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Assessment of rock slope stability by probabilistic-based Slope Stability Probability Classification method along highway cut slopes in Adilcevaz-Bitlis (Turkey)

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Abstract: Rock slope stability is of great concern along highway routes as stability problems on cut slopes may cause fatal events as well as loss of property. In rock slope engineering, stability evaluations are commonly performed by means of analytical or numerical analyses, principally considering the factor of safety concept. As a matter of fact, the probabilistic assessment of slope stability is progressively getting popularity due to difficulties in assigning the most appropriate values to design parameters in analytical or numerical methods. Additionally, the effect of heterogeneities in rock masses and discontinuities on the analysis results is minimized through the probabilistic concept. In this study, slope stability of high and steep sedimentary rock cut slopes along a state highway in Adilcevaz-Bitlis (Turkey) was evaluated on the basis of probabilistic approach using the Slope Stability Probability Classification (SSPC) system. The probabilistic assessment indicates major slope stability problems because of discontinuity controlled and discontinuity orientation independent mass movements. Almost all studied cut slopes suffer from orientation-independent stability problems with very low stability probabilities. Additionally, the probability of planar and toppling failures is significantly high with respect to the SSPC system. The stability problems along the investigated rock slopes were also verified by field reconnaissance. Remedial measures such as slope re-design and reinforcement at the studied locations should be taken to prevent hazardous events along the highway. On the other hand, the probabilistic approach may be a useful tool during rock slope engineering to overcome numerous uncertainties when probabilistic and analytic results are compared.

Keywords: Slope stability; Probabilistic approach; SSPC; Rock; Discontinuity

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Introduction

Precision of slope stability analyses and safe slope design are strongly related to the accuracy of shear strength parameters assigned during analyses. However, in many cases, precise estimation of these parameters is challenging due to the heterogeneity of slope masses. On the other hand, in heavily jointed rock masses, slope failure occurs on multi-surfaces rather than a single plane and the failure surface is almost circular similarly in soils. Therefore, discontinuity properties and rock mass behavior should be jointly well-defined in highly jointed rock units.

Rock slope stability assessments are mostly performed on the basis of kinematical, analytical and numerical analyses. Additionally, several rock mass classification systems such as Rock Mass Rating (RMR) (Bieniawski 1989), Slope Mass Rating (SMR) (Romana et al. 2003), Chinese Slope Mass Rating (CSMR) (Chen 1995), Mining Rock Mass Rating (MRMR) (Laubscher 1990) can be applied to predict the stability of slope masses. Kinematical analyses should only be taken into consideration for the preliminary estimation of slope instabilities in rock masses as they do not involve geometrical solutions and are not specifically design-based. Limit equilibrium or numerical analyses should be performed in case of a potential discontinuity controlled stability problem revealed by kinematical analyses. Shear strength parameters of the discontinuity causing instability, slope geometry, groundwater condition and maximum horizontal ground acceleration are the major parameters considered during the limit equilibrium analyses (Karaman 2013; Kentli and Topal 2004; Gürocak et al. 2008).

Rock slope engineering involves numerous uncertainties due to heterogeneities in rock masses which should be well-defined for the precise estimation of slope stability (Ramly et al. 2002). For instance, the variation of shear strength of discontinuities in macro scale as a result of inconsistent surface roughness has great impact on the results of rock slope stability evaluations. On the other hand, discontinuity shear strength may be decreased by weathering in regional scale or probable changes in pore water pressure may lead to variations in slope stability estimations.

The factor of safety retrieved by deterministic approaches is the major concept in slope stability analyses. The ratio between the forces resisting slope movement and the forces driving instability is investigated during the factor of safety calculation. Although the threshold value for the factor of safety may be increased with respect to the importance of project or in case of a high risk of loss of property and life, a factor of safety value of slightly over 1.0 is generally accepted as stable in slope engineering. Nevertheless, the factor of safety may not be precisely defined due to regional variations of intact rock strength, discontinuity surface geometry and pore water pressure as well as insufficient laboratory testing and poor slope stability modeling (Park and West 2001). Therefore, the factor of safety value of a specific slope determined by various approaches may be inconsistent due to variable geotechnical parameters.

A gradual increase in the use of probabilistic approach in rock slope engineering to eliminate the problems in factor of safety concept is noticeable in recent decades (Hack 1998; Gökçeoğlu et al. 2000; Lindsay et al. 2000; Park and West 2001; Ramly et al. 2002; Hack et al. 2003; Park et al. 2005; Duzgun and Bhasin 2009; Irigaray et al. 2012; Li and Xu 2016). Instead of a factor of safety value, a percentage probability is assigned to the inspected slope in probabilistic stability analyses to indicate the probability of stability. Application of probabilistic approach to rock slope stability allows the consideration of uncertainties and variations in the geotechnical properties of rock masses. Analyses in this method are carried out for the random variables of rock mass and discontinuities. The input parameters of analyses are determined on the basis of statistical evaluations of geological and/or geotechnical data (Park et al. 2005). Whereas a number of researches on probabilistic approach make use of the deterministic method as a part of the study and express the stability in probabilistic manner, all data for the evaluation of slope stability are interpreted by statistical methods and the results are presented in percentage probability in other studies about probabilistic analyses.

In this study, slope stability probabilities of sedimentary rock cut slopes along Adilcevaz state highway (Bitlis-Turkey) were evaluated using the Slope Stability Probability Classification (SSPC) system firstly introduced by Hack (1998). Accordingly, scanline discontinuity surveys were carried out at 7 different cut slope locations to obtain discontinuity properties which are major input parameters for the SSPC system. In addition, intact rock mass strength was acquired by means of point load tests performed on irregular lump specimens. As a particular note, slope location 2, which is almost 30 m-high cut slope, was classified into two sections (upper and lower) as discontinuity properties are quite distinct at these zones.

1 Study Area

The studied highway is located along the northern shores of Lake Van adjacent to the town of Adilcevaz, which is one of the major districts of Bitlis province in Turkey. Slope stability assessments were performed for a total of 7 different cut slope locations along a 4 km-long section of the highway with an elevation of almost 1700 m. The location map of the study area also depicting the investigated cut slope locations is shown in Figure 1.

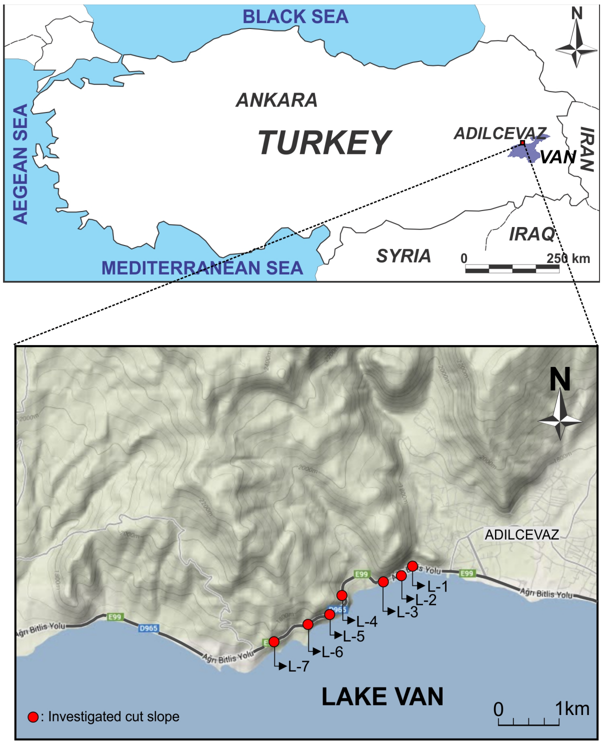

Figure 1 Location map of the study area.

This sector of the highway is prone to several types of instability problems due to the fact that the rock mass unit where the slope cuts were excavated is jointed. Furthermore, at several locations, the slope mass is cut by highly persistent vertical discontinuities which may lead to toppling or rockfall problems. As evidence, fallen blocks along the ditches of the highway can be easily noticed at first glance during field reconnaissance. The discontinuities daylighting towards the highway may also trigger discontinuity controlled planar or wedge type instabilities. Severe slope stability problems along the investigated highway may come up with the loss of life and property. Several views of the examined slopes are depicted in Figure 2.

Slope cuts in the study area were excavated in Lower Miocene aged limestone unit which is locally known as the Adilcevaz Limestone. They were typically deposited in a shelf environment. The limestone layers are yellowish-beige and whitish in color and a rich fossil content is noticeable (Demirtaşlı and Pisoni 1965; Büyükutku 2003). Shear zones and fault slip surfaces can also be observed along the highway indicating a NE-SW trending compressive regional tectonism. As a result of significant tectonism around the region, the sedimentary units were heavily jointed at some locations. The geological map of the study area modified after 1:25,000 scale map of General Directorate of Mineral Research and Exploration is shown in Figure 3.

2 Discontinuity and Rock Mass Properties

2.1 Discontinuity properties

Discontinuity properties of the limestone unit along the studied highway were determined by means of scanline surveys performed at 7 locations. Accordingly, spacing, aperture, persistence, roughness, infill and weathering properties of discontinuities were revealed in accordance with International Society for Rock Mechanics (ISRM 2007) suggestions. The rate of discontinuity parameters are major input for the SSPC system. The percentage distribution of spacing, aperture and persistence of discontinuities at different slope locations is presented in Tables 1, 2 and 3, respectively.

As seen in Table 1, the spacing of the discontinuities in the study area is variable at different slope locations. In general, it can be stated that discontinuities are very closely to moderately spaced in locations 1, 2 3 and 4 except the upper section of location 2. Furthermore, a significant increase in the spacing of the discontinuities is obvious for the rest of the slope locations (locations 5, 6 and 7). Most discontinuities along the investigated highway even bedding planes are commonly open or moderately wide in aperture to some extent, whereas tight apertures can rarely be observed at certain

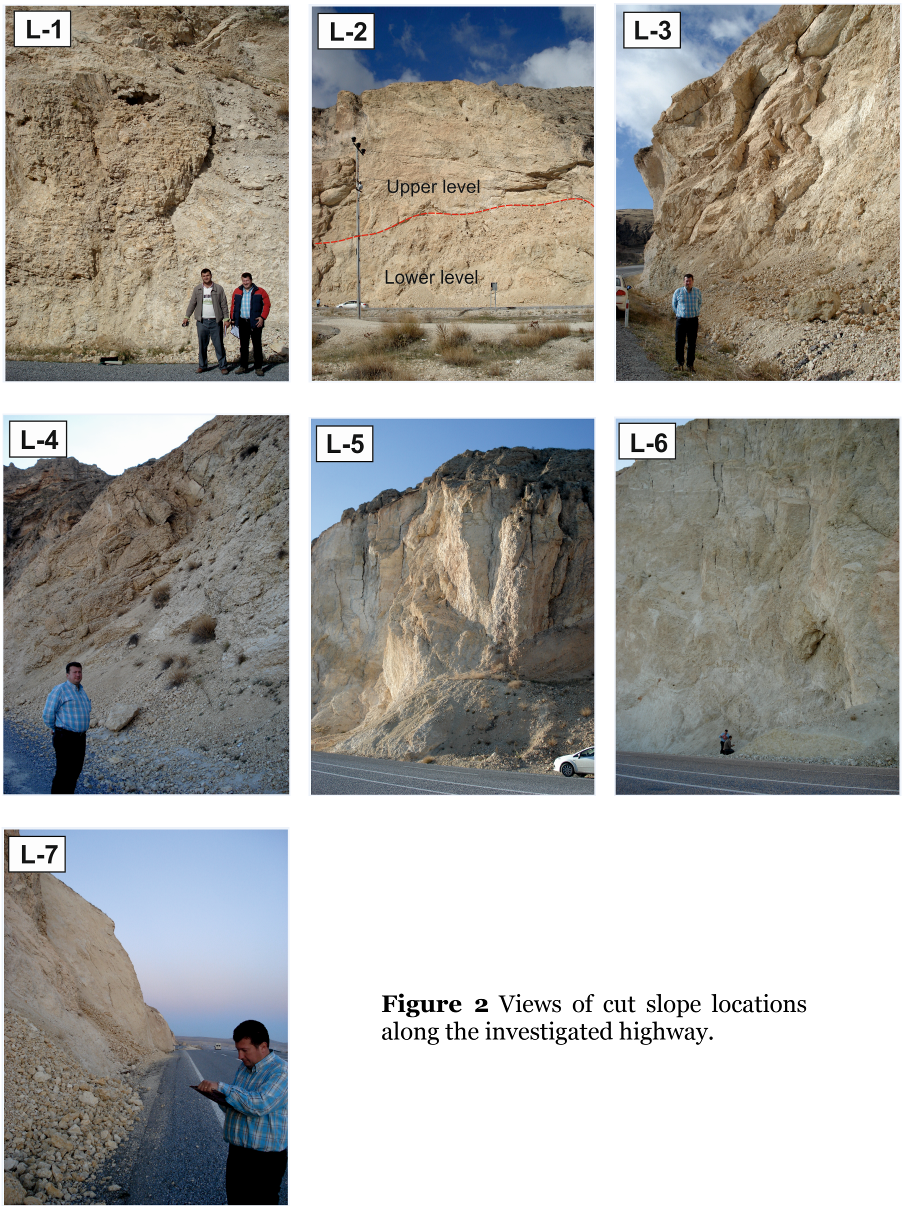

Figure 2 Views of cut slope locations along the investigated highway.

locations (Table 2). The open discontinuity apertures are closely related with tectonic events in the region. The discontinuities in the slope masses are not very persistent in the first four locations (Table 3). However, highly persistent and almost vertical fractures exist at locations 5 to 7 which may trigger toppling failures.

Roughness and waviness parameters are of great importance for the stability of discontinuity controlled failures. An increasing roughness profile of a discontinuity results in a substantial rise in discontinuity shear strength. The limestone unit in the study area presents commonly smooth and rough discontinuity surfaces (profiles IV and V) in accordance with ISRM (2007). Besides, the roughness of the same discontinuities can be classified as slightly curved (i=2°-4°) in large-scale and undulating (amplitude roughness>2-3 mm) in small-scale roughness profiles with respect to the SSPC system (Figure 4).

Commonly, no infill material in the discontinuities can be observed in Adilcevaz limestones, whereas rarely noticed infilling at several locations consists of clay. In addition, the thickness of the infilling is mostly lower than the amplitude of discontinuity aperture at the considered slopes. Furthermore, the weathering of the discontinuities is moderate. However, karstic features are present in the study area which may considerably decrease the shear strength of the rock mass as well as the discontinuities.

The discontinuity surface strength was estimated using an L-type Schmidt hammer at the field except the upper section of location-2 where is personally impossible to reach due to steep and high slope surface. The Schmidt hammer rebound values vary between 27 and 44. When the average unit weight of the Adilcevaz limestone is considered as 23.5 kN/m^3 with respect to laboratory data, the uniaxial compressive strength of the discontinuity surfaces ranges between 32 and 74 MPa in accordance with the graph of Deere and Miller (1966). Accordingly, the Adilcevaz limestone is classified as low to medium strong rock regarding ISRM (2007). Several views of discontinuities from slope locations are shown in Figure 5.

2.2 Rock mass characterization

Rock mass properties of the limestone were revealed by Geological Strength Index (GSI) originally proposed by Hoek and Brown (1997). In the GSI system, rock masses can be classified with regards to block size controlled by discontinuities as well as discontinuity surface condition. On the other hand, determination of block size and discontinuity surface condition is quite subjective in the original GSI chart of Hoek and Brown (1997). Therefore, the GSI chart was modified by Sönmez and Ulusay (2002) to overcome bias. A couple of new parameters were added to the system namely structure rating (SR) and discontinuity surface rating (SCR)

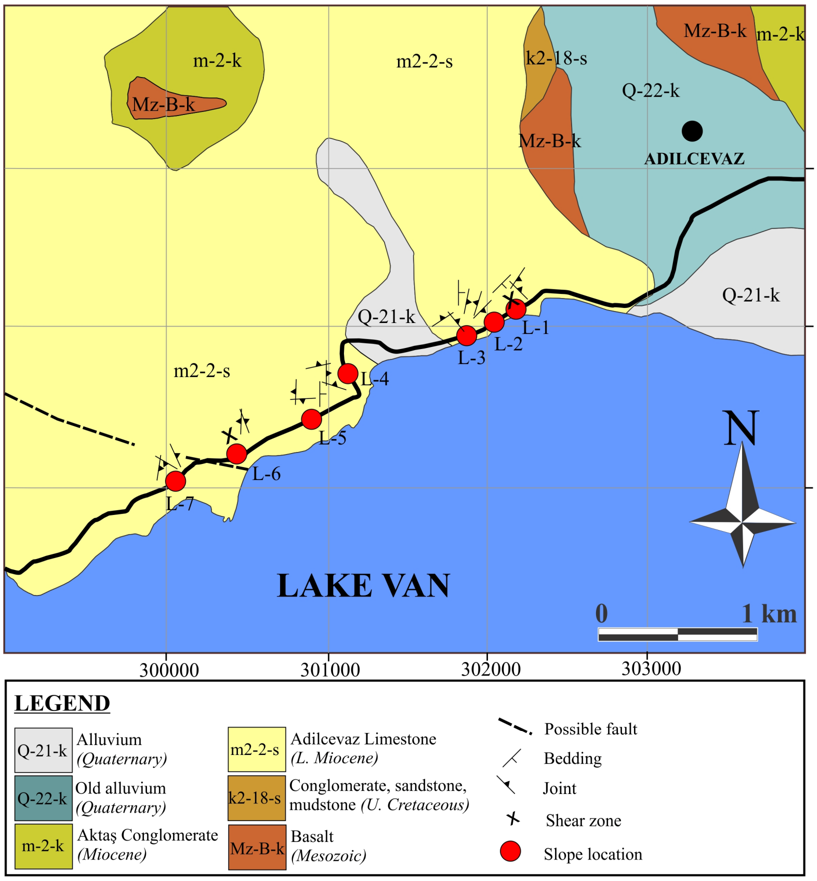

Figure 3 Geological map of the study area (modified after MTA 2000).

Table 1 Percentage distribution of discontinuity spacing at the studied slope locations in accordance with ISRM (2007)

Spacing (mm)	Description	Location							
		1	2 Lower	2 Upper	3	4	5	6	7
<20	Extremely close spacing	10*	5	-	-	-	-	-	-
20-60	Very close spacing	**70**	25	-	5	**40**	-	-	-
60-200	Close spacing	10	**55**	-	25	30	10	5	5
200-600	Moderate spacing	5	10	10	**40**	25	20	10	10
600-2000	Wide spacing	5	5	10	20	3	25	25	**70**
2000-6000	Very wide spacing	-	-	**80**	10	2	**45**	**60**	15
>6000	Extremely wide spacing	-	-	-	-	-	-	-	-

Notes:*All values are in percentage; Bold values identify the maximum percentages.

Table 2 Percentage distribution of discontinuity aperture at the studied slope locations in accordance with ISRM (2007)

Aperture (mm)	Description	Location							
		1	2 Lower	2 Upper	3	4	5	6	7
<0.1	Very tight	-	-	**60**	-	-	-	-	-
0.1-0.25	Tight	-	-	30	5	5	10	5	5
0.25-0.5	Partly open	25*	30	10	15	5	10	5	10
0.5-2.5	Open	**40**	**30**	-	**40**	**30**	15	25	35
2.5-10	Moderately wide	15	20	-	20	30	20	**35**	**30**
>10	Wide	10	10	-	10	10	25	20	10
10-100	Very wide	10	10	-	10	20	**20**	10	10
100-1000	Extremely wide	-	-	-	-	-	-	-	
>1000	Cavernous	-	-	-	-	-	-	-	

Note:*All values are in percentage; Bold values identify the maximum percentages.

Table 3 Percentage distribution of discontinuity persistence at the studied slope locations in accordance with ISRM (2007)

Persistence (m)	Description	Location							
		1	2 Lower	2 Upper	3	4	5	6	7
<1	Very low persistence	20	20	-	20	20	-	-	5
1-3	Low persistence	**35**	**35**	5	**40**	30	10	5	35
3-10	Medium persistence	25	25	10	35	**40**	30	35	**50**
10-20	High persistence	20	20	**45**	5	10	**50**	**40**	10
>20	Very high persistence	-	-	40	-	-	10	20	-

Notes: *All values are in percentage; Bold values identify the maximum percentages.

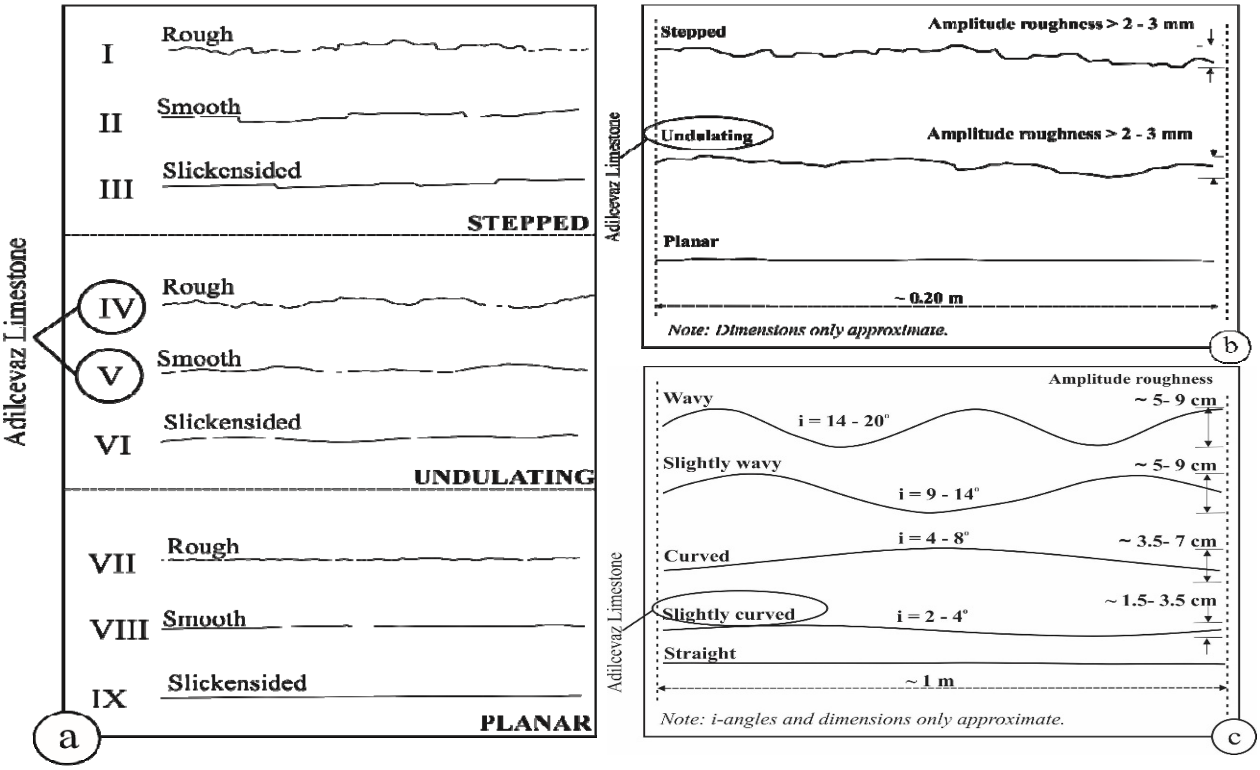

Figure 4 Roughness profiles by International Society for Rock Mechanics (ISRM 2007) (a), large-scale roughness by Slope Stability Probability Classification (SSPC) (b) and small-scale roughness by SSPC (c).

to more accurately define block size and discontinuity surface conditions, respectively. The SR value is assigned based on the volumetric joint count (Jv), whereas the SCR of discontinuities is identified considering roughness, weathering and infilling parameters of discontinuities.

The rock masses along the investigated highway were classified in accordance with the quantitative GSI chart by Sönmez and Ulusay (2002) (Figure 6). The volumetric joint count (Jv) values of the rock masses vary between 1 and 23 with respect to the discontinuity scanline surveys. In addition, the discontinuity surfaces are generally smooth, moderately weathered and do not contain infill. Accordingly, the SR and SCR values were determined to reveal the GSI value of slope masses. As seen in Figure 6, the GSI values of the rock masses at the study area range between 28 and 58. Then, the rock mass at locations 1, 2-lower, 3, 4 and 7 is blocky and disturbed. An increasing rock mass quality is evident at locations 2-upper, 5 and 6 due to less volumetric joint count. A summary of volumetric joint count, structure rating, surface condition rating and GSI value of each location is presented in Table 4.

The shear strength parameters of the slope rock masses in the study area were calculated as a function of GSI values using the Hoek-Brown empirical failure criterion (Hoek et al. 2002) via RocLab v.1.0 software (Rocscience Inc. 2002). The results of rock mass shear strength parameters as well as deformation moduli are given in Table 5. Rock mass uniaxial compressive strength of Adilcevaz limestone is found to be between 0.18 and 3.5 MPa. Besides, the deformation moduli of the rock masses vary between 334 MPa and 4.8 GPa. The rock mass cohesion could be as low as 0.08 MPa while a maximum cohesion value of 0.59 MPa is noticed. Moreover, the rock mass friction angle is in between 34.2° and 50.8° in accordance with the Hoek-Brown failure criterion. It should be emphasized that the shear strength parameters obtained from non-linear failure criterion are instantaneous.

Table 4 Summary of volumetric joint count (J_v), structure rating(SR), surface condition rating(SCR) and GSI value of each location

Location	J_v	SR	SCR	GSI	Rock mass definition
1	23	26	6	28	Blocky-D
2-lower	23	26	6	28	Blocky-D
2-upper	1	80	8	58	Blocky-M
3	18	30	8	35	Blocky-D
4	14	35	8	37	Blocky-D
5	2.8	61	8	47	Blocky
6	2.8	61	8	47	Blocky
7	18	30	8	35	Blocky-D

Note: Blocky-D: Blocky-disturbed; Blocky-M: Blocky-massive.

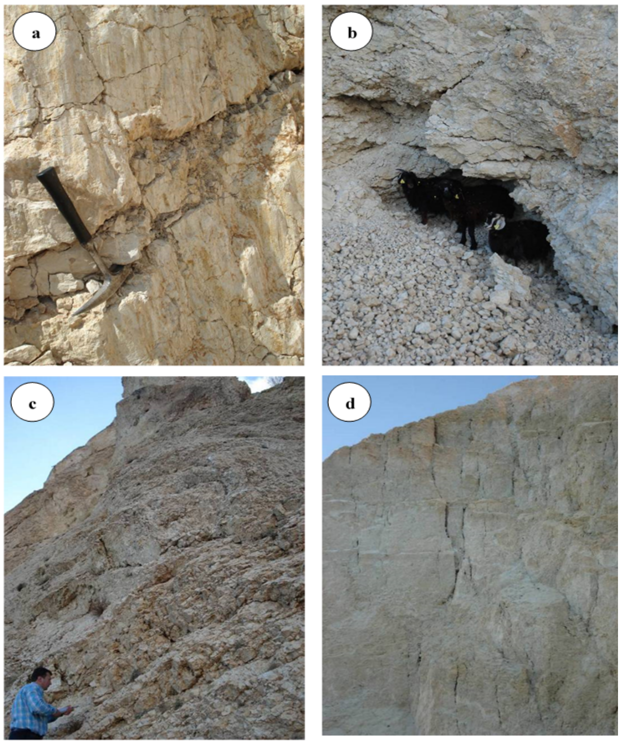

Figure 5 Views showing general properties of discontinuities at the slope locations, fault surface with slickensides (a), open (b), closely spaced (c), highly persistent discontinuities (d).

3 Kinematical Analyses of the Investigated Slopes

Kinematical analyses were carried out with respect to discontinuity and slope orientations for 7 different cut slopes to delineate potential discontinuity controlled failures. Eventually, planar, wedge and toppling type instabilities were investigated by means of Dips v.5.0 software (Rocscience Inc. 1998). It is important to note that the friction angle of discontinuities is accepted to be 30° in kinematical analyses in accordance with the in-situ performed tilt tests. Kinematical analyses of planar, wedge and toppling failures are shown in Figure 7 for the slope location-3 as an example. Furthermore, potential failure types for all slope locations in accordance with the kinematical analyses are summarized in Table 6.

Table 5 Summary of the rock mass shear strength parameters obtained by the Hoek-Brown failure criterion (Hoek et al. 2002)

Location	GSI	UCS_r (MPa)	m_i	D	H (m)	UCS_{rm} (MPa)	E_m (MPa)	c (MPa)	Φ (°)
1	28	28	8	0.5	15	0.18	334	0.08	39.2
2-lower	28	28	8	0.5	30	0.18	334	0.12	34.2
2-upper	58	59	8	0.5	30	3.5	4804	0.59	50.8
3	35	59	8	0.5	20	0.68	629	0.17	45.9
4	37	59	8	0.5	20	0.79	753	0.19	46.7
5	47	59	8	0.5	25	1.64	1842	0.32	48.9
6	47	59	8	0.5	30	1.64	1842	0.34	47.7
7	35	59	8	0.5	20	0.67	629	0.17	45.9

Notes: UCS_r: Intact rock uniaxial compressive strength; m_i: Hoek-Brown constant; D: Disturbance factor; H: Slope height; UCS_{rm}: Rock mass uniaxial compressive strength; E_m: Rock mass deformation modulus; c: Rock mass cohesion; Φ: Rock mass friction angle.

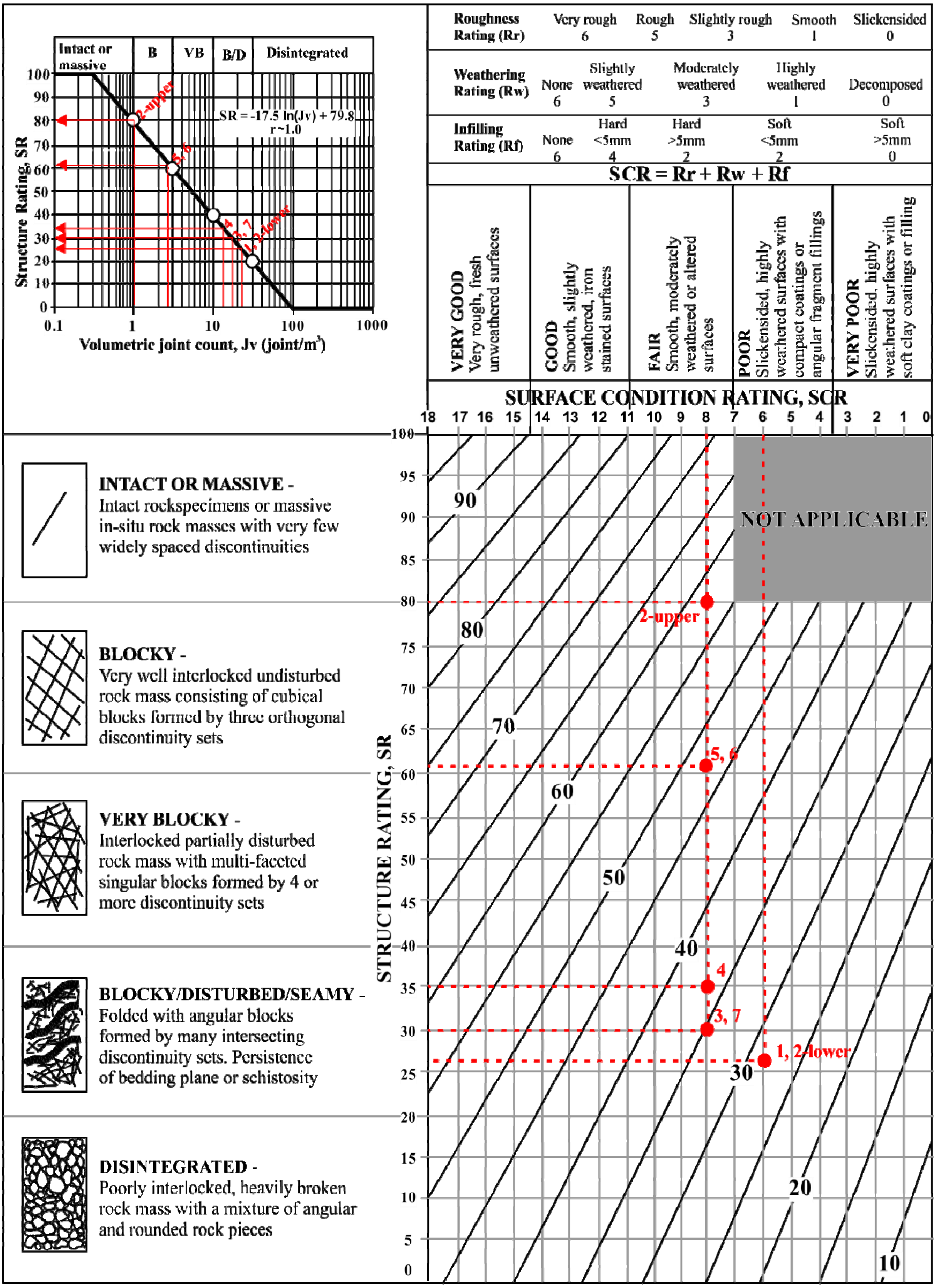

Figure 6 Rock mass classification of slope masses at the study area by the quantitative GSI chart (Sönmez and Ulusay 2002).

As seen in Table 6, all slope locations suffer from various discontinuity controlled slope instabilities particularly planar failure. Wedge failure and toppling are potentially obvious for several locations as well. It should be emphasized that the results of the kinematical analyses coincide with the field observations. Additionally, slides with circular failure surfaces should also be considered at the specified site since the rock mass unit in the study area is highly jointed at certain locations as a result of intense regional tectonism.

4 Probabilistic Slope Stability Assessment Using the SSPC system

4.1 Fundamentals of the SSPC system

The SSPC system was developed for the stability assessment of rock slopes using probabilistic approach. Additionally, the SSPC system classifies and assigns adjustment factors to the rock mass parameters required to classify a slope mass (Lindsay et al. 2000). Furthermore, the rock mass shear strength parameters (cohesion and internal friction angle) as well as the maximum height for a stable slope can also be estimated by the same method. Detailed information about the SSPC system can be found in Hack (1998) and Hack et al. (2003).

Contradictory to other rock mass classification systems, the SSPC method encompasses a three-step evaluation of slope stability probability and

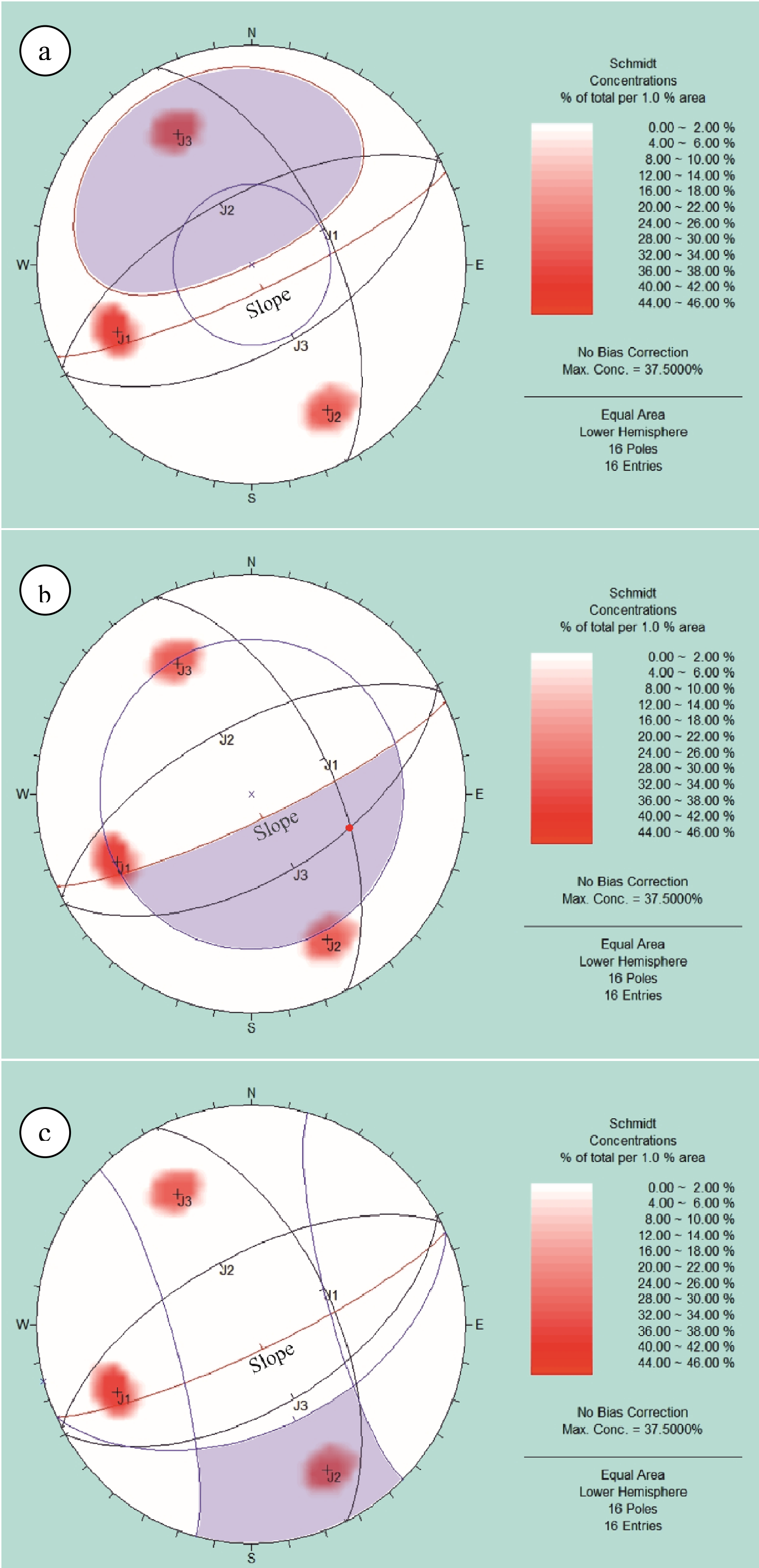

Figure 7 Kinematical analyses of planar (a), wedge (b), toppling (c) failures for the slope location-3.

Table 6 Summary of the kinematical analyses for the investigated slope locations

Location	Failure type		
	Planar	Wedge	Toppling
1	Yes	-	-
2-lower	Yes	Yes	Yes
2-upper	Yes	-	-
3	Yes	Yes	Yes
4	Yes	-	Yes
5	Yes	Yes	Yes
6	Yes	-	-
7	Yes	Yes	-

accordingly a total of 3 different rock masses are identified in the system during probabilistic evaluation (Hack 1998).

a) Exposure Rock Mass (ERM) which is directly observed on the outcrop of a slope.

b) Reference Rock Mass (RRM) which is an imaginary undisturbed and fresh rock mass before the excavation.

c) Slope Rock Mass (SRM) which is the existing or new slope to be excavated (Figure 8).

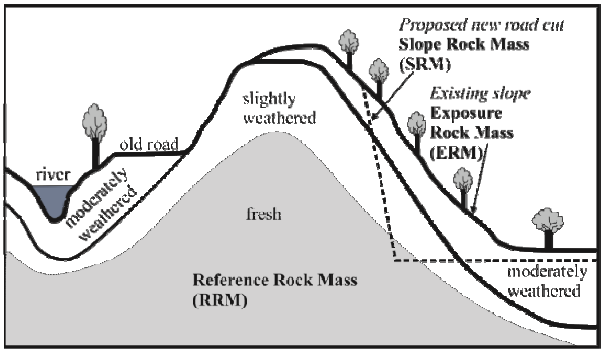

Figure 8 Three different rock masses considered in the concept of the SSPC system (modified after Hack et al. 2003).

It should be noted that the exposure rock mass (ERM) and slope rock mass (SRM) are same if the stability of an existing slope is evaluated. In addition, rock mass parameters of the investigated slope are described and characterized at the exposure namely exposure rock mass. It is obvious that weathering or excavation disturbance significantly affect the rate of parameters measured on the outcrop. Therefore, the parameters of the ERM are converted to those of theoretical fresh rock mass (reference rock mass) in the system that is beneath the weathering zone. Therefore, the parameters of material in the same geotechnical unit with variable weathering and excavation disturbance degrees are converted back to original parameters of fresh rock mass (Hack et al. 2003). The stability of the slope rock mass is determined from the reference rock mass with the adjustment of rock mass parameters (Das et al. 2010).

In the SSPC system, slope stability probability is evaluated on the basis of intact rock material and discontinuity properties such as intact rock strength (IRS), discontinuity spacing (SPA) and condition of discontinuities (CD) obtained from the exposure rock mass. One of the advantages of the SSPC system is that the shear strength parameters of the rock mass (ϕ_{mass} and Coh_{mass}) can also be calculated to be used for slope stability assessment together with the height of investigated slope (H_{slope}) (Das et al. 2010). Moreover, if ϕ_{mass} is smaller than the dip of slope (dip_{slope}), the maximum height of a stable slope (H_{max}) can be determined. The parameters of H_{max}, H_{slope}, ϕ_{mass} and dip_{slope} are used to delineate the orientation independent stability probability. The orientation independent stability expresses the stability of slopes against failures which are not controlled by discontinuities such as falls, raveling or circular slides in highly jointed rock masses. In addition to orientation independent stability, the probability of orientation dependent stability (sliding and toppling) can also be estimated using the probability graphs of the system. Unfortunately, the SSPC method is unable to evaluate the probability of wedge failures (Hack et al. 2003). The simplified flowchart of the SSPC system is depicted in Figure 9.

The characterization parameters of the exposure rock mass (ERM) in the SSPC system are depicted in Appendix 1 with the correction factors of various parameters used for conversion to reference rock mass (RRM).

4.2 Determination of rock mass shear strength parameters by the SSPC system

Besides stability probability, the shear strength parameters of a rock mass can be estimated by mans of the SSPC system. Intact rock strength, discontinuity spacing, roughness, infill and karst parameters are considered during the calculation of rock mass shear strength (Hack 1998). The intact rock strength (IRS) determined in laboratory or in-situ is divided by the weathering

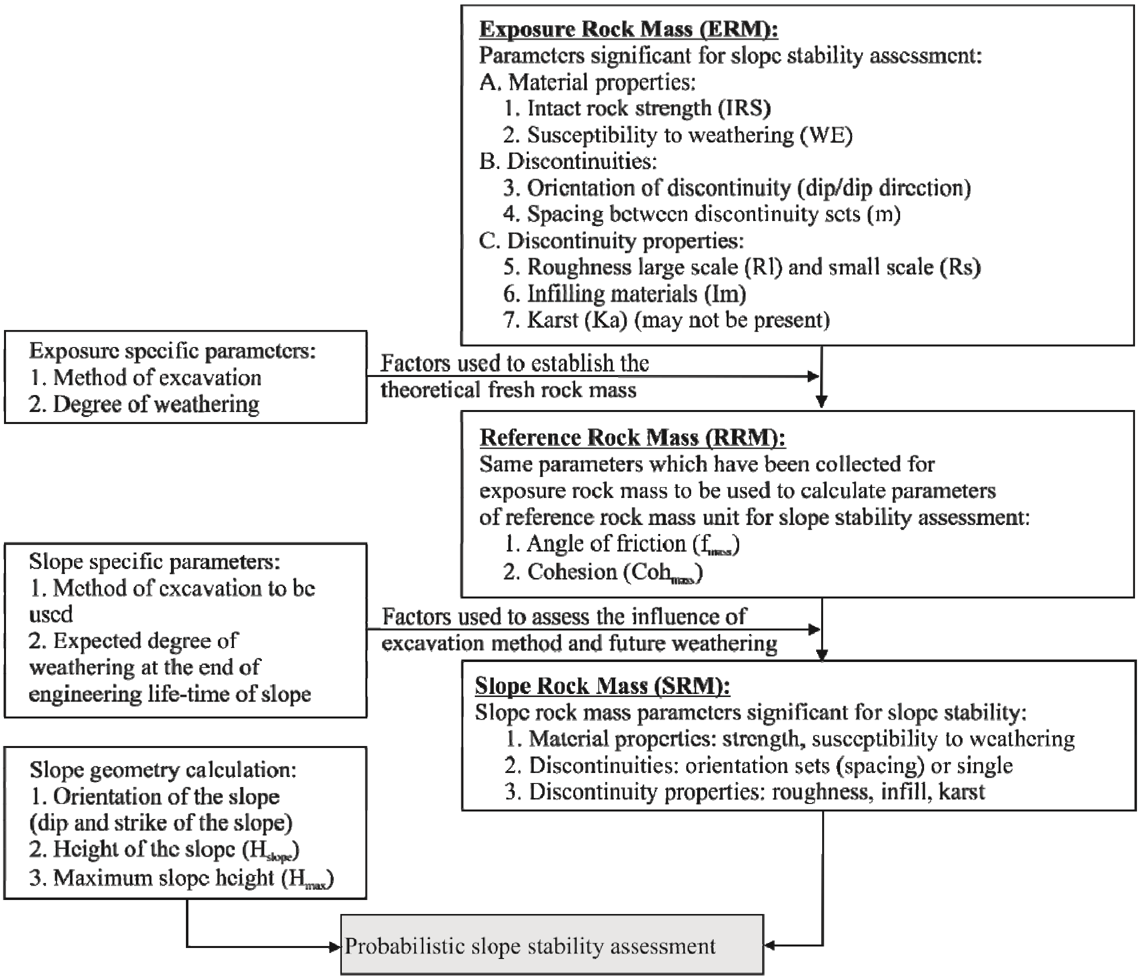

Figure 9 Flowchart of the SSPC system (modified after Hack et al. 2003 and Das et al. 2010).

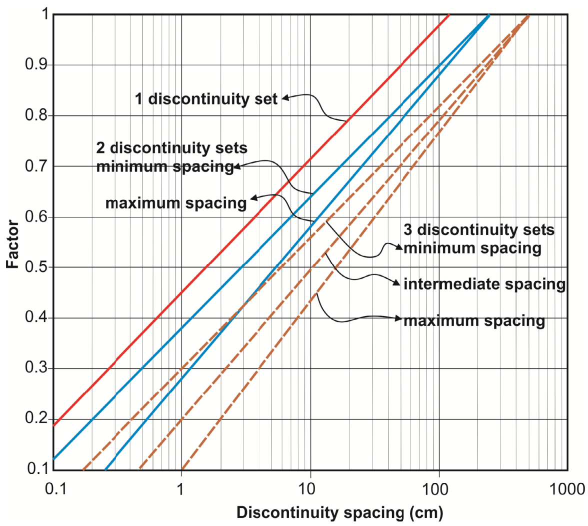

Figure 10 Factor determination graph in accordance with discontinuity spacing (modified after Hack 1998).

degree factor (WE) to calculate the reference rock intact rock strength (RIRS).

$$RIRS = IRS/WE \tag{1}$$

Reference discontinuity spacing (RSPA) is determined considering three discontinuity sets which reveal minimum spacing values (if there are more than three discontinuity sets) in the investigated rock mass. Consequently, a number of three correction factors (factor 1, 2 and 3) are obtained using the graph presented in Figure 10.

Then, the reference discontinuity spacing (RSPA) is calculated considering factors of weathering degree (WE) and method of excavation (ME) by the following equations. It should be noted that the number of factors will be equal to the number of discontinuity sets during the calculation of SPA if less than three discontinuity sets exist in a rock mass.

$$SPA = (\text{Factor } 1)\times(\text{Factor } 2)\times(\text{Factor } 3) \tag{2}$$

$$RSPA = SPA/(WE\times ME) \tag{3}$$

The condition factor (*TC*) of a discontinuity is calculated by a simple multiplication of large-scale roughness (*Rl*), small-scale roughness (*Rs*), infill (*Im*) and karst (*Ka*) factors.

$$TC = Rl \times Rs \times Im \times Ka \quad (4)$$

Discontinuity condition of reference rock mass (RTC) is defined after weathering degree correction by the formula below. It is important to note that the RTC is valid for only one discontinuity set in the reference rock mass (Hack 1998).

$$RTC = TC/(1.452 - 1.220 \times e^{-WE})^{0.5} \quad (5)$$

Then, weighted discontinuity condition (CD) is calculated using TC and discontinuity spacing (DS) based on the following formula with the mean of condition factors of three discontinuity sets weighted against the spacing of discontinuity sets (Hack 1998; Das et al. 2010).

$$CD = \frac{\frac{TC_1}{DS_1} + \frac{TC_2}{DS_2} + \frac{TC_3}{DS_3}}{\frac{1}{DS_1} + \frac{1}{DS_2} + \frac{1}{DS_3}} \quad (6)$$

Subsequently, reference discontinuity condition (RCD) is estimated considering the factor of weathering degree in the following equation.

$$RCD = CD/WE \quad (7)$$

The friction angle (ϕ_{RM}) and cohesion (C_{RM}) of a rock mass is calculated optimizing the Mohr-Coulomb failure criterion with the intact rock strength (RIRS), spacing (RSPA) and condition of discontinuities (RCD) (Hack 1998). If intact rock strength is higher than 132 MPa, then RIRS is accepted as 132 MPa since a RIRS value of about 132 MPa, the stability of slopes do not further increase with an increment in RIRS (Hack et al. 2003; Das et al. 2010).

Table 7 Reference rock mass shear strength parameters for all slope locations determined by the SSPC system

Location	RIRS (MPa)	RSPA	RCD	C_{RM} (MPa)	ϕ_{RM} (°)
1	31.1	0.12	0.43	0.008	16.5
2-lower	31.1	0.17	0.43	0.009	19.0
2-upper	65.6	1.22	0.49	0.04	82.3
3	65.6	0.45	0.49	0.02	42.0
4	65.6	0.18	0.49	0.01	28.0
5	65.6	0.80	0.49	0.03	60.1
6	65.6	0.94	0.49	0.035	67.8
7	65.6	0.55	0.49	0.02	47.3

$$\phi_{RM} = (RIRS \times 0.2417) + (RSPA \times 52.12) + (RCD \times 5.779) \quad (8)$$

$$C_{RM} = (RIRS \times 94.27) + (RSPA \times 28629) + (RCD \times 3593) \quad (9)$$

As an example, the calculation steps of shear strength parameters of reference rock mass for location-3 in the study area are depicted in Appendix 2. In addition, reference rock mass shear strength parameters for all slope locations are summarized in Table 7.

4.3 Determination of orientation dependent stability probability

The final step of the SSPC system is the determination of stability probability. As aforementioned, stability is evaluated for both orientation dependent and orientation independent conditions in the system. Orientation dependent stability indicates discontinuity controlled failures (planar and toppling) while orientation independent stability defines the failures occur irrespective of a certain sliding surface.

Discontinuity condition (STC) and the apparent angle of the dip of discontinuity plane in the direction of slope dip (AP) are taken into consideration during the orientation dependent stability probability evaluation in the SSPC system (Hack 1998). The AP is expressed as follows:

$$AP = \arctan(\cos\delta \times \tan\beta_j) \quad (10)$$

where δ is the dip direction difference between slope and discontinuity (δ = Slope direction– discontinuity dip direction) and β_j is the discontinuity dip.

Discontinuity condition of the slope (STC), which significantly controls the shear strength of discontinuity, can be determined as a function of RTC determined during rock mass characterization in Eq. 5. The STC is determined as follows with respect to RTC and the weathering factor of slope in the engineering life time (SWE).

$$STC = RTC \times (1.452 - 1.220 \times e^{-SWE})^{0.5} \quad (11)$$

Having determined the parameters of AP and STC, stability condition is initially defined considering the kinematical relations presented in Table 8. Subsequently, the probability graphs of the SSPC system for planar and toppling failures are employed to determine the stability probability with respect to STC and AP parameters. Slope stability probability against planar and toppling failures for dominant discontinuity sets at the investigated slope locations are presented on default SSPC probability graphs in Figure 11 and Figure 12. Furthermore, the probability results and the necessary parameters for probability calculations in the study area are summarized in Table 9. The stability probability evaluations executed by means of the SSPC system indicate that planar and toppling stability probabilities of slopes 2-lower, 2-upper, 3, 6 and 7 are lower than 5% as seen in Table 9. In other words, stability probabilities are considerably low at these slopes specifying severe stability problems. Planar failure probability is quite high at slopes 2-lower and upper, 3 and 7, while slopes 3 and 6 suffer from toppling failures. On the other hand, stability probability against toppling failure is around 80-90% at slope locations 1 and 5 revealing a low failure probability. Although a planar failure potential can be attributed to slopes 5 and 6 regarding kinematical analyses (see Table 6), the SSPC system reveals high stability probability for those locations. Similarly, there is no toppling potential for the slope locations 6 and 7 despite low stability probability against toppling in the SSPC system. Nevertheless, high probability of planar and toppling failure at other slopes (2, 3 and 7) were verified by kinematical analyses.

Table 8 Orientation dependent stability conditions in the SSPC system (Hack 1998)

Conditions	Discontinuity orientation	Planar	Toppling
AP > 84° or AP < -84°	vertical	100%	100%
(slope dip+5°) < AP < 84°	with	100%	100%
(slope dip-5°) < AP < (slope dip+5°)	equal	100%	100%
0 < AP < (slope dip-5°)	with	Planar*	100%
AP<0° and (-90°-AP+slope dip)<0°	against	100%	100%
AP<0° and (-90°-AP+slope dip)>0°	against	100%	Toppling**

Note:*Use planar failure probability graph;**Use toppling failure probability graph.

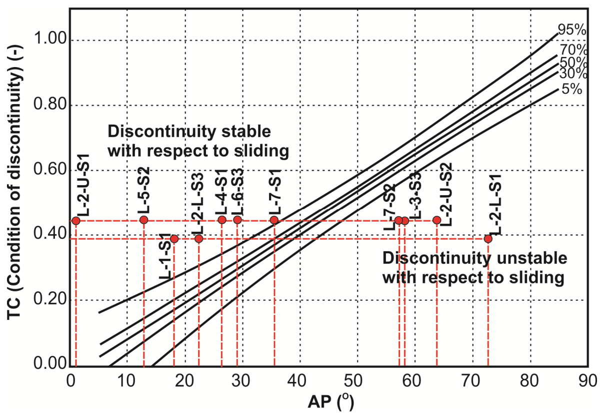

Figure 11 Slope stability probability against planar (sliding) failure at the investigated slopes.

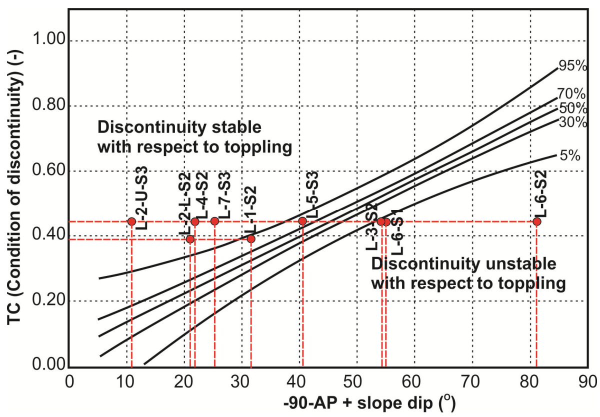

Figure 12 Slope stability probability against toppling failure at the investigated slopes.

4.4 Determination of orientation independent stability probability

The stability of highly jointed rock slopes mainly depends upon the shear strength of rock mass and slope height, and the failure surface passes partially through intact rock and partially along existing discontinuities. For such slope masses, the SSPC system considers a linear shear plane model following the Mohr-Coulomb failure criterion to assess stability (Hack et al. 2003; Li and Xu 2016). In this case, the internal friction angle and cohesion in Mohr-Coulomb failure criterion are represented by rock mass shear parameters which directly depend on intact rock strength, block volume and shear strength of all discontinuities in the rock mass.

In the SSPC system, intact rock strength of slope mass (SIRS), discontinuity spacing (SSPA) and discontinuity condition (SCD) are determined as a function of reference rock mass values corrected by the factors of weathering degree (SWE) and method of excavation (SME) for existing/excavated slope. It is important to note that the weathering grade and method of

Table 9 Orientation dependent stability probabilities of discontinuities at the investigated slopes

Location	Discontinuity	STC	AP	Potential failure type	Stability probability (%)
1	S1	0.39	18.83	Planar	100
	S2	0.39	-46.93	Toppling	90
	S3	0.39	77.25	-	100
2-lower	S1	0.39	72.58	Planar	<5
	S2	0.39	-25.86	Toppling	100
	S3	0.39	21.48	Planar	100
2-upper	S1	0.44	1.00	Planar	100
	S2	0.44	63.59	Planar	<5
	S3	0.44	-15.84	Toppling	100
3	S1	0.44	-1.66	-	100
	S2	0.44	-63.97	Toppling	<5
	S3	0.44	57.90	Planar	<5
4	S1	0.44	24.88	Planar	100
	S2	0.44	-57.40	Toppling	100
	S3	0.44	68.68	-	100
5	S1	0.44	85.43	-	100
	S2	0.44	12.99	Planar	100
	S3	0.44	-46.29	Toppling	80
6	S1	0.44	-60.26	Toppling	<5
	S2	0.44	-85.76	Toppling	<5
	S3	0.44	28.57	Planar	100
7	S1	0.44	36.48	Planar	100
	S2	0.44	56.85	Planar	<5
	S3	0.44	-44.56	Toppling	100

excavation factors are similar to those determined for the exposure rock mass in case the stability of an existing slope is assessed and future weathering is neglected.

$$\mathrm{SIRS} = \mathrm{RIRS}\times\mathrm{SWE} \quad (12)$$

$$\mathrm{SSPA} = \mathrm{RSPA}\times\mathrm{SWE}\times\mathrm{SME} \quad (13)$$

$$\mathrm{SCD} = \mathrm{RCD}\times\mathrm{SWE} \quad (14)$$

Accordingly, friction angle and cohesion of slope mass are defined by the following equations which are similar to those of reference rock mass presented in Eqs. 8 and 9.

$$\phi_{SRM} = (\mathrm{SIRS}\times0.2417) + (\mathrm{SSPA}\times52.12) + (\mathrm{SCD}\times5.779) \quad (15)$$

$$C_{SRM} = (\mathrm{SIRS}\times94.27) + (\mathrm{SSPA}\times28629) + (\mathrm{SCD}\times3593)\ (\mathrm{Pa}) \quad (16)$$

It can be concluded that there will be no orientation independent stability problem if slope dip is less than or equal to ϕ_{RM} for the investigated slope. Contrarily, stability condition is evaluated on the basis of maximum slope height (H_{max}) (Hack 1998).

$$H_{max} = [1.6\times10^{-4}\times C_{SRM}\times\sin(\text{slope dip})\times\cos(\phi_{SRM})]/[1-\cos(\text{slope dip} - \phi_{SRM})] \quad (17)$$

The orientation independent slope stability is determined by means of a probability chart presented by Hack (1998) which considers the ratios between maximum slope height (H_{max}) and existing slope height (H_{slope}), internal friction angle of slope (ϕ_{SRM}) and slope dip. The orientation independent stability probabilities are illustrated on the probability chart of the SSPC system in Figure 13. In addition, necessary data for the orientation independent probability evaluation at the specified site are summarized in Table 10.

Orientation independent stability probabilities of all slopes except the upper section of slope-2 are around 5%. Low probability values for the investigated locations along a major highway point out severe stability problems which may be extremely hazardous and result in extensive loss of life and property. Fallen blocks with large volume can be observed along the highway verifying the results of probabilistic stability assessment.

The actual slope height values of existing slopes are considerably higher than those of maximum height values calculated by the SSPC method as seen in Table 10. In other words, very high rock slopes were excavated in poor rock mass conditions at these locations which are very

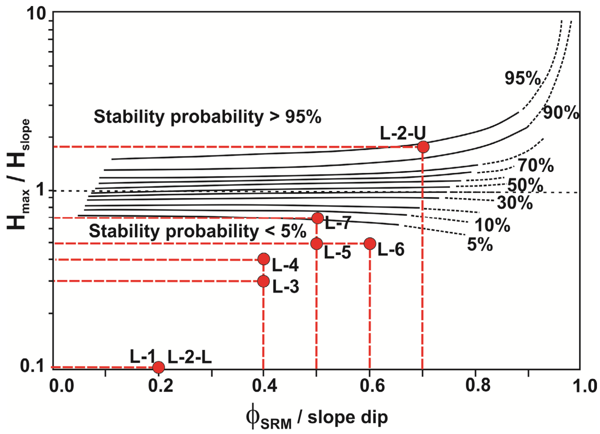

Figure 13 Orientation independent stability probability at the investigated slopes.

Table 10 Summary of orientation independent stability probability at the investigated slopes.

Location	Slope height (m)	ϕ_{SRM} (°)	H_{max} (m)	ϕ_{SRM}/Slope dip	H_{max}/Slope height	Stability probability (%)
1	15	13.4	1.9	0.2	0.1	<5
2-lower	15	15.2	1.7	0.2	0.1	<5
2-upper	15	60.3	26.9	0.7	1.8	93
3	20	32.7	6.6	0.4	0.3	<5
4	20	23.2	8.5	0.4	0.4	<5
5	25	45.5	11.2	0.5	0.5	<5
6	30	50.9	15.1	0.6	0.5	<5
7	20	36.6	13.2	0.5	0.7	5

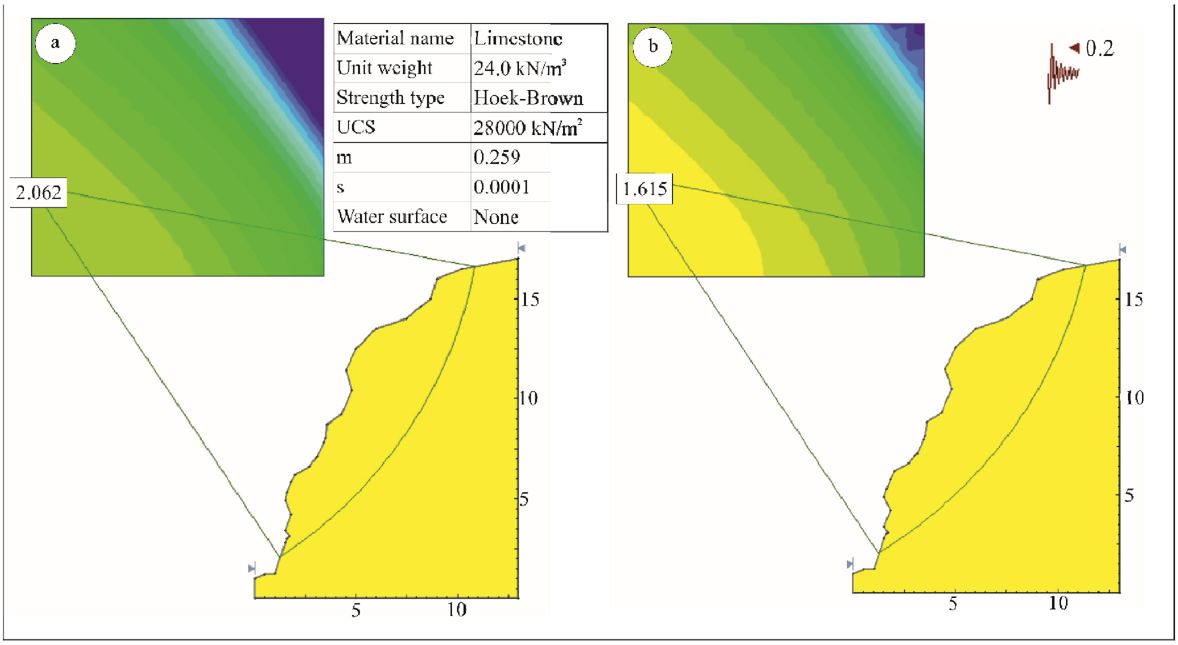

Figure 14 Limit equilibrium analysis of location-1 for static (a) and dynamic (b) conditions.

sensitive to stability problems.

5 Limit Equilibrium Analysis

In addition to probabilistic assessment of slope stability along the investigated highway, limit equilibrium analysis of circular failure in heavily jointed rock mass was carried out for location-1 in order to compare probabilistic and deterministic methods. Slide v.5.0 software (Rocscience Inc. 2002) was employed during analysis considering the Hoek-Brown failure criterion. The uniaxial compressive strength of the rock mass is 28 MPa while m and s constants of the criterion are accepted to be 0.259 and 0.0001, respectively. The results of limit equilibrium analysis for static and dynamic conditions (for horizontal a_{max} 0.2g) are shown in Figure 14.

Eventually, the factor of safety values in static and dynamic conditions are 2.0 and 1.6. High factor of safety values indicate that the slope (location-1) is stable against a circular failure even in very poor rock mass conditions. It can be concluded that large-scale circular failures are not very probable in the study area with respect to limit equilibrium analysis. Therefore, low probability of orientation independent stability determined by the SSPC method can be attributed to rockfall or raveling type of instabilities rather than circular failures.

6 Conclusions

Slope stability probability of very high (15-

30m) and steep (50°-90°) cut slopes excavated in the limestone unit was investigated in this study using the SSPC system. As a result of probabilistic analyses, it is concluded that the investigated rock slopes are prone to stability problems which may occur on discontinuity surfaces or irrespective of discontinuities. Therefore, urgent remedial measures should be taken such as re-designing of slope, rock bolting and rockfall barriers along the investigated highway against possible failures to prevent loss of life and property.

As a major disadvantage, the SSPC system is unable to analyze the probability of wedge failure. Besides, the effect of groundwater on the shear strength of discontinuities is neglected in the system. Furthermore, no certain approvals are valid for probabilistic analyses while accepting a safe probability threshold value similar to factor of safety concept in analytical methods. Although certain probability values can be selected satisfying the designers of a rock slope excavation project, a minimum of 70% slope stability probability is necessary in the SSPC system to define a slope as stable in accordance with the authors' experiences.

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