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# SUPERVISORY CONTROL OF RIVER FLOOD: NETWORK MODELING WITH VARYING TIME DELAY

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**ABSTRACT:** *Food phenomena become an usual phenomenon occurring in the world and causing serious human and material damage. One way to protect urban zones from river floods is to equipped river with flood control area used as reservoirs in order to reduce the velocity of water and to attenuate the flood wave. The work presented in this paper concerns the supervisory control of such an equipped river. The supervisory scheme consists in four blocks connected to the river process: a Supervisory Control and Data Acquisition block, a Dynamic Parameterization block, a Diagnosis-Decision-Correction block and a Management Objectives and Constraints Generation block. The proposed method is based on a dynamic method combining a reduced transportation network and a temporized matrix from which the water volumes to be stored or released in time are calculated. It makes possible the water storage and release adapted to each river flood scenario, and preservation of agriculture in these floodplains. It takes into account the variation of the time delay with the flow without any modification in the structure of the network.*

**KEYWORDS:** *Water systems management, Supervisory control, Transportation networks, Time delay, Flood control.*

## 1 INTRODUCTION

Flood is an usual phenomenon all over the world. Extreme rainfall events become more frequent and the induced damages more severe. Recently, at the end of April 2014, rainstorm caused floods in the North of Florida, as well as in the east of the United States, rains reached up to 550 millimeters of water. Roads were cut, a street collapsed in Baltimore, flights were delayed, hundred of people have been evacuated, power failure affected more than 28000 homes. This disaster caused the loss of 35 lives (source AFP). Flooding due to excessive rains can cause significant human and material damages around the world.

One way to prevent these flood problems is to equipped river with flood control area used as reservoirs. The reservoirs are emptying with water in order to reduce the water velocity in the river and to attenuate the flood wave. Various research works have been proposed in order to reduce flood peaks and volumes involving linear programming (Needham et al., 2000), folded dynamic programming (Nagesh Kumar et al., 2009), hybrid analytic/rule-based approach (Karbowski et al., 2005) for example. Most of these methods do not allow controlling the duration of water storage in the reservoir, the storage and

release dates ... In order to improve the managers' decisions during these abrupt climatic phenomena, optimization techniques were proposed such as linear programming (Karamouz et al., 2003), fuzzy optimization (Fu, 2008; Cheng and Chau, 2001), stochastic optimization (Ratnayake and Harboe, 2007) and multi-objective optimization (Chuntian and Chau, 2002). Rivers are equipped with sensors and actuators and Supervisory Control and Data Acquisition systems (SCADA) are developed in order to improve their control. Such SCADA systems are used to collect data from sensors, communicate with operators through a Human Machine Interface, and send control values to actuators in many kind of systems such as irrigation canals (Figueiredo et al., 2013; Puerto et al., 2013; Pfitscher et al., 2012), inland navigation networks (Duviella et al., 2013), or energy management (Mora et al., 2012) ; network vulnerabilities of such systems are studied in (Amin et al., 2013a,b).

In order to manage the water volumes in case of flood in river area, a supervisory control scheme is proposed in this paper. The structure of the paper is as follows. Section 2 describes the proposed scheme which includes water storage and release in the reservoirs and the variation of time delay with discharge. The effectiveness of the proposed supervisory scheme

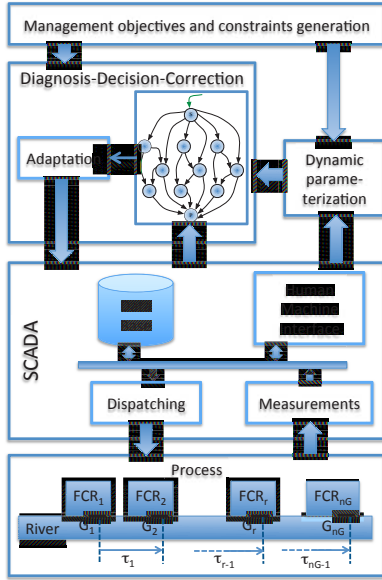


Figure 1 – Supervisory Control Scheme.

is shown for different flood situations on a simulated river system in section 3. Finally, conclusion is given and some future works are suggested.

## 2 SUPERVISORY CONTROL SCHEME

### 2.1 General Scheme

In order to manage the water volumes in case of flood arising in river area, a supervisory control scheme depicted in Figure 1 is proposed. The process is a river system along which  $n_G$  reservoirs are distributed. These reservoirs, denoted  $FCR_r$  (flood control reservoir) are floodplains provided with a controlled gate  $G_r$ ,  $r = 1, \dots, n_G$ , and are used to absorb the flood. The time delay,  $\tau_r$ , from the gate  $G_r$  to the following gate  $G_{r+1}$  depends on the flow discharge. The gate opening should be computed in order to maintain the river discharge under a predefined flow value called the attenuation flow. Thus the control of discharges leads to limit the level of the river.

The SCADA system is connected to the river process. It transmits the sensors' values to the Diagnostic-Decision-Correction and the Dynamic Parameterization blocks, and receives the gate opening values in order to send them to the process. The measurements considered herein are levels and discharges.

The Diagnostic-Decision-Correction block (DDC) permits the determination of the gate opening setpoints. It is composed of a transportation network which diagnose the process state, depending on the flow in the river, decide if a correction must be carried out and execute it. The transportation network includes time delays, moreover, if the time delays vary, it is not necessary to change the network structure (no need to add node or arc, only network param-

eters are modified); as described in the next section (section 2.2). Setpoints values are adapted in order to be understood by the SCADA system.

The Dynamic Parameterization (DP) block provides the DDC block with all the necessary dynamic characteristics such as time delays.

The Management Objectives and Constraints Generation block supplies the DDC and DP blocks with management constraints and rules such as the thresholds, the attenuation flow value, the priority parameters.

### 2.2 Network With Time Delay

The river with FCR is modeled with a transportation network including time delay. In previous work (Nouasse et al., 2012), we firstly proposed the use of a static transportation network, where time delay where neglected. Because of the importance of the flow delay in the river, the method was improved to include time delay (Nouasse et al., 2013a) and storage and release operations for reservoirs (Nouasse et al., 2013b). The method is composed of a transportation network  $\mathbb{G}$  and a Temporization Matrix ( $TM$ ), as described in Figure 2. In these works, time delays were constant during the simulation; in the present paper, the method is proved to be efficient with delay varying during the simulation.

The objective of the method is to maintain the flow under a predefined attenuation flow denoted  $Q_{lam}$ . The attenuation flow is a flow threshold under which the river flow should remain as, expected by the river system manager. Moreover, in order to protect agriculture and to be able to control a new flood episode, when the reservoirs are not empty, and when the discharge level in the river is lower than the attenuation threshold, the stored water can be released. For this purpose a threshold,  $Q_{do}$ , is defined as the discharge level under which the water is released from the reservoir. In the case of release, gates are opened in a way that the discharge level in the river remains under the attenuation threshold  $Q_{lam}$ .

In order to determine the optimal attenuation flow that satisfies the management objectives and the flood occurring case, we formulated the problem as a *Min-Cost-Max-Flow* problem. The cost function to minimize is subject to the constraints of flow conservation and minimal and maximal capacities in the network. The network  $\mathbb{G}$  enables water storage and release from reservoirs. In order to include variable transfer time in the network without oversizing it, the values of delayed flow are stored in the  $n \times 2n_G$   $TM$  matrix. Each column represents the evolution of a gate or a FCR, and each line represents an instant of the evolution of the state of the system,

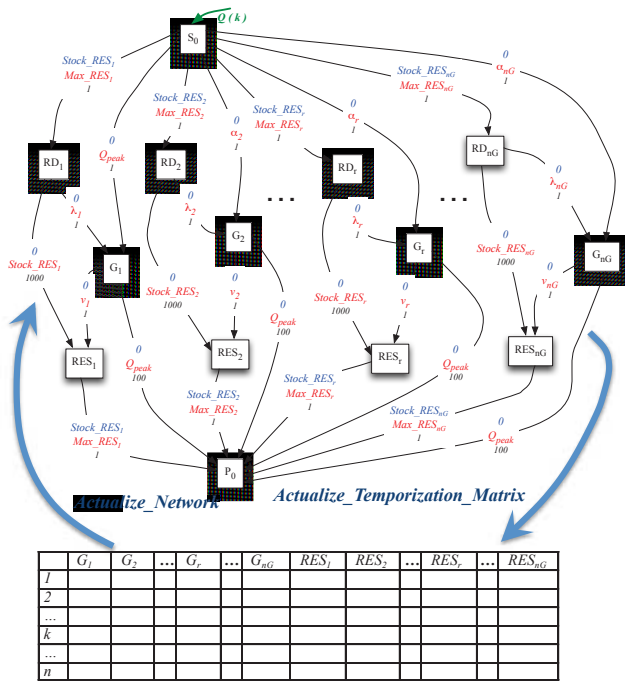


Figure 2 – Network for Diagnosis-Decision-Correction Block.

at each  $kT_c, k = 0, \dots, n$ , in the horizon  $H_f$ , with  $H_f = n \times T_c, n \in \mathbb{N}^+$ .

The network  $\mathbb{G} = \{\mathcal{N}, \mathcal{A}\}$  where  $\mathcal{N}$  is a set of  $3n_G + 2$  nodes defined as follows, with  $r = 1, \dots, n_G$ :

- The node  $G_r$  represents the gate  $G_r$ ;
- The node  $RD_r$  is a release decision node;
- The node  $RES_r$  is a reservoir node;
- The node  $S_0$  is a source node corresponding to the fictive entry point of the system;
- The node  $P_0$  is a sink node corresponding to the fictive exit point of the system.

The arcs belonging to the set  $\mathcal{A}$  between the nodes of  $\mathcal{N}$  are valued, each value is between a maximum capacity and a minimum capacity written respectively in blue and red in Figure 2. The arc value at  $kT_c$  is computed using an optimization algorithm as detailed in the following. The arcs describe the following connections:

- The arc between the node  $S_0$  and the node  $RD_r, r = 1, \dots, n_G$ , represents the water volume already stored in the reservoir linked to it. Its maximum capacity is  $Max\_RES_r$ , the maximum storage capacity of the reservoir, which depends on the maximum value of the peak flow-rate of the flood ( $Q_{peak}$ ). Its minimum capacity is  $Stock\_RES_r$ , and it corresponds to the amount of water already present in the reservoir.

- The arc between the node  $RD_r$  and the node  $G_r, r = 1, \dots, n_G$ , represents the draw-off flow leaving the reservoir. Its maximum capacity is denoted  $\lambda_r$ .
- The arc between the node  $RD_r$  and  $RES_r, r = 1, \dots, n_G$ , represents the water volume not released and remaining in the reservoir at the end of the concerned period. Its maximum capacity is  $Stock\_RES_r$ , the amount of water already present in the reservoir.
- The arc between the node  $S_0$  and the node  $G_r, r = 2, \dots, n_G$ , takes into account the discharge upstream from the gate  $G_r$  in the system. Its maximum capacity is  $Q_{peak}$ .
- The arc between the node  $G_r$  and the node  $RES_r$ , connects the gate with its reservoir. It represents the flow leaving the river through the gate  $G_r$  towards the reservoir, i. e. the stored water. Its maximum capacity is denoted  $\nu_r$ .
- The arc between the node  $G_r$  and the node  $P_0, r = 1, \dots, n_G - 1$ , represents the flow transferred from the gate  $G_r$  to the next gate  $G_{r+1}$ . This discharge is stored in the column of  $TM$  associated to the gate  $G_{r+1}$  at line  $k + k_r$ , with  $k_r = \lfloor \frac{r}{T_c} \rfloor$ . Its maximum capacity is  $Q_{peak}$ .
- The arc between the node  $G_{nG}$  and the node  $P_0$ , corresponds to the flow-rate downstream from the exit point of the system. Its maximum capacity is  $Q_{peak}$ .
- The arc between the node  $RES_r$  and the node  $P_0, r = 1, \dots, n_G$ , represents the total volume of water remaining in the reservoir. It respects transportation network conservation flow rules. Its maximum capacity is  $Max\_RES_r$ , its minimum capacity is  $Stock\_RES_r$ .

In Figure 2, the cost of each arc is written in black. In order to limit overflow downstream, the cost of the  $G_{nG}P_0$  arc is set to a high value, here 100. Similarly, the cost of the  $G_rP_0$  arcs ( $r = 1, \dots, n_G - 1$ ) are set to a value lower or equal to the  $G_{nG}P_0$  arc cost, here 100. In order to release water only in the case where there is no overflow risk, the costs of the  $RD_rRES_r$  arcs ( $r = 1, \dots, n_G$ ) were set to a value higher than the cost value of the  $G_{nG}P_0$  arc, here 1000. Finally, the three reservoirs are considered to have a similar role, thus, all other costs were set equal to 1.

The Flood-Attenuation algorithm, described in algorithm 1, permits to determine the gate opening setpoint values. The computation of the setpoints is based on the arc values. Firstly, the temporization matrix is initialized. Thereafter, at each  $k$ , the network and the temporization matrix are actualized (see algorithm 2 and algorithm 3), the optimal flow is

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**Algorithm 1: Flood Attenuation**

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**input :**  
 $\mathbb{G}$  the network  
 $n = \lfloor \frac{H_f}{T_c} \rfloor + 1$  the number of samples  
 $n_G$  the number of gates and FCR in the system  
 $k_r = \lfloor \frac{r}{T_c} \rfloor, r = 1, \dots, n_G - 1$   
 $Q_{input}(k)$  the flow of flood at  $kT_c, k = 1 \dots n$   
 $Q_{lam}$  the attenuation flow  
**output:**  
 $\mathbb{G}$  the network  
 $TM$  the  $n \times 2n_G$  temporization matrix  
 $\varphi^*$  the optimal flow for each arc in  $\mathbb{G}$   
**begin**  
  % Initialization phase one  
  **for**  $k = 1$  **to**  $n$  **do**  
     $TM(k, 1) \leftarrow Q_{input}(0)$   
    **for**  $r = 2$  **to**  $2n_G$  **do**  
       $TM(k, r) \leftarrow 0$   
    **end**  
  **end**  
  % Initialization phase two  
  **for**  $r = 1$  **to**  $n_G - 1$  **do**  
    **for**  $k = 1$  **to**  $k_r - 1$  **do**  
       $TM(k, r + 1) \leftarrow \min(Q_{input}(0), Q_{lam})$   
    **end**  
  **end**  
   $k \leftarrow 1$   
  **while**  $(k \leq n)$  **do**  
     $Actualize\_Network(\mathbb{G}, k, TM)$   
     $\varphi^*(k) \leftarrow Compute\_Optimal\_Flow(\mathbb{G}, k)$   
     $Actualize\_Matrix(\varphi^*(k), k, TM)$   
     $k \leftarrow k + 1$   
  **end**  
**end**

---

computed. In order to compute the optimal flow, the Min cost Max flow problem resolution for this network is done, using a Linear Programming formulation (as described in (Nouasse et al., 2012)), according to our management objectives.  $\varphi_{xy}^*(k)$  is the obtained optimal flow from node  $x$  to node  $y$  in the network  $\mathbb{G}$  at  $kT_c$ ; thus, at  $kT_c$ , we can derive the gate  $G_r$  opening setpoint value which is equal to  $\varphi_{G_r, RES_r}^*(k)$  in the storage case and to  $\varphi_{RD_r, G_r}^*(k)$  in the release case. During the phase one of the initialization of the Flood-Attenuation algorithm, the first column of  $TM$  matrix is set to the value of the upriver flow at each  $kT_c$  ( $k = 1, \dots, n$ ), which is the flow upstream from the first gate  $G_1$ . The initialization phase two allows us to introduce the flow values for all the gates  $G_r (r = 2, \dots, n_G)$  during the non-stationary phase i. e. before  $k = k_{n_G-1}$ , with  $k_{n_G-1} = \lfloor \frac{\tau_{n_G-1}}{T_c} \rfloor$ . We chose in this case to set these upstream discharges to the upriver flow except when it is higher than the attenuation flow. In the algorithm 2,  $Q(k)$  is the flow-rate entering the network at  $kT_c$ . It is equal to the sum of flows entering the gates added to the sum of

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**Algorithm 2: Actualization Network**

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**input :**  
 $TM$  the  $n \times 2n_G$  temporization matrix  
 $\mathbb{G}$  the network  
 $n_G$  the number of gates and RES in the system  
 $k$  the iteration number  
 $Q_{lam}$  the attenuation flow  
 $Q_{do}$  the draw-off discharge threshold  
 $Max\_RES_r$  the maximum storage capacity of  $RES_r$   
**output:**  
 $\mathbb{G}$  the network  
**begin**  
  **for**  $r = 1$  **to**  $n_G$  **do**  
    **if**  $TM(k, r) \geq Q_{lam}$  **then**  
       $\gamma_r \leftarrow 1; \mu_r \leftarrow 0$   
    **else**  
      **if**  $(TM(k, r) < Q_{do})$  **then**  
         $\mu_r \leftarrow 1; \gamma_r \leftarrow 0$   
      **else**  
         $\mu_r \leftarrow 0; \gamma_r \leftarrow 0$   
      **end**  
    **end**  
  **end**  
   $Q(k) \leftarrow 0$   
  **for**  $r = 1$  **to**  $2n_G$  **do**  
     $Q(k) \leftarrow Q(k) + TM(k, r)$   
  **end**  
  **for**  $r = 2$  **to**  $n_G$  **do**  
     $\alpha_r \leftarrow TM(k, r)$   
  **end**  
  **for**  $r = 1$  **to**  $n_G$  **do**  
     $\nu_r \leftarrow \min[\max(0, TM(k, r) - Q_{lam}), \max(0, Max\_RES_r - TM(k, n_G + r)) \times \gamma_r]$   
     $Stock\_RES_r \leftarrow TM(k, n_G + r)$   
     $\lambda_r \leftarrow \min[Stock\_RES_r, \max(0, Q_{lam} - TM(k, r)) \times \mu_r]$   
  **end**  
**end**

---

the discharge corresponding to the water volumes already stocked in all the reservoirs. In order to choose which strategy to implement, we introduced management parameters  $\mu_r$  and  $\gamma_r$ . The storage phase and release phase cannot occur at the same time for each gate, thus parameters values are set according to the following equation:

$$\begin{cases} \gamma_r = 1, \mu_r = 0 & \text{if water storage} \\ \gamma_r = 0, \mu_r = 1 & \text{if water release} \\ \gamma_r = 0 & \text{if no water storage} \\ \mu_r = 0 & \text{if no water release} \end{cases} \quad (1)$$

The network  $\mathbb{G}$  is updated at each  $kT_c$  ( $k = 1, \dots, n$ ). The network parameters values at  $k-1$  such as the adjacency matrix, the costs and the constraints vector



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**Algorithm 3:** Actualization TM Matrix

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**input :**

$TM$  the  $n \times 2n_G$  temporization matrix  
 $n_G$  the number of gates and FCR in the system  
 $k$  the iteration number  
 $k_r = \lfloor \frac{\tau_r}{T_c} \rfloor, r = 1, \dots, n_G - 1$   
 $\varphi^*(k)$  the optimal flow in  $\mathbb{G}$  at  $kT_c$

**output:**

$TM$  the  $n \times 2n_G$  temporization matrix

**begin**

**for**  $r = 1$  **to**  $n_G - 1$  **do**

$TM(k + k_r, r + 1) \leftarrow$

$TM(k + k_r - 1, r + 1) + \varphi_{G_r P_0}^*(k)$

**end**

**for**  $r = 1$  **to**  $n_G$  **do**

$TM(k + 1, n_G + r) \leftarrow$

$TM(k, n_G + r) + \varphi_{RES_r P_0}^*(k)$

**end**

**end**

---

(arc minimum and maximum values) are input parameters. The strategy parameters,  $\gamma_r$  and  $\mu_r$  are set depending on the discharge values in the matrix  $TM$  compared to the thresholds  $Q_{lam}$  and  $Q_{do}$ . The flow entering the network is updated with the sum of the line  $k$  of the  $TM$  matrix. In order to take into account the transfer time between gates, the maximum flow,  $\alpha_r$ , upstream from each gate  $G_r$  ( $r = 2, \dots, n_G$ ), is set to the  $TM$  matrix previous stored value. The maximum capacity,  $\nu_r$ , of the arc  $G_r RES_r$ , is set to the amount of flow overtaking  $Q_{lam}$  and lower than the remaining  $RES_r$  capacity (only if the  $RES_r$  can be used i. e.  $\gamma_r = 1$ ). The value of  $Stock\_RES_r$  ( $r = 1, \dots, n_G$ ) is set to its previous value stored in the  $TM$  matrix. Finally, the maximum capacity,  $\lambda_r$ , of the arc  $RD_r G_r$  is set to the amount of flow to be released from  $RES_r$ , provided that it remains under  $Q_{lam}$ , and that it is lower than the amount of water in the  $RES_r$  (only if the  $RES_r$  can be released i. e.  $\gamma_r = 1$ ).

The matrix  $TM$  is updated at each  $kT_c$  ( $k = 1, \dots, n$ ) in the Actualization Temporization Matrix algorithm described in algorithm 3. In this matrix, the temporized flow values are stored and actualized such that transfer times can be introduced in the network. In order to take into account the transfer time between gates, the optimal flow from each gate  $G_r$  ( $r = 1, \dots, n_G - 1$ ) to the sink  $P_0$ ,  $\varphi_{(G_r, P_0)}^*(k)$ , is stored in the  $TM$  matrix as the future flow upstream from the next gate  $G_{r+1}$  at  $k + k_r$ . The flow feeding each  $RES_r$  ( $r = 1, \dots, n_G$ ) at  $k$ ,  $\varphi_{RES_r, P_0}^*(k)$ , is added to the flow already stored in order to obtain the new stored value. This value is written in the  $TM$  matrix as the future  $RES_r$  stored value i. e. at  $k + 1$ .

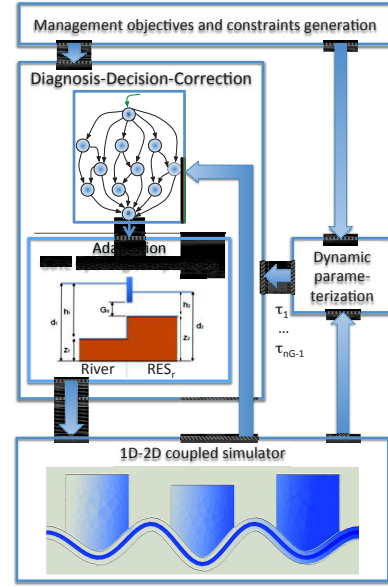


Figure 3 – Simulator and supervisory control scheme

### 3 IMPLEMENTATION AND RESULTS

#### 3.1 Implementation

In order to evaluate the efficiency of the proposed model, a simulation for several cases of flood was done. A test case river was used and the implementation of this river was performed by using a 1D-2D coupled numerical model according to the description given in (Garcia-Navarro et al., 2008; Morales-Hernandez et al., 2012; Morales-Hernández et al., 2013). 1D and 2D models are formulated using a conservative upwind cell-centred finite volume scheme. The discretization was based on cross-sections for the 1D model and with triangular unstructured grid for the 2D model. Coupling techniques were based on mass and momentum conservation. The cross section geometry and topography was derived from the Ebro river, however, the shape of the river was simplified. As summarized in Figure 3, the process and SCADA systems were replaced by a 1D-2D coupled simulator (developed by (Garcia-Navarro et al., 2008)). In the 1D-2D coupled simulator, each gate is modeled considering that the flow discharge that crosses the gate is governed by the difference between the water level in both side of the gate. The 1D-2D coupled simulator entries are the values of the gate opening thus, the Adaptation Block consisted in the computation of the gate opening values from the optimal flow, by means of a static inversion of the free flow open channel equations (Litrico et al., 2008). The Dynamic Parameterization block was used in order to compute the time delays at each  $kT_c$  ( $k = 1, \dots, n$ ). The transfer time,  $\tau_r$ , from the gate  $G_r$  to the gate  $G_{r+1}$ ,  $r = 1, \dots, n_G$  was approximated by the following equation, (see (Karamouz et al., 2003) for exam-

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**Algorithm 4:** Actualization TM and Simulator

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**input :**

$n_G$  the number of gates and FCR in the system  
 $TM$  the  $n \times 2n_G$  temporization matrix  
 $k$  the iteration number  
 $k_r = \lfloor \frac{\tau_r}{T_c} \rfloor$ ,  $r = 1, \dots, n_G - 1$   
 $V_{RES_r}^{mes}(k)$  the measured amount of water stored in the  $RES_r$  at  $kT_c$   
 $Q_{(G_r, RES_r)}^{mes}(k)$  the measured discharge from gate  $G_r$  to  $RES_r$  at  $kT_c$   
 $\mathbb{G}$  the network

**output:**

$TM$  the  $n \times 2n_G$  temporization matrix

**begin**

**for**  $r = 1$  **to**  $n_G - 1$  **do**  
     $TM(k + k_r, r + 1) \leftarrow$   
     $TM(k + k_r - 1, r + 1) + \varphi_{G_r, P_0}^*(k) +$   
     $\max(0, \varphi_{G_r, RES_r}^*(k) - Q_{(G_r, RES_r)}^{mes}(k))$

**end**

**for**  $r = 1$  **to**  $n_G - 1$  **do**  
     $TM(k + 1, n_G + r) \leftarrow$   
     $TM(k, n_G + r) + V_{RES_r}^{mes}(k)$

**end**

**end**

---

ple):

$$\tau_r = \frac{Q_{G_r}}{S \cdot d_{G_r, G_{r+1}}} \quad (2)$$

where  $Q_{G_r}$  is the discharge measured at gate  $G_r$ ,  $S$  is the wetted cross section, and  $d_{G_r, G_{r+1}}$  is the distance traveled from  $G_r$  to  $G_{r+1}$ . In order to evaluate time delays, methods such as the ones developed in (Romera et al., 2013) or in (Bautista and Clemmens, 2015) can also be used. The values measured with the simulator were introduced in the algorithm for actualization of temporized matrix, as described in algorithm 4.

### 3.2 Performance criteria

The flood wave attenuation can be defined as the decrease in the downstream peak flow, due to the attenuation of the flood (Bedient, P. B. et al., 2013). In order to evaluate the performances of the proposed flood attenuation method, three indicators were defined : the attenuation rate (AR), the rate of filling (RF) and the attenuation wave rate (AWR). These measures allow us to evaluate how we prevent downstream flood risk by using the proposed method. All these measures are computed over the time horizon  $H_f$ , i. e. for  $k = 0, \dots, n$ ; and we denote  $Q_{out}$  the downstream flow. The AR permits to measure the difference between the attenuation threshold objective and the obtained attenuation threshold. It is defined as the ratio between the mean effective at-

tenuation flow,  $Q_{mea}$ , and the predefined attenuation flow  $Q_{lam}$ , as given in equation (3) and equation (4).

$$AR = \frac{Q_{mea}}{Q_{lam}} \quad (3)$$

$$\begin{cases} \text{if } \exists k | Q_{out}(k) > Q_{lam} & Q_{mea} = \underset{Q_{out}(k) > Q_{lam}}{\text{mean}} Q_{out}(k) \\ \text{else} & Q_{mea} = \max_{k=1 \dots n} Q_{out}(k) \end{cases} \quad (4)$$

Another estimator of the attenuation capacity is the AWR which compares the case where the gates are always closed to the case in which a strategy is involved and is expressed by equation (5).

$$AWR = \frac{\sum_{Q_{cg}(k) > Q_{lam}} Q_{cg}(k) - \sum_{Q_{out}(k) > Q_{lam}} Q_{out}(k)}{\sum_{Q_{cg}(k) > Q_{lam}} Q_{cg}(k)} \quad (5)$$

The downstream flow when the gates are closed is denoted  $Q_{cg}$ . Finally, RF indicates the water volume storage efficiency. It is computed as the ratio between the water volume stored in all the reservoirs,  $V_s$ , and the estimated water volume to be stored,  $V_{lam}$ , as indicated in equation (6), and assuming that  $V_{RES_r}$  is the maximum volume stored in the reservoir  $RES_r$  ( $r = 1, \dots, n_G$ ), during the time horizon  $H_f$ .  $V_{lam}$  is approximated by the trapezoidal numerical integration of the input flow,  $Q_{input}$ , above  $Q_{lam}$ .

$$RF = \frac{V_s}{V_{lam}}, \quad V_s = \sum_{r=1}^{n_G} V_{RES_r} \quad (6)$$

### 3.3 Results

Simulation were done within the horizon  $H_f = 86400s$  corresponding to 24 hours,  $T_c = 100s$  thus  $n = 864$ . The simulated river was equipped with  $n_G = 3$  flood control reservoirs, each one controlled by a gravitational gate.

The first case studied is a flood episode with one peak flow of  $790.31m^3/s$  occurring at  $k = 330$  i. e. around 9 hours after the beginning of the simulation. The values of attenuation and draw-off flows were set to  $Q_{lam} = 675m^3/s$  and  $Q_{do} = 600m^3/s \approx 90\%Q_{lam}$ . For this one peak flood, the measured time delays varied between  $11T_c$  and  $16T_c$  as illustrated in Figure 4. Thus in order to compare the results obtained when the strategy involved constant time delay or varying time delay, we realized simulation for constant time delays underestimated or overvalued:  $\tau_1 = \tau_2 = 10T_c$ ,  $\tau_1 = \tau_2 = 11T_c$ ,  $\tau_1 = \tau_2 = 14T_c$ ,  $\tau_1 = \tau_2 = 16T_c$ ,  $\tau_1 = \tau_2 = 18T_c$ . In Figure 5, the  $Q_{input}$  value is given in red and results obtained for the four following cases



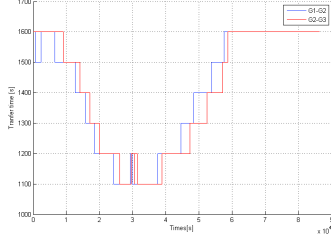


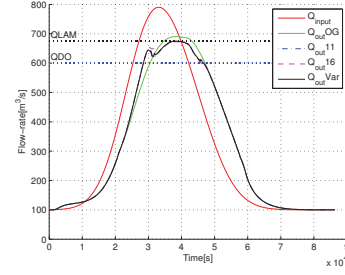
Figure 4 –  $\tau_1$  and  $\tau_2$  evolution for a 1-peak simulation with  $Q_{lam} = 675m^3/s$  and  $Q_{do} = 600m^3/s$ .

are compared. Case one, the gates always open (unregulated reservoirs) is given in green. Case two, the proposed strategy applied with constant time delays:  $\tau_1 = \tau_2 = 11T_c$  is given in blue. Case three, the proposed strategy applied with constant time delays:  $\tau_1 = \tau_2 = 16T_c$  is given in magenta. Case four, the proposed strategy applied with varying time delays, expressed as function of flow and computed thanks to the Dynamic Parameterization block is given in black. When the gate are always open, the peak flood is reduced however, the discharge exceeds the  $Q_{lam}$  value. When time delays are computed, the  $Q_{out}$  curve is between the  $Q_{out}$  curves obtained for the time delays set to their variation interval bounds. In all these cases, the  $Q_{out}$  maximum value is given, and denoted  $Q_{max}$  in the second column of the Table 1. Without the use of flood control reservoirs the peak flow reaches  $777.08m^3/s$ , when the gates are always open, the peak flow reaches  $690.39m^3/s$ . When the proposed strategy is applied, the peak flow decreases and it is lower than the  $Q_{lam}$  value when the time delays are computed. When time delays are set to constant values, performance decreases, and we can conclude that it is preferred to overestimate the time delays.

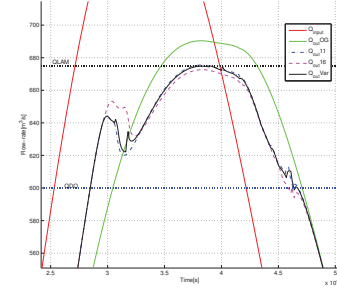
Case	$Q_{max}$	AR%	AWR%	RF%
Open Gates	690.39	101.65	36.83	123.42
$\tau_r = 10T_c$	679.26	100.39	65.09	115.80
$\tau_r = 11T_c$	675.46	100.05	91.12	112.99
$\tau_r = 14T_c$	673.83	99.83	100	112.21
$\tau_r = 16T_c$	672.64	99.65	100	111.65
$\tau_r = 18T_c$	675.47	100.04	90.45	113.94
Varying $\tau$	674.92	99.99	100	112.59

Table 1 – AR, AWR and RF values for the 1 peak scenario.

The values of the performance criteria obtained in the studied cases are given in Table 1. Whatever the method used for the time delays computation, the ability to absorb the flood is increased when using the transportation network. Indeed  $AWR = 65.09\%$  when the time delays are underestimated, and  $AWR = 90.45\%$  when the time delays are overvalued. When the time delays are set to the minimum value of their variation interval  $AWR = 91.12\%$ . When the time delays are computed or set to values high



(a) Original scale



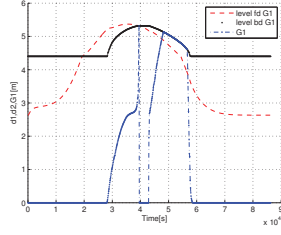
(b) Zoom

Figure 5 –  $Q_{input}$  and  $Q_{out}$  for a 1-peak simulation with  $Q_{lam} = 675m^3/s$  and  $Q_{do} = 600m^3/s$ .

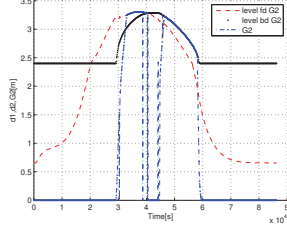
enough,  $AWR = 100\%$ , the peak flow is under the  $Q_{lam}$  value. Finally,  $AWR = 36.83\%$  when the gates are not regulated. AR value is better if it is as close as possible to 100% which is the case for computed time delays. Finally, in all cases the water stored in the reservoir is upper than the estimated needed volume.

The gates' opening height computed by the algorithm with varying time delays is given in blue in Figure 6(a) for the gate  $G_1$ , in Figure 6(b) for the gate  $G_2$  and in Figure 6(c) for the gate  $G_3$ . The water level inside the reservoir is represented in black and the water level in the river in front of the gates in red. The water levels are measured with regard to the river bed. In each Figure, the gate is first opened in order to store water, thereafter, during the phase when the discharge is between  $Q_{lam}$  and  $Q_{do}$  the gate is closed and finally, the gate is opened in order to empty the reservoir.

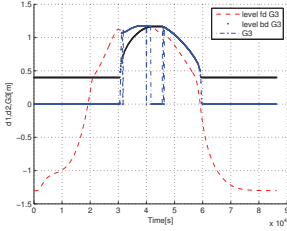
In the fourth illustrated cases, the water level inside the reservoirs is superimposed in Figure 7(a) for the gate  $G_1$ , in Figure 7(b) for the gate  $G_2$  and in Figure 7(c) for the gate  $G_3$ . The always open gates case is given in green. The proposed strategy applied with constant time delays:  $\tau_1 = \tau_2 = 11T_c$  is given in blue, with  $\tau_1 = \tau_2 = 16T_c$  in magenta and with varying time delays in black. For each one of the three gates, the green curve is always above the other ones, which indicates that the needed reservoirs' capacity is lower when using the regulation scheme. Moreover, the reservoirs are filled later in that case and the water remains less time in the reservoirs, thus the agri-



(a)  $G_1$  Gate opening and water levels inside and outside  $FCR1$



(b)  $G_2$  Gate opening and water levels inside and outside  $FCR2$

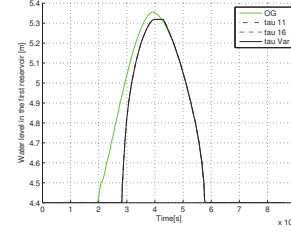


(c)  $G_3$  Gate opening and water levels inside and outside  $FCR3$

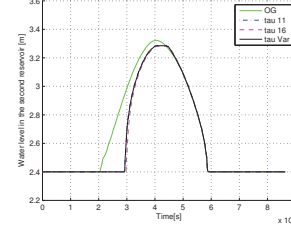
Figure 6 – Gate opening and water levels for a 1-peak simulation with  $Q_{lam} = 675m^3/s$  and  $Q_{do} = 600m^3/s$ . The water levels inside (outside) the reservoirs are denoted bd (fd) respectively.

cultural zone are better preserved. The water level curve in the case of computed time delays is between the curves obtained for the time delays set to their variation interval bounds.

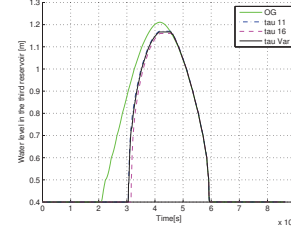
The second case studied is a flood episode with two peak flows, the first one is of  $838.79m^3/s$  occurring at  $k = 324$  i. e. around 9 hours after the beginning of the simulation, the second is of  $753.79m^3/s$  and occurs at  $k = 570$  i. e. around 16 hours after the beginning of the simulation. The values of attenuation and draw-off flows were set to  $Q_{lam} = 710m^3/s$  and  $Q_{do} = 680m^3/s \approx 95\%Q_{lam}$ . This case was proposed in order to evaluate the ability of the method to attenuate a second flood episode. Moreover,  $Q_{do}$  is set high enough to allow for a water draw-off from the reservoir after the first peak and before the second one and so that the ability to absorb the second flood exists. Because results obtained in the one peak flood episode shown that results were better in the computed time delay case, we compared for the two peaks flood episode only this case and the case when gates are always open. For this two peak flood, the



(a) First reservoir



(b) Second reservoir



(c) Thrid reservoir

Figure 7 – Comparison of water levels inside the reservoirs for a 1-peak simulation with  $Q_{lam} = 675m^3/s$  and  $Q_{do} = 600m^3/s$ .

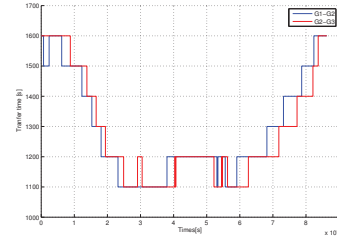
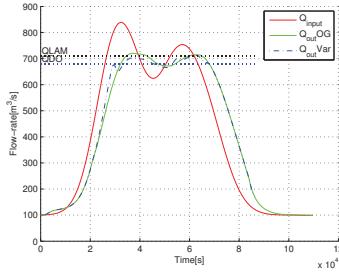


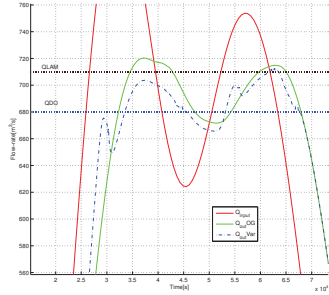
Figure 8 –  $\tau_1$  and  $\tau_2$  evolution for a 2-peaks simulation with  $Q_{lam} = 710m^3/s$  and  $Q_{do} = 680m^3/s$ .

measured time delays varied between  $11T_c$  and  $16T_c$  as illustrated in Figure 8.

In Figure 9, the  $Q_{input}$  value is given in red, the always open gates case in green. The proposed strategy applied with varying time delays is given in black. When the gate are always open, the peak flood is reduced however, the discharge exceeds the  $Q_{lam}$  value. When time delays are computed, the  $Q_{out}$  curve is between the  $Q_{out}$  curves obtained for the time delays set to their variation interval bounds. Without the use of flood control reservoirs the peak flow reaches  $823.01m^3/s$  for the first wave and  $746.19m^3/s$  for the second one. When applying the strategy, the peak flow reaches  $703.65m^3/s$  for the first wave and



(a) Original scale



(b) Zoom

Figure 9 –  $Q_{input}$  and  $Q_{out}$  for a 2-peak simulation with  $Q_{lam} = 710m^3/s$  and  $Q_{do} = 680m^3/s$ .

$712.56m^3/s$  for the second one. Applying the strategy allows the discharge to remain under the  $Q_{lam}$  value for the first wave and very close to it for the second wave. The values of the performance criteria

Case	AR%	AWR%		RF%
		1st pic	2nd pic	
Open Gates	100.76	64.34	77.34	103.17
Varying $\tau$	100.16	100	91.90	99.07

Table 2 – AR, AWR and RF values for the 2 peaks scenario in the two different cases.

computed for each case are given in Table 2. As in the first test, the ability to absorb the both flood waves is increased when using the proposed method. Indeed, for the first wave,  $AWR = 100\%$  when gates are regulated whereas  $AWR = 64.34\%$  when gates are not regulated. For the second wave  $AWR = 91.90\%$  when the strategy is used whereas  $AWR = 77.34\%$  when the gates remain open. Before the arrival of the second flood, we take advantage of the decrease of the water level in the river to release a certain amount of water from FCRs in the river. This enables us to better accommodate the second wave of flooding.

## 4 CONCLUSION

In this paper, we propose a supervisory control scheme for the management of a river section in a flood situation. The Diagnosis-Detection-Correction block is based on a transportation network model-

ing including time delay. It permits to account for the variation of time delays without any modification in the network structure. The results of the connection between the method and the 1D-2D-coupled river simulator were displayed, highlighting the benefits of the strategy. The proposed simulated case permitted to attest the feasibility of including varying time delays in the network. Results are expected to be most obvious when considering a more extended network with longer delays. The strategy consists of two phases: water storage and water release. The storage phase keeps the flow below the attenuation discharge threshold imposed: the flood is attenuated, and the draw-off phase, enables us to preserve the floodplain. The strategy can be used in order to estimate the capability of river systems equipped with flood control reservoirs to control floods. One of the most important problem to be studied, beyond quantitative flood management, is the quality of water in the river and in the reservoirs. Future research will focus on the integration of pollution problems into the strategy.

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