Stability research on reservoir slope based on geomechanical parameters back-analysis and monitoring data: A case study in China

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Abstract. Stability evaluation of reservoir slope is a key issue affecting the safe operation of water conservancy projects, especially the new unexpected slope deformable bodies caused by changes in the hydrodynamic environment during the reservoir operation. The present study investigated the slope stability evaluation based on geomechanical parameters back-analysis and monitoring data of the left bank deformed slope of Guandi reservoir in China, of which the soil mechanical parameters result of laboratory test do not match the actual operating. In order to accurately evaluate the stability of this slope under the complex external environment, the deformation characteristics and instability mode of the slope were analysed based on the systematic study of the relationship between monitoring data and reservoir water level as well as rainfall. The failure mode of deformed slope was proposed, and the geomechanical parameters were obtained through back-analysis according to monitoring data of slope deformation. Based on the back-analysis results, the detailed analysis of the slope stability was studied, and the stability sensitivity analysis of the deformed slope under the sudden drop of the reservoir water level was carried out combining with the reservoir dispatch requirements. The results show that the slope stability using the back-analysis parameters are objective and reasonable. The research results could provide scientific references for stability evaluation of similar slopes.

1. Research background
China is rich in hydropower resources, especially in the southwest region surrounding the Qinghai-Tibet Plateau. With the implementation of China's western development strategy, dozens of hydropower stations have been built. Affected by the uplift belt of the Tibetan Plateau, the new tectonic activities in the southwest region are strong, making these regions present the characteristics of mountainous landforms with deep valleys and steep bank slopes. Therefore, the stability of reservoir bank slopes is a key issue in the construction and operation of these hydropower stations. Since the 1963 Vajont landslide in Italy [1], Qianjiangping landslide [2] and Tangyanguang slope [3] in China, the reservoir slope stability has attracted more and more attention, especially the new deformed bodies caused by changes in the hydrodynamic environment because of the operation of reservoirs and rainfall [4,5]. Accurate determination of slope stability has important theoretical significance and application value for the prevention and control of slopes in reservoir areas of hydropower projects [6,7].
The rigid body limit equilibrium method is the basic method of slope stability analysis with long development history. But the accuracy of its results largely depends on the accuracy of the geotechnical parameters. For natural slopes, due to the complex composition of rock and soil mass, it is difficult to measure its geotechnical parameters directly and accurately through tests. Especially after the reservoir impoundment, the fine phase in the soil-rock mixture will soften under the action of water immersion, resulting in the decrease of macroscopic mechanical parameters. Therefore, how to accurately select the geotechnical parameters is not only a complicated theoretical problem, but also has important engineering practical significance [8,9,10].

In this paper, taking the left bank deformed slope of Guandi reservoir as research object, the influence of reservoir water level and rainfall on slope deformation was studied. Failure mode of deformed slope was proposed, according the deformation monitoring data and inspection situation. The geomechanical parameters were obtained through back-analysis, and the results were applied to the slope stability evaluation.

2. The basic geological conditions of the slope and its deformation characteristics after reservoir impoundment

2.1. Engineering Geological Characteristics of Overburden Slope in Front of Dam on the Left Bank
The main task of the hydropower station is to generate electricity. The normal storage level of the reservoir is 1,330 m, and the water level for sand discharge is 1,328 m. It is a daily regulating reservoir with a total storage capacity of 760 million m$^3$. The barrage dam is a roller compacted concrete gravity dam. The dam crest elevation is 1,334 m. The maximum dam height is 168 m. The dam crest length is 516 m.

The overburden distribution area of the left bank in front of the dam is located in the left bank slope section of the river bend section on the upstream side of the dam. The front-edge arc of the bank slope is about 1.5 km long along the valley. The flow rate is S20$^\circ$ W direction flows into the front edge of the bank slope, and then gradually deflects to S25$^\circ$ E-direction outflow bank slope. The slope direction of the overburden is between the valley (at an elevation of about 1,200 m) and the steep wall of the bedrock at the rear edge (at an elevation of about 1,900 m). The middle part of the bank slope is a ridge in the east-west direction. To the north of the ridge, the terrain has little fluctuation, and the dome is relatively flat, with a slope of about 45$^\circ$~55$^\circ$. To the south, the surface of the bank slope alternates with gullies and ridges, and the terrain is slightly undulating.

The exposed strata of the bank slope in this area are mainly the basalt formation of the upper Permian system. There are a wide range of colluvium, slope deposit, residual deposit and alluvial proluvial deposit on the surface. The contact relationship is complex and they are mostly mixed distribution. According to the deformation and failure characteristics of bank slope, landform of the bank slope, genetic types and development and distribution characteristics of the overburden, the overburden in front of the left bank dam is divided into A~E areas as shown in Figure 1. The area B studied in this paper is located in the middle and upper part of the overburden.

Area B is mainly composed of overburden, with front elevation of about 1,240 m, back elevation of about 1,690 m and bottom width of about 230 m along the river. According to the drilling results, the overburden below the surface is as follows:

①(col+dlQ4): Colluvial and Deluvial solitary gravelly soil. The colluvium is composed of colluvium boulder and colluvium gravelly soil. The diameter of colluvium boulder is generally about 1-3 m, and a few can exceed 10m. The surface of colluvium boulder is gray black with weathering halo of 1-3 mm. This layer is thicker above the elevation of 1,580 m, with an average thickness of about 18 m, and thinner below the elevation of 1,580 m, with an average thickness of about 10 m. At the elevation of 1580m, there is an obvious change in the boundary between foundation and overburden.

②(colQ4): Colluvial rock. The diameter is 15-24 cm accounting for 35%, 8-10 cm for 35%, 2-4 cm for 30%. The lithology is composed of near source dense basalt and breccia agglomerated lava in the posterior margin.
3 (dLQ4): Slope gravel soil. The diameter of gravel is 5-13 cm, accounting for about 55%. The lithology is near source dense basalt, and the soil is yellowish brown clay.

4 (col+dLQ4): The characters are the same as that of ①.

The overburden composition above and below the elevation of 1,360 m is slightly different, and the typical section is shown in Figure 2.

2.2. Deformation characteristics of area B after impoundment

The hydropower station began to store water in the lower sluice reservoir of diversion tunnel in November 2011. The water level reached 1,328 m in July 2012. After impoundment, due to the influence of reservoir water level, the deformation and cracking of the slope occurred. Monitoring data is analysed up to the end of 2019.

2.2.1. Deformation monitoring data. Monitoring instruments are arranged in the overburden Slope in Front of Dam on the Left Bank. There are external deformation and internal deformation monitoring instruments in area B. The monitoring arrangement is shown in Figure 3.

Figure 1. Plane sketch of overburden slope distribution area on left bank in front of dam.

Figure 2. Typical section geological map of area B.

Figure 3. Layout of monitoring installations and horizontal deformation vector diagram up to the end of 2019 (including monitoring data outside of area B).
2.2.1.1 Exterior deformation
The exterior observation points in area B of the overburden layer are TP6FGC ~ TP11FGC. The deformation process line is shown in Figure 4, and the distribution diagram is shown in Figure 3. The deformation direction is specified as: X is the direction along the river, and deform to downstream is positive, Y is the left and right bank direction, and deform to left bank is positive, H is the vertical direction, sinking is positive and the opposite direction is negative. The following points can be learned from the figures:
(1) After the reservoir is impounded, local deformation of the slope below 1,500 m is triggered, namely TP6FGC, TP7FGC, TP10FGC, TP11FGC. The slope above 1,500 m is relatively small in deformation, namely TP8FGC, TP9FGC.
(2) There is no uniform deformation direction in the X-direction. Preliminary analysis suggests that it is related to the topography. The maximum cumulative value is 109 mm.
(3) All monitoring points in the Y direction point to the right bank (cross-river direction), which is the main deformation direction. The monitoring points of local deformation show a trend of continuous displacement toward the riverbed. The cumulative deformation increases, but the deformation rate is relatively stable without sudden changes. The maximum cumulative value is ~252.5 mm.
(4) All monitoring points in the H direction point to settlement. The monitoring points of the local deformation continues. The cumulative deformation increases, but the deformation rate is relatively stable without sudden change. The maximum cumulative value is 202.6 mm.
(5) According to statistics, the rate of change of local deformation measurement points is basically maintained at the same level. The rate of change along the river is about 2 mm/month. The rate of change of settlement is about 1.5 mm/month.

![Deformation processes of area B.](image)

2.2.1.2 Internal deformation
There are IN2, IN4 ~ IN6 inclinometers arranged in area B. Up to the end of 2019 year, the typical displacement distribution with depth of IN5 (1,405 m elevation) and IN6 (1,354 m elevation) are shown in Figure 5 and Figure 6. Other inclinometers have smaller measured values. The direction A is the horizontal river direction, and the horizontal direction of the reservoir area (right bank) is positive. The direction B is the upstream and downstream direction, and the horizontal direction upstream is positive; otherwise, the deformation is negative. In addition, due to the deformation, IN5 cannot detect the deformation of the hole depth below 39 m after June 2016. Therefore, a set of inclinometer was added near the original position and the initial value was obtained in March 2017.
Figure 5. Deformation distribution of IN5 at different times.

Figure 6. Deformation distribution of IN6 at different times.

The following points can be learned from the figure:

1. The A-direction deformation of IN5 is generally in the horizontal river direction (opening direction), with varying degrees of disturbance deformation at about 10~20 m below the orifice, and obvious shear slip zone at about 40~38.5 m below the orifice. The deformation in the cross-river direction increases continuously after impounding. The cumulative deformation currently monitored has reached 154.7 mm. Considering the data connection processing in 2016 and 2017 years, it is estimated that the actual deformation is about 240 mm and the external deformation is consistent. Disturbance and deformation of varying degrees appeared about 15 m below the IN6 orifice, and the soil began to deform about 38 m below the orifice.

2. The B-direction deformation orifice value of IN5 and IN6 is very small. There is internal local disturbance and deformation.
(3) Although there is an obvious shear slip zone about 40~38.5 m below the IN5 orifice, considering the spatial distribution and deformation law of the two measuring points, the slope has not yet formed a clear slip surface.

(4) Considering of the soil composition, the internal deformation disturbances are all located in the soil layer containing fine-grained soil, which is the layer ③.

2.2.1.3 Relationship between deformation and rainfall

The slope of the reservoir area is greatly affected by the fall of the reservoir water level. But since the power station is a daily regulating reservoir, the normal water level fall depth is within 2 m. The fall of the reservoir water level within 2 m has little impact on the slope of area B of several hundred meters. Rainfall is another major factor affecting the slope stability of area B during operation. Taking appearance deformation as an example, the relationship between rainfall and Y-direction displacement of a typical measuring point is shown in Figure 7.

![Figure 7. Process of Y-direction deformation and rainfall.](image)

According to rainfall statistics, rainfall is relatively large from May to October each year, which is divided into one period. November to April of the following year is divided into another period. It is concluded that the deformation of the surface deformation mainly occurs in rainy season. Rainfall causes deformation of the slope surface.

2.2.2. Inspection results. Slope inspection is carried out once a month for the slope. A side slope inspection is organized every year after the pre-flood news and every year. The inspection found that there were local tensile cracks in the drainage ditch at the elevation of 1,440~1,450 m, and the cracks developed in a small range, depicted in Figure 8. No transverse tensile cracks were seen in other parts. The macroscopic deformation was slight and no penetrating cracks were found.

![Figure 8. Crack distribution of the 1,440 m drainage ditch.](image)

3. Back-analysis of geomechanical parameters
3.1. Test results of soil physical mechanics

In the early stage, physical and mechanical tests were carried out on this part. The sample numbers SF11 and SF12 are all collapsed deposits. The physical and mechanical parameters are shown in Table 1.

In order to study the influence of valley deformation of the arch dam, the parameters of the fixed dam and the base material remain unchanged, as shown in Table 2. In the case that the valley width takes different magnitudes, the three-dimensional finite element method is used to calculate the deformation of the arch dam under its own weight and different valley narrowing magnitudes.

| test numbering | sample preparation control conditions | unit weight | direct shear test (saturated, consolidated, fast) |
|----------------|---------------------------------------|-------------|-----------------------------------------------|
|                | dry density $\rho_d$ g/cm$^3$ | void ratio $e$ | water content $\omega$ % | natural weight $\gamma$ kN/m$^3$ | saturated weight $\gamma_{sat}$ kN/m$^3$ | cohesion $C$ kPa | internal friction angle $\phi$ $^\circ$ |
| SF11           | 2.14                                 | 0.25        | 4.1 | 21.86 | 23.40 | 45.0 | 29.0 |
| SF12           | 2.01                                 | 0.34        | 6.5 | 21.41 | 22.64 | 35.0 | 27.3 |

In addition, during the on-site investigation, the comprehensive internal friction angles of the slopes with different genesis in each subarea were also counted, as shown in Table 2.

Table 2. Statistical of comprehensive internal friction angle

| Causes of Overlay | comprehensive internal friction angle |
|-------------------|--------------------------------------|
| slope sediments   | $34^\circ$ $\sim$ $38^\circ$         |
| Colluvial deposit | $30^\circ$ $\sim$ $37^\circ$         |

According to the indoor test parameters, the stability safety factor of the slope under normal operating conditions before the water storage is 0.95. According to the outdoor test parameters, the stability safety factor of the slope after the water storage under the rainstorm conditions is about 1.3. Both are inconsistent with the actual situation of the slope. Therefore, in order to accurately determine the stability of the slope, it is necessary to determine the mechanical parameters based on the in-situ monitoring data.

3.2. Back-analysis of mechanical parameters based on monitoring data

3.2.1. Analysis of slope instability mode. According to deformation monitoring data and patrol inspection, the deformation of area B has not yet formed a penetrating sliding boundary or a clear slip surface. Combined with the soil composition of the overburden layer, the slope instability mode is presumed to be water softening-induced retrogressive landslides. During the reservoir impoundment, the fine phase in the soil-rock mixture soften under the action of water immersion, causing the slope deform and unevenly sink.

3.2.2. Inversion mode selection. Considering the slope deformation, failure characteristics and patrol inspections, the two parts(points) of the sliding surface can be basically determined: the first part (point) is rear the edge of the deformed body near the 1,440m~1,450 m drainage ditch; the second part (point) is located 38.5 m below the orifice (about 1,367 m elevation) revealed by IN5. In addition, the inclination IN6 also monitored the disturbance deformation at a depth of 40 m, but there was no obvious slip surface. Considering the accuracy of monitoring instruments, a distribution area is set up to block searching. The arc search is carried out based on these two areas, as shown in Figure 9, to find the most dangerous slip surface.
3.2.3. Determination of the inversion target. The monitoring data of the slope shows that the slope of area B has a relatively obvious displacement under rainfall condition. According the deformed slope, the slope safety factor is between 1.0 and 1.05. Therefore, in this paper the safety factor of the most dangerous sliding surface in the back analysis is taken as 1.03 under the rainfall condition.

3.2.4. Back-analysis method. Based on the analysis of the above-mentioned, it is inferred that the possible failure mode is circular sliding. The simplified Bishop arc method was applied using the “Slide” software. Figure 2 is used for the calculation profile. During the inversion, the trial calculations are firstly performed to obtain a series of $\Phi$ and $c$ value combinations under specified inversion target. Because there is no clear slip zone on the slope of area B, the physical and mechanical parameters of each layer of soil are finally determined based on the calculation results, slope engineering geological characteristics and test results, combining with engineering experience and analogy.

3.2.5. Inversion results. According to the back-analysis method and the inversion target, the inversion safety factor and the corresponding $\Phi$ and $c$ values of the potential slip zone are shown in Table 3. According to Table 3, the recommended values of the physical and mechanical parameters of each layer of the slope in the B area are drawn up, shown in Table 4.

### Table 3. Comprehensive inversion parameters under rainfall conditions

| $\Phi$ (°) | 30 | 31 | 32 | 33 | 34 | 35 | 36 |
|---|---|---|---|---|---|---|---|
| C (kPa) | 70 | 75 | 80 | 60 | 64 | 70 | 55 |
| K | 1.02 | 1.038 | 1.055 | 1.016 | 1.032 | 1.052 | 1.033 | 1.05 | 1.032 | 1.066 | 1.032 | 1.049 | 1.049 |

### Table 4. Suggested values of physical and mechanical

| Strata codename | natural weight ($\gamma$) (kN/m³) | saturated weight ($\gamma_{sat}$) (kN/m³) | natural shear strength | Shear strength saturation |
|---|---|---|---|---|
| ①col+dlQ4 | 20.0 | 21.5 | 45-55 | C (kPa) | 34-36 | 35-40 | 32-34 |
| ②colQ4 | 21.5 | 22.5 | 30-40 | 35-37 | 25-35 | 34-36 |
| ③dlQ4 | 19.5 | 21 | 60-70 | 31-33 | 45-55 | 29-31 |
| ④col+dlQ4 | 20.5 | 22 | 60-65 | 34-36 | 40-50 | 32-35 |
4. Evaluation of Slope Stability and Sensitivity Analysis of Sudden Drop of Reservoir Water Level

4.1. Slope stability evaluation

According to the physical and mechanical parameters of each layer of soil obtained above, the slope stability is analyzed and evaluated. When the sliding surface is approximate to a circular arc, the simplified Bishop method is uniformly used for calculation; when the sliding surface is a composite sliding surface, the Morgenstein-Price method is uniformly used.

4.1.1. Calculation conditions and assumptions. According to the needs of slope evaluation, long-term design conditions (normal operating conditions), short-term design conditions (rainfall conditions), and accidental design conditions (earthquake conditions) are selected for stability analysis. Under the rainfall condition, the soil body adopts saturation parameters. The groundwater level is obtained by monitoring. Under the case of earthquake conditions, ignoring the amplification effect of the seismic inertial force of the slope, the horizontal peak acceleration is taken as 0.172g according to the 50-year probability of exceeding 10%.

4.1.2. Slope stability review. The overall sliding along the rock boundary and the arc sliding surface before and after the reservoir impoundment are calculated. The stability safety factor is shown in Table 5.

| Slip mode                                      | Normal working condition | Rainstorm conditions | Seismic conditions | Remarks                  |
|------------------------------------------------|--------------------------|----------------------|--------------------|--------------------------|
| The whole slides along the rock boundary after impoundment | 1.16                      | 1.06                 | 1.06               | Morgenstein-Price method  |
| The results of the predict arc slip surface before impoundment | 1.22                      | 1.07                 | 1.13               | Simplified Bishop Method |
| The results of the predict arc slip surface after impounding | 1.10                      | 1.03                 | 1.00               | Simplified Bishop Method |

It can be seen from the table that the slope of area B is stable after reservoir impoundment. Before reservoir impoundment, the slope of area B is stable. After reservoir impoundment, the stability is reduced, the slope is basically stable under normal conditions and are in an under-stable state under rainfall and earthquake conditions. The results are consistent with the actual operating of the slope and the monitoring data. The mechanical parameters obtained from the back-analysis are objective and reasonable.

According to the calculation results, the slope in area B is stable as a whole. Although there is deformation in the local slope, the deformation rate is relatively small, which is much smaller than that of the conventional overburden landslide. Besides, the slopes of this type mostly show a layered disintegration failure, and will not slide on a large scale at one time. Therefore, the risk management are adopted instead of reinforcement treatment.

4.2. Sensitivity analysis of reservoir water level drop

The impact of reservoir water level changes on slope stability is largely determined by the rate of rise and fall of reservoir water level. In this paper, the “B-bar” method is used to analyze the sensitivity of the impact of sudden reservoir water level drop on slope safety on the inferred circular slip surface.

Combining the operation mode of the reservoir, and fully studying the impact of the sudden drop of the reservoir water level on the slope stability, we select the reservoir water level from the normal water level drop of 2 m, 3 m, 4 m and 5 m for analysis. Since the sudden drop of water level is an accidental working condition, this paper will no longer superimpose the earthquake. We select two working conditions of normal and heavy rain for calculation. The results are shown in Table 6.
Table 6. Slope stability analysis results under sudden water level drop

| Serial number          | Safety factor K |
|------------------------|-----------------|
|                        | Normal working condition | Rainstorm conditions | Remarks            |
| Base condition         | 1.099            | 1.032               |                    |
| Working condition one  | 1.089            | 1.020               | Sudden drop 2 m    |
| Working condition two  | 1.083            | 1.016               | Sudden drop 3 m    |
| Working condition three| 1.078            | 1.012               | Sudden drop 4 m    |
| Working condition four | 1.073            | 1.008               | Sudden drop 5 m    |

Sensitivity analysis results show that the natural conditions of a sudden drop of 2~5 m in the water level are in a basically stable state, and the rainstorm conditions are all in an under-stable state. Moreover, the greater the height of the reservoir water level drop, the greater the reduction in the safety factor of the slope. Therefore, the reservoir water level should strictly control the sudden drop rate, and the recommended water level sudden drop rate is less than 5 m/d for the slope in area B.

5. Conclusion
Overburden slope dominated by soil-rock mixture is a common geological body in nature. During the water storage process of a hydropower project, these slopes may deform and affect the operation of the hydropower station. Due to the unique genetic characteristics of the overburden slope, its spatial geological structure is relatively complex. The geomechanical parameters determined by field and laboratory tests do not match the actual operating of the slope.

In this paper, the deformation characteristics and instability modes of the research slope are analysed, and the influence of reservoir water level and rainfall on slope deformation is studied, based on monitoring data and inspections, combining with engineering geological characteristics. Furtherly, the geomechanical parameters are obtained through back-analysis, and the inversion results are applied to the slope stability evaluation. The research results show that reservoir impoundment is the inducing factor of slope deformation and rainfall is the main factor of slope stability during reservoir operation for many reservoir overburden slopes. The back-analysis method based on monitoring data is a good selection for determining geomechanical parameters. In the meantime, the sudden drop of the reservoir water level has a greater impact on the stability of the overburden slope. The rate of decrease of the reservoir water level should be reasonably controlled. The research results also provide a scientific reference for further understanding such deformable bodies and taking corresponding risk management measures.

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