Analysis of the stability of the Xingguang bank slope in the Xiluodu Hydropower Station, China, under dry–wet cycles

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Abstract. The first impoundment of the Xiluodu Hydropower Station on the Jinsha River in Southwest China commenced in May 2013. A month after the first impoundment, the Xingguang bank slope began to deform and continues to do so to this day. This endangers the lives and properties of residents as well as the operation of the reservoir. The change in the shear strength index of the rock mass in the water level fluctuation zone under multiple saturation and drying conditions was studied using the dry–wet cycle test, and it was concluded that after 25 cycles, the shear strength index of the rock mass in the water level fluctuation zone of the bank slope decreased to the minimum. The cohesion and internal friction angle decreased by 29.77% and 20.31%, respectively. The Fast Lagrangian Analysis of Continua in three dimensions (FLAC3D) software was used to analyze the influence of water level change on bank slope stability. According to the simulation calculation, the stability coefficient of the bank slope decreased from 1.87 to 1.76 when the water level increased from 540 m to 600 m. However, the interior of the bank slope did not develop into a consistent shear strain increment concentration zone, indicating that an increase in the water level resulted in a decrease in the stability coefficient of the bank slope, but the bank slope remained stable as a whole. The results of the dry–wet cycle test were introduced into the numerical simulation. The results demonstrate that when the calculation time was more than 25 years, the maximum displacement of the extremely strong dumping area of the bank slope suddenly increased from 169 cm to 1691 cm, and the stability coefficient decreased to 1.11. According to the dry–wet cycle test and numerical simulation results, it can be determined that water storage in the reservoir area is a major cause of bank slope deformation and failure, and the bank slope may be unstable in the next 25 to 30 years. The bank slope may be destroyed in advance if the reservoir water level rises and falls frequently.

1. Introduction
Many large hydropower stations have been built to fully utilize the rich hydropower resources in Southwest China, causing significant changes in the original geological conditions of the bank slope of the reservoir area and resulting in serious deformation of many bank slopes, such as the left bank slope of the Jinping I Hydropower Station, the slope of the Xiaowan Hydropower Station, the right abutment slope of the Huangdeng Hydropower Station, and the Xingguang bank slope of the Xiluodu Hydropower Station [1-4]. If these large bank slopes are unstable and destroyed, they may produce high-speed surges, endangering the lives and
properties of residents as well as the normal operation of the reservoir. The stability analysis of bank slopes is mainly divided into two categories: qualitative analysis and quantitative analysis. The qualitative analysis mainly analyzes the slope stability and evolution trend based on basic geological conditions and other influencing factors, whereas the quantitative analysis mainly analyzes the slope stability through mathematical calculations, physical simulation tests, and numerical calculations, where the numerical calculation and physical simulation test have been widely used [5]. The coupling model of the seepage and stress fields of the Hefeng landslide is constructed using the Fast Lagrangian Analysis of Continua in three dimensions (FLAC3D) software, based on the groundwater flow equation and static equilibrium equation. According to the results, the decrease in the reservoir water level is the main reason for the aggravation of the Hefeng landslide deformation. [6]. The response of the deformation of the Muyubao landslide to the changes in the reservoir water level is explored using a centrifugal model test. The test results demonstrate that the buoyancy force controls the landslide deformation. The rock and soil masses of the bank slope are in a long-term dry–wet cycle of soaking and drying because of the periodic rise and fall of the reservoir water level. The mechanical parameters of rock masses in the fluctuation zone of bank slopes gradually decrease over time because of the long-term dry–wet cycle [7-8] [9]. The slip failure of the Huangnan north slope in the Three Gorges Reservoir has been studied using the discrete element method. The results reveal that the dry–wet cycle causes an irreversible attenuation of the mechanical strength of rock masses. Owing to the attenuation of the mechanical parameters of rock masses in the fluctuation zone of bank slopes, the bank may have variable degrees of deformation, which can lead to the occurrence of serious geological disasters [10]. The current research results reveal that reservoir water level has a significant effect on bank slope stability, which provides a suitable basis for rational analysis and evaluation of long-term bank slope deformation stability. However, little research has been done on the low shear strength of rock masses in the fluctuation zone of bank slopes under dry–wet cycle conditions. Since the impoundment of the Xiluodu reservoir area, the deformation range of the Xingguang bank slope has continued to expand, and the number of surface cracks has increased significantly. Since the implementation of professional monitoring in June 2014, the deformation has been continuous, posing a direct threat to 256 local villagers. Therefore, from the viewpoint of the attenuation of the shear strength of bank slope rock masses under dry–wet cycle conditions, the stability of the Xingguang bank slope in the Xiluodu reservoir area was evaluated using three-dimensional simulation numerical calculations, based on field investigation and laboratory tests. The results of this study can be used to provide technical and theoretical support for the treatment of the Xingguang bank slope.

2. Study area

The Xingguang bank slope is located on the right side of the Jinsha River in Xingguang Group 3 of Yongshan County, Yunnan Province, China, roughly 25 km from the Xiluodu Hydropower Station dam (Figure 1a). The bank slope has a longitudinal length of 1600 m, a transverse width of about 980 m, an elevation range of 420–1400 m, and a height difference of 980 m. Above 1360 m, the slope is flat, between 1000 m and 1360 m, the slope is 25°–30°, and below 1000 m, the slope is steep (40°–45°). The two sides of the bank slope are cut by gullies, revealing three empty faces (Figure 1b).

The deformation body has a complicated lithology. According to the field investigation and data analysis, the front of the bank slope is mainly composed of mudstone and argillaceous sandstone from the Hongshiya formation of the Ordovician system (O1h), and the lithology is soft. The middle and lower parts of the bank slope are mainly composed of hard rocks such as dolomite, limestone, and dolomitic limestone from the Erdaoshui formation of the Cambrian system (Є3E). The middle part of the bank slope is mainly composed of argillaceous siltstone and silt like mudstone from the Xiwangmiao formation of the Cambrian system (Є2X), and the lithology is soft. The middle and upper parts of the bank slope are mainly composed of hard rocks such as dolomite and limestone from the Cambrian system [4]. The dumping deformation of the Xingguang bank slope is divided into a strong dumping area,
an upper section of the strong dumping area, a lower section of the strong dumping area, a weak dumping area, and a normal bedrock, based on the deformation and failure characteristics of the bank slope, rock mass structure, development degree of cleavage, and change in the dip angle (Figure 1c).

![Figure 1](image1.png)

Figure 1. Engineering geological plan.

There were mainly five large-scale cracks in the Xingguang bank slope before the first impoundment, with a distribution crack elevation of 1085–1255 m. However, when the reservoir began storing water in May 2013, new cracks appeared on the bank slope surface as the scale of the earlier cracks continued to increase. By the end of 2018, the number of cracks had increased from 5 before impoundment to 11 (Table 1, Figure 2), indicating that reservoir...
Impoundment is a major cause of deformation aggravation in the Xingguang bank slope.

### Table 1. Crack statistics.

| Serial number | Crack extension length (m) | Crack width (cm) | Distributed elevation (m) |
|---------------|---------------------------|------------------|--------------------------|
|               | 6/2013                    | 8/2019           |                          |
| LF1           | 500                       | 540              | 5~20                     | 1255~1195 |
| LF2           | 150                       | 190              | 5~15                     | 1210~1180 |
| LF3           | 700                       | 770              | 10~20                    | 1225~1020 |
| LF4           | 230                       | 285              | 10~15                    | 1135~1090 |
| LF5           | 320                       | 330              | 5~10                     | 1085~1050 |
| LF6           | 135                       | 160              | 5~12                     | 980~890   |
| LF7           | 100                       | 485              | 5~8                      | 1385      |
| LF8           | 50                        | 510              | 2~5                      | 1370      |
| LF9           | 160                       | 205              | 5~10                     | 950~850   |
| LF10          | 200                       | 240              | 35~45                    | 890~845   |
| LF11          | 250                       | 265              | 15~35                    | 885~810   |

Figure 2. Typical cracks.

3. Dry–wet cycle test

The fluctuation zone of the Xingguang bank slope is mainly composed of sandstone. Therefore, the sandstone samples collected above the fluctuation zone of the bank slope were used to create seven sets of patterns according to the specifications. Each set was tested 0 times, 5 times, 10 times, 15 times, 20 times, 25 times, and 30 times for dry–wet cycle conditions. The sample was placed in a vertical tank, and water was added to a quarter of the sample height. After 2 h of soaking, the water reached the sample height. After 6 h, the sample was completely submerged in water (Figure 3a). After 48 h of soaking, the sample was placed in a drying oven, and the drying temperature was maintained at 60 °C (Figure 3b). After 48 h, the moisture content of the sample was determined to be about 0.1%, and the dry–wet cycle test was completed.

After the pattern had completed the required number of cycles, the shear strength index of the pattern was obtained via direct shear tests. Figure 4 depicts the test results. Table 1 shows the relevant results obtained by linear fitting the shear strength index of sandstone and changing the dry–wet cycle times (n). The shear strength index of the sandstone decreased as the dry–wet cycle time (n) increased but tended to stabilize when n > 25 times. The cohesion decreased by 29.77% and the internal friction angle decreased by 20.31% when the number of cycles (n) increased from 0 to 30.
4. Stability analysis of the deformed body under dry–wet cycles

4.1. Numerical calculation scheme

The fluctuation range of water level elevation in the Xiluodu reservoir area is about 540–600 m, the highest water level is 600 m, and the lowest water level is 540 m. Therefore, this numerical calculation primarily simulated the Xingguang bank slope in the following two aspects:

1. The pore water pressure distribution, displacement field, and maximum shear strain increment of the Xingguang bank slope were calculated when the lowest water level was 540 m and the highest water level was 600 m. The deformation and stability of the bank slope were analyzed when the lowest and highest reservoir water levels were 540 m and 600 m, respectively.

2. The strength deterioration law of the rock in the fluctuation zone under dry–wet cycle conditions was introduced into the numerical calculation, and the deformation and stability of the bank slope under dry–wet cycle conditions were analyzed.

4.2. Model establishment and calculation parameter selection

As shown in Figure 5, the bank slope model is divided into four deformation areas in the plane: intensified strong toppling deformation zone, strong toppling deformation zone, moderate toppling deformation zone, and slight toppling deformation zone. The model
contains 61,104 nodes and 99,017 elements.

![Figure 5. Calculation model.](image)

4.2.1. Imposition of boundary conditions
When setting the mechanical boundary conditions, the normal constraint was adopted around the bank slope model, and the fixed hinge support was used at the bottom. When setting the seepage boundary conditions, the impervious boundary was adopted around the bank slope model and the bottom, and the slope surface was the permeable boundary.

In the establishment of interface elements on the model surface, the FISH was used to lock the water level rise and fall areas of the interface elements. Thereafter, the node information of this part of the interface element was extracted, and the node information in the complex three-dimensional (3D) model was further locked through the node information of the interface elements. Finally, these nodes used the “apply pp” command to apply the water-head boundary and simulate the rise and fall in the water level of the reservoir area and the hydrostatic pressure on the surface of the water level rise and fall area.

4.2.2. Calculation parameters
The Mohr–Coulomb criterion was used to determine the strain, and the strength reduction method was used to calculate the stability of the bank slope. The mechanical parameters of the rock and soil mass of the calculation model were comprehensively determined using indoor rock and soil tests and related engineering experience analogies, based on the deformation degree of the slope’s rock mass. Table 2 lists the parameters in detail.

| Rock Mass                        | Density (kg/m³) | Bulk modulus (GPa) | Shear modulus (GPa) | Cohesion (MPa) | Friction angle (Deg) | Tensile strength (MPa) |
|----------------------------------|-----------------|--------------------|---------------------|----------------|----------------------|-------------------------|
| Intensified strong toppling zone| 2340            | 1.45               | 1.33                | 2.19           | 23                   | 0.223                   |
| Strong toppling zone            | 2460            | 2.25               | 1.65                | 3.32           | 25                   | 0.372                   |
| Moderately toppling zone        | 2460            | 2.45               | 2.75                | 4.32           | 28                   | 0.482                   |
| Slightly toppling zone          | 2590            | 7.39               | 6.68                | 7.54           | 31                   | 0.691                   |
| Mother rock                     | 2590            | 8.39               | 9.68                | 9.68           | 35                   | 0.881                   |
According to the water level data of the reservoir area, the water level in the reservoir area rises and falls about once a year; therefore, the time was taken as 0a, 5a, 10a, 15a, 20a, 25a, and 30a for calculation, and the dry–wet cycle times of the rock mass in the corresponding fluctuation zone were 0, 5, 10, 15, 20, 25, and 30, respectively. The deterioration law of the rock mass in the fluctuation zone under working cycle conditions was introduced into the calculation using the strength reduction method and FISH.

4.3. Analysis of the calculation results

4.3.1. Stability analysis under a stable water level

Figures 6a and 6b show the pore water pressure distribution of the stable seepage field with reservoir water levels of 540 m and 600 m. The pore water pressure increased gradually from top to bottom, the flooded part is the fluctuation zone, and the blue part is the unsubmerged area.

Figure 6. Pore water pressure distribution of the bank slope. a) Pore water pressure distribution of the bank slope at the 540 m water level and b) pore water pressure distribution of the bank slope at the 540 m water level

Figure 7 shows that when the water level of the reservoir is 540 m, the displacement is primarily distributed in the extremely strong dumping area of the slope, and the maximum displacement is about 34.2 cm. However, when the reservoir water level is 600 m, the displacement range of the slope increases, from the extremely strong dumping area to the normal bedrock, and the maximum displacement increases from 34.2 cm to 76.7 cm. This is because as the flooding area of the bank slope increases, the strength of the flooded rock mass decreases due to water softening. However, changes in the seepage field cause changes in the stress field, which further affects the displacement field of the slope.

Figure 7. Displacement contours of the calculation model at the 540 m water level. a) Displacement contours of the calculation model at the 540 m water level and b) displacement contours of the calculation model at the 600 m water level

Figure 8a shows that when the reservoir water level is 540 m, there is no shear strain increment concentration on the bank slope, and the stability coefficient of the bank slope is 1.87, indicating good overall stability. When the reservoir water level is 600 m (Figure 8b), there is a noticeable shear stress increment zone below the water level of the bank slope, but no continuous shear strain increment zone is formed at this time. The bank slope has a stability coefficient of 1.76, indicating that it remained stable. When the water level in the reservoir area is 540 m, the slope as a whole is stable; however, when the reservoir water level increases to 600 m, the slope stability exhibits a downward trend, but the slope as a whole is
still stable.

Figure 8. Contours of the maximum shear strain increment. a) Contour of the maximum shear strain increment at the 540 m water level and b) contour of the maximum shear strain increment at the 600 m water level.

4.3.2. Stability analysis of the bank slope under dry–wet cycle conditions

Figure 9 shows the displacement cloud map of the bank slope for different dry–wet cycles. When the number of dry–wet cycles is \( n = 5, 10, 15, \) and \( 20 \), the maximum displacements of the bank slope are 81.8 cm, 100 cm, 138 cm, and 169 cm. When the number of dry–wet cycles is \( n = 25 \), the maximum displacement of bank slope is about 169 cm, and the deformation zone gradually converges to the extremely strong dumping area. When the number of dry–wet cycles is \( n = 30 \), the maximum displacement of the bank slope increases from 169 cm to 2551 cm, and the deformation zone is concentrated in the extremely strong dumping area.

As the number of dry–wet cycles increases, the maximum displacement of the bank slope gradually increases, and the displacement gradually converges to the extremely strong dumping area. When the number of dry–wet cycles is \( n = 25 \), the displacement of the bank slope changes abruptly, and the deformation is concentrated in the extremely strong dumping area, and the maximum displacement increases abruptly from 169 cm to 1691 cm. It can be estimated that when the number of dry–wet cycles is \( n > 25 \), there is a possibility of instability and failure in the extremely strong dumping area of the bank slope [11].
As shown in Figure 10, the maximum shear strain increment is mainly concentrated in the extremely strong dumping area. As the number of dry–wet cycles increases, the shear strain increment expands backward, and the slope stability coefficient decreases (Figure 11). When the number of dry–wet cycles is \( n = 25 \), the maximum shear stress increment zone is basically concentrated in the extremely strong dumping area of the bank slope, and when the number of dry–wet cycles is \( n = 30 \), the shear strain increment of the bank slope is completely concentrated in the extremely strong dumping area. As a result, the stability coefficient of the bank slope decreased to 1.11.

**Figure 9.** Displacement contours of the calculation model. a) \( n = 5 \), b) \( n = 10 \), c) \( n = 15 \), d) \( n = 20 \), e) \( n = 25 \), and f) \( n = 30 \).

**Figure 10.** Shear strain increment contours of the calculation model. a) \( n = 5 \), b) \( n = 10 \), c) \( n = 15 \), d) \( n = 20 \), e) \( n = 25 \), and f) \( n = 30 \).
5. Conclusion
The following conclusions are drawn based on the field investigation, dry–wet cycle test, and FLAC3D numerical calculation results:

1. After the impoundment of the reservoir, the number of large-scale cracks on the slope surface increased from 5 to 13, and the scale of the cracks before impoundment increased significantly, indicating that slope deformation and development are closely related to reservoir impoundment.

2. The dry–wet cycle test revealed that, under dry–wet cycle conditions, the strength of rock mass in the fluctuation zone of the bank slope deteriorated noticeably. The relationship between the cohesion (C) and the number of dry–wet cycles (n) is \( C = 14.14 - 1.131 \ln(n + 1) \), and the relationship between the internal friction angle (φ) and the number of dry–wet cycles (n) is \( \phi = 47.62(0.29e^{-0.045n} + 0.72) \).

3. When the reservoir water level is stable at 540 m and 600 m, the bank slope as a whole is stable. As the number of dry–wet cycles increases, the maximum displacement of the bank slope gradually increases, the plastic zone continues to expand backward, and the displacement gradually converges to the extremely strong dumping area. There is a possibility that the bank slope will become unstable and fail in the next 25–30 years. If the reservoir water level rises and falls frequently, the failure time of the bank slope may advance.

4. In this study, only the effects of reservoir water on bank slope stability were considered, but the effects of earthquakes and rainfall were not considered. In this region, there have been 22 destructive earthquakes in the last 70 years, with six of them exceeding magnitude 8. Moreover, earthquakes have a significant effect on bank slope stability. The influence of seismic action on the stability of the Xingguang bank slope will be considered in a follow-up study to further improve the prediction of the stability of the Xingguang bank slope.

5. Currently, the common treatment measures for the slope in the reservoir area are a large number of anti-slide piles, slope cutting, and drainage, but the fluctuation zone area is rarely treated; thus, it is recommended that the rock mass of the fluctuation zone be strengthened by grouting and anchoring.

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