Representation of the urban terrain and its use in quantifying the hydrologic response

Application d'une description morphologique de l'espace urbain pour la modélisation hydrologique

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RÉSUMÉ

Dans les modèles d'assainissement pluvial urbain, la simulation des collecteurs est réalisée par des méthodes hydrauliques alors que les apports des sous-bassins versants sont modélisés par des méthodes hydrologiques qui négligent la présence des petits collecteurs et des caniveaux. Plus précisément, le comportement des sous-bassins est paramétré à partir des MNT qui n'apportent pas d'informations sur les écoulements canalisés. Cet article présente un nouveau modèle, dénommé U-McIUH (Hydrogramme Unitaire Instantané Morpho-Climatique) qui définit l'IUH (Hydrogramme Unitaire Instantané) comme la densité de probabilité des temps de parcours à l'exutoire. Les cheminements de l'eau de tout point du bassin versant à l'exutoire sont obtenus à partir d'un MNT et en prenant en compte les propriétés des surfaces contributives et du réseau artificiel de collecte des eaux pluviales. Le calcul des temps de parcours à l'exutoire fait à appel à la méthode de l'onde cinématique en prenant en compte la contribution amont et l'intensité de la pluie. Le modèle reproduit la réponse à une pluie par une convolution quasi-linéaire des impulsions pluvieuses et de leurs IUHs respectifs. L'application du modèle à un bassin versant instrumenté montre la bonne reproduction des hydrogrammes observés. Les résultats suggèrent que la non-linéarité de la réponse du bassin versant est importante et que l'ajout des éléments artificiels au modèle numérique de terrain joue un rôle majeur dans le calcul de la réponse hydrologique.

ABSTRACT

In urban stormwater models, large pipes and channels are simulated with hydraulic methods, but contributing subcatchments are usually modeled with hydrologic methods that neglect the presence of smaller pipes and streets. Specifically, subcatchments are parameterized from digital elevation models (DEMs) that do not resolve such features. This paper presents a new model called the U-McIUH (Urban Morpho-climatic Instantaneous Unit Hydrograph), which defines the IUH as the probability density function of travel times to the catchment outlet. Flow paths are extracted from a specially processed DEM that includes the properties of contributing surfaces and artificial conduits. This information is utilized to compute travel times using kinematic wave-based expressions that depend on the upstream contribution and the rainfall intensity. The model generates the response to a storm through a quasi-linear convolution of the rainfall pulses and their respective IUH. The application of the model to a real catchment shows a good reproduction of observed hydrographs. Results suggest that the nonlinearity of the catchment response is important and that artificial elements represented in the urban terrain play a major role in determining the hydrologic response.

KEYWORDS

Kinematic wave, Non-linear response; Rainfall-Runoff model, Terrain models; Urban hydrology

1 INTRODUCTION

Stormwater models are valuable tools for the prediction of discharges, the assessment of downstream impacts associated with land-use changes, and the design of various elements of the stormwater system. Most of these models provide a detailed description of the sewer system hydraulics, yet they still rely on simplistic parameterizations of the upstream runoff formation and concentration (Rodriguez et al., 2003). In these models, the watershed is divided into subcatchments in which a hydrologic method is used to accumulate the flow that discharges into the major elements of the drainage system. The subcatchment is often conceptualized as a simple geometric shape or reservoir that neglects the actual spatial configuration, and the conduits or channelized elements contained in each subcatchment (i.e. streets, small pipes, and small streams are neglected). However, Rodriguez et al. (2003 and 2005), Lhomme et al. (2004), and Gironás et al. (2009a) demonstrated that these elements, and the spatial structure of the drainage system, play an important role in directing flow within subcatchments and in generating the hydrologic response of urban subcatchments.

Maidment et al. (1996) explicitly considered the spatial structure of the catchment in the identification of the unit hydrograph (UH). They incorporated the geomorphic features of a rural catchment derived from the Digital Elevation Model (DEM) in the definition of the UH obtained from a spatially-distributed velocity field. Many studies in rural catchments have been developed after this seminal work; however, the use of geomorphologic features to identify the UH in urban catchments has been much less explored. Several studies (e.g., Smith et al., 2005; Javier et al., 2007) have applied the Network Model (Morrison and Smith, 2001), which divides the catchment into hillslope and channel components, each one characterized by a constant flow velocity, that are used to determine the travel time from each cell to the outlet. Melesse and Graham (2004) developed a spatially-distributed travel time method that considers a field of spatially-distributed hillslope and channel velocities. Finally, Rodriguez et al. (2003, 2005) developed a method in which the IUH was obtained by adding the runoff generated by a unit of effective precipitation over the hydrologic elements based on the travel times along the drainage system explicitly defined (streets and pipes).

These previous methods to determine the distributed unit response all include at least one of three significant drawbacks. First, the evidence of non-linearity in the rainfall-runoff response in urban catchments (Hossain et al., 1978) is not typically considered (i.e. the dependence of the unit hydrograph on the rainfall intensity). A second problem is the omission or simplification of flow in pipes, which can be the main component of the drainage network in small to mid-sized urban catchments. Finally, the velocities in hillslope cells are typically assumed to be constant or independent of the amount of overland flow that is entering the cell. Lhomme et al. (2004) successfully addressed this issue by incorporating the upstream area into an empirical expression for the velocity field in an urban catchment. Nonetheless, this aspect should be formally considered in a hydrologic model for urban catchments. Rodriguez et al. (2003, 2005) addressed the first two issues, but some aspects in their formulation can be further improved (i.e. a more flexible parameterization of the nonlinear dependence between the runoff response and the rainfall intensity, and a simpler morphologic characterization of the entire drainage system).

This paper develops an Urban Morpho-climatic Instantaneous Unit Hydrograph (U-McIUH), oriented to overcome these drawbacks. Non-linearity in the rainfall-runoff response is modeled by including the dependence on the rainfall intensity. The different role of surfaces, streets and pipes in the generation of the response is considered through a more realistic representation of the urban terrain. Finally, the effect of upstream contribution on travel times in hillslope cells is formally incorporated through a more physically-based expression. The paper is structured as follows. In the next section the representation of the urban terrain is discussed. Then we present a description of the model and we introduce the study area and the associated dataset. The testing of the model is presented in the section “Performance of the model”. Finally, the last section of the paper presents the main conclusions.

2 REPRESENTATION OF THE URBAN TERRAIN

A detailed description of the flow paths is of great importance in determining the hydrologic response in urban catchments (Lhomme et al., 2004; Rodriguez et al., 2003, 2005, Gironás et al., 2009a). In this study, we use the method proposed by Gironás et al. (2009a) to pre-process urban DEMs for the generation of urban terrains. They showed that this method better reproduces the real flow paths through artificial conduits (streets and pipes) and demonstrated the impacts of this improvement in hydrologic modelling applications. In this method, the streets and pipes are "burned" into the DEM by reducing the elevation of the cells where these artificial conduits are located. Unlike the more common approach in which a constant depth is subtracted, the method uses the local conduit depths when making the elevation adjustments. This approach allows the flow in the conduits to have different directions than the flow on the ground surface. The data required for implementing the method are the raw DEM and vector layers representing conduits (i.e. streets, pipes, and channels) with the planar and vertical coordinates of the upstream and downstream ends of each element. After processing the DEM, the post-processing routines available in Arc Hydro are used to fill pits, compute the local slopes, and generate the flow directions using the direction of steepest descent.

The methodology for the terrain representation also generates a second raster with the classification of each cell in the catchment. Thus, cells representing overland flow (hillslopes) and flow in conduits (streets, pipes, and open channels) can be easily identified. If both a street and a pipe occur in a cell, it is labeled a pipe. Similarly, if a street or a pipe occurs together with a channel, the cell is identified as a channel. Under this approach conduit cells received the totality of the flow coming from upstream, and although representing transport elements, these cells still generate runoff because they have an associated area. The raster file that identifies the cell classifications is linked to other raster files containing the geometrical attributes of the conduits (i.e. slopes and other geometric properties). This approach allows the U-McIUH to use the actual values of these properties when calculating the hydrologic behavior of the conduit in each cell. In the case of a hillslope cell, all the required geometric attributes can be determined directly from the DEM.

3 DESCRIPTION OF THE MODEL

3.1 The morphoclimatic instantaneous unit hydrograph

Rodríguez-Iturbe and Valdés (1979) introduced the Geomorphologic Instantaneous Unit Hydrograph (GIUH) theory, which uses the spatial structure of the basin to identify the IUH. In the theory, the IUH is defined as the PDF of the travel times to the catchment outlet. In this study, we reinterpret the IUH as the PDF of the travel time to the catchment outlet for a perturbation in the flow. An explicit approach can be adopted to define the GIUH, in which the individual flow paths from each location in the catchment to the outlet are defined using a DEM. This approach also facilitates the use of spatially-variable flow velocities in artificial conduits within the subcatchment. Rodríguez-Iturbe et al. (1982) extended the GIUH to the geomorphoclimatic instantaneous unit hydrograph (GcIUH), which incorporates the excess rainfall intensity and duration in the determination of the IUH. This revision was made because flow velocities typically increase for more intense excess rainfall, which results in shorter travel times and a modified IUH. With this addition, the original formulation departed from the linearity assumptions because the IUH becomes variable for different rainfall inputs. Rodriguez et al. (2003 and 2005) observed a strong dependence of the shape of the IUH in urban catchments on the return period of the rainfall event. Therefore, a morphoclimatic approach is more suitable than a purely morphologic approach in identifying the IUH in urban catchments (as suggested by Rodriguez et al. (2005), the term "morphological" is used here instead of "geomorphological" because artificial conduits are a major portion of the urban drainage network). What follows in this section is the description of the model developed for this study, and each one of its components. A detailed description of the model was presented by Gironás et al. (2009b).

3.2 Excess rainfall representation

The U-McIUH depends on the excess rainfall rate. Thus, although this study focuses on the travel times in a subcatchment, a method is needed to generate excess rainfall. Two losses are considered: an initial abstraction (i.e. depression storage and interception) and infiltration. The initial abstraction IA is constant over the catchment and must be filled completely prior to the occurrence of infiltration. The infiltration capacity is represented by a simple model in which the initial and final infiltration rates are the same and equal to a single infiltration rate f_o. Because the U-McIUH will be applied to individual events, f_o is represented as $f_o = f_c/C$, where f_c is an infiltration capacity (a property of the soil) and C is an event-dependent calibration parameter that is used to account for the antecedent moisture.

The impervious area is also included in this simple model for excess rainfall. In a cell j of area A_j, the imperviousness H_j is defined as the ratio A_j^I / A_j, where A_j^I is the impervious area of the cell. It is assumed that all rainfall occurring on the impervious area becomes runoff. Thus, after the initial abstraction occurs, the excess rainfall intensity produced by the cell *j* at a time *i* ($E_{j,i}$) is computed as:

$$E_{j,i} = \begin{cases} I_i H_j & \text{if } I_i \le f_c / C \\ \left(1 - H_j\right)\left(I_i - \dfrac{f_c}{C}\right) + I_i H_j & \text{if } I_i > f_c / C \end{cases} \tag{1}$$

where I_i is the rainfall intensity. $E_{j,i}$ is computed for all the cells in the urban terrain including those classified as conduits, and this spatially distributed field is used to generate the different IUHs.

3.3 Overland flow

Overland flow is simulated using a travel time for each cell that is derived from kinematic wave theory, widely used in small to mid-size urban drainage systems (Overton and Meadows, 1976; Singh, 2001). For hillslope cells, it is assumed that the equilibrium in each individual cell is always reached before the end of a rain pulse. This approximation has been widely used to represent overland flow and travel time in hillslope cells (e.g. Muzik, 1996; Melesse and Graham, 2004).

In a typical basin, many hillslope cells are linked together with flow being transferred from one to the other. This connectivity is addressed by using an expression for the time of equilibrium that depends on the precipitation over the cell and the upstream contribution of flow. Using kinematic wave theory, Wong (1995) derived an expression for the wave-celerity-based travel time τ for a rectangular plane subjected to a uniform excess rainfall pulse of intensity E and with a constant upstream inflow per unit width q_u. This expression, which is also an expression for the time to equilibrium, can be written:

$$\tau = \left(\frac{\ell E^{1-\beta_h}}{\alpha_h} \right)^{1/\beta_h} \left[(\lambda + 1)^{1/\beta_h} - \lambda^{1/\beta_h} \right] \tag{2}$$

where ℓ is the flow length across the plane and $\lambda \equiv q_u / (E\ell)$ is a dimensionless inflow relating q_u and the excess rainfall produced per unit width. α_h and β_h are parameters that relate the discharge per unit width $q = E\ell + q_u$ to the flow depth y, so that $q = \alpha_h y^{\beta_h}$. Using Manning's equation, the discharge-depth parameters are $\alpha_h = S^{0.5}/n$ and $\beta_h = 5/3$, where S is the slope of the plane and n is Manning's resistance coefficient. Eq. (2) becomes:

$$\tau = 6.99 \left(\frac{n\ell}{\sqrt{S}} \right)^{0.6} E^{-0.4} \left[(\lambda + 1)^{0.6} - \lambda^{0.6} \right] \tag{3}$$

where τ is in min, S in m m^{-1}, ℓ in m, and E in mm h^{-1}.

If the upstream inflow to the plane is zero ($q_u = \lambda = 0$), Eq. (3) reduces to the traditional kinematic wave expression of τ for a rectangular plane of length ℓ and slope S (Viessman and Lewis, 1995), which does not consider the effects of upstream flow on the computation of travel times. Such a expression (or similar) has been used in previous DEM-based models for urban areas (e.g. Melesse and Graham, 2004; Kute and Stuart, 2008). The effect of the upstream contribution on the travel time of a hillslope cell was evaluated in detail by Gironás et al. (2009b).

Generalizing the nomenclature to allow variations in time and between cells, we obtain $\tau_{j,i}$ the travel time at a hillslope cell *j* for an excess rainfall pulse $E_{j,i}$ at time *i* as:

$$\tau_{j,i} = 6.99 \left(\frac{n_j \ell_j}{\sqrt{S_j}} \right)^{0.6} E_{j,i}^{-0.4} \left[(\lambda_j + 1)^{0.6} - \lambda_j^{\,0.6} \right] \tag{4}$$

where *j* is an index of cells and *i* is an index of times, so that the only term with temporal variability considered in the formulation is the excess rainfall. In this expression, the slope S_j and the flow length ℓ_j in the grid cell *j* are obtained from the processed urban terrain, n_j is assumed to be a function of the landuse, and $E_{j,i}$ is obtained using Eq. (1). $\ell_j = \Delta x$ (the grid resolution) when the downstream flow is vertical or horizontal, and $\ell_j = \Delta x\sqrt{2}$ when the flow is in the diagonal direction. In Eq. (4), we have assumed that λ_j is constant in time. In reality, this dimensionless ratio would change as the relative contributions of upstream flow and local flow change. To simplify the method, we use a temporally invariant "effective" value of λ_j by first assuming that precipitation is homogeneous over the catchment. In this case, the upstream discharge is dependent on the contributing area (Rodríguez-Iturbe et al., 1992). In addition, because impervious areas play a major role in the generation of runoff in urban areas, particularly for frequent storms, we neglect the contribution of flow from the pervious areas. Note that these assumptions are only applied for the purpose of obtaining the value of λ_j. Under these assumptions, λ_j is the ratio between the total impervious area contributing to cell *j* and the impervious area of the cell itself:

$$\lambda_j = \frac{H_{u,j}A_{u,j}}{H_j A_j} \tag{5}$$

where $A_{u,j}$ is the contributing area for cell *j*, $H_{u,j}$ is the total imperviousness of the contributing area $A_{u,j}$, and H_j is the imperviousness of cell *j*. The final expression for travel time in hillslope cells, $\tau_{j,i}$, is obtained by substituting Eq. (5) in Eq. (4):

$$\tau_{j,i} = 6.99\left(\frac{n_j \ell_j}{\sqrt{S_j}}\right)^{0.6} E_{j,i}^{-0.4}\left[\left(\frac{H_{u,j}A_{u,j}}{H_j A_j}+1\right)^{0.6} - \left(\frac{H_{u,j}A_{u,j}}{H_j A_j}\right)^{0.6}\right] \tag{6}$$

The units of Eq. (6) are the same as those in Eq. (3), and the areas A_j and $A_{u,j}$ are defined in m^2. This expression depends on *E*, so the climatic dependence of the IUH identified by Rodríguez-Iturbe et al. (1982) is partially incorporated for the hillslope cells. In this respect, the Mc-IUH relaxes the linearity assumption used in UH theory.

3.4 Conduit flow

Travel times in cells representing conduits are also computed using kinematic wave theory. We consider three types of conduit elements: streets (represented as gutters), pipes, and channels. Wong (2003) derived the following expression for the wave's travel time t_L in a conduit of length *L*:

$$t_L = \left(\frac{L}{\alpha_c q_L^{\beta_c - 1}}\right)^{1/\beta_c}\left[(\lambda_c + 1)^{1/\beta_c} - \lambda_c^{1/\beta_c}\right] \tag{7}$$

where α_c and β_c are parameters relating the discharge in the conduit *Q* to the cross-sectional flow area A_c in the expression $Q = \alpha_c A_c^{\beta_c}$. λ_c is a dimensionless ratio relating the upstream inflow Q_u to the uniformly distributed lateral inflow q_L:

$$\lambda_c \equiv \frac{Q_u}{q_L L} \tag{8}$$

In order to adapt Eq. (7) to a grid cell formulation, we need to estimate the conduit length that should be associated with each conduit cell. To avoid the overestimation of conduits lengths obtained from the grid resolution, Gironás et al. (2009b) proposed the use of a factor ϕ, given by the ratio of the total length of the conduits measured from the raster representing these conduits using ℓ, $L_{T,\ell}$, and the true total length of the conduits belonging to the drainage system, $L_{T,L}$. The conduit length *L* in a cell *j* could then be estimated by ℓ_j multiplied by $1/\phi$. A second consideration in adapting Eq. (7) to a grid cell is the value for q_L. This variable is calculated as the flow produced by the excess rainfall in the cell $E_{j,i}A_j$ divided by the conduit flow length ℓ_j/ϕ. This approach implies that cells identified as conduits are also able to generate runoff, and the travel time depends on the excess rainfall rate. Finally, like hillslopes, λ_c is computed using the ratio of the upstream and local impervious areas (see Eq. (5)). From Eq. (7), we obtain the final expression for the travel time in a conduit cell *j* at time *i*, $\tau_{j,i}$, as:

$$\tau_{j,i} = \frac{\ell_j}{\phi}\left[\frac{1}{\alpha_{c,j}(E_{j,i}A_j)^{\beta_{c,j}-1}}\right]^{1/\beta_{c,t}}\left[\left(\frac{H_{u,j}A_{u,j}}{H_jA_j}+1\right)^{1/\beta_{c,j}} - \left(\frac{H_{u,j}A_{u,j}}{H_jA_j}\right)^{1/\beta_{c,j}}\right] \tag{9}$$

where the notation of $\alpha_{c,j}$ and $\beta_{c,j}$ implies that the values of the kinematic wave parameters for conduits vary depending on whether the cell *j* represents a street, pipe, or channel.

Using Manning's equation, Wong and Zhou (2003, 2006) proposed expressions of the kinematic wave parameters for different cross sections. By replacing these expressions in Eq. (9), Gironás et al. (2009b) obtained the following expressions for travel time in cells representing streets (Eq. 10), pipes (Eq. 11) and open channels (Eq. 12):

$$\tau_{j,i} = \frac{0.86}{\phi}\left(\frac{n_j}{\sqrt{S_j}}\right)^{3/4}\frac{\ell_j}{(E_{j,i}A_j)^{1/4}}\left(\frac{1+\sqrt{1+z_j^2}}{\sqrt{z_j}}\right)^{1/2}\left[\left(\frac{H_{u,j}A_{u,j}}{H_jA_j}+1\right)^{3/4} - \left(\frac{H_{u,j}A_{u,j}}{H_jA_j}\right)^{3/4}\right] \tag{10}$$

$$\tau_{j,i} = \frac{0.59}{\phi}\left(\frac{n_j}{\sqrt{S_j}}\right)^{4/5}\frac{\ell_j}{(E_{j,i}A_j)^{1/5}D_j^{2/15}}\left[\left(\frac{H_{u,j}A_{u,j}}{H_jA_j}+1\right)^{4/5} - \left(\frac{H_{u,j}A_{u,j}}{H_jA_j}\right)^{4/5}\right] \tag{11}$$

$$\tau_{j,i} = \frac{6.99}{\phi}\,\ell_j\left(\frac{n_j}{\sqrt{S_j}}\right)^{3/5}\left(\frac{B_j}{E_{j,i}A_j}\right)^{2/5}\left[\left(\frac{H_{u,j}A_{u,j}}{H_jA_j}+1\right)^{3/5} - \left(\frac{H_{u,j}A_{u,j}}{H_jA_j}\right)^{3/5}\right] \tag{12}$$

Where *z* is the side slope (H/V) of a gutter with a vertical size, *D* is the diameter of the pipe and *B* is the rectangular width of the open channel. These three properties together with *S* and *n* can vary between cells. The units of the previous equations are min for $\tau_{j,i}$, m m^{-1} for S_j, m for ℓ_j, m for *D*, m for *B*, mm h^{-1} for $E_{j,t}$ and m^2 for A_j and $A_{u,j}$. Again, *j* is an index of cells and *i* is an index of times.

3.5 Instantaneous unit hydrograph and convolution

To compute the total travel time T_j of the wave going from each cell *j* to the outlet, we add the travel times along the cells belonging to the flow path starting at that cell. The travel time in each cell τ_r is obtained using Eqs. (6), (10), (11) and (12), depending on its classification. Both τ_r and T_j depend on the excess rainfall rate. Following the definition proposed by Rodríguez-Iturbe and Valdés (1979), the IUH corresponds to the PDF of the travel times, $f(T_j)$. From the IUH theory, Chow et al. (1988) derived *h*(*t*), which is the UH for a constant rainfall pulse of duration Δ*t*, as:

$$h(t) = \frac{1}{\Delta t}\int_{t-\Delta t}^{t} f(T_j)\,dT = \frac{F(t)-F(t-\Delta t)}{\Delta t} \approx \frac{1}{\Delta t}\frac{n_t}{N_t} \tag{13}$$

where $F(t)$ is the cumulative probability of the travel time for points within the catchment, and Δ*t* and *h*(*t*) have the units of T and T^{-1} respectively. Additionally n_t is the number of cells whose travel time to the outlet is in the range [*t*-Δ*t*, *t*], and N_t is the total number of cells in the catchment. Finally, Q_v, the flow rate at the v^{th} time interval of length Δ*t* is given by the discrete convolution between the rainfall pulses and the corresponding UHs calculated from Eq. (13)

$$Q_v = A\sum_{i=1}^{v}\overline{E}_i h_{v-i+1} \tag{14}$$

where *A* is the catchment area, *h* is the UH corresponding to the i^{th} excess rainfall pulse and $\overline{E}_i$ is the spatially averaged excess rainfall depth of the i^{th} pulse. If *A* is in m^2, Δ*t* in s and $E_{j,i}$ in mm s^{-1}, the units of $\overline{E}_i$ are mm and the units of *Q* are l s^{-1}. Note that we are convoluting a UH derived from a spatially-distributed excess rainfall field with the spatially-averaged excess rainfall depth. This quasi-linear approach was proposed by Muzik (1996) and allows the incorporation of the climatic-dependence of the IUH and thus the nonlinear response of catchments to excess precipitation.

4 STUDY AREA

The U-McIUH method was studied by simulating a series of events in the Gohard catchment, located in the metropolitan area of Nantes, France (Fig. 1). The Gohard catchment has an area of 163.7 ha and an overall imperviousness of 30.8%. It is a subcatchment of the Aubinière catchment, whose area is 10.9 km^2 and overall imperviousness is 31.7%. The catchment has loamy/sandy loam soil. The Aubinière catchment was used by Gironás et al. (2009a) to validate the terrain representation used in the model to generate the urban terrain and the boundary of the Gohard catchment. The catchment contains residential areas, commercial areas and industrial zones, and has been used previously to develop and test hydrologic models (Rodriguez et al., 2003, 2005). The available data include all the information required to define the urban terrain and apply the model (i.e. a DEM with a resolution of 20 m, an imperviousness raster with the same resolution, and layers delineating streets, storm sewer pipes, and natural channels). The Gohard catchment was used to test the U-McIUH model because it has reliable discharge records with a 5-min time step. Rainfall records from a single rain gauge were used in this study and intensities were assumed to be spatially homogenous over the catchment. Table 1 summarizes the beginning date ($date_{beg}$), beginning time (t_{beg}), total precipitation (P_{total}), duration (Dur), and maximum intensity (I_{max}) for the 17 events within the 2001-2003 period that were used in the evaluation of the model. Base flow, which is typically small or nonexistent at the event time scale, was removed from observed discharge hydrographs using the straight line method (Viessman and Lewis, 1995). The Gohard has no open channels, so only the hillslope, street and pipe components of the U-McIUH are tested here.

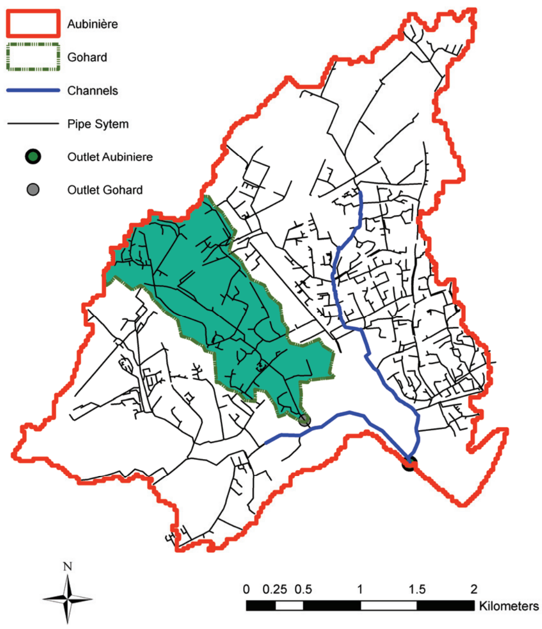

Figure 1. The Gohard and Aubinière catchments.

Storm event	$date_{beg}$ (dd-mm)	t_{beg} (hh:mm)	P_{total} (mm)	Dur (hr)	I_{max} (mm/h)
1	03-Sept	10:30	2.24	2.50	3.36
2	28-Sept	21:35	3.66	4.33	2.76
3	02-Oct	20:55	12.70	3.33	13.31
4	06-Oct	3:35	5.33	13.42	6.82
5	07-Oct	14:30	16.20	4.83	36.24
6	31-Jan	19:15	5.20	8.17	3.82
7	13-Feb	14:05	10.54	11.17	6.00
8	17-Apr	18:10	1.80	0.92	6.05
9	01-Jul	9:30	3.89	7.33	2.14
10	09-Jul	3:35	10.14	7.75	7.37
11	02-Aug	5:20	10.50	6.50	7.18
12	26-Aug	5:15	22.00	18.75	4.82
13	24-Oct	3:25	48.33	39.83	29.87
14	24-May	7:25	7.53	10.08	15.82
15	30-Nov	17:45	13.40	15.33	15.60
16	13-Dec	15:15	5.2	4.67	3.60
17	20-Dec	11:00	8.34	8.17	4.88

Table 1. Storm events. Storms 1-5 occurred in 2001 and were used for calibration; Storms 6-13 occurred in 2002, and Storms 14-17 occurred in 2003.

5 PERFORMANCE OF THE MODEL

The raw DEM was processed to generate the urban terrain according to the method previously summarized, and the U-McIUH was developed using this information. Flow direction, accumulation, length, and slope raster files were obtained from the urban terrain. The flow accumulation and imperviousness raster files were used in Eq. (5) to generate the raster file with values of λ_j. The saturated hydraulic conductivity K_s = 46.8 mm h^{-1} measured at a depth of 0.2 m under natural conditions is used as an estimate of f_c. Values of n at hillslope cells were generated as the imperviousness weighted average of the roughness coefficients for pervious and impervious areas (n_P and n_I, respectively). Initial values of n_P = 0.2 and n_I = 0.015 were obtained from the literature

(Viessman and Lewis, 1995). Preliminary values for the roughness coefficients for streets (n = 0.0161) and pipes (n = 0.0152) were obtained from a previous study in the area (Rodriguez et al., 2003). For conduit cells, the pipe diameters D, channel widths B, and slopes S, were determined from the complementary raster files generated from the original vector representations of the conduits. A constant side slope z = 25 was assumed for the street gutters, which was adopted from typical ranges given in the literature (Nicklow, 2001). A value of ϕ = 1.475 was computed.

The five rainfall events available in year 2001 were used to manually calibrate the model. We adopted n_P as the calibration variable for hillslope cells. Likewise, n was used for calibration of street and pipe cells. The calibration aimed to maximize the value of the modified coefficient of efficiency (MCE), which is defined as (Legates and McCabe, 1999):

$$\mathrm{MCE} = 1 - \frac{\sum_{v} \left| Q_{obs,v} - Q_{sim,v} \right|}{\sum_{v} \left| Q_{obs,v} - \overline{Q}_{obs,v} \right|} \qquad (15)$$

where $Q_{obs,v}$ is the observed discharge, $Q_{sim,v}$ is the simulated discharge, $\overline{Q}_{obs,v}$ is the average of the observed discharges, and v indicates each time interval belonging to the period of simulation. According to this coefficient, a value of one indicates an exact match with the observations. In the end, the calibrated value of the pervious surface roughness was n_P = 0.1, and calibrated values of n for the streets and pipes were 0.0289 and 0.0242, respectively.

Because the U-McIUH is an event-based model, values of IA and C were adjusted for each storm. These parameters depend on the soil moisture conditions at the beginning of an event and could eventually be estimated through a continuous formulation of the model. In this application, values of IA for all the storms ranged between 0.2 mm and 1.1 mm, with an average of 0.61 mm. Values of C ranged between 2.9 to 29.3, with an average of 13.7. These values imply an average effective infiltration rate (f_c/C) of 5 mm h^{-1}, a rate quite lower than the value of K_s initially used. Compaction of urban soil as well as reductions in the infiltration capacity at deeper layers can explain this small value. Fig. 2 shows the observed and simulated hydrographs for the calibration period.

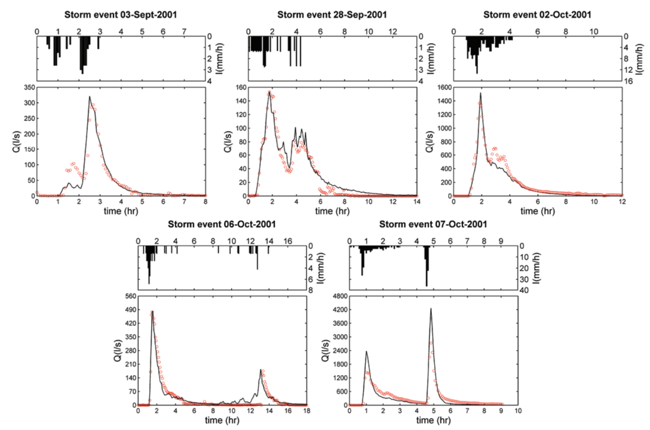

Figure 2. Comparison of the observed (circles) and simulated (lines) flows for the calibration events.

The calibrated model was tested using the 12 storm events in 2002 and 2003 (Fig. 3). Overall, the simulated and observed flows are quite similar, and the model performance appears to be similar among the events, irrespective of their magnitude. The MCE ranges between 0.52 and 0.82, with half of the storms having efficiencies greater than 0.75. The average values for the calibration and validation periods are 0.70 and 0.72, respectively. The lowest MCE values were obtained for events 5 and 13, in which Q_{max} was greatly overestimated. During these particularly intense events, it is likely that the pipe system surcharged or simply flooded, causing flow conditions to differ from those assumed by the model. A low MCE (0.55) was also obtained for event 8. Observed flows for that event show a second, smaller peak after the main one, which is not well simulated. However, the computed hydrograph is consistent with the structure of the storm event, which has only one peak.

Underestimation of a later secondary peak is also observed for events 3, 6, 7, 12, 15, 16, 17, but a subset of these (events 7, 8, 15, and 17) show an overestimation of the preceding peak. Thus, some storage in the system may not be adequately simulated by the model. Furthermore, this difference might also be due to the existence of base flow not properly removed from the observed hydrographs. Events 15, 16 and 17 have the highest base flows among the 12 events. The prediction of the peak flows is reasonably good and the average error for the volume (7%) is also fairly low. The model also does well in predicting the response time of the catchment and its dependence on the different pulse's intensities. The error in estimating t_{max} is 5 min or less in 11 out of the 17 events, and the maximum error is 15 min. Overall, the model generates good results in spite of the simple representation of the excess rainfall and the absence of a hydraulic module to explicitly route the flows.

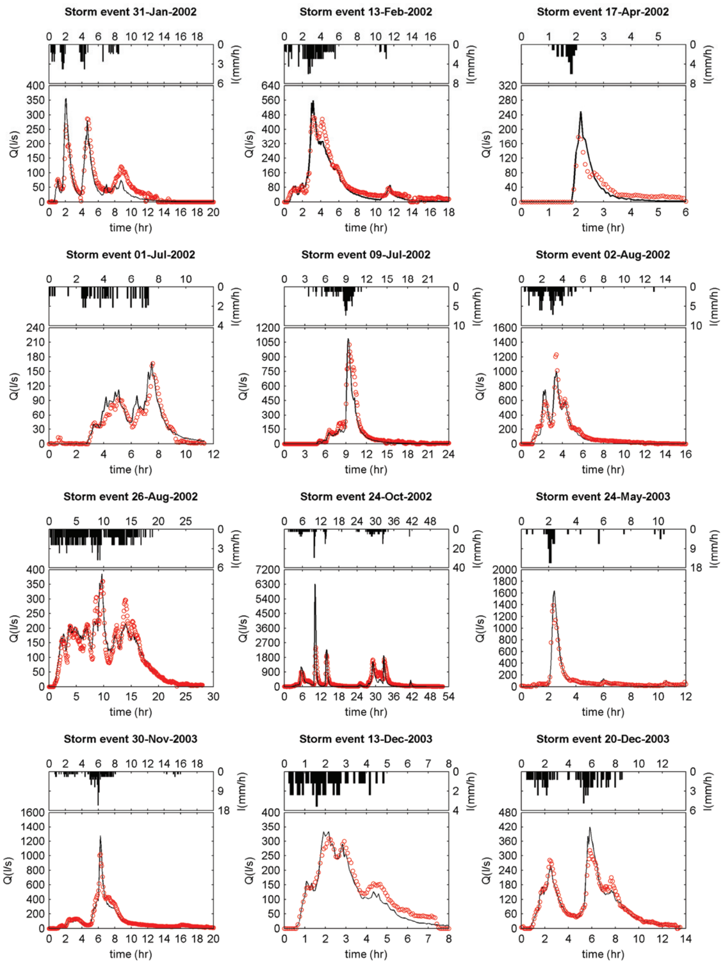

Figure 3. Comparison of the observed (circles) and simulated (lines) flows, period (2002-2003).

6 CONCLUSIONS

In this paper, we present a new hydrologic model for urban catchments. The model is based on the explicit representation of the GIUH, which defines the IUH as the PDF of the travel times from each cell in a raster representation of the catchment to the outlet. By including the excess rainfall intensity in the travel time calculation, the model includes the so-called climatic dependence of the IUH (i.e. faster travel times associated with higher excess rainfall rates). Additionally, the upstream contribution of flow is also considered and is estimated using the upstream impervious. Using this quasi-linear approach, spatially-averaged rainfall pulses of different intensities are convoluted with their respective IUHs to generate the response to storm events. Overall, the results demonstrate that an explicit representation of the urban morphology including the spatial structure of the catchment, the different elements contained in the catchment and the associated impacts on the travel times, can produce benefits in the simulation of the hydrologic response. Additionally, we conclude that the consideration of both the effect of the upstream contribution on the travel time, and the climatic dependence between the travel times and the rainfall intensity, is of the highest relevance and can be incorporated within a morphologic IUH approach. Given its spatial morphologic capabilities but simple formulation, some of the possible applications of the model, other than those of traditional models, are the evaluation of impacts associated with different spatial patterns of imperviousness and different types of drainage networks strategies, both in terms of spatial extension and types of conduits used.

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