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

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14 *Running Head* : Computing flooding of crossroads with obstacles

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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

31 ## 1 Introduction

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

38 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   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:

61   • The impact of the obstacle on the discharge distribution is strongly dependent on its location
62   with regards to the crossroad. Indeed, obstacles located within the upstream channel increase
63   the streamwise flow velocity and thus tend to reduce the lateral and increase the downstream
64   discharges. Oppositely obstacles located within the downstream channel tend to block off the
65   flow in this channel and to reduce the corresponding discharge while increasing the lateral
66   discharge. Finally for obstacles located within the branch channel, their impact depends on the
67   side of the channel in which they are introduced.

68   • This impact is a direct consequence of the modifications when introducing the obstacle of the:
69   (i) streamwise and centrifugal flow acceleration, (ii) width of the recirculation zone and (iii)
70   wake downstream the obstacle.

71   • The impact of an obstacle appears to increase as the upstream Froude number increases while
72   the modification of the normalized water depth hardly affects the results and the impact of the
73   initial flow distribution depends on the obstacle location.

74   Given the potential impacts of a single obstacle located in the vicinity of the bifurcation
75   (modification of the flow distribution up to 12%), the present paper aims at verifying the capacities of
76   operational 2D numerical models to reproduce these impacts.

77   2D modelling of subcritical dividing flows in a three branch bifurcation without obstacle was
78   carried out by Shettar and Murthy (1996) and Khan, Cadavid, and Wang (2000). Both models proved
79   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

88 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

126   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

128   same 14 flows, sidewalks (of dimensions 6cm x 2cm, Fig. 1b) were also added (without obstacles) by

129   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

131   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

133   all obstacles, and finally with the sidewalks. All these experimental configurations can be considered

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

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

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

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 K

147   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,

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

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

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

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

160 The usual range of equivalent Strickler coefficients is 85-105 $m^{1/3}s^{-1}$. 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 $m = 3.5$
183 to 5 cm in the junction, leading to 7 cells across the channels. This mesh size is indeed representative
184 of 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

191   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)

196   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} m^{2}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

203   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

205   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 δ and root

207   mean square deviation σ of the relative error of the lateral discharge Q^*_{b0} with:

208
$$Q^*_{b0} = \frac{Q_{b0,SIM} - Q_{b0,MES}}{Q_{b0,MES}} \tag{6}$$

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

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

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 ($\delta(Q^*_{b0})$<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)

219   and a minimum $\sigma(Q^*_{b0})$. Nevertheless, the optimum run without obstacle may not be the optimum run

220   when considering the obstacles. For instance, the computation of the wake created by the obstacle may

221   require a finer mesh than that of run B. As a consequence, four runs are kept for future analysis with

222   obstacles: the three cases with the coarse mesh with different eddy viscosity values (A, B and C) and a

223   fourth one with the finer mesh (D) in order to verify whether a mesh refinement improves the

224   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~0.71), 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

238   measured data and computed runs A and C (K=0 and K=10^{-3} m^{2}s^{-1}). For run A, the recirculation area

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.

242   To conclude, the numerical model appears to predict the experimental flows without obstacle

243   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***

246   Here below, the 14 flows are simulated with the 7 obstacles configurations from Fig. 1a and the 4

247   parameter runs A, B, C and D, representing a total of 14x7x4=392 calculations. First, each simulation

248   is assessed calculating the simulated and measured discharge distribution modification due to the

249   obstacle:

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

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

252 and then comparing the simulated and measured cases:

$$\Delta R_{qi} = \Delta R_{qi,SIM} - \Delta R_{qi,MES} \qquad (11)$$

254 with i=1…7 the obstacle number (Fig.1). Then, for each parameter set the average δ and root mean

255 square deviation σ of these errors are computed over the 7 obstacle configurations and the 14 flow

256 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

258 columns 6-7 of Table 3 with:

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

$$\sigma(Q^*_{b1-7}) = \sqrt{\frac{\sum_{k=1}^{14}\sum_{i=1}^{7}\left(Q^{*}_{bi}{}^2\right)}{14\text{x}7}} \qquad (13)$$

261 Table 3 reveals an overall fair ability to predict the discharge distribution when obstacles are

262 included with a typical average error lower than 3% and a root mean square error of about 3%. It

263 appears that:

264 • using the fine (D) and coarse (B) meshes results in a similar agreement with the
265   measurements.
266 • the bias δ decreases as the eddy viscosity coefficient decreases (C-B-A) which means that the
267   calculated branch discharge is reduced
268 • the higher values of the eddy viscosity coefficient (runs B-C) seem to be the best options
269   although the differences using the three values remain weak.
270 • the error of the simulation with obstacles is of the same order as the error for the flows without
271   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

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

281 Discrepancies mainly occur for:

282 • 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

284	junction. The simulation of this partially supercritical flow with obstacles 1, 2, 6 and 7 is less
285	accurate than for other flows with lower Froude numbers. This limit is related to a slight
286	uncertainty in the prediction of the actual hydraulic jump location, as already noticed by
287	Mignot, Paquier, and Rivière (2008).
288	• 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=3cm) 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 $|\Delta R_{q2}|$ 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 ΔR_{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*
307	As the sidewalks containing vertical edges are submerged, the meshing challenge lies in an adequate
308	representation of their topography. Cells slope in 2D models is limited by their dimensions for
309	practical reasons (computation times), so an adaptation of the mesh is required. The mesh considered
310	in the present section is made of regular 2 cm width square elements everywhere except on the
311	sidewalks edges, where finer elements of 0.5 cm or 1 cm are used (Fig.7). Using this mesh, two
312	different topographical representations are used (Fig. 7):

313	• the reference topography (*Ref*) similar to the experimental geometry.
314	• the simplified averaged topography (*Avg)*, for which the whole channel bottom elevation is
315	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 δ and root mean
320 square error σ 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:

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

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.
337   Measured and simulated modifications of the discharge distribution due to the sidewalks are
338 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
339 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 ΔR_q
341   decrease as F_{u0} increases) observed in the experiments for F_{u0}~0.6-0.8 is rather occurring for
342   F_{u0}~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.
350   • for flow 11 that is the shallowest flow configuration (h_{u0}/b=0.08) for which the water depth
351   (h_{u0}=2.4cm) hardly exceeds the elevation of the sidewalk (z=2cm), so that the flow conditions
352   do not fit the shallow water equations hypothesis.

4   Conclusions

Numerical simulations of experimental bifurcation flows have been carried out with the Rubar20 code that solves the two-dimensional shallow water equations. Comparison of simulation results with experimental measurements allowed to assess the code ability to model nine series (one without obstacle, seven with one single obstacle and one with sidewalks) of 14 subcritical dividing flows with varying Froude number, discharge distribution and water depths. The computation of the discharge distribution for the flows without obstacles/sidewalks was achieved with a fair accuracy (error 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 5×10^{-4} to 10^{-3} m^2/s for the cases considered herein). Therefore, the prediction of the discharge distribution for subcritical 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., 2011).

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

403   the exact order of magnitude of the other sources of uncertainty, the number, size and location of the

404   obstacles and finally the availability of obstacles data.

405   **Notation**

406   b =   channel width (m)

407   C =   weir crest height (m)

408   f =   friction factor (-)

409   F   =   Froude number (-)

410   g =   constant of gravity (m s^{-2})

411   h =   water depth (m)

412   K =   eddy viscosity coefficient (m^2 s^{-1})

413   K_s =   Strickler coefficient (m$^{1/3}$ s^{-1})

414   k_s =   roughness height (m)

415   m =   mesh resolution (cm)

416   n =   Manning roughness coefficient (s m$^{-1/3}$)

417   Q =   discharge (m^3 s^{-1})

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   u, v =   velocity components (m s^{-1})

424	Z_b	=	bottom elevation (m)
425	z	=	elevation (m)
426	δ	=	average
427	υ	=	kinematic viscosity ($m^2\ s^{-1}$)
428	σ	=	root mean square deviation
429	ΔR_q	=	discharge modification (%)
430			

431 Subscripts

432	u	=	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			

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498

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

Series	Flow	Q_{u0} Ls^{-1}	F_{u0} -	R_{q0} -	h_{u0}/b -
	1	6.01	0.79	0.39	0.13
	2	4.99	0.60	0.39	0.14
	3	**4.01**	**0.45**	**0.39**	**0.15**
S1	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**
	13	5.38	0.45	0.39	0.18
	14	7.00	0.45	0.39	0.22

500

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

Run	m cm	K m^2s^{-1}	$\delta(Q^*_{b0})$ %	$\sigma(Q^*_{b0})$ %
A	3.5-5	0	-1.88	2.50
B	3.5-5	5.0×10^{-4}	-1.73	2.34
C	3.5-5	1.0×10^{-3}	-1.71	2.36
E1	0.5	0	-0.98	1.75

E2	0.5	2.0×10^{-4}	-1.19	2.14
D	0.5	5.0×10^{-4}	-0.99	1.99
E3	0.5	1.0×10^{-3}	-1.11	1.97

502

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

Run	m	K	$\delta(\Delta R_{q1\text{-}7})$	$\sigma(\Delta R_{q1\text{-}7})$	$\delta(Q^*_{b1-7})$	$\sigma(Q^*_{b1-7})$
	cm	$m^2\,s^{-1}$	%	%	%	%
A	3.5-5	0	-0.37	1.13	-2.80	3.79
B	3.5-5	5.0×10^{-4}	-0.01	0.86	-1.70	2.71
C	3.5-5	1.0×10^{-3}	0.17	0.85	-1.21	2.67
D	0.5	5.0×10^{-4}	-0.08	0.71	-1.27	2.50

504

505

506   Table 4: Numerical parameters and indicators on the discharge distribution evolution for the

507   simulations of flows with sidewalks

Run	Topography	K	$\delta(\Delta R_{q10})$	$\sigma(\Delta R_{q10})$	$\delta(Q^*_{b10})$	$\sigma(Q^*_{b10})$
		$m^2 s^{-1}$	%	%	%	%
1	*Ref*	5.0×10^{-4}	-0.21	2.08	-2.24	5.72
2	*Ref*	1.0×10^{-3}	0.01	1.88	-1.14	5.28
3	*Avg*	5.0×10^{-4}	-0.48	2.05	-2.90	5.58
4	*Avg*	1.0×10^{-3}	-0.37	1.99	-2.30	5.32

508

509

510

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=3cm (Exp) and computed (depth averaged)

520   for runs A (K=0) and C (K=10^{-3} m^2 s^{-1}), for flow 6 without obstacles.

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 (■) and simulated (runs B with coarse mesh (Δ) 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=0.15m profile (b) and extent of the recirculation zones (c).

543

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