An assessment of long-term overtopping risk and optimal termination time of dam under climate change

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ABSTRACT

Reservoir management faces a wide range of new challenges resulting from the impact of climate change. One set of challenges arises from the non-stationary nature of hydrological conditions. Another crucial issue is watershed sedimentation, which can significantly influence the sustainability and safety of reservoirs. To address these concerns, this study developed a framework for the management of reservoir risk. An analytical conceptual model coupling physical governing relationships and economic tools was proposed, which was then applied to the Shihmen Reservoir in Taiwan. We adopted a statistical representation of future hydrologic conditions with the assumption of time-variant moments and focused on evaluating the impact of an increase in the frequency of extreme hydrological events caused by climate change and used a stochastic approach to quantify the risk factors. Our results confirm that this approach can be used to identify reservoir-related risks and generate appropriate options for strategy and policy.

We determined that the major source of risk is the hydrological conditions, especially the extreme events. More severe intra-annual climatic change is much more dominant in the risk compared to inter-year trends. The influence of reservoir characteristics on risk is associated mainly with the availability of flood control capacity, but limited due to the limitation of its volume and potential to regulate the flow. Engineering may provide an option for mitigating the risk, but integrated, watershed-level approaches, such as providing systematic detention or land use management, are better suited to reducing the storm peak from a long-term perspective. With a critical increase in the risk of overtopping, a high probability of dam failure and corresponding losses may precipitate the need to retire or remove the facility. However, because the benefits and costs are both huge, the decision may be biased by a conservative attitude. The outcome of small facilities failing may be considered more acceptable than similar events besetting larger systems.

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1. Introduction

As an essential element in the management of water resources, reservoirs have made an important contribution to human development. Currently, reservoir management faces a wide range of new challenges resulting from the impact of climate change. One set of challenges arises from the non-stationary nature of hydrological conditions, which in recent decades has caused an increase in the frequency and severity of extreme weather events globally. These extreme events, such as flooding, increase the risk of reservoir safety. One crucial concern is watershed sedimentation, which can significantly influence the sustainability and safety of reservoirs. Deposited sediment reduces reservoir storage capacity, which limits functionality and diminishes the lifespan of the facility. Furthermore, the loss of flood control volume required to attenuate peaks in supply can compromises the safety of the impoundment structure. When flood levels exceed the designed spill capacity of reservoirs, the likelihood that dams will be overtopped increases.

Risk analysis is useful in quantifying the risks associated with dams and determining the impact of dam failure (Bowles et al., 1997; Thompson et al., 1997). Risk-based analysis methods are employed in a wide range of applications associated with hydraulic engineering (Tung and Mays, 1980, 1981; Yen and Tung, 1993; Stedinger, 1993). Thompson et al. (1997) compiled a systematic/classified list of major studies dealing with risk analysis for the evaluation of dam safety. Kuo et al. (2007) conducted risk analysis for overtopping events in the Feitsui Reservoir using five different computational methods.
Hydrological conditions, particularly flood characteristics, are driving forces in dam overtopping (Bowles et al., 1998; Stedinger, 1993). Traditionally, hydrologic frequency analysis and engineering designs have relied on the assumption that hydrologic patterns conform to a stationary, independent, identically distributed random process (Hossain et al., 2010). Today, this assumption is being called into question due to the effects of climate change and human disturbance (Milly et al., 2008; Sveinsson et al., 2009; Pielke, 2009). Atmosphere–Ocean General Circulation Models (AOGCMs) have been widely adopted to represent climatic conditions in the future, and coupled AOGCMs form the basis of the vast majority of impact studies. Nonetheless, the reliability of AOGCM projections remains an issue of debate (Randel and Wiedlicki, 1997; Shackley et al., 1998; Henderson-Sellers and McGuffie, 1999; Petersen, 2000). Besides AOGCMs, time-dependent stochastic models have proven a useful alternative for dealing with hydrological non-stationarity due to their rigorous statistical formalism. This approach relies on time series of observation and measurement data as the primary source of information. One such method involves the application of an identification of distribution trend (IDT) model to simulate trends according to the statistical moments of the probability density function (PDF), rather than its original parameters (Strupczewski et al., 2001; Rigby and Stasinopoulos, 2005; Villarini et al., 2009). This approach has proven an effective alternative to more sophisticated AOGCMs and is far more easily applied (Wilks, 1992).

Reservoir sediment distribution tends to increase the severity of sediment-related problems. As mentioned in Morris and Fan (1998) and Salas and Shin (1999), accurately predicting sediment yield in reservoirs is extremely difficult due to the complexity of the process, involving factors such as sediment production, sediment transportation rate, sediment type, mode of sediment deposition, reservoir operations, reservoir geometry, and streamflow variability. Various uncertainties are inherent in the estimation of sedimentation and accumulation in reservoirs, preventing the process from being treated as deterministic. Obtaining a long-term perspective on reservoir sedimentation requires a stochastic description (Meade, 1982; Kelsey et al., 1987; Salas and Shin, 1999; White, 2005). Several studies have used stochastic theory to explore the problem of sedimentation. Gani and Yakowitz (1989) developed a probabilistic model to estimate the life of reservoirs and the probability of failure by considering the rainfall-runoff process, sedimentation, sediment transport, and reservoir storage and release. Salas and Shin (1999) performed uncertainty analysis of sediment data related to deposition in large reservoirs. Their findings suggest that uncertainty in the estimation of sediment type and trap efficiency is small compared to the difficulties associated with the estimation of annual streamflow and sediment inflow required for the application of sediment rating curves.

A better understanding of reservoir sediment and the potential risk of failure could help to evaluate the life span of reservoirs more effectively. Baecher et al. (1980) stated that potential future losses resulting from dam failure should be incorporated in a cost-benefit analysis, where the risk of dam failure is recognized as a real cost in evaluation. They also indicated that estimating the probability of failure is critical. Paté-Cornell and Tagaras (1986) studied how the probability of failure varies with time to demonstrate that expected risk-related costs can be significant when the dam has a high potential for flood loss and relatively small benefits. In a continuation of previous studies, Cochrane (1989) addressed problems associated with double counting in the computation of costs related to the risks and benefits of rebuilding after a dam break. Evans et al. (2000) indicated that a loss of storage capacity is a long-term progressive process, making the dam increasingly vulnerable to failure during high-magnitude flooding and continuously reducing the economic benefits provided by the installation. Palmieri et al. (2001) explored this issue by providing a framework for assessing the life of dams. Their results indicate that the economic viability of a reservoir depends on physical, hydrological, and financial parameters. Accordingly, they applied the framework to evaluate the current situation rather than potential problems, and therefore excluded the risk of dam failure.

To address the above concerns, this study developed a framework for the management of reservoir risk, adopting a statistical representation of future hydrologic conditions with the assumption of time-variant moments. We focused on evaluating the impact of an increase in the frequency of extreme hydrological events caused by climate change and used a stochastic approach to quantify the risk factors. First, we developed an analytical conceptual model coupling physical governing relationships and economic tools, which was then applied to the Shihmen Reservoir in Taiwan. Our results confirm that this approach can be used to identify reservoir-related risks and generate appropriate options for strategy and policy.

2. Methodology

This study first developed an abstract model capable of providing a reasonable representation of the response that reservoir watershed systems exhibit with regard to long-term climatic conditions. Based on these results, we proposed an economic framework to evaluate the possible outcome of system failure, which was applied to a discussion of potential management implications.

2.1. A stochastic precipitation model

To obtain an appropriate representation of future climatic conditions, we applied an identification of distribution trend (IDT) model, using probability distribution and a time trend in the first two statistical moments to account for climatic trends. For annual precipitation, gamma distribution generally provides a good fit to rainfall data and enables the amounts to be expressed probabilistically (Thom, 1958; Bradley et al., 1987; Clarke, 1979; Aksoy, 2000). Using the IDT model, a time-variant distribution of P_t, annual precipitation in different years, is assigned by presenting the probability of failure during high-magnitude floods. In a continuation of this work, we applied an identification of distribution trend (IDT) model to simulate trends according to the statistical moments of precipitation in different years, is assigned by presenting the probability of failure during high-magnitude floods. Evans et al. (1999, 2000) explored this issue by providing a framework for assessing the economic benefits associated with double counting in the computation of costs related to the risks and benefits of rebuilding after a dam break. Evans et al. (2000) indicated that a loss of storage capacity is a long-term progressive process, making the dam increasingly vulnerable to failure during high-magnitude flooding and continuously reducing the economic benefits provided by the installation. Palmieri et al. (2001) explored this issue by providing a framework for assessing the life of dams. Their results indicate that the economic viability of a reservoir depends on physical, hydrological, and financial parameters. Accordingly, they applied the framework to evaluate the current situation rather than potential problems, and therefore excluded the risk of dam failure.

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2.2. Annual maximum peak discharge

Extreme weather records have been broken every year in the last decade and changes in the magnitude and frequency of annual maximum peak discharge events increases the risk to dams. In many areas of the world, these events account for a significant proportion of annual precipitation; rainfall accumulated from a single heavy rain during the monsoon, a typhoon, or hurricane, can contribute several deciles of annual rainfall, in some cases exceeding 50%. To represent annual extreme events, this study applied a linear statistical disaggregated model, which considers the amount of rainfall in extreme events as a portion of annual precipitation. For simplicity, the portion is assumed to increase linearly with time, as supported by several previous studies (Groisman et al., 2005; Karl and Knight, 1998; Manton et al., 2001; Tam, 2006). The model is formulated as,

\[ P_{E,t} = c_t \cdot P_t + \xi \]

\[ c_t = c_0 + w_3 \cdot t \]  

(3)

where \( P_{E,t} \) is the amount of rainfall during extreme events, \( c_t \) is the ratio of \( P_{E,t} \) to \( P \) and \( c_0 \) is its initial value. When \( c_t \) is zero, the annual maximum rainfall event and the annual precipitation are fully independent. \( \xi \) represents the independent variation in the amount of rainfall associated with extreme events. Independent variation is determined in the form of normal and extreme value type I (EVI) distribution, with zero mean and \( \sigma_E \) as the standard deviation.

The rational equation is the simplest but most useful method for determining peak discharge from drainage basin runoff. The rational equation includes stochastic elements to provide a realistic simulation of peak runoff discharge (Chow et al., 1988; Arnold et al., 1998). The equation is written as

\[ Q_{p,t} = \frac{2}{T_d \cdot C_D} \cdot A \cdot C_R \cdot P_{E,t} \]  

(4)

where \( Q_{p,t} \) is peak discharge, \( A \) is the area of the reservoir basin, and \( C_R \) is the run-off coefficient. \( T_d \) is the duration of the rainfall event. The direct runoff duration is \( T_d \cdot C_D \) where \( C_D \) is a dimensionless coefficient. Except \( P_{E,t} \) and \( Q_{p,t} \), other parameters are treated as time-invariant. Fig. 1 presents a schematic diagram of the concept behind the process of rainfall-runoff.

2.3. Reservoir sedimentation

Estimating annual reservoir sedimentation and accumulated reservoir sedimentation through time. Sediment supply, transport, and deposition (the primary aspects of the sedimentation process) are not linearly dependent on causal factors. A comprehensive understanding of the dynamic nature of sediment yield or predictions to that end has yet to be achieved. The state of the art, the Universal Soil Loss Equation (USLE) provided by Wischmeier and Smith (1978) has been widely applied on the watershed scale as the basis for a lumped approach to catchment scale (Williams and Berndt, 1972, 1977; Griffin et al., 1988; Dickinson et al., 1998; Jain et al., 2001). Salas and Shin (1999) confirmed that uncertainty in annual streamflow is the most important factor influencing uncertainty in reservoir sedimentation. As a result, this study applied a model modified from the USLE method as well as Salas and Shin (1999) to approximate the sedimentation and accumulation of reservoirs as,

\[ V_t = C_{S1} \cdot P_{t}^C{S2} \]

\[ \sum_{t} V_t = C_{S1} \cdot \sum_{t} P_{t}^C{S2} \]  

(5)

where \( V_t \) is annual reservoir sedimentation as a power function of annual precipitation \( P_t \). \( C_{S1} \) and \( C_{S2} \) are the two coefficients involved in watershed sedimentation. \( C_{S1} \) is recognized as a lumped factor of watershed characteristics, such as erodibility, topographic and geological conditions, land use, and land cover. \( C_{S2} \) determines how sediment production relates to hydrological conditions.

Assuming that the initial capacity of the reservoir at \( t = 0 \) is \( S_0 \), the effective capacity at \( t \) year can be calculated as

\[ S_t = S_0 - \sum_{1}^{t} V_t \]  

(6)

In Equation (6), \( S_t \geq 0 \). A decrease in \( S_t \) to zero means that the reservoir is filled entirely, thereby losing any available capacity and its original function. Sediment management approaches such as sediment routing/flushing can mitigate the loss of the capacity. However, the efficiency of these approaches is still limited and sediment balance is still difficultly achieved. This study would only conservatively consider this issue without modeling any management practices.

2.4. Reduction of flood control volume

In particular, a reduction in effective flood control volume due to reservoir sedimentation may increase the risk of dam failure (Bashar and Khalifa, 2009; López-Pujol and Pnseti, 2006; Navas et al., 2009). This study estimated the loss of effective flood control volume by assuming geometric similarity in the deltaic deposition (Fan and Morris, 1992). The reservoir’s flood control capacity, \( D_t \), at \( t \) year is represented as

\[ D_t = D_0 \cdot \left( \frac{S_t}{S_0} \right)^{\eta} = a_2 \cdot S_0 \cdot \left( \frac{S_t}{S_0} \right)^{\eta} \]  

(7)

where \( a_2 \) is the ratio of flood control volume, \( D_0 \), to reservoir capacity, \( S_0 \), in the initial year and \( \eta \) is the geometric coefficient of deposition between of 0 and 1. \( \eta = 1 \) represents identical deltaic deposition.

Flood control capacity equals the amount of water the reservoir can regulate during flooding. The regulation can be difficult due to
the need for reliable inflow forecasting, which is not always available. Fig. 1 demonstrates the possible regulation of inflow and outflow from the reservoir. The solid line represents the triangular inflow hydrograph used in this model; the dashed line represents the regulated outflow hydrograph; the shadow area presents the flood control volume. The original peak of inflow is \( Q_p \) and the outflow peak after regulation is \( Q'_p \). The relationship between \( Q_p \), \( Q'_p \), and \( D_t \) is expressed as
\[
Q'_{p,t} = Q_p - 2D_t \quad (8)
\]

2.5. Risk of overtopping

Two direct factors, insufficient flood storage and inadequate spillway capacity are the main causes of dam overtopping. After the flood has been regulated, if the flood outlet facilities remain unable to release water quickly enough, the water level will rise above the dam crest, resulting in overtopping. We define the overtopping risk, \( R_{OT} \), at year \( t \) as
\[
R_{OT,t} = P\{Q'_{p,t} > Q_s\} \quad (9)
\]
where \( P\{\cdot\} \) is the exceedance probability, and \( Q_s \) is the spillway capacity. The dam is assumed to have failed when it has overtopped.

3. Numerical simulation

Providing analytic derivation of risk \( R \) is nearly impossible due to the complexity of the system. As an alternative, this study employs numerical simulation to investigate the impact of various factors on risk. Latin hypercube sampling (LHS) (McKay, 1988; Iman, 1999) was adopted to represent the uncertainty of various input factors to determine the probabilistic risk of overtopping. A general case was studied using this framework. Maintaining generality allows the consideration of various parameters to gain a deeper understanding of the issue. A degree of standardization in the parameters was required to simplify the analysis. Precipitation, \( P_t \), was standardized with respect to \( \mu_P \); therefore, the initial mean was \( \mu_P/\mu_0 = 1 \). The \( \sigma_P/\mu_0 \) is a standardized representation of standard deviation of annual precipitation. The parameters of the baseline scenario and other scenarios tested in the numerical model are listed in Table 1. According to previous studies, the estimation of Probable Maximum Precipitation (PMP) and Probable Maximum Flood (PMF) return periods is approximately \( 10^5 \)-\( 10^7 \) years (NRC, 1994; Koutsoyiannis, 1999; Douglas and Barros, 2003). This study used a return period of 2000 years as \( Q_s \), the threshold for overtopping. The ranges of other parameters are also shown in the table.

Fig. 2 demonstrates the concept used to determine the risk of overtopping. Using LHS, the overtopping risk \( R_{OT} \) can be estimated, as shown in Fig. 2(a). By making probabilistic \( P_t \) and \( \xi \) the two parameters driving uncertainty and the other parameters deterministic quantities, we can determine the risk of overtopping, which is equivalent to the probability of regulated outflow exceeding spillway capacity. When the distributions of \( P_t \) and \( \xi \) vary with time, the risk of overtopping changes each year. Risk versus time is plotted in Fig. 2(b). In the initial year, the overtopping risk was 0.0033, as 3000 year return period, when the effect of flood control capacity was considered. With the passage of time, the long-term risk increases with a power exponent of 0.018.

\( \sigma_P/\mu_0 \) and \( \sigma_\xi/\sigma_0 \) and \( c_0 \) are parameters characterizing the present hydrological conditions. Their influence is very straightforward. \( \sigma_P/\mu_0 \) is the annual fluctuation in precipitation, and \( \sigma_\xi/\sigma_0 \) is the variability in extreme events. Both increase the risk of overtopping under more severe flood conditions. \( c_0 \) is the initial relative importance of an extreme event in annual precipitation. These parameters determine the initial risk but have less influence on long-term changes in risk.

Long-term change is described using three temporal parameters of the IDT model, \( w_1w_2w_3 \). The corresponding change in the risk of failure with respect to these parameters is on the order of one, and the return period can reduce from 2000 years to a hundred years in the coming decades. Fig. 3(a) shows the influence of \( w_1 \) (the increase rate of annual average precipitation), \( w_1 \) is set between 0.1% and 1% per year. The increase in risk gradually increases with time. A larger value for \( w_1 \) is accompanied by an increase in the rate of risk. As with \( w_1 \), the value of \( w_2 \) (increase in annual precipitation S.D.) is given between 0.1% and 1%. As shown in Fig. 3(b), \( w_3 \) leads to a relatively lower increase in risk in the initial few decades, but quickly accelerates later. Fig. 3(c) shows the results of an increasing rate of annual rainfall for extreme events over the long-term. This study assigned the value for \( w_3 \) to be between 0.05% and 0.25%. Compared with \( w_1 \) and \( w_2 \), the risk has a relatively high degree of sensitivity toward \( w_3 \), which resulting in a significant increase in the overtopping risk over the entire period.

Following a discussion of reservoir characteristics, we included changes in flood control capacity as well as other factors, which indirectly result in risk trends associated with overtopping. \( S_0/\mu_0 \) is the initial standardized reservoir capacity that provides greater potential with regard to alleviating the negative impact of dam safety. As shown in Fig. 4(a), case \( S_0/\mu_0 = 0.1 \) represents a small reservoir completely filled with sediment in approximately the 40th year. The risk of overtopping in this case increased steadily over time. When \( S_0/\mu_0 \) 0.15 and 0.25, the reservoir is completely filled in approximately the 70th year and at the end of the period, respectively. After losing all reservoir capacity, the risk trends should theoretically coincide. Because the reservoir has no flood control function, hydrological conditions are the only factors influencing the risk of overtopping. The figure shows that the risk trends of \( S_0/\mu_0 = 0.15 \) and 0.25 approach \( S_0/\mu_0 = 0.1 \) after the reservoir is filled with sediment. Similar phenomena can be observed in the other two factors related to reservoir characteristics: the type of deposition \( \eta \) and the ratio of flood control volume to reservoir capacity (\( D_0/S_0 \)). In Fig. 4(b), the risk shows nearly no change with differences in \( \eta \) because it is the amount of sediment,
not the way sediment accumulates, that alters the lifespan of reservoirs. Fig. 4(c) illustrates the influence of the initial ratio of flood control volume to reservoir capacity, $D_0/S_0$. In the first seven decades before the reservoir is completely filled with sediment, a higher flood control volume may be helpful to a degree. Afterward, the risk trend is indifferent toward the loss of flood control capacity. The influence of reservoir characteristics on risk is associated mainly with the availability of flood control capacity. Due to the limitation of its potential to regulate the flow, risk only slightly increases compared to the temporal parameters discussed.

This study also investigates the impact of watershed characteristics on the pattern of sediment production. Fig. 5 presents the results of various runoff coefficients and lag times, both describing the hydrological conditions of the watershed. Our results show that the runoff coefficients are negatively correlated to risks. This study found that $C_R = 0.8$ is several times riskier than the initial value of $C_R = 0.6$, while only a 33% increase in runoff is expected (Fig. 5(a)). Fig. 5(b) shows the long-term risk results of the lag time factor. The coefficient and risk are negatively correlated. With $C_S$ lying in the range of 1.1–1.5, the return period of overtopping could change from 2000 years to 300 years. The influence of sediment production is similar to that of reservoir characteristics, which reflect a reduction in available flood control capacity. Fig. 6 shows the long-term influence of the coefficients of sediment production, $C_{S1}$ and $C_{S2}$. As shown in the figure, both parameters slightly alter the risk, but only before the reservoir is completely filled with sediment. In addition, the scale parameter, $C_S$, may have a more direct impact on reservoir sedimentation, compared to the power law exponent $C_{S2}$.

In addition to the overtopping risk, this study simulated changes in reservoir capacity over time, as a result of sedimentation. Fig. 7 illustrates the temporal change in standardized reservoir capacity ($S_t/\mu_0$) of the baseline scenario. The probabilistic characteristics of capacity are presented using the mean, as well as the 5th and 95th percentiles. The results show that the change in reservoir capacity is somewhat concave, but very nearly linear with respect to time. The distribution becomes less concentrated due to the uncertainty accumulation. As a result, the 90% confidence interval of reservoir
capacity gradually widens. However, we did not observe obvious nonlinearity in the change of confidence interval. The impact of losing reservoir capacity is analyzed in Fig. 8. Because our model recognizes sedimentation as an annual phenomenon, we only discussed the following parameters: $S_0/\mu_0$, $w_1$, $w_2$, $C_{S1}$ and $C_{S2}$. For the sake of clarity, we only plotted two extreme scenarios in the figure to provide a comparison with the base scenarios.

As mentioned, the influence of initial capacity, $S_0/\mu_0$, is straightforward, although this characteristic has a nonlinear influence on the risk of overtopping. As shown in Fig. 8(a), a larger initial capacity linearly extends the life span of the reservoir without altering the temporal patterns of sedimentation. This result also demonstrates that climatic parameters, $w_1$ and $w_2$, have less influence on the capacity of the reservoir, compared to the parameters of watershed sedimentation, $C_{S1}$ and $C_{S2}$. The increase in annual average precipitation ($w_1$) accelerates the sedimentation rate and the annual increase in precipitation S.D ($w_2$) exacerbates uncertainty related to the loss of capacity. With regard to the coefficient of sediment production, scale parameter, $C_{S1}$, reflects the characteristics of the watershed, with a direct influence on the quantity of sediment produced in the watershed. When $C_{S1}$ increases by approximately 1.66 times (equal to 1/0.6; from 0.015 to 0.025), the time taken by the sediment to fill the reservoir is reduced by a factor of 0.6 (63 year $\sim$ 38 years). A simple linear relationship can be found between the time required for the reservoir to fill and scale parameter, $C_{S1}$. The power law exponent coefficient, $C_{S2}$, which dominates the nonlinearity of sediment production with respect to rainfall, could also increase the rate of reservoir sedimentation; however, it has a more significant influence in uncertainty. As shown in Fig. 8(e), when $C_{S2}$ increases from 1.5 to 5, the time required for the reservoir to be entirely filled by sediment could be as much as a decade less; however, considerable uncertainty remains. The 90% confidence interval of the time required for sediment to fill the reservoir completely increases from 7 years to approximately 20 years. The change in reservoir capacity still shows a strong linear relationship with time. Barring a sudden alteration to the hydrology or characteristics of the watershed, sedimentation can generally be treated as temporally consistent behavior within the gradual change of climatic conditions.

4. Economic benefit-cost analysis of dam failure model

After a critical increase in the risk of overtopping, maintaining reservoir operations may not be advisable. A high probability of dam failure and corresponding losses are serious concerns that may precipitate the need to retire or remove the facility. This raises the question, when is the optimal time to retire the reservoir? The model below is intended to answer this question.
Modifying the framework proposed by Palmieri et al. (2001), we assumed the decision making is based on the benefit over the operating lifespan of the facility. This model considers net benefits $B_t$ in each period $t$, construction cost $C_i$, maintenance cost $C_M$, and the salvage value $U$. Assume that the reservoir has been safely operated for period $T = \tau$, the present value of overall benefit can be written as

$$U_T = \sum_{t=0}^{T-\tau} \left( \frac{B_t - C_M}{(1 + r)^t} + C_i + \frac{U}{(1 + r)^{(T-1)}} \right)$$  \hspace{1cm} (10)

where $r$ is the discount rate used to account for the time preference.

At the end of $\tau$th period, we need to determine whether to continue operating the reservoir. Operating the reservoir for another period, $\tau + 1$, would result in an additional benefit $B_{\tau+1}$ as well as a regular maintenance cost $C_M$, and most importantly, the risk of overtopping and associated costs $COT$. The overall expected benefit in operating the reservoir will become,

$$U_{T=\tau+1} = \sum_{t=0}^{T-\tau-1} \left( \frac{(B_t - C_M)}{(1 + r)^t} + C_i + \frac{COT_{\tau+1}}{(1 + r)^{(\tau+1)}} \right) + \frac{U}{(1 + r)^{T}}$$  \hspace{1cm} (11)

The decision is straightforward if we compare the overall expected benefits of horizons of periods $\tau$ and $\tau + 1$. Subtracting Eq. (10) from Eq. (11), we obtain the condition of optimal terminal time:

$$\begin{align*}
\text{Fig. 5.} & \text{ The change of risk of failure with respect to the characteristics of the watershed.} \\
\text{(a) } C_R & \\
\text{(b) } C_D
\end{align*}$$

$$\begin{align*}
\text{Fig. 6.} & \text{ The change of risk of failure with respect to the sediment production parameters.} \\
\text{(a) } C_{S1} & \\
\text{(b) } C_{S2}
\end{align*}$$
Eq. (12) can be understood as the additional benefit obtained in period \( t + 1 \). A negative value means that continuing reservoir operations is ineffective; therefore, retiring the reservoir is more economical.

One essential objective of this study was to investigate the impact of dam failure as a result of overtopping. In Eq. (12), both \( C_M \) and \( \Omega \) could be treated as fixed. The operating benefit, \( B_t \), is a function of available capacity within the reservoir. As a result, \( B_t \) monotonically decreases with time \( t \). \( C_{OT} \) is equal to the expected risk multiplied by the loss due to failure, as follows:

\[
C_{OT} = R_{OT} \cdot L
\]  

(13)

This term is the key factor influencing the system. According to previous analysis, the risk increases exponentially with time. The trend of change in the losses related to dam failure over time might be ambiguous and require further examination.

Previous studies have considered the losses due to dam failure, as they pertain to flood damage downstream due to a sudden release of water from the reservoir. In the event of a dam failure, the energy of the water stored behind the dam is capable of causing rapid and unexpected flooding downstream, resulting in a loss of life and immense property damage. From this perspective, the failure of a sediment-filled dam may be negligible because no water is stored behind it. However, this inference is not consistent with previous experience (Costa, 1988; Evans et al., 2000; Macías et al., 2004; Alcrudo and Mulet, 2006; Capart et al., 2007). A break in a sediment-filled dam could result in a surge of sediment into channels, discharging debris or streamflow heavily laden with sediment downstream. A less obvious but persistent threat is the
mobilization of fine-grained sediment trapped within the reservoir. The failure of an upstream structure may alter the sediment budget and threaten the integrity of downstream structures.

Considering the points raised in the above discussion, assessing losses due to dam failure requires that the impact of both water and sediment volume be taken into account. This study simply separates them into two individual terms, \( L_Q \) and \( L_S \), losses due to water and sediment release, respectively. Losses due to water release flooding are usually estimated using a damage function according to the depth of inundation, approximately proportional to the flood peak discharge (Baecher et al., 1980; Costa, 1985; Paté-Cornell and Tagaras, 1986). Many empirical formulations have been developed for the prediction of dam breach characteristics and peak outflow, based on the hydraulic and geometrical properties of dams and reservoirs. In general, peak discharge downstream can be expressed as a power function of the product of volume and head of water in the reservoir triggering the failure with some exponent (Hagen, 1982; MacDonald and Monopolis, 1984; Costa, 1985, 1988; Froehlich, 1995; Broich, 1998; Zagonjolli and Mynett, 2006). Because the head of the reservoir remains unchanged, the loss of failure can be represented as

\[
L_Q = \alpha_Q \left( \frac{S_t}{S_0} \right)^{\beta_Q} \tag{14}
\]

\( \alpha_Q \) is a scalar and \( \beta_Q \) is the exponent coefficient. Because the reservoir capacity, \( S_C \), decreases with time, \( L_Q \) also decreases until the storage is completely filled.

As for the loss due to sediment, \( L_s \), very limited guidance is available in the literature. Only a few studies can be found, and these provide only observations and qualitative descriptions (Costa, 1988; Evans et al., 2000; Macías et al., 2004). This study assumed \( L_s \) is a power function of sediment held in the reservoir in a form similar to that of \( L_Q \) which can be written as

\[
L_S = \frac{N S}{S_0} \alpha_S \left( 1 - \frac{S_t}{S_0} \right)^{\beta_S} \tag{15}
\]

\( \sum V_i \) is the accumulated sedimentation in the reservoir, as in Eq. (6). Combining Eqs. (13)–(15), the expected cost of dam failure can be estimated as

\[
C_{OT} = R_{OT} \cdot \left[ \alpha_Q \left( \frac{S_t}{S_0} \right)^{\beta_Q} + \alpha_S \left( 1 - \frac{S_t}{S_0} \right)^{\beta_S} \right] \tag{16}
\]

Several possibilities are for the shape of \( C_{OT} \) which can be categorized according to the relationship between \( \alpha_Q \) and \( \alpha_S \), the maximum loss due to flooding and sediment. When the reservoir begins operations, no sediment has been accumulated; therefore, the loss is \( \alpha_Q \). On the other hand, when the reservoir is completely filled with sediment, the loss is \( \alpha_S \). The total loss, \( L \), may occasionally be smaller or larger than \( \alpha_Q \) and \( \alpha_S \), but usually falls between them. As shown in Fig. 9(a), when \( \alpha_S \) is approximately equal to or considerably larger than \( \alpha_Q \) (I and II), \( C_{OT} \) could only be monotonically increasing (V), because the rate of change of \( L \) is relatively small and thus \( R_{OT} \) dominates the trend. \( \alpha_S \leq \alpha_Q \) would mean that losses due to sedimentation are unimportant, indicating two possible trends for \( C_{OT} \): unimodal (VI) or monotonically decreasing (VII and VIII). The latter situation exists when \( L \) is convex (IV) or has a large initial decay rate. However, according to previous studies, \( L_Q \) is generally concave, such that \( C_{OT} \) is unimodal and the maximum occurs within the life span of the reservoir.

![Diagram](a) The relationship between risk, potential loss and risk cost

![Diagram](b) The relationship between benefit and cost of reservoir

**Fig. 9.** The determination of optimal termination time.
By simplifying our approach, we can estimate the time of the maximum unimodal \( C_{\text{OT}} \). As mentioned previously, the risk increases exponentially and \( \alpha_S \leq \alpha_Q \). Thus, Eq. (12) can be rewritten as

\[
C_{\text{OT}} = R_0 \left( \frac{S_0}{S_L} \right)^{\beta_Q} = R_0 e^{w_L \cdot t} \cdot \alpha_Q \left(1 - \frac{t}{T} \right)^{\beta_Q}
\]  

(17)

\( R_0 \) is the initial risk, \( w_L \) is the exponent coefficient and \( T \) is the lifespan of the reservoir. In Eq. (17), \( S_t \) is approximated as \( S_0 / T \) because it changes linearly with time. The time at which the maximum \( C_{\text{OT}} \) occurs can be determined by taking a log-transform and applying a first order condition to Eq. (17).

\[
\frac{\partial C_{\text{OT}}}{\partial t} = \left( \ln(R_0) + w_L \cdot t + \ln(\alpha_Q) + \beta_Q \ln(1 - \frac{t}{T}) \right) / \partial t = w_L - \beta_Q \frac{1}{T - t} = 0
\]  

(18)

The point at which \( C_{\text{OT}} \) attains the maximum is

\[
t^{*}_{\text{OT}} = T - \frac{w_L}{\beta_Q}
\]  

(19)

Under the condition of \( t^{*}_{\text{OT}} \), the time at which the maximum \( C_{\text{OT}} \) occurs will be a function of the concavity of the loss function and the rate at which risk increases. When the rate increases quickly (large \( w_L \)) or the flood loss function is relatively concave, \( t^{*}_{\text{OT}} \) will be smaller, which means that the maximum loss occurs earlier. On the other hand, if risk increases slowly or the loss function approaches linearity, \( t^{*}_{\text{OT}} \) may be larger or not even exist in the feasible region, \( 0 \leq t^{*} \leq T \).

Differences in the shapes of curve \( C_{\text{OT}} \) and \( t^{*}_{\text{OT}} \) could influence the selection of the terminal time. Fig. 9(b) plots \( B_{t+1} - C_M - \Omega \cdot (r(1 + r) \cdot \alpha_S \) and \( C_{\text{OT}} \) together. As shown in the figure, even with a rapid increase in the risk of dam failure, the reservoir does not necessarily need to be retired, as long as the potential loss decays considerably with time (VII and VIII) or the benefits of the facility are relatively high. However, if the potential loss is large or fails to decays with time, a reservoir may need to be considered for retirement. The most important issue to determine is perhaps \( L_s \), loss due to reservoir sedimentation. However, as discussed previously, very little is known with regard to this issue.

5. Application

5.1. Study area

The Shihmen Reservoir is located in the Tahan River watershed of northern Taiwan (Fig. 10). The drainage area of the watershed is 760.2 km\(^2\). The average annual rainfall is approximately 2580 mm, highly concentrated between May and September, as a result of heavy rainfall from typhoons. High intensity precipitation is the primary cause of disasters associated with those typhoons. The reservoir has a total storage capacity of 252 million cubic meters (mcm) with an effective storage capacity of 236 mcm. The main purposes of this reservoir include irrigation, water supply, flood control, power generation and recreation. Approximately 26.78 mcm of the storage is reserved for flood control during the typhoon season. The reservoir is managed by Water Resources Agency. Shihmen dam outlet facilities include spillways, spill-tunnels, power plant intakes, river outlets, and an intake for the Shihmen canal. The capacity of the spillway is 11,400 cms (using 6 sluice gates) and the spill-tunnel capacity is 2400 cms (using 4 sluice gates).

5.2. Model parameters and climate scenarios

Implementing the overtopping risk model to the case study required inputting parameters specific to the Shihmen Reservoir watershed system, as listed in Table 2. Most parameters were obtained from analysis of historical data related to precipitation, streamflow, and sedimentation (WRA, 1963–2004). Other parameters describing the physical and operational characteristics (ex. \( D_0 \) and \( n \)) were estimated according to the current conditions of the reservoir. This study simulated the 24-h peak inflow into Shihmen Reservoir and performed frequency analysis. Table 3 shows the results of frequency analysis obtained from the simulated peak flow associated with flooding. According to our estimations, the current spill capacity (13,800 cms) should be sufficient to defend against flooding in a 973-year return period (0.0010%). Taking into account the detention of flood control, the return period of the current facility could reach 1313 years (0.00076%).

To estimate the long-term risk, the study provided four possible simulation scenarios, with stationary climatic condition and three degrees of alteration, as shown in Table 4. Scenario I is stationary climatic condition, in which the three temporal parameters of the IDT model were set to zero. Scenarios II, III, and IV provided differences for \( w_L \) and \( \alpha_Q \) representing minor, moderate, and severe changes in the prevailing conditions, in accordance with the findings of previous studies (Wang, 2004; Tam, 2006).

According to a government report (WRA, 2008), the values for annual operating benefit and maintenance costs in 2010 were 6.45 and 13.6 billion NTD (New Taiwan dollar) with a 5% discount rate. The relationship between these economic variables and reservoir capacity is also provided. The current value of maintenance costs was assumed to be constant over time, while the annual operating benefit was assumed to decrease linearly with reservoir capacity. The salvage value was not taken into account in this application.

As discussed, this study differentiates between losses due to water and those associated with sediment release. The potential losses resulting from dam failure flooding, including direct and indirect losses, are approximately 6.61 billion NTD, as of 2010 (WRA, 2004). Equation (13) was used to fit the loss functions of dam failure flooding, in which \( \beta_Q \) was set to 0.4, according to the relationship between the peak flow associated with dam failure and reservoir capacity determined in previous studies (Hagen, 1982; MacDonald and Monopolis, 1984; Costa, 1985, 1988; Froehlich, 1995; Broich, 1998; Zagornjoli and Mynett, 2006). This study also took into account additional losses due to dam failure resulting from sediment accumulation, as represented in Eq. (15). For simplicity, sediment loss was assumed to be linearly related to the volume of sediment trapped in the reservoir (\( \beta_S = 1 \)) and the scalar \( \alpha_S \) is the ratio of the scalar of flood loss, \( \alpha_Q \). In this study, cases in which \( \alpha_S \alpha_Q = 1 \) were used to simulate the effect.

5.3. Estimation of overtopping risk

Fig. 11 shows the trends related to the risk of reservoir overtopping, under various scenarios. The results for Scenario I show an indistinguishable change in risk in the following decades. Under stationary climatic conditions, the main factor influencing overtopping risk is the reduction in flood control capacity due to sediment deposition. Despite the reservoir being completely filled, the risk is only reduced from 0.00076% to 0.001%, as no detention storage is present. In Scenario II, the exceedance probability increases to 0.002% (500 year return period) in approximately the 70th year. The risk increases to a 500-yr return period, the average annual rainfall increased by 7.4%, and S.D. increased to 44%. In Scenario III, the risk trends in the 10th, 20th, and 30th years are 1109, 762, 509 years, respectively. In the 63rd yr, the return period
decreased to 100 years, the average annual rainfall increased by 1.31 times, S.D. increased 2.88 times, and the ratio of extreme events to annual rainfall was 16.3%. In Scenario IV, the risk trend in a 1000-yr return period was in the 10th year, increasing to a 500-yr return period in the 22nd year. The risk increased to a 100-yr return period in the 44th year, in which annual rainfall averages increased 1.31 times, S.D. increased 2.84 times, and annual rainfall due to extreme events increased by 16.6%.

Fig. 12 presents the exceedance probability at a time when the Shihmen Reservoir is completely filled with sediment. In Scenario I, the average time to full sedimentation is approximately 58 years. The 5th and 95th percentiles of the fill time are 62 and 55, respectively. In Scenario II, the average time drops by 3 years and in Scenario III, this is shortened by 7 years, compared to Scenario I. Nonetheless, the estimated time also becomes increasingly uncertain due to the increase in annual rainfall variability (S.D.). In Scenario IV, average time to full sedimentation is estimated to be 47 years, the exceedance probability is more spread out, and the 5th and 95th percentiles of the fill time are 51 and 42, respectively.

5.4. The estimation of optimal termination time

This study attempted to estimate the optimal termination time of the Shihmen Reservoir under four scenarios related to climate change. As shown in Fig. 13, by comparing the benefits \(B_{t+1}\), and costs \(C_M + C_{OT} + \Omega \cdot r/(1 + r)\), the optimal terminal time is determined using the conditions proposed in Eq. (12). In this case, the optimal termination times for Scenarios I~IV are the 48th, 46th, 40th, and 37th years, respectively. The main factor influencing climate change is a loss of reservoir capacity and corresponding operational benefits. The increase in costs due to the risk of overtopping is not obvious.

The influence of increasing risk could become increasingly important with higher flood losses; therefore, this study assumed a case in which flood loss is fifty times that of the original (13.32 billion NTD). We also assumed that losses due to sediment could be overlooked, \(a_Q = 0\), such that a unimodal cost curve occurs. Fig. 14 shows the results. In this case, the optimal termination time is, in the 46th, 44th, 36th, and 33rd
years, respectively, earlier than in the original case. Under this adjusted condition, the maximum cost occurs within the life span of the reservoir, between the 40th and 50th years. According to Eq. (19), when risk increases more quickly, the time to reach maximum costs is also extended. In this case, the maximum costs for Scenarios I to IV occur in the 5th, 31th, 41th and 43th years, respectively. However, some cannot be clearly observed because the magnitude of risk and the corresponding costs are too small.

Table 2
The characteristics of the Shihmen Reservoir and corresponding parameters.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean annual precipitation</td>
<td>2400 mm</td>
</tr>
<tr>
<td>S.D. annual precipitation</td>
<td>600 mm</td>
</tr>
<tr>
<td>Annual inflow</td>
<td>1390 × 10^6 m³</td>
</tr>
<tr>
<td>Storage</td>
<td>211.38 × 10^6 m³</td>
</tr>
<tr>
<td>Mean annual sediment</td>
<td>3.53 × 10^6 m³</td>
</tr>
<tr>
<td>Spillway</td>
<td>13,800 cms</td>
</tr>
<tr>
<td>Dam crest</td>
<td>252.1 m</td>
</tr>
<tr>
<td>Watershed area</td>
<td>763.4 km</td>
</tr>
<tr>
<td>Design storm duration</td>
<td>24 h</td>
</tr>
</tbody>
</table>

Corresponding parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>σ₀/μ₀</td>
<td>0.167</td>
</tr>
<tr>
<td>c₀</td>
<td>0.1</td>
</tr>
<tr>
<td>σ₀/σ₀</td>
<td>0.55</td>
</tr>
<tr>
<td>S₀/μ₀</td>
<td>0.152</td>
</tr>
<tr>
<td>D₀/S₀</td>
<td>0.138</td>
</tr>
<tr>
<td>η</td>
<td>0.5</td>
</tr>
<tr>
<td>C₀</td>
<td>0.7</td>
</tr>
<tr>
<td>C₁</td>
<td>1.2</td>
</tr>
<tr>
<td>C₅₁</td>
<td>0.017</td>
</tr>
<tr>
<td>C₅₂</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table 3
Frequency analysis in 24-h peak inflow in the Shihmen Reservoir.

<table>
<thead>
<tr>
<th>Return period (yr)</th>
<th>25</th>
<th>50</th>
<th>100</th>
<th>200</th>
<th>500</th>
<th>1000</th>
<th>2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak inflow (cms)</td>
<td>7158</td>
<td>8431</td>
<td>9677</td>
<td>10,849</td>
<td>12,479</td>
<td>13,647</td>
<td>14,874</td>
</tr>
</tbody>
</table>

Table 4
Scenarios of reservoir model with climate change.

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Changing condition</th>
<th>w₁</th>
<th>w₂</th>
<th>w₃</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>No change</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>II</td>
<td>Minor</td>
<td>0.001</td>
<td>0.001</td>
<td>0.0005</td>
</tr>
<tr>
<td>III</td>
<td>Moderate</td>
<td>0.005</td>
<td>0.005</td>
<td>0.001</td>
</tr>
<tr>
<td>IV</td>
<td>Severe</td>
<td>0.007</td>
<td>0.007</td>
<td>0.0015</td>
</tr>
</tbody>
</table>

Fig. 11. The risk of overtopping in Shihmen Reservoir for the various scenarios.

Fig. 12. The exceedance probability of time when the reservoir capacity is completely filled by sediment in different scenarios.

Fig. 13. The optimal termination time of reservoir in different scenarios (original case).
variability has been observed worldwide, often leading to compared to inter-year trends. A general increase in intra-annual annual climatic change, could be much more dominant, extreme hydrological conditions. The increasing weight of extreme determining that the major source of risk is the magnitude of overseeing resulting from time-variant climatic factors. We these factors on reservoir management. deterministic methodology to explore the long-term impact of deterministic mechanisms, and other risk factors also remain unbuffering. The risk of overtopping and sedimentation are both nonlinear effect; however, it is not very strong in most cases. The nonlinearity depends on the characteristics of the watershed. As a result, some reservoirs face greater threats when the problem of sedimentation is highly sensitive to the effects of precipitation. The Shihmen Reservoir is such a case, showing a very significant non-linear relationship with annual precipitation. As a result, the risk of the Shihmen dam overtopping has increased to such an extent that maintenance operations are no longer justified. In the following, we employ economic analysis to explore this issue, identify the potential risks associated with the reservoir, and provide appropriate suggestions for strategy and policy.

The impact of climate change imposes a number of challenges to reservoir continued operation in Taiwan. An important issue in Taiwan is reservoir safety because extreme hydrological events are occurring with increased frequency and dams face a greater likelihood of overtopping. The traditional design principle does not take into account the increasing risk under climate change and the high sedimentation rate in the reservoir, which significantly decreases reservoir capacity and the protection afforded by flood buffering. The risk of overtopping and sedimentation are both non-deterministic mechanisms, and other risk factors also remain uncertain. This necessitates the use of risk management instead of a deterministic methodology to explore the long-term impact of these factors on reservoir management.

This study developed a framework to investigate the risk of dam overtopping resulting from time-variant climatic factors. We determined that the major source of risk is the magnitude of extreme hydrological conditions. The increasing weight of extreme events on annual precipitation, which implies more severe intra-annual climatic change, could be much more dominant, compared to inter-year trends. A general increase in intra-annual variability has been observed worldwide, often leading to flooding and droughts of higher intensity and frequency. Unfortunately, both physical and statistical modeling have greater interpretation capacity for long-terms trends than intra-annual variability. As a result, a lack of reliable predictions for intra-annual change may bias our estimation of hydrological risk/threat to infrastructure. Engineering may provide an option for mitigating hydrologic risk under changing climatic conditions. Integrated, watershed-level approaches, such as providing systematic detention or land use management, are better suited to reducing the storm peak from a long-term perspective. According to our analysis, managing watersheds using this approach could extend the protection afforded by such predictive methods. Engineering works to upgrade the risk resistance of dams, such as increasing the capacity of spillways, designing reservoirs beyond anticipated needs, or removing sediment, could provide benefits to increase dam safety. However, due to on-going climatic changes, the benefits of such actions cannot be extended indefinitely.

Maintaining reservoir capacity helps to mitigate the risk of flooding/overtopping and provides continual operational benefits. According to our economic analysis, operational benefits are key factors in deciding whether a reservoir should be retired. Sediment removal immediately increases capacity; however, this is invariably a short-term measure. Similarly, efforts made at the watershed-level help to reduce the amount of sediment arriving in the reservoir every year. Longer-term watershed-level efforts are probably a better option for reservoirs with a relatively long remaining lifespan, while sediment removal may suffice to extend the life of reservoirs nearing the end.

Several studies have addressed the potential losses associated with failure in the benefit-cost analysis of dam design (Baecher et al., 1980; Paté-Cornell and Tagaras, 1986; Cochrane, 1989; Evans et al., 2000). Determining whether losses due to failure are a significant factor in the retirement of dams is still a matter of some concern. It is possible that this will become a critical issue only when the risk of failure or/and the potential losses are very high. In our case study, the potential losses due to dam failure and annual operational benefits of the reservoir are on the same order (approximately 6 billion NT). Therefore, even when risk increases dramatically, potential costs do not increase excessively. It should be noted that the risk of dam failure is usually around $10^{-4}$ to $10^{-2}$. When we discuss a very high risk of overtopping, it is seldom less than 1%; therefore, expected losses due to overtopping (risk multiplied by potential loss) would be very small, compared to the benefits.

In this study, losses were increased by a factor of 50 to observe their influence. As expected, the effects became more obvious with a rapid increase in risk. However, this raises the question of whether the loss due to failure could be several tens or hundreds of times the annual operational benefit. Paté-Cornell and Tagaras (1986) used three cases to discuss the issue of risk cost in dam construction: (1) Teton Dam: annual benefits of $5.16 million and estimated property loss due to dam failure of approximately $700–900 million; (2) Dickey-Lincoln School Lakes project: annual benefits of $79 million and estimated property loss due to dam failure of $36–330 million; and (3) Auburn Dam: annual benefits of $88 million and estimated property loss due to dam failure of $4.0–7.9 billion. Only two of these cases satisfy the condition that the potential losses are relatively high; therefore, we have to assume that not all dam failures inevitably lead to losses comparable to the costs preventing its occurrence. For such cases, losses due to failure are relatively high, compared to the benefits. If climate change is not serious, then the hydrological risk may still be around several hundreds or thousands. Thus, only high-hazard dams that threaten catastrophic losses are a concern for dam retirement.

Another concern about the costs due to risk is the threat of sediment in the reservoir. A sediment filled dam may cause greater damage to downstream areas; however, sediment can only be
delivered over relatively short distances. These influences completely alter the decision-making process. If losses due to sediment are not significant, the associated costs may reach a maximum during the lifespan of the reservoir. According to Eq. (19) and the case study, while the risk increases more rapidly, the peak of risk-associated costs is reached later and vice versa. Under conditions in which the peak in risk-associated costs is still less than the operation benefits, a failure of the dam is not a serious concern in the determination of dam retirement. This situation could occur, for example, in cases where reservoirs are used for power generation. Although the reservoir is filled with sediment, it continues providing benefits through in-stream power generation.

To ensure the sustainable management of reservoirs, we need to determine whether the level of protection against such risk is reasonable. Because the benefits and costs are both huge, the decision may be biased by a conservative attitude. Could the failure of the dam be an option requiring further examination? If the climate continues changing, avoiding an increase in the risk to these water resources infrastructures will be difficult. The outcome of small facilities failing may be considered more acceptable than similar events besetting larger systems because large systems would probably lead to disastrous consequences after failure. In the future, rather than developing systems that are completely safe (using principles such as PMP and PMF), engineers may need to accept the possibility of failure within the infrastructure, but use control damage to an acceptable level.

References


