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Practices on rockburst prevention and control in headrace tunnels of Jinping II hydropower station

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Abstract: Rockburst problems induced by high in-situ stresses were prominent during construction of the headrace tunnels of Jinping II hydropower station. The rockbursts occurred in various forms, and it is necessary to take pertinent measures for integrated prevention and control of rockbursts. In view of the rockburst characteristics during tunnel construction of Jinping II hydropower station, the engineering geological conditions were presented, and the features, mechanisms and forms of rockbursts observed during construction were analyzed in detail. A large number of scientific researches, experiments and applications were conducted. Multiple measures were adopted to prevent and control rockbursts, including the prediction and early warning measures, stress relief by blasting in advance, optimized blasting design and optimized tunnel support in the tunnel sections prone to strong rockbursts. The effectiveness of these prevention and control measures was evaluated. Experiences have been accumulated through a great number of helpful explorations and practices for rockburst prevention and control. A comprehensive rockburst prevention and control system has been gradually established.

Key words: long and deep tunnel; rockburst prevention and control; stress relief by blasting; microseismic monitoring

1 Introduction

Jinping II hydropower station is located on the Jinping bend of the Yalong River at the junction of Muli, Yanyuan and Mianning Counties of Liangshan Yi Autonomous Prefecture, Sichuan Province, China. It is a super-large underground hydropower station with four headrace tunnels, and is one of the main power stations for the West-to-East Electricity Transmission Project in China. It takes advantage of the natural elevation drop at the 150 km long Jinping bend of the Yalong River and water is diverted by a sluice dam to headrace tunnels for power generation. The maximum hydraulic head is 321 m and the rated head is 288 m. Eight Francis turbine generator sets were installed with each unit capacity of 600 MW and the total installed capacity of 4 800 MW. The long-term annual average power output is predicted to hit 24.23×10^9 kW·h, which can ensure the annual power output of 1 972 MW and the operative duration of 5 048 h. Jinping II hydropower station is presently the one with the highest hydraulic head and the maximum installed capacity along the Yalong River. In this hydropower station, eight generators are arranged in four headrace tunnels. The inlet is located near the Jingfeng Bridge at the west end of the Jinping bend and the station is located in Dashuigou at the east end. Four headrace tunnels are in parallel arrangement and cross over the Jinping Mountain. The average tunnel length is about 16.67 km from the inlet to the upstream surge chamber. The tunnel diameter is 12.4–13 m and the center-to-center distance of any two adjacent tunnels is 60 m. The azimuth of the tunnel axis is N58°W. The vertical layout of the tunnels is in gentle slope and the base slope gradient is 3.65%. The overburden depth is basically 1 500–2 000 m, with the maximum of approximately 2 525 m. The tunnels are characterized by great cover depth, long length and large diameter. Jinping II hydropower station can be classified as an ultra-deep and super-large underground hydropower station.

The four headrace tunnels with two auxiliary tunnels and one construction drainage tunnel form a large tunnel group, namely, from south to north the auxiliary tunnel A, the auxiliary tunnel B, the construction

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drainage tunnel, and the headrace tunnels Nos. 4, 3, 2 and 1, respectively. The two auxiliary tunnels were excavated by the drill-and-blast method and were completed in August 2008. The construction drainage tunnel was excavated by TBM, heading from east to west. In order to ensure drainage during construction, the working face of the construction drainage tunnel must keep ahead of that of the headrace tunnel. The four headrace tunnels were excavated from the east or west end toward the center. The east sections of the headrace tunnels Nos. 1 and 3 were constructed by TBM and the other two tunnels were excavated by the drill-and-blast method. In order to ensure that the headrace tunnel No. 1 can start to operate as schedule, it has to be completed first, followed by the headrace tunnels Nos. 2–4.

With the developments of various engineering projects in transportation, water resources, hydropower and energy engineering, many engineering practices have shown that the in-situ stresses increase greatly with depth (Jing et al., 2008). During the construction of the four headrace tunnels and the two auxiliary tunnels, the rockbursts with various magnitudes occurred. Different types of rockbursts were observed in the construction of drainage tunnel and the headrace tunnels currently under construction, which facilitates the understanding of rockburst phenomenon and presents challenges to the prediction and control of rockbursts. Rockburst is a complex dynamic hazardous phenomenon (Xu et al., 2008) and has become one of the key problems restricting engineering safety and progress. It is necessary to take comprehensive measures to control rockbursts according to the risk levels and occurrence conditions.

2 Geological conditions and mechanical properties of rocks

2.1 Lithology

The outcrop strata in the project site are pre-Devonian-Jurassic alternating marine and terrestrial formations of shallow marine-littoral facies. In this area, the Triassic series are widely distributed, which account for more than 90%. Among the Triassic series, the outcrop area of carbonate rock occupies 70%–80% in total. The Triassic strata are the major surrounding rocks of the auxiliary tunnels and headrace tunnels, mainly including the western lower Triassic series (T_1), Yantang Group (T_{2y}), Zagunao Group (T_{2z}), Baishan Group (T_{2b}), and the upper Triassic series (T_3) and Quaternary breccia.

2.2 Geological structures

It can be inferred from the distribution of geological structures that the stress field in the project area is approximately in NWW-SEE direction (Fig. 1). A series of approximately NS-compacted complex folds and compressional or compressional-shear faults with large dip angles were formed, accompanied by NWW tensile or tensile-shear structural planes. More faults were larger scale and developed in the east region than those in the west. The dips of most folds in the east region are toward the west, while shearing and corrugation are more prominent for the folds in the west region.

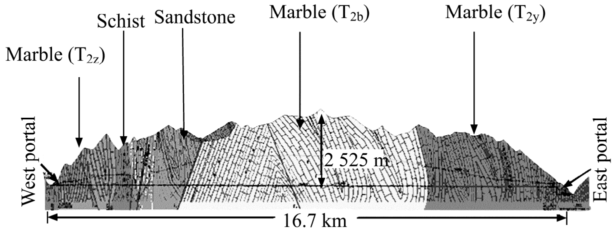


Fig. 1 Simplified geological profile along the headrace tunnel No. 1 (Zhang et al., 2011).

2.2.1 *Folds*

Most folds in the project area are approximately SN (NNE) compacted folds. The major folds include the Luoshuidong anticline, the Jiefanggou complex syncline, the Yangzhuchang complex syncline, the Zumu anticline, the Madang syncline and the Dashugou complex anticline from west to east. Four folds along the alignment of headrace tunnels are the Luoshuidong anticline, the Jiefanggou complex syncline, the Laozhuangzi complex syncline, and the Dashuigou complex anticline.

2.2.2 *Structural planes*

Among the 281 structural planes exposed along the auxiliary tunnels, the fault F6 is Type I structural plane, 15 are Type II, 62 are Type III-1, 127 are Type III-2, and 76 are Type IV. They can be roughly classified into SN, NE, NW and EW groups. The majority is approximately EW, accounting for about 40.9%, followed by 28.1% of approximately SN structural planes, 21.4% of NE planes and 9.6% of NW planes. According to the statistics on the dip angles of structural planes, the majority are deep dip angles, accounting for 77.9%, followed by 21.4% of moderately deep dip angles and 0.7% of gentle dip angles. Most structural planes are thrust faults and few are strike-slip faults. The general characteristics are summarized as follows: (1) the width of most structural planes is within 50 cm, (2) 71.2% of them are 0–50 cm in width, 8.9% are 50–100 cm in width and 19.9% are more than 100 cm in width, and (3) except the three regional structural planes F5, F6 and F27, the length of the structural planes is usually a few hundred meters. The filling materials in the ruptures are mainly cataclasite, breccia or schist and debris. Some contain fault gouge and secondary yellow mud. The filling materials of structural planes are dominated by rock debris and debris silt.

2.3 Mechanical properties of marble in depth

Laboratory triaxial tests indicate that the Jinping marble exhibits characteristics of brittle-ductile-plastic transformation. Fig. 2 shows the stress-strain curves of Baishan Group marble under different confining pressures (Zhang et al., 2010). It can be seen that the stress-strain curves of marble transfer from brittle to ductile after peak stresses with the increase in confining pressure. When the confining pressure is low, for instance, 2 MPa, the marble specimen fails shortly after the peak stress is reached. The stress-strain curve exhibits obvious brittle features. When the confining pressure increases to 10–15 MPa, the marble specimen becomes relatively ductile after the peak stress is reached. When the confining pressure continues to increase up to 40 MPa, the post-peak stress-strain curve shows evidently plastic response. The characteristics of brittle-ductile-plastic transformation significantly differ from those of some igneous rocks. For instance, the well-studied Lac du Bonnet marble shows typical brittle features even under high confining pressure of 60 MPa and its stress-strain curve drops rapidly after the peak stress.

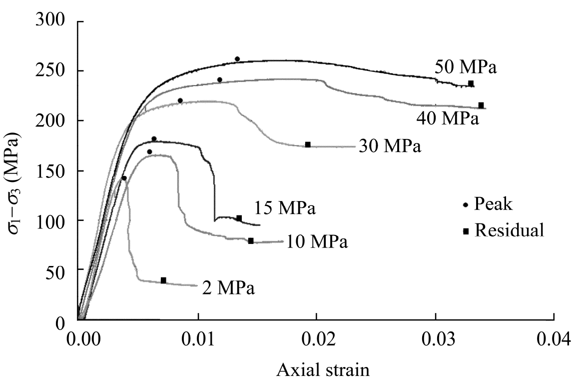


Fig. 2 Triaxial test results of Baishan Goup marble in the auxiliary tunnel of Jinping II hydropower station (Zhang et al. 2010).

Apparently, bulk rocks also have the characteristics of brittle-ductile-plastic transformation and the threshold confining pressures for transformation from brittle to ductile and from ductile to plastic are lower than those of the corresponding rock blocks. The characteristics of brittle-ductile-plastic transformation exhibited by marble would markedly affect the failure process and the secondary stress distribution in the surrounding rock masses under high in-situ stresses after tunnel excavation. As a result, it would affect the distance between the potential rockburst location and the working face. Fig. 3 shows the general relationship between rock mechanical properties and responses of surrounding rock masses after excavation of a deep tunnel (Zhu, 2009). Fig. 3(a) represents a typical brittle rock such as granite. For this type of rock masses, the post-peak stress-strain curve shows brittle characteristics as the confining pressure increases. The bearing capacity after yielding decreases significantly. Hence, the depth of brittle failure after excavation is usually great. Fig. 3(c) shows the response of perfectly elastoplastic rock masses. For the rock masses of this type, the bearing capacity is almost constant after yielding. Therefore, the failure depth under high in-situ stresses is relatively shallow. The mechanical behavior

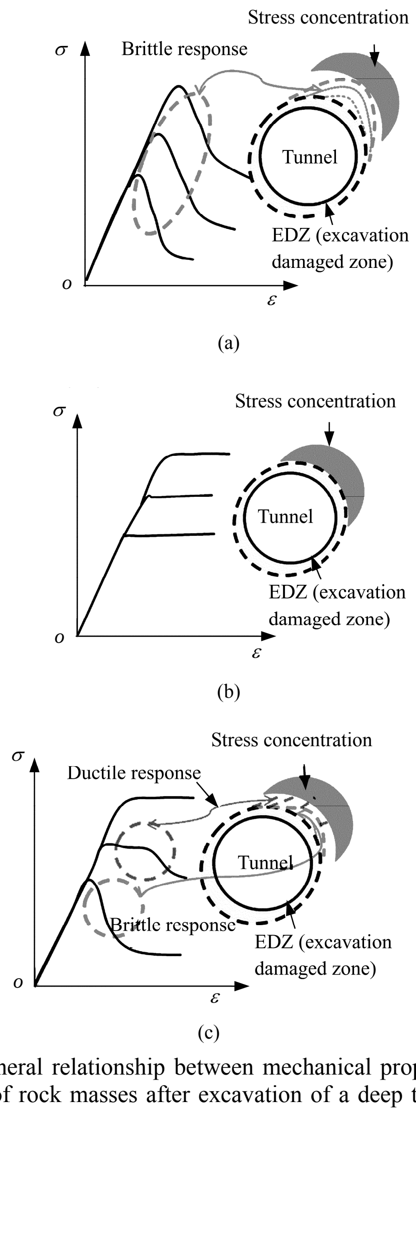


Fig. 3 General relationship between mechanical properties and response of rock masses after excavation of a deep tunel (Zhu, 2009).

of Jinping marble is between brittle and perfectly elastoplastic. The post-peak stress-strain curve varies with the increase in confining pressures. Correspondingly, its depth of brittle failure is between the brittle and the perfectly elastoplastic states.

It is commonly recognized that the high-stress region at a certain distance from the excavated tunnel wall is the dynamic source of potential strain rockburst. The stress concentration regions in Fig. 3 represent the high-stress regions for various types of rock masses after excavation. The characteristics of brittle-ductile-plastic transformation of marble determine relatively shallow damage or failure depth for the surrounding rock masses and closer distance of the high-stress region to the working face. Therefore, some low-energy microseismic events may lead to large-scale failures in the surrounding rock masses. This is the basic feature of the strain rockbursts observed in Jinping II hydropower station, which is completely different from the rockbursts taking place during deep tunnel excavation in igneous rocks such as granite. This is of significant meaning to the formulation of prevention and control measures for rockbursts in Jinping II hydropower station.

2.4 In-situ stresses

The maximum overburden depth is 2 375 m for the auxiliary tunnels and 2 525 m for the headrace tunnels. The measured major principal stress of this region is 46 MPa. Hydraulic fracturing method was adopted for rock stress testing in the auxiliary tunnels. Various measurement methods were employed in different exploratory tunnels to measure and analyze in-situ stresses, including aperture method, bored wall method, laboratory acoustic emission method, hydraulic fracturing method and back-analysis of displacement convergence. Based on the finite element simulations (Fig. 4) and multiple regression analysis of the three-dimensional (3D) in-situ stress field in the project area, the following understandings are obtained:

(1) The regression analysis of in-situ stress during the feasibility study shows that the major principal stress (σ_1) along the longitudinal profile of the

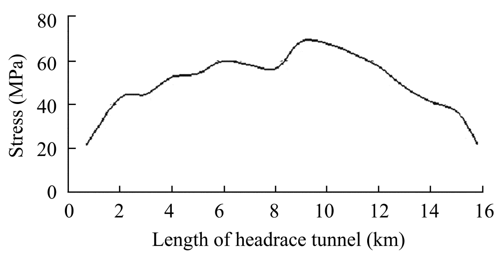


Fig. 4 Distribution of major principal stress along a headrace tunnel.

headrace tunnel No. 1 is approximately 70.1 MPa, the maximum intermediate principal stress (σ_2) is about 38 MPa, and the minor principal stress (σ_3) is approximately 31 MPa. The dominant factors controlling the in-situ stress field are gravity and the horizontal compressional structure in approximately WE direction.

(2) It can be observed from the analyses of the stress field in the tunnel excavation area that the stress field can be classified into three typical stress zones: (i) the valley stress zone in the tunnel entrance and exit regions; (ii) the gravity stress zone in the middle tunnel sections; and (iii) the fault stress zone in the tunnel sections crossing faults. The features of the three stress zones vary from each other. It is recommended that the stress zones are treated discriminatingly during design and construction.

(3) In a global sense, the intermediate principal stress of the regional in-situ stresses is basically consistent with the direction of tunnel axis in the middle tunnel sections, while the major and minor principal stresses are approximately perpendicular to the tunnel axis.

3 Statistics, classification, mechanism, and criterion of rockbursts

3.1 Statiscics of rockbursts

Based on the statistics of rockburst events occurred during construction of the auxiliary tunnels, the accumulative tunnel length with rockburst events in the auxiliary tunnel A is 3 222.5 m, accounting for 18.48% of the total tunnel length; while in the auxiliary tunnel B, it is 2 838.7 m, accounting for 16.29% of the total tunnel length. The rockbursts in the auxiliary tunnels are predominantly mild rockbursts (level I), accounting for 12.54% of the total tunnel length in the auxiliary tunnel A and 10.32% in the auxiliary tunnel B. Moderate rockbursts (level II) account for 4.13% and 4.67% of the total tunnel length in the auxiliary tunnels A and B, respectively. Some intensive rockbursts (level III) occurred in the auxiliary tunnels A and B, accounting for 1.73% and 1.12%, respectively. Few very strong rockbursts (level IV) occurred, and accounted for 0.09% and 0.17% of the total tunnel length in the auxiliary tunnels A and B, respectively.

Rockbursts along the auxiliary tunnels mainly occurred in the strata T_{2z} and T_{2b}. 19.44% and 15.41% of the total tunnel length with rockbursts are located in the stratum T_{2z} in the auxiliary tunnels A and B, respectively. 63.48% and 63.07% of the total tunnel length with rockbursts are located in the stratum T_{2b} in the auxiliary tunnels A and B, respectively. 7.66% and 10.44% of the total tunnel length with rockbursts are located in the stratum T_{2y}^{5} in the auxiliary tunnels A and B, respectively. 5.85% and 4.47% of the total tunnel length with rockbursts are located in the stratum T_{2y}^{6} in the auxiliary tunnels A and B, respectively. Few rockbursts occurred in the strata T_{2y}^{4} and T_3. The most intensive rockbursts in the stratum T_{2b} are very strong (level IV) and intensive rockbursts (level III) in the stratum T_3, and mild (level II) or moderate rockbursts (level I) in other strata. Most rockbursts occurred within 6–12 m away from the working face and 5–20 hours after excavation.

With the headrace tunnels advancing, both the number and intensity of rockbursts increase. Especially at the tunnel crown and spandrel, rockbursts of levels II–III were observed. The fractured surface was rough, step-like or dome-shaped. Tensile and shear failure resulted in wedge or dome-shaped fractured surfaces and its influential depth may reach 160 cm. Strong crackling and sometimes dull sound similar to blasting were captured during rockburst events. Since the start of the headrace tunnels construction, 77 rockbursts of various levels have been observed in the headrace tunnel No. 1, which caused serious damage to TBM equipment. About 200 rockbursts of various levels happened in the headrace tunnel No. 2. About 100 rockbursts occurred since the commencement of the headrace tunnel No. 3 in November 2008 and more than 100 rockbursts occurred in the headrace tunnel No. 4. Since the tunnels were excavated in marble of Baishan Group, rockbursts of levels III–IV occurred in some tunnels with the increasing depth. The maximum ejection distance of very strong rockbursts reached 5.0 m, and the crater depth ranged from 3 to 5 m, and the rockburst sound lasted for about 5 hours. Table 1 shows the statistics of rockbursts occurred in headrace tunnels (east end). According to Table 1, most intensive rockbursts occurred within 10–30 m behind the working face. In this region, the stress is redistributed in a violent manner and highly concentrated. Most rockbursts occurred 0.5–8 hours after blasting. Intermittent rockburst events still occurred even after the surrounding rock masses were supported with shotcrete at some tunnel sections where equilibrium had not been reached for stress redistribution. However, the intensity and frequency reduced to a certain degree. Most rockbursts occurred on the north spandrel (1–3 o'clock position) and the south arch corner (7–10 o'clock position). This is mainly related to the direction of the principal stresses and the major geological structures.

The geological conditions of the headrace tunnels are basically the same as those of the drainage tunnel and the auxiliary tunnels, but the dimensions of the headrace tunnels are larger than those of the drainage tunnel and the auxiliary tunnels. The size effect played an important role in rockbursts under high in-situ stresses during the headrace tunnels excavation. Preliminary studies showed that tunnels with diameter of about 8 m were more prone to rockbursts. Nevertheless, for the same level of rockbursts, a larger range of damage in the surrounding rock masses would occur in tunnels with larger dimensions. Hence, the damage due to rockbursts in the headrace tunnels may be more severe than those in the drainage tunnel and the auxiliary tunnels.

3.2 Classification of rockbursts

Rockbursts can be classified according to the damage potential (intensity), scale, and violence. In China, rockbursts are primarily classified based on the damage potential, i.e. weak rockburst, medium rockburst, strong rockburst and very strong rockburst. Rockburst classification for Jinping II hydropower station is based on the "Code for hydropower engineering geological investigation". In terms of scale (length of rockburst pit), rockbursts can be divided into sparse rockburst (L<10 m), large-area rockburst (10 m ≤ L≤20 m), continuous rockburst (L >20 m). According to the failure patterns, rockbursts can be divided into: (1) flaky or slabby spalling, (2) bending failure, (3) dome/wedge-shaped failure, and (4) cavern collapse. Based on the controlling factors, rockbursts can be

Table 1 Statistics of rockbursts occurrence in headrace tunnels (east end).

Rockburst level	Headrace tunnel No. 1		Headrace tunnel No. 2		Headrace tunnel No. 3		Headrace tunnel No. 4		Accumulative length (m)	Percentage (%)
	Accumulative length (m)	Percentage (%)	Accumulative length (m)	Percentage (%)	Accumulative length (m)	Percentage (%)	Accumulative length (m)	Percentage (%)		
No rockburst	6 816.091	92.62	7 890.586	88.39	6 732.88	92.48	6 780.275	87.44	28 219.832	90.1
Level I	497.7	6.76	680.5	7.62	524.5	7.2	815.5	10.52	2 518.2	8.04
Level II	45.5	0.62	300	3.36	23	0.32	138	1.78	506.5	1.62
Level III	0	0	45	0.50	0	0	20.4	0.26	65.4	0.21
Level IV	0	0	11	0.12	0	0	0	0	11	0.04

classified into strain burst, slip burst and pillar burst. The rockbursts in Jinping II hydropower station are mainly of strain bursts, with some controlled by tectonic structures such as faults. Due to cavern excavation, a small number of pillar rockbursts occurred during construction. The statistical data indicate that the causes of rockbursts were complicated and various forms of rockbursts from weak to very strong rockbursts occurred. The failure patterns varied from slight spalling and ejection to cavern collapses. The rockbursts are characterized by complex mechanism, various scales and forms.

3.3 Mechanism and criterion of rockbursts

Rockburst is a catastrophic phenomenon triggered by a progressive failure process. Its mechanism is very complicated. Most studies on rockburst mechanism are still of hypothetical or empirical level. Currently, the strength theory, the energy theory, the stiffness theory, the rockburst proneness theory, the instability theory, the fracture damage theory, and the catastrophe theory have been proposed for rockburst mechanism. The strength theory, the energy theory, and the rockburst proneness theory have been applied to the Jinping II hydropower station project. Due to the complex geological conditions at the construction site, a single criterion evidently cannot satisfy the requirements. Comprehensive judgments have to be made in combination of the field conditions.

Currently, a few methods are available for rockburst prediction, mainly including the discreminant index method, the field measurement method and numerical analysis. So far, no unified and mature prediction theories or methods are available, for each method has its own advantages and drawbacks. Based on this, the microseismic monitoring systems, from ESG Company of Canada and ISS of South Africa, have been employed for rockburst prediction in the Jinping II project, which is the first application in China. Field measurements were applied for prediction and early warning of rockburst risks at the excavation face and good results were achieved.

4 Prevention and control measures for rockbursts

In order to effectively prevent and control potential rockbursts, various measures such as geological early-warning prediction, microseismic monitoring, optimization of blasting network, optimization of excavation and support scheme, adoption of new materials, are taken. At present, some achievements have been made in rockburst prevention and control during construction of the headrace tunnels.

4.1 Rockburst monitoring

A large number of researches indicate that the energy accumulated in rocks is gradually released in the form of acoustic emission for a certain period of time before failure. Each microseismic event contains useful information for the variation in internal state of rock masses. Processing and analyzing of received microseismic signals can provide a basis for the assessment of rock mass stability. It is well known that rocks loaded in testing machine and rock masses stressed near underground excavations both emit detectable acoustic or seismic signals, and microseismic monitoring techniques have been thus used to locate damage in rock engineering practice (Cai et al., 2001). Therefore, this feature of microseism can be utilized to monitor the stability of rock masses and in turn to predict rock mass collapsing, caving, slabbing, sliding, and rockburst. The microseismic technique has been applied abroad for many years (Ge, 2005). In some countries, microseismic monitoring is implemented as a mandatory standard in mining engineering. The application of microseismic monitoring technique in China started a few years ago (Hirata et al., 2007). The microseismic monitoring devices manufactured by ISS Company, South Africa, were adopted to establish a microseismic monitoring system in Dongguashan copper mine at great depth (Tang et al., 2006). Jiang et al. (2008) reported a domestic microseismic monitoring system employed to monitor water inrush in a coal mine. The microseismic monitoring technique can also be used to study the crack patterns under compression in an oilfield (Liu et al., 2004). In a global sense, the microseismic monitoring technique has been applied widely in more fields, especially in mining engineering (Lu and Zhang, 1989; Li et al., 2005; Mercera and Bawden, 2005; Cheng et al., 2007; Wang and Ge, 2008).

A full set of microseismic monitoring system, special data acquisition system and data processing software developed by ESG Company, Canada, and ISS Company, South Africa, were adopted to assist in monitoring excavation-induced and natural seismic events. The drainage tunnel and the headrace tunnels Nos. 1–4 under construction in Jinping II hydropower station are monitored by this system. For the first time, a movable microseismic monitoring system moving with advancement of TBM or drill-and-blast tunneling has been established. The microseismic events in front of and behind the working face are continuously monitored in real time. The monitoring data were analyzed for rockburst prediction. On October 7, 2009, the first prediction was made for a possible rockburst in the construction of drainage tunnel. On October 9, 2009, an intensive rockburst occurred. The approximate ranges of a number of rockbursts were predicted.

4.2 Characteristics of blasting vibration and stress release effect in the tunnel sections with rockbursts

The presence of rockburst is affected and triggered by various factors in site. The degree of stress release and the release process are the major factors affecting the rockburst levels. Therefore, the study on proper blasting network and parameters for tunnel sections prone to rockbursts is helpful in reducing the risk level and intensity of rockbursts. The characteristics of blasting vibration and blasting effects are carried out in this study. In combination with the typical layout of blasting holes and sequential detonation for tunnel excavation, the damage effect of blasting under extremely high in-situ stresses is investigated by feedback of the monitoring results, which mainly cover blasting vibrations, loosening of the surrounding rock masses, theoretical analysis and numerical simulations. The scope of the study covers the effects of blasting on: (1) the surrounding rock masses and support of the blasted tunnel under high in-situ stresses, (2) the surrounding rock masses of the adjacent tunnels, (3) the optimization of the excavation sequence, (4) the blasting parameters, and (5) the blasting network for tunnels under high in-situ stresses. By optimizing the blasting parameters and network, the risk level of rockburst in the tunnel under construction can be reduced.

At present, inclined radial blasting holes are drilled in the surrounding rock masses. Two blasting schemes were proposed: (1) holes are drilled at the right of the crown and the bottom of the left sidewall; and (2) holes are drilled at the left and right of the crown and the bottom of both sidewalls. The main blasting parameters are determined by experiments.

A tunnel section (length of 50 m) at (2)10+800–10+750 in the headrace tunnel No. 2 was selected as the research background for the blasting schemes. Three groups of blasting tests were carried out for each scheme. The blasting parameters and excavation method for optimal stress release were determined based on the acoustic emission and blasting vibration monitoring results.

The scope of the blasting tests covers: (1) selection of blasting parameters; (2) detection of blasting effect (including the stress release degree in the surrounding rock masses, the probability and intensity of rockbursts, the disturbance and damage to the bedding rock); and (3) the effect of blasting on the adjacent buildings and shotcrete area.

The preliminary experimental results indicate that the risk level and the intensity of rockbursts can be reduced to a certain extent by adjusting blasting parameters and network, including the layout of radial holes for stress release, the charge, the precise calculation and the optimization of blasting network.

4.3 Support of tunnel sections characterized by potential rockbursts

The correlation between the mechanical damage of deep tunnels under high in-situ stresses and the damage of engineering works in the surrounding rock masses is in good agreement with those near the seismic sources and at the ground surface (buildings). Hence, the design idea of the rockburst prevention and control in the headrace tunnels should be consistent with that of aseismic design, i.e. “no damage under minor earthquakes, reparable under moderate earthquakes, no collapse under strong earthquakes”. The design and construction principles are, considering active and passive rockburst prevention and control measures, to realize that:

(1) no damage should occur in the bearing system consisting of shotcrete and anchors in case of mild rockbursts (level I);

(2) local small-scale damage is allowed in the bearing system in case of moderate rockbursts (level II), and the tunnel structural stability can be ensured by local reinforcement afterwards;

(3) no outbursting collapse should occur in the load bearing system consisting of shotcrete and anchors (steel frames) in case of intensive rockbursts (level III) and very strong rockbursts (level IV); and

(4) the long-term stability can be ensured by removal of damaged support and implementation of overall heavy-duty reinforcement by combining shotcrete, concrete lining and grouting, etc.

In order to provide a scientific basis for support design, geophysical investigations were carried out for the headrace tunnels. According to the geophysical investigations, the relaxation conditions for the sections at the east and west portals of the headrace tunnels are shown in Tables 2 and 3.

Table 2 Statistics of relaxed and unrelaxed rock masses zones at the west end of headrace tunnels.

Stratum	Relaxation depth (m)		Average wave velocity (m/s)		Reduction rate of wave velocity (%)	Range of cover depth (m)	Section No.
	Range	Average	Relaxed zone	Unrelaxed zone			
T_1	1.4–6.4	4.09	3 777–4 179	4 484–4 898	17.18	1 554–1 790	7
T_{2z}	1.2–2.2	1.8	4 201	5 440	22.78	831	1
	0.28–2.8	2.91	3 841–5 457	5 025–6 309	16.83	1 184–1 232	8
	1.0–6.2	3.15	4 069–4 638	5 201–5 922	21.32	1 641–1 827	4

Table 3 Statistics of relaxed rock masses at the east end of headrace tunnels.

Stratum	Relaxation depth (m)		Average wave velocity (m/s)		Reduction rate of wave velocity (%)	Range of cover depth (m)	Section No.
	Range	Average	Relaxed zone	Unrelaxed zone			
T_{2b}	2.22–5	3.6	3 307–3 885	5 720–5 940	46.44	2 007–2 011	2
T_{2y}^6	1.4–5.2	2.39	2 522–3 963	5 400–6 054	48.13	1 744–1 845	5
	1.2–3.6	2.44	2 904–3 892	5 372–6 456	42.77	588–955	3
T_{2y}^5	0.8–4.8	2.42	2 306–3 360	5 223–6 923	48.91	1 517–1 912	8
	0.8–4.2	2.45	2 718–5 154	5 668–6 532	46.37	1 019–1 384	9
T_{2y}^5	1.0–4.8	2.48	2 297–4 040	4 748–6 190	47.17	220–1 428	5

Based on previous practices and experiences on the support design of the auxiliary tunnels, the implementation process of support scheme was adjusted and optimized according to the site-specific conditions some time after the construction of the headrace tunnels started. Two-phase support scheme has currently been adopted for the tunnel sections with rockbursts, which were constructed by the drill-and-blast method and TBM.

(1) The first phase is to use temporary support. The temporary support includes “shotcrete or steel meshes + anchors”. The specific parameters are determined according to the site conditions. In general, the temporary support is mainly applied to the area in the vicinity of the tunnel crown according to the current practices and experiences. In detail, “steel meshes + water expansion anchors” are adopted in the headrace tunnel No. 1 excavated by TBM; “steel meshes + grouted hollow anchor rods” are installed in the headrace tunnel No. 3 excavated by TBM; “shotcrete + water expansion anchors” are employed in the headrace tunnels Nos. 2 and 4 excavated by the drill-and-blast method.

(2) The second phase is the permanent support. On the basis of the temporary support, the TBM-excavated tunnels are further reinforced by steel meshes and anchors, and then covered by shotcrete. The drill-and-blasted tunnels are further reinforced by permanent rock bolts and covered by shotcrete. The support designed for marble tunnels with rockbursts level II or III in the original design scheme will serve as the permanent support system, and lining structure will be installed for other conditions.

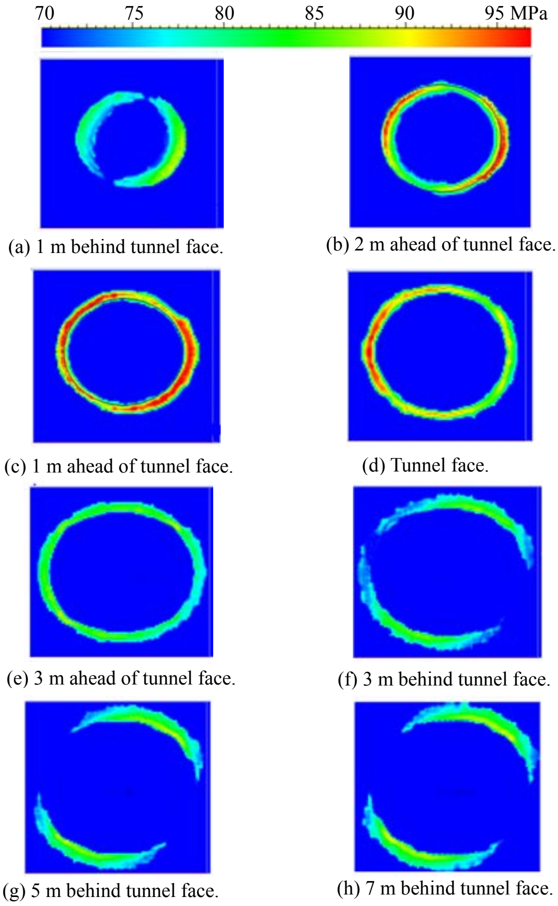
The effectiveness and rationality of the support system consisting of “shotcrete + steel mesh + anchor + shotcrete” have been well demonstrated in the earlier studies. So far, the construction practices have indicated that the support system is sufficient to maintain the stability and safety of the surrounding rock masses during construction. According to the geological data exposed by the excavated headrace tunnel sections, and in view of the evident relaxation phenomenon and temporal effect (especially in the deep tunnel sections) of the surrounding rock masses under high in-situ stresses, the presences of some local karst sections, water-rich sections, chlorite schist, slate, and highly corroded sections should be carefully considered to ensure the long-term stability of the surrounding rock masses of the headrace tunnels. Full-length concrete lining can be adopted for the headrace tunnels, including the systematic support, backfill, cement grouting and waterproof grouting.

In addition, as rockbursts are most likely to occur within 30 minutes to 8 hours after excavation, the support needs to be installed rapidly so that the support force is in place shortly after excavation. For this reason, some new materials, for instance, inorganic nano-materials and organic steel fibers, were adopted for the temporary support after several field experiments and investigations.

4.3.1 *Tunnel sections excavated by TBM*

For tunnel sections excavated by TBM, the purposes of rockburst control are to avoid the occurrence of intensive or very strong rockbursts as possible to prevent outbursting collapses due to rockbursts, and reduce negative impact on the safety and procedure of

TBM construction, and minimize the degree of damage to the bearing capacity of surrounding rock masses (Fig. 5).



(a) 1 m behind tunnel face. (b) 2 m ahead of tunnel face.

(c) 1 m ahead of tunnel face. (d) Tunnel face.

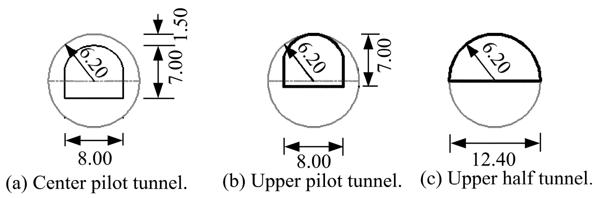
(e) 3 m ahead of tunnel face. (f) 3 m behind tunnel face.

(g) 5 m behind tunnel face. (h) 7 m behind tunnel face.

Fig. 5 General characteristics of stress distribution in the surrounding rock masses in front of and behind the tunnel working face (Zhu, 2009).

In response to the rockburst problems when TBM advanced to the core of Jinping Mountain, a contingent plan for rockburst control in case of intensive rockburst was proposed. The main idea of the plan was to release the high in-situ stresses with pilot tunnels by drill-and-blast method for the tunnel sections highly prone to intensive rockbursts. More importantly, the pilot tunnel can improve the condition that TBM equipment can do nothing when intensive rockbursts happen. At the same time, the pilot tunnels can serve as geological exploratory pits and the working faces for preconditioning and microseismic monitoring. The pilot tunnel provides a good opportunity for revealing, monitoring, analyzing and treating intensive and very strong rockbursts.

The pilot tunnels can be roughly classified into three types, namely, the center pilot tunnel (slightly upper), the upper pilot tunnel and the upper half tunnel. The numerical results indicate that the pilot tunnel (Fig. 6)



(a) Center pilot tunnel. (b) Upper pilot tunnel. (c) Upper half tunnel.

Fig. 6 Sketch maps of pilot tunnels (unit: m).

can significantly reduce the risk of strain rockburst during the secondary excavation by TBM. For drill-and-blasted tunnels, the advanced pilot tunnel can reduce the risk of fracture rockbursts. However, it cannot eliminate the occurrence of fracture rockbursts during secondary excavation.

The scheme employing pilot tunnel for Jinping II hydropower station project was implemented from the viewpoint of mechanical adaptability of TBM and economical benefits. With microseismic monitoring and further investigation, if some tunnel sections are exposed to the high risk of rockbursts that may trigger outbursting collapses, these sections are suggested to be excavated at full dimensions by drill-and-blast method so that TBM can pass through safely.

4.3.2 *Tunnel sections excavated by drill-and-blast method*

For the headrace tunnel section excavated by the drill-and-blast method, proactive prevention and control measures and strong support are needed to ensure construction safety. The measures mainly include control blasting, shotcrete, and anchor support that can be taken as follows: (1) control blasting with short footage, in combination with stress release for sections prone to very strong rockbursts; (2) removal of dangerous rock blocks and high-pressurized water cleaning; (3) timely coverage of rock surface by shotcrete; (4) timely implementation of anchorage measures against rockbursts (including fast bolt, hanging mesh, steel rib, etc.); and (5) follow-up installation of systematic rock bolts.

The detailed process can be described as follows:

(1) The construction sequence of the tunnel group before excavation should be properly arranged so as to avoid the deterioration of the stress state in the tunnels. We can appropriately adjust the construction activities if too many rockbursts occur, and wait for construction restarting until the major rockbursts are completed.

(2) Smooth blasting has to be adopted for tunnel excavation. If necessary, the working face has to be modified (the working face is excavated in a volute shape with a concave center). Fig. 7 shows the

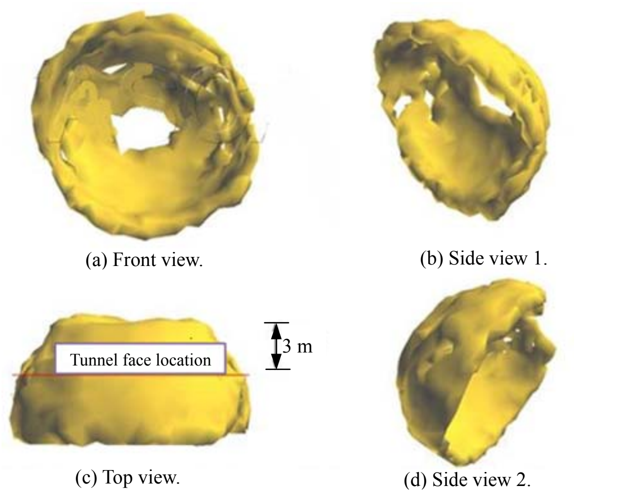


Fig. 7 The shape of the working face in tunnel sections with rockbursts (Zhu, 2009).

shape of the working face in tunnel sections with rockbursts (Zhu, 2009). The difference in the footage between the center and the periphery is controlled within 2 m so as to facilitate construction. Smooth transition from the center to the periphery and a generally arc-shaped working face both have to be maintained. The stress relief blasting technology is taken as a routine blasting operation in the high in-situ stress regions.

(3) Removal of instable rock blocks shall be carried out for the working face and the tunnel walls in the close proximity immediately after mucking, followed by high-pressurized water cleaning.

(4) The necessity of rockburst prevention and control is determined according to site inspection and understanding of the monitoring data. For tunnel sections prone to intensive or very strong rockbursts, advanced rock bolt support and shotcrete of the working face should be considered.

(5) Shotcrete should be applied immediately after excavation if the site is stable. The thickness and additives of the shotcrete should satisfy the design requirements. The thickness and quality of shotcrete have to be ensured to make up for the shortcomings in the anchorage construction. The thickness of preliminary shotcrete is generally 8 cm. For the sake of safety, the thickness can be increased to 10–20 cm if necessary. Other reinforcement measures such as shotcrete with hanging meshes can be adopted to ensure the safety of follow-up operations. Each steel mesh has to be prefabricated at the dimensions of 2.0 m× 2.0 m. The mesh should be overlapped or welded according to the design requirements.

(6) Rock bolts should be installed timely after the completion of shotcrete layer. If grouted hollow anchor rods or water expansion anchor bolts are used as the temporary support, the permanent support should be principally installed close to the working face with a distance less than the tunnel diameter. When it is difficult to follow up, thread bolts with early setting explosive cartridge (cement or epoxy) should be considered for the temporary support, or at least this type of bolts is used for some sections to increase the anchorage force. In addition, steel frames can be used to form a heavy-duty protection system according to the levels of rockbursts.

(7) As the headrace tunnels need further excavation at the bottom, the anchorage has to be installed close to the floor of upper bench as possible. Special attention should be paid to the area in the vicinity of the south abutment.

(8) As the installation of screw thread bolts may be time-consuming, they can be installed after the water expansion bolts are in place for the temporary support.

(9) The stress variation in the surrounding rock masses for the excavated sections and the microseismic events induced by deep rock fracturing should be monitored and pre-warned to guide the support reinforcement and excavation.

Due to the large excavated cross-section of the headrace tunnels and great time-consumption of rock bolt installation, field mechanized installation is employed to improve efficiency and quality of construction. Mechanized dobby drilling machine should be adopted by considering drill-and-blast method. Manual drilling is prohibited so as to avoid casualties due to the intensive rockbursts.

5 Conclusions

The headrace tunnels and the drainage tunnel of Jinping II hydropower station on the Yalong River are featured by great overburden depth, large length and diameter. The station is a large-scale underground hydropower project with ultra-deep-long tunnels. Based on the engineering practices in this project, the following conclusions can be drawn:

(1) By comprehensive analysis with microseismic monitoring techniques, most precursory information of rockbursts can be captured and reasonably accurate prediction can be made. It can provide important reference for rockburst prevention and control.

(2) During TBM excavation of the tunnel sections prone to strong rockbursts, the drill-and-blast method was employed to excavate a pilot tunnel before TBM

excavation was employed. It can effectively reduce the impact of strong rockbusts on economic losses and construction period for TBM construction.

(3) Effective support and stress relief in advance during excavation of tunnel sections prone to strong rockbursts can reduce the probability (intensity) of rockbursts, which is important to rockburst prevention and control.

A great number of explorations and practices have been performed for rockburst problems encountered in the Jinping II project, and finally comprehensive rockburst prevention and control system is established. These efforts are of great importance to mitigate the risks in construction safety and progress of the headrace tunnels of Jinping II hydropower station. Meanwhile, they can serve as a useful reference for similar projects.

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