The Jinping I hydropower station is a huge water conservancy project consisting of the highest concrete arch dam to date in the world and a highly complex and large underground powerhouse cavern. It is located on the right bank with extremely high in-situ stress and a few discontinuities observed in surrounding rock masses. The problems of rock mass deformation and failure result in considerable challenges related to project design and construction and have raised a wide range of concerns in the fields of rock mechanics and engineering. During the excavation of underground caverns, high in-situ stress and relatively low rock mass strength in combination with large excavation dimensions lead to large deformation of the surrounding rock mass and support. Existing experiences in excavation and support cannot deal with the large deformation of rock mass effectively, and further studies are needed. In this paper, the geological conditions, layout of caverns, and design of excavation and support are first introduced, and then detailed analyses of deformation and failure characteristics of rocks are presented. Based on this, the mechanisms of deformation and failure are discussed, and the support adjustments for controlling rock large deformation and subsequent excavation procedures are proposed. Finally, the effectiveness of support and excavation adjustments to maintain the stability of the rock mass is verified. The measures for controlling the large deformation of surrounding rocks enrich the practical experiences related to the design and construction of large underground openings, and the construction of caverns in the Jinping I hydropower station provides a good case study of large-scale excavation in highly stressed ground with complex geological structures, as well as a reference case for research on rock mechanics.

1. Introduction

The Jinping I hydropower station is located in Liangshan Yi Autonomous Prefecture, Sichuan Province, Southwest China (Fig. 1). As a key project located in the downstream region of the Yalong River, it has an installed capacity of 3600 MW, with a 305 m-high hyperbolic concrete arch dam and a normal pool level of reservoir of 1880 m (PowerChina Chengdu Engineering Corporation Limited, 2003). In 2014, all generators at the Jinping I hydropower station were put into operation. Due to the complicated geological conditions, i.e. high in-situ stress and low rock strength, large deformation of rocks and severe failures (such as onion skinning, spalling and buckling around caverns) pose a major threat to the safety of caverns during construction. To ensure the safety of the caverns, excavation is sometimes interrupted to implement timely support, and special investigations have been conducted to determine how the deformation and failure occur (PowerChina Chengdu Engineering Corporation Limited, 2009). Thus the deformation and failure of rocks during the excavation of the Jinping I hydropower underground project are the most challenging issues.

Peeling and buckling of the rock mass around caverns have been rarely reported in hydropower projects due to the fact that the caverns are located in massive to moderately jointed hard rock masses that experience low to intermediate in-situ stress. However, such failures have been reported for the mine-by experiment tunnel at a depth of 420 m in AECL’s Underground Research Laboratory (Martini et al., 1997; Martino and Chandler, 2004; Read, 2004), at a large chamber at a depth of 3400 m in the East Rand Proprietary...
2. Engineering geology, layout of caverns, and excavation and support designs

2.1. Engineering geology overview

In the project region, the Yalong River flows in the direction of N25°E, with a water level of 1635 m in the dry season. The top elevation of the mountain reaches 3200–3600 m, and the relative height difference of the topography is 1500–2000 m. High and steep slopes are present in the precipitous gorges and sharply incised V-shaped valleys with a declination of 45°–80°.

The outcropping strata in the project region are the middle and upper groups of the Triassic Zagunao Formation (T2–3Z), which can be classified into three rock groups according to the lithology. The first group is the green schist (T2–3Z1), which is deeply buried in the riverbed and does not outcrop on the ground. The second is the...
marble ($T_2/C_0$) consisting of eight layers, which lies along both banks of the river, with a thickness of approximately 600 m. Interlayer dislocation belts develop in this rock group. The third group is the metasandstone and slate ($T_3/C_0$) consisting of six layers, with a thickness of approximately 400 m, which is formed in the core strata of the Santan Syncline and can be found in the upper left abutment. The strikes of these strata are basically consistent with the flow direction of the river, dipping towards the left bank, with inclinations of about 40°. The strikes of these strata result in the consequent slopes on the right bank and the inverse slopes on the left bank (Fig. 1). Weak discontinuities developed in the dam site mainly consist of faults (i.e. f5, f2, f18, f13, f14) and lamprophyre dyke X (Fig. 2).

The underground caverns are located in the marble strata (i.e. the formations $T_2/C_0$, $T_2/C_0$ and $T_2/C_0$), in which the rock mass is classified as grade III with the deformation modulus ranging from 10 GPa to 20 GPa. The mechanical properties of the rock mass are listed in Table 1. The faults (f13, f14 and f18) and lamprophyre dyke X outcrop in the caverns and have a consistent strike of N60°E and a dip of 70°E (Fig. 3). The faulted zone is composed of breccias, mylonite (main composition) and mud with a thickness of several centimeters. The thickness of the faulted zone ranges from...
Fig. 8. Failure phenomena of the rock mass surrounding the caverns.
1 m to 3 m. The joints outcropping in the caverns can be classified into 4 groups: (1) those developed in the marble stratum $\text{T}_2^3$–$\text{T}_2^7$ with the same strike and dip direction as those of the rock strata, (2) those developed widely in the powerhouse cavern with strikes of N50°–70°E and dips of 60°–80°, (3) those outcropping mainly in the assembly room with strikes of N25°–40°W and dips of 80°–90°, and (4) those outcropping sparsely with strikes approximately parallel to the powerhouse cavern axis.

At the Jinping I dam site, in-situ stress measurement using the stress relief technique was conducted at 19 locations. The results showed that the directions of the maximum and minimum principal stresses range from 120° to 150° and from 20° to 60°, respectively. However, the direction of the intermediate principal stresses was dispersed and could be approximately divided into two groups, specifically, the directions ranging from 110° to 120° and from 170° to 190°, respectively. The magnitude of the principal stress irregularly varied with the horizontal distance from the slope surface on the right bank of the Yalong River (Fig. 4). Obviously, these values are significantly influenced by the topography of the Yalong River valley and complicated geological structures. In the area of the powerhouse cavern, the magnitude of the maximum principal stress ranged from 20 MPa to 35 MPa. It is noted that the area of the powerhouse cavern, the magnitude of the maximum.

2.2. Layout of underground caverns

The water diversion and power generation system were excavated inside the mountain on the right bank of the Yalong River (Fig. 3). The underground caverns were mainly composed of the powerhouse, transformer chamber and surge tank, which are laid out in parallel with each other. The principle was followed that the strike of the main cavern should be approximately parallel to the dip direction of the maximum in-situ stress and nearly perpendicular to the strike of the major joint sets at the same time. In this regard, the strike of the powerhouse cavern was determined as N65°W. The angles between the axis of the powerhouse and the dip direction of the maximum in-situ principal stress, the strikes of the major joint set 1, and the faults (f13, f14 and f18) and dyke X are 15°, 65° and 50°, respectively (Fig. 5).

The dimensions of the main powerhouse cavern and transformer chamber are 204.52 m $\times$ 28.90 m $\times$ 68.80 m (length $\times$ width $\times$ height) and 197.1 m $\times$ 19.3 m $\times$ 32.7 m, respectively. The two separate surge tanks are in the shape of an upright cylinder with a dome and have the dimensions of $\phi 41 $ m $\times$ 80.5 m (diameter $\times$ height) and $\phi 37 $ m $\times$ 79.5 m, respectively. The rock pillars between the powerhouse and transformer chamber is 43.75 m thick and that between the transformer chamber and surge tank is 46 m thick (Fig. 6).

2.3. Excavation methods

For the main caverns, a middle pilot heading was first excavated and then an enlargement excavation was carried out to form a crown. After that, bench excavation was performed until the floor of the caverns. The bench is 4–10 m in depth per stage, with 11, 5 and 14 stages for the powerhouse cavern, transformer chamber and surge tank, respectively (Fig. 6). The details of the main cavern excavation are described as follows.

For the powerhouse cavern, the bench excavation between elevations of 1666.1 m and 1654.5 m, where the rock anchor beam was located, was designed with a depth of 3.5 m per stage. The central zone, approximately 21 m wide, was first excavated and then the zones about 2 m wide on either side of the central zone, namely the protective layer (as illustrated in Fig. 6), were excavated using a pre-split blast method to form the neat sidewall without overbreak or underbreak. The bench excavation of the rock mass between elevations of 1654.5 m and 1635.3 m was designed to be 4.6 m in depth per stage.

For the transformer chamber, the bench excavation was 5 m deep per stage with pre-split line striking along the sidewall. For the surge tanks, the excavation of dome followed a similar process as that of the crown of the powerhouse cavern as described above. The middle part of the surge tank (between elevations of 1668 m and 1625.5 m) was excavated in a way that a central pilot shaft was completed first and then the bench excavation was carried out.

The excavation sequence of the main caverns was arranged in the order that the transformer chamber was excavated after the crown of the powerhouse cavern was completed, and then the surge tank, after that the crown of the transformer chamber was excavated.

2.4. Support design

The support components in the main caverns consist of fiber-reinforced shotcrete (or mesh-reinforced shotcrete), grouted rebar and pre-stressed bolt cables to maintain the stability of the
surrounding rock mass. Basically, the exposed roof and walls were immediately reinforced with shotcrete, and then the rebar and bolt cables installation was implemented. The details of the support design are described as follows.

(1) Mesh-reinforced shotcrete, i.e. 20-cm thick shotcrete (with the compressive strength of 25 MPa, hereafter referred to as C25 shotcrete) in combination with wire mesh (8 mm in diameter and a spacing of 20 cm) was implemented at the crown of the powerhouse and surge tank caverns, whereas in the transformer chamber, the thickness of the shotcrete was reduced to 15 cm. On the sidewalls, fiber-reinforced shotcrete (15-cm or 10-cm thick C25 shotcrete) was used. At lower elevations of the powerhouse cavern, 10-cm thick C25 shotcrete was used.

(2) The staggered layout of grouted rebar in the sidewalls was 32 mm in diameter, and 6 m or 8 m in length, with a spacing of 1.5 m.

(3) The pre-stressed bolt cables in the sidewall of the powerhouse cavern were arranged at elevations of 1662.5 m, 1656.5 m, 1653.5 m, 1650.5 m, 1647.5 m, 1644.5 m, 1640.5 m and 1635.5 m, with a spacing of 3 m for elevations above 1644.5 m, and 4.5 m for the remainder. In the upstream sidewall of the powerhouse, the drill holes for settling bolt cables at the elevation of 1662.5 m stretched out to the sidewall of the drainage tunnel on the upstream side. On the downstream sidewall of the powerhouse, such drill holes at elevations of 1662.5 m, 1656.5 m and 1653.5 m stretched out to the upstream sidewall of the transformer chamber. These bolt cables are 33/46 m in length, with a load-bearing

Fig. 9. Failure phenomena of shotcrete and steel rib.
capacity of 2000 kN and anchor piers mounted on two ends. The others were 15/20 m in length with a load-bearing capacity of 1750 kN.

(4) On the downstream sidewall of the transformer chamber, three rows of pre-stressed bolt cables were arranged at elevations of 1644.5 m, 1650 m and 1655.5 m. The bolt cables at the elevation of 1664.5 m stretched out to the downstream sidewall of the drainage tunnel and had the same load-bearing capacity (2000 kN) and spacing (3 m) as those at the elevation of 1662.5 m on the sidewall of the powerhouse cavern, with anchor piers on both ends of the holes (24 m in length). The bolt cables at elevations of 1660 m and 1655.5 m were 20 m in length, with a load-bearing capacity of 1750 kN and a spacing of 4.5 m.

(5) In the surge tanks, four rows of pre-stressed bolt cables with a load-bearing capacity of 1500 kN were arranged at elevations of 1656 m, 1651 m, 1646 m and 1641 m. The length and spacing of these bolt cables were 15/20 m and 4.5 m, respectively.

(6) In the area near the outcrop of the faults, support intensity was enhanced. The weak rock mass in this area was reinforced by applying a dense pattern of rebar, increasing the length of bolt cables, applying shotcrete with higher compressive strength, using the steel arch and anchor rebar pile, carrying out consolidation grouting and replacing the weak rock mass with concrete when necessary.

3. Analyses of deformation and failure of surrounding rocks and supports

Displacement monitoring data showed that the deformations of the crown and sidewalls were small after the third bench excavation of the powerhouse cavern. Cracking of the shotcrete was observed at the lower part of the crown on the downstream side. After the seventh bench excavation of the powerhouse cavern, failure was observed in a large area of surrounding rock mass and support. The displacement of the shallow rock mass was considerably large, and small displacement was observed in the deep rock mass of the powerhouse cavern, transformer chamber and bus tunnels when the entire excavation and support process had been completed. Inevitably, large deformation of rock mass led some rebars and bolt cables to be overloaded.

3.1. Failure of surrounding rock mass, shotcrete and steel arch

The failure of rock mass and shotcrete occurred at the lower parts of the cavern crown on the downstream side and the sidewall on the upstream side. The observed failure patterns were spalling (or slabling) and buckling induced by splitting, detachment of the discontinuities and slipping of rock mass along the joint surfaces (PowerChina Chengdu Engineering Corporation Limited, 2009; Huang et al., 2011). The distributions of the failed rock mass and shotcrete in the crown and sidewalls of the powerhouse cavern are shown in Fig. 7.

For the slabbing and buckling induced by rock mass splitting, which is aligned parallel to the orientation of the cavern face, the depth of failed zone usually ranges from 20 cm to 30 cm and could reach 60 cm or 70 cm in extremity with a crack width of 2–6 cm (Fig. 8a–f). The cracks or discontinuities induced by rock mass relaxation (the term ‘relaxation’ is used to describe conditions where stress adjustment leads to a reduction in tangential stress near the excavation) were mainly found in the middle and lower parts of the upstream sidewall, downstream sidewall and end wall. The dip direction and dip angle of such cracks were N50°–70° W (approximately parallel to the cavern axis) and NE < 40°–50°, respectively.

The width of the cracks was usually 5–10 cm and could reach 20 cm in extremity (Fig. 8g–i). In the case of slipping along the discontinuities (Fig. 8j), the slipping distance was approximately 5 cm.

The support failure occurred in the same locations where the rock failure was observed. The types of support failures included shotcrete cracking, buckling of the steel rib (or I-type steel beam) (Fig. 9b, d and f), and anchor pier invagination. The jagged and intermittent shotcrete cracks propagating along the direction of the cavern axis mainly occurred in the lower parts of the cavern crown, with crack widths of 2–6 cm (Fig. 9a, c and e). It was noted that on the upstream sidewall of the transformer chamber, the tensile fracture of shotcrete was significant and there were a large number of inclined shotcrete cracks with dip angles between 50° and 70° (Fig. 10). Most of the inclined cracks were distributed in a zone 3–4 m high above the floor of the transformer chamber. Only a few inclined cracks stretched out and joined the shotcrete cracks at the crown of the bus tunnels.

3.2. Analysis of surrounding rock mass displacements

Nine instrumentation arrays were set up along the axis direction of the powerhouse cavern at chainages of 0–35.62, 0 + 0.00, 0 + 31.70, 0 + 63.40, 0 + 79.20, 0 + 95.10, 0 + 126.80, 0 + 158.50, and 0 + 196.27. The instrumentation consisted of a multipoint extensometer set in the roof and sidewalls of the caverns, and cable load cells on selected bolt cables.

Based on the data obtained over the period from the day of installation of the extensometers to the middle of October 2009 when the seventh bench excavation of the powerhouse cavern was completed, the displacement of the surrounding rock mass was analyzed. It should be noted that the displacement here means the relative displacement between two points. One point was located at the entrance of the hole where the extensometer was settled and the other at the bottom of the hole.

3.2.1. Displacement distribution in the powerhouse cavern

Among the 59 extensometers, 57.6% showed a displacement of less than 10 mm, 16.9% of 10–30 mm, 15.3% of 30–50 mm, and 10.2% of larger than 50 mm.

Fig. 11 indicates that the downstream sidewall displacements were basically larger than 30 mm, whereas the upstream sidewall ones were usually less than 30 mm. On the upper part of the
downstream sidewall, there were three separated zones where the displacements were larger than 50 mm, and in the middle part of the upstream sidewall (between elevations of 1650 m and 1660 m), there was also a zone where the displacements were larger than 40 mm.

In summary, the downstream sidewall displacements were larger than the upstream sidewall ones and the latter were larger than the crown displacements.

3.2.2. Displacement distribution in the transformer chamber

Among the 34 extensometers, 50% showed a displacement of less than 10 cm, 26.5% of 10–30 mm, 8.8% of 30–50 mm, and 14.7% of larger than 50 mm.

Fig. 12 indicates that the displacements in the upper part of the downstream and upstream sidewalls were 30–100 mm and 30–50 mm, respectively. There were three zones where the displacements were the largest, with the maximum displacements of 155.1 mm, 61.4 mm and 59.3 mm, respectively.

To investigate the displacement distribution of the surrounding rock mass, the displacements of the points (related to the bottom of the hole) at different depths along the direction of the hole axis were obtained, and are plotted in Fig. 13. It can be seen that the displacement of the point inside the rock mass on the downstream side was generally larger than that of the symmetric point on the upstream side. In addition, the displacement gradient (especially in the shallow region) on the downstream side was larger than that on the upstream side.

The above analysis shows that:

(1) The displacement distribution was asymmetric with respect to the cavern axis. On one hand, the displacements on the upstream side were generally smaller than those on the downstream side. On the other hand, the displacements in the lower part were larger than those in the upper part on the upstream side; whereas on the downstream side, the displacement distribution trend was oppositely.

(2) Severe failure of rock mass and support was reported at the same location where large displacements were observed.

3.3. Analysis of monitoring data from anchor cable load cells

For simplicity, the actual load, the pre-tension force and the bearing capacity of bolt cables are denoted by $f_a$, $f_p$ and $f_{lim}$, respectively, in the following sections.

Among the 81 cable load cells installed in the powerhouse cavern, the ones with $f_a < f_p$ (which means that the bolt cables were in a relaxation state) and the ones with $f_a > f_p$ (denoted by the symbols “<” in the case of $f_a < f_p < f_{lim}$ and “>” in the case of $f_a > f_{lim}$, respectively, in Fig. 11) accounted for 23.5% and 76.5%, respectively. The cable load cells showing an increasing load amplitude (related to the pre-tension force) of 0.2–0.4 accounted for 65.2% and 25.1%, respectively, on the upstream side. On the downstream side, however, nearly half of the cable cells showed that the relative increasing amplitude ranged from 0.2 to 0.4.

![Fig. 11. Distribution of rock mass and bolt cables displacements in the powerhouse cavern.](image1)

![Fig. 12. Distribution of rock mass and bolt cables displacements in the transformer chamber.](image2)
In the transformer chamber, the ratio of the load cells with $f_a < f_p$ was nearly the same as that in the powerhouse cavern. In addition, the load cell with $f_a > f_{lim}$ accounted for 21.4%.

### 3.4. Analysis of the excavation damaged zone

During the excavation of the caverns, the acoustic wave testing technique and digital borehole camera technique were employed to determine the depth of the damaged zone around the caverns. The results obtained from the acoustic wave tests are listed in Tables 2 and 3. The acoustic wave testing showed that for the upper part (above the elevation of 1665 m) of the powerhouse cavern, the boundary of the damaged zone on the downstream side was farther away from the cavern perimeter than that on the upstream side, with the maximum depth of the damaged zone of 16.2 m and 4 m on the downstream and upstream sides, respectively. On the upstream side, the depth of the damaged zone increased as the altitude descended, with a sharp ascent (from 4 m to 13.4 m) followed by a steady ascent (from 13.4 m to 16.2 m) at the elevation of 1641 m. The asymmetry of the damaged zone distribution around the powerhouse cavern was similar to that of the displacement.

![Graphs and charts showing displacement vs. depth for various sections of the caverns.](image)

**Fig. 13.** Variation of displacement with depth based on the data from multipoint extensometers in the powerhouse and transformer caverns.

<table>
<thead>
<tr>
<th>Elevation (m)</th>
<th>Depth of damaged zone on the upstream sidewall (m)</th>
<th>Depth of damaged zone on the crown and upstream sidewall (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Severe damage Slight damage</td>
<td>Severe damage Slight damage</td>
</tr>
<tr>
<td>1670</td>
<td>1.4–2.6 2–4</td>
<td>4–7.8 7.2–16.2</td>
</tr>
<tr>
<td>1665</td>
<td>2–9.8 7–13.4</td>
<td>4–9.6 5.2–16</td>
</tr>
<tr>
<td>1657</td>
<td>4.2–8.2 7.2–14.2</td>
<td>5.2–8 14.2–16.8</td>
</tr>
<tr>
<td>1649</td>
<td>2–9.2 7–16.2</td>
<td>6–9.6 12.6–19</td>
</tr>
<tr>
<td>1641</td>
<td>13.7–14.8</td>
<td>1.6–3.4 4–8</td>
</tr>
<tr>
<td>1634</td>
<td>6.8–13.8</td>
<td>4–6.4 4.6–15</td>
</tr>
<tr>
<td>1660</td>
<td>5–10.6 8–18</td>
<td>4.8–8.6 7.6–14</td>
</tr>
<tr>
<td>1652</td>
<td>3.6–8 7.2–14</td>
<td>2.4–6 6–11.8</td>
</tr>
</tbody>
</table>

**Table 2** Relaxation depths of rock mass surrounding the powerhouse cavern.

<table>
<thead>
<tr>
<th>Elevation (m)</th>
<th>Depth of damaged zone on the downstream sidewall (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Severe damage Slight damage</td>
</tr>
<tr>
<td>1668</td>
<td>4–6.8 13.8–17.2</td>
</tr>
<tr>
<td>1660</td>
<td>5–10.6 8–18</td>
</tr>
<tr>
<td>1652</td>
<td>3.6–8 7.2–14</td>
</tr>
</tbody>
</table>

**Table 3** Relaxation depths of rock mass surrounding the transformer chamber.
For the transformer chamber, there was no significant difference between the distribution of the damaged zone on the upstream and downstream sides.

3.5. Mechanisms of deformation and failure of rock mass around the caverns

In order to explore the mechanisms of deformation and failure of the surrounding rock mass, a stress ellipse was drawn and analyzed based on the in-situ stress data obtained before excavation by Lu et al. (2010, 2012). In this section, the in-situ stress data from 10 sites were used to produce stress ellipses. First, the principal stresses on the three planes with normal directions parallel or perpendicular to the cavern axis were obtained. For simplicity, the planes were denoted as "XZ", "YZ" and "XY" in the following section. Then, the stress ratios (σ_max/σ_min) and dips of σ_max were calculated and are listed in Table 4.

It can be seen that in each plane, there is a considerably consistent dip direction of σ_max at different sites. The stress ratios range in 0.26–0.69, 0.3–0.93, and 0.37–0.79 for the XY, XZ and YZ planes, respectively. The corresponding dips of σ_max range from 18.84° to 40.94°, 1.47° to 38.79°, and 21.38° to 57.15° for the three respective planes. The average stress ratio and dip of σ_max in each plane are depicted in the form of stress ellipse as illustrated by Fig. 14.

The in-situ stress state, especially the orientation of the principal stresses relative to the tunnel axis, is the most important indicator among the factors that have a significant influence on the failure characteristics of surrounding rock mass (Read, 2004; Diederichs, 2007). The failure characteristics around the circle tunnel were well revealed by Read (2004). According to the study conducted by Read (2004), the damage induced by excavation was significant in the area of compressive stress concentration. In

![Table 4](https://example.com/table4.png)

<table>
<thead>
<tr>
<th>Measuring points</th>
<th>XY</th>
<th>XZ</th>
<th>YZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>σ_max/σ_min Dip of σ_max (°)</td>
<td>σ_max/σ_min Dip of σ_max (°)</td>
<td>σ_max/σ_min Dip of σ_max (°)</td>
<td></td>
</tr>
<tr>
<td>0.42 20.04 0.3</td>
<td>21.58 0.65 57.15</td>
<td>0.52 24.72 0.68 1.47 0.79 39</td>
<td></td>
</tr>
<tr>
<td>0.36 –15.79 0.49 2.32 0.72 26.41</td>
<td>0.52 38.38 0.45 22.37 0.41 58.38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.48 40.94 0.76 7.44 0.67 57.09</td>
<td>0.45 18.84 0.48 2.4 0.37 48.22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.33 38.63 0.54 38.79 0.54 40.36</td>
<td>0.45 24.24 0.76 9.89 0.77 –8.19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.26 18.89 0.55 17.24 0.48 21.38</td>
<td>0.69 23.24 0.93 1.63 0.68 30.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.45 23.21 0.59 12.51 0.61 37</td>
<td>Average 0.45 23.21 0.59 12.51 0.61 37</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

![Fig. 15](https://example.com/fig15.png)

Distributions of the maximum principal stress around the powerhouse cavern after the seventh bench excavation.

![Table 5](https://example.com/table5.png)

<table>
<thead>
<tr>
<th>Rock mass grade</th>
<th>E (GPa)</th>
<th>Poisson’s ratio</th>
<th>Friction angle (°)</th>
<th>Cohesion (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>II</td>
<td>35</td>
<td>0.25</td>
<td>53.5</td>
<td>2</td>
</tr>
<tr>
<td>III1</td>
<td>19</td>
<td>0.25</td>
<td>46.9</td>
<td>1.5</td>
</tr>
<tr>
<td>III2</td>
<td>13</td>
<td>0.3</td>
<td>45.5</td>
<td>0.9</td>
</tr>
<tr>
<td>IV</td>
<td>3</td>
<td>0.35</td>
<td>35</td>
<td>0.6</td>
</tr>
<tr>
<td>V</td>
<td>1</td>
<td>0.35</td>
<td>16.7</td>
<td>0.02</td>
</tr>
</tbody>
</table>

![Fig. 16](https://example.com/fig16.png)

Distribution of rock mass displacement along a typical cross-section.
Fig. 14a, it is clear that the excavation results in compressive stress concentration, which occurs mainly in the lower part of the cavern crown, the upper part of the sidewall on the downstream side, and the lower part of the sidewall on the upstream side. With respect to the bus tunnels and headrace tunnels with tunnel axis perpendicular to that of powerhouse cavern, as shown in Fig. 14c, it is evident that the lower part of the tunnel crown on the right side and the foot of the sidewall on the left side will be the compressive stress concentration zones.

The above discussion on the compressive stress concentration around the caverns (or tunnels) is also verified by the stress distribution contours (Fig. 15) obtained by the numerical simulations in which a Mohr–Coulomb elastoplastic constitutive model was used. The mechanical parameters for the simulations are listed in Table 5.

The zones where moderate to severe relaxation (or damage) occurs are highlighted by large displacement of the surrounding rock mass. It is shown in Fig. 16 that after the seventh bench excavation, the displacement distribution is asymmetrical with respect to the cavern axis. For the powerhouse cavern, the wall displacements on the upstream side and downstream side are usually 50–80 mm and 70–90 mm, respectively. At the sites where faults or dykes outcrop, the wall displacement reaches a maximum value exceeding 100 mm. For the transformer chamber, the rock mass displacement is larger (70–100 mm) in the middle and upper parts of the downstream sidewall compared with that at the upstream side. The area where the zone with wall displacement on the downstream side exceeds 100 mm is significantly larger than that on the upstream side. The maximum displacement on the downstream sidewall is 183 mm.

4. Measures for controlling rock mass deformation and their evaluation

It is well known that intact rock (especially hard rock) undergoes a small deformation before failure. Therefore, rock mass deformations are mainly caused by rock fractures (specifically the opening of macroscopic cracks in rock) and the opening (or slipping) of weak discontinuities (e.g. joints). In the case of a low ratio of the rock strength to the maximum in-situ stress, the openings of macroscopic cracks in rocks account for a substantial part of rock mass deformation and lead to the so-called large deformation. It is noted that the large deformation of brittle hard rock mass in a hydropower project is considerably different from that of soft rock mass in coal mines. The deformation of soft rock mass can easily reach dozens of centimeters; however, the deformation of hard rock mass exceeding 100 mm is rarely reported in hydropower caverns. To guarantee the safety of workers during excavation and to ensure that the hydropower station functions correctly during the service period, the deformation of rock mass exceeding several hundreds of millimeters is usually not allowed to occur. Such a rigorous requirement poses a great challenge to the design and adjustment of support and excavation procedures.

After the seventh bench excavation of the powerhouse cavern, clear signs of large deformation were observed, which resulted in the requirement that certain measures should be implemented in a timely manner. The following measures in both support and excavation procedures aim to prevent the rock splitting from further developing, such that the deformation caused by rock fracturing can be effectively controlled.

4.1. Measures for controlling rock mass deformation

Based on the investigations of rock deformation and failure mechanism, some adjustments in both support and excavation procedures were proposed. Support adjustments were mainly made in the areas where compressive stress was concentrated (i.e. cavern crown on the downstream side) and relaxed (i.e. the cavern sidewall on the upstream side). In these areas, the rock mass was reinforced mainly by applying a dense pattern of rebar (especially the pre-stressed rebar) and a combination of concrete frame beams with bolt cables, and consolidation grouting was also used to reinforce the severely damaged rock mass (Li et al., 2009; Hou et al., 2012a, 2012b).

Detailed adjustments of support and excavation procedures are described as follows:

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Fig. 17. Illustration of modified support scheme (only bolt cables and rebar are shown) in the powerhouse and transformer caverns.
(1) In the downstream crown (the zone between elevations of 1674.5 m and 1665.8 m) of the powerhouse cavern, 9-m long rebar (or pre-stressed rebar) was added to further reduce the spacing of rebar and 5 (or 6) rows of pre-stressed bolt cables with a bearing capacity of 2000 (or 1500) kN were also applied (Fig. 17). The length of these bolt cables is 20–30 m, varying with the site-specific geological conditions. In order to achieve the goal of integrated system support, the combination of concrete frame beams with pre-stressed bolt cables was also implemented in this area (Fig. 17). In addition, consolidation grouting was also carried out to reinforce the broken rock mass. The grouting hole was 8 m deep inside surrounding rocks, with an average spacing of 3 m.

Similar to the powerhouse cavern, a total of 5 rows of pre-stressed bolt cables and a dense pattern of rebar were implemented at the lower part of the downstream crown and upper part of the downstream sidewall of the transformer chamber (Fig. 17).

(2) The middle parts of the upstream sidewall of the powerhouse cavern were reinforced by applying a denser pattern of rebar, and the other rebar was mainly pre-stressed rebar (9 m in length). Meanwhile, the bearing capacity of some bolt cables was increased from 2000 kN to 2500 kN. Consolidation grouting was also used in this area.

(3) The rock pillars between the neighboring bus tunnels were reinforced by the pre-stressed bolt cables. The holes for...
settling these bolt cables were drilled through the pillars to install anchor piers on both sides of the neighboring tunnels. (4) The excavation of the lower part (between elevations of 1635.3 m and 1625.8 m) of the powerhouse cavern was adjusted. For example, the eighth bench excavation was subdivided into two halves, with one half on the upstream side excavated ahead of another on the downstream side by some distance as the excavation advanced along the cavern axis. Such adjustment results in a phased releasing of in-situ stress and helps to reduce the rock mass damage in the presence of blasting excavation.

In fact, the above-mentioned support adjustments imply an important idea that in the case of highly stressed ground, it is better to take advantage of the role of proactive support (e.g. the pre-stressed rebar) and adopt the integrated system support in which the combination of support elements and utilization of the combined synergies are emphasized. In addition to enhancing the capability of resisting shear failure, pre-stressed rebar can also improve the stress state of the superficial rock mass around the cavern by reducing the difference between the major and minor principal stresses to some extent. Therefore, using the pre-stressed rebar to reinforce the rock mass not only enables the rock mass to support itself but also controls the process of bulking as the pre-stressed rebar.

4.2. Evaluation of the support and excavation procedure adjustments

The monitoring data obtained from the multipoint extensometers and cable load cells are shown in Figs. 19 and 20, respectively. It can be seen from these figures that during the period of follow-up bench excavation, the displacements of the surrounding rock mass converge gradually, and the loads of most of the cables fluctuate within a narrow range. This implies that the support and excavation procedure adjustments are reasonable and effective.

5. Conclusions

(1) During the excavation of underground caverns in Jinping I hydropower station, large deformation and severe damage,
which have rarely been reported previously, occurred in a wide area of the surrounding rock mass. The depth of the damaged zone was also large. Large deformation and severe failure made the Jinping I hydropower underground project the most challenging in the world.

(2) The high in-situ stress relative to the low strength of the rock mass (the ratio of strength to stress is 1.89—4.18) resulted in the occurrence of severe rock mass failure. The asymmetry of the surrounding rock mass failure with respect to the cavern axis was mainly determined by the orientation of the principal stresses relative to the cavern axis.

(3) The support and excavation procedure adjustments were effective in controlling the rock mass deformation and preventing the area and depth of the damaged zone from further expanding. Both the rock mass deformation and cable loads were basically stable.

(4) This case study implies an idea regarding the cavern excavated in highly stressed strata, where both the support and excavation procedures need to be designed carefully, that is to say, it is better to take advantage of the role of proactive support (e.g., the pre-stressed rebar) and adopt integrated system support, whereas the disturbance to the rock mass caused by the excavation should be reduced as possible.

The measures for controlling the large deformation in the Jinping I hydropower station not only enrich the experience related to the design and construction of large caverns in hydropower projects, but also provide a good reference case for the excavation and support of deep tunnels in traffic projects and underground openings in mines that suffer high in-situ stress.

Conflict of interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

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