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Contribution to a flood situation management: a supervisory control scheme to reduce disaster impact.

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Abstract: Inundations due to river overflows are becoming more frequent; management of flood is thus an important task belonging to the set of preventive measures allowing the protection of people and goods downstream. The flood situation management method proposed in this paper was designed to reduce the flood impact at its early arising stage. The river is supposed to be equipped with reservoirs in which water excess are stored and then released only when the flood episode ends. The supervisory control scheme allows calculating the water volumes through the use of a network flow. The management objectives such as the maximum discharge level allowed in the river, the order of priority for the reservoir storage or release; the measured levels and discharge in the river and in the reservoirs; and the assessed parameters such as time delays, are combined to configure the network flow. Then, the optimal flow in the network is computed and supplies the reservoirs' gate opening setpoints. Finally, the method was applied to a simulated case for which the time delay during the flood varied and remained efficient for flood attenuation compared to the case when the gates were always open, thanks to the network configuration.

Keywords: Flood control; Network flows; Time delay; Water systems management.

INTRODUCTION

Flooding due to excessive rains can cause important human and material damages around the world. The frequency of these events and their scale is increasing, as well as the importance of the human and material damages caused (Wagenknecht and Rueppel, 2013). In this context, the term of crisis is generally used for floods leading to an actual inundation in a limited geographical zone and for which numerous assistance interventions are needed in order to help the inhabitants either to protect residential areas or to proceed to evacuations. It is essential to consider that the crisis began at the early occurrence of the flood phenomenon. It permits to study and implement the means leading to a fast recovery and to inform the inhabitants, to prepare and dispense the protection, even in high-risk areas (Merz et al., 2010; Plate, 2002).

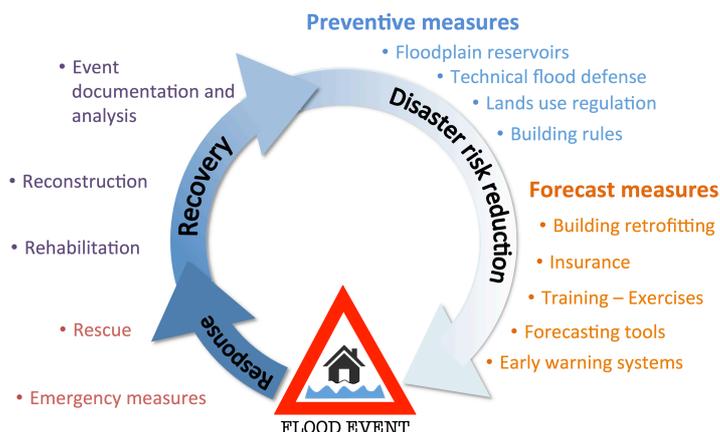


Figure 1: Management crisis cycle, adapted from (Thieken et al., 2007).

Crisis management is the set of organizational methods, techniques and means that enable an organization to prepare for and to effectively manage the occurrence of a crisis; and, in a prospective vision, to capitalize the lessons of the event to improve procedures and structures. In order to set up an effective management of crisis, three principal phases

must be considered (see Figure 1): before the flood where it is necessary to plan, to prevent, to prepare, to protect and to anticipate crisis situations, during the flood where the effective management of crisis is performed and finally after the flood the where a relevant feedback of the learning experience should be performed in order to improve the first phase and to implement the means for the resilience (Hooijer et al., 2004; Thielen et al., 2007).

Forecast and prevention programs are provided by states in order to face the flood events undergone in the world. Thus, the European Commission supports and finances projects in order to develop forecasting and alerting systems to warn communities of impending floods. The various projects described in the literature focuses on different aspects of the crisis management and numerous software were developed (Alfieri et al., 2013; Liechi et al., 2013; Pengel et al., 2013).

The present paper focuses on the phase preceding the inundation event. The potential flood is detected and the peak flow is reduced in order to limit the downstream flood impact, and if possible to avoid the inundation. For this purpose, flood control areas existing along the river are used as reservoirs. In order to reduce the water velocity in the river, the reservoirs are filled with water thus the flood wave is attenuated.

Various research works have been proposed to reduce flood peaks and volumes, involving linear programming (Needham et al., 2000), or 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), and multi-objective optimization (Chuntian and Chau, 2002).

Herein, a supervisory control scheme is proposed to handle the water volumes. This scheme, including the variation of time delay with discharge, is described in Section 2. Different flood situations are compared for a simulated river system in section 3, showing the effectiveness of the scheme.

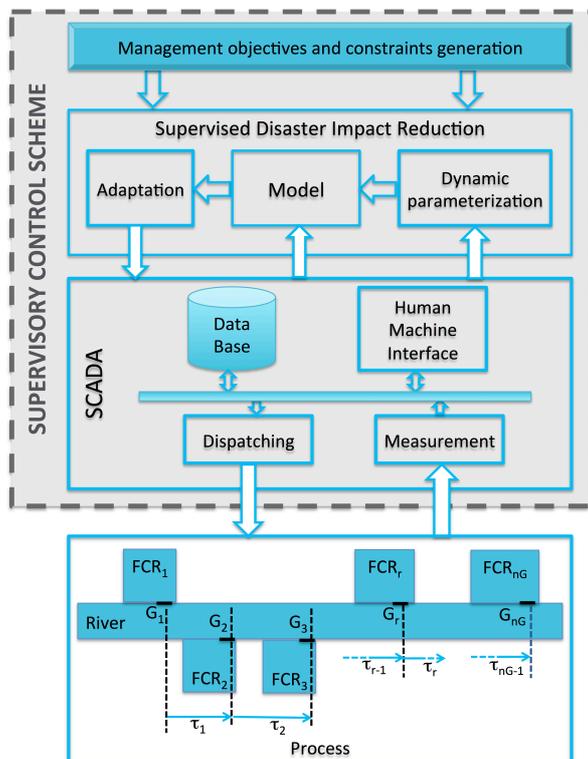


Figure 2: The river process and the Supervisory Control Scheme.

SUPERVISORY CONTROL SCHEME

Supervisory control methods permit to combine optimization, regulation, and simulation techniques. In order to help the decision-making process, the supervision, detection and diagnosis tools are integrated and diverse schemes and architectures are proposed in the literature (Isermann, 1997). The supervision step consists in the detection, the estimate, the prognosis of the system state, the diagnosis of this state, the computation of the setpoints and, if necessary, the control law reconfiguration.

General Scheme

The supervisory control scheme proposed in this paper is depicted in Figure 2. It is composed of three interconnected blocks: the SCADA (Supervisory Control and Data Acquisition) system, the management objectives and constraints generation (MOCG), and the supervised disaster impact reduction blocks. This scheme was designed in order to reduce the impact of a flood downstream a river. For this purpose the river is equipped with n_G flood control reservoirs located along the river, denoted FCR_r .

The reservoirs are used to store the excess of water such that the output river discharge, Q_{out} , remains under a predefined flow value, Q_{lam} : the attenuation threshold. Each reservoir is provided with a controlled gate G_r , $r = 1, \dots, n_G$. The opening value of each gate is computed by the proposed scheme.

When the reservoirs are not empty, the stored water can be released if the discharge level in the river is lower than the attenuation threshold Q_{lam} . In order to detect when the water can be released from the reservoir, a threshold, Q_{do} , is defined.

The storage and release phases are exemplified in the Figure 3, where Q_{input} is the input discharge in the river.

Moreover, the threshold Q_{do} can be defined in order to include the protection of farming usually present in the reservoirs in the release objective, and to be able to control flood episodes with near occurrences.

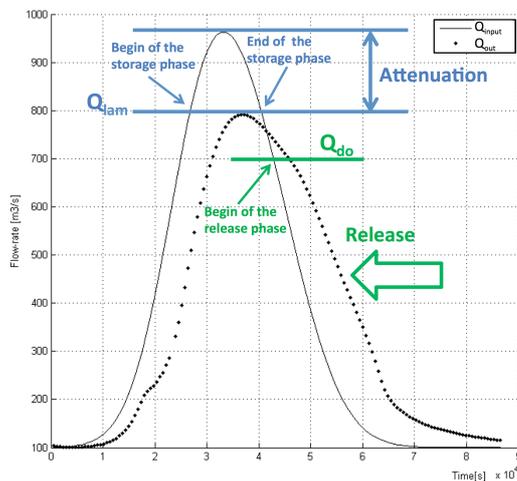


Figure 3: Storage and release phases.

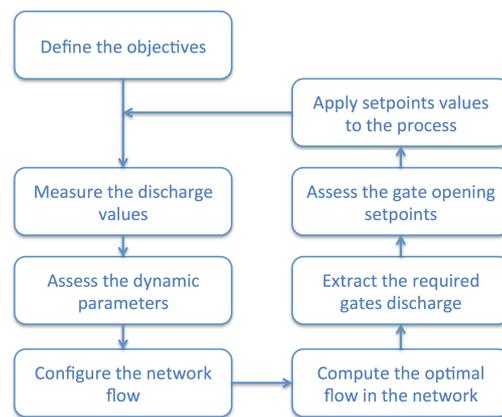


Figure 4: Supervisory process.

Based on the network flow model of the system composed by the river and its reservoirs, the functioning of the scheme is sequenced with eight steps. After defining the management objectives and constraints, the process follows an infinite closed loop including the activities given in Figure 4.

Supervisory Control and Data Acquisition system (SCADA) block

The SCADA block is connected to the river process. It permits to collect data from sensors and to send control values to actuators. Measured and setpoint values can be sent

to or given by an operator through a Human Machine Interface. Such SCADA system can be found in various kinds of systems such as irrigation canals (Figueiredo et al., 2013; Pfitscher et al., 2012), inland navigation networks (Duviella et al., 2013), or energy management (Mora et al., 2012). In our scheme, it transmits the sensors' values to the Supervised Disaster Impact Reduction (SDIR) block, and receives the gate opening setpoint values in order to send them to the process. The measurements considered herein are levels and discharges.

Management Objectives and Constraints Generation (MOCG) block

The MOCG block supplies the SDIR block with management constraints and rules such as threshold values, Q_{lam} , Q_{do} , and the priority parameters allowing, for example, to favor one of the reservoirs, or to define a reservoir assignment order. Some of these values are defined depending on the government organization directives. Moreover, in the network modeling the system, the arc can be weighted with costs in order to evaluate the strategies and take decisions improving the management. The cost values defined in the MOCG block depends on cost-benefit analysis including an estimation of the costs of the various water usages and risks (Karamouz et al., 2003; Loucks et al., 2005). The costs definition in the objective function relies on:

- The need to avoid, reduce or delay as long as possible the inundation downstream the river,
- The reservoir nature (agricultural zone, fallow...),
- The reservoir capacity,
- The reservoir usability,
- The protection of the farming existing in the reservoir,
- The maximal duration of the water retention,
- And on the necessity of preserving the water quality in the reservoirs.

Supervised Disaster Impact Reduction (SDIR) block

The management constraints are taken into account, according to the measured values, thanks to the SDIR block, which is detailed in the upper part of Figure 5. In order to manage the flood episode, control method must be associated to a scheduling method (Baldea and Harjunoski, 2014). Indeed, while tracking the overflow of the discharge in case of floods, we need to establish a diagnosis of the process state, to optimize the storage and the release of the water volumes in the flooding reservoirs and to control the opening of their gates. We choose to implement a management method based on the network flow describes in detail in (Nouasse et al., 2013a, b, c). This SDIR block includes:

- The Dynamic Parameterization (DP) block allowing to supply the SDIR with all the necessary dynamic parameters such as the costs and the time delays obtained by the use of estimation techniques for example.
- The Model block producing the setpoint values for each reservoir. The model involved in this block is based on a network flow modeling the network, the reservoirs and their management. For each kT_c , $k = 0, \dots, n$, in the horizon H_f , with $H_f = nT_c$, $n \in \mathbb{N}^+$. This model is firstly configured according to the measures and to the dynamic parameters values: maximum and minimum arc capacities are set depending on the delayed flow and on the reservoirs dimensions; the release or storage functioning mode are defined on the basis of the diagnose state of the flow. Then, in the data exploitation phase, the optimal flow is computed by applying a Min cost Max flow problem resolution for this network, producing the setpoint discharge values. The proposed implementation of the network flow includes time delays. If the time delays vary, the network structure is not impacted

thus it is not necessary to add node or arc, only network parameters are modified (Nouasse et al., 2013b).

- The Adaptation block converting the setpoint values supplied by the model in values adapted with the process actuators controller and thus understood by the SCADA system. In fact, the water crosses the gravitational reservoirs gates thanks to the difference between the levels inside the reservoir and in the river. Thus discharge setpoint values need to be converted in level values. The Bernoulli equation is applied to the flow between the river and the reservoir to derive the non-linear static equation representing the dynamic behavior of this structure.

Using the measured flow values, the reservoirs' configuration, the time delays, and the objectives, the SDIR block computes the gate opening setpoint values allowing the output flow to remain under the attenuation threshold.

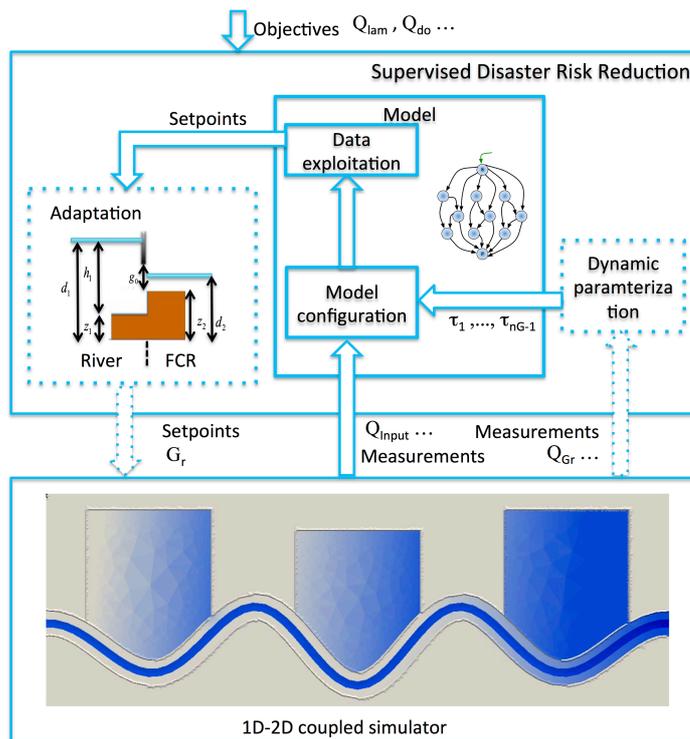


Figure 5: Simulator and supervisory control scheme.

IMPLEMENTATION AND RESULTS

In order to evaluate the efficiency of the proposed model, a simulation for several cases of flood was done. More often, the dimensioning of the reservoirs is done such that they can attenuate a potential flood; thereby the gates are not regulated. Thus, in each simulated flood case, the method was compared with the case when the gates are always open, which is often the case.

Implementation

The process and SCADA systems were replaced by the implementation of a test case river performed by using a 1D-2D coupled numerical model, according to the description given in (Morales-Hernandez et al., 2013), as illustrated in Figure 5.

In this simulator, each gravitational gate is modeled considering that the flow discharge that crosses the gate is governed by the difference between the water levels 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. The

Dynamic Parameterization block was used in order to compute the time delays at each $kT_c (k = 1, \dots, n)$. The time delay, τ_r , from the gate G_r to the following gate G_{r+1} ($r = 1, \dots, n_G$) depends on the flow discharge. It was approximated by the following equation, (see (Karamouz et al., 2003) for example):

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

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) can also be used.

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, two indicators were defined: the attenuation rate (AR), and the attenuation wave rate (AWR). These indicators allow us to evaluate how we prevent downstream flood by using the proposed method. All these indicators 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 attenuation flow, Q_{mea} , and the predefined attenuation flow Q_{lam} , as given in equation (2) and equation (3).

$$AR = \frac{Q_{mea}}{Q_{lam}} \quad (2)$$

$$\begin{cases} \text{if } \exists k | Q_{out}(k) > Q_{lam} & Q_{mea} = \text{mean}_{Q_{out}(k) > Q_{lam}} Q_{out}(k) \\ \text{else} & Q_{mea} = \max_{k=1 \dots n} Q_{out}(k) \end{cases} \quad (3)$$

Q_{mea} is the mean of all the Q_{out} which value is greater than Q_{lam} . In case of flood: $Q_{input} > Q_{lam}$, if $AR > 1$, the attenuation is not complete and if $AR < 1$, too much water is stored.

Another estimator of the attenuation capacity is the AWR , which compares the case where the gates are always closed (indexed cg) to the case in which a strategy is involved. It is illustrated in Figure 6 and is expressed by equation (4).

$$AWR = \frac{\sum_{Q_{cg}(k) > Q_{lam}} Q_{cg}(k) - \sum_{Q_{out}(k) > Q_{lam}} Q_{out}(k)}{\sum_{Q_{cg}(k) > Q_{lam}} Q_{cg}(k)} \quad (4)$$

The downstream flow when the gates are closed is denoted Q_{cg} . The AWR value is a relative estimation of the not attenuated volumes.

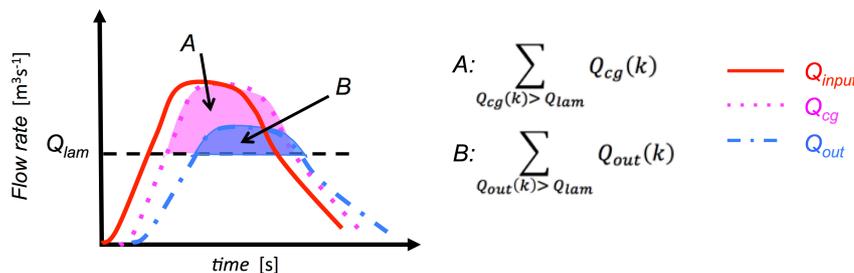


Figure 6: AWR computation.

Results

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

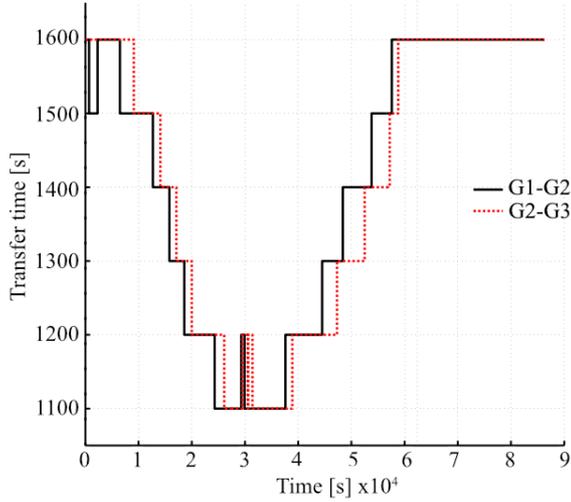
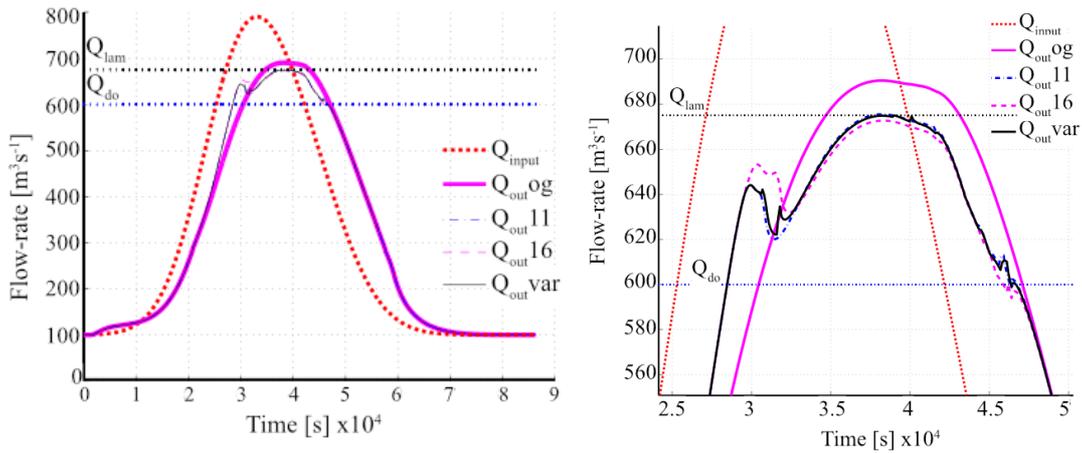


Figure 7: τ_1 and τ_2 evolution for a 1-peak simulation with $Q_{lam} = 675m^3s^{-1}$ and $Q_{do} = 600m^3s^{-1}$.



(a) Original scale.

(b) Zoom.

Figure 8: Q_{input} and Q_{out} for a 1-peak simulation with $Q_{lam} = 675m^3s^{-1}$ and $Q_{do} = 600m^3s^{-1}$.

The first case studied is a flood episode, Q_{input} , with one peak flow of $790m^3s^{-1}$ occurring at $k = 330$ i. e. around 9h after the beginning of the simulation. The values of attenuation and draw-off flows were set to $Q_{lam} = 675m^3s^{-1}$ and $Q_{do} = 600m^3s^{-1} \approx 90\%Q_{lam}$. For this one peak flood, the measured time delays varied between $11T_c$ and $16T_c$ as illustrated in Figure 7. 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 8, the Q_{input} value is given in red (dotted) and results obtained for the four following cases are compared. The case one, when the gates are always open (unregulated reservoirs), is given in magenta. The case two, when the proposed strategy is applied with constant time delays: $\tau_1 = \tau_2 = 11T_c$, is given in blue (dotted – dashed). The case three, when the proposed strategy is applied with constant time delays: $\tau_1 = \tau_2 = 16T_c$, is given in magenta (dashed). The

case four, when the proposed strategy is applied with varying time delays expressed as function of flow and computed thanks to the Dynamic Parameterization block, is given in black. When the gates 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 $777\text{m}^3\text{s}^{-1}$, when the gates are always open, the peak flow reaches $690\text{m}^3\text{s}^{-1}$. 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	$Q_{max}(\text{m}^3\text{s}^{-1})$	AR(%)	AWR(%)
Open Gates	690	102	37
$\tau_r = 10T_c$	679	100	65
$\tau_r = 11T_c$	675	100	91
$\tau_r = 14T_c$	674	100	100
$\tau_r = 16T_c$	673	100	100
$\tau_r = 18T_c$	675	100	90
Varying τ_r	675	100	100

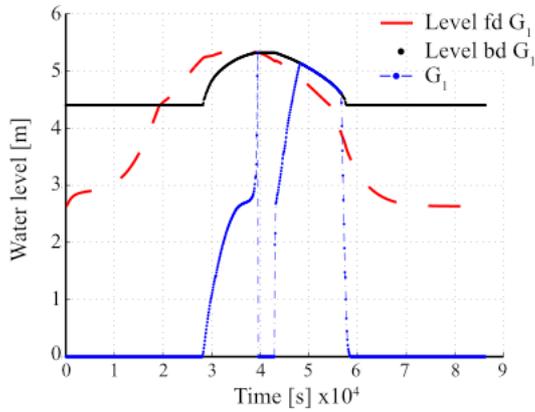
Table 1: AR, and AWR values for the 1 peak scenario.

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 is, the ability to absorb the flood is increased when using the network flow. Indeed $AWR = 65\%$ when the time delays are underestimated, and $AWR = 90\%$ when the time delays are overvalued. When the time delays are set to the minimum value of their variation interval $AWR = 91\%$. When the time delays are computed or set to high enough values, $AWR = 100\%$, the peak flow is under the Q_{lam} value. Finally, $AWR = 37\%$ when the gates are not regulated. The 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 volume stored in the reservoir is upper than the estimated needed volume.

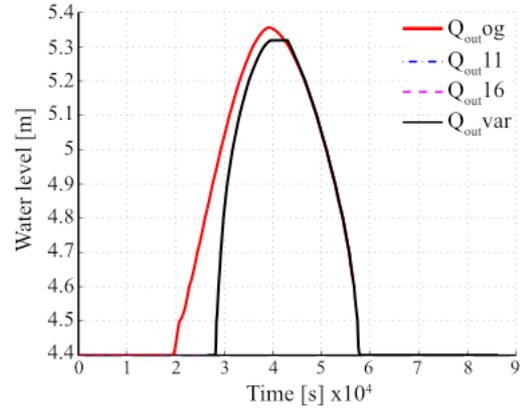
The gates' opening height computed by the algorithm with varying time delays is given in blue in Figure 9(a) for the gate G_1 , in Figure 9(c) for the gate G_2 and in Figure 9(e) for the gate G_3 . The water level inside the reservoir is represented in black and the water level in the river in front of the gates in red. The water levels are measured with regard to the riverbed. 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 9(b) for the gate G_1 , in Figure 9(d) for the gate G_2 and in Figure 9(e) for the gate G_3 . The always-open gates case is given in red. The proposed strategy applied with constant time delays: $\tau_1 = \tau_2 = 11T_c$ is given in blue (dotted – dashed), with $\tau_1 = \tau_2 = 16T_c$ in magenta (dashed) 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 agricultural zone are better preserved. The water level curve in the case of computed time

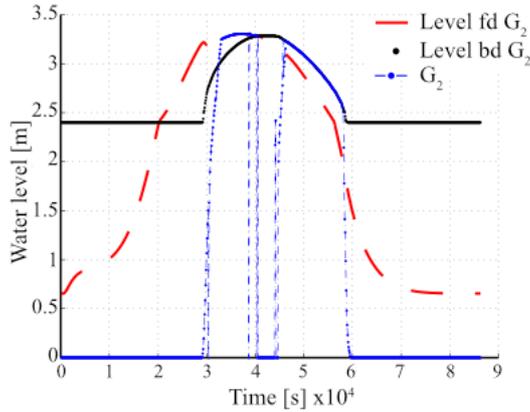
delays is between the curves obtained for the time delays set to their variation interval bounds.



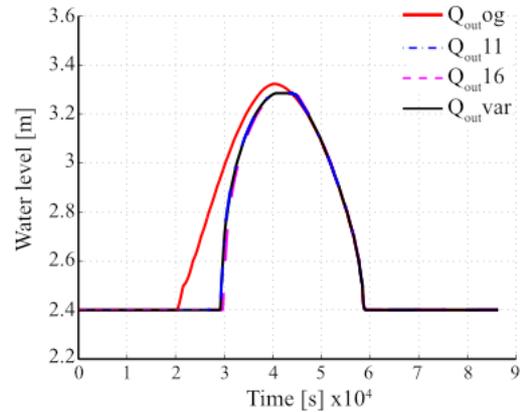
(a) G_1 Gate opening and water levels inside and outside FCR_1 .



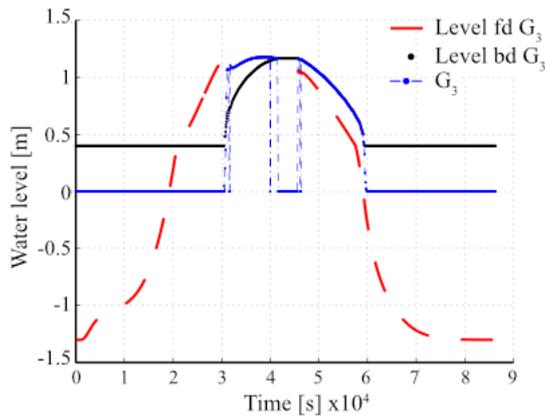
(b) Comparison of water levels inside FCR_1 .



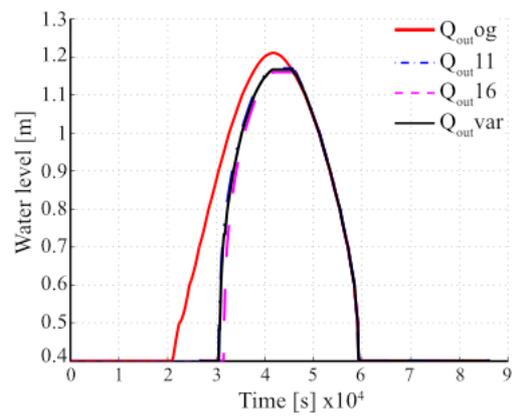
(c) G_2 Gate opening and water levels inside and outside FCR_2 .



(d) Comparison of water levels inside FCR_2 .



(e) G_3 Gate opening and water levels inside and outside FCR_3 .



(f) Comparison of water levels inside FCR_3 .

Figure 9: Gate opening and water levels for a 1-peak simulation with $Q_{lam} = 675m^3s^{-1}$ and $Q_{do} = 600m^3s^{-1}$. The water levels inside (outside) the reservoirs are denoted bd (fd) respectively.

The second case studied is a flood episode with two peak flows, the first one is of $839m^3s^{-1}$ occurring at $k = 324$ i. e. around 9h after the beginning of the simulation, the second is $754m^3s^{-1}$ and occurs at $k = 570$ i. e. around 16h after the beginning of the simulation. The values of attenuation and draw-off flows were set to $Q_{lam} =$

$710\text{m}^3\text{s}^{-1}$ and $Q_{do} = 680\text{m}^3\text{s}^{-1} \approx 95\%Q_{lam}$. That case was proposed in order to evaluate the ability of the method to attenuate a second flood episode. Moreover, Q_{do} was 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 peaks flood, the measured time delays varied between $11T_c$ and $16T_c$, as illustrated in Figure 10.

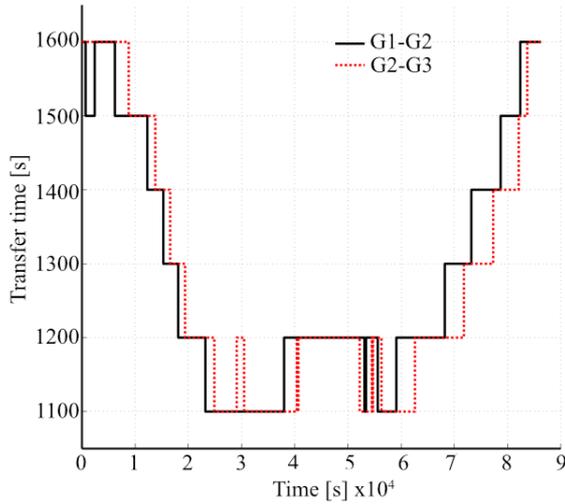
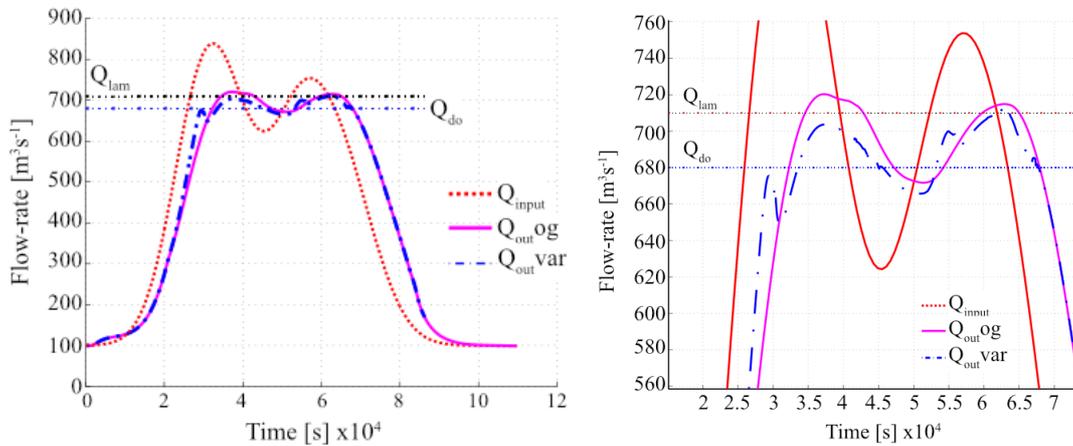


Figure 10: τ_1 and τ_2 evolution for a 2-peaks simulation with $Q_{lam} = 710\text{m}^3\text{s}^{-1}$ and $Q_{do} = 680\text{m}^3\text{s}^{-1}$.



(a) Original scale.

(b) Zoom.

Figure 11: Q_{input} and Q_{out} for a 2-peaks simulation with $Q_{lam} = 710\text{m}^3\text{s}^{-1}$ and $Q_{do} = 680\text{m}^3\text{s}^{-1}$.

In Figure 11, the Q_{input} value is given in red (dotted), the always-open gates case in magenta. The proposed strategy applied with varying time delays is given in blue (dotted – dashed). When the gates 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 $823\text{m}^3\text{s}^{-1}$ for the first wave and $746\text{m}^3\text{s}^{-1}$ for the second one. When applying the strategy, the peak flow

reaches $704m^3s^{-1}$ for the first wave and $713m^3s^{-1}$ 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 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 $AWR = 64\%$ when gates are not regulated. For the second wave $AWR = 92\%$ when the strategy is used whereas $AWR = 77\%$ 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 the reservoirs in the river. This enables us to better accommodate the second wave of flooding.

Case	AR(%)	AWR(%)	
		1st pic	2nd pic
Open Gates	101	64	77
Varying τ_r	100	100	92

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

CONCLUSION

In this paper, a crisis management method included in a supervisory control scheme has been proposed.

- It consists in three blocks connected to a river process using reservoirs allowing the management of the flood situation.
- It allows calculating the water volumes to be stored or released through the use of a network flow.
- The variation of the time delays does not impact the network structure.
- Simulation results, for the case of a river with three reservoirs, have showed the effectiveness of the proposed method.
- The proposed simulated case has permitted to attest the feasibility of including varying time delays in the network.
- Future research will study the case of an extended catchment, thus the proposed scheme will consider a river network with longer delays, bifurcations and confluences.

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APPENDIX

Symbol	
Q_{out}	The output river discharge
Q_{lam}	The attenuation threshold
n_G	The number of flood control reservoirs and gates.
FCR_r	The r^{th} flood control reservoir
G_r	The gate controlling the r^{th} flood control reservoir
τ_r	The time delay from the gate G_r to the following gate G_{r+1}
Q_{do}	The release threshold
T_c	The control period
H_f	The time horizon
n	The number of control period in the time horizon
Q_{G_r}	The discharge measured at the r^{th} gate
S	The wetted cross section
$d_{G_r, G_{r+1}}$	The distance between the r^{th} gate and the $(r + 1)^{\text{th}}$ gate
$AR(\%)$	The Attenuation Rate
$AWR(\%)$	The Attenuation Wave Rate
Q_{mea}	The mean effective attenuation flow
$Q_{out}(k)$	The output river discharge measured at the date kT_c
$Q_{cg}(k)$	The output river discharge measured at the date kT_c when the gates are closed
Q_{max}	The maximum value of the output river discharge during the time horizon

Table 3: List of symbols.