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Numerical determination of deformability and strength of 3D fractured rock mass

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ABSTRACT

We used a systematic numerical method to determine deformability and strength parameters of a fractured rock mass with irregular and stochastic fracture systems in 3D. In this case, different DFN realizations were generated based on Monte Carlo simulation and then compliance tensors and Representative Elementary Volumes (REV) for rock mass deformability and strength were determined in 3D. Stress-dependent strength of fractured rock mass was examined and different theoretical and empirical failure criteria were adopted. The numerical modeling results show anisotropy in deformability parameters in different directions. The obtained rock mass deformability moduli and peak strength of rock mass are reduced expectedly compared to intact rock. The results were compared to the analysis in 2D which had been performed in the previous stages of this research work. The approximated REV of rock mass deformability and strength properties in three-dimensions were found to be smaller than those in 2D numerical simulations. The results of rock mass behavior under different sets of confinement stresses indicate that characteristic parameters are completely stress-path-dependent. The approximated characteristic parameters from Empirical Hoek & Brown (EHB) criterion shows that rock engineering design based on EHB generally is more conservative. The Mogi-Coulomb criterion, considering the intermediate principal stress, estimates different values of cohesion and friction angle of rock mass compared to the conventional and extensional tri-axial tests. Therefore, care should be taken when such parameters are used in rock engineering projects.

1. Introduction

Evaluating mechanical parameters of fractured rock mass is necessary in designing surface and underground structures in rock engineering, particularly underground reservoirs of water, oil and gas, geothermal reservoirs, road tunnels, water transmission tunnels, and dam sites. Fractured rock mass, which is naturally a discontinuum and complex medium, consists of intact rock matrix and discontinuities and behaves as a discontinuous, inhomogeneous, anisotropic, and nonlinear elastic (DIANE) medium.[1] As a consequence, a rock mass is usually weaker, anisotropic, and more deformable than an intact rock. Generally, in fractured rock masses in which the deformability of intact matrix is small compared to displacements of fractures, the deformability and strength behavior of rock mass is mainly affected by discontinuities and geometrical properties of fracture networks. Due to the fact that stress distribution in a fractured rock can be controlled by geometrical parameters of a Discrete Fracture Network (DFN) such as trace length, orientation, and position of fractures, the mechanical behavior of rock mass is controlled by fracture patterns. It should be noted that, theoretically, acquiring realistic results for mechanical behaviors of fractured rocks needs large volumes of rock mass containing fractures to be tested at desired stress levels. Existence of large numbers of discontinuities with inherent complex geometrical and mechanical properties could be a drawback for directly measuring mechanical properties of fractured rocks in laboratory or field scale conditions.[2] The considered size of the sample is an effective parameter on mechanical properties of rock mass such as strength and elastic modulus.[3] Empirical[4] and numerical[5] studies about the block size effect on mechanical properties of rock mass, show that with increasing rock mass volume with a large number of discontinuities, usually aforementioned parameters are decreased significantly. Even though, the density of fractures remains constant, still the size effect on mechanical properties exists. DFN models are generated by different statistical distribution functions of geometrical parameters of fracture network. In this research, stochastic modeling using different realizations of fracture networks with various sizes was conducted. Therefore, it is necessary to

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determine the Representative Elementary Volume (REV) of mechanical parameters which is defined as a minimum volume of a sampling domain beyond which the properties remain constant.[6] Determination of rock mass equivalent mechanical properties and behavior in field scale is difficult, expensive, time consuming, and needs accurate control of boundary conditions. Moreover, it includes largely unknown effects of the fractures nearby the test site and hidden fractures which induce a high degree of uncertainty. Nevertheless, it is necessary to put a precise interpretation on the results.[2] Various indirect methods including empirical, analytical and numerical methods are available for determination of equivalent mechanical properties of rock masses. In the literature, some of the most common empirical equations have been reviewed.[7,8] The evaluated results by empirical equations are based on rock mass classifications[9,10] and consider a rock mass as an isotropic and continuum medium. Analytical methods are mostly adopted on a regular fracture sets and consider a rock mass as a continuous anisotropic medium.[11,12] Obviously, both aforementioned methods suffer from simplicity and unrealistic assumptions and cause an unknown degree of uncertainty.

Nowadays, numerical methods are able to calculate the deformability and strength of fractured rock mass considering a more realistic fracture network and interaction of intact matrix and discontinuities when constitutive models are well identified. Some of the main numerical methods are Finite Element Method (FEM), Boundary Element Method (BEM) and Discrete Element Method (DEM). In,[13] FEM was used to determine the strength and deformability of fractured rock mass in 2D models via implementing a damage-softening statistical constitutive model of intact rock. However, finite element and boundary element methods are usually limited in characterization and number of discontinuities in addition to simulating the large displacements and rotations of rock blocks that generally occur in fractured rock masses. DEM which was firstly introduced by Cundall in 1971[14] and later was developed by Lemos et al.,[15] is a beneficial approach which explicitly simulates complex geometries for fracture networks with simple or complicated constitutive models of intact rock and fractures. Moreover, large deformations are possible in discontinuities.[16]

A number of studies were carried out to investigate the mechanical behavior of intact rock[17] as well as rock mass as a discontinuum medium with one or a few regular joint sets with oversimplified assumptions[12,18,19] which may not represent a realistic complex fracture network geometry. Therefore, a hybrid DFN-DEM[20] modeling which considers a more realistic geometrical model with a sophisticated numerical method is more eligible.

In the literature, the DFN-DEM is used to determine equivalent elastic parameters of fractured rock mass with representation of DFN explicitly alongside investigating the application of equivalent continuum method in modeling the mechanical behavior of rock mass. It is proved that determining rock mass elastic parameters is approximately possible by compliance tensor through numerical simulations.[21] Along completion of previous studies, this method is used to investigate the effects of fracture intensity on fractured rock mass deformability parameters and elastic strength in 2D and consequently the compliance tensor and REV were determined.[22,23] In[24] variations of rock mass deformability modulus and the rock mass Poisson's ratio under different confining pressures, loading directions and water pressure conditions were determined. Also, the effects of axial stress loading and axial velocity loading boundary conditions on the strength and stress-strain behaviors of fractured rock mass were studied using aforementioned method in two dimensions.[24] A comparison between 2D and 3D numerical modeling for a fractured rock mass with a special 3D realization model of fractures has been conducted and reported in.[25]

In earlier studies for determining the rock mass deformability and strength characteristics, as previously mentioned, either the medium is a simplified version of real cases[26] or with plain strain assumptions where it is attempted to extend the 2D parameters to the third dimension. In addition, in three dimensional studies, the fractured rock mass is considered only with a unique pattern of discontinuities.[25]

The objective of this paper is to model a fractured rock mass in 3D medium with multiple realizations of fractures as well as to find the REV for deformability and strength parameters. In this case, various realizations of DFN models in 3D were generated using the geometrical data of a specific site and the block models of rock mass were constructed according to these fracture patterns. The complete compliance tensors in 3D were calculated by applying appropriate boundary conditions using 3DEC software and the REV for deformability of rock mass in three principal directions were determined. The REV for rock mass strength was determined and stress-dependent strength was studied and fitness of different failure criteria on the numerical results was examined. Finally, the obtained results were compared with reported 2D simulation results in the literature.[22,23]

2. Numerical methodology for determining mechanical properties of a 3D fractured rock mass

2.1. Constitutive models of anisotropic elastic material and compliance tensor

By considering the rock mass as an equivalent continuum medium for assessment of the overall deformability of rock masses, it is possible to define some relations between the stresses, strains, geometrical, and mechanical properties of rock discontinuities. In an anisotropic elastic material, the strain tensor (ε_{ij}) is generally related to stress tensor (σ_{kl}) according to the following stress-strain law[27]:

$$\varepsilon_{ij} = S_{ijkl}\sigma_{kl} \tag{1}$$

where S_{ijkl} is a fourth-order tensor known as compliance tensor including 36 independent components. Eq. (1) can be expressed as follows,[28,29] in which the ε_{ii}^m is the strain component ($i = x,y,z$), γ_{ij}^m is the engineering shear strain ($i,j = x,y,z$), and σ_{ij}^m is the stress component ($i,j = x,y,z$) for $m = 1,\ldots,6$:

$$\begin{bmatrix} \varepsilon_{xx}^1 & \varepsilon_{xx}^2 & \varepsilon_{xx}^3 & \varepsilon_{xx}^4 & \varepsilon_{xx}^5 & \varepsilon_{xx}^6 \\ \varepsilon_{yy}^1 & \varepsilon_{yy}^2 & \varepsilon_{yy}^3 & \varepsilon_{yy}^4 & \varepsilon_{yy}^5 & \varepsilon_{yy}^6 \\ \varepsilon_{zz}^1 & \varepsilon_{zz}^2 & \varepsilon_{zz}^3 & \varepsilon_{zz}^4 & \varepsilon_{zz}^5 & \varepsilon_{zz}^6 \\ \gamma_{yz}^1 & \gamma_{yz}^2 & \gamma_{yz}^3 & \gamma_{yz}^4 & \gamma_{yz}^5 & \gamma_{yz}^6 \\ \gamma_{xz}^1 & \gamma_{xz}^2 & \gamma_{xz}^3 & \gamma_{xz}^4 & \gamma_{xz}^5 & \gamma_{xz}^6 \\ \gamma_{xy}^1 & \gamma_{xy}^2 & \gamma_{xy}^3 & \gamma_{xy}^4 & \gamma_{xy}^5 & \gamma_{xy}^6 \end{bmatrix}$$

$$= \begin{bmatrix} S_{11} & S_{12} & S_{13} & S_{14} & S_{15} & S_{16} \\ S_{21} & S_{22} & S_{23} & S_{24} & S_{25} & S_{26} \\ S_{31} & S_{32} & S_{33} & S_{34} & S_{35} & S_{36} \\ S_{41} & S_{42} & S_{43} & S_{44} & S_{45} & S_{46} \\ S_{51} & S_{52} & S_{53} & S_{54} & S_{55} & S_{56} \\ S_{61} & S_{62} & S_{63} & S_{64} & S_{65} & S_{66} \end{bmatrix} \begin{bmatrix} \sigma_{xx}^1 & \sigma_{xx}^2 & \sigma_{xx}^3 & \sigma_{xx}^4 & \sigma_{xx}^5 & \sigma_{xx}^6 \\ \sigma_{yy}^1 & \sigma_{yy}^2 & \sigma_{yy}^3 & \sigma_{yy}^4 & \sigma_{yy}^5 & \sigma_{yy}^6 \\ \sigma_{zz}^1 & \sigma_{zz}^2 & \sigma_{zz}^3 & \sigma_{zz}^4 & \sigma_{zz}^5 & \sigma_{zz}^6 \\ \sigma_{yz}^1 & \sigma_{yz}^2 & \sigma_{yz}^3 & \sigma_{yz}^4 & \sigma_{yz}^5 & \sigma_{yz}^6 \\ \sigma_{xz}^1 & \sigma_{xz}^2 & \sigma_{xz}^3 & \sigma_{xz}^4 & \sigma_{xz}^5 & \sigma_{xz}^6 \\ \sigma_{xy}^1 & \sigma_{xy}^2 & \sigma_{xy}^3 & \sigma_{xy}^4 & \sigma_{xy}^5 & \sigma_{xy}^6 \end{bmatrix} \tag{2}$$

To calculate such matrices' components, solving a set of equations with 36 unknowns is mathematically vital. Therefore, six linearly independent stress boundary conditions are sufficient to determine all components in the matrix shown in Eq. (2). After each set of loading boundary conditions, one column of stress and strain matrices will be completed gradually. Finally, after six distinct sets of loading, the stress and strain matrices are completed and all of the components of compliance tensor and consequently deformation parameters in all directions are calculated.

In this case, six linear independent stress boundary conditions (BC) are applied on 3D models as shown in Fig. 1. In the BC 1, cubic samples are loaded symmetrically by axial stresses on all faces as a tri-axial normal stress test. The first BC is considered to load the models in x-direction; however, to restrict the model boundaries in all six directions

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