Research on Deformation Mechanism of Underground Chamber Surrounding Rock Based on the Influence of Surrounding Rock Stress Path

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Abstract During the construction of the pumped-storage power station, problems such as the stability of the surrounding rock of the underground chamber of the powerhouse appeared, which put forward higher requirements for the safety of the construction of the project. How to reasonably analyze and evaluate the stability of the surrounding rock of the underground chamber became a constraint to the smooth construction of the project. Obstacles to progress. In response to this problem, a typical underground powerhouse chamber of a pumped storage power station was selected, and for the problems caused by the stress unloading of the surrounding rock during the excavation of the powerhouse chamber, field sampling, indoor triaxial unloading test, and numerical analysis were adopted. The stress path test results are applied to the numerical simulation analysis of different types of surrounding rock and different excavation conditions. The calculation analysis shows that the types of surrounding rock and the excavation conditions affect the arches, arch abutments and spans of underground chambers. Key positions such as the intersection of the arch bottom and the side wall have a significant impact. When the surrounding rock grade is the same, the displacement field distribution characteristics and maximum displacement of the two step-by-step excavation schemes are not much different; when the surrounding rock grade changes from type II to type III, the maximum displacements of the arch top and the bottom of the surrounding rock increase doubled.

1. Introduction
A large number of rock mechanics issues will be involved in the construction of pumped storage power stations, such as the stability of underground powerhouse surrounding rock, the stability of high slopes, the leakage of upper and lower reservoirs, and the seepage damage of underground powerhouses. These problems are in actual engineering. It is often encountered during construction, and it may even cause heavy losses to the construction of the project. The stability of the surrounding rock of the underground powerhouse is one of the key issues. A reasonable evaluation of the stability of the surrounding rock of the underground powerhouse is extremely important to the project construction. The excavation of a large-span underground powerhouse is a manifestation of rock
unloading. The researches on loading rock mass parameters, unloading rock mass size effect, anisotropy and rheological properties, etc. are mainly carried out through the observation and analysis of engineering unloading phenomena such as laboratory rock tests, rock mass model tests and other experimental methods.

The stress path has a great influence on the triaxial test. In the 1970s and 1980s, domestic and foreign scholars explored the influence of the stress path on the triaxial test. Jaeger [1] pointed out in 1967 that whether the strength of the rock is related to the stress path is a question to be studied. Swanson [2], Crouch [3] used conventional triaxial apparatus to study the influence of unloading confining pressure on rock strength. Their conclusion is that the stress path has no effect on rock strength. Xu Dongjun et al. [4] conducted rock deformation and failure experiments with different stress paths under true triaxial stress conditions. The experimental results show that the changes in the three principal stresses can cause rock deformation and failure; and the strength, ductility and brittleness, volume change and failure precursors of the rock are all related to the stress path. Li Tianbin and Wang Lansheng [5-7] studied the deformation and failure characteristics of basalt in the unloaded state. The main conclusion is: in the unloaded state, as the confining pressure increases during failure, the failure mode of the sample gradually changes from tension to strength. Hydropower Station's deep-buried diversion tunnel, selected typical marble in the area, and carried out unloading confining pressure failure tests and unloading confining pressure based on the actual stress environment of the tunnel surrounding rock Multi-level destruction test.

"Rock mass structure" was proposed in the 1960s. It is believed that there are many geological factors that affect the stability of rock masses, which are mainly various weak structural surfaces that damage the integrity of the rock mass structure. Types, genetic types and combination relationships, and the structural plane is considered to be an important factor affecting the stability of rock masses, laying the foundation for the formation of rock mass structure theory. Sun Guangzhong [8] further proposed "Rock Mass Structure Cybernetics" in 1984, and established the basic theory of rock mass structural mechanics comprehensively and systematically. Huang Runqiu et al. [9] conducted a more detailed study on rock mass structure based on large-scale engineering examples. Abroad, the failure of Malpasset in France in 1959 and the vajont dam in Italy in October 1963 caused great shocks in engineering geology and rock mechanics, and people began to realize the stability of the dam foundation rock mass. Performance is as important as the strength of the structure, and the systematic study of the mechanical properties of rock masses arouses people's attention. It is opposed to using rock as a continuous medium and simply using the principles of solid mechanics and material mechanics to analyze the mechanical properties of rock masses. Instead, it emphasizes that joints, cracks, and faults must be considered. In order to study the geological structure, the role of groundwater must also be considered.

2. Geological conditions of Luoning pumped storage plant site

2.1 Overview
Luoning Pumped Storage Power Station is located in Jiankou Township, Luoning County, Henan Province, and belongs to Baimajian, a first-level tributary of the Luohe River in the Yellow River system. Luoning Pumped Storage Power Station has an installed capacity of 1400MW. The power station hub includes four buildings, the upper reservoir, the lower reservoir, the water delivery system, and the underground powerhouse, with a rated water head of 604.0m. The rainwater collection area of the upper reservoir is 0.69km², the annual average runoff is 126,100 m³, the normal storage level is 1230.00m, and the corresponding storage capacity is 8.33 million m³.

2.2 Engineering geological conditions
The power generation system buildings of the powerhouse include underground powerhouses, main transformer rooms, bus tunnels and tail gate rooms. The underground powerhouse adopts a central development method and is arranged in a strong mountain body with a horizontal distance of about
1450m from the inlet/outlet of the lower warehouse, with a buried depth of more than 550m. The layout of the main underground cavern avoids the relatively developed area of faults, and the longitudinal axis of the plant is 110°. The installation height of the unit is 498m, and the excavation size of the main and auxiliary powerhouse is 164.3m×24.5m (26.0m)×54.7m.

The plant layout area has a strong mountain body, the surface elevation is 1078m-1176m, the highest peak elevation is 1466m, and there is no large gully cut along the line. The vertical buried depth of the cavern is about 570m-658m, and the overlying rock mass is about 560m-650m thick. The lithology is late Yanshanian porphyritic granite, with tight and hard rocks. According to the flat hole drilling ZK110~ZK113 and its acoustic test data, the rock mass of the cavern surrounding rock has good integrity, the rock quality index (RQD) is generally 83%-95%, and the rock mass sonic wave velocity is greater than 5000m/s. The coefficient is greater than 0.67.

3. Indoor stress path test of porphyritic granite
50mm, which are semi-automatic plate-shaped; plagioclase has a better degree of self-shaped, which is semi-automatic plate columnar crystal. It is relatively fresh, with zonal structures and albite twin crystals commonly seen. The mineral composition and the content of phenocrysts vary with different parts, generally from the center phase to the edge phase, the mineral particles (the phenocryst grain size) change from coarse (large) to fine (small), potassium feldspar and quartz content Gradually decrease, plagioclase gradually increases. Potash feldspar is 45.3% to 10%, plagioclase is 25% to 40%, and quartz is 33.1% to 15%. The matrix is longer stone (25%-30%), feldspar (10%), biotite (5%), quartz (15%-20%) and common hornblende.316mm, and a few larger than 40mm38mm~12mm3The rock is grayish white, light flesh red, medium-coarse semi-automorphic granular granite structure (like porphyritic granite structure), and massive structure. The main mineral components are potash feldspar, plagioclase and quartz; secondary minerals are biotite, common hornblende and chlorite; accessory minerals include magnetite, sphene, apatite, fluorite and pyrite, zirconium Stone etc. The phenocrysts are mainly potash feldspar (micro feldspar), and a small amount of longer stones, the content of the two is 30%-40%. Potassium feldspar phenocrysts have a large grain size, mostly 6mm

3.1 Typical full-process stress-strain curve
The full-process stress-strain curve of the intact limestone under different confining pressures in its natural state is shown in Figure1. It can be seen from the figure that under the condition of triaxial loading, the deformation of the intact limestone in the elastic stage is axial deformation Mainly, after reaching a certain deviatoric stress value before the peak value, the radial deformation rate of the sample gradually increases, which makes the sample appear obvious expansion phenomenon, and finally causes the damage of the rock sample. It can be seen from the figure that the strength of the rock mass increases with the increase of confining pressure. The slope of the elastic phase under different confining pressures increases with the increase of confining pressure, and the slope difference is not particularly large.
3.2 Stress path test analysis

Uniaxial and conventional triaxial compression failure means that the axial stress reaches the bearing capacity of the rock sample, while the unloading confining pressure failure reduces the bearing capacity to the maximum axial stress that the rock sample can withstand when the confining pressure is unloaded. Due to the path difference when the rock reaches the final failure, the peak stress during the unloading confining pressure test is no longer the so-called peak strength in the traditional loading test. As the confining pressure continues to decrease, the rock failure stress more reflects the ultimate load after the confining pressure is reduced ability. In the process of unloading the confining pressure, the deviator stress remains unchanged, but the axial strain and the radial strain show an increasing trend during the unloading of the confining pressure, and the axial strain, radial strain and body strain are The relationship is shown as the volume strain> radial strain> axial strain when unloading confining pressure failure for intact limestone or fractured limestone; when the confining pressure is unloaded, the rock sample does not immediately enter the plastic deformation stage, stress and strain are concerned Presents an ideal elastoplastic model.

Table 1 Deformation parameters of loading stages

| No. | Pressure/MPa | E50/GPa | μ 50 |
|-----|--------------|---------|------|
| 3-1 | 5            | 50.58   | 0.31 |
| 3-2 | 10           | 43.65   | 0.41 |
| 3-3 | 15           | 71.03   | 0.34 |
| 4-1 | 5            | 75.9    | 0.42 |
| 4-2 | 10           | 77.6    | 0.31 |
| 4-3 | 15           | 44.71   | 0.68 |
| 4-4 | 10           | 43.65   | 0.41 |
Fig 2 Stress and strain curve of unloading confining pressure of fractured rock mass

It can be seen from Table 1 that the deformation parameters of the complete rock sample when loaded and the loading deformation parameters obtained in the conventional loading test in Chapter 2 are not much different, and as the confining pressure increases, the unloading point of the complete rock sample gradually rises. During the transverse axial pressure unloading confining pressure test, the unloading point is the so-called peak strength, so it can be judged that the complete rock sample used in the unloading confining pressure test has similar mechanical properties and mechanical parameters as the rock sample used in the conventional loading test. From the stress-volume strain curve, it can be seen that the rock samples under different initial confining pressures have the expansion phenomenon during the loading stage, indicating that the rock samples have already undergone plastic deformation when the confining pressure is unloaded.

4. Analysis of surrounding rock deformation during excavation of underground powerhouse

The model size of this simulation is 500mX500mX600m (length X width X height). The model is divided into 1,160,356 units and 193,820 nodes. According to the calculation requirements of excavation, a total of 14 groups are divided.

Through the above analysis of the maximum displacement field and vertical displacement cloud diagram of the underground caverns after the completion of different excavation time steps, it can be seen that after the cavern excavation, due to the unloading effect, the cavern surrounding rock undergoes rebound deformation towards the inside of the cave. The chamber vault has a large sinking displacement, and the displacement value tends to increase with the excavation. The chamber floor has a large rebound deformation, and the side wall deformation is dominated by horizontal displacement. The vertical displacement of the top arch and floor is the Lord. With the progress of layered excavation, the displacement value of surrounding rock continuously increases, and the influence range of the displacement field is also continuously expanded. The maximum settlement after the seven-step excavation is 14.51mm, which is located at the top arch of the main power house, and the
The deformation value of the bottom heave of the bottom plate of the underground power house is 11.55mm.

Through the analysis of the results of the maximum principal stress and minimum principal stress in a typical section, it can be concluded that the radial stress around the tunnel is released due to the excavation of the tunnel, and the tangential stress increases. After each step of excavation, the vault and bottom of the tunnel appear in a certain range of tensile stress concentration area, if the tensile strength of the surrounding rock exceeds the tensile strength of the surrounding rock, it will cause tensile failure of the surrounding rock; the arch shoulder, side wall, and the intersection of the arch bottom and the side wall will all have obvious compression stress concentration. After the excavation of the busbar tunnel, a large area of tensile stress will be generated at the top and bottom of the tunnel, and with the progress of layered excavation, the tensile stress range will gradually expand, especially at the intersection of the busbar tunnel bottom plate and the factory building.

![Contour Of Displacement](image1)

![Contour of Max. Principal Stress](image2)

**Fig4 Computing Analysis Cloud Diagram**
5. conclusion

Through the analysis of the maximum displacement field of the underground cavern of the first excavation scheme for type II granite, it can be seen that the displacement value of the surrounding rock of the underground cavern increases with the progress of layered excavation. Comparing the final displacement field characteristics of different working conditions, the maximum displacement of full-face excavation is significantly increased compared with the two step-by-step excavation schemes; the displacement field distribution characteristics of the two step-by-step excavation schemes when the surrounding rock levels are the same and the maximum displacement is not much different; when the surrounding rock grade changes from type II to type III, the maximum displacement of the surrounding rock vault changes from 14.57mm to 29.39mm; the maximum deformation of the arch bottom changes from 17.55mm to 34.36mm.

Through the analysis of the plastic zone of the first excavation scheme for type II granite, it can be seen that the plastic zone expands continuously with the excavation. The plastic zone around the main powerhouse and the main transformer tunnel is about 2m; after the seventh step of excavation, the excavation of the arch bottom of the main powerhouse has caused a significant expansion of the plastic zone at the junction of the busbar tunnel and the main powerhouse side wall. It is about 4m. By comparing the characteristics of the plastic zone under different working conditions, the plastic zone of the surrounding rock is obviously larger than that of the two step-by-step excavation schemes in full-face excavation; the plastic zone changes of the two step-by-step excavation schemes are different when the surrounding rock grade is the same.

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