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A Comparative Study on the Estimation of Shear Strength of Rock Masses Using Rock SSPC System and Hoek-Brown Criterion

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ABSTRACT

The shear strength of rock masses has vital importance for geotechnical projects such as slope stability, foundation and tunnels. The shear strength of the jointed rock mass can be estimated by different methods including large-scale testing, back calculation, rock mass classification system and empirical criterion. In this study, the peak and residual shear strength envelopes of jointed magmatic rock masses assessed using the Hoek-Brown criterion were compared with those assessed using the SSPC system The rocks masses evaluated in this study were outcropped at Gümüşhane-Giresun highway, NE of Turkey. The geotechnical units were separated from the rock masses using the lithological features, the weathering state and the frequency of discontinuity. It is determined that the meaningful relationships can be obtained between the values of shear strength parameters of the geotechnical units obtained by Hoek-Brown failure criterion and SSPC system .

Key Words: : *jointed rock masses, magmatic rocks, shear strength, SSPC, Hoek-Brown criterion.*

1. INTRODUCTION

Shear strength of rock masses has a significant effect on the design of engineering constructions such as slopes, foundations and tunnels. Therefore, it can be said that a better understanding of the engineering property of the rock can provide a base for a more rational approach to the design of the engineering constructions. In slopes where closely jointed rock masses are encountered, failure can occur both through the rock mass, as a result of a combination of macro and micro jointing, and through the rock substance [1]. Determination of the strength of this category of rock mass is extraordinarily difficult since the size of representative specimens is too large for laboratory testing [1]. The shear strength of the rock mass can be determined by different methods including large-scale testing, back calculation, rock mass classification system and empirical criterion. Two of these methods are Hoek-Brown failure criterion [2] and Slope Stability Probability Classification System [3].

The Slope Stability Probability Classification (SSPC) system developed by Hack [4] has two distinctive components as output parameters. The first component of the analysis is the slope stability probability assessments, including orientation independent and orientation dependant stability assessments based upon kinematics

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and probability analysis. The second component includes the rock mass cohesion, friction angles and rock mass strength [3], [4], [5].

The Hoek–Brown criterion is one of the non-linear criteria widely accepted and used by engineers to estimate the strength and deformation of a rock mass [2], [6], [7], [8], [9], [10]. In the last decade, the index was further developed and modified, particularly for poor and heterogeneous rock masses for designing projects such as tunnels, slopes and foundations in rocks [8], [9], [10], [11]. An explanation for the applicability of Hoek–Brown criterion to rock slopes is displayed in Figure 1. Guidelines given by the Geological Strength Index (GSI) system are for the estimation of the peak strength of jointed rock masses named Group III in Figure 1.

In general, rock masses, except when highly disturbed, exhibit strain-softening post-peak behavior, so that the residual strength parameters are lower than the peak parameters [14]. Both are required for design. The peak and residual strengths are, respectively, the maximum and minimum stresses of a rock mass that can be sustained under a given confinement condition [14]. The residual strength is, generally, only reached after considerable plastic deformation [14].

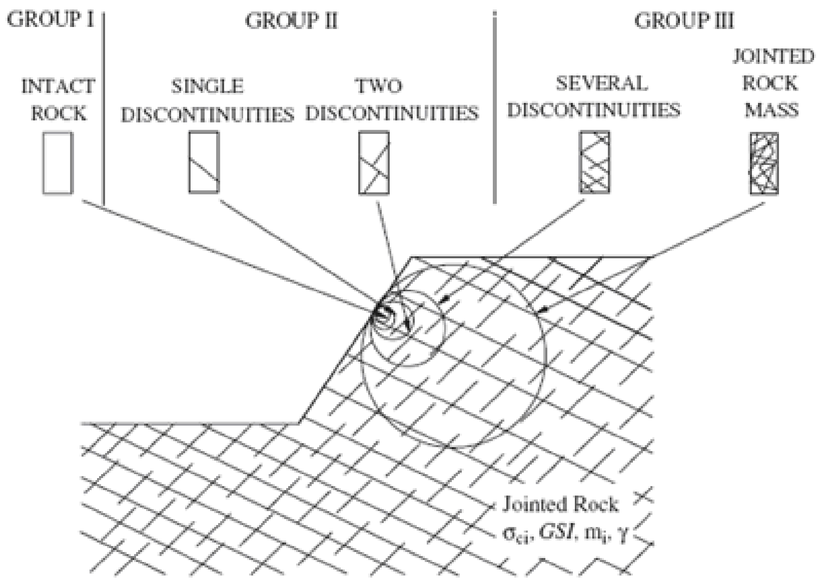


Figure 1. Applicability of the Hoek-Brown criterion for slope stability problem [13].

Many researchers have tried to correlate the various classification systems and some relations have been proposed as the outcomes of those studies (e.g. [15], [16], [17], [18], [19], [20], [22], [23], [24], [25]). The shear strength envelopes obtained from laboratory direct shear testing of relatively large, jointed soft rock masses were compared to those assessed using the Hoek-Brown GSI criterion in the study practiced by Szymakowski Haberfield [22]. In the abovementioned study, the correlation between two envelopes suggests that the GSI envelope slightly underestimates the shear strength envelope. The authors stated that this shows the importance of the scale on rock mass behavior and the effect of joint spacing on rock mass strength.

Tzamos and Sofianos [24] investigated four classification systems: Rock Mass Rating System (RMR), Q-System, Geological Strength Index and Rock Mass Index. According to the authors, the common parameters of these systems, which concern and characterize solely the rock mass, are those used for rating the rock structure and the joint surface conditions. Rock structure is quantified by the block size or the discontinuity spacing ratings while the joint surface conditions are quantified by the joint conditions ratings. The authors defined A Rock Mass Fabric Index as a scalar function of the components rock structure and joint conditions and explained that all rock mass classification systems' ratings are grouped together in a common Fabric Index chart. The validity of the chart was tested using data extracted from various projects in the study. They suggested the use of the chart to simplify the input, to correlate rock mass classification systems and improve their utility.

In this study, the peak and residual shear strength envelopes of jointed magmatic rock masses selected from Gümüşhane-Giresun highway, NE Turkey assessed using the Hoek-Brown criterion were compared with those obtained using the SSPC system. Firstly, the geotechnical units were separated from the jointed rock masses using the lithological features, the weathering state, and the frequency of discontinuity. The properties of rock materials and discontinuities were investigated for each geotechnical units and the shear strength parameters of the geotechnical units were estimated using Hoek- Brown criterion and SSPC system. Later, the relationships were investigated between the shear strength envelope of the geotechnical units estimated using Hoek-Brown criterion and SSPC system. While deriving these relationships, A? Rock Mass Fabric Index suggested by Tzamos and Sofianos(2007) is not considered.

2. MATERIAL AND METHODS

In this study the rocks masses outcropped at Gümüşhane-Giresun highway, NE Turkey were evaluated (Figure 2).

In this area, the oldest rocks are Turonian-Santonian andesite and its pyroclastics that are interbedded with dark red colored clayey limestone, sandy limestone and tuffit. These series of rocks called as Catak Formation that concordantly overlie Kizilkaya Formation, is consist of mainly Turonian??-Santonian dacite and its pyroclastics with some sedimentary rock lenses. Sarıosman Granitoid aged Campanian-Maastrichtian Çağlayan Formation concordantly covers all these series. All these rocks concordantly overlie Travertine aged Quaternary (Figure 3) [26].

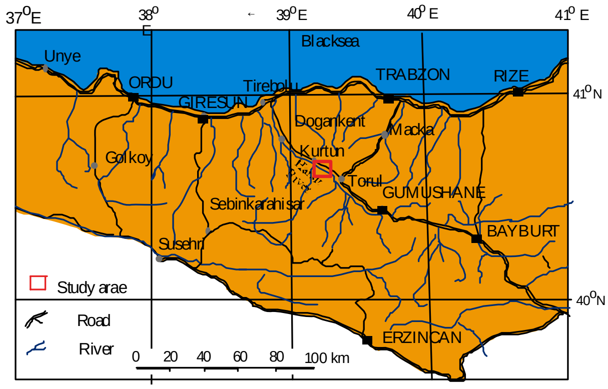


Figure 2. Location maps of the study area..

In this study, twelve geotechnical units are determined taking into consideration the lithological features, the weathering degree and the frequency of discontinuity in the jointed magmatic rocks, granite, dacite and andesite, selected. (Figure 4). The properties of rock materials and discontinuities were investigated for each geotechnical unit. In order to estimate the shear strength parameters of the geotechnical units, Hoek- Brown criterion and SSPC system were used.

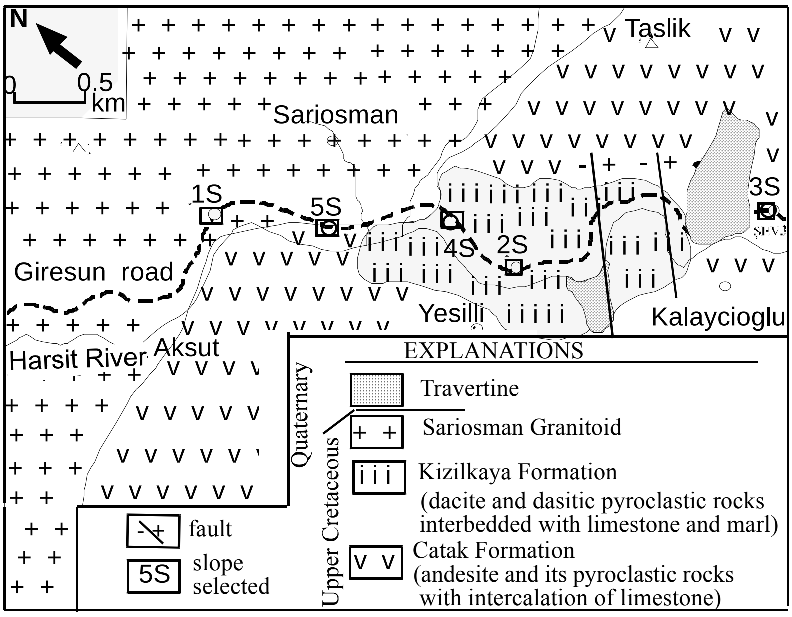


Figure 3. Geological map of the study area [26].

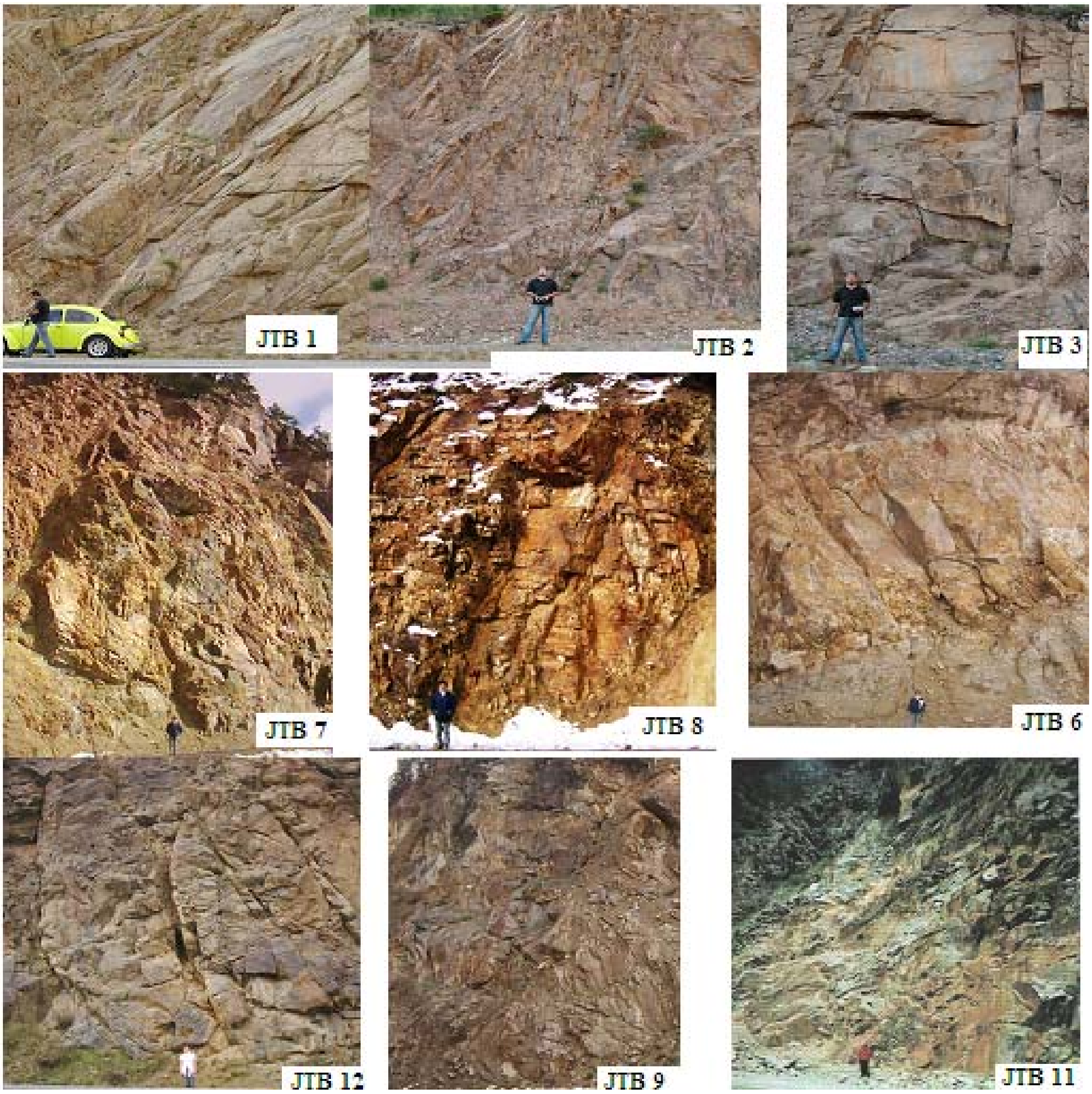


Figure 4. Geotechnical units (JTB) determined in magmatic rocks exposed in the selected slope.

3. ESTIMATION OF THE SHEAR STRENGTH OF THE JOINTED ROCK MASSES USING SSPC SYSTEM AND HOEK-BROWN CRITERION

In the SSPC, rock mass cohesion (c*, Pa) and friction angle (ϕ*, degree) are calculated as follows ([3], [4],).

$$\phi^* = 0.2417\sigma_{ci} + 52.12SPA + 5.779CD \qquad (1)$$

$$c^* = 94.27\sigma_{ci} + 28629SPA + 3593CD \qquad (2)$$

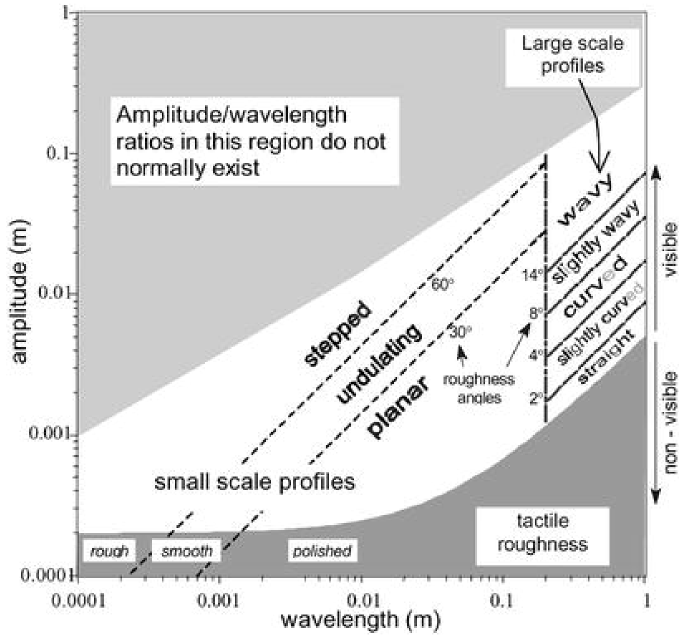
$$CD = \frac{\frac{TC1}{DS1} + \frac{TC2}{DS2} + \frac{TC3}{DS3}}{\frac{1}{DS1} + \frac{1}{DS2} + \frac{1}{DS3}} \qquad (3)$$

TC=(Rl) (Rs) (Im) (Ka) (4)

$$SPA = (0.30 + 0.259 \log_{10} Sa)(0.20 + 0.296 \log_{10} Sb)(0.10 + 0.333 \log_{10} Sc) \qquad (5)$$

Where, SPA is spacing factor, CD is condition of discontinuities, Sa is minimum spacing (m), Sb is intermediate spacing (m), Sc is maximum spacing (m), TC1, TC2 and TC3 are discontinuity condition of three discontinuity sets, DC1, DC2 and DC3 are discontinuity spacing of three discontinuity sets, Rl and Rs are large-scale roughness and small scale roughness, respectively Im is infill material and Ka is karst (Figure 5).

Definition of roughness



Rating of large-scale roughness (Rl)

wavy (1,00)
slightly wavy (0,95)
curved (0,85)
slightly curved (0,80)
straight (0,70)

Rating of small-scale roughness (Rs)

rough stepped/irregular	(0,95)
smooth stepped	(0,90)
polished stepped	(0,85)
rough undulating	(0,80)
smooth undulating	(0,75)
polished undulating	(0,70)
rough planar	(0,65)

Rating of Karst (Ka)

No (0,92); Karst (1,00)

Rating of Infilling Materials(Im)

No infill-surface staining (1,00);non softening and sheared material: coarse (0,95), medium (0,90), fine (0,85); soft sheared material: coarse (0,75), medium (0,65), fine (0,55)

Figure 5. Definition of roughness and rating of Infilling Material, karst and roughness [3].

In order to estimate the shear strength parameters, cohesion and friction angle of the geotechnical units using SSPC system, the properties of discontinuities were measured in the field (Table 2).

Table 2. The properties of discontinuities measured in the field and the shear strength parameters of the geotechnical units obtained using SSPC system.

Rock type		Granite				Dacite				Andesite			
JTB		1	2	3	4	5	6	7	8	9	10	11	12
The properties of discontinuities measured in the field													
	φ_j	167	151	295	214	103	177	287	308	150	146	162	291
	ß$_j$	85	87	25	57	61	62	52	57	87	48	86	72
	Js	0,34	0,16	0,36	0,18	0,21	0,35	0,25	0,18	0,18	0,32	0,2	0,39
JS1	RI	0,8	0,8	0,85	0,75	0,8	0,75	0,8	0,8	0,8	0,8	0,8	0,85
	Rs	0,8	0,8	0,8	0,75	0,75	0,8	0,8	0,8	0,8	0,8	0,8	0,8
	Im	1	0,9	0,9	1	1	0,55	1	0,65	0,65	1	0,9	1
	TC	0,64	0,58	0,61	0,56	0,6	0,33	0,64	0,42	0,42	0,64	0,54	0,68
	φ_j	76	72	252	315	220	276	302	212	295	162	142	168
	ß$_j$	86	15	80	50	77	56	28	76	85	38	44	58
	Js	0,56	0,18	0,47	0,27	0,24	0,44	0,38	0,21	0,19	0,44	0,2	0,56
JS2	RI	0,75	0,8	0,8	0,75	0,8	0,85	0,75	0,8	0,8	0,85	0,9	0,85
	Rs	0,8	0,85	0,8	0,75	0,8	0,8	0,8	0,8	0,85	0,8	0,8	0,8
	Im	1	0,9	0,9	1	1	0,75	1	0,9	0,9	0,65	0,9	0,9
	TC	0,6	0,61	0,58	0,56	0,64	0,51	0,6	0,58	0,61	0,44	0,61	0,61
	φ_j	144	68	332	122	315	325	115	104	162	287	282	108
	ß$_j$	36	86	80	82	38	32	38	42	38	32	85	78
	Js	0,36	0,18										
JS33	RI	0,8	0,8	0,8	0,8	0,95	0,75	0,95	0,75	0,85	0,8	0,9	0,8
	Rs	0,8	0,8	0,8	0,85	0,8	0,75	0,8	0,75	0,8	0,8	0,8	0,8
	Im	1	1	0,9	1	1	0,55	0,55	1	0,9	1	0,6	1
	TC	0,64	0,64	0,58	0,68	0,76	0,31	0,42	0,56	0,61	0,64	0,37	0,64
Strength Properties of Rock Mass													
σ_{ci}(MPa)		98,5	56,7	12,37	47,8	98,43	122	132	48,9	58,7	93,43	43,5	112,2
SPA		0,26	0,18	0,343	0,23	0,223	0,15	0,28	0,202	0,209	0,33	0,189	0,151
CD		0,63	0,61	0,593	0,59	0,661	0,39	0,57	0,514	0,537	0,575	0,518	0,648
Φ*(degree)		41	26	24	27	39	39	49	25	28	43	23	39
c*(MPa)		0,02	0,01	0,013	0,01	0,018	0,02	0,02	0,012	0,0134	0,0203	0,0114	0,0172

(JTB: geotechnical unit, JSi Discontinuity set, φ_j mean dip direction of the discontinuity set (degree), ß$_j$ mean dip of the discontinuity set (degree), Js mean spacing of the discontinuity set, σ_{ci}: unconfined compressive strength of rock material, TC: discontinuity condition of discontinuity set)

Hoek-Brown criterion, defined by the following equation [2];

$$\sigma'_1 = \sigma'_3 + \sigma'_{ci}\left(m_b \frac{\sigma'_3}{\sigma'_{ci}} + s\right)^a$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \qquad (6)$$

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{7}$$

where σ'_1 and σ'_3 are the major and minor effective principal stresses at failure, σ/c_i is the uniaxial compressive strength of the intact rock material and m and s are material constants where s=1 for intact rock, m_i value of granite, dacite and andesite are given as 29, 25 and 23, respectively[2], [11] ,[14].. D is a factor which depends on the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation [2].

The following equations give the angle of friction (ϕ') and cohesive strength (c') from the Mohr-Coulomb failure criterion [2], [11] ,[14]:

$$\phi' = \sin^{-1}\left[\frac{6am_b(s + m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b(s + m_b\sigma'_{3n})^{a-1}}\right] \tag{8}$$

$$c' = \frac{\sigma_{ci}\left[(1+2a)s + (1-a)m_b\sigma'_{3n}\right](s + m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + (6am_b(s + m_b\sigma'_{3n})^{a-1})/((1+a)(2+a))}} \tag{9}$$

$$\sigma_{3n} = \sigma'_{3\max} / \sigma_{ci} \quad , \quad \frac{\sigma'_{3\max}}{\sigma'_{cm}} = 0.72\left[\frac{\sigma'_{cm}}{\gamma H}\right]^{-0,91} \text{ for slopes)} \tag{10}$$

$$GSI(Vb, jC) = \frac{26.5 + 8.79\ln(jC) + 0.9\ln Vb}{1 + 0.0151\ln(jC) - 0.0253Vb} \tag{11}$$

$$jC = \left(\frac{jR}{jA}\right) = \left(\frac{js.jw}{jA}\right) \tag{12}$$

$$Vb = \frac{s1 \;\; s2 \;\; s3}{\sqrt[3]{p1 \;\; p2 \;\; p3} \;\; \sin\gamma_1 \sin\gamma_2 \sin\gamma_3} \tag{13}$$

$$pi = \begin{cases} \dfrac{li}{L} & li<L \\ 1 & li\geq L \end{cases}$$

Where GSI: Geological Strength Index, jC: joint condition factor jw, js and jA are the joint large-scale waviness factor, small-scale smoothness factor, and alteration factor, respectively (jA, js, jw and rating of the Joint Size Factor are taken from [27]), si, pi and γi are the joint spacing and the angle between joint sets, joint persistence, respectively. li and L are the average joint spacing and the accumulate joint spacing

There are some guidelines for the estimation of the rock mass' residual strength given by some researchers [28]. In this study, the peak shear strength of the selected geotechnical units was calculated by the method proposed by Cai et al. [14] given by the following equations.

$$GSI(Vb, jC) = \frac{26.5 + \ln 8.79\, jCr + 0.9\ln Vbr}{1 + 0.0151\ln jCr - 0.0253Vbr} \tag{15}$$

$$jCr = \left(\frac{jRr}{jAr}\right) = \left(\frac{jsr.jwr}{jAr}\right) \tag{16}$$

Where jwr, jsr and jAr are the residual joint large-scale waviness factor, residual small-scale smoothness factor, and residual alteration factor, respectively. jCr is the residual joint condition factor. jA is equal to jAr and residual block volume size is equal to 10 cm^3 [14].

Joint condition of the discontinuity, joint volumetric count, block size, Geological Strength Index of the geotechnical units selected was measured in the field (Table 4). Then, the peak and residual shear strength parameters of the geotechnical units selected were obtained using Hoek-Brown criterion (Table 4). In this study, GSI is equal to 5 when GSI<5

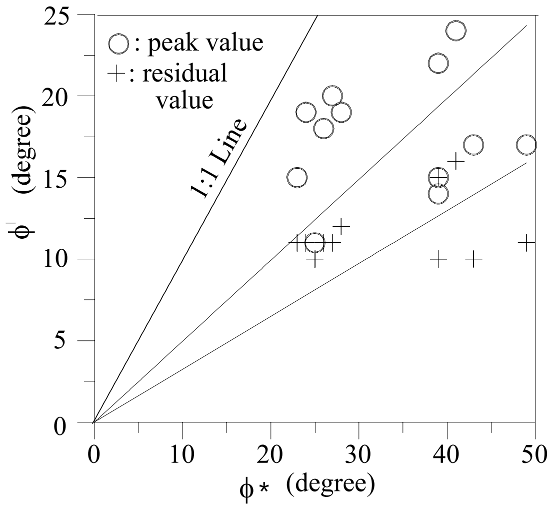
Table 4. The shear strength parameters of the geotechnical units obtained using Hoek-Brown criterion.

Rock type	Granite				Dacite				Andesite			
JTB	1	2	3	4	5	6	7	8	9	10	11	12
Jv	4,14	23,75	1,51	11,43	5,4	4,27	11,97	20,64	11,67	17,36	7,85	7,71
jA	1	3	4	3	1	6	4	8	3	4	8	8
jL	0,75	1	1	0,75	1	1	1	3	1	1	2	0,75
jR	2,5	2	2	2,5	2,5	2	3	1,5	3	2,5	3	2
jC	2,5	0,666	0,5	0,833	2,5	0,33	0,75	0,187	1	0,625	0,375	0,25
Vb	0,141	0,016	2,108	0,013	0,07	0,114	0,04	0,008	0,019	0,088	0,022	0,127
GSI	33	19	21	21	32	15	21	7	23	20	14	12
$\phi^{/}$	24	18	19	20	22	15	17	11	19	17	15	14
$c^{/}$	3,36	1,24	0,29	1,13	3,05	2,06	2,89,	0,52	1,41	2,03	0,73	1,69
jRr	1,25	1	1,25	1,25	1,25	1	1,5	0,75	1,5	1,25	1,5	1
jCr	1,25	0,333	0,313	0,417	1,25	0,167	0,375	0,094	0,5	0,312	0,187	0,125
GSIr	16	5	5	5	15	5	7	5	9	6	5	5
$\phi^{/}_{r}$	16	11	11	11	15	10	11	10	12	10	11	10
$c^{/}_{r}$	1,896	0,565	0,124	0,476	1,694	1,251	1,421	0,452	0,619	0,893	0,384	0,988

(JTB: geotechnical unit, $\phi^{/}$ and $c^{/}$: peak shear strength parameters, the angle of friction and cohesive strength, respectively, Jv: the volumetric joint count (m^{-3}), jA: the alteration factor (1: Clean Joints; Fresh rock walls, 3: Coating or thin filling*;* coating of frictional material without clay as sand, silt, calcite , 4: Coating or thin filling*; C*oating of softening and cohesive minerals such as cay, chlorite, talc, etc, 6 and 8: Filled joints with partial or no contact between the rock wall surfaces. 6 representing compacted clay materials, 8 representing soft clay materials), jR: the joint roughness factor, jL: the joint size factor (jL=2 for 0,1<L≤1m, jL=1 for 1<L≤10 m, jL=10,75 for 10<L≤30 m, jL=10,5 for L>30m, L: Joint length), jC: joint condition factor, Vb: block volume size (m^3), GSI: Geological strength Index. r is representing residual value).

4. REGRESSION ANALYSES AND COMPARISON OF PERFORMANCES OF THE PREDICTIONS

In the first stage in the regression analyses, the differences between friction angle and cohesion values of the geotechnical units determined by using Hoek-Brown criterion and SSPC system were evaluated (Figure 4). In the second stage, the relationships between friction angle and cohesion values from Hoek Brown criterion and SSPC system were obtained (Eq. 17-20). The simple regression analyses provide a means of summarizing the relationship between two variables. During the simple regression analyses, linear ($y=ax+b$), power ($y=ax^b$), through origin (y=ax), logarithmic ($y=a \ln x+b$), and exponential ($y=ae^{bx}$) functions were employed. The relationship with the highest coefficient of determination was taken into account. The number of data points used in Figure 4 is equal to 12.



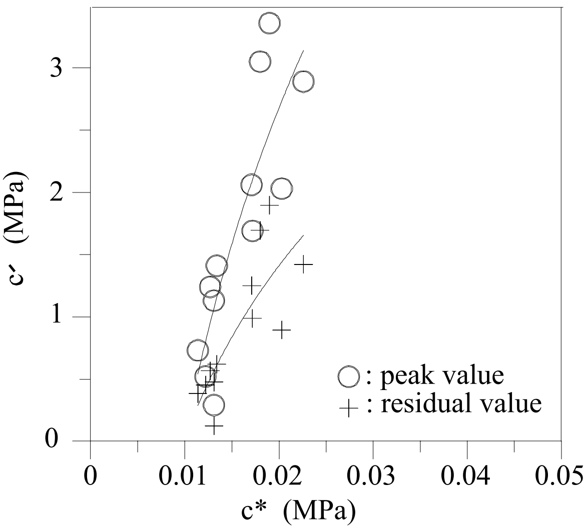


Figure 4. The relationships between the shear strength values obtained by Hoek-Brown criterion and SSPC system ($\phi^/$and $c^/$ are frictional angle and cohesion derived by Hoek-Brown criterion, respectively. $\phi^*$and c* are frictional angle and cohesion derived by SSPC system, respectively).

$$\phi^/_{peak} = 0.493\phi^* \quad (17)$$

(R^2=0.921)

$$\phi^/_{residual} = 0.325\phi^* \quad (18)$$

(R^2=0.932)

$$c^/_{peak} = 3.799\ln(c^*) + 17.54 \quad (19)$$

(R^2=0.734)

$$c^/_{residual} = 1.996\ln(c^*) + 9.22 \quad (20)$$

(R^2=0.654)

And then, in order to obtain the peak and residual shear strength given at Eq. 21-22, the relationships given at Eq. 17-20 were used.

$$\tau^/_{peak} = (3.799 \ \ln(c^*) + 17.54) + \sigma \ \tan(0.493\phi^*) \quad (21)$$

$$\tau^/_{residual} = (1.996 \ \ln(c^*) + 9.22) + \sigma \ \tan(0.325\phi^*) \quad (22)$$

In the above equations, $\phi^/$and $c^/$ are frictional angle and cohesion derived by Hoek-Brown criterion, respectively. ϕ*and c* are frictional angle and cohesion derived by SSPC system, respectively. R^2 is coefficient of determination. Coefficient of determination values that are higher than 0.64 are considered statistically significant [29]. R^2 values between 0.49 and 0.64 are not considered to be significant, but are taken to provide rough estimates of engineering properties involved in the correlation. R^2 at the 95% confidence level was determined for all available data.

In Figure 5, the relationships between the shear strength value of the geotechnical unit derived by Hoek-Brown criterion (shear strength_measured) and shear strength predicted by Eqs. 21-22 were given.

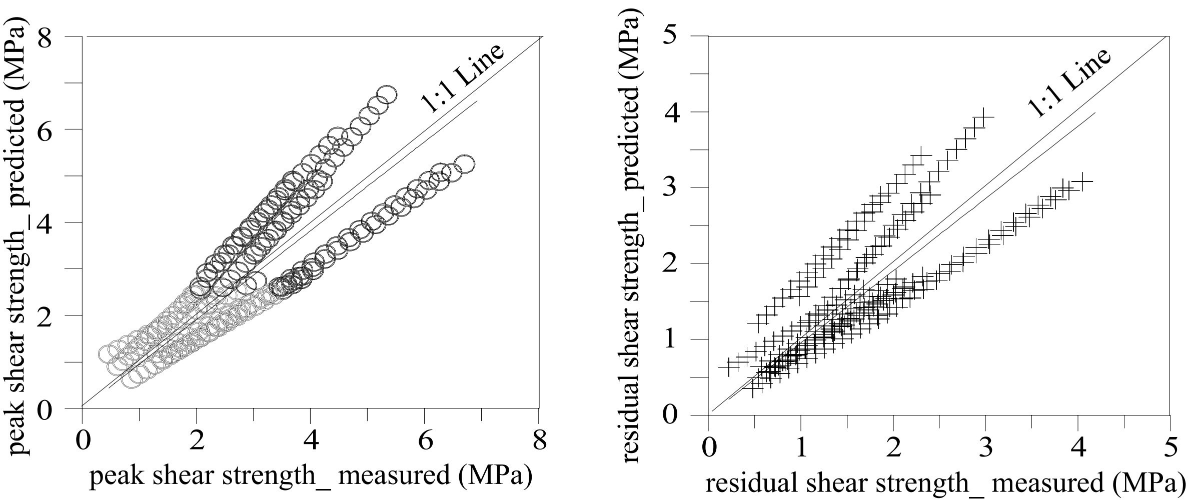


Figure 5. The relationships between the shear strength value derived by Hoek-Brown criterion (shear strength_measured) and shear strength predicted by Eqs. 21-22 .

Performance control of a prediction model is an important issue. For this reason, a series of performance analyses were carried out considering various performance coefficients such as coefficient of determination, R^2. R^2 and best-fit curves were calculated by the ''least squares curves fit'' method. In addition, variance account for (VAF) given Equation 23 and root mean square error (RMSE) given Eq.24 indices were also calculated to control the performance of the prediction capacity of the regression model given in this study as employed by Gokceoglu [30] and Gokceoglu and Zorlu [31].

$$VAF = (1 - \frac{\mathrm{var}(y - y')}{\mathrm{var}\, y}) x100 \qquad (23)$$

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(y - y')^2} \qquad (24)$$

Where y and y' are measured and predicted values, respectively. If VAF is 100 and RMSE is 0, then model will be excellent.

VAF was determined as 72%) , RMSE was determined as 0.18 for the model which is given at Eq.21. This situation showed that the performance of both models is sufficient.

5. RESULTS AND CONCLUSIONS

In this study, the peak and residual shear strength envelopes of jointed magmatic rock masses assessed using the Hoek-Brown criterion compared to the strength envelopes assessed using the Stability Probability Classification system (SSPC). The rocks masses evaluated in this study outcropped at Gümüşhane-Giresun highway,NE Turkey. Twelve geotechnical units were separated from the jointed rock masses exposed at excavated slope selected using the lithological features, the weathering state and the frequency of discontinuity. The shear strength parameters of the geotechnical units were obtained using Hoek- Brown criterion and SSPC system. Then, the relationships were investigated between the shear strength envelope of the geotechnical units estimated using Hoek-Brown criterion and SSPC system. There are meaningful differences between the values of shear strength of the units found with Hoek-Brown failure criterion and SSPC system. On the other hand, according to the results of the regression analyses, it is found that the meaningful relationships exist between the values of shear strength parameters of the geotechnical units obtained by Hoek-Brown failure criterion and SSPC system. In order to predict the peak and residual shear strength, the models were created using the meaningful relationships. Failure envelope is linear in this model. And then, the performance analyses were carried out considering various performance coefficients such as coefficient of determination, variance account for (VAF) and root mean square error (RMSE). As a result, the model proposed in this study can be used reliably in finding peak and residual values of shear strength of frequently fractured rocks.

In the evaluation of possible failure of jointed rock masses, the relationships given in this study allows the comparison of the results of the methods which are based on the Hoek-Brown criterion and the outputs of SSPC system. However, it should be underlined that the performances of the models developed in this study should also be checked using some additional data which can be obtained through in situ test and available literature.

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