

Slope Stability Considering Joint Networks and Creep Parameters of Rock Mass

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Abstract: Research on rock slope stability, particularly considering joints and creep parameters, plays an important role. In this paper, the authors present an experimental procedure for determining the instantaneous mechanical parameters of rock using uniaxial compression and Brazilian tests, as well as for identifying creep parameters through three-point bending tests on beam-shaped rock specimens. The creep parameters obtained from laboratory tests were converted for use in numerical simulations. The Itasca 3DEC software was then employed to evaluate slope stability, incorporating both joint structures and the creep behavior of rock masses

Keywords: Rock mass, equivalent internal friction angle, deformation modulus of rock mass, Itasca 3DEC

1. Introduction

Rock slope stability is a crucial issue in structural stability calculations. With the advancement of computer science, numerical methods are widely used in rock slope stability design, taking into account rock mass parameters. Currently, numerical methods used in geotechnical engineering include the finite element method (FEM), boundary element method (BEM), finite difference method (FDM), and discrete element method (DEM). Major universities around the world, such as TU Bergakademie Freiberg (Germany), TU Delft (Netherlands), and Moscow State University of Civil Engineering (Russia), have been developing methods that incorporate rock mass parameters.

The advantages of DEM-based numerical methods include their ability to simulate and solve complex problems such as non-uniformity, anisotropy, nonlinearity, and multi-scale behavior. They are suitable for problems that cannot be addressed using analytical or physical modeling. DEM provides detailed analysis and helps researchers gain a deeper understanding of the physical mechanisms involved. However, one limitation of DEM is that it requires users to have a deep understanding of the method. All models used in geotechnical calculations must be verified and validated.

Itasca 3DEC is an advanced numerical modeling tool specifically designed for geotechnical analysis of discontinuous materials such as rock, masonry, and soil. It uses a hybrid simulation approach combining the distinct element method (DEM) and continuum mechanics. Itasca 3DEC allows for the simulation of blocky materials with joints that can deform, slide, rotate, or detach.

Based on conversion formulas and custom-made equipment, the authors conducted uniaxial compression and Brazilian splitting tensile tests to determine rock parameters in accordance with the Mohr–Coulomb model. These parameters were then converted, using a simplified conversion theory, for application to rock masses.

2. Uniaxial compression test and Brazilian splitting test.

2.1. Rock sample uniaxial compression test

The procedure for the unconfined compression test is as follows: the load is gradually applied to the surface of the sample until failure. The load-bearing capacity is determined as the maximum compressive load the sample can withstand before failure. The uniaxial compressive stress is calculated using Equation.

$$\sigma_N^* = \frac{4F_1}{\pi D^2} \quad (1)$$

where:

σ_N^* - Uniaxial compressive strength of the rock, determined experimentally

F_1 - Maximum axial compressive force;

D - diameter of the sample.

Based on the results of the uniaxial compression test and using empirical equations, the deformation modulus and Poisson's ratio of the rock samples are determined.

2.2. The Brazilian splitting tensile strength test

The behavior of soil and rock under compression and shear has not been extensively studied, partly due to limitations in experimental techniques and partly because these behaviors often manifest on a small scale prior to sample failure. Additionally, these behaviors are often specific cases that can be inferred from simpler compression tests.

Tensile and shear tests on rock samples are generally conducted only to assess their load-bearing capacity under such conditions. Published data on the deformation behavior of rocks under tensile and shear loading remain scarce. A typical example is the Brazilian splitting tensile test, first proposed by F. Carneiro and A. Barcellos (Brazil) in 1949. When the sample is compressed along its vertical axis, stress components act along the diameter, and the minimum principal stress (σ_3) develops in the horizontal direction.

The maximum tensile stress at the center of the sample is calculated using the following equation:

$$\sigma_k = \frac{2F_2}{\pi DL} \left[\frac{\sin 2\alpha}{\alpha} - 1 \right] \approx \frac{2F_2}{\pi DL} \quad (2)$$

where:

σ_k - indirect tensile strength of the rock (Brazilian test);

F_2 - applied horizontal force;

L- length of the sample.

Deformation perpendicular to the loading direction is tensile in nature and is governed by the tensile stress σ_3 . When the tensile stress at the center reaches the rock's tensile strength, the sample fails, and a crack propagates along its diameter, splitting it into two halves.

The maximum tensile stress σ_k corresponds to the maximum applied horizontal force F_{2max} as follows:

$$\sigma_k^* = \frac{2F_{2max}}{\pi \cdot D \cdot L} \quad (3)$$

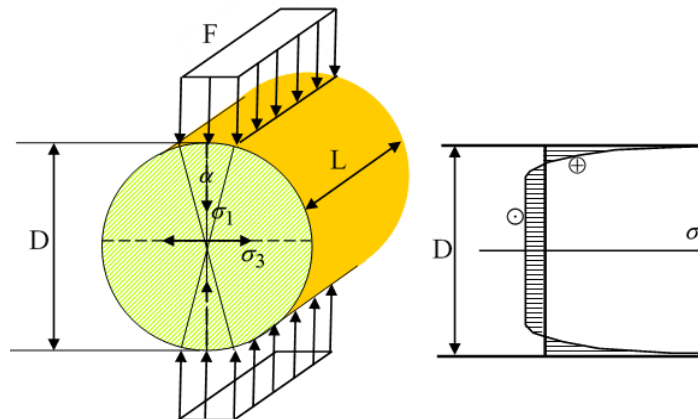


Fig. 1. The Brazilian splitting tensile strength test

The effective cohesion (c') and angle of internal friction (ϕ) of the sample were determined directly using Equation (1), which incorporates the maximum tensile stress σ_k^* :

For the cohesion:

$$c' = \frac{1}{2} \cdot \sqrt{\sigma_N^* \cdot \sigma_K^*} \quad (4)$$

For the internal friction angle:

$$\phi' = \arcsin \left(\frac{\sigma_N^* - \sigma_K^*}{\sigma_N^* + \sigma_K^*} \right) \quad (5)$$

Thus, based on the results of uniaxial compression and Brazilian splitting tensile tests, the parameters of the Mohr-Coulomb model, including the elastic modulus E, Poisson's ratio ν , cohesion c' , and internal friction angle ϕ' , can be determined.

3. Conclusions



Fig. 3. Sample machining and loading, installation of a displacement measuring gauge, compression force.

The experimental rock samples were collected from a coal mine in Quang Ninh Province at a depth exceeding 100 meters. The samples were subsequently processed and tested in the laboratory located at 443 Pham Van Dong Street, Co Nhue.



Fig. 4. Rock sample failure
Tab. 1. The Input rock sample

	Deformation parameters	Units of measurement	Values
01	Unit weight, γ	T/m ³	2.5
02	Sample height, H	cm	6
03	Sample diameter, D	cm	3

Tab. 2. The parameters for The Brazilian splitting tensile strength test and uniaxial compression gained from equations (8) and (9).

Parameters	Units of measurement	Sampe 1	Sample 2	Sample 3	Average values
σ_N^*	MPa	29	29.1	32.07	30
σ_K^*	MPa	0.77	0.78	0,85	0.8
The cohesion of rock mass c'_i	MPa	2.362	2.382	2.61	2.45
The internal friction angle of rock sample ϕ'_i	Degree	71.52	71.44	71.54	71.5

Tab. 3. Processed deformation parameters

	Deformation parameters	Units of measurement	Values
01	Deformation modulus, E_i	MPa	2300
02	Poisson ratio, ν_i		0.25

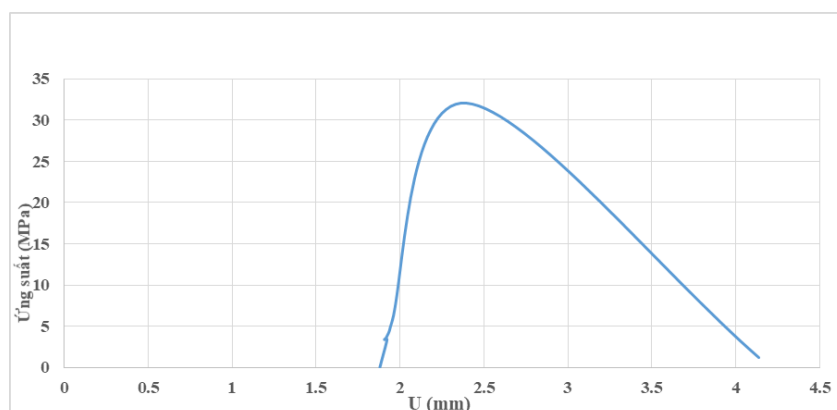


Fig. 6. Stress- axial strain curve

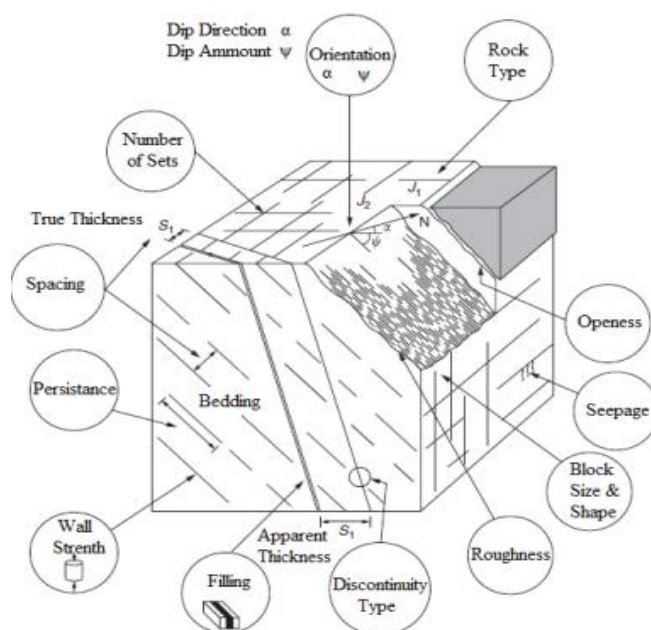


Fig. 7. Rock mass model

The calculated compressive strength of the rock mass is determined using the following equations:

$$\sigma_N^{*mass} = \sigma_N^* \cdot k_c \cdot k_{yl} \quad (6)$$

$$\sigma_K^{*mass} = \sigma_K^* \cdot k_c \cdot k_{wl} \quad (7)$$

where:

k_c - coefficient accounting for the degree of structural weakening due to discontinuities such as slip surfaces, cracks, or interbedded weak layers (see Tab. 4);

k_{vl} - coefficient reflecting the reduction in strength due to water saturation (see Tab. 5).

Then the cohesion - $c'_{\max}$ and the internal friction angle $\phi'_{\max}$ of rock mass are calculated using the following equations

$$c'_{mass} = \frac{1}{2} \cdot \sqrt{\sigma_N^{*mass} \cdot \sigma_K^{*mass}} \quad (8)$$

$$\phi'_{mass} = \arcsin \left(\frac{\sigma_N^{*mass} - \sigma_K^{*mass}}{\sigma_N^{*mass} + \sigma_K^{*mass}} \right) \quad (9)$$

Tab. 4. The determination of the coefficient k_c

The average distance between the rock-weakening surfaces, m	Values of k_c
Greater than 1,5	0.9
Less than 1,5 to 1	0.8
Less than 1 to 0,5	0.6
Less than 0,5 to 0,1	0.4
Less than 0,1	0.2

Tab. 5. The determination of the coefficient k_{vl}

Rock types	Values of k_{vl}
Siliceous sandstone and flagstone	0.8
Limestone sandstone and limestone	0.7
Clayey siltstone and silty sandstone	0.6
Claystone and mudstone	0.3
Peat clay	0.2
Kaolinite clay	0.6

The deformation modulus of the rock mass can be estimated from the deformation modulus of the intact rock using the following relation:

$$\frac{1}{E_m} = \frac{1}{E_i} + \frac{1}{k_n S} \quad (10)$$

$$\frac{1}{G_m} = \frac{1}{G_i} + \frac{1}{k_s S} \quad (11)$$

$$\nu_m = \nu_i$$

$$G_i = \frac{E_i}{2(1 + \nu_i)} \quad (12)$$

where:

- E_m - deformation modulus of the rock mass;
- E_i - deformation modulus of the intact rock sample;
- S - average spacing between structural weaknesses;
- k_n - normal stiffness of discontinuities;
- G_m - shear modulus of the rock mass;
- G_i - shear modulus of the intact rock;
- k_s - shear stiffness of discontinuities;
- ν_i - Poisson's ratio of the intact rock;
- ν_m - Poisson's ratio of the rock mass.

The effect of normal stress on joint stiffness was neglected, as laboratory and numerical simulation results have shown that normal stiffness increases with applied normal stress. Moreover, while average normal and shear stiffness values are commonly used for evaluating joints, little research has focused on determining instantaneous stiffness. Therefore, it is necessary to propose a method for accurately determining the real stiffness, which can offer a deeper understanding of the mechanical behavior of rock joints.

$$k_n = k_s = 10 \max \left[\frac{K_i + \frac{4}{3} G_i}{\Delta z_{\min}} \right] \quad (13)$$

where:

K_i - The bulk modulus of rock sample;

$$K_i = \frac{E_i}{3(1 - 2\nu_i)}$$

$\Delta z_{\min}$ - The minimum size of the mesh of contact surfaces.

4. Numerical modelling in Itasca 3DEC

Applying the parameters converted to rock mass parameters and simulating fractures:

Tab. 6. Parameters of rock mass

	Parameter	Units of measurement	Values
01	The deformation modulus of rok sample	MPa	2300
02	Poisson ratio of rok sample, ν_i	-	0.25
03	The shear modulus of rok sample, G_i	MPa	920
04	The bulk modulus of rok sample, K_i	MPa	1533
05	The determiation of the coefficient, k_c	-	0.9
06	The determination of the coefficient k_{ν}	-	0.8
07	The average distance between the rock-weakening surfaces, S	m	1.5
08	The minimum size of the mesh of contact surfaces	m	6
09	The normal stiffness, k_n	MPa/m	4580
10	The shear stiffnesses, k_s	MPa/m	4580
11	The shear modulus of rok mass, G_m	MPa	811
12	The deformation modulus of rok mass, E_m	MPa	1723
13	The bulk modulus of rok mass, K_m	MPa	1148
14	The cohesion of rock mass, c'_m	MPa	1.764
15	The internal friction angle of rock mass, ϕ'_m	Degree	71.5
16	Dilatancy angle	Degree	41.5

Creep parameters of rock samples:

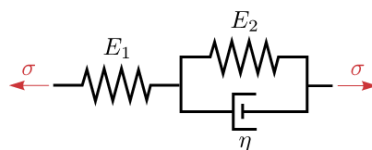


Fig. 8. Zener linear viscoelastic model

Currently, there are various laboratory methods available for determining the creep parameters of rock specimens. One such method is the bending compression test performed on beam-shaped rock specimens with a cross-sectional dimension of 20×20 mm and a length of 160 mm. The testing setup is illustrated in Fig. 9.

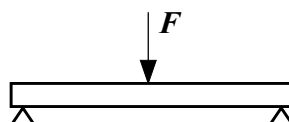


Fig. 9. The scheme of bending and compressing test of a rock specimen

The deflection of the rock beam is determined by using the following formula:

$$y = \frac{F.l^3}{48.E.J} \quad (14)$$

Where: J - moment of inertia of the beam -cross-section.

The tensile stress arising from bending the rock beam is calculated using the following formula:

$$\sigma = \frac{M.h}{2J} \quad (15)$$

Where: M - The maximum bending moment at the center beam cross-section; h - beam height.

$$M = \frac{F.l}{4} \quad (16)$$

Combining (16), (15) and (14) the formula could be presented as follows:

$$y = \sigma \cdot \frac{l^2}{6E.h} \quad (17)$$

The initial instantaneous beam deflection gets the values of:

$$y_0 = \sigma \frac{l^2}{6E.h} \quad (18)$$



Fig. 10. Processing and loading of the sample

The equation with Zener model:

$$y(t) = \frac{\sigma}{E_{\infty}} \times \left[1 - \left(1 - \frac{E_{\infty}}{E_1} \right) \right] e^{\left(-\frac{E_{\infty}}{E_1 n} t \right)} \quad (19)$$

$$E_{\infty} = \frac{E_1 E_2}{E_1 + E_2} \quad (20)$$

$$n = \frac{\eta}{E_1 + E_2} \quad (21)$$

$$E(t) = \frac{E_{\infty}}{\left[1 - \left(1 - \frac{E_{\infty}}{E_0} \right) \right] e^{\left(-\frac{E_{\infty}}{E_0 n} t \right)}} \quad (22)$$

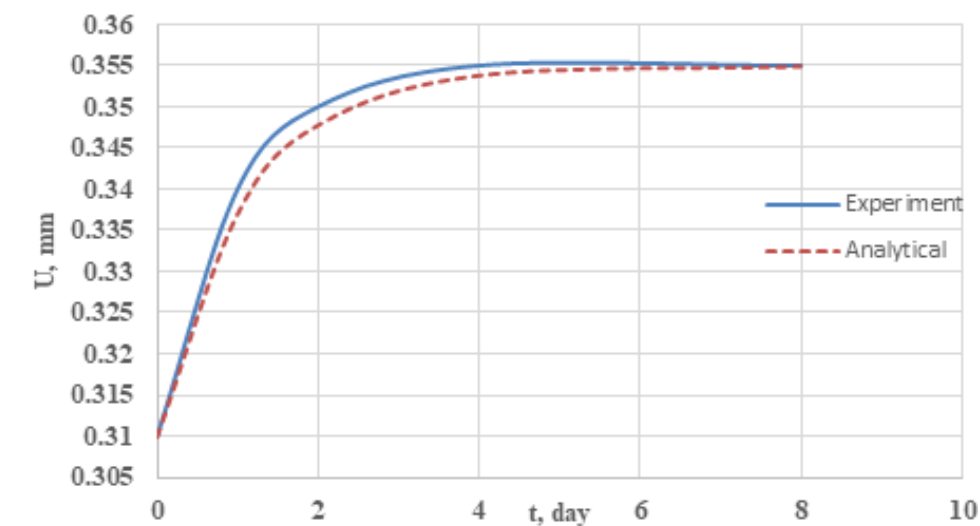


Fig. 11. Creep parameters of rock samples

Creep parameters of rock mass, combining (22), (10):

$$\frac{1}{E_m(t)} = \frac{\left[1 - \left(1 - \frac{E_\infty}{E_0}\right)\right] e^{\left(\frac{E_\infty t}{E_0^n}\right)}}{E_\infty} + \frac{1}{k_n S} \quad (23)$$

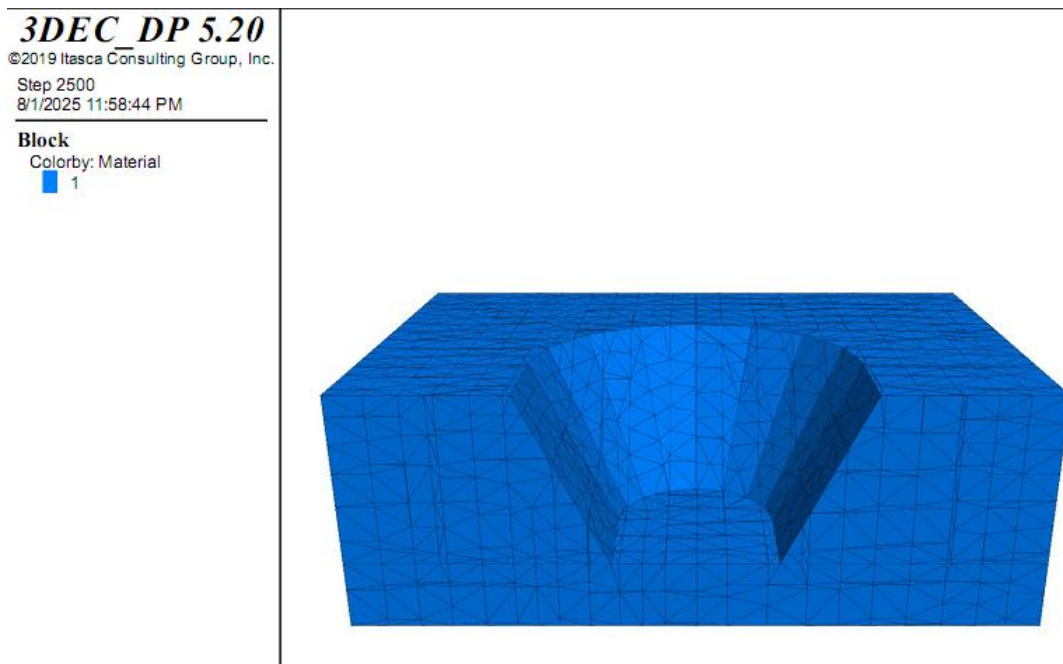


Fig. 12. Model in Itasca 3DEC

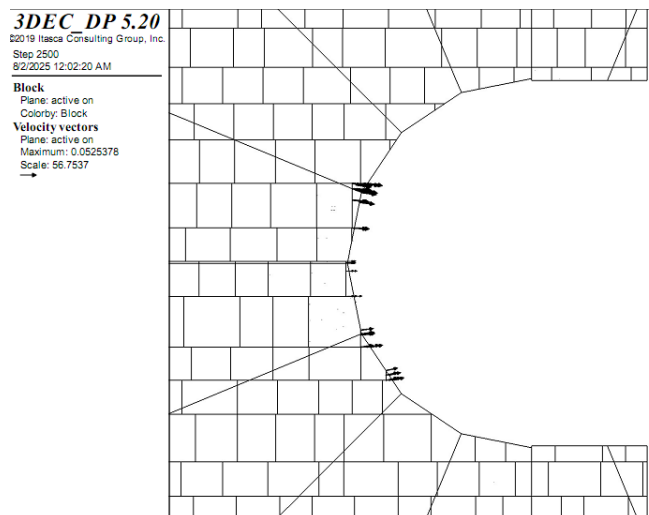


Fig. 13. 3DEC plot of velocity vectors on a plan section through the pit – rigid blocks

5. Conclusion

In Vietnam, the collection of laboratory rock data, conversion to rock mass parameters accounting for fracturing, and interface characterization are not yet widely adopted. In this study, the authors used custom-built equipment to conduct uniaxial compression and Brazilian splitting tensile tests on rock samples. A procedure is also proposed for converting the parameters of the Mohr–Coulomb model from rock specimens to corresponding rock mass parameters. Fracture data were obtained based on field observations and relevant technical guidelines.

Using numerical modeling with the Itasca 3DEC software, the parameters of the fractured rock mass and the interfaces between rock blocks were incorporated. The 3D slope stability was then calculated, taking into account the interaction between these blocks. This modeling approach can be extended to real-world scenarios involving mixed soil–rock media and other complex geological conditions.

Acknowledgements

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