



Stability analysis of bank slope under conditions of reservoir impounding and rapid drawdown

Xiaoping Chen^{1,2*}, Jingwu Huang^{1,3}

¹ College of Science and Engineering, Jinan University, Guangzhou, 510632, China

² Key Laboratory of Disaster Forecast and Control in Engineering, Jinan University, Guangzhou, 510632, China

³ Guangdong Hydropower Planning and Design Institute, Guangzhou, 510170, China

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Abstract: Stability of an ancient landslide in a reservoir area is analyzed by using centrifugal model tests, soil laboratory tests and numerical analysis. Special attention is paid to variation in water level, simulation of large-scale heterogeneous prototype slope, and strength reduction of sliding zone soils after slope sliding. The results of centrifugal model test show that reservoir impounding can reduce sliding resistance at the slope toe, followed by toe collapsing and front cracking of slope. Rapid drawdown can produce hydrodynamic pressure towards reservoir at the front of slope. Deformation is observed in the middle and upper slope, which reduces the slope stability further and forms the pull-typed landslide trend. Reinforcement of slope toe is effective for preventing the progressive failure. The results of laboratory test show that slope toe sliding will lead to the redistribution of soil density and moisture content, which will reduce the shear strength of soil in sliding zone, and the cohesion of immersed soil is reduced gradually and finally vanishes with time. The numerical results show that the strength reduction method used in finite element method (FEM) is very effective in capturing the progressive failure induced by reservoir water level fluctuations, and the evolution of failure surface derived from numerical simulation is very similar to that observed in centrifugal model test.

Key words: slope stability; reservoir impounding; centrifugal model test; strength reduction; progressive failure

1 Introduction

The landslide triggered by reservoir impounding and rapid drawdown is of great concern in geotechnical engineering. Since the centrifugal model test can reproduce the same stress level and plastic zone in a small-scale model as those presented in a full-scale prototype, it is a useful tool in slope stability analysis. The principle of centrifugal model test is to select a centrifugal acceleration to be N times that of gravity, thus the large prototype slope under gravity and the model slope with the scale of $1/N$ in the centrifugal model test experience identical physical effects. Researches on slope stability by

centrifugal model tests were focused on estimation of influential factors, such as dimensions and gradient, soil properties, excavation, particle size, boundary effect, time effect, etc. [1–6]. In addition, slope instability induced by rainfall or excess pore water pressure during earthquakes was also analyzed [7, 8].

With particular attention paid to bank slopes, most landslides are related to reservoir impounding and rapid drawdown. As a result, water level fluctuation must be simulated in a high-speed rotating centrifuge, which is very difficult. Zhang and Hu [9] designed a water control facility for water supply and drainage in centrifugal tests. Timpong et al. [10] proposed a centrifugal model, which could reflect water level fluctuation. Li et al. [11] reported a slope instability test concerning reservoir water level fluctuation. However, up to now, there is few reports on heterogeneous soil slope under rapid water level fluctuation. Another difficulty in bank slope tests is the large size of slopes, i.e. there is a contradiction

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*Corresponding author. Tel: +86-20-85220986;

E-mail: chenxp@jnu.edu.cn

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between model and prototype because the box where model bodies are placed is limited by the centrifugal acceleration N . Some different techniques were adopted to obtain similar results, such as partial simulation method and small-scale simulation method [12–15]. The former uses local slope for the whole, thus a relatively bigger error is introduced; while the latter uses the geometric scale N_s , which is larger than the gravity scale N , leading to a relatively smaller error when proper ratio of N_s / N is chosen. Because of variable conditions in field and nonlinear failure characteristics of slopes, the ratio has less universal application.

This paper aims at an ancient landslide in a reservoir area. Improved centrifugal model tests were conducted to compare the responses of natural slope and reinforced slopes under conditions of reservoir impounding and rapid drawdown, considering the real-time control of water level, heterogeneous simulation of prototype slope, and scale method with geometric scale larger than gravity scale. The strength parameters of soils in sliding zone were determined experimentally before and after centrifugal model tests. Numerical analysis of prototype slope was performed based on testing results, focusing on the strain-softening behaviors of soils after slope toe sliding.

2 Details of bank slope

2.1 Project and geology

The ancient landslide is located at the upstream of the Lechang Gorge dam, a flood control project on the Wujiang River in Guangdong Province, China. The main body of the slope is more than 200 m high and 400 m wide. Its representative cross-section is shown in Fig.1. The subsoil is constituted of sericite quartz sandstones and slates, interbedded with a thin ancient sliding zone with a thickness of 30–100 cm (sandy/gravel clay or clayey sand/gravel). When the reservoir impounds to the normal water level (154.6 m), the ancient landslide will be submerged for more than 50 m, and it may be reactivated or partially reactivated, although the slope is stable at present. Stability analysis is expected to provide a direct proof for the landslide prevention.

2.2 Soil properties

Table 1 shows the grain size distribution of the soils taken in field, and their basic physico-mechanical parameters are listed in Table 2. It can be seen that the sliding zone soil is classified as clayey soil containing sand with low liquid limit, and the slope body soil is mainly of clayey gravel.

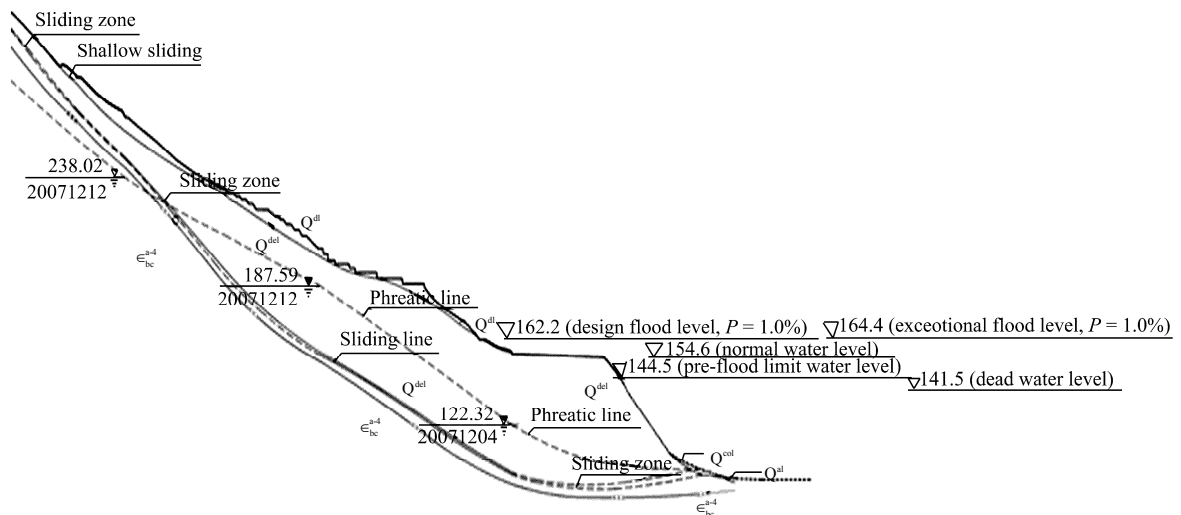


Fig.1 Schematic geological section of the slope (unit: m).

Table 1 Particle size distribution of soils.

d (mm)	Particle proportion (%)		d (mm)	Particle proportion (%)		d (mm)	Particle proportion (%)	
	Sliding zone soil	Slope body soil		Sliding zone soil	Slope body soil		Sliding zone soil	Slope body soil
> 20	0	0.7–1.9	0.5–2	12.1	31.2–55.8	0.05–0.075	34.2	57.7–66.5
5–20	4.4	3.6–33.6	0.25–0.5	13.7	32.5–59.1	0.005–0.05	69.3	73.9–92.3
2–5	7	8–34	0.075–0.25	28.7	51.6–62.6	0.002–0.005	82.1	82–95.3

Table 2 Physico-mechanical parameters of soils.

Soil	w (%)	ρ (g/cm ³)	ρ_{dmax} (g/cm ³)	e	w_L (%)	w_P (%)	I_P	I_L	w_{op} (%)
Sliding zone	16	2.06	1.87	0.567	29	16.8	12.2	-0.06	12.7
Slope body	15	2.08	2.02	0.524	34.8	18	16.8	-0.18	10

Soil	k (cm/s)	e_0	$E_{s,1-2}$ (MPa ⁻¹)	Field shear strength		Consolidated quick shear strength		Residual strength	
				c (kPa)	φ (°)	c (kPa)	φ (°)	c (kPa)	φ (°)
Sliding zone	4.78×10^{-4} – 1.60×10^{-7}	0.57	6.3	25	26	25	26	0	25
Slope body	8.07×10^{-4} – 1.40×10^{-5}	0.32–0.85	7.8–10.9			22	29		

3 Centrifugal model test

3.1 Physical similarity between model and prototype

Based on similarity law and dimensions analysis, the common similarity relation of centrifugal model test was summarized by Fuglsang and Ovesen [16]. It is defined that the gravity scale $N = a/g$, where a is the centrifugal acceleration, and g is the gravity acceleration. The general test method is dependent upon the increasing deadweight of soils in centrifuge, making $L_p g = L_m g N$, where L_p and L_m are the geometric lengths of prototype and model, respectively. This method is called “small-scale modeling” (SSM). Since the prototype slope is too large for centrifuge box, the improved method of “scaling small-scale model (SSSM)” is adopted. In this way, the geometric scale N_s is bigger than the gravity scale N , expressed as $N_s = R_s N$, where R_s is larger than 1.0. The necessary condition for R_s is the consistent plastic strain between scaled-prototype slope and its prototype, that is, the model is built with dimensions reduced by $1/(R_s N)$, but with the same density. Then an acceleration field of N times of the gravity acceleration would generate the same stresses in centrifugal model and prototype. Under the similarity principle, $L_p g = L_m N R_s$ may be taken in centrifugal model tests.

In order to perform the SSSM method in this case, R_s was first defined using numerical simulation approach, and the behavior of soil/rock was described by an elastoplastic model with the Mohr-Coulomb failure criterion. The parameters used in numerical simulations are shown in Table 3, which are obtained from laboratory tests.

Table 3 Model parameters used in numerical simulation.

Sample	ρ (g/cm ³)	E (MPa)	μ	c (kPa)	φ (°)
Sliding zone	2.06	30	0.30	25	26
Slope body	2.08	40	0.25	22	29
Bedrock	2.30	1 000	0.25	300	35

$R_s = 3$ was attained by a series of numerical simulations. It may keep the consistent plastic strain between model slope (1/3-scale prototype) and prototype slopes. The comparison is shown in Fig.2. Therefore, considering the limited volume in the centrifugal test device, $N=120$ and $N_s=360$ were taken. The physical similarity ratios of main parameters are listed in Table 4, and the dimensions of model slope are shown in Fig.3.

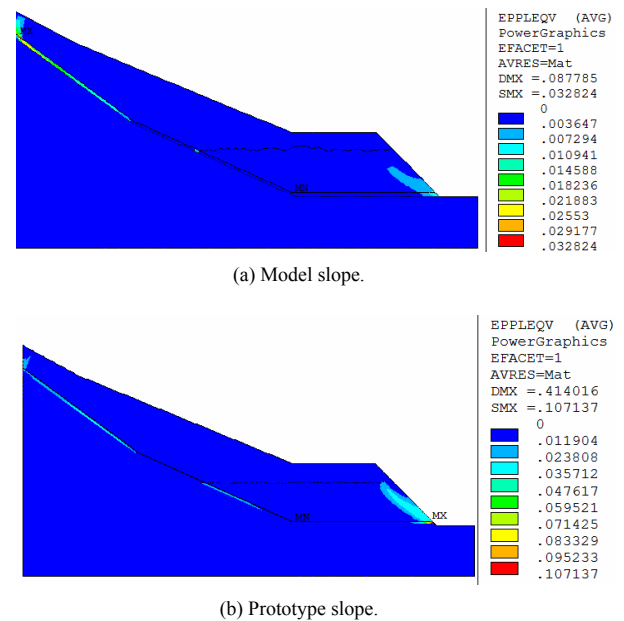


Fig.2 Comparison of consistent plastic strain between model slope and prototype.

Table 4 Physical similarity ratios of SSSM test (model/prototype).

Parameter	Similarity ratio	Parameter	Similarity ratio
Length	$1/N_s$	Dynamic time	$1/N_s$
Area	$1/N_s^2$	Static time	$1/N_s^2$
Acceleration	N	Force	N/N_s^3
Seepage velocity	N	Stress	N/N_s
Permeability	N	Strain	N/N_s
Strength index	1	Displacement	N/N_s^2

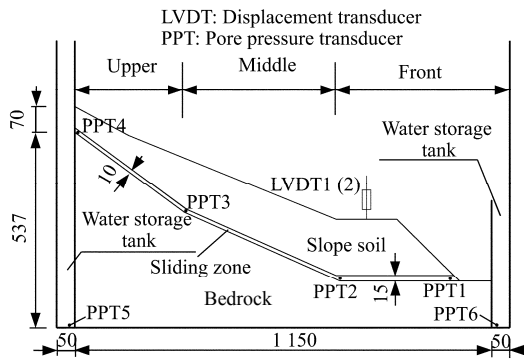


Fig.3 Model slope used in centrifugal model tests (unit: mm).

3.2 Instrument arrangement and monitoring

The real-time control of water level is important for the centrifugal model test. The control system is composed of water storage cylinder, pneumatic valves, water storage tanks, and water level measurement equipments. The water storage tanks are installed at both ends of the model box, whose walls facing the slope body have permeable holes for water flowing in and out freely. The walls are covered with a layer of geotextile so as to prevent clogging. The inlet and drainage of each water storage bank are controlled by a storage cylinder with a pneumatic valve. The flow rate and water level can also be controlled by adjusting air pressure. In addition, several pore water pressure gauges and a ruler are installed at the bottom of the tanks to monitor the water level.

The deformation and pore water pressure were continuously monitored in testing process by displacement transducers (LVDTs), pore pressure transducers (PPTs), and an auto-image storage and analysis system. The control system and monitoring devices are presented in Fig.3.

3.3 Testing procedure

The centrifugal model tests were conducted by two different models. One was the natural slope or original bank slope, showing the deformation and failure procedures with water level fluctuations. Another was the reinforced slope by the backing rock ballast at slope foot to indicate the reinforcement effect. The natural slope was constructed by compacted field soils obtained from the prototype slope. The compacted dry density of soils was considered by orthogonal tests of dry density, moisture content and shear strength. The reinforced slope was constructed by increasing the dry density in the front of the slope and decreasing the moisture content in the front of the sliding zone to simulate the field reinforcement measures. The dry densities and moisture contents after compaction are listed in Table 5.

Table 5 Dry densities and moisture contents after compaction.

Soils		Natural slope		Reinforced slope	
		w (%)	ρ_d (g/cm ³)	w (%)	ρ_d (g/cm ³)
Sliding zone	The slope	16	1.86	16	1.86
	The front slope	20	1.59	16	1.86
Slope body	The slope	15.4	1.99	15.4	1.99
	The front slope	15.4	1.65	15.4	1.99

Note: the slope means the upper and middle slope, as shown in Fig.3.

The testing procedure is listed in Table 6. The acceleration was increased to 120g in seven stages to ensure the normal consolidation of slope soil, and the deformation and pore water pressure had to be stabilized between successive stages.

Table 6 Testing procedure of centrifugal model test.

Slope	Test objective	Varying water level conditions (prototype)
Natural slope	To simulate deformation and failure occurring with water level fluctuations	(1) From the initial level at the elevation of 95.6 m to the normal level in 180 days, lasting for 60 days, then rapid dropdown to the pre-flood limit level in one day.
		(2) From the pre-flood limit level to the exceptional flood level in 2 days, lasting for 15 days, then rapid dropdown to the pre-flood limit level in 2 days, lasting for 15 days.
Reinforced slope	To simulate reinforced effect	(3) Again from the pre-flood limit level to the exceptional flood level in 2 days, lasting for 15 days, then rapid dropdown to the pre-flood limit level in one day, lasting for 30 days.

3.4 Laboratory tests on soils after centrifugal model tests

To compare the behaviors of soils involved in the sliding zone before and after slope toe sliding, laboratory tests were performed. The soil samples after slope toe sliding were obtained from the model slope after centrifugal model tests.

4 Analysis of testing results

4.1 Natural slope under reservoir impounding

Under reservoir impounding, the front border of the slope was submerged gradually with water level rising, which reduced geostatic stress and shear strength, and the submerged soil was influenced by uplift pressure (buoyancy). Therefore, the rising of reservoir water level can induce the sliding resistance

of slope. The centrifugal model tests for the natural slope reveal that some cracks appear in the front slope when the water level rises to 146.9 m, followed by local sliding at the slope toe when the reservoir water level reaches the normal water level (154.6 m). The screenshots of cracks and local sliding captured by the video of centrifugal model tests are shown in Figs.4(a) and (b), respectively. Based on the data obtained from the auto-image storage device, a series of post processings are done by professional softwares. Taking the bedrock as a reference surface, the displacement vector field of the natural slope under the normal water level is given in Fig.4(c). It can be observed that the displacement features vary along the slope: the front slope swells and slides, the

middle slope subsides, and the upper slope moves towards the reservoir. The results show that the slope soil above the water level experiences deformation when the slope toe slides, and the shear strength would be reduced gradually with the increasing displacement. Interactions between slope toe sliding and progressive reduction of soil strength are verified.

4.2 Natural slope under rapid drawdown

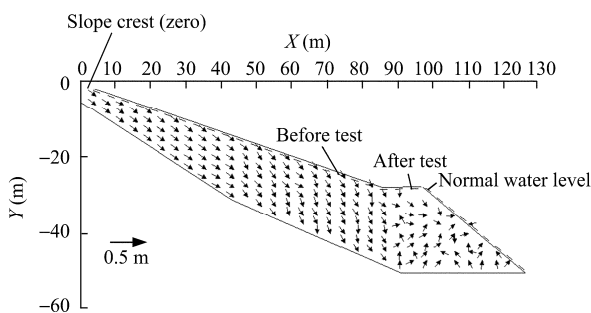
Using the same model slope, the centrifugal model tests under rapid drawdown were carried out. Figure 5 shows the testing video screenshots of landslide in the front slope and the corresponding displacement vector field. Compared with those under reservoir impounding, the slope toe sliding develops when the water level varies from 154.6 to 144.5 m (Fig.5(a)). When the drawdown is accelerated, the sliding speeds up (Fig.5(b)). Using the same post processing method described in Fig.4(c), Fig.5(c) can be obtained. Compared with Fig.4(c), the middle slope swells but does not subside, showing the failure tendency of the front slope. The displacement along the slope is characterized by expanded sliding range, increasing displacement and direct upward movement in the front slope.



(a) Cracks at the water level of 146.9 m.



(b) Slope toe sliding at the normal water level.



(c) Displacement vector field at the normal water level.

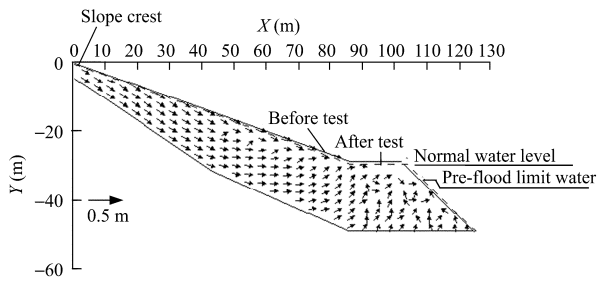
Fig.4 Natural slope under reservoir impounding.



(a) From the normal water level to the pre-flood limit level.



(b) Accelerated water drawdown.



(c) Displacement vector field with water level varying from 154.6 to 144.5 m.

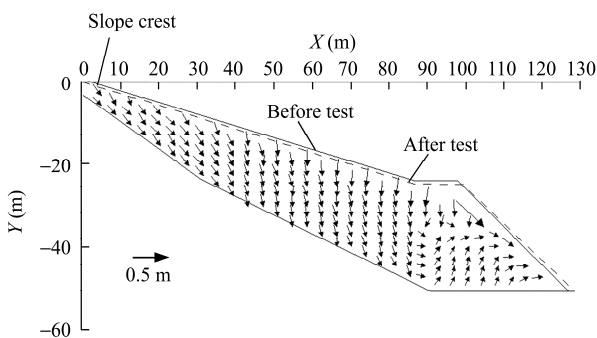
Fig.5 Natural slope under rapid water drawdown.

The testing results of natural slope indicate that the reservoir impounding causes the slope toe to slide and the slope body to deform, and rapid drawdown causes intensified sliding and increasing deformation. Cracks across the front slope are observed, and the upper sliding surface is vertical and the lower is arched. The characteristics show that the progressive failure occurs owing to soil strength reduction and dynamic water pressure, which may lead to retrogressive landslide if no reinforcing measure is taken for the slope toe.

4.3 Reinforced slope under reservoir impounding and rapid drawdown

By the comparison of various reinforcement measures, backfilling rock ballast at the slope toe was taken in the project. In order to study the reinforcement measures, the construction of reinforced model was different from that of natural slope. The dry density in the front slope was increased and the moisture content in the front of the sliding zone was decreased. The parameters of soils after compaction are listed in Table 5. Same as the natural slope, the centrifugal model tests of the reinforced slope were conducted during the processes of reservoir impounding and rapid drawdown.

The slope deformation after tests is shown in Fig.6. In Fig.6(a), the deformation can be characterized in different directions along the slope: the deformation



(a) Displacement vector field.



(b) No crack and collapsing.

Fig.6 Reinforced slope after centrifugal model tests.

of the upper slope is mainly horizontal, the middle is mainly vertical, and the front has a heaving tendency. In spite of displacement response to water level fluctuations, there is no crack or collapsing (Fig.6(b)). Compared with those of natural slope, the reinforcement measures play an important role in improving slope stability, increasing soil strength and reducing permeability. The tensile area of the upper slope is reduced, and the sliding resistance of the lower slope is enhanced. The stability of reinforced slope is improved significantly.

It can be concluded that if the submerged part of the slope could keep stable under reservoir impounding or water drawdown, the slope stability may be maintained.

4.4 Influence of slope toe sliding on soil properties

Based on the similarity principle of centrifugal model tests, the same stress level of the prototype slope was reproduced. Laboratory tests were carried out after centrifugal model tests to understand the change of soil properties. The sampling position and testing results are shown in Fig.7. The values of c and ϕ were determined by consolidated quick direct shear test.

Figure 7 shows that the soil properties are greatly affected by the slope toe sliding. Compared with the initial values of natural slope after compaction, both the density and strength of soils are reduced (Fig.7(a)). Because the slope body displacement is towards the reservoir, the soil density shows a decreasing trend from upper to lower slope sequentially, and in contrast, soil moisture content increases. The change in the surface slope is greater than that in the inner part. When the slope toe is reinforced, there is little change in soil density and moisture content, and the strength remains the same as that before tests (Fig.7(b)).

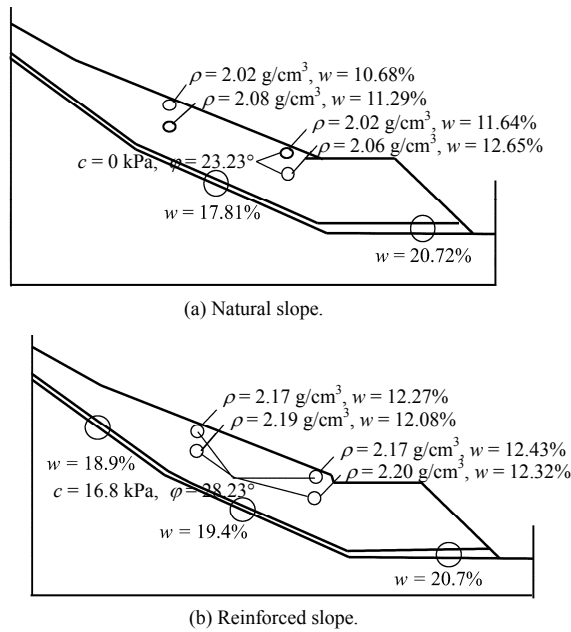


Fig.7 Results of laboratory tests after centrifugal model tests.

In order to further explore the cohesion characteristics of soils under water, a test was carried out after centrifugal model tests. A soil sample with dimensions of 20 cm×20 cm×20 cm was submerged in a pitcher, and it was observed that about 50% of the sample collapsed after 19 hours. With time elapsing, the soil sample further collapsed until it dissolved into the water totally. This indicates that the cohesion determination of the submerged soil is unreliable, which reduces drastically with the immersion time until vanishing.

The above testing results reveal that after the slope sliding, the shear strength of soils is decreased, causing the resistance of sliding zone to be reduced progressively. Consequently, a progressive failure was triggered by slope toe sliding under reservoir water level fluctuation.

5 Numerical analysis

A plane strain finite element analysis of prototype slope was performed, considering the strength reduction behaviors of soils under reservoir impounding and rapid drawdown. The ANSYS software was used in the numerical analysis, and the Mohr-Coulomb plastic law was adopted.

5.1 The slope geometry and soil parameters

The numerical analysis refers to the natural prototype slope shown in Fig.8, which is reconstructed on the basis of topographic section and centrifugal model indicated in Figs.1 and 3.

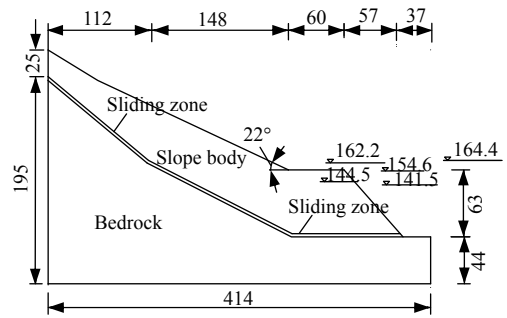


Fig.8 Cross-section considered in the numerical analyses (unit: m).

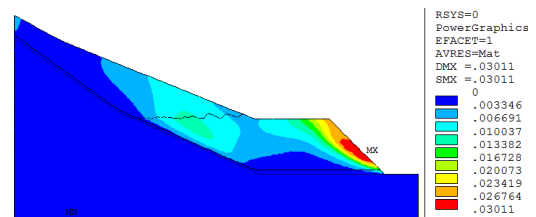
The peak and residual strength parameters considered for the soils can be derived from Table 2 and Fig.7. The strength reduction of soils was simulated by reducing the strength parameters c and ϕ with the accumulated shear strain. The soil parameters used in the numerical analysis are listed in Table 7. The values of Poisson’s ratio had reasonably been assumed for absence of specific experimental values.

Table 7 Soil parameters used in the analyses.

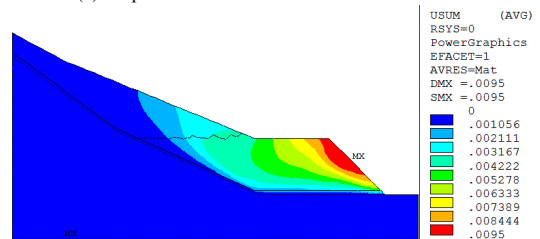
Soil	E (MPa)	μ	ρ (g/cm^3)	Peak strength		Residual strength	
				c_p (kPa)	ϕ_p ($^\circ$)	c_r (kPa)	ϕ_r ($^\circ$)
Slip zone	30	0.3	2.06	25	26	0	25
Slope body	40	0.25	2.08	22	29	12	24
Bedrock	1 000	0.25	2.3	300	35		

5.2 Analyses of numerical results

The numerical analyses for the natural slope was first performed using the peak strength parameters c_p and ϕ_p . The behavior of soils was described by an perfectly elastoplastic model with the Mohr-Coulomb failure criterion. The calculation provides the displacement field of natural slope under reservoir impounding and rapid drawdown (Fig.9). It can be



(a) Displacement field at water level of 154.6 m.



(b) Displacement field with water level from 154.6 to 144.5 m.

Fig.9 Displacement fields of natural slope under different water levels (unit: m).

seen from Fig.9 that the maximum displacement is concentrated in the slope toe (Fig.9(a)). With the rapid drawdown, it propagates to the front of the slope (Fig.9(b)).

In order to understand the strength reduction of soil, the slope stability analysis was then conducted using the strength reduction method. The initial stress state within the slope was presented by the peak strength. Then, for all the soils involved, an elastoplastic strength reduction model associated with the Mohr-Coulomb plastic law was built by reducing the shear strength parameters as the functions of shear strain ($c = c_p f(g)$, $j = j_p f(g)$). Figure 10 shows the evolution of plastic strain in the front of the slope. It can be seen that the plastic strain field develops with a clear localization in the front of the slope. The factor of safety is 0.82 (Fig.11). These imply that a progressive failure occurs, with the strength parameters varying between the peak and residual values. The results have been observed in the

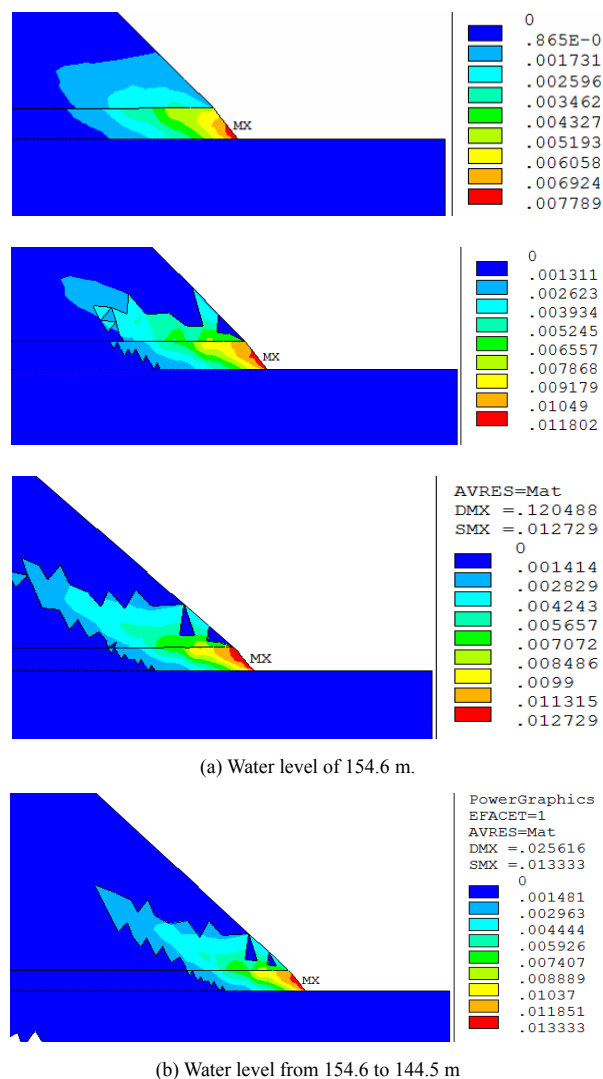


Fig.10 Evolution of equivalent plastic strain of natural slope.

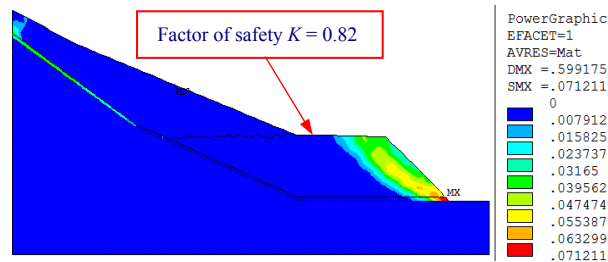


Fig.11 Critical slip surface and factor of safety under rapid drawdown.

centrifugal model tests. Therefore, the numerical analysis based on the elastoplastic model with strength reduction is effective in capturing the failure process.

6 Conclusions

Stability analysis of bank slope under reservoir impounding and rapid drawdown was presented with centrifugal model tests, laboratory tests and numerical analysis. Some conclusions can be drawn as follows:

(1) The improved centrifugal model test program was effective in real-time control of water level, considering the geometric scale greater than the gravity scale. The reconstructed reinforced slope by increasing soil density and decreasing moisture content in the front of model slope was feasible for simulating the practical reinforcement measures of backfilling rock ballast.

(2) The laboratory tests after centrifugal model tests are necessary. By comparing the test results before and after slope toe sliding, it is shown that the density and moisture content of soils redistribute, and the shear strength of soils involved in the sliding zone is reduced.

(3) The strength reduction method used in the numerical analysis is effective in capturing the progressive failure induced by reservoir water level fluctuation. The strength reduction behavior of soils can be simulated by reducing the strength parameters with accumulated shear strain. The evolution of failure surface deduced from the numerical simulation is found to be similar to that observed in the centrifugal model tests.

(4) The progressive failure of slope can be illustrated as follows: (i) reduction of slope toe resistance; (ii) slope toe sliding; (iii) reduction of soil strength near the sliding zone; (iv) further sliding of slope; and (v) further reduction of soils strength. Therefore, reinforcement of slope toe is effective for

preventing the failure of bank slope.

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