Study on the preloading effect of grouting on the tunnel lining

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Abstract. Grouting can effectively improve the stress state of the tunnel lining tensile areas under high external water pressure. To determine the relation between the permanent preloading effect and the post-grouting preloading effect on tunnel linings, several groups of grouting experiments were conducted using a grouting simulation test device developed in this research. Theoretical analysis was then performed on the basis of the experiments. Results showed that the preloading effect was severely affected by microcracks in the grouting area, although the effect gradually decreased after grouting. The ratio of the permanent effects against the post-grouting effects was shown to be a function of the drainage volume and deformation of the grouted rock mass.

1. Foreword
Recently, with the development of the hydropower industry, particularly the rapid development of the pumped storage power stations, many large-diameter headrace tunnels with high internal water pressure have been reported. Traditional steel-plate lining structures have limitations in many respects such as transportation, assembly, and welding. Thus, the increasing numbers of these plates in usage has led to the consideration of reinforced concrete linings owing to their effective construction technology and economic benefits. The cracking of reinforced concrete lining is inevitable under the impact of high internal water pressure. Moreover, the thickness required to prevent the lining from cracking is too large. Thus, consolidation grouting on the rock mass outside the lining is usually required to strengthen the surrounding rock of the tunnel and seal the fissures in the rock near the tunnel to improve the integrity and non-deformability of the surrounding rock. This will also increase the permeability of the surrounding rock to avoid the seepage of internal water and prevent hydraulic infiltration damage from occurring in the adjacent hydraulic structures. Geotechnical engineering practice has shown that grouting, particularly high-pressure consolidation grouting, can produce certain preloading effect on the lining. However, although this preloading effectively limits the cracking of reinforced concrete lining, it gradually dissipates with the operation of the project. Thus, its effectiveness in engineering design is low. Several domestic and foreign scholars have studied the mechanical behavior of the interaction between the tunnel lining and the surrounding rock from various perspectives. Detournay\textsuperscript{1} analyzed the mechanical behavior of the stress–strain orthogonal relation in a circular tunnel based on semi-analytical results obtained using a complex function. Previous research has systematically investigated the interaction between the lining and the surrounding rock of the diversion tunnel as well as the influence of water pressure and grouting pressure on the lining\textsuperscript{2–7}. In addition, various power stations, such as the Nanyang Huilong Pumped Storage Power Station, Guangzhou Pumped Storage Power Station, Tianhuangping Pumped Storage...
Power Station, and Huizhou Pumped Storage Power Station, provide certain monitoring data and regular outcomes. In summary, the preloading effect of grouting on concrete lining is restricted by many factors, and there are some difficulties associated with the precise determination of this effect. The long-term retention ratio coefficient of preloading stress is quite discrete, with 0.1 and 0.6 times the grouting pressure used in the designs of the Guangzhou and Huizhou pumped storage power stations and the Baishan hydropower station, respectively. Therefore, the empirical coefficient is still popular. The preloading stress has a strong influence on the working characteristics of the concrete lining; thus, the influence of consolidation grouting on the mechanical factors of the headrace tunnel lining still requires further deep study. Accordingly, the present study uses model experiments to study the preloading effect of grouting on lining, deduce the theoretical analysis method of the preloading effect, and research the changing rules of lining stress under various conditions. The results of this study provide a valuable reference for designs of the surrounding rock grouting and lining.

2. Model Experiment

2.1. Model Design
A combined barrel structure was adopted in the analysis model used for testing the mechanical properties of the grouted surrounding rock outside the tunnel lining. For the intake tunnel under a high water head, the lining produces large tensile stress during operation. Compared with the surrounding rock grouting circle, the lining thickness is smaller. The lining will incur obvious deformation during the high-pressure grouting and water conveyance operation; therefore, a steel pipe with a radius of 100 mm and a wall thickness of 3.0 mm was used to simulate the lining, as per the testing analysis. Because the rock mass outside the grouting circle is thick, the influence of its deformation on the lining preloading effect can be ignored. According to theoretical calculation and previous pressure testing, when the wall thickness is 15 mm under 2 MPa pressure, the deformation of the barrel wall is small and can be ignored. Therefore, a steel pipe with a radius of 200 mm and a wall thickness of 15.0 mm was used to simulate the non-reinforced rock mass outside the grouting circle, as shown in Figure 1.

![Figure 1 Schematic diagram of the model](image)

High-strength concrete was used to fill the space between the inner and outer sides of the barrel to simulate the rock mass to be grouted in the grouting circle. The fusible pieces were first designed with different standard thicknesses according to the predetermined requirements, and then the concrete inside was cast. After seven days of pouring, crevices at the desired width in the rock mass were created in the concrete by using methods of heating and washing. To improve the drainage conditions of the model, reduce the loss of grout, and build up the typical features of the drainage channel, a double-layer of geotextile was placed in the area near the drainage hole on the outer wall of the cylinder. Then, compacted fine sand was piled in the drainage pipe, and the corresponding valves were connected to simulate the crevice drainage channel. A strain gauge was used to monitor the change in prestress in the corresponding position of the lining during and after grouting.
2.2. Model Verification
To verify the rationality and sealing status of the model and to determine the effectiveness of the model size, structure, sealing, and monitoring parameters, a packer test with water was executed. The water pressure was maintained for 4 h at a maximum of 1.2 MPa, and the water leakage and tightness of the model were observed. After 4 h, the water pressure was turned off, and the stress change at each position was continuously monitored, as shown in Figure 2.

![Figure 2 Stress versus time curve]

The test yielded the following results. 1) The stress at each monitoring position was different owing to the influence of fracture locations in the rock mass in the grouting circle, which were to be grouted. 2) The stress change trend at each monitoring point was closely related to the water pressure. The stress value of monitoring point #1 was about 95% of the water pressure because it was located just a short distance from the pressurized water inlet; the stress change trend at other monitoring points conformed to the basic rock permeability theory. 3) When the water pressure was shut off, the stress at each monitoring point was essentially unchanged during the pressure maintenance period; therefore, the model was well sealed.

2.3. Experiment Scheme
To analyze the preloading effect and dissipation process of the tunnel lining after the high-pressure grouting of the fracture rock mass in the reinforcement ring and to study the relationships among these factors, such as the retained long-term prestress and grouting pressures as well as the maximum preloading stress, the following three testing schemes were determined: scheme 1: only one crack could be grouted; scheme 2: two cracks could be grouted and two could not; and scheme 3: three cracks could be grouted and five could not.

Ordinary Portland cement slurry with a water:cement ratio of 1:1 was used for grouting, and the drainage hole was kept open during grouting. The cement slurry is a granular type that generally cannot be used in cracks with openings less than 0.2 mm. According to the grout ability of the cement slurry, the groutable and non-groutable crack openings were 0.30 mm and 0.10 mm, respectively.

3. Model Experiment Results and Analysis

3.1. Experiment Results
The crack was connected to the drainage channel, with a circular length of 20 cm. The drilling hole intersected with the crack, and the grouting pipe was buried at the top. When the grouting pressure increased to 2 MPa, casting was continued for 30 min under constant pressure. Then, the grout pipe was closed to stop the grouting, and water discharge and stress data at each position were monitored. The monitoring results of scheme 1 are shown in Figs. 3 and 4.
With the consolidation of the cement slurry, the groutable cracks were filled by the concrete, which blocked the drainage channel. Because scheme 1 has no non-groutable cracks, the water was unable to be discharged; thus, the accumulated volume of water discharged from the cracks did not increase. Accordingly, the corresponding deformation in volume also showed no change, and the preloading stress on the simulated lining essentially did not dissipate. Conversely, because schemes 2 and 3 have non-groutable cracks, and the water in the slurry was gradually discharged under the pressure, as shown in Figs. 5–8.
The following results were noted:
1) After grouting, the discharge volume of model gradually increases, and the preloading stress at each measuring point gradually decreases;
2) the maximum preloading stress value obtained by the two schemes of the model test were 1.6 Mpa and 1.4 Mpa, respectively;
3) the drainage obtained by the two schemes of the model test were 1.82 ml and 3.08 ml, respectively.

3.2. Experiment Analysis

Based on the calculation principle of equivalent mechanical parameters of a rock mass with penetrable fractures, the equivalent deformation parameters of a complex fracture rock mass were derived, as shown in Figure 9. One inclined fracture was observed that did not penetrate the entire rock block. The length of the fracture trace is \( l \), the opening is \( d \), and the horizontal angle is \( \theta \).

According to the generalized Hooke’s law and that shown in Fig. 8 (a), the strain is assumed to be equal according to the displacement coordination conditions and under uniaxial pressure. Thus,

\[
\varepsilon = \frac{\sigma_x}{E_x} = \frac{\frac{\sigma_x}{E_x}(h-x)}{h} + \frac{\frac{\sigma_x}{E_x}x}{h} = \frac{\frac{\sigma_x}{E_x}(h-d') + \frac{\sigma_x}{E_x}d'}{h}.
\]

The generalized linear Hooke’s law indicates \( \varepsilon = \frac{\sigma}{E} \). According to the principle of strain equivalence, we get
As per the static equilibrium conditions,
\[ \sigma \omega = \sigma_s (\omega - 2a - b) + \sigma_s b + 2 \int_0^\omega \sigma_s \, dx. \]  
(3)

According to the geometrical relationship,
\[ a = d \sin \theta \]
\[ b = l \cos \theta - d \sin \theta \]
\[ d = d / \cos \theta \]

Then,
\[ E = E_r \left[ w - 2a - b + \frac{E_s h b}{E_s (h - d) + E_s d} + \frac{2E_s h}{E_s - E_r} \ln \left( \frac{E_s h + (E_r - E_s) a}{E_s h} \right) \right]. \]  
(5)

After substitution, we get
\[ E = E_r \left[ 1 - \frac{l \cos \theta}{w} - \frac{d \sin \theta}{w} + \frac{E_s h (l \cos \theta - d \sin \theta) \cos \theta}{w \left( E_s (h \cos \theta - d) + E_s d \right)} + \frac{2E_s h}{E_s - E_r} \ln \left( 1 + \frac{d (E_r - E_s) \sin \theta}{h} \right) \right]. \]  
(6)

Considering that the opening \( d \) is a very small compared with other geometric parameters of the fracture,
\[ \frac{d}{w} \rightarrow 0 \]
\[ \frac{d}{h} \rightarrow 0. \]  
(7)

By substituting the above formula, we get
\[ E = E_r \left[ 1 + \frac{(E_r - E_s) ld + E_s h d \sin \theta \cos \theta}{w \left( E_s (h \cos \theta - d) + E_s d \right)} \right]. \]  
(8)

Equation (8) is exactly the equivalent deformation modulus \( E \) calculation formula of a single-fracture rock mass.

If \( \theta = \pi / 2 \), the model of the fractured rock mass will be a rock mass with vertical fracture; thus, \( E = E_r \), which indicates that the vertical fracture has no contribution to the vertical deformation modulus of the rock mass. As a result, the equivalent deformation modulus of the rock mass can be taken as the deformation modulus of the complete rock mass, which is consistent with the empirical conclusion. Similarly, when \( \theta = 0 \), \( w = l \), the fractured rock mass model becomes a rock mass with a completely penetrating horizontal fracture. Then,
\[ E = \frac{E_r E_s h}{E_s (h - d) + E_s d}. \]  
(9)

The length of the crack in the rock mass is far larger than the crack opening. To facilitate calculating the equivalent Poisson’s ratio \( \mu \), the influence of the stress transition section on the equivalent model can be ignored. The Poisson ratio derived is
\[ \mu = \frac{\varepsilon_s}{\varepsilon_v} = \frac{\mu \cos \theta + \mu \chi (1 - \eta - \xi \cos \theta) + \mu \chi \eta \xi \cos \theta}{\cos \theta + \chi (1 - \eta)}, \]  
(10)

where \( \chi = \frac{d}{l}, \eta = \frac{l}{w} \), and \( \xi = \frac{E_s}{E_r} \).
Considering that the crack opening width is very small compared with the length, that is, \( \chi = \frac{d}{l} \to 0 \), the substitution in the above formula is
\[
\mu = \mu_r. \tag{11}
\]
Therefore, the equivalent Poisson ratio of the rock mass is exactly the same as that of the rock block. The roof, floor, and outer wall of the testing model cannot be deformed. According to the elastic theory, the volume change in the model can be calculated as \( \Delta V = \frac{\Delta P_{hilb}}{E} \). The trace lengths of the groutable and non-groutable fractures in the experimental model are both 10 cm; the horizontal angle \( \theta \) is 15\(^\circ\); the width of the fracture in the expanding direction is 35 cm; and the height of each of the three models is 40 cm. The equivalent deformation modulus and Poisson ratio of the corresponding test scheme can be obtained by programming. Table 1 gives the values for \( \Delta Q \), \( \Delta V \), \( \Delta P_m \), and \( \Delta P \).

**Table 1.** Parameters of schemes 2 and 3 including drainage volume \( \Delta Q \), volume change \( \Delta V \), maximum prestress after grouting \( \Delta P_m \), and prestress change in the lining \( \Delta P \).

| Scheme | Drainage volume \( \Delta Q \) (ml) | Volume change \( \Delta V \) (cm\(^3\)) | Max. prestress after grouting \( \Delta P_m \) (MPa) | Prestress change on lining \( \Delta P \) (MPa) | \( \frac{(\Delta Q/\Delta V)/(\Delta P/\Delta P_m)}{\alpha} \) |
|--------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|
| Scheme 2 | 1.82 | 2.96 | 1.6 | 1.51 | 0.58 |
| Scheme 3 | 3.08 | 4.80 | 1.4 | 1.38 | 0.63 |

It is assumed that the ratio of the drainage volume \( \Delta Q \) to the initial volume change \( \Delta V \) of the rock mass in the consolidation circle after grouting has a certain relationship with the change in prestress on the lining \( \Delta P \) and the maximum prestress after grouting \( \Delta P_m \). Thus,

\[
\alpha = \frac{(\Delta Q/\Delta V)/(\Delta P/\Delta P_m)}{\alpha}. \tag{12}
\]

The \( \alpha \) value calculated in schemes 2 and 3 is 0.58 and 0.63, respectively. Because the drainage of non-groutable fracture is the main influencing factor in the dissipation of preloading stress in the lining, the ratio coefficient \( \alpha \), at less than 1, is affected by the surrounding rock type, consolidation grouting pressure, and development degree of the microfractures. According to the results of numerous model experiments and analyses, the \( \alpha \) value range is 0.55–0.80.

### 4. Analysis on Dissipation Mechanism of Lining Preloading Stress

#### 4.1. Factors Analysis

Numerous microcracks were observed in the rock mass with openings less than 0.2 mm. Thus, because the cement slurry could not be grouted, water was discharged through the channel. The opening and connectivity of cracks affect the distance and speed of the water discharge. Under the grouting pressure, the slurry is consolidated, and the water is separated and gradually drained. The degree of drainage is related mainly to the grouting pressure, drainage conditions, and consolidation time. The preloading stress and its dissipation in the lining caused by grouting are affected mainly by the following factors.

1) The drainage of water in microconnected cracks after high-pressure grouting. The microcracks in rock mass are usually developed and connected, and the rock mass after grouting has certain interspace and passage, resulting in a certain degree of water permeability in the rock mass (generally not less than \( 10^{-6} \text{ cm/s} \)). The results of acoustic wave testing of the rock mass after grouting also confirmed the presence of numerous microcracks in the rock mass after grouting. Under the prestress action of the grouted rock mass, the water in these tiny cracks gradually drains along the cracks, and the microcracks are further compressed. A certain degree of volume expansion occurs in the rock mass after grouting, which reduces the prestress on the lining. Thus, with the water drained from the non-groutable cracks, the prestress on the lining gradually dissipates.
2) Self-influence of the stone body produced by the cement slurry. Ordinary cement slurry forms into a hard stone body under high pressure. The stone body can shrink under the prestress, which gradually dissipates the preloading effect on the lining.

3) Effects of prestress on the lining after grouting on the surrounding rock outside the consolidation grouting circle. Its deformation and internal water discharge under the prestress also affect the dissipation of the preloading on the lining.

4) Continuous deformation of the concrete stone and lining body under long-term prestress. This deformation, as well as creep in the deep rock mass, gradually releases the prestress on the lining. The analysis summary indicates that the third and fourth factors given above have small influence. The shrinkage of the stone body formed by the cement slurry showed similar research results of negligible influence. Therefore, the objective of this research to study the influence of water drainage and dissipation in microconnected fractures in consolidation grouting caused by preloading.

4.2. Basic Assumption
In this study, the following assumption were applied:

1) The crack in the rock mass was simplified as a smooth, parallel plate crack with a constant opening to calculate the diffusion process of water flow in the rough crack of rock mass; and

2) The equivalent hydraulic opening $2b_h$ was used to replace the opening $2b$ in the fracture. Under a certain hydraulic gradient and flow mode, the volume flow of the same fluid in a rough fracture is equal to that in a flat smooth fracture with an opening of $2b$; thus, $2b_h$ is referred to as the equivalent hydraulic opening of the rough fracture. Therefore, the cubic law and corresponding formulas are also true for rough fractures\(^8\).

The equivalent hydraulic opening of $2b_h$ can be determined by means of borehole hydraulic catalog and packer testing. The results of relevant research show that the relationship between the equivalent hydraulic opening $2b_h$ and mechanical opening $2b_m$ is related to the dimensionless roughness coefficient $\sigma_h/\sigma_m$ and that the mechanical opening $2b_m$ is always larger than the equivalent hydraulic opening $2b_h$. The smaller $\sigma_h/\sigma_m$ is, the greater the difference between the mechanical opening and the equivalent hydraulic opening. With an increase in $\sigma_h/\sigma_m$, the equivalent hydraulic and mechanical openings tend to be consistent.

4.3. Analysis of Dissipation Mechanism of Preloading
Numerous fractures are developed that do not completely penetrate the natural rock mass. These fractures are randomly distributed in space, and their geometric parameters also satisfy certain random distribution. The existence of these cracks indicates that the mechanical parameters of the rock mass are obviously smaller than those of the completed rock block, and they show the characteristics of anisotropy, heterogeneity, and size effect. In practical engineering, many structural planes occur in a rock mass; thus, it is impossible to accurately determine their spatial locations, geometry, and scale individually. Numerous statistical analysis results indicate that the spatial position and geometric characteristics of this type of structural surface have the characteristics of random distribution; thus, the Monte Carlo principle can be used for random simulation.

According to the results of random network simulation, each fracture of each structural plane is equivalent one by one as per the two formulas above, and the equivalent result of the latter is applied here as the rock deformation parameter so that the deformation parameter of complex fractures in a rock mass can be equivalent.

\[
E_{r,n} = E_i \prod_{j=1}^{n} \left\{ 1 + \frac{\left[ (E_{r_{ij}} - E_{r_{ij}})d + E_{r_{ij}}hd \sin \theta \right] \cos \theta}{w[E_{r_{ij}}(b \cos \theta - d) + E_{r_{ij}}d]} \right\}
\]

(13)

\[
\mu = \mu_r
\]

(14)

Based on the assumption 1) above, the cumulative water discharge of a rock mass under grouting pressure in each time period can be calculated by applying the cubic law as
\[ \Delta Q = w \sum q \Delta t, \]  
\[ q = \frac{gb'}{12v} J, \]  
where \( g \) is the acceleration of gravity; \( V \) is the kinematic viscosity coefficient of water, which could be \( 8.5 \times 10^{-3} \text{ m}^2/\text{s} \) at 25 \( ^\circ\text{C} \); and \( J \) is the hydraulic gradient along the direction of fracture diffusion, which can be calculated by formula (17) as

\[ J = \frac{\Delta P}{\Delta \rho g l}, \]  
where \( \rho \) is the density of water, \( g \) is the acceleration of gravity, and \( l \) is the length in the direction of fracture diffusion.

### 4.4. Case Study

According to the actual data of a project measured before and after grouting, when the cement slurry is injected into the rock mass under high pressure, the elastic modulus of the fresh rock mass is \( E = 25 \text{ GPa} \), the Poisson ratio is \( \mu = 0.15 \), and the unit weight is \( \gamma = 24 \text{kN/m}^3 \). When the structural surface is under compression, the elastic modulus is \( E = 2 \text{ GPa} \); under tension, the elastic modulus is \( E = 0.1 \text{ GPa} \), the Poisson ratio is \( \mu = 0.15 \), and the unit weight is \( \gamma = 15 \text{kN/m}^3 \). The field test after grouting shows that the elastic modulus of the structural surface of which the slurry is poured can be taken as 15 GPa; the peak value of preloading stress \( Pm \) in the rock mass after grouting is usually 0.5–0.7 times the grouting pressure (0.6 is taken in the calculation); and the permeability coefficient of the rock mass after grouting is \( k = 1 \times 10^{-4} \text{ cm/s} \).

Combined with the permeability coefficient of the grouting circle after grouting, the flow rate of the fissure water in the grouting circle, the total drainage volume and corresponding hydraulic gradient within the unit width satisfy equations (18) and (19).

\[ u = kJ \]  
\[ Q = kJA = kJ \pi R^2 \]  
\[ P_m - H_w \]  
\[ J = \frac{\rho g}{R} \]  

In these equations, \( R \) is the radius of grouting circle, \( m \) is the thickness of grouting circle outside the lining, which can be regarded as 10 m according to the design data; and \( H_w \) is the groundwater level. Then, we can obtain the unit discharge of the grouting circle in the corresponding period by applying the linear iterative method in the programming. According to the indoor model, \( \Delta Q/\Delta V = (0.55–0.8) \) (\( \Delta P/\Delta P_m \)), the grouting pressures are 2 MPa, 6 MPa, and 10 MPa, respectively. The lining prestress in the corresponding period is calculated as shown in Table 2.

| Grouting pressure (MPa) | 7 days after grouting | 28 days after grouting | 90 days after grouting | 180 days after grouting |
|-------------------------|-----------------------|------------------------|------------------------|------------------------|
|                        | \( \Delta Q \) (ml)   | \( \Delta P \) (MPa)  | \( \Delta P_m \) (%)   | \( \Delta Q \) (ml)   | \( \Delta P \) (MPa)  | \( \Delta P_m \) (%)   | \( \Delta Q \) (ml)   | \( \Delta P \) (MPa)  | \( \Delta P_m \) (%)   |
| 2                       | 32.8                  | 0.62–0.90              | 51–75                  | /                      | /                      | /                      | /                      | /                      | /                      |
| 6                       | 120                   | 0.67–0.97              | 19–27                  | 275                    | 1.53–2.23              | 42–63                  | 343.6                  | 1.92–2.79              | 53–77                  | /                      | /                      | /                      | /                      |
| 10                      | 169                   | 0.77–1.13              | 13–19                  | 435                    | 1.99–2.90              | 33–48                  | 642.1                  | 2.94–4.28              | 49–71                  | 710                    | 3.25–4.73              | 54–78                  |

The following results were derived on the basis of the information given in the table.

1. Under the same fissure condition, the higher the grouting pressure, the slower the dissipation of the prestress.
(2) Seven days after grouting, the dissipation of the prestress (caused by drainage of grouting ring) generated under 2 MPa grouting pressure was 51%–75% of the peak value of the prestress.

(3) Ninety days after grouting, the dissipation of the prestress generated under 6 MPa grouting pressure was 53%–77% of the peak value of the prestress.

(4) Ninety days after grouting, the dissipation of the prestress generated under 10 MPa grouting pressure was 49%–77% of the peak value of the prestress. The value after 180 days was 54%–78%. Moreover, if we consider the prestress effect on the surrounding rock outside the consolidation grouting ring and the influence of long-term deformation such as creep in the concrete, lining, and deep rock mass after grouting, the retention value of the prestress will be even lower.

5. Conclusion

The present study used a model experiment to deduce a method for analyzing the preloading effect of the lining caused by grouting based on a case study. The major conclusions are summarized below.

(1) Deformation of non-groutable cracks is the main factor affecting the dissipation of preloading stress in the lining.

(2) Under the same fissure condition, the higher the grouting pressure, the larger the deformation of the non-groutable fissure during grouting and the slower the dissipation of prestress in lining after grouting.

(3) Under grouting pressure, an essentially linear relationship occurs between the ratio of the drainage volume $\Delta Q$ of the rock mass in the reinforced circle to the volume deformation $\Delta V$ of the rock mass after grouting and the ratio of prestress change $\Delta P$ on the lining to the maximum prestress $\Delta P_m$ after grouting. The range of the ratio coefficient $\alpha$ in these cases is 0.55–0.8.

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