Key rock mechanical problems of underground powerhouse in Shuibuya hydropower station

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Abstract: The complicated rock structures and the stability of surrounding rocks of the underground powerhouse were the key rock mechanical problems in Shuibuya hydropower station. In order to overcome the related rock mechanical problems encountered during its construction, a comprehensive research was carried out for the underground powerhouse in Shuibuya hydropower station based on a detailed geological survey. It covers the investigations on the initial in-situ stress distribution features, rock mechanical properties, engineering rock mass classifications by different methods, numerical modeling for stability and support analysis, proper measures for rock excavation and support. The results show that the rock excavations of the underground powerhouse under the given geological conditions can be controlled effectively. Some measures, suggested by the designers, are proved to be rational and effective. These measures mainly consist of: (1) the soft rock replacements by concrete in local area below the crane beam, (2) the shotcrete and reinforcement by rock bolts and anchor cables in surrounding rocks, and (3) 2 m concrete placement on the rock bench between adjacent tailrace tubes. The engineering practice shows that the treated surrounding rocks have a good overall stability. The deformation behaviors observed by safety equipments are within the designing limits. The research conclusions on the related rock mechanical problems, prior to the underground powerhouse excavations, are reliable.

Key words: Shuibuya hydropower station; underground powerhouse; stability of surrounding rocks; rock excavation and support; soft rock replacement

1 Introduction

The site of a proposed hydropower stations usually has many unfavorable geological structures, threatening the stability of surrounding rocks of underground powerhouses. According to engineering practices in underground powerhouse at present in China, four typical cases are listed as follows: (1) unfavorable fault intersections with the surrounding rocks, such as fault faults intersections of f16 and f1 in the downstream wall of the main underground powerhouse in Baishan hydropower station [1, 2], and potential unstable rock blocks formed by fault intersections of F22, F20, f84, f10, etc., in downstream wall of main powerhouse in the Three Gorges project [3]; (2) siltized interlayers located nearly above the roof of the underground powerhouse in sedimentary rocks in Xiaolangdi project [1, 4]; (3) large-scale karst caverns near surrounding rocks in Goupitan hydro-power station [5, 6]; and (4) steeply dipped bedding planes in surrounding rocks with their orientations nearly parallel to the axis of underground powerhouse in Pengshui hydropower station [7]. For these unfavorable geological conditions, some special engineering measures have been proven to be successful. They include (1) reinforced concrete tunnel replacement combined with anchor cables used in Baishan hydropower station; (2) concrete replacement in controlled faults combined with anchor cables in the downstream wall of underground powerhouse in the Three Gorges project; (3) systematic anchor cables installed in the roof and sidewalls of main underground powerhouse in Xiaolangdi project and Penshui hydropower station; and (4) treatment mainly with
concrete backfill in karst caverns in Goupitan hydropower station [1–7].

The underground powerhouse is the main structure in Shuibuya hydropower station. Its stability problems were concerned mainly due to the unfavorable geological conditions. They include the soft and hard layers of rocks alternately distributed in surrounding rocks, especially with the soft rock in the lower part of the powerhouse, and several interlayer shear zones developed between the soft and hard rock layers or in the soft rocks. These shear zones behave obviously the mechanical properties as those of soft rock. They have lower shear strength and creep property when they are exposed to the excavation surface or met with water. It is shown that these unfavorable geological features could have great side effects on the stability of the underground powerhouse if no special reinforcements in addition to rock bolt and shotcrete are applied.

In order to make a reasonable evaluation for the stability of underground powerhouse, a comprehensive investigation on key rock mechanical problems of underground powerhouse in Shuibuya hydropower station has been carried out. It is based on a detailed scheme including the initial in-situ stress distribution features, rock mechanical properties, engineering rock mass classifications by different methods, numerical modeling for stability and support analysis, safety margin for overloading and failure features, proper measures for rock excavation and support [8–15]. Based on a general introduction to the key rock mechanical problems of the underground powerhouse, related to the complicated rock structures and the soft rock on the surrounding rocks, a brief summary of the investigation on stability and rock support for the underground powerhouse in Shuibuya hydropower station is presented in this paper. Its practical engineering supports and the related deformation behaviors are also analyzed and discussed.

2 Layout and geological conditions

Shuibuya hydropower station is the third hydropower station developed in cascade in Qingjiang River, a sub-branch of Yangtze River. It has a concrete faced rockfill dam (CFRD). Its main underground structures include underground powerhouse, spillway, discharging tunnel and navigation structures. The maximum dam height is 233 m, and it is one of the highest dams at present. It has 4 units of generators with a total installed capacity of 1 600 MW.

The underground powerhouse is located in the right bank of the dam. It consists of intake tunnels, main powerhouse, busbar galleries, the 500 kV translators installed in the ground surface, the tailrace tunnels and the related structures for its intake and outlet designs. The main powerhouse is 23 m in span, 68 m in height and 150 m in length. The elevations of the main powerhouse are 230.47 m at its roof and 162.80 m at its lowest excavation surface. Its overburden thickness is 170 to 220 m.

The axial direction of the main powerhouse opening is 296°, and the attitude of rock stratum is 245°±8°–15°. The surrounding rocks of the main powerhouse mainly consist of limestone, bioclastic limestone, sandstone and marly shale. The rock strata extending from the roof to the bottom of the opening consist of Qixia group (P1i, P1i, P1ma, P1ms), Ma’an group (P1ma), Huanglong group (C2h), and Xiejingsi group (D3x).

Joints in the underground powerhouse area are developed and categorized into four groups according to their orientations. Joints are steeply inclined in most cases, and have an average width of 0.2–2.0 cm. Figure 1 shows the cross-section of the underground powerhouse where rock masses in the strata P1i, P1i, P1ma, lower C2h and D3x usually behave as moderately soft rocks or interlayer shear zones.

In addition, except for the horizontal exploratory adits excavated directly from river bank above water level, a special exploratory adit named PD42, designed to investigate in detail the geological conditions especially for the lower soft rocks, was excavated prior to the excavation of the underground powerhouse. The special adit PD42 was 24° downward inclined and 280 m long and extended from the elevation of 225 m nearly to the roof of the main powerhouse at the elevation of 145 m, much lower than the bottom of the powerhouse excavation. It supplied us not only an adit for geological survey for different strata, but also a place for in-situ rock mechanical tests directly.

3 Characteristics of in-situ stress around underground powerhouse

In-situ stress measurements were conducted using hydro-fracturing test in three crossed boreholes in adit PD42 located in the upper part of powerhouse. The three crossed boreholes consisted of a horizontal borehole with a depth of 22.2 m, an inclined borehole with 45° downward from horizontal direction and a depth of 26.1 m, and a vertical borehole with a depth of 62.5 m. The above three boreholes were located
in a vertical plane with its strike of N46°E, normally perpendicular to the axis of powerhouse. By using hydro-fracturing test in the three crossed boreholes, two-dimensional (2D) stresses of 13 different intervals were obtained. With above measured data, three-dimensional (3D) stresses of the test points could be calculated by using elasticity mechanics.

The 3D rock stress measurements with the over-coring stress relief method were carried out separately at three strata, i.e. 

### 4 Mechanical properties of engineering rock mass

The rock mass mechanical properties in terms of

<table>
<thead>
<tr>
<th>Elevation (m)</th>
<th>zkf113</th>
</tr>
</thead>
<tbody>
<tr>
<td>480</td>
<td>460</td>
</tr>
<tr>
<td>440</td>
<td>420</td>
</tr>
<tr>
<td>400</td>
<td>380</td>
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<tr>
<td>360</td>
<td>340</td>
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<tr>
<td>320</td>
<td>300</td>
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<tr>
<td>280</td>
<td>260</td>
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<tr>
<td>240</td>
<td>220</td>
</tr>
<tr>
<td>200</td>
<td>180</td>
</tr>
<tr>
<td>160</td>
<td>140</td>
</tr>
</tbody>
</table>

The magnitude of stress acquired from hydro-fracturing test in the three crossed boreholes is about 5.62 MPa, and its dip direction and dip angle are 253.5° and 58.9°, respectively. Test data from the vertical borehole indicate that the direction of the maximum principal stress is between N76°E and N80ºE, parallel to the stream direction of Qingjiang River in general.

The magnitudes of the maximum principal stress obtained by the over-coring stress relief method through boreholes in strata P1q, P1q and C2h are 4.99, 3.90 and 6.44 MPa, respectively. Their direction angles vary from 197° to 221° and their dip angles are less than 5°.

The magnitude distribution of stress acquired from the three stress relief boreholes mentioned above indicates the nonuniform characteristics of stress distribution in alternately distributed soft and hard rock layers. The in-situ stress in soft rock is relatively low, but that in hard rock the stress is relatively high. For the stratum P1q, the rock mass is relatively hard, homogeneous and considerably thick, and the distribution of the in-situ stress is relatively uniform. For the stratum P1q, the rock mass is weaker than both the upper stratum P1q and the lower stratum P1q, and many shear zones are developed in this soft stratum. So the stress magnitude is relatively low. The limestone stratum C2h is harder than both the upper stratum P1ma and the lower argillaceous sandstone stratum C2h. Its stress is relatively high. Figure 2 shows the curves of stress coefficients Kx = σx/σz and Ky = σy/σz versus overburden depth obtained by regression calculation where σx, σy are the normal stress components, with the right-angle coordinates x and y parallel and perpendicular to the axis of underground powerhouse, respectively.

![Fig.1 Cross-section of underground powerhouse in Shuibuya hydropower station.](image-url)

![Fig.2 Curves of stress coefficients Kx and Ky versus overburden depth.](image-url)
deformation and strength characteristics were studied through rock mechanical tests in laboratory and field. By using the inclined adit PD42, the in-situ rock mechanical tests for different strata of surrounding rocks were conducted in convenience. The tests included rock mass deformation test, shear strength test of rock mass and shear zones, triaxial compression test with specimen’s dimensions of 50 cm × 50 cm × 100 cm or 30 cm × 30 cm × 60 cm (length × width × height). In the inclined adit, ten testing adits located respectively in different strata of surrounding rocks of the underground powerhouse were excavated. The tests for these adits could objectively reflect the mechanical properties of different rock strata exposed to the underground powerhouse region. In-situ tests, conducted in the inclined adit PD42, included 34 plate bearing tests with the plate’s dimension of 50 cm in diameter, and 4 groups of shear strength tests with 25 specimens in total for rock mass with shear area of 2500 cm², and 4 groups of shear strength tests with 24 specimens in total for soft interlayers, and 2 groups of triaxial compressive tests with 11 specimens in total for triaxial strength properties of rock masses.

In addition, in order to realize the rheological properties of the surrounding rocks, especially for soft rocks, unconfined compressive rheological tests were conducted in laboratory for the strata 1qP, 1qP, 1qP, 3qP, C2h and D3x and shear zone No.031, 25 specimens in total were conducted in laboratory. Rock samples taken from very soft rock or shear zones were processed into non-standard specimens. The non-standard specimen had a rectangle block shape with the pressure bearing area normally from 22 to 176 cm². Rock samples taken from hard rock strata were processed into standard core specimens with dimensions of φ60 mm × 100 mm.

The surrounding rocks could be classified into the following four types according to the hardness and intact index:

1. Hard rock with good homogeneity. It includes medium-thick, thick bedded and micro-crystalline limestone with bioclastic limestone inclusions in the stratum 1qP, cherty limestone in the stratum 1qP, quartz sandstone with calcareous shale intercalated layer in C2h. Its deformation modulus, friction coefficient and cohesion are 9–14 GPa, 1.2–2.0, and 0.8–1.0 MPa, respectively.

2. Less hard rock with medium-thick and thick bedded limestone. It includes medium-thick and thick carbon-argillaceous limestone rich in biogenic debris in the stratum 1qP, and medium-thick limestone with thin carbon-argillaceous limestone intercalated layer in the stratum 1qP. Its deformation modulus, friction coefficient and cohesion are 9–14 GPa, 1.2–2.0, and 0.8–1.0 MPa, respectively.

3. Composite rock mass with soft and hard thin layers is deposited alternately. It includes thin carbon-argillaceous limestone rich in biogenic debris in stratum 1qP, quartz sandstone with calcareous shale intercalated layer in C2h. In this case, the deformation modulus represents the comprehensive modulus of different rock strata, and it is influenced mainly by the thin carbon-argillaceous limestone rich in biogenic debris at the lower part. Its deformation modulus is 1–2 GPa. Its friction coefficient and cohesion by in-situ test are 1.19 and 0.98 MPa, respectively.

4. Shear zones. It includes shear zones No.031, No.001 and F205 developed in strata 1qP, 1maP and C2h, respectively. As the results of interlayer shear actions, the rock mass behaves as cataclastic structure characterized with scaly or slab features. Its deformation modulus is 0.01–0.55 GPa.

Based on the geological investigation and rock mechanical tests in different strata, the suggested parameters of surrounding rocks are listed in Table 1. According to the rheological laboratory test of specimens, the rheological properties in terms of compressive strength are evaluated and listed in Table 2, where the ratios of creep strength to instantaneous strength of different rock layers are also listed. The in-situ deformation testing curves in typical strata are shown in Fig.3.

5 Engineering rock mass classification of surrounding rocks

In order to obtain a comprehensive evaluation of quality of the surrounding rocks quantitatively and qualitatively, the rock mass classification was conducted using three different methods, i.e. BQ method, Q system and RMR system. All of the works associated with field investigations were carried out in the inclined adit PD42.

5.1 BQ method based on Chinese national standard (GB50218-94) [16]

According to Chinese national standard (GB50218-94) for engineering rock mass classification, determination of rock mass basic quality is the first step. The rock mass basic quality is determined by two parameters, i.e. rock solidness and rock mass integrity.
### Table 1 Suggested parameters of surrounding rocks.

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Compressive strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Deformation modulus (GPa)</th>
<th>Poisson’s ratio</th>
<th>Shear strength</th>
<th>Stratum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bioclastic limestone</td>
<td>60–65</td>
<td>1.0–1.5</td>
<td>15.0–20.0</td>
<td>0.25</td>
<td>1.1–1.2</td>
<td>p&lt;sub&gt;1qP&lt;/sub&gt;</td>
</tr>
<tr>
<td>Carbon-argillaceous limestone</td>
<td>25–30</td>
<td>0.5–0.8</td>
<td>3.0–5.0</td>
<td>0.30</td>
<td>0.8–0.9</td>
<td>p&lt;sub&gt;1qP&lt;/sub&gt;</td>
</tr>
<tr>
<td>Microcrystalline limestone</td>
<td>70–80</td>
<td>1.0–1.5</td>
<td>15–20.0</td>
<td>0.25</td>
<td>1.2–1.3</td>
<td>p&lt;sub&gt;1qP&lt;/sub&gt;</td>
</tr>
<tr>
<td>Coal bed</td>
<td>15–20</td>
<td>0.2–0.5</td>
<td>0.4–0.8</td>
<td>0.35</td>
<td>0.5–0.55</td>
<td>p&lt;sub&gt;1q&lt;/sub&gt;</td>
</tr>
<tr>
<td>Quartz sandstone with calcareous</td>
<td>40–45</td>
<td>1.0</td>
<td>15.0–20.0</td>
<td>0.25</td>
<td>1.2–1.3</td>
<td>C&lt;sub&gt;2h&lt;/sub&gt;</td>
</tr>
<tr>
<td>limestone with shale intercalated</td>
<td>40–45</td>
<td>0.2–0.5</td>
<td>3–4</td>
<td>0.30</td>
<td>0.8–0.9</td>
<td>D&lt;sub&gt;3x&lt;/sub&gt;</td>
</tr>
<tr>
<td>Coal zones</td>
<td>0.01–0.55</td>
<td>0.40–0.45</td>
<td>0.25–0.30</td>
<td>0.02–0.03</td>
<td></td>
<td>No.001, No.031, No.041</td>
</tr>
<tr>
<td>Faults</td>
<td>1.5–2.0</td>
<td>0.30</td>
<td>0.35–0.45</td>
<td>0.01–0.05</td>
<td></td>
<td>F&lt;sub&gt;3&lt;/sub&gt;, F&lt;sub&gt;4&lt;/sub&gt;, F&lt;sub&gt;5&lt;/sub&gt;</td>
</tr>
</tbody>
</table>

### Table 2 Ratios of creep strength to instantaneous strength for different strata.

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Ratio (%)</th>
<th>Mean value (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>p&lt;sub&gt;1qP&lt;/sub&gt;</td>
<td>40.38–76.58</td>
<td>62.57</td>
</tr>
<tr>
<td>p&lt;sup&gt;1q&lt;/sup&gt;</td>
<td>40.83–59.08</td>
<td>52.47</td>
</tr>
<tr>
<td>p&lt;sub&gt;1qP&lt;/sub&gt;</td>
<td>55.16–69.45</td>
<td>59.98</td>
</tr>
<tr>
<td>C&lt;sub&gt;2h&lt;/sub&gt;</td>
<td>33.56–53.24</td>
<td>43.53</td>
</tr>
<tr>
<td>D&lt;sub&gt;3x&lt;/sub&gt;</td>
<td>30.43–69.94</td>
<td>47.51</td>
</tr>
<tr>
<td>No.031</td>
<td>40.53–59.94</td>
<td>46.81</td>
</tr>
</tbody>
</table>

### Fig.3 In-situ deformation testing curves in strata p<sub>1qP</sub> and p<sub>1qP</sub>.

Rock solidness and rock mass integrity should be determined by both parallel methods, namely the qualitative classification and the quantitative indices. After the rock mass basic quality is evaluated, the rock mass class is determined by combining qualitative characteristics of rock mass and basic quality index (BQ). In order to achieve a detailed classification of engineering rock mass, the necessary correction parameters should be considered on the basis of classification of rock mass. They include the features of different rock engineering types, the state of groundwater, the initial in-situ stress, and the combination of relative orientation of the structural axis with the occurrence of the main weak discontinuity sets. Based on the corrective parameters, the corrected rock mass basic quality index [BQ] is calculated according to the supplied formula.

The rock mass index BQ is calculated with two parameters, i.e. saturated uniaxial compression strength R<sub>c</sub> (MPa) and intactness index K<sub>v</sub>:

\[
BQ = 90 + 3R_c + 250K_v
\]  

Then the values of the corrected index [BQ] are calculated for all the strata exposed in inclined adit PD42, as shown in Fig.4.

### 5.2 Evaluation based on Q system [17]

The classification based on the Q system was carried out along the inclined adit PD42. Figure 5 shows the...
values of $Q$ along the adit passing through different rock layers. The values of $Q$ range from 0.4 to 23. The values of $Q$ for most strata are greater than 5 while those for soft layers are less than 5.

5.3 Evaluation based on RMR system [17]

The classification by RMR system was also carried out in the same inclined adit. Figure 6 shows the results of RMR classification. Values of RMR of rock masses normally range from 40 to 60, indicating that the rock could be described as a fair rock. For soft rock masses, values of RMR are between 20 and 40, which indicate poor rock. Good rock mass with a value of RMR greater than 60 occupies a low proportion.

![Fig.6 Values of RMR along adit PD42 passing through different rock layers.](image)

5.4 Comprehensive analysis of rock mass evaluation

The detailed results of rock mass classification for individual stratum of surrounding rocks by different methods are listed in Table 3. In order to comprehensively compare the classification results obtained by above three methods, the rock mass classification by $Q$ system is categorized into five grades. The values of $Q$ either < 1, or 1–4, or 4–10, or 10–40, or > 40 are corresponding to the grades V, IV, III, II, I, respectively. It has been shown that there is a good agreement for evaluating the quality of rock masses by using different methods. The rock mass classification reflects comprehensively the complicated characteristics of surrounding rocks of the underground powerhouse.

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Value</th>
<th>Rock class</th>
<th>Value</th>
<th>Rock class</th>
<th>Value</th>
<th>Rock class</th>
</tr>
</thead>
<tbody>
<tr>
<td>$p_{1d}$</td>
<td>399–587</td>
<td>III–I</td>
<td>8–22</td>
<td>III–II</td>
<td>45–62</td>
<td>III</td>
</tr>
<tr>
<td>$p_{1a}$</td>
<td>287–361</td>
<td>IV–III</td>
<td>4–8</td>
<td>III</td>
<td>37–57</td>
<td>IV–III</td>
</tr>
<tr>
<td>$p_{1s}$</td>
<td>393–503</td>
<td>III–II</td>
<td>9–13</td>
<td>III–II</td>
<td>43–62</td>
<td>III–II</td>
</tr>
<tr>
<td>$P_{1ma}$</td>
<td>127–150</td>
<td>V</td>
<td>1–2</td>
<td>IV</td>
<td>30–60</td>
<td>IV–III</td>
</tr>
<tr>
<td>$C_{2b}$</td>
<td>163–549</td>
<td>IV–II</td>
<td>0.43–14</td>
<td>V–II</td>
<td>34–67</td>
<td>IV–II</td>
</tr>
<tr>
<td>$D_{3b}$</td>
<td>163–170</td>
<td>V</td>
<td>1–2</td>
<td>IV</td>
<td>27–31</td>
<td>IV</td>
</tr>
</tbody>
</table>

6 Numerical modeling on excavation stability and its support effects

During the designing stage, numerical simulations in 2D and 3D conditions with both FEM and FLAC were carried out for underground powerhouse excavation and its reinforcements. The deformation of surrounding rocks, stress state and the distribution of plastic zone with/without reinforcement were obtained through numerical modeling. On the basis of modeling results, the reinforcement effects of different support schemes were evaluated. At the same time, support design was optimized.

The support schemes were the combination of different measures as follows: (1) Reinforced concrete replacement of the soft rock in stratum $p_{1d}$ with cross-section of $7 \, \text{m} \times 7 \, \text{m}$ in the upstream and downstream sidewalls. (2) When the excavations of the underground powerhouse reached the installation elevation of turbine machine, the elevation of 179.0 m, 2 m thick concrete placement was adopted for the remained rock surface between adjacent turbines. (3) Shotcrete with a thickness of 20 cm for surrounding rocks support. (4) Systematic rock bolts are set with alternate dimensions of $\varnothing 25/32 \, \text{mm}$, and $L = 6/8 \, \text{m}$. (5) Seven rows of 2 000 kN grade anchor cables ($L = 25 \, \text{m}$) were installed in the roof of the main powerhouse, two rows of 2 000 kN grade anchor cables ($L = 20, 25 \, \text{m}$) in the upstream sidewall, and three rows of 2 000 kN grade anchor cables ($L = 20, 25 \, \text{m}$) in the downstream sidewall. (6) Seven rows of 1 500 kN grade anchor cables ($L = 25 \, \text{m}$) were installed in the roof of the main powerhouse, and six rows of 1 500 kN anchor cables ($L = 20, 25 \, \text{m}$) in the upstream and downstream sidewalls. The reinforcement of surrounding rocks with schemes (5) and (6) were for comparative study.

The following four calculation cases considering the combination of excavation and supports were conducted in numerical simulations:

(1) Case 1: step-by-step excavation without any reinforcement;
(2) Case 2: step-by-step excavation with supports formed by schemes (1) and (2);
(3) Case 3: step-by-step excavation with supports formed by schemes (1)–(5);
(4) Case 4: step-by-step excavation with supports formed by schemes (1)–(4) and (6).

The results of numerical simulations showed that replacements of soft rock and systematic rock bolting would enable the overall deformation of the surrounding rocks to decrease effectively. Comparing the cases 1 and 2, the maximum horizontal displacement of the upstream sidewall decreased from 40.07 mm (case 1) to 31.12 mm (case 2), and the maximum horizontal displacement of the downstream...
sidewall decreased from 45.48 mm (case 1) to 35.07 mm (case 2). Meanwhile, the maximum rebounding displacement of the excavation of bottom surface decreased from 110.9 mm (case 1) to 50.18 mm (case 2). The displacements of the other parts of the surrounding rocks were decreased by 3.0–10.0 mm. With shotcrete and rock bolts installed, the displacements decreased more. Comparing the cases 3 and 2, the displacements of the roof was and the sidewalls of the main powerhouse decreased obviously. The maximum vertical displacement of the roof decreased by about 10.09 mm, and the displacements of the upstream and downstream sidewalls of the turbine pits were decreased by about 5.13 and 12.88 mm, respectively. The displacements of sidewalls, intake tunnels and the intersections of the main powerhouse with the busbar galleries and the tailrace tunnels all decreased with different scales. The average deformations decreased by 3.0–6.0 mm. The support in the case 4 behaved the best effect. In this case, the maximum reduction of displacement of the surrounding rocks due to excavation was 20.54 mm.

With the replacement of the soft rock and installation of systematic bolts, the surrounding rocks had a better stress state after excavation than that without any support. Comparing the cases 3 and 1, the range of tensile zone in the side walls was decreased greatly. The plastic zones developed at the intersection of the headrace tunnels and the upstream sidewall of the main powerhouse, the intersections of main powerhouse with the busbar galleries and the tailrace tunnels decreased largely, too. Comparing the cases 4 and 3, the plastic zone further decreased. This indicated that the installation of anchor cables (support scheme (6)) played an important role in improving the stress condition of the surrounding rocks and the overall stability of the underground caverns.

In general, with the current support design scheme (case 4), the maximum displacement of the key position of the surrounding rocks could be reduced by 15%–30%. The displacement of the surrounding rocks ranged typically from 40 to 60 mm, except for the one where soft rock layers were exposed. With the support schemes (1) and (2), if shotcrete and rock bolt were installed immediately after excavation, the displacement of the surrounding rocks would range from 20 to 35 mm, with the maximum value less than 60 mm.

The 3D numerical model, the maximum deformations at different places corresponding to related calculation case and the plastic distribution around the surrounding rocks in calculation of case 4 are shown in Figs.7–9, respectively.

7 Actual rock support and deformation behaviors

7.1 Actual rock support

Based on the detailed survey of the distribution of the soft rock and the comprehensive researches on rock mechanical problems, the actual excavation scheme and rock support of the underground powerhouse were proposed as follows:

(1) Replacement of the soft rock below elevation of the crane beam. The rock bolted crane beam is a load-bearing structure and has a strict limit for the deformation magnitude of the surrounding rocks. It is suggested to replace the stratum $p_{1q}^3$ with concrete C25, and the replacement area is $7 \text{ m} \times 7 \text{ m}$ for both the upstream and downstream sidewalls. Replacement of the soft rock stratum $p_{1q}^3$ and its contacting
consolidation grouting should be completed prior to the main powerhouse excavation.

(2) Supporting with shotcrete and rock bolts. For roof and sidewalls of the main powerhouse and the tailrace tube excavation surface, it is suggested to systematically install rock bolts with length \( L = 6/8 \) m, combined with steel fiber shotcrete. The thickness of the shotcrete is 15–20 cm.

(3) Grouting and protection of the soft rock exposed to the lower part of the main powerhouse. Except for stratum \( p_{1q}^4 \), the soft rock is mainly exposed below elevation of the tailrace tube inlet. Numerical results show that if the excavation is preformed from roof to bottom of the main powerhouse with normal step-by-step method, the deformations of the side walls of the turbine pit will be considerably large and the maximum displacement will reach 16 cm. This will cause a negative effect on the stress, deformation and stability of the upper part of the surrounding rocks. Therefore, when the full-face excavation is preceded to the elevation of 179 m, the anchor piles are installed in rock masses around all the turbine pits. Then reinforced concrete C25 with a thickness of 2 m is backfilled on the rock bench between adjacent tailrace tubes to protect the excavation surface. Thus, rock benches with a width of 16 m and a height of 14 m will be formed between adjacent tailrace tubes, which contact directly the upstream and downstream sidewalls, and a strong support between the two walls will be supplied. After the reinforced concrete placement is completed, the grouting of the rock masses around tailrace tubes can be performed, and a strictly controlled excavation along tailrace tubes is preceded.

(4) Reinforcement of shear zones and faults by anchor cables. The roof of the main powerhouse is located at stratum \( p_{1q}^4 \) where several shear zones and faults are fully developed. In order to prevent the negative effect of shear zones on the stability of the roof, the 1 500 kN grade anchor cables, 23 rows in longitude and 7 rows in vertical have been installed. The length of the cable is 25 m, and the spacings are 4.2 m × 4.5 m.

For the upstream sidewall, 6 rows of 1 500 kN grade anchor cables have been installed with a length of 20–25 m and the spacing in longitude direction is 4.5 m. For the downstream sidewall, 5 rows of 1 500 kN grade anchor cables have been installed with a length of 25 m and the spacing in longitude direction is 4.5 m. The purpose of installing anchor cables on the sidewalls is to prevent the shear deformation caused by shear zones exposed to side walls.

(5) Locking support of intersections prior to excavation. For the intersections of the main powerhouse with the intake tunnels, busbar galleries and tailrace tunnels, the excavation and locking support should be completed prior to the excavation of main powerhouse when proceeding to corresponding elevations. This will reduce the deformations of the sidewalls of the main powerhouse and the stress states of the intersections can be improved.

Figure 10 shows the practical rock support scheme. In Fig.10, the characters in the small circle symbol represent different types of rock supports. “MS” represents the anchor cables with capacity force of 1 500 kN and cable length of 25 m. “MS1” represents the anchor cables with capacity force of 1 500 kN and cable length of 20 m. “A” represents the rock bolts with spacing of 1.5 m and length of 8 m. “A1” represents the rock bolts with spacing of 1.5 m and length of 9.3 m. “B” represents the rock bolts with spacing of 1.5 m and length of 6 m. “C” represents the rock bolts with spacing of 1.5 m and length of 10 m. “F” represents the rock bolts with spacing of 1.5 m and length of 7 m.

![Fig.10 Practical rock support scheme (unit: m).](image-url)

7.2 Deformation behaviors

The construction of the underground powerhouse started in 2002. Till mid 2005, all the excavation and support were completed. During the excavation of the underground powerhouse, different kinds of safety monitoring equipments were used to monitor the deformation behaviors of the surrounding rocks. They included the convergence meters, multi-point extensometers, cracking measurement especially focused on the crane beams, and anchor force measurement of anchor cables and rock bolts.

It was shown that the reinforced surrounding rocks had a good overall stability. The maximum displacements observed by multi-point extensometers...
at upstream and downstream sidewalls and in the roof of the main powerhouse were 6.3, 9.2 and 4.9 mm, respectively. Figure 11 shows the monitoring results by multi-point extensometers installed at downstream sidewall at the section of unit No.2, where the curves of L1, L2 and L3 represent, respectively, the anchorage positions with 1, 6 and 11 m from excavation surface to the downstream wall in one borehole.

Fig.11 Time-history curves of the maximum displacement observed by multi-point extensometers.

8 Conclusions

The key rock mechanical problems of the underground powerhouse in Shuibuya hydropower station have been presented and discussed in detail. In order to resolve the related rock mechanical problems encountered prior to and during its construction, a detailed study in terms of rock mechanics, based on carefully issued schemes, was carried out. It included the initial in-situ stress distribution features, rock mechanical properties, engineering rock mass classifications by different methods, numerical modeling for stability and support analysis, proper measures for rock excavation and support. The results show that rock excavations of the underground powerhouse under the given geological conditions can be handled with technically. The measures suggested by the designers are proved to be reasonable and effective.

The engineering practice shows that the surrounding rocks have a good overall stability. The deformation behaviors observed by safety equipments are under the designing limits. The research conclusions on the related rock mechanical problems, prior to underground powerhouse excavations, are reliable.

References


