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# Modelling progressive failure in fractured rock masses using a 3D discrete element method

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ABSTRACT

Instabilities in rock structures involve coupled mechanisms related to both deformations along existing discontinuities and brittle fracture of intact rock. Conventional kinematic and limit equilibrium techniques used to study rock slope stability suffer from oversimplifications. Mass strength degradation and progressive failure mechanisms in rock bridges cannot be ignored and must be considered to predict the overall slope behaviour. A 3D numerical model based on the discrete element method has been developed to overcome these limitations. Pre-existing discontinuities as a Discrete Fracture network (DFN) can be initially plugged into a set of discrete elements combined with the use of a modified contact logic which provides an explicit representation of joints. Both fracturing of intact material and yielding within discontinuities can therefore be reproduced, depending on the loading conditions and material strength. Simulations of referenced experimental tests are presented here to show the capabilities of the model in tackling the failure mechanisms of intact rock in the presence of pre-existing discrete fractures, with an emphasis on the initiation and propagation processes. This model proves to be a promising tool in understanding and predicting instabilities that could lead to the failure of fractured rock slopes.



## 1. Introduction

Instabilities occurring in rock slopes are generally related to the presence of pre-existing discontinuities and the limit equilibrium method which involves a representation of the slope as a set of rigid blocks remains the most commonly adopted approach to assess potential slope failure [1]. However, back analyses have shown that for numerous cases, non-persistent discontinuities (or often observed joints) are involved in the failure process and it is the breakage of rock bridges between those joints that is mainly at the origin of the rock mass destabilization [1–3]. This progressive failure mechanism would also explain the time delay between the initial localised surface deformation and the entire slope failure. Thus, if in most cases progressive failure mechanisms of rock bridges lead to global failure, a model to study rock slope stability should encompass the nucleation or activation of cracks within the rock matrix and their possible coalescence which would then lead to the creation of critical fractures connecting the pre-existing ones.

Different techniques have been developed to deal with sets of non-persistent discontinuities. For example, using stochastic techniques, Einstein et al. [4] made an attempt to relate rock mass stability with persistence in the geometry and the spatial variability of discontinuities. However, this approach was based on the limit equilibrium analysis and therefore, remained limited in reproducing and understanding the progressive nature of slope failure. More recently, numerical methods have led to significant enhancements in rock slope stability analysis, wherein some of the characteristic features of a fractured rock mass (e.g., Discrete Fracture Network, anisotropy, 3D effects or non-linear behaviour, hydro-mechanical coupling). Although classical continuous or discrete approaches in their initial formulation do not seem adapted to describe the progressive failure mechanisms in jointed rock, several attempts have been made to extend their capabilities. For example, Wang et al. [5] demonstrated that the application of a particle flow code can provide valuable insights into the stability analysis of heavily jointed rock slopes. Eberhardt et al. [2] have shown that a coupled FEM/DEM formulation [6,7] can reproduce observed failure mechanisms by taking advantage of both continuous and discrete approaches (case study: 1991 Randa rockslide). The RFPA code recently developed by Tang et al. has also been enhanced to simulate the actual failure of fractured rock slopes [8]. However, to our knowledge, all these previous studies were performed in 2D. A further step is therefore to develop 3D

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models that can reproduce the complex combination of intact material fracturing and yielding within discontinuity planes. Using a discrete approach, a 3D simulator for jointed rock slopes which accepts DFN has been recently developed by Itasca [9]. It is based on a lattice formulation where fractures can propagate through the intact medium. Although this new approach seems very promising for its dedicated purpose (i.e. large scale simulations), it still needs to be thoroughly tested before real stability studies should be considered. As part of this validation process, the objective of the present work is to show that this type of approach (i.e. discrete model) can provide an accurate and reliable description of the key mechanisms that lead rock slopes to failure.

A 3D model for fractured rock is presented here. Implemented into YADE Open DEM [10,11], the model uses a discrete representation of the intact rock mass, in which a DFN can be plugged in a straightforward way as a set of planes representing the discontinuities (joints). These joints can then be simulated with both rough surfaces resulting from DE shapes, or using a modified contact logic where interactions between Discrete Elements (DE) are setup depending on the orientations of the joint surfaces [12].

First, the formulation of the model will be given and its ability to simulate a typical rock-like behaviour will be shown. Then, uniaxial compression tests on a pre-flawed sample will be presented to emphasise the relevance of the model in reproducing the so-called "wing crack" extensions usually observed around penny shaped cracks. The model capabilities in terms of crack propagation and coalescence will then be discussed on the basis of direct shear test simulations performed on a discontinuous joint made up of pre-existing "en echelon" cracks as depicted in Gehle and Kutter experiments [13]. Finally, model predictions will be investigated at the scale of a rock slope with an emphasis on its capacity to reproduce two of the keys mechanisms usually involved in the development of the progressive failure surface, namely step-path and flexural toppling failure modes.

## 2. Formulation of the model

The algorithm used in the present Discrete (or Distinct) Element Method (DEM) [14], involves two steps. First, based on constitutive laws, interaction forces between Discrete Elements (DE) are computed. Second, Newton's second law is applied to determine, for each DE, the resulting acceleration, which is then time integrated to find the new position. This process is repeated until the simulation is finished. All developments and simulations described in this paper have been implemented into the YADE Open DEM platform [10,11].

### 2.1. Constitutive law

For small deformations, rock exhibits a linear elastic response characteristic of geomaterials. To reproduce this behaviour, linear elastic interaction forces between the discrete elements are sufficient. In the present model, the interaction force $F$ which represents the action of DE $a$ on DE $b$ may be decomposed into a normal force $F_n$ and a shear force $F_s$ which are related to the relative normal and incremental shear displacements through the stiffnesses, $K_n$ and $K_s$, in the normal and the tangential directions, respectively.

Once the DE's packing has been generated to form the numerical sample, pairs of initially interacting Discrete Elements are identified within an interaction range $\gamma_{\text{int}}$, such that,

$$D_{eq} = \gamma_{\text{int}}(R_a + R_b) \tag{1}$$

where $D_{eq}$ is the initial equilibrium distance, $R_a$ and $R_b$ the radius of elements $a$ and $b$ respectively, and $\gamma_{\text{int}} \geq 1$.

Contrary to classical DEM which only considers strictly contacting DE (i.e. $\gamma_{\text{int}} = 1$), the initial number of interacting links (also referred to here as the coordination number) can thus be controlled, whatever the granular packing, by simply varying $\gamma_{\text{int}}$ before the first timestep of the computation cycle [15].

The normal interaction force is calculated through the local constitutive law (see Fig. 1) and can be split into two parts, the compressive and the tensile components. In compression, $F_n$ is linear and it is given by,

$$F_n = K_n \ \Delta D \tag{2}$$

where $K_n$ is the normal stiffness and $\Delta D$ is equal to the relative displacement between the interacting DE defined as $\Delta D = D - D_{eq}$, with $D$ being the distance between the centroids of the interacting DE.

In tension, the normal interaction force is also linear using the same stiffness for small deformations. The maximum acceptable tensile force $F_{n,\max}$ is defined as a function of the tensile strength $T$ such that,

$$F_{n,\max} = -TA_{\text{int}} \tag{3}$$

where $A_{\text{int}} = \pi(_{\min}(R_a, R_b))^2$ is the interacting surface between DE $a$ and $b$.

After the tensile force reaches its maximum value, the stiffness may be modified by a softening factor $\zeta$ to control the energy released due to link breakage:

$$F_n = (\Delta D - D_{rupture})\frac{K_n}{\zeta} \tag{4}$$

When $\Delta D > D_{rupture}$, rupture occurs and interaction forces are set to zero.

The shear force $F_s$ is computed in an incremental manner by updating its orientation and intensity depending on the increment of shear force $\Delta F_s = K_s \Delta u_s$ which develops at the interaction point, as defined in [16]:

$$F_s = \{F_s\}_{updated} + K_s \Delta u_s \tag{5}$$

with $K_s$ being the shear stiffness and $\Delta u_s$ the relative incremental tangential displacement.

To model the nonlinear behaviour of geomaterials, a modified Mohr–Coulomb model is used (Fig. 2). The maximum shear force $F_{s,\max}$ is characterised by the normal force $F_n$, the cohesion $C$, the local frictional angle $\varphi_b$ and the local residual frictional angle $\varphi_c$.

The maximum shear force is calculated for a bonded interaction according to,

$$F_{s,\max} = F_n \tan \varphi_b + CA_{\text{int}} \tag{6}$$

whereas only purely frictional new contacts can appear during the simulation, with a maximum shear force defined as,

$$F_{s,\max} = F_n \tan \varphi_c \tag{7}$$

Because the proposed DEM is a fully dynamic formulation, a local non-viscous type damping is used in the model in order to

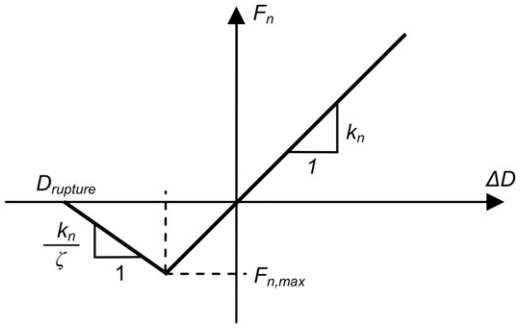


**Fig. 1.** Normal interaction force between DE.