Integrated techniques for behavior evaluation of large-scale underground openings by using DEM(distinct element method)

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1. INTRODUCTION

Understanding the mechanical behaviors of discontinuous rock masses is important for the development and utilizations of underground engineering such as radioactive waste disposal facilities, power plants and petroleum reservoirs. The deformation and failure behaviors of an underground structure are principally governed by the discontinuities existing in the host rock masses. During the excavation process, the change of stress state (primary the relaxation of stress) in the rock masses surrounding the underground structure may induce the generation of new cracks, which combining with the existing discontinuities could play an important role in endangering the stability of the structure. To effectively assess the stability of underground structures, the thorough understanding of the mechanical behaviors of existing discontinuities as well as the mechanism for the generation and propagation of new cracks is required.

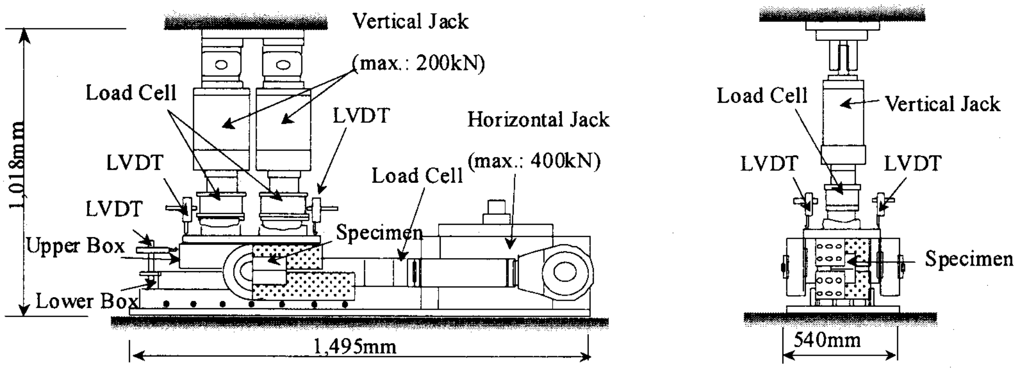
In this study, an automated servo-controlled direct shear test apparatus was developed to accurately assess the mechanical properties of rock joints. This apparatus is capable of shear tests on natural and artificial rock samples on either Constant Normal Load (CNL) or Constant Normal Stiffness (CNS) boundary condition. An Expanded Distinct Element Method (EDEM) was developed for simulating the generation and propagation of new cracks due to the shear and tension failures in the matrix blocks based on the distinct element code UDEC. Using this method, excavation simulations of a deep underground cavern have been carried out on a series of models with differing depths and differing geometrical distributions of existing discontinuities. A new approach to evaluate the fractal feature of jointed rock masses was proposed and the influences of geometrical distribution of rock joints on the behavior of underground openings were evaluated by using DEM.

2. DEVELOPMENT OF A DIRECT SHEAR TEST APPARATUS

Correct assessment of the shear strength of rock joints is essential for many rock-engineering projects. The shear behavior of rock joints is usually investigated in the laboratory by using a direct shear apparatus wherein the forces or stresses acting normal to the direction of shear displacement are kept constant during the shear process. However, they may be quite inappropriate for the situations where the normal stress on the joint surfaces changes considerably during the shear process. Shear testing under a constant normal load (CNL) boundary condition is only beneficial for the cases such as the non-reinforced rock slopes, etc. For deep underground opening or rock anchor-reinforced slopes, however, shear tests under CNL conditions are not appropriate. A more representative behavior of joints would be achieved if the shear tests were carried out under boundary conditions of constant normal stiffness (CNS).

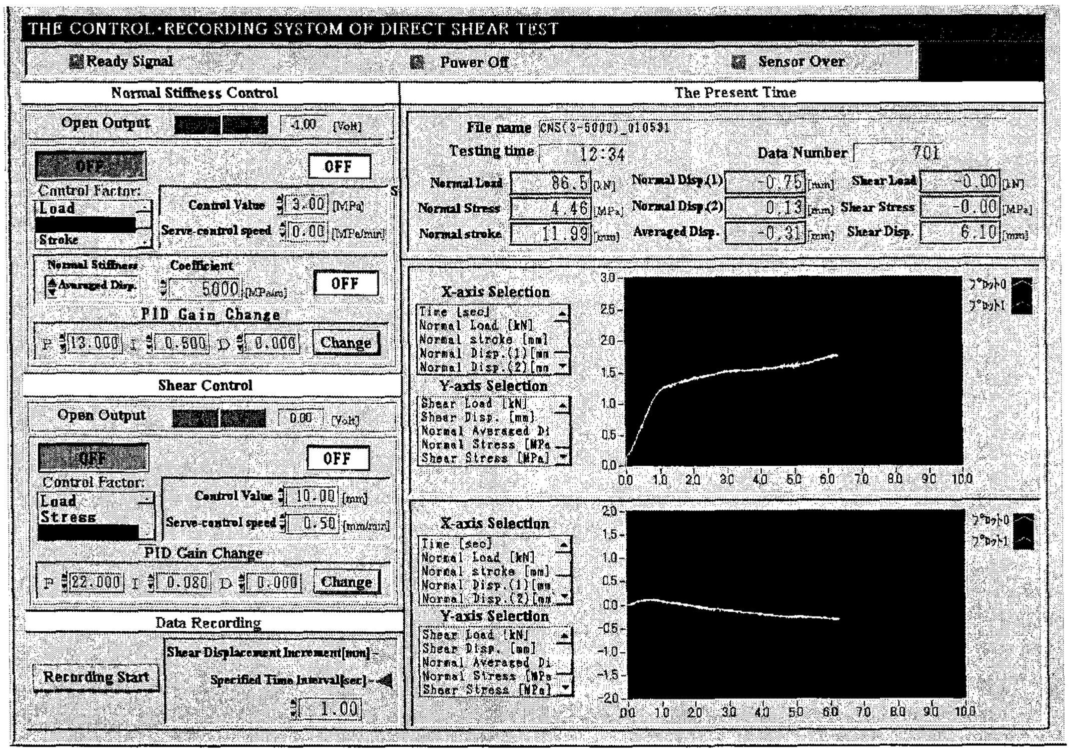
2.1 Hardware of the apparatus

A novel servo-controlled direct shear apparatus shown in Figure 1 is designed and fabricated for the purpose of testing both natural and artificial rock joints under various boundary conditions. It consists of a hydraulic-servo actuator unit, an instrument package unit and a mounting shear box unit. In this novel apparatus, the constant and variable normal stiffness control conditions are reproduced by the digital closed loop control with electrical and hydraulic servos. The control and measurement are carried out on PC Windows, through the multifunction analog-to-digital, digital-to-analog and digital input/output (A/D, D/A and DIO) board, graphical programming language LabVIEW, custom built 'virtual instrument' (VI), and a PID control toolkit. Digital control program was designed by using LabVIEW programming language for building data acquisition and instrumentation systems. With LabVIEW, the interactive control of the system can be created quickly. Figure 2 shows the innermost unit of 'shear main' program



(a) side view (b) front view

Fig.1 Digital-controlled shear testing apparatus.


THE CONTROL·RECORDING SYSTEM OF DIRECT SHEAR TEST

Normal Stiffness Control

Open Output -1.00 [Volt]

Control Factor: Load / Stroke

Control Value 3.00 [MPa]

Serve-control speed 0.00 [MPa/min]

Normal Stiffness: Avaraged Disp.

Coefficient 5000 [MPa/m]

PID Gain Change: P 13.000 I 0.500 D 0.000

Shear Control

Open Output 0.00 [Volt]

Control Factor: Load / Stress

Control Value 10.00 [mm]

Serve-control speed 0.50 [mm/min]

PID Gain Change: P 22.000 I 0.080 D 0.000

Data Recording

Shear Displacement Increment[mm]

Specified Time Interval[sec] 1.00

The Present Time

File name CNS(3-5000)_010531

Testing time 12:34

Data Number 701

Normal Load 86.5 [kN]

Normal Stress 4.46 [MPa]

Normal stroke 11.99 [mm]

Normal Disp.(1) -0.75 [mm]

Normal Disp.(2) 0.13 [mm]

Averaged Disp. -0.31 [mm]

Shear Load -0.00 [kN]

Shear Stress -0.00 [MPa]

Shear Disp. 6.10 [mm]

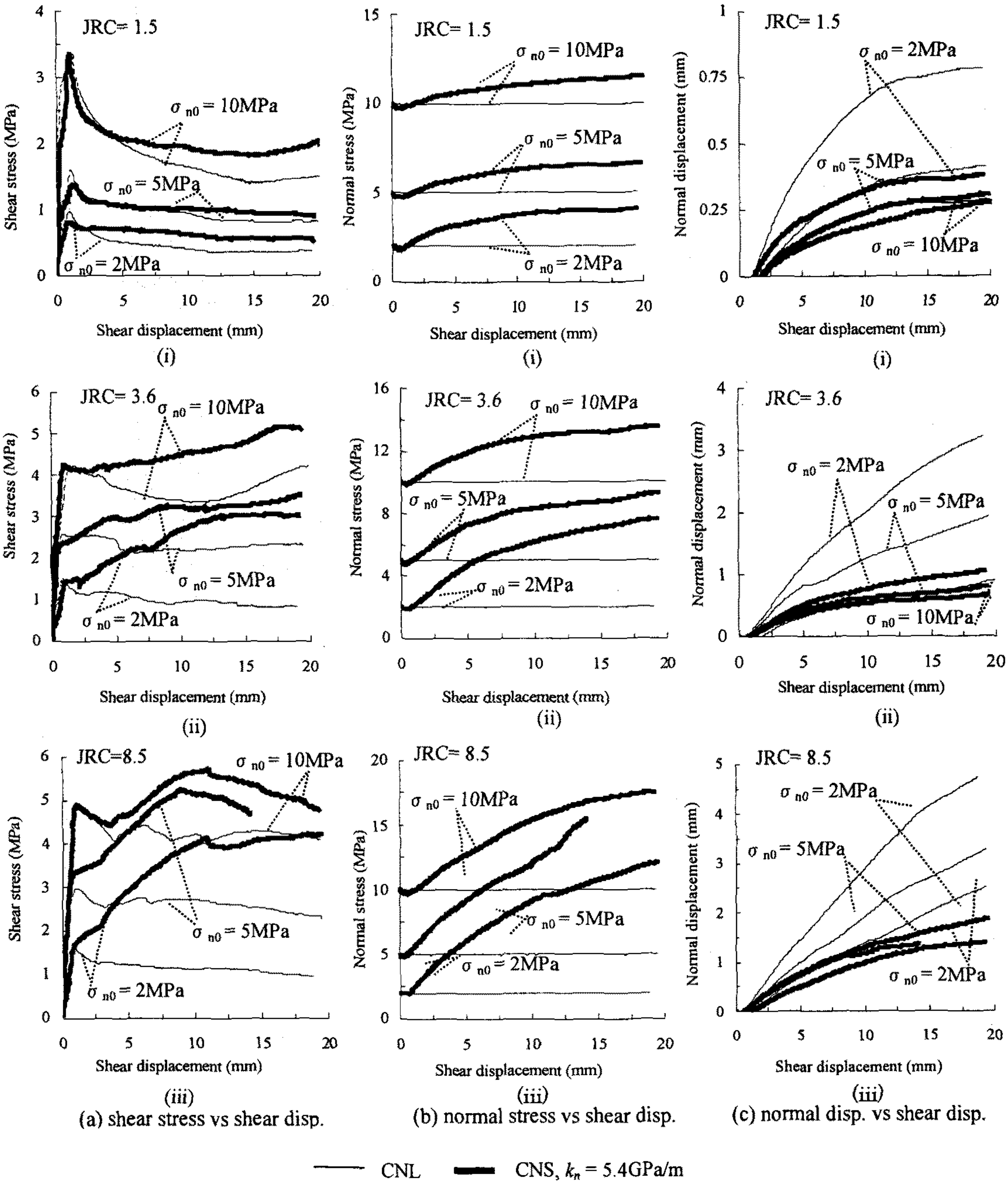
Fig. 2 The LabVIEW front panel of the 'shear screen' VI.

diagram. The block diagram with its associated sub-VIs, represents the executable program of the control and instrument. Commands and subroutines are represented by icons, which are wired to each other according to the type of variables. The control system included about 100 sub-VIs and having an overall size of about 26.4 MB.

By using the developed hardware and software systems, the experimental environment was greatly improved. The software described above is more user-friendly and much simpler to operate than the past ones. Flexible control of experiment can be accomplished through a PC with a customized LabVIEW interface.

2.2 Direct shear tests on artificial rock joints

A series of shear tests were performed on irregular artificial rock joints by using this apparatus. In order to compare the shear behaviors of rock joints with the natural roughness profiles under CNS and CNL conditions, shear tests under the different initial normal stresses of 2, 5, 10 MPa were conducted. Figure 3(a) illustrates the shear stress-shear displacement relations under different roughness profiles and initial normal stresses. It is observed that CNL tests always present the low peak shear strength at a small shear



(a) shear stress vs shear disp.
(b) normal stress vs shear disp.
(c) normal disp. vs shear disp.

Fig. 3 Shear behaviors of the artificial rock joints under CNL and CNS conditions.

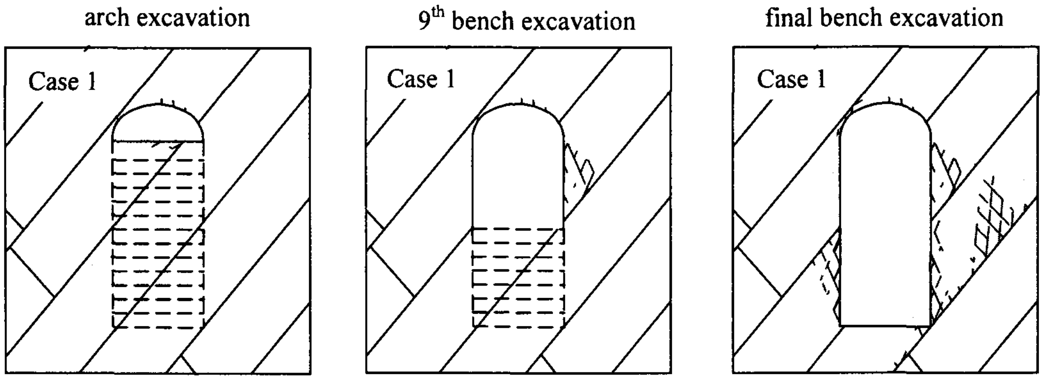
displacement. In CNS tests, the normal stiffness, k_n= 5.4 GPa/m, were selected to clarify the effects of normal stiffness of the surrounding rock mass on the shear behaviour of joints. The peak shear stress is found to be greater than that of CNL tests as a result of the increased normal stress during shear process (Figure 2(b)). It is also clear that an increasing in normal stiffness increases the normal stress so as to reduce the dilation of the rough joints. The more detailed investigations into the shear results show some dependence between the evaluated parameters and change in the initial normal stress and normal stiffness. The change of normal displacement during shear process, i.e. the dilation angle shows no changing while the peak shear stress is significantly affected by both the initial normal stress and the normal stiffness. The peak shear stress increased at an augmented initial stress and normal stiffness of the surrounding rock mass, such as soft, medium hard and hard rock.

The relations of normal displacement (volume change) and shear displacement are shown in Figure 3 (c). It is shown that the mechanical aperture of joint alters significantly when a normal stiffness is applied to the sample and the mechanical aperture increases with shear displacement. The change of the mechanical aperture in CNS tests is smaller than that in CNL tests and is also influenced by JRC.

By the above-observed experimental results, it is clarify that the shear strength of rock joint is greatly controlled by boundary conditions.

3. CRACK GENERATION SIMULATION OF ROCK MASSES CONTAINING UNDERGROUND OPENING

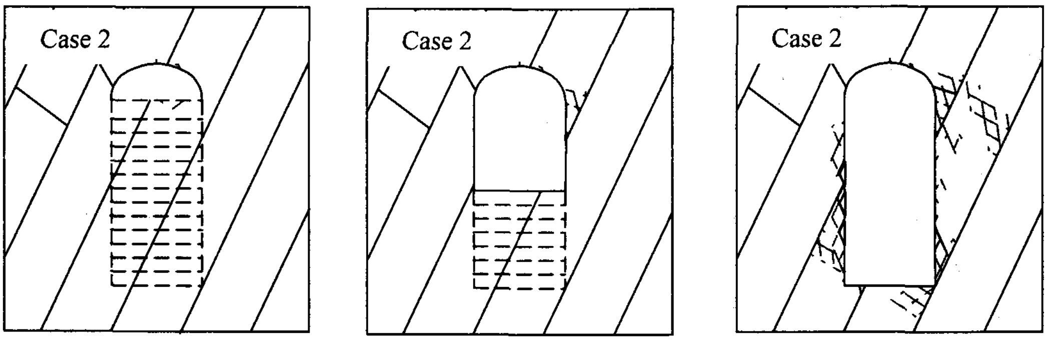
During the excavation process of an underground cavern, the change of stress state (primary the relaxation of stress) in the rock masses surrounding the underground structure may induce the generation of new cracks, which combining with the existing discontinuities could play an important role in endangering the stability of the structure. To effectively assess the stability of underground structures, the thorough understanding of the mechanical behaviors of existing discontinuities as well as the mechanism for the generation and propagation of new cracks is required. In this study, an Expanded Distinct Element Method (EDEM) was developed for simulating the generation and propagation of new cracks due to the shear and tension failures in the matrix blocks based on the distinct element code UDEC. Using this method, excavation simulations of a deep underground cavern have been carried out on a series of models with differing depths and differing geometrical distributions of existing discontinuities. The support effects of rock bolts on controlling the deformations of the rock masses surrounding the cavern and the movements of key blocks were evaluated in the numerical analyses.


arch excavation

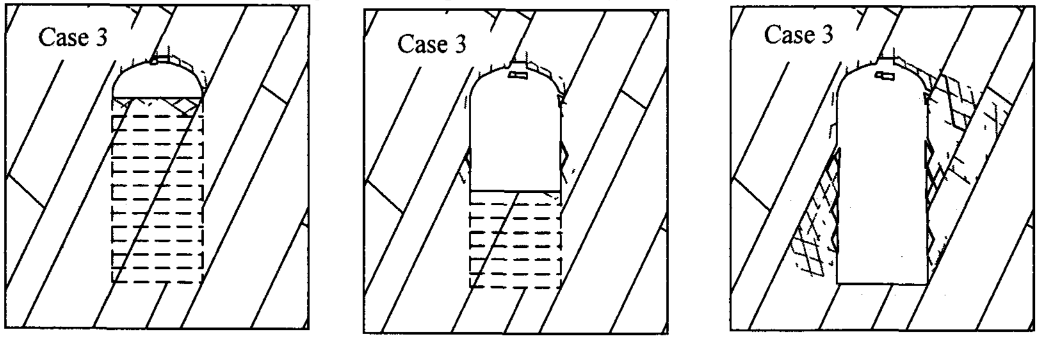
9th bench excavation

final bench excavation

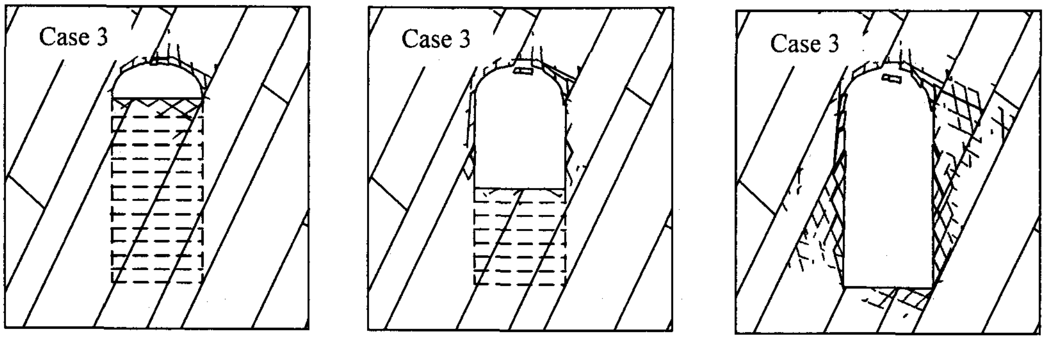
(a) Depth of underground cavern (case 1): H = 158 m



(b) Depth of underground cavern (case 2): H = 158 m

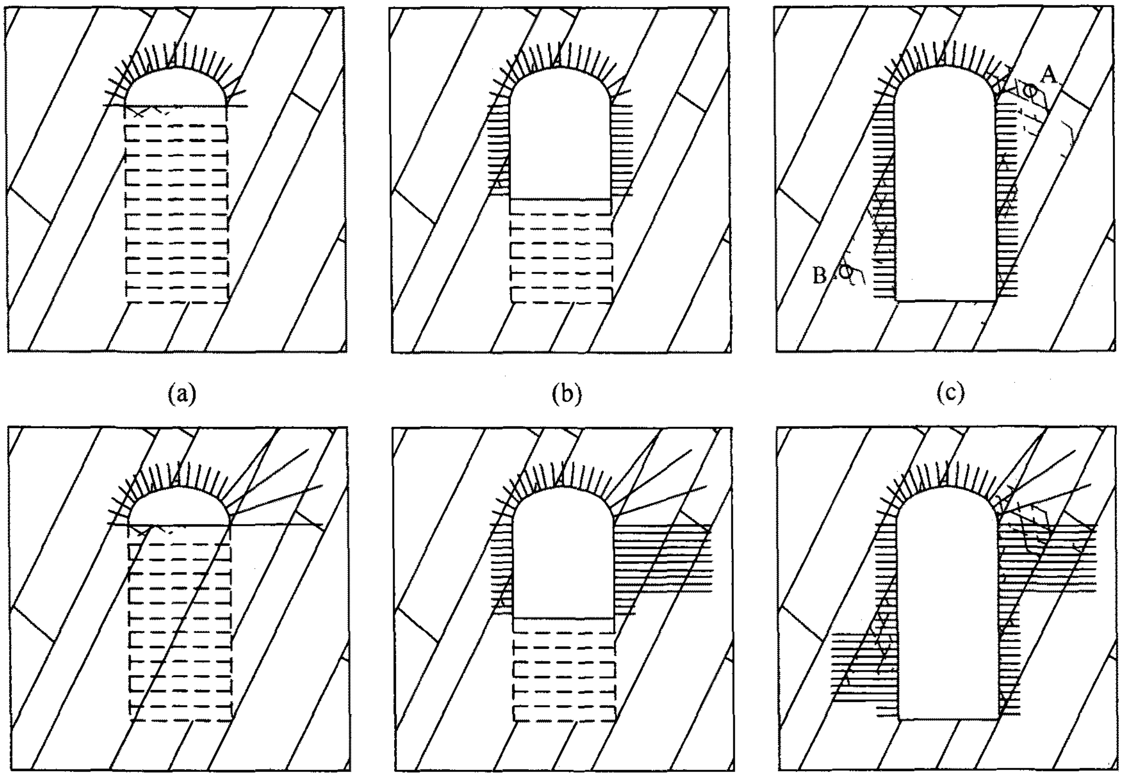


(c) Depth of underground cavern (case 3): H = 158 m



(d) Depth of underground cavern (case 3): H = 262 m

Fig. 4 Generation of new cracks during excavation process.



(a) (b) (c)

(d) (e) (f)

Fig. 5 Illustration of the crack generation in the rock masses subjected to two patterns of rock bolting system (pattern (1): (a), (b), (c); pattern (2): (d), (e), (f)). (A, B: weak zones)

3.1 Description of EDEM

Special treatment in terms of defining the potential cracks, which could change to true cracks when satisfying the failure criterions, was applied to the DEM simulation model to accomplish the function of generating new cracks. Two failure criterions are adopted according to the principal stresses acting on each potential crack to define the failure modes for these cracks. Criterion functions for shear failure f_s and tension failure f_t can be expressed as equations (1) and (2), respectively.

$$f_s = (1.0 - \sin\phi)\sigma_1 - (1.0 + \sin\phi)\sigma_3 - 2.0c\cos\phi \tag{1}$$

$$f_t = \sigma_t - \sigma_3 \tag{2}$$

where c is the cohesion force, ϕ is the friction angle of intact rock. σ_t is the tension strength of intact rock.

The mechanical properties of the potential cracks satisfying either one of the failure criterions are reduced to that of the existing joints automatically so that these potential cracks could perform the same mechanical behaviors with the existing joints.

3.2 Simulation of excavation induced crack generation

Simulation on 3 distribution patterns of rock joints were carried out and compared to assess the influences of geometrical characteristics of rock joints on the performance of underground cavern.

Figures 4 illustrate the propagation of new cracks and the process of local failure around the deep underground cavern due to excavation. The new cracks generated after arch excavation are few in Case 1 and Case 2. At the same time, Case 3 produces more new cracks along the roof wall due to the higher density of existing joints. During the bench excavation, Case 1 and Case 2 produce almost same new cracks in amount on the right side with the differences that the new cracks in Case 2 are in higher position comparing to Case 1 on the right side and are larger in amount on the left side because the existing joints in Case 2 have steeper dip angle than that of Case 1.

On the other hand, because of the steeper dip angle and the shorter spacing of existing joints in Case 3, the amount and area of newly generated cracks are much larger than that in Case 1 and Case 2 after the whole excavation steps finished, and separation of key blocks takes place along the new cracks on the wall of cavern. The amount and area of new cracks are furthermore increased by locating the cavern in deeper underground by comparing Figure 4(c) (H=158m) to (d) (H=262m).

For all the cases, the positions where the existing joints encountering the walls of rock cavern are favorable for the generation and propagation of new cracks. The local failure zones are significantly influenced by the dip angle, spacing and the relative position of the existing joints to the cavern as well as the depth of underground cavern through the results above. The construction of reinforcements is therefore necessary to taking these factors into account in engineering practice.

The grouted rock bolts are embedded vertically to the wall of cavern after each excavation step. In pattern (1), the rock bolts are 5m long, and embedded in 1.5m intervals along the roof and sidewalls as shown in Figures 5(a), (b), (c). Comparing with the no bolting pattern (see Figure4(c)), the bolted pattern successfully inhibits the generation of key blocks on the walls of roof and sidewalls and the amount of new cracks is significantly decreased. The new cracks are mainly generated near the lower part of left sidewall (B) and the upper part of right sidewall (A), which could be considered as the weak zones in this model. To effectively control the deformation and crack generation in these weak zones, a further reinforced pattern (2) is evaluated in EDEM as shown in Figures 5(d), (e), (f). In pattern (2), longer rock bolts (15m) are placed on the lower left sidewall and upper right sidewall to control the degradation of weak zones. The amount of new cracks as well as the displacements above roof has been slightly decreased in pattern (2), exhibiting good support effect comparing to the model without reinforcements.

4. INFLUENCE OF GEOMETRICAL DISTRIBUTION OF ROCK JOINTS ON DEFORMATIONAL BEHAVIOR OF UNDERGROUND OPENING

Mechanisms of deformation and failure of underground opening in jointed rock masses are mainly governed by the characteristics of the geometrical distribution of rock joints. In order to evaluate the deformational behavior and stability mechanism of underground opening appropriately, the key block theories, DDM (Discontinuous Deformation Method) and DEM have been considered as the useful tools for design assistant and stability assessment of underground opening. In this study, the extracting method of geometrical distribution of discontinuities in rock masses by image processing is presented and used to make networks of rock joints from the construction field. Using those networks, the fractal characteristics of the discontinuities are described by using the box-counting method for quantitatively evaluating the state of the discontinuous distribution. Finally, numerical analysis based a case of the excavation of underground power plant is carried out to find the relation between geometrical distribution of rock joints and deformational behavior of underground opening.

4.1 Fractal measure methods

In this study, the box-counting method is used to determine the fractal dimension of geometrical distribution of the existing rock joints and the newly generated cracks. As a general way, first, the joint trace data area is covered by a square box; then, the box size is decreased and the number of boxes needed to completely cover the feature of joint trace is counted; at last, a log-log plot of number of boxes needed to cover the feature versus box size is drawn and the slope of this plot presents the fractal dimension of this shape. Fractal dimension D_{Box} in the box-counting method can be written by the following equation.

$$D_{Box} = \lim_{\delta \to 0} \frac{\log N_{\delta}}{-\log \delta} \tag{3}$$

where D_{Box} is the fractal dimension; N_{δ} is the number of boxes needed to cover the object; and δ is the box size.

Depending on this calculation procedure and equation (3), a simple computer program was written for performing the box-counting method. The networks used in this program are first transformed into square cell (360*360 pixels), in which the curves are expressed by the fine lines with width of 1 pixel. Then, as the box size decreased, the log-log plot can be created.

4.2 The relation between fractal dimension and deformational behavior of

underground cavern

Three steps were carried out in the excavation simulation. First, the numerical models were built and input in UDEC, and three joint network cases with different joint densities or orientation were created; Then, using the proposed measure method, the fractal dimension D_{Box} of the three cases were calculated; At last, these models, with uniform material behaviour, were executed in UDEC and their deformational behaviors were recorded.

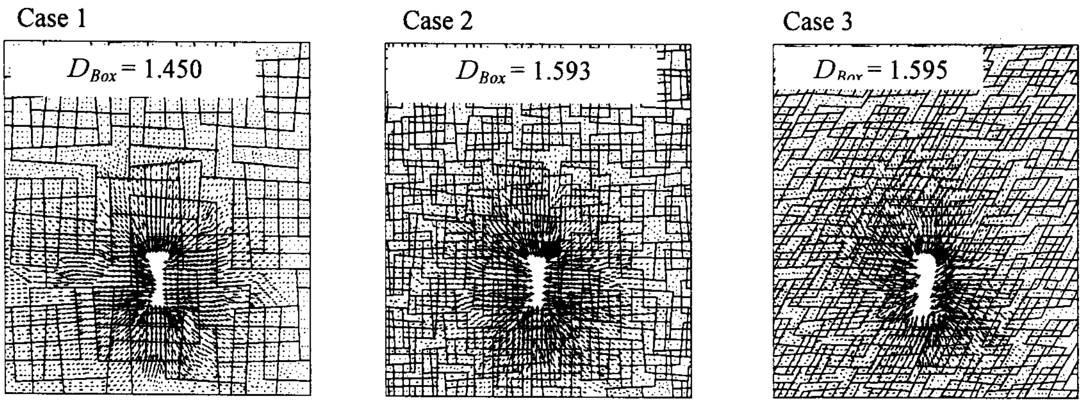
Figure 6 shows the analyzed deformational behavior of rock masses around opening according to different fractal dimension D_{Box} and orientation. UDEC supported a good plot view of the different deformational states in different cases. The displacement vectors in three cases have different distribution characters, especially case 3, which have different orientation with cases 1 and 2, the direction of displacement vectors around opening trends to the center direction of two joint sets. It means that the total displacement of a block in the model is combined of the displacements along the directions of two joint sets, which agrees with the in-situ conditions when the properties of two joint sets are the same. The plastic zone distributions of three cases are shown in Figure 6 (b), it can be seen from cases 1 and 2 that the main plastic zones concentrate at the cross areas of joints and the wall of opening.

In case 3, the plastic zone spread to broader area and mainly distribute along the joint sets. For an easy comparison, by changing the space and trace of joint sets, another cases with lower fractal dimension and different set orientation were created. After executed by simulation program, the maximum displacements of all cases were obtained, their relationship with fractal dimension D_{Box} are shown in Figure 7. The angle θ between two joint sets are set to be 40°, 60° and 90° to estimate the influence of orientation. It can be found that the displacement is significantly influenced by D_{Box} when θ is smaller than 90°, especially in the case of θ=60°. It shows that an underground opening with two joint sets, one is horizontal and the other one is vertical or close to vertical, as cases 1 and 2, is in a relative stable state and not influenced by the joint set density too much. For the cases that θ located between 0° and 90°, the increase of displacement with fractal dimension should be noted.

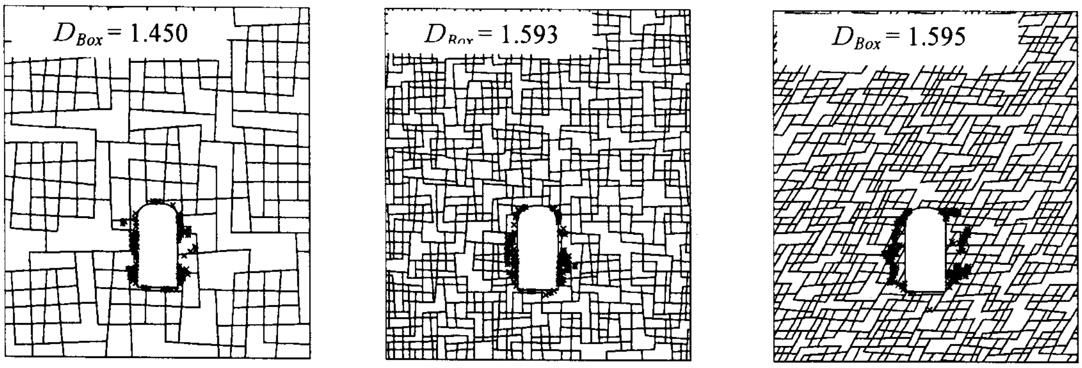
On the other hand, as shown in Figure 8, a series of numerical experiments are carried out by changing θ and α. Herein, θ is the angle between set1 and set2, α is the angle between set1 and the horizontal direction.

Comparing and investigating those results, followings are clarified: (1) when $\alpha = 30^\circ$ and $\theta > 60^\circ$, the change of the maximum displacement around the rock cavern is small. (2) there are peak values of the maximum displacement on the curves while θ=60° and α=0°, 60°, respectively.

From these results, it could be found out that a joint set, the angle between which and opening wall is around 60°, may induce a great displacement.



(a) Displacement vector



(b) Plastic zone

Fig. 6 Deformational behavior of rock masses around deep underground opening according to fractal dimension D_{Box} and orientation.

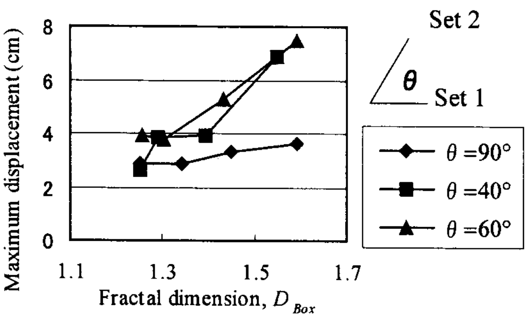


Fig. 7 Maximum displacement around underground opening versus fractal dimensions.

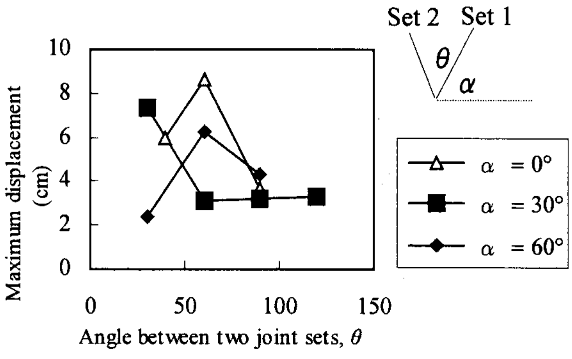


Fig. 8 Maximum displacement around underground opening versus fractal dimensions.

5. CONCLUSION

The reinforcement effect and stability of large rock caverns in deep underground are often affected by discontinuity-related deformation. The presence of rock joints or faults in the vicinity of a cavern can significantly increase the area of plastic zones and the local deformation especially when the joints encountering the walls of cavern. The DEM is capable of presenting the separation, rotation, slip and compression of discontinuities and therefore could be used to evaluate the local deformational behavior such as the movements of key blocks, which attract high concern in ensuring the stability of a cavern especially at arch in the jointed rock system.

In the present study, a high-performance direct shear test apparatus was developed to effectively assess the mechanical behaviors of rock joints. A numerical approach by using the DEM has been proposed for analyzing the stability of underground openings in the jointed rock masses. Based on fractal analysis and numerical simulation of underground opening in the jointed rock masses, the relationship between deformational behavior and fractal dimension and orientation of joint sets are discussed. The expanded distinct element method is also proposed to represent the generation and propagation of the new cracks around the deep underground cavern in the discontinuous rock masses encompassing a large-scale cavern due to excavation.

From the results above, the direct shear test-UDEC based approach has successfully modeled a number of rock caverns in the deep underground, which has the potential to be used as a routine design tool in the underground constructions.

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