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An Experimental Study to Determine the Hoek-Brown Constant (m_i) for Tuffs

Tüflerin Hoek-Brown Sabitinin (m_i) Belirlenmesi için Bir Deneysel Çalışma

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ABSTRACT

Nonlinear strength criteria are handy tools for determining the whole range of rock material behavior such as tensile, unconfined compressive, confined compressive and brittle-ductile transition. Hoek-Brown (H-B) failure criterion is one of the most widely used strength criteria for the intact rock material and m_i constant is an important parameter of the H-B criterion. The m_i constant in the H-B criterion can be determined from reference tables, triaxial compression tests and empirical studies for different rock materials. However, experimental studies for determining the m_i constant for tuffs are limited. On this context, a series of triaxial compressive strength tests were carried out on two different tuff samples under varying confining pressures. Using the least squares regression, H-B criterion parameters were computed. As a result, it was seen that computed m_i constants were lower than the tabulated constants and unconfined compressive strength of the tuffs were overestimated by the H-B equation. Furthermore, post-failure images of the samples indicated that failure angle increased with increasing confining pressure.

Keywords: Rock mechanics, Hoek-Brown failure criterion, triaxial compression test, confining pressure, m_i constant.

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ÖZ

Lineer olmayan yenilme ölçütleri, kaya malzemelerinin çekme, basma, yanal basınç altında basma ve kırılgan-sünek geçişi gibi tüm aşamalarını temsil etme noktasında kullanışlı araçlardır. Hoek-Brown (H-B) yenilme ölçütü sağlam kaya malzemesi için önerilmiş olan en önemli ölçütlerden birisidir ve mi sabiti ölçütün önemli bir parçasıdır. Hoek-Brown yenilme ölçütünde farklı kaya malzemeleri için m_i sabiti çizelgede önerilen değerlerden, üç eksenli basınç deneylerinden ya da görgül çalışmalardan belirlenebilmektedir. Ancak tüflerin m_i sabitinin belirlenebilmesi için yapılan deneysel çalışmalar sınırlıdır. Bu amaçla, iki farklı türde tüf örnekleri üzerinde farklı yanal basınçlar kullanılarak bir seri üç eksenli basınç dayanımı deneyleri gerçekleştirilmiştir. En küçük kareler regresyon yöntemini kullanarak H-B yenilme ölçütünün parametreleri tespit edilmiştir. Sonuç olarak, tespit edilen m_i değerlerinin tabloda verilen değerden daha düşük olduğu belirlenmiştir. Ayrıca, tek eksenli basınç dayanımı değerinin H-B denklemi ile gerçek değerden daha yüksek olarak hesaplandığı görülmüştür. Buna ek olarak, yenilme sonrası numune görüntüleri, yenilme açısının artan yanal basınç ile arttığını göstermiştir.

Anahtar Kelimeler: *Kaya mekaniği, Hoek-Brown yenilme ölçütü, üç eksenli basınç deneyi, yanal basınç, m_i sabiti.*

INTRODUCTION

Intact rock strength and failure characteristics have utmost significance on basic understanding about characteristics of rock materials and rock masses (Hoek and Martin, 2014). Besides, it is a difficult and costly work to define pre- and post-failure behavior of rocks. Moreover, it is a well-known fact that rock materials have delicacy to experimental conditions like the confining pressure such that the failure mode (brittle or ductile) of rock samples might change due to increasing confining pressure (Byerlee, 1968). Numerous strength criteria have been proposed so far to describe the rock material behavior under complex stress conditions which can be grouped under theoretical (Griffith, 1921) and empirical (Hoek and Brown, 1980) strength criteria, and these criteria are well-reviewed by different authors (Lade, 1993; Andreev, 1995; Lakirouhani and Hasanzadehshooiili, 2011).

Among these criteria, Mohr-Coulomb (M-C) and Hoek-Brown (H-B) strength criteria are most widely used. The M-C strength criterion is a linear strength criterion which defines the shear strength of the material with the internal friction angle and the cohesion interpreted through a series of triaxial tests (Labuz and Zang, 2012). Besides, the H-B strength criterion is a widely used nonlinear criterion which is generated using the triaxial compressive strength test data and describes the major principal stress (σ_1) with the minor principal stress (σ_3), unconfined compressive strength of the intact rock (σ_{ci}) and the m_i constant. The m_i constant has gained a significant amount of attention from the rock engineering society as a material property and it can be determined using series of triaxial tests, reference tables and empirical models (Hoek, 2007; Aladejare and Wang, 2019; Zuo and Shen, 2020). The m_i parameter might be taken from proposed tables to establish the H-B equation however, direct triaxial testing is the most reliable source for the determination m_i. Hoek (2007) provided m_i values for different rock types, however there is a paucity of the experimental data for some rock materials such as tuffs. This paper intended to supply experimentally found m_i values for two different tuffs. Additionally, variations of the failure angle with increasing confining pressure were investigated.

On this context, a series of triaxial compressive strength tests were carried out on two types of tuffs to determine m_i values of these rock materials. Different confining pressures (σ_3 =3, 6 ,9, 12 and 15 MPa) were employed during the study and two replications were realized for each confining pressure level. Least squares regression method was utilized to determine the m_i constants of the tuff samples and their H-B equations. Determined m_i values were compared with the tabulated m_i values and previous models. Additionally, post-failure images of the samples were used to examine the effect of the confining pressure on the failure angle.

HOEK-BROWN (H-B) FAILURE CRITERION AND m_i CONSTANT

Mohr-Coulomb is one of the most widely used strength criteria for describing the behavior of friction materials such as rocks. However, due to overestimation of the tensile strength and being unable to define the strength of rocks under high confining pressures, nonlinear strength criteria are much more precise to estimate the rock material behavior and strength. Hoek-Brown failure criterion was inspired by the nonlinear character of the Griffith fracture theory (Hoek and Brown, 2019). Murrell (1958) has expanded the Griffith theory to the rock materials and Hoek (1965) modified the Griffith theory to the closed cracks for application for compressive confining pressures. Afterwards, Hoek and Brown (1980) proposed the H-B criterion for intact rock using the triaxial test data and defined the major principal stress for a given minor principal stress as given in Eq. 1. H-B failure criterion, at the same time, is used in conjunction with the Geological Strength Index (Hoek, 1994) to evaluate the strength and behavior of rock masses (Sönmez and Ulusay, 2002; Hoek and Brown, 2019).

$$\sigma_1 = \sigma_3 + \sigma_{ci}\sqrt{m_i \frac{\sigma_3}{\sigma_{ci}} + 1} \qquad (1)$$

where σ_1 is the major principal stress, σ_3 is the minor principal stress, σ_{ci} unconfined compressive strength of the intact rock and m_i is the H-B material constant. m_i in the Eq. 1 is a fundamental intact rock parameter to estimate the strength and behavior. Larger m_i values are generally associated with brittle behavior while lower values indicate ductile failure (Hoek, 1983). Hoek (1983) demonstrated how a Mohr envelope of a rock sample varies due to variations in the m_i constant which can be seen from Figure 1.

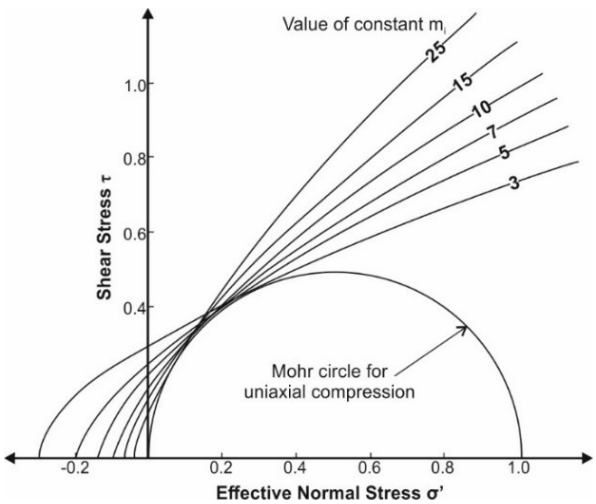


Figure 1. Effect of m_i value on shape of the Mohr failure envelope (Hoek, 1983).
Şekil 1. m_i değerinin Mohr zarfının şekline olan etkisi (Hoek, 1983).

Figure 1 demonstrates the importance of correct assumption of the m_i constant which can result in overestimation or underestimation of the strength of rock mass or rock material. This situation matters by means of the project safety and economics. The m_i constant, on the other hand, is widely used in numerical modelling tools (e.g. Phase, FLAC, UDEC and Plaxis) as a rock material property.

In the absence of triaxial testing equipment, which is an expensive and time-consuming experimental procedure, researchers rely on reference tables and empirical models. The latest version of the reference table was proposed by Hoek (2007) along with a lithological classification which can be seen in Table 1. Note that the values in parenthesis are approximations.

Another method for estimation of the m_i parameter is the empirical models. The ratio of the uniaxial compressive strength to the tensile strength (R) was used as an estimator for m_i by different researchers (Hoek and Brown, 1980; Mostyn and Douglas, 2000; Cai, 2010; Sari, 2010; Richards and Read, 2011; Read and Richards, 2014). On the other hand, a number of researchers utilized the uniaxial compressive strength and the tensile strength to estimate the m_i constant (Shen and Karakus, 2014; Vásárhelyi et al. 2016; Arshadnejad and Nick, 2016; Wang and Shen, 2017). Among these, Arshadnejad and Nick (2016) proposed a universal plot with compiling the previously published data and the equation given below is asserted to be used for igneous rocks:

$$m_i = e^{[1.2(\frac{\sigma_{ci}-2\sigma_t}{\sigma_t})^{0.3}]} \quad (R^2 = 0.96) \qquad (2)$$

where m_i is the H-B material constant, σ_{ci} is the uniaxial compressive strength of intact rock and σ_t is the tensile strength of intact rock. Zuo et al. (2008), on the other hand, stated that the m_i parameter has a physical meaning. Afterwards, Hoek and Martin (2014) and Hoek and Brown (2019) showed that m_i is an indicator for rock brittleness (R) which is defined with the σ_{ci}/σ_t ratio and they suggested several plots. Additionally, it was proposed that the higher m_i values result in smaller plastic zones in underground excavations (Cai et al., 2007).

Table 1. m_i values for intact rock (Hoek, 2007).

Çizelge 1. Sağlam kaya malzemesi için m_i değerleri (Hoek, 2007).

Rock type	Class	Group	Texture			
			Coarse	**Medium**	**Fine**	**Very fine**
SEDIMENTARY	Clastic		Conglomerates (21±3)	Sandstones 17±4	Siltstones 7±2	Claystone 4±2
			Breccias (19±5)		Greywackes 18±3	Shales (6±2)
						Marls (7±2)
	Non-clastic	Carbonates	Crystalline limestone (12±3)	Sparitic limestone (10±2)	Micritic limestone (9±2)	Dolomites (9±3)
		Evaporites		Gypsum 8±2	Anhydrite 12±2	
		Organic				Chalk 7±2
METAMORPHIC	Non-foliated		Marble 9±3	Hornfels (19±4)	Quartzites 20±3	
					Metasandstones (19±3)	
	Slightly foliated		Migmatite 29±3	Amphibolites 26±6		
	Foliated		Gneiss 28±5	Schists 12±3	Phyllites (7±3)	Slates 7±4
IGNEOUS	Plutonic	Light	Granite 32±3	Diorite 25±5		
			Granodiorite (29±3)			
		Dark	Gabbro 27±3	Dolerite (16±5)		
			Norite 20±5			
	Hypabyssal		Porphyries (20±5)		Diabase (15±5)	Peridotite (25±5)
	Volcanic	Lava		Rhyolite (25±5)	Dacite (25±3)	Obsidian (19±3)
				Andesite 25±5	Basalt 25±5)	
		Pyroclastic	Agglomerate (19±3)	Breccia (19±5)	Tuff (13±5)	

ROCK SAMPLES

Two different homogenous tuff samples were used in the study. Both tuffs are massive, fine-grained and have fragmental textures. Tuff 1 (T1) was obtained from a quarry located in Rize and has a commercial name of Iyidere Tuff. T1 has a gray color and has an ultra-fine-grained matrix which experienced argillisation contains pumice particles, pyroxene, rock pieces, feldspars and carbonate. Alteration degree of the sample is high.

Tuff 2 (T2) was obtained from a quarry in Bayburt and it is generally named as Bayburt Tuff. T2 is a vitric tuff and has a yellowish color. The sample has an ultra-fine-grained matrix which experienced silicification and argillisation and the matrix contains plagioclase, biotite and quartz minerals. Alteration degree of the sample is moderate. Tuff samples can be seen from Figure 2.

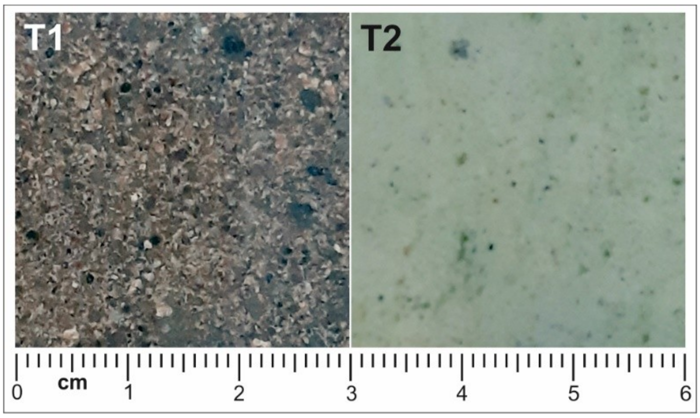


Figure 2. Images of the tuffs used in the study.

Şekil 2. Çalışmada kullanılan tüflerin görüntüleri.

Triaxial compressive strength tests were carried out in the context of this study. Additionally, engineering properties of the samples were also presented which were taken from a previous study. Physical and mechanical properties of the rock samples were determined according to recommendations of ISRM (2007). The apparent porosity (n), the dry density (ρ), the uniaxial compressive strength (σ_{ci}), the indirect tensile strength (σ_t) and the tangent elasticity modulus (E) of the samples are given in Table 2. It should be noted that the mechanical properties of the samples were determined using oven-dry samples.

Table 2. Engineering properties of the tuffs (Yasar, 2020).

Çizelge 2. Tüflerin mühendislik özellikleri (Yasar, 2020).

Rock Samples	ρ (g/cm^3)	n (%)	σ_{ci} (MPa)	σ_t (MPa)	E (GPa)
T1	2.14	11.08	74.66	8.18	23.59
T2	1.70	21.00	74.13	7.29	13.46

TRIAXIAL COMPRESSION TESTS

A series of triaxial compressive strength tests were carried out on tuff samples. NX diamond core bits were used to obtain high quality core samples and height to diameter ratio was selected as 2 for this study. At least two core samples were prepared for each confining pressure level and tests were executed on the oven-dry samples which contain no natural moisture.

Different confining pressures (σ_3 = 3 MPa, 6 MPa, 9 MPa, 12 MPa, and 15 MPa) were applied during tests using the Hoek cell shown in Figure 3. It was stated that at least five confining pressures should be used in the H-B criterion (Hoek, 2007). Confining pressures were applied using an external servo-hydraulic confining pressure loading unit shown in Figure 4. For the execution of tests, a servo-hydraulic testing machine was used which has a capacity of 2000 kN.

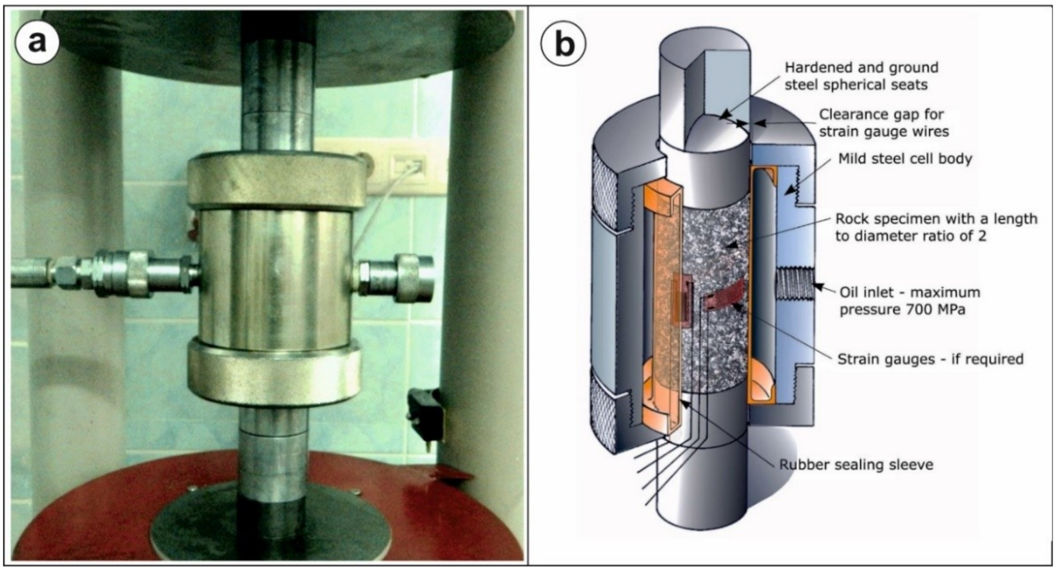
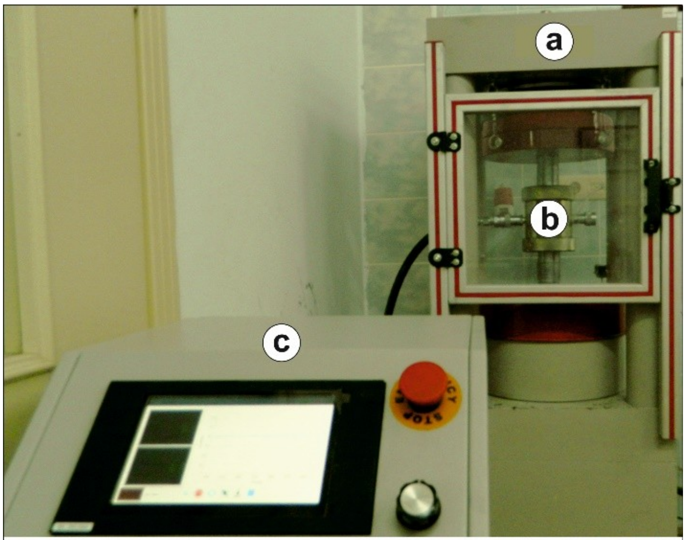


Figure 3. (a) The Hoek cell used in the study (b) Inner structure of the cell (Hoek, 2007).

Şekil 3. (a) Çalışmada kullanılan Hoek hücresi (b) Hücrenin iç yapısı (Hoek, 2007).

The core samples were carefully placed into the rubber sleeve seen in Figure 3 and end caps of the cell body was screwed to avoid any oil leakage. Prior to each testing, the rubber sleeve was checked for the air bubbles which can result in a deviation in the confining pressure. It should be noted here that strain measurements were not realized during the triaxial compression.



a. The loading unit **b.** The Hoek cell
c. The lateral pressure unit

Figure 4. The experimental setup.

Şekil 4. Deney düzeneği.

Triaxial compressive strength tests were carried out according to the recommendations of ISRM (2007). Three procedures can be applied during a triaxial compressive strength test which can be grouped in accordance with the application of the confining pressure. These are individual, multiple failure state and continuous failure state testing procedures. The individual testing procedure was applied in the study which involves the application of the confining pressure individually and using individual samples for each confining pressure. The confining pressure and the axial stress were applied simultaneously up to the selected confining pressure value. Afterwards, axial stress was increased up to the sample failure. Axial loading rate was applied as 1 kN/s. Post-failure photos of the samples were given in Figure 5. It is clear from the figure that all samples demonstrated a shear failure during triaxial compression. However, Mohr circles and envelopes were not provided since the study has no intention to discuss the Mohr-Coulomb failure criterion.

Figure 5, on the other hand, demonstrated the angle between the failure surface and the major principal stress (σ_1). It is pertinent to state that the increasing confining pressure resulted in an increase in the failure angle (β) as a general trend. Results of the triaxial tests were summarized in Table 3. Note that the σ_1 values in the table are the average of two individual tests.

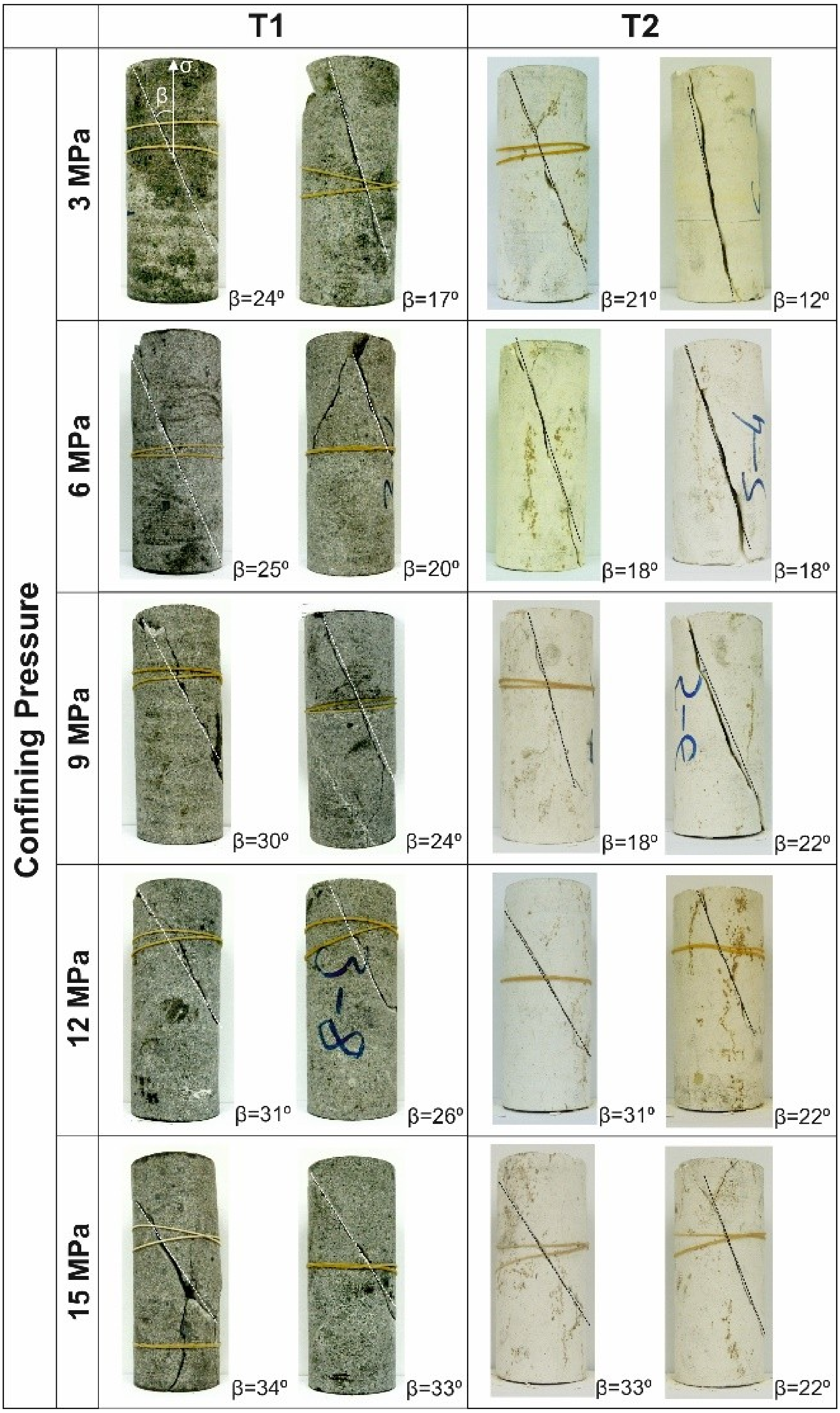


Figure 5. Post-failure images of the samples.

Şekil 5. Numunelerin yenilme sonrası görüntüleri.

Table 3. Results of the triaxial compression tests.
Çizelge 3. Üç eksenli basınç dayanımı deneylerinin sonuçları.

Rock Sample	σ_3 (MPa)	σ_1 (MPa)
T1	3	108.72
	6	118.42
	9	127.24
	12	138.63
	15	151.32
T2	3	111.00
	6	114.43
	9	137.01
	12	146.87
	15	150.09

INTERPRETING THE m_i CONSTANT

m_i constants were computed using the triaxial test data with the aid of the least squares regression method. Firstly, Eq.1 was reorganized and following parameters were accordingly calculated.

$$\boldsymbol{\sigma_1 = \sigma_3 + (m_i\, \sigma_{ci}\, \sigma_3 + \sigma_{ci}^2)^{1/2}} \tag{3}$$

$$\boldsymbol{(\sigma_1 - \sigma_3)^2 = m_i\, \sigma_{ci}\, \sigma_3 + \sigma_{ci}^2} \tag{4}$$

$$\boldsymbol{y = (\sigma_1 - \sigma_3)^2} \tag{5}$$

$$\boldsymbol{x = \sigma_3} \tag{6}$$

$$\boldsymbol{y = m_i\, \sigma_{ci}\, x + \sigma_{ci}^2} \tag{7}$$

$$\boldsymbol{\sigma_{ci}^2 = \frac{\sum y}{n} - \left[\frac{\sum xy - (\sum x \sum y/n)}{\sum x^2 - ((\sum x)^2/n)}\right]\frac{\sum x}{n}} \tag{8}$$

$$\boldsymbol{m_i = \frac{1}{\sigma_{ci}} - \left[\frac{\sum xy - (\sum x \sum y/n)}{\sum x^2 - ((\sum x)^2/n)}\right]} \tag{9}$$

$$\boldsymbol{R^2 = \frac{[\sum xy - (\sum x \sum y/n)]^2}{[\sum x^2 - ((\sum x)^2/n)][\sum y^2 - ((\sum y)^2/n)]}} \tag{10}$$

where σ_1 is the major principal stress, σ_3 is the minor principal stress, σ_{ci} unconfined compressive strength of the intact rock, m_i is the H-B material constant, n is the sample number, and R^2 is the determination coefficient of the regression model. It is generally acceptable to have 0.9 or higher determination coefficients (R_2) for the acceptability of the models (Hoek, 2007). Using the least squares regression model and above-given formulas, σ_{ci}, m_i and R^2 parameters were computed. Results of the regression study are demonstrated in Table 4.

Table 4. Results of the regression study.
Çizelge 4. Regresyon çalışmasının sonuçları.

Rock Sample	H-B Parameter	Value
T1	m_i	6.39
	σ_{ci}	94.97 MPa
	R^2	0.98
T2	m_i	6.75
	σ_{ci}	96.79 MPa
	R^2	0.87

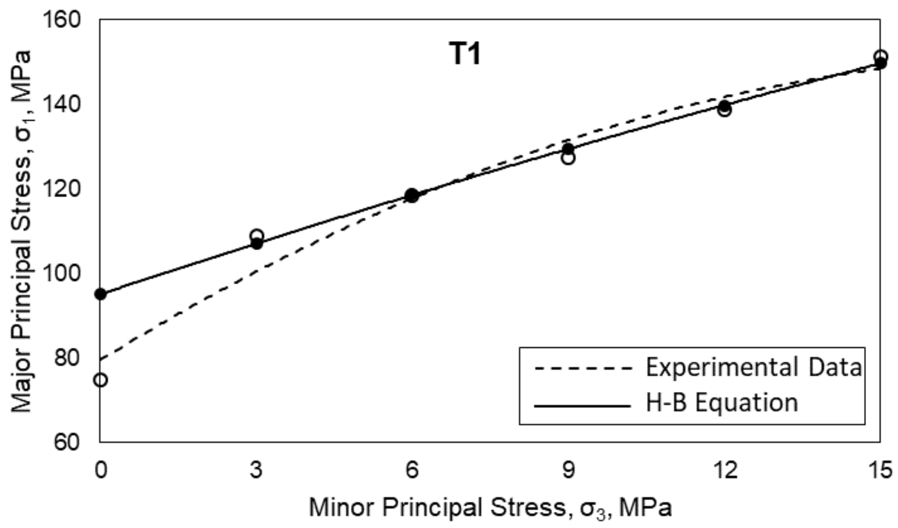
DISCUSSIONS

Initially, tabulated and determined mi values are compared. m_i values for tuffs were given as 13±5 by Hoek (2007) as seen from Table 1 and it was found as 6.39 and 6.75 for T1 and T2, respectively. Considering the standard deviation, it can be easily stated that the determined m_i values are still lower than the tabulated values. H-B equations are, besides, given below which demonstrates the strength under different confining pressures for T1 and T2, respectively.

$$\boldsymbol{\sigma_1 = \sigma_3 + 94.97\sqrt{6.39\,\frac{\sigma_3}{94.97} + 1}} \quad (R^2 = 0.98) \tag{11}$$

$$\boldsymbol{\sigma_1 = \sigma_3 + 96.79\sqrt{6.75\,\frac{\sigma_3}{96.79} + 1}} \quad (R^2 = 0.87) \tag{12}$$

It should be noted here that R^2 values of the regressions satisfy the required level which corresponds to the high representation of the major principal stress (σ_1) with the minor principal stress (σ_3). Triaxial strengths for different confining pressures were calculated using the Eq. 11 and 12 and results were plotted in Figure 6.



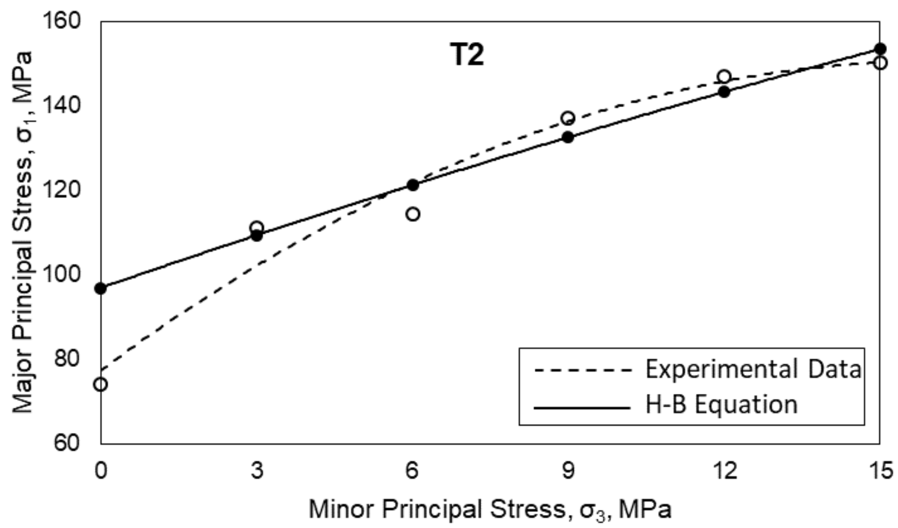


Figure 6. A comparative representation of experimental and H-B results.
Şekil 6. H-B denkleminden ve deneysel olarak elde edilen verilerin karşılaştırılması.

It is clear from Figure 6 that the triaxial strength values calculated from the H-B equation is in a good fit with the experimental results. However, it should be stated here that the unconfined compressive strength values of the rocks were overestimated by the H-B equations. Unconfined compressive strengths of the rocks are 74.66 MPa and 74.13 MPa for T1 and T2, respectively. On the other hand, estimated values are 94.97 MPa and 96.79 MPa as seen from Figure 6.

Tensile strength representation of a failure criterion, also, plays a key role on the ability to represent the failure characteristics of an intact rock sample, such that the major shortcoming of the Mohr-Coulomb failure criterion is the tensile strength overestimation. This is due to the linearity of the envelope which fails to explain the behavior of the rocks. And this issue is, generally, overcome with a tension cut-off. Nonlinearity, on the other hand, seems the best fit for a failure criterion as proposed by Shen et al. (2018) or Hoek and Brown (2019). However, tension part of the H-B equation was not considered in this study and should be investigated in further studies. On the other hand, higher confining pressures can be applied to the tuff samples to check for the further parts of the proposed equations. Additionally, it should be also kept in mind that the m_i constant has delicacy to the mineralogy, the composition, and the grain size of the rock sample (Hoek et al., 2002). Hence, the experimentally proposed m_i constants in this study might vary for different tuff samples with varying mineralogy and grain size.

On the other hand, Shen and Karakus (2014), Hoek and Martin (2014), and Hoek and Brown (2019) described the m_i constant as an indicator for rock brittleness and Hoek and Brown (2019) suggested the following relationship. Note that the σ_{ci}/σ_t ratio is generally stated as an indicator for the rock brittleness and higher ratio corresponds to higher brittleness (Andreev, 1995).

$$R = 0.81\, m_i + 7 \tag{13}$$

where R is the brittleness ratio (σ_{ci}/σ_t) and m_i is the H-B material constant. Computed R (σ_{ci}/σ_t) ratios of the T1 and T2 samples using the Eq.13 are 12.17 and 12.47, respectively. Actual values of the σ_{ci}/σ_t which were calculated using Table 2 are 9.13 and 10.31 for T1 and T2. It can be stated that the computed brittleness ratios are close to the actual ratios.

Additionally, Eq. 2 was used to compute the m_i constant using σ_{ci} and σ_t to compare the experimentally determined values with previous studies. Computed m_i values using the Eq.2 were found as 8.70 and 9.52 for T1 and T2, respectively. Although the experimentally determined m_i values are close the computed m_i values, experimental m_i vales are still lower than the m_i values determined from Eq.2. It supports the fact that the triaxial testing procedure is the most reliable source for the determination of m_i constant if an experimental facility is available.

CONCLUSIONS

The paucity of the triaxial experimental data for tuffs motivated the study. Hence, two homogenous fine-grained tuff samples were subjected to triaxial compression tests under different confining pressures. Least squares regression method was used to derive the intact Hoek-Brown equations for the tuff samples and m_i values for both samples were computed. It was seen that tabulated mi values are higher than the computed ones (6.39 and 6.75 for T1 and T2). Additionally, intact compressive strengths of the tuffs were estimated through the H-B equations and it was also determined that compressive strengths are overestimated. Additionally, it was seen from the triaxial compressive strength tests that failure angles of the samples tend to increase with increasing confining pressure.

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