FE analysis of the shaft lock chamber in Mengdigou Hydropower Project considering construction and service cases

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Abstract: The geological conditions of the diversion tunnel of Mengdigou hydropower station are very complex. Faults, joint fissures and groundwater are developed, and the underground chambers in the entrance end of the diversion tunnel are large scale, and the mechanical deformation induced by excavation and unloading is complicated. In order to investigate the stress and deformation in the shaft chamber surrounding rock during the period of construction and running, numerical simulation method was adopted to simulate construction process, discharging, water impoundment and power generation cases. The following conclusions are obtained: 1) during construction, the surrounding rock around the transition section of shaft lock chamber is stable; 2) during the period of discharging, water storage and power generation, the tensile stress in the inner part of the joint of gate slot, side wall and bottom plate exceeds the tensile strength of lining concrete. It is suggested to strengthen the reinforcement in these zones, and the concrete compressive stress in the compression area does not exceed the compressive strength. The results from this research could be taken as the reference for the design and construction of the project.

1. Introduction

Numerical calculation method is widely used in highway, railway, water conservancy and hydropower engineering construction. Especially, when the scale of the project is large, the geological conditions are complex, and the traditional design method is not clear, it has become an important link to check the design scheme, simulate the construction process and evaluate the stability of the project operation by using the numerical simulation method[1-5]. The inlet of the diversion tunnel of Mengdigou hydropower station is located on the right bank of Yalong River, it is about 500 meters upstream of the dam axis. Two diversion tunnels with a spacing of 50m are arranged at the inlet, numbered 1# and 2#. The underground shaft lock chamber of diversion tunnel is a large size underground structure with complex working cases and high surrounding rock stress, the faults, joint fissures and groundwater are relatively developed. The stability of surrounding rock is affected by rock class, stress state, groundwater status, cavern size and shape, excavating scheme and supporting measures, etc.. Therefore, it is necessary to comprehensively analyse the rationality of surrounding rock supporting parameters during construction and operation.

In order to investigate the stress and deformation of shaft chamber and lining concrete structure,
verify the rationality of structural reinforcement under various cases (construction, discharging, water impounding and power generation), a three-dimensional finite element analysis method was applied to study the stress and deformation of shaft chamber structure and surrounding rock under various loading conditions, so as to determine the safety, feasible, economical and reasonable layout and construction scheme.

2. Geological conditions of lock chamber

2.1. Lock chamber Foundation
The lithology of lock chamber foundation is medium fine grained granodiorite (γδ52) in early Yanshanian period. A small amount of random granitized and clayed altered rocks were developed, which are slightly new, mainly in massive structure and mainly in type II rock mass. It is a good foundation, and the integrity of rock mass in local random altered rock (belt) and exposed part of small fault is poor.

2.2. Surrounding rock of lock chamber
The vertical buried depth of the lock chamber is 55~115m, and the horizontal buried depth is 70~85m. The lithology of surrounding rock is medium fine grained granodiorite (γδ52) in early Yanshanian period. A small amount of random granitized and clayized altered rocks were developed. The rock quality is uniform and slightly new. No large-scale faults and altered rocks (zones) were seen passing through this location. Small faults were distributed randomly. Joint fissures were relatively developed. The chimerism degree of structural plane is close to relatively close. Groundwater is not developed. Under the influence of random altered rock (zone), faults and fissures, the local stability conditions of side wall and crown arch are poor.

3. Calculation Scheme

3.1. Calculation scope and model
① In the upstream direction, the boundary is connected by a straight line and a curve at the top of the diversion tunnel, and the downstream boundary extends 100m from the end of the downstream transition section of the lock chamber.
② The axis of No.1 diversion tunnel extends horizontally to the valley for 80 m, and the axis of No.2 diversion tunnel extends horizontally to the mountain for 80 m.
③ The bottom plate of the gate chamber excavation extends downward to 60 m (about twice the height of the chamber), and upward to the slope surface.
④ The geological conditions are mainly considered in the model, including grade II, III1, III2 and grade IV rock masses, and the local overburden on the slope surface is incorporated into grade IV rock mass and fault F10 within the scope of analysis.
⑤ Boundary conditions: normal constraints are imposed on the left and right sides, upstream and downstream sides and the bottom, and the top surface is free.
⑥ Finite element mesh: the element size of excavation area and lining concrete is limited within 1.5m. The element size of the first floor anchorage zone (the area where the short bolt is located) is limited within 2m. The element size of the second anchorage zone (the area where the long bolt is located) is limited within 2.5m; The size of the edge element near the model boundary to the excavation area is less than 15m, while the size of the edge element far from the model boundary to the excavation area is less than 20m. From the anchorage zone to the model boundary, the element size is gradually transitioned.

According to the above element size control standards, the three-dimensional finite element mesh model established for the shaft lock chamber section and the front and rear gradual change section are shown in Figure 1. A total of 347,071 elements and 115,066 nodes were meshed.
3.2. Calculation parameters

The physical and mechanical parameters of rock and rock interfaces near the dam site of Mengdigou Hydropower Station are listed in table 1 and table 2.

Table 1. Mechanical parameters of rock interfaces

| rock interfaces category | Shear Strength($c \neq 0$) | Shear Strength($c=0$) |
|--------------------------|-----------------------------|------------------------|
| class                    | subclass                    | $f'$ | c'/MPa   | $f$ | c/MPa |
| A Rigid rock interfaces  | A1  Closed fresh            | 0.65~0.75 | 0.10~0.20 | 0.55~0.65 | 0           |
|                          | A2  Slight alteration       | 0.55~0.65 | 0.05~0.15 | 0.50~0.55 | 0           |
|                          | A3  Rusty, slightly open    | 0.50~0.55 | 0.05~0.10 | 0.45~0.50 | 0           |
| B Weak rock interfaces   | B1  Rock block and cutting type | 0.45~0.50 | 0.05~0.10 | 0.40~0.45 | 0           |
|                          | B2  Cuttings with mud       | 0.35~0.45 | 0.04~0.08 | 0.35~0.40 | 0           |
|                          | B3  Mud with cuttings       | 0.30~0.35 | 0.02~0.04 | 0.25~0.30 | 0           |
|                          | B4  Mud type               | 0.20~0.25 | 0.002~0.01 | 0.18~0.25 | 0           |

Table 2. Physical and mechanical parameters of surrounding rock

| Rock type | modulus/ $E_0$/GPa | Poisson's ratio/ $M$ | f' | c'/MPa | $f$ | c/MPa | Elastic resistance coefficient $K_0$/ MPa/cm | Firmness coefficient $f_k$ |
|-----------|---------------------|----------------------|-----|--------|-----|-------|---------------------------------------------|--------------------------|
| II        | 20~25               | 0.22~0.24            | 1.2~1.40 | 1.70~2.30 | 0.75~0.85 | 0 | 50~60 | 5~7                                  |
| III       | 10~13               | 0.25~0.29            | 1.00~1.20 | 1.00~1.50 | 0.65~0.75 | 0 | 35~45 | 4~5                                  |
| II1       | 4~6                 | 0.30~0.33            | 0.80~1.00 | 0.70~1.00 | 0.60~0.65 | 0 | 20~30 | 3~4                                  |
| IV        | 1.50~3.0            | 0.33~0.35            | 0.60~0.80 | 0.30~0.60 | 0.45~0.55 | 0 | 10~20 | 2~3                                  |
| V         | 0.25~0.55           | >0.35                | 0.40~0.55 | 0.05~0.25 | 0.35~0.45 | 0 | <10   | <1                                   |

3.3. Calculation conditions and simulation process

The calculation of the lock chamber section mainly considers construction conditions (simplified to 8-step excavation), over-current conditions, water impounding and power generation conditions.

The simulation process is as follows: 1) The initial in-situ stress field is simulated by applying self
weight of rock mass; 2) Excavating the first section of rock mass; 3) Excavating the second section and anchoring the surrounding rock excavated in the first section (And so on to the eighth section of the rock mass); 4) Anchoring the surrounding rock excavated in the eighth step and pouring the lining concrete at the bottom layer; 5) Pouring the second layer of lining concrete (And so on to the seventh layer); 6) Pouring of lining concrete for top arch; 7) Pouring the second stage concrete of the lower part; 8) Pouring the second stage concrete of the upper part; 9) Calculating over-current condition of diversion tunnel; 10) Calculating the retaining water condition (90m) of gate; 11) Calculating the power generation condition of gate.

4. Discussion of results

The sign appointment of coordinate system and stress:

1) The tensile stress is positive, and the compressive stress is negative.

2) In the lead-straight direction, the lead-straight upward is positive, and the lead-straight downward is negative. Along the river direction, pointing to the downstream is positive, pointing to the upstream is negative. In the direction of the river, pointing to the right bank is positive, pointing to the left bank is negative.

4.1. Construction case

It can be seen from Fig. 2 and Fig. 3 that most of the rock mass around the shaft tunnel in the transition section of the diversion tunnel is under pressure after the excavation. There is a little tension area in the shallow layer around the hole, and the tensile stress level is also low, less than 1.00 MPa. After excavation, the surrounding rock of the tunnel is deformed to the inside of the tunnel, and the deformation of surrounding rock around the tunnel is small. Due to the limited space, only vertical displacement contour of vertical section of shaft chamber axis is listed (Fig. 3).

Figure 2. The first principal stress contour in the longitudinal section of shaft chamber axis (MPa)

Figure 3. Vertical displacement contour of vertical section of shaft chamber axis (mm)

The maximum displacement of the upstream transition section of No. 1 diversion tunnel is no more than 3.50 mm. The rock deformation around the tunnel in the downstream transition section is larger than that in the upstream transition section. However, the maximum deformation of surrounding rock in the downstream transition section is less than 5.00 mm. The maximum deformation of surrounding rock of shaft is not more than 4.30 mm.

The buried depth of No. 2 diversion tunnel is larger than that of No. 1 diversion tunnel, and the excavation deformation of surrounding rock around No. 2 diversion tunnel is slightly larger than that of No. 1 diversion tunnel. The maximum displacement of the upstream transition section is not more than 4.40 mm, and the maximum deformation of the rock around the downstream tunnel is not more than 5.60 mm. The maximum deformation of surrounding rock of shaft is not more than 4.60 mm.
According to figure 4, the point safety factors of surrounding rock around the tunnel are all greater than 1.00, indicating that there is no yield zone in the surrounding rock after excavation. Combined with the stress and deformation analysis results of surrounding rock of the tunnel, it can be judged that the surrounding rock is stable on the whole after excavation.

4.2. Discharging case of diversion tunnel

It can be seen from the calculation results that:

1. The compressive stress level of the lock chamber section and the upstream and downstream transition section lining are relatively low. The maximum principal compressive stress of the lining is about 1.01 MPa, which is lower than the design value of the compressive strength (11.9 MPa), meeting the requirements of compressive strength.

2. The main tensile stress at the connection of side wall and bottom plate in the upstream transition section is relatively large, and the maximum principal tensile stress is 1.44 MPa, which is slightly higher than the design value of tensile strength (1.27 MPa), which can meet the requirements of bearing capacity after reinforcement.

3. The main tensile stress of concrete in lock chamber section is lower than the tensile strength.

4. The maximum principal tensile stress of the transition section of No.1 diversion tunnel is 3.29 MPa, which is higher than the design value of tensile strength (1.27 MPa). (Fig. 5)

5. The main tensile stress and compressive stress of the lining concrete in the upper part of the shaft are low, which are lower than the tensile strength and compressive strength, meeting the requirements of bearing capacity (Fig. 6).

It is suggested to strengthen the reinforcement at the junction between the side wall and the bottom plate of the lock chamber (gray part of figure 5).
4.3. Gate closed and water storage case

It can be seen from figures 7 and 8 that: ① The main tensile stress in the local area of the connection between the side wall and the bottom plate of the upstream transition section is greater than the design value of the tensile strength (1.27MPa). The main tensile stress of concrete in the local area of the junction between the upstream edge of the intermediate pier and the top arch and bottom plate are greater than the design value of the tensile strength (1.27MPa). The maximum principal tensile stress is 1.95MPa. ② The main tensile stress of concrete at the downstream corner of gate slot, the connection of side wall and bottom plate are higher than the design value of tensile strength (1.27MPa), and the maximum principal tensile stress is 1.68MPa; ③ The main tensile stress of concrete in the downstream transition section of sluice chamber section is less than the tensile strength. ④ The main compressive stress of concrete in lock chamber, upstream and downstream transition section are less than the compressive strength. ⑤ The main tensile stress and compressive stress of the lining concrete in the upper part of the shaft are lower than the design values of the tensile and compressive strength, meeting the requirements of the bearing capacity.

4.4. Power generation conditions

It can be seen from figures 9 and 10 that: ① The main tensile stress of lining concrete at the connection between the side wall and bottom plate in upstream transition section is greater than the design value of tensile strength (1.27MPa). The main tensile stress of lining concrete in the inner wall near the bank side arch foot, the upstream edge of intermediate pier and the junction of top arch and bottom plate are...
greater than the design value of tensile strength (1.27 MPa). The maximum principal tensile stress is 3.41 MPa. ② The main tensile stress of lining concrete at the downstream corner of gate slot, the connection between the side wall and bottom plate, and the connection between the side wall and roof near the river bank are greater than the design value of tensile strength (1.27 MPa). The maximum principal tensile stress is 3.26 MPa. ③ The main tensile stress of the lining concrete in the downstream transition section of the lock chamber section is less than the tensile strength. ④ The main compressive stress of concrete in lock chamber, upstream and downstream transition section are less than the compressive strength. ⑤ The main tensile stress and compressive stress of the lining in the upper part of the shaft are lower than the tensile and compressive strength, meeting the requirements of the bearing capacity.

5. Conclusion

(1) During the construction period, the rock around the transition zone of shaft and lock chamber is stable;
(2) During the discharging period of the diversion tunnel, the tensile stress of lining concrete at the junction of the top arch, side wall and bottom plate of the downstream transition section of the sluice chamber exceed the tensile strength. The tensile stress of concrete at the junction of the middle pier end, roof arch and bottom plate exceed the tensile strength.
(3) During the normal water storage (91 m water retaining condition) and power generation period, the tensile stress of some structural joints exceeds the tensile strength, it includes: the connection between the side wall and the bottom plate of the upstream transition section; the junction between the upstream edge of the intermediate pier, the top arch and the bottom plate; the downstream corner of gate slot in lock chamber section; joint between the side wall and bottom plate.
(4) The compressive stress of concrete in the compression zone does not exceed the compressive strength of concrete
(5) It is suggested to strengthen the reinforcement in the area where the tensile stress of concrete exceeds the tensile strength.

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Reference
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