Stress-induced brittle failure of surrounding rock mass and its engineering countermeasures in the Baihetan underground powerhouse

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Abstract. Stress-induced brittle failures, such as spalling, slabbings, and rockburst, are common failure modes of underground caverns in high-stress zones. In the Baihetan Hydropower Station, the brittle failures, embodied as spalling, rock fracturing, and time-dependent deformation, bring new challenges to the excavation and support system design of underground caverns. Based on the New Austrian Tunneling Method, the specific design principles for the underground caverns of Baihetan Hydropower Station are proposed, the keys of which are preconditioning surrounding rock mass, reducing the excavation damage, supporting quickly in the proper sequence, and controlling loosen zones. These principles are applied during construction, and a series of excavation and support methods are developed for the key parts of the powerhouse, such as the roof arch, the rock wall of rock-bolted crane girder, the high side wall, and the partition wall of the generator pit. Data obtained from visual appearance, displacement measurement, and acoustic testing show that the proposed design principles are appropriate and that the developed excavation and support methods can control the range and depth of stress-induced failure of the surrounding rock mass.

1. Introduction

Spalling is a mode of instability commonly observed in brittle hard rocks. It is caused by stress-induced crack and fracture initiation, propagation, and coalescence, as well as fractures growing parallel to the free surface producing thin slabs of rock[1]. This fracturing is generally referred to as a tensional and brittle failure process without obvious rock ejection[2]-[3][4]. Many hard rock mines and tunnels in Australia, Canada, China, Chile, South Africa, Sweden, Switzerland, and other countries have experienced rock spalling and even rockburst to various degrees. Three deep underground experimental tunnel cases have been widely studied at an international scale: the Mine-by experiment tunnel in AECL’s Underground Research Laboratory[5][7], the Åspö pillar stability experiment conducted in Hard Rock Laboratory in Sweden[8]-[10], and the in situ Posiva’s Olkiluoto Spalling Experiment[11] conducted in Finland. In China, hydropower projects that have encountered severe spalling or rockburst damage to the surrounding rock mass include Guangdi, Laxiwa, Jinping I, Jinping II, and Baihetan powerhouses and tunnels.

The rock support system design in high-stress grounds differs from conventional rock support, which controls gravity-induced instability and manages shallow zones of loose rock. The rockbolts and/or rebars not only strengthen the surrounding rock mass, thus enabling the rock mass to support itself, but also control fracture initiation and propagation, leading to spalling, bulking, and large deformation. Some principles of rock support design have been proposed for tunnels in burst-prone ground[12]-[15]. These principles include the weakest link principle, the integrated system support principle, and the
simplicity principle, some of which could also be applied in large underground caverns that are prone to spalling, stress-induced fracturing, and large deformation. This paper is divided into the following parts. First, the types of stress-induced failure and surrounding rock mass deformation observed in the Baihetan underground powerhouse are summarized. Then, the principles of surrounding rock mass stability control for brittle basalt in high-stress ground are proposed according to characteristics of stress-induced fracturing and spalling. Finally, the support system design and engineering countermeasures conducted following those principles are introduced.

2. Overview of the Baihetan underground powerhouse
The Baihetan Hydropower Station is located in the lower reaches of the Jinsha River and at the boundary of Ningnan County Sichuan Province and Qiaojia County Yunnan Province. The total storage capacity of the reservoir is 20.627 billion cubic meters, and the total installed capacity of the power station is 16,000 MW. It is the second largest hydropower station in China.

![Figure 1](image1.png)

*Figure 1*. Layout of the Baihetan Hydropower Station. (a) Layout of main structures; (b) glance of the left powerhouse; and (c) glance of the right powerhouse.

The underground powerhouses are arranged in the mountain on the left and right banks of the dam. Each is equipped with eight 1000 MW Francis turbines. The underground caverns mainly include the powerhouse cavern, the transformer cavern, the draft gate maintenance gate chamber, and the tailrace surge chamber, which are arranged in parallel, as shown in Figure 1. The underground powerhouses on the left and right banks are the same size, which are 438 m long, 88.7 m high, and 34.0 m wide. The left underground powerhouse is covered under the rock mass at a horizontal depth of 600–1000 m and a vertical depth of 260–330 m. The right underground powerhouse is at a horizontal depth of 420–800 m and a vertical depth of 420–540 m.

The underground caverns are in a monoclinic stratum. The surrounding rock of the underground powerhouse is mainly basalt, which can be subdivided into oblique porphyry basalt, cryptocrystalline basalt, columnar joint basalt, almond-shaped basalt, breccia lava, and tuff. For the left underground powerhouse, the surrounding rock is mainly oblique porphyry basalt, and the right underground powerhouse is mainly cryptocrystalline basalt.

The in-situ stress field is dominated by tectonic stress, with the horizontal stress greater than the vertical stress. The maximum principal stress in the left underground powerhouse is about 19–23 MPa, and the measured maximum value is 33.39 MPa. For the right underground powerhouse, the maximum principal stress is about 15–24 MPa, and the measured maximum value is 30.99 MPa. The average saturated compressive strength of the surrounding rock is 74–112 MPa, and the strength stress ratio ($R_s/m$) of the rock is 3.22–5.89 on the left bank and 2.85–5.09 on the right bank. This result shows that Baihetan is in a high-stress area and has possible stress-induced failure. Baihetan basalts are characterized by brittle failure, especially cryptogaphic basalts, and even ejection failure. Triaxial compression test and the AE event monitoring results revealed that the uniaxial compression strength of the cryptic basalt can reach more than 200 MPa and its initial crack strength is
only around 42 MPa, which is relatively low compared with the strength (Figures 2a and 2c). In general, Baihetan basalts show hidden crack development, low cracking strength, high peak strength, post-peak brittle failure, and low residual strength.

The surrounding rock of the underground powerhouse on the left and right banks is intact, and the rockmass quality class is III in Chinese code. However, under the influence of hidden microcracks, as shown in the CT scan image Figure 2(c), the surrounding rock has low cracking strength and obvious brittleness. Those rock characteristics make the rock prone to form fracturing and spalling, and even unraveling caused by microcracks in shallow stress concentration zones, when the stress concentration during cavern excavation exceeds the cracking strength of the rock.

![Figure 2](image)

**Figure 2.** Test results of Baihetan typical basalt rocks. (a) Triaxial compression test results of basalt; (b) Laboratory AE monitoring events; (c) CT scan image of sample.

![Figure 3](image)

**Figure 3.** Prone locations and stress-induced brittle failure types of Baihetan powerhouse. (a) Stress raiser of surrounding rock; (b) Spalling at the upstream half part of roof; (c) Spalling at the sidewall; (d) Spalling at cover rock of rock wall for rock-bolted crane girder; (e) Fracturing of surrounding rock; (f) loosening and collapse of downstream sidewall.

3. **Stress-induced failure characteristics of brittle basalt**

3.1. **Prone locations and stress-induced brittle failure types**

For Baihetan in situ stress field, the horizontal stress is greater than the vertical stress. The maximum principal stress on the cross section of the powerhouse is inclined to be upstream influenced by the river valley. This characteristic makes the upstream part of the roof and the downstream side wall corner stress concentration zones. Therefore, given the powerhouse layout and excavation scheme, the powerhouse locations, such as the upstream part roof, the downstream side wall corner, the rock wall of the rock-bolted crane girder, and the partition wall of the generator pit, are likely to undergo stress-
induced failure. The stress-induced failure of the Baihetan underground powerhouse can be divided into spalling, fracturing, and loosen collapse, as shown in Figure 3.

3.2. Time-dependent characteristic
The time-dependent characteristic of stress-induced failure has two performance types, as shown in Figure 4. The first performance type is that the stress state of the surrounding rock mass gradually increases due to excavation, and the strength maintains a certain level. For example, during the downward powerhouse excavation, the stress concentration of the roof gradually increases, and the range of the stress raiser zone where stress exceeds 42 MPa would gradually increase. This phenomenon indicates that the possibility of stress-induced fracturing and spalling at the roof arch gradually increases with the downward excavation of the underground powerhouse. Cracking, propagation, and flaking of shotcrete can be observed at the powerhouse roof.

The other type performance is that when the stress level is constant, the mechanical properties of rock mass degenerate with time, or the strength of rock mass decreases with the disappearance of the confining pressure, leading to the brittle failure of the surrounding rock. For example, the range of old spalling increase and new spalling occurred 2–3 years after excavation of exploration tunnels of Baihetan. After excavation of the powerhouse, the spalling notch became deeper and larger, the shotcrete cracked, the cracks extended, and the loosen zone depth increased. The time-dependent displacement of the shallow fractured surrounding rock can be observed.

![Figure 4. Time-dependent characteristic of stress-induced fracturing.](image)

(a) stress concentration of the roof gradually increasing with downward excavation.
(b) Spalling of shotcrete at the powerhouse roof during downward excavation.
(c) Cracking of shotcrete at the powerhouse roof during downward excavation.

3.3. Local deep fracturing
In the condensation of basalt, the presence of sealing stress forms a local abnormal high-stress zone. Moreover, the basalt’s lithology is complex and hidden cracks, interlayers, and layer bedding faults develop in basalt. The secondary excavation stress field of the opening may also produce a local abnormal stress zone close to some rigid cracks, interlayers, and layer bedding faults. While the stress state of the local abnormal stress zone reaches the rock mass strength, the brittle fracturing in the deep rock would appear influenced by excavation or stress fluctuation. Macroscopically, it shows the local relatively large deformation and the local reduction of the acoustic velocity of the deep surrounding rock.

At the roof of the right underground powerhouse, displacement of the measuring points in the deep surrounding rock of section (0 + 076) and section (0 + 133) sharply increased, and the borehole TV inspection at section (0 + 133) showed that new cracks appeared and old cracks extended at the deformation growing range zone, as shown in Figure 5. At the underground powerhouse and transformer chamber of Jinping I, the fracture depth of the surrounding rock at some location greatly exceeded conventional experience[16].
Figure 5. Local deep fracturing at the powerhouse roof. (a) Deformation of the roof surrounding rock at section 0 + 076 m; (b) Deformation of the roof surrounding rock at section 0 + 133 m; (c) Borehole TV photos of roof at section 0 + 133 m.

3.4. Large deformation due to rock fracturing
Rock mass dilation may induce large deformation of the surrounding rock in high-stress ground. The rock mass dilation deformation and the damage induced by spalling and fracturing are smaller than those induced by bulking, which is a rock burst type. However, similar to bulking, the surface rock’s volume increases because of the geometry un-coordination in fracturing. Near the excavation surface, the expansion deformation of rock mass is a type of unidirectional deformation perpendicular to the excavation surface, which is related to tangential strain, and highly depends on the confining pressure. In the Baihetan underground powerhouse, the deformation appeared in the surrounding rock whose depth is less than 3.5 m at the roof arch, the proportion of monitoring points whose depth is 1.5 m and deformation is greater than 30 mm is 23%, and the maximum deformation is about 60 mm. The proportion of monitoring points whose depth is 0–1.5 m and deformation is greater than 50 mm is 31%, and the maximum deformation is about 104 mm at the upstream wall. The proportion of monitoring points whose depth is 0–1.5 m and deformation is greater than 50 mm is 30%, and the maximum deformation is about 106 mm at the downstream wall.

4. Stability control principles for basalt brittle failure
In high geo-stress environment, the excavation and support principle of underground caverns is different from the conventional support design principle aiming at controlling the instability of the
surrounding rock driven by gravity[15]. In high geo-stress environment, aside from solving the conventional problems, the support design of caverns controls the cracking and crack propagation of the surrounding rock, as well as the large deformation induced by cracking, to meet the functional requirements of the underground caverns.

Based on the New Austrian Tunneling Method, specific design principles for the underground caverns of Baihetan Hydropower Station in high-stress zones are proposed, the key principles of which are as follows.

4.1. Precondition of the surrounding rock mass
Before the excavation, necessary engineering measures can be taken to reduce the stress state of the surrounding rock or strengthen the load bearing capacity of the surrounding rock to lower the risk of stress-induced failure after excavation or reduce the damage of the surrounding rock.

4.2. Reduction of stress superposition effect through reasonable excavation
Large-scale underground caverns are generally excavated in parts and benches, and the stress distribution is influenced by the excavation of surrounding caverns. Excavation procedures should be planned reasonably to prevent the rock mass in the critical region from experiencing unfavorable stress paths. The excavation sequence that makes the same region repeatedly experience stress concentration should also be avoided to reduce the risk of stress-induced failure in the critical region.

4.3. Provision and strengthening of initial and final support in the correct sequence
After excavation, the tangential stress on the boundary increases, whereas the lateral pressure of the surrounding rock decreases, thereby reducing the strength of the surrounding rock and increasing the risk of stress-induced failure. Stress-induced fracturing and rock mass expansion are highly dependent on the confining pressure of the surrounding rock. When the surrounding rock is excavated, applying confining pressure quickly can help improve the surrounding rock state, lower the risk of stress-induced fracturing, and reduce the failure degree of the surrounding rock.

The support should be implemented in proper sequence. According to the different functions of various types of supports, the support implementation steps, procedures, and time should be rationally organized and designed. Support that can be quickly and easily implemented should be given priority, and the subsequent support should be implemented in an orderly manner.

Stress-induced failure has obvious time-dependent characteristics. When the stress concentration degree at the roof gradually increases due to the downward excavation of the underground powerhouse and the stress-induced fracturing at the powerhouse roof has obvious time-dependent characteristics, reasonable staged support enhancement should be implemented sequentially to increase gradually the lateral pressure on the surrounding rock, control effectively the damage degree of the surrounding rock at each stage, and thus ensure the overall safety of the entire support system.

4.4. Actively control the fracturing and dilation deformation synchronously
Stress-induced fracturing reduces the mechanical properties of the surrounding rock and produces large deformation, which decreases the stability of the surrounding rock. Engineering measures should be taken to limit the rupture and expansion of the rocks and to control the deformation of the surrounding rock, thereby preventing excessive deformation and catastrophe of the surrounding rock, which may cause unrecoverable losses at the post-processing stage.

Chain strength is always determined by the weakest link. Therefore, controlling the surrounding rock rupture and deformation through limiting shallow relaxation cracking and controlling deep layer rupture and deformation is crucial to regulate transient and time-dependent deformation.

5. Treatment measures for basalt brittle failure

5.1. Support system for quickly applied and in stage implement
The support system can be divided into three units and effects, namely, reinforcement, retain, and hold units, which are used to support the damaged rock mass and anchor the maintenance unit onto deep
stable rocks [13][15]. On the basis of the function of the support system and the brittle failure characteristics of the basalt, a support system suitable for high-geostress environment and for the low initiation cracking strength of brittle basalt is proposed, as shown in Figure 6. The key components and functions are summarized as follows.

**Figure 6.** Schematic of controlling loosen zone depth in shallow and fracturing-induced deformation in deep surrounding rockmass.

5.1.1. Initial strengthened shotcrete. The initial shotcrete must have a certain strength and ductility that can adapt to the deformation of the surrounding rock to slow down the crack initiation and expansion of the shallow surrounding rock. The thickness of the initial shotcrete layer can be about 5–8 cm, and the steel fiber shotcrete can be used to improve the strength of the initial shotcrete. The nanomaterials can be used to increase the thickness of the shotcrete in one shot. The initial shotcrete should be implemented immediately after excavation and as quickly as possible.

5.1.2. Systematic prestressed rock bolts quickly applied and implemented in stages. The systematic rock bolts can partially or fully be prestressed. To quickly provide active initial confining pressure to mitigate or control the crack initiation and propagation of shallow surrounding rock, priority should be given to the prestressed rock bolts, and the common mortar rock bolts may be implemented simultaneously. To quickly applied support, the early-strength grouting material may be used in the prestressed rock bolts. Rock bolts should be implemented immediately after initial shotcrete is completed, and the excavation face should be followed in time to provide active support for the surrounding rock.

The systematic rock bolts should be implemented in stages. If the final systematic bolts are implemented at one time, then during downward excavation of the underground powerhouse, the bolt stress is prone to the excessive design allowable value due to time-dependent fracturing and deformation, which may cause the overall failure of the system support structure. Therefore, the systematic bolts should be implemented in stages according to the powerhouse excavation stage and the time-dependent fracturing development stage, which can not only limit the development of the surrounding rock fracturing but also ensure the overall safety of the final support structure.

5.1.3. Steel mesh strengthened with steel strap. Steel mesh is an effective link to form surface support force, and it is often the weakest unit. A steel strap is set up and firmly connected with the rock bolts to enhance the restraining effect of the maintenance unit on the surrounding rock.

5.1.4. Systematic prestressed tendon and frame beam. Pattern prestressed tendons can be designed and implemented in an orderly manner to form an integral system with a shallow support. Such a system prevents the shallow damage from developing into the deep surrounding rock and strengthens the deep surrounding rock to limit its damage degree. Frame beams can be designed between the tendons to
enhance the support system to limit the large deformation of the surrounding rock or optimize the distribution of the supporting force applied by the tendons to the surrounding rock. The frame beams should be reliably connected to the outer anchor head of the tendons.

5.2. Cracking and relaxation control of the surrounding rock at the powerhouse roof
The powerhouse roof is a stress concentration area, and the direction of the major principal stress on the cross section is nearly horizontal. The horizontal oriented interlayer shear zones are also developed at the powerhouse roof, which makes the surrounding rock at the powerhouse roof prone to cracking, loosening, and rockfalls. In addition, the span of the powerhouse is relatively large. Once the V-shaped spalling failure zone is formed and the rockfall occurs, the arch effect at the roof of the powerhouse will be reduced, which is detrimental to the stability of the surrounding rock.

5.2.1. Precondition of the surrounding rock. As shown in Figure 7, an anchoring observation tunnel above the powerhouse is set up, and the tendons connecting the anchoring observation tunnel and the roof of the powerhouse are arranged to precondition the surrounding rock at the roof of the powerhouse, thereby improving the stability of the surrounding rock at the powerhouse roof. The anchoring observation tunnel is excavated before the powerhouse excavation. It can be used to pre-drain the surrounding rock of the powerhouse together with drainage tunnels to improve the hydrological environment and increase the overall bearing capacity of the surrounding rock. The borehole drilling is completed in advance. Thus, the prestressed tendons can be implemented immediately after the roof of the powerhouse is excavated. The surrounding rock should be reinforced before the full section of the powerhouse is exposed, and the deformation observation equipment should be installed in advance to capture the response of the surrounding rock during excavation.

5.2.2. Reasonable excavation of the powerhouse roof to reduce stress superposition. When planning the excavation procedure, the corners of excavation boundaries within the stress concentration area should be avoided. The excavation block should be large enough to cover the stress concentration area to protect the surrounding rock at the upstream roof of the powerhouse from the stress concentration by multiple times.
After the excavation of the powerhouse pilot tunnel, priority should be given to the excavation of the upstream side, and the width of the excavation block should be large enough to cover the stress concentration zone on the upstream side. Excavating the downstream side first increases the degree of stress concentration at the corner of the upstream side of the pilot tunnel and, consequently, the damage degree of the surrounding rock at the upstream roof of the powerhouse.

5.2.3. Treatment measures for time-dependent failure of the surrounding rock. On the basis of the characteristics of the time-dependent cracking of the surrounding rock and shotcrete, the cracking part of the shotcrete layer should be cleaned in time, and the damaged support should be restored. The prestressed rock bolts, tendons, or steel arches should be arranged dynamically to enhance the surrounding rocks. Therefore, the support force of the surrounding rock is increased in stages, and the damage development toward the deep surrounding rocks can be prevented.
The construction platform at the ceiling should be used in advance, as shown in Figure 8. During the excavation of the powerhouse, the construction platform can provide a working platform for the dynamic and stage support enhancement to implement the support enhancement smoothly.
5.3. Damage and cracking control of the surrounding rock of crane girder

Excavation of the crane girder is a key point in the excavation of the powerhouse. Cracking and bulking, especially at the downstream side wall, challenge the excavation of the crane girder.

5.3.1. Precondition measures. Pre-splitting on the side walls, excavation in the middle part, and reserving protective layers on both sides are adopted to release the stress of the surrounding rock in advance and reduce the risk of stress-induced failure. Through pre-splitting and middle trench excavation, the stress of the powerhouse surrounding rock is partially released; the reserved protective layer helps control the cracks and relaxation of the surrounding rock during excavation and maintain the integrity of the surrounding rock of the crane girder.

After the excavation of middle trench of the powerhouse and before the excavation of the protective layer, the fiberglass rock bolts are used to prevent the rock from rupture and damage, thereby maintaining the integrity of the surrounding rock of the crane girder and improving the excavation shape, as shown in Figure 9(a).

The additional rock bolts should be implemented below the inflection point and completed before the rock wall of the crane girder is exposed, as shown in Figure 9(b), to limit rock mass rupture and relaxation and prevent rock over-excavation.

5.3.2. Reasonable excavation procedure to reduce stress superposition. In a high geo-stress ground, when planning the excavation schemes and procedures, the surrounding rock of the crane girder should be far from the high-level stress concentration area as much as possible. The height of the excavation bench should be tall enough to cover the crane girder area. The excavation part boundaries should not be set near the crane girder to avoid the stress concentration, lower the risk of stress-induced failure, and ensure that the excavation angle of the inclined rock wall of the crane girder meets the design requirements.

Figure 9. Treatments for the surrounding rock of crane girder. (a) Reinforcing rock by fiberglass bolts before excavation; (b) Reinforcing wall under rock-bolted crane girder before excavation; (c) Prestress cables/tendons on the rock wall of the rock-bolted crane girder.
5.3.3. Measures to limit the relaxation of the surrounding rock. The crane girder surrounding rock is the foundation of the crane girder. After excavation, only the rock bolts and bar anchors are implemented, and the crane girder surrounding rock is uncovered without shotcrete until the cast of the crane girder concrete. Therefore, after the excavation of the crane girder rock wall, the pattern tendons are arranged on the vertical rock wall to provide active pressure to the surrounding rock, as shown in Figure 9(c). It can not only increase the stability but also limit the rupture and relaxation of the surrounding rock instead of using shotcrete and prestressed rock bolts.

5.4. Cracking and relaxation control of the surrounding rock of the generator pit
Corresponding to the stress concentration area at the upstream roof of the powerhouse, the bottom plate on the downstream side of the powerhouse is also the stress raiser area, which is the location of the generator pit. During the excavation of the generator pit, the stress adjustment of the surrounding rock is likely to induce damage, relaxation, or even failure of the surrounding rock.
Maximizing the surrounding rocks, keeping the partition rock mass of generator pit as complete and intact as possible, and limiting the rupture and relaxation of the surrounding rock of the generator pit are important to provide a stable foundation for the unit and to ensure the stability of the lower part of the powerhouse.

5.4.1. Precondition measures. Before the excavation of the generator pit, a concrete cover layer is casted on top of the partition rock, and then the consolidation grouting is conducted. The downward prestressed tendons are also constructed, and the outer anchor head is embedded in the cover concrete. These combined measures pre-strengthen the surrounding rock and limit the cracking and relaxation of the surrounding rock, as shown in Figure 10.

5.4.2. Measures to limit the relaxation of the surrounding rock. As the generator pit is gradually excavated downward, the left and right sides of the generator pit become exposed. The compression from the upstream and downstream side walls of the powerhouse increases the stress concentration state of the partition rock of the generator pit. Tendons are arranged horizontally on both sides of the pit to limit the degree of rupture and relaxation of the surrounding rock mass of the generator pit. In addition, the generator pit should be excavated in a staggered sequence to excavate the adjacent generator pits in different benches. Therefore, the system support and tendons can be implemented conveniently.

![Figure 10](image_url)

**Figure 10.** Treatments for the surrounding rock of the generator pit. (a) Cover concrete and cables on the partition rock of the generator pit; (b) Consolidation grouting conducted before excavation; (c) Excavation shape of the generator pit.
5.5. Deformation and loosen zone depth of Baihetan powerhouse

The keys of principles mentioned above are preconditioning surrounding rockmass, reducing the excavation damage, supporting quickly in the proper sequence, and controlling the loosen zone. These principles were applied in the whole process of Baihetan powerhouse construction and achieved a satisfactory effect.

The key locations of the Baihetan underground powerhouse, such as the roof, rock wall of rock-bolted crane girder, and the generator pit partition, have all achieved good excavation shapes.

The deformation of the surrounding rock of the underground powerhouse has also been well controlled. For the left underground powerhouse, the maximum deformation of the roof is 63 mm, and the side wall is 107 mm. In addition to the section influenced by the interlayer shear zones, the maximum deformation of the roof of the right underground powerhouse is 59 mm, and the side wall is 192 mm. After the support is completed, the deformation of the surrounding rock converges.

For the loosen zone, the average loosen depth of the roof of the left underground powerhouse is 1.3–1.7 m, and the upstream side top arch is relatively large. The depth of the side wall is generally 1.0–3.0 m. On the right underground powerhouse, the depth of the roof is generally 1.0–1.6 m, and the side wall is generally 2.0–4.0 m. Baihetan underground powerhouse. The local depth of the loosen zone is greater, the roof can reach 7.0 m, and the side wall can reach 7.6 m. After strengthening the support and reinforcing surrounding rockmass, the surrounding rock is stable.

6. Conclusion

Stress-induced failures, such as spalling, slabbing, and rockburst, are common failure modes of underground caverns in high-stress zones. Predicting and treating these failures are always the difficulties in underground engineering. The treatment measure and rock support for stress-induced failure in the Baihetan underground powerhouse have been summarized in a few guiding principles in this paper.

Brittle failure of underground excavations in hard rock is related to the in situ stress magnitude and rock mass quality. The best strategy is to precondition the surrounding rock, either strengthening the surrounding rock before excavation to increase the strength of the surrounding rock or reducing the stress state of the surrounding rock. For a large-span hydro underground powerhouse, the anchoring observation tunnel is useful for preconditioning the crown. It can be used to pre-drain the surrounding rock with drainage tunnels and to implement tendon pre-strengthening of the crown. For the surrounding rock of the crane girder and generator pit, fiberglass rock bolts on the protective rock block and concrete cover with downward tendons on the partition rock can achieve effective pre-reinforcement of the surrounding rock. Pre-splitting and middle trench excavation, reserved protective layer or block at a critical location, excavation of one part covering the stress concentration area, and smooth excavation boundary are necessary to reduce the stress state and excavation damage of the surrounding rock.

Stress-induced fracturing and rock dilation deformation are highly dependent on the confining pressure of the surrounding rock. Quickly-applied steel fiber shotcrete or nano-steel fiber shotcrete and prestressed rock bolts with early-strength grouting slurry can quickly provide relatively high active initial confining pressure to the surrounding rock after excavation. The measures can restore a certain strength of the surrounding rock and effectively limit the initiation and propagation of the shallow surrounding rock fracture.

For a huge hydro underground powerhouse, the time-dependent stress-induced failure and fracturing exhibit two modes. One is that the stress state of the surrounding rock mass increases gradually due to excavation, and the strength maintains a certain level. The other is that when the stress level is constant, the mechanical properties of rock mass degenerate with time, or the strength of rock mass decreases with the disappearance of the confining pressure, leading to the brittle failure of the surrounding rock. Systematic support implemented in stages can gradually increase the lateral pressure on the excavation boundary and the strength of the surrounding rock. It can not only effectively control the stress-induced failure of the surrounding rock at each excavation stage but also eventually ensure the overall safety of the entire support system. Moreover, using the ceiling construction platform in advance provides a good way to strengthen the crown of the powerhouse in stages.
Effective principles and engineering measures are based on the complete understanding of the geological environment, rock mechanical properties, excavation response characteristics, mechanism of support elements, and function of treatment methods. Three effective working methods can be used to maintain the stability of the surrounding rock in underground caverns: (1) preconditioning according to the advance cognition of the surrounding rock, (2) actively regulating the unfavorable response of the surrounding rock generated during excavation, and (3) actively adjusting the surrounding rock excavation sequence, support, and engineering measures according to the evolution of the surrounding rock mass characteristics and the variation of the stress state of the support and protective structures during excavation.

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