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1 **Quantifying peakflow attenuation/amplification in a karst river using**
2 **the diffusive wave model with lateral flow**

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16 **Key words**

17 Flood; Lateral flow; Karst, Diffusive wave model; Peakflow attenuation or amplification;

18 Surface water / groundwater interactions

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20

21 **Abstract**

22 The aim of this paper is to quantify peakflow attenuation and/or amplification in a river,
23 investigating lateral flow from the intermediate catchment during floods. This is a challenge
24 for the study of the hydrological response of permeable/intermittent streams, and our
25 contribution refers to a modelling framework based on the inverse problem for the diffusive
26 wave model applied in a karst catchment. Knowing the upstream and downstream
27 hydrographs on a reach between two stations, we can model the lateral one, given information
28 on the hydrological processes involved in the intermediate catchment. The model is applied to
29 33 flood events in the karst reach of the Iton River in French Normandy where peakflow
30 attenuation is observed. The monitored zone consists of a succession of losing and gaining
31 reaches controlled by strong surface-water / groundwater (SW/GW) interactions. Our results
32 show that, despite a high baseflow increase in the reach, peakflow is attenuated. Model
33 application shows that the intensity of lateral outflow for the flood component is linked to
34 upstream discharge. A combination of river loss and overbank flow for highest floods is
35 proposed for explaining the relationships. Our approach differentiates the role of outflow
36 (river loss and overbank flow) and that of wave diffusion on peakflow attenuation. Based on
37 several sets of model parameterization, diffusion is the main attenuation process for most
38 cases, despite high river losses of up to several m^3/s (half of peakflow for some
39 parameterization strategies). Finally, this framework gives new insight into the SW/GW
40 interactions during floods in karst basins, and more globally in basins characterized by
41 disconnected river-aquifer systems.

42

43 **1 Introduction**

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

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

68 simple typology can mask a greater complexity of lateral flows. In most cases, several
69 processes are spatially and temporally combined, leading to mixed (and maybe compensated)
70 out- and in-flows during flooding, as shown on Figure 1c.

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

81 When carbonate formations are highly karstified, which can be the case in chalk formations,
82 groundwaters contribute to the fast-flow component in streams (Maréchal et al., 2008;
83 Charlier et al., 2015b), playing a significant role in flooding, even for flash floods. In karst
84 basins, flooding may also occur by the cessation of rainfall infiltration because of the small
85 retention capacity of a karst massif (Maréchal et al. 2008; Fleury et al. 2013), or because the
86 infiltration capacity of the underground conduit network is exceeded (Lopez-Chicano et al.
87 2002; Bonacci et al. 2006; Bailly-Comte et al. 2009). As chalk aquifers are often karstified,
88 specific types of groundwater flooding can occur seasonally, as is reported from Ireland in
89 low-lying topographic depressions, known as turloughs, that are fed by underground flow
90 from karst aquifers (Naughton et al., 2012; Gill et al., 2013).

91 Another frequent feature of flood generation in karst basins is a contrasted evolution of
92 peakflow, i.e. amplification or attenuation, in a same reach depending upon the prior aquifer

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

105 Many modelling methods exist for investigating SW/GW interactions on groundwater
106 flooding. As the hydrogeological context of chalk aquifers can be favourable for applying
107 physically-based models, lumped approaches (Pinault et al., 2005; Upton & Jackson, 2011)
108 were completed by distributed models, such as MODCOU (Korkmaz et al., 2009), SIM
109 (Habets et al., 2010), MIKE SHE (House et al., 2016), or MARTHE (Thiéry et al., 2018).
110 However, the specifics of karst basins with their high degree of complexity prevent an
111 efficient application of such distributed models, and few such modelling approaches have
112 been used for characterizing SW/GW interactions during flood events in karst basins. To our
113 knowledge, the only published work on the application of flood-routing models to channel
114 reaches, is based on the simplified Saint-Venant equation that describes unsteady flow in
115 partially filled channels; a first modelling approach for karst areas was proposed by Bailly-
116 Comte et al. (2012), using the Kinematic Wave Equation coupled with a linear underground
117 reservoir for simulating lateral inflow. Recently, Charlier et al. (2015b) tested the relevance of

118 the Diffusive Wave Equation (DWE) for assessing lateral in- and outflows in karst rivers.
119 DWE is more adapted to floods with large wavelengths, common in medium and large
120 catchments (Ponce, 1990). Moreover, Moussa (1996) extended the analytical solution of the
121 DWE under the Hayami (1951) hypothesis (Moussa & Bocquillon, 1996b) to the case of
122 uniformly distributed lateral flow. The model can calculate the temporal distribution of lateral
123 flow in a river reach by an inverse problem approach, using as input the flow in both the
124 upstream and downstream gauging stations. The solution of the inverse problem proposed is
125 part of the hydrological MHYDAS model (Distributed Hydrological Modelling of
126 AgroSystems ; Moussa et al., 2002). Using this inverse problem for the DWE we can simulate
127 the global dynamics of lateral flow during floods, which provides information on the
128 hydrological processes involved, such as tracking loss and gain reaches in rivers (Charlier et
129 al., 2015b), or characterizing matrix/conduit relationships through the underground network
130 of a karst aquifer (Cholet et al., 2017); both these publications showed promises in the ability
131 of such models to quantify lateral flow in a karst basin.

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

142 2 Modelling approach

143 2.1 The diffusive wave model

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

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

149 where x [L] is the length along the channel, t [T] is the time, and the celerity C [L.T⁻¹] and
150 the diffusivity D [L² T⁻¹] are functions of the discharge Q [L³ T⁻¹]. In this case, we have a
151 simple two-parameters (C, D) model.

152 At the scale of a river reach – delimited by an input I and an output O station – the model is
153 applied to the flood component ($Q_{I_f}(t)$ and $Q_{O_f}(t)$) of the total discharge ($Q_f(t)$ and $Q_o(t)$)
154 recorded at the input and output stations, respectively. $Q_{I_f}(t)$ and $Q_{O_f}(t)$ are estimated by
155 removing the baseflow components $Q_{I_b}(t)$ and $Q_{O_b}(t)$ from $Q_f(t)$ and $Q_o(t)$.

156 The resolution of Eq. 1 is obtained using the convolution approach proposed by Moussa
157 (1996) based on the Hayami assumptions (Hayami, 1951), i.e. considering C and D as
158 constant parameters over time along a channel of length l :

159 Eq. 2:
$$Q_{I_f r}(t) = \int_0^p Q_{I_f}(t - \tau) K(\tau) d\tau = Q_{I_f}(t) * K(t)$$

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

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

165 Eq. 2 is then used in a direct approach to define the C and D parameters, comparing $Q_{I_f,r}(t)$
 166 with $Q_{O_f}(t)$. In a theoretical case without lateral flows, under diffusive wave and Hayami
 167 assumptions, the routed input hydrograph is equal to the observed output one ($Q_{I_f,r}(t) = Q_{O_f}(t)$)
 168 as shown on Figure 2a.

169 **2.2 The diffusive wave model with lateral flows**

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

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

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

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

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

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

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

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

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

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

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

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

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

200 with

201 Eq. 11
$$K^i(t) = K * K * \dots * K \quad (i \text{ times})$$

202 $Q_{A f}(t)$ can be easily calculated using Eq. 12, after the identification of $K(t)$ in Eq. 3:

203 Eq. 12
$$Q_{A f}(t) = \frac{l}{c} \frac{d\Phi}{dt}$$

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

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

214 **2.3 Quantification of the peakflow amplification/attenuation**

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

$$222 \text{ Eq. 13} \quad E = Q_{x_{O f}} - Q_{x_{I f}} = E_D + E_A$$

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

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

$$229 \text{ Eq. 14} \quad E_D = Q_{x_{I f r}} - Q_{x_{I f}} \quad \text{with } E_D \leq 0$$

230 with $Q_{x_{Ifr}}$, the peakflow of the routed input hydrograph $Q_{Ifr}(t)$ without lateral flow (dashed
231 grey curve). Lateral flow is responsible for an attenuation/amplification expressed as:

232 Eq. 15
$$E_A = Q_{x_{Of}} - Q_{x_{Ifr}}$$

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

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

238 **2.4 Sensitivity analysis on a virtual example**

239 To illustrate the model behaviour and its calibration, various parameterization sets of C and D
240 parameters were used to apply the inverse model on the same couple of theoretical
241 hydrographs Q_{If} and Q_{Of} (Figure 3). Figure 3a and 3b present the simulations carried out by
242 fixing D and varying C . In the case of a gaining reach (Figure 3a), the results show that the
243 highest lateral peakflow is simulated for the lower C . When increasing C , lateral peakflow
244 decreased and became constant when C exceeded a threshold (here $C > 0.4$ m/s) at the same
245 time that outflow was simulated at the start of the flood event. This illustrates a dynamics of
246 compensation of out- and inflows during the same flood event due to a conservation of the
247 flood volume (total lateral flood volume was equal for all simulation tests). The model
248 behaviour is simpler in the case of a losing reach, because Figure 3b shows that the higher the
249 C , the higher and the earlier will be the lateral outflow peak. Figure 3c and 3d show a similar
250 test, but now fixing C and varying D . In the case of lateral inflow (Figure 3c), the results
251 showed that the higher the D , the higher will be lateral peakflow. Similar to Figure 3a,
252 temporal lateral outflow is simulated at the beginning of the flood when D increases. In the
253 case of a losing reach (Figure 3d), the higher the D , the higher and the earlier will be the

254 lateral outflow peak. Globally, this sensitivity analysis shows that various lateral hydrographs
255 are simulated for different couples of C and D parameters, due to equifinality in the modelling
256 approach. As observed in previous studies (Moussa & Bocquillon, 1996a; Yu et al., 2000;
257 Charlier et al., 2009; Cholet et al., 2017), C is more sensitive than D , as a variation of C by 4
258 in our test case generated a same range of lateral peakflow variations as a variation of D by
259 10,000.

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

272 **2.5 Framework of the modelling approach used**

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

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

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

292 **3 Case study**

293 **3.1 Field site**

294 **3.1.1 Basin presentation**

295 The Iton basin is located in Normandy, north-west France (Figure 4a). Land use consists
296 mainly in cereal crops and grassland, and the only important urban area is Evreux (100,000
297 inhabitants) on the Iton in the downstream part of the basin. The topographic catchment is
298 1050 km² at the Normanville gauging station, 7 km downstream Evreux city (Figure 4b).

299 **3.1.2 Climate**

300 The climate is of the humid temperate oceanic type. Annual rainfall ranges between 500 and
301 1000 mm, with an inter-annual average of 600 to 715 mm between the upstream and
302 downstream areas of the catchment, respectively. The intra-monthly variations are relatively

303 low, with slightly wetter months in autumn (60 to 80 mm/mo) compared to other periods of
304 the year (40 to 65 mm/mo), but in general rainfall is quite regularly distributed throughout the
305 year. The region has also been characterized in the past by exceptional rainfall in 2000-2001
306 (about 100 mm of rainfall depth in 2 to 4 days only), generating catastrophic flood events
307 enhanced probably by two previous years of accumulated wetness (Pinault et al., 2005).

308 **3.1.3 Geology and hydrogeology**

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

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

324 **3.1.4 Conceptual model of lateral flow**

325 The conceptual model presented in Figure 4c is the result of hydrogeological and hydrological
326 studies, highlighting surface-water / groundwater (SW/GW) interactions of various origin
327 (Charlier et al, 2015a; David et al., 2016). The main horizontal line represents the Iton River

328 for the 75-km reach between the two gauging stations Bourth and Normanville (input and
329 output in Figure 4c; see Figure 4b for location). Surface flow is in light blue colour and
330 groundwater flow in dark blue. The dashed line represents ephemeral streams due to river
331 losses. For surface flow, the main properties of the Iton are: i) Drying-up of the drainage
332 network (as well as the Rouloir) where it crosses the karst zone; and ii) Contribution of the
333 main tributary (Rouloir) and of the two groups of springs near the confluence. Groundwater
334 flow is composed of infiltrated river losses as well as aquifer contributions via several springs.
335 Finally, in this conceptual scheme of SW/GW interactions in the Iton basin in its karst part,
336 lateral flow is defined by outflow from river losses and inflow from groundwater origin.

337 **3.2 Data**

338 **3.2.1 Hydrological and hydrogeological time series**

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

349 **3.2.2 Flood selection and processing for model application**

350 For flood event analysis, the highest 33 peakflows at the Bourth gauging station (Input
351 station) were selected from the dataset (Table 1). Rainfall events ranged between 16 and
352 100 mm, and peakflows between 9 and 26 m³/s at the Input station and 5 to 17 m³/s at the

353 Output one. Minimum discharge-inducing overbank flow is 14 and 10 m³/s for the Bourth and
354 Normanville stations, respectively. These thresholds correspond to discharge generating flows
355 in the flood plain without rapid return towards the channel. Table 1 shows that the 10 highest
356 flood events were partially subject to overbank flow.

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

361 **4 Results**

362 **4.1 Groundwater influence on surface flow**

363 **4.1.1 Base and flood components**

364 An example of the base and flood flow separation is given in Figure 5 for the 2000-2001
365 hydrological cycle at the Bourth Input station in black (Figure 5a) and the Normanville
366 Output station in blue (Figure 5b). The input-output relationships for the base and flood
367 components account for a 2-day delay (Figure 5c), corresponding to the mean delay of
368 peakflows. It shows a contrasted behaviour: the baseflow increases 3 to 4 fold from input to
369 output station, whereas flood flow decreases by several m³/s for the highest flood events. This
370 means that lateral groundwater inflow contributed highly to stream flow for the base
371 component, at the same time that strong lateral outflow occurred for the flood component.

372 **4.1.2 Baseflow analysis**

373 Figure 6 shows the relationships between baseflow calculated at the output gauging station
374 and groundwater levels for six piezometer wells. We observe a slight to fair correlation for
375 wells located on the plateau, having mainly multi-year cycles (Normanville and Cierrey
376 piezometers Figure 4b for location). Best correlations are obtained for wells with annual
377 cycles at Nogent-le-Sec, Moisville, and Coulonges (best linear correlation with R² >0.7). The

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

381 **4.1.3 Flood analysis**

382 The input-output relationships for peakflow are plotted in Figure 7 according to two factors
383 used as key indices of the catchment saturation level: the soil-humidity index (HU2, Figure
384 7a) and the karst saturation index (GW depth z in the Coulonges well; Figure 7b). Before
385 assessing the effect of such factors, it is interesting to observe that output peakflow is always
386 less than, or equal to, the input one. A peakflow attenuation is also observed for the highest
387 flood events when $Q_{xI} > 15 \text{ m}^3/\text{s}$. This value corresponds to the threshold of overbank flow at
388 the input station, meaning that this process may explain part of the attenuation of the highest
389 flood events. In the first case, HU2 seems not to be a discriminant factor for explaining the
390 data variability, as most events are characterized by indices close to the saturation level (i.e.
391 HU2 ~ 60 , HU2 ranging between 40 and 65). In the second case, the initial GW depth seems
392 to explain the attenuation variation for a given input peakflow: the higher the initial GW
393 depth, the higher will be output peakflow. Globally, this analysis shows that the soil saturation
394 index is not a suitable factor for differentiating flood intensity. On the contrary, these results
395 show that the peakflow attenuation is related to the antecedent groundwater level.

396 **4.2 Simulation of a lateral flood hydrograph**

397 **4.2.1 Model application to a mono-peak flood event**

398 Figure 8 shows an example of the DW model application for the mono-peak flood event of
399 06/01/2001 with the optimized parameters $C = 0.35 \text{ m/s}$ and $D = 500 \text{ m}^2/\text{s}$. From top to
400 bottom rainfall, soil saturation index (HU2), groundwater depth in the Coulonges well,
401 discharge for the flood component, and total discharge are shown. On each discharge plot,
402 four hydrographs correspond to the observed input hydrograph (Q_{If} and Q_I ; black curve), the

403 observed output hydrograph (Q_{Of} and Q_O ; blue curve), the routed input hydrograph (Q_{Ifr} and
404 Q_{Ir} ; dashed grey curve) using the direct model without lateral flows, and the simulated lateral
405 hydrograph (Q_{Af} and Q_A ; dotted red curve) using the inverse model. Both soil saturation
406 index (HU2 >58) and groundwater levels (GW >-15 m AGL) were saturated before the
407 beginning of the flood. The discharge analysis shows a strong attenuation of the flood
408 hydrograph along the reach, halving the peakflow from 26.8 m³/s (Q_{xI}) to 13.7 m³/s (Q_{xO}).
409 Before the flood, lateral inflow ($Q_A(t)$) was 3.7 m³/s; but during the flood it became negative.
410 This may be interpreted as a continuous contribution of lateral baseflow hidden by occasional
411 high losses in the flood component. Using the minimum values of the lateral flood
412 component, we can quantify the maximum intensity of lateral outflow Q_{nAf} as -6.8 m³/s. The
413 peakflow attenuation due to outflow (losses+overbank flow) E_A is -6.6 m³/s and the
414 attenuation due to diffusion E_D is -9.2 m³/s, leading to a total peakflow attenuation E of -
415 15.8 m³/s.

416 Analysis of this single flood event shows that, despite lateral baseflow, the outflow associated
417 to the flood component can be quantified. As outflow by overbank flow may occur during the
418 highest discharge, outflow by river losses is probably continuous as long as the stream flows
419 at the input station. This suggests that losses are compensated by highest baseflow discharge
420 during recession periods, leading to under-estimating their real values. The other interesting
421 point is that we can compare peakflow attenuation by hydraulic processes (diffusion) and by
422 outflow (river loss+overbank flow), quantifying it (for a given parameterization set) equal to
423 57% and 43%, respectively.

424 **4.2.2 Distribution of model parameters**

425 Following the above example, the model was applied to the 33 main flood events (Table 1),
426 using the parameterization strategy presented in Section 2.5. Several values of D were
427 selected for optimizing the C parameter, for inputting phase peakflows of the routed input

428 hydrograph (Q_{I_r}) and of the output one (Q_O). Figure 9 shows C distribution using boxplots for
429 five D values of 500, 1000, 2500, 5000, and 10,000 m^2/s , which is the classic range for
430 streams and rivers (Todini, 1996), such as in our study, knowing that D increases with the size
431 of the river. Boxplot analysis shows that C values range between 0.1 and 0.4 m/s with a
432 relative small variability for a given D . The higher the D , the lower will be the range of C
433 values from 0.35 to 0.11 m/s . As shown in the sensitivity analysis (Section 2.4 above), a
434 lower limit of C values to almost 0.15 m/s is observed for D values above 2500 m^2/s .
435 Knowing that lateral flows are highly sensitive to C values and less so to D values, various
436 parameterization sets will be tested in the following section.

437 **4.3 Quantification of peakflow attenuation**

438 **4.3.1 Assessment of lateral outflow**

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

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

462 4.3.2 Factors influencing peakflow attenuation

463 Two phenomena participate in peakflow attenuation (E): hydraulic processes due to flood
464 wave diffusivity (E_D) and hydrological processes due to outflow (E_A) (cf. Eq. 13). To quantify
465 their respective roles, Figure 11 presents the E_A vs. E_D relationships for the same five
466 parameterization strategies as in Figure 10. We see that E_D ranges between -18.0 to $-0.5 \text{ m}^3/\text{s}$
467 whereas E_A ranges between -9 to $4.0 \text{ m}^3/\text{s}$. As expected, the higher the D (light blue colour),
468 the higher the $|E_D|$ values (negative in the graph since E_D is inevitably an attenuation of
469 peakflow). Except for cases with the lowest D values ($D=500 \text{ m}^2/\text{s}$), and some cases with
470 $D=1000 \text{ m}^2/\text{s}$ (dark blue colour, Figure 11), the points lie below the $E_D=E_A$ line, i.e.
471 attenuation due to diffusivity is often higher than that due to outflow. It is interesting to note
472 that when D is high and thus $|E_D|$ is high, E_A is positive (lateral flow became inflow) while
473 being below the $E_D=-E_A$ line. This means that in these cases, despite flood amplification due
474 to lateral inflow, peakflow attenuation is finally observed because the effect of high
475 diffusivity overtakes it.

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

478 D , calculated with the following formula proposed by Chow (1959) that considers simple
479 network descriptors: $D = \bar{Q} / (2 \times slope \times width)$, where \bar{Q} is the mean flow discharge for a
480 rectangular section. Varying the mean slope of the river from 0.0010 to 0.0015, the mean river
481 width from 5 to 10 m, and \bar{Q} from 5 to 20 m³/s, D ranges between 250 and 2000 m²/s, over a
482 quite small range of values compared to the tested ones (500 to 10,000 m²/s). In this
483 theoretical case, E_D and E_A are roughly equal according to Figure 11, but this result should be
484 taken with care due to the high spatial variability of channel properties along the Iton River.

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

490 **5 Discussion**

491 **5.1 On the interest of using a diffusive wave model to assess peakflow attenuation** 492 **and/or amplification**

493 Although aquifer's recharge by river losses can attenuate floods in arid/Mediterranean
494 environment (Sorman et al., 1997; Hughes & Sami, 1992; Lange, 2005; Dahan et al., 2007;
495 Vázquez- Suñé et al., 2007) or in karst basins (Jourde et al., 2013; Charlier et al., 2015b;
496 Brunet and Bouvier, 2017), we cannot neglect hydraulic diffusion processes when we are
497 interested in peakflow forecasting. In the case of permeable basins, our results show that,
498 despite the presence of an infiltration zone in the river bed with significant losses (several
499 m³/s), peakflow attenuation is mainly related to diffusion of the flood wave. This raises the
500 question of the mechanisms favouring such attenuation, which may be related to the
501 meandering morphology of the drainage network, as well as to the zones of temporary storage
502 for highest flood events (Moussa & Chevion, 2015).

503 Our example shows the added value of using a diffusive model—combining direct and
504 inverse problem approaches—for better understanding and quantifying SW/GW interactions
505 during floods, and which deserves to be tested on other types of basins where significant
506 lateral losses and/or gains are observed (Martin & Dean, 2001; Ruehl et al., 2006; Payn et al.,
507 2009). For instance, the inverse DWE model has also been used for investigating lateral flow
508 in underground karst conduits, and for defining the exchanges between conduits and the
509 fissured matrix (Cholet et al., 2017). In parallel to these considerations that promote the model
510 as a tool for diagnosing SW/GW exchanges at different scales, our results highlight the
511 inaccuracies that can be generated by using non-diffusive models, as is frequently the case for
512 flood modelling (see review in Singh, 2002) and for karst basins (Bailly-Comte et al., 2012;
513 Dvory et al., 2018). Our results are coherent with Naulin’s work (2012) in the Cévennes
514 region (southern France), who showed that DWE was more suitable in lowland areas,
515 including karst formations, than in mountains with less permeable hard-rock formations.

516 **5.2 Surface-water / groundwater interactions in permeable basins**

517 **5.2.1 River losses and overbank flow**

518 The relationship between lateral outflow and input peakflow has established a function of
519 river losses that improves the understanding of SW/GW interactions in permeable basins. In
520 fact, below the discharge threshold for overbank flow, outflow is mainly generated by losses
521 that account in our case for several m^3/s during a flood. Although this process is well known
522 in many catchments when studying low water-level periods, it is generally not considered
523 during flood events, because inflow conceals it. Despite some works on infiltrating floodwater
524 in basins characterized by disconnected river-aquifer systems (Hughes & Sami, 1992; Lange,
525 2005; Dahan et al., 2007; Vázquez- Suñé et al., 2007), the estimation of infiltration rate
526 during the flood (i.e. at a high temporal resolution) is generally disregarded. Thus, our study

527 brings a relevant approach to help quantify the loss intensities as well as the recharge rate of
528 the underlying aquifers.

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

537 The ceiling of outflow intensity with the increase of discharge into the river is most probably
538 explained by the occurrence of overbank flow, knowing that flood plain attenuation can play a
539 key role on the modification of hydrograph shape (Sholtes & Doyle, 2011; Valentova et al.,
540 2010; Rak et al., 2016; Fleischmann et al., 2018). Indeed in the study case, outflow on the
541 flood plain appears to be an important process of peakflow attenuation for the highest floods,
542 since the increase in infiltration with an increased input peakflow is stopped (Figure 10).
543 Another concept may explain this outflow ceiling: several studies of karst hydrology reported
544 a limitation of infiltration during rainfall events due to small void diameters or constricted
545 conduits at depth (Bonacci, 2001; Bailly-Comte et al., 2009). However, such a process
546 requires specific monitoring that fell outside the scope of our study.

547 **5.2.2 Loss and gain in river reaches**

548 The specific features of the studied river, including both loss and gain reaches, render an
549 analysis of lateral exchanges difficult. The occurrence of both out- and in-flow in some
550 reaches has been conceptualized as hydrologic turnover (Covino et al. 2011; Mallard et al.,
551 2014) in simultaneous loss and gain reaches. This pattern was highlighted both in chalk

552 catchments, where it was found that groundwater flooding consists of a combination of
553 intermittent stream discharge and anomalous springflow (Hughes et al., 2011), and in karst
554 rivers characterized by successive loss and gain reaches from a multi-layered aquifer in deep
555 canyons (Charlier et al., 2015b). Improving the understanding of lateral exchange during
556 floods, our modelling approach opens a novel way to help deciphering the various
557 contributions of loss and gain.

558 **5.3 Saturation state in a karst catchment: soil moisture vs. aquifer storage**

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

570 Comparing this pattern with classic catchment hydrology, it is interesting to note that the soil
571 moisture index HU2 (which only reflects the supposed behaviour of the soil cover) doesn't
572 influence the runoff-runoff relationships in our case, even for events for which the saturation
573 state of the aquifer is low. This is finally consistent with the fact that, with lateral
574 contributions being mainly of underground origin, it is the initial antecedent saturation of the
575 aquifer that is the best indicator of the saturation state of the catchment. This result reflects the
576 specificities of karst catchments as compared to other types of catchment. Most production

577 functions in hydrological models are designed to consider the role of soil moisture (e.g.
578 Horton, 1933; Philip, 1957; SCS, 1972; Morel-Seytoux, 1978). Our results show, however,
579 that such models cannot be generalized for carbonate basins with significant SW/GW
580 interactions, when neglecting deep infiltration and groundwater storage in the bedrock.

581 **5.4 Implications for flood forecasting**

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

597 **6 Conclusions**

598 We propose a framework for quantifying peakflow attenuation and/or amplification in a river,
599 based on defining lateral flow during floods in the case of a highly permeable basin that
600 favours surface-water/groundwater interactions. The novelty of our research is the use of the
601 inverse problem of the DWE proposed by Moussa (1996) to simulate a lateral flow

602 hydrograph in a river reach draining karst formations (Normandy, France), knowing the
603 hydrographs from both upstream and downstream gauging stations. Application of the model
604 to several flood events of various intensity shows that, despite a high groundwater
605 contribution to the baseflow component, the peakflow was strongly attenuated. Our approach
606 was designed to differentiate between attenuation generated by wave diffusion and that
607 generated by outflow related to river loss and overbank flow.

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

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

623 **Data Availability Statement**

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

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632

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856 **Figure captions**

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

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

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

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

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

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

887 Figure 7: Effect of (a) initial soil humidity and (b) of initial karst saturation on the input-
888 output peakflow relationships (Q_{x_O} vs. Q_{x_I} , respectively). Soil saturation is expressed from
889 the HU2 index (n.d.) and karst saturation from the groundwater depth 'z' below ground level
890 (m BGL) in the Coulonge piezometer well; circle size is proportional to the initial saturation
891 value. The discharge threshold for overbank flow is indicated for each station.

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

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

898 Figure 10: Outflow intensity ($Q_{n_A f}$) vs. input peakflow ($Q_{x_I f}$) of the flood components, for
899 different diffusivity values.

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

904 Appendix A: List of symbols

Symbols	Dimensi on	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 $Q_{x_{If}} - Q_{x_{Of}}$
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_{Ib}, Q_{If}	$[L^3.T^{-1}]$	discharge, base, and flood components at the input station I , respectively
Q_{Ifr}, Q_{Ir}	$[L^3.T^{-1}]$	routed Q_{If} and routed Q_i , respectively
$Q_{n_{Af}}$	$[L^3.T^{-1}]$	maximum intensity of lateral outflow
Q_o, Q_{Ob}, Q_{Of}	$[L^3.T^{-1}]$	discharge, base, and flood components at the output station O , respectively
$Q_A, Q_{Ab}, Q_{Af}, Q_{Afr}$	$[L^3.T^{-1}]$	discharge, base, and flood components of lateral exchanges, respectively
Q_{Afr}	$[L^3.T^{-1}]$	routed Q_{Af}
$Q_{x_{Af}}, Q_{x_b}, Q_{x_{If}}, Q_{x_{Ifr}}, Q_{x_o}, Q_{x_{Of}}$	$[L^3.T^{-1}]$	peakflow of $Q_{Af}, Q_i, Q_{If}, Q_{Ifr}, Q_o$, and of Q_{Of} , respectively
t	$[T]$	time
x	$[L]$	length along the channel
z	$[L]$	Groundwater depth
λ	$[L]$	time
ϕ	-	function related to C and Q_{Afr}
τ	$[T]$	time

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