Stability control for surrounding rock of the underground powerhouse of Baihetan hydropower station in China

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Abstract. The geological conditions of Baihetan hydropower station are very complex, under which the stability control of surrounding rock encounters two major challenges: stress-induced large deformation and brittle failures, and dislocation deformation along interlayer shear zones. Brittle failure control is achieved through proper excavation sequence and fine excavation method to reduce the damage of excavation and avoid the superposition of the stress concentration area, and pre-treatment measures are applied for key parts of the powerhouse to control the excavation shape, with proper support sequence and parameters being used to strengthen support in stages and thus limit the development of fractures. Dislocation deformation control is achieved through the use of concrete replacement tunnels to promote shear resistance, proper excavation sequences to avoid rapid stress relaxation, and targeted supports on hanging wall to reduce rock mass relaxation to limit shear displacement and promote the stability of surrounding rock. The data obtained from visual appearance observation, displacement measurement, and acoustic testing show that such measures are effective.

1. Introduction

In recent years, large-scale hydropower projects in China are mainly concentrated in Sichuan Province and Yunnan Province, and mainly located along the Jinsha River, Yalong River, YaluZangbu River, and Dadu River. Since the Tertiary, the intense tectonic movement in the western region of China has produced abundant hydropower resources and created an engineering geological environment with high geostress, complex geological conditions, and high seismic intensity.

Stress-induced failure of rock masses is common in the western hydropower projects. Houziyan, Guandi, Laxiwa[1], Jinping I[2] and II[3], and Baihetan all exhibit spalling and fracturing of surrounding rock. Strong rock bursts even occurred in Jinping II diversion tunnels. Stress-induced fracturing or cracking also causes continuous large deformation of surrounding rock, which increases significantly with time. Moreover, this phenomenon causes the loose zone to deepen and the stress of blops and cables to increase, thereby adversely affecting the stability of the cavern.

Western China is characterized by strong geological activities, well-developed geological structures, and complex and diverse lithology. Large-scale weak structures, such as interlayer shear zones in basalt and large-scale faults, are often inevitably encountered, such as Xiangjiaba[4], Xiluodu, Guandi[5], and Baihetan. Large-scale weak shear planes or faults usually have poor properties, low mechanical parameters, and a tendency to degenerate when exposed to water[6][7]. Weak shear planes or structures with weak penetrability easily encounter various problems such as shear deformation, concrete spray cracking, and the bolt or anchor cable load exceeding the design value during excavation.

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The Baihetan underground powerhouse cavern group also encounters the two major challenges mentioned above. It is a typical representative of underground engineering in western hydropower stations. This paper presents an overview of Baihetan underground powerhouse caverns, discusses the characteristics, and proposes treatment principles and measures to address stress-induced failure and shear deformation of weak shear plane.

2. Overview of Baihetan underground powerhouses

Baihetan hydropower station is located in the lower reaches of Jinsha River in southwest China, and it will be the second largest hydropower station in China after construction. The project includes the construction of a concrete double-curvature arch dam with a maximum dam height of 289 m and two underground powerhouses with a total installed capacity of 16,000 MW. The dam crest elevation is 834 m, the impoundment level is 825 m, and the reservoir capacity is 20.627 bn m³, as shown in Figure 1. The underground powerhouses are arranged in the mountain on the left and right banks, and each is equipped with eight 1,000 MW Francis turbines that are locally manufactured in China (Figure 2).

Underground powerhouse caverns contain the powerhouse, transformer house, draft tube bulkhead gate chamber, and four cylinder-shaped tailrace surge chambers, which are arranged in parallel. Two powerhouses have the same size, which are 438 m long, 88.7 m high, and 34 m wide. The left underground powerhouse is covered by a rock mass with a horizontal depth of 600 m–1000 m and a vertical depth of 260 m–330 m. The right underground powerhouse has a horizontal depth of 420 m–800 m and a vertical depth of 420 m–540 m. The lithologies of underground caverns mainly include cryptocrystalline and amygdaloidal basalts, which show significant brittle characteristics [8][9]. The ratio of crack initiation strength to peak strength is 0.45 to 0.55. Many different-scale geological structures, such as interlayer shear zones, faults, joints, and cracks, are well developed (Figure 3). Gently dipping interlayer shear zones—C2, C3, C4, C5—run through the whole powerhouse caverns, (Figure 4). The maximum principal stress of the left bank underground powerhouse caverns is 19 MPa–23 MPa, and that of the right bank is 22 MPa–26 MPa. The average saturated compressive strength of surrounding rock is 74 MPa–112 MPa, and the strength stress ratio \( \left( R_b / \sigma_1 \right) \) of the rock is 3.22–5.89 on the left bank and 2.85–5.09 on the right bank.

![Figure 1. Concrete double-curvature arch dam of Baihetan hydropower station.](image1)

![Figure 2. Underground powerhouse caverns of Baihetan hydropower station.](image2)

![Figure 3. Longitudinal section of the left powerhouse of Baihetan hydropower station.](image3)

![Figure 4. Interlayer shear zone C2 cutting through the main caverns.](image4)
3. Brittle failure of basalt and stability control of surrounding rock

3.1. Stress-induced brittle failure of basalt
The in-situ stress field of Baihetan is dominated by tectonic stress, with horizontal stress being greater than vertical stress, and influenced by deep valley. The maximum principal stress on the cross section of powerhouse is inclined to upstream, both for the left and right banks. The upstream part of roof and the downstream side wall corner are stress concentration zones. The possibility of stress-induced failure is greater in these zones, such as the upstream part roof, downstream side wall corner, rock wall of rock-bolted crane girder, and the partition wall of the generator pit.

The stress-induced failure of Baihetan can be divided into spalling, fracturing, and loosen collapse[10], as shown in Figure 5.

The stress-induced failure of Baihetan shows the following characteristics:
- **Time dependence.** The stress state gradually increases due to excavation, or the mechanical properties of rock mass degenerates with time, which lead to the development of brittle failure. The spalling and cracking of surrounding rock increase with time.
- **Local deep fracturing.** When the stress in the local abnormal stress zone of the deep surrounding rock reaches the strength of the rock, the deep surrounding rock may exhibit brittle fracture. This characteristic shows the local relatively large deformation and the local reduction of acoustic velocity of rock in deep surrounding rock.
- **Large deformation due to rock fracturing.** The surface rock’s volume increases because of the uncoordinated geometry in the fracturing process. Rock mass dilation may induce large deformation of surrounding rock in high-stress ground. This characteristic shows that the overall deformation is relatively larger than the deformation of a similar project in a low-stress zone.

![Figure 5](image)

**Figure 5.** Prone locations and stress-induced brittle failure types of Baihetan powerhouse[10]. (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 sidewalk.

3.2. Stability control principles for basalt brittle failure
On the basis of the characteristics of stress-induced failure, specific design principles for the underground caverns of Baihetan are proposed[10], and the key principles are as follows:
- **Precondition the surrounding rock.** Before excavation, necessary engineering measures can be taken to reduce the stress state of surrounding rock or to strengthen the load bearing capacity of the surrounding rock to lower the risk of stress-induced failure or reduce the damage of surrounding rock.
Reduce the stress superposition effect through reasonable excavation procedure. Excavation procedures should be planned reasonably to prevent the rock mass in the critical region from experiencing unfavorable stress paths and stress concentration by multiple times.

Provide the initial support quickly, and implement the final support in proper sequence and strengthen the support in stages. Stress-induced fracturing and rock mass expansion are highly dependent on the confining pressure of the surrounding rock. After excavation, quickly applied confining pressure can help reduce the failure degree. Stress-induced failure has obvious time-dependent behaviors, so reasonable staged support enhancement should be implemented sequentially to gradually increase the lateral pressure on the surrounding rock and effectively control the damage degree of the surrounding rock at each stage.

Actively control the fracturing and dilation deformation synchronously. Engineering measures should be taken to limit the rupture and expansion of the rocks, and actively control the surrounding rock to prevent excessive deformation and catastrophe, which may cause unrecoverable losses at the costly post-processing stage.

3.3. Treatment measures for basalt brittle failure

3.3.1. Precondition the surrounding rock at the key parts of the powerhouse. The common measures can be summarized into three categories: pre-reinforcement, pre-drainage, and pre-grouting. For the roof of the powerhouse, an anchoring observation tunnel is set up above the powerhouse, and the tendons connecting the anchoring observation tunnel and the roof of the powerhouse are arranged to pre-treat the surrounding rock. The anchoring observation tunnel is excavated before the powerhouse excavation and can be used to pre-drain the surrounding rock together with drainage tunnels to improve the hydrological environment. For the surrounding rock of crane girder, fiberglass rock bolts are used before the protective layer excavation to protect the rock from rupture and damage. For the surrounding rock of generator pit, the concrete cover layer is cast on top of the partition rock before the excavation, and the consolidation grouting is conducted. Then, the downward prestressed tendons are constructed, and the outer anchor head is embedded in the cover concrete. These combined measures achieve the purpose of pre-strengthening the surrounding rock and limiting the cracking and relaxation of the surrounding rock.

3.3.2. Reasonable excavation procedure to reduce stress superposition. During excavation of the powerhouse roof, the excavation block is set to be large enough to cover the stress concentration area so that the surrounding rock at the upstream roof does not have to experience stress concentration many times. For the surrounding rock of crane girder, presplitting and middle trench excavation method are adopted to partially release the stress of the surrounding rock. The height of the excavation bench is set to be tall enough to cover the crane girder area to make sure that the surrounding rock is kept out of the high-level stress concentration area. For the generator pits, the rock is excavated in a staggered sequence so that the adjacent generator pits can be excavated in different benches, thus ensuring that the systematic bolts and tendons can be implemented conveniently to quickly provide active initial confining pressure.

3.3.3. Support system for quickly applied implementation in stages. According to the brittle failure characteristics of the basalt, a supporting system suitable for a high geostress environment and the low initiation cracking strength of brittle basalt is proposed in Baihetan[10], as shown in Figure 6. The core of the supporting system is quick implementation, enough strength to apply sufficient confining pressure, and phased strengthening according to the state of the surrounding rock.

The initial shotcrete should be implemented immediately after excavation. Steel fiber or nanomaterials can be used to improve the strength of the initial shotcrete. Then, systematic prestressed rock bolts are applied quickly. The systematic bolts should be implemented in stages to enhance the surrounding rock according to the excavation stage and the time-dependent fracturing development stage. For local deep fracturing and large deformation, systematic prestressed tendon can be used to provide relatively strong supporting force to the surrounding rock. Pattern prestressed tendons can be
designed and implemented in an orderly manner to form an integral system with the shallow support and thus prevent shallow damage from developing into the deep surrounding rock and strengthen the deep surrounding rock to limit its damage degree.

Support sequence
1. excavation
2. Steel fiber shotcrete
3. Prestress rock bolts first, common mortar rock bolts simultaneous, and quickly applied
4. Steel mesh and secondary shotcrete
5. Systematic cables or second stage rock bolts
6. n th stage support enhancement
7. Final support

**Figure 6.** Schematic of controlling loosen zone depth in shallow and fracturing induced deformation in deep of surrounding rock [10].

3.4. Deformation and loosen zone depth of Baihetan powerhouse
The key locations of the underground powerhouse, such as the roof, rock wall of rock-bolted crane girder, and the generator pit partition, all achieved the desired excavation shapes. The deformation of the surrounding rock was also well controlled.

The maximum deformation of the roof of the left underground powerhouse is 63 mm, and the side wall is 107 mm. The maximum deformation of the roof of the right underground powerhouse is 59 mm, and the side wall is 192 mm due to the influence of interlayer shear zones. The average loosen depth of the roof is 1.0–1.7 m, and the local can reach 7.0 m. The side wall is generally 1.0 m–4.0 m, and the local can reach 7.6 m. After the support is completed, the deformation converges.

4. Shear displacement along interlayer shear zone and stability control of surrounding rock

4.1. Shear deformation characteristics of interlayer shear zone
The interlayer shear zone has poor mechanical properties and softens easily when exposed to water. The interlayer shear zone C2 runs through the left powerhouse, which controls the deformation distribution of surrounding rock. Numerical analysis results show that the surrounding rock dislocates along the interlayer shear zone. The shear deformation can reach 50 mm–70 mm, and the dislocation depth can reach twice the span of the powerhouse. The deformation of surrounding rock can reach more than 100 mm in the influenced location of the interlayer shear zone C2. The interlayer shear zone increases the deformation of surrounding rock and the loosen depth of the high side wall.

**Figure 7.** Shear displacement along the interlayer shear zone C2[11].

**Figure 8.** Dislocation phenomenon of interlayer shear zone C2.
The influence of interlayer shear zone C2 on the downstream side wall is greater than on the upstream. The spatial feature of C2 inclined from the downstream of the powerhouse to the upstream makes it easier to cut out the rock mass of the hanging wall of the downstream side wall[11] (Figure 7). This condition indicates that after excavation, the hanging wall rock mass deformation in the downstream side wall is larger than the footwall, and the risk of failure and loosening is higher. The controlling deformation and loosen zone depth of the hanging wall rock mass is the key to control the shear deformation.

The obvious dislocation phenomenon of the hanging wall and footwall rock masses was observed during the interlayer shear zone C2 being exposed (Figure 8).

4.2. Treatment principles for dislocation control of interlayer shear zone

According to the influence of the interlayer shear zone on the deformation of the surrounding rock, the following treatment principles are proposed:

- **Actively control the hydrological environment of the interlayer shear zone.** Through necessary engineering measures, the water channel formed by the interlayer shear zone is cut off and the groundwater in the powerhouse area is drained to improve the hydrological environment and reduce the risk of deterioration of its mechanical parameters.

- **Pre-control the mechanical properties of the interlayer shear zone and actively control the depth of shear deformation.** Before the interlayer shear zone is exposed at the side wall of the powerhouse, necessary engineering methods should be applied to improve its mechanical properties and shear resistance. At the same time, engineering measures should be taken to control the dislocation and deformation of the deep surrounding rock to limit the influence range of the shear deformation, and minimize its adverse effects on the stability of the surrounding rock.

- **Actively control the relaxation and deformation of the surrounding rock on the hanging wall of the interlayer shear zone.** When the interlayer shear zone is exposed at the side wall, support measures should be taken to improve the integrity of the hanging wall rock mass, actively control its relaxation and deformation, improve the hanging wall and footwall rock mass deformation synergy, and reduce shear deformation.

4.3. Treatment measures for dislocation deformation of interlayer dislocation zone

Various engineering measures are used flexibly and rationally to form a combined control measure for controlling shear deformation of the interlayer shear zone C2.

4.3.1. **Jointly set up anti-seepage interception tunnels, grouting curtains, drainage hole curtains, and other measures to improve the hydrological environment of the interlayer shear zone.** Comprehensive measures such as anti-seepage interception tunnels, grouting curtains, and drainage hole curtains are adopted to improve the hydrological environment of the interlayer shear zone. A grouting curtain system and an anti-seepage interception tunnel are used to intercept the seepage channel from the front point of the seepage path, a drainage system is set up to drain the groundwater in the powerhouse area, and drainage holes are specially set up for drainage of the interlayer shear zone. This approach is called the “interception–blocking–draining” comprehensive treatment technology, which actively regulates and controls the hydrological environment (Figure 9).

4.3.2. **Set up replacement tunnels for the interlayer shear zone to improve the shear resistance and control the influence range of dislocation deformation.** To reduce the shear deformation along the interlayer shear zone C2 at the side wall of the underground powerhouse, replacement tunnels are set up 12 m away from the side wall and they were implemented in advance (Figure 10). The excavation section of the replacement tunnel is 6 m × 6 m. After the excavation is completed, the primary concrete lining is carried out, consolidation grouting is applied, and finally, the middle gallery is backfilled with concrete.
4.3.3. Improve the integrity of the surrounding rock in the hanging wall of the interlayer shear zone and control the relaxation and deformation of the deep and shallow surrounding rock. The interlayer shear zone is excavated and exposed in sections, and support is applied immediately after excavation to avoid rapid stress relaxation of hanging rock mass. Along the trace of the interlayer shear zone on the side walls of the powerhouse, anchor cables are used to improve the rock mass integrity and limit the relaxation and deformation of the hanging surrounding rock. At the same time, anchor piles are used to lock the exposed traces to prevent the shallow surrounding rock from loosening and collapsing, thereby promoting the stability of the hanging rock mass.

![Figure 9](image9.png) 
**Figure 9.** Measures to improve the hydrological environment of the interlayer shear zone C2.

![Figure 10](image10.png) 
**Figure 10.** Replacement tunnels and monitoring inclinometers for the interlayer shear zone C2.

4.4. Shear deformation along interlayer shear zone C2

An inclinometer and dislocation meter are used to observe the shear deformation of the interlayer shear zone. The inclinometers are buried in the busbar tunnel before the exposure of the interlayer shear zone, and they are about 5 m (3#, 4#) and 35 m (1#, 2#) away from the downstream side wall of the powerhouse (Figure 10). The dislocation meters are buried in the exposed trace of the interlayer shear zone at the side walls of the powerhouse.

![Figure 11](image11.png) 
**Figure 11.** Shear displacement along the interlayer shear zone C2 obtained by inclinometers.

The shear deformation changes with the excavation and exposure of the interlayer shear zone C2. The hanging rock mass above C2 is gradually exposed during downward excavation of the powerhouse, and the shear deformation gradually increases. When the hanging rock mass is completely exposed, the value of shear deformation reaches the maximum. During the continued excavation process, the rock mass of the footwall was gradually exposed, but the shear deformation value decreased slightly due to the increase in the displacement of the footwall. After the excavation of powerhouse is completed, the shear deformation along C2 converges, being 38 mm for the 3# inclinometer and 26 mm for the 4# inclinometer, as shown in Figure 11.

The monitoring data obtained by 8 dislocation meters arranged on the surface of the surrounding rock show that, except for the two larger measuring points, which are 20 and 6 mm, respectively, the absolute values of the remaining measuring points are all less than 2 mm, indicating that more shear deformation occurred during the excavation process, whereas the shear deformation produced over time after the excavation is completed is relatively small.
5. Conclusions
The properties of the Baihetan underground powerhouse caverns, such as high-stress field, brittle basalt, developed joints, and gently dipping shear planes, pose two challenges to the stability control of surrounding rock during construction: stress-induced failure and shear displacement along interlayer shear zones.

For stress-induced failure, the best strategy is to precondition the surrounding rock before excavation, which involves either strengthening the surrounding rock or reducing the stress state of the surrounding rock. The most common measures are pre-reinforcement, pre-drainage, and pre-grouting. During excavation, the excavation block and sequence are properly set to avoid encountering stress concentration, especially stress concentration experienced multiple times. Strong support is then implemented immediately to apply sufficient confining pressure to the surrounding rock after excavation, and phased strengthening is applied according to the state of the surrounding rock to reduce or mitigate fracture propagation and failures.

For dislocation of the interlayer shear zone, the hydrological environment should be pre-treated before excavation to improve the mechanical parameters of the surrounding rock, and the replacement tunnel should be set up before excavation to improve the shear resistance, which is the key to controlling the magnitude and scope of the dislocation deformation. Excavation relaxation is reduced and the integrity of the interlayer shear zone hanging wall rock is improved to control the relaxation deformation of deep and shallow surrounding rock, which can also help control dislocation deformation.

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