DEM analyses of shear behaviour of rock joints by a novel bond contact model

M J Jiang\textsuperscript{1,2,3,5}, J Liu\textsuperscript{1,2,3}, C Sun\textsuperscript{1,2,3} and H Chen\textsuperscript{4}

\textsuperscript{1} State Key Laboratory for Disaster Reduction in Civil Engineering, Tongji University, Shanghai, China, 200092
\textsuperscript{2} Key Laboratory of Geotechnical and Underground Engineering of Ministry of Education, Tongji University, Shanghai, China, 200092
\textsuperscript{3} Department of Geotechnical Engineering, College of Civil Engineering, Tongji University, Shanghai, China, 200092
\textsuperscript{4} Yunnan traffic planning design and research institute, Kunming, China

E-mail: mingjing.jiang@tongji.edu.cn

Abstract. The failure of rock joints is one of the potential causes for the local and general rock instability, which may trigger devastating geohazards such as landside. In this paper, the Distinct Element Method (DEM) featured by a novel bond contact model was utilized to simulate shear behaviour of centre/non-coplanar rock joints. The DEM results show that the complete shear behaviour of jointed rock includes four stages: elastic shearing phase, crack propagation, the failure of rock bridges and the through-going discontinuity. The peak shear strength of centre joint increases as the joint connectivity rate decreases. For intermittent non-coplanar rock joints, as the inclination of the rock joints increases, its shear capacity decreases when the inclination angle is negative while increase when positive. Comparison with the experimental results proves the capability of this DEM model in capturing the mechanical properties of the jointed rocks.

1. Introduction

Shear failure of rock joints can induce the instability of rock mass, a common medium in the field of mining, oil exploitation, water conservancy and hydropower. Thus, the shear behavior of rock joints becomes a topic of the geological engineering. Lajtai [1-2] conducted direct shear tests to examine and analyze the strength of discontinuous rocks, and concluded that the total shear strength is composed of two parts, namely, the joint friction along the separated parts of the weakness plane and the fundamental shear strength and internal friction in solid bridges. Gehle and Kutter [3] investigated the shear behavior of intermittent rock joints, arguing that the failure pattern and the shear resistance of jointed rock are significantly affected by both the orientation of the joints and the normal stress. Cao et al. [4] further found that the rock specimen takes a turn from wing crack propagation failure to crack coalescence failure with the increase of the angle of the rock bridge.

Although theoretical and experimental studies have given some promising results of the shearing of rock joints, some very important microscopic information controlling the failure patterns still remain unknown. More and more researchers have used numerical methods to further investigate this problem [5-11]. Among these methods, the Distinct Element Method (DEM) proposed by Cundall and Strack

\textsuperscript{5} To whom any correspondence should be addressed.
[8] is a feasible method to study the shear behavior and failure mechanism of sand and rocks [9-12]. Park et al. [9] examined the effects of the geometrical features and the micro-properties of a joint on its shear behavior by using a bond model. Potyondy and Cundall [10] proposed a numerical model for rock and investigated the sensitivity of the results to micro-properties. Jiang et al. [11] investigated the shear behavior and strain localization in cemented sands by 2D DEM. More recently, Bewick et al. [12] investigated mechanism of rupture by comparing DEM direct shearing result with experimental one under different normal pressures.

In this study, a new DEM bond model proposed by one of the authors has been adopted to simulate mechanical response of jointed rock mass under direct shearing condition. The shear curves and shearing capacities of the DEM virtual rock joints are qualitatively compared with the experimental results. Also, the force chains inside the jointed rock sample are presented to provide a view in the shear behaviour microscopically.

2. DEM bond contact model

The DEM bond contact model is proposed by Jiang et al. [13] for the purpose of simulating heavily cemented materials such as granite. The bond contact model was first derived theoretically [14-16] and then further verified by laboratory tests on bonded aluminum rods [17-18].

The total bond contact is assumed to be composed by two paralleled parts: inter-particle and inter-bond contacts. Contact forces are assumed to be transferred through not only the bond but also the inter-particle contact:

\[ F_n = F_{bn} + F_{pm}, \quad F_s = F_{bs} + F_{ps}, \quad M = M_b + M_p \]

where \( F_{bn}, F_{bs} \) and \( M_b \) are the normal force, the shear force and the moment transferred through the bond, respectively, while \( F_{pm}, F_{ps} \) and \( M_p \) are the corresponding components transferred through the inter-particle contact. The inter-particle contact laws can be approached in reference [15] and the mechanical response of inter-bond contact can be expressed as:

\[
F_{bn} = \begin{cases} 
\min[k_{bn}u_n, R_{bn}], & (u_n \geq 0) \\
\min[k_{bn}u_n, R_{bn}], & (u_n < 0) 
\end{cases}
\]

\[ F_{bs} = \min[k_{bs}u_s, R_{bs}] \quad (3) 
\]

where \( u_n, u_s, \) and \( \theta \) are the overlap, the shear displacement and the relative rotation angle, respectively; \( k_{bn}, k_{bs}, k_{bm} \) are the normal, shear and rolling contact stiffness of bonds, respectively; \( R_{bc}, R_{bs}, R_{bs} \) and \( R_{tu} \) are the compressive, tensile, shear and rolling bond strengths, respectively.

The projections of the strength envelope in \( F_c F_n, M F_n \) planes and critical normal force \( R_c \) can be expressed as:

\[ R_c = \mu_f R_{bc} [1 + g_f (\ln \frac{1}{f})^\alpha] + \mu_p F_{ps} \]

\[ R_v = \frac{1}{6} \beta_p f R_{bc} [1 + g_f (\ln \frac{1}{f})^\alpha] + \frac{1}{6} \beta_p f R_{pm} F_{bs} = \min[k_{bn}u_s, R_{bs}] \]

where \( f = (F_{bn} + F_{ps})/(R_{bc} + R_{ps}) \) is the stress ratio, which can be used to define the types of bond failures: when \( f = 0 \), compressive failure occurs; when \( 0 < f < 1 \), tensile-shear-torsional failure \((F_n < 0)\) or compressive-shear-torsional failure \((F_n > 0)\) occurs; when \( f = 1 \), tensile failure occurs; \( \mu_f \) and \( \mu_p \) are friction coefficients of bonds and particles; \( \beta_p \) and \( \beta_f \) are rolling resistance coefficients of bonds and particles; \( F_s, F_c, g_s \) and \( g_f \) are the envelope shape factors; \( R_c \) is the common radius of two particles in contact and \( k_{pm} \) is inter-particle normal contact stiffness. It should be noted that the strength in Eqs. (5)
and (6) includes both of the components of bond and inter-particle contacts when \( F_n > 0 \), which means that when \( F_n < 0 \) the second term disappears as inter-particle contact cannot transfer tensile force.

If forces reach strength envelope/critical values, they will fall to the residual state. The residual shear strength \( R_{sr} \) and rolling strength \( R_{rr} \) can be expressed by:

\[
R_{sr} = \mu_b F_{ts} + \mu_p F_{pm}
\]

\[
R_{rr} = \frac{1}{6} \beta_p r F_{ts} + \frac{1}{6} \beta_p r F_{pm}
\]

which also contain two terms corresponding to bond and inter-particle components.

3. DEM modelling

The DEM rock sample containing 10,000 particles was generated by Under-compaction Method (UCM) proposed by Jiang et al. [18], with maximum, minimum and average diameters being 2.0, 0.5 and 1.3 mm respectively. The size of the specimen is 12.23 cm×7.82 cm with planar void ratio being 0.20. The synthetic intact rock material has already been calibrated by comparison with uniaxial compression and Brazilian test results of granite [19]. A set of joints in the given arrangement are then generated within the rectangular DEM sample, and the joints were identified by a 2 mm thick layer of particles including no bond effect. The assigned parameters for both intact rock and joints are listed in Table 1.

Table 1 Microscopic material parameters used in DEM simulations

| Parameters                          | Values for intact rock | Value for joint |
|------------------------------------|------------------------|-----------------|
| Particle density \( \rho_s \) (kg/m\(^3\)) | 2700                   | 2700            |
| Planar void ratio \( e \)          | 0.20                   | 0.20            |
| Normal stiffness of particles \( k_{np} \) (N/m) | \( 1.8 \times 10^{11} \) | \( 1.8 \times 10^{11} \) |
| Tangential stiffness of particles \( k_{tp} \) (N/m) | \( 9.47 \times 10^{10} \) | \( 9.47 \times 10^{10} \) |
| Normal stiffness of bonds \( k_{nb} \) (N/m) | \( 9.0 \times 10^{10} \) | \( 9.0 \times 10^{10} \) |
| Tangential stiffness of bonds \( k_{tb} \) (N/m) | \( 4.735 \times 10^{10} \) | \( 4.735 \times 10^{10} \) |
| Tensile strength of bonds \( R_{tb} \) (N) | \( 6.5 \times 10^4 \) | 0.0             |
| Compressive strength of bonds \( R_{bc} \) (N) | \( 8.0 \times 10^7 \) | 0.0             |
| Interparticle friction coefficient \( \mu_p \) | 1.0                   | 0.2             |
| Interparticle rolling resistance coefficient \( \beta_p \) | 1.5                   | 0.0             |
| Friction coefficient of bonds \( \mu_b \) | 0.5                   | 0.1             |
| Rolling resistance coefficient of bonds \( \beta_b \) | 0.5                   | 0.0             |

Figure 1 and Figure 3 present the schematics and the DEM samples with the centre and non-coplanar rock joints respectively. The parameter \( g \) in Fig. 2 is the joint connectivity rate and is determined by Eq. (10):

\[
g = \frac{L_s - L_j}{L_j}
\]

During the direct shear test, shear load was applied to the DEM jointed rock sample by moving walls #2 and #3 towards each other at a quasi-static velocity while maintaining walls #1 and #4 under a constant normal pressure. For the centre joints, the influence of joint connectivity rate (e.g., 0.6, 0.8, 0.9) is investigated with normal stress being 1.0 MPa, while for intermittent non-coplanar joints with a given joint length \( l \) and spacing \( e \), the effect of the inclination of joints \( i \) (varying from -75\(^\circ\) to 90\(^\circ\)) is explored.
4. Results

4.1 Centre joint
Figure 3 illustrates the relationships between the shear stress/number of bond breakages and shear displacements for the specimen with $g = 0.6$, it shows that the deformation and failure process of DEM specimens can be divided into four stages: (1) elastic shearing phase: shear stress increases almost linearly with the increase of shear displacement, while the no bond breakages happen during this shearing phase; (2) crack propagation: as the shear displacement increases, shear stress fluctuates until reaching the peak stress, and a large number of bond breakages occur with tensile-shear-torsional failures dominating in number; (3) the failure of rock bridges: the shear stress quickly drops to a residual value after arriving the peak; (4) through-going discontinuity: the shear stress remains at a relatively low residual value in spite of some fluctuations, during this period of shearing, the number of the microscopic bond breakages consistently grows. The shear behaviour above is quite typical compared with that normally obtained from experimental results [20].

Figure 4 shows the relationships between the peak shear strength and joint connectivity rate, the shear strength in numerical and experimental results are normalized by their own peak stresses for the case with $g = 0.6$ separately. It can be observed that both of the peak shear stresses significantly decrease with the increase of normal stress, and the trends of the DEM and experimental results are similar. This phenomenon can be reasoned that growth of the proportion of joint weakens the shearing capacity within the shear axis, causing the decrease of the shear strength of the jointed samples.
4.2 Intermittent non-coplanar joints

Considering that all the cases with different inclinations share almost the same trend, only the shear curve of non-coplanar intermittent rock joints with inclination $i = 45^\circ$ is showed in Fig. 6. The shear curve of the non-coplanar joints is quite similar to that of the centre ones, except that no apparent yielding when approaching the peak stress. The reason for this might be that the number of the bond breakages rises exponentially when the jointed sample arrives yielding state, so the stress drops sharply simultaneously, but not the case for the centre joint.

Figure 3 Relationship between shear stress/broken bond number and shear displacement by DEM simulation

Figure 4 Relationship between the normalized peak shear stress and the joint connectivity rate

Figure 5 Relationship between shear stress/broken bond number and shear displacement by DEM simulation with joint inclination angle of 45°

Figure 6 The relationship between the joint angle and the normalized peak value of shear stress

Figure 6 illustrates the shear loading capacities of non-coplanar intermittent joint assemblages with different inclinations. The numerical and experimental results are both normalized by the maximum peak stresses obtained in experimental tests (i.e., -30° for experimental results). The results show that in the DEM simulation, the peak stress decrease as the inclination angle changes from -75° to 15°. It reaches its lower limit value at around 15° and then increase till 75°. The experimental results show almost the same trend for inclination angles varying from -30° to 75° except for the values near -45°. The difference can be partially explained that in the DEM simulation, the joints are filled with particles, whereas in the laboratory, the man-made joints are just open cracks with nothing inside.

5. Conclusions

The numerical simulation results indicate that the complete shear behaviour of jointed rock includes elastic shearing phase, crack propagation, the failure of rock bridges and the through-going discontinuity. For centre rock joints, the decrease of the joint connectivity rate will enhance the peak shear stress. As to intermittent non-coplanar rock joints, the relationship between shear stress and shear displacement presents the elastic-brittle-plastic behavior. The shear capacity drops when the joint inclination angle is negative but rises when positive with the increases of rock joints inclination. The failure process of intermittent non-coplanar rock joints begins with the transmission of compressive forces through the tips of rock joints and ends with the breakage of rock bridges. The comparison between DEM and experimental results proves that this bond contact model is capable of describing the shear behaviour of jointed rocks during direct shearing.
Acknowledgements
This research is supported by the Major Project of Chinese National Programs for Fundamental Research and Development (973 Program) (Nos. 2011CB013504 and 2014CB046901), China National Funds for Distinguished Young Scientists (No. 51025932), State Key Lab. of Disaster Reduction in Civil Engineering (No. SLDRCE14-A-04). These supports are greatly appreciated.

References
[1] Lajai E Z 1969 Shear strength of weakness planes in rock Int. J. Rock Mech. Min. 6 499-515
[2] Lajai E Z 1969 Strength of discontinuous rocks in direct shear Géotechnique 19 218-233
[3] Gehle C, Kutter H K 2003 Breakage and shear behavior of intermittent rock joints Int. J. Rock Mech. Min. 40 687-700
[4] Cao P, Liu T Y, Pu C Z and Lin H 2015 Crack propagation and coalescence of brittle rock-like specimens with pre-existing cracks in compression Eng. Geol. 187 113-121
[5] Zhang S, Leech C 1986 FEM analysis on mixed-mode fracture of CSM-GRP Eng. Fract. Mech. 23 521-535
[6] Hayashi K, Ono A, Abe H 1989 BEM Analysis of a Cylindrical Three-Point Bend Specimen with a Chevron Crack for Fracture Toughness Test of Rock JSME Int. J. A-Solid M. 32 427-431
[7] Hatzor Y, Arzi A, Zaslavsky Y and Shapira A 2004 Dynamic stability analysis of jointed rock slopes using the DDA method: King Herod's Palace, Masada, Israel Int. J. Rock. Mech. Min. 41 813-832
[8] Cundall P A, Strack O D L 1979 A discrete numerical model for granular assemblies. Géotechnique 29 47-65
[9] Park J W, Song J J 2009 Numerical simulation of a direct shear test on a rock joint using a bonded-particle model Int. J. Rock Mech. Min. 46 1315-1328
[10] Potyondy D O, Cundall P A 2004 A bonded-particle model for rock Int. J. Rock Mech. Min. 41 1329-1364
[11] Jiang M J, Yan H B, Zhu H H and Utili S 2011 Modelling shear behaviour and strain localisation in cemented sands by two dimensional Distinct Element Method Analyses Comput. Geotech. 38 14-29
[12] Bewick R P, Kaiser P K, Bawden W F and Bahmani N 2014 DEM simulation of direct shear: 1. Rupture under constant normal stress boundary conditions Rock Mech. Rock Eng. 47 1647-1671
[13] Jiang M J, Chen H, Crosta G B 2015 Numerical modeling of rock mechanical behavior and fracture propagation by a new bond contact model Int. J. Rock Mech. Min. (accepted for publication)
[14] Jiang M J, Yu H S, Harris D 2006 Bond rolling resistance and its effect on yielding of bonded granulates by DEM analyses Int. J. Numer. Anal. Met. 30 723-761
[15] Jiang M J, Yu H S, Harris D 2005 A novel discrete model for granular material incorporating rolling resistance Comput. Geotech. 32 340-357
[16] Jiang M J, Yu H S, Leroueil S 2007 A simple and efficient approach to capturing bonding effect in naturally micro-structured sands by discrete element method Int. J. Numer. Meth. Eng. 69 1158-1193
[17] Jiang M J, Sun Y G, Li L Q and Zhu H H 2012 Contact behavior of idealized granules bonded in two different interparticle distances: An experimental investigation Mech. Mater. 55 1-15
[18] Jiang M J, Sun Y G, Xiao Y 2012 An experimental investigation on the mechanical behavior between cemented granule Geotech. Test J. 35 678-690
[19] Jiang M J, Konrad J M, Leroueil S 2003 An efficient technique for generating homogeneous specimens for DEM studies Comput. Geotech. 30 579-597
[20] Martin C D 1993 The strength of massive Lac du Bonnet granite around underground openings: Winnipeg (Canada: University of Manitoba)
[21] Bai S W, Ren W Z, Feng D X and Zhou S H 1999 Research on the strength behavior of rock containing coplanar close intermittent joints by direct shear test Rock Soil Mech. 20(2): 10-16