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Key words

Flood; Lateral flow; Karst, Diffusive wave model; Peakflow attenuation or amplification; Surface water / groundwater interactions

Abstract

The aim of this paper is to quantify peakflow attenuation and/or amplification in a river, investigating lateral flow from the intermediate catchment during floods. This is a challenge for the study of the hydrological response of permeable/intermittent streams, and our contribution refers to a modelling framework based on the inverse problem for the diffusive wave model applied in a karst catchment. Knowing the upstream and downstream hydrographs on a reach between two stations, we can model the lateral one, given information on the hydrological processes involved in the intermediate catchment. The model is applied to 33 flood events in the karst reach of the Iton River in French Normandy where peakflow attenuation is observed. The monitored zone consists of a succession of losing and gaining reaches controlled by strong surface-water / groundwater (SW/GW) interactions. Our results show that, despite a high baseflow increase in the reach, peakflow is attenuated. Model application shows that the intensity of lateral outflow for the flood component is linked to upstream discharge. A combination of river loss and overbank flow for highest floods is proposed for explaining the relationships. Our approach differentiates the role of outflow (river loss and overbank flow) and that of wave diffusion on peakflow attenuation. Based on several sets of model parameterization, diffusion is the main attenuation process for most cases, despite high river losses of up to several m^3/s (half of peakflow for some parameterization strategies). Finally, this framework gives new insight into the SW/GW interactions during floods in karst basins, and more globally in basins characterized by disconnected river-aquifer systems.

1 Introduction

In a catchment densely monitored for hydrological surveying, flood-flow generation can be investigated at the network scale in reaches between two gauging stations. In such a runoff-runoff approach, flows from the intermediate catchment can be significant contributors of flood flow at the outlet, depending on catchment geometry and hydrological conditions. Although lateral flow from the intermediate catchment is well documented for low-water periods, investigating for instance surface-water / groundwater (SW/GW) interactions in successive reaches (Covino et al., 2011; Mallard et al., 2014), there is a lack of information on how to define lateral flow during flood events.

As shown on Figure 1, several types of lateral flow occur during floods, depending upon catchment descriptors such as climate, relief, geology, soil, etc. For highest peakflows, lateral outflow (Figure 1a) is generally considered in the case of overbank flow when waters from the river flow to the flood plain, without been returned to the river during the flood (Jothityangkoon & Sivapalan, 2003; Moussa and Bocquillon, 2009), modifying the shape of the hydrograph (Rak et al., 2016; Fleischmann et al., 2018). Another case favouring lateral outflow is specific to permeable basins, where river losses infiltrate and recharge the underlying aquifer (Sorman et al., 1997; Charlier et al., 2015b; Dvory et al., 2018). This is notably the case in arid/Mediterranean environment, where the importance of infiltrating floodwater for aquifer recharge has been highlighted in disconnected river- aquifer systems (Hughes & Sami, 1992; Camarasa Belmonte & Segura Beltrán, 2001; Lange, 2005; Dahan et al., 2007; Vázquez- Suñé et al., 2007). Lateral inflow (Figure 1b) is often taken into account in flood modelling due to its common occurrence in all types of catchments (e.g., Cimorelli et al., 2014;2018). Most inflow comes from tributaries (point lateral flow) or from hillslope runoff (distributed lateral flow). In permeable basins, groundwater flooding can also cause large lateral inflows (Finch et al., 2004; Pinault et al. 2005; Naughton et al., 2012), but this simple typology can mask a greater complexity of lateral flows. In most cases, several processes are spatially and temporally combined, leading to mixed (and maybe compensated) out- and in-flows during flooding, as shown on Figure 1c.

Investigating flood generation in a carbonate catchment is of great interest because it is highly controlled by SW/GW interactions. Following Robins and Finch (2012), a true groundwater flood—where the groundwater level rises above ground surface—is distinguished from a groundwater-induced flood—which occurs as an intense groundwater discharge through springs and permeable shallow horizons into surface waters. These types of flooding notably occur in chalk terrains of Northern Europe (Finch et al., 2004; Pinault et al., 2005; Hughes et al., 2011; Morris et al., 2015; Thiéry et al. 2018), where extreme events were caused by exceptionally high groundwater levels in 2000-2001. Flood generation in these cases was driven by saturation of the matrix porosity (Price et al., 2000), and is associated to long-duration floods due to the aquifer inertia.

When carbonate formations are highly karstified, which can be the case in chalk formations, groundwaters contribute to the fast-flow component in streams (Maréchal et al., 2008; Charlier et al., 2015b), playing a significant role in flooding, even for flash floods. In karst basins, flooding may also occur by the cessation of rainfall infiltration because of the small retention capacity of a karst massif (Maréchal et al. 2008; Fleury et al. 2013), or because the infiltration capacity of the underground conduit network is exceeded (Lopez-Chicano et al. 2002; Bonacci et al. 2006; Bailly-Comte et al. 2009). As chalk aquifers are often karstified, specific types of groundwater flooding can occur seasonally, as is reported from Ireland in low-lying topographic depressions, known as turloughs, that are fed by underground flow from karst aquifers (Naughton et al., 2012; Gill et al., 2013).

Another frequent feature of flood generation in karst basins is a contrasted evolution of peakflow, i.e. amplification or attenuation, in a same reach depending upon the prior aquifer saturation level (Maréchal et al., 2008; Charlier et al., 2015b). In catchments characterized by a high storage potential in thick unsaturated zones of karst aquifers, peakflow attenuation is common (Jourde et al., 2013; Ladouche et al., 2014; Brunet & Bouvier, 2017), which may add to the attenuation due to flood-wave diffusion in the channel. Attenuation is also enhanced in medium and large catchments, where diffusion is favoured due to the development of drainage networks in lowland areas (e.g., Moussa & Cheviron, 2015; Trigg et al., 2009) and due to the occurrence of overbank flow (see a synthesis in Bates & De Roo, 2000; Hunter et al., 2007; Moussa & Bocquillon, 2009). In karst massifs, the drainage is often developed in canyons that cross-cut the carbonate plateau, where the channel commonly is rougher. We can suppose that this will favour a velocity decrease of the flood wave due its meandering feature. Finally, there is a need in such a context to understand the respective roles of lateral outflow (losses, overbank flow) and flood routing in peakflow attenuation.

Many modelling methods exist for investigating SW/GW interactions on groundwater flooding. As the hydrogeological context of chalk aquifers can be favourable for applying physically-based models, lumped approaches (Pinault et al., 2005; Upton & Jackson, 2011) were completed by distributed models, such as MODCOU (Korkmaz et al., 2009), SIM (Habets et al., 2010), MIKE SHE (House et al., 2016), or MARTHE (Thiéry et al., 2018). However, the specifics of karst basins with their high degree of complexity prevent an efficient application of such distributed models, and few such modelling approaches have been used for characterizing SW/GW interactions during flood events in karst basins. To our knowledge, the only published work on the application of flood-routing models to channel reaches, is based on the simplified Saint-Venant equation that describes unsteady flow in partially filled channels; a first modelling approach for karst areas was proposed by Bailly-Comte et al. (2012), using the Kinematic Wave Equation coupled with a linear underground reservoir for simulating lateral inflow. Recently, Charlier et al. (2015b) tested the relevance of the Diffusive Wave Equation (DWE) for assessing lateral in- and outflows in karst rivers. DWE is more adapted to floods with large wavelengths, common in medium and large catchments (Ponce, 1990). Moreover, Moussa (1996) extended the analytical solution of the DWE under the Hayami (1951) hypothesis (Moussa & Bocquillon, 1996b) to the case of uniformly distributed lateral flow. The model can calculate the temporal distribution of lateral flow in a river reach by an inverse problem approach, using as input the flow in both the upstream and downstream gauging stations. The solution of the inverse problem proposed is part of the hydrological MHYDAS model (Distributed Hydrological Modelling of AgroSystems ; Moussa et al., 2002). Using this inverse problem for the DWE we can simulate the global dynamics of lateral flow during floods, which provides information on the hydrological processes involved, such as tracking loss and gain reaches in rivers (Charlier et al., 2015b), or characterizing matrix/conduit relationships through the underground network of a karst aquifer (Cholet et al., 2017); both these publications showed promises in the ability of such models to quantify lateral flow in a karst basin.

The aim of this paper is therefore to quantify peakflow attenuation/amplification in a river, and to investigate lateral flow during floods. For that, we propose a modelling framework to simulate lateral flow from an intermediate catchment, using the inverse problem for the DWE. The model is applied to 33 flood events in the karst reach of the Iton River in French Normandy, where peakflow is attenuated. After analysing the role of groundwater on flood generation, the model is used for quantifying the peakflow attenuation. The respective roles of outflow and wave diffusion on peakflow attenuation are described according to various parameterization strategies. Investigating the sensitivity of model parameterization, we provide a new way for better understanding and quantifying flood routing through accounting for lateral flow.

2 Modelling approach

2.1 The diffusive wave model

In order to model 1-D unsteady flow in open channels, the Saint-Venant equations can be simplified under some assumptions (acceleration terms neglected), leading to the Diffusive Wave Equation (DWE) (Moussa & Bocquillon, 1996a), which corresponds to the case of long wave-length flood events generally observed in medium and large basins:

Eq. 1 $$\frac{\partial Q}{\partial t} + C\frac{\partial Q}{\partial x} - D\frac{\partial^2 Q}{\partial x^2} = 0$$

where x [L] is the length along the channel, t [T]) is the time, and the celerity C [L.T^{-1}] and the diffusivity D [L^2 T^{-1}] are functions of the discharge Q [L^3 T^{-1}]. In this case, we have a simple two-parameters (C, D) model.

At the scale of a river reach – delimited by an input I and an output O station – the model is applied to the flood component ($Q_{I\,f}(t)$ and $Q_{O\,f}(t)$) of the total discharge ($Q_I(t)$ and $Q_O(t)$) recorded at the input and output stations, respectively. $Q_{I\,f}(t)$ and $Q_{O\,f}(t)$ are estimated by removing the baseflow components $Q_{I\,b}(t)$ and $Q_{O\,b}(t)$ from $Q_I(t)$ and $Q_O(t)$.

The resolution of Eq. 1 is obtained using the convolution approach proposed by Moussa (1996) based on the Hayami assumptions (Hayami, 1951), i.e. considering C and D as constant parameters over time along a channel of length l:

Eq. 2: $$Q_{I\,f\,r}(t) = \int_0^p Q_{I\,f}(t-\tau)\,K(\tau)\mathrm{d}\tau = Q_{I\,f}(t) * K(t)$$

Where $Q_{I\,f\,r}(t)$ is the routed input hydrograph on the flood component, p is the time memory of the system, and the symbol * represents the convolution operator. As there is no problem of calculation time, the term p must be large in comparison to the travel time along a channel reach. In Eq. 2, the Hayami kernel function $K(t)$ is expressed as follows:

Eq. 3 $$K(t) = \frac{l}{2(\pi D)^{1/2}} \frac{exp\left[\frac{Cl}{4D}(2 - \frac{l}{Ct} - \frac{Ct}{l})\right]}{t^{3/2}}$$

Eq. 2 is then used in a direct approach to define the *C* and *D* parameters, comparing $Q_{I\,f\,r}(t)$ with $Q_{O\,f}(t)$. In a theoretical case without lateral flows, under diffusive wave and Hayami asumptions, the routed input hydrograph is equal to the observed output one ($Q_{I\,f\,r}(t) = Q_{O\,f}(t)$) as shown on Figure 2a.

2.2 The diffusive wave model with lateral flows

Several solutions exist to resolve the DWE with lateral flows (see Cimorelli et al. (2014) for a review). In our paper, the one proposed by Moussa (1996) has been selected because an analytical solution has been proposed to solve the inverse problem, as explained herein. Moreover recently, an experimental evaluation of the solution has been performed by Moussa and Majdalani (2019) on a large variety of hydrograph scenarios. The model is based on Hayami assumptions, considering i) diffuse lateral flows as uniformly distributed along the channel reach, and ii) the two *C* and *D* parameters as constant over time during the flood. It is used to route the input hydrograph to the output station accounting for lateral flows. A solution to the inverse problem of the DWE is used to model lateral flows, knowing the input and the output hydrographs. This last case is suitable to be used when no observations on lateral flows are available. Indeed, it gives information on the potential contributions of the intermediate catchment, and notably on the estimation of peakflow of the lateral hydrograph.

Implementing lateral flows in the DWE needs to account for the lateral flow rate *q* per unit length [L^2 T^{-1}] along a channel reach, as proposed by Moussa (1996):

Eq. 4 $$\frac{\partial Q}{\partial t} + C\left(\frac{\partial Q}{\partial x} - q\right) - D\left(\frac{\partial^2 Q}{\partial x^2} - \frac{\partial q}{\partial x}\right) = 0$$

In the particular case of uniform distribution of *q* along the reach, under the hypotheses used in the Hayami model (*C* and *D* constant), Moussa (1996) proposes an analytical resolution:

Eq. 5 $$Q_{I\,f\,r}(t) = \Phi(t) + \left(Q_{I\,f}(t) - \Phi(t)\right) * K(t)$$

Eq. 6 with $$\Phi(t) = \frac{C}{l}\int_0^t \left[Q_{A\,f}(\lambda) - Q_{A\,f}(0)\right] d\lambda$$

Eq. 7 and with $$Q_{A\,f}(t) = \int_0^l q(x,t).\mathrm{dx}$$

where $q(x, t)$ [$L^2\ T^{-1}$] is the lateral flow rate per unit length as a function of distance along the channel reach x. $Q_{A\,f}(t)$ represents the flood component of the lateral hydrograph $Q_A(t)$. As illustrated on Figure 2b,c, the expression $q(x, t)$ may be positive or negative depending upon the lateral flow direction from the channel when inflow or outflow occurred, respectively.

According to Moussa (1996), the inverse problem identifies $Q_{A\,f}(t)$ by knowing $Q_{I\,f}(t)$ and $Q_{O\,f}(t)$, and according to a predetermination of the two parameters C and D. Given that,

Eq. 8 $$Q_{A\,f\,r}(t) = Q_{O\,f}(t) - Q_{I\,f\,r}(t)$$

Eq. 9 with $$\Phi(t) - \Phi(t) * K(t) = Q_{A\,f\,r}(t)$$

Using the Laplace transforms, an approximation of the solution of Eq. 9 is

Eq. 10 $$\Phi(t) = Q_{A\,f\,r}(t) + Q_{A\,f\,r}(t)\sum_{i=1}^{\infty} K^i(t)$$

with

Eq. 11 $$K^i(t) = K * K * \ldots * K \qquad (i\ times)$$

$Q_{A\,f}(t)$ can be easily calculated using Eq. 12, after the identification of $K(t)$ in Eq. 3:

Eq. 12 $$Q_{A\,f}(t) = \frac{l}{C}\frac{d\Phi}{\mathrm{dt}}$$

The lateral hydrograph $Q_A(t)$ is simply calculated from $Q_{A\,f}(t)$ by adding $Q_{A\,b}(t)$, the difference of baseflow ($Q_{O\,b}(t)$ - $Q_{I\,b}(t)$) between O and I stations.

When no observations of $Q_A(t)$ are available to calibrate the model, this simple approach with two parameters (C, D) is favoured. This choice is made in comparison with more complex approaches adding a further degree of freedom, by considering additional parameters. Thus, according to the principle of parsimony, we choose the most simplest and robust model, considered as more reasonable. For the inverse problem, many couples of solutions (C, D) exist to simulate $Q_A(t)$. In our paper, the sensitivity on the various couples of solutions (C, D) is then characterized to assess whether the different solutions brings some different interpretations or not on the lateral contributions.

2.3 Quantification of the peakflow amplification/attenuation

Based on the conceptual schemes of Figure 1, we expect that the global lateral flow at a given time is the sum of simultaneous negative and positive q values originating from various processes. Knowing this, we now must understand the causes of the attenuation or amplification of peakflow (without the baseflow component) shown on Figure 2, from the upstream station $Qx_{I\,f}$ [peakflow of the input hydrograph $Q_{I\,f}(t)$: black curve] to the downstream one $Qx_{O\,f}$ [peakflow of the output hydrograph $Q_{O\,f}(t)$: blue curve]. The difference between both peakflows is noted E:

Eq. 13 $$E = Qx_{O\,f} - Qx_{I\,f} = E_D + E_A$$

with E>0 in the case of peakflow amplification (Figure 2b), and with E<0 in the case of attenuation (Figure 2c). E is composed of two terms linked to the hydraulic properties of the channel E_D (controlled by D parameter), and linked to the lateral flows (E_A). Noted that in the case of no-lateral flow component (Figure 2a), $E_A = 0$ and $E = E_D \leq 0$.

Diffusion is responsible for a peakflow attenuation E_D of the input hydrograph expressed as follows:

Eq. 14 $$E_D = Qx_{I\,f\,r} - Qx_{I\,f} \quad \text{with } E_D \leq 0$$

with $Qx_{I\,f\,r}$, the peakflow of the routed input hydrograph $Q_{I\,f\,r}(t)$ without lateral flow (dashed grey curve). Lateral flow is responsible for an attenuation/amplification expressed as:

Eq. 15 $$E_A = Qx_{O\,f} - Qx_{I\,f\,r}$$

Depending upon the importance of out- and in-flows, E_A may be negative or positive, respectively.

Hereafter, we investigate the flood processes in a catchment favouring peakflow attenuation (E<0) according to high diffusion (E_D<0) and the occurrence of river losses and overbank flow (E_A<0).

2.4 Sensitivity analysis on a virtual example

To illustrate the model behaviour and its calibration, various parameterization sets of C and D parameters were used to apply the inverse model on the same couple of theoretical hydrographs $Q_{I\,f}$ and $Q_{O\,f}$ (Figure 3). Figure 3a and 3b present the simulations carried out by fixing D and varying C. In the case of a gaining reach (Figure 3a), the results show that the highest lateral peakflow is simulated for the lower C. When increasing C, lateral peakflow decreased and became constant when C exceeded a threshold (here C>0.4 m/s) at the same time that outflow was simulated at the start of the flood event. This illustrates a dynamics of compensation of out- and inflows during the same flood event due to a conservation of the flood volume (total lateral flood volume was equal for all simulation tests). The model behaviour is simpler in the case of a losing reach, because Figure 3b shows that the higher the C, the higher and the earlier will be the lateral outflow peak. Figure 3c and 3d show a similar test, but now fixing C and varying D. In the case of lateral inflow (Figure 3c), the results showed that the higher the D, the higher will be lateral peakflow. Similar to Figure 3a, temporal lateral outflow is simulated at the beginning of the flood when D increases. In the case of a losing reach (Figure 3d), the higher the D, the higher and the earlier will be the lateral outflow peak. Globally, this sensitivity analysis shows that various lateral hydrographs are simulated for different couples of *C* and *D* parameters, due to equifinality in the modelling approach. As observed in previous studies (Moussa & Bocquillon, 1996a; Yu et al., 2000; Charlier et al., 2009; Cholet et al., 2017), *C* is more sensitive than *D*, as a variation of *C* by 4 in our test case generated a same range of lateral peakflow variations as a variation of *D* by 10,000.

Consequently, for a given flood event, the *C* and *D* parameters should be optimized. In our case, we expect that, whatever the flood, the fast component of lateral flow will contribute to peakflow at the output station. Thus, we chose to optimize *C* and *D* in order to put in phase peakflows of the routed inflow and outflow hydrographs. Figure 3e and 3f show the effect of the inverse model, varying *D* and calibrating *C* under these conditions. Regarding the evolution of parameters, it shows that *C* decreases when *D* increases, but up to a lower limit (of $C = 0.22$ m/s in our case for $D > 150$ m²/s). In the case of a gaining reach (Figure 3e), the higher the *D*, the higher will be the lateral peakflow, but for a losing reach (Figure 3f), the higher is the *D*, the lower will be lateral outflow peakflow. These results show that, contrary to varying *D* and fixing *C* (Figure 3b), lateral outflow peakflow decreases when *D* increases, provided the corresponding *C* is optimized following the hypothesis of a same routing scheme for lateral out- and in-flows.

2.5 Framework of the modelling approach used

We propose a framework in this paper for defining amplification/attenuation of peakflows in a river reach. Though interpretation of the results will obviously be better when confronting them with field knowledge, the model application can stand alone in order to help decipher some hydrological processes in catchments with a complex behaviour. The modelling framework has four steps:

- First, the base and flood components of the hydrographs at the two gauging stations must be separated;
- Second, calibration of the DW model parameters (*C* and *D*) applying the direct approach without lateral flow (Eq. 2). We saw above how the calibration strategy may influence the results, and we thus must optimize *C* and *D* by inputting phase peakflow of the routed inflow $Qx_{I\,f\,r}$ and of the outflow $Q_{O\,f}$ (as illustrated on Figure 1);
- Third, is the calculation of the lateral hydrograph, applying the inverse approach of the DW model using in Eq. 10 the pre-calibrated *C* and *D* values from Eq. 2;
- Fourth, we quantify peakflow amplification/attenuation using Equations 12 and 13.

The choice of the modelling calibration on peakflows proposed in the second step appears to be the most likely in the absence of monitoring lateral flow along the reach. In order to account for the uncertainty on this choice, different sets of simulation by varying *D* values (and corresponding calibrated *C* values; see Figure 3f) should be carried out. This is tested in the following case study.

3 Case study

3.1 Field site

3.1.1 Basin presentation

The Iton basin is located in Normandy, north-west France (Figure 4a). Land use consists mainly in cereal crops and grassland, and the only important urban area is Evreux (100,000 inhabitants) on the Iton in the downstream part of the basin. The topographic catchment is 1050 km² at the Normanville gauging station, 7 km downstream Evreux city (Figure 4b).

3.1.2 Climate

The climate is of the humid temperate oceanic type. Annual rainfall ranges between 500 and 1000 mm, with an inter-annual average of 600 to 715 mm between the upstream and downstream areas of the catchment, respectively. The intra-monthly variations are relatively low, with slightly wetter months in autumn (60 to 80 mm/mo) compared to other periods of the year (40 to 65 mm/mo), but in general rainfall is quite regularly distributed throughout the year. The region has also been characterized in the past by exceptional rainfall in 2000-2001 (about 100 mm of rainfall depth in 2 to 4 days only), generating catastrophic flood events enhanced probably by two previous years of accumulated wetness (Pinault et al., 2005).

3.1.3 Geology and hydrogeology

The geomorphological context of the basin can be described as a plateau cross-cut by the Iton River and its main tributary, the Rouloir (Figure 4b). On the plateaux, Late Cretaceous chalk formations are covered by a clayey formation associated with loess, up to several tens of metres thick. In the valley bottom, chalk formations may also be covered by alluvium. Thus, aquifers in the basin are located in the karstified chalk that is mostly covered by shallow formations, as indicated by the non-exposed karst-aquifer symbol in the extract of the World Karst Aquifer map (Chen et al., 2017) in Figure 4a.

The underground karst networks in the chalk are fed by diffuse infiltration waters through swallow-holes developed in the (non-cohesive) shallow formations on the plateau, but also from river losses where the chalk is exposed in the river bed. This can generate a drying up of the stream, as in the “Dry-Iton” reach and the Rouloir tributary (Figure 4b). Outlets of these karst aquifers are the springs at the foot of the hillslopes close to the river, and feeding it. The use of artificial tracers (yellow arrows; Figure 4b) showed that infiltrated Iton waters bypass the streambed underground, to reappear downstream in the same bed via resurgence springs located near the confluence with the Rouloir (David et al, 2016).

3.1.4 Conceptual model of lateral flow

The conceptual model presented in Figure 4c is the result of hydrogeological and hydrological studies, highlighting surface-water / groundwater (SW/GW) interactions of various origin (Charlier et al, 2015a; David et al., 2016). The main horizontal line represents the Iton River for the 75-km reach between the two gauging stations Bourth and Normanville (input and output in Figure 4c; see Figure 4b for location). Surface flow is in light blue colour and groundwater flow in dark blue. The dashed line represents ephemeral streams due to river losses. For surface flow, the main properties of the Iton are: i) Drying-up of the drainage network (as well as the Rouloir) where it crosses the karst zone; and ii) Contribution of the main tributary (Rouloir) and of the two groups of springs near the confluence. Groundwater flow is composed of infiltrated river losses as well as aquifer contributions via several springs. Finally, in this conceptual scheme of SW/GW interactions in the Iton basin in its karst part, lateral flow is defined by outflow from river losses and inflow from groundwater origin.

3.2 Data

3.2.1 Hydrological and hydrogeological time series

Mean rainfall and soil-humidity indices (HU2) over the Bourth and Normanville sub-basins were obtained from METEO FRANCE, available on the COMEPHORE/ANTILOPE and SAFRAN ISBA MODCOU (Habets et al., 2008) databases, respectively. Even if HU2 is not derived from observation data sets, this index is widely used by modelers to initialize hydrological models. Thus, despite the uncertainty on the model output, this is a pertinent data set of soil wetness, that is used as it by end-users (forecasters). Streamflow hydrographs were obtained from the “Service de Prévision des Crues” (SPC) for the Bourth (code: H9402030) and Normanville (code: H9402040) stations, available on Banque Hydro (2015). Groundwater levels were obtained from ADES (2015). All hydrological and hydrogeological time series were synchronized at an hourly time step over the 1999-2014 period.

3.2.2 Flood selection and processing for model application

For flood event analysis, the highest 33 peakflows at the Bourth gauging station (Input station) were selected from the dataset (Table 1). Rainfall events ranged between 16 and 100 mm, and peakflows between 9 and 26 m^3/s at the Input station and 5 to 17 m^3/s at the

Output one. Minimum discharge-inducing overbank flow is 14 and 10 m^3/s for the Bourth and Normanville stations, respectively. These thresholds correspond to discharge generating flows in the flood plain without rapid return towards the channel. Table 1 shows that the 10 highest flood events were partially subject to overbank flow.

The inverse approach of the DW model was applied to the karst portion of the Iton River from hydrographs of the Input (Bourth) and Output (Normanville) stations. Model application requires in a first step a separation of the base and flood components (Section 2.5), which was done with the BFI method (Gustard et al., 1992) using ESPERE software (Lanini et al., 2016).

4 Results

4.1 Groundwater influence on surface flow

4.1.1 Base and flood components

An example of the base and flood flow separation is given in Figure 5 for the 2000-2001 hydrological cycle at the Bourth Input station in black (Figure 5a) and the Normanville Output station in blue (Figure 5b). The input-output relationships for the base and flood components account for a 2-day delay (Figure 5c), corresponding to the mean delay of peakflows. It shows a contrasted behaviour: the baseflow increases 3 to 4 fold from input to output station, whereas flood flow decreases by several m^3/s for the highest flood events. This means that lateral groundwater inflow contributed highly to stream flow for the base component, at the same time that strong lateral outflow occurred for the flood component.

4.1.2 Baseflow analysis

Figure 6 shows the relationships between baseflow calculated at the output gauging station and groundwater levels for six piezometer wells. We observe a slight to fair correlation for wells located on the plateau, having mainly multi-year cycles (Normanville and Cierrey piezometers Figure 4b for location). Best correlations are obtained for wells with annual cycles at Nogent-le-Sec, Moisville, and Coulonges (best linear correlation with R^2 >0.7). The

Graveron well, with both multi-year and annual cycles, reflects an intermediate behaviour. These results show that SW/GW interactions on the baseflow component is driven by lateral exchanges with the karst aquifer, best shown by the Coulonges piezometer well.

4.1.3 Flood analysis

The input-output relationships for peakflow are plotted in Figure 7 according to two factors used as key indices of the catchment saturation level: the soil-humidity index (HU2, Figure 7a) and the karst saturation index (GW depth *z* in the Coulonges well; Figure 7b). Before assessing the effect of such factors, it is interesting to observe that output peakflow is always less than, or equal to, the input one. A peakflow attenuation is also observed for the highest flood events when $Qx_I > 15$ m^3/s. This value corresponds to the threshold of overbank flow at the input station, meaning that this process may explain part of the attenuation of the highest flood events. In the first case, HU2 seems not to be a discriminant factor for explaining the data variability, as most events are characterized by indices close to the saturation level (i.e. HU2 ~60, HU2 ranging between 40 and 65). In the second case, the initial GW depth seems to explain the attenuation variation for a given input peakflow: the higher the initial GW depth, the higher will be output peakflow. Globally, this analysis shows that the soil saturation index is not a suitable factor for differentiating flood intensity. On the contrary, these results show that the peakflow attenuation is related to the antecedent groundwater level.

4.2 Simulation of a lateral flood hydrograph

4.2.1 Model application to a mono-peak flood event

Figure 8 shows an example of the DW model application for the mono-peak flood event of 06/01/2001 with the optimized parameters C = 0.35 m/s and D = 500 m²/s. From top to bottom rainfall, soil saturation index (HU2), groundwater depth in the Coulonges well, discharge for the flood component, and total discharge are shown. On each discharge plot, four hydrographs correspond to the observed input hydrograph ($Q_{I\,f}$ and Q_I; black curve), the observed output hydrograph ($Q_{O\,f}$ and Q_O; blue curve), the routed input hydrograph ($Q_{I\,f\,r}$ and $Q_{I\,r}$; dashed grey curve) using the direct model without lateral flows, and the simulated lateral hydrograph ($Q_{A\,f}$ and Q_A ; dotted red curve) using the inverse model. Both soil saturation index (HU2 >58) and groundwater levels (GW >-15 m AGL) were saturated before the beginning of the flood. The discharge analysis shows a strong attenuation of the flood hydrograph along the reach, halving the peakflow from 26.8 m^3/s (Qx_I) to 13.7 m^3/s (Qx_O). Before the flood, lateral inflow ($Q_A(t)$) was 3.7 m^3/s; but during the flood it became negative. This may be interpreted as a continuous contribution of lateral baseflow hidden by occasional high losses in the flood component. Using the minimum values of the lateral flood component, we can quantify the maximum intensity of lateral outflow $Qn_{A\,f}$ as -6.8 m^3/s. The peakflow attenuation due to outflow (losses+overbank flow) E_A is -6.6 m^3/s and the attenuation due to diffusion E_D is -9.2 m^3/s, leading to a total peakflow attenuation E of -15.8 m^3/s.

Analysis of this single flood event shows that, despite lateral baseflow, the outflow associated to the flood component can be quantified. As outflow by overbank flow may occur during the highest discharge, outflow by river losses is probably continuous as long as the stream flows at the input station. This suggests that losses are compensated by highest baseflow discharge during recession periods, leading to under-estimating their real values. The other interesting point is that we can compare peakflow attenuation by hydraulic processes (diffusion) and by outflow (river loss+overbank flow), quantifying it (for a given parameterization set) equal to 57% and 43%, respectively.

4.2.2 Distribution of model parameters

Following the above example, the model was applied to the 33 main flood events (Table 1), using the parameterization strategy presented in Section 2.5. Several values of D were selected for optimizing the C parameter, for inputting phase peakflows of the routed input hydrograph (Q_{Ir}) and of the output one (Q_O). Figure 9 shows C distribution using boxplots for five D values of 500, 1000, 2500, 5000, and 10,000 m²/s, which is the classic range for streams and rivers (Todini, 1996), such as in our study, knowing that D increases with the size of the river. Boxplot analysis shows that C values range between 0.1 and 0.4 m/s with a relative small variability for a given D. The higher the D, the lower will be the range of C values from 0.35 to 0.11 m/s. As shown in the sensitivity analysis (Section 2.4 above), a lower limit of C values to almost 0.15 m/s is observed for D values above 2500 m²/s. Knowing that lateral flows are highly sensitive to C values and less so to D values, various parameterization sets will be tested in the following section.

4.3 Quantification of peakflow attenuation

4.3.1 Assessment of lateral outflow

In order to quantify outflow during floods, we express the maximum lateral outflow intensity as a function of the input peakflow. Figure 10 presents $Qn_{A\,f}$ (i.e. maximum losses as negative values attributed to outflow) *vs.* $Qx_{I\,f}$ for all the 33 flood events and for five calibration strategies that vary D (and the corresponding optimized C) from 500 to 10,000 m²/s; dark blue colours refer to the lowest D values. The first result confirms that outflow for the flood component is simulated for all flood events, regardless of input peakflow. This means that lateral outflow intensities of the fast component are systematically higher than potential lateral inflow values during the flood, i.e. flood flow from karst springs and tributaries, or surface runoff. As expected, the second result confirms that, globally, the lateral outflow intensity is higher for a lower D. Outflow increases with increasing input peakflow, but the relationship stabilizes when $Qx_{I\,f}$ >12 m^3/s, very close to the threshold value of 14 m^3/s for starting overbank flow when considering peakflow of the total discharge Qx_I (Qx_I=$Qx_{I\,f}$+$Q_{I\,b}$). For D=500 m²/s, $Qn_{A\,f}$ reaches a ceiling of ~9 m^3/s, against 7 m^3/s for D=1000 m²/s and 2 m^3/s for the highest D value of 10,000 m²/s.

The influence of the initial karst saturation level on outflow intensity has been tested, and any concluding results were highlighted to validate this hypothesis. Consequently, input peakflow seems to be the main driver of outflow intensity. An interesting result is that when discharge is below the overbank flow threshold ($Qx_{I\,f}$ <12 m^3/s), outflow is mainly due to river losses, following a linear relationships between $Qn_{A\,f}$ and $Qx_{I\,f}$. Depending upon the parameterization strategy, these river losses may reach high values of up to 9 m^3/s, corresponding to half of the peakflow at the input station. When discharge exceeds this threshold, the highest discharge outflow ceiling may be explained by a limitation of the infiltration rate into the stream bed, due to a ceiling of the water-level increase in the river bed when overbank flow occurs.

4.3.2 Factors influencing peakflow attenuation

Two phenomena participate in peakflow attenuation (E): hydraulic processes due to flood wave diffusivity (E_D) and hydrological processes due to outflow (E_A) (cf. Eq. 13). To quantify their respective roles, Figure 11 presents the E_A *vs.* E_D relationships for the same five parameterization strategies as in Figure 10. We see that E_D ranges between -18.0 to -0.5 m^3/s whereas E_A ranges between -9 to 4.0 m^3/s. As expected, the higher the D (light blue colour), the higher the $|E_D|$ values (negative in the graph since E_D is inevitably an attenuation of peakflow). Except for cases with the lowest D values (D=500 m²/s), and some cases with D=1000 m²/s (dark blue colour, Figure 11), the points lie below the E_D=E_A line, i.e. attenuation due to diffusivity is often higher than that due to outflow. It is interesting to note that when D is high and thus $|E_D|$ is high, E_A is positive (lateral flow became inflow) while being below the E_D=-E_A line. This means that in these cases, despite flood amplification due to lateral inflow, peakflow attenuation is finally observed because the effect of high diffusivity overtakes it.

These simulation tests replicate a wide range of classic stream and river D values found in the literature. Finally, we should evaluate the proportion of E_D and E_A for the theoretical range of

D, calculated with the following formula proposed by Chow (1959) that considers simple network descriptors: $D = \bar{Q}$ / (2 x *slope* x *width*), where $\bar{Q}$ is the mean flow discharge for a rectangular section. Varying the mean slope of the river from 0.0010 to 0.0015, the mean river width from 5 to 10 m, and $\bar{Q}$ from 5 to 20 m^3/s, D ranges between 250 and 2000 m^2/s, over a quite small range of values compared to the tested ones (500 to 10,000 m^2/s). In this theoretical case, E_D and E_A are roughly equal according to Figure 11, but this result should be taken with care due to the high spatial variability of channel properties along the Iton River.

In summary, these results show that, in the case of low D values, peakflow attenuation is equally due to diffusion and to outflow, but in cases with high D values, most of this attenuation is caused by diffusion. They also show that, despite lateral inflow in the case of highest D values, these contributions are compensated by a strong attenuation due to flood wave routing.

5 Discussion

5.1 On the interest of using a diffusive wave model to assess peakflow attenuation and/or amplification

Although aquifer's recharge by river losses can attenuate floods in arid/Mediterranean environment (Sorman et al., 1997; Hughes & Sami, 1992; Lange, 2005; Dahan et al., 2007; Vázquez- Suñé et al., 2007) or in karst basins (Jourde et al., 2013; Charlier et al., 2015b; Brunet and Bouvier, 2017), we cannot neglect hydraulic diffusion processes when we are interested in peakflow forecasting. In the case of permeable basins, our results show that, despite the presence of an infiltration zone in the river bed with significant losses (several m^3/s), peakflow attenuation is mainly related to diffusion of the flood wave. This raises the question of the mechanisms favouring such attenuation, which may be related to the meandering morphology of the drainage network, as well as to the zones of temporary storage for highest flood events (Moussa & Cheviron, 2015).

Our example shows the added value of using a diffusive model—combining direct and inverse problem approaches—for better understanding and quantifying SW/GW interactions during floods, and which deserves to be tested on other types of basins where significant lateral losses and/or gains are observed (Martin & Dean, 2001; Ruehl et al., 2006; Payn et al., 2009). For instance, the inverse DWE model has also been used for investigating lateral flow in underground karst conduits, and for defining the exchanges between conduits and the fissured matrix (Cholet et al., 2017). In parallel to these considerations that promote the model as a tool for diagnosing SW/GW exchanges at different scales, our results highlight the inaccuracies that can be generated by using non-diffusive models, as is frequently the case for flood modelling (see review in Singh, 2002) and for karst basins (Bailly-Comte et al., 2012; Dvory et al., 2018). Our results are coherent with Naulin's work (2012) in the Cévennes region (southern France), who showed that DWE was more suitable in lowland areas, including karst formations, than in mountains with less permeable hard-rock formations.

5.2 Surface-water / groundwater interactions in permeable basins

5.2.1 River losses and overbank flow

The relationship between lateral outflow and input peakflow has established a function of river losses that improves the understanding of SW/GW interactions in permeable basins. In fact, below the discharge threshold for overbank flow, outflow is mainly generated by losses that account in our case for several m^3/s during a flood. Although this process is well known in many catchments when studying low water-level periods, it is generally not considered during flood events, because inflow conceals it. Despite some works on infiltrating floodwater in basins characterized by disconnected river-aquifer systems (Hughes & Sami, 1992; Lange, 2005; Dahan et al., 2007; Vázquez- Suñé et al., 2007), the estimation of infiltration rate during the flood (i.e. at a high temporal resolution) is generally disregarded. Thus, our study brings a relevant approach to help quantify the loss intensities as well as the recharge rate of the underlying aquifers.

Outflow due to river losses is an important process as it may represent up to half of peakflow in models, depending upon the parameterization strategy. The estimated value loss of several m^3/s is important, but not exceptional as it is coherent with observations made on other ephemeral karst rivers in southern France (Ladouche et al. 2002, 2004), or in the Jura Mountains (Charlier et al., 2014). The linear relationship between outflow intensity and input peakflow (below the overbank flow threshold) argues for the control of loss rate by water height in the river. This implies that the aquifer fed by the losses is disconnected with the river, agreeing with the absence of influence of groundwater level on this relationship.

The ceiling of outflow intensity with the increase of discharge into the river is most probably explained by the occurrence of overbank flow, knowing that flood plain attenuation can play a key role on the modification of hydrograph shape (Sholtes & Doyle, 2011; Valentova et al., 2010; Rak et al., 2016; Fleischmann et al., 2018). Indeed in the study case, outflow on the flood plain appears to be an important process of peakflow attenuation for the highest floods, since the increase in infiltration with an increased input peakflow is stopped (Figure 10). Another concept may explain this outflow ceiling: several studies of karst hydrology reported a limitation of infiltration during rainfall events due to small void diameters or constricted conduits at depth (Bonacci, 2001; Bailly-Comte et al., 2009). However, such a process requires specific monitoring that fell outside the scope of our study.

5.2.2 Loss and gain in river reaches

The specific features of the studied river, including both loss and gain reaches, render an analysis of lateral exchanges difficult. The occurrence of both out- and in-flow in some reaches has been conceptualized as hydrologic turnover (Covino et al. 2011; Mallard et al., 2014) in simultaneous loss and gain reaches. This pattern was highlighted both in chalk catchments, where it was found that groundwater flooding consists of a combination of intermittent stream discharge and anomalous springflow (Hughes et al., 2011), and in karst rivers characterized by successive loss and gain reaches from a multi-layered aquifer in deep canyons (Charlier et al., 2015b). Improving the understanding of lateral exchange during floods, our modelling approach opens a novel way to help deciphering the various contributions of loss and gain.

5.3 Saturation state in a karst catchment: soil moisture *vs.* aquifer storage

An influence of the aquifer-saturation state on peakflow attenuation is observed in the karst part of the catchment. This agrees with several papers on the exceptional groundwater flooding of 2000-2001, in karstified chalk areas of northern Europe (Finch et al., 2004; Pinault et al., 2005; Hughes et al., 2011; Morris et al., 2015; Thiéry et al. 2018). However, we did not see this influence on the relationship between lateral outflow intensity and input peakflow on the flood component, as might have been expected. This means that aquifer saturation probably influences the baseflow component, which increases strongly in the karst reach (Figure 5). Even if river losses recharge the underlying aquifer, which in turn feed the river downstream, these results are not contradictory. In fact, losses are controlled by the infiltration zone, which is never fully saturated, whereas baseflow is linked to groundwater levels (Figure 6).

Comparing this pattern with classic catchment hydrology, it is interesting to note that the soil moisture index HU2 (which only reflects the supposed behaviour of the soil cover) doesn't influence the runoff-runoff relationships in our case, even for events for which the saturation state of the aquifer is low. This is finally consistent with the fact that, with lateral contributions being mainly of underground origin, it is the initial antecedent saturation of the aquifer that is the best indicator of the saturation state of the catchment. This result reflects the specificities of karst catchments as compared to other types of catchment. Most production functions in hydrological models are designed to consider the role of soil moisture (e.g. Horton, 1933; Philip, 1957; SCS, 1972; Morel-Seytoux, 1978). Our results show, however, that such models cannot be generalized for carbonate basins with significant SW/GW interactions, when neglecting deep infiltration and groundwater storage in the bedrock.

5.4 Implications for flood forecasting

On the basis of our results and the available data, several insights can be proposed for reliable flood forecasting in permeable basins including karst as well as more generally ephemeral streams. As a karst aquifer is a complex hydrogeological medium, an analysis of its hydrogeological behaviour and its role in runoff at the basin scale is an essential prerequisite. Our first recommendation is not to neglect the influence of hydraulics (diffusivity) on flood routing. For example, floodplains or karst canyons promote meandering networks that are supposed to be an exacerbating factor in wave diffusion. Knowing this, the main risk in forecasting is thus to significantly over-estimate peakflow when applying non diffusive flood-routing models. The second recommendation is to account for river loss in the modelling approach if flood analysis shows significant outflow. The relationship we propose between input peakflow and lateral flow intensity can serve as a basis for such an infiltration function to be implemented in a model. The third recommendation is to use groundwater level as an index of basin saturation for initializing hydrological models. This has to be considered in preference to a soil moisture index, which appears inappropriate for such a basin with fast infiltration at depth.

6 Conclusions

We propose a framework for quantifying peakflow attenuation and/or amplification in a river, based on defining lateral flow during floods in the case of a highly permeable basin that favours surface-water/groundwater interactions. The novelty of our research is the use of the inverse problem of the DWE proposed by Moussa (1996) to simulate a lateral flow hydrograph in a river reach draining karst formations (Normandy, France), knowing the hydrographs from both upstream and downstream gauging stations. Application of the model to several flood events of various intensity shows that, despite a high groundwater contribution to the baseflow component, the peakflow was strongly attenuated. Our approach was designed to differentiate between attenuation generated by wave diffusion and that generated by outflow related to river loss and overbank flow.

Our results provide new insight in flood routing processes in a karst context and more generally in permeable basins favouring ephemeral streams and disconnected river-aquifer systems. First, the model restituted the global dynamics of lateral flow, given information on the hydrological processes involved. Second, we could propose a relationship quantifying outflow intensity as a function of peakflow discharge at the upstream gauging station. Based on previous experimental work investigating the hydrological processes at the origin of loss and gain in rivers, we could highlight the importance of river losses and then of overbank flow for highest flood events. Third, as lateral flow is characterized for unsteady-state conditions, the relative contribution of outflow compared to attenuation due to diffusion was characterized for several sets of model parameterization, allowing interpretations according to parameter sensitivity.

In a more global way, our approach deserves to be tested as a diagnostic tool before applying hydrological models for flood forecasting in permeable—karst—basins. The conclusions provided by our model can help modellers in selecting the best tool in terms of hydrological processes to be simulated as well as of parameterization strategy.

Data Availability Statement

The datasets used in this article can be obtained by contacting Jean-Baptiste Charlier (j.charlier@brgm.fr).

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Figure captions

Figure 1: Lateral outflow (a) and inflow (b) during floods; two examples of combined cases are also given (c); dark and light blue colours are used to differentiate water levels before and during the flood, respectively, in surface water and groundwater (dashed lines).

Figure 2: Diffusive wave model on a reach without lateral flow (a), and with uniformly distributed lateral flows along a channel reach according to two cases: gains (b) and losses (c). The black curve $Q_{I\,f}$ depicts the input hydrograph, and the blue curve $Q_{O\,f}$ the output one at the end of the reach. The direct approach of the DWE is used to route $Q_{I\,f}$ at the end of the reach without lateral exchanges $Q_{I\,f\,r}$ (dashed grey curve). In the case of lateral flows, the inverse approach is used to simulate lateral flow $Q_{A\,f}$ (dotted red curve), which is positive for lateral inflow into the reach (b), or negative for lateral outflow from the reach (c). Terms E, E_D and E_A are attenuation and/or amplification terms explained in the text (Section 2.3).

Figure 3: Sensitivity analysis of the inverse problem of the DWE to simulate lateral flow ($Q_{A\,f}$ - red dotted line) for various parameterization sets of C (in m/s) and D (in m²/s). The analysis is based on two theoretical mono-peak flood events used as input ($Q_{I\,f}$ - black curve) and output ($Q_{O\,f}$ - blue curve) on a 500-m-long reach and at a computed time step of 120 s. For gaining and losing reaches, respectively, a) and b) show the effect of varying C and fixed D; c) and d) the effect of varying D and fixed C; and e) and f) the effect of varying D and calibrating C so that peakflow time of the routed input ($Q_{I\,f\,r}$ – dashed grey line) is in phase with the output ($Q_{O\,f}$ - blue curve) one.

Figure 4: a) Location of the Iton River in France (karst aquifers map from Chen et al., 2017); b) Hydrogeological map of the Iton basin, and c) Scheme illustrating the main lateral surface flows (light blue colour) and lateral groundwater flows (dark blue colour) along the karst reach of the Iton River (adapted from Charlier et al., 2015a; David et al., 2016).

Figure 5: Daily input $Q_I(t)$ (a) and output $Q_O(t)$ (b) time series for the 2000-2001 hydrological cycle along the reach delimited by the two gauging stations at Bourth and Normanville. (c) Input-output relationships for the base component Q_b (grey squares) and flood component Q_f (red circles) are shown for a 2-day delay, corresponding to the mean peakflow delay.

Figure 6: Baseflow at the Normanville gauging station *vs*. groundwater level at daily time steps for six piezometer wells in the Iton catchment, showing a best correlation for Coulonges piezometer (right bottom).

Figure 7: Effect of (a) initial soil humidity and (b) of initial karst saturation on the input-output peakflow relationships (Qx_O *vs.* Qx_I, respectively). Soil saturation is expressed from the HU2 index (n.d.) and karst saturation from the groundwater depth ‘z’ below ground level (m BGL) in the Coulonge piezometer well; circle size is proportional to the initial saturation value. The discharge threshold for overbank flow is indicated for each station.

Figure 8: Hydrological time series and simulated lateral flow (C = 0.35 m/s and D = 500 m²/s) during the flood event of 06 January 2001. From top to bottom, rainfall P (at input station P_I and for lateral catchment P_A), soil humidity index HU2, groundwater depth in the Coulonges piezometer well, discharge for the flood component, and total discharge.

Figure 9: Boxplot of the C parameter calibrated for various diffusivity D values (n=33 flood events)

Figure 10: Outflow intensity ($Qn_{A\ f}$) *vs*. input peakflow ($Qx_{I\ f}$) of the flood components, for different diffusivity values.

Figure 11: Peakflow attenuation generated by diffusion E_D *vs*. peakflow amplification or attenuation generated by lateral exchanges E_A. Positive E_A values indicate amplification due to lateral inflow, but compensated by a highest attenuation due to diffusion E_D when circles are below the E_D=-E_A line.

Appendix A: List of symbols

Symbols	Dimension	Definitions
*	-	convolution operator
C	$[L.T^{-1}]$	flood wave celerity
D	$[L^2.T^{-1}]$	flood wave diffusivity
E	$[L^3.T^{-1}]$	Difference of peakflows $Qx_{I\,f}$ - $Qx_{O\,f}$
E_A, E_D	$[L^3.T^{-1}]$	Difference of peakflows linked to the lateral flows, and to the hydraulic properties of the channel, respectively
$HU2$	-	Soil humidity index
I	-	Input station
K	-	Hayami kernel function
l	$[L]$	length of the channel
O	-	Output station
P	$[L]$	total rainfall
p	$[T]$	time memory of the system
q	$[L^2.T^{-1}]$	lateral flow per unit length
Q, Q_b, Q_f	$[L^3.T^{-1}]$	discharge, base, and flood components of discharge
$\bar{Q}$	$[L^3.T^{-1}]$	mean flow discharge for a rectangular section
Q_I, $Q_{I\,b}$, $Q_{I\,f}$	$[L^3.T^{-1}]$	discharge, base, and flood components at the input station I, respectively
$Q_{I\,f\,r}$, $Q_{I\,r}$	$[L^3.T^{-1}]$	routed $Q_{I\,f}$, and routed Q_I, respectively
$Qn_{A\,f}$	$[L^3.T^{-1}]$	maximum intensity of lateral outflow
Q_O, $Q_{O\,b}$, $Q_{O\,f}$	$[L^3.T^{-1}]$	discharge, base, and flood components at the output station O, respectively
Q_A, $Q_{A\,b}$, $Q_{A\,f}$, $Q_{A\,f\,r}$	$[L^3.T^{-1}]$	discharge, base, and flood components of lateral exchanges, respectively
$Q_{A\,f\,r}$	$[L^3.T^{-1}]$	routed $Q_{A\,f}$,
$Qx_{A\,f}$, Qx_I, $Qx_{I\,f}$, $Qx_{I\,f\,r}$, Qx_O, $Qx_{O\,f}$	$[L^3.T^{-1}]$	peakflow of $Q_{A\,f}$, Q_I, $Q_{I\,f}$, $Q_{I\,f\,r}$, Q_O, and of $Q_{O\,f}$, respectively
t	$[T]$	time
x	$[L]$	length along the channel
z	$[L]$	Groundwater depth
λ	$[L]$	time
ϕ	-	function related to C and $Q_{A\,f\,r}$
τ	$[T]$	time