

# Numerical and experimental research of rock failure mechanism and its dependence on $m_i$ Hoek-Brown failure criterion

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## Abstract

Rock failure mechanism is one of the most important issues in rock mechanics engineering which plays a key role in the stability analysis of various structures. Therefore, different failure criteria have been proposed to understand the failure mechanism of rocks. One of the most commonly used rock failure criteria is the Hoek-Brown criterion, in which there is a parameter called  $m_i$ , which is very important to the response provided by this criterion. Due to the importance of conducting extensive studies on this parameter, in this current research, by performing a series of experimental triaxial compressive strength test and numerical simulating in PFC-2D code, the effect of the Hoek-Brown constant  $m_i$  on the failure mechanism and crack growth of different rocks has been studied. Based on the results of this study, it was found that the effect of parameter  $m_i$  on the failure mechanism of different rocks varied according to the type of rocks, and the greatest effect of this parameter was on the peak strength of rocks. In addition, it was found that under higher lateral pressures, there are less destructive cracks in rocks, and as a result, they show more ductile behaviour.

## Keywords:

Failure Mechanism, Hoek-Brown criterion,  $m_i$  parameter, Triaxial Compressive Strength, PFC-2D.

## 1. Introduction

A great number of rock strength criteria have been proposed over the past years (Hoek and Brown, 1980; Yudhbir et al., 1983; Sheorey et al., 1989; Yoshida, 1990; Ramamurthy, 2001; Dunda et al., 2003; Song et al., 2019). Among the various strength criteria, Hoek-Brown (HB) strength criterion is one of the most well-known, which applies to any type of rock and has been used in various aspects of rock engineering (Merifield et al., 2006; Bagheripour and Hakimipour, 2009; Osgoui and Ünal, 2009; Sari, 2012; Shen et al., 2013; Feng and Jimenez, 2015; Bertuzzi et al., 2016; Jiang et al., 2016; Sun et al., 2016; Wu et al., 2017; Peng et al., 2017; He et al., 2020). HB failure criterion is a non-linear one, which is expressed in terms of major and minor principal stresses at failure ( $\sigma_1$  and  $\sigma_3$ ), and the mathematical equation for an intact rock is expressed as follows (Hoek & Brown, 1980):

$$\sigma_1 = \sigma_3 + \sigma_{ci} \left( m_i \frac{\sigma_3}{\sigma_{ci}} + 1 \right)^{0.5} \quad (1)$$

where:

$\sigma_1$  – the major principal stress,

$\sigma_3$  – the minor principal stress or confining pressure,

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$m_i$  – HB material constant,

$\sigma_{ci}$  – the uniaxial compressive strength of the intact rock.

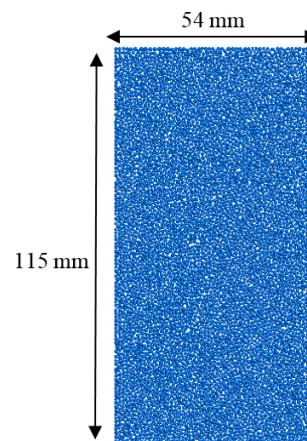
As can be seen in Equation 1, the equation contains two intact rock properties, namely, the uniaxial compressive strength (UCS) of the intact rock  $\sigma_{ci}$  and the HB constant  $m_i$ . The values of  $m_i$  due to its significance in HB failure criterion, have concerned many researchers. Generally, it is assumed that  $m_i$  is a curve-fitting parameter to achieve the HB failure envelope. However, some research showed that  $m_i$  is not just a curve-fitting parameter, but has physical meanings which can be derived from micro-mechanics principles (Hoek and Martin, 2014; Zuo et al. 2008, 2015). The values of  $m_i$  are distributed from 7 to 35, depending upon the rock material characteristics, however, several factors influence these values. For intact rocks, the  $m_i$  value varies with the grain size, mineral composition, cementation of rocks, and applied stress (Hoek, 2007). Research by (Singh et al., 2011 and Shen and Karakus, 2014) indicated that the range of the confining stress  $\sigma_3$  has a significant influence on  $m_i$  values. In the case of anisotropic intact rocks, the  $m_i$  value is also influenced by weak planes (Colak and Unlu, 2004; Saroglou and Tsiambaos, 2008; Bagheripour and Hakimipour, 2009; Peng et al., 2014; Perras and Diederichs, 2014; Briševac et al., 2015; Singh et al., 2015; Hoek and Brown, 2019;

**Jafari Mohammadabadi et al., 2021; Wen et al., 2021**). Since the  $m_i$  parameter is very important in the HB strength criterion, it is necessary to conduct comprehensive studies on this parameter. In the present study, an extensive series of experimental Triaxial Compressive Strength (TCS) tests, which follow the ISRM recommendations on three different rock samples, were carried out, and then by an extensive series of numerical simulations, the influence of HB constant  $m_i$  on the failure mechanism and crack growth of different rocks was studied. **Jafari Mohammadabadi et al., 2021** reported that the bond cohesion and tensile strength and friction coefficient are very important to increase rock strength and failure mechanism. As we know, for designing a safe and stable underground or surface mining project, rock failure mechanism and failure criterion are the most important issues which should be understood clearly. So, rock failure criteria's parameters such as  $m$  and  $s$  in the Hoek-Brown failure criterion should be defined, otherwise, it may affect the stability, and finally, a collapse could occur. For this reason, in this current research, three types of rock were selected and rock failure mechanism and crack initiation were evaluated, both experimentally and numerically. In numerical simulations where the effects of micro-parameters were not taken into account, the simulations were precisely analyzed and then the results were compared with experimental tests.

## 2. Material and methods

To conduct the studies, three types of rock samples, namely andesite, limestone and sandstone have been tested, which include rocks with different strength values. Altogether, 27 rock samples were tested. Numerical simulations have been performed using particle flow code 2D (PFC2D). This code, which is one of the most famous software based on the Discrete Element Method (DEM), simulates the materials as a set of bonded particles that follow the law of motion. In PFC2D, the constitutive relations have been replaced with simpler particle contact logic (which do not require plasticity rules), and so there is no need for pre-defined complex empirical relationships (**Bahrani et al., 2011**). Also, all stages of fracture mechanism could be simulated (from crack initiation, propagation, to coalescence explicitly with bond breakage) without any specific crack-criteria (**Zhang and Stead, 2014**).

Due to its advantages, PFC2D has been used in a wide range of numerical studies by other researchers in recent years (**Hama et al., 2017; Calvetti, 2008; Gu et al., 2017; Lin et al., 2017; Akram and Sharrock, 2010; Cui et al., 2019; Potyondy and Cundall, 2004; Asadi et al., 2012; Bahaaddini et al., 2013; Bahaaddini et al., 2014; Bahaaddini et al., 2016; Bahaaddini, 2017; Yang and Huang, 2018; Deisman et al., 2010; Mas Ivars et al., 2011; Bahaaddini et al., 2013b; Bahaad-**



**Figure 1:** Geometry of PFC 2D model in TCS test

**dini et al., 2016; Jiang et al., 2018; Jafari Mohammadabadi et al., 2021**).

PFC2D code has three main bonding models (Contact Bond (CB), Parallel Bond (PB), and Flat-Joint (FJ) models) that simulate the cementation between particles. In the current study, the FJ model is used for numerical modeling. This model was proposed by **Potyondy (2012)** to fix the intrinsic problems that had been raised in the standard PB model and can reproduce the mechanical behaviour of rock materials under different loading conditions (**Bahaaddini et al., 2019; Jafari Mohammadabadi et al., 2021**).

In this research, the diameter and height of samples were considered 54 mm and 115 mm, respectively (see **Figure 1**). In the process of generating a sample, as the first stage, a sample vessel including frictionless walls was created, and then it was filled with a set of particles that were placed randomly. The particle size obeys a normal distribution which is determined by the minimum particle radius ( $R_{\min} = 0.45$  mm) and the ratio of maximum to the minimum radius ( $R_{\max}/R_{\min} = 1.5$ ). For decreasing the number of locked-in stresses, isotropic stress is applied between all of the particles through the grain scale processing. Finally, bonds are installed at the particle contacts with a gap equal to or less than the installation gap (**Itasca Consulting Group Inc., 2017**).

After the generation of specimens, the micro-properties of particles and bonds must be calibrated. To perform the calibration process in PFC2D, the mechanical properties of the synthetic material, i.e. the macro response of its behaviour, must be set up, and then micro-properties are defined and calibrated with an appropriate method depending on the bond type used in the model. Both uniaxial and triaxial compressive strength tests can be modeled during the calibration process (**Itasca Consulting Group Inc., 2017**). Therefore, the mechanical behaviour of the synthetic samples under the compression test is reproduced and compared with the experimental tests. In this paper, the TCS test has been used for calibrating the micro-properties of synthetic samples.

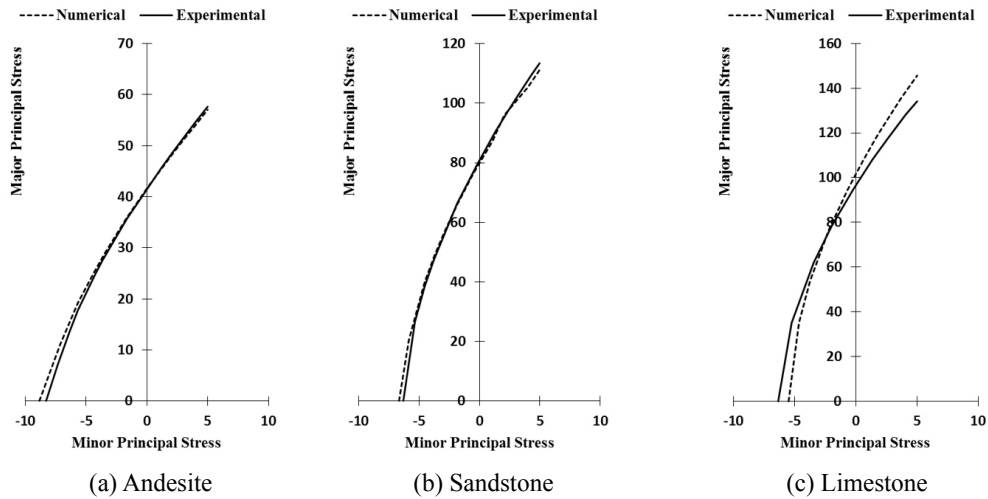


Figure 2: Comparison of failure envelope of numerical and laboratory tests

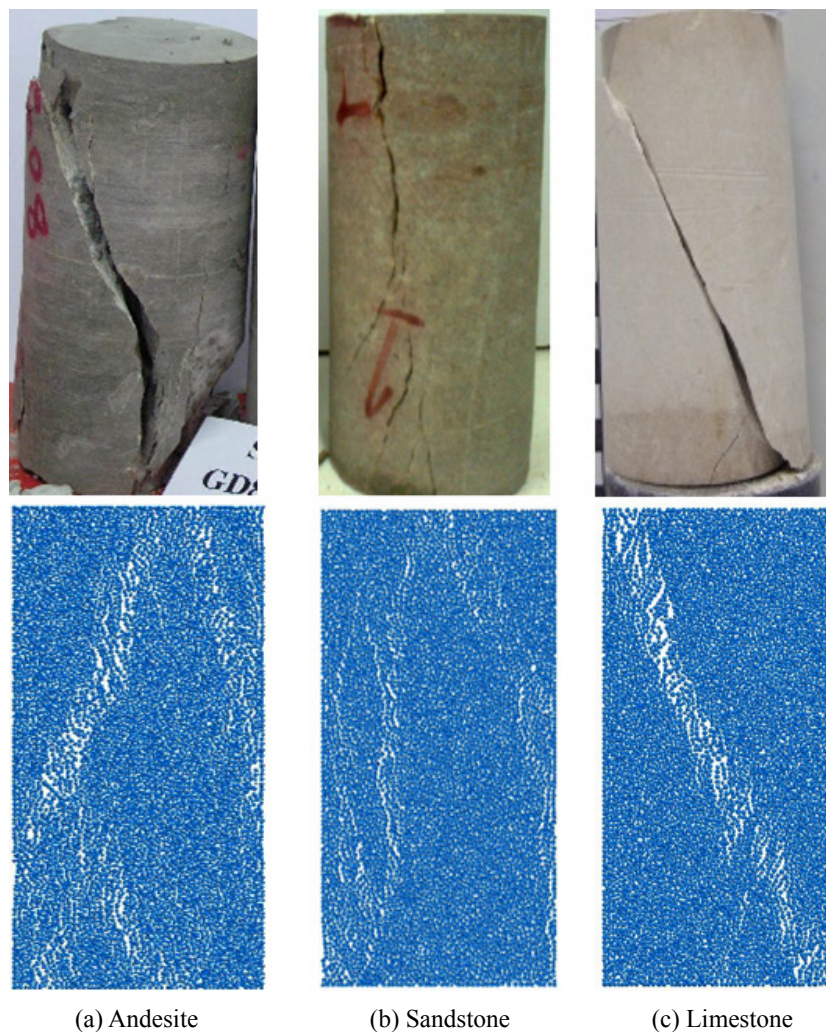


Figure 3: Comparison of failure platen of numerical and laboratory tests

To ensure the accuracy of the numerical results, the failure envelopes of the rock samples have been compared with laboratory tests, which are shown in Figures 2 and 3. Also, in Table 1, the values of rock strength

parameters in both numerical and laboratory modes are compared with each other. According to Figures 2 and 3, and Tables 1 and 2, the values obtained from numerical models are very close to the laboratory in all three



**Table 1:** Comparison of numerical and experimental results

Rock	$m_i$			C (MPa)			$\Phi$ (°)		
	Experimental	Numerical	STD* (%)	Experimental	Numerical	STD (%)	Experimental	Numerical	STD (%)
Andesite	5.00	6.47	2.94	12.35	12.42	0.56	31.40	35.22	12.16
Sandstone	12.86	12.04	6.37	17.30	18.99	9.76	46.86	45.83	2.19
Limestone	15.25	17.96	17.77	17.29	17.9	3.52	49.57	51.94	4.5

\*STD: standard deviation

**Table 2:** Average value of principal stresses on each sample

Rock number	Rock samples								
	Andesite			Limestone			Sandstone		
	Ave.S <sup>1</sup>	Ave.S <sup>2</sup>	Ave.S <sup>3</sup>	Ave.S <sup>1</sup>	Ave.S <sup>2</sup>	Ave.S <sup>3</sup>	Ave.S <sup>1</sup>	Ave.S <sup>2</sup>	Ave.S <sup>3</sup>
$\sigma_1$ (MPa)	47.3	53.3	60	102.7	113.1	132.2	94.2	106.3	119.8
$\sigma_3$ (r)	1	3	5	1	3	5	1	3	5

\*Ave.S = Average for each sample

**Table 3:** Calibrated values from each rock in PFC 2D models

Property	Andesite	Limestone	Sandstone	Unit
Density	2450	2600	2450	(kg/m <sup>3</sup> )
Bonded fraction	0.93	0.98	0.99	---
Ec	6.0	15.2	14.0	(GPa)
$K_n/K_s$ *	1.7	2	1.7	---
Friction coefficient	0.45	0.8	0.3	---
Cohesion	33.5	64.5	82.0	(MPa)
Cohesion std	3.35	6.45	8.2	(MPa)
Tensile strength	5.0	12.6	4.5	(MPa)
Tensile strength std	0.5	1.26	0.45	(MPa)
Friction angle	5	15	5	Degree

\*  $K_n$  and  $K_s$  are normal and shear stiffness between grains respectively

rock samples. Therefore, it can be concluded that the created models are sufficiently accurate to perform more research. The calibrated micro-parameter values from each rock type are given in **Table 3**. As shown in **Figure 2**, the maximum principal stress versus minimum principal stress is drawn based on Hoek-Brown rock failure criterion. Even though andesite and sandstone have a lower value of  $m_i$  compared to limestone, they are more brittle and axial and tensile fractures were developed through the samples, despite the low value of their strength. Limestone is denser and stiffer than the other samples. As it was found from both experimental and numerical simulations, a major fracture is developed rather than some wing cracks and micro fractures, which shows a higher rigidity of the rock. In addition, it is also

because of calibration accuracy in PFC, andesite and sandstone have a high compatibility, while for limestone, the scatter between the graphs is slightly increased. This small amount may be ignored.

### 3. Results and discussion

After the calibration process and ensuring the accuracy of numerical results, the effect of the  $m_i$  parameter on the mechanical properties of the rocks was studied. For this purpose, first, the behaviour of stress-strain curves of all three rock types under a confining pressure of 3 MPa was studied. The graphs of these results are shown in **Figure 4**.

As can be seen in **Figure 4**, the obtained stress-strain curves for andesite, sandstone and limestone are shown. The andesite rock reached its peak of strength after withstanding 0.95% strain at a stress of 60 MPa and then failed. In this rock, tensile cracks with 91 percent of total cracks had the most influence on the failure mechanism and the rock had a linear behaviour. The first crack (which is a tensile one) was created at a stress of 9.9 MPa and around 0.2% strain and the destructive cracks started at 53.9 MPa. However, shear cracks started around 0.8% strain, which started four times later than the tensile cracks.

In sandstone, after withstanding 0.79%, the rock reached peak strength at 114 MPa. This rock has shown brittle behaviour. Also, tensile cracks have the greatest impact on the failure of the rock sample (99% of total cracks), and the crack density suddenly increased from the moment of  $\sigma_{peak}$  to the moment of sample failure. In this sample also, shear cracks started at 0.8% of strain while the tensile cracks started at less than 0.2% of strain.

In limestone, the rock sustained a 0.90% strain, and then at a stress of 132 Mpa, it reached the peak of its

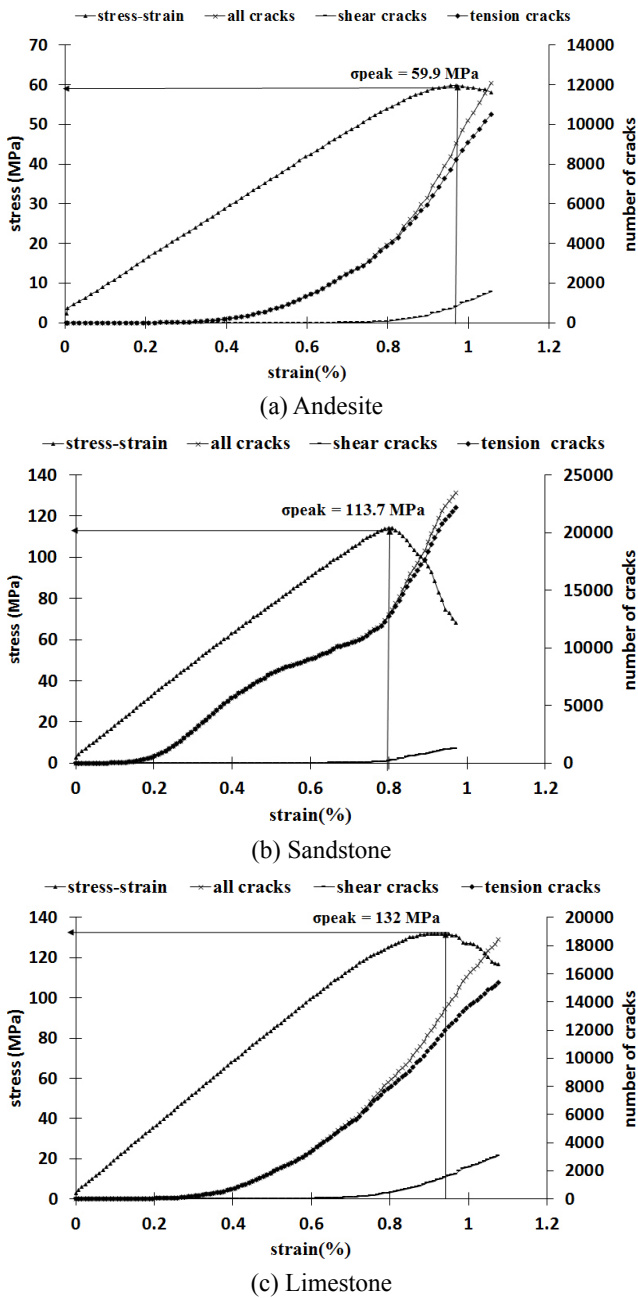


Figure 4: Obtained stress-strain curves for (a) andesite (b) sandstone (c) limestone

strength. This rock had a ductile behaviour and tensile cracks have the most influence on the failure mechanism (91% of total cracks). In this rock type, tensile cracks started around 0.3% of strain but shear cracks began at 0.6% of the strain value.

In Figure 5, a comparison of the mechanical responses in each rock sample under different lateral pressures is shown. The mechanical response of these three rock samples has been coordinated under different lateral pressures. In these graphs, the values of the ultimate strength of rocks and the equivalent strain are researched. According to the following diagrams, it can be seen that limestone (which had a larger  $m_i$ ) has greater strength

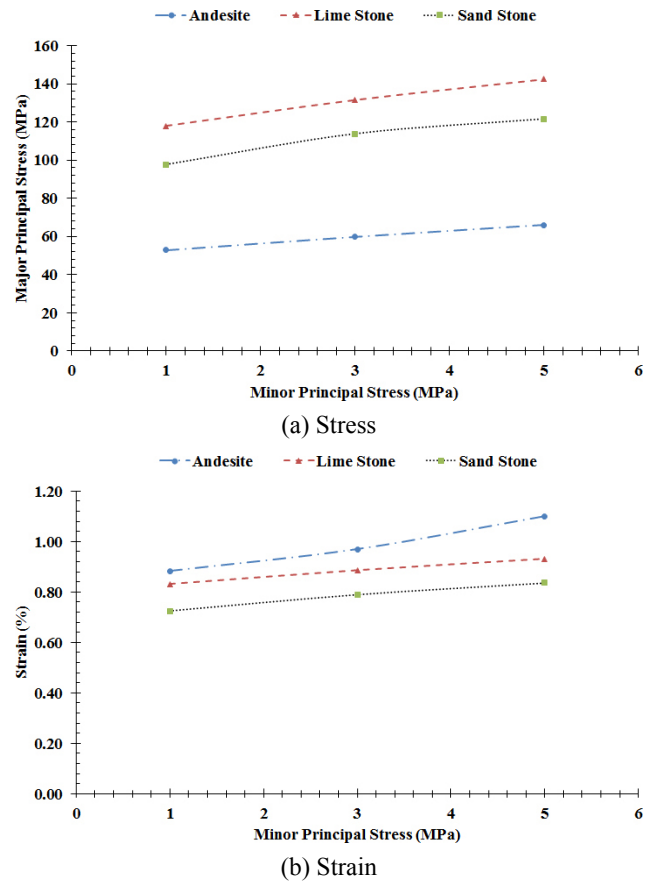
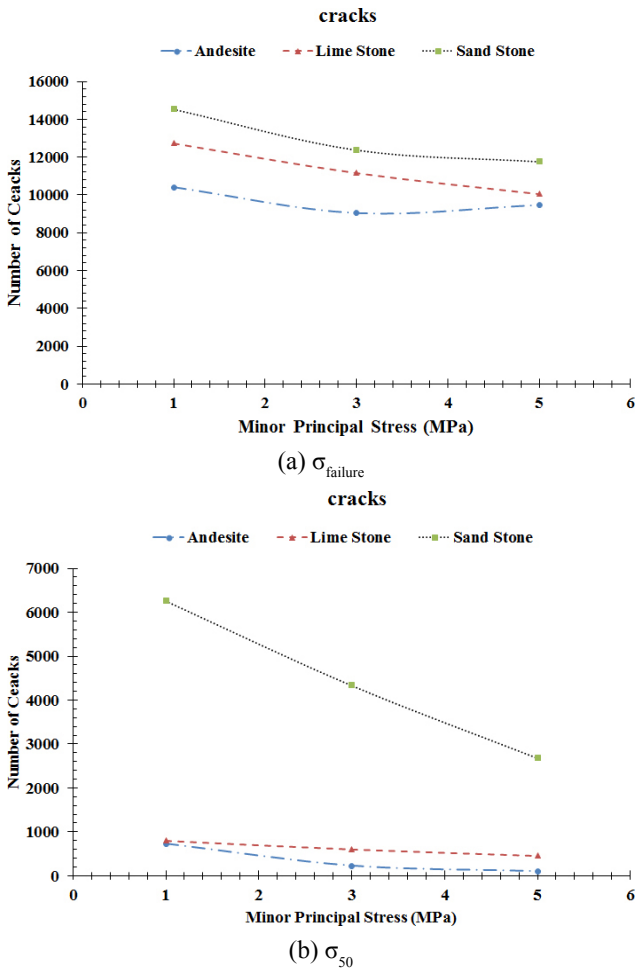


Figure 5: Comparison of the mechanical responses in each rock sample under different lateral pressures: (a) stress (b) strain

than the other two rocks. This relation is consistent in the two other rocks as well. The strain values of the rocks in andesite (which had a smaller  $m_i$ ) were higher than in the other cases. However, in the other two rock samples, this relationship was not consistent, and limestone withstood more strain than sandstone.

Also, the density of destructive cracks at the moment of  $\sigma_{failure}$  and  $\sigma_{50}$  has been studied, which is shown in Figure 6. According to Figure 6, the density of cracks at the moment of failure in andesite rock is less than the other rocks, but limestone had fewer cracks than sandstone, and this density decreased with the increasing lateral pressure in all three types of rocks. These relations were also consistent at the  $\sigma_{50}$  of all 3 rock types. However, the density of destructive cracks in sandstone is much higher than in the other rocks.

Moreover, the condition of the specimen samples at the time of failure under different lateral pressures is also shown in Figure 7. According to Figure 7, it can be seen that the andesite rock has a shear fracture, which becomes more visible with increasing lateral pressure. Also, sandstone and limestone have splitting and multiple shear fractures, respectively. The tensile cracks (indicated by red balls) had the greatest impact on the failure mechanism in all three cases of these rock samples,



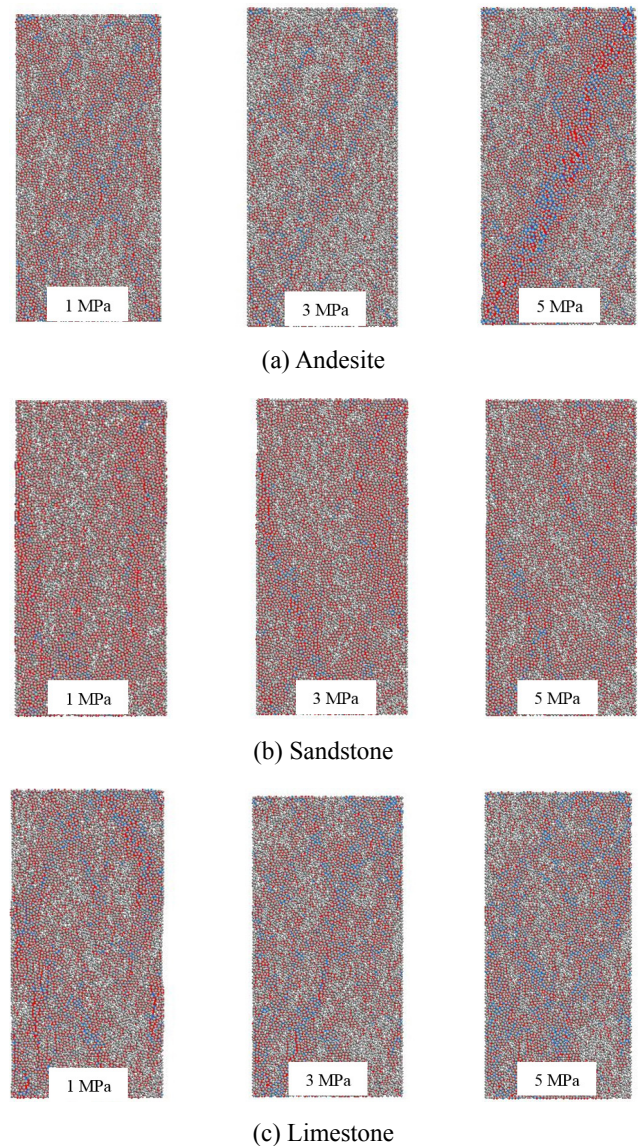
**Figure 6:** Comparison of the density of destructive cracks in rock samples under different lateral pressures: (a) At  $\sigma_{failure}$  and (b) At  $\sigma_{50}$

and with increasing lateral pressure in all three cases, the behaviour of the rock has become more ductile.

#### 4. Conclusion

In this paper, using experimental tests and numerical simulations of Triaxial Compressive Strength tests in PFC 2D software, and the effects of the HB parameter  $m_i$  on the mechanical behaviour of three rock types with different mechanical properties and failure mechanisms were studied. For this purpose, after ensuring the accuracy of the numerical results compared to experimental results, the stress-strain curves of the rocks and the macro responses of their behaviour were studied. The most important results of this study can be summarized as follows:

According to the studies, it was found that the effect of parameter  $m_i$  on the failure mechanism of different rocks varied according to the type of rocks. However, the greatest effect of the  $m_i$  parameter was on the ultimate strength of rocks so that the lowest change in its value can change the amount of ultimate strength of the rocks. Increasing this parameter reduces the value of



**Figure 7:** Condition of shear and tensile cracks in rock samples under different lateral pressures: (a) andesite (b) sandstone (c) limestone

strains that occurred depending on the type of rock. This could change the rock behaviour dramatically if the right value could not be taken into account. In addition to this, increasing the rock failure criterion parameter of  $m_i$  increases the strength of rocks, which is not the case in reality.

Also, it was found that under higher confining pressures, the destructive cracks in the rocks are smaller, which could lead to ductile behaviour in the rock. From the results, it was found that at low values of confining pressures, the number of developed cracks in all three different rock types are almost the same at the peak strength value, however at the elastic range of the applied load, the number of cracks which developed in sandstone are significantly greater than the other rocks, nearly six times. This value reduces sharply with increasing confining pressure.

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## SAŽETAK

### Numerička i eksperimentalna istraživanja mehanizma loma stijene te ovisnosti toga procesa o Hoek-Brownovu kriteriju loma $m_i$

Mehanizam loma stijene jedno je od najvažnijih pitanja u mehanici i inženjerstvu stijena te je ključno u analizi stabilnosti raznih struktura. Za različite kriterije loma predloženi su različiti mehanizmi odgovorni za takav proces. Jedan od najčešće korištenih kriterija kojim se opisuje lom stijena jest Hoek-Brownov kriterij, u kojemu postoji varijabla zvana  $m_i$ , koja je vrlo važna kod ocjene tim kriterijem. Zbog važnosti provođenja opsežnih studija o prilagođavanju te varijable u prikazanim istraživanjima ona je određena izvođenjem serije eksperimentalnih ispitivanja troosne tlačne čvrstoće i numeričke simulacije u PFC-2D kodu. Time je istražen učinak vrijednosti  $m_i$  na lom stijene i rast pukotina različitih stijena. Na temelju rezultata utvrđeno je kako je utjecaj varijable  $m_i$  na lom različitih vrsta stijena varijabilan, a najveći je učinak na vršnu čvrstoću stijena. Također, utvrđeno je da su pod većim bočnim pritiscima razarajuće pukotine u stijena manje (uspoređeno s prethodnim testom), a kao rezultat pokazuju više duktilno ponašanja.

#### Ključne riječi:

mehanizam loma, Hoek-Brownova varijabla  $m_i$ , troosna tlačna čvrstoća, PFC-2D

#### Author's contribution

**Sina Salajegheh** (Ph.D. student): author and performed numerical tests and laboratory experiments. **Shahriar** (Full Professor): interpreted and analysed the numerical and laboratory results. **Jalalifar** (Full Professor): created the ideation and monitored the execution of work from the beginning to the end. **Ahangari** (Assistance Professor): assisted in writing and in the history of literature.