Research Article

Influence of Wind-Generated Wave Action on Mountain Reservoir Bank Collapse: A Case Study at the Lancang River, Western China

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Slopes facing reservoir water often collapse, affecting reservoir capacity. It is important to understand the factors affecting such collapses and predict the locations of collapse after impoundment for practical purposes. However, the mechanism of bank collapse in mountainous areas after reservoir impoundment remains unclear. To address this issue, field geotechnical property surveys, laboratory tests, and numerical simulations were employed to evaluate bank collapse susceptibility. The results showed that (i) bank collapse mainly occurs in slopes with a small cohesion and internal friction angle of less than 30°; (ii) the main effects of wave action on bank collapse are scour, erosion, and cyclic impact on the bank slope: wave erosion occurs in the bank slope with a wave height of less than 1.0 m, cohesion of less than 90 kPa, and slope angle of less than 30°, and wave impaction mainly occurs at the area with a wave height of more than 2.0 m and a slope angle of more than 30°; (iii) for valley reservoirs, softening of water on the slope rock and soil mainly reduces cohesion, which commonly occurs within 30 days; the cyclic action of wave impact load mainly damages the structure and reduces the internal friction angle. The influence of waves on bank collapse is mainly related to the wave height and wave velocity. The antiwave impact broken energy of rock and soil is an important index to evaluate whether bank collapse occurs in the long term, which is related to the strength, structure, and impact resistance. The present research is helpful to better understand the mechanism and evolution process of reservoir bank collapse.

1. Introduction

Reservoir bank collapse is one of the most important geological disasters, affecting the security of immigration engineering in the reservoir areas and reservoir capacity [1–4]. For example, on 3 October 2001, a reservoir collapse in Sichuan Province in southwest China caused 12 deaths, with 13 people missing and 12 injured. The wave induced by the “720” bank collapse of Xiaowan reservoir in the Lancang River also led to 13 people missing. On 11 August 2012, a 180,000 m\(^2\) reservoir collapsed in Zhoushan, Zhejiang province, killing at least 10 people and injuring 27 others. Therefore, accurately predicting reservoir bank collapse is a key issue in the construction of water conservation projects. However, it is difficult to quantitatively describe the effect of geological and hydrological factors on bank collapse because of the complex mechanism of such a collapse [5, 6]. Intensive investigation or modelling of bank collapse predictions has not been performed. The mechanism of bank collapse must be studied to analyse the sections in which bank collapse may occur, and it is necessary to use the corresponding analysis results to predict the scope of collapse.
Since reservoir bank collapse was first proposed by Savarenski in 1935, such research has become the main work to be performed before the impoundment of large reservoirs. Until now, most research has focused on plain area reservoirs or rivers [7, 8]. Through geological investigation and property analysis of bank collapse sites, the main factors affecting bank collapse have been found to be hydrological, geological, geomorphological, and groundwater level changes on bank slopes [9–12]. In recent years, experimental methods and numerical simulation methods have been applied to study the stability of slope under the rapid change of reservoir water level or rainfall. Based on centrifuge tests on rainfall-induced instabilities in variably saturated slopes, Wang et al. indicated that rainfall-induced slope failures usually follow the slide-to-flow or flowslide failure modes [13]. Song et al. have investigated the influence of rapid water drawdown on the seismic response characteristics of a reservoir rock slope by using numerical dynamic analyses and shaking table tests [14]. Biniyaz et al. investigated the influence of the water level fluctuation on the stability of soil slopes using coupled seepage and slope stability analysis [15]. Li et al. found DEM can explain rock mass deformation and is suitable for numerical stability analysis [16].

In contrast to the plain-type reservoir, whose bank slope is mainly composed of alluvial and proluvial soil, the bank slope of the river-type reservoir is mainly composed of weathered residual and strong unloading rock mass owing to rapid river cutting and significant stress release. The bank slopes are unsaturated, the groundwater level in the bank slope is deep, and the influence of seepage during reservoir impoundment on the bank collapse is not obvious. Large amounts of observation data show that bank collapse of large reservoirs is a long-term changing process, and it first occurs near the water-level fluctuation zone [9–12]. The effect of waves on bank collapse is mainly reflected in the impact of periodic wave loads and erosion on the bank slope. Wave loading on the bank slope is related to the wave type, wave height, wave length, wave period, and bottom shear stress [17–19]. Although significant progress has been made on bank collapse in plain-type reservoirs, very few investigations have examined the mechanism of bank collapse in river-type reservoirs in mountainous areas. Existing research has mainly focused on the genetic types and influencing factors [11, 12, 18–20]. Some achievements have been made in terms of factors such as rainfall and reservoir water-level change, slope geotechnical structure, and topography of bank collapse [21]. The wave effect is one of the main factors of bank collapse in river-type reservoirs in mountainous areas. However, the wave action is only attributed to the erosion of the loose bank slope. No further quantitative studies have focused on the relationship between bank collapse and saturation or wave impaction. Most of them have suggested prediction models for the scale of bank collapse, and there are no relevant reports on the prediction of bank collapse occurrence time and possible location by considering the strength change rate of rock and soil. The mechanism of the wave-induced reservoir bank collapse has not been extensively understood. Although most studies propose prediction models for the scale of bank collapse, an accurate evaluation of the possible location and time of bank collapse in the process of bank storage has not been proposed.

According to the different geological structures of the bank slope, the river bank can be divided into hard rock bank slopes and loose soil bank slopes. Corresponding to the type of bank slope, waves can be divided into breaking waves and nonbreaking waves [22]. The waves in mountainous reservoirs are generated mainly by the action of wind. The wave parameters are related to the wind speed, fetch, and water depth at the bank slope. However, the relationship is not a simple function and must be corrected using actual monitoring data [23, 24]. By comparing the predicted and measured waves, some improvements in wave prediction methods have been proposed, and the nonbreaking or breaking wave run-up was determined more accurately for ocean areas [25, 26]. There are few corresponding prediction models for wind-induced waves in mountainous river reservoirs.

In the present study, the mechanism of bank collapse caused by saturation and wave impaction of river-type reservoirs in mountainous areas was investigated. The main objectives of the present research are as follows: (i) to develop a damage law of rock and soil under saturation and wind-generated wave dynamic action, (ii) to explain the mechanism of bank collapse with different lithologies, and (iii) to propose a collapse prediction model of critical wave load. This study contributes to a more comprehensive understanding of the bank collapse mechanism in mountainous reservoirs.

2. Research Site Descriptions

The whole basin of the Lancang River (including the Mekong River) is located at 94° to 107°E and 10° to 34°N. The study area investigated is located in the middle and lower reaches of the Lancang River, wherein the Huangdeng and Miaowei reservoirs are located in the middle reaches. It is a high mountain gorge area, and the main valley is a typical “V” shaped valley. The Xiaowan and Nuozhadu reservoirs are located in the lower reaches of the river. The relative elevation difference of high mountains and gorges ranges from 3500 m to 1000 m.

Three field wave-induced collapse monitoring reservoirs were first arranged in 2008 in the Lancang River, Southwest China (Figure 1). One site is the Nuozhadu hydropower station, the normal water level for this site is 812 m, and the total reservoir capacity is 227.41 × 10⁶ m³. The length of the dam commissioning reservoir was 240 km. Reservoir storage began on 20 April 2012. During these years, more than 98 collapses were observed and monitored in accordance with the reservoir storage stages, which corresponded to different slope materials and wave strengths. The second research field site was the Xiaowan Hydropower Station. The total length of the reservoir bank was 587 km (295 km and 292 km on the right and left banks, respectively). The normal reservoir water level was 1240 m, and the total reservoir capacity was 150 × 10⁶ m³. The third research field site is the Miaowei hydropower station reservoir, with a length of approximately 62 km, a normal storage level of 1408 m,
and a corresponding storage capacity of $6.60 \times 10^8 \text{m}^3$. In this case, the highly weathered rock mass with a thickness ranging from 2 to 20 m on the bank slope suffered toppling failure. After reservoir impoundment to 1401 m, bank collapse occurred near the water level on the side of the local highway, critically affecting highway safety along the river.

3. Research Methods

3.1. Field Wave Element Monitoring and Geological Survey. Field monitoring includes studying wave parameters and collapse properties. The geological conditions of 233 collapse sites in three typical reservoirs during different impoundment stages were surveyed.

To monitor the wave parameters, a Mag-310 wave sensor and manual observations were used in typical bank collapse areas of the Nuozhadu, Miaowei, and Xiaowan reservoirs. The Mag-310 wave sensor is a high-precision sensor based on MEMS, which can output wave height, wave period, wave direction, and other information through COM1, and output original MEMS data through com2. It can monitor wave heights from 1 m to 20 m and wave periods from 2 s to 25 s. It includes 16 direction waves. The original MEMS data were further analysed using MATLAB software to obtain the time series of the wave height. For key bank collapse points, a high-resolution digital camera, a handy wind direction meter, and a wind dynamometer were arranged on a boat to monitor the wave image at a given section. Using the wave image observed in the field, the wave elements were determined using the Ulead GIF Animator graphics processing software.

Then, the action of waves on bank slopes was determined for soil or fully weathered rock bank slopes and for the intake rock slope [27–29]. The cyclic action of the wave impact total energy on the bank slope can be expressed as a product of one impact force and the number of wave actions (Eq. (1)).

$$E = N \cdot F_w \cdot L,$$

where $N$ is the wave number, $L$ is the wave length (m), and $F_w$ is the one-wave impact force acting on the bank slope.

3.2. Wave Scour Rate Measurement of Loose Slope. Four sites were selected in the Nuozhadu reservoir to measure the scouring rate under different wave conditions. Terrain data were obtained through a 3D laser topographic survey. The specific methods were as follows: taking the slope toe line at the static water surface as the starting line, the measuring height ($y$) was 3 m, one measuring point was arranged every 1 cm on the measuring line, and one survey line was arranged every 2 cm along the $X$-direction. Measured point data before and after scour were added to the ArcMap layer. The Kriging method in the spatial analysis toolbox was selected to interpolate the point data to obtain the terrain grid file, and then, the surface volume of the processed terrain grid file was calculated. The observation time was 15 minutes.

3.3. Sampling and Laboratory Tests of Bank Material

3.3.1. Sampling Sites. Representative rock and soil samples were selected from the banks of three research reservoirs, Miaowei, Xiaowan, and Nuozhadu. The compressive strength and impact tests of the rock and shear strength of the residual soil were carried out after different saturation times. The sampling locations and methods used are listed in Table 1.

3.3.2. Laboratory Testing. The uniaxial compressive strength of the rock at different saturation times was measured using a YDS-2 multifunction test instrument. The maximum output of the vertical jack was 500 kN, the maximum displacement was 60 mm, and the piston area was 122.7 cm$^2$. The sampling size was 50 mm × 50 mm × 100 mm, and the loading direction was perpendicular to the bedding plane.

A modified impact instrument was used for the rock saturated impact test. The ZS-100 vertical drilling and coring machine, a DQ-4 automatic rock-cutting machine, and SHM–200 double-face grinding machine were used for coring, cutting, and two face grinding and polishing, respectively. The sample was processed into a standard specimen with a processing size of 50 mm × 25 mm. During the test, the drop hammer was lifted to a certain height and then allowed to fall freely. When the initial crack occurred, the impact number was recorded as the initial crack impact.
number $N_1$, and the initial crack resistance energy rocks were calculated.

The shear strength of the residual slope soil, weathered soil, and other gravel soil was obtained by direct shear tests after different wave impaction times.

### 3.4. Numerical Simulation

Recently, numerical simulation methods have been widely applied to predict the rate and scope of bank collapse under the action of waves [30]. The discrete element method is commonly used to simulate the wave loads acting on the bank slope [31], as proposed by Cundall [32]. Based on the structural characteristics of the rock mass at the bank collapse site and wave elements during failure, three models were developed to explore the influence mechanism of the wave cyclic load on bank collapse. When the wave dynamic analysis was carried out in the UDEC software, the bottom of the model was set as a viscous boundary, and the other boundary of the model was a free-field boundary. The stress and deformation values of each element or node at each step can be calculated using an iterative solution, and then, the entire process of slope deformation and failure can be simulated. In the calculation steps, the wave load was added at the slope surface as a dynamic load according to the wave period and action time.

### 4. Results Analysis

#### 4.1. Factors Affecting Bank Collapse of Reservoirs in Mountainous Areas

The bank collapse of the three reservoirs is related to the lithology of the bank slope and the intensity of wave action. The three reservoirs investigated here (Miaowei, Xiaowan, and Nuozhadu) are located in the middle and lower reaches of the Lancang River, from upstream to downstream. In the reservoir area of the Miaowei hydropower station, 90% of the bank collapses occurred in the range of 10–20 km from the dam (15% of the total length of the reservoir), and 90% were distributed in the residual soil; strong toppling was observed where strong wave action of the total 33 collapses occurred. When the water storage of the Xiaowan reservoir reaches 1160 m, the total length of the reservoir is approximately 100 km, 103 bank collapses occurred with a height of more than 5 m, 38.3% of them were developed in colluvial soil bank slopes, 35.6% in fully weathered rock bank slopes, and 13.7% in strongly weathered rock bank slopes. Most of them were within the range of 15–50 km from the dam, where the width of the water surface was large, the wind fetch is large, and the wave action is strong. When the water storage of the Nuozhadu reservoir reaches 812 m, the total length of the reservoir is approximately 200 km, 20 km away from the dam. The shear strength of the residual slope soil, weathered soil, and other gravel soil was obtained by direct shear tests after different wave impaction times.

#### Table 1: Sampling sites

| Reservoir | Sampling location | Lithology | Sample number | Test type |
|-----------|-------------------|-----------|---------------|-----------|
| Miaowei   | Right             | Slate     | M01-1~M01-14  | Compression test |
|           | Left              | Metamorphic quartz sandstone | M02-1~M02-14 | Compression test |
| Nuozhadu  | Right             | Granite   | N01-1~N01-14  | Compression test |
|           | Right             | Gravelly sandstone | N02-1~N02-14 | Compression test |
| Xiaowan   | Right             | Silty mudstone | X01-1~X01-14 | Compression test |
| Nuozhadu  | Right             | Granite   | N11-1         | Impact test |
|           | Right             | Sandstone | N13-1         | Impact test |
|           | Right             | Granite gneiss | N15-1       | Impact test |

| Reservoir | Sampling location | Lithology | Sample number | Test type |
|-----------|-------------------|-----------|---------------|-----------|
| Miaowei   | Left              | Metamorphic quartz sandstone | Left (27 samples) | Completely weathered granite | NL02-1~NL02-12(A4) |
|           | Right             | Slate     | Right (57 samples) | Completely weathered sandstone | NL03-1~NL03-15(B5) |
|           |                   |           |               | Completely weathered granite | NL04-1~NL01-18(B6) |
|           |                   |           |               | Completely weathered mudstone | NL06-1~NL01-12(A4) |
|           |                   |           |               | Completely weathered sandstone | NL01-7~NL01-12(A4) |
| Nuozhadu  | (23)              |           |               | Q^{col-dl} | MR01-1~MR01-12(A4) |
|           |                   |           |               |             | MR03-1~MR03-18(B6) |
|           |                   |           |               | Completely weathered slate | MR04-1~MR04-12(A4) |
|           |                   |           |               |               | MR05-1~MR05-15(B5) |
| Miaowei   | (19 groups)       |           |               | Q^{col-dl} | X01-1~X01-12(A4) |
|           |                   |           |               |             | XR02-1~XR02-12(A4) |
| Xiaowan   | (8 groups)        | Silty clay | Right (24 samples) | 1 group in natural state; | 4 groups of saturated direct shear test, 6 groups of direct shear after wave impact |
|           |                   | Silt      |               | 3 groups of direct shear after saturated impact |
from the dam, mainly distributed in sandstone, with only one bank collapse. Within the range of 20–50 km away from the dam, the lithology of the bank slope is mainly fully weathered granite and eluvial slope, and the number of bank collapses in this area accounts for 50% of the total 97 collapses. The average wave height in this area was approximately 2.5 m, the wave period was 2 s, and the wave impact was strong.

The volume difference before and after scouring is the accumulated scouring sediment volume during monitoring, which is the scouring rate (see Table 2). According to the wave scour rate measurement of loose slopes, the scouring effect of waves on the bank slope shows that the higher the silt content, the greater the scouring rate, while the percentage of coarse sand increases, and the scouring rate decreases. The influence of velocity on the scouring rate is very obvious, and the scouring rate increases gradually with an increase in velocity. The influence degree of scour rate from high to low is the flow direction, velocity, percentage content of 0.5-1 mm particles, density, and percentage content of 0.075-0.25 mm particles. Because the bank of the Nuozhadu reservoir is mainly composed of strongly weathered granite with high density, the wave-scouring effect is not obvious. This means that wave scouring is not the main factor of bank collapse in mountainous reservoirs.

Ten typical bank collapses caused by wave cycle impact have the same characteristics as the logarithmic helix shape in the Nuozhadu reservoir (Figure 2(a)). Figure 2(a) shows that within 5 m above the stable water level, the bank collapse surface is anticoncave, which is consistent with the height of wave action. A good correlation exists between the wave height and the bank collapse scale (Figure 2(b)). The statistical results (Table 3) of the Miaowei reservoir show that there is a good correlation between the water depth and wave height, with a correlation coefficient of 0.944. Most of the collapses occur at a water depth of more than 50 m in the Miaowei reservoir. This indicates that saturation and wave action are the main factors leading to bank collapse.

Field investigations show that bank collapse has the following characteristics: (1) bank collapse mainly occurs within 30 days after the stable water level is reached at the section where the strength of rock and soil changes significantly after saturation. (2) The bank collapse was located in the area where the water surface was wide, the water was deep, and the wave action was strong. Generally, the average wave height is more than 2.0 m and the wave period is 2–5 s. (3) The bank collapse sites all have groove shapes near the wave action zone caused by wave erosion and impact. Wave scouring is not the main cause of collapse.

### 4.2. Strength Degrading under Saturation Time

#### 4.2.1. Uniaxial Saturated Compressive Strength with Different Saturation Time

Slate, granite, metamorphic quartz sandstone, silty mudstone, and sandstone were selected for the compressive strength test under different saturation times. The saturation time was arranged as 0.1, 6, 12, 24, 36, 48, 84, 168, 252, 336, 432, and 588672 h. The test results show that the rate of change in compressive strength after 28 days (672 h) was very small (Figure 3). After 168 h (7 d) of saturation, the strengths of metamorphic quartz sandstone, slate, and silty mudstone were 43%, 35.5%, and 37.9% of the natural compressive strength, respectively. Strongly weathered granite and gravelly sandstone were 52.2% and 64.3%, respectively. The rate of change of the rock uniaxial compressive strength under saturation time is mainly affected by the rock mass strength and structure. During the first 144 h of saturation, the compressive strength decreased rapidly. The change law of strength is similar for hard and soft rock, which shows a logarithmic function with natural saturation time by regression analysis (Eq. (2)):

\[
R_c(t) = (\frac{-2.247lnlnR_{0s} + 5.4247}{ln(t) + 0.849R_{0s} + 3.3884})
\]

where \(R_c(t)\) is the uniaxial compressive strength under saturation time (MPa), \(R_{0s}\) is the initial uniaxial compressive strength (MPa), and \(t\) is the saturation time under natural conditions (h).

#### 4.2.2. Results of Direct Shear Test of Saturated Loose Bank Slope

As can be seen from Table 4, cohesion changes faster in different saturated states. The internal friction angle changes slowly, and the soil saturation weakens the cohesive strength between particles.

### Table 2: Scouring rate measurement results.

| Testing sites | 0.075-0.25 mm particle content (%) | 0.5-1 mm particle content (%) | Density (g/cm³) | Average wave velocity (cm/s) | Average scour rate (cm²/cm²·min) |
|---------------|----------------------------------|-------------------------------|----------------|-------------------------------|----------------------------------|
| No. 1         | 65                               | 0                             | 1.45           | 15                            | 0.0241                           |
|               | 65                               | 10                            | 1.65           | 25                            | 0.0959                           |
| No. 2         | 70                               | 0                             | 1.55           | 30                            | 0.0926                           |
|               | 70                               | 10                            | 1.75           | 20                            | 0.0747                           |
| No. 3         | 75                               | 5                             | 1.75           | 15                            | 0.0067                           |
|               | 75                               | 15                            | 1.55           | 25                            | 0.1281                           |
| No. 4         | 80                               | 5                             | 1.65           | 30                            | 0.1772                           |
|               | 80                               | 15                            | 1.45           | 20                            | 0.0769                           |
4.3. Strength Deterioration under the Coupling Action of Wave and Saturation

4.3.1. Saturated Rock Impact Test Results. According to the wave monitoring results (see Section 3.1), three types of impact energy were used to simulate the wave action in the test. The samples were saturated for 21 days before the impact test. The impact hammer is 0.5 kg; the impact distances are 0.5 m, 0.3 m, and 0.1 m; and the corresponding impact energies are 2.5, 1.5, and 0.5 J, respectively. The impact frequency was set to 5 s. The initial crack energy of the rock was calculated according to Equation (3). The appearance of a crack is the mark to judge the initial crack, and the rock impact test results are listed in Table 5.

\[ W = N_1 mgh, \]  

(3)

where \( W \) is the initial crack energy of impact crushing, \( N_1 \) is the impact time, \( m \) is the impact hammer mass, \( h \) is the impact height, and \( g \) is the gravitational acceleration.

As shown in Figure 4, the total energy of the initial broken increases with the strength impact energy ratio. Using the ratio \( E_0 \) of compressive strength \( R_c \) to the initial impact energy \( J \) (named as strength impact energy ratio), the relationship between rock types and the initial strength impact energy ratio can be established as Equation (4), with a correlation coefficient of 0.90. This can be used to judge the antiwave impact energy of slope rocks under wave cycle impaction.

\[ E_0 = 782.9 \left( \frac{R_c}{J} \right)^{0.3312}. \]  

(4)

Under the condition of low impact energy, the damage variable of granite increases slowly with an increase in the cyclic impaction number. However, when the single impact energy increases to a certain extent, the damage variable increases rapidly with an increase in the number of cyclic impaction. The cumulative damage of sandstone increases with an increase in cyclic loading times, and the impact damage effect is cumulative. That is, the damage degree of the first time is the largest, and those of the last two times gradually decrease. Under different impact loads, the failure characteristics of the slate after impact were microjoints and small cracks. Most of the joints and fissures intersect with each other. Under cyclic loading, the failure of the rock mainly started from the sample end. With an increase in the number of cyclic impacts, the specimen absorbs more energy. This promotes the increase in microcrack nuclei, expansion, merging, and the formation of macrocracks. When the cumulative damage increases to the macro level, the stiffness of the rock decreases, and the compressive strength decreases owing to the large number of macrocracks. When the damage reaches the macro level, the sandstone can be broken with less energy, and the failure end will be crushed owing to the increase in crack propagation.

4.3.2. Strength Deterioration of Loose Slope under the Coupling Action of Wave and Saturation

Samples were taken from the bank slope above the wave action zone in the natural state and were saturated after 30, 60, and 90 days of wave action. The wave energy is obtained by field measurements and statistics, and the load of a single wave acting on the bank slope is referred to in the literature [29]. The test results are presented in Table 6.

Table 6 indicates that for the highly weathered rock mass and accumulated soil, the internal friction angle reduction
factor  is approximately 0.70, for a duration of 30 days under saturation and wave action. When the initial internal friction angle of the soil was greater than 30°, the strength change under wave action was slow (Figure 5(a)). The internal friction angle of the soil is proportional to the antiwave-impact broken energy (Figure 5(b)). The reduction rate of

Table 3: Main collapse under wave action in the Miaowei reservoir.

| SWL (m) | Collapse no. | Depth of water (m) | Max wave height (m) | Height above SWL (m) | Horizontal depth (m) | Angle (°) | Rock character |
|---------|--------------|--------------------|---------------------|---------------------|----------------------|-----------|----------------|
| 1364    | L006         | 45.0               | 2.2                 | 30.0                | 3.0                  | 84.0      | Toppling and highly weathered sandstone and slate |
|         | L013         | 42.5               | 1.8                 | 4.0                 | 0.5                   | 82.9      | Toppling and highly weathered sandstone and slate |
|         | L014         | 40.3               | 1.8                 | 5.0                 | 0.5                   | 84.2      | Colluvium |
|         | R004         | 49.0               | 2.4                 | 5.0                 | 1.0                   | 78.7      | Colluvium |
|         | R011         | 47.0               | 2.0                 | 6.0                 | 1.0                   | 78.0      | Toppling and highly weathered sandstone and slate |
|         | QD14 (first) | 50.0               | 2.7                 | 137.0               | 5.0                   | 81.0      | Colluvium |
| 1401    | K84 + 980    | 60.0               | 3.0                 | 8.0                 | 1.0                   | 82.9      | Colluvium |
|         | K86 + 900    | 59.0               | 3.0                 | 30.0                | 3.0                   | 84.2      | Toppling and highly weathered sandstone and slate |
|         | K89 + 900    | 55.0               | 2.9                 | 20.0                | 5.0                   | 75.9      | Toppling and highly weathered sandstone and slate |
|         | K96 + 700    | 39.0               | 1.7                 | 50                  | 2.0                   | 85.4      | Colluvium |
|         | K71 + 400    | 87.0               | 3.5                 | 140.0               | 5.0                   | 87.9      | Colluvium |
|         | L006         | 82.0               | 3.2                 | 66.0                | 10.0                  | 81.4      | Toppling and highly weathered sandstone and slate |
|         | QD14 (second)| 87.0               | 5.1                 | 102.0               | 9.0                   | 80.0      | Colluvium |

Table 4: Shear test results of loose soil with different saturation times (22 groups).

| Lithology                     | Parameter     | Unsaturated | 1 day | 3 days | 7 days | 14 days | 28 days |
|-------------------------------|---------------|-------------|--------|--------|--------|---------|---------|
| Granitic residual soil        | Cohesion      | 36.4        | 36.1   | 35.5   | 33.2   | 29.4    | 25.3    |
|                               | Internal friction angle | 25.3        | 24.8   | 24.5   | 24.1   | 23.8    | 23.2    |
| Completely weathered slate    | Cohesion      | 25.2        | 23.7   | /      | 18.7   | 16.4    | 15.1    |
|                               | Internal friction angle | 21.0        | 21.0   | /      | 20.4   | 20.2    | 20.1    |
| Completely weathered sandstone| Cohesion      | 39.0        | 37.6   | /      | 34.3   | 31.2    | 28.2    |
|                               | Internal friction angle | 23.0        | 22.8   | /      | 22.0   | 21.6    | 21.2    |
| Colluvial soil                | Cohesion      | 90.3        | 85.5   | 80.1   | 74.8   | 70.2    | 67.2    |
|                               | Internal friction angle | 24.7        | 24.6   | 24.3   | 23.8   | 23.2    | 22.2    |

Figure 3: Relationship between saturation time and compressive strength.
Table 5: Rock impact test results (sample: diameter 50 mm, height 25 mm, and volume 50 cm³).

| Lithology                  | Sample number | Impact energy J/mm | Saturated uniaxial compressive strength/MPa | Impact number of initial cracks | Initial splitting energy/J | Failure characteristics |
|----------------------------|---------------|--------------------|--------------------------------------------|--------------------------------|----------------------------|--------------------------|
| Granite                    | N11-1         | 0.5 kg * 0.5 m = 2.5 | 160                                       | 1132                           | 2830.0                    | 5 cracks                 |
|                            | N11-2         | 0.5 kg * 0.3 m = 1.5 | 168                                       | 2490                           | 3735.0                    | 1 crack                  |
| Granite gneiss             | N15-1         | 0.5 kg * 0.1 m = 0.5 | 165                                       | 11257                          | 5628.5                    | 1 crack                  |
| Sandstone                  | N13-1         | 0.5 kg * 0.1 m = 0.5 | 84.8                                      | 8786                           | 4393.0                    | 1 crack                  |
| Metamorphic quartz sandstone | M11          | 0.5 kg * 0.3 m = 1.5 | 137                                       | 2043                           | 3064.5                    | 1 crack                  |
| Slate                      | M12-1         | 0.5 kg * 0.5 m = 2.5 | 18.1                                      | 475                            | 1187.5                    | Multiple cracks          |
|                            | M12-2         | 0.5 kg * 0.3 m = 1.5 | 13.3                                      | 1327                           | 1990.5                    | Multiple cracks          |
|                            | M12-3         | 0.5 kg * 0.1 m = 0.5 | 12.2                                      | 5369                           | 2684.5                    | 1 crack                  |

Figure 4: Relationship between different initial strength impact energy ratio and total broken energy.

The internal friction angle under the coupling effect of saturation and wave action is smaller (Figure 5(c)) than that of cohesive, which indicates that bank collapse is less likely to occur. The saturated friction angle $\varphi^*$ under wave action energy $E$ with the initial friction angle $\varphi$ can be expressed as follows:

$$\frac{\varphi^*}{\varphi} = (0.0001\varphi - 0.005)E^3 - (0.0007\varphi - 0.0295)E^2$$
$$+ (0.0012\varphi - 0.0527)E - 0.001\varphi^2 + 0.0656,$$

where $\varphi$ is the initial friction angle (°), and $E$ is the wave action energy ($\times 10^p$).

Equation (5) indicates that the friction angle decreases under wave action, and saturation is the mechanism of collapse for the loose bank slope.

4.4. Numerical Simulation of Wave Action on the Slope Stability. The QD14 toppling body collapse of the Miaowei hydropower station was used to analyse the mechanism of a wave acting on a bank slope via numerical simulation. The reservoir began to impound water on 29 July 2016. Monitoring points BQ1-2 and BQ1-3 were arranged in the strongly toppling area and extremely toppling area, and BQ1-1 was arranged in the weakly toppling area. After 1 December 2016, TP-BQ1-2 and TP-BQ1-3 were damaged due to collapse. Three electric wind speed meters and hand-held wind meters were arranged to monitor the wind speed and direction near the upstream area of QD14. QD14 first failed on 1 December 2016 at 3:00 p.m. From 29 July to 1 December 2016, during these 123 days, the wind-generated wave height was 2.70 m on average, and the wave period was 5 s. The second failure occurred on 13 June 2018 from the early morning of 12 June 2018 to the night of 13 June 2018. There was a strong wind of 27.8 m/s (wind force grade 10), resulting in a wave height of 5.1 m and duration of 2 days, with an average wave period of 2 s. The water saturation and general wave action time were 345 days, and an extreme storm generated a wave action time of two days. The failure surface was between strongly toppling and weakly toppling rock mass (Figure 6(a)). From June 2016 to August 2018, the accumulated deformation of BQ1-1 was less than 35 mm (Figure 6(b)).

4.4.1. Geological Condition and Calculation Parameters. The QD14 toppling slope is composed of purplish red and greyish green slates mixed with siltstone and fine sandstone. From the surface to the inside, the slope was divided into an extremely toppling area (BQ = 175), strongly toppling area (BQ = 225), weakly toppling area (BQ = 322), and untoppled area (BQ = 410). The initial calculation parameters are presented in Table 7.

4.4.2. Calculation Scenarios and Boundary Condition. Calculation scenarios are detailed in Table 8.

The UDEC model was defined from 1275 m to 1550 m (275.0 m in Y and 300.0 m in X directions). The slope material was assumed to be elastic-plastic. After fixing the displacement boundary on both sides and at the bottom, the initial stress field of the slope was solved only under gravity.

4.4.3. Analysis of Results. Model 1: Deformation Analysis of Slope under Pure Saturation State. In this model, only the rock mass softening of the slope was considered (according
to 123 days of saturation). The deformation of the rock mass below 1340 m elevation in the extremely toppling area increased slightly with an increase in the water level, the top displacement of the rock mass in the Y-direction in the extremely toppling area was 40–75 cm, the middle area displacement was 40–70 cm, and the bottom area displacement was 28–40 cm. The X-direction and Y-direction displacement of the B2-1 and C-1 series monitoring points were at the top of the strongly toppling area and weakly toppling area. The X-direction displacement of the slope top in the weakly toppling area was 5–8 cm (Figure 7(a)), the Y-direction displacement was 2–6 cm, the X-direction displacement of the top of the strongly toppling area was 10–15 cm, and the Y-direction displacement was 14–24 cm. Furthermore, no continual failure surface was formed, but the displacement was larger than that before the reservoir impounded.

Model 2: Deformation Analysis of Slope at a Wave Height of 3.0 M and Period of 5 s. When the reservoir impoundment reached 1360 m, 123 days of softening and 123 days of normal wave action were considered, and there was an obvious displacement differentiation zone between the extremely toppling area and the strongly toppling area. The X-direction deformation in the extremely toppling area is close to 100 cm near the water surface (Figure 7(b)) and 50–100 cm in the strongly toppling area, and the weak toppling weakly toppling area is less than 5 cm.

Model 3: Deformation of the Slope under General and Rain Storm. After a saturated and normal wave action of 345 days and an extreme storm wave action duration of 2 days, the reference point 5A-1 in the middle of zone A and near the water surface and wave action area, the maximum displacement in the X-direction was 260 cm (Figure 7(c)) and 200 cm in the Y-direction. The maximum displacement is 290 cm in the X-direction and 240 cm in the Y-direction of reference monitoring point 8A-1, located at the top of the extremely toppling area. The Y-direction displacement of reference monitoring point 8C-1, located at the top of the weakly toppling area, is 2–6 cm, which is close to the actual monitoring displacement value. The entire rock mass in the extremely toppling area of the slope exhibits sliding deformation toward the valley.

The numerical simulation results show that the bank slope plastic strain at the shallow part near the wave action zone is directly affected by the wave cyclic loads. This results in an extreme increase in the displacement, with the first damage of the bank slope occurring near the wave action zone. Under the action of waves generated by extreme storms, the toppling body produces accelerated deformation in an extremely toppling area near the wave action zone.

### 5. Discussions

The plain reservoir bank is dominated by coarse-grained soil. To date, many researchers have studied the main factors and mechanisms of coarse-grained, noncohesive, and cohesive homogenous soil bank collapse. These factors usually include slope angles, material diameters, water-level fluctuations, wave heights of reservoir water, and soil density degrees [33]. The mechanism of bank collapse is the change in the physical and mechanical properties of the rock and soil under the action of reservoir water, buoyancy force when the water level is still, and seepage force when the water level rises and falls on the stability of the bank slope [12, 16, 34]. Some researchers have analysed the erosion effect of waves on bank slopes by considering the wave-current interactions, but few have studied the mechanism of bank collapse from the perspective of wave impact damage. Only a few experimental studies on cohesive lateral bank erosion [35, 36] and mountainous river-type reservoirs have been reported in the literature [37, 38]. This is not

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### Table 6: Direct shear test results of saturated and wave impaction.

| Lithology of bank slope | Shear strength | Natural state | Saturated wave action energy/J | μ₀<sub>30</sub> 30 days |
|------------------------|---------------|---------------|-------------------------------|--------------------------|
| Silty                  | Internal friction angle (°) | 25.0 | 21.0 | 18.0 | 17.4 | 0.72 |
|                        | Cohesive (kPa)          | 12.5 | 11.7 | 11.3 | 10.9 | 0.90 |
| Silty clay             | Internal friction angle (°) | 23.0 | 19.3 | 16.5 | 16.2 | 0.72 |
|                        | Cohesive (kPa)          | 67.8 | 65.3 | 63.5 | 62.4 | 0.94 |
| Residual and colluvial soil | Internal friction angle (°) | 28.0 | 22.4 | 18.3 | 17.5 | 0.65 |
|                        | Cohesive (kPa)          | 31.3 | 30.1 | 28.7 | 27.9 | 0.92 |
| Highly weathered marl  | Internal friction angle (°) | 38.6 | 37.0 | 35.0 | 34.2 | 0.91 |
|                        | Cohesive (kPa)          | 56.8 | 55.2 | 54.9 | 54.3 | 0.96 |
| Highly weathered mudstone | Internal friction angle (°) | 40.3 | 38.7 | 36.8 | 34.0 | 0.91 |
|                        | Cohesive (kPa)          | 21.4 | 20.6 | 20.1 | 19.7 | 0.94 |
| Highly topped slate    | Internal friction angle (°) | 25.0 | 23.0 | 19.0 | 17.0 | 0.76 |
|                        | Cohesive (kPa)          | 25.0 | 23.5 | 22.4 | 22.1 | 0.90 |
| Highly weathered granite | Internal friction angle (°) | 26.5 | 24.2 | 20.0 | 16.3 | 0.75 |
|                        | Cohesive (kPa)          | 13.8 | 13.2 | 12.7 | 12.5 | 0.92 |

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Figure 5: (a) Relationship between friction angle and wave action energy. (b) Friction angle under saturation and wave action energy. (c) Friction angle reduction rate with wave action energy.
quantitatively sufficient to fully understand the mechanism of bank collapse in mountainous areas. However, owing to the deep unloading of the river, the bank slope in the mountainous area is mainly composed of residual and completely weathered rock mass. The mechanism of bank collapse after impoundment is significantly different from that of a plain-type reservoir. Most bank collapse is concave in shape, which is consistent with the flow pattern of the wave impact. So far, there is a lack of research on bank collapse caused by periodic wave impacts.

5.1. Mechanism of Bank Collapse in the Mountainous Reservoir. Bank collapse mainly occurs in the water-level rising stage, accompanied by an increase in the sloping underwater depth and water surface width. In this process, the wind-induced wave action on a steep bank slope is

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Figure 6: (a) Collapse of QD14. (b) Vertical displacement with reservoir water-level change.
strengthened. Owing to the impact of waves, the geometric shape of the bank slope after bank collapse is concave in a certain height range. The bank collapse in the mountainous area is mainly related to the rock and soil structure of the bank slope, submergence depth, water surface width, and the intensity of wind-induced wave action. The height and angle of the bank slope were not the main factors. Wind-induced wave action is the main external force of mountain-angle of the bank slope were not the main factors. Wind-induced wave action is the main external force of mountain- 

### 5.1.1. Rock Failure Process under Wave Action

The results of the in situ wave scour rate and laboratory strength test show that the influence of waves on bank collapse of river-type reservoirs is mainly manifested in two aspects. One is scouring (similar to a plain reservoir), which causes the bank slope to form a cavity in the wave action zone. This effect mainly occurs in the loose soil or completely weathered residual slope with a slope angle of less than 30° and a cohesive slope of less than 90 kPa. Because the cohesion between particles rapidly decreases after the slope saturates, it is easy to carry loose particles under the action of waves and coastal currents, leading to slow bank collapse [40]. Wave velocity had the most significant effect on wave scouring. The second is the damage to the rock and soil structure caused by the striking wave cyclic impact load. For a rock bank slope, the cyclic impact load leads to the opening of existing joints or the generation of microcracks in the rock body, which causes the rock structure to break gradually. This effect is related to the strength of a single wave and the characteristics of the rock structure. A loose bank slope can easily lead to the generation of internal microcracks. This leads to an increase in the saturation velocity and a reduction in the internal friction angle, directly leading to local block collapse of the bank slope. Wave amplitude, velocity, and slope structural characteristics were the main factors determining the failure mode (Table 9). A strong impact load causes cracks on the surface of the soil mass, structural damage, and further splitting of water. This significantly reduces the internal friction angle of the soil mass. When the average wave velocity is 2.5 m/s and the wave height is 2.0 m, bank collapse occurs at the bank slope with an internal friction angle of less than 30°. When the extreme hurricane wave height is 5.1 m and the wave velocity is 3.5 m/s, bank collapse occurs on the fully weathered slate slope. The same result was obtained by a numerical simulation.

#### 5.1.2. Failure Mechanism of Rock Slope under Wave Impact Action

If the primary energy of the wave cyclic impact is small, the impact is not sufficient to break the rock mass. However, when the concentrated stress of the stress wave at the crack end exceeds the strength of the rock, new cracks will occur in the rock, and the damage in the rock will continue to accumulate [41]. With an increasing number of impact cycles, the internal damage to the rock increases. The damage variable of the rock increases slowly with the cyclic impact number at a small impact energy condition. The damage variable increases rapidly with an increase in the impact cycles, the internal damage to the rock increases. The degree of weakening of the impact resistance varies with rock strength. For example, granite is brittle and has strong cementation; the tensile stress caused by mineral expansion is also strong; therefore, the impact failure energy required is large. However, slate, with a low compressive strength, requires less fracture energy. The rock strength is the main

### Table 7: Calculation of initial rock mechanics parameters in UDEC.

| Material                | Deformation modulus/GPa | Shear modulus/GPa | Poisson’s ratio | Bulk density/kN•m⁻³ | Internal friction angle/° | Cohesion/MPa |
|-------------------------|-------------------------|-------------------|----------------|---------------------|---------------------------|--------------|
| Extremely toppling area | 0.5                     | 0.18              | 0.35           | 22.0                | 27                        | 0.08         |
| Strongly toppling area  | 1.0                     | 0.37              | 0.34           | 23.0                | 26                        | 0.34         |
| Weakly toppling area    | 3.75                    | 1.50              | 0.28           | 26.0                | 36                        | 0.40         |
| Untopped area           | 18.0                    | 7.44              | 0.21           | 26.5                | 41.7                      | 0.65         |

### Table 8: List of calculation scenarios.

| Numerical model | Water level/m | Wave height/m | Wave period/s | Circumstances                  | Duration of wave action/h | Wave energy/kJ |
|-----------------|---------------|---------------|---------------|--------------------------------|---------------------------|----------------|
| Model 1         | 1360          | 0.00          | 0.0           | No wave action                 | 0                         | 0              |
| Model 2         | 1360          | 2.70          | 5.0           | Normal wave action             | 2952                      | 63763.2        |
| Model 3         | 1401          | 5.10/strong 2.0/strong | Normal wave + short-term extremely storm wave action | 2952+8280+48 | 49,582,808,640+4320 |

†Reference Eq. (1) to calculate wave energy.
Figure 7: (a) X-direction displacement in the extremely toppling area under the pure saturation state. (b) X-direction displacement in the extremely toppling area under saturation and wave action. (c) X-direction displacement in the extremely toppling area under general and stream wave action.
Wind-generated wave height has a significant influence on loose soil banks, the soil cohesion decreases after saturation and wave scour. According to Table 2, the scouring rate has a significant strength reduction after saturation and a strong wave-scouring effect.

5.1.3. Mechanism of Loose Bank Slope Collapse. When waves act on loose soil banks, the soil cohesion decreases after saturation, and the first stage of wave action is scouring to carry particles, which is generally completed within 30 days after impoundment. Scouring causes small-scale collapse. After the collapse forms a relatively steep slope, the continuous action of the wave impact load plays a major role. Furthermore, the internal friction is reduced, and the structure is destroyed. Collapse mainly occurs with a significant strength reduction after saturation and a strong wave-scouring effect.

Bank collapse that first occurs in the soil is easy to scour. According to Table 2, the scouring rate has a significant causal relationship with wave velocity and soil density. From high to low, the factors influencing the scour rate are wave velocity, wave height, percentage of 0.5-1 mm particles, density, and percentage of 0.075-0.25 mm particles. For the silt content (A), coarse content (B), soil density (C), and wave velocity (E) in the bank slope as independent variables, the mathematical regression analysis method was used to establish a prediction model of the four factors and scouring rate. Through standard normalisation, the relationship between each factor and scour rate was established, and the nonlinear formula of the total scour rate can be obtained as follows.

\[
S_r = 0.25 \left[ 0.35e^A + 0.50(B + 0.90)^2 - 0.18(1.75C - 095)^2 + 0.05E - 1.65 \right].
\]

The regression coefficient \( R^2 \) is 0.915.

The change in the internal friction angle after the impact load is related to the type of soil. The change in colluvial soil is the largest, followed by silt, silty clay, and fully weathered granite. The change in fully weathered marlstone and fully weathered argillaceous sandstone is the smallest. When the initial internal friction angle of the soil is more than 30°, the change rate under the wave impact is small. The change in cohesion under wave impact was not obvious when the wave load was small. For loose soil with small initial cohesion, the cohesion decreases significantly after saturation and wave impact, and the impact of waves on the loose soil is more obvious than that on the soil with large cohesion. A bank slope with low cohesion is also easily eroded by waves. With the reflection of waves, the particles are removed, the bank slope forms a scouring trough, and the upper rock or soil is prone to collapse.

A typical feature of weathered residual soil is that it contains a large number of microcracks. The macrodamage under an impact load is essentially a process of damage accumulation and crack development. Damage to the sample is reflected in two aspects. At the macro level, each impact produces an irrecoverable residual plastic strain. At the micro level, the energy absorbed by the sample is used for friction loss between mineral particles, the propagation of original microcracks, and the initiation of new cracks. The essence of the wave cyclic impact is to damage the soil structure and reduce the friction between particles.

The strain-time relationship of typical loose soil under the action of dynamic load when the soil sample was damaged is shown in Figure 8(a). Taking strongly weathered granite soil as an example, when the amplitude is \( F_{\omega} \), \( f = 5 \text{ Hz} \), the relationship between the number of impacts and the strain is illustrated in Figure 8(b) [43]. It is obvious that the soil broken time is related to the amplitude and frequency of the wave action.

5.1.4. Antiwave Impact Broken Energy of Saturated Rock and Soil. The evolution of reservoir bank collapse is rock or soil strength reduction and structural damage under saturation.
and wave cyclic impact action, forming a certain depth cavity in the bank slope. With an increase in the cavity depth, the upper rock and soil mass lose support and collapse. Therefore, bank slope collapse is mainly related to the antiwave impact broken energy of the bank slope material and wave action strength.

The antiwave impact broken energy refers to the total energy of wave action corresponding to the failure of saturated rock or soil mass, which is related to the quality index BQ of the rock mass [44] or soil mass. When the bank slope is composed of a rock mass with an intact structure and large antiwave impact broken energy, collapse does not occur easily. Generally, when the wave impact energy of a rock mass is greater than the critical antiwave impact broken energy of bank collapse, bank collapse stops and the bank becomes stable.

Geological statistics of bank collapse and the statistics of cumulative wave action intensity indicate that there is no collapse within 3 years after reservoir water impoundment, under the condition of a BQ value greater than 280. The rock mass quality index BQ of the bank slope collapse is less than 280, and the wave action energy is mostly larger than $45 \times 10^6$ J. According to the impact tests, the relationship between the wave impact energy and the uniaxial compressive strength of the intact rock sample is shown in Figure 9, and the final unit broken energy $E_{r0}$ can be calculated using the following equation.

$$E_{r0} = 0.0009R_c^{1.7358}.$$  \hspace{1cm} (7)

For loose soil bank slopes, the deformation and failure process of soil under cyclic impact load is a process of energy dissipation and damage generation and accumulation. Generally, it shows the growth of cumulative plastic deformation and soil failure. According to the results of field monitoring and laboratory tests, when the cumulative strain reached 90%, the slope collapsed. Equation (8) can then be used to judge whether bank collapse occurs in the soil [43].

$$E_s = \frac{(\sigma_1 - \nu\sigma_3)n}{E(t)} \geq 0.90.$$  \hspace{1cm} (8)

![Figure 8](image-url)  \hspace{1cm} Figure 8: (a) Strain-time relation of typical loose soil under dynamic load action. (b) $\varepsilon_a - \lg N$ relationship.

![Figure 9](image-url)  \hspace{1cm} Figure 9: Unit broken energy of common rock with uniaxial compressive strength under cycle loads.
where \( \sigma_3 = \sigma_1 \tan^2(45^\circ - \phi'/2) - 2\sigma_1 \tan(45^\circ - \phi'/2) \), \( \sigma_1 = F_{w}/A \), \( \nu \) is Poisson’s ratio of soil (\( \nu = 0.30-0.35 \) for completely weathered residual soil of granite), \( E \) represents the antiwave impact broken energy of soil, \( n \) is the wave action number, \( E_i(t) \) is the deformation modulus at the failure time, and \( c \) is the cohesion.

5.2. Collapse Evolution in the Mountainous Reservoir. In view of the coarse-grained soil bank slope being widely distributed in plain reservoirs, owing to the material and structural conditions of the bank slope, the time of bank collapse is short [45, 46]. Some researchers divide bank collapse into three stages [36, 38]. For river-type reservoirs, the softening of rock and soil and the wave impact makes the bank collapse take a longer time, and the bank collapse process can be described in six steps. First, when the reservoir water rises to a certain level, the slope material is saturated by the wave acting near the SWL (step I). After a certain period of time, the strength of the slope material decreased, and a small collapse occurred near the water surface (step II). Next, after the wave acts straight on the bank slope for a certain duration, small collapse induces a larger collapse above the SWL (step III). In step IV, the wave action becomes increasingly strong on the steeper slope, resulting in a larger collapse in step V. Finally, when the bank collapse material accumulates at the foot of the slope, the intensity of the wave action decreases, and the bank collapse becomes weak. With continuous accumulation of the upper bank collapse material at the slope foot, the slope angle underwater decreases. To a certain extent, the bank slope is subjected to a shallow water wave. After further deposition, the effect of the wave on the original bank slope disappears, and it changes to that of a shallow water wave, which causes scour and siltation of the silted slope. When the energy of the wave action disappears, the bank slope becomes stable. At this time, the bank collapse process is terminated. After underwater accumulation, a stable slope angle is formed, and the bank collapse ceases to develop (step VI).

Site monitoring also shows that bank collapse mainly occurs on a bank slope 5–30 km away from the dam site. The upstream area of the reservoir pertains to a small water storage depth and a narrow water surface. The shallow water wave was the main wave under the same wind force. Therefore, a few bank collapses occurred in this area. Under the action of the wave, the area with the smallest wave and strongest wind wave action corresponds to a reservoir impoundment depth of more than 50 m.

The duration of bank collapse of river-type reservoirs in mountainous areas is much longer than that of plain-type reservoirs. It has been nearly 30 years since the impoundment of the Manwan Reservoir in 1993, and there is still scope for new bank collapse and expansion of the original bank collapse. Since the impoundment of the Nuozhadu reservoir in 2012, bank collapse is also increasing, all of which are due to the failure of the underwater bank slope to make wave action disappear. Based on the lithology, the collapse occurrence time can be divided into four stages: less than 30 days, 30 days to 1 year, 1 to 5 years, and more than 5 years.

6. Conclusions

In this study, field geotechnical property surveys, laboratory tests, and numerical simulations were employed to investigate the mechanism of bank collapse and propose a collapse prediction model of the critical wave load in a mountainous area reservoir. The main conclusions are summarised as follows:

1. The bank slope in mountainous reservoirs is mainly composed of weathered rocks and residual soils. Deep unloading is a serious problem. The effect of waves on bank collapse is mainly reflected in the impact of periodic wave loads. The collapse occurrence times of different lithologies can be divided into four stages: less than 30 days, 30 days-1 year, 1-5 years and more than 5 years, and 85% occurs within 30 days during the reservoir water-level rising stage.

2. Bank collapse first occurs in the fully weathered and colluvial soil slope, where the cohesion was less than 90 kPa, the internal friction angle was less than 30°, and the strength decreased significantly after saturation. The cohesion of the loose slope reduces quickly under saturation after reservoir water impoundment, and the internal friction angle changes due to the action of wind-induced wave impaction.

3. The structure of the unloading rock mass is damaged by cyclic wave loading impacts. When the wave height is less than 2 m, the influence of waves on bank collapse is negligible. When the wave height is larger than 5 m, the collapse in unloading and toppling rock slope is highly likely to occur under the impact of waves. The velocity and height of the wind-generated waves are the key factors causing structural cracking. Bank collapse occurs in the area 20–50 km away from the dam of the reservoir and a water depth of more than 50 m.

4. The antiwave impact broken energy of different rocks and soils has been proposed based on laboratory experiments and field monitoring. This can be used to predict the location and occurrence time of bank collapse. The present study provides a framework for the further prediction of bank collapse. However, the proposed method is lack of considering the topographic features, thus, it needs to improve the prediction accuracy on considering the bank slope terrain on its stability in the future research.

Data Availability

The data used to support the findings of this study were supplied by [Faming Zhang] under license and so cannot be made freely available. Requests for access to these data should be made to [Faming Zhang, zhangfm@hhu.edu.cn].
Conflicts of Interest

The authors declared that they have no conflicts of interest to this work. We declare that we do not have any commercial or associative interest that represents a conflict of interest in connection with the work submitted.

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References

[1] H. M. Tang, “Study on reservoir bank collapse and its engineering prevention in the Three Gorges areas, Changjiang River,” Quaternary Sciences, vol. 23, no. 6, pp. 648–656, 2003.
[2] J. Du, K. Yin, and S. Lacasse, “Displacement prediction in colluvial landslides, Three Gorges Reservoir, China,” Landslides, vol. 10, no. 2, pp. 203–218, 2013.
[3] H. M. Tang, J. Wasowski, and C. H. Juang, “Geohazards in the three Gorges Reservoir Area, China: Lessons learned from decades of research,” Engineering Geology, vol. 261, article 105267, 2019.
[4] Y. P. Yin, B. L. Huang, X. T. Chen, G. N. Liu, and S. C. Wang, “Numerical analysis on wave generated by the Qianjiangping landslide in Three Gorges Reservoir, China,” Landslides, vol. 12, no. 2, pp. 355–364, 2015.
[5] S. Dapporto, M. Rinaldi, and N. Casagli, “Mechanisms of riverbank failure along the Arno River, Central Italy,” Earth Surface Processes and Landforms: The Journal of the British Geomorphological Research Group, vol. 28, no. 12, pp. 1303–1323, 2003.
[6] S. B. Wu and M. H. Yu, “Experimental study on bank failure process and interaction with riverbed deformation due to fluctuating hydraulic force,” Journal of Hydraulic Engineering, vol. 45, no. 6, pp. 649–657, 2014.
[7] M. L. Chu-Agor, M. L. Wilson, and G. A. Fox, “Numerical modeling of bank instability by seepage erosion undercutting of layered streambanks,” Journal of Hydrologic Engineering, vol. 13, no. 12, pp. 1133–1145, 2008.
[8] F. Ji, C. Liu, H. Zhou, H. Liu, and Y. Liao, “Identifying the influences of geological factors on reservoir bank collapse by a model test,” Bulletin of Engineering Geology and the Environment, vol. 77, no. 1, pp. 127–139, 2016.
[9] D. Huang, D. M. Gu, Y. X. Song, D. F. Cen, and B. Zeng, “Towards a complete understanding of the triggering mechanism of a large reactivated landslide in the Three Gorges Reservoir,” Engineering Geology, vol. 238, pp. 36–51, 2018.
[10] G. W. Jia, T. L. Zhan, Y. M. Chen, and D. G. Fredlund, “Performance of a large-scale slope model subjected to rising and lowering water levels,” Engineering Geology, vol. 106, pp. 92–103, 2009.
[11] X. P. Chen and J. W. Huang, “Stability analysis of bank slope under conditions of reservoir impounding and rapid drawdown,” Journal of Rock Mechanics and Geotechnical Engineering, vol. 3, no. 5, pp. 429–437, 2011.
[12] G. H. Sun, Y. T. Yang, W. Jiang, and H. Zheng, “Effects of an increase in reservoir drawdown rate on bank slope stability: a case study at the Three Gorges Reservoir, China,” Engineering Geology, vol. 221, pp. 61–69, 2017.
[13] S. Wang, G. Idinger, and W. Wu, “Centrifuge modelling of rainfall-induced slope failure in variably saturated soil,” Acta Geotechnica, vol. 16, no. 9, pp. 2899–2916, 2021.
[14] D. Song, X. Liu, B. Li, J. Zhang, and J. J. V. Bastos, “Assessing the influence of a rapid water drawdown on the seismic response characteristics of a reservoir rock slope using time–frequency analysis,” Acta Geotechnica, vol. 16, no. 4, pp. 1281–1302, 2021.
[15] A. Biniyaz, A. Azmoon, and Z. Liu, “Coupled transient saturated–unsaturated seepage and limit equilibrium analysis for slopes: influence of rapid water level changes,” Acta Geotechnica, 2021.
[16] Y. Li, J. Chen, F. Zhou et al., “Stability evaluation of rock slope based on discrete fracture network and discrete element model: a case study for the right bank of Yigong Zangbu Bridge,” Acta Geotechnica, vol. 17, 2021.
[17] X. Zhang, L. Chen, F. Zhang, C. Lv, and Y. Zhou, “Impact of fluid turbulent shear stress on failure surface of reservoir bank landslide,” Arabian Journal of Geosciences, vol. 11, no. 22, pp. 1–13, 2018.
[18] X. Zhang and R. Simons, “Experimental investigation on the structure of turbulence in the bottom wave-current boundary layers,” Coastal Engineering, vol. 152, article 103511, 2019.
[19] J. S. Que, Prediction For Reservoir Bank Collapse in Fulina Area of the Three Gorges Project, [Ph.D. thesis], Jilin University, 2007.
[20] G. H. Sun, Y. T. Yang, S. G. Cheng, and H. Zheng, “Phreatic line calculation and stability analysis of slopes under the combined effect of reservoir water level fluctuations and rainfall,” Canadian Geotechnical Journal, vol. 54, no. 5, pp. 631–645, 2016.
[21] M. G. Tang, Q. Xu, and R. Q. Huang, “Types of typical bank slope collapses on the Three Gorges Reservoir (in Chinese),” Journal of Engineering Geology, vol. 14, no. 2, pp. 172–177, 2006.
[22] M. S. Kirkgöz, “Breaking wave impact on vertical and sloping coastal structures,” Ocean Engineering, vol. 22, no. 1, pp. 35–48, 1995.
[23] B. Golding, “A wave prediction system for real-time sea state forecasting,” Quarterly Journal of the Royal Meteorological Society, vol. 109, no. 460, pp. 393–416, 1983.
[24] S. Bal, “Prediction of wave pattern and wave resistance of surface piercing bodies by a boundary element method,” International Journal for Numerical Methods in Fluids, vol. 56, no. 3, pp. 305–329, 2008.
[25] Y. Li and F. Raichlen, “Non-breaking and breaking solitary wave run-up,” Journal of Fluid Mechanics, vol. 456, pp. 295–318, 2002.
[26] T. Tomotuka and K. Yuihiro, “Evaluation of simplified wave prediction method for wind generated waves in Bohai Bay,”
A. Farhadi, H. Emdad, and E. G. Rad, "On the numerical simulation of the nonbreaking solitary waves run up on sloping beaches," Computers & Mathematics with Applications, vol. 70, no. 9, pp. 2270–2281, 2015.

J. R. Morison, J. W. Johnson, and S. A. Schaaf, "The forces exerted by surface waves on piles," Journal of Petroleum Technology, vol. 189, pp. 145–154, 1950.

U. S. Army, Coastal Engineering Research Centre. Shore Protection Manual, U.S. Government Printing Office, Washington, DC, 4th edition, 1984.

N. Nagata, T. Hosoda, and Y. Muramoto, "Numerical analysis of River channel processes with bank erosion," Journal of Hydraulic Engineering, ASCE, vol. 126, no. 4, pp. 243–252, 2000.

B. Maihemuti, E. Wang, T. Hudan, and Q. Xu, "Numerical simulation analysis of reservoir bank fractured rock-slope deformation and failure processes," International Journal of Geomechanics, vol. 16, no. 2, article 04015058, 2016.

P. A. Cundall, The Measurement and Analysis of Acceleration in Rock Slopes, Imperial College Science and Technology, University of London, London, 1971.

A. P. Shu, F. H. Li, and K. Yang, "Bank-collapse disasters in the wide valley desert reach of the upper Yellow River," Procedia Environmental Sciences, vol. 13, pp. 2451–2457, 2012.

Q. Xu, T. X. Liu, and M. G. Tang, "A new method of reservoir bank-collapse prediction in the Three Gorges Reservoir—river bank structure method," Hydrogeology & Engineering Geology, vol. 34, no. 3, pp. 110–115, 2007.

M. H. Yu, H. Y. Wei, and S. B. Wu, "Experimental study on the bank erosion and interaction with near-bank bed evolution because of fluvial hydraulic force," International Journal of Sediment Research, vol. 1, pp. 81–89, 2015.

F. Ji, Y. Shi, R. Li, H. Zhou, D. Wang, and J. Zhang, "Progressive geomorphic evolution of reservoir bank in coarse-grained soil in East China - Insights from long-term observations and physical model test," Engineering Geology, vol. 281, article 105966, 2021.

K. J. Shen and Y. W. Meng, "Stability analysis and treatment study on reservoir bank collapse area," Applied Mechanics and Materials, vol. 737, pp. 437–440, 2015.

J. Zhao, H. Zhang, C. Yang, M. Lee, X. Zhao, and Q. Lai, "Experimental study of reservoir bank collapse in gravel soil under different slope gradients and water levels," Natural Hazards, vol. 102, no. 1, pp. 249–273, 2020.

V. K. Das, S. Roy, K. Barman, S. Chaudhuri, and K. Debnath, "Cohesive river bank erosion mechanism under wave-current interaction: a flume study," Journal of Earth System Science, vol. 129, no. 1, p. 99, 2020.

D. M. Lawler, "The measurement of river bank erosion and lateral channel change: a review," Earth Surface Processes and Landforms, vol. 18, no. 9, pp. 777–821, 1993.

D. E. Kipp and E. Gradym, "The micromechanics of impact fracture of rock," International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, vol. 16, no. 5, pp. 293–302, 1979.

A. M. Rubin and T. J. Ahrens, "Dynamic tensile failure induced velocity deficits in rock," Geophysical Research Letters, vol. 2, pp. 219–223, 1991.