Research on the deformation and failure pattern of rock masses in dam foundations controlled by continuous gently inclined and discontinuous steeply inclined unconformities

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Abstract: The rock masses of the sluice gate of the Datengxia Hydropower Station in Guangxi Province are selected in this study to conduct stability analysis by using Universal Distinct Element Code (UDEC) software. The investigated rock masses are composed of continuous soft layers with low dip angles as well as discontinuous structural fractures with high dip angles. The strength reduction method is used to determine whether the failure pattern is shear or compression type according to the displacement of key points, force imbalance, and plastic yield zone. The results show that the failure pattern of rock masses with continuous gently inclined and discontinuous steeply inclined unconformities is compression failure rather than regular shear failure. The reliability and rationality of this method are verified by comparing the results with those obtained by using a physical experiment model. Therefore, the results of the method proposed in this study can serve as a reference for similar engineering projects.

Keywords: Strength reduction method, Stability, UDEC, Discrete element

1. Introduction

China has a vast territory and abundant hydropower resources. Accordingly, significant developments have been made in the hydropower industry, which has caused the geological conditions for dam site selection to become increasingly complicated. In particular, dam foundations and abutments in hydraulic engineering projects are often embedded in rock masses. Therefore, the stability of the rock mass is a key factor affecting the operation of hydropower projects. In a dam foundation, this stability is affected mainly by discontinuities manifested as soft interlayers, bedding planes, or fractures; thus, it is necessary to study the geometric and mechanical characteristics of such unconformities in the rock mass.

In the last 10 years, the discrete element method has been increasingly used in engineering analysis. The boundary between blocks in this model can interact to reflect its discontinuity and fracture characteristics. The discrete element method applies an explicit integral iteration algorithm, thus accommodating large displacement and rotation. Furthermore, discrete element methods can more
accurately reflect the geometric characteristics of fractured rock masses compared with other numerical methods, which is beneficial for considering nonlinear deformation and the destruction of rock masses containing discontinuities. In recent years, many scholars have used the discrete element method to analyze the safety of dams. Yang \cite{1} proposed a stability calculation formula of a gravity dam with multi-slip surfaces based on boundary simulation of the discrete blocks. Chen \cite{2} used the particle flow discrete element method to analyze the stability of a dam body and obtained the factor of safety. Wang \cite{3} et al. used the strength reserve method of discrete element analysis to evaluate the stability of a dam foundation, assign a safety factor, and determine the final failure mode. Zhang et al. \cite{4} used the 3DEC software to analyze the overload and strength reduction in an arch dam. Hou et al. \cite{5} used the discrete element method to simulate the collapse of China’s Meihua arch dam and analyze the dam failure process.

Moreover, the strength reduction method is commonly used to evaluate the cause of instability in a rock mass and has been widely applied in engineering projects. Pei et al. \cite{6} used this method to study the deep anti-sliding stability of a gravity dam. Other scholars \cite{7-14} analyzed the stability of gravity dams by using the strength reduction method based on the discrete element method. In particular, the strength reduction method is used to simulate the instability of rock masses by decreasing the strength parameters such as cohesion and the internal friction angle in the model while keeping the load unchanged. When the system reaches the limit equilibrium state, the failure mechanism of the entire system can be studied. The ratio of the shear strength applied at this time to the original shear strength is the factor of safety $F_s$ of the slope as in the following formula:

$$\begin{align*}
c' &= \frac{c}{F_{Sr}} \\
\varphi' &= \arctan\left(\frac{\tan\varphi}{F_{Sr}}\right)
\end{align*}$$

(1)

The rock masses of the sluice gate of the Datengxia Hydropower Station exhibit a complex discontinuity system. The main discontinuities affecting the stability of the dam foundation include soft interlayers with gentle dip angles and structural fractures with steep dip angles, which will be described in section 2. In general, the failure of a rock mass of dam foundation is multi-slip surface failure. However, fractures with steep dip angles more frequently affect the stability of the dam foundation rock mass in many hydropower projects. This type of discontinuity does not easily slip under the action of high water pressure. Therefore, it is necessary to study the deformation and failure mode of rock masses characterized by discontinuities with continuous gently inclined layers and discontinuous steeply inclined unconformities.

Because a discontinuity system in a rock mass affects the deformation failure mode, the influence of discontinuities must be fully considered in the analysis. At present, only the discrete element method can be employed for this purpose; thus, we use Universal Distinct Element Code (UDEC) software to conduct the aforementioned analysis. In addition, the strength reduction method is applied to study the deformation and failure mode of the dam foundation rock mass. It is determined that if a decrease in the strength parameters causes significant changes in the force and displacement of the dam foundation rock mass, the failure mode is shear failure.

2. Study area

The lithology of the rock masses of the Datengxia Hydropower Station mainly includes argillaceous limestone, limestone, and dolomite. The weak interlayers observed in the field are essentially the same as those of the rock formations. The composition of the mud zone is mostly mud containing cuttings, and its thickness is generally less than 3 mm. Section #28, as shown in Figure 1, was selected as the focus of the present study. The dam foundation of the sluice gate in the study area is exposed mainly in the D1y1-2 and D1y1-3 sub-formations of the Yujianjiang Stage. The lithology D1y1-3, which belongs to the lower Yujianjiang Stage of the lower Devonian system, is mainly gray–black limestone and dolomitic limestone. As shown in Figure 1, the karst phenomenon of the rock mass on the right side of the karst demarcation line is very well developed, whereas that on the left side is not developed. The beddig is developed with a thickness of 15–30 cm with an occurrence of the bedding at about N10°–
20°E, $\angle$SE10°–15°. The lithology of $Dy^1$-$2$, which belongs to the same stage, is mainly grayish black limestone. The bedding of the rock mass is developed, also with a thickness of 15–30 cm; its occurrence is about N5°–20°E, $\angle$SE10°–15°.

![Figure 1. Panoramic view of section #28](image1.jpg)

In addition, fractures are well developed in the fine sandstones and mudstone-bearing fine sandstones in the field. The groundwater occurrence in the rock mass alternates frequently, and the weathering of the rock mass is relatively strong. In the siltstone, argillaceous siltstone, and mudstone, however, fractures are not developed in the siltstone, argillaceous siltstone, or mudstone. The groundwater activity in this rock mass is weak, and its weathering is relatively poor compared with that in the other rock mass.

The geological conditions of the dam foundation of the Datengxia Hydropower Station are complex with active crustal movement. Fractures are extremely well developed in the field, and their characteristics differ. Field observation revealed that the fracture walls are mostly flat and smooth and are filled with fine debris and mud. The fractures exhibit obvious shear characteristics with small openings and steep dip angles. The fractures near the weak interlayer and the stratigraphic boundary are characterized mainly by their inability to extend throughout the soft layer and the stratigraphic boundary. The fractures observed in the field are shown in Figure 2. Numerous fractures oriented parallel to the dam axis in section #28 tend to form shear surfaces or break surfaces under the action of high water pressure, which adversely affects the stability of rock masses. Therefore, section #28 was selected as a case study for analysis in which the UDEC numerical model was established.

![Figure 2. Developed fractures observed in the field](image2.jpg)

3. Data and Method
The UDEC program provides several commonly used discontinuity models, such as the Mohr–Coulomb model for ideal elastic–plastic materials, the continuous yielding model, and the Barton–Bandis (BB) model. The strength reduction method introduced in this project is based on the Mohr–Coulomb model, through which the shear yielding, opening, and dilation effects can be realized.

The establishment of the model considers many factors, including seepage simulation, weathering degree of rock formations, and occurrence of discontinuities in the rock mass of the dam foundation.
Specifically, water infiltrates downward along the weak interlayers. According to the engineering geological conditions discussed in section 2, however, because the weak interlayers are in a mostly closed state, it is assumed that they are impermeable. To consider the seepage of rock formations and fractures, the simulation length of section #28 upstream was extended to 3.5 times the height of the dam (i.e., 150 m). The simulation length of section #28 downstream is approximately three times the height of the dam (i.e., 125 m), and the burial depth of the dam foundation rock mass is 122 m. In addition, the analysis range should be sufficiently large for avoiding the boundary effect in the model, which ensures that the actual analysis results will not be influenced. The final model, as shown in Figure 3, had a length of 340 m and a height of 144 m. The simulation of the dam body was performed under the action of an anti-seepage curtain. The calculation of the model adopted the normal water levels of the sluice gate, which are 61 m and 22.71 m upstream and downstream, respectively. When the ratio of the maximum unbalance force to the system characteristic force was greater than a certain allowable value, the block system was considered to be in an unbalanced state; otherwise, the system remained in the balanced state. In this study, the allowable value was $10^{-5}$. The failure of the rock mass can be determined by the corresponding displacement vector map.

![Figure 3. Numerical model of section #28](image)

| Rock stratum         | Weathering degree   | C’ (Mpa) | f   | Deformation modulus | Elastic modulus |
|----------------------|---------------------|----------|-----|---------------------|-----------------|
| $D_1y^1 - 3$         | Weak weathering     | 0.82     | 0.60| 5                   | 8               |
| $D_1y^1 - 2$         | Weak weathering     | 0.85     | 0.62| 8                   | 12              |
| $D_1y^1 - 1$         | Weak weathering     | 0.80     | 0.60| 5                   | 8               |
| $D_1n_{13-3}$        | Weak weathering     | 0.79     | 0.58| 3                   | 4               |
Table 2. Physical and mechanical parameters of the discontinuities

| Discontinuity types | $K_a(Pa/m)$ | $K_s(Pa/m)$ | $\varphi(^\circ)$ | $c(Pa)$ | $\sigma^i_1(a)$ |
|---------------------|-------------|-------------|-------------------|---------|----------------|
| Fissure             | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 26.5              | 0       | 0              |
| $D_1n_{11-7}$       | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 14.5              | $1.5 \times 10^{5}$ | 0              |
| $D_1n_{13-1}$       | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 15.6              | $2 \times 10^{4}$  | 0              |
| $D_1y_{1-1}$        | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 17.7              | $4 \times 10^{4}$  | 0              |
| $D_1y_{1-2}$        | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 16.7              | $3 \times 10^{4}$  | 0              |
| Stratigraphic boundary | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 24.2              | $1.5 \times 10^{5}$ | 0              |
| Curtain             | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 42                | $7.5 \times 10^{5}$ | $2 \times 10^{6}$ |
| Fault boundary      | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 0                 | 0       | 0              |
| Dam bottom          | $1 \times 10^{11}$ | $1 \times 10^{10}$ | 42                | $7.5 \times 10^{5}$ | 0              |

4. Results and Discussion

(1) Seepage calculation

To ensure that the calculation results are close to the actual values, it is necessary to simulate the seepage flow. Thus, to study the displacement changes in directions $x$ and $y$, one key monitoring point from each of the five areas was selected after the end of calculation as the most likely point to be damaged. These areas include the gate chamber, left cogging bedrock, rock mass at the bottom of the gate chamber, right cogging bedrock, and near-surface bedrock downstream. To express the specific locations of the key monitoring points, A, B, C, D, and E were used to denote the respective locations of the five selected key monitoring points (Figure 4).

The strength reduction method was used to study the deformation and failure mode of the dam foundation, which is discussed in the following subsection. The influence of the seepage path under this condition, as shown in Figure 4, was considered in the calculation of dam foundation stability under same condition. The calculation method and steps are discussed above.
With an increase in the calculation time steps, the maximum unbalance force in the model tended to be zero at the end of calculation. Figure 5 shows the time-step curve of the displacement of section #28 under water level conditions of normal storage. The displacements in directions x and y of each point converged, with those of point A (gate chamber) showing the largest values. However, the displacements were not more than 1 cm. Therefore, the model finally achieved the equilibrium state, which indicates that the water level of section #28 is safe under normal working conditions.

(2) Strength reduction calculation
The strength of the fractures and weak interlayers in dam section #28 was reduced, as were \( c \) and \( \varphi \), and the results were compared with the displacement vector map and analyzed. Then, the influence of the strength reduction on the stability of the dam body and bedrock were obtained.

In the simulation of section #28, no obvious fluctuation in the maximum unbalance force curve was noted in the calculation process. The maximum unbalance force curve converged after the calculation, which means that the dam foundation remained stable even though the strength of the fractures and weak interlayers was reduced.

Figure 6 shows the displacement curves of the monitoring points when the strength of section #28 was reduced to different multiples. Because the displacement of monitoring point A was the largest, this point is taken as an example. When the strength of section #28 was not reduced, and the convergence displacement in direction x of monitoring point A was \( 9.0 \times 10^{-3} \) m toward the right, that in direction y was \( -2.0 \times 10^{-3} \) m vertically downward. As the strength of the fractures and weak interlayers was reduced step-by-step, the displacement in directions x and y uniformly increased. Finally, when the strength of the section was reduced to 0.1 times that of the previous value, the convergence displacement in direction x was \( 12.8 \times 10^{-3} \) m toward the right, and that in direction y was \( -3.5 \times 10^{-3} \) m vertically downward. The displacement in directions x and y of the other four monitoring points showed the same deformation trend as that of monitoring point A when the strength of the fractures and weak interlayers was reduced gradually.

The ultimate displacement of each monitoring point tended to be constant value and did not change significantly after strength reduction in the fractures and weak interlayers. The maximum displacement
was only 12.8 × 10⁻³ m under this condition. The plastic zone did not occur in the model; therefore, it can be concluded that the model is stable and will not be destroyed, which means section #28 is safe under the strength reduction condition. Therefore, the deformation failure mode of the dam foundation is compression failure rather than shear failure.

Figure 6. Displacement curves of the monitoring points when the strength of section #28 was reduced to different multiples

(3) Sensitivity analysis of deformation and failure of dam foundation

According to the above analysis, although the reduction in the strength of the fractures and weak interlayers affected the displacement of the model, the plastic zone did not occur. Therefore, the model generally remained stable, which means the model is intact. Therefore, to determine the main factors that affect the deformation and failure of the dam foundation and to analyze its deformation and failure mode, sensitivity analysis of the deformation and failure of the dam foundation need to be conducted by reducing other parameters. The rock mass conducts huge water pressure through the dam foundation and acts on the bedrock; therefore, the characteristics of the rock mass in addition to those of the discontinuities play a key role in the deformation and stability of the dam foundation. Because the rock mass is subjected to mainly compressive force, the shear strength parameters and elastic modulus have strong influences on the deformation and stability of the dam foundation. In this section, we focus on the effects of these parameters.

The shear strength and elastic modulus of the bedrock of section #28 were reduced. The displacement and plastic zone diagram were compared and analyzed, and the factors affecting the sensitivity of the dam foundation were evaluated.

Figure 7(a) presents a broken-line diagram of the displacement variation at each monitoring point as the shear strength of the rock is reduced to different multiples; c and φ were also reduced at the same time. Compared with that shown in Figure 3, when the rock strength was reduced to 0.4 times the previous value, the displacement of each monitoring point exhibited obvious increases with rapid deformation. In addition, the maximum unbalance force was in a state of fluctuation, as shown in Figure 7(b), which means that the entire model was unstable. Moreover, the 27 grid elements in the lower right corner of the sluice gate appeared to be in a compressive plastic zone, with the entire plastic zone exhibiting coalescence, as shown in Figure 7(c). In contrast to the reduction in the shear strength of the fractures and weak interlayers, the model was destroyed when the rock strength was reduced to 0.4 times the previous value. Therefore, the shear strength of the bedrock had a stronger influence on the stability of the dam foundation than that of fractures and weak interlayers.
(a) Broken-line diagram of the displacement variation at each monitoring point as the shear strength of the rock is reduced to different multiples

(b) Maximum unbalance force curve when the shear strength of the bedrock was reduced to 0.4 times the previous value

(c) Plastic yield zone occurring when the shear strength of bedrock was reduced to 0.4 times previous value

Figure 7. Displacement diagram of shear strength reduced to different multiples, the maximum unbalance force curve of the model at failure, and the yield zone diagram

The elastic modulus of section #28 was reduced, and the maximum unbalance force in the model was moderate and constant. No plastic zone occurred in the model; thus, the entire model remained still stable. Figure 8 presents a broken-line diagram of the displacement variation of each monitoring point as the elastic modulus of bedrock is reduced to different multiples. The displacement value of monitoring point A was the largest; therefore, monitoring point A was selected again as an example.

Compared with the shear strength reduction in the fractures and weak interlayers, when their shear strength is reduced to 0.1 times the previous value, the convergence displacement in direction x of monitoring point A was $12.8 \times 10^{-3}$ m toward the right, that in direction y was $-3.5 \times 10^{-3}$ m vertically...
downward. When the elastic modulus of the bedrock was reduced to 0.1 times the previous value, the direction-x convergence displacement of monitoring point A was $43.7 \times 10^3$ m toward the right, which is 3.41 times that of the fractures and weak interlayers and that in direction y was $-9.4 \times 10^3$ m, toward right, which is 2.69 times that of fractures and weak interlayers. In other words, the increment of displacement with the reduction in the elastic modulus of rock mass was obviously larger than that with the reduction in the fractures and weak interlayers under the same condition. Therefore, the elastic modulus of bedrock had a stronger influence on the deformation of the dam foundation than the shear strength of the soft layers and the structural plane. The shear strength parameters of the bedrock had a stronger influence on the stability of the dam foundation rock mass. In fact, only slight differences were noted in the displacement variation in the y direction between the two cases. However, that in direction x changed significantly, which also indicates that the failure mode of the model is compression failure rather than shear failure.

![Figure 8. Broken-line diagram of the displacement variation of each monitoring point as the elastic modulus of bedrock is reduced to different multiples](image)

4) Physical simulation verification

To verify the numerical simulation results, a three-dimensional geo-mechanical model test of the dam section of the sluice gate was conducted. In the model, all types of rock mass in the dam foundation as well as the unfavorable geological structures affecting the stability of the dam foundation were simulated, and the controlling discontinuities affecting the stability were weakened. The geo-mechanical model test is a nonlinear failure test that must meet similar requirements as that for the failure test, including geometric, stress–strain, shear strength, and load similarities. The model’s geometric scale was determined to be 100 according to engineering characteristics and those of section #28, the task requirements of the test, the scale of the test site, and the test accuracy requirements. The simulation scope of the three-dimensional model of the dam section was determined mainly according to the terrain characteristics of section #28, main geological structure characteristics of the dam foundation, and requirements of the test tasks. The final model size was $2.15 \times 0.303 \times 1.26$ m (longitudinal × transverse × height), which is equivalent to the scope of the actual project, at $215 \times 30.3 \times 126$ m. In addition, different rock types were simulated with different model materials that are not described here. The failure process, failure form, and failure mechanism of the section #28 foundation were obtained according to the comprehensive failure test results, and the failure mode of the dam foundation was revealed. The failure pattern of the gate dam is shown in Figure 9, which indicates that the dam foundation failure occurred mainly in the two toes of the gate dam. The fractures of the two toes were coalesced with the weak interlayers; however, the damaged discontinuities were generally not completely coalesced with the upper and lower reaches of the discontinuities, implying that the damaged plane is non-persistent. The physical simulation results further demonstrate that failure mode of the model is compression failure rather than shear failure.
Figure 9. Failure of bedrock in the left and right dam chambers of the sluice gate

Because of the fixed effect of the concrete cogging trenches in the upper and lower reaches, the rock stratum bears the pressure of the pier, which produces deformation and destruction of the compressive plastic zone. The failure area in the physical and numerical simulations was mainly the compression failure zone of the bedrock below and to the right of the two cogging trenches; deformation and destruction of the persistent discontinuities did not occur.

5. Conclusion
We conducted simulation analysis of section #28 of the Datengxia Hydropower Station. The results are summarized below.

1) The strength reduction analysis in the numerical simulation of section #28 was conducted. The shear strength parameter of the discontinuities was reduced to 0.1 times the previous value, and the deformation of the model was small. The failure mode of the dam foundation was determined to be compression failure rather than shear failure.

2) To identify the main factors affecting the deformation and failure of the dam foundation and analyze its deformation and failure mode, we conducted sensitivity analysis on section #28. The elastic modulus of the bedrock had a stronger influence on the deformation of the dam foundation than the shear strength of the fractures and weak interlayers, and the shear strength parameters of the bedrock had a greater influence on the stability of dam foundation rock mass. These experimental results also indicate compression failure as the failure mode.

3) The physical simulation results were similar to those obtained through numerical simulation; therefore, the conclusions obtained in the numerical simulation are reliable.

4) The failure area of physical and numerical simulations was mainly compression failure of the bedrock below and to the right of the two cogging trenches, and deformation and destruction of the persistent discontinuities did not occur. The rock mass at the surface of the lower reaches increased in elevation. In addition, the rock formations on both sides of the weak interlayers produced non-uniform deformation under high water pressure, which can be captured by the extension of the plastic zone of the rock mass and the crack deformation of the fractures and weak interlayers near the upper and lower reaches. Owing to the fixed effect of the concrete cogging trenches in the upper and lower reaches, the dam foundation bore the pressure of the pier, which produced deformation and destruction of the compressive plastic zone. Therefore, in the sensitivity analysis, the deformation and failure of the dam foundation was determined not only by the rock discontinuity system, which could cause shear failure in the dam foundation, but also by the shear strength and elastic modulus of the bedrock.

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