Seismic stability of an underground cavern in jointed rock mass based on the discrete fracture network

Chu Cun*1,2, Chen Peishuai1,3, Yuan Qing1,4, Jiang Hong1 and Feng Deding1

1CCCC Second Harbour Engineering Co., Ltd., Wuhan, 430040, China
2State Key Laboratory of Water Resources and Hydropower Engineering Science, Wuhan University, Wuhan, 430072, China
3Key Laboratory of Large-span Bridge Construction Technology & Ministry of Communications, Wuhan, 430040, China
4CCCC Highway Bridges National Engineering Research Center Co., Ltd., Beijing, 100088, China

*Corresponding author: 84862655@qq.com

Abstract. In the construction of underground structures in high seismic intensity areas, it is inevitable that a fault fracture zone will pass through the construction site. Therefore, it is very important to evaluate and analyze the stability of the unstable block, which is cut by the fault and the dominant joint group. In this study, the discrete element method was used to simulate the seismic response of an underground powerhouse of a hydropower station near a high seismic zone. The discrete fracture network (DFN) model was introduced to simulate the joint group. Moreover, to optimize the number of joints and improve computational efficiency under dynamic conditions, the joints in the DFN with little influence on the stability of the cavity, were deleted by the algorithm, and a nonlinear time-history response analysis method was used for dynamic calculation. Based on the simulation results obtained, it was possible to classify the instability of the unstable block, and corresponding suggestions for construction were provided. Additionally, these results can be used as a reference for the seismic design of similar underground structures.

1. Introduction

Hydropower projects have very long operation cycles and are extremely cost intensive; moreover, unfavorable geology is usually the primary reason that leads to the instability of the surrounding rocks when earthquakes occur. Therefore, it is necessary to evaluate the influence of joints and fractures on the stability of underground caverns at the sites of hydropower stations. Numerical simulation is a suitable method for studying the stability of the rocks surrounding an underground powerhouse under earthquake action. Dowding et al. investigated the response of cavities in jointed rock mass stimulated by sinusoidal vibration due to earthquake action [1]. Wang Xiaowei et al. studied the deformation and failure of underground cavern groups under seismic load and proposed a dynamic strength criterion considering the influence of strain rate on cohesive force [2]. Based on the block theory, Zhu Zeqi et al. studied the seismic response characteristics and stability evaluation methods for key blocks in the rocks surrounding underground workshops using the Newmark method [3]. Additionally, Cui Zhen et al. studied the influence of the structure effect of the random jointed rock mass on the seismic response of underground caverns and the influence of the control effect of the rock mass structure on
the seismic dynamic response of the underground cavern [4, 5]. On the basis of discussions related to the failure characteristics and the associated mechanism of underground structures, Chen Weizhong et al. summarized research progress on the evaluation of the seismic performance of underground structures [6]. Abundant results have been obtained in previous studies [7–9]; however, it is worth noting that for simplifying calculations, only some aspects of the underground powerhouse cavern group have been investigated, whereas the nonlinear failure behavior of the joints under earthquake action has not yet been considered. In the present study, an underground plant in southwest China was selected for the simulation calculation of stability under the influence of earthquake action, and 3DEC software was used for the simulation calculation. Additionally, time history analysis was used to perform the dynamic calculation, for which a nonlinear continuously yielding structural plane model was adopted.

2. Continuously Yielding Model and DFN

2.1. Continuously yielding model
In previous numerical simulation studies, linear elastic models, such as the Mohr–Coulomb structural plane model, with normal stiffness ($k_n$) and shear stiffness ($k_s$) parameters of the joints expressed as constants, have been frequently used for structural plane analysis. However, according to engineering practice and laboratory test, joints are a progressive damage resulting from shear action, and their mechanical behavior is nonlinear. Cundall and Hart proposed a continuously yielding joint model that can reflect the nonlinear mechanical behavior of joints under dynamic load more realistically than a linear elastic model [10]. The main mechanical parameters of this continuously yielding model are as follows:

$$
\Delta \sigma_n = k_n \Delta \mu_n \tag{1}
$$

$$
k_n = a_n \sigma_{en} \tag{2}
$$

$$
\Delta \tau = F k_s \Delta \mu_s \tag{3}
$$

$$
k_s = a_s \sigma_{es} \tag{4}
$$

where $\sigma_n$ represents the normal stress of the structural plane, $\mu_n$ represents the normal deformation of the structural plane, $k_n$ represents the normal stiffness of the structural plane, $a_n$ represents the initial normal stiffness of the structural plane, $e_n$ represents the normal stiffness index of the structural plane, $\tau$ represents the shear stress on the structural plane, $k_s$ represents shear stiffness, $\mu_s$ represents shear deformation, $a_s$ represents the initial tangential stiffness of the structural plane, $e_s$ represents the normal stiffness index of structural plane, and $F$ represents a specific parameter that is related to stress history, current shear stress, initial friction angle ($\phi$), effective friction angle ($\phi_m$), and roughness of structural surface ($R$). A more detailed derivation of this model can be obtained from a previous study. Figure 1 shows a typical stress–displacement curve for monotonic loading under constant normal stress.

2.2. Discrete fracture network
3DEC (three-dimensional distinct element code) can be used to simulate a discrete fracture network (DFN) model, which is a statistical description of the joints. With the DFN model, the fracture population embedded in a rock mass can be visualized as a set of discrete, planar, and finite-sized fractures [11]. By default, the discrete fractures are disk-shaped as shown in Figure 2. Additionally, a DFN template is a statistical description of a DFN and not an actual representation of fractures. The geometrical characteristics currently supported by the DFN module are fracture size (diameter), orientation, and position distributions [12].
3. Project Overview

The installed capacity of the pumped-storage power station in southwest China is 1200 MW. Its underground powerhouse, which is buried at a depth of approximately 410 m, is located approximately 280 m from the upstream reservoir. The underground powerhouse group mainly comprises the main powerhouse, main transformer tunnel, tailgate tunnel, bus tunnel, diversion tunnel, and construction branch tunnel. The major and auxiliary powerhouse holes are arranged in parallel with the main transformer holes at 40-m intervals. The actual situation after excavation is as follows: the surrounding rock is mainly granite and a group of joints is mainly developed (dip 80°, dd 10°). The parameters of the surrounding rock and the structural plane obtained from the field test and reverse calculation are given in the Tables 1 and 2. The three-dimensional arrangement of the cave group is shown in Figure 4. The maximum horizontal principal stress in the factory area is ~9.74–19.30 MPa and the minimum is ~4.87–10.90 MPa. Additionally, the maximum and minimum principal stresses are close to the horizontal and verticals direction, respectively. Basically, the principal stress increases gradually from top to bottom, and the stress concentration phenomenon is not obvious.

Table 1. Physical and mechanical parameters of rock mass.

| Rock Mass | Grade | Bulk Modulus (GPa) | Shear Modulus (GPa) | Density (kN/m³) | Cohesion (MPa) | Friction Angle (°) |
|-----------|-------|--------------------|---------------------|-----------------|----------------|-------------------|
| Granite   | III   | 10.2               | 6.7                 | 27              | 2.6            | 51                |

Table 2. Physical and mechanical parameters of joints (continuously yielding model).

| $a_n$    | $e_n$    | $a_s$    | $e_s$    | $\varphi$ | $\varphi_m$ | $R$               |
|----------|----------|----------|----------|-----------|-------------|-------------------|
| 0.43     | 0.247    | 0.32     | 0.648    | 17.2      | 6.8         | $1 \times 10^{-8}$ |

4. Excavation Calculation Under Static Conditions

Prior to the dynamic calculation, the calculation of the plant excavation under the static conditions was performed first. The excavation was conducted in 10 phases (Figure 3). For the rock mass, the Mohr–Coulomb constitutive model was adopted, and for the structural plane, the Coulomb slip constitutive model was adopted. The displacement cloud map of the rock surrounding the plant after the excavation was completed as shown in Figure 5. As can be seen from the calculation results, the deformation of the surrounding rock was controlled by the structural plane, and its displacement increased significantly in the joint distribution area. Under the influence of structural plane cutting and
the effect of the tunnel group, the maximum displacement of the surrounding rock (approximately 3.9 cm) was observed in the intersection area between the right-side wall of the main powerhouse and the entrance of the bus tunnel.

![Excavation zoning diagram.](image)

5. Dynamic Analysis

5.1. Boundary conditions and damping

To perform the dynamic calculation, the time history analysis method was used [13, 14]. The purpose of using the time history analysis method was to introduce dynamic action into the differential equation in the form of a time function. This is a dynamic analysis method for obtaining the direct integral solution for differential equations, which facilitate the calculation of the nonlinear failure of the model under dynamic action and obtaining the influence of time-holding factors under seismic action. In this study, this method proved to be a relatively accurate dynamic response analysis method. Regarding the dynamic calculation, the free-field boundary was set around the model, and the damping, which was performed with a damping ratio of 1%, was Rayleigh damping [15, 16]. The seismic waves were Wenchuan earthquake waves, from which 16 s of high energy were selected for
the calculation. According to the code [17], seismic waves need to be reduced proportionately when they are input from the ground. The reduced horizontal and vertical peak acceleration values were 0.6 and 0.4g, respectively. The time-history curve of acceleration after filtering and the baseline correction are shown in Figures 6 and 7. The seismic wave was input from the bottom of the model, and to perform the dynamic calculation, the continuously yielding constitutive model was adopted for the structural plane (Table 2).

![Time-history curve of horizontal acceleration](image1)

**Figure 6.** Time-history curve of horizontal acceleration.

![Time-history curve of vertical acceleration](image2)

**Figure 7.** Time-history curve of vertical acceleration.

5.2. Analysis of dynamic calculation results

To show the displacement increment of the surrounding rock under earthquake action more intuitively, the displacement generated by the excavation of the surrounding rock under static conditions was cleared prior to the dynamic calculation. With the action time of the seismic wave, the cloud diagram of the displacement increment deformation of the surrounding rock was as shown in Figure 8.

![Seismic displacement (m) contour at different seismic](image3)

**Figure 8.** Seismic displacement (m) contour at different seismic
moments.

Figure 8 shows that when the action time of the seismic wave was 3 s, the overall displacement increment of the surrounding rock was not large, and the maximum displacement, which was observed at the 1# tail gate chamber, was approximately 3.2 mm. When the action time of the local seismic wave was 6 s, the displacement of the rock surrounding the tunnel chamber increased considerably, and the maximum displacement at the 1# tail gate chamber reached 4.6 cm. As the earthquake action continued, the displacement of the surrounding rock constantly fluctuated and at the end of the earthquake, the maximum residual displacement was approximately 2.2 cm. From these displacement variations due to earthquake action, it is evident that the displacement increment is composed of both elastic and plastic deformation. In the region without joints, the surrounding rock primarily produced elastic deformation under earthquake action, and the plastic deformation was very small after the earthquake. In the region where the joints are developed, unstable blocks were formed under cave intersection. Additionally, facing surface and joint cutting had a larger displacement increment and a larger plastic displacement value under the action of earthquake and were prone to instability. Considering the displacement variation of the surrounding rock under static and dynamic conditions, the evaluation of the stability of the surrounding rock can be divided into three levels. The first level is the high-risk area, which generates a large displacement under both static and dynamic conditions. The second level is the medium-risk area, which generates a small displacement under static conditions, but a large displacement under dynamic action. Finally, the third level is a low-risk region, which generates a small displacement under both static and dynamic conditions, as shown in Figure 9.

Figure 9. Stability risk assessment of surrounding rock.

6. Conclusions
In this study, the stability of an underground cavern under earthquake action was analyzed via numerical simulation, and some useful conclusions were derived:
Under both static and dynamic conditions, the structural plane plays a controlling role in the displacement of the surrounding rock. Further, the deformation of the surrounding rock under the dynamic conditions consists of both elastic and plastic deformation. Plastic deformation mainly occurs in the region where the structural plane is developed. Compared with the surface structure, which was seriously damaged by the Wenchuan earthquake, the overall displacement increment of the underground powerhouse is small, indicating that the underground structure has a better seismic performance.
Additionally, the deformation of the surrounding rocks could be divided into three levels. In the high-risk areas, it is necessary to improve support measures, including both static excavation and seismic support measures. In the medium-risk areas, it is necessary to improve seismic support measures, and in the low-risk areas, no support-improvement measures are necessary.
References

[1] DOWD C H, BEL T B 2000 Dynamic response of million block cavern models with parallel processing. *Rock Mechanics and Rock Engineering*. 33 207-14

[2] Wang X W, Chen Q T and Xiao M 2016 Seismic response analysis of underground cavern in layered rock mass. *J. Huazhong univ. of Sci. & Tech. (Natural Science Edition)* 44 18-24

[3] Zhu Z Q, Sheng Q, Leng X L, Zhu F G 2010 Study of seismic response of key block in large underground opening group. *Rock and Soil Mechanics*. 31 254-64

[4] Cui Z, Sheng Q and Leng X L 2018 Effects of a controlling geological discontinuity on the seismic stability of underground caverns subjected to near-fault ground motions. *Bulletin of Engineering Geology and the Environment*. 77 265-82

[5] Cui Z, Sheng Q and Leng X L 2017 Analysis of S wave propagation through a nonlinear joint with the continuously yielding model. *Rock Mechanics and Rock Engineering*. 50 112-23

[6] Chen W Z, Song W P, Zhao W S, Yang D S, Zhao K and Sheng Q 2017 Research progress of seismic analysis methods and performance evaluation in underground engineering. *Chinese Journal of Rock Mechanics and Engineering*. 36 311-25

[7] CHEN Z Y and SHEN H 2014 Dynamic centrifuge tests on isolation mechanism of tunnels subjected to seismic shaking. *Tunnelling and Underground Space Technology*. 42 67-77

[8] Wang R B, Xu W Y, Shi C and Zhou X Q 2009 Dynamic response analysis of rock underground caverns in highly seismic region. *Chinese Journal of Rock Mechanics and Engineering*. 28 569-75

[9] Zhang Z G, Xiao M and Chen J T 2011 Simulation of earthquake disaster process of large-scale underground caverns using three-dimensional dynamic finite element method. *Chinese Journal of Rock Mechanics and Engineering*. 30 510-23

[10] CUNDALL P A, LEMOS J V. 1990 Numerical simulation of fault instability with the continuously-yielding joint model[C]// *Rock bursts and Seismicity in Mines*. Rotter dam: A. A. Balke ma.

[11] Wang Tao, Wu H G, Su K 2017 DEM modeling of the stability of the Jurong underground powerhouse cavern[C]// *American Rock Mechanics Association*, Geomechanics Symposium held in San Francisco, California, USA, 448-54.

[12] Wang T, Chen X L and Yu L H 2005 Discrete element calculation of surrounding rock mass stability of underground cavern group. *Rock and Soil Mechanics*. 26 1936-40

[13] Zhang Y T, Xiao M and Liu B 2011 Analysis of Seismic Damage Mechanism of Underground Powerhouse Structures of Hydropower Plants in High Seismic Intensity Region. *Journal of Sichuan University (Engineering Science Edition)*. 43 71-76

[14] Zhu Y S, Zhu H C, Shi A C and Meng G T 2011 Complicated block stability analysis of baihetan hydropower station based on distinct element method. *Chinese Journal of Rock Mechanics and Engineering*. 30 2069-75

[15] Melih G 2010 Assessment of the dynamic stability of the portals of the Dorukhan tunnel using numerical analysis. *International Journal of Rock Mechanics & Mining Sciences*.47 1231-41

[16] Xiao J, Zhou Y, Han Y H and Wang T 2018 Seismic responses of Pompeii colonnade structure based on three-dimensional distinct element method. *Engineering Journal of Wuhan University*.51 689-02

[17] The National Standards Compilation Group of People’s Republic of China. GB51247—2018 Code for seismic design of buildings[S]. Beijing: China Architecture and Building Press, 2018.(in Chinese)