

Adaptive multi-objective simulation–optimization framework for dynamic flood control operation in a river–reservoir system

Om Prakash, K. Srinivasan and K. P. Sudheer

ABSTRACT

An adaptive simulation–optimization (S–O) framework enables dynamic reservoir operational decision-making process during the different phases (time stages) of flood control operation during the passage of a flood event in a river–reservoir system is proposed. This is achieved by incorporating the changing priorities of the reservoir operator/manager at each phase of the flood mitigation operation into the S–O framework by evoking the appropriate set of objective functions and dynamically reconstructing the multi-objective optimization model. Five different objective functions are formulated within the S–O framework, out of which two are concerned with the mitigation at the reservoir; two more deal with the mitigation at the control point; and one ensures sufficient water is stored for meeting future demands. The non-dominated sorting genetic algorithm-II (NSGA-II) is employed to obtain the trade-off solutions from the multi-objective optimization model at each time stage. The results from the study show that the dynamic flood operation model yields a significant level of improvement in flood peak mitigation over the static model both at the reservoir as well as at the control point. The proposed S–O framework can be used in developing either deterministic or probabilistic optimal reservoir release policies for flood control operation, especially where damage functions and penalty functions are not developed.

Key words | dynamic flood control operation, multi-objective, NSGA-II, optimization model, reservoir operation, simulation–optimization framework

Om Prakash (corresponding author)
WEST Consultants,
Folsom,
CA 95630,
USA
E-mail: oprakash@westconsultants.com

K. Srinivasan
K. P. Sudheer
Department of Civil Engineering,
Indian Institute of Technology Madras,
TN 600 036,
India

INTRODUCTION

A river–reservoir system operation for flood control is generally complex, especially in practical applications, due to the presence of a large number of uncertain factors, multiple objectives, complex dynamics of the problem, non-linear and state dependent constraints on control variables. Several researchers in the past have adopted a simulation approach to model the operation of river–reservoir systems during flood events. For instance, the HEC-ResSim model (USACE 2013) obtains the reservoir releases by applying a fixed set of heuristic operation rules and priorities to simulate the flood control operation of the system. Sigvaldason (1976) developed a flexible reservoir simulation model that used penalty coefficients to account for deviations from

ideal conditions. The simulations were based on the operator's experience. Stam *et al.* (1998) presented user interactive decision support system (DSS) for reservoir management, which blended the release rules obtained from optimization model or simulation-based scenario analysis, with the expert input from the experienced reservoir manager. Shim *et al.* (2002) have listed the typical DSS models for integrated river basin flood mitigation as: Spatial DSS; Basin Runoff and Streamflow Simulation model (Colon & McMahon 1987), Ford & Killen (1995), Biddle (1999); and CalSim model (Draper *et al.* 2004). Jain *et al.* (1998) employed simulations to fine tune the existing reservoir rule curves so as to determine the safe releases from

multi-purpose reservoirs. Ahmad & Simonovic (2000) proposed a feed-back based object-oriented simulation approach that can incorporate the inputs from the end users (reservoir operators) in the model. Prakash *et al.* (2006) developed an adaptive rule based simulation model, which determines the best rule for the current time period based on a number of trial (dummy) simulations.

Alternatively, a number of researchers have proposed/developed reservoir or river–reservoir system optimization models and have demonstrated the same in the context of flood mitigation. Detailed reviews of such applications are provided by Yeh (1985), Wurbs (1993), Labadie (2004), and Rani & Moreira (2010). In general, the commonly employed methods for optimal flood mitigation can be classified as follows: linear programming (Windsor 1973; Wasimi & Kitanidis 1983; Needham *et al.* 2000), non-linear programming (Unver & Mays 1990), goal programming (Can & Houck 1984), dynamic programming (Jain *et al.* 1992; Shim *et al.* 2002), linear regression technique (Li *et al.* 2013), evolutionary optimization algorithms (Chang & Chen 1998), machine learning methods (Cheng & Chau 2001; Wu *et al.* 2011), tree-based optimal operation rules (Wei & Hsu 2008), and Liu *et al.* (2006) applied dynamic programming neural-network simplex (DPNS) model for refill-operating rules to optimize the hydropower generation in the Three Gorges Reservoir (TGR), China. A simplex method based non-linear programming technique was used to refine the output from the dynamic programming neural network (DPN) model. Improvements in the probability of refill as well as the mean hydropower generation were noted. Guo *et al.* (2011) proposed a joint operation of the Three Gorges and Qingjing cascade reservoir in China to maximize the hydropower generation and hydropower revenue objective function by defining flood limiting water levels, using progressive optimality algorithms. Yun & Singh (2008) proposed the ‘multiple duration limited water level’ and ‘dynamic limited water level’ approaches to increase the water supply storage at the reservoir while maintaining its security for flood control. The dynamic limited water level for flood control was fixed based on the conditional probabilities of large storms obtained using a lag-one multivariate autoregressive model. Li *et al.* (2010) proposed dynamic control of reservoir flood limited water level considering the inflow forecasting error and uncertainty of the

flood hydrograph shape, and it was applied in the China’s TGR. The Monte Carlo simulation technique was used to estimate the boundary of the dynamic control bound of reservoir flood limited water level. They demonstrated better flood water utilization without increasing the flood risk.

A real-time river–reservoir flood mitigation model was proposed by Hsu & Wei (2007) to obtain optimal real-time release during typhoon periods considering two objectives regarding maximizing peak flow reduction at downstream control points and meeting conservation storage requirements at the end of the flood. Three-stage flood operation guidelines were incorporated into the mixed-integer linear programming model. Chen *et al.* (2014) developed a real-time optimal flooding operation model for the Tseng-wen reservoir in Taiwan. The study uses artificial neural networks (ANNs) for reservoir inflow forecasting and genetic algorithm (GA) techniques to find the optimal release considering the minimization of downstream flood loss. The study compares the model results with the real operations during Typhoons Sepat, Krosa, Kalmaegi, Fung-wong, Sinlaku, and Jangmi in Taiwan. Jia *et al.* (2015) applied a third-order hierarchical optimization decomposition-coordination model in the Huaihe river basin, China, to solve the multi-objective optimization problem proposed for the objectives of real-time flood control operation in reservoirs and flood storage basin, considering the maximum safety of the reservoir and minimum losses of flood storage basin. Chou & Wu (2013) have proposed a model for deciding the target pre-release for reservoir flood control with the aim of reducing downstream flood potential, while achieving the end-of-flood-operation storage target for water supply. Chang *et al.* (2011) have built two artificial intelligence techniques, namely, knowledge acquisition and implementation and fuzzy inference system, into the real-time reservoir operational model they proposed. Some of the other notable optimal flood control models proposed in the last decade made use of the advanced computing tools/techniques such as ANN, ANFIS, GA (Cheng & Chau 2001, 2004; Ngo *et al.* 2007). Kumar *et al.* (2013) applied ANN, Fuzzy Logic and Decision Tree algorithms for the development of reservoir operating rules for irrigation and hydro-power generation for the Bhakra reservoir located in northern India. The revised elevation-area-capacity

curves developed for the sediment volumes for the future 25 years are also used in obtaining the optimal set of reservoir releases. More recently, a survey of the application of several artificial intelligence based optimization techniques to reservoir operation has been presented by Hossain & El-Shafie (2013). A detailed and general review on theory and applications of simulation–optimization modeling was presented by Tekin & Ihsan (2004). Rani & Moreira (2010) have reviewed the application of simulation–optimization modeling to reservoir systems operation.

Most of the available methods for flood mitigation in a river–reservoir system aim at minimizing the flood damage at the control points, and hence require the availability of the damage functions at those control points (Sigvaldson 1976; Yeh 1985; Wurbs 1993; Labadie 2004). In addition, while representing the flood control operation in reservoir optimization models, considerable approximations are to be made to represent the relevant objectives and constraints related to the operation of the system (Valdes & Marco 1995). While developing compromise solutions for the optimal flood control operation problem in a river–reservoir system, it has been the practice to employ linear or dynamic programming techniques in which the multiple objectives are transformed into a single objective function using penalizing coefficients or weightages. These coefficients or weights are quite implicit and it is difficult to determine the relative importance of these on the optimal release decisions (Can & Houck 1984; Cheng & Chau 2001). However, such flood damage functions at control points are not always available, especially in the case of developing countries. In such cases, the knowledge and the experience of the reservoir operators play a vital role and they need to be incorporated appropriately into the optimal release decisions (Matsumura *et al.* 2012).

The key to any successful real-time flood mitigation operation in a river–reservoir system lies in quickly generating some feasible and effective near-optimal alternative release strategies and then selecting the most appropriate one among them for implementation. In general, the objectives of such alternatives will be to achieve the maximum flood peak attenuation as well as the minimum accumulated flood volume over the flood horizon at the reservoir and/or control points during the flood events. Another objective may be to minimize the absolute deviation of the actual

reservoir water surface elevation at the end-of-flood operation-horizon from the desired target water surface elevation. This ensures that sufficient water is stored for meeting future demands or sufficient space in the reservoir is available to store the oncoming floods as the case may be. Alternatively, a target elevation constraint may be included to achieve the same effect. It is to be mentioned that during the passage of a flood in a river–reservoir system, the physical situations keep changing dynamically. Hence, the priorities of the reservoir operator/manager also keep changing dynamically, as the flood passes. In fact, this can be achieved by integrating a real-time flood forecasting model with an adaptive optimal reservoir operation model that should enable dynamic reservoir release decision-making. The focus of the present study is to develop an adaptive simulation–optimization (S–O) framework for flood mitigation operation in a river–reservoir system that would enable the dynamic reservoir operational decision-making that reflects the operator’s changing priorities during the various stages of the flood event. The motivation behind the development of such a framework is to consider multiple objectives of flood mitigation operation in the optimization formulation and to effect dynamic triggering of the appropriate combination of objectives according to the changing state of the river–reservoir system as the flood passes through.

Flexibility is built within the proposed S–O framework to reconstruct the optimization model in an adaptive manner after certain phases during the passage of the flood, by means of selecting the appropriate set of objective functions. This feature enables a reasonable representation of the dynamic nature of the decision-making process during the passage of any flood event and in the time between consecutive events of flood in a season. There are no damage, penalty or weightage functions used in the multi-objective optimization model developed in this study. The robust non-dominated sorting genetic algorithm-II (NSGA-II) has been used to solve the multi-objective optimization model developed. The usefulness of the proposed framework is demonstrated by considering a river–reservoir system with one reservoir and a single downstream control point, where flood protection is to be achieved. The effect of initial reservoir water surface elevation on the effectiveness of flood mitigation in the river–reservoir system, and

the influence of the end-of-flood horizon target reservoir elevation constraint on the total flood volume and the peak flood mitigation at the control point are discussed. Also, a brief discussion is presented as to how the adaptive S–O framework proposed herein can be easily extended for flood control operation in a river–reservoir system with multiple reservoirs and control points.

MODEL FORMULATION

Methodology

The dynamic change that can occur in the physical system generally, as the flood progresses in time, are the hydro-meteorological conditions in the upstream watershed, the space available in the reservoir for flood control, the river stage at the control point, the residual channel capacities available below the bankfull stage and that below the damage-causing stage. These changes call for changes in the operational priorities at the reservoir, as the flood passes through the river–reservoir system (Matsumura et al. 2012; Chou & Wu 2013). These changes in the physical status of the system during the flood passage are proposed to be handled through appropriate choice of different objective functions within the framework. However, it is not necessary that all the objective functions would be activated simultaneously in the optimization throughout the flood event. A combination of one or more of these objective functions is triggered depending on the physical state of the river–reservoir system, as the flood passes. In other words, the proposed framework enables reconstructing the optimization model in an adaptive manner as the flood passes. In the current study, five objective functions are considered, of which the first four functions deal with reductions in total excess volume and flood peak mitigation at the reservoir or the control point or both. The fifth objective function is concerned with minimizing the absolute deviation of the actual reservoir water surface elevation at the end of the flood horizon from the prefixed target water surface elevation. A number of constraints are specified for every period of operation and with increase in the number of periods of operation, the total number of constraints run into a few thousands. The reservoir storage–elevation

relationship and the outlet discharge functions are non-linear. Furthermore, the optimization model has to access the reservoir routing and the channel routing modules in every time period of operation. In addition, decisions regarding incremental or decremental release from the reservoir include ‘if-then’ conditions. Thus, the model proposed in this study is a highly constrained non-linear optimization formulation, for which the non-dominated elitist-based sorting genetic algorithm is a promising evolutionary solution technique. The objective functions and constraints that are considered in the current study are discussed below.

Objective functions

Objective I: Maximize the cumulative volume of water drained from the reservoir during the flood operation horizon

This objective aims to release as much volume of water from the reservoir as possible during the period of flood operation to make the final reservoir elevation low, thus by promoting the safety of the dam against the succeeding flood.

$$\text{Maximize } \sum_{t=1}^{T_R} O_t - \sum_{t=1}^{T_R} I_t \quad (1)$$

where I_t = reservoir inflow (m^3/s); O_t = outflow from the reservoir (m^3/s); T_R = flood operation horizon at reservoir; and t = time index (varies from 1 to T_R).

Objective II: Minimize the cumulative excess flow volume (in excess of the specified channel capacity) at the control point during the flood operation horizon

This objective has a tendency to store water in the reservoir with a view to minimize the control point flooding.

$$\text{Minimize } \sum_{t=1}^{T_C} [\max(0, Q_{Ct} - CC)] \quad (2)$$

where Q_{Ct} = flow at the control point at time t , including local flow contribution (m^3/s) if any; CC = specified channel capacity (m^3/s) at control point, and T_C = flood operation horizon at control point; In the present study, it is assumed that $T_R = T_C$.

Objective III: Maximize the flood peak attenuation at the reservoir

This objective aims to obtain maximum flood mitigation in terms of reduction in peak flood discharge at the reservoir.

$$\text{Maximize } (I_p - O_p) \quad (3)$$

where I_p = peak inflow discharge into the reservoir (m^3/s) =

$$\sum_{T_R} \max\{I_1, I_2, \dots, I_{T_R}\}$$

O_p = peak reservoir outflow discharge (m^3/s) =

$$\sum_{T_R} \max\{O_1, O_2, \dots, O_{T_R}\}$$

Objective IV: Maximize the flood peak attenuation at the control point

The usefulness of this objective is to reduce the flood peak discharge as much as possible at the control point with a view to reduce the damage to the developments in the flood plain.

$$\text{Maximize } (I_p - Q_{Cp}) \quad (4)$$

where Q_{Cp} = control point peak flow (m^3/s) =

$$\sum_{T_C} \max\{Q_{c1}, Q_{c2}, \dots, Q_{cT_C}\}$$

Objective V: Minimize the deviation of the reservoir water surface elevation at the end of the flood operation horizon from the desired target water surface elevation at the reservoir

The target elevation at the reservoir will depend on whether flood control space is to be provided for subsequent floods to arrive at the reservoir or conservation storage is to be provided for the upcoming season if the flood season is likely to end.

$$\text{Minimize } (|E_{\text{targ}} - E_{TR}|) \quad (5)$$

where E_{targ} = target water surface elevation at the reservoir (m) and E_{TR} = actual water surface elevation (m) at the end of the operation horizon (T_R).

Constraints

The following constraints are included in the optimization model: storage-continuity equation used in routing the

floods through the reservoir (Equation (6)); upper limit of the release from bottom outlet (Equation (8)); upper limit on the hourly increase in the release from the bottom outlet (Equation (10)); upper limit on the hourly decrease in the release from the bottom outlet (Equation (11)); upper and lower limits of reservoir storage at time t (Equation (12)); and channel routing equation as per the Coefficient Routing Method (Equation (13)).

$$S_t = S_{t-1} + \left(\frac{I_{t-1} + I_t}{2} \Delta t\right) - \left(\frac{O_{t-1} + O_t}{2} \Delta t\right) \quad (6)$$

$$O_t = f_1(H_t), \text{ where } H_t = f_2(S_t) \quad (7)$$

$$R_t^b \leq R_{t\text{max}}^b \quad (8)$$

$$O_t = R_t^b + W_t \quad (9)$$

If $R_t^b \geq R_{t-1}^b$ then

$$R_t^b - R_{t-1}^b \leq RI \quad (10)$$

If $R_t^b \leq R_{t-1}^b$ then

$$R_t^b \leq R_{t-1}^b \quad (11)$$

$$S_{\text{min}} \leq S_t \leq S_{\text{max}} \quad (12)$$

$$Q_{Ct} = C_0 O_t + C_1 O_{t-1} + C_2 O_{t-2} + LF_t \quad (13)$$

where S_t = reservoir storage (Mm^3); Δt = routing time interval; W_t = uncontrolled spill discharge from the reservoir (m^3/s) over the ogee spillway crest; R_t^b = release through the bottom outlet (m^3/s); $R_{t\text{max}}^b$ = maximum release possible through the bottom outlet (m^3/s); RI = upper limit of the hourly rate of increase in the release from the bottom outlet (m^3/s); RD = upper limit of the hourly rate of decrease in the release from the bottom outlet (m^3/s); S_{min} = inactive storage (Mm^3); S_{max} = maximum reservoir storage (Mm^3); Q_{Ct} = control point flow (m^3/s) at time t ; LF_t = local flow contribution at the control point (m^3/s) at

time t ; C_0 , C_1 and C_2 are the routing coefficients for the channel reach between the reservoir outlet and the control point; and O_t , O_{t-1} , O_{t-2} are the reservoir outflows at the times t , $t-1$ and $t-2$, respectively.

It is to be noted that instead of Objective-I, the following constraint restricting the upper limit of the target elevation at the reservoir at the end of the operation horizon may be specified (Equation (14)).

$$E_{TR} \leq E_{\text{targ}} \quad (14)$$

where E_{TR} = the actual reservoir elevation at the end-of-operation horizon (m); and E_{targ} = target elevation to be maintained at the reservoir (m) at the end of the flood operation horizon.

Solution technique

As discussed earlier, the optimization formulation in the proposed S–O framework is a highly constrained, multi-objective mixed integer non-linear type. The fast elitist-based NSGA-II (Deb et al. 2002) is one of the frequently-used multi-objective evolutionary algorithms, which has a very good search ability. The NSGA-II uses a fast non-dominated sorting procedure, an elitist-preserving approach and a crowding distance measure based on multiple objectives and it has been successfully used by a number of researchers in recent years in the field of water resources planning and management for solving complex multi-objective optimization problems (Reed & Minsker 2004; Baran et al. 2005; Kim et al. 2005, 2008). The NSGA-II also has the capability to handle highly constrained multi-objective formulations (Farmani et al. 2005; Murty et al. 2006; Jayaram & Srinivasan 2008) such as the one formulated in the present study. Hence, NSGA-II is employed as the optimization tool in this research study. A detailed account of the working of the algorithm of NSGA-II can be found in Deb et al. (2002).

The proposed S–O framework can be used for the static as well as the dynamic flood control operation (which is based on operator/decision-maker's changing priorities during the passage of a flood) in a river–reservoir system. The scheme of the S–O framework for both static and dynamic flood control operations is shown in Figure 1.

Each release decision vector generated from the NSGA-II module is sent to the reservoir simulator to compute the current status of the river–reservoir system and the fitness functions of the selected objective functions. After that, the solutions are sorted in the NSGA-II module, according to the fast non-dominated sorting approach, and new populations are created using the binary tournament selection operator, uniform crossover and mutation. These processes are repeated until the pre-specified stopping criterion is achieved and then the final set of non-dominated solutions is stored in an output file (Figure 1). Furthermore, in case of the dynamic flood control operation, the framework will obtain from the operator/decision-maker the model choice (that indicates the changing priority) for the next phase of flood event operation, and update the river–reservoir system based on the selected solution of the previous phase (Figure 1).

MODEL APPLICATION

The applicability of the proposed flood mitigation operation framework is illustrated through a case example in this section. The single river–reservoir system presented herein is adapted from the Hayes Basin system given in HEC-ResSim v3.1 model (USACE 2013). The river–reservoir system considered in this paper (Figure 2(a)) has one upstream flood control reservoir and one downstream control point. The reservoir has been divided into three storage zones or pools, as indicated in Figure 2(b). The total release from the reservoir consists of the controlled releases through the bottom outlet ports and the uncontrolled releases over the ungated ogee spillway crest (Figure 2(b)). The physical and the operational characteristics of the reservoir, the river and the control point, are shown in Table 1. In this study, two flood events E-1 (Figure 3) and E-2 (Figure 4) are considered for the investigations regarding flood mitigation. The flood event E-1 (Figure 3) has a quick flood peak and short flood duration (given in the HEC-ResSim model), while the inflow flood event E-2 (Figure 4) is resembling a plausible realistic flood event and has high local flood peak and longer total flood duration to test the efficacy of the S–O framework. The operating time interval of the reservoir is considered to be 1 hour because a shorter interval is required during the flood.

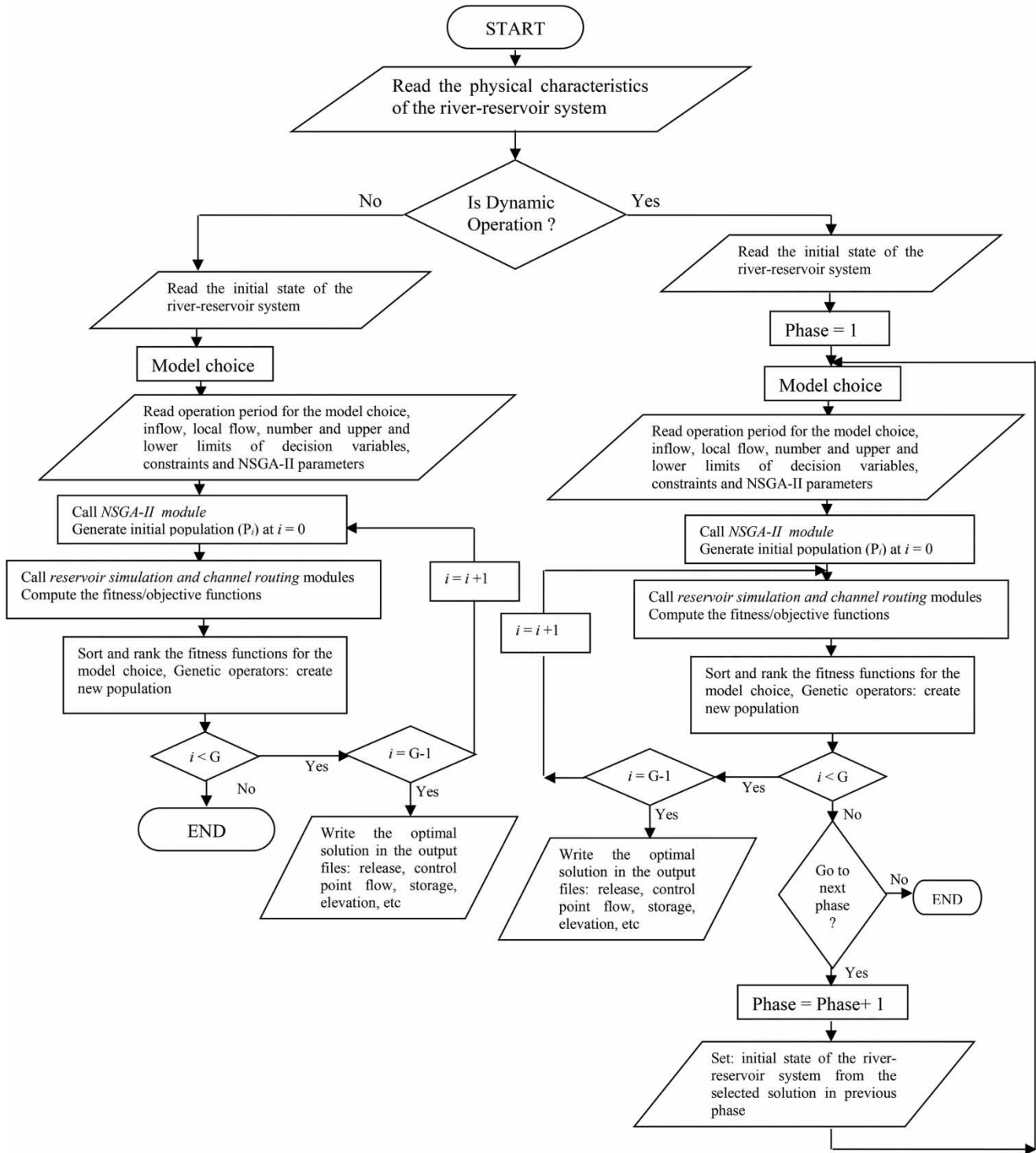


Figure 1 | Block diagram of simulation–optimization (S–O) framework for static/dynamic flood mitigation operation.

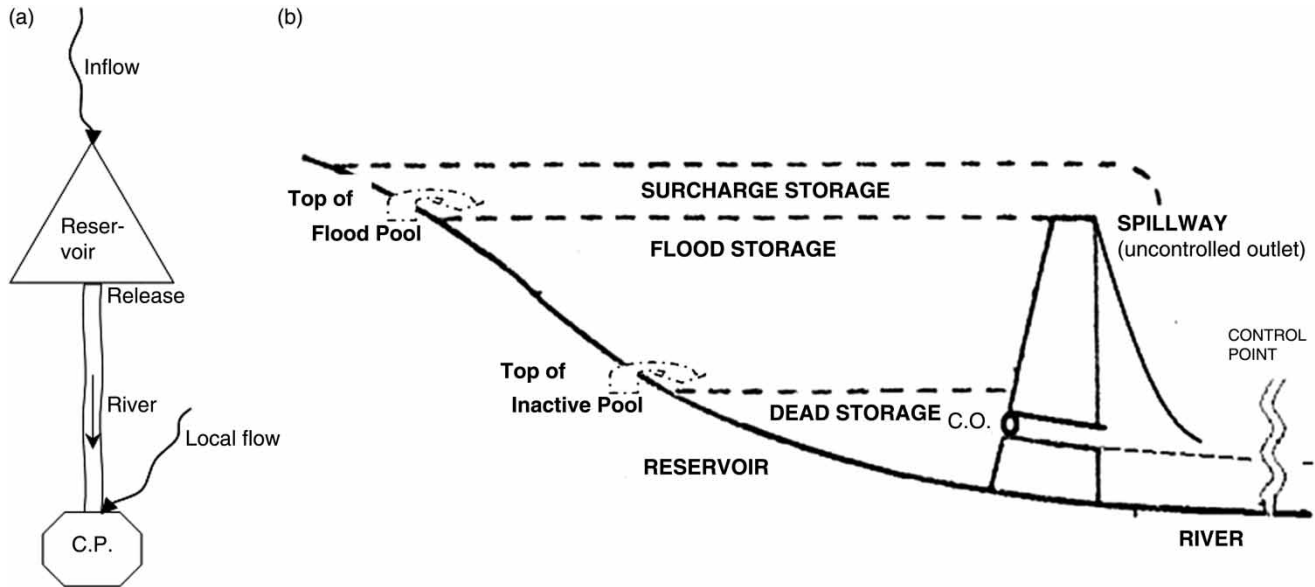


Figure 2 | Schematic of single river–reservoir system. C.P.: Control point; C.O.: Controlled outlet. (a) Plan view of system. (b) Side view of river–reservoir system.

For the event E-1, the inflow peak is $690.7 \text{ m}^3/\text{s}$ that occurs at 10 hours and the local flow hydrograph peak magnitude of $100.9 \text{ m}^3/\text{s}$ that occurs at 11 hours (Figure 3). While for the event E-2, the incoming flood wave is assumed to have an inflow peak of $553.1 \text{ m}^3/\text{s}$ occurring at 66 hours and a higher local flow peak (than that of E-1) of $241.7 \text{ m}^3/\text{s}$ occurring at 95 hours (Figure 4). The outflow hydrograph from the reservoir is computed by routing the given inflow flood hydrograph through the reservoir, using the storage-indication method. Moreover, the coefficient routing method (the coefficient values are assumed as 0.333, 0.333 and 0.334 (USACE

2013)) is used to route the flood along the channel from the reservoir to the control point downstream.

RESULTS AND DISCUSSION

Using the static flood mitigation model (that uses a single objective function through the entire flood control operation horizon), (i) the effect of the initial reservoir water surface elevation on the effectiveness of flood mitigation in the river–reservoir system and (ii) the effect of the end-of-period target reservoir elevation constraint on the excess volume passing at the control point and the flood peak mitigation at the control point are investigated. Following this, a comparison of the results between the static flood mitigation model and the dynamic flood mitigation model is presented to bring out the improvement achieved in flood mitigation due to dynamic decision-making that makes use of different sets of objective functions during the different phases of the flood mitigation operation. The investigations concerning the static flood mitigation model are carried out using the flood data corresponding to the event E-1. To illustrate the improvement achieved in flood mitigation due to the dynamic model, the flood data corresponding to the event E-2 are used. The NSGA-II parameters obtained based on

Table 1 | River–reservoir system characteristics

| | | | |
|---|------------------------------|---|------------------------------|
| Elevation at top of the dam | 456.44 m | Reservoir storage capacity | 36.76 Mm^3 |
| Spillway crest elevation (Top of the flood pool) | 447.60 m | Flood storage capacity | 12.19 Mm^3 |
| Top of the inactive pool | 437.39 m | Inactive storage capacity | 11.78 Mm^3 |
| Upper limit of hourly rate of increase of release through bottom outlet | $28.32 \text{ m}^3/\text{s}$ | Upper limit of hourly rate of decrease of release through bottom outlet | $56.63 \text{ m}^3/\text{s}$ |

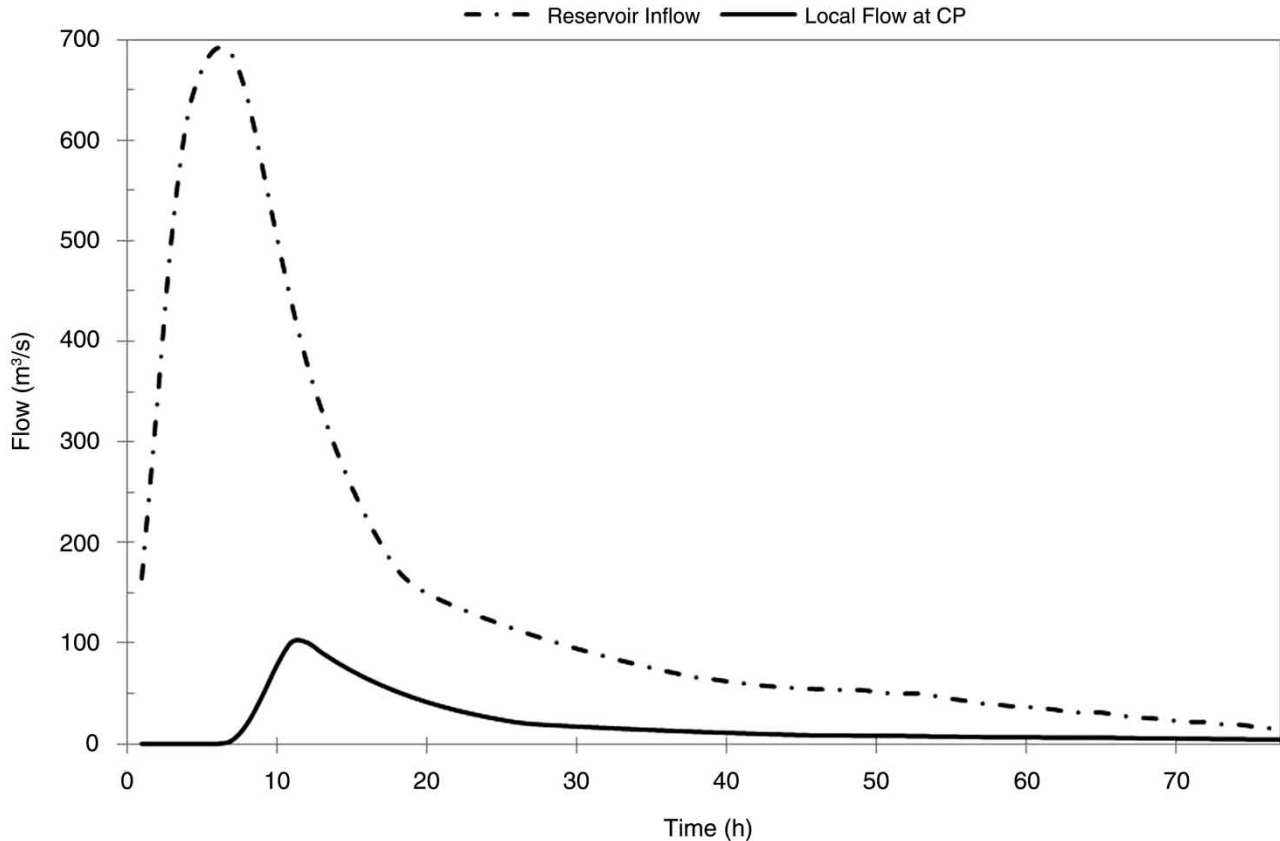


Figure 3 | Inflow time series at the reservoir and the control point for flood event E-1.

a detailed sensitivity analysis for both the flood events E-1 and E-2 are as follows: (i) crossover probability = 0.8; (ii) mutation probability = 0.012; (iii) population = 200; and (iii) generation = 1,000.

Static flood mitigation model

Three static multi-objective models, M1, M2 and M3 (Table 2) are formulated using the objective functions listed in the model formulation section above and these models are run for the flood event E-1 using the S–O framework. For each of the three static multi-objective models, the optimal values taken by the participating objective functions of the respective model are given in italics in Table 3; the values of the other objective functions (those not participating in the respective optimization models) are also presented in Table 3. It may be noted from Table 3 that the M1 model does not offer significant flood peak mitigation either at the reservoir or at the control point, since

this model considers only two volume related objectives A and B, and does not consider the flood peak mitigation related objectives C and/or D. On the other hand, the other two multi-objective models M2 and M3 offer significantly higher flood peak mitigation at the reservoir and at the control point. Out of the three static multi-objective models considered (listed in Table 3), the model M2 is seen to perform competitively in respect of peak flood mitigation at the reservoir as well as the control point, while offering considerable band width for the two volume objectives and a reasonable range for the peak mitigation objectives. Hence, the static model M2 is chosen to investigate the effect of initial water surface elevation, and for comparison of the static model with the dynamic decision-making model. For each of the considered models, the most compromising non-dominant solution can be selected from the band of non-dominant solutions obtained for that model. Then, out of all the selected compromising solutions from the various models considered, the appropriate

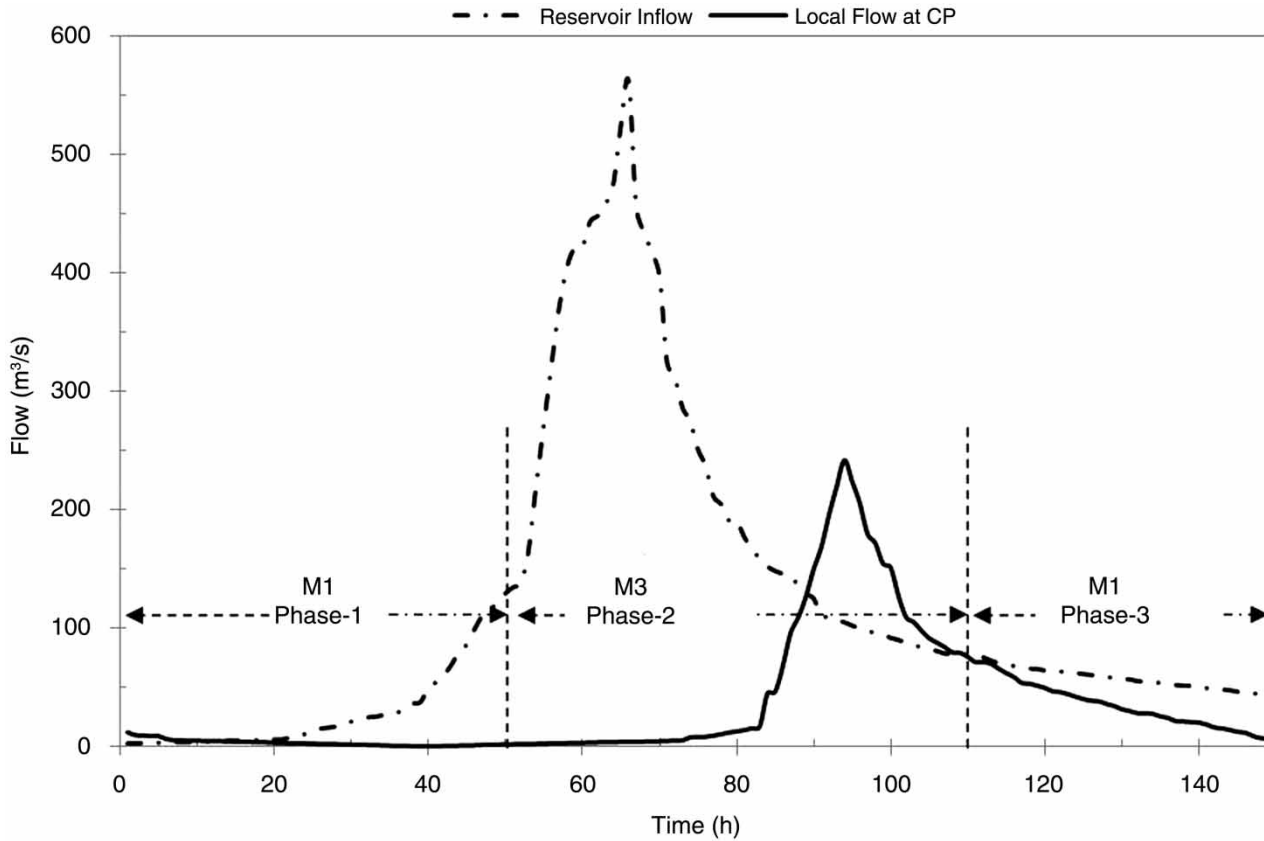


Figure 4 | Inflow time series at the reservoir and the control point for flood event E-2.

compromising solution can be chosen by the decision-maker for possible implementation, after a rigorous evaluation.

Effect of initial reservoir water surface elevation on flood mitigation

With a view to investigate the effect of initial reservoir water surface elevation on the effectiveness of flood mitigation in

Table 2 | Combinations of the objectives for the four multi-objective models considered

| Models ^a | Obj.-I ^b (A) | Obj.-II (B) | Obj.-III (C) | Obj.-IV (D) | Number of objectives |
|---------------------|-------------------------|-------------|--------------|-------------|----------------------|
| M1 | ✓ | ✓ | | | 2 |
| M2 | ✓ | ✓ | | ✓ | 3 |
| M3 | ✓ | ✓ | ✓ | ✓ | 4 |
| M4 ^c | | ✓ | | ✓ | 2 |

^aM1, M2, M3, and M4 indicate some of the possible multi-objective optimization models that are constructed by evoking one or more of the objective functions considered.

^bObj.-I refers to Objective-I and so on.

^cModel M4 has the additional constraint on the end-of-horizon target elevation at the reservoir.

the river–reservoir system, six runs were made using the three objective model M2 (see Table 1), by varying initial reservoir water surface elevation from 438.91 to 446.53 m at 1.52 m interval. The initial state of the outflow from the reservoir and that of the flow at the control point downstream were kept identical over all the six runs. The flood event used for this investigation was event E-1. The results obtained by running the M2 model are presented in Table 4. It may be noted from Table 4 that for the lowest reservoir water surface elevation considered (438.91 m), the band of non-dominant solutions obtained provide a high degree of flood mitigation at the reservoir as well as the control point downstream in terms of significant attenuation of the inflow flood peak (35 to 33% of the inflow flood peak at the reservoir and 22 to 21% of the reservoir inflow flood peak at the control point) and considerable increase in lag time. With increase in the initial reservoir water surface elevation to 446.53 m, the degree of flood mitigation effected is seen to drop progressively and reach a minimum

Table 3 | Role of objective functions in flood mitigation operation of the river-reservoir system-flood event E-1

Initial state of the river-reservoir system

| Reservoir storage (Mm ³) | | Reservoir elevation (m) | | Reservoir release (m ³ /s) | | Control point flow (m ³ /s) | | |
|--------------------------------------|------------|-------------------------|----------------------|---------------------------------------|-----------------------|--|-----------|---------------|
| 16.36 | | 441.96 | | 3.45 | | 3.57 | | |
| Value of objective functions | | | | | | | | |
| Model (participating objectives) | Band width | A (Mm ³) | B (Mm ³) | C (m ³ /s) | D (m ³ /s) | E (hours) | F (hours) | Elevation (m) |
| M1 (A; B) | Maximum | 0.22 | 23.32 | 137.74 | 51.37 | 4 | 5 | 445.14 |
| | Minimum | -4.18 ^a | 19.55 | 113.60 | 33.37 | 2 | 5 | 441.67 |
| M2 (A; B; D) | Maximum | 0.54 | 23.61 | 169.99 | 79.94 | 4 | 5 | 446.16 |
| | Minimum | -5.60 ^a | 19.55 | 151.72 | 76.59 | 2 | 5 | 441.37 |
| M3 (A; B; C; D) | Maximum | 0.02 | 23.11 | 168.99 | 76.83 | 4 | 5 | 446.84 |
| | Minimum | -6.61 ^a | 19.55 | 157.61 | 70.57 | 2 | 5 | 441.85 |

^aNegative values in column (3) indicate that the cumulative outflow from the reservoir is less than the cumulative inflow into the reservoir, during the flood operation horizon.

(8 to 5% of the inflow flood peak at the reservoir and 5 to 4% of the reservoir inflow flood peak at the control point); also, the lag time is seen to drop considerably.

Effect of the end-of-horizon target reservoir elevation constraint

The effects of the end-of-horizon target reservoir water surface elevation constraint on the total excess volume passing through the control point during the flood operation

horizon and the flood peak mitigation at the control point are investigated, for flood event E-1, by making four runs using the two-objective model M4 (refer Table 1). The variation in the end-of-period target reservoir elevation considered for the investigation is from 438.91 to 446.53 m at 1.52 m intervals. The initial reservoir water surface elevation is kept at 441.96 m (corresponding initial reservoir storage = 16.36 Mm³). The initial state of the outflow from the reservoir and that of the flow at the control point downstream are kept identical over all the four runs (Table 5). The

Table 4 | Effect of initial reservoir elevation on the flood peak mitigation-flood event E-1

Initial state of the river-reservoir system

| Reservoir elevation (m) | Reservoir storage (Mm ³) | Model (participating objectives) | Band width | Value of objective functions | | | | | | |
|-------------------------|--------------------------------------|----------------------------------|------------|------------------------------|---------------------|-----------------------|-----------------------|-----------|-----------|---------------|
| | | | | A (m ³) | B (m ³) | C (m ³ /s) | D (m ³ /s) | E (hours) | F (hours) | Elevation (m) |
| 438.91 | 13.14 | M2 (A; B; D) | Maximum | 2,380.21 | 5,600.30 | 243.07 | 152.02 | 5 | 6 | 445.95 |
| | | | Minimum | 810.25 | 4,535.69 | 230.15 | 145.72 | 4 | 6 | 441.59 |
| 440.44 | 14.75 | M2 (A; B; D) | Maximum | 1,912.44 | 5,892.07 | 209.14 | 110.62 | 4 | 5 | 445.91 |
| | | | Minimum | 519.21 | 4,983.06 | 194.60 | 105.08 | 3 | 5 | 442.10 |
| 441.96 | 16.36 | M2 (A; B; D) | Maximum | 148.71 | 6,559.33 | 169.99 | 79.94 | 4 | 5 | 446.16 |
| | | | Minimum | -1,555.57 ^a | 5,430.64 | 151.72 | 76.59 | 2 | 5 | 441.37 |
| 443.48 | 18.19 | M2 (A; B; D) | Maximum | 516.34 | 6,926.57 | 123.49 | 62.68 | 2 | 5 | 445.68 |
| | | | Minimum | -861.74 ^a | 5,940.19 | 115.48 | 57.62 | 2 | 4 | 441.85 |
| 445.01 | 20.18 | M2 (A; B; D) | Maximum | 1,101.50 | 7,512.66 | 88.72 | 45.59 | 2 | 5 | 446.21 |
| | | | Minimum | -516.44 ^a | 6,491.38 | 70.59 | 39.19 | 1 | 4 | 441.74 |
| 446.53 | 22.33 | M2 (A; B; D) | Maximum | 1,859.21 | 8,254.16 | 57.29 | 31.15 | 1 | 5 | 445.61 |
| | | | Minimum | 313.32 | 7,059.55 | 32.69 | 26.75 | 0 | 4 | 441.19 |

^aNegative values in column (5) indicate that the cumulative outflow from the reservoir is less than the cumulative inflow into the reservoir, during the flood operation horizon.

Table 5 | Summary of the multi-objective solutions for different target reservoir elevations at the end of the operation horizon- flood event E-1

| Initial state of the river–reservoir system | | | | |
|---|-------------------------|---------------------------------------|--|--|
| Reservoir storage (Mm ³) | Reservoir elevation (m) | Reservoir release (m ³ /s) | Control point flow (m ³ /s) | |
| 16.36 | 441.96 | 3.45 | 3.57 | |

| Model (participating objectives) | End-of-horizon target reservoir elevation (m) | Value of objective functions | | |
|----------------------------------|---|------------------------------|-----------------------|-----------------------|
| | | B (m ³) | C (m ³ /s) | D (m ³ /s) |
| M4 (B; D) | 441.96 | 6,382.82 | 139.17 | 79.49 |
| | 443.48 | 5,871.72 | 143.98 | 84.26 |
| | 445.01 | 5,430.21 | 147.18 | 85.02 |
| | 446.53 | 5,430.30 | 148.45 | 85.35 |

results obtained by running the M4 model are presented in Table 5. It may be noted from Table 5 that in order to reach the lowest end-of-period target reservoir water surface elevation considered (441.96 m), a large excess volume of water needs to be passed through the control point and less flood peak mitigation is effected at the control point. With an increase in the end-of-period target reservoir elevation, the excess volume to be passed through the control point reduces, while the flood peak mitigation at the control point increases (Table 5).

Improvement in flood mitigation due to dynamic decision-making

In this analysis, in order to find out the level of improvement in flood mitigation effected by the dynamic model over the static models, the S–O framework (Figure 1) is run for both the static and the dynamic models for the event E-2 (Figure 4). It may be noted that for both the static and the dynamic models, the initial state of the river–reservoir system (in terms of reservoir water surface elevation, reservoir outflow and the flow at the control point downstream is kept identical (Table 6) to facilitate the comparison.

The *dynamic* model is run in different phases considering the prevailing situation and the expected condition in the river–reservoir system during the passage of the flood. In case of flood event E-2, three distinct phases of flood control operation are identified after a careful inspection of the reservoir inflow flood hydrograph and the local flow hydrograph at

the control point, which are assumed to be available as a result of perfect forecast. The first phase starts when the flood arrives at the reservoir (time = 0 h) and is assumed to extend until 50 hours (when the rising limb of the inflow flood hydrograph starts ascending with a steep slope). During this phase, it is essential to provide sufficient storage space at the reservoir for moderating the peak flood that is anticipated within some hours, while minimizing the excess flow volume at the control point, located downstream of the reservoir where flood protection is to be achieved. This is best achieved by using the M1 model (Table 1). The results from phase-1 are presented in Table 6, from which it can be seen that the non-dominant solutions form a narrow band. A typical non-dominant solution from this band that has an end-of-period reservoir elevation of 437.89 m (slightly above the bottom of the flood control pool) is selected (Table 6). For this selected solution, the reservoir outflows and elevations and the control point flows are computed using the river–reservoir simulation model in the framework. Following this, the state of the system at the end of the first phase is transferred to the start of the second phase.

Once the inflow hydrograph rises steeply (from 50 hours onwards), the M1 model has been switched to the M3 model since the flood peak at the reservoir as well as the flood peak at the control point downstream are to be mitigated to the maximum extent. These two objectives are given priority until the time the local flow peak at the control point passes (110 hours-end of phase-2 in Figure 4). The results from phase-2 are also presented in Table 6, from which it can be seen that the non-dominant solutions form a narrow band. A typical non-dominant solution from this band that yields a flood peak mitigation of 202.15 m³/s at the reservoir and 198.37 m³/s at the control point and with an end-of-period reservoir elevation of 447.71 m (just above the top of the flood control pool) is selected (Table 6). For this selected solution, the reservoir outflows and elevations and the control point flows are computed using the river–reservoir simulation model in the framework. Following this, the state of the system at the end of the second phase is transferred to the start of the third phase.

After the passage of the local flow peak at the control point, the phase-3 starts at 110 hours and continues until the end of the flood event. At this time, it becomes essential to drain the surplus volume from the reservoir in order to

Table 6 | Dynamic decision-making operation of the river–reservoir system and comparison of the dynamic model solution with the static model solution – flood event E-2**Initial state for first phase**

| Reservoir storage (Mm ³) | | Reservoir elevation (m) | | Reservoir release (m ³ /s) | | | Control point flow (m ³ /s) | | |
|--------------------------------------|--------------------|-------------------------|----------------------|---------------------------------------|-----------------------|-----------------------|--|-----------|---------------|
| 16.36 | | 441.96 | | 3.45 | | 12.44 | | | |
| Full event (Static) | Model M2 (A; B; D) | Band width | A (Mm ³) | B (Mm ³) | C (m ³ /s) | D (m ³ /s) | E (hours) | F (hours) | Elevation (m) |
| | | Maximum | –4.84 | 34.85 | 136.43 | 143.04 | 4 | 5 | 447.22 |
| | | Minimum | –6.91 | 33.09 | 127.38 | 141.13 | 1 | 3 | 445.84 |
| Phase-1 | M1 (A; B) | Maximum | 4.11 | 0.04 | – | – | – | – | 437.98 |
| | | Minimum | 4.00 | 0.00 | – | – | – | – | 437.89 |
| | | Selected | 4.10 | 0.03 | – | – | – | – | 437.89 |

Initial state for second phase, data taken from M1 at 50 hours of flood simulation time

| Reservoir storage (Mm ³) | | Reservoir elevation (m) | | Reservoir release (m ³ /s) | | | Control point flow (m ³ /s) | | |
|--------------------------------------|-----------------|-------------------------|----------------------|---------------------------------------|-----------------------|-----------------------|--|-----------|---------------|
| 12.22 | | 437.89 | | 115.76 | | 93.56 | | | |
| Phase-2 | Model M3 (C; D) | Band width | A (Mm ³) | B (Mm ³) | C (m ³ /s) | D (m ³ /s) | E (hours) | F (hours) | Elevation (m) |
| | | Maximum | –11.98 | 30.90 | 202.16 | 198.39 | 5 | 5 | 447.72 |
| | | Minimum | –12.09 | 30.81 | 201.96 | 198.28 | 1 | 3 | 447.66 |
| | | Selected | –12.08 | 30.82 | 202.16 | 198.28 | 5 | 5 | 447.71 |

Initial state for third phase, data taken from M1–M3 at 110 hours of flood simulation time

| Reservoir storage (Mm ³) | | Reservoir elevation (m) | | Reservoir release (m ³ /s) | | | Control point flow (m ³ /s) | | |
|--------------------------------------|--------------------------|---|--|--|---|------------------------|--|--|-------------------|
| 24.34 | | 447.71 | | 106.70 | | 171.68 | | | |
| Phase-3 | Model M1 (A; B) | Band width | A (Mm ³) | B (Mm ³) | C (m ³ /s) | D (m ³ /s) | E (hours) | F (hours) | Elevation (m) |
| | | Maximum | 2.52 | 3.67 | – | – | – | – | 447.50 |
| | | Minimum | 0.41 | 1.94 | – | – | – | – | 446.32 |
| | | Selected | 0.56 | 1.94 | – | – | – | – | 447.43 |
| Selected dynamic model solution | Dynamic model (M1–M3–M1) | End-of-horizon reservoir storage (Mm ³) | End-of-horizon reservoir elevation (m) | End-of-horizon reservoir release (m ³ /s) | Reservoir-peak mitigation (m ³ /s) | Reservoir-lag time (h) | End-of-horizon CP flow (m ³ /s) | C.P.-peak mitigation (m ³ /s) | C.P.-lag time (h) |
| | | 23.92 | 447.43 | 95.16 | 202.16 | 5 | 84.83 | 198.28 | 5 |
| Selected static model solution | Static model (M2) | 23.42 | 447.22 | 126.20 | 134.81 | 1 | 107.86 | 141.33 | 5 |

ensure the safety of the dam, while keeping the excess volume passing at the control point during phase-3 under check. In other words, the M1 model (Objectives I and II) is reactivated and the M3 model that deals with flood peak mitigation is disabled. A typical non-dominant solution from this band that has an end-of-period reservoir elevation of 447.43 m is chosen (Table 6). For this selected solution,

the reservoir outflows and elevations and the control point flows are computed using the river–reservoir simulation model. The basic details concerning the selected solutions for the static model and the dynamic model are also presented in Table 6.

The comparative results regarding the outflow hydrograph at the reservoir and the flow hydrograph at the

control point obtained from the *dynamic* (M1-M3-M1) and the *static* (M2) models are depicted in Figures 5 and 6 respectively, for the flood event E-2. It is to be noted that the *dynamic* model gives 33% more mitigation over the *static* model at the reservoir (Figure 5), while 28% more mitigation is obtained at the control point (Figure 6). In the case of the dynamic model, once the M1 model is turned off and the M3 model is activated at 50 hours (Figures 5 and 6), the flood peak is mitigated effectively, since the objectives work towards that. In order to achieve this, the M3 model proactively discharges significant volume of water from the reservoir between 50 and 64 hours (Figure 5) through the bottom outlets and provides sufficient storage space in the reservoir. This, in turn, results in effective flood peak mitigation at the control point (Figure 6). Moreover, at the end of the second phase (110 hours), the M3 model is switched off, since the control point peak flow has already passed, and the M1 model is activated. As a result, the two volume objectives come into play and the volume of excess flow passing over the control point is reduced. This continues for nearly 10 hours (Figure 6).

In the case of the static model (M2), the two volume objectives and the one objective that specifies the control point flood peak mitigation are active during the entire operation horizon. Thus, the system dynamics effect is not reflected in this operation strategy. Hence, there is less flood peak mitigation at the reservoir as well as the control point, as indicated earlier (Figures 5 and 6). Moreover, even after the flood peaks of the reservoir inflow and the control point flow have passed, the outflows from the reservoir continue to be high for some more hours (Figure 5), as the levels in the reservoir continue to be slightly above the spillway crest during this period, unlike in the case of the dynamic model. In turn, the outflows at the control point are also more during this period (Figure 6).

The computational time for a single run using the simulation–optimization framework for the event E-1 (which has an operating horizon of 77 hours) is approximately 7 minutes. For the event E-2 (which has an operating horizon of 150 hours), the computational time is approximately 14 minutes for the static model and 20 min for the dynamic model, on a Pentium 4 computer with 3.2 GHz processor and 1 GB

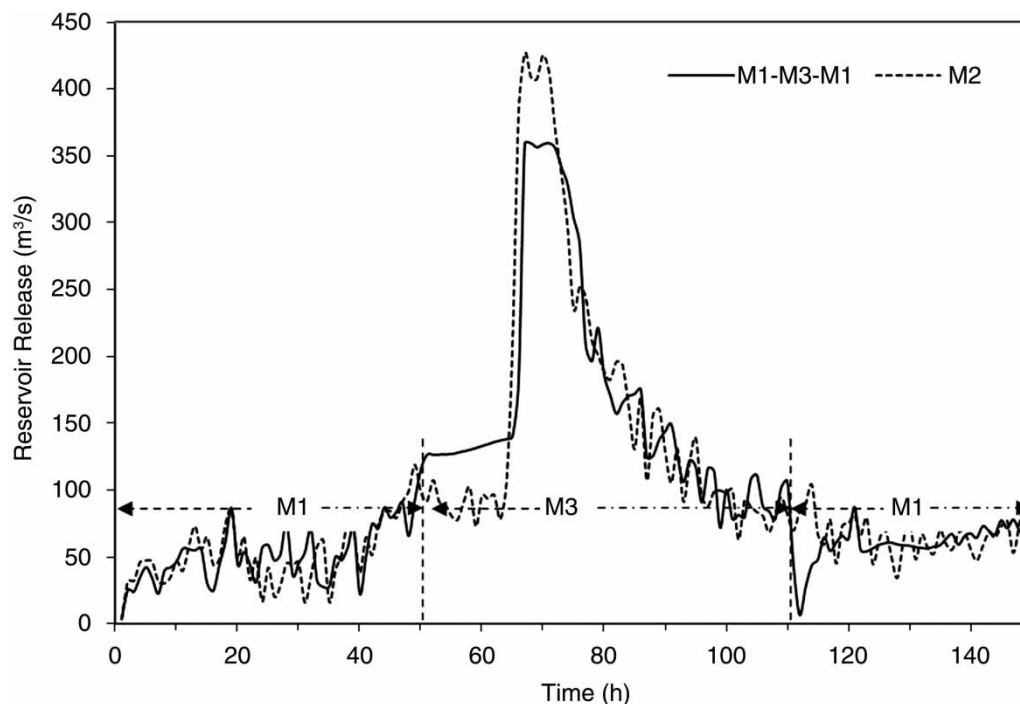


Figure 5 | Comparison of the outflow hydrographs at the reservoir between the dynamic model (M1-M3-M1) and the static model (M2) for the flood event E-2.

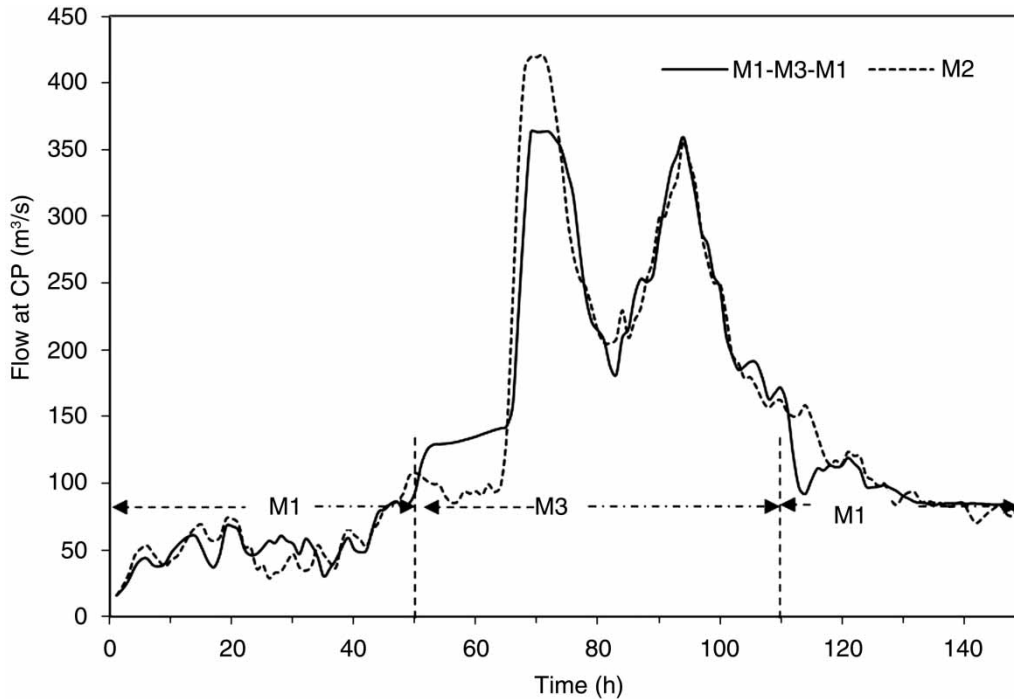


Figure 6 | Comparison of the outflow hydrographs at the control point between the dynamic model (M1-M3-M1) and the static model (M2) for the flood event E-2.

RAM. However, with the availability of computers having higher configuration, this greatly reduces the run time, which is critical for any dam operator during the flood event. Hence, the proposed framework can be used in real-time flood operation and management applications.

EXTENSION OF THE S–O FRAMEWORK TO MULTIPLE RIVER–RESERVOIR SYSTEMS

The complexity of the river–reservoir system increases with higher numbers of reservoirs and control points and their relative locations, since each one of them may have different physical and operational characteristics, highly variable temporal patterns of flood flows, and damage reaches with distinctly different characteristics. Moreover, local flows may be contributing at various control points along the river system and at various times, which may or may not coincide or overlap in time with the upstream reservoir releases and/or inflows. Also, some of the reservoirs may be of multi-purpose type while some others may be

exclusively for flood control. The reservoir elevation at the end of a flood event is important from the view point of minimizing the risk due to dam overtopping (which is catastrophic), especially when the time gap between successive flood events is less or when sufficient conservation storage is to be provided in certain reservoirs at the end of the flood season. Considerable variation in these boundary conditions may exist across the different reservoirs in a basin. All these complexities make the dynamic operation of multi-reservoir flood management quite challenging. Moreover, during the passage of floods in the multiple river–reservoir system, the physical situation that exists keeps changing dynamically, resulting in changing priorities of the reservoir operators. A brief discussion regarding the extension of the adaptive multi-objective framework proposed for the dynamic flood control operation of river–reservoir systems to multiple reservoirs and control points is presented in the following paragraphs.

The multiple objectives that would be addressed in the extended framework are: (i) maximize the sum of the total volume of water drained from the reservoirs during the operation horizon of the flood; (ii) minimize the sum of the

cumulative flood volumes (above the channel capacity) passing the control points during the operation horizon of the flood; (iii) maximize the sum of flood peak mitigation at all the reservoirs; (iv) maximize the sum of flood peak mitigation at the control points; and (v) minimize the sum of the deviations of the end-of-operation horizon water surface elevations of the reservoirs from the respective target water surface elevations. The formulation also consists of the physical system constraints concerning lower and upper limits of reservoir storages and release restrictions at the reservoirs, channel carrying capacities, rate of increase/decrease of outflows from the outlets of the reservoirs, channel routing equations for the Coefficient Routing Method, upper and lower limits of target water surface elevation at the reservoirs at the end of the flood simulation period. The mass balance between the inflow and the outflow from each of the reservoirs is described through the storage-continuity equations written in a difference form that is useful in routing the flood flows through each reservoir. Due to lack of space, these equations are not presented here. The storage-continuity equations and the river routing equations are framed once the information concerning the network configuration of the river–reservoir system is known.

Dynamic reconstruction of the multi-objective flood control optimization model is enabled within the framework at each time stage (phase) by combining different sets of objective functions along with the physical and operational constraints of the multiple river–reservoir system. Each of these optimization models will be executed during different time stages of the flood event. The number of phases of flood operation is to be decided by the river–reservoir system operator based on the forecast information regarding the reservoir inflows and the local flows at the control points, and a good knowledge about the overall system characteristics including the flood damage potential at the control points. The identification of the objective functions to be activated at each reservoir and control point at each phase of the flood control operation will have to be done by a system operator who is knowledgeable and experienced with the multiple river–reservoir system. Especially while dealing with multiple reservoirs and control points, the heuristic knowledge about the operation of the river–reservoir system concerning the operational priorities of the reservoir operators/managers at various stages of

the flood events and the accurate contemporary multi-period ahead flow forecasts at the various gauges/control points in the system are vital for the effective functioning of this adaptive framework. This knowledge would help in deciding the number of time stages and the actual times at which the alternative model combinations are to be run and also in reducing the number of alternative combinations of models to be tried out at each time stage. This would provide a clear focus and also effect considerable saving of time for the operator/decision-maker, which is very crucial in reducing damage to life and property during the floods.

During the different time stages (phases) of the flood control operation, one or more of these model variations can be set up within the model framework and executed in different computers/terminals placed in the control station and competent operational decisions can be taken during each phase by comparing the output results from these models. It is to be mentioned that this demands coordinated operation among the system of reservoirs by knowledgeable and experienced decision-makers/reservoir operators with the river–reservoir system. The availability of accurate flow forecasting system and efficient communication system would make this kind of coordinated dynamic operation of the system of reservoirs in a basin more effective. This kind of a dynamic operational framework proposed can also be used by the reservoir system operators to gain expertise regarding the combinations of objectives as well as models to be activated during the various possible scenarios of flood flows, initial reservoir levels and expected target reservoir elevations at the end of the flood event or flood season.

The authors have successfully applied this extended framework to a multiple river–reservoir system case example with two parallel upstream flood control reservoirs and three downstream control points (Prakash *et al.* 2014). The results of the same are not provided here due to brevity. However, the ideas presented and demonstrated in this research work concerning the adaptive and dynamic flood control operation of river–reservoir systems can be developed further to deal with the real-time flood control operation of complex river–reservoir systems (consisting of serial-parallel combinations of reservoirs and several control points and damage reaches within a river basin).

SUMMARY AND CONCLUSIONS

This study proposes an adaptive simulation–optimization framework that facilitates a rational representation of the dynamic nature of the decision-making process during the different phases (time stages) of any flood control operation during a flood event in a single river–reservoir system. This framework enables the use of one or more of the following five objective functions adaptively during the different phases of the operation: (i) maximize the cumulative volume of water drained from the reservoir during the flood operation horizon; (ii) minimize the cumulative excess flow volume (in excess of the specified channel capacity) at the control point during the flood operation horizon; (iii) maximize the flood peak attenuation at the reservoir; (iv) maximize the flood peak attenuation at the control point; and (v) minimize the deviation of the reservoir water surface elevation at the end of the flood operation horizon from the desired target water surface elevation at the reservoir. The identification of the different phases and the selection of the appropriate set of objective functions to be employed at each phase of the flood control operation will have to be done by the system operator/decision-maker who is supposed to be knowledgeable and experienced with the river–reservoir system. The robustNSGA-II has been employed as the search technique to solve the multi-objective optimization model within the adaptive simulation–optimization framework.

The following investigations are carried out using the adaptive simulation–optimization framework developed: (1) the role of the different objective functions in the flood mitigation operation in the river–reservoir system; (2) the effect of the initial reservoir water surface elevation on the effectiveness of flood mitigation in the river–reservoir system; (3) the effect of the end-of-horizon target reservoir elevation constraint on the excess volume passing at the control point and the flood peak mitigation at the control point; and (4) the level of improvement in flood mitigation due to dynamic decision-making (that makes use of different sets of objective functions) during the different phases of the flood mitigation operation.

The results from the case example show that the developed framework is effective in representing the river–

reservoir system dynamics during flood control operations and also to achieve the target elevation at the reservoir for conservation use or to provide the space for the oncoming flood as the case may be. The dynamic flood control operation model yields a significant level of improvement in flood peak mitigation over the static model both at the reservoir as well as at the control point.

A brief discussion on the extension of the proposed river–reservoir system framework for flood control operation during flood events to multiple river–reservoir systems is presented. While dealing with multiple reservoirs and control points, the heuristic knowledge of the operation of the river–reservoir system concerning the operational priorities of the reservoir operators/managers at various stages of the flood events and the accurate contemporary multi-period ahead flow forecasts at the various gauges/control points in the system are vital for the effective functioning of this adaptive framework. This knowledge would help in deciding the number of time stages and the actual times at which the alternative model combinations are to be run and also in reducing the number of alternative combinations of models to be tried out at each time stage. This would provide a clear focus and also effect considerable saving of time for the operator/decision-maker, which is very crucial in reducing damage to life and property during the floods.

The proposed S–O framework can be a useful tool in developing either deterministic or probabilistic optimal reservoir release policies for flood mitigation during flood events in river–reservoir systems. Also, it can be useful in training the reservoir operators to develop a good knowledge base regarding the response of the river–reservoir system to the various plausible flood events and the operational scenarios, which in turn will be beneficial in the operation of the real-time flood events. This framework can be easily extended to real-time flood control operation of multiple river–reservoir systems, by introducing a state-of-the-art real-time flood forecasting model, which will be able to yield reliable forecasts for a lead time of 6–8 hours during floods. Also, improvements can be made to the framework by way of replacing the hydrologic routing component employed for channel routing in this study with an appropriate hydraulic routing method such as dynamic flood wave routing.

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