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Fusegates selection and operation: simulation–optimization approach

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ABSTRACT

Fusegates present a reliable and cost-effective alternative, which increase flood protection and water supply benefits. This article develops a comprehensive simulation–optimization framework for practical selection, installation, and operation of fusegates. The computational model simulates the complicated hydraulic behavior of fusegates systems with varying design characteristics and consequential anomalous routing process in case of flood events. An efficient mixed genetic algorithm (GA) is subsequently developed and coupled with the highly nonlinear hydraulic simulation model to minimize the overall expected annual cost under structural, hydraulic, and operational constraints. Types, heights, and tipping heads of gates are explicitly treated as optimization decision variables. Furthermore, the frequent practice of installing all gates in the same level is practically improved to favorably help minimize water loss in case of moderate discharge floods. The proposed model is demonstrated and discussed for a case study of the Taleghan Dam fusegates installation project in Iran. A series of sensitivity analyses are also conducted to assess routing effect and uncertainty in water unit price and replacement costs and provide more insight and understanding of the design problem.

Key words | flood routing, fusegates, hydraulic simulation, optimization, spillway

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NOTATION

A, B, and μ	the discharge coefficients
C_d	discharge coefficient for sharp or broad-crested weirs which is a function of (h/P)
CE_i	excavation cost
CF_i	installation cost of each fusegate type
CR_i	expected annual gate replacement cost
CRF	capital recovery factor
CW_i	expected annual cost of water loss
ED	excavation depth
H	height of fusegate (m)
h_{a_i}	the difference between the reservoir water level and the crest elevation of the ith gate
h_c	one of the hydraulic characteristics of fusegates
i	gate number
IMWL	mean water level after fusegates installation
L_a	apron length of fusegate
MP_{sw}	the maximum values of $\sum_{i=1}^{N} W_i - W_t$ in every generation
MP_{wl}	the maximum values of (MWL−IMWL) in every generation
MWL	mean water level before fusegates installation
N	number of fusegates
NT	number of tipped gates
P	height of the crest above the upstream approach channel invert elevation
Q	discharge (m^3/s)
RI	required increase in normal water level
Tr	flood return period associated with tipping head h
U_r	unit replacement cost
U_w	unit water cost
V_w	water loss due to gate tipping
W	width of fusegate (m)

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W_e effective width of spillway crest which equals to sum of tipping fusegates widths producing an open space for discharging the flood

INTRODUCTION

Extreme drought events, groundwater overdraft, widespread water pollution, and population increase make water crises a challenging worldwide issue. Among the many available options to counteract these crises such as conservation policies, reservoir construction, wastewater recycling, and seawater desalination, increasing the storage capacity of existing reservoirs might be considered an economical and effective alternative for their alleviation. Reservoir dredging and dam heightening have traditionally been implemented to recover the lost storage of existing reservoirs due to sedimentation and/or to provide extra storage capacity. Fusegates installation is a comparatively new alternative, which has increased in popularity during recent years due to its numerous advantages. Since their first real-world application in the Lussas Dam in 1991, they have been widely used in over 50 dams all over the world and have gained considerable recognition as a safe and economical tool for providing extra water supply. Operational reliability, self-operation with low maintenance cost, limited impact on the neighboring environment, as well as reasonably low investment costs are among the major practical advantages that have contributed to their increasing popularity (Chevalier 2004).

Fusegates are essentially a technical method to increase the maximum water level without structural dam heightening. Fusegates may be efficiently implemented to increase spillway capacity without sacrificing existing reservoir storage. In fusegates system, gates are placed side-by-side to fill in the original spillway width. Before tipping, fusegates are put in stable places, operating like an ungated spillway. They remain in place by the force of gravity and concrete toe abutments cast on the spillway sills prevent them from sliding. The principal advantage of fusegates over fuseplugs lies in their operational schedule. In contrast to the fuseplugs system that stops working entirely after overtopping, fusegates tip independently dependent on the flooding condition and their design tipping heads. The tipping recurrence of a fusegate is selected so that the imposed replacement cost does not adversely affect the total annual project cost.

So far, most of the reported works in this field have focused on fusegates hydraulics, reliability, and cost-effectiveness. Falvey & Treille (1995) investigated their discharge characteristics as well as hydraulic design and operation. Lacroix & Walz (2009) later updated these characteristics and hydraulic behavior analyses and also discussed that the associated costs of replacement are actually reasonable over long periods between two tipping events. Ait Alla (1996), Barcouda *et al.* (2006), Chevalier & Rayssiguie (2003), and Jones *et al.* (2006) illustrated the advantages of fusegate technology and its successful applications for enhancing dam safety. Jones *et al.* (1996) reported on the reliability of fusegates technology in ice-affected environment. Chevalier (2004) compared reservoir dredging with fusegate installation to increase the water supply of reservoir in order to mitigate the silting effects and demonstrated economical preference of fusegates.

Nevertheless, little attention is devoted to some critical aspects of fusegates design and operation. Computational analysis of flood routing process alterations due to fusegates installation, for instance, has not been addressed effectively in the existing literature. Furthermore, despite the successful application of optimization approaches to the design of hydraulic structures such as spillways (Afshar & Marino 1990), sewer systems (Diogo & Graveto 2006), stilling basins (Bakhtyar *et al.* 2007), and flood diversion systems (Rasekh *et al.* 2010), very few studies of optimizing fusegates configuration have been reported in the literature. Afshar *et al.* (2003) employed a linear optimization model to minimize the overall cost of fusegates comprising installation and excavation costs as well as the price of expected water loss. Their model selects the best fusegate type and height together with the tipping head of the first gate and is limited to gates with the same heights and types. Moreover, only the tipping head of the first fusegate is considered as a decision variable and the tipping head of the other gates are indirectly obtained by merely adding a constant value to the tipping head of the neighboring gate. In this way, the first gate, unavoidably, tips at the minimum level and the last one tips at the maximum allowable level. As all gates are set at the same level, once the first gate tips the total volume of water stored between the normal water level and the gate apron elevations is lost.

Furthermore, the flood routing in the reservoir is carried out using the free spillway elevation–discharge curve that disregards the significant hydraulic effect of fusegates. Afshar & Takbiri (2009) employed a genetic algorithm (GA) model to minimize the overall cost of the fusegates project instead of the linear method, which is used in Afshar *et al.* (2003) for optimization. Using GA in their model eases the linking of the routing model to the optimization model. But the results of the optimization model are still negatively affected by the constraint of the tipping head for gates explained in previous model by Afshar *et al.* (2003). Due to both this and the constraint of equality of all gates in their types and heights, the GA as with the flood routing model does not properly illustrate the situation of fusegates in a real situation. In fact, the search space of an optimization algorithm in this model is roughly confined; therefore, a large feasible area which may contain the optimum solution cannot be definitively searched.

This study presents a simulation–optimization model for the optimum design of a set of fusegates to increase storage capacity of existing reservoirs while enhancing dam safety during flooding conditions. The model links with the hydraulic simulation, which accounts for the changes in flood routing procedure due to installation of fusegates on the free spillway. In the proposed model, the gates are arranged in such a way that not only is the total water volume preserved by the gates not lost until the last gate tips, but also the dam safety is enhanced. Moreover, an efficient mixed GA model is utilized as an optimizer to minimize the overall cost comprising the expected costs of water loss and replacement costs as well as the installation and excavation cost under structural, hydraulic, and operational constraints. The water loss and replacement cost which is not properly addressed in previous work (Afshar *et al.* 2003; Afshar & Takbiri 2009) are both accurately computed with association of the developed hydraulic simulator. A variety of design parameters including type, height and tipping head of each gate are explicitly treated as decision variables in the optimization model to find the best possible design. As the total number of gates, their heights, types, and tipping heads are free to change from one solution to another so, the total number of decision variables is not fixed and may vary. Application of the model to Taleghan Dam in Iran, demonstrates its promising performance in dealing with actual problems.

FUSEGATES: HYDRAULICS AND OPERATION

General description

A fusegate consists of three major components: water retaining structure, pressure chamber with drain holes to discharge incidental inflow (due to leaking seals for example), and water collecting well. The water-retaining element is a concrete or steel structure with a downstream abutment block, which prevents the gate from sliding. Water flows from the collection well into the pressure chamber at the bottom of the fusegate, resulting in uplift pressure. The collection well ensures that water can enter the chamber when the water level in the reservoir exceeds the inlet elevation. When the reservoir is in normal operating conditions, the fusegates behave as a watertight barrier. The moderate floods freely pass over the fusegate crest, as they would do over a free weir. When the water surface reaches a predetermined level, water flows into the inlet well. Flow inside the inlet well exceeds the flow out of the drain holes and develops an increasing water head in the chamber. It causes an uplift pressure, which results in a tipping moment on the gate. When the water elevation above the gate reaches a specific level, the overturning moment exceeds the restoring moment and the fusegate tips. If the water level continues to rise after the first breach, more fusegates may tip (Lacroix & Walz 2009).

Most fusegates are made of concrete and can break apart and become unrecoverable as they tumble along the downstream. Simultaneous tipping of all gates may result in a large flood downstream, losing a large of volume of stored water, and imposing a high expected replacement cost. Therefore, the well crest in each fusegate is installed at different elevations to prevent the tipping of all gates at the same time. These elevations are dictated by a predetermined design tipping head of each gate.

Three types of labyrinth fusegates with varying standard heights are commercially available that are known as NLH, WLH, and WHH according to the ratio of gate width to its height and tipping head (Lacroix & Walz 2009). The first letter (W or N) refers to wide or narrow gate, the second two letters (LH or HH) are used to address the relatively low or high head. The standard gate heights are 1.50, 1.80, 2.15, 2.60, 3.10, 3.75, 4.50, 5.40, and 6.50 m respectively.

An appropriate combination of types and heights will satisfy the range of required conditions in a specific application. Some required design parameters are given in Table 1 (adapted from Lacroix & Walz 2009) in which L_a is the apron length of fusegate; H is the height of the fusegate; and W is the width of the fusegate.

Hydraulic studies related to the labyrinth fusegates discharge characteristics were conducted in various laboratories around the world. After the fusegates installation, the flow discharges through fusegate crest, which is always wider than its width (see Figure 1). As a result, due to this increase in spillway length, the labyrinth fusegates can remarkably pass 1.5 to 2.5 times more water than a conventional ogee crest (Lacroix & Walz 2009).

Discharge characterization

The discharge characteristics of labyrinth fusegates have been investigated experimentally. The discharge of flood over a fusegate before tipping (Figures 2(a) and (b)) may be computed as (Lacroix & Walz 2009):

$$Q = \begin{cases} W[A\sqrt{H}h + BH^{3/2}] & \text{if } h > H \cdot h_c \\ \mu_1 \sqrt{2g} W\, h^{2/3} & \text{otherwise} \end{cases} \quad (1)$$

Table 1 | Main characteristics of labyrinth fusegates

Type	W	L_a
NLH	1.0 *H*	1.0 *H*
WLH	1.5 *H*	1.0 *H*
WHH	1.8 *H*	1.2 *H*

Source: Adapted from Lacroix & Walz (2009).

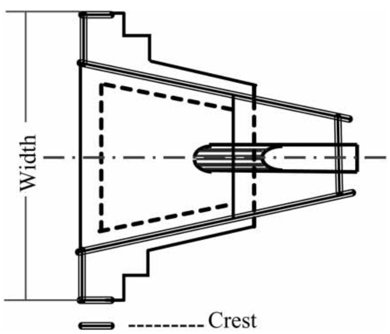

Figure 1 | Plan view of a labyrinth fusegate (Adapted from Falvey & Treille 1995).

where Q is the discharge, g is the acceleration of gravity, H is the fusegate height, h is the difference in reservoir and crest elevation of gates, W is the width of the fusegate, h_c is one of the hydraulic characteristics of fusegates, and A, B, and μ are the discharge coefficients given in Table 2 for various fusegates types.

When the level reaches the predetermined tipping value (Figure 2(c)) during a specific flood event, the fusegate tips over as the result of the development of an abrupt uplift pressure in the base chamber (Figure 2(d)). As the first gate tips, the head over the sill suddenly increases which in turn causes a drastic and rapid change in discharge from the reservoir. In other words, with tipping a fusegate, an open space is created leading to more flow discharge. This varying phenomenon needs to be traced in the modeling routing process in the reservoir. In this stage, when only one or a small numbers of fusegates have tipped, the ratio of water head above the apron of the tipped fusegates to their width is more than 1.8, so the weir behaves as a sharp-crested weir and the discharge can be obtained as (Lacroix & Walz 2009):

$$Q = (2/3)\sqrt{2g}C_d h^{3/2} W_e \quad (2)$$

where W_e is the effective width of the spillway crest which equals the sum of tipping fusegates widths producing an open space for discharging the flood, and C_d is the discharge coefficient for sharp-crested weirs which is a function of h/P where P is the height of the crest above the upstream approach channel invert elevation. As the water level rises, more fusegates tip therefore, W_e increases and subsequently, the ratio of water head to W_e is no longer larger than 1.8. Therefore, a new situation emerges in which this ratio is approximately less than 0.5 and the weir behaves as a broad-crested weir. Similarly, discharge for broad-crested weirs can be computed from Equation (2) in which C_d is the discharge coefficient for broad-crested weirs which is a function of h/W (Johnson 2000).

Flood routing simulation

The effect of flood routing in fusegates design modeling is too important to be ignored. Regardless of the type of spillway, flood routing is affected by reservoir dimensions, the

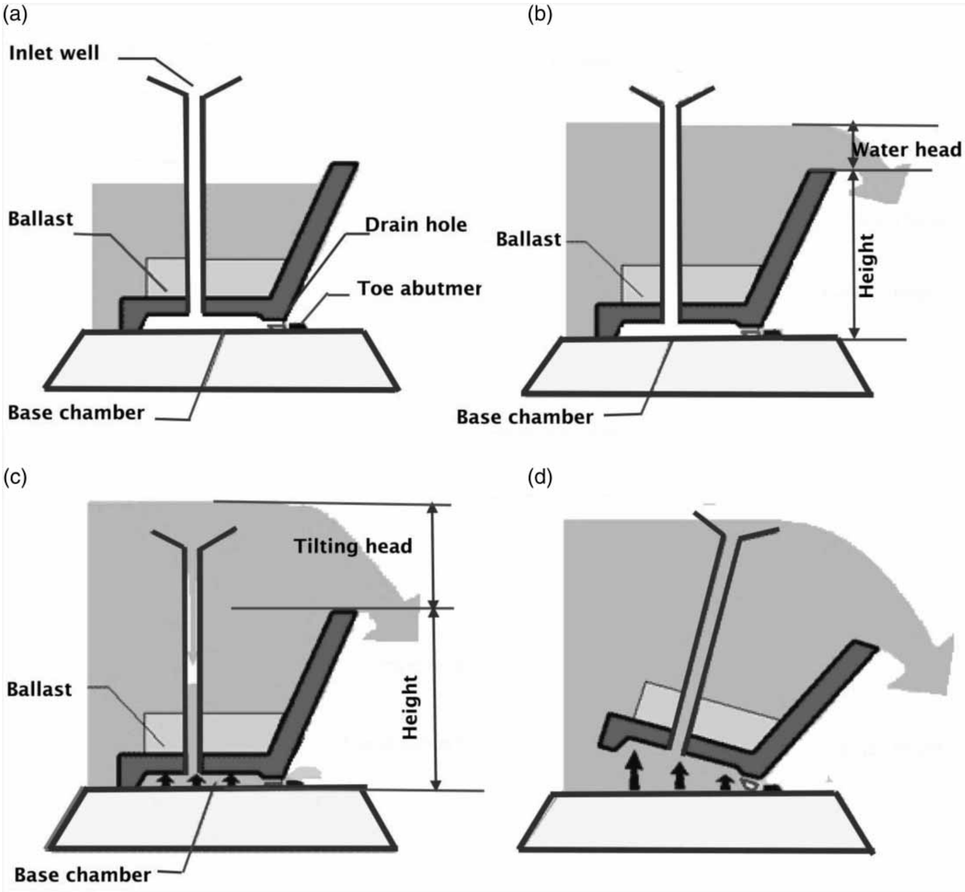

Figure 2 | Fusegate's cross-sections through a fusegate: (a) the water level is increasing, (b) fusegate acts as an ungated spillway, (c) the water head reaches to the predetermined elevation (design tipping head), and (d) the fusegate tips (Adapted from Hydroplus site www.hydroplus.com).

Table 2 | Discharge coefficient

Fusegate type	μ_l	A	B	h_c
NLH	1.334	2.671	0.080	0.12
WLH	1.178	2.497	−0.051	0.18
WHH	1.192	3.095	−0.155	0.13

Source: Adapted from Lacroix & Walz (2009).

elevation–discharge relationship for the spillway, and the flood hydrograph characteristics. Owing to the complexity of fusegates operation, the construction of the elevation–discharge curve and, subsequently, the outflow hydrographs requires much more elaboration compared to the ungated spillways. Therefore, a special module is developed to generate the specific elevation–discharge curve for a unique combination of gates with distinct operation and tipping characteristics. This generated elevation–discharge curve might be used afterwards to model the flood routing process for corresponding combinations of tipped and in-rest gates.

When all fusegates are in rest (no tip-off occurs), the total discharge is the summation of flood passing over individual fusegates. In this case, C_d, L_c and h may change from one gate to another due to their characteristics in each solution. Therefore, total discharge is presented as:

$$\begin{aligned} &\text{If } h_{a_i} > H \cdot h_{c_i} \quad Q = \sum_{i=1}^{N} [W_i(A_i\sqrt{H_i}h_{a_i} + B_i H_i^{3/2})] \\ &\text{If } h_{a_i} < H \cdot h_{c_i} \quad Q = \sum_{i=1}^{N} [\mu_{l_i}\sqrt{2g}W_i h_{a_i}^{2/3}] \end{aligned} \tag{3}$$

where N is the total number of gates installed on the spillway, h_{a_i} = the difference between the reservoir water level and the crest elevation of the ith gate, and i refers to the gate number. As the water level in the reservoir increases, the flood discharge over the fusegates increases accordingly. The first gate (or set of gates) tips, when the water level above the fusegates reaches a minimum predetermined design parameter for tipping heads of gates. Therefore, the total discharge over tipped and in-rest gates is presented as:

If $h > H \cdot h_c$

$$Q = \sum_{i=1}^{N-\mathrm{NT}} [W_i(A_i\sqrt{H_i}h_{a_i} + B_iH_i^{3/2})] + \frac{2}{3}\sqrt{2g} \times \sum_{i=N-\mathrm{NT}+1}^{N} (Cd_iW_ih_{b_i}^{3/2})$$

If $h < H \cdot h_c$

$$Q = \sqrt{2g}\sum_{i=1}^{N-\mathrm{NT}} (\mu_{l_i}W_ih_{a_i}^{2/3}) + \frac{2}{3}\sqrt{2g}\sum_{i=N-\mathrm{NT}+1}^{N} (Cd_iW_ih_{b_i}^{3/2}) \quad (4)$$

where NT is the number of tipped gates, h_{b_i} is the difference in the reservoir water level and apron elevation of the ith gate. Cd_i is the discharge coefficient for sharp or broad-crested weirs depending on the number of tip-off fusegates and the water head above the fusegates' apron. Equation (3) is a special case of general discharge equation for all gates in-rest (i.e. $\mathrm{NT} = 0$).

The elevation–discharge curve for an illustrative configuration of fusegates is shown in Figure 3. This design includes 12 fusegates with three tipping steps installed at the same apron elevation of 1,780.7 m. Every set of four gates tips at a different level and this results in four zigzagging sets of discontinuous curves. The reservoir water level naturally rises as the flood event begins. From point A to point B all gates are

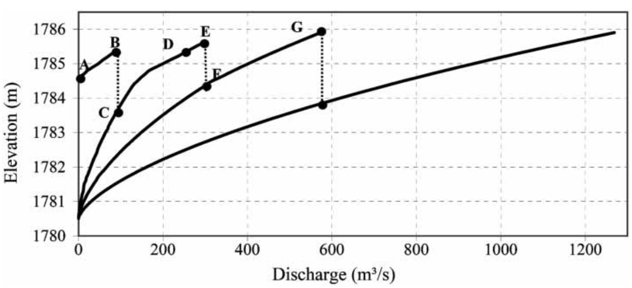

Figure 3 | Elevation–discharge curve.

in rest and Equation (3) controls the discharge from the reservoir. The first set of gates tilt as the water level reaches the predetermined tipping elevation of 1,785.4 m (point B). The reservoir level needed to discharge the flow rate corresponding to point B decreases rapidly due to extra discharge capacity created by the first set of tipped gates. The head required for this discharge rate with the new governing conditions is represented by point C. If the inflow to the reservoir exceeds the discharge rate at point C, then the water surface in the reservoir rises further and the second set of gates tip at point E and this trend continues until the third set of gates tip at point G. Point D, illustrates the water level at which for the same elevation of point C, the discharge is significantly more due to first set of gates tipping. It is noteworthy that whenever the reservoir water level stops rising, the current governing discontinuous zigzagging curve is used to calculate the continuously decreasing discharge rate until the water level reaches the apron elevation. At this point, the discharge rate is equal to zero. These conceptual descriptions need to be extended to derive the elevation–discharge curves for the different scenario of installing gates at distinct apron elevations.

As observed, a new discharge–elevation curve must be generated with even small changes in fusegates types, heights, tipping head, or arrangement. A hydraulic model is developed to evaluate this challenging combinatorial design and operation of a fusegated spillway. This model keeps track of the complex behavior of fusegates system for every expected flood event and evaluates the outflow hydrograph. As an illustrative example, Figure 4 indicates inflow and corresponding outflow hydrographs for sample floods with 2-, 10-, 200-, and 1,000-year return periods for the fusegates system described above. Figure 4(a) shows that the 2-year flood with an approximate peak flow rate of 300 m^3/s passes over the fusegates without tipping any of the gates. For floods with 10- and 200-year return periods, one and two sets of gates tip, resulting in one and two jumps in the outflow hydrographs, respectively (see Figures 4(b) and (c)). For the 1,000-year return period flood, however, all three sets of gates tip in three consecutive steps (see Figure 4(d)). It is possible to appreciate the significant variation in outflow hydrograph shape as the fusegates are installed on an ungated spillway.

Besides evaluating the peak outflow discharge, this flood routing model is required for determining fusegates design

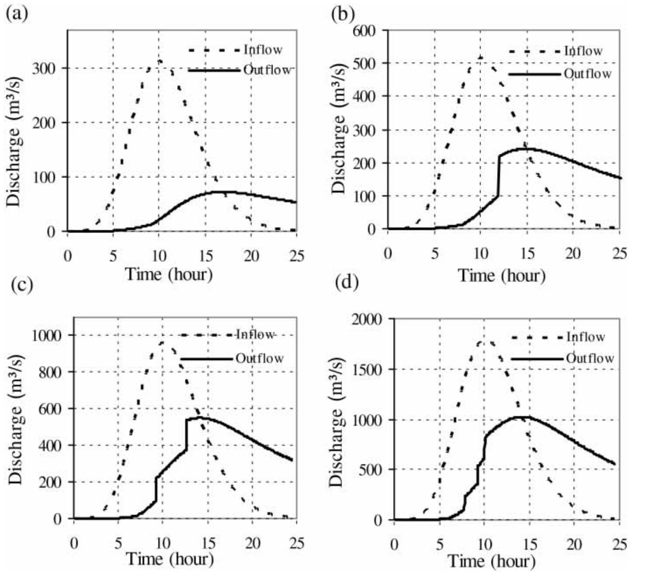

Figure 4 | Inflow and outflow hydrographs for return periods of (a) 2 years, (b) 10 years, (c) 200 years, and (d) 1,000 years.

with proper operation. For smooth transition from low outflows to very large ones, the simultaneous tipping of all gates should be avoided. When high head gates tip, the drop in the elevation–discharge curve and consequently the jumps in outflow hydrograph are rather greater than when the short gates are used to cover the spillway width. Figure 5, shows the resultant outflow hydrographs for inflow hydrographs with peaks ranging from 500 to 4,000 m^3/s. As observed, outflow hydrographs have smoother jumps when the gates are designed with uniformly increasing tipping heads (Figure 5(b)) than the case some gates are set to significantly distinct tipping heads (Figure 5(a)). Additionally, due to the particular behavior of fusegates in case of flood event, design of gates should be performed carefully to avoid releasing discharges that would be greater than would occur before installation of fusegates system. Figure 6, for example, illustrates a specific design for which peak discharge exceeds the peak inflow that might consequently threaten dam safety.

OPTIMIZATION MODEL

The effectiveness of the fusegates system to restore lost reservoir storage and improve flood control has long been recognized (Hite & Mifkovic 2000). Methodological optimization of fusegates, however, remains a problem. Having developed the hydraulic model for simulating the complex behavior of fusegates, an optimization framework is structured here to find the optimal design of fusegates system considering a variety of economic measures and safety aspects. The overall objective is to minimize the sum of expected annual installation cost as well as the expected lost benefit due to water loss and expected gates replacement costs while meeting dam safety constraints. The expected annualized cost equation for a fusegates system configuration might be expressed as:

$$\mathrm{Min\,TAEC} = \sum_{i=1}^{N} [\mathrm{CRF} \times (\mathrm{CE}_i + \mathrm{CF}_i) + \mathrm{CW}_i + \mathrm{CR}_i] \tag{5}$$

where i is the gate number; N is the number of fusegates; CE_i is the excavation cost; CF_i is the fusegate installation cost; CW_i is the expected annual cost of water loss; CR_i is the expected annual gate replacement cost; and CRF is the capital recovery factor. The excavation cost for every gate is directly related to its excavation depth, which is equal to difference between gate height H and the required increase in normal water level RI. Gate installation cost also varies depending on its type and height. Expected replacement and water loss costs can be calculated as:

$$\begin{aligned} \mathrm{CW}_i &= \frac{1}{\mathrm{Tr}_i} \times V_{\mathrm{w}_i} \times U_{\mathrm{w}} \\ \mathrm{CR}_i &= \frac{1}{\mathrm{Tr}_i} \times U_{\mathrm{r}} \end{aligned} \tag{6}$$

where Tr is the flood return period associated with tipping head h; V_{w} is the water loss due to gate tipping; U_{w} is the unit water cost; and U_{r} is the unit replacement cost. The return period of the associated flood which causes the tipping of gate is evaluated in a hydraulic simulator. It should be mentioned that as each gate tips according its own design tipping head, Tr should be computed for each gate in every design configuration.

The minimization of total expected cost is limited by several types of physical and operational constraints. Depending on the fusegate type, the tipping head over height ratio must be in a permissible range:

$$\begin{array}{lll} \mathrm{Type} = \mathrm{NLH\ or\ WLH} & \text{for which} & 0.3 \le (h_i/H_i) \le 0.7 \\ \mathrm{Type} = \mathrm{WHH} & \text{for which} & 0.7 \le (h_i/H_i) \le 1.4 \end{array} \tag{7}$$

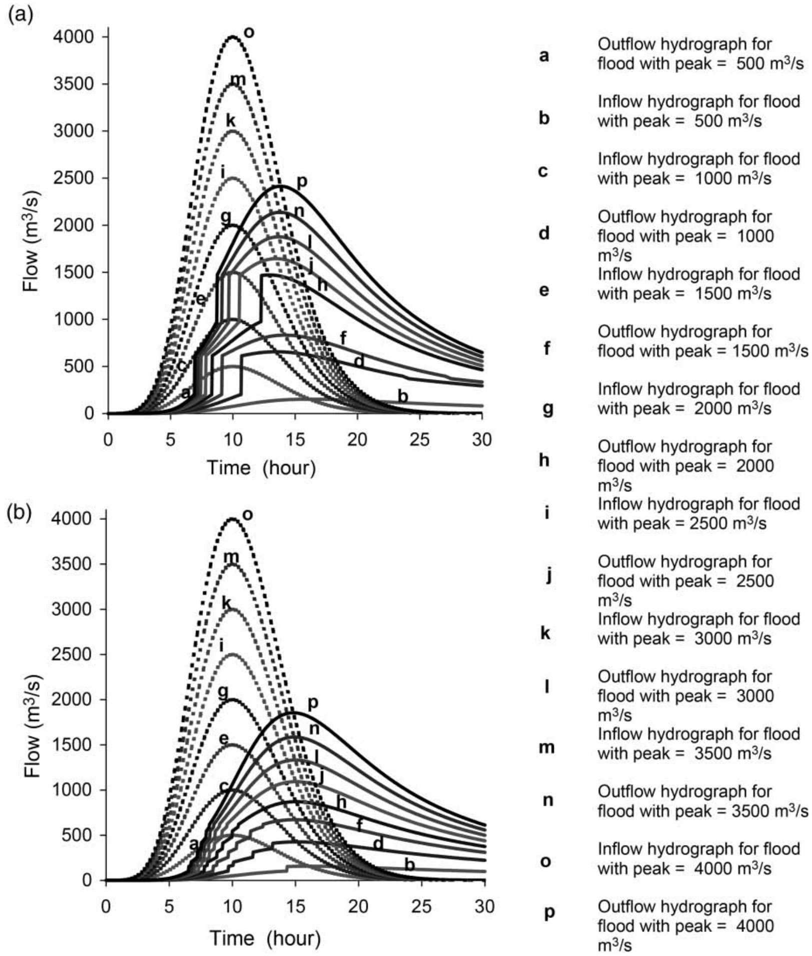

Figure 5 | The outflow hydrographs for eight various inflow hydrographs in two different combinations of fusegates.

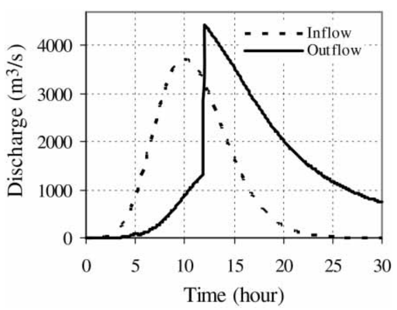

Figure 6 | One example of flood routing in which the outflow hydrograph exceeds the inflow hydrograph.

To guarantee dam safety after gate installation, the maximum water level after routing the design flood through the reservoir may not exceed the reservoir MWL before fusegates installation (IMWL). Note that MWL must be evaluated for all combinations of possible gate selections and their initially unknown tipping heads in a complicated flood routing process through the reservoir. As the gates might tip one by one in each time step, the discharge capacity of the spillway will be automatically defined based on the number of tipped gates with different design characteristics, as well as the sudden drop in reservoir storage. Therefore, the developed special flood routing simulation model is coupled with an optimization model in an interactive loop.

From a practical standpoint, the cumulative width of all fusegates with widths W_i should not exceed the original spillway width W_t:

$$\sum_{i=1}^{N} W_i < W_t \tag{8}$$

Under the assumption of uniform gate installation, when the first gate tips, the total water stored between the gate's apron and existing water level is lost. In other words, the tipping schedule of the other gates has no single effect on the total volume of wasted water. In the proposed modeling scheme, however, the assumption of single type and equal gate heights are favorably relaxed. When the lost water is considered in the optimization process, installation of gates with different heights and different apron elevations is almost inevitable. In price competitive situations, sequential gate tipping with different aprons and heights may result in stepped water loss, which essentially saves some fraction of the stored water before the last gate tips.

The hydraulic simulation model developed to evaluate the annual expected costs and safety of dam is highly nonlinear and complex. Evolutionary algorithms have been successfully applied for solving such complicated problems in the technical literature (ASCE Task Committee 2009). Thus, an appropriate version of a GA is tailored to cope with the combination of the discrete and real-value decision variables of the proposed model. This proposed evolutionary-based optimizer is coupled with the hydraulic model, which simulates the system behavior for every design solution and determines return periods of floods that cause gates to tip.

Genetic algorithm

GA are stochastic search methods that imitate natural biological evolution. GAs operate on a population of potential solutions following the survival principle of the fittest generation to reproduce with more probability than other. This repeated process leads to the evolution of populations of individuals that are better than previous populations. GAs are used broadly in optimization problems (ASCE Task Committee on Evolutionary Computation in Environmental and Water Resources Engineering 2009).

The elitist real-coded GA applied in this paper uses tournament selection. In tournament selection, each tournament is carried out between four solutions to enhance convergence rate. The crossover type used in this work is scattered-type. In the scattered-type crossover, first a random binary vector is generated whose length is equal to the size of the GA solutions. The value of each gene which is 0 or 1 is randomly generated. When the random number is 0, the value of the corresponding gene in offspring solution is selected from the first parent and oppositely, random number 1 indicates that this value should be from the second parent. Moreover, the penalty method proposed by Deb *et al.* (2002) is employed to enforce existing constraints. Total normalized values of constraints violations are used to penalize the infeasible solutions in the constraint handling process. The penalty cost for any solution that violates the constraints on the maximum water level, and maximum width of free spillway occupied by fusegates is calculated based on their distance from the feasible region. The total normalized penalty for every solution is sum of normalized penalties for maximum water level (NP_{wl}) and spillway fixed width (NP_{sw}) defined as:

$$
\begin{aligned}
NP_{wl} &= \begin{cases} \dfrac{MWL - IMWL}{MP_{wl}} & \text{if } MWL > IMWL \\ 0 & \text{otherwise} \end{cases} \\
NP_{sw} &= \begin{cases} \dfrac{\sum_{i=1}^{N} W_i - W_t}{MP_{sw}} & \text{if } \sum\limits_{i=1}^{N} W_i > W_t \\ 0 & \text{otherwise} \end{cases}
\end{aligned} \tag{9}
$$

where MP_{wl} and MP_{sw} are the maximum values of (MWL−IMWL) and $\sum_{i=1}^{N} W_i - W_t$ in every generation, respectively.

The fusegated spillway is favored to pass the design flood with the maximum water level not exceeding that of original free spillway. Different gate combinations with distinct design characteristics should be employed to fulfill this requirement. The decision vector for this problem, therefore, consists of a number of gates, type, heights, and tipping heads for all nominated gates. Among these decision variables, the number of gates, types, and heights are discrete variables, whereas the fusegates tipping heads are real numbers bounded within maximum and minimum permissible values. Once the type and height of a gate is nominated, its width is known as a dependent variable. In fact, for each combination of types (NLH, WLH, and WHH) and heights (1.50, 1.80, 2.15, 2.60, 3.10, 3.75, 4.20, 5.40, and 6.50) a predetermined width is available from the Hydroplus. Thereafter, depending on the types and heights of gates, the widths of gates are computed and a new solution is generated. Clearly, the number of genes in a solution is not fixed and varies as the total number of gates decreases or increases. In this study, the population size, number of elites, and mutation probabilities are set to 100, 2, and 0.08, respectively. Detailed descriptions of the computational procedure and the interaction between the simulation and optimization modules are shown in Figure 7.

MODEL APPLICATION AND DISCUSSIONS

In order to demonstrate the effectiveness and applicability of the proposed model, a case study is now introduced and the results are analyzed. The proposed model was implemented to upgrade the Taleghan Dam in northwestern Iran. The 103 m high rockfill dam is located at latitude 36, 05 to 36, 25 and longitude 50, 10 to 51, 37 on the Taleghan River. Its total storage volume is about 460×10^6 m^3. The 48 m wide spillway is designed to safely pass 10,000 years flood with peak flow of 3,700 m^3/s.

Due to an increase in demand, it has been decided to increase the total storage volume by 18.1×10^6 m^3 which leads to a 1.35 m increase in the normal water level (NWL). According to Figures 1 and 2, the excavation volume is approximated as:

$$V_{\mathrm{exc}_i} = 0.5 \times \sum_{i=1}^{N} \{W_i \times \mathrm{ED}_i \times (2L_{\mathrm{a}_i} + \mathrm{ED}_i \times \tan\alpha)\}$$
$$\tan\alpha = \frac{(L_\mathrm{c} - L_\mathrm{a})}{H} \tag{10}$$

where α is the angle between diagonal wall of bucket and the vertical plate; and ED is the excavation depth. The fusegate technology is exclusive to Hydroplus. Due to unavailability of true cost data, the following hypothetical costs are

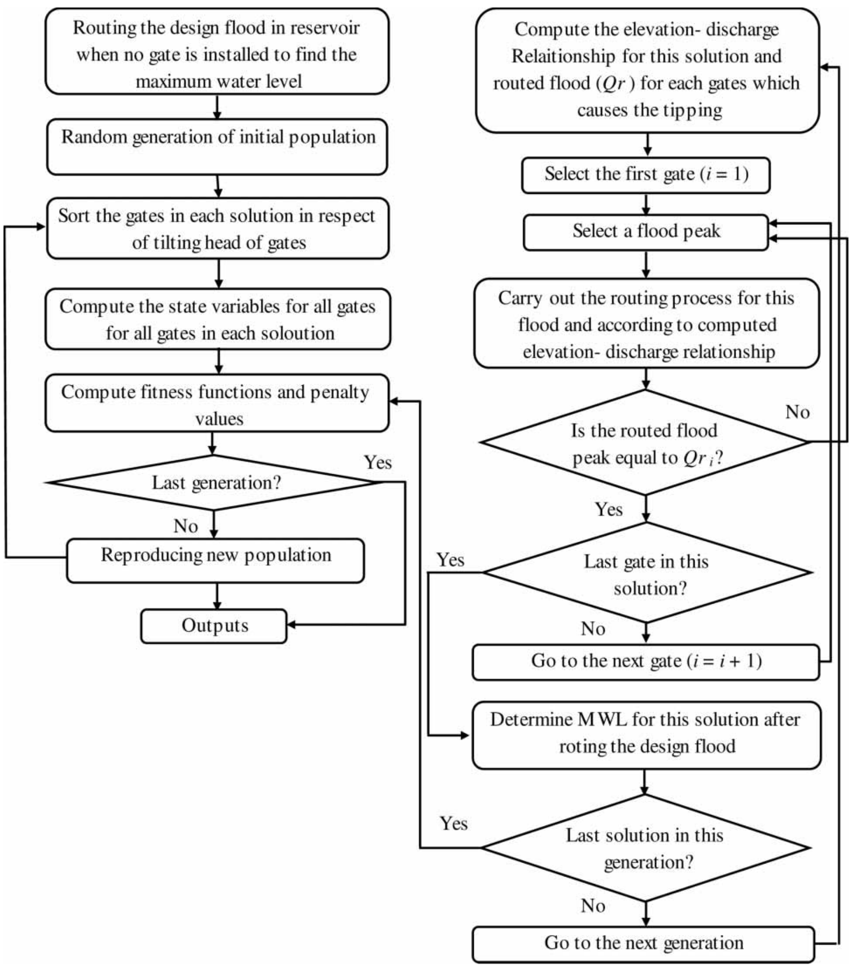

Figure 7 | Flowchart for the proposed simulation–optimization model.

considered in this study:

$$\mathrm{CF}_i = \begin{cases} 0.75 \times 10^5\, H_i & \text{for NLH} \\ 1.00 \times 10^5\, H_i & \text{for WLH} \\ 1.25 \times 10^5\, H_i & \text{for WHH} \end{cases} \tag{11}$$

Recall that CF_i is the installation cost of each fusegate type in dollars. Replacement of a tipped and destructed gate is assumed to cost 5 times as much as the initial installation cost. It is further assumed that 1 m^3 of lost water costs US$0.8.

The proposed simulation–optimization scheme is employed to develop optimal solution to this critical fusegate design which intends to increase reservoir storage while enhancing dam safety. To illustrate the performance of the model, two different cases are considered. Case I assumes that all the gates are of the same type with equal heights while Case II is free to select different types and heights from the available set. The optimization model of Case I is like the developed model of Afshar & Takbiri (2009) in which all gates have to be equal in their heights and types. However, the difference between these two models (Case I and developed model of Afshar & Takbiri 2009) is in the limitation for tipping heads. Case I has no limitation mentioned for tipping heads in Afshar *et al.* (2003) and Afshar & Takbiri (2009). By developing Case I, the effect of removing only the constraint of equality of heights and types from previous models is clearly demonstrated. For a spillway with 48.0 m width, the total number of fusegates may vary from 4 to 32 for gate widths ranging from 1.5 to 11.70 m, respectively. As the total number of gates, their heights, types, and design tipping heads are free to change from one solution to another, the total number of decision variables is not fixed and may vary from 12 to 96.

The number of decision variables in Case II is much more than that of Case I. Although the tipping head might vary from one gate to another, selection of only one type and height for all gates significantly reduces the total number of decision variables. Therefore, Case I is expected to converge to a near optimal solution in smaller number of function evaluations compared to Case II. This expectation is supported by presenting the rate of convergence of the solutions with respect to the number of generations for Cases I and II (Figure 8).

The optimal solutions found for the Taleghan spillway rehabilitation are presented in Table 3. In addition to Cases I and II, another case is considered to show the significant improvement of optimum results of Case II to the optimization models of Afshar & Takbiri (2009) and Afshar *et al.* (2003). This model is represented as Case III in which the heights and types of gates should be equal (like Case I) and the explained constraint of tipping heads in previous models exists. In Case III there are three decision variables; height, type, and tipping head of the first gate. The results show that for gate replacement costs the ratio equals 5, the total annual expected costs for the best solutions of Cases III, II and I are US$2.39 million, US$1.90 million, and US $2.28 million, respectively. The considerable different percentage between the results of Case II with routing consideration and Case III without routing effect is near 27% because of the more limited search space of Cases III to II and the effect of flood routing, as well.

The replacement cost ratio (RCR) refers to the ratio of replacement cost to the initial installation cost. It is obvious that the total cost of the project in Case II is approximately 16% cheaper than that of Case I. In fact, forcing the system to select only one type and height from the available set imposes a hard constraint on the model which results in a

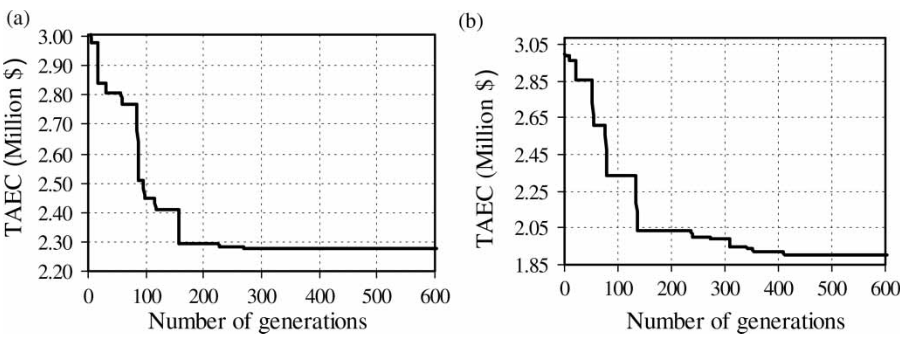

Figure 8 | Rate of convergence to near-optimal solutions for (a) Case (I), (b) Case (II) (averaged over 5 runs).

Table 3 | Optimal heights, types, and tipping heads of fusegates

Case I	Gate number	1–2	3–4	5–7	8–9	10
	Height	3.10	3.10	3.10	3.10	3.10
	Type	WLH	WLH	WLH	WLH	WLH
	Tipping head	1.02–1.58	1.70–1.77	1.82–1.84	1.85–1.90	1.96
Case II	Gate number	1–8	9–11	12–13	14	15
	Height	1.5	1.8	2.15	3.1	4.5
	Type	WLH (5)	WLH (2)–WHH (1)	WLH (1)–WHH (1)	WHH (1)	WLH (1)
	Tipping head	0.45–0.60	0.63–1.50	2.17–2.95	3.11	3.15

very small feasible space. Although the installation of gates with different types and heights might be practically more difficult, its lower annual cost, flexibility in operation, and smaller wasted water resulting from gate tipping justify their selection as a more desirable solution. To assess the routing effect on optimal total annual expected cost of fusegates project, the optimization model is performed without routing consideration. The total annual expected costs for Cases II and I without considering the routing process are US$2.37 million and US$2.75 million, respectively. These values stipulate that the flood routing phenomenon in reservoir must be considered in fusegated spillway design, because, for example, for Case II, total annual expected cost reduces about 24% with considering the flood routing effect (see Table 4 for more details). The flood routing simulation model, however, should be quite reliable and avoid jeopardizing the safety of the dam. To achieve this goal, the routing model should accurately simulate the complicated hydraulic behavior of fusegates during the flood event.

To assess significance of the expected lost benefit due to water loss using the proposed simulation–optimization scheme, a series of sensitivity tests were conducted. Sensitivity of the optimum annual cost to water unit cost for Cases II and I are presented in Figure 9. It shows that, for very small water unit cost (US$0.2), the differences in optimum annual costs of two cases are smaller than larger water unit costs. In fact, Case II is relatively adaptive to water unit cost increase, whereas the same increase in water unit cost significantly affects the optimal solution of Case I. This is mainly due to the loss of the entire amount of water stored between the normal water level and apron elevation of the gates as tipping occurs. Varying apron elevations in Case II limits the lost water volume to that stored between normal water level and the apron elevation of the first gate as it tips. As more gates tip by flood flows of larger return periods, more water might be lost.

To test the sensitivity of the solution to gate replacement cost, a series of tests are conducted. Results for two

Table 4 | Optimal values for total annual expected cost

	TAEC ($)	CF ($)	CW ($)	CR ($)	CE ($)
Case I – with routing (a)	2.28E + 06	1.55E + 06	2.63E + 05	3.80E + 05	9.00E + 04
Case I – without routing (b)	2.75E + 06	1.61E + 06	4.28E + 05	6.01E + 05	1.11E + 05
Case II – with routing (c)	1.90E + 06	1.51E + 06	1.40E + 05	1.66E + 05	8.40E + 04
Case II – without routing (d)	2.37E + 06	1.54E + 06	3.28E + 05	4.36E + 05	7.00E + 04
Case III – with routing (e)	2.41E + 06	1.52E + 06	4.07E + 05	4.20E + 05	6.23E + 04
Case III – without routing (f)	2.60E + 06	1.65E + 06	4.63E + 05	4.23E + 05	6.43E + 04
Difference between (a) and (c)	16.7%	2.6%	46.7%	56.4%	6.7%
Difference between (b) and (d)	13.8%	4.3%	23.4%	27.5%	36.9%
Difference between (a) and (b)	17.1%	3.7%	38.6%	36.8%	18.9%
Difference between (c) and (d)	19.8%	1.9%	57.3%	61.9%	−20.0%
Difference between (c) and (f)	26.9%	8.5%	69.8%	60.8%	−30.6%

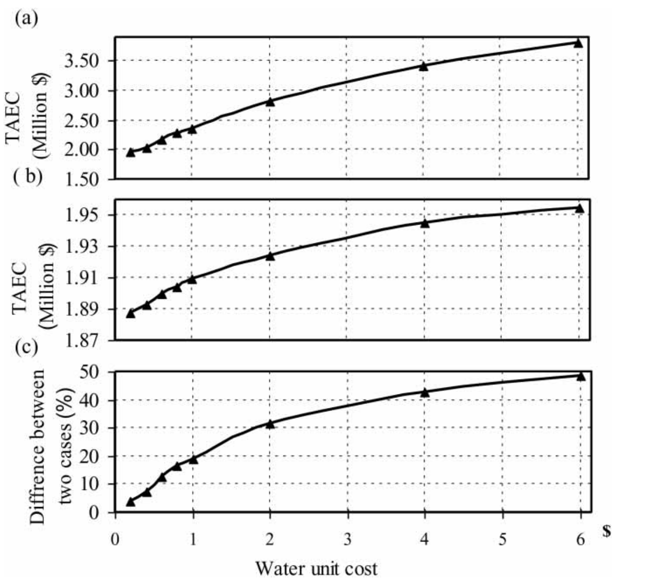

Figure 9 | Sensitivity of the optimum values of total annual expected cost (TAEC) with respect to water unit cost for Case I (a), and Case II (b), as well as difference of TAEC between Case I and II (c).

extreme cases are presented in Figure 10. Figure 10(a) presents the combination of gates resulting from the optimal solution for case II with a RCR of 3. As it shows, except one gate, all gates are relatively short. The widths of the gates vary from 2.25 to 4.65 m for type WHH. Except the last gate, which tips with 650-year flood, the others mostly tip with 300-year flood. Figure 10(b) presents the same information for Case II with a RCR of 10. Although the RCR of 10 is unrealistic for most practical proposes, results effectively illustrate the sensitivity of the solution to high value of RCR. Without any exception, all gates are from type WHH and the first gate tips for 6,800-year flood. In other words, no gate tips for floods smaller than 6,800-year flood. Thereafter, the gates tip one by one as more severe floods enter the reservoir. With increasing the replacement cost, the optimization algorithm selects the taller gates from WHH type, because the ratio of tipping head to the height of gates in this type is rather greater than two other types. Taller gates are selected because they tip with the floods of larger return periods, which consequently reduce the total expected cost of replacement. The straight dotted lines display the normal water level before gate installation. The block area below the dotted lines show the excavation needed to fix the apron for gate installation.

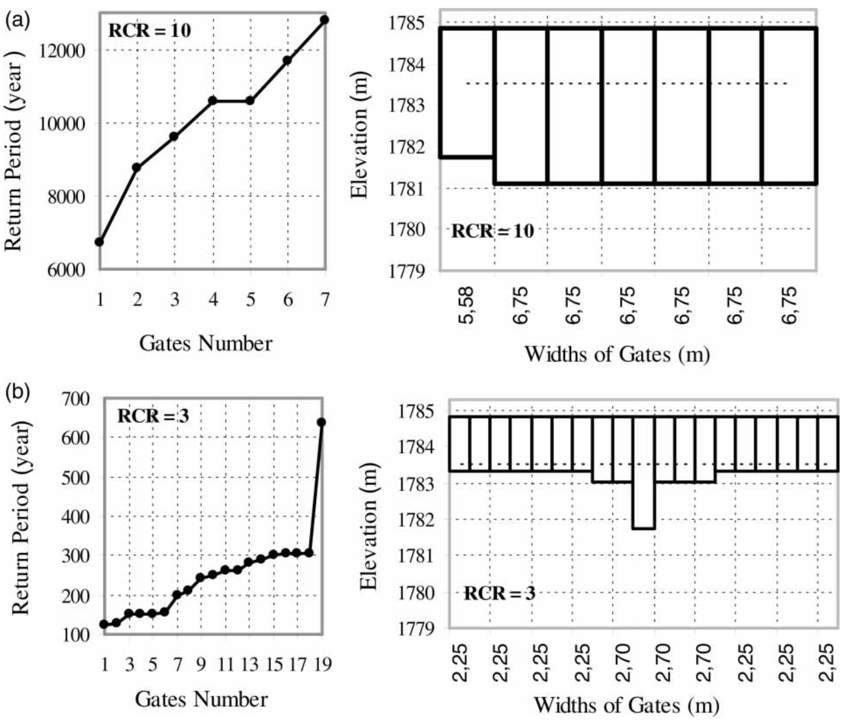

Figure 10 | (a) The optimum return periods of floods which cause the gates tipping, (b) the optimum combination of fusegates included types, and heights with respect to replacement cost ratio.

CONCLUDING REMARKS

The principal advantage of fusegates over fuseplugs lies in their operational schedule. Fuseplugs completely fail when they overtop whereas a number of tipping fusegates depends on flooding conditions and design tipping head of the individual gates. In price-competitive situations, sequential gate tipping with different aprons and heights should be used which essentially saves some fraction of the stored water before the last gate tips. Owing to the complexity of fusegates operation, construction of the elevation–discharge curve and, subsequently, the outflow hydrographs requires much more elaboration compared with ungated spillways. As the reservoir water level rises, more gates tip, causing a zigzag curve for elevation–discharge. A detailed analysis to explain the zigzag shape of the discharge–elevation curve is needed to address the physical behavior of the fusegated spillway under flooding condition.

The fusegated spillway is favored to pass the design flood with maximum water level not exceeding that of original free spillway. Different gates combinations, their setting aprons, and varying routing characteristics of the fusegated spillway should be employed to fulfill this requirement. A mixed evolutionary-computation-based algorithm is developed in this study as an efficient tool for finding the optimal design considering all existing constraints and abnormal hydraulic behavior of fusegates is case of flood events. Application of the proposed model to the Taleghan dam revealed that the model might be effectively used to increase storage capacity of an existing reservoir while enhancing dam safety. Although the installation of gates with different types and heights might practically be difficult, its lower annual cost, flexibility in operation, and smaller wasted water resulting from gates tipping justify their selection as a more desirable solution.

REFERENCES

Afshar, A. & Marino, M. A. 1990 Optimizing spillway capacity with uncertainty in flood estimator. *J. Water Res. Plann. Manage.* **116** (1), 71–84.

Afshar, A. & Takbiri, Z. 2009 Optimal design and operation of fuse-gates considering water loss due to gates tilting. In: *World Environmental and Water Resources Congress*. Great Rivers: ASCE, Kansas City, pp. 3053–3060.

Afshar, A., Marino, M. A. & Jalali, M. R. 2003 Optimum design of fusegates; reconciling dam safety and increasing storage capacity. *Iranian J. Sci. Technol.* **1** (1), 28–32.

Ait Alla, A. 1996 The role of fusegates in dam safety. *J. Hydropower Dams* **3** (6), 67–72.

ASCE Task Committee on Evolutionary Computation in Environmental and Water Resources Engineering 2009 State of the art for genetic algorithms and beyond in water resources planning and management. *J. Water Res. Plann. Manage. ASCE* **136** (4), 412–432.

Bakhtyar, R., Mousavi, J. & Afshar, A. 2007 A dynamic-programming approach to optimal design of cascade stilling basins. *ASCE J. Hydraul. Eng.* **113** (8), 949–954.

Barcouda, M., Cazaillet, O., Cochet, P., Jones, B. A., Lacroix, S., Odeyer, C., Laugier, F. & Vigny, J. P. 2006 Cost effective increase in storage and safety of most dams using Fusegates or P.K. Weirs. *Transactions of the International Congress on Large Dams Barcelona* **22** (1), 1289–1326.

Chevalier, S. 2004 Fusegates mitigate silting at Beni Amrane reservoir. *J. Hydropower & Dams* **11** (5), 118–119.

Chevalier, S. & Rayssiguier, J. 2003 Optimization of existing dams. In *Proceedings of ICOLD Montreal*, **81** (3), 529–539.

Deb, K., Pratap, A., Agarwal, S. & Meyarivan, T. 2002 A fast and elitist multi-objective genetic algorithm NSGA-II. *IEEE Trans. Evol. Comp.* **6** (2), 182–197.

Diogo, A. F. & Graveto, V. M. 2006 Optimal layout of sewer systems: a deterministic versus a stochastic model. *ASCE J. Hydraul. Eng.* **132** (9), 927–943.

Falvey, H. & Treille, P. 1995 Hydraulics and design of Fusegates. *ASCE J. Hydraul. Eng.* **121** (7), 512–518.

Hite, J. & Mifkovic, C. 2000 Increasing reservoir storage or spillway capacity using fusegates. US Army Corps of Engineers ERDC/CHL CHETN-VIII-1.

Johnson, M. C. 2000 Discharge coefficient analysis for flat-topped and sharp-crested weirs. *J. Irrigation Science* **19**, 133–137.

Jones, S. J., Spencer, D., Rodionov, V. B., Lounatsi, M. E. & Ait Alla, A. 1996 The reliability of fusible gates in ice-affected environments. *J. Hydropower and Dams* **3** (3), 78–82.

Jones, B., Cazaillet, O. & Hakin, B. 2006 Safety measures adopted to increase spillway potential at Jindabyne dam in the Snowy Mountains. *Transactions of the International Congress on Large Dams Barcelona* **22** (3), 229–252.

Lacroix, S. & Walz, A. 2009 Engineering a fusegated spillway. In *Proceedings of ICOLD Brasilia*. 23rd Congres des Grands Barrages Brazilia, May 2009, Report no. CFBR/Q. 90.

Rasekh, A., Afshar, A. & Afshar, M. H. 2010 Risk-cost optimization of hydraulic. Structures: methodology and case study. *J. Water Res. Plann. Manage.* **24** (11), 2833–2851.

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