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Research Paper

Characterization of strength and damage of hard rock pillars using a synthetic rock mass method

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ABSTRACT

In this research a 3D Synthetic Rock Mass (SRM) method is used to numerically characterize the strength and damage of hard rock pillars. The SRM is an integrated model incorporating a Discrete Fracture Network (DFN) within a Particle Flow Code 3D (PFC3D) particle assembly. Based on the numerical results of a joint-free pillar model, laterally fixed loading platens are suggested to simulate uniaxial compression tests on rock pillars. An internal-strain loading method is meanwhile used to ensure more realistic model behaviour. The peak strength, post-peak strain-softening gradient and deformation modulus of a series of jointed pillar models are then quantified, in order to investigate the effects of the inserted joint sets. The simulated peak strengths demonstrate a U-shape relationship when the joint sets are rotated; the peak strength also decreases with increasing joint size. A brittle post-peak behaviour is observed for pillar models with vertical joint sets of low persistence, the post-peak behaviour becoming more ductile when the joint sets are inclined and of higher persistence. A correlation is identified between the post-peak pillar behaviour and the simulated tensile cracking events, where a brittle post-peak corresponds to a high cracking rate. The effects of the joint sets on the pillar deformation modulus are observed to be similar to the effects on the pillar peak strength. Particular attention is given to the characterization of the crack initiation stress (σ_{ci}) and crack damage stress (σ_{cd}) thresholds of each pillar model, where the ratio of the crack initiation stress/peak strength is between 0.3 and 0.45, and the ratio of the crack damage stress/peak strength is between 0.75 and 0.98. The simulated cracks are compared between the jointed pillars and detailed cracking modes are plotted as 3D views and as 2D thin layers for selected pillar models.

1. Introduction

Pillar strength and associated damage characteristics are of major importance in hard rock mine design. To date, there is however arguably no accurate and efficient in situ testing method with which to characterize the pillar strength. Numerical methods are therefore increasingly applied to provide a better understanding of the pillar mechanical behaviour under external loading. In most models the pillar is treated as a continuum material, where in comparison with the rock sample strength, the pillar strength is reduced in order to implicitly involve pre-existing joint sets [1–9]. Although the continuum method can be efficient in simulating the pillar strength and damage mechanisms, it may not be appropriate when pre-existing joints impose a significant kinematic influence on the pillar stability. For these pillars, discontinuum and hybrid numerical models which support explicit joint insertion may be preferred options. As a demonstration, Elmo and Stead [10] emphasized the effects of the joint orientation and size on mine-based pillar models using a hybrid ELFEN code [11]. The results were encouraging, but the 2D plane strain assumption may not always accurately reflect the pillar failure characteristic. A more realistic pillar model should consider 3D deformation by explicitly incorporating non-persistent joint sets.

In order to explore a more robust pillar design tool, we use a state-of-the-art 3D Synthetic Rock Mass (SRM) approach to characterize the rock pillar strength and damage [12–16]. The importance of the model boundary condition and loading scheme is initially emphasized based on the results of uniaxial compression tests on a joint-free pillar. The mechanical properties of a series of jointed pillars are then characterized to demonstrate the effects of the joint set orientation and size, including peak strength, post-peak

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strain-softening gradient and deformation modulus. Particular attention is focussed on characterization of pillar stress thresholds during the compressive loading process. Simulated cracking modes are finally plotted for selected pillar models.

2. The SRM model

2.1. Basic concept and configuration

The SRM model is constructed by integrating a Discrete Fracture Network into a PFC3D-based particle assembly [17]. Using non-deformable particles as the basic element, the SRM simulates elastic and plastic deformation through particle overlap. The particles are generally bonded together for intact rock blocks, and a crack is simulated when the external stress exceeds the assigned bond strength. The particles are also assigned a friction coefficient to control the particle movement after bond breakage. This is suggested to be a more realistic configuration for hard rock failures, as the friction component of a rock is not mobilized unless the rock is sufficiently fractured to allow for relative shear displacement [18–21]. The SRM focuses on particle scale failure and requires no constitutive mechanical model, simulation involving automatic strength weakening in the post-peak stage. A calibration process is however required for intact rock parameters to fit the rock properties measured in the laboratory [22]. A Coulomb sliding criterion is used to control the strength of joints in the DFN model, and the joint properties are usually characterized based on field measurements. A "smooth joint" logic is applied when inserting a DFN model into a particle assembly, overcoming the bumpiness of the joint surface encountered in earlier versions of PFC models. When a "smooth joint" is assigned to a particle contact, the two particles are allowed to slide smoothly and the sliding direction is controlled by the "smooth joint" orientation. The original contact properties are meanwhile replaced by the properties of the "smooth joint".

The boundary condition and loading scheme are two important factors in rock pillar simulations. The stiffness of loading platens has a significant influence on the model behaviour, as verified by Tang et al. [23]. The platen stiffness variation essentially changes the friction resistance between the platens and model ends, which in turn controls the model numerical behaviour. In order to eliminate numerical uncertainties caused by the boundary condition, we suggested using platens that are directly bonded to the pillar ends, where the difficulty of determining the friction coefficient is avoided. The deformation modulus of the loading platens is the same as the modulus of the intact rock within the pillar. Considering the host rocks confining pillar ends in practice, this configuration could be more appropriate than separated platens. In addition to the boundary condition, the loading scheme has an important influence on the simulated pillar behaviour. The pillar can be uniaxially compressed at a constant loading rate using platens (regular loading), or it can be compressed using a more efficient and accurate internal-strain loading method [13,15–16]. The entire loading process of the internal-strain loading is divided into several load-relaxation cycles. In each cycle the pillar is compressed at a high loading rate for a small amount of the axial strain, and then sufficiently relaxed to simulate the static pillar behaviour. The monitored stresses and strains are retained only after each relaxation cycle. Detailed comparisons of the regular loading and internal-strain loading methods will be presented in Section 3.

2.2. Pillar model setup

A prism-shaped joint-free pillar was initially constructed as a base element of the subsequent jointed pillars (Fig. 1(a)). The pillar Width/Height, W/H, ratio was selected as 0.57 (W/H = 4.0 m/ 7.0 m). Two 0.3 m thick layers of particles are placed at both the pillar top and bottom to simulate the host rocks and loading platens simultaneously. The axial stress is monitored within a large measurement sphere, and the axial strain calculated from the two loading platens. The platen-based axial stress was found to be less robust in comparison with the sphere-based stress measurement due to the relatively random definition of the contact area between the platens and pillar ends. The platen-based axial strain measurement is selected due to the lower computing expense. Target mechanical properties of the pillar intact rock are a Uniaxial Compressive Strength (UCS) of 122.0 MPa, a deformation modulus of 86.1 GPa and a Poisson's ratio of 0.28. These rock properties used for particle parameter calibration are based on laboratory tests on limestone samples from the Viburnum Trend in Southeast Missouri, USA [16]. The particle properties, including particle size, stiffness, bond strength and friction coefficient, are calibrated until comparable macroscopic rock properties are achieved (Table 1). The particle radius is varied from 4.0 cm to 8.0 cm considering the pillar scale, in order to ensure a balance between the model resolution and run time. General guidelines for calibrating the particle parameters are: (i) pillar strength is positively correlated to the bond strength and friction coefficient between particles; (ii) pillar modulus increases with higher stiffness assigned to particles and bonds; (iii) Poisson's ratio increases with a higher ratio of normal/shear stiffness of the particles and bonds [22].

After the joint-free pillar construction, a SRM pillar can be built by incorporating a DFN into the particle assembly. For real pillar simulations, the distribution of each joint set is determined using the scanline mapping on simulated DFN traces in the same manner as in the in situ mapping campaign [12]. For conceptual pillar simulations in this paper, the joint distribution forms are directly assumed. Eighteen SRM pillar models were constructed to allow for a systematic investigation of the pillar strength and damage assuming varied joint set input. Each inserted DFN contains 450 joints, representing three joint sets (A, B and C). This was clearly a simplification of complex real joint networks, and was implemented to ensure practical computing times for the preliminary models, while still capturing realistic pillar behaviour (the run time of each pillar model is about 60 h using a Dell workstation computer). Each DFN was generated in a zone chosen to be two times greater in dimension than the pillar scale, in order to involve sufficient joints with centres outside the pillar zone. Joint sets A and B have the same dip, and dip directions of 0° and 90° respectively. Joint set C is orthogonal to the joint sets A and B. No orientation deviation was considered but the joint centres are randomly distributed. The dip of the joint sets A and B was then simultaneously varied from 15° to 90° in order to investigate the joint orientation effect. The joint radius distribution for a real pillar can be assessed by analytical methods [24], provided that the distribution of fracture traces on the pillar surface is known. In this research, a reference radius R and an associated standard deviation for each joint set were directly assumed as 0.500 m and 0.500 m respectively, and the radius was assumed to follow a lognormal distribution. This distribution was comparable to the joint radius distribution based on mapping on real pillars by Elmo and Stead [10]. An exponential or power-law distribution for the joint radius can also be applied [25–29], but the widely scattered radius values may be inappropriate for the current pillar scale (realised joints may readily cut through the pillar model and cause absolutely structure-controlled pillar failures). The mean joint radius was then decreased/increased to 0.375 m and 0.625 m in order to characterize the effects of the joint size. For convenience in presenting the results, the jointed pillar models were denoted by the dip of the joint sets A/B and mean radius, i.e., R1-45 denotes a pillar with a joint dip of 45° and R 0.375 m; R3-75 represents a dip of 75° and