

# Computing flooding of crossroads with obstacles using a 2D numerical model

Pierre-Henri Bazin, Emmanuel Mignot, André Paquier

### ► To cite this version:

Pierre-Henri Bazin, Emmanuel Mignot, André Paquier. Computing flooding of crossroads with obstacles using a 2D numerical model. Journal of Hydraulic Research, 2017, 55 (1), pp.72-84. 10.1080/00221686.2016.1217947. hal-01724983

## HAL Id: hal-01724983 https://hal.science/hal-01724983

Submitted on 21 Nov 2018

**HAL** is a multi-disciplinary open access archive for the deposit and dissemination of scientific research documents, whether they are published or not. The documents may come from teaching and research institutions in France or abroad, or from public or private research centers. L'archive ouverte pluridisciplinaire **HAL**, est destinée au dépôt et à la diffusion de documents scientifiques de niveau recherche, publiés ou non, émanant des établissements d'enseignement et de recherche français ou étrangers, des laboratoires publics ou privés.

- 1 Computing flooding of crossroads with obstacles using a 2D numerical model
- 2
- 3 PIERRE-HENRI BAZIN, PhD, Irstea, UR HHLY, 5 rue de la Doua BP 32138, 69616 Villeurbanne
- 4 Cedex, France
- 5 Email : pierrehenri.bazin@gmail.com.
- 6 EMMANUEL MIGNOT, Assistant Professor, Laboratoire de Mécanique des Fluides et d'Acoustique
- 7 (LMFA), CNRS-Université de Lyon, Ecole Centrale de Lyon/Université Lyon 1/INSA de Lyon, 20
- 8 Avenue A. Einstein, 69621 Villeurbanne, France
- 9 Email : emmanuel.mignot@insa-lyon.fr (author for correspondence)
- 10 ANDRE PAQUIER (IAHR Member), Engineer, Irstea, UR HHLY, 5 rue de la Doua BP32138, 69616
- 11 Villeurbanne Cedex, France
- 12 Email : andre.paquier@irstea.fr.
- 13
- 14 Running Head : Computing flooding of crossroads with obstacles
- 15
- 16

#### 17 Computing flooding of crossroads with obstacles using a 2D numerical model

#### 18 ABSTRACT

19 Typical urban flood flow features are usually computed using two-dimensional numerical models. How such 20 modelling can be implemented in dense urban areas with obstacles is investigated. A strategy for representing 21 the effect of urban obstacles in various flow conditions is defined. The comparison between the available 22 laboratory measurements and the model results show that if the water depth is high enough and the flow remains 23 subcritical, two-dimensional modelling with constant eddy viscosity provides the effect of the obstacles on the 24 flow distribution accurately, even with a coarse mesh; moreover, an over-simplified representation of the 25 sidewalks averaging the street cross section elevations seems sufficient. Oppositely, if the water depth is low 26 and/or the flow becomes supercritical, the description of the flow is not relevant enough and it generates errors in 27 the flow distribution at the crossroads.

28

*Keywords*: Flood modelling; flow-structure interaction; junction flow; two-dimensional models; urban
 flood

#### 31 1 Introduction

According to the European Environment Agency report (EEA, 2010), floods have caused 1126 deaths
in the period 1998-2009. Besides, floods remain the most costly natural hazard, with cumulated
damages evaluated to EUR 52 billion in the same period. Actually, most of the human losses and
economic damages occur in the urban areas that concentrate a large part of the stakes: population,
economic activities and industries, historical centres, road networks... because cities are often located
in flood-prone areas such as river floodplains and coastal areas.
When considering a dense urban area, the flood can be assumed to occur mainly in the street

39 network so that surface flow models can be restrained to this network (e.g. Leandro, Chen, Djordjevic, 40 & Savic, 2009; Mignot, Paquier, & Haider, 2006; Mignot, Paquier, & Ishigaki, 2006). The standard 41 approach to simulate these events relies on a depth averaged two-dimensional model (2D) solving the 42 full shallow water equations which provides the global flood extent and the spatial distribution of the 43 maximum water levels, along with detailed time series of local flow depths and velocities. The ability 44 of standard 2D models to represent the global surface flow pattern during urban floods has been 45 assessed by comparing numerical simulations and laboratory experiments for idealized and well-46 controlled urban configurations corresponding to flooded urban crossroads (El Kadi Abderrezzak, 47 Lewicki, Paquier, Riviere, & Travin 2011; Ghostine et al., 2010; Mignot, Paquier, & Riviere, 2008) or

48 schematic flooded urban areas (Mignot, Paquier, & Ishigaki, 2006; Soares-Frazão, & Zech 2008; Van

49 Emelen et al., 2012).

- 50 Nevertheless, obstacles of dimensions much smaller than the buildings such as bus-stops, trees 51 or parked cars, that are common street furniture, are never included in the flood simulations even if 52 these obstacles may strongly modify the flow pattern and thus the risk distribution in the city. 53 Moreover, given the increasing computing capacities and spatial data acquisition methods from 54 existing GIS databases or Lidar land survey, such introduction of urban details is not a limiting 55
- 55 process anymore.

56 Mignot et al. (2013) recently measured and computed (using a 3D code solving the unsteady 57 Reynolds-averaged Navier-Stokes equations with a Spalart-Allmaras model for turbulence closure) the 58 impact of obstacles located in a crossroad on flow distribution to the downstream streets. The chosen 59 geometry was an idealized subcritical 3-branch dividing flow. The main results in terms of impact on 60 the flow distribution are listed below:

- The impact of the obstacle on the discharge distribution is strongly dependent on its location with regards to the crossroad. Indeed, obstacles located within the upstream channel increase the streamwise flow velocity and thus tend to reduce the lateral and increase the downstream discharges. Oppositely obstacles located within the downstream channel tend to block off the flow in this channel and to reduce the corresponding discharge while increasing the lateral discharge. Finally for obstacles located within the branch channel, their impact depends on the side of the channel in which they are introduced.
- This impact is a direct consequence of the modifications when introducing the obstacle of the:
  (i) streamwise and centrifugal flow acceleration, (ii) width of the recirculation zone and (iii)
  wake downstream the obstacle.
- The impact of an obstacle appears to increase as the upstream Froude number increases while
   the modification of the normalized water depth hardly affects the results and the impact of the
   initial flow distribution depends on the obstacle location.
- Given the potential impacts of a single obstacle located in the vicinity of the bifurcation
  (modification of the flow distribution up to 12%), the present paper aims at verifying the capacities of
  operational 2D numerical models to reproduce these impacts.
- 2D modelling of subcritical dividing flows in a three branch bifurcation without obstacle was
  carried out by Shettar and Murthy (1996) and Khan, Cadavid, and Wang (2000). Both models proved
- an excellent ability to compute the discharge distribution. In particular, Shettar and Murthy (1996)
- 80 performed an extensive validation of the numerical model, using vertically-averaged velocity fields,
- 81 water surface profiles at the intersection, as well as more global flow characteristics such as the size of
- 82 the branch separation zone and the energy loss in the junction.
- 83 On the other hand, simulations of flows around obstacles in the literature mainly concern 84 flows around bridge piers or groynes and are preferably computed using 3D numerical models (as in 85 Mignot et al., 2013), but 2D models are also used along with adequate turbulence closure models.

86 Yulistiyanto, Zech, and Graf (1998) used a 2D model to simulate the flow around an emerging

- 87 cylinder using a specific treatment of the dispersion stresses due to vertical velocity profiles and their
- simulations appear to fairly predict the velocities and water depths around the cylinder, without any
- 89 calibration. Jiang, Yang, and Liang (2009) performed 2D modelling of the flow past a vertical plate,
- 90 and proved that an eddy viscosity model computed via the friction velocity can achieve reasonable
- 91 prediction of the velocity field. Stansby (2006) performed simulations of the flow past a conical island
- 92 with a 2D model, including a horizontal mixing-length turbulence eddy viscosity model and the ability
- 93 of the model was found to depend on the wake type, with discrepancies increasing when predicting the
- 94 occurrence and length of stable wakes.

95 Nevertheless, operational 2D numerical models used for urban flood modelling rarely consider 96 turbulence effects, or use simple turbulence models because accurate modelling of turbulence would 97 require computational efforts that are not affordable for large-scale flood studies. Moreover, the 98 modellers often neglect the presence of obstacles and simply do not consider them in the 99 geometry/meshing process. Then the present paper aims at modelling the impact in the crossroad of 100 obstacles from Mignot et al. (2013) and sidewalks from Bazin (2013) using a 2D operational 101 numerical model with a very simple turbulence model, as used when modelling the urban flood events, 102 in order to reveal that the consideration of such obstacle could improve the usual urban flood 103 calculation. This model is thus much simpler than the 3D model previously used by Mignot et al. 104 (2013) to compute these flows but this latter model could not be applied to urban flooding 105 calculations. The objective is here twofold: 1) to identify for which obstacle location and flow 106 configuration a 2D operational model is able to fairly estimate the large-scale effects (mostly the effect 107 of the discharge distribution to the downstream branches) of an impervious obstacle or a sidewalk in 108 the vicinity of a 3-branch open-channel bifurcation and 2) to identify which meshing strategy (method 109 for including obstacles and selected mesh dimensions) is required to achieve this fair estimate. The 110 first section presents experimental data used herein along with the 2D numerical model and its 111 application in cases where no obstacle is included, the second section presents the calculation of flows 112 in bifurcations with each obstacle configuration and then with the sidewalks.

#### 113 2 Experimental data and numerical model

#### 114 2.1 Experimental data

115 The experimental data are derived from measurements performed by Mignot et al. (2013) and Bazin

- 116 (2013). The chosen geometry was an idealized subcritical 3-branch dividing flow with one inlet named
- 117 "upstream channel" with subscript "u" and two outlets, one aligned with the inlet named "downstream
- 118 channel" with subscript "d" and the second forming a 90° angle, named "branch channel" with
- 119 subscript "b". The experiments included 14 flow configurations listed in Table 1 with varying
- 120 dimensionless parameters  $F_{u0}$  (series S1),  $R_{q0}$  (S2) and  $h_{u0}/b$  (S3), where F is the Froude number,  $R_q$ =

- 121  $Q_b/Q_u$  is the flow distribution with Q the discharge, b is the channel width, h is the water depth and
- 122 with the 0 subscript referring to the case without obstacle. For each of these 14 flows, the discharge
- 123 distribution  $R_q$  was measured without obstacle (denoted as  $R_{q0}$ ) and after adding one obstacle at each
- 124 of the 7 locations sketched in Fig. 1, replacing the subscript 0 (without obstacle) by the subscripts 1 to
- 125 7 corresponding to the obstacle location number. The obstacles are square shape impervious and
- emerging blocks of base dimensions 5 cm x 5 cm. Note that Mignot et al. (2013) also measured
- 127 configurations with two obstacles but these configurations are not considered herein. However, for the
- same 14 flows, sidewalks (of dimensions 6cm x 2cm, Fig. 1b) were also added (without obstacles) by
- Bazin (2013) and this additional configuration is referred to by the subscript 10 as sketched in Fig.1.
- 130 In the end, a total of 14 flows times 9 cases were measured (1 without obstacle, 7 with one obstacle
- and 1 with sidewalk), that is 126 flow configurations. Additionally, the 2D horizontal velocity field
- 132 was measured at one selected elevation (z=3cm, z/h=0.71) for flow 6 (Table 1) without obstacle, with
- all obstacles, and finally with the sidewalks. All these experimental configurations can be considered
- to be representative of a crossroad of narrow streets in a city at scale 1/25 (Mignot et al., 2013).

#### 135 2.2 Equations and numerical model

136 The Rubar20 code solves the 2D shallow water equations including the continuity equation (Eq. 1) and 137 the conservation of momentum along orthogonal axes x and y (Eqs 2 and 3):

138 
$$\frac{\partial h}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} = 0$$
(1)

139 
$$\frac{\partial(hu)}{\partial t} + \frac{\partial(hu^2)}{\partial x} + \frac{\partial(huv)}{\partial y} + gh\frac{\partial h}{\partial x} = -gh\frac{\partial Z_b}{\partial x} - g\frac{u\sqrt{u^2 + v^2}}{K_s^2 h^{1/3}} + K\left[\frac{\partial}{\partial x}\left(h\frac{\partial u}{\partial x}\right) + \frac{\partial}{\partial y}\left(h\frac{\partial u}{\partial y}\right)\right]$$
(2)

140 
$$\frac{\partial(hv)}{\partial t} + \frac{\partial(hv^2)}{\partial y} + \frac{\partial(huv)}{\partial x} + gh\frac{\partial h}{\partial y} = -gh\frac{\partial Z_b}{\partial y} - g\frac{v\sqrt{u^2 + v^2}}{K_s^2 h^{1/3}} + K\left[\frac{\partial}{\partial x}\left(h\frac{\partial v}{\partial x}\right) + \frac{\partial}{\partial y}\left(h\frac{\partial v}{\partial y}\right)\right]$$
(3)

141 where t is the time, h is the water depth, u and v are the depth averaged velocities along respectively x 142 and y axis, g is the gravity acceleration,  $Z_b$  is the bottom elevation,  $K_s$  is the Strickler coefficient with 143  $K_s = 1/n$ , *n* being the Manning roughness coefficient, and *K* is the eddy viscosity coefficient. The eddy 144 viscosity coefficient K represents effects of diffusion, depth-averaging of the velocities as well as 145 turbulent stresses; it does not form an elaborate turbulence model but it offers a way to calibrate 146 simulations in the case where turbulence effects have to be considered. The simplest formulation for K147 assumes a constant value in time and space. Use of such simple eddy viscosity model can lead to 148 acceptable results once K is calibrated, as shown for flows including strong two-dimensional patterns 149 such as separations zones (Bravo & Holly, 1996; Papanicolaou, Elhakeem, & Wardman, 2011). 150 Depending on the experimental flow configurations, the flow regime is here either 151 hydraulically smooth or in the transition toward fully rough flow. Therefore, as for Mignot, Paquier,

and Rivière (2008), the following explicit approximation of the Colebrook-White formula given by Yen (2002) is considered for computing the friction factor f:

154 
$$f = \frac{1}{4} \left[ -log_{10} \left( \frac{k_s}{12R_h} + \frac{6.79}{R^{0.9}} \right) \right]^{-2}$$
(4)

where R is the local Reynolds number calculated as  $4h\sqrt{u^2 + v^2}/v$  with v the water kinematic viscosity,  $R_h$  is the hydraulic radius, taken here as the local water depth h and  $k_s=0.1$  mm the channel roughness height. The computed friction factor is then transformed in an equivalent Strickler coefficient for Eqs 2 and 3 using:

159

$$K_s = \sqrt{\frac{8g}{h^{1/3}f}} \tag{5}$$

160 The usual range of equivalent Strickler coefficients is 85-105 m<sup>1/3</sup>s<sup>-1</sup>. Moreover, for cells 161 located along solid boundaries (including the obstacles), wall friction is added, considering it equal to 162 half the bottom friction for the same water depth (for simplicity and because the averaged water 163 pressure along the wall should be half the water pressure at the bottom). Finally, the obstacles are 164 represented as impervious cells in which no flow is calculated. The sidewalks are represented in the 165 topography increasing the bottom elevation of all the points of the sidewalks by their elevation, i.e. by 166 0.02 m.

167 The numerical scheme is detailed in Mignot, Paquier, and Haider (2006). The computational 168 mesh is made of quadrilaterals and triangles using an unstructured grid so that the mesh can be 169 adapted to any complex obstacle shape. The code solves the above equations using an explicit second-170 order finite-volume scheme, adapted from MUSCL approach. The numerical scheme can run with a 171 fixed time step, or with an adaptive time step respecting a Courant number below 1 so that the scheme 172 remains stable. Originally developed for simulations of dam-break waves, the code is robust for all the 173 simulations of shallow flows. Treatment of the drying/wetting processes is by setting null water depths 174 whenever the computed ones are lower than a threshold (0.001 mm for computations described in the 175 present paper). The mass conservation is achieved with typical errors less than 0.01 % of the total 176 mass. Code validation against experimental and field data includes simulations of dividing 177 supercritical and transcritical flows (Mignot, Paquier, & Rivière, 2008; El Kadi Abderrezzak et al., 178 2011), floods in dense urban areas (Mignot, Paquier, & Ishigaki, 2006) and dam-break type flows 179 around obstacles (El Kadi Abderrezzak, Paquier, & Mignot, 2009). The main discrepancies observed 180 in previous comparisons with laboratory measurements concern the exact prediction of the location 181 and size of the hydraulic jumps. 182 In the present study, the reference mesh consists of a square grid with a resolution of m = 3.5

- to 5 cm in the junction, leading to 7 cells across the channels. This mesh size is indeed representativeof a model used for simulating urban flood events. This mesh permits to capture the time-averaged
- 185 flow around each obstacle, but may not represent finer phenomenon such as vortex shedding (note that

- 186 Lloyd & Stansby (1997) and Yulistiyanto et al. (1998) use around 30 cells across their obstacles). A
- 187 refined mesh (m = 0.5 cm in the junction and 2 cm in the channels) is used for verifying the mesh
- 188 influence on the results. The inlet flow discharge  $Q_{u0}$  experimentally measured in the pumping loop is
- 189 imposed on the total width of the upstream channel inlet, with a uniform velocity distribution across
- 190 the boundary. The downstream boundary conditions imposed at the outlet sections of the downstream
- and lateral branches are the experimental weir equations, i.e. the measured stage-discharge
- 192 relationships  $Q_b = f(C_b, h_b)$  and  $Q_d = f(C_d, h_d)$  with C the weir crest height and h the water depth measured
- 193 two channel widths upstream from the weir.

#### 194 2.3 Numerical validation: modelling flows without obstacles

- 195 First of all, the measured channel roughness height equals 0.1 mm and it was verified (not shown here)
- that a variation of this parameter within the range of uncertainty (0.05 to 0.2 mm) does not impact the
- 197 calculations. Now, the parameters subject to calibration and their acceptable ranges are the mesh size
- 198 (from 0.5 cm to 3.5-5 cm) and the eddy viscosity (from 0 to  $10^{-3} \text{ m}^2 \text{s}^{-1}$ ). In order to validate the
- 199 numerical model ability to simulate the discharge distribution and predict the general flow patterns
- 200 before introducing the obstacles and to identify the best option for the two calibration parameters,
- 201 seven parameter runs detailed in Table 2 are used to compute the 14 flows without obstacles (subscript
- 202 0), considering the two mesh sizes and four values of eddy viscosity coefficient K. First, the measured
- water depths are compared to calculated ones resulting in an average difference of about 0.1 mm and a
- 204 mean quadratic difference of 0.6 mm (not shown here), which can be considered as small compared to
- an average water depth of about 40 to 50 mm and the accuracy of the water depth measurements.
- 206 Then, the capacity to predict the discharge distribution is assessed calculating the average  $\delta$  and root
- 207 mean square deviation  $\sigma$  of the relative error of the lateral discharge  $Q^*_{b0}$  with:

208 
$$Q_{b0}^* = \frac{Q_{b0,SIM} - Q_{b0,MES}}{Q_{b0,MES}}$$
(6)

209 
$$\delta(Q_{b0}^*) = \frac{1}{14} \sum_{k=1}^{14} Q_{b0}^*$$
(7)

210 
$$\sigma(Q_{b0}^*) = \sqrt{\frac{\sum_{k=1}^{14} Q_{b0}^{*^2}}{14}}$$
(8)

211 where  $Q_{b0,SIM}$  and  $Q_{b0,MES}$  are respectively the simulated and measured values of the branch channel 212 flow discharge obtained without obstacles. These indicators included in columns 4-5 of Table 2 213 confirm the capacities of the numerical model to estimate the discharge distribution, with typical 214 errors about 2%, and a bias generally negative ( $\delta(Q_{h0}^*) \le 0$ ). These results do not promote a specific 215 value for the eddy viscosity K (runs A-B-C or E1-E2-D-E3) and reveal that the mesh refinement (runs 216 E1-E2-D-E3) only slightly improves the calculation adequacy with regards to the use of a coarser 217 mesh (runs A-B-C). The optimum seems to be run B as it considers a coarser mesh (of same size as 218 the obstacle dimension, that is the minimum size that could be used for simulations with obstacles)

- and a minimum  $\sigma(Q^*_{b0})$ . Nevertheless, the optimum run without obstacle may not be the optimum run when considering the obstacles. For instance, the computation of the wake created by the obstacle may require a finer mesh than that of run B. As a consequence, four runs are kept for future analysis with obstacles: the three cases with the coarse mesh with different eddy viscosity values (A, B and C) and a fourth one with the finer mesh (D) in order to verify whether a mesh refinement improves the
- calculations of the flow with obstacles.
- 225 Figure 2 compares the horizontal measured and simulated velocity fields for flow number 6 226 (Table 1). Note that experimental velocities are measured at the elevation z=3 cm ( $z/h\sim0.71$ ), slightly 227 above the mid elevation, which are likely to be representative of the depth-averaged velocity, whereas 228 the computed velocities are depth-averaged velocities. Within this comparison framework, the data 229 from the 4 parameter runs appear to be in fair agreement with experimental measurements both 230 quantitatively and qualitatively except for the transverse extension (along x) of the recirculation cell in 231 the downstream branch (Fig. 2b). The coarse mesh (runs A, B and C) leads to a coarse representation 232 of the velocity field, but the global flow pattern remains well predicted without any clear advantage 233 while varying the eddy viscosity coefficient.
- 234 The recirculation area located along the upstream wall of the branch channel is a main flow 235 structure in a subcritical dividing flow (Mignot et al., 2014). From the experimental and numerical 236 data, the contour of the recirculation zone is computed, assuming it can be taken as the "zero-237 discharge area". Figure 3 shows the contour of this recirculation area for flow 6 (Table 1), for measured data and computed runs A and C (K=0 and  $K=10^{-3}$  m<sup>2</sup>s<sup>-1</sup>). For run A, the recirculation area 238 239 almost reaches the downstream weir of the branch channel, whereas its length (along y axis) strongly 240 decreases for run C. Oppositely, the maximum width of the recirculation appears to be similar for both 241 K values.
- To conclude, the numerical model appears to predict the experimental flows without obstacle with enough accuracy to allow the introduction of obstacles and sidewalks in the next section.

#### 244 **3** Modelling of flows with obstacles or sidewalks

#### 245 3.1 Introduction of obstacles

Here below, the 14 flows are simulated with the 7 obstacles configurations from Fig. 1a and the 4
parameter runs A, B, C and D, representing a total of 14x7x4=392 calculations. First, each simulation
is assessed calculating the simulated and measured discharge distribution modification due to the
obstacle:

$$\Delta R_{qi,SIM} = R_{qi,SIM} - R_{q0,SIM} \tag{9}$$

$$\Delta R_{qi,MES} = R_{qi,MES} - R_{q0,MES} \tag{10}$$

and then comparing the simulated and measured cases:

$$\Delta R_{qi} = \Delta R_{qi,SIM} - \Delta R_{qi,MES} \tag{11}$$

with i=1...7 the obstacle number (Fig.1). Then, for each parameter set the average  $\delta$  and root mean

square deviation  $\sigma$  of these errors are computed over the 7 obstacle configurations and the 14 flow

configurations, in a similar way as in Eqs. 7 and 8 and are indicated in columns 4-5 of Table 3.

257 Moreover, the accuracy of the model to compute the discharge distribution with obstacle is included in

columns 6-7 of Table 3 with:

253

259

$$\delta(Q_{b1-7}^*) = \frac{1}{14} \sum_{k=1}^{14} \left( \frac{1}{7} \sum_{i=1}^{7} Q_{bi}^* \right)$$
(12)

260 
$$\sigma(Q_{b1-7}^*) = \sqrt{\frac{\sum_{k=1}^{14} \sum_{i=1}^{7} (q_{bi}^{*2})}{\frac{14x7}{}}}$$
(13)

Table 3 reveals an overall fair ability to predict the discharge distribution when obstacles are included with a typical average error lower than 3% and a root mean square error of about 3%. It appears that:

- using the fine (D) and coarse (B) meshes results in a similar agreement with the
   measurements.
- the bias δ decreases as the eddy viscosity coefficient decreases (C-B-A) which means that the
   calculated branch discharge is reduced
- the higher values of the eddy viscosity coefficient (runs B-C) seem to be the best options
   although the differences using the three values remain weak.
- the error of the simulation with obstacles is of the same order as the error for the flows without
   obstacles provided in Table 2

272 Scatter plots comparing measured and simulated evolutions of the discharge distribution are 273 shown on Fig.4 for runs B (coarse mesh) and D (fine mesh). A linear regression using a least-square 274 method is carried out and the slope *s* is included, showing that the calculations fairly estimate the 275 discharge distribution for most of the 98 calculations using each mesh.

276 Still using runs B and D, Fig.5 details for each obstacle and each series from Table 1 the

277 measured and simulated modifications of the discharge distribution as each obstacle is introduced.

278 Figure 5 confirms that for both parameter runs (corresponding to both mesh refinements), most

279 obstacle configurations (each line) and most flows (each column), the discharge distribution is well

estimated by the numerical model; i.e. computed and measured symbols are in agreement.

281 Discrepancies mainly occur for:

• flow 1 with  $F_{u0}=0.79$ , which is the configuration with highest upstream Froude number, for 283 which the flow without obstacle includes a significant part of supercritical regime in the

- junction. The simulation of this partially supercritical flow with obstacles 1, 2, 6 and 7 is less
  accurate than for other flows with lower Froude numbers. This limit is related to a slight
  uncertainty in the prediction of the actual hydraulic jump location, as already noticed by
  Mignot, Paquier, and Rivière (2008).
- 20

• the upstream obstacle 2 for which  $|\Delta R_{q2}|$  is overestimated when computed with the fine mesh 289 (run D) and underestimated when computed with the coarser one (run B).

290 In order to gain information on the computational errors when introducing obstacle 2, Fig.6 291 shows the horizontal velocity fields measured (at z=3 cm) and computed (depth-averaged velocity 292 field) for flow 6 with the 4 parameter runs (to be compared with the measured flow without obstacle in 293 Fig.2). It appears that when using the coarse mesh (runs A to C), the size of the wake with small 294 velocity downstream of the obstacle is strongly overestimated, which limits the flow capacity to rotate 295 into the branch channel. Thus, there is an increased effect of obstacle 2, hence an increase of  $|\Delta R_{a2}|$  but 296 the coarse mesh also smoothens the flow around obstacle 2, so that finally the effect of the obstacle on 297 the discharge distribution is rather fairly estimated for this flow 6, as shown on Fig.5. Oppositely, the 298 use of the fine mesh (D) on Fig.6a leads to a shorter wake and thus to a higher deviation of the flow 299 towards the branch and thus a slightly better estimate of  $\Delta R_{q2}$  shown on Fig.5. Moreover, Fig. 6b 300 reveals that both meshes lead to a fair estimation of the velocity field within the intersection (x < 0.3m 301 and y < 0.3m) but not to a precise extension of the recirculation zone, as already observed without 302 obstacle (Fig. 2b). For this obstacle 2, additional simulations were carried out for all flows with the 303 fine mesh using friction velocity-dependent eddy viscosity formulations. None of these additional runs 304 achieved a better agreement with the measured discharge distribution (not shown here for the sake of 305 scarcity).

#### 306 3.2 Introduction of sidewalks

As the sidewalks containing vertical edges are submerged, the meshing challenge lies in an adequate representation of their topography. Cells slope in 2D models is limited by their dimensions for practical reasons (computation times), so an adaptation of the mesh is required. The mesh considered in the present section is made of regular 2 cm width square elements everywhere except on the sidewalks edges, where finer elements of 0.5 cm or 1 cm are used (Fig.7). Using this mesh, two different topographical representations are used (Fig. 7):

- the reference topography (*Ref*) similar to the experimental geometry.
- the simplified averaged topography (*Avg*), for which the whole channel bottom elevation is
   constant and equals the average elevation of the channel cross section, i.e.
- 316 2x(6 cmx2 cm)/(30 cm) = 0.8 cm. This representation is the simpler representation of the
- 317 topographical change induced by the sidewalks.

318 4 parameter runs are defined in Table 4, using both topographical representations and two 319 eddy viscosity coefficient values *K*. As for obstacles in Table 3, the average error  $\delta$  and root mean 320 square error  $\sigma$  of the predicted discharge distribution and evolution of the discharge distribution when 321 including the sidewalks are included in Table 4, with the subscript 10 referring to the sidewalks. It 322 appears that:

- the model is able to predict the discharge distribution when sidewalks are included with a
   typical average error lower than 3% and a root mean square error of about 5%.
- the simplified topographical representation *Avg* gives similar results as the reference
   configuration, without main deterioration of the results.
- slightly better simulation results are obtained using a higher eddy viscosity coefficient  $K=10^{-3}$ 328 m<sup>2</sup>s<sup>-1</sup>.

329 Figure 8 shows the measured and horizontal simulated velocity fields for flow 6, for parameter 330 runs 1 and 2 from Table 4 carried out with the *Ref* topography (to be compared with the measured 331 flow without sidewalk in Fig.2). The acceleration of the flow in the upstream channel central area is 332 well simulated but this acceleration does not extend within the intersection in the numerical model, 333 unlike in the measurements (Fig. 8b). Moreover, the lateral extension of the recirculation and adjacent 334 acceleration zones in the branch channel are fairly (Fig. 8c) but not perfectly (Fig. 8b) estimated. 335 Moreover, increasing the eddy viscosity (run 2) hardly affects the velocity distribution across the 336 channel width, and only slightly changes the branch channel recirculation shape.

Measured and simulated modifications of the discharge distribution due to the sidewalks are
shown for each of the 14 flows on Fig.9 in the same way as for Fig.5. Results with the 4 runs appear to
be close to each other, and stand well within the range of experimental uncertainties, except:

- 340 for flow 2 that has a large upstream channel Froude number. The discontinuity (sudden  $\Delta R_a$ • 341 decrease as  $F_{u0}$  increases) observed in the experiments for  $F_{u0} \sim 0.6-0.8$  is rather occurring for 342  $F_{u0} \sim 0.4$ -0.6 in the numerical simulations. Figure 10 shows that this discontinuity is due to the 343 occurrence of an oblique hydraulic jump attached to the upstream corner of the junction 344 (Mignot, Riviere, Perkins, & Paquier, 2008), which strongly affects the flow. The location of 345 the hydraulic jump is not perfectly estimated by the numerical model and leads to the strong 346 error in the discharge distribution for flow 2; but to a fair estimation for flows 1 and 3 (Fig. 9). 347 This result confirms the conclusion raised by Mignot, Paquier, and Rivière (2008) that a slight 348 error in the hydraulic jump location can (but not necessarily) lead to strong errors in the 349 discharge distribution.
- for flow 11 that is the shallowest flow configuration  $(h_{u0}/b=0.08)$  for which the water depth 351  $(h_{u0}=2.4 \text{ cm})$  hardly exceeds the elevation of the sidewalk (z=2 cm), so that the flow conditions 352 do not fit the shallow water equations hypothesis.

#### 353 4 Conclusions

354 Numerical simulations of experimental bifurcation flows have been carried out with the Rubar20 code 355 that solves the two-dimensional shallow water equations. Comparison of simulation results with 356 experimental measurements allowed to assess the code ability to model nine series (one without 357 obstacle, seven with one single obstacle and one with sidewalks) of 14 subcritical dividing flows with 358 varying Froude number, discharge distribution and water depths. The computation of the discharge 359 distribution for the flows without obstacles/sidewalks was achieved with a fair accuracy (error 360 typically less than 2%, Table 2), without specific calibration of the numerical model although better results are obtained using a small positive value of the eddy viscosity K (typically  $5 \times 10^{-4}$  to  $10^{-3}$  m<sup>2</sup>/s 361 362 for the cases considered herein). Therefore, the prediction of the discharge distribution for subcritical 363 dividing flows with the shallow water equations (code Rubar20) appears to be more accurate than for supercritical (Mignot, Paquier, & Rivière, 2008) or transcritical flows (El Kadi Abderrezzak et al., 364 365 2011).

366 The simulations of the flows with obstacle proved the capacities of the code to fairly predict 367 the effects of one obstacle on the flow (deflections, contractions and accelerations) and the 368 consequences on the discharge distribution to the downstream channels. It appeared that a relatively 369 coarse mesh (3.5 to 5 cm cells, i.e. of same size as the typical size of the obstacle) is sufficient to fairly 370 compute the large-scale effects (mostly the flow distribution to the downstream branches) of the 371 obstacle. Deeper analysis shows that errors mainly arise i) when hydraulic jumps take place in the 372 crossroad and/or ii) when the obstacle is introduced in the upstream channel (obstacles 1 and 2), the 373 wake of which strongly impacts the flow in the area where it is deflected towards the branch channel. 374 Improving this calculation may require the use of a finer mesh and a more elaborate turbulence model, 375 but both aspects would be too costly to be included when simulating urban floods in large urban areas.

The authors wish to warn any potential user of the present model that, regarding the test functions  $\delta$  and  $\sigma$  indicated in Tables 2 and 3, the model appears to be substantially insensitive to the calibrated parameters within their range of variations considered herein. Nevertheless, we showed that these parameters do affect specific details of the flow patterns such as the recirculation zone (an increasing *K* value tends to reduce its length as shown in Fig. 3) or the wake downstream the obstacle (a refined mesh tends to reduce the wake streamwise extension in Fig. 6a and leads to a better estimate of the discharge distribution in Fig. 5).

Modelling of flows with sidewalks was performed with two topographical models and both fairly predict the impact of the sidewalks on the discharge distribution. The calculations with the simplified geometry show that the simulation of the average flow acceleration in a channel section is sufficient to predict the impact of the sidewalks. The main discrepancies appear to be related to 1) the occurrence of supercritical flows in the crossroad or 2) a very limited water depth over the sidewalk. This suggests that the effect of the sidewalks is mainly related to the upstream flow acceleration, and 389 that there is no significant impact on the mechanism of flow division (except when the flow in the 390 junction becomes supercritical).

391 Finally, the potential change in flow distribution due to obstacles of typical dimension equal to 392 1/6 of the street in a subcritical flow with aspect ratios ranging between 0.08 and 0.22 can exceed 393 10%. These results are likely to be generalized to a large set of urban cases because the laboratory 394 experiments are scaled to a specific crossroad in a city and are using physically based shallow water 395 equations. On the other hand, the literature (Mignot, Paquier, & Haider, 2006; Paquier & Bazin, 2014) 396 showed that the uncertainty associated to other parameters of the urban flood calculation such as the 397 hydrographs, neglecting the sewage network, some open areas within blocks, or the rain falling on the 398 computed domain may lead to comparable levels of uncertainty. Because the inclusion of obstacles 399 does not require a more detailed mesh, modellers could easily explicitly integrate such obstacles into 400 their urban flood models, which would improve the calculation of the flow distribution and the 401 representation of the local flow patterns. Nevertheless, the error made when neglecting obstacles of 402 smaller dimensions should be lower than 10%. The decision to include obstacles or not will depend on the exact order of magnitude of the other sources of uncertainty, the number, size and location of the 403 404 obstacles and finally the availability of obstacles data.

#### 405 Notation

406	b	=	channel width (m)
407	С	=	weir crest height (m)
408	f	=	friction factor (-)
409	F	=	Froude number (-)
410	g	=	constant of gravity (m s <sup>-2</sup> )
411	h	=	water depth (m)
412	Κ	=	eddy viscosity coefficient (m <sup>2</sup> s <sup>-1</sup> )
413	$K_s$	=	Strickler coefficient $(m^{1/3} s^{-1})$
414	k <sub>s</sub>	=	roughness height (m)
415	т	=	mesh resolution (cm)
416	п	=	Manning roughness coefficient (s m <sup>-1/3</sup> )
417	Q	=	discharge (m <sup>3</sup> s <sup>-1</sup> )
418	$Q^*$	=	discharge distribution error (%)
419	R	=	Reynolds number (-)
420	$R_h$	=	hydraulic radius (m)
421	$R_q$	=	discharge distribution (-)
422	t	=	time (s)
423	<i>u</i> , <i>v</i>	=	velocity components (m s <sup>-1</sup> )

424	$Z_b$	=	bottom elevation (m)
425	Ζ	=	elevation (m)
426	δ	=	average
427	υ	=	kinematic viscosity (m <sup>2</sup> s <sup>-1</sup> )
428	σ	=	root mean square deviation
429	$\Delta R_q$	=	discharge modification (%)
430			
431	Subsci	<u>ripts</u>	
432	и	=	upstream channel
433	d	=	downstream channel
434	b	=	branch channel
435	0	=	without obstacle
436	1-7	=	obstacle number
437	10	=	with sidewalks
438	SIM	=	simulated
439	MES	=	measured
440			

#### 441 References

Bazin, P.H. (2013). *Flows during floods in urban areas: Influence of the detailed topography and exchanges with the sewer system* (PhD thesis). University Claude Bernard - Lyon I, Lyon.

Bravo, H. R., & Holly, F. M. (1996). Turbulence model for depth-averaged flows in navigation
installations. *Journal of Hydraulic Engineering*, *ASCE*, *122(12)*, 718-727.

446 EEA (2010). Mapping the impacts of natural hazards and technological accidents in Europe An
447 overview of the last decade. Copenhagen, Denmark, European Environment Agency (EEA), 146.

El Kadi Abderrezzak, K., Paquier, A., & Mignot, E. (2009). Modelling flash flood propagation in
urban areas using a two-dimensional numerical model. *Natural Hazards*, *50(3)*, 433-460.

450 El Kadi Abderrezzak, K., Lewicki, L., Paquier, A., Riviere, N., & Travin, G. (2011). Division of

- 451 critical flow at three-branch open-channel intersection. *Journal of Hydraulic Research*, 49(2),
  452 231-238.
- Ghostine, R., Mignot, E., Abdallah, M., Lawniczak, F., Vazquez, J., Mose, R., & Gregoire, C. (2010).
  Discontinuous Galerkin Finite-Element Method for Simulation of Flood in Crossroads. *Journal of Hydraulic Engineering*, *ASCE*, *136(8)*, 474-482.
- Jiang, C., Yang, C., & Liang, D. (2009). Computation of shallow wakes with the fractional step finite
  element method. *Journal of Hydraulic Research*, *47(1)*, 127-136.

- Khan, A. A., Cadavid, R., & Wang, S. S. Y. (2000). Simulation of channel confluence and bifurcation
  using the CCHE2D model. *Proceedings of the Institution of Civil Engineers-Water Maritime and Energy*, *142(2)*, 97-102.
- Leandro, J., Chen, A. S., Djordjevic, S., & Savic, D. A. (2009). Comparison of 1D/1D and 1D/2D
  Coupled (Sewer/Surface) Hydraulic Models for Urban Flood Simulation. *Journal of Hydraulic Engineering, ASCE, 135(6),* 495-504.
- Lloyd, P. M., & Stansby; P. K. (1997). Shallow-water flow around model conical islands of small side
  slope .1. Surface piercing. *Journal of Hydraulic Engineering*, *ASCE*, *123(12)*, 1057-1067.
- 466 Mignot, E., Paquier, A., & Haider, S. (2006). Modeling floods in a dense urban area using 2D shallow
  467 water equations. *Journal of Hydrology*, *327(1-2)*, 186-199.
- 468 Mignot, E., Paquier, A., & Ishigaki, T. (2006). Comparison of numerical and experimental simulations
  469 of a flood in a dense urban area. *Water Science and Technology*, *54(6-7)*, 65-73.
- 470 Mignot, E., Paquier, A. & Riviere, N. (2008). Experimental and numerical modeling of symmetrical
  471 four-branch supercritical cross junction flow. *Journal of Hydraulic Research*, *46(6)*, 723-738.
- 472 Mignot, E., Riviere, N., Perkins, R., & Paquier, A. (2008). Flow patterns in a four-branch junction
  473 with supercritical flow. *Journal of Hydraulic Engineering*, *ASCE*, *134(6)*, 701-713.
- 474 Mignot, E., Zeng, C., Dominguez, G., Li, C.-W., Rivière, N., & Bazin, P.-H. (2013). Impact of
  475 topographic obstacles on the discharge distribution in open-channel bifurcations. *Journal of*476 *Hydrology*, 494(28), 10-19.
- Mignot, E., Doppler, D., Riviere, N., Vinkovic, I., Gence, J.N., & Simoens, S. (2014). Analysis of
  flow separation using a local frame-axis: application to the open-channel bifurcation. *Journal of Hydraulic Engineering, ASCE, 140 (3),* 280-290
- Papanicolaou, A. N., Elhakeem, M., & Wardman, B. (2011). Calibration and Verification of a 2D
  Hydrodynamic Model for Simulating Flow around Emergent Bendway Weir Structures. *Journal of Hydraulic Engineering*, *ASCE 137(1)*, 75-89.
- 483 Paquier, A.,& Bazin, P.H. (2014). Estimating uncertainties for urban floods modelling. *Houille*484 *Blanche-Revue Internationale de l'eau*, 6, 13-18.
- 485 Shettar, A. S., & Murthy, K. K. (1996). A numerical study of division of flow in open channels.
  486 *Journal of Hydraulic Research*, 34(5), 651-675.
- 487 Soares-Frazão, S., & Zech, Y. (2008). Dam-break flow through an idealised city. *Journal of Hydraulic*488 *Research*, 46(5), 648-658.
- 489 Stansby, P. K. (2006). Limitations of depth-averaged modeling for shallow wakes. *Journal of*490 *Hydraulic Engineering, ASCE, 132(7),* 737-740.
- 491 Van Emelen, S., Soares-Frazao, S., Riahi-Nezhad, C. K., Chaudhry, M. H., Imran, J., & Zech, Y.
- 492 (2012). Simulations of the New Orleans 17th Street Canal breach flood. *Journal of Hydraulic*
- 493 *Research, 50(1),* 70-81.

- 494 Yen, B. C. (2002). Open Channel Flow Resistance. *Journal of Hydraulic Engineering, ASCE, 128(1),*495 20-39.
- 496 Yulistiyanto, B., Zech, Y. & Graf, W. H. (1998). Flow around a cylinder: Shallow-water modeling
  497 with diffusion-dispersion. *Journal of Hydraulic Engineering, ASCE, 124(4),* 419-429

Corrigo	Flow	$Q_{u0}$	$F_{\mathrm{u0}}$	$R_{q0}$	$h_{u0}/b$
Series		Ls <sup>-1</sup>	-	-	-
	1	6.01	0.79	0.39	0.13
	2	4.99	0.60	0.39	0.14
<b>S</b> 1	3	4.01	0.45	0.39	0.15
51	4	3.00	0.33	0.39	0.15
	5	2.51	0.28	0.40	0.15
	6	2.00	0.23	0.38	0.14
	7	4.00	0.44	0.23	0.15
	3	4.01	0.45	0.39	0.15
S2	8	4.00	0.45	0.51	0.15
	9	4.00	0.44	0.65	0.15
	10	3.99	0.45	0.80	0.15
	11	1.66	0.44	0.40	0.08
	12	2.77	0.45	0.38	0.12
S3	3	4.01	0.45	0.39	0.15
S3	<b>3</b> 13	<b>4.01</b> 5.38	<b>0.45</b> 0.45	<b>0.39</b> 0.39	<b>0.15</b> 0.18

499 Table 1: Main parameters of the 14 flow configurations without obstacles.

501 Table 2: Numerical parameters for the 7 runs and simulation quality indicators without obstacle.

Dum	т	Κ	$\delta(Q^*_{b0})$	$\sigma(Q^*_{b0})$
Kull	cm	$m^{2}s^{-1}$	%	%
А	3.5-5	0	-1.88	2.50
В	3.5-5	5.0x10 <sup>-4</sup>	-1.73	2.34
С	3.5-5	1.0x10 <sup>-3</sup>	-1.71	2.36
E1	0.5	0	-0.98	1.75

E2	0.5	2.0x10 <sup>-4</sup>	-1.19	2.14
D	0.5	5.0x10 <sup>-4</sup>	-0.99	1.99
E3	0.5	1.0x10 <sup>-3</sup>	-1.11	1.97

503 Table 3: Numerical parameters for the 4 runs and simulation quality indicators with obstacle.

Run	т	Κ	$\delta(\Delta R_{q1-7})$	$\sigma(\Delta R_{q1-7})$	$\delta(Q^*_{b1-7})$	$\sigma(Q^*_{b1-7})$
Kull	cm	$m^2 s^{-1}$	%	%	%	%
А	3.5-5	0	-0.37	1.13	-2.80	3.79
В	3.5-5	$5.0 \times 10^{-4}$	-0.01	0.86	-1.70	2.71
С	3.5-5	1.0x10 <sup>-3</sup>	0.17	0.85	-1.21	2.67
D	0.5	5.0x10 <sup>-4</sup>	-0.08	0.71	-1.27	2.50

Table 4: Numerical parameters and indicators on the discharge distribution evolution for thesimulations of flows with sidewalks

Run	Run Topography	Κ	$\delta(\Delta R_{q10})$	$\sigma(\Delta R_{q10})$	$\delta(Q^*_{b10})$	$\sigma(Q_{b10}^*)$
Kull		$m^2 s^{-1}$	%	%	%	%
1	Ref	5.0x10 <sup>-4</sup>	-0.21	2.08	-2.24	5.72
2	Ref	1.0x10 <sup>-3</sup>	0.01	1.88	-1.14	5.28
3	Avg	5.0x10 <sup>-4</sup>	-0.48	2.05	-2.90	5.58
4	Avg	1.0x10 <sup>-3</sup>	-0.37	1.99	-2.30	5.32

5	1	0
-		-

511 Figure 1: Obstacles (a) and sidewalks (b) configurations

512

513 Figure 2 : a: Measured velocities at elevation z = 3 cm (Exp) and simulated depth averaged velocities 514 (runs A, B, C, D) around the crossroad for flow 6 without obstacle. For the fine mesh run, only a 515 selection of the computed velocities is shown. b: Comparison of the measured and simulated (using 516 runs B and D) of the streamwise (u) and transverse (v) mean velocity components along the x=0.15m 517 profile. 518 519 Figure 3: Extent of the recirculation zones measured at z=3 cm (Exp) and computed (depth averaged) for runs A (K=0) and C ( $K=10^{-3}$  m<sup>2</sup> s<sup>-1</sup>), for flow 6 without obstacles. 520 521 522 Figure 4: Comparison of simulated (SIM) and measured (MES) evolutions of discharge distribution for 523 each obstacle for runs B (coarse mesh) and D (fine mesh). Result of the linear regression is indicated 524 with the grey line, along with its slopes. 525 526 Figure 5: Measured ( $\blacksquare$ ) and simulated (runs B with coarse mesh ( $\Delta$ ) and D with fine mesh (o)) 527 evolutions of the discharge distribution for each obstacle configuration (7 lines) and each flow 528 grouped according to the 3 series (3 columns). 529 530 Figure 6: a: Measured velocities at elevation z=3 cm (Exp) and simulated depth averaged velocities 531 (runs A, B, C and D) around the crossroad for flow 6 with obstacle 2. b: Comparison of the measured 532 and simulated (using runs B and D) of the streamwise (u) and transverse (v) mean velocity 533 components along the x=0.15m profile; note that runs A and C exhibit quite similar velocity profiles as 534 run B and were thus omitted for sake of simplicity. 535 536 Figure 7: Top view of the mesh (left) and cross section (right, shown along the dotted line on the left 537 plot) with the channel bottom elevation used in models *Ref* and *Avg* for flows with sidewalks 538 simulations. 539 540 Figure 8: Measured at z=3cm (Exp) and simulated (depth averaged, with runs 1 and 2) flow velocity 541 magnitude for flow 6 with sidewalks: around the junction (a), 2D velocity components along the 542 x=0.15 m profile (b) and extent of the recirculation zones (c).

544	Figure 9: Measured (Exp) and simulated (runs 1, 2, 3 and 4) evolution of the flow discharge
545	distribution for the 14 flows with sidewalks
546	
547	Figure 10: Computed (with run 1) water levels around the junction for flows 1, 2 and 3 without
548	sidewalks (top) and with sidewalks (bottom). Supercritical flow areas ( $F>1$ ) are shown as hashed.
549	