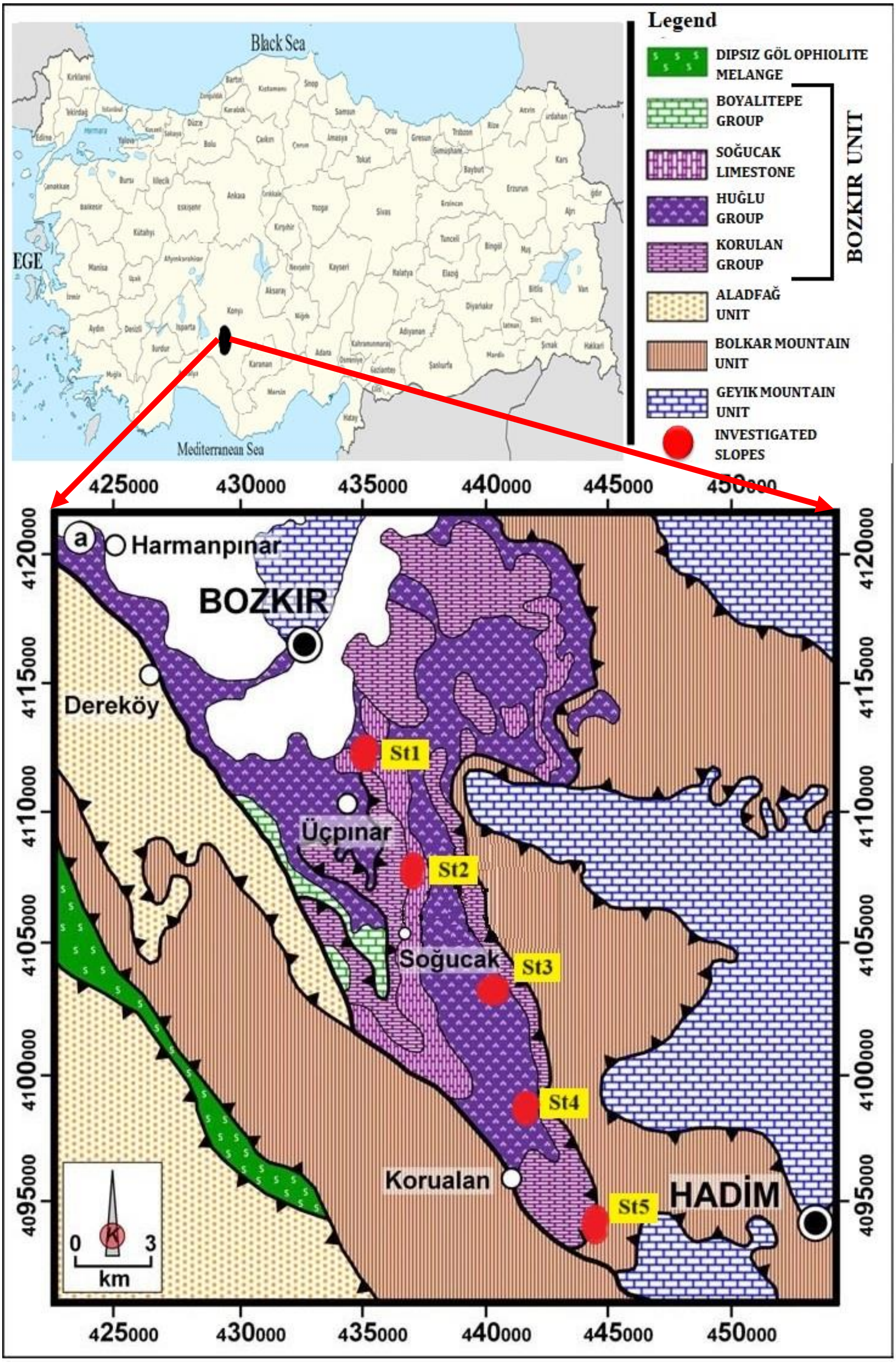

Fig. 1. Location and simplified geological map of the investigated road segment (MTA, 2002).

Methods and Materials

Methods

Field Investigation

After a reconnaissance study of the area, five potential cut slopes are selected along the Alanya roadway segment. A detailed field survey was carried out to characterize the engineering, geological, and geotechnical observations, in addition to the hydrogeological parameters regarding rock mass, as summarized in Table 1.

Table 1: Summary of the geotechnical field and laboratory testing conducted in the present work.

1	Dry density and Specific gravity test	laboratory	To investigate the dry and bulk density of the rocks	ASTM D792-20
2	joint roughness coefficient (JRC)	Field	Joint surface identification	Barton-type wire profilometer Barton and Choubey (1977).
3	compressive strength of joint (JCS)	Field	compressive strength of the joint wall	Schmidt hammer equipment
4	Direct shear strength of rock	laboratory	Shear Strength parameters determination, including cohesion and internal friction angle	ASTM D5607-16
5	Uniaxial Compressive Strength (σ_c) of Rocks	laboratory	Estimation of Uniaxial Compressive Strength of Intact Rocks, core specimen	ASTM D2938-95

Geotechnical laboratory testing

This stage is followed by performing many lab tests, including dry and bulk unit weight, uniaxial compressive strength test, and direct shear strength tests of the intact rock. All of these lab tests are carried out after the rock core specimens have been prepared according to the ASTM (D-4543-19) standard specifications mentioned in Table 1. The compression test is conducted according to ASTM (D2938-95) by applying an axial loading continuously until failure occurs. The data obtained from the field are integrated with the laboratory results and then used as input into the RocLab software to perform analysis for rock mass characterization. This study can be summarized with a flow chart outlined below in Figure 2):

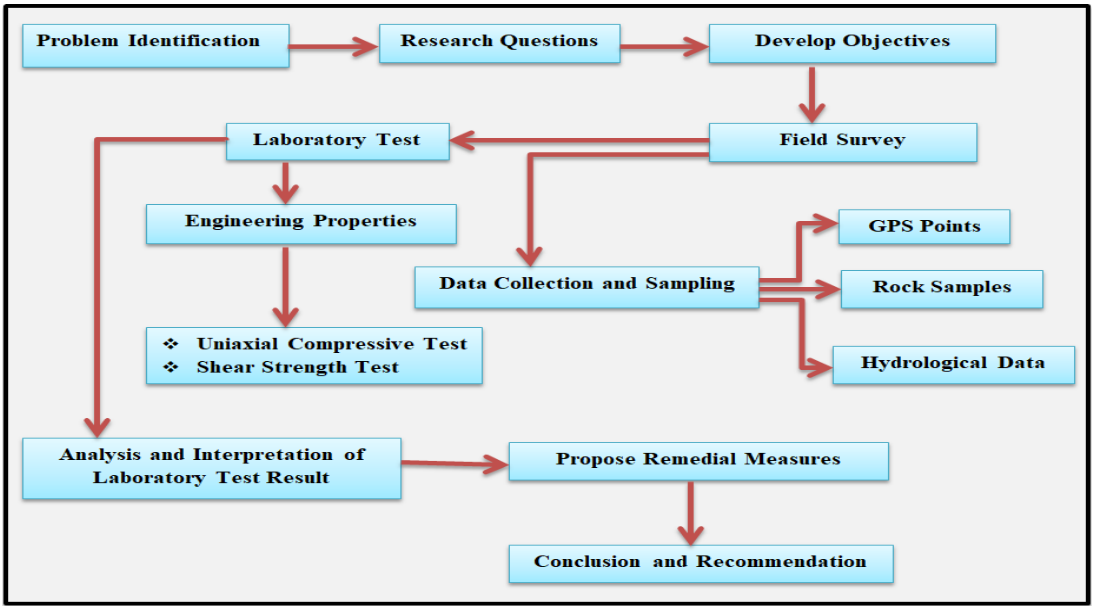

Fig. 2. Flow chart used for estimating rock mass properties in this study.

Materials

In this study, as mentioned above, many laboratory tests, including uniaxial compressive strength (σc), shear strength parameters (c, ϕ) of rock discontinuity, and unit weight, were performed on a representative rock sample, which has been prepared from blocks collected from 5 selected slope stations. The cores are prepared from samples with their required dimensions according to the methods suggested by ISRM (2007). Furthermore, for compression test and to obtain accurate results, the diameters and lengths of all core specimens are prepared in accordance with ASTM-D 4543-19 standards, then checked and measured (perpendicular to bedding) (L/D= 2.5). The measurements were taken in two directions perpendicular to each other with a caliper.

Results and Discussion:

Geological and Geotechnical features:

The engineering geological studies were conducted through both field and laboratory tests. Site investigations were carried out, encompassing field observation, discontinuity survey, and sampling. During the field survey, it was observed that the majority of rock masses at the slope under investigation are controlled by joints (structurally controlled failures). Therefore, discontinuity and joint set data 70 were taken with a scan-line survey manner, and engineering geological properties of rock mass were also characterized in terms of orientation, dip, and surface features of discontinuities and joint sets in a massive rock mass (Tables 1 and 2).

Shear Strength of Rock Masses Discontinuities:

In samples preparation for the direct shear strength test, a genuine discontinuity plane could not directly be obtained from the field, so a flat surface was prepared using a cutting machine in the laboratory by pouring of cut specimens with cement into a mold to be suitable within the test apparatus (Fig. 3). In this way, 18 test samples with a diameter of 54 mm have been prepared. A shear test was performed on these prepared samples. Finally, test results are plotted (Fig.4. a, b) to determine the parameters of the shear strength (cohesion, friction angle), which are inserted in the rock mass properties.

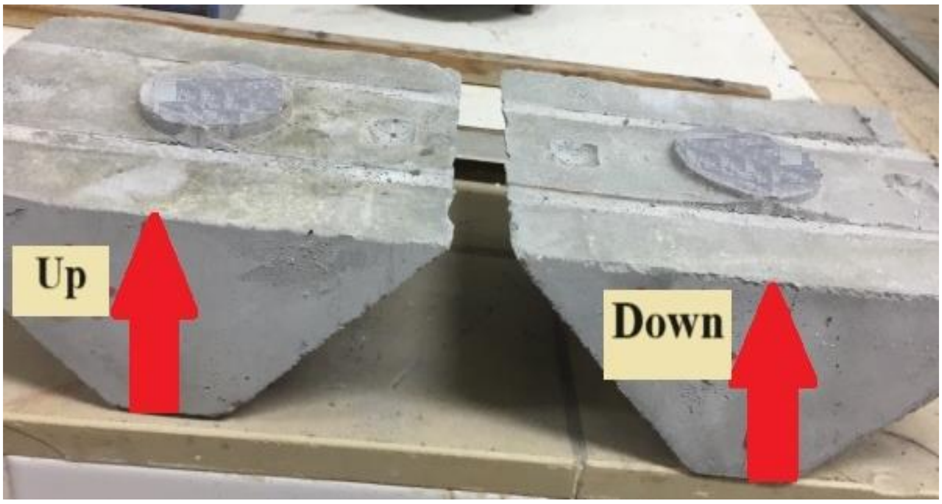

Fig. 3. Preparation of shear test specimen.

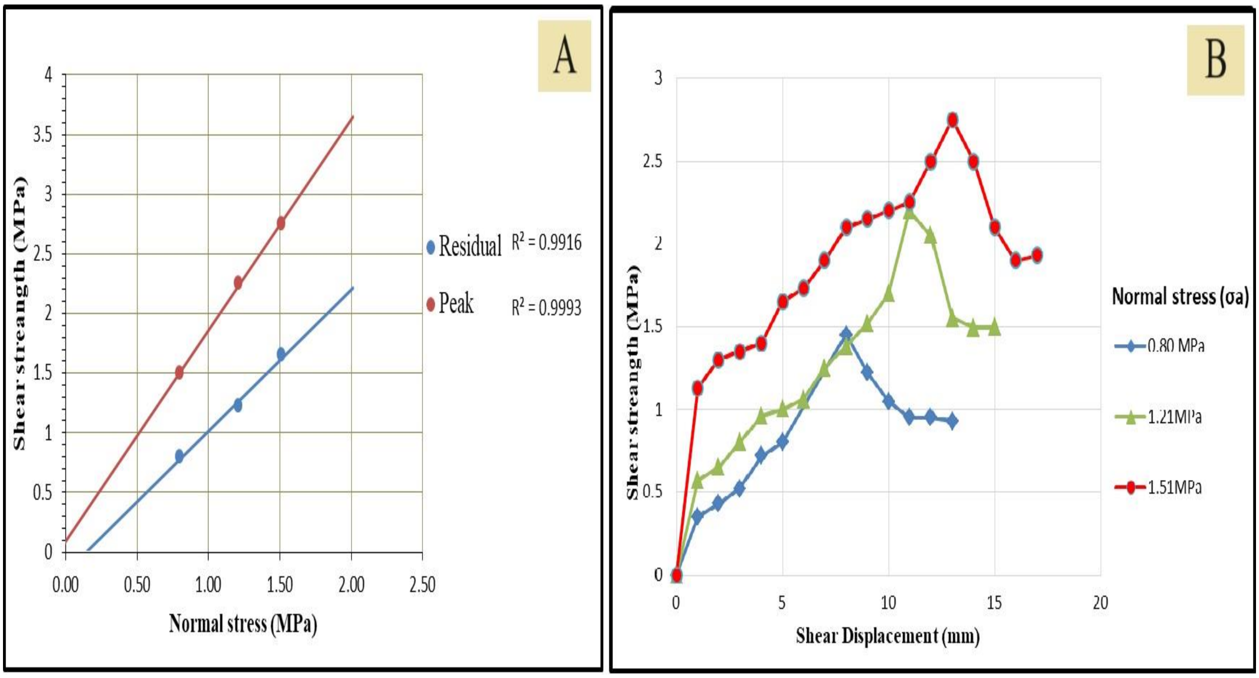

Fig. 4. a., b. Plots of peak and residual strengths of the shear vs. normal stress for a tested sample in the present study.

The mean values table (2) shows that the unit weight and uniaxial compressive strength of the limestone are close to each other, with an average of 2.66 g/cm^3 and 67.82 MPa, respectively.

The overall results of the geotechnical properties conducted during this study are summarized in Table 2.

Table 2: Summary of rock mass and intact rock properties for jointed rock units in the present study.

	Parameter/ Location		1	2	3	4	5
Field tests	JCS (MPa)		47.5	45.5	44.5	56	51
	Joint Roughness coefficient		9	14	12	11	13.75
	Rock Quality Designation		79.78	79.78	58.06	51.36	72.46
Laboratory Tests	Unit weight γt (g/cm^3)		2.64	2.76	2.61	2.66	2.65
	Avg.σ_c (Mpa)		69.59	50.68	66.63	44.42	67.24
	Cohesion (Mpa)	Peak (c_p)	0.848	1.210	1.263	1.441	1.981
		Peak (c_P)	0.545	0.765	0.621	1.336	1.659
	Friction Angle (deg.)	Peak (ϕ_P)	33	30	30	32	37
		Residual (ϕ_r)	29	28	28	30	35

* JRC: joint roughness coefficient; ** JCS: joint wall compressive strength.

Many measurements are carried out in the field to identify discontinuity properties in terms of bedding-joint orientation (dip/strike), spacing, joint conditions, groundwater condition, rock strength of rock material, and evaluation of rock quality designation (RQD%) (Table 2). The studied cut slope passes through limestone lithology that varies between massive to crushed limestone beds (Fig. 1). Limestone beds are moderately to highly jointed in nature (problematic), and vary in thickness as observed in the field. Limestone beds are characterized mostly by thick-well bedding, compact and massive, hard, generally grey to brown colored with aperture up to 10 mm in width, partly open to wide near surface, then it may be less at depth. The discontinuity parameters have also been evaluated; they are smooth to undulating rough. The histogram of discontinuity spacing plotted against cumulative frequency indicates that the spacing generally falls between 0.06 to 6m; then, according to

ISRM (2007), it can be classified as a very close spacing and has 3-10-m persistence and thus classified as medium-high persistence (Table 3).

Table 3: Prevailing joint and rock mass rating data measured on the investigated slope faces (after Bieniawski, 1989).

Station No.	Rock strength		Prevailing bedding plane (B) / Joint set (J) (Dip Direction /Dip amount)	Spacing (cm)	Persistence (m)	Roughness	Aperture (mm)	RQD Value Rating	Infilling (mm)	Weathering conditions	Groundwater conditions	*RMR-b
	Is(50)	USC rating										
St1	4.95	12	J1-100/60 J2- (150/58) B-046/71	Wide (6- 20) 8	Low-Medium (1-5) 3	Undulating Rough 3	Moderately wide (2.5-10) 5	79.78 17	RF/Soft clay 4	MW 5	Dry-damp 10	Good 67
St2	4.51	11	B- 270/80 J2- (055/51) J3- (344/52)	20-60 10	Medium-high (3-10) 2	R/P 3	0.5-2.5 4	79.78 17	None 6	MW 5	damp 10	Good 68
St3	4.45	10	J2- (338/85) J3- (115/60) B- 090/57	Close to Moderate (10-50) 15	1-7 3	R/P 3	Moderately wide (2.5-10) 5	58.06 13	Soft clay 2	MW 5	damp 10	Good 66
St4	5.06	13	B- 270/80 J2- (055/51) J3- (344/52)	60-200 15	1-8 4	R/P 3	2.5-10 6	51.36 13	RF 4	SW 5	damp 10	Good 73
St5	4.49	11	J2- (063/87) J3- (220/82) B- 210/45	60-200 14	3-10 2	S 1	0.5-2.5 2	72.46 13	None 6	HW 1	damp 10	Fair 60

* NV: near to vertical; R/S: Rough/smooth; R/P: Rough/planar; S: smooth; SL: Slickensided; RF: Rock fragments; HW: high weathering.

Furthermore, Rock Mass Rating (RMR_{basic}) (Bieniawski, 1973-1989), RQD, and GSI of the rock mass parameters are calculated for each slope**.** It is found that the estimated RMR_{basic} values are close to each other, ranging between 60 and 73 (Table 3), which corresponds to a "fair rock quality". RQD parameter is estimated by measuring spacing on rock mass exposure as shown in Figure 5, and then calculated using the common equation proposed by Palmström (2005).

$$RQD = 115 - 3.3Jv \quad ....(1)$$

RQD estimation is based on a new correlation between RQD and Jv, which is founded by Palmström (2005) by including computer-generated blocks of different sizes and shapes. The new correlation used in this research is given by the following expression:

$$RQD = 110 - 2.5\ Jv \ldots\ldots\ .... (2)$$

The estimated RQD values of the limestone in the current study range between 51.36 and 79.78 % (see Table 2), which corresponds to a moderate-high rock quality.

Also, it is noted that there is weathered debris accumulated on the slope face with replaced rock fragments that range in size from small to boulders, which moved from the slope face downwards under the gravity effect. These detached blocks are described following Varnes' classification as free rock falling (Varnes, 1984).

Fig. 5. Estimation of RQD.

Pelagic and neritic limestones belonging to the Bozkir units are exposed locally and are encountered at most of the investigated slopes. These outcrops, besides being fractured and jointed, are also weathered in different degrees, ranging from low, medium, to high, with no bedding planes locally. The majority of slopes consists of a thick succession of low-grade metasediments made up of very fine-grained crystallized limestone, extremely folded and faulted, hard-highly strength, moderately-highly weathered rocks interlayered with radiolarite and colored shale.

Site Rock Mass Characterization

With inadequate information available about the geo-structural features, an understanding of rock mass properties is difficult or impossible (Wyllie, 2017); therefore, in this research, to predict the engineering behavior of rock masses with reasonable accuracy, a careful examination of the rock masses is carried out through detailed field and laboratory measurements. Rock mass examination is performed in each road-cut slope by collecting discontinuity-slope orientation data and then by measuring the geometrical discontinuity properties listed in Tables 1, 2, and 3. During this examination, no water-availability signs were seen in all tested rock masses, so it is accepted as "humid-dry". All joint sets are continuous and persistent with smooth–planar to rough-undulating and rough-slickensides with slightly to highly weathered joint walls.

The joint sets exhibited a tight proximity ranging from fairly close to widely dispersed. The rock mass can be classified into fractured, blocky, or massive rock mass based on the observed spacing of discontinuities. The joint sets display a moderate to extremely high level of persistence. The prominent joint sets exhibit a near-vertical or steeply inclined orientation. The joint planes exhibit a combination of modest roughness and smoothness with localized, little clay infilling.

These clay infillings consist of rock fragments and sometimes weathered clay (Table 3). Accordingly, rock masses at the investigated road-cut slopes are highly jointed by three or four joint sets forming angular blocks (Table 4).

Table 4: Summarized data for rock mass characterization in the present study (DD = Dip direction, DA = Dip amount).

Station No.	Slope features				Joints features			Rock Mass Quality	
	Attitude DD /DA	H_{max} (m)	Slope dimensions (H/W)	Ground water at the slope	No. of Joints measured	Joint surface condition (SCR)	Structure Rating (SR)	GSI value	class
St1	065/70	13.5	0.41	Damp	20-26	7	50	40	Blocky/Disturbed: faulted
St2	300/56	20.2	1.64	Dry-	32	5	50	34	with angular blocks formed

				damp					by many intersecting joint sets
St3	090/57	13.8	2.17	Dry	35	4	37	26	
St4	280/58	22.7	2.19	Dry	28	7	33	30	
St5	087/63	22.6	1.41	Dry-damp	38	9	48	39	

Empirical Analysis of Rock Mass Characterization using RocLab Software

To investigate rock masses characterization and strength, field and laboratory results are used as input data into the RocLab software. As is well known, RocLab is a Rocscience Inc. software product commonly used for determining rock mass strength parameters based on the generalized Hoek-Brown failure criterion. (Rocscience, 2010). The advantages of RocLab, in addition to calculating rock mass parameters (tensile strength, uniaxial compressive strength, and deformation modulus) for the Generalized Hoek-Brown criterion, it always calculates equivalent Mohr-Coulomb parameters (cohesion and friction angle) for the rock mass.

RocLab also identifies the Generalized Hoek-Brown strength parameters of a rock mass (mb, s, and a) based on the following input data:

- Unconfined compressive strength of intact rock (σ_c)
- The intact rock parameter (m_i)
- The geological strength index (GSI)
- The disturbance factor D and
- The intact modulus (E_i)

Estimating Input Parameters

Each of the above 5 input parameters (σ_c, m_i, GSI, D, and E_i) is estimated according to the field conditions conveniently by RocLab software from its "Pick Dialogs" and through "Interactive Data Input" charts and tables based on rock type, geological conditions, etc. The σ_c is estimated from laboratory tests on intact rock samples. Based on the particular rock type and geological conditions, m_i values can also be estimated from a pick dialog. In the same way, the GSI is determined depending on joint surface condition (SCR) and structure rating (SR) values using conveniently designed charts and tables. The values of the D range between 0.7 and 1.0 according to the disturbance degree of the slope. Most of the rock mass slopes investigated in this study have been excavated with poor blasting, thus, referred to as a highly disturbed slope with a disturbance factor D = 1. Moreover, the intact modulus (Ei) is estimated based on rock type and texture, and it can also be estimated mathematically using the following equation:

$$Ei = MR * \sigma_c \quad \ldots\ldots\ldots\ldots(3)$$

All these parameters investigated during this study are presented in Table 5.

Table 5: Estimated empirical values of Hoek - Brown failure criterion used as input data into the RocLab software.

Station No.	Inputs							
	σc Mpa	GSI	mi (material constant)	D (Disturbance factor)	Ei = MR*σc (Mpa)	Unit weight of intact rock (MN/m3)	slope height (m)	Application
St1	69.59	39	12	0.7	37375	0.026	22.75	Slope
St2	50.68	34	17	0.7	14487	0.026	20	Slope
St3	66.63	26	12	0.7	34340	0.027	13.8	Slope
St4	44.42	30	10	0.7	32480	0.026	19.2	Slope
St5	67.24	39	12	0.7	34630	0.026	22.6	Slope

After the selection of the five input parameters, all output data will be immediately recalculated based on rock type, geological conditions, etc. (Table 6).

Table 6: Rock mass strength parameters and Hoek –Brown failure criteria values obtained with RocLab software.

Station No.	Output parameters: Hoek –Brown constants: mb	s	a	σ3max: $\sigma 3_{max}$ (MPa)	Mohr-Coulomb criteria parameters: Cohesion C (MPa)	Internal Friction angle Φ (degrees)	Rock Mass parameters (Mpa): Tensile Strength σt (MPa)	Compressive Strength of intact rock (σc) (Mpa)	Compressive Strength of rock mass (σ_{cm}) (Mpa)	Elasticity Modulus (Em) (Mpa)
St1	0.420	0.0001	0.512	0.525	0.221	47.74	- 0.026	0.808	6.08	2058.89
St2	0.452	0.0001	0.517	0.453	0.160	46.60	- 0. 008	0.375	4.328	618.87
St3	0.206	0.00022	0.529	0.317	0.098	43.53	- 0.007	0.235	3.483	1070.74
St4	0.214	0.00032	0.522	0.415	0.112	39.19	- 0.009	0.232	2.506	1168.09
St5	0.420	0.0001	0.512	0.518	0.212	47.25	- 0.024	0.748	5.639	1907.68

Prediction of Failure Envelope for Rock Masses

An ability to identify rock mass behavior specifically at a pre-failure stage can contribute to predicting or avoiding the potential for geotechnical failure and thus preventing a loss of life and property (Szwedzicki, 2003). Although many studies indicate that the failures on large-scale rock mass behavior do not happen randomly and unpredictably, in several cases, geotechnical precursors prove that when categorized as 'unanticipated' found to be predictable and could have been curbed, or at least the effects of failure could have been relieved (Zhang et al., 2019). In this research, a prediction of rock mass behavior with its stability is attempted graphically through a principal stress (major and minor) - based chart using RocLab software (Fig. 6). The chart is constructed based on obtained (output) results presented in Table 6, then evaluated for each investigated site by its parameters. The results of rock mass analysis show that most of the major stress failure envelopes lie below Mogi's line (which defines the ratio of major and minor effective main stresses where a transition occurs from brittle to ductile failure) (Fig. 6). Thus, it is inferred that "a ductile failure form" could happen for the investigated slopes. This may be due to low values of GSI ranging between 26 and 41.

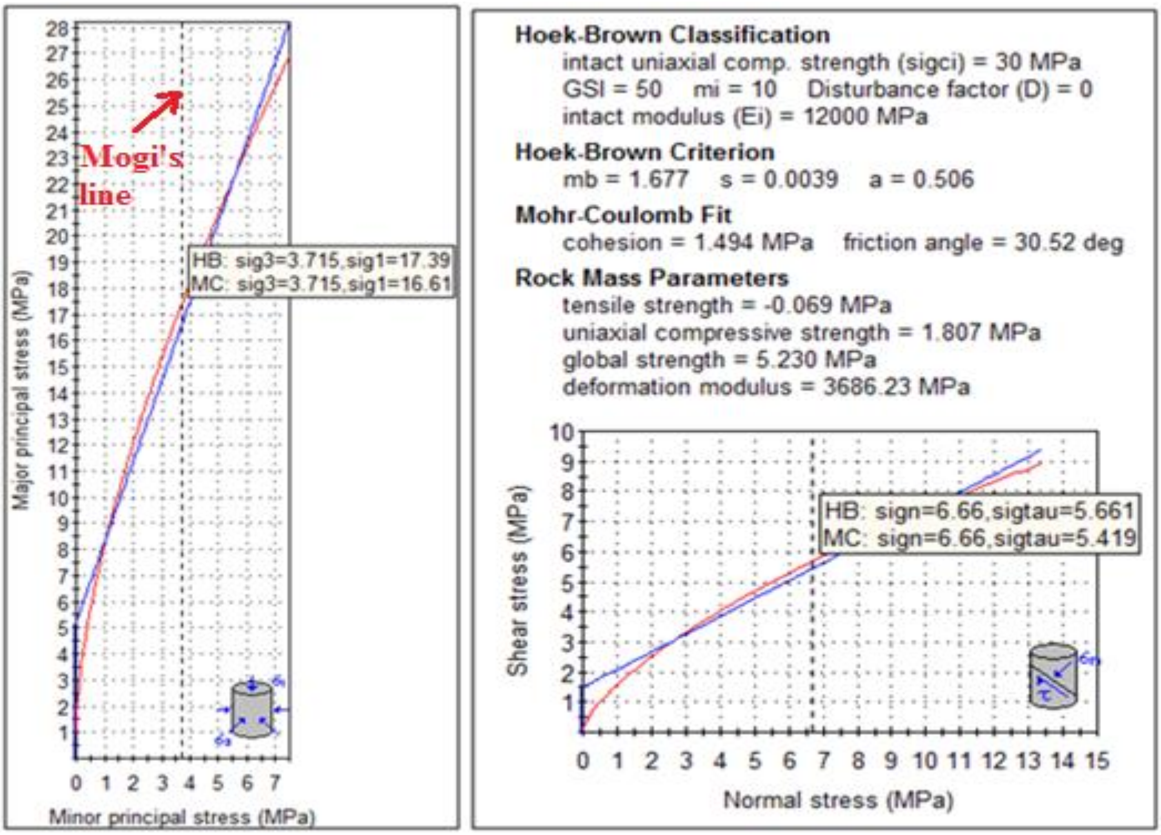

Fig. 6. Rock mass failure envelopes drawn based on the Generalized Hoek-Brown criteria.

Conclusion and Recommendations

In summary, this study focused on the integration of experimental (UCS, ϕ, etc.) and empirical (GSI, RMR) parameters for rock mass characterization along the Bozkir-Hadim Road (Turkey).

In this work, the engineering geological investigation-based rock mass characterization reveals that the rock masses are highly jointed by joint sets, which not only control the rock masses but also control structural failure of road-cut slopes, where the investigated slopes are currently partially stable, but could lose their strength with time under various external effects. Thus, further failures in the rock mass are likely to occur unless remedial measures are taken.

To obtain a substantial assessment of the properties of rock fracture and numerical analysis of rock fracture networks and to get a better understanding of jointed rock mass behavior, researchers, practitioners, and geologists should give more attention to the use of site manners. When detailed investigations are inaccessible or affordable, the preliminary analysis using RocLab software with RMR and GSI-limited data is more suitable.

Finally, when the tri-axial test cannot be conducted at the laboratory, major and minor stresses at any point of failure stages from Mogi's line using RocLab software can be estimated.

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Conflict of Interest

The authors declare that there are no conflicts of interest regarding the publication of this manuscript.

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