Modeling the Effect of Water, Excavation Sequence and Reinforcement on the Response of Tunnels

Kim, Yong-Il*

요 지

본 논문에서는 기존의 불연속 변형 해석(DDA) 방법에 대한 세가지 방향의 새로운 개선 방법들이 제시되었다. 이 개선 방법들은 암반 균열에서 암석 블록과 지하수 흐름의 수리-역학적 커플링, 연속적인 하중 재하 또는 제하, 그리고 록볼트, 숏크리트와 콘크리트 라이닝에 의한 보강으로 구성되었다. Shi (1988)와 Lin (1995)에 의한 기존 DDA 프로그램은 이 방법들에 의하여 추가로 개선되었으며, 이 새로운 DDA프로그램에 대한 몇 가지 적용예들이 제시되었다. 또한, 경부고속철도 공사의 일부인 운주 터널의 지하굴착에 대한 시뮬레이션을 통하여 절리를 통한 지하수의 흐름, 굴착순서, 그리고 록볼트와 숏크리트에 의한 보강이 터널안정에 미치는 영향을 연구하였다. 그 결과 절리를 통한 지하수의 흐름과 부적절한 굴착순서는 터널의 안정성에 악영향을 미치나, 한편 록볼트와 숏크리트에 의한 보강은 터널을 안정화 시킨다는 사실을 밝혀냈었다. 그 결과 세가지 개선방법이 추가된 DDA 프로그램은 지하구조물 설계에 있어서 유용한 해석방법으로 사용될 수 있다는 사실을 보여주었다.

Abstract

A powerful numerical method that can be used for modeling rock-structure interaction is the Discontinuous Deformation Analysis (DDA) method developed by Shi in 1988. In this method, rock masses are treated as systems of finite and deformable blocks. Large rock mass deformations and block movements are allowed. Although various extensions of the DDA method have been proposed in the literature, the method is not capable of modeling water-block interaction, sequential loading or unloading and rock reinforcement: three features that are needed when modeling surface or underground excavation in fractured rock. This paper presents three new extensions to the DDA method. The extensions consist of hydro-mechanical coupling between rock blocks and steady water flow in fractures, sequential loading or unloading, and rock reinforcement by rockbolts, shotcrete or concrete lining. Examples of application of the DDA method with the new extensions are presented. Simulations of the underground excavation of the "Unju Tunnel" in Korea were carried out to evaluate the influence of fracture flow, excavation sequence and reinforcement on the tunnel stability. The results of the present study indicate that fracture flow and improper selection of

* Member, Senior, R.E., DAEWOO Institute of Const. Tech.

excavation sequence could have a destabilizing effect on the tunnel stability. On the other hand, reinforcement by rockbolts and shotcrete can stabilize the tunnel. It is found that, in general, the DDA program with the three new extensions can now be used as a practical tool in the design of underground structures. In particular, phases of construction (excavation, reinforcement) can now be simulated more realistically.

Keywords : Discontinuous deformation analysis, Blocky rock masses, Hydro-mechanical coupling, Excavation sequence, Rock reinforcement, Rockbolts, Shotcrete, Tunnel stability, Construction phases.

1. Introduction

Several numerical methods are used in rock mechanics to model the response of rock masses to loading and unloading. These methods include the Finite Element Method (FEM), the Boundary Element Method (BEM) and the Discrete Element Method (DEM). Compared to the FEM and BEM methods, the DEM method is tailored for structurally controlled stability problems in which there are many material discontinuities and blocks. The DEM method allows for large deformations along discontinuities and can reproduce rock block translation and rotation quite well.

The Discontinuous Deformation Analysis (DDA) method is a recently developed technique that can be classified as a DEM method. Shi (1988) first proposed the DDA method in his doctoral thesis; computer programs based on the method were developed and some applications were presented in the thesis as well as in more recent papers. Various modifications to the original DDA formulation have been published in the rock mechanics literature over the past ten years. For instance, Lin (1995) improved the original DDA program of Shi (1988) by including four major extensions: improvement of block contact, calculation of stress distributions within blocks using sub-blocks, block fracturing, and viscoelastic behavior. Despite these improvements, the DDA method still suffers from several major limitations when used to model the interaction of engineering structures (dams, tunnels) with fractured rock masses. In this paper, three limitations are addressed: hydro-mechanical coupling, sequential loading or unloading, and rock reinforcement.

In rock masses there are generally many discontinuities that are preferred pathways for groundwater. Water flow induces hydrostatic pressure and seepage forces that affect the state of stress in the rock masses. At the same time, changes in the state of stress induce rock mass deformation, which result in changes in the rock mass hydraulic properties. This hydro-mechanical coupling is critical and cannot be ignored when modeling rock-structure interaction in engineering problems where water is present.

As discussed more extensively by Szechy (1967), the arrangement of underground openings and their excavation sequence depend on the necessary operations to be conducted in them (excavation method, installation and construction of temporary and permanent reinforcement, use, etc.), the nature of the rock mass, and the rock pressure conditions encountered. Therefore, there is a practical need to simulate the different phases of underground construction and, if possible, find the optimal construction procedure considering not only rock mechanics issues but also construction time and cost.

Three extensions to the DDA method have been made and implemented into the original program of Shi and modified by Lin in 1995. First, seepage can take place in the fracture space between rock blocks; flow is assumed to be steady and laminar or turbulent. The hydro-mechanical coupling across rock fracture surfaces is also taken into account. The program computes water pressure and seepage throughout the rock mass of interest. Second, the program allows for sequential loading or unloading and therefore can be used to simulate construction sequence. When block elements are removed(excavation) or added(loading), the algorithm modifies the initial block elements and the initial stress data. The new data are then used as input data for the next construction step. Finally, the program allows to include different types of rock reinforcement (shotcrete, rockbolts, concrete lining). The shotcrete or concrete lining algorithm creates lining elements along the excavated rock surface of the underground opening with specified thickness and material. The rockbolt algorithm suggested by Shi(1988) was modified to be applicable to the cases of sequential excavation and reinforcement, in which axial forces of rockbolts at a previous step are applied to the rockbolts as preloading in the next step. The DDA program with the three new extensions can now be used as a practical tool in the design of underground structures. In particular, phases of construction(excavation, reinforcement) can now be simulated using this new program.

2. Fundamentals of Discontinuous Deformation Analysis(DDA) Method

In the DDA method, the formation of blocks is very similar to the definition of a finite element mesh. A finite element type of problem is solved in which all the elements are physically isolated blocks bounded by pre-existing discontinuities. The elements or blocks used by the DDA method can be of any convex or concave shape whereas the FEM method uses only elements with predetermined topologies. When blocks are in contact, Coulomb's law applies to the contact interfaces, and the simultaneous equilibrium equations are selected and solved at each loading or time increment. The large displacements and deformations are the accumulation of small displacements and deformations at each time step. Within each time step, the displacements of all points are small, hence the displacements can be reasonably represented by first order approximations.

In general, the DDA method has a number of features similar to the FEM. For instance, the DDA and FEM methods minimize the total potential energy of a system(of blocks or elements) to establish the equilibrium equations; the displacements are the unknown of those simultaneous equations. Also, both methods add stiffness, mass and loading submatrices to the coefficient matrix of the simultaneous equations. Finally, the DDA method's use of displacement locking of contacting blocks resembles in many ways the method of adding bar elements to element contacts in the FEM. The main attraction of the DDA method is its capability of reproducing large deformations along discontinuities and large block movement: two features that are restricted with the FEM.

2.1 Block Deformations

By adopting first order displacement approximations, the DDA method assumes that each block has constant stresses and strains throughout. The displacements(u,v) at any point(x,y) in a block, i, can be related in two dimensions to six displacement variables

$$\mathrm{D_i} = (d_{1i} \;\; d_{2i} \;\; d_{3i} \;\; d_{4i} \;\; d_{5i} \;\; d_{6i})^T = (u_0 \;\; v_0 \;\; r_0 \;\; \varepsilon_x \;\; \varepsilon_y \;\; \gamma_{xy})^T \tag{1}$$

where (u_0, v_0) is the rigid body translation at a specific point(x_0, y_0) within the block. r0 is the rotation angle of the block with the rotation center at(x_0, y_0) and ε_x, ε_y and γ_{xy} are the normal and shear strains in the block. As shown by Shi (1988), the complete first order approximation of block displacements takes the following form

$$\begin{bmatrix} u \\ v \end{bmatrix} = \mathrm{T_i D_i} \tag{2}$$

where

$$\mathrm{T_i} = \begin{bmatrix} 1 & 0 & -(y-y_0) & (x-x_0) & 0 & (y-y_0)/2 \\ 0 & 1 & (x-x_0) & 0 & (y-y_0) & (x-x_0)/2 \end{bmatrix} \tag{3}$$

This equation enables the calculation of the displacements (u,v) at any point (x,y) within the block (in particular, at the corners), when the displacements are given at the center of rotation and when the strains (constant within the block) are known. In the two-dimensional formulation of the DDA method, the center of rotation with coordinates (x_0, y_0) coincides with the block centroid.

2.2 Simultaneous Equilibrium Equations

In the DDA method, individual blocks form a system of blocks through contacts among blocks and displacement constraints on single blocks. Assuming that n blocks are defined in the block system. Shi (1988) showed that the simultaneous equilibrium equations can be written in matrix form as follows

$$\begin{bmatrix} \mathrm{K_{11}} & \mathrm{K_{12}} & \mathrm{K_{13}} & \cdots & \mathrm{K_{1n}} \\ \mathrm{K_{21}} & \mathrm{K_{22}} & \mathrm{K_{23}} & \cdots & \mathrm{K_{2n}} \\ \mathrm{K_{31}} & \mathrm{K_{32}} & \mathrm{K_{33}} & \cdots & \mathrm{K_{3n}} \\ \vdots & \vdots & \vdots & \ddots & \vdots \\ \mathrm{K_{n1}} & \mathrm{K_{n2}} & \mathrm{K_{n3}} & \cdots & \mathrm{K_{nn}} \end{bmatrix} \begin{Bmatrix} \mathrm{D_1} \\ \mathrm{D_2} \\ \mathrm{D_3} \\ \vdots \\ \mathrm{D_n} \end{Bmatrix} = \begin{Bmatrix} \mathrm{F_1} \\ \mathrm{F_2} \\ \mathrm{F_3} \\ \vdots \\ \mathrm{F_n} \end{Bmatrix} \tag{4}$$

where each coefficient $\mathrm{K_{ij}}$ is defined by the contacts between blocks i and j. Since each block i has six degrees of freedom defined by the components of $\mathrm{D_i}$ in equation (1), each $\mathrm{K_{ij}}$ in equation (4) is itself a 6 x 6 sub-matrix. Also, each F_i is a 6 x 1 sub-matrix that represents the loading on block *i*. Equation(4) can also be expressed in a more compact form as KD = F where K is a 6n x 6n stiffness matrix and D and F are 6n x 1 displacement and force matrices, respectively. In total, the number of displacement unknowns is the sum of the degrees of freedom of all the blocks. It is noteworthy that the system of equations(4) is similar in form to that in finite element problems.

The solution to the system of equations(4) is constrained by a system of inequalities associated with block kinematics(e.g. no penetration and no tension between blocks) and Coulomb friction for sliding along block interfaces. The system of equations(4) is solved for the displacement variables. The final solution to that system is obtained as follows. First, the solution is checked to see how well the constraints are satisfied. If tension or penetration is found along any contact, the constraints are adjusted by selecting new locks and constraining positions and a modified version of K and F is formed from which a new solution is obtained. This process is repeated until no tension and no penetration is found along all of the block contacts. Hence, the final displacement variables for a given time step are actually obtained by an iterative process.

The simultaneous equations(4) were derived by Shi(1988) by minimizing the total potential energy(of the block system. The *i* -th row of equation(4) consists of six linear equations

$$\frac{\partial \Pi}{\partial d_{ri}} = 0, \quad r = 1-6 \tag{5}$$

where the d_{ri} are the deformation variables of block *i*. The total potential energy(is the summation over all the potential energy sources, i.e. individual stresses and forces. The potential energy of each force or stress and their derivatives are computed separately. The derivatives

$$\frac{\partial^2 \Pi}{\partial d_{ri} \partial d_{sj}}, \quad r,s = 1-6 \tag{6}$$

are the coefficients of the unknown d_{sj} of the equilibrium equations (4) for variable d_{ri}. All terms of equation(6) form a 6 x 6 sub-matrix, which is the sub-matrix K_{ij} in the global equation(4). Equation (6) implies that matrix K in equation (4) is symmetric. The derivatives

$$-\frac{\partial \Pi(0)}{\partial d_{ri}}, \quad r = 1-6 \tag{7}$$

are the free terms of equation(5) which are shifted to the right hand side of equation(4). All these terms form a 6 x 1 sub-matrix, which is added to the sub-matrix Fi.

Shi's thesis(1988) covers the details for forming sub-matrices K_{ij} and F_i, for elastic stresses, initial stresses, point loads, line loads, volume forces, bolting forces, inertia forces and viscosity forces. Both static and dynamic analyses can be conducted with the DDA method. For static analysis, the velocity of each block in the blocky system at the beginning of each time step is assumed to be zero. On the other hand, in the case of dynamic analysis, the velocity of the blocky system in the current time step is an accumulation of the velocities in the previous time steps.

3. Analysis of Fluid Flow through a Rock Joint Network

A numerical model was developed to study fluid flow in deformable naturally fractured rock masses. The model consists of a two-dimensional, linear-elastic rock mass dissected by a large number of joints with various apertures, lengths, and orientations. Fluid flow, which occurs when pressure gradients exist, is assumed to be one-dimensional parallel plate flow. The flow may be laminar or turbulent according to the values of the Reynolds number and relative roughness of the joint walls(Louis, 1969). The fluid flow and the rock deformation are fully coupled. Variations in fluid pressure and quantity of fluid result in joint deformation. In turn, joint deformation changes the joint properties, which therefore changes the fluid pressures and the resistance to fluid flow.

3.1 Assumptions

When implementing the hydro-mechanical coupling in the DDA program, several simplifying assumptions were made. The major assumptions are as follows: (a) the fluid is incompressible; (b) the blocks of intact rock are impervious, and fluid flow takes place in the joint space only; (c) the joints deform elastically; (d) a finite number of joints exist; (e) the intact rock is linearly elastic; and(f) joint displacements are small relative to the joint dimensions.

3.2 Water-Block Interaction Model

The model consists of two major components: the DDA method for the rock mass and the FEM method for joint flow. The main algorithm of the model is shown as a flowchart in Fig. 1. The initial mechanical properties of the joints such as aperture, length, orientation, and boundary conditions from the DDA program are used to compute the total heads and fluid quantities at the joints with a FEM subroutine called RFLOW. The numerical algorithm of subroutine RFLOW is shown in Figure 2. The seepage forces acting on each rock block are computed from the total heads and elevation heads using a subroutine called WPRESSURE. The numerical algorithm of subroutine WPRESSURE is shown in Kim's thesis(1988). The seepage forces for each rock block are used to compute joint deformations with the DDA program. The joint deformation data are used to compute changes in the joint properties such as aperture, length, and orientation. A computational loop is followed until the results converge according to a criterion decided by the user. As a validity check of the RFLOW subroutine, a comparison was made with the experimental work reported by Grenoble(1989), who constructed a physical laboratory model to simulate two-dimensional flow through a jointed rock mass(Kim, 1988). As a validity check of the WPRESSURE subroutine in the DDA program, the problem of the opening of a crack(joint) in an infinite domain subjected to an internal pressure was considered and compared with an analytical solution(Kim, 1998).

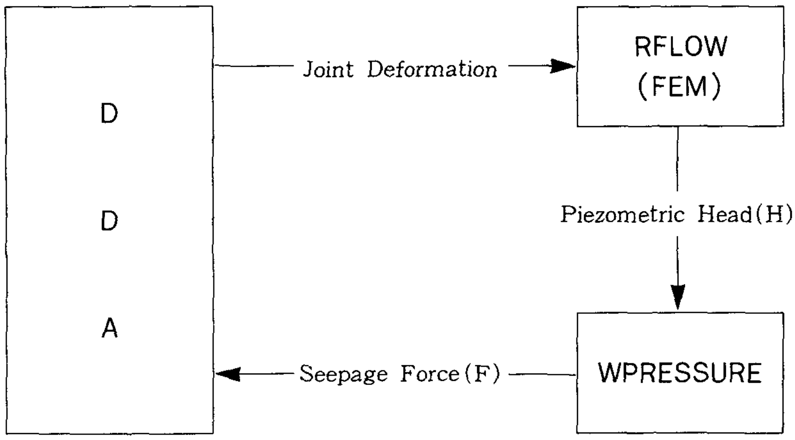


Fig. 1 Water-block interaction model

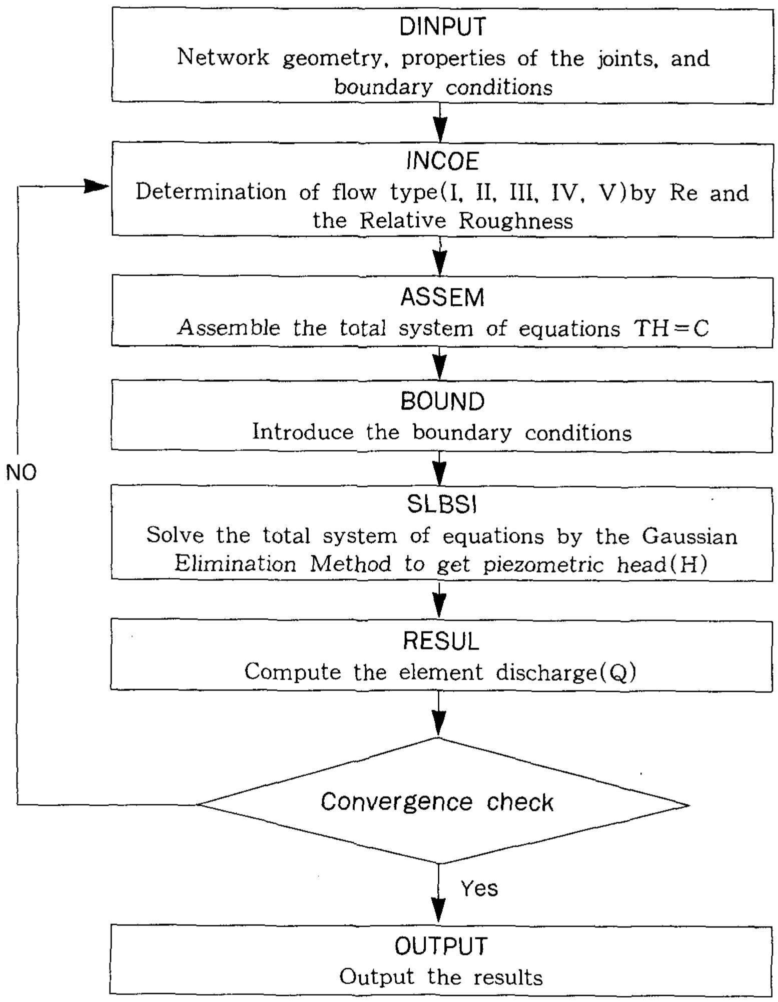


Fig. 2 Numerical algorithm of subroutine RFLOW

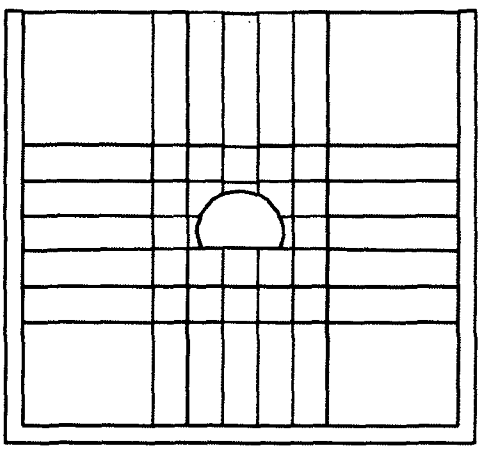
3.3 Effect of Water Level on Tunnel Stability

The excavation of a(half) circular tunnel was considered as a numerical example. The tunnel has a diameter, D, of 10m and is located at a depth of 409.6m below the surface. The water level varies between 100 and 500m. The domain of analysis is 5D(50m) wide and 4D plus tunnel height(46.71m) high. A vertical compressive stress of 10 MPa is applied on the top boundary of the domain(to simulate the load associated with 384.6m of rock) and no lateral deformation is allowed. The intact rock has a unit weight of $26KN/m^3$, a Young's modulus of 3.6 GPa, and a Poisson's ratio of 0.2. The joints have a spacing of 4m, a friction angle of 35° and a cohesion of 0.5MPa. No reinforcement is applied in this example.

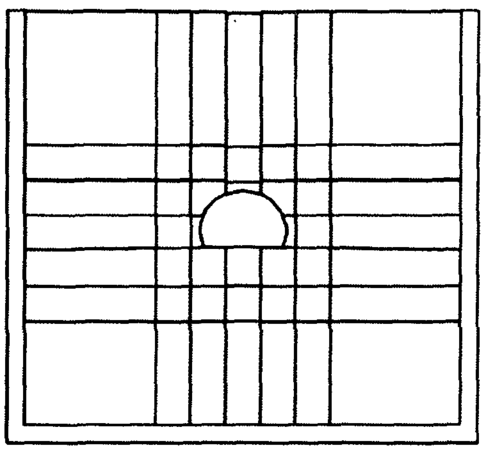
Without water the settlement of the tunnel roof was found to be equal to 165.2mm(Table 1). The tunnel roof deforms but does not collapse as shown in Figure 3(a). As the water level increases, the tunnel roof settlement increases, which results in collapse of the tunnel(see Table 1 and Figures 3(b)-(f)).

Table 1. Effect of water level

Water level(m)	Roof settlement(mm)
0	165.2
100	237.6
200	706.4
300	1632.8
400	2088.2
500	4514.0



(a)



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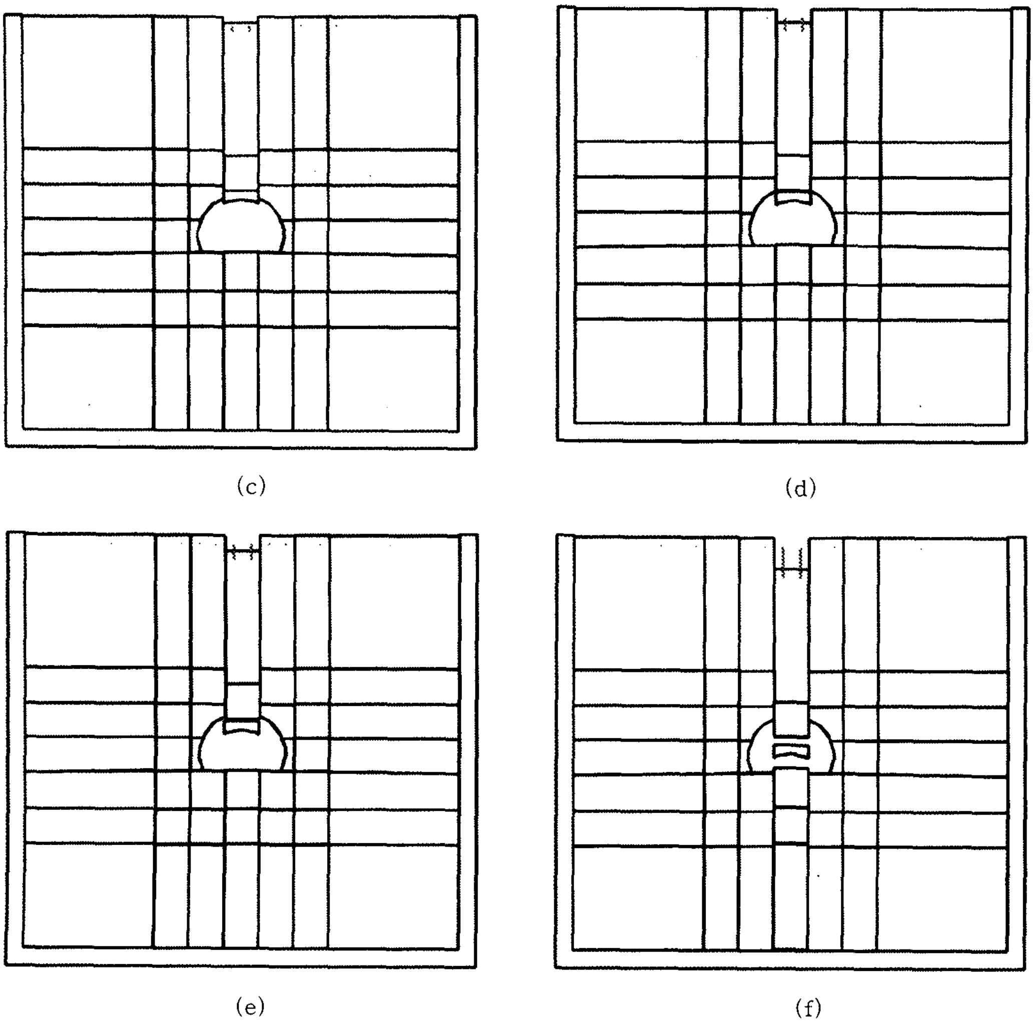


Fig. 3 Effect of water level on tunnel stability (a) No water, (b) W.L.: 100m, (c) W.L.: 200m, (d) W.L.: 300m, (e) W.L.: 400m, (f) W.L.:500m

4. Analysis of Excavation Sequence of Tunnels

A new algorithm to simulate sequential loading or unloading has been developed and implemented into the DDA program. The algorithm can be used, for instance, to model the different phases of underground excavation. The algorithm consists of an iterative procedure where in situ stresses are first computed in all rock blocks before excavation. Then, the new stress distribution is determined by following the first excavation step. The new stresses are then taken as initial stresses for the next excavation step. This iterative procedure continues until the end of the excavation process.

As an example of application of the method, a rock mass 30m wide and 20m high consisting of 17 rock blocks is considered(Figure 4). Three point loads are applied on block #15, to simulate structural loads at the ground surface. The intact rock has a unit weight of 23.6 kN/m^3, a Young's modulus of 48GPa, and a Poisson's ratio of 0.3. The joints have a friction angle of 40(and a cohesion of 4.0MPa. No water flow was considered. Two sequences of excavation of a horseshoe tunnel in the rock mass of Figure 4 were considered and are referred to as top-to-bottom and bottom-to-top. In the top-to-bottom(ex-t) excavation sequence, five blocks(numbered #32, #33, #31, #30, #29 in Figure 4) were removed sequentially. In the bottom-to-top(ex-b) excavation sequence, five different blocks(numbered #29, #30, #31, #32, #33) were removed sequentially. The major principal stress(1 in blocks #18, #19, #23, #22 & #20 located on the left of the excavation surface was calculated for both excavation sequences. The results are plotted in Figures 5(a)-(e).

Figures 5(a)-(e) indicate that the top-to-bottom excavation sequence induces much less stress concentration in the rock blocks adjacent to the excavated rock surface than the bottom-to-top excavation sequence. The results also show that in the bottom-to-top excavation sequence, the stress increases rapidly, which is more critical for the stability of the final excavated rock surface. At the final excavation step(step 5), the top-to-bottom excavation sequence shows slightly smaller

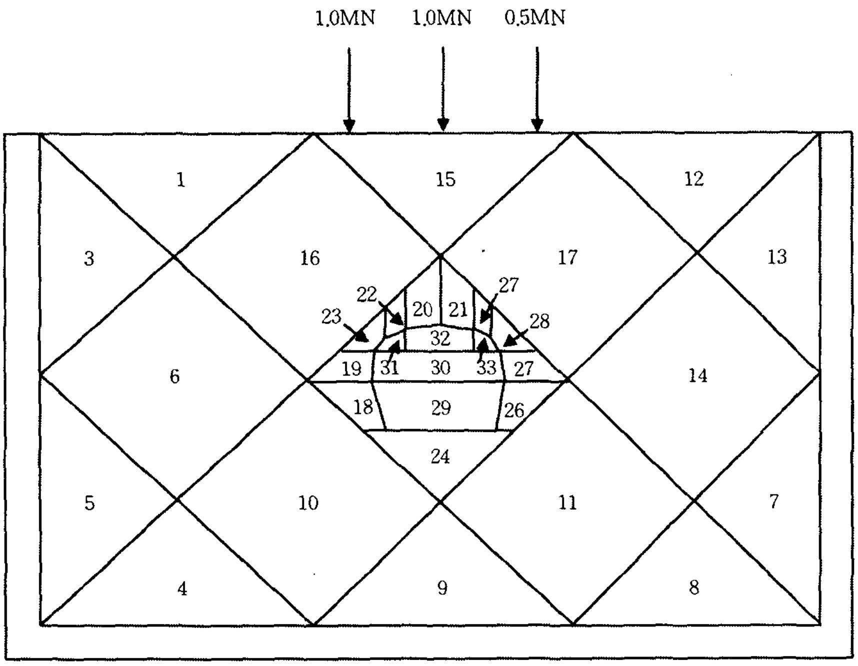


Fig. 4 Initial configuration for analysis of tunnel excavation sequence

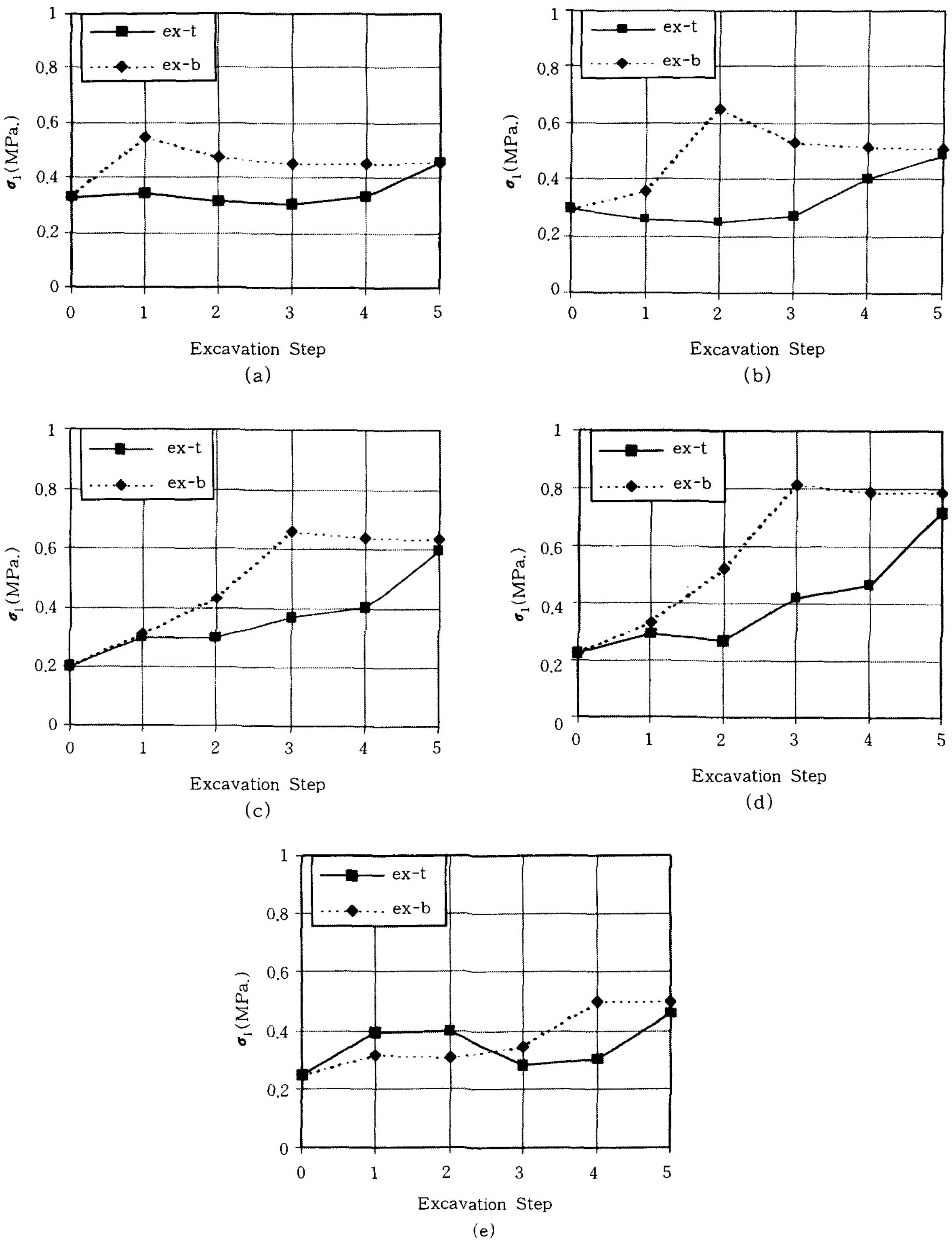


Fig. 5 Variation of σ_1 with step of excavation for the two excavation sequences (a) Block #18, (b) Block #19, (c) Block #23, (d) Block #22, (e) Block #20

stresses than the bottom-to-top excavation sequence. These results indicate that the final stress distribution is influenced by the stress history induced by the excavation. This stress-path dependency is associated with geometrical non-linearity in the DDA method as energy losses take place by friction along the joints and relative deformation of the blocks.

5. Case Study: Modeling The Excavation and Reinforcement of The UNJU Tunnel

The Unju tunnel was selected as a case study(Daewoo Institute of Construction Technology, 1995). The tunnel is located in Yeongigun Chungchungnamdo(Korea) and is part of the "Kyungbu High Speed Railway Project". Its depth ranges between 0 and 277.6m(109K820) and has a length of 4.02km. A tunnel section(Station 109K 440) was selected for numerical analysis using our DDA program.

The half circular tunnel was excavated by the single bench cut method, and reinforced by shotcrete(thickness of 10cm), rockbolts (diameter of 25mm and length of 4m) and concrete lining (thickness of 0.4m). Rockbolts (untensioned end-bearing type) were installed along the upper half surface of the tunnel only. The spacing of the rockbolts was 2m longitudinally and 11.3o latitudinally(with a total number of 15). The ground condition above the tunnel is represented in Figure 6. The radius of the excavation is 7.6m(tunnel radius = 7.1m, shotcrete thickness = 0.1m, lining thickness = 0.4m). The geology of the station consists of augen and banded gneiss. From the geological site investigation report, the ground above the tunnel is mostly hard rock with a very small depth of weathered rock and soft rock. The intact rock has a unit weight of 26kN/m^3, a Young's modulus of 3.6 GPa, and a Poisson's ratio of 0.2. The joints have a friction angle of 33.5o

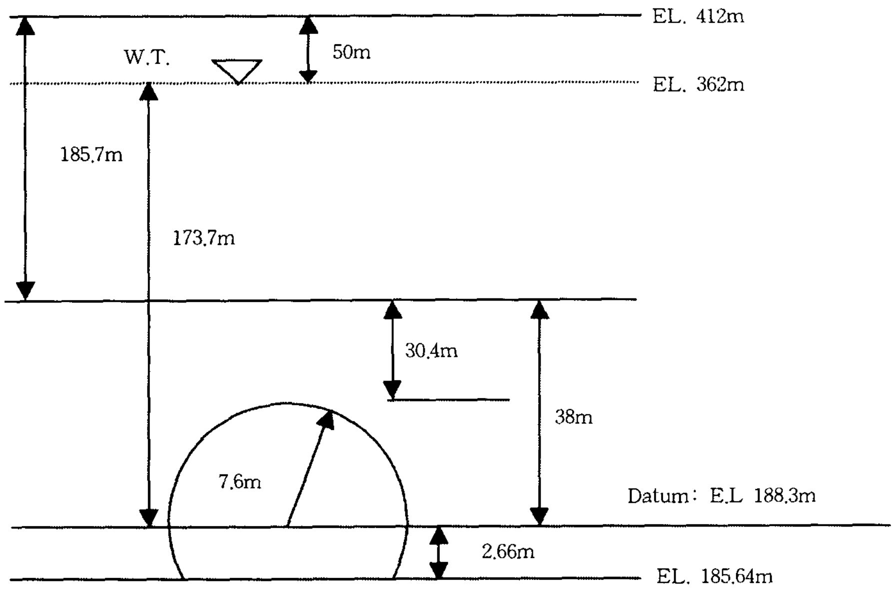
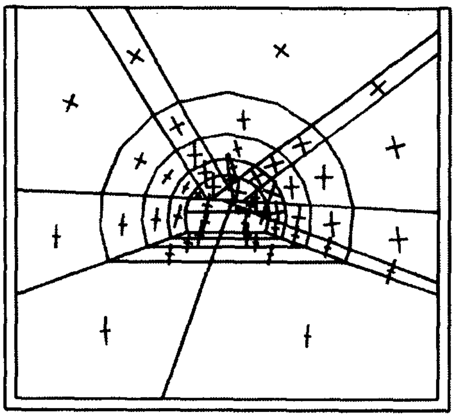


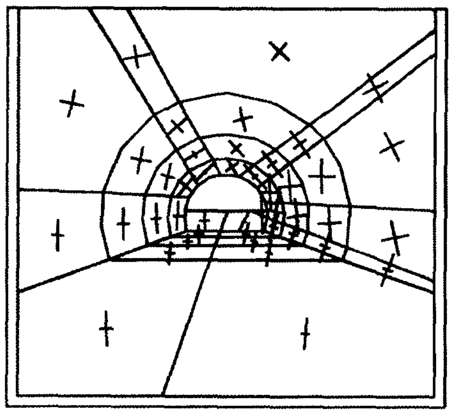
Fig. 6 Ground condition(Station 109K 440)

and a cohesion of 0.11MPa. The shotcrete has a unit weight of $23kN/m^3$, a Young's modulus of 15GPa, and a Poisson's ratio of 0.25. The Young's modulus of the rockbolts is 214GPa. The domain of analysis was set as 5D horizontally and 4D (plus tunnel height) vertically, where D is the tunnel diameter equal to 15.2m. A vertical compressive stress of 4.83MPa was applied on the top boundary of the domain (to simulate the load associated with 185.7 m of rock).

The joint location and orientation data for station 109K 440 used in the DDA analysis were obtained from tunnel face mapping photographs at Station 109K 442 located near the station of interest. The joints were assumed to be continuous and to extend over the entire domain of analysis. The initial geometry is shown in Figure 7(a). Construction (excavation and reinforcement) of the tunnel was conducted in five steps outlined in Table 2. These five steps were simulated using our DDA program. Three-dimensional effects due to longitudinal and transverse arching were considered by load distribution method, in which different internal pressures were applied along the excavated surface of the tunnel according to the construction steps. The stress distribution at the end of each step is shown in Figures 7(b)-(f).



(a)



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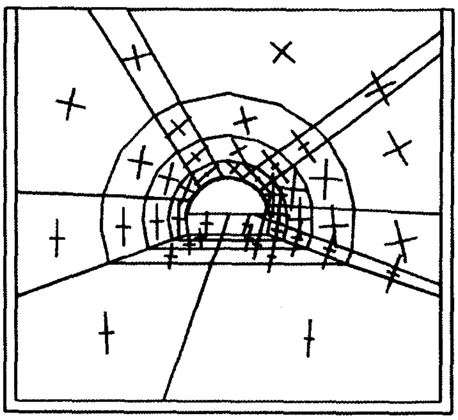

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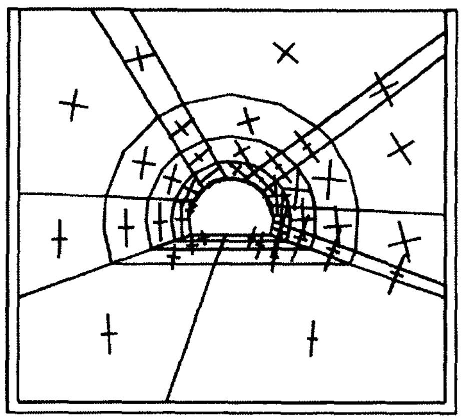

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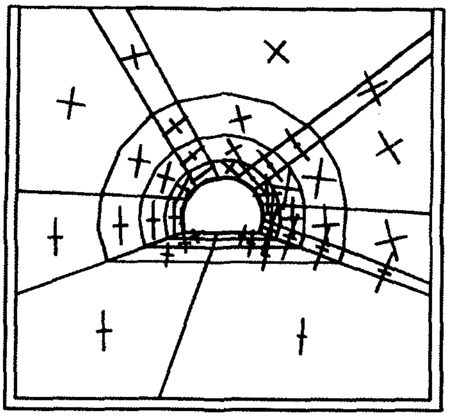
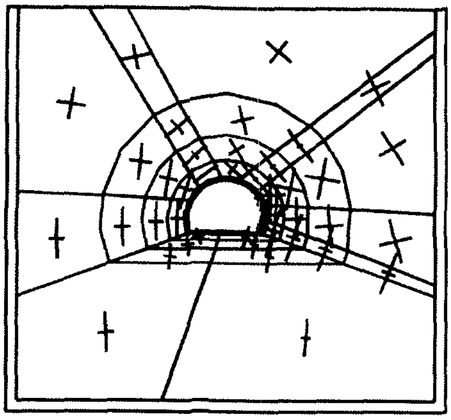


(e)



(f)

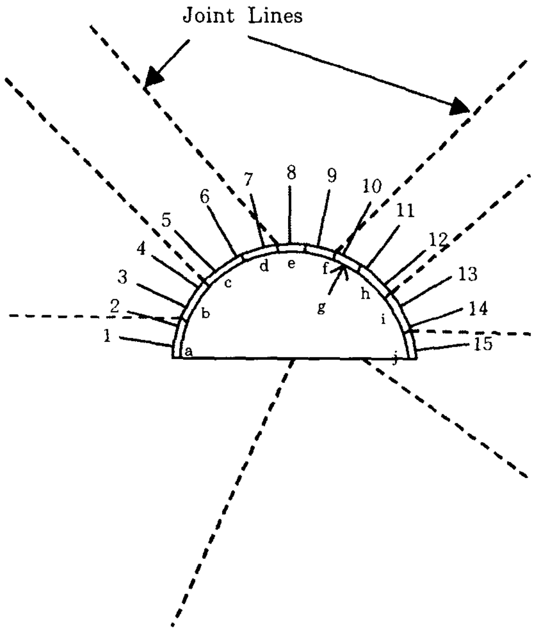
Fig. 7 Construction sequence of the Unju tunnel(109K 440) (a) Step 0, (b) Step 1, (c) Step 2, (d) Step 3, (e) Step 4, (f) Step 5

Table 2. Steps of excavation and reinforcement

Step	Contents
0	Initial state without excavation
1	Excavation of upper half section
2	Reinforcement of the upper half section with shotcrete and rockbolts
3	Excavation of lower half section
4	Reinforcement of lower half section with shotcrete
5	Reinforcement of full section with concrete lining

The tunnel roof settlement, the axial force in the 15 rockbolts, and the tangential stress in the shotcrete segments were predicted using the DDA analysis and compared with actual field measurements. The computed and measured data are listed in Table 3. These results indicate that rockbolts #7 and #10, which intersect natural rock joints, develop large tensile forces(42kN and 81 kN). This observation shows the effectiveness of rockbolt reinforcement in preventing relative movement between blocks in a rock mass. The stresses in the shotcrete show small compression. The results in Table 3 indicate that rockbolts act as a major reinforcement of rock blocks and shotcrete functions as auxiliary reinforcement such as sealing of rock surface, preserving inherent ground strength and providing a structural arch.

Table 3. Comparison between computed and measured data

Station	109K 440		
Item	DDA	Measured Data	App.
Roof Settlement (mm)	4.3	4	+ : Down —: Up
Rock Bolt Axial Force(kN)	-3.97(1) -0.70(2) 4.82(3) 3.03(4) -0.85(5) 0.93(6) 41.98(7) -0.51(8) 9.68(9) 80.74(10) 6.07(11) 7.01(12) -4.86(13) -4.58(14) 2.87(15)	-3.88(1) -0.20(8) -4.08(14)	—: Compression + : Extension 
Tangential Shotcrete Stress (MPa)	-0.007(a) -0.013(b) -0.015(c) -0.015(d) -0.013(e) -0.035(f) -0.035(g) -0.036(h) -0.026(i) -0.009(j)	 -0.014(b) -0.012(e) -0.011(j)	

6. Conclusion

Three major extensions were implemented into the original DDA program of Shi and modified by Lin in 1995. The extensions include hydro-mechanical coupling between rock blocks and water flow in fractures, sequential loading and unloading, and rock reinforcement by shotcrete, rockbolt and concrete lining.

The hydro-mechanical coupling algorithm is very important in rock engineering problems if seepage takes place in natural fractures and joints. Seepage forces and water pressure can be controlling factors in rock mass stability as illustrated in the tunnel example presented in this paper. The algorithm is limited to steady state flow and needs to be modified to include transient flow phenomena.

The sequential loading(or unloading) algorithm allows for changes in loading conditions that can occur in rock engineering problems. In the examples presented herein, the changes were associated with the excavation of underground openings. The same algorithm could also be used to model surface excavations associated with road cuts, open pit mining, quarrying, etc. It is likely that the overall final stability of a rock mass depends on the excavation sequence and the corresponding stress history. This interesting phenomenon can now be studied and researched using our new DDA program.

The shotcrete or concrete lining algorithm creates shotcrete or concrete lining elements along the excavated rock surface with specified thickness and material properties, which simulate applying shotcrete or installing concrete lining on already reinforced and excavated rock surfaces. The rockbolt algorithm suggested by Shi(1988) was modified so that it may be applicable to the cases of sequential excavation and reinforcement, in which axial forces of rockbolts at a previous step are applied to the rockbolts as preloading in the next step.

The DDA program with the three new extensions can now be used as a practical tool in the design of underground structures such as tunnels or caverns. It can also be used to analyze the stability of concrete dams on fractured rock masses. The main contribution of this paper is that phases of construction(excavation, reinforcement) can now be simulated in a more realistic way.

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