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Risk analysis for earth dam overtopping

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Abstract: In this paper, a model of overtopping risk under the joint effects of floods and wind waves, which is based on risk analysis theory and takes into account the uncertainties of floods, wind waves, reservoir capacity and discharge capacity of the spillway, is proposed and applied to the Chengbihe Reservoir in Baise City in Guangxi Zhuang Autonomous Region. The simulated results indicate that the flood control limiting level can be raised by 0.40 m under the condition that the reservoir overtopping risk is controlled within a mean variance of 5×10^{-6}. As a result, the reservoir storage will increase to 16 million m^3 and electrical energy generation and other functions of the reservoir will also increase greatly.

Key words: *overtopping risk analysis*; *earth dam*; *flood*; *wind wave*; *risk standard*

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1 Introduction

More than 86 000 reservoirs have been constructed in China, with a total storage capacity of over 4 500 trillion m^3. This is a greater number of reservoirs than in any other country in the world. They play an essential role in controlling and harvesting benefits from floods throughout China. However, for historical reasons, more than 90% of the dams are embankment dams. Accidents are potential threats to people's lives and property downstream, due to overtopping and dam breaches caused by insufficient information, inadequate survey and design, poor construction quality or improper management. From the 1950s to the 1990s, 1 147 dams suffered from overtopping during floods, accounting for 46.6% of the total number of dam failures in China during that period (Zhang and Wen 1992). About one-third of the world's dam failures have been caused by flood overtopping, which indicates that flood overtopping is an important factor affecting reservoir projects' safety. Moreover, because of a poor understanding on the randomness of floods, reservoir water levels during flood seasons are often lowered artificially in order to avoid overtopping and protect the lives and property of downstream residents. Excessive flood discharge during flood seasons leads to insufficient reservoir storage after the flood season, making the reservoir incapable of providing the expected benefits of its design. If things go on like this, not only are valuable hydropower resources wasted and the reservoir project's profit lowered, but a lot of construction capital is

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also stagnant and water enterprises are trapped in a poor economic condition in which they cannot survive and develop soundly. Therefore, the solution of the conflicts between a project's safety and its profit is a permanently important and complex problem.

Reservoir overtopping risk analysis technique is the key to solving this problem. Three different computing methods emerging from current analysis and research on reservoir overtopping risk can be summarized as follows:

(1) The method based on an observed water level sequence estimates the risk of dam overtopping by analyzing the frequency of the highest annual frontal water level. The Nonparametric Hypothesis Testing Method is used to determine the distribution of the highest frontal water level. Then, the overtopping risk can be computed according to the chosen level distribution $U(z)$. If the overtopping risk is $P_{\mathrm{f}} = P(z>Z)$, then $P_{\mathrm{f}} = 1-P(z\leqslant Z) = 1-U(Z)$, where z is the highest frontal water level and Z is the elevation of the dam crest. The highest frontal water level is the factor with the most direct influence on dam safety, so studying its variability is useful for overtopping risk analysis. However, it is a non-natural sequence affected by human factors, and the feasibility of this method is still under discussion.

(2) In the method based on the design flood, the flood control limiting level is taken as the initial flood-regulating level, the regulation model is built according to the reservoir's operation rules, reservoir routing is conducted by analyzing design floods of different frequencies, and the corresponding highest frontal water levels are calculated. Consequently, a relationship between the highest annual frontal water levels and their probabilities is established. Thus, overtopping risk can be calculated through the dam crest elevation. The obvious defect of this method lies in the assumption that the design flood frequency is equal to the frequency of the highest frontal water level, which means that the randomness of the initial flood-regulating level is ignored, as are the wind and wave factors. Therefore, this method does not correspond to the actual circumstances of the project.

(3) In the method based on random flood simulation, the annual maximum reservoir inflow flood process sequence, which comprehensively represents the statistical characteristics of the observed reservoir inflow flood, is simulated using the reservoir inflow flood randomness model, according to which the sequence of the highest frontal water level can be obtained to calculate the overtopping risk. The difficulty of this method lies in building a model reflecting the general features of the flood, so that the probability can be reliably estimated and a comprehensive analysis can be conducted. In order to meet computation accuracy needs, the simulation is very complex. Therefore, this method is limited in its practical applicability.

In this paper, a model of overtopping risk under the joint effects of floods and wind waves is established based on previous research of overtopping. In the model, the uncertainties of floods, wind waves, storage capacity of the reservoir and discharge capacity of spillways are comprehensively considered. A corresponding computer program has been developed to calculate the risk of overtopping, considering the joint effects of floods and storms on the flood control and scheduling plan of the proposed reservoir. The reservoir water level can be adjusted to improve the reservoir's profit on the condition that the dam overtopping risk is within the standard values.

2 Earth dam overtopping risk model considering the joint effects of floods and wind waves

2.1 Overtopping risk model

Dam overtopping is defined as the situation in which water flows over the dam crest because the frontal water level is higher than the elevation of the dam crest. If

$$z(t) \geqslant Z \tag{1}$$

where Z is the elevation of the dam crest and $z(t)$ is the frontal water level at time t, then dam overtopping occurs.

Overtopping risk ($R(T)$) refers to the probability of the frontal water level being higher than the elevation of the dam crest during the analytical period T (usually one year), which can be expressed as

$$R(T) = P\left(z(t) \geqslant Z\right) \quad 0 \leqslant t \leqslant T \tag{2}$$

The dam overtopping risk model that considers the joint effects of floods and wind waves can be written as

$$R(T) = P\left(z(t) \geqslant Z\right) = P\left(z_{\max} + e + h_{\mathrm{p}} \geqslant Z\right) \tag{3}$$

where $z_{\max}$ is the highest frontal water level caused by floods, e is the backwater height caused by wind and h_{p} is the swash height of wave.

2.2 Uncertainty analysis

2.2.1 $z_{\max}$

The uncertainty of $z_{\max}$ is derived from the flood, water discharge and reservoir capacity, etc. For a given reservoir dispatching regime, $z_{\max}$ is determined from flood discharge $Q(t)$, reservoir area $F(z)$ and water discharge $S(z)$, through the following relationship:

$$Q(t) - S(z) = \mathrm{d}v/\mathrm{d}t = F(z)\mathrm{d}z/\mathrm{d}t = F(z)\, f(z,t) \tag{4}$$

where $f(z,t)$ is the probability density function of water level z at time t and $Q(t)$, $S(z)$ and $F(z)$ are stochastic functions. The reservoir water level $z(t)$ at time t is computed using Eq. (4). However, Eq. (4) is a stochastic differential equation, and obviously $z(t)$ cannot be expressed as an analytical function of $Q(t)$, $S(z)$ and $F(z)$. Thus, Eq. (4) must be transformed into a linear equation through difference methods. For an unsolved function like Eq. (4), the stochastic differential equation for flood regulation can be obtained through the Runge-Kutta method, and, consequently, a four-order formula with higher accuracy is formed：

$$z(t_{i+1}) = z(t_i) + h\left(K_1 + 2\left(K_2 + K_3\right) + K_4\right)/6 \tag{5}$$

where

$$K_1 = \left(Q(t_i) - S\left(z\left(t_i\right)\right)\right) / F\left(z\left(t_i\right)\right)$$
$$K_2 = \left(Q\left(t_{i+1/2}\right) - S\left(z\left(t_i\right) + 0.5hK_1\right)\right) / F\left(z\left(t_i\right) + 0.5hK_1\right)$$
$$K_3 = \left(Q\left(t_{i+1/2}\right) - S\left(z\left(t_i\right) + 0.5hK_2\right)\right) / F\left(z\left(t_i\right) + 0.5hK_2\right)$$
$$K_4 = \left(Q\left(t_{i+1}\right) - S\left(z\left(t_i\right) + hK_3\right)\right) / F\left(z\left(t_i\right) + hK_3\right)$$

in which $z\left(t_i\right)$ represents the reservoir water level at time t_i in m, $Q\left(t_i\right)$ is the reservoir inflow in m^3/s, $S\left(z\left(t_i\right)\right)$ is the reservoir outflow in m^3/s, $F\left(z\left(t_i\right)\right)$ is the reservoir surface area in m^2 and h is time step in s.

The mean variance of $z\left(t_{i+1}\right)$ at time t is calculated using a superposed second moment method, namely

$$D\left(z\left(t_{i+1}\right)\right) = D\left(z\left(t_i\right)\right) + \left(h^2/36\right)\left(D\left(K_1\right) + 4\left(D\left(K_2\right) + D\left(K_3\right)\right) + D\left(K_4\right)\right) \tag{6}$$

where

$$D\left(K_1\right) = \left[\frac{Q\left(t_i\right) - S\left(z\left(t_i\right)\right)}{F^2\left(z\left(t_i\right)\right)}\right]^2 F_1^2\left(z\left(t_i\right)\right)\left[D\left(z\left(t_i\right)\right)\right] + \left[\frac{Q\left(t_i\right) - S\left(z\left(t_i\right)\right)}{F^2\left(z\left(t_i\right)\right)}\right]^2 D\left[F\left(z\left(t_i\right)\right)\right] + \frac{D\left(S\left(z\left(t_i\right)\right)\right)}{F^2\left(z\left(t_i\right)\right)}$$

$$D\left(K_2\right) = \left[\frac{Q\left(t_{i+1/2}\right) - S\left(z\left(t_i\right) + 0.5hK_1\right)}{F^2\left(z\left(t_i\right) + 0.5hK_1\right)}\right]^2 F_1^2\left(z\left(t_i\right) + 0.5hK_1\right)\left[D\left(z\left(t_i\right)\right) + 0.25h^2 D\left(K_1\right)\right] + \left[\frac{Q\left(t_{i+1/2}\right) - S\left(z\left(t_i\right) + 0.5hK_1\right)}{F^2\left(z\left(t_i\right) + 0.5hK_1\right)}\right]^2 D\left[F\left(z\left(t_i\right) + 0.5hK_1\right)\right] + \frac{D\left(S\left(z\left(t_i\right) + 0.5hK_1\right)\right)}{F^2\left(z\left(t_i\right) + 0.5hK_1\right)}$$

$$D\left(K_3\right) = \left[\frac{Q\left(t_{i+1/2}\right) - S\left(z\left(t_i\right) + 0.5hK_2\right)}{F^2\left(z\left(t_i\right) + 0.5hK_2\right)}\right]^2 F_1^2\left(z\left(t_i\right) + 0.5hK_2\right)\left[D\left(z\left(t_i\right)\right) + 0.25h^2 D\left(K_2\right)\right] + \left[\frac{Q\left(t_{i+1/2}\right) - S\left(z\left(t_i\right) + 0.5hK_2\right)}{F^2\left(z\left(t_i\right) + 0.5hK_2\right)}\right]^2 D\left[F\left(z\left(t_i\right) + 0.5hK_2\right)\right] + \frac{D\left(S\left(z\left(t_i\right) + 0.5hK_2\right)\right)}{F^2\left(z\left(t_i\right) + 0.5hK_2\right)}$$

$$D\left(K_4\right) = \left[\frac{Q\left(t_{i+1}\right) - S\left(z\left(t_i\right) + hK_3\right)}{F^2\left(z\left(t_i\right) + hK_3\right)}\right]^2 F_1^2\left(z\left(t_i\right) + hK_3\right)\left[D\left(z\left(t_i\right)\right) + h^2 D\left(K_3\right)\right] + \left[\frac{Q\left(t_{i+1}\right) - S\left(z\left(t_i\right) + hK_3\right)}{F^2\left(z\left(t_i\right) + hK_3\right)}\right]^2 D\left[F\left(z\left(t_i\right) + 0.5hK_3\right)\right] + \frac{D\left(S\left(z\left(t_i\right) + hK_3\right)\right)}{F^2\left(z\left(t_i\right) + hK_3\right)}$$

in which $F_1(z) = \mathrm{d}F(z)/\mathrm{d}z$. When discharge in the spillway is $S = MB\sqrt{2g}\left(z\left(t_i\right) - Z_0\right)^{1.5}$ (M is a discharge coefficient, B is the overflow width of the weir, $z\left(t_i\right)$ is the reservoir water level and Z_0 is the elevation of the weir crest), then

$$D\left(S\left(z\left(t_i\right)\right)\right) = \left(B\sqrt{2g}\left(z\left(t_i\right) - Z_0\right)^{1.5}\right)^2 D\left(M\left(z\left(t_i\right)\right)\right) + \left(1.5MB\sqrt{2g\left(z\left(t_i\right) - Z_0\right)}\right)^2 D\left(z\left(t_i\right)\right)$$

$$D\left(S\left(z\left(t_i\right)+0.5hK_1\right)\right)=\left(B\sqrt{2g}\left(z\left(t_i\right)+0.5hK_1-Z_0\right)^{1.5}\right)^2 D\left(M\left(z\left(t_i\right)+0.5hK_1\right)\right)+$$
$$\left(1.5MB\sqrt{2g\left(z\left(t_i\right)+0.5hK_1-Z_0\right)}\right)^2\left[D\left(z\left(t_i\right)\right)+0.25h^2D\left(K_1\right)\right]$$
$$D\left(S\left(z\left(t_i\right)+0.5hK_2\right)\right)=\left(B\sqrt{2g}\left(z\left(t_i\right)+0.5hK_2-Z_0\right)^{1.5}\right)^2 D\left(M\left(z\left(t_i\right)+0.5hK_2\right)\right)+$$
$$\left(1.5MB\sqrt{2g\left(z\left(t_i\right)+0.5hK_2-Z_0\right)}\right)^2\left[D\left(z\left(t_i\right)\right)+0.25h^2D\left(K_2\right)\right]$$
$$D\left(S\left(z\left(t_i\right)+0.5hK_3\right)\right)=\left(B\sqrt{2g}\left(z\left(t_i\right)+0.5hK_3-Z_0\right)^{1.5}\right)^2 D\left(M\left(z\left(t_i\right)+0.5hK_3\right)\right)+$$
$$\left(1.5MB\sqrt{2g\left(z\left(t_i\right)+0.5hK_3-Z_0\right)}\right)^2\left[D\left(z\left(t_i\right)\right)+0.25h^2D\left(K_3\right)\right]$$

where the values of $z(x)$, $S(x)$, $F(x)$ and $F_1(x)$ are all obtained from their own mean values.

2.2.2 *e* and *h*P

Generally, the rise of the water level caused by wind waves will not result in overtopping. Only when a flood raises the water level to a certain height, can the wind waves cause overtopping. Therefore, the precondition to calculating the wind speed series is the occurrence of a flood. As for the overtopping risk, only the wind blowing towards the dam body will cause overtopping, which will not happen in non-flood seasons or if the wind is blowing away from the dam body, however strong the wind is. Thus, the effective wind should be defined as wind that blows towards the dam body during a flood.

The wind-related factors causing overtopping include the backwater height caused by the wind e and the wave swash height h_{P}. Due to the randomness of wind blowing from different directions and at different speeds, the backwater height and the wave swash height are random as well.

According to the *Rolled-Earth Dam Design Standard* (MWRPI 1985) the backwater height caused by the wind can be computed as follows:

$$e=\left(KW^2D/\left(2gH\right)\right)\cos\beta \tag{7}$$

where K is a comprehensive friction coefficient whose value ranges from 1.5×10^{-6} to 5.0×10^{-6}, and is usually 3.6×10^{-6}; W is the wind speed at 10 m above the water surface in m/s; D is the fetch length of the reservoir in m; H is the average water depth of the reservoir in m; and β is the included angle between wind direction and the fetch length, which in general is $\beta=0°$.

Thus, the mean value and mean variance of e can be calculated using the first-order second-moment method:

$$\overline{e}=K\overline{W}^2D/\left(2gH\right) \tag{8}$$
$$\sigma_e=\left(K\overline{W}D/\left(gH\right)\right)\times\sigma_W \tag{9}$$

where $\overline{W}$ is mean wind speed and σ_W is the mean variance of the wind speed series.

The mean value of the swash height can be computed with the formula recommended by the *Rolled-Earth Dam Design Standard* (MWRPI 1985):

$$h_{\mathrm{P}} = \left(K_{\Delta} K_{W} / \sqrt{1+m^2}\right)\sqrt{\bar{h}\bar{\lambda}} \tag{10}$$

where

$$\bar{h} = h/1.71$$
$$h = 0.0166 W^{5/4} D^{1/5}$$
$$\bar{\lambda} = 0.389 W D^{1/5}$$

and in which K_{Δ} is the roughness permeability coefficient of the slope; K_W is an empirical coefficient determined by the non-dimensional value $W/\sqrt{gH}$, composed of wind speed W, average water depth of the water area H and gravity acceleration g; m is a gradient coefficient; h and $\bar{h}$ are, respectively, wave height and its mean value (m); and $\bar{\lambda}$ is mean wave length (m).

The mean variance of h_{P} can be converted through following formula:

$$\begin{cases} M(x) = \sqrt{0.5\pi}\mu \\ \sigma(x) = \sqrt{0.5(4-\pi)}\mu \end{cases} \tag{11}$$

where $M(x)$ and $\sigma(x)$ are the mean value function and mean variance function, respectively, and μ is a wave height distribution coefficient.

2.3 Solution for overtopping risk model

A dam's overtopping risk is calculated using the Integration-JC method based on Eq. (3). The central task of this method is the numerical integration of the discharge series Q, which is divided into several intervals, $[Q_{i-1}, Q_i]$. The probability of Q within these intervals is $P_i = f(Q_i)\mathrm{d}Q_i$, which can be used to calculate $f(Q_i)$. Q_i can be seen as a fixed value within a certain interval that is small enough. If described by a graph, $Q_i(t)$ can be seen as a definite flood hydrograph. For $Q_i(t)$, the mean value and the mean variance, $D(z_{\max})$, of the highest frontal water level $z_{\max}$, are calculated using Eq. (5) and Eq. (6), respectively. As for wind speed, W, the mean value and mean variance of e are calculated according to the maximum effective wind speed using Eq. (8) and Eq. (9), while the mean value and mean variance of h_{P} are calculated with Eqs. (10) and (11). As such, the dam overtopping risk, $P_{\mathrm{f}i}$, caused by the joint effects of $[Q_{i-1}, Q_i]$ and the maximum effective wind speed is

$$P_{\mathrm{f}i} = P\left(z_{\max i} + e + h_{\mathrm{P}} \geqslant Z\right) \tag{12}$$

in which Z and $z_{\max i}$ have a normal distribution, e has an extremum I distribution and h_{P} has a Rayleigh distribution. Because normal and abnormal variables exist in Eq. (12), P_{fi} is computed using the JC method (Wu 1990). The $P_{\mathrm{f}i}$ of each interval for the discharge series Q and wind speed series W is calculated and superposed. Then the overtopping risk is obtained as follows:

$$R = \sum_{i=1}^{n} f(Q_i) P_{\mathrm{f}i} dQ_i \tag{13}$$

3 Discussion of earth dam overtopping risk criteria

Calculation of earth dam overtopping risk aims at judging whether the risk is acceptable or not. Therefore, an overtopping risk evaluation standard is needed. When the risk assessment method was originally introduced in the dam safety field, it was based purely on economic risk criteria, including casualties estimated using an economic value index. At present, this method has basically been abandoned. The value of risk criteria should be determined on the basis of the risk levels widely accepted by the society, as the ability to bear different risks varies across countries and industries. With changes in social, economic and environmental states as well as people's psychological conditions, the allowable risk criteria change correspondingly. Different countries and organizations will propose allowable risk criteria in accordance with their own circumstances (Salmon and Hartford 1995; Krenzer 2000; Rettemeier et al. 2000).

According to data from the Australian Bureau of Statistics, the maximum mortality rate of the Australian population is approximately 1.0×10^{-4} per year, on the basis of which the *Australian Risk Assessment Guidance* (ANCOLD 2003) proposed that individual risk above 1.0×10^{-4} per year was intolerable for operating dams, and individual risk above 1.0×10^{-5} per year was intolerable for newly-constructed dams and extension projects of operating dams. In 1983, David E. Langseth pointed out in his thesis concerning spillway flood design criteria that, in order to guarantee dam safety, the failure risk should be at a level of 10^{-4}, whereas the overtopping risk caused by inadequate flood discharge should be at a level of 10^{-5} (Zhu et al. 2003). In addition, according to the statistics, 1 105 serious dam damage accidents had happened abroad by the end of 1975, including 145 accidents caused by flood overtopping. If the dam failure risk is 10^{-4}, then the dam overtopping failure risk is 10^{-5}.

From 1954 to 2001, the mean annual dam breach rate in China was 8.79×10^{-4} per year, about 4 times higher than the worldwide rate (2.0×10^{-4} per year). Since 1980, China's mean annual dam breach rate has gradually declined due to the reinforcement of dam safety management. From 1981 to 2001, the rate was 5.54×10^{-4} per year, still higher than the worldwide rate, with 1.1×10^{-4} per year for medium reservoirs (0.01–0.1 billion m^3), 2.8×10^{-4} per year for mini (Ⅰ) reservoirs (1.0–10 million m^3) and the relatively high rate of 6.4×10^{-4} per year for mini (Ⅱ) reservoirs (0.1–1.0 million m^3) (Xie et al. 2007). Generally speaking, China's mean annual dam breach rate is approaching that of Western developed countries, and many Chinese scholars have conducted significant studies on the value of dam overtopping risk criteria (Sun and Huang 2005; Zhu et al. 2003; Hao et al. 2003; Sheng and Peng 2003). However, in the absence of national and professional overtopping risk standards, an overtopping risk level of 10^{-6} may be acceptable. This is equal to the risk level of an earthquake, which means that the acceptable safety reliability of overtopping exceeds 99.999%.

In summary, a dam breach risk level of 10^{-5} is preferred abroad, while a 10^{-6} level is preferred in China. The lower the dam breach risk level is, the more easily it will be accepted.

If individual risk reaches the level of 10^{-6} , people no longer have to worry about dam overtopping risk, which is a reason for China to identify this value. At present, this level of dam overtopping risk is still difficult to achieve in China. There is a large difference between the economy, technology and public awareness of risks in China and those in Western developed countries; a level of 10^{-5} is not consistent with actual situation in China. Besides floods and wind waves, there are many other factors that can lead to dam overtopping: (1) the abatement of the gate lifting device can cause the gate to fail to open on time; (2) a flood from the watershed area of the reservoir can be underestimated, so that the flood exceeds the defensive capability of the reservoir; (3) the inflow flood caused by an upstream dam breach can exceed design flood results in a chain reaction; and (4) the reservoir can be inappropriately operated. According to statistics, dam overtoppings caused by floods and wind waves account for half of the total. Therefore, at present, it is appropriate to consider 5.0×10^{-6} the limit of dam overtopping risk caused by the joint effects of floods, wind and waves. In this study, this value was applied to a case of dam project risk research.

4 Case study

The Chengbihe Reservoir in Guangxi Province was used as a case study to compute and analyze the dam overtopping risk.

4.1 Introduction of the Chengbihe Reservoir

The Chengbihe Reservoir, located 7 km north of Baise City in Guangxi Zhuang Autonomous Region, lies on the Chengbi River, a tributary of the Youjiang River. It is part of the large-scale reservoir of grade Ⅰ project (ABCR 1998) and has a storage capacity of 1.15 billion m^3. It is a carryover storage multi-function reservoir, not only for power generation, but also for water supply, flood control, irrigation, fisheries and reservoir tours. The dam is an earth-rock dam with a maximum height of 70.40 m whose seepage prevention measure is a concrete core wall. A power station with a total capacity of 30 000 kW and an average annual energy output of 123.73 million kW/h is at the dam toe. The flood control limiting level of the reservoir coincides with its normal high-water level, 185.00 m.

The Chengbihe Reservoir uses a gate dam to discharge the flood. While the flood control limiting level of the reservoir is 185.00 m, the crest elevation of the spillway is 176.00 m. The flood control operation rule is that the opening gates will be used to keep the water level of the reservoir at 185.00 m when the reservoir inflow is less than the reservoir outflow corresponding to the flood control limiting water level; when the reservoir inflow is larger than the reservoir outflow corresponding to the flood control limiting level, the flood will be discharged through the gates.

4.2 Computation of wind regime

According to the maximum wind speed series of the Chengbihe River in flood seasons from 1970 to 2005, the mean value and mean variance of the maximum omni-directional wind speed in the flood season are calculated as $W_1 = 5.40$ m/s and $\sigma_1 = 1.46$ m/s, respectively; the mean value and mean variance of effective wind obtained through conversion are $W_y = 5.01$ m/s and $\sigma_y = 1.57$ m/s, respectively.

The wind speed values above must be converted into wind speed at 10 m above the reservoir surface, which means the wind elevation is 200.40 m. According to *The Hydraulic Design Manual* (MWRPI 1984), the wind speed can be converted from 205 m (anemoscope elevation) to 200.40 m with a conversion coefficient of K =0.96. Then the mean value and mean variance of the wind speed (m/s) at 10 m above the reservoir surface are

$$W_{10} = KW_1 = 0.96 \times 5.40 = 5.18$$
$$\sigma_{10} = K\sigma_1 = 0.96 \times 1.46 = 1.40$$

Next, the wind speed at 10 m above the reservoir surface is converted into wind speed on the reservoir surface. On the basis of *The Hydraulic Design Manual* (MWRPI 1984), the concealed coefficient of the anemoscope is $K_1 = 1.4$ and its topographic coefficient is $K_2 = 0.9$. According to W_{10} and the product value $K = K_1 \times K_2 = 1.3$, the wind speed on the reservoir surface, $W_2 = 6.70$ m/s, and its mean variance, $\sigma_2 = W_2 / W_{10} \cdot \sigma_{10} = 6.70/5.18 \times 1.40 = 1.81$ m/s, are obtained from *The Hydraulic Design Manual* (Volume Ⅳ), Table 17-4-2 on pages 4-13 (MWRPI 1984).

As a matter of fact, only the wind blowing towards the dam body will cause the overtopping. Thus, the omni-directional wind must be converted into effective wind, whose speed conversion coefficient is

$$K_W = W_y / W_1 = 5.01/5.40 = 0.93$$
$$K_\sigma = \sigma_y / \sigma_1 = 1.57/1.46 = 1.08$$

The mean value and the mean variance of the maximum effective wind speed on the reservoir surface in the flood season are, respectively:

$$\overline{W} = K_W W_2 = 0.93 \times 6.70 = 6.23$$
$$\sigma = K_\sigma \sigma_2 = 1.08 \times 1.81 = 1.95$$

4.3 Handling of various uncertain factors

(1) When the flood series satisfies P-Ⅲ distribution, its distribution density function can be calculated as

$$f(Q) = 0.0372(Q - 475.50)^{-0.705} \exp(-0.00062(Q - 475.50)) \tag{14}$$

(2) The uncertainty of the flood discharging capacity can be expressed by the discharge coefficient M, which has a normal distribution, and its mean value $\overline{M}$, which can be determined from Wu (1994). Experimental data generated by the spillway flow model indicate that the difference in flow coefficients after stabilization is 10% and the mean variance of M is $0.10\overline{M}$.

(3) The reservoir area function $F(z)$ shows a normal distribution, with the observed

value as its mean value $\overline{F}$ and $0.05\overline{F}$ as its mean variance. According to statistics, the simulated risk is apt to be safe as long as the relative error of the observed length is controlled to less than 0.1% and the mean variance is $0.05\overline{F}$.

(4) The uncertainties of the initial flood-regulating level and dam crest elevation caused by reservoir operation, management, survey and construction errors are relatively low. The mean values of uncertainties can be design values, while both of their mean variances are $0.01\ \text{m}$.

4.4 Results analysis

The wind height and wave swash height are calculated according to the mean value, 6.23 m/s, and mean variance, 1.95 m/s, of the maximum effective wind speed during the flood season. As the wind height and wave swash height are less than the depth of the reservoir, their mean value and mean variance hardly vary with changes in the initial flood-regulating level, so it is assumed that they don't vary with changes in the flood control limiting level. After normalization, the mean value and mean variance of the wind height are 0.069 m and 0.007 m, respectively, while the mean value and mean variance of wave swash height are 0.365 m and 0.165 m, respectively. The risk values of the Chengbihe Reservoir dam overtopping with different flood control limiting levels in flood seasons are calculated using Eq. (13) and listed in Table 1.

Table 1 Calculated Chengbihe Reservoir overtopping risks

Flood control limiting level (m)	Frontal level (m)		Overtopping risks	
	Mean value	Mean variance	No consideration of wave wall	In consideration of wave wall
185.00	188.57	0.0300	1.918×10^{-6}	$<10^{-8}$
185.20	188.63	0.0296	3.065×10^{-6}	$<10^{-8}$
185.40	188.75	0.0290	4.768×10^{-6}	$<10^{-8}$
185.60	188.82	0.0286	7.151×10^{-6}	$<10^{-8}$

Table 1 shows that if the flood meets the dam at the original flood control limiting level of 185.00 m, the overtopping risk without consideration of the wave wall is 1.918×10^{-6}, smaller than the allowable risk of 5.0×10^{-6}, while the overtopping risk that considers the impervious wall is lower than 10^{-8}, much smaller, illustrating that the dam overtopping risk to the Chengbihe Reservoir is apt to be minimal when the dam is operating at the original flood control limiting level of 185.00 m during flood seasons. With the continuous rise of the flood control limiting level, the overtopping risk becomes greater. When the flood control limiting level rises from 185.00 m to 185.40 m, the overtopping risk is 4.768×10^{-6} without regard to the role of the wave wall, smaller than the allowable risk of 5.0×10^{-6}. The overtopping risk is still lower than 10^{-8} when the wave wall function is taken into consideration. But when the flood control limiting level rises to 185.60 m, the overtopping risk is 7.151×10^{-6}, greater than the allowable risk. Thus, the overtopping risk to the Chengbihe Reservoir is acceptable when the flood control limiting level fluctuates between 185.00 m and 185.40 m throughout the flood season.

Based on the analysis above, a water level of 185.40 m during floods is recommended. Because the flood control limiting level of the Chengbihe Reservoir is equal to its normal storage level, the increase of 0.40 m when the reservoir level rises from 185.00 m to 185.40 m causes a storage increase of 16 million m^3 and a corresponding mean annual direct economic profit of about 100 million CNY.

5 Conclusions

In this paper, the basic theory and computation method for determining the risk of earth dam overtopping under the joint effects of floods and wind waves, in consideration of the uncertainties of floods, wind waves, storage capacity of the reservoir and discharge capacity, are proposed and applied to the Chengbihe Reservoir using reliability mathematics, stochastic hydraulics, stochastic hydrology and other related knowledge. The results indicate that increasing the reservoir level by 0.40 m can increase storage by 16 million m^3 and lead to a corresponding mean annual direct economic profit of about 100 million CNY while at the same time protecting the reservoir dam from overtopping. The authors have analyzed the overtopping risk of the Chengbihe Reservoir in a previous paper (Mo et al. 2003), in which the "simplified calculation method", which only takes into account a thousand-year frequency design flood and the maximum effective wind in flood seasons, was used to calculate the overtopping risk. The overtopping risk standard, with consideration of economic loss in a dam breach, was selected as 7.120×10^{-5}. As a result, the flood control limiting level increase was 1.60 m and the storage increase was 64 million m^3. As opposed to the method used in Mo et al. (2003), the method presented in this paper considers the interval probability combination of floods and wind waves. Thus, the computation accuracy is higher and the results are safer. In addition, as 5.0×10^{-6} is chosen as the overtopping risk limit, taking no account of economic loss factors, its result is more reasonable than that of the simplified calculation method. Consequently, the results can provide some decision-making support for the reservoir management department, and the overtopping risk analysis method proposed by the authors can be applied to other operating earth dams as well.

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