Stability Analysis of Surrounding Rock of Large Semi-underground Pump House under Complex Conditions

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Abstract. The stability of the surrounding rock of large underground water-diversion projects is a critical safety concern. The water intake pumping station of a water-transfer project in Guizhou Province adopts the form of semi-underground pump house. The plane size of the main pump house is 93.5 × 40.0 m, and the maximum excavation depth is nearly 85 m, making it a large underground cavern. The project site is in a karst area with a complex geological environment, and multiple faults cross the main pump house. As a result, the groundwater seepage field affects the stability of the surrounding rock after the reservoir is impounded. We used the connotative composite element method to simulate faults, and numerical simulations to analyze the stability of the surrounding rock during multi-stage excavations. The results demonstrate that the effects of the faults and seepage are significant, the influence of complex geologic faults on the stability of surrounding rock is reflected effectively by using the connotative fault element method, and using shotcrete bolt support and optimizing construction can make caverns under complex conditions safer.

1. Introduction

Although southwest China is rich in water resources, it faces severe pressures associated with supply and demand. In recent years, several droughts have brought to attention the problem of engineering water shortages in the region. Many large-scale water-diversion projects are being planned and constructed to ensure regional water security and high-quality development. Because of the local topographic conditions, underground structures are mostly used for water-diversion projects. However, because of the complex geological environment and rich groundwater, the stability of the surrounding rock of large underground caverns is a critical safety concern for underground constructions and operation.

Numerical simulation method can simulate various types of rock and soil mass, structural planes, boundary conditions, and complex cavern models, and it has been one of the main research methods for the surrounding rock stability of underground caverns. Many domestic and foreign scholars have carried out a lot of fruitful studies in this field. Cai et al. [1] used the coupling method of FLAC and PFC software to simulate the excavation process of large underground caverns. Yossef et al. [2] used the DDA method to predict and analyze the stability of karst caves. Swoboda. G et al. [3] studied the influence of different supporting time and conditions on the stability of surrounding rock of underground caverns. Xiao et al. [4] used elastoplastic finite element method to simulate the
excavation and support process of underground caverns. Fan et al. [5] used UDEC software to study the stability of jointed rock caverns under dynamic condition. Zhao et al. [6] and Sheng et al. [7] analyzed the seismic response characteristics of underground caverns of hydropower stations by using ABAQUS software. However, there are only few research cases on the stability analysis of surrounding rock of large underground pump house under complex conditions.

This study uses a water-diversion project in Guizhou Province as an example of a large semi-underground pump house to analyze the surrounding rock stability using the three-dimensional elastoplastic damage finite element and connotative fault element methods and seepage calculation theory. Construction optimization methods for underground chambers under complex conditions are explored, and the findings can provide a reference for the safe construction and operation of similar projects.

2. Numerical simulation of rock mass

2.1. Three-dimensional elastoplastic damage finite element method

The semi-underground pump house was simulated using the three-dimensional elastoplastic damage finite element method [8]. According to related research on damage mechanics [9], micro-cracks occur in rock and concrete materials and gradually expand over time. Stress releases in the micro-crack area result in the creation of a stress damage zone. The differential increase of stress in the damaged area can be represented as follows:

$$d\sigma^D_{ij} = (1-D)\sigma_{ij}^p d\varepsilon_{ij} + \frac{D}{3} \delta_{ij}^p \varepsilon_{ij} - S_{ij} dD,$$

where $D$ is the internal variable of the plastic damage, $\sigma_{ij}^p$ the elastoplastic stress matrix, $d\varepsilon$ the strain differential increment, $\delta$ the displacement component, and $S$ the deviatoric stress tensor.

The iterative method of incremental variable stiffness damage was used to solve the equation. When the surrounding rock enters a damaged state and the incremental load is very small, the damage coefficient $D$ can be considered as a constant for this iteration step, that is, $dD=0$. Therefore, Equation (1) can be simplified as:

$$d\sigma^D_{ij} = ([H_e] - [H_D]) d\varepsilon_{ij},$$

$$[H_e] = (1 - D + D \frac{\delta_{ij}^p}{3})[H_p] + (D - D \frac{\delta_{ij}^p}{3})[H_D],$$

where $[H_p]$ is the elastic stress matrix, $[H_p]$ the plastic stress matrix, and $[H_D]$ the damage stress matrix.

In each iteration, the stiffness matrix of the damage elements was modified according to Equation (4):

$$[K_p] = \int [B]^T [H_D][B] dv,$$

where $[K_p]$ is the stiffness matrix of the damage elements and $[B]$ the strain matrix of the elements.

Similar to the calculation of the elastoplastic finite element method [10], the iteration format for the damage finite element method can be derived from the above equations as:

$$[K_p][\delta]_i = \{\Delta R_p\} + [K_D][\delta]_i,$$

where $[K_p]$ is the elastic stiffness matrix, $[K_D]$ the damage stiffness matrix, and $\{\Delta R_p\}$ the incremental load of the elastoplastic finite element iteration.

2.2. Connotative fault element simulation method

Faults are important structural factors, which cause instability of the surrounding rock in underground engineering. At present, simulated faults are simplified by using long and thin solid elements in most numerical calculations [11, 12]. However, faults in real-world, large underground caverns are
generally crisscrossed and complex, greatly increasing the difficulty of finite element modelling. Moreover, as the aspect ratio of these long and thin model elements increases, singular elements tend to appear, which increases calculation errors and can cause non-convergence.

This study adopted the connotative composite element method to simulate faults. This method does not need to divide the fault into solid elements in the finite element mesh model because the fault is hidden in the mesh model (which gives the method its name).

We assumed that a fault passes through a rock mass element, as shown in Figure 1, and as a result, the element becomes an anisotropic rock mass element perpendicular and parallel to the fault plane. A homogeneous anisotropic rock mass equivalent element can be used to simulate the structural effect of the fault. The specific calculation principles and reasoning behind the connotative composite element method are detailed in [13].

![Composite rock and fault element](image)

**Figure 1** Composite rock and fault element

The elastic modulus and Poisson's ratio parallel and perpendicular to the fault plane can be derived as follows:

\[
E_h = \left[ \sum \frac{\phi_i E_{ij}}{1 - \mu_{ij}} \right] - \left( \sum \frac{\mu_{ij} \phi_i E_{ij}}{1 - \mu_{ij}} \right)^2 \left( \sum \frac{\phi_i E_{ij}}{1 - \mu_{ij}} \right),
\]

(6)

\[
\bar{\mu}_h = \left( \sum \frac{\mu_{ij} \phi_i E_{ij}}{1 - \mu_{ij}} \right) \left( \sum \frac{\phi_i E_{ij}}{1 - \mu_{ij}} \right) - \left( \sum \frac{\mu_{ij} \phi_i E_{ij}}{1 - \mu_{ij}} \right)^2,
\]

(7)

\[
E_v = 1 \left( \sum \phi_i \left[ \frac{1 - 2\mu_{ij}^2}{E_{ij} (1 - \mu_{ij})} + \frac{2 \mu_{ij}^2}{E_{ij} (1 - \mu_{ij})} \right] \right),
\]

(8)

\[
\bar{\mu}_v = \left( \sum \frac{\mu_{ij} \phi_i E_{ij}}{1 - \mu_{ij}} \right) \left( \sum \frac{\phi_i E_{ij}}{1 - \mu_{ij}} \right) - \left( \sum \frac{\mu_{ij} \phi_i E_{ij}}{1 - \mu_{ij}} \right)^2,
\]

(9)

where \( E_h \) and \( E_v \) are the elastic moduli parallel and perpendicular to the fault plane, respectively, and \( \mu_h \) and \( \mu_v \) the Poisson's ratio parallel and perpendicular to the fault plane, respectively; \( \phi_i \) is the layer thickness coefficient, and \( \phi_i = H_i / H \), \( \sum \phi_i = 1 \). \( H_i \) is the thickness of the rock mass of layer \( j \).

The equivalent parameters of the equivalent element were calculated according to (6)–(9), and the anisotropic finite element method was used for structural analysis to calculate the rock mass stress.

2.3. FEM for seepage calculation

Assuming that the seepage in anisotropic rock mass media satisfies Darcy's law, the basic seepage equation for three-dimensional steady seepage is as follows:
adopts the boundary, exploration. Engineering
\[ \frac{\partial}{\partial x} (k_x \frac{\partial H}{\partial x}) + \frac{\partial}{\partial y} (k_y \frac{\partial H}{\partial y}) + \frac{\partial}{\partial z} (k_z \frac{\partial H}{\partial z}) = 0 \]
\[ H_{|1} = \phi(x,y,z) \]
\[ k \frac{\partial H}{\partial n} _{|2} = q(x,y,z) \]
\[ H_{|3} = Z(x,y), k \frac{\partial H}{\partial n}_{|3} = 0 \]
\[ H_{|4} = Z(x,y), k \frac{\partial H}{\partial n}_{|4} \leq 0 \]
where \( H \) is the head; \( k_x, k_y, k_z \) and are the permeability coefficients in the three directions of \( x, y, \) and \( z \), respectively; \( \Gamma_1, \Gamma_2, \Gamma_3, \Gamma_4 \) and are the head boundary, discharge boundary, free surface boundary, and overflow boundary, respectively; \( n \) is the direction vector.

The mechanical effect of the seepage field is reflected in the seepage load due to the hydraulic gradient in the seepage process, which acts on the surrounding rock structure in the form of seepage body force. The change in the hydraulic gradient also caused a change in the seepage body force. The seepage body force was calculated as follows:

\[
\begin{bmatrix}
    f_x \\
    f_y \\
    f_z 
\end{bmatrix} = \begin{bmatrix}
    -\frac{\partial p}{\partial x} \\
    -\frac{\partial p}{\partial y} \\
    -\frac{\partial p}{\partial z}
\end{bmatrix} = -\gamma_w \begin{bmatrix}
    \frac{\partial H}{\partial x} \\
    \frac{\partial H}{\partial y} \\
    \frac{\partial H}{\partial z}
\end{bmatrix} ,
\]
where \( p \) is the pressure, \( H \) the head, \( \gamma_w \) and the unit weight of water.

3. Engineering case study
3.1. Project profile
The selected water-diversion project in Guizhou Province adopts the water transmission mode of "pump station pressurization + non-pressure gravity flow + pressure gravity flow," with a designed water-supply capacity of 2.3 million m³/day and average annual volume of 672 million m³. The project includes a reservoir intake pump station and secondary boosting pump station. The former is equipped with five vertical centrifugal pumps: four in active use, and one standby. The design discharge is 26.62 m³/s and the design head is 324 m.

Considering the terrain and engineering layout of the site, the water intake pump station of the reservoir was constructed as a semi-underground rectangular pump house. The plane size of the main chamber is 93.5 × 40.0 m and elevation of the foundation 908.0 m. The current ground elevation of the site is 967.1–993.0 m, and the maximum excavation depth of the pump house reaches 85 m. The exposed stratum in the pump station area is the Lower Triassic Yelang Formation (T₁y) limestone with bare bedrock, and the thickness of the strongly weathered layer is 3.0–5.0 m. The stratum lithology is limestone medium-hard rock, and the overall lithology is mainly Class III surrounding rock. There are three influential faults: F1, F2, and F3, in the pump station area, which are all approximately 2 m wide. Moreover, the groundwater of the project area is rich, as determined by geological exploration. Therefore, it is necessary to study the stability of the rock surrounding the pump station.

3.2. FEM model and calculation conditions
A three-dimensional finite element calculation model containing the main pump house was established, and the three major faults, F1, F2, and F3, were simulated with connotative fault elements. The calculation model is divided into 378,189 8-node isoparametric elements, among which 74,649 were excavation elements. The model ranges along x, y, and z-axis are 190.0 m, 148.8 m and 153.4 m, respectively. The boundary conditions of the FEM model are as follows: the bottom is vertical constraint, the surrounding is horizontal constraint, and the ground is free. According to the construction organization design, the top–down excavation of the pump house comprises a total of five stages. The calculation adopts the numerical simulation system developed by State Key Laboratory of Water Resources and Hydropower Engineering Science of Wuhan University.

The initial in situ stress field of the project area was obtained by inversion of the measured in situ stress. Geological exploration showed that the conditions of the rock mass in the pump house area are good, with Class III surrounding rock as the main rock mass. The values of the physical and mechanical parameters of the rock mass and the concrete materials are listed in Table 1.

The shotcrete bolt support scheme of this project is as follows:

C20 sprayed concrete with a thickness of 15 cm and 2000 kN prestressed anchor cables are employed. The length of the anchor cables is 25 m, and the spacing is 4.5 × 4.5 m. Additionally, system bolts of φ28@3 m × 3 m, L = 9 m (12 m) are used.

To study the stability characteristics of the surrounding rock of the semi-underground pump house, three calculation conditions were designed in this study:

Condition 1: excavation without support;
Condition 2: excavation with shotcrete bolt support, without considering the influence of groundwater seepage; and
Condition 3: excavation with shotcrete bolt support and considering the influence of groundwater seepage.

![Entire calculation model](image1)
![Staged excavation model](image2)

**Figure 2.** 3D computational finite element model of the semi-underground pump house

| Material       | Deformation modulus (GPa) | Poisson ratio | Cohesion (MPa) | Friction angle (°) | Compressive strength (MPa) | Tensile strength (MPa) | Unit weight (kN/m³) |
|----------------|---------------------------|---------------|----------------|-------------------|---------------------------|------------------------|--------------------|
| Rock mass      | 8                         | 0.25          | 1.2            | 49.2              | 60                        | 2.2                    | 25.5               |
| Concrete       | 25                        | 0.167         | 1.5            | 42.0              | 12.5                      | 1.2                    | 25                 |

**Table 1.** Mechanical parameters of rock mass and concrete
3.3. Analysis of calculation results

3.3.1. Analysis of the fault simulation effect. The deformation and damage of the surrounding rock were analyzed through the calculation results of the excavation without support (Condition 1). Taking the y = 50 m cross section as the research object, the distribution of the damage zone, stress, and displacement were obtained as shown in Figure 3(b), (c), and (d), respectively. The distribution of the damage zone of the surrounding rock is not even. The damage elements near the fault crossing area were highly concentrated, and the depth of the damage zone was larger. The main reason for this is that the rock mass near the fault is relatively broken leading to it having low mechanical parameters and poor bearing capacity; therefore, it is greatly affected by the excavation disturbance.

The principal stress distribution is shown in Figure 4. The surrounding rock stress at the fault undergoes a sudden change due to cutting of the fault, and a stress concentration phenomenon occurs at the intersection of the fault and the cavern.

The displacement distribution of the side walls is shown in Figure 5. The overall displacement gradually increases from top to bottom, reaching its maximum at the lower middle section, and then gradually decreases. Near the surrounding rock through which the fault passes, the displacement of the side walls changes abruptly, which is relatively larger than that of the adjacent areas.

The rock mass near the fault is greatly affected by excavation disturbance, and the simulation of the connotative fault reflects the influence of the fault on the stability of the surrounding rock.

![Damage zone of the right-side wall](image1)

![Damage zone of a typical cross section](image2)

**Figure 3** Damage zone of surrounding rock under Condition 1
3.3.2. **Analysis of the shotcrete bolt support effect.** Figure 6 shows the distribution of the damage area of the surrounding rock after excavation under Conditions 1 and 2. As seen in Figure 6 (a), (c), and (e), the damage volume of the surrounding rock after the first-stage excavation in Condition 1 is very small, only 10.3 m$^3$. As the excavation continues, the damage zone of the surrounding rock increases prominently. When the excavation is completed, the volume of the damage zone is 33,664.8 m$^3$ and the maximum depth of the damage zone appears in the lower middle part of the right-side wall, reaching 11.5 m. This is deep enough to threaten the stability of the surrounding rock of the pump house. From Figure 6 (b), (d), and (e), we observe that the damage zone of the surrounding rock in Condition 2 is much smaller than that of Condition 1, and there is almost no damage zone after the first-stage excavation. In the subsequent excavation process, the damage volume of the surrounding rock gradually increases, but the growth rate in Condition 2 is slow compared to that in Condition 1. At the end of the excavation, the volume of the damage zone was 11,628.8 m$^3$, and the maximum damage depth was 5.5 m, which decreased from that under Condition 1 by 65.5% and 46.6%, respectively.
According to the displacement distribution of the typical section $y = 50$ m (Figure 7), the displacement of the side walls in Conditions 1 and 2 are $8.0–20.9$ mm and $5.0–13.8$ mm, respectively, and the maximum displacement value decreases by 34.0% after applying the shotcrete bolt support.
3.3.3. Seepage influence analysis. Figure 8 shows the damage to the surrounding rock during the excavation under Conditions 2 and 3. It can be seen from Figure 8 that the damage zone of the surrounding rock under Condition 3 significantly increased when compared with that under Condition 2. After the first-stage excavation, the volume of the damage zone of the surrounding rock under Condition 3 is still very small, indicating that the previous excavation had little influence on the surrounding rock. As the subsequent excavation continues, the damage zone of the surrounding rock gradually increases, and at the end of the fifth-stage excavation, the total damage volume of the rock mass reaches 14,941.7 m$^3$, which is 28.5% higher than that under Condition 2. The maximum depth of the damage zone in the lower middle part of the side walls is 7.0 m, increased from 27.3% under Condition 2.

Figure 9 shows the displacement distribution of the surrounding rock of cross section $y = 50$ m at the end of the excavation under Conditions 2 and 3. The deformation under Condition 3 is greater compared to that under Condition 2. The displacement of the left-side wall of the pump house increases by 0.5–2 mm, whereas that of the right-side wall increases by a smaller amount. This is because the left-side wall is closer to the reservoir area and the seepage of the rock mass is more significant than for the right-hand wall, which intensifies the deformation of the surrounding rock. Therefore, anti-seepage and drainage measures should be taken during the excavation of underground pump houses to reduce the influence of seepage on the stability of the surrounding rock.
Figure 8 Damage zone distribution and damage volume changes under Condition 2 and 3.
4. Conclusions

In this study, the stability of the cavern and its surrounding rock under complex conditions in the presence of faults and groundwater in karst areas was studied. A semi-underground pump house excavation project was simulated, and the following conclusions are drawn and contributions made:

1. The simulation of complex fault structures by connotative fault elements not only makes modelling simple and fast, but also effectively reflects the influence of faults on the stability of the surrounding rock of complex underground caverns.

2. Shotcrete and rock bolt supports can significantly reduce the disturbance of excavations on rock mass, reduce the damage taken by the rock mass, effectively limit the deformation of the surrounding rock, and ensure the stability of the surrounding rock of the pump house.

3. The damage zone of the surrounding rock increases significantly and the deformation of the side walls of the cavern also increases to a certain extent due to seepage. Therefore, the effect of seepage cannot be ignored, and adequate seepage prevention steps and drainage are necessary during construction.

However, the actual geological conditions of underground engineering projects are complex, faults are widely distributed, and groundwater is abundant. Issues such as the local stability of faults, the softening effect of seepage on the surrounding rock, and the coupling of seepage damage and damaged rock mass still need to be further studied.

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