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A discussion on the Hoek-Brown failure criterion and suggested modifications to the criterion verified by slope stability case studies

Hoek-Brown yenilme ölçütü üzerine bir tartışma ve şev duraylılığı vakaları ile sınanmış ölçüte ilişkin değişiklik önerileri

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ABSTRACT

Estimation of the mechanical behavior of closely jointed rock masses is one of the fundamental problems in rock mechanics since the size of representative specimens is too large for laboratory testing. Among the empirical strength criteria suggested for intact rocks and rock masses, the Hoek-Brown criterion has become highly popular. Since its introduction in 1980, the criterion has been refined and expanded over the years, particularly due to some limitations in its application to poor-very poor quality rock masses. In 199,7 the Geological Strength Index (GSI) was introduced into the criterion by its originators as a scaling parameter. In addition, some modifications to the GSI System to provide a more quantitative estimate of GSI and methods of parameter estimation have also been previously suggested by the authors of this paper in 1999. However, the authors considered that some improvements seem to be necessary in order to avoid the gap between failure envelopes of the rock masses with the GSI values between 25 and 26, intersection between failure envelopes of the rock masses of high and low strengths at a certain normal stress level, and a uniaxial compressive strength of zero when s=0 for GSI<25. In this study, some improvements to the equations providing the rock mass parameters 's' and 'a' for the criterion were proposed. Further, a modification to the quantitative GSI chart, which was adopted from the original GSI chart by the authors, was also suggested by considering intact or massive rock mass. The validity of the proposed improvements was verified by applying the criterion to a hypothetical slope and to failed open pit mine and spoil pile slope case studies. The results particularly indicated that the switch GSI value of 25 between poor and good to reasonable quality rock masses in the criterion should be replaced by 30 and the parameter 's' should not be assumed as zero for poor quality rock masses. In addition, the improvements suggested in this study were also compared with the new equations in the 2002 version of the criterion, which was published by Hoek and his co-workers after this study has been completed, in conjunction with the use of case history examples.

Key words: Disturbance factor, geological strength index, Hoek-Brown failure criterion, rock mass, slope instability, strength gap.

ÖZ

Yakın aralıklı eklemlerle bölünmülş kaya kütlelerinin mekanik davranışının tahmini, laboratuvar deneyleri için gerekli olan örnek boyutlarının büyük olması nedeniyle kaya mekaniğinin temel sorunlarından biridir. Kaya malzemeleri ve kaya kütleleri için önerilmiş olan görgül (ampirik) yenilme ölçütleri arasında yer alan Hoek-Brown ölçütü oldukça popüler olmuştur. Ölçüt, özellikle zayıf kaya kütlelerine uygulanmasıyla ilgili bazı sınırlamaları nedeniyle, ilk kez önerildiği 1980'den bu yana farklı zamanlarda tekrar düzenlenmiş ve genişletilmiştir. 1997 yılında Jeolojik Dayanım İndeksi (GSI), kaya kütlesinde ölçek etkisini dikkate alan bir parametre olarak geliştiricileri tarafından ölçüte dahil edilmiştir. Ayrıca GSI'nın daha niceliksel şekilde belirlenebilmesi ve GSI ile ilgili girdi parametrelerinin tahmininde kullanılabilecek yöntemler konusunda 1999 yılında bu makalenin yazarları tarafından mevcut GSI Sistemi'ne yönelik bazı değişiklikler önerilmiştir. Bununla birlikte yazarlar, GSI değeri 25 ve 26 olan kaya kütlelerinin ye-

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nilme zarfları arasında ortaya çıkan bir dayanım boşluğu, belirli bir normal gerilme düzeyinde yüksek ve düşük dayanımlı kaya kütlelerinin yenilme zarflarının birbirlerini kesmesi ve GSI<25 koşulunda s=0 olduğu zaman tek eksenli sıkışma dayanımının sıfır olarak elde edilmesi gibi sorunların giderilmesi gerektiğini düşünmüşlerdir. Bu çalışmada, yenilme ölçütünde 's' ve 'a' gibi kaya kütlesi parametrelerinin belirlendiği eşitliklerle ilgili bazı değişiklik önerileri yapılmıştır. Ayrıca, yazarların daha once orijinal GSI Sistemi'nden adapte ederek geliştirdikleri niceliksel GSI Sistemi de masif kaya kütlesi kavramı da dikkate alınarak yeniden düzenlenmiştir. Önerilen değişikliklerin geçerliliği; ölçüt, kuramsal bir şeve, yenilmiş açık işletme ve döküm harmanı şevlerine uygulanarak sınamıştır. Sonuçlar; özellikle zayıf ve iyi kaliteli kaya kütleleri arasındaki 25 olan sınır GSI değerinin 30 olarak alınmasının ve zayıf kaya kütleleri için 's' parametresinin sıfır olarak kabul edilmemesinin gerektiğini göstermiştir. Ayrıca, önerilen değişiklikler, şev duraysızlığı vakaları esas alınarak, bu çalışmanın tamamlanmasından sonra Hoek ve çalışma grubunca yayımlanan ve ölçütün 2002 yılına ait son versiyonuda önerilen eşitliklerle karşılaştırılmıştır.

Anahtar kelimeler: *Örselenme faktörü, jeolojik dayanım indeksi, Hoek-Brown yenilme ölçütü, kaya kütlesi, şev duraysızlığı, dayanım boşluğu.*

INTRODUCTION

Determination of the strength of jointed rock masses is difficult since the size of representative specimens is too large for laboratory testing. This restriction forced the investigators to develop practical methods, particularly empirical strength criteria, which can provide good estimate for the strength of closely jointed rock masses. Amongst the empirical strength criteria formulated both for intact rock material and rock masses, the Hoek-Brown criterion has become popular. This empirical criterion (Hoek and Brown, 1980) is widely used in conjunction with the Bieniawski's RMR System (Bieniawski, 1989) as an attempt to address the problem, and has been refined and expanded over the years (Hoek, 1983 and 1994; Hoek and Brown, 1988; Hoek et al., 1992 and 1995). Hoek and Brown (1997) proposed a new classification called Geological Strength Index (GSI), instead of RMR, due to the limitations in the RMR System for very poor quality rock masses. The GSI System based upon the visual impression on the rock mass structure has twenty codes to identify each rock mass category and estimates the GSI value ranging between 10 and 85. On the basis of the studies on the Athens schist by Hoek et al. (1998), a new rock mass category was introduced into the GSI System called "foliated/laminated rock mass structure". Hoek (1999[a]) also inserted an upper row to the GSI System to deal with "intact or massive" rock. The papers by Marinos and Hoek (2000, 2001) put more geology into the Hoek-Brown failure criterion and introduced a new GSI chart for heterogeneous weak rock masses. Marinos and Hoek (2000) also slightly changed the uppermost part of the current GSI chart.

The 1997 and the latest versions of the GSI chart are sufficient for field observations, since it is only necessary to note the code that identifies the rock mass category. It is also noted that the intention of Hoek and his co-workers was to present an approximate method for rock mass characterization using the GSI. However, due to lack of measurable and more representative parameters, and related interval limits or ratings for describing the surface conditions of the discontinuities, the GSI for each rock mass category in the chart represents a range of values. In other words, it is possible to estimate different GSI values from the chart for the same rock mass by different persons, depending on their personal experience. Therefore, an attempt has been made by Sönmez and Ulusay (1999) to provide a more quantitative numerical basis for evaluating the GSI and to suggest quantities that make more sense than that of the RMR System when used for the estimation of the rock mass strength as an additional tool. In addition, same investigators also considered that another particular issue is the use of undisturbed and disturbed rock mass categories for determining the parameters in the criterion, for which clear guidelines are lacking. Because average undisturbed in-situ conditions have been considered only to estimate the rock mass parameters in conjunction with GSI without the use of any adjustment for disturbance effect since 1997. Therefore, Sönmez and Ulusay (1999) proposed a method to assess the influence of disturbance on rock mass constants depending on the method of excavation and consequently to account for strength. Applying the criterion to slope instability case histories selected from Turkey by performing back analysis has validated these modifications to the GSI System and the suggested method. Consequently, after the 1988 version of the criterion (Hoek and Brown, 1988), the use of the disturbance factor was suggested by the authors of this paper.

The rock mass constants 's' and 'a' in the original Hoek-Brown criterion also depend empirically on the value of GSI. For the GSI values less than 25, the parameter 's' is taken as zero and the exponent 'a' becomes GSI-dependent. However, the authors of this recent paper recognized an evident gap between the strength envelopes of weak rock masses with GSI values of 25 and 26. It is also noted that the generalized equation of the failure criterion yields a uniaxial compressive strength of zero for the rock masses with GSI values less than 25. During this work, from the review of the literature on the criterion, it was clear that no consideration had been given to avoid these difficulties. In this study, therefore, an attempt is made to avoid some difficulties related to the estimation of rock mass constants, particularly for weak rock masses with GSI values varying between 5 and 30. In addition, the quantitative GSI chart previously suggested by the authors of this paper is re-arranged by considering the latest GSI charts suggested by Marinos and Hoek (2000). The improvements are then validated by applying the criterion to a hypothetical slope, and to failed mining slopes of pits and spoil piles with a blocky nature from Turkey.

This work is an original study completed in 2001. However, after the study has been completed an article, which was dealing with the deficiencies pointed out in this study, was published by Hoek et al. (2002) and presented in NARMS-TAC Joint Conference 2002 (Canada, July 2002). Although the authors of this paper were unaware of this new publication, it is surprisingly noted that there are some similarities between these two works. By considering the suggested modifications to the criterion by the authors of this paper and those suggested in the 2002 edition of the criterion were compared, and the case history examples of the authors were also reworked using the equations released by Hoek et al.(2002). The main attempts by the authors are to address some difficulties of the criterion, particularly encountered in its application to weak rock masses, to suggest a methodology to avoid them, and to provide some contributions for the performance and practical use of this strength criterion.

THE DIFFICULTIES RELATED TO THE HOEK-BROWN PARAMETERS FOR WEAK ROCK MASSES

The Hoek-Brown failure criterion has found wide practical application as a method of defining the stress conditions under which a rock mass will deform in elastically and, if not supported adequately, collapse. The parameters defining the Hoek-Brown criterion can be estimated from a combination of laboratory tests on intact rock cores and empirical adjustment to account for the reduced strength of the rock mass due to presence of discontinuities. According to Hoek and Brown (1980), the original failure criterion is given by the following parabolic law.

$$\sigma'_1 = \sigma'_3+(m\sigma'_3\ \sigma_{ci} + s\sigma_{ci}^2)^{0.5} \qquad (1)$$

Where σ'_1 and σ'_3 are the major and minor principal stresses at failure, respectively, σ_{ci} is the uniaxial compressive strength of the intact rock material, and 'm' and 's' are the dimensionless material and rock mass parameters. The most general form of the criterion, which incorporates both the original and the modified forms, is given by following equation.

$$\sigma'_1 = \sigma'_3+\sigma_{ci}(m_b \frac{s'_3}{s_{ci}} +s)^a \qquad (2)$$

Where 'm_b' is the value of constant 'm' for the rock mass and 'a' is the constant depending upon the characteristic of the rock mass. The parameter 'm_b' in Eqn.(2) depends on both the intact rock parameter 'm_i' and the GSI value, as defined by the following equation.

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right) \qquad (3)$$

The parameters 's' and 'a' also empirically depend on the GSI value as follows:
for GSI>25, i.e. rock masses of good to reasonable quality,

$$s = \exp\left(\frac{GSI - 100}{9}\right) \quad (4)$$

$$a = 0.5 \quad (5)$$

and for GSI<25, i.e. rock masses of very poor quality, the criterion applies with

$$s=0 \quad (6)$$

$$a = 0.65\left(\frac{GSI}{200}\right) \quad (7)$$

From the above equations it is clear that the rock mass strength parameters are sensitive to the GSI value. It should also be noted that in Eqns. (3) to (7), only undisturbed rock mass condition is considered. The originators of the criterion (Hoek and Brown, 1997) indicated that the choice of GSI=25 for the switch between the original and modified criteria was purely arbitrary, and it could be argued that GSI=30 provide a continuous transition in the value of 'a'. As also stated by the same investigators, extensive trials have shown that the exact location of this switch GSI value has negligible practical significance. However, a questionable issue is that the validity of this boundary value of GSI (=25) has not been confirmed yet by case studies. In this study, therefore, as a first step, the validity of this boundary value is discussed. For the purpose, strength envelopes of the rock masses with GSI values ranging from 20 to 30 are constructed by considering normal stress levels between 0 and 2 MPa for the intact rock material properties of σ_{ci}=10 MPa and m_i=10 (Figure 1). Employing the Balmer's equation (Balmer, 1952) the strength envelopes are drawn as suggested by Hoek and Brown (1997). Figure 1 suggests that the envelopes show a regular decreased pattern in terms of GSI values from 30 to 26 and from 25 to 20, while an evident strength gap occurs between the envelopes of the rock masses represented by the GSI values of 25 and 26. It is concluded that this situation probably arises from the equations employed for the estimation of 'a' (Eqns. 5 and 7) at GSI >25 and GSI<25. The effect of the strength gap appearing in Fig. 1 on stability assessments is also investigated by means of a hypothetical drained slope. The analyses are carried out by a computer program, HOBRSLP, developed and described by Sönmez et al. (1997) for eleven GSI values varying from 20 to 30. The program can handle slope stability analysis of circular and non-circular slip surfaces using Bishop's (Bishop, 1955) and Janbu's (Janbu, 1973) methods, respectively, for slopes involving many benches with different geometries, various materials and different groundwater conditions. The geometry of the slope examined, intact rock material properties and the variation of the factor of safety (FOS) with the GSI are given in Figure 2. The 'FOS-GSI' plot indicates that as the GSI regularly increases FOS also does, but a sudden jump in the FOS from 1.05 to 1.23 appears between the GSI values of 25 and 26. Therefore, this situation suggests that the criterion results in a practical question for the assessment of stability of slopes in rock masses with a boundary value of GSI=25. Hoek (1998) indicates that the GSI can be represented by normal distribution. But it is clear from Figure 2 that the values of cohesion, internal friction angle and factor of safety obtained from the stability analyses, which employ an average value of GSI of 25, may not be defined by the normal distribution.

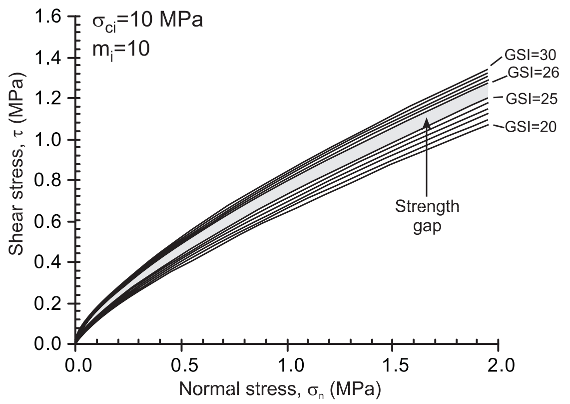


Figure 1. Failure envelopes obtained from the Hoek-Brown failure criterion for the rock masses with the GSI values between 20 and 30, and the strength gap appearing between the envelopes of the rock masses with the GSI values of 25 and 26.

Şekil 1. GSI değerleri 20 ile 30 arasında değişen kaya kütleleri için Hoek-Brown yenilme ölçütüyle belirlenmiş yenilme zarfları ile GSI=25 ve GSI=26 olan kaya kütlelerinin yenilme zarfları arasındaki dayanım boşluğu.

The Hoek-Brown criterion suggests that s=0 when GSI<25. In this study, the effect of 's' on the uniaxial compressive strength of the rock

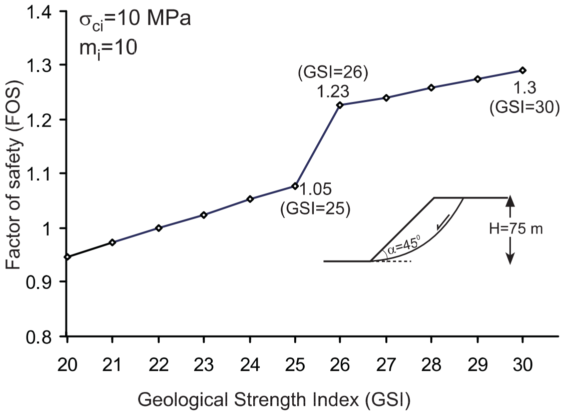


Figure 2. Variation of the factor of safety with the GSI for a hypothetical drained slope based on the assessments by the generalized Hoek-Brown criterion.

Şekil 2. Genelleştirilmiş Hoek-Brown yenilme ölçütüne göre değerlendirilmiş drene olmuş kuramsal bir şev için güvenlik katsayısının GSI'a bağlı değişimi.

mass is also investigated. By putting σ'_3=0 in Eqn.(2), the uniaxial compressive strength of the rock mass is obtained by the following expression.

$$\sigma_{cmass} = \sigma_{ci} \, (s)^a \tag{8}$$

Eqn.(8) suggests that the parameter 's' and, consequently, the uniaxial compressive strength of the rock mass suddenly drop to zero when GSI<25. Hoek (1999[b]) suggested an alternative approach for the estimation of the uniaxial compressive strength of the rock masses. In this approach, the data pairs of the normal stress and shear stress are obtained for the minor principal stress (σ'_3) levels in the range of $0<\sigma'_3<0.25\sigma_{ci}$, and these data pairs are evaluated by linear regression for the determination of c' (average cohesion) and ϕ' (average internal friction angle). The corresponding uniaxial compressive strength of the rock mass (σ_{cmass}), based on c' and ϕ' is calculated from the following expression.

$$\sigma_{cmass} = \left(\frac{2c'\cos\phi'}{1-\sin\phi'} \right) \tag{9}$$

In order to examine the validity and applicability of Eqn.(9), an example from Hoek (1999[b]) is selected. This example includes a calculation for a tangent to the Mohr's envelope defined by the criterion. The GSI of the rock mass is 45, and the intact rock material properties σ_{ci} and m_i are 85 MPa and 10, respectively. For the range of σ'_3 between 0 and 21.25 MPa (=$0.25\sigma_{ci}$), linear regression analysis yields c' = 3.27 MPa and ϕ' = 30.1^0. When these values are substituted into Eqn. (9), the uniaxial compressive strength of the rock mass is determined as 11.35 MPa. However, depending on the rock mass parameters 's' and 'a', Eqn.(8) yields a lower uniaxial compressive strength (σ_{cmass}= 4.0 MPa) for the same GSI value. The strength envelope and the Mohr's circles in terms of uniaxial compressive strength values obtained from Eqns. (8) and (9) are shown in Figure 3. The Mohr's circle representing the uniaxial compressive strength derived from Eqn. (9) intersects the strength envelope of the rock mass. This situation suggests that the use of Eqn. (9) results in considerably higher values of σ_{cmass} than those obtained from the failure criterion itself.

The empirical failure criterion suggests an expression for the ratio between the uniaxial compressive strengths of rock mass and intact rock material (Hoek and Brown, 1980). If a value of 0.5 is put into Eqn. (8) for the parameter 'a', the following expression is obtained for the rock masses with GSI values greater than 25.

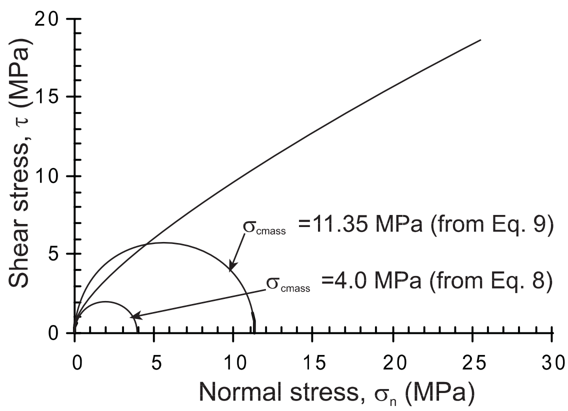


Figure 3. Comparison of the failure envelope and Mohr's circles determined from Eqs. (8) and (9) for a rock mass of moderate quality (GSI=45, σ_{ci}=85 MPa, m_i=10).

Şekil 3. Orta kalitedeki bir kaya kütlesi (GSI=45, σ_{ci}=85 MPa, m_i=10) için Eşitlik 8 ve 9'dan elde edilen yenilme zarfları ve Mohr dairelerinin karşılaştırması.

$$\left(\frac{\sigma_{cmass}}{\sigma_{ci}}\right) = \sqrt{s} \tag{10}$$

If Eqn. (4) given for 's' is put into Eqn.(10), the resulting expression based on the GSI for '$\sigma_{cmass}/\sigma_{ci}$' ratio is as follows.

$$\sqrt{s} = \sqrt{e^{\left(\frac{GSI-100}{9}\right)}} = e^{\left(\frac{-100}{18}\right)} e^{\left(\frac{GSI}{18}\right)}$$

$$\text{or} \quad \left(\frac{\sigma_{cmass}}{\sigma_{ci}}\right) = 0.003866e^{0.0555GSI} \tag{11}$$

On the other hand, Hoek (1999[b]) suggests that the ratio of the uniaxial compressive strengths in the field and the laboratory can be estimated from the following equation, which involves the GSI.

$$\left(\frac{\sigma_{cmass}}{\sigma_{ci}}\right) = 0.022e^{0.038GSI} \tag{12}$$

Eqn. (12) can also be re-written in the following form as suggested by Gerçek (2001):

$$\left(\frac{\sigma_{cmass}}{\sigma_{ci}}\right) = e^{-3.81671} e^{\left(\frac{GSI}{26.31579}\right)} = e^{\left(\frac{GSI-100.44}{26.31579}\right)} \approx \exp\left(\frac{GSI-100}{26}\right) \tag{13a}$$

Square root of the right hand side of the above equation represents 's' which can be given as follow.

$$s = \exp\left(\frac{GSI-100}{13}\right) \tag{13b}$$

The contradiction between Eqns. (4) and (13) indicates the presence of difficulties in the estimation of the uniaxial compressive strength of the rock mass.

For GSI<25, Eqn. (7) suggests 'a' values ranging between 0.525 and 0.65. The values of 'a' approaching to 0.65 result in a decrease in the curvature of the failure envelope. However, another difficulty, which arises from the current equation for 'a', suggested by the criterion (Eqn. 7) for GSI<25, is the intersection of the strength envelopes of the rock masses with the GSI values below and above 25 at a certain normal stress level. This difficulty is demonstrated by some examples in this study. While such a problem is not expected at normal stress levels between 0 and 2 MPa as can be seen in Figure 1, crossing between envelopes of different rock masses occurs at higher normal stresses (Figure 4). The example in Figure 4 suggests that the failure criterion yields slightly higher strength values for very poor quality rock masses with GSI values of 22 and 24 than those of stronger rock masses (GSI = 26 and 28) at certain normal stress levels. The effect of σ_{ci} on this problem is also investigated for σ_{ci} values of 4 and 100 MPa for two rock masses (GSI=24 and 26) having the same m_i (=10) value. From Figures.5a and 5b, it is evident that as σ_{ci} increases, intersection of the strength envelopes occurs at higher normal stresses. From the comparison of the intersection points shown in Figures 4 and 5c, that illustrate the strength envelopes of two rock masses with same σ_{ci} but different m_i constants, it is concluded that a decrease in m_i results in intersections at lower normal stress levels. Although a minute jump in 'a' from 0.5 to 0.525 results in intersection of the strength envelopes and strength differences of a few percent only (see Fig. 4), deviations between the envelopes at higher normal stress levels beyond the intersection points tend to show a considerable increase, particularly for the rock masses with lower uniaxial compressive strengths and lower m_i values (see Figures. 5a and 5c). In other words, although the effect of intersection of the strength envelopes on rock mass strength is small, this situation seems to be opposite to the nature of the criterion. On the other hand, the stress levels around deep underground openings are considerably high when compared to those at the surface excavations. On the basis of the above discussions, if the problem related to the intersection of the strength envelopes of the poor quality rock masses with high intact rock material strength is taken into consideration, it seems that the boundary condition defined by GSI=25 will also result in incorrect assessments of the rock mass parameters for deep underground excavations.

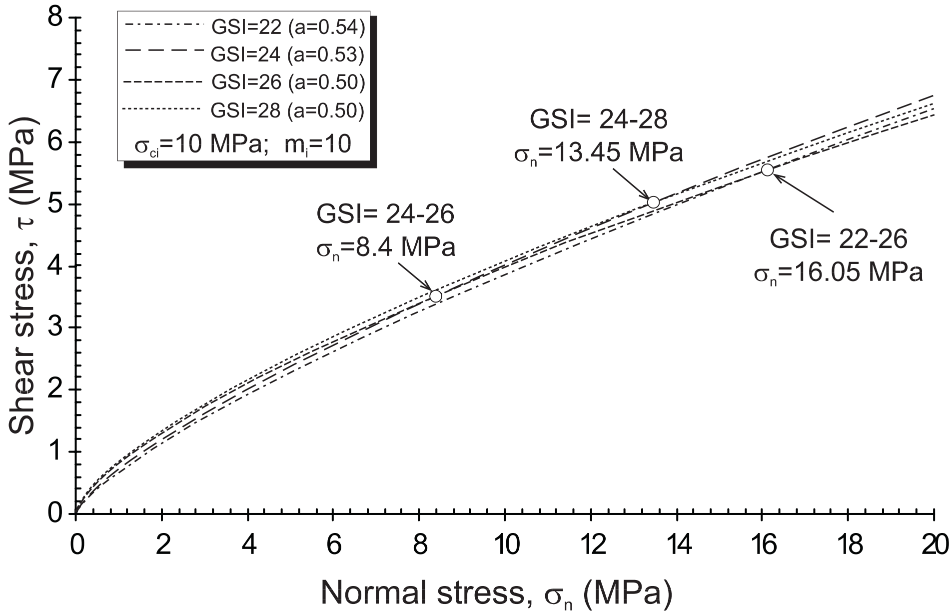


Figure 4. Intersection of the failure envelopes derived from the Hoek-Brown criterion for a poor quality rock mass (GSI=22-28, σ_{ci}=10 MPa, m_i=10).

Şekil 4. Zayıf bir kaya kütlesi (GSI=22-28, σ_{ci}=10 MPa, m_i=10) için Hoek-Brown yenilme ölçütünden elde edilmiş yenilme zarflarının kesişmesi.

The above-mentioned conclusions suggest four difficulties arising from the generalized Hoek-Brown failure criterion. These are as follows.

(a) When the input parameters of the criterion are kept constant, a strength gap occurs between the GSI values of 25 and 26. This situation calls an improvement to the equations employed for the estimation of ‘a’.

(b) For GSI<25, the generalized equation of the criterion suggests a questionable concept that the rock mass has an uniaxial compressive strength of zero. This problem occurs when zero is assigned to the parameter ‘s’ for the rock masses of very poor quality (i.e. GSI<25). It is, therefore, clear that an improvement to the equations suggested by the criterion for the parameter ‘s’ seems to be necessary for avoiding this problem. It is also noted that another alternative proposed by Hoek (1999[b]) for the estimation of the uniaxial compressive strength of the rock mass results in values greater than those obtained from the original criterion.

(c) The strength envelopes of the rock masses with GSI values smaller and greater than 25 intersect each other at certain normal stress levels. In other words, the criterion, in its existing form, suggests that a weaker rock mass may possess higher strength than that of a stronger one when the normal stresses corresponding to the intersections are exceeded. Because it is possible to encounter with such normal stress levels in engineering practice, the boundary value of 25 assigned to the GSI, which governs the use of the equations of the rock mass parameters, should be re-considered.

(d) As mentioned in the Introduction of this paper, some modifications to the GSI System have been suggested by Sönmez and Ulusay (1999) to provide a more quantitative basis for evaluating the GSI values as an additional tool. However, the quantitative GSI chart does not include “intact or massive” rock category. Therefore, the authors of this recent study considered that the introduction of this category of rock mass into the quantitative GSI System would be useful. For the purpose, the previously suggested ratings and their intervals by the authors in 1999 and the the recent GSI chart suggested by Marinos and Hoek(2000) were employed.

The following paragraphs include the modifications to the quantitative GSI chart and improvements to the rock mass constants. The approac-

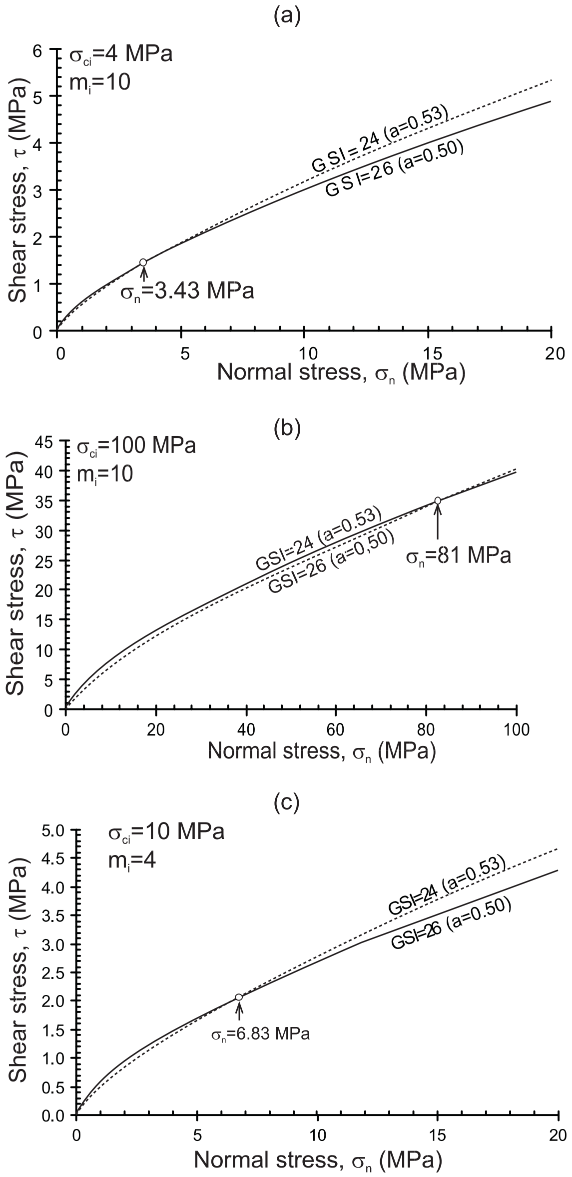


Figure 5. (a) and (b) variation of the intersections of the failure envelopes with normal stress and uniaxial compressive strength of the intact rock, and (c) an example illustrating the effect of intact rock parameter m_i on intersection of the failure envelopes of two different rock masses.

Şekil 5. (a) ve (b) normal gerilmeye ve kaya malzemesinin tek eksenli şıkışma dayanımına bağlı olarak yenilme zarflarının kesişme noktalarının değişimi ve (c) kaya malzemesi sabiti m_i'nin iki farklı kaya kütlesinin yenilme zarfının kesişmesi üzerindeki etkisi.

hes suggested to check the validity of these and the methodology of parameter estimation by employing well studied slope instability case histories in rock masses are also presented. In addition, the results obtained from the improvements in this study and those from the use of the 2002 edition of the criterion are compared using the case history examples of the authors.

MODIFICATIONS TO THE QUANTITATIVE GSI CHART

Due to lack of the parameters to describe surface conditions of the discontinuities and the rock mass structure in the GSI System, two terms namely, 'structure rating, SR' based on volumetric joint count (J_v) and 'surface condition rating, SCR', estimated from the input parameters (e.g., roughness, weathering and infilling) were suggested by Sönmez and Ulusay (1999). A new rock mass category to accommodate thinly foliated or laminated, folded and predominantly sheared weak rock of non-blocky structure proposed by Hoek et al. (1998) has not been included into this quantitative modified GSI System. The quantitative GSI System, and associated approaches and input parameters for its construction are given by Sönmez and Ulusay(1999) in detail, and therefore, they are not repeated herein. However, it is concluded by the authors that a top row on 'intact or massive' rock which was included into the original GSI chart by Hoek(1999[a]) and then slightly modified by Marinos and Hoek (2000) is also considered necessary to be introduced into the quantitative GSI System in order to provide a complete GSI rating ranging between 5 and 100, and the upper boundary of the GSI should theoretically be 100. Based on the structure and surface condition ratings, the modified quantitative GSI System proposed by the authors is transformed into a modified form including five rock mass categories (Figure 6). For the modification, the latest boundaries for structure and surface conditions and the lines for the GSI ratings defined by Marinos and Hoek (2000) are taken into consideration. However, due to the reasons explained by Sönmez and Ulusay (1999) laminated/sheared category rock mass is not included into the system.

Based on the intervals of J_v and corresponding descriptions for the blockiness ratings, structure rating (SR) was assigned to each category by the procedure suggested by Sönmez and Ulusay (1999). However, in this study, since the bo-

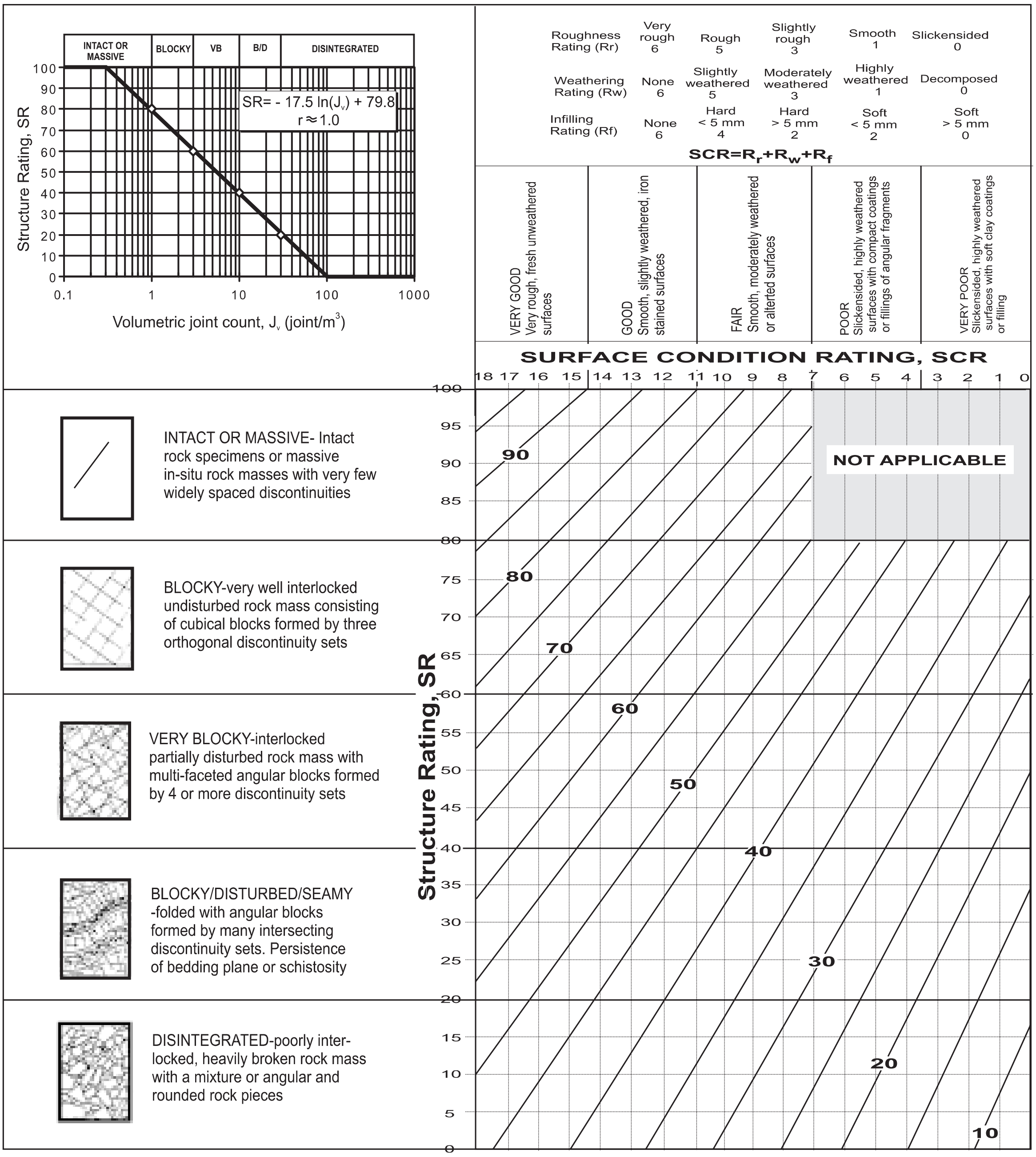


Figure 6. The modified quantitative GSI System suggested in this study.
Şekil 6. Bu çalışmada önerilen modifiye edilmiş niceliksel GSI Sistemi.

undaries between the structural categories in the 1999 quantitative GSI chart are equally divided, the SR limits between five rock mass groups are selected as 80, 60, 40 and 20, respectively. The relationship given in Figure 6 between these SR limits and corresponding J_v values (1, 3, 10 and 30 joints/m^3) is obtained. The relationship or the plot of J_v-SR given in the left margin of Figure 6 can be used to assign a rating for SR of any rock mass using the value of J_v. For the upper and lower bounds of SR, the corresponding J_v values are 0.3 and 100 joints/m^3, respectively. J_v is estimated by one of the expressions given in the paper by Sönmez and Ulusay (1999).

In addition to heavily jointed rock masses consisting of small rock pieces, some spoil piles composed of a mixture of angular and rounded rock pieces free from high proportion of fines caused by hauling, dumping and subsequent deformation may possess similarities to heavily jointed, crushed and disturbed rock masses. The authors consider that categorizing such materials as disintegrated rock masses in the GSI classification seems to be possible. But it is impossible to estimate the number of discontinuity sets in spoil piles and consequently the equations previously suggested by the authors in 1999 cannot yield realistic J_v values for such medium. In order to overcome this difficulty, it is considered to be logical and more practical counting the faces of individual rock pieces involved by the spoil piles. For the purpose, by assuming that parallel or nearly parallel surfaces represent the same discontinuity set, such parallel surfaces should be counted once. In the case of a rock piece from a rock mass including three joint sets approximately perpendicular to each others, prismatic blocks with six surfaces are formed and if the parallel surfaces are considered from a single discontinuity set the number of discontinuity sets (D_n) is estimated as 3. While in the case of a tetrahedral rock pieces of which surfaces are not parallel to each others, the number of discontinuity sets is considered as 4. Assuming that heavily jointed rock masses and spoil piles are homogenous and isotropic, the following expression is suggested for the approximate estimation of J_v in conjunction with D_n.

$$J_v = D_n\left(\frac{1}{S}\right) \tag{14}$$

Where, D_n is the estimated number of discontinuity sets as mentioned above and S is the average size of the block or rock pieces, which represents average spacing of discontinuities and estimated from the selected pieces of the rock mass or spoil pile.

CONSIDERATION ON THE VALIDITY OF THE ROCK MASS CONSTANTS AND SUGGESTED MODIFICATIONS

Considerations on the Use of the Parameter 's'

Once the GSI has been estimated, 's' is calculated from Eqn. (4) or taken as zero, depending on the conditions of GSI>25 and GSI<25 conditions, respectively. At this boundary,GSI=25, Eqn. (4) yields $s=2.4\times10^{-4}$. For a rock mass with $s=2.4\times10^{-4}$ and intact rock material properties of σ_{ci}=100 MPa and m_i=10, the uniaxial tensile and compressive strengths are calculated as 0.035 MPa and 1.55 MPa, from Eqn. (2) by putting $\sigma'_1=0$ and $\sigma'_3=0$, respectively. These results suggest that a rock mass with the GSI value of 25 has a uniaxial tensile strength 2.2 % of its uniaxial compressive strength. Therefore, it can be concluded that when 's' approaches to zero, depending on the GSI values less than 25, the uniaxial tensile strength of the rock mass will be become nil. But it does not seem to be correct to assume this approach.

The effect of the parameter 's' on σ'_1 in the generalized equation of the criterion (Eqn. 2) is also so examined. For the purpose, the values for 's', σ_{ci} and m_i given in the previous paragraph are employed in Eqn. (2), and variation in the major principal stress σ'_1 as percentage in terms of s=0 and $s=2.4\times10^{-4}$ conditions is calculated for a range of σ'_1 and σ'_3 values. Figure 7a suggests that the increase in σ'_1 at σ'_3=0.2 MPa is 8 %. While this increase at σ'_3=1 MPa is only about 1 %, when the parameter 's' is taken 2.4×10^{-4} instead of zero for GSI=25. Considerable variations occurring in σ'_1 arise from the criterion itself which yields zero value for σ'_1 when $\sigma'_3=0$ and s=0. Therefore, asymptotes appear for the curves shown in Figures 7a and 7c at $\sigma'_3=0$ condition. On the other hand, σ'_1 values obtained from Eqn. (2) for s=0 and $s=2\times10^{-4}$ are compared in Figures 7b and 7d. These figures reveal that there is no margin of error as indicated by the plots of data on 1:1 line.

If the intact rock material strength (σ_{ci}) considerably decreases from 100 MPa to 10 MPa, the variation in σ'_1 depending on σ'_3 becomes very low (about 0.8 %), even σ'_3 is very low (0.2 MPa) (see Figure 7c). This situation indicates that the effect of s=0 condition on the estimation of rock mass strength will be less than 1 % at normal stress levels commonly encountered in rock engineering applications. On the other hand, as indicated by Hoek et al. (1992), most rock mechanics engineers consider that the type of jointed rock mass, to which the Hoek-Brown failure criterion applies, should have zero tensile strength. For the past 30 years, finite element

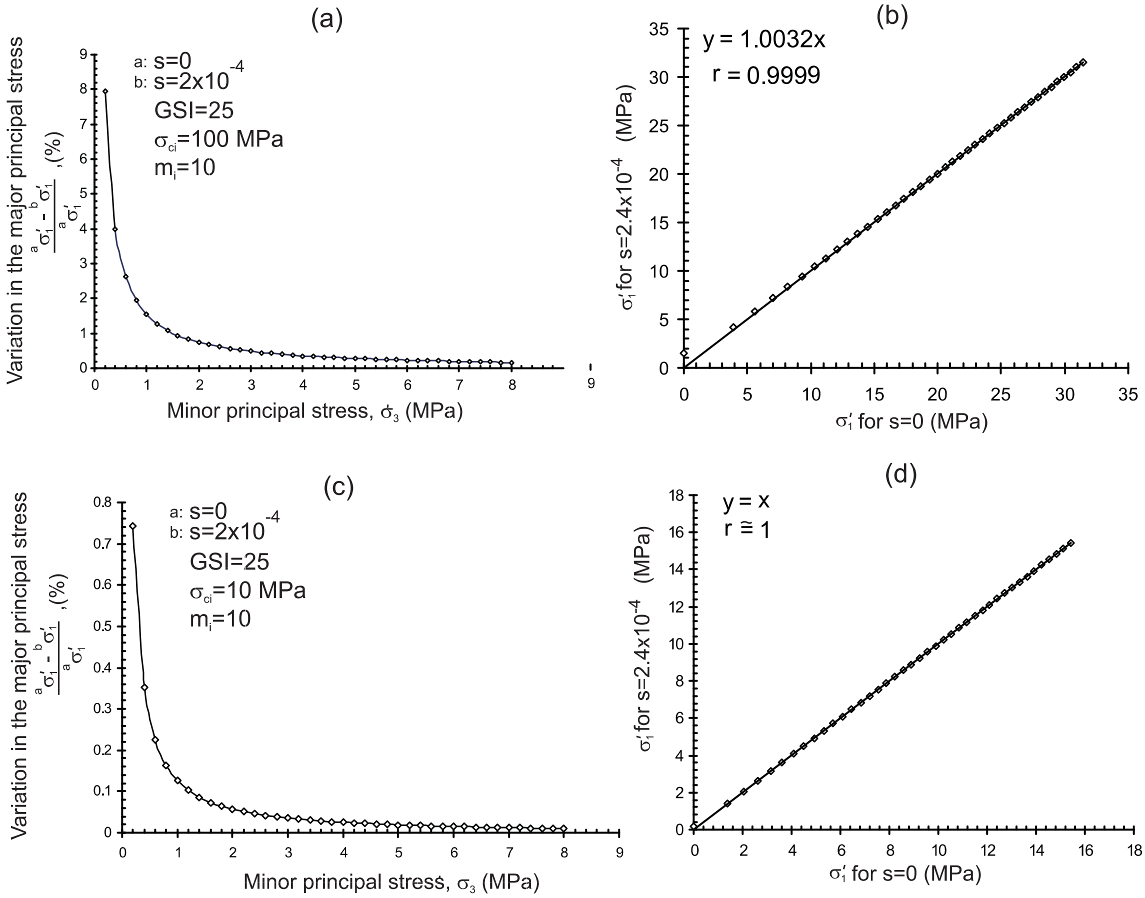


Figure 7. Variation in the major principal stress in terms of s=0 and s=2.4x10^{-4} with minor principal stress (GSI=25).
Şekil 7. s=0 ve s=2.4x10^{-4} koşulları için en büyük asal gerilmenin en küçük asal gerilmeye bağlı değişimi.

numerical models for use in rock mechanics have included a 'no tension' option, which allows tensile stresses developed in the model to be transferred onto adjacent elements. Thus, the originators of the criterion considered this as a deficiency and modified the criterion. But the authors of the recent study have some suspicions about this approach, because s=0 condition at GSI<25 suggests a uniaxial compressive strength of zero for the rock mass. Therefore, s=0 condition to obtain a tensile strength of zero for the numerical models should be avoided. The authors believe that this problem can be avoided, if a single equation is used for the estimation of 's' regardless of a switch GSI value (i.e. GS<25). For this purpose, the following equations of which derivation were given in detail by Sönmez and Ulusay (1999) are suggested.

$$s = \exp\left(\frac{GSI-100}{b_s}\right) \tag{15}$$

$$b_s = 0.67\ln\left(\frac{d_f}{d_f+340(1-d_f)}\right)+9 \tag{16}$$

Where, d_f is disturbance factor depending on the method of excavation, and b_s is a unitless parameter ranging between 6 and 9 which also appear in the denominator of the equations of the parameter 's' in the 1988 version of the criterion (Hoek and Brown, 1988). It should be taken into account that the values between 5 and 100 (see Figure 6) can be assigned to the GSI in Eqn. (15) without any consideration on GSI<25 condition, and d_f values ranging between 0.8 and 1 depending on the degree of disturbance can be selected from the literature dealing with rock mass disturbance (Kendorski et al., 1983; Laubscher, 1990) depending on the method of excavation, presence of major planes of weakness or change in stress.

Considerations on the Use of the Parameter 'a'

As discussed in the previous paragraphs, a strength gap occurs between the GSI values of 25 and 26 due to the effect of corresponding 'a' values to the GSI. In order to avoid this problem, four approaches for the estimation of 'a' with the use of the modified quantitative GSI chart in Figure 6 have been considered and applied to the failed slopes from Turkey. The authors previously investigated five slope instability cases for the modifications to the GSI (Sönmez and Ulusay, 1999). In this recent study, among five cases, only three cases with GSI values less than 30 are employed to investigate the effects of the switch at GSI=30. Two cases selected from the failures of open pit slopes in heavily jointed rock masses with GSI values 16.5 and 29 and one from a failure occurred in spoil piles (GSI=27) were examined. Input parameters used for the estimation of the GSI and instability conditions in these cases are briefly outlined in the following paragraphs. The details of the cases with the slope and failure surface geometries can be provided from Sönmez and Ulusay (1999) and the associated references cited below.

The first case is a slope failure at Başkoyak (Şarkikarağaç) barite open pit mine in a schist rock mass. Ulusay and Yücel (1989) carried out a comprehensive slope stability project to determine the rock mass parameters and to assess alternative measures for the improvement of overall stability in this pit. The scan-line surveys and geotechnical logging indicated that the rock mass consisted of heavily broken schist by closely spaced discontinuities and schistosity planes, but not sheared or foliated. Characteristics of the discontinuity surfaces and their average spacing are tabulated in Table 1. Due to the heavily jointed nature of the schist, the rock mass was categorized as homogeneous and isotropic (disintegrated rock mass) with an average discontinuity spacing of 0.04 m in all directions. Since the overburden material and the ore are removed by excavators without any blasting, an adjustment factor of 0.97 according to Kendorski et al.(1983) was considered. A slope failure through a single bench was selected to study in more detail. The failure was in circular form and no sign of groundwater was encountered through the boreholes and on bench faces.

The second case investigated occurred as a local bench failure in the eastern slope of the Himmetoğlu (Göynük-Bolu) coal mine located north-

Table 1. The parameters employed in the GSI classification for the case examples considered in this study.
Çizelge 1. Bu çalışmada yararlanılan örnek vakalar için GSI sınıflamasında kullanılan parametreler.

Parameters	Case 1	Case 2	Case 3
Spacing [(a)] (m)	$S_{average}$ =0.04	S_1=0.37, S_2=0.65 S_b=0.11	$S_{average}$=0.083 [(d)]
Condition of discontinuities and ratings	Smooth to slickensided surfaces (1), highly weathered (1), soft coating<5 mm (2)	Slickensided surfaces (0), moderately weathered (3), soft coating<5 mm (2)	Same as in Case 1
SCR	4	5	8
J_v (joint/m^3)	75	13.3	72.3
SR	4.2	34.5	4.9
GSI [(b)]	**16.5**	**29**	**27**
d_f [(c)]	0.97	0.97	0.80
m_i	7	9.87	9.87
σ_{ci} (MPa)	5.2	4.8	4.15

[a] True spacing (S_1, S_2, S_3 for joints, S_b for bedding planes)
[b] GSI determined from the modified chart in Fig. 6 (this study)
[c] Adjustment factor for disturbance effect
[d] Estimated by the method of photoanalysis along x, y and z axes (Sönmez and Ulusay, 1999)
(Case 1: Başkoyak barite pit; Case 2: Göynük lignite open pit; Case 3: Spoil pile instability at Eskihisar open pit)

west Anatolia. The failure was due to steepening of the slope in a heavily jointed rock mass. Site investigations (Ulusay et al., 1998) revealed a mode of failure by combination of a planar sliding surface along the weak floor strata and circular failure through the rock mass. Because the upper part of the sliding surface passed through the jointed marly rock mass in the form of circular sliding, this case was considered to be suitable for the purpose of the study. Scan-line surveys along the benches of the pit revealed a heavily jointed rock mass with discontinuity characteristics given in Table 1. Based on these, input parameters and the GSI of the rock mass were estimated (see Table 1). Detailed hydrogeological investigations suggested that the slope was drained. Since the excavators remove the overburden without blasting, a disturbance factor of 0.97 was considered for this case. The back analysis of the multiplanar failures along both bedding planes and faults in the other parts of the pit indicated that the residual shear strength of the weak and slickensided bedding planes (c_r=1.4 kPa and $\phi_r=12^0$) has mobilized at the time of failure (Ulusay et al., 1998). Therefore, residual shear strength of the bedding planes was utilized for the structurally controlled part of the failure surface through the back analysis, while rock mass parameters were considered for its upper part.

The third case history example is from the Eskihisar (Yatağan-Muğla) strip coal mine where spoil piles suffer from numerous stability problems as reported by Ulusay et al. (1995[a], 1995[b], 1996) The selected spoil pile instability occurred near the haul road and consisted of heavily broken angular and rounded marly rock pieces with small amount of fines. Average block size of the material was estimated with the aid of photo-analysis technique and statistical methods (Sönmez and Ulusay, 1999). On the basis of the estimated number of natural discontinuity sets and average block size, a J_v value of 72.3 was obtained from Eqn. 14. Parameters of the rock mass and intact rock material with the GSI are given in Table 1. It was a shallow-seated instability occurred in a spoil pile with an in-place unit weight of 14 kN/m^3. The cross-sections prepared from the instability plan revealed that the circular failure did not involve the foundation material. No water table or seepage was encountered in the pile.

As a first step, the circular slope failure in a closely jointed schist rock mass (Case 1) with a GSI value less than 25 (GSI=16.5) was back analyzed. The rock mass and intact rock material properties tabulated in Table 1 were employed in the analyses, and ‘s’ is calculated as 1.19x10^{-5} from Eqns. (15) and (16). Stability analysis of the failure surface was performed for different ‘a’ values varying between 0.55 and 0.58. The variation of the factor of safety (FOS) with ‘a’ is illustrated in Figure 8a. This figure suggests an ‘a’ value of 0.5765 corresponding to a FOS satisfying limiting equilibrium condition.

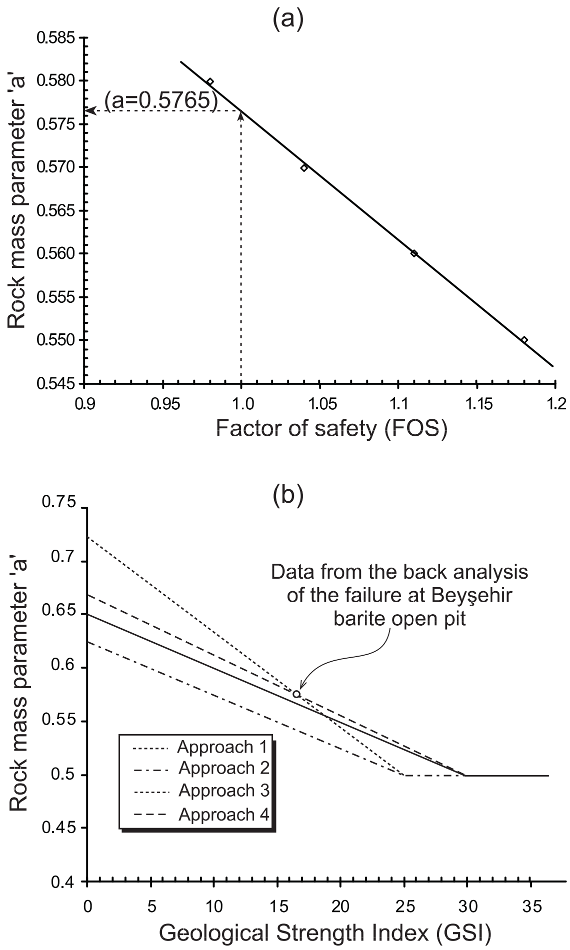


Figure 8. (a) variation in ‘a’ with factor of safety (FOS) for the slope instability at Başkoyak barite open pit (Case 1), and (b) the relationship between ‘a’ and the GSI for four approaches considered in this study.

Şekil 8. (a) Başkoyak barit açık işletmesindeki (vaka no.1) şev duraysızlığı için güvenlik katsayısına bağlı olarak ‘a’ parametresindeki değişim ve (b) bu çalışmada dikkate alınan dört yaklaşım için ‘a’ ve GSI arasındaki ilişki.

The following approaches are employed to investigate a possible improvement for the use of the equations given for ‘a’:

(a) Approach 1: from the linear relationship established between the data pairs of a=0.5 and GSI=25, and a=0.5765 and GSI=16.5, the following expression is obtained for further assessments.
$$a = 0.725 - 0.009\ GSI \quad (17)$$
(b) Approach 2: the following equation suggested by Gerçek (1996) to obtain a=0.5 for GSI=25 is considered.
$$a = (125 – GSI)/200 \quad \text{or} \quad a= 0.625-0.005\ GSI \quad (18)$$
(c) Approach 3: Considering that Eqn. (7) yields a=0.5 for GSI=30, it is assumed that Eqn. (7) will be valid for GSI<30 .
(d) Approach 4: The following expression, obtained from the linear relationship between the data pairs of a=0.5 and GSI=30, and a=0.5765 and GSI=16.5, is considered as an alternative approach.
$$a = 0.67 – 0.0057\ GSI \quad (19)$$

The values of ‘a’ obtained from these approaches are plotted against GSI (Figure 8b). Figure 8b indicates that approaches 3 and 4 suggest ‘a’ values very close to each others, while ‘a’ values derived from approach 3 (original equation of the criterion) are higher than those obtained from approach 2, about 0.025. Approaches 1 and 4 result in identical ‘a’ values only at GSI=16.5 which is taken as the common GSI value by these approaches.

Four approaches were assessed by the back analysis of the above-mentioned three slope instabilities. In the analyses, the quantitative GSI chart (see Figure 6) and Eqns. (15) and (16) for the estimation of ‘s’ are employed.

The results obtained from the back analysis of the slope failures by considering four approaches are given in Table 2. Except the FOS (= 1.23) obtained from the approach 2 for the schist rock mass (barite open pit, GSI=16.5), the values of FOS around unity were obtained for all cases from all approaches. Due to this, it was considered that the back analysis of the instability occurred in the schist rock mass could not be an efficient tool to check the validity of these three approaches.

The use of the first approach results in lower strength values than those obtained from the equations by the approaches 3 and 4 for the rock masses with GSI<16.5. On the contrary, it yields higher strengths for GSI values between 16.5 and 25. For an undisturbed rock mass with intact rock material properties of σ_{ci}=100 MPa and m_i=10 by considering $s\neq0$, the approach 1 is used to compare with the generalized form of the criterion for assessing the variation in shear strength with normal stress for different GSI values. Figure 9a suggests that high shear strength estimations reaching up to 30 % for the GSI values between 16.5 and 25 are possible at low normal stress levels when the approach 1 is employed for the estimation of ‘a’, while the decrease in shear strength is about 30 % for GSI<16.5.

Although the expressions employed for the parameter ‘a’ by the approaches 3 and 4 are quite similar, Eqn. (18) considered in approach 4 is based on only a single back analysis. Therefore, the use of Eqn. (21) for GSI<30, instead of GSI<25, seems to be realistic in order to avoid the strength gap appearing between the GSI values of 25 and 26. The assessments carried out

Table 2. The results of the back analysis of the failed slopes based on different approaches.
Çizelge 2. Duraysızlığa uğramış şevler için farklı yaklaşımlar esas alınarak yapılmış geriye dönük analizlerin sonuçları.

Case No.		GSI	Factor of safety (FOS)			
			Approach 1	Approach 2	Approach 3	Approach 4
1. Beyşehir barite pit		16.5	1.01	1.23	1.06	1.01
2. Himmetoğlu lignite pit		29	1.01	1.01	1.00	1.00
3. Spoil	Section (1-1)	27	1.02	1.02	0.94	0.93
pile	Section (2-2)		0.99	0.99	0.91	0.90
(Eskihisar)	Section (3-3)		1.00	1.00	0.92	0.91
	Section (4-4)		1.02	1.02	0.93	0.92

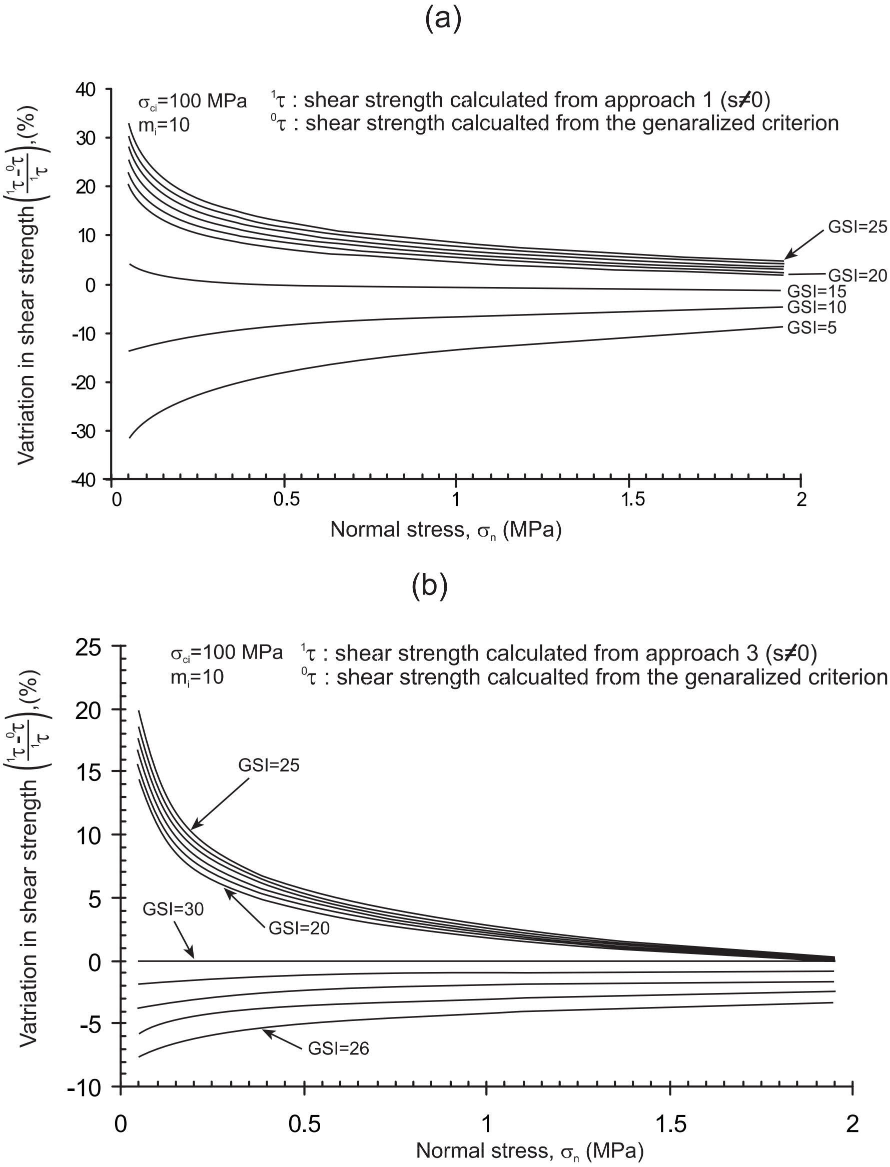


Figure 9. Variation in shear stress with normal stress based on the calculations from approaches 1 (a) and 3 (b), and from the generalized criterion for different of the GSI values.

Şekil 9. (a) 1 ve (b) 3 numaralı yaklaşımlar ile ölçütün genelleştirilmiş versiyonundan farklı GSI değerleri için yapılan hesaplamalara göre makaslama gerilmesinin normal gerilmeye bağlı değişimi.

for the rock masses with different GSI values by considering the intact rock material properties given in Figure 9b indicate that the use of the approach 3 ($s\neq0$ condition) for GSI<30 results in values of rock mass strength only 5 % lower than those obtained from the criterion. It should also be taken into consideration that this difference will decrease at high normal stresses. It is also noted that approach 3 yields FOS values equal to 1, and around 1 for the cases 2 and 3, respectively (see Table 2).

Validity of the Proposed Improvements to the Hoek-Brown Estimates

In this study, as a first attempt the quantitative GSI chart previously modified by the authors of this recent paper was re-arranged. In addition, it was suggested that the parameter 's' should be estimated by the expressions proposed by Sönmez and Ulusay (1999) by considering the disturbance effect, regardless of the boundaries for GSI>25 and GSI<25 conditions. 30 for the estimation of the parameter ‘a’ should also replace the switch GSI value of 25. In order to check the validity of these improvements to the Hoek-Brown estimates of the rock mass, ‘normal stress-shear stress’ plots in terms of different GSI values shown in Figures 1 and 4 and stability of the hypothetical slope (see Figure 2) were re-evaluated.

Figure 10a shows the failure envelopes of the rock masses with GSI values varying from 20 to 35. It is evident from Figure 10a that the strength gap appearing in Figure 1 does not occur by the application of the suggested modifications. Similarly, intersection of the failure envelopes of the rock masses with GSI values of 29 and 30 (selected as an example for the suggested switch GSI value of 30 for ‘a’) also disappears when the suggested modifications are taken into consideration (Figure 10b). The variation of the FOS with the GSI obtained from the analysis of a hypothetical slope (see Fig. 2) by employing the estimates of the original criterion and by those suggested the authors of this study is shown in Figure 10c. It is evident that the use of the improvements results in a regular increase in FOS without any jump as the GSI increases.

COMPARISON OF THE SUGGESTED IMPROVEMENTS WITH THE 2002 EDITION OF THE CRITERION

After this study has been completed another work, i.e. the 2002 edition of the criterion, which was dealing with the deficiencies pointed out by the authors of this paper, was presented by Hoek et al.(2002) in NARMS-TAC Joint Conference held in Canada in July, 2002. Although the authors of this paper were unaware of this new publication, it is surprisingly noted that there are some similarities between the approaches suggested by these two works. On the other hand, the reasons behind the equations given in the 2002 edition of the criterion have not been clearly explained and a guidelines for estimating the disturbance factor, which shows some similarities with that suggested by Sönmez and Ulusay, 1999, was also introduced into the criterion. By considering this situation, the suggested modifications to the criterion in this study and those suggested in its 2002 edition were compared, and the case history examples of the

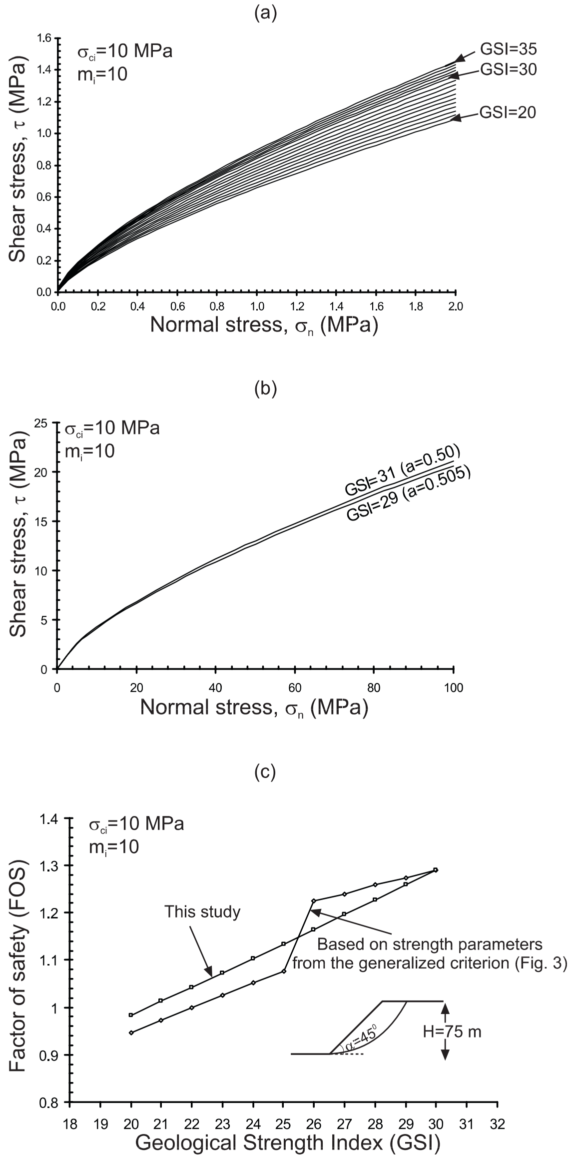


Figure 10. Applications of the improvements suggested in this study to the Hoek-Brown criterion: (a) failure envelopes for the GSI values between 20 and 35 without any strength gap, (b) failure envelopes of two rock masses constructed by assuming a switch GSI value of 30, and (c) comparison of the ‘FOS-GSI’ plots derived from this study and the generalized criterion for the hypothetical slope shown in Figure 2.

Şekil 10. Bu çalışmada Hoek-Brown ölçütü için yapılan önerilerle ilgili uygulamalar: (a) dayanım boşluğu olmaksızın, GSI’nın 20 ile 30 arasında değişen değerleri için yenilme zarfları, (b) iki kaya kütlesi için GSI=30 eşik değerine göre çizilmiş yenilme zarfları ve (c) Şekil 2’de verilen kuramsal şev için bu çalışmada yapılan önerilerden ve ölçütün genelleştirilmiş versiyonundan elde edilen FOS-GSI grafiklerinin karşılaştırılması.

authors were also re-worked using the equations released by Hoek et al. (2002).

In the 2002 edition of the criterion the "switch" at GSI=25 (Hoek and Brown, 1997) for 's' and 'a' has been eliminated as can be seen from Eqns. (20) and (21) which give smooth continuous transitions for the entire range of GSI values.

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

$$s = \exp\left(\frac{GSI-100}{9-3D}\right) \quad (21)$$

The parameter m_b, which is a reduced value of the material constant m_i, is given by the following equation in the 2002 edition of the criterion. Eqn. (21) becomes valid for all GSI values between 0 and 100, and consequently prevents the decrease of 's' estimated at GSI=25 to s=0.

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \quad (22)$$

D in Eqns. (21) and (22) is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. This factor is similar to b_m and b_s previously suggested by Sönmez and Ulusay (1999) and varies from 0 for undisturbed in-situ rock masses to 1 for very disturbed rock masses. Hoek et al. (2002) provide guidelines for the selection of D.

Based on the improvements by the authors of this study and Hoek et al. (2002), variation of the constant 'a' with GSI is compared in Figure 11. The "GSI vs. a" relationships in Figure 11 suggest that the modifications by these two different works show a clear similarity. The maximum difference in 'a' values between both modifications is 4 % at GSI = 30 which was selected as a switch value in this study. Although Hoek et al.

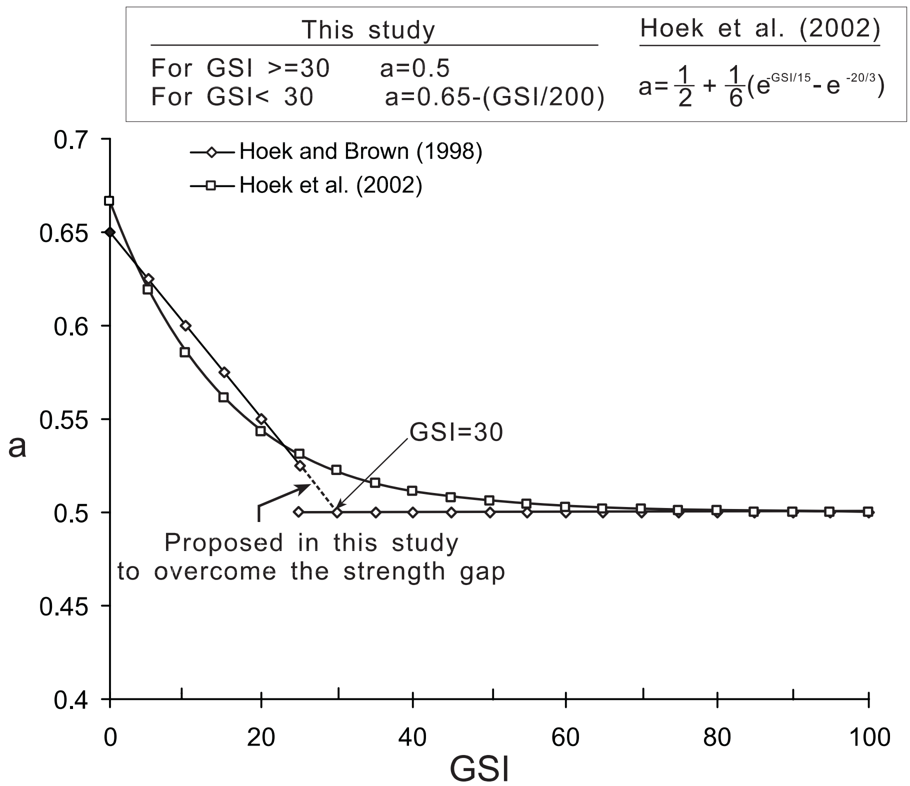


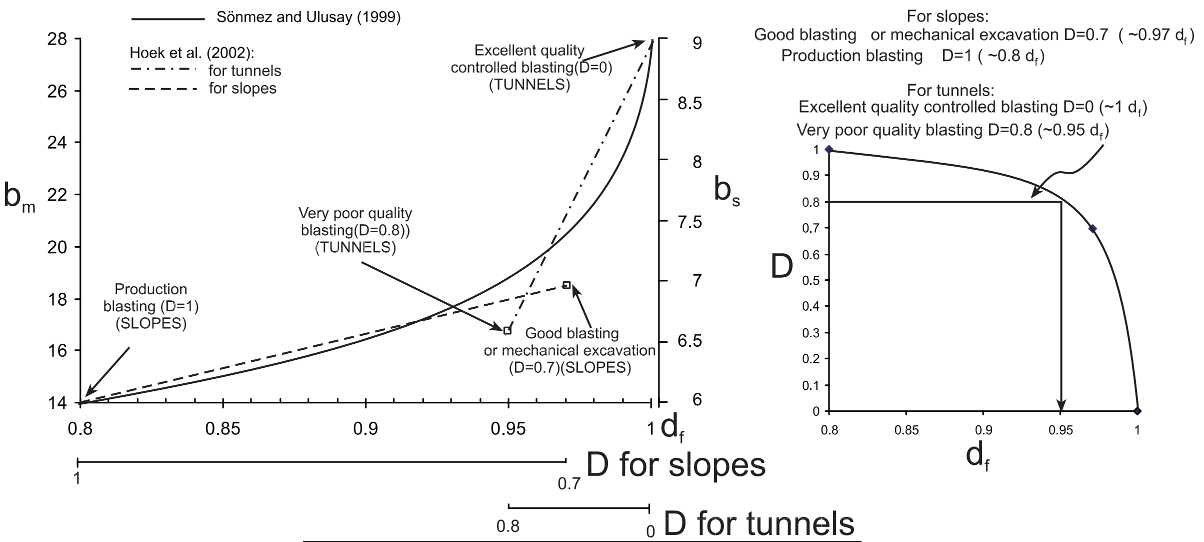
Figure11. Comparison of the variation of 'a' with GSI, based on the improvements in this study and the equations the 2002 edition of the criterion.

Şekil 11. Bu çalışmada yapılan öneriler ve ölçütün 2002 versiyonunda verilen eşitlikler esas alınarak belirlenen 'a' değerlerinin GSI ile değişiminin karşılaştırılması.

(2002) indicated that re-examination of the equation for 'a' was necessary particularly for slope stability assessments, they do not indicate the presence of the strength gap emphasized by the authors of this study, and their suggestions have not been validated by case history examples. They also state that numerical values of 'a' and 's', given by Eqns. (20) and (21), respectively, are very close to those given by the equations of the previous edition of the criterion and it is not necessary to revisit and make corrections to old calculations. Based on these new modifications, it is difficult to understand the reasons behind this statement.

The relationships between b_m-d_f and b_s-d_f suggested by Sönmez and Ulusay (1999) with the disturbance factor D proposed by Hoek et al (2002) are shown in Figure 12a. Denominators of the Eqns. (21) and (22), which include D, are also given in Figure 12a in terms of b_m and b_s. It is clear from Figure 12a that the disturbance factor D is defined by two straight lines and shows an approximation to d_f-b_m-b_s curve drawn by using the equations suggested by Sönmez and Ulusay (1999). However, D is not defined as a continuous function in the equations of s and m_s and given only by the upper and lower bounds. These bounds suggest D=0 (b_m=28; b_s=9) and D=0.8 (b_m=16.8; b_s=6.6) for tunnels while D=0.7 (b_m=18.1; b_s=6.9) and D=1 (b_m=14; b_s=6) for slopes. Therefore, assignment of intermediate D values for blasting conditions as described by Kendorski et al. (1983) seems to be difficult from the guidelines proposed in the 2002 edition of the criterion. On the contrary to

(a) (b)



Sönmez and Ulusay (1999)

$$m_b = m_i \exp\left(\frac{GSI-100}{b_m}\right) \qquad s = \exp\left(\frac{GSI-100}{b_s}\right)$$

Sönmez and Ulusay (1999)	Hoek et al. (2002)
$b_m = 3.14\ln\left(\frac{d_f}{d_f+340(1-d_f)}\right)+28$	$b_m = 28-14D$
$b_s = 0.67\ln\left(\frac{d_f}{d_f+340(1-d_f)}\right)+9$	$b_s = 9-3D$

Figure12. (a) Comparison of the disturbance factors suggested by Sönmez and Ulusay (1999) and Hoek et al.(2002), and (b) d_f-D relationships.

Şekil 12. (a) Sönmez ve Ulusay (1999) ile Hoek vd. (2002) tarafından önerilen örselenme faktörlerinin karşılaştırması ve (b) d_f-D ilişkisi.

this, the concept of disturbance factor d_f suggested by the authors in 1999 considers the effects of different types of disturbance with a continuous relationship between d_f and b_m-b_s and is based on the descriptions for different disturbance effects by Kendorski et al. (1983) for this purpose. The relationship between D and d_f is shown in Figure 12b.

The case history examples considered in this study were re-worked using the equations released by Hoek et al. (2002) to compare the FOS values from stability analyses of the case study examples considered. For the purpose, approach 3, which is the most suitable approach to avoid the strength gap, is selected. In addition to three case examples examined in this study, two cases with GSI>30 are also considered for comparison. These cases involve the instabilities of open pit coal mines and the rock masses composed of marl have failed along the circular failure surfaces (Case 4: Kısrakdere open pit coal mine - GSI=37, and Case 5 : Eskihisar strip coal mine - GSI=43). Because the rock mass characteristics and failure conditions of these two cases are given by Sönmez and Ulusay (1999) in detail, these are not outlined herein. Input parameters and the values of FOS calculated for five case history examples based on the suggestions of the authors and those by Hoek et al (2002) are given and compared in Table 3. In addition, deviations of the calculated FOS values from FOS=1 (i.e. limiting equilibrium condition) for each case are also compared in Figure 13 in the form of a histogram. It is evident from Figure 13 that deviations of the calculated values of FOS, based on the equations in Hoek et al. (2002) from FOS=1 condition are greater than those in this study and range between 7.5 % and 17.4 %. While the deviations in FOS calculated considering the equations suggested in this study are very low and range between 0.1 % and 8.5 %. This comparison suggests that FOS values calculated using the equations in the 2002 edition of the criterion seem to be slightly underestimated. This situation is resulted from the differences, which reach up to 4 %, between 'a' values from this study and the 2002 edition of the criterion (see Figure 12a). It is also noted that, as can be seen from Figure 11, this effect or difference can be ignorable for GSI values greater than 50; while for the rock masses with GSI<40, it becomes important. However, introducing the disturbance factor, D, into the 2002 edition of the criterion, which has similarities to d_f suggested by Sönmez and Ulusay (1999), results in a reduction in this difference.

CONCLUSIONS

The GSI classification scheme, in its existing form, is sufficient as a means of obtaining a first estimate of rock mass properties and highly practical. However, it leads to rough estimates of the GSI values particularly by practitioners who are not well experienced. Another particular issue is the use of undisturbed and disturbed rock mass categories for determining the parameters in the criterion for which more clear guidelines are lacking. Due to this, a few years ago some modifications were suggested to the GSI System of the Hoek-Brown criterion by the authors of this present paper in order to provide more quantitative basis for evaluating the GSI values. In this study, by preserving the original form of the latest GSI chart, the quantitative GSI chart has been re-arranged by introducing a top row on "intact or massive rock", and by changing the rating intervals. It is also noted that the aim of this modification is to provide an additional tool for the practitioners and to estimate GSI values, which are based on SCR (surface condition rating) and SR (structure rating) values determined at various locations, rather than to make the system more complicated.

The other issues concluded in this study were the difficulties arising from the assumptions made by the criterion for the rock mass parameters 'a' and 's' depending on a switch GSI value (=25). One of these difficulties is a strength gap appearing between the failure envelopes of the rock masses with the GSI values of 25 and 26. On the other hand, the generalized equation of the criterion suggests a uniaxial compressive strength of zero when s=0 for GSI<25. It was also shown that the strength envelopes of the rock masses with GSI < 25 intersect those of the rock masses with GSI > 25 at certain normal stress levels. To overcome these difficulties in the generalized Hoek-Brown criterion, some approaches and improvements to the estimates of rock mass parameters have been suggested. The validity of the proposed improvements has been checked by the application of them to different rock masses, and to hypothetical and re-

Table 3. Comparison of the results of the stability analyses based on the improvements suggested in this study and by Hoek et al. (2002) considering approach 3.

Çizelge 3. 3 numaralı yaklaşım esas alınarak bu çalışmada ve Hoek vd. (2002) tarafından yapılan önerilere göre şev duraylılığı analizlerinden elde edilen sonuçların karşılaştırılması.

Location	This study	Hoek et al. (2002)
	d_f=0.97 (mechanical excv.)	D=0.7 (mechanical excv.)
	FOS=**1.062**	FOS=**0.925**
Beyşehir barite mine (Case 1)	b_m=20.3	b_m=18.2
	b_s=7.36	b_s=6.9
	m_b=0.1151	m_b=0.07122
	s=1.1877x10^{-5}	s=5.551x10^{-6}
	a=0.5675	a=0.5553
	d_f=0.97 (mechanical excv.)	D=0.7 (mechanical excv.)
	FOS=**0.998**	FOS=**0.891**
Göynük open pit coal mine (Case 2)	b_m=20.3	b_m=18.2
	b_s=7.36	b_s=6.9
	m_b=0.1151	m_b=0.1996
	s=1.1877x10^{-5}	s=3.3976x10^{-5}
	a=0.505	a=0.5239
	d_f=0.8 (lower bound value)	D=1 (upper bound value)
	FOS=**0.941**	FOS=**0.870**
Spoil pile – Eskihisar strip	b_m=14.01	b_m=14
coal mine (Case 3)	b_s=6.02	b_s=6
(Section 1-1)	m_b=5.3943x10^{-2}	m_b=5.3676x10^{-2}
	s=5.3675x10^{-2}	s=5.201x10^{-6}
	a=0.515	a=0.5273
Spoil pile	FOS=**0.905**	FOS=**0.834**
(Section 2-2)	Same as above	Same as above
Spoil pile	FOS=**0.915**	FOS=**0.845**
(Section 3-3)	Same as above	Same as above
Spoil pile	FOS=**0.930**	FOS=**0.857**
(Section 4-4)	Same as above	Same above
	d_f=0.9 (modarete blasting)	D=0.93 (from Figure 12b)
	FOS=**0.977**	FOS=**0.852**
Kısrakdere open pit coal mine	b_m=16.51	b_m=14.9
(Case 4)	b_s=6.55	b_s=6.21
	m_b=0.19925	m_b=0.1348
	s=6.6425x10^{-5}	s=3.9275x10^{-5}
	a=0.5	a=0.5139
	d_f=0.94 (controlled blasting)	D=0.3 (from Figure 12b)
Eskihisar strip coal mine	FOS=**1.001**	FOS=**0.859**
(Case 5) (Section 1-1/1)	b_m=18.2	b_m=16.38
	b_s=6.91	b_s=6.51
	m_b=0.43035	m_b=0.30411
	s=2.60904x10^{-4}	s=1.5755x10^{-4}
	a=0.5	a=0.50927
Eskihisar strip coal mine	FOS=**0.977**	FOS=**0.838**
(Case 5) (Section 1-1/2)	Same as above	Same as above
Eskihisar strip coal mine	FOS=**0.963**	FOS=**0.826**
(Case 5) (Section 2/2)	Same as above	Same as above

al slope cases. These applications indicated that the parameter 's' should be estimated from the suggested equation which considers the effect of disturbance on the rock mass and the GSI values varying between 5 and 100, regardless of any switch value for the GSI. It was also validated that the boundary of GSI<25 suggested by the criterion should be replaced by GSI<30 to overcome the difficulties related to the estimation of the parameter 'a'.

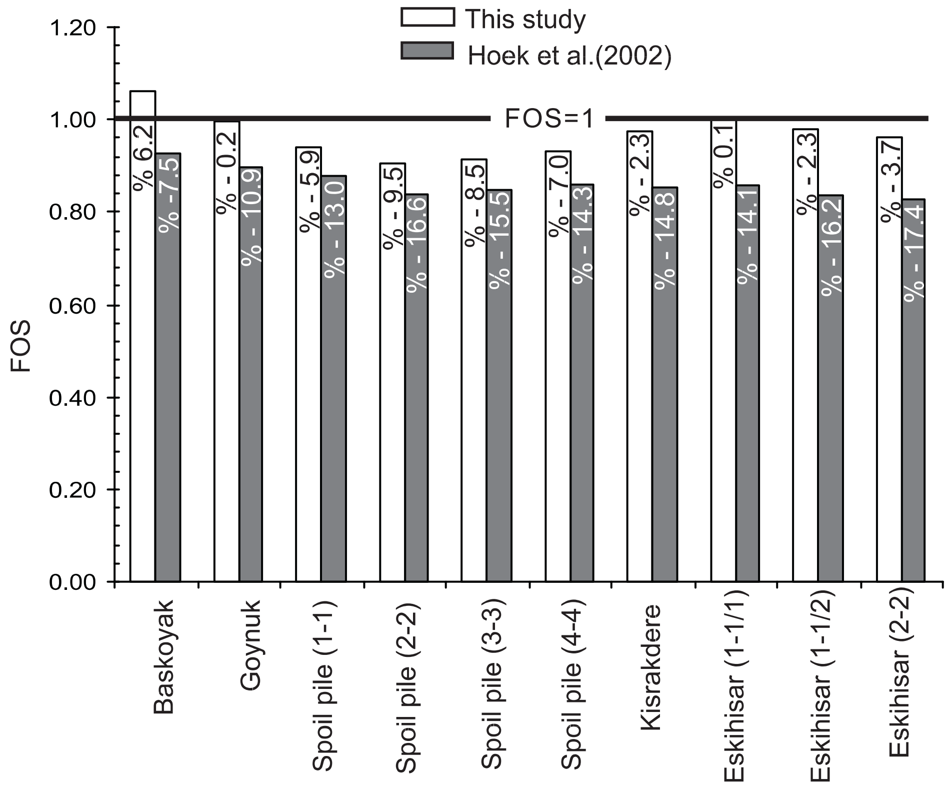


Figure 13. Histograms showing the deviation of the factor of safety values, which were calculated using the equations suggested in this study and by Hoek et al.(2002), from the limiting equilibrium condition.

Şekil 13. Bu çalışmada ve Hoek vd. (2002) tarafından önerilen eşitlikler kullanılarak hesaplanmış güvenlik katsayılarının limit denge koşuluna göre farklılığını gösteren histogramlar.

The improvements to the criterion suggested in this study were also compared to the new equations in the 2002 edition of the criterion. This comparison suggests that the modifications in both studies are similar and introducing the disturbance factor into the criterion results in more realistic conclusions as validated by case history examples. However, the disturbance factor, d_f, suggested by Sönmez and Ulusay (1999) provides the consideration of various disturbance conditions when compared to the guidelines suggested for D in the 2002 edition of the criterion. It is also noted that, as validated by the instability case examples, the use of the equations in the 2002 edition of the criterion yields slightly underestimated factor of safety values for the slopes in rock masses with the GSI<50.

One of the cases employed in this study is from spoil piles, which have properties similar to those of disintegrated rocks masses. The authors do not state that the Hoek-Brown criterion can be applied to all kinds of spoil piles. It is just a single case, which involves heavily broken rock masses with a mixture of angular and rounded rock pieces with very low proportion of fines. Therefore, it is suggested that the validation of this application should be tested on similar spoil pile materials.

A better understanding of the mechanics of the jointed rock mass behavior is a problem of major significance in rock engineering. The authors believe that the Hoek-Brown failure criterion provides a good estimate for the shear strength of closely jointed rock masses and even for spoil piles mainly consisting of rock material. However, the attempt by the authors is to address the discussion of the Hoek-Brown estimates of the rock mass and some of their limitations and to suggest some improvements to the criterion. It is hoped that the application of the recently suggested improvements in this study and by Hoek et al.(2002) to various failure case histories, from both surface and underground excavations, may lead to provide a better tool for more precise guidelines and to check further validity of the equations employed by the non-linear failure criterion.

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