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Mechanism of evolution of stress–structure controlled collapse of surrounding rock in caverns: A case study from the Baihetan hydropower station in China

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ABSTRACT

During the excavation of the underground powerhouse in the Baihetan hydropower station, which is currently still under construction, stress–structure controlled collapse has occurred frequently. In order to study the mechanism behind the evolution of this kind of collapse, an *in situ* experiment involving microseismic (MS) monitoring was carried out in the left main/auxiliary powerhouse. In this paper, the spatiotemporal characteristics of stress–structure controlled collapse are summarized and presented. A field survey, scanning electron microscopy and MS monitoring have been used to investigate a typical stress–structure controlled collapse that occurred during the monitoring period. These methods provided a consistent set of results, namely, that tensile fracturing is the rock-mass fracturing mechanism that is most active during the process of evolution of stress–structure controlled collapse. In addition, the evolution of the microseismicity during the development of the studied collapse was also obtained. The results provide a direct case history that will assist the prediction and support of stress–structure controlled collapse disasters and contribute to excavation of deeply-buried caverns in the field.

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1. Introduction

Collapse is a common and serious form of engineering disaster that occurs during the process of excavation of underground caverns. In general, the occurrence of collapse is mainly affected by the *in situ* properties of the rock and presence of structural planes in the rock banks (Fraldi and Guarracino, 2009), and can be treated as structure-controlled damage. Accordingly, collapse commonly occurs in special rock-mass structures, e.g. stratified and cataclastic structures, where the rock has poor quality and low stability (Xiang et al., 2011). The rock mass surrounding most collapses tends to be fractured. As a matter of fact, as the burial depth of an underground cavern increases, the rock-mass failure mode (for the same structure) changes in a manner corresponding to the *in situ* geostress from low, to moderate, to high. For example, when the surrounding rock mass has good quality (rock-mass rating, RMR > 75) the trend in the failure mode is from stable, to spalling, to rockburst (Hoek et al., 1995). Similarly, a surrounding rock mass with good self-supporting ability which may be stable at shallow depths, can be damaged (in the form of collapse) during excavation in deeply-buried caverns. For this kind of collapse, adverse stress conditions cannot be neglected and constitute, in fact, one of the controlling factors. This type of collapse is therefore defined as 'stress–structure controlled' (Martin et al., 1999). As the excavation depth develops, stress–structure controlled collapse occurs more frequently. The collapses can cause mechanical damage, delays to projects, and economic loss. As an example, several large stress–structure controlled collapses have occurred during construction of the underground powerhouse of the Baihetan hydropower station in China (with a rough burial depth of 330 m). On July 22, 2014, an intense collapse caused the total destruction of a drilling–blasting stair vehicle as well as an approximately ten-day delay to construction.

There has been much research on the evolution laws and mechanisms of such collapses, including tests on physical models and numerical simulations. Atkinson et al. (1975, 1977) reported some early investigations of the mechanism of collapse in shallowly-buried sandstone tunnels based on centrifugal model experiments. Using comprehensive methods of model testing and numerical simulation, the evolution laws in compressed zones (Nazimko et al., 1997), arching effects (Lee et al., 2006), and displacement during the process of collapse (Seokwon et al., 2004) have also been investigated. Diederichs (2003) stated that tensile damage plays an important role in the collapse mechanism in the failure mode of structurally controlled and stress-driven collapse in

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underground excavation in a hard-rock environment. Li et al. (2014) argued that shearing wedge damage in the direction of minimum principal stress is the cause of collapse in tunnels with crushed surrounding rock. They also suggested that the collapse is the result of the combined action of tension and shearing stress. However, the subject matter covered in the existing research is basically structure-controlled collapse itself. The laws of evolution and mechanisms of stress–structure controlled collapse have not yet been reported.

Monitoring microseismicity is important for understanding the *in situ* process of rock-mass fracturing associated with geological engineering disasters (e.g. rockbursts, gas outbursts, water inrushes, and landslips) that occur during rock engineering activities (Martin et al., 1997; Young and Collins, 2001; Feng et al., 2012; Cristina et al., 2014). Information from microseismic (MS) monitoring has indeed clarified the mechanism of fracturing in monitored rock masses (Feignier and Young, 1992; Cai et al., 1998; Feng et al., 2012, 2013). This suggests that if we can identify the type of rock mass fracturing involved during the development of stress–structure controlled collapse (tensile, mixed, or shear), then the mechanism of evolution of the stress–structure controlled collapse may be obtained directly.

The purpose of this paper is to explore the mechanism of evolution of stress–structure controlled collapses. To this end, an *in situ* MS monitoring experiment was carried out in the left main/auxiliary powerhouse in the Baihetan hydropower station in China. By chance, a typical stress–structure controlled collapse occurred in the monitored region during monitoring and this permitted us to acquire a significant amount of information about the macro-failure characteristics, micro-failure modes on the rupture planes, and microseismicity (presented in this paper). Accordingly, the mechanism of evolution associated with this stress–structure controlled collapse was analyzed.

2. Failure characteristics of the stress–structure controlled collapse

2.1. Background information on the underground Baihetan powerhouse

The Baihetan hydropower station is located in the second cascade of four hydropower stations in the downriver region of Jinsha River (Fig. 1a). It has a total capacity of $2.06 \times 10^{10}\ m^3$ and is designed to have an installed capacity of 1600 MW. Its underground powerhouse, which is still under construction, will be the largest in the world. The left- and right-bank parts of the powerhouse are located either side of the mountain, respectively. The powerhouse is composed of a main/auxiliary powerhouse, main transformer room, tail gate, and surge tank, as depicted in Fig. 1b. The dam area of the Baihetan station is mainly composed of basalt of the Upper Permian Emeishan Formation. The volcanic sequence of basalt flow layers can be divided into 11 rock layers ($P_2\beta^1$–$P_2\beta^{11}$) according to historic lava eruption episodes that have been recognized and identified. The underground powerhouse coincides with monoclinal strata with a strike of 40°N–50°E, SE tendency, and a dip of 15–25°.

The lengths of the left and right main/auxiliary powerhouses are 438 m and 434 m, respectively. The heights and widths of the two powerhouses are the same (86.7 m and 31 m, respectively; the width above the rock beam is 34 m). Excavation of the main/auxiliary powerhouse was carried out using bench excavating in layers and traditional drilling and blasting (D&B). A central pilot tunnel was excavated primarily in the first layer of the main/auxiliary powerhouse. Rock spallings occurred frequently on the east spandrel during the excavation of this pilot tunnel. Most of these spallings had a depth of 10–30 cm, and the maximum depth reached to about 100 cm. The frequency, region of distribution, and intensity of the spallings in the left bank were greater than those in the right bank. Therefore, an *in situ* experimental area was established in the left bank for real-time MS monitoring to monitor the process of rock-mass fracturing, possible spalling, and collapse as the excavation expanded towards the sides in the first layer of the main/auxiliary powerhouse. The strike of the main/auxiliary powerhouse where the experimental region was located is N 20°E and the thickness of the overlaying strata amounted to at least 300 m. The surrounding rock masses (mainly composed of fresh aphanitic basalt, porphyritic basalt, amygdaloidal basalt, and breccia lava) are hard and intact. The *in situ* stress measurements showed that the maximum major principal stress in the region of the main/auxiliary powerhouse (left bank) reaches 22.0 MPa and is approximately horizontal. Therefore, the stress level in the experimental region is high. A three-dimensional representation of the engineering geological conditions and position where the experimental region was located are shown in Fig. 1c.

2.2. Description of stress–structure controlled collapse

Many instances of stress–structure controlled collapse were observed during excavation of the underground powerhouse in the Baihetan hydropower station. For example, five and eight stress–structure controlled collapses of various scale occurred in the left and right bank, respectively, during excavation of the first layer of the main/auxiliary powerhouse during the month of August, 2014. The main spatiotemporal features of a typical stress–structure controlled collapse can be summarized as follows:

- Tend to occur in the region affected by excavation unloading and near the working face.
- Mainly occur on the spandrel or junction area between the spandrel and sidewall and are concentrated on one side of the cavern where rock spalling frequently occurs.
- The hard and integrated surrounding rock mass tend to contain only a few rigid structural planes (no more than two or two sets).
- The rock faces of the collapse pits are typically fresh and rough. The pits are typically nested or V-shaped. The side boundaries of the collapse pits are often controlled by the rigid structural plane.

A typical stress–structure controlled collapse is shown in Fig. 2.

2.3. The 7.22 collapse

The main/auxiliary powerhouse in the left bank was excavated using the D&B method. The excavation process in the first layer can be divided into five steps (I_1–I_5, as shown in Fig. 3b). In step I_1, a central pilot tunnel was first excavated (12 × 10 m). After completion of systematic support on the vault and spandrel of the central pilot tunnel, the floor of the central pilot tunnel was excavated downwards 1 m (I_2). Then, both sides of the central pilot tunnel were expanded by 6 m (I_3). The floor was, again, excavated downwards (by 2.6 m) when the systematic support (except on the sidewalls) was accomplished (step I_4). Finally, in step I_5, the excavation of the first layer was finished by excavating 5 m towards both sides. The systematic support used on site included: pre-stressed bolts (the diameter, length, and spacing of the bolts were 32 mm, 9 m, and 1.2 × 1.2 m, respectively), reinforcing fabric (the diameter and spacing were 8 mm and 150 × 150 mm, respectively), and reinforced concrete (with a thickness of 200 mm). Otherwise, two anchor cables (the row spacing along the tunnel heading

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