Coupled hydro-mechanical effect of a fractured rock mass under high water pressure

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Abstract: To explore the variation of permeability and deformation behaviors of a fractured rock mass in high water pressure, a high pressure permeability test (HPPT), including measuring sensors of pore water pressure and displacement of the rock mass, was designed according to the hydrogeological condition of Heimifeng pumped storage power station. With the assumption of radial water flow pattern in the rock mass during the HPPT, a theoretical formula was presented to estimate the coefficient of permeability of the rock mass using water pressures in injection and measuring boreholes. The variation in permeability of the rock mass with the injected water pressure was studied according to the suggested formula. By fitting the relationship between the coefficient of permeability and the injected water pressure, a mathematical expression was obtained and used in the numerical simulations. For a better understanding of the relationship between the pore water pressure and the displacement of the rock mass, a 3D numerical method based on a coupled hydro-mechanical theory was employed to simulate the response of the rock mass during the test. By comparison of the calculated and measured data of pore water pressure and displacement, the deformation behaviors of the rock mass were analyzed. It is shown that the variation of displacement in the fractured rock mass is caused by water flow passing through it under high water pressure, and the rock deformation during the test could be calculated by using the coupled hydro-mechanical model.

Key words: fractured rock mass; permeability under the condition of high water head; hydro-mechanical coupling effect

1 Introduction

The underground tunnels in a pumped storage power station are typically subjected to high water head greater than 200 m. With the development of engineering technology, the reinforced concrete lining becomes more widely employed as a substitute of steel lining in many hydropower projects in China and abroad. In most cases, cracks will be developed in the reinforced concrete lining during operation, and the surrounding rocks behind the reinforced concrete lining will be in a high water pressure environment (Zhang, 2005; Jiang et al., 2007). Therefore, it is important to understand the permeability and deformation characteristics of the fractured rock masses under the condition of high water head for estimating the leakage and stability of hydropower tunnels.

The deformation induced by seepage could be analyzed with a coupled hydro-mechanical theory (Biot, 1942, 1954, 1956; Verrijt, 1969; Sheng and Su, 1998). Seepage field has various influences on the stress and displacement of a rock mass, which has been comprehensively examined in nuclear waste repository projects (Fairhurst, 2002; Rutqvist et al., 2009). It is shown that the seepage-induced deformation is not remarkable when the pore water pressure in the rock mass is low, but with the increase in pore water pressure, the deformation could result in a serious problem in engineering (Shen et al., 2009; Xu et al., 2009). Therefore, it is very important to evaluate the deformation and stress of the surrounding rocks of a concrete-lined tunnel subjected to high water pressure.

In this paper, a special high pressure permeability test (HPPT) was performed to study the permeability and deformation of a rock mass due to high water pressure.
heads at Heimifeng pumped storage power station (Jiang, 2007). The pressure-dependence of the permeability and the deformation behavior of the rock mass during the test were investigated.

2 The HPPT scheme

The high pressure Y-pipe segment of Heimifeng pumped storage power station is 215.0 m deep and is near the fault F15. The fault F15 includes 2 branch faults. The distance between the two branch faults is about 5.0 m. The HPPT was used to investigate the permeability characteristics of the fault and fractured rock mass near the Y-pipe segment. In the testing system, pore water pressure sensors and displacement sensors were designed to monitor the variation of pore water pressure and deformation of the rock mass in slave borehole during the HPPT. Fig. 1 shows the layout of the HPPT system, with one host borehole for injecting water and several slave boreholes for measuring pore water pressure and displacement in the rock mass. Fig. 2 shows the location of the monitoring sensors for displacement and pore water pressure in the rock mass.

During the test, the water pressure in the host borehole was controlled, ranging from 1.0 to 7.8 MPa. According to the duration of water injection under different water pressures, three methods were considered in the HPPT, i.e. a high-speed injection method, a medium-speed injection method and a low-speed one. The test durations were designed to be 5, 30 and 360 minutes for each injection method. The pore water pressures and displacements in the slave boreholes and the water pressure in the host borehole were recorded every minute.

3 Coefficient of permeability

Since no formulae are available to calculate the coefficient of permeability corresponding to the HPPT, a theoretical expression is established in this section for determining the coefficient of permeability according to the water pressures in the host and slave boreholes.

It is assumed that radial water flow dominates in the surrounding rock of the host borehole and the flow satisfies Darcy’s law, as shown in Fig. 3. When the water injected in the host borehole reaches a stable state, the total discharge passing through any cylindrical section parallel to the axis of the borehole could be expressed as

\[ Q_{R0} = Q_r = Q_R \]

(1)

where \( Q_{R0} \), \( Q_r \) and \( Q_R \) are the discharges through the cylindrical surfaces with radii of \( R_0 \), \( r \) and \( R \), respectively; \( R_0 \) is the radius of boreholes; \( r \) is the radius of the cylindrical section; and \( R \) is the distance between the host borehole and the slave ones for measurement of pore water pressure.
According to Darcy’s law, the following relation holds:

\[ 2\pi r K_p i_r L_r = 2\pi R K_p i_r L_r \]  \hspace{1cm} (2)

where \( K_p \) is the coefficient of permeability of the rock mass (cm/s); \( i_r \) and \( i_r \) are the hydraulic gradients at the locations of \( r \) and \( R \), respectively; \( L_r \) and \( L_r \) are the lengths of cylindrical surfaces at the locations of \( r \) and \( R \), respectively.

Assuming \( L_r = L_r \), we have \( i_r = (R i_r) / r \). The increment of water pressure at the location of \( r \), far away from the host borehole, is

\[ dp = i_r dr \]  \hspace{1cm} (3)

Integrating Eq. (3) yields the following relation:

\[ i_r = \frac{P_{r_0} - P_r}{R \ln(R / R_0)} \]  \hspace{1cm} (4)

where \( P_r \) and \( P_{r_0} \) are the total water heads (converted from water pressure, cm) in the slave boreholes and the host borehole, respectively.

The discharge passing through the section with a distance of \( R \) from the host borehole is

\[ Q = 2\pi R L_0 K_p i_r \]  \hspace{1cm} (5)

where \( Q \) is the discharge injected in the host borehole (cm³/s), and \( L_0 \) is the length of water injected in the host borehole (cm).

Substituting Eq. (4) into Eq. (5), we obtain the formula for calculating the coefficient of permeability as follows:

\[ K_p = \frac{2\pi R L_0 i_0}{Q} = \frac{2\pi (P_{r_0} - P_r) L_0}{Q} \ln(R / R_0) \]  \hspace{1cm} (6)

If the pore water pressure is zero at the location of the slave borehole \( (R) \), and \( R \) is equal to \( L_0 \), Eq. (6) can be rewritten as

\[ K_p = \frac{2\pi R L_0 i_0}{Q} = \frac{2\pi P_{r_0} L_0}{Q} \ln(R_0 / R) \]  \hspace{1cm} (7)

Eq. (7) is the same as the formula proposed in the code (SL31—2003). However, it is rather difficult to guarantee in the test a zero value of the water pressure at the location of \( L_0 \), far from the host borehole. Therefore, it is more reasonable to use Eq. (6) to calculate the coefficient of permeability.

The coefficient of permeability of the rock mass is calculated according to Eq. (6) in the HPPT at Heimifeng pumped storage power station. Fig. 4 shows the variation trend of the coefficient of permeability versus water pressure. The fitting relation between the coefficient of permeability and the water pressure can be written as

\[ K_p = \begin{cases} 4 \times 10^{-6} P - 2 \times 10^{-5} & (P > 5.25 \text{ MPa}) \\ 1 \times 10^{-6} & (P \leq 5.25 \text{ MPa}) \end{cases} \]  \hspace{1cm} (8)

where \( P \) is the water pressure (MPa).

One observes from Fig. 4 that the coefficient of permeability almost remains constant before the water pressure is increased up to 5.25 MPa. In this case, no new cracks will be induced during the HPPT. However, when the water pressure is greater than 5.25 MPa, the coefficient of permeability increases with increasing water pressure because of the presence of some new cracks resulting from hydraulic fracturing. This phenomenon should be considered in numerical simulations.

4 Displacement analysis during HPPT

There are two boreholes designed to record the deformation of the rock mass during the HPPT. Fig. 5 shows the evolutions of displacement at the monitoring sensors installed in the borehole #4. The whole duration for recording the rock mass deformation was about 140 hours. In the test, the measured displacements were relative values with respect to the deformation at the near end of the borehole. When the rock mass moves towards the near end of the borehole, the displacement is regarded as negative, and vice versa. Fig. 5 indicates that the relative deformation of the rock mass along the axis of the measuring borehole is not large, and the maximum relative displacement is about 0.12 mm. The displacement sensors WYJ4-1 and WYJ4-2 were placed in the rock mass at opposite sides of the fault, and the other two displacement sensors, WYJ4-2 and WYJ4-3, were placed in the faults F15-2 and F15-1, respectively. The sensors WYJ4-1 and WYJ4-2 moved towards the near end of the borehole, while the sensors WYJ4-3 and WYJ4-4 moved towards the far end of the borehole because the permeability of the faults was higher than that of the surrounding rock mass.
rock mass. This deformation trend was induced by the dominating water flow and high water pressures in the faults after injection of water into the host borehole. The different pore water pressures in the rock mass near the faults formed a high hydraulic gradient and then exerted a seepage force on the rock mass at both sides of the faults in opposite direction. The opposite direction of the seepage force demonstrated the reason that the sensors WYJ4-1 to WYJ4-4 moved to opposite directions.

Though the water pressure in the host borehole experienced several pressure cycles from 0 to about 7.0 MPa during the test, the deformation of the rock mass at the measured locations did not recover to its original zero value when the water pressure in the host borehole was decreased to zero. This phenomenon indicates that the deformation caused by the water pressure gradient could not be restored to its initial state. To further interpret this phenomenon, a numerical model was employed to simulate the rock mass response in the HPPT.

5 Numerical simulation of HPPT

It is obvious that the pore water pressure and deformation of the rock mass will evolve in the HPPT. However, it is difficult to capture the variations in stress and seepage field of the rock mass in the vicinity of the host borehole by only analyzing the measured data during the test because of the limited measuring points. 3D numerical simulation of the HPPT process is therefore a better alternative for completely understanding the rock mass response.

5.1 A coupled hydro-mechanical theory

The coupled hydro-mechanical method based on poroelasticity could be employed to simulate the seepage flow and deformation process involved in the HPPT. The CODE_ASTER was used for the calculations in this paper. The basic theory used in this software is introduced as follows.

(1) Cauchy’s equation

The Cauchy’s equation can be expressed as

\[ \nabla \sigma + b = 0 \]  \hspace{1cm} (9)

where \( \sigma \) represents the Cauchy stress tensor, and \( b \) is the body force vector.

(2) Fluid mass balance equation

The Coussy’s method (Coussy 1995) is used to describe the saturated fluid flow in porous media. In this method, the fluid balance is described by

\[ \nabla w + \frac{d m}{d t} = 0 \]  \hspace{1cm} (10)

where \( w \) is the discharge vector of fluid, \( m \) is the increase of fluid mass relative to its initial state, and \( t \) is the time.

(3) Coupling constitutive model

The coupling constitutive models are given by

\[ \sigma - \sigma_0 = (K - 2G/3) e_0 + 2G e - b ((P - P_0) + \rho_0 \varepsilon_0) \]  \hspace{1cm} (11)

\[ P - P_0 = M ( - b e_0 + m / \rho_0 \varepsilon_0 ) \]  \hspace{1cm} (12)

where \( \sigma_0 \) is the stress tensor in the initial state, \( K \) is the bulk modulus under undrained condition, \( e_0 \) is the volumetric strain, \( G \) is the shear modulus, \( e \) is the strain tensor due to external force, \( b \) is the Biot’s coefficient, \( P_0 \) is the pore water pressure in the initial state, \( M \) is the Biot’s modulus, and \( \rho_0 \varepsilon_0 \) is the fluid density at reference state.

5.2 Numerical model

The numerical model includes two fault zones, an influenced zone of faults and a zone of fractured rock mass, in which the host borehole of the HPPT system is located. The distance between the two faults is about 5.0 m. The width of the influence zone of the faults is 2.0–15.0 m. The original point of the numerical model is set at the center of the near end of the host borehole. The diameter of the host borehole is 110 mm. The positive direction of \( X \)-axis points to the upstream of the tunnel. The positive direction of \( Y \)-axis points to the far end of the host borehole, and the positive direction of \( Z \)-axis is vertically upward.

The size of the numerical model is 80.0 m in the \( X \)-direction, 82.0 m in the \( Y \)-direction, and about 60.0 m in height. The host borehole is placed in the center of the \( XZ \) plane in the numerical model. The 3D finite element mesh is plotted in Fig. 6, with 49 864 brick elements and 52 311 nodal points in total.

5.3 Initial and boundary conditions

According to the results of hydraulic fracturing
method, the initial stresses in the $X$- and $Y$-directions are both estimated as 2.2 MPa, and the initial stress in the $Z$-direction is 4.4 MPa.

According to the measured data in the slave boreholes before the HPPT, the values of pore water pressures were all around 0.1 MPa. Therefore, in the numerical analysis, the initial pore water pressure is considered as 0.1 MPa due to the difficulty in obtaining the exact value on each node. Although there is a discrepancy between the specified initial condition and the measurements, the main purpose of the numerical study is to obtain the increment of pore water pressure, and such a simplification of the initial condition will not significantly influence the simulation results.

As for boundary conditions, the horizontal displacement on all the vertical faces is regarded as zero, and the vertical displacement on the bottom of the numerical model is also prescribed as zero. All the pore water pressure on the external faces of the numerical model is constant by assuming that the external faces far away from the host borehole will not be influenced by the water injection.

5.4 Material parameters

The values of mechanical parameters are employed according to the experimental results of bearing plate test and density test of wax-sealing method. The coefficient of permeability of the rock mass is estimated from the data of water pressure tests and the HPPT. The Biot’s modulus, which has a remarkable impact on the coupled hydro-mechanical processes, is back analyzed. Table 1 lists the hydro-mechanical properties needed in the numerical simulations.

5.5 Simulation steps

Five calculation steps are selected for simulation of the high-speed injected water process and the rock mass response induced by high water pressure:

1. $t = 0$, simulation of initial stress and seepage fields;
2. $t = 0$, simulation of the host borehole excavation;
3. $t = 0 - 60$ minutes, simulation of the increasing process of the water injection pressure with the coupled hydro-mechanical method;
4. $t = 60 - 120$ minutes, simulation of the stable process of the water injection pressure with the coupled hydro-mechanical method;
5. $t = 120 - 220$ minutes, simulation of the decreasing process of the water injection pressure with the same coupled approach.

6 Calculation results and analysis

6.1 Variations in pore water pressures

Fig. 7 shows the variations in pore water pressures at the location of sensor SYJ5-1. The calculated and measured pore water pressures have the same trend of variation and the same order of magnitude. The modeling results also partly reflect the delayed variation in pore water pressure in the rock mass with respect to the water pressure in the host borehole. The results indicate that the material parameters and the coupled hydro-mechanical model employed in the 3D numerical analysis are reasonable, and the coupling analysis could reflect the actual variations in pore water pressures in the rock mass during the HPPT. The measured pore water pressure shows a sharp variation at the time of about 60 minutes. This phenomenon may result from hydraulic fracturing occurring suddenly in the rock mass during the test. In the presence of the hydraulic fracturing, the coefficient of permeability of the rock mass also increases sharply, as shown in Fig. 4. The increase in the coefficient of permeability leads
to the reduction in resistance of the rock mass to water flow and the sharp increase in the water pressure. Although a fitting function of the coefficient of permeability versus the water pressure in the host borehole is employed in the numerical modeling, it fails to completely simulate the hydraulic fracturing phenomenon that may happen in the rock mass subjected to high water pressure.

Fig. 8 depicts the variation curves of pore water pressures at some points 3.5 m away from the host borehole. The pore water pressure on the plane crossing the center of water injection segment normally, is obviously higher than that on any other planes crossing the borehole normally. This is because that the water flow is restricted by both sides of the rock mass and can only move along the radial direction of the host borehole. The water flow at the other two points could move to other places along the radial direction of the host borehole. Hence the water is dissipated much quickly and the pore water pressure is relatively low. The measured water pressure in the host borehole rose up to 8.43 MPa quickly within 166 minutes and then decreased. The descending process lasted for 20 minutes. The pore water pressure in the rock mass rose again during the falling process of the water pressure in the host borehole, but its maximum value was smaller than the previous maximum value of the pore water pressure. This indicates that the transmission of pore water pressure needs more time.

6.2 Variations in displacement

Fig. 9 shows the variation in displacement at the location of sensor WYJ4-1. In Fig. 9, the positive value of the displacement means that the rock mass moves along the Y-direction. The calculated and measured displacements also have the same variation trend and good agreement as time evolves. At the same time, the calculated displacement can also reflect the delaying effect of the rock mass deformation with respect to the water pressure in the host borehole. This indicates again that the calculation parameters and the coupled hydro-mechanical model employed in the 3D numerical analysis could reflect the actual variation processes of stress and displacement in the rock mass during the HPPT.

Fig. 10 shows the displacement distribution in X-, Y- and Z-directions at the end stage of steady pressures. Under the water pressure of 7.0 MPa in the host borehole, the horizontal displacement due to the water injection occurs in the rock mass far from the host borehole. Because of the symmetry of the 3D numerical model and the applied loads, the deformation of the rock mass in X-, Y- and Z-directions appears to be symmetric, too. Because the maximum water pressure has been increased up to 7.8 MPa and maintained for about 20 minutes before the water pressure is decreased to 7.0 MPa and kept for the following 60 minutes, the maximum X-displacement appears at the location of 1.0 m away from the host borehole. The predicted maximum values of X- and Y-displacements are 1.3 and 0.6 mm, respectively. The value of Y-displacement is zero on the plane crossing normally the center of water injection segment, which indicates that the rock mass moves to opposite directions.

On the vertical profile (Y = 35.0 m), the vertical displacement is symmetric with respect to the
The maximum vertical displacement does not appear at the wall of the host borehole, because the existence of 7.80 MPa water pressure before the water pressure of 6.0 MPa is kept for 60 minutes in the test. The maximum vertical displacement appears at the location of 1.0 m far away from the host borehole too, and its value is up to 1.0 mm.

The zone of deformation larger than 0.1 mm induced by the seepage flow under high water pressure is about 13.0 m. The influence zone with a deformation larger than 0.01 mm is approximately 30 m.

6.3 Comparison of coupled and non-coupled modeling results

It has been shown that the water pressure in the host borehole could lead to deformation of the rock mass far away from the host borehole, but this effect could not be simulated if only a mechanical model is employed in the 3D numerical analysis. To illustrate this case, a non-coupled model that only accounts for the mechanical aspect is employed for a comparison. The boundary condition on the wall of the water injection segment is kept at 7.80 MPa in the non-coupled analysis. The difference of the boundary condition between the coupled and non-coupled models is that, besides the normal stress of 7.0 MPa applied on the surface of the borehole at the steady water pressure stage, a water pressure boundary is also exerted on the surface of the borehole simultaneously in the coupled model. Table 2 shows the results predicted by the two numerical models.

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In the non-coupled model, even the maximum loads are kept at 7.80 MPa, the deformation of the rock mass only appears in a limited area in the vicinity of the host borehole. The maximum displacements in X- and Y-directions are 0.6 and 0.012 mm, respectively. Both of them are smaller than the results of the coupled model. At the same time, the influence extent due to water injection is only 3–3.5 m, again much smaller than the values, 13–15 m, predicted by the coupled model. This suggests that the coupled model could better reflect the deformation response of the rock mass.

Fig. 11 shows the contours of the effective minor principal stress at the profile of Y = 40 m (in the horizontal plane (Z = 0.0 m). The maximum vertical displacement does not appear at the wall of the host borehole, because the existence of 7.80 MPa water pressure before the water pressure of 6.0 MPa is kept for 60 minutes in the test. The maximum vertical displacement appears at the location of 1.0 m far away from the host borehole too, and its value is up to 1.0 mm.

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Fig. 11 shows the contours of the effective minor principal stress at the profile of Y = 40 m (in the
middle of water injection segment). It shows that the distributions of the minor principal stress obtained by different calculation models on the same profile are different from each other. Particularly, the tensile area estimated by the coupled model due to water injection under the same water pressure is larger than that by the non-coupled model. The tensile area is 1.0–1.3 m away from the wall of the host borehole. The maximum tensile stress is over 3.0 MPa, which means that fractures may be generated near the borehole. However, no tensile area is predicted by the non-coupled model near the borehole due to the effect of the initial stress.

7 Discussions

The coupled hydro-mechanical effect is a very important phenomenon in geotechnical engineering, especially for rock masses under high water pressure. According to the results of in-situ tests and numerical simulations, there exist significant differences in stress and deformation using the traditional non-coupled analysis method and the coupled analysis theory. To better understand what happens in rock mass in the conditions of high water pressure, it is necessary to take into account the coupled hydro-mechanical effect when designing a high water head power station. It is difficult, however, to convince engineers to accept these results, because of the complexity of the coupled hydro-mechanical model and the lack of any suggestion on the coupling effect in codes for design of hydropower stations in the condition of high water head.

8 Conclusions

In most existing studies, the attention was paid to the hydraulic characteristics of rock masses in HPPTs, and the deformation response of the rock masses was often ignored. However, the stress redistribution and the induced deformation of a tunnel under high water head could affect the stability of the surrounding rocks. Based on the results of HPPT, variation of permeability of rock mass under high water head is studied. To explore the responses of rock mass in high water pressure environment, a 3D hydro-mechanical coupled model is employed to simulate the HPPT process. Through this study, the following conclusions associated with the HPPT can be drawn:

(1) The coefficient of permeability of the rock mass changes significantly under the condition of high water head. An empirical relation between the coefficient of permeability of the rock mass and the injected water pressure was obtained by a fitting method, which was used in the seepage analysis to better reflect the seepage flow process.

(2) Significant deformation may occur in the rock mass under high water head. The deformation response could be calculated by using the coupled hydro-mechanical model.

(3) If only the mechanical aspect is considered, numerical simulation of water tunnels under high water head condition may lead to an inaccurate judgment on the stability of the surrounding rocks.

Based on the above observations, it is strongly recommended that a coupled hydro-mechanical method be employed in the engineering analysis for
rock masses subjected to high water head.

References


