Study on evolution process of water inrush disaster of tunnel based on the distinct element method

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Abstract. On the basis of the analysis of the geological characteristics of the construction adit of a water diversion project, in conjunction with the phenomena of water gushing, mud bursting and collapse encountered in the construction process of the tunnel, the fracture network model of the typical rock mass in the project area is established. Based on the seepage stress coupling mechanism of the fractured rock mass, the UDEC discrete element method is used to analyze the gradual development process of the surrounding rock fractures in the excavation process of the tunnel. The dynamic evolution law of tunnel water inrush is revealed, which could provide a good reference for the safe construction of underground engineering in water rich area and the design and control of surrounding rock stability.

1. Introduction
During the construction of deep-buried tunnel in water-rich area, it is unavoidable to cross fractured rock mass with different hydrogeological and engineering geological conditions. The in-situ stress field of tunnel rock mass is relatively high. At the same time, there is a seepage flow field in the fractured rock mass traversing the water-rich area, which constitutes a specific dynamic equilibrium system with the ground stress field. The original groundwater dynamic balance system tunnel of is usually destroyed by the excavation, resulting in the underground gallery for groundwater discharge. Inrush water disasters are easy to occur in the area, which brings great harm to the safe construction and operation of the tunnel. Taking the famous Qinghan Subsea Tunnel as an example, the maximum inflow is 8.64×10^4 m^3/d during the inrush water disaster, resulting in 3015m submergence of auxiliary tunnel and 1493m submergence of main tunnel. Another project is the Alladases Railway Tunnel in Sweden, with a maximum inrush of 2.42×10^4 m^3/d and water pressure of 0.8MPa. Therefore, the problem of groundwater in water-rich areas has attracted much attention of tunnel engineering [1-6].

Figure 1. Water inrush photo of a construction adit
2. General situation of the project

A diversion project is located in Yunnan Province, in which the tunnel length is 612 km, accounting for 92% of the total line length. The studied tunnel in this paper is one of the tunnel of the diversion project. During the construction period, the tunnel is used as the construction passage to deal with the tunnel fracture and its influence zone. It is also used as the permanent maintenance tunnel and the ventilation tunnel during the operation period.

The construction branch tunnel crosses strata mainly Nβ、Pβ Basalt, Tβ carbonate rocks and the East-West limeyard fault zone. The maximum buried depth of the tunnel is 585 meters. The surrounding rocks are strongly influenced by NNE-NE and two groups of near-east-west fault structures. The rock mass is poor, and the karst and hydrogeological conditions are complex. The water inrush mud along the fracture zone and the carbonate tunnel section are prominent and the external water pressure are relatively high. Surrounding rock cracking and water inrush in different parts of the tunnel occurred many times during construction. In 2020, large settlement displacement occurred on the right vault with maximum value of 30cm. The side wall is deformed and cracked to form staggered stage, the maximum staggered stage is about 40cm. The top of the palm face bursts with water and local bulges, which carries a large amount of breccia and macadam and causes submergence of the palm face. Long-term penetration damage brings out a large amount of crushed rock and gravel rock, which causes cavities in the right side wall to the top arch and also causes overall settlement damage of the steel support in this area.

The tunnel section is located in the contact zone between limestone and fault breccia and crushed rock, close to the core of limestone kiln fracture zone. The tunnel is also located below the groundwater level of more than 200 meters. Groundwater is rich and has large water pressure. The top arch of pile at one section has developed a long fissure with a width of 1-2 cm. The fissure surface is open and without filling (filling material is washed away by water flow). The long fractured zone may penetrate the karst water systemin the area, forming a smooth groundwater seepage channel. Under construction excavation disturbance, the high-head groundwater will flow out together along the long fractured zone and cause seepage damage, forming a concentrated water inrush channel, causing water inrush. At the same time, groundwater outflow and pressure release further disturb the rock mass structure, resulting in deformation and destruction of surrounding rocks.

Based on the strata lithology, hydrogeology and geological structure of the tunnel, a rock mass structure model for typical section is established in this paper. Discrete element numerical method is used to analyze the permeability characteristics of surrounding rock mass under the conditions of stress and water pressure interaction. The seepage-stress coupling mechanism of fractured rock mass under high water pressure and complex stress path are then simulated. Research on change of seepage flow field of rock mass and formation process of water inrush passage during construction are studied. The results of this paper could reflect possible parts of water inrush, and provide good reference for safe construction and protection design of the tunnel.

3 Theory and method of seepage analysis of fractured rock mass based on discrete element

UDEC is a two-dimensional calculation program based on discrete element method to simulate discontinuous media. The mechanical behavior of discontinuous media under static or dynamic loads can be described from a two-dimensional perspective. Discontinuous media can be represented by aggregates of discrete blocks. Structural planes can be seen as boundaries between blocks, allowing large displacements and rotations of blocks along the structural planes. Fluid flow in impermeable block fissures can be analyzed. UDEC can be used to effectively simulate fluid movement in non-permeable fractured rock mass. During the flow process, the fissure changes with the change of stress field. The fluid pressure in the fissure determines the change of seepage field, which in turn controls the change of stress field.

For a closely packed system, there is a network of domains, each of which is assumed to be filled with fluid at uniform pressure and which communicates with its neighbors through contacts. As shown in Fig2, five domains are numbered, where domain 3 and 4 represent joints, domain 2 is located at the intersection of two joints, and domain 5 is a void space. Domains are separated by the contact points
(designated by letters A to F in the figure), which are the points at which the forces of mechanical interaction between blocks are applied. Because deformable blocks are discretized into a mesh of triangular elements, grid points may exist not only at the vertices of the block, but also along the edges. A contact point will be placed wherever a grid point meets an edge or a grid point of another block. For example, in the same figure, contact D implies the existence of a grid point along one of the edges in contact. As a consequence, the joint between the two blocks is represented by two domains: 3 and 4. If a finer internal mesh were adopted, the joint would be represented by a larger number of contiguous domains. Therefore, the degree of refinement of the numerical representation of the flow network is linked to the mechanical discretization adopted, and can be defined by the user.

![Figure 2. Flow in joints modeled as flow between domains](image)

**Figure 2.** Flow in joints modeled as flow between domains

In the absence of gravity, a uniform fluid pressure is assumed to exist within each domain. For problems with gravity, the pressure is assumed to vary linearly according to the hydrostatic gradient, and the domain pressure is defined as the value at the center of the domain. Flow is governed by the pressure differential between adjacent domains. The flow rate is calculated in two different ways, depending on the type of contact. There are two methods to calculate the flow

1) For a point contact (i.e., corner-edge, as contact F in Figure 2, or corner-corner), the flow rate from a domain with pressure $p_1$ to a domain with pressure $p_2$ is given by

$$ q = - k_c \Delta p $$

where $k_c$ is a point contact permeability factor, and

$$ \Delta p = p_2 - p_1 + \rho_w g (y_2 - y_1) $$

where $\rho_w$ is the fluid density; $g$ is the acceleration of gravity (assumed to act in the negative $y$-direction); and $y_1$, $y_2$ are the $y$-coordinates of the domain centers.

2) In the case of an edge-edge contact, a contact length can be defined (e.g., in Figure 2, ID and IE denote the lengths of contacts D and E, respectively). The length is defined as half the distance to the nearest contact to the left plus half the distance to the nearest contact to the right. In this case, the cubic law for flow in a planar fracture can be used. The flow rate is then given by

$$ q = - k_f a^3 \frac{\Delta p}{T} $$

where $k_f$ is a joint permeability factor (whose theoretical value is $1/12\mu$); $\mu$ is the dynamic viscosity of the fluid; $a$ is the contact hydraulic aperture; and $l$ is the length assigned to the contact between the domains.

The hydraulic aperture is given, in general, by

$$ a = a_0 + u_n $$

where $a_0$ is the joint aperture at zero normal stress; and $u_n$ is the joint normal displacement. A minimum value, $a_{r_{ex}}$, is assumed for the aperture, below which mechanical closure does not affect the contact permeability. The variation of aperture with normal stress on the joint is depicted in Fig 3.

![Figure 3. Relation between hydraulic aperture and joint normal stress](image)

**Figure 3.** Relation between hydraulic aperture and joint normal stress

4. **Research on Crack Expansion and Evolution Process of Inrush Water in Surrounding Rock of Tunnel Excavation**
According to the description of existing engineering fissure structural plane, the geological general model of typical section is established, which covers a range of 80m×80m. The tunnel is city-gate shaped tunnel with a width of 8.6m and a height of 7.75m. Two lithological rock masses are considered in the model, corresponding to Rock1 and Rock2 in Figure 4 respectively. The corresponding physical and mechanical parameters of rock mass are shown in Table 1. The random fracture network generated by Monte-Carlo method is used in the calculation area. In the application of Monte-Carlo method, it is considered that the length, inclination, width and spacing of fracture tracks follow normal distribution. Two groups of dominant fracture planes are pre-set in Rock2 rock body on the right side of the tunnel center. The included angles with the horizontal axis are 65 ° and 105 ° respectively, and the fracture spacing is about 2.0m and 2.5m respectively. It is assumed that the rock fractures are smooth and parallel, regardless of the roughness of the fractures and the material filling between the cracks.

4.1 Calculation conditions
The buried depth of the top arch of the tunnel is assumed to be 300m. According to the distribution law of the in-situ stress field, the vertical in-situ stress is taken gravity stress, and the horizontal lateral pressure coefficient is 1.1. Stress boundary condition is applied in the upper boundary of the model adopts stress boundary condition, and the displacement boundary condition is applied in the left, right and bottom boundary. adopts displacement boundary condition, and the constitutive model of rock mass and joint adopts Mohr Coulomb model. Mohr Coulomb model is adopted for the constitutive model of rock mass and joints. The tunnel is deeply buried in the ground, and the groundwater level is in the upper part of the tunnel. In the model, it is assumed that the height difference between the groundwater level and the vault of the tunnel is 200m, and the corresponding hydrostatic pressure is applied on the boundary to simulate groundwater in fractured rock mass.

| rock mass | joints |
|-----------|--------|
|           | Rock1  | Rock2  | kn(GN/m) | 1000 |
| density (kg/m³) | 2900   | 2100   | 1000     |
| bulk(MPa)   | 3788   | 1852   | 1000     |
| shear modulus MPa) | 1953   | 758    | friction angle(°) | 25 |
| Cohesion (MPa) | 0.8    | 0.4    | Cohesion (MPa) | 0 |
| friction angle(°) | 40     | 30     | Residual joint width (m) | 0.0005 |
| tension(MPa) | 0.6    | 0.2    | Zero stress joint width (m) | 0.002 |
| Poisson's ratio | 0.28   | 0.32   | Permeability coefficient(pa-1.s-1) | 100 |

4.2 evolution process analysis of tunnel water inrush disaster
UDEC provides an empirical algorithm, which makes it possible to analyze large time history transient flow, make large time history transient flow analysis possible. In this paper, This the calculation model assumes that the fluid is incompressible, and runs iterative steps in each flow time step to adjust the pore pressure and domain volume to ensure the continuity of fluid flow. In order to simulate the phenomenon of water inrush caused by tunnel excavation in fractured rock mass, the real-time velocity of water in fractured rock mass is mainly concerned. Therefore, the transient analysis model of UDEC is adopted in this study to solve the seepage problem.

The most intuitive expression of the evolution process of tunnel water inrush is reflected by the change process of flow, water pressure and displacement of characteristic points around the tunnel. In particular, 1 # to 4 # characteristic points are selected in the right crown, right side wall and surrounding rock of the tunnel, and the location of the characteristic points is shown in Figure 5.
4.2.1 water pressure and flow rate of surrounding rock fracture The sign of flow indicates the recharge or outflow of fracture water at this point. After the completion of tunnel excavation, the unloading deformation of surrounding rock causes the rock mass crack to open or close, which will affect the distribution of seepage field of rock mass. The water pressure and water flow at the 1 # point of the tunnel top arch change with time. In the early stage of tunnel excavation, the flow rate at 1 point is about 0.007m³/s~0.021m³/s, and the flow rate is basically stable after 215 seconds. The corresponding water pressure is about 2.3MPa ~ 3.5MPa. After that, the variation range of fracture water pressure increases, the pressure increases, and the fracture width adjusts accordingly with the change of water pressure. The fracture reduces pressure through water flow discharge or extravasation. Each time there is water seepage, the water pressure and drainage flow have obvious changes. Before water seepage, the maximum water pressure and flow gradually increase with the number of water seepage, and the instantaneous water pressure of the fracture causing water inrush in the tunnel is about 16MPa. The maximum instantaneous flow is 50m³/s.

The pore pressure and fracture water flow rate at 2 # point in the middle of the right side wall of the tunnel change with the expansion or closure of the cracks in the surrounding rock. From the initial excavation to 365 seconds of seepage drainage, the flow rate of this part experiences four abrupt changes, and obvious water inrush occurs in the 240th ~ 260th second, the 345th ~ 352th second, the 352th ~ 357th second and the 360th ~ 365th second respectively. The maximum instantaneous flow rates of the four times are 4.0m³/s, 5.1m³/s, 16.8m³/s and 3.6m³/s respectively, and the corresponding instantaneous water pressures of fractures are 10.4MPa, 16.4MPa, 7.75 MPa and 7.36MPa respectively.

The fracture water pressure and flow at 3 # and 4 # characteristic points in the surrounding rock at the top of the tunnel vary with time (as shown in Fig.7). When the hydraulic coupling is 365 seconds after the completion of the tunnel excavation, the flow and water pressure at this part change more than 10 times, and the corresponding instantaneous flow is generally less than 1m³/s, and the corresponding...
water pressure is generally 6.0–9.0MPa. The maximum instantaneous flow of 3# and 4# are 50.0m³/s and 1.8m³/s respectively, and the corresponding maximum instantaneous water pressure is 16.0MPa and 13.4MPa.

![Figure 7](image)

**Figure 7.** Curves of water pressure and flow rate with coupling time for 3# and 4#

It can be seen from the time curve of water pressure and flow rate at characteristic points that before water inrush and instability of surrounding rock, the flow and hydrodynamic pressure at observation points around the tunnel are relatively large, the maximum instantaneous hydrodynamic pressure is about 16.0MPa, and the maximum instantaneous flow is about 50m³/s. In the initial stage of excavation, the water pressure of each measuring point is relatively stable, the water flow is small, and the change is basically small. Then, with the expansion and conduction of cracks in the surrounding rock, a new seepage channel is gradually formed. The water flow of each measuring point increases, the water pressure increases rapidly, and the water flow moves from high head to low head. When the flow reaches a certain amount, then the water pressure is basically in a stable state, and a small amount of water seepage occurs in the tunnel. The dynamic water pressure causes the shear displacement and opening deformation of the fracture surface, which leads to the re evolution and expansion of the fracture. The rapid growth of the recharge water in the fracture leads to the sudden increase of the dynamic water pressure. When the water barrier of the surrounding rock is broken, the surrounding rock of the tunnel is in the state of instability and water inrush. It can be seen that the occurrence of water inrush has a strong "mutation", with the characteristics of short-term sudden.

4.2.2 crack propagation process of surrounding rock  After the tunnel is excavated, the reason of water inrush is that the change of gap width leads to the formation of local hydraulic channel, which causes the high-pressure water head to gush out from local area and causes water inrush disaster. Figure 8 shows the process of fracture gradual expansion in the process of transient flow calculation. The figure shows the joint fracture with a width greater than 2.0 mm. The width of the joint fracture is expressed by the number of parallel lines, and a single line corresponds to a width of about 2.0 mm.

![Figure 8](image)

**Figure 8.** Crack opening and expansion of tunnel surrounding rock in different periods

It can be seen from figure 8 that the deformation of surrounding rock after tunnel excavation causes the opening or closing of cracks, which leads to the obvious change of hydraulic gradient of structural plane. Under the multiple effects of crack seepage and stress coupling, the crack width of surrounding rock of the right top arch and side wall of the tunnel increases gradually. From the 240th second, the crack expansion and opening change significantly, to the 345th second. The maximum crack width on the right side of the tunnel crown is about 26mm, and the maximum crack width on the right side wall
is about 14mm. Under the action of high water pressure, hydraulic fracturing occurs in the rock mass cracks of the right side wall, the water flowing cracks expand and the crack penetration is further strengthened, thus forming a water inrush channel. At the 365th second, the maximum width of the cracks in the right side wall rock is about 50 mm. Under the action of high water pressure, the blocks on the right side of the vault and the right wall begin to fall off with other blocks, and eventually lead to mud inrush and water gushing in the tunnel.

4.2.3 deformation of surrounding rock  The tunnel excavation causes disturbance in the mechanical model (the total residual stress on the tunnel boundary is reduced to zero) and the seepage model (the pore water pressure on the tunnel excavation boundary is reduced to zero). The mechanical process time scale in rock mass and fluid process time scale in joint have different orders of magnitude, so the mechanical response in the model is relatively instantaneous. The first step response of surrounding rock caused by tunnel excavation is undrained deformation: when pore water pressure in joint is a function of rock mass deformation and bulk modulus of groundwater, surrounding rock deformation occurs. Secondly, the seepage and drainage deformation caused by groundwater entering the tunnel is the time effect under the control of groundwater consumption.

As shown in Figure 9, after the tunnel excavation, the surrounding rock masses all deform towards the tunnel. At the initial stage of unloading, the deformation of surrounding rock around the tunnel is about 8.0-22.4mm. The mechanical properties of surrounding rock on the right side of the tunnel are poor, the fissures are developed, and have high permeability. The water pressure is changed correspondingly by the opening and closing of the crack joint width. With the coupling of the stress and seepage of the surrounding rock, the top arch and the side wall on the right side of the tunnel will move towards the tunnel. When the hydraulic coupling is 365 seconds, the incremental deformation of the surrounding rock towards the tunnel is about 500 ~ 970mm. The large deformation value of the right side of the crown arch is about 300 ~ 700mm, and the large deformation of the right side wall is about 500 ~ 970mm.

![Figure 9. Vector diagram of rock deformation around the tunnel](image)

See figure 10, after more than 240 seconds of seepage and drainage deformation calculation, the displacements of characteristic points around the tunnel are small. From 245 seconds, the displacements of 1# and 2# points show different degrees of rapid growth, and then the deformation is stable for about 100 seconds. After 350 seconds, the displacements of 1# and 2# points surge again, until the top arch and side wall rock on the right side of the tunnel lose stability. At this time, the displacement of characteristic point 1# is about 400 mm, The displacement of 2# is close to 1000mm.
5 Conclusion

The water-rock coupling of fractured rock mass in water-rich areas under high water pressure and complex stress paths is a very complex process. In the early stage of water inrush, there is a long period of scattered seepage phenomena, and the water inflow is generally gradual from small to large, and the leakage points are gradually concentrated. The formation of water inrush channel generally experiences a slow seepage failure process. The formation process of water inrush channels is the evolution process of fracture initiation, expansion, and connection in rock mass. In the process of tunnel excavation, the redistribution of surrounding rock stress and groundwater seepage cause the damage, deformation, and destruction of surrounding rock, which eventually leads to the instability of surrounding rock and water inrush.

In the process of research, the roughness of fracture and the effect of filling material are not considered, and the three-dimensional state of fracture is not considered, so there are many deficiencies in theory and simulation. The study can reveal the evolution process of water inrush disaster of deep-buried tunnel in water-rich areas, and provide reference for the construction safety of fractured rock mass in water-rich areas.

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