Author response to Anonymous Referee #1 for “Hydrodynamics of long-duration urban floods: experiments and numerical modelling”

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We deeply acknowledge the Referees for their detailed analysis of our manuscript and their valuable inputs. We provide hereafter a point-by-point response to the main comments by Anonymous Referee #1. The corresponding changes will be implemented in the revised version of the manuscript.

General comments

The paper deals with the computation of large urban floods; it is thus perfectly in the scope of the journal. The paper provides firstly an interesting database gathered on a huge laboratory set-up reproducing several streets crossing to form a block. This database is then used to evaluate a 2Dh computational code based on shallow water equations. This code is used to perform a sensitivity analysis to roughness, grid refinement and turbulence models. A discussion addresses the problem of upscaling and the improvements with to the use of a porosity model. The paper is easy to read and well written. It would benefit from addressing some remaining issues. **I would recommend the paper for publication after minor revisions.**

Thank you.

In the present form, the paper is classically written with (i) experiments, (ii) code evaluation and (iii) additional simulations. Yet, this code was already used and evaluated in several studies cited in the reference list. The novelty of the paper, apart from the experiments in a very original facility, would take benefits from addressing more deeply some physical aspects, with the help of the additional flow features provided by the simulations. First, the results are very little influenced by the roughness. But, what was the expected effect of the roughness and why it has no influence? This could be attributed to a predominant role of control sections, but the modification of the recirculation zone when changing the turbulence modelling seems to hardly affect the discharge distribution. This should be commented. The two preceding questions are connected to a last one: is there an explanation to have a 60%-40% downstream distribution instead of 50%-50%? Is it possible to compare this distribution to the one of a single crossroad with the same boundary conditions, using one of the references cited?

The Referee recommends taking more benefit from the additional flow features made accessible by the numerical simulations. In this respect, we undertook several additional simulations and/or analyses of the results, such as:

- Figures 5 and 6 of the original manuscript have been revised to incorporate the computed crosswise variations of the flow depth, which were not available from the experimental measurements (see Figures 1 and 2 of our response to the comments of Anonymous Referee #2);
- new unsteady simulations have been performed to discuss more quantitatively to which extent the experimental observations can be transposed to real-world flood events regarded as a series of successive steady states (see Figure 1 and our response to Specific comment Q2 below);
- we have conducted extra simulations in the simplified configuration of a single "equivalent" intersection (see our response to the Specific comment Q6 below, as well as our response to Anonymous Referee #2), to better elucidate the role of different aspects of the geometry (streets width, location of wide streets, streets inclination) on the partition of the outflow discharge (~ 40 % through the east fact *vs*. ~ 60 % through the south face);
- …

The effects of roughness and control sections are also further discussed in our responses to the specific comments of the Referees.

2 Specific comments

Q1/ Up to 7 authors co-signed the paper. This paper has a very significant experimental part. From references throughout the manuscript and from the acknowledgements, it can be understood that a significant part of these experiments were performed by Araud. Yet, (s)he is not one of the co-authors. This is maybe justified but is mandatory to be checked.

As detailed in the Acknowledgement section of the original manuscript, M. Debaucheron helped with numerical simulations and Q. Araud performed experimental measurements; but they both did not take part in the analysis presented in this paper nor contributed to writing the paper. Therefore, we deemed appropriate to acknowledge their contributions.

Q2/ Page 3, it is stated that "flash floods" are out of scope of the study. Nevertheless, the time scales associated with present study and flash floods should be detailed (can't it be considered as a succession of steady states), so as the time scale which is considered to discriminate the two kinds of floods. This time scale could be addressed in the section about upscaling, and discussed along the time scales characterizing the laboratory small-scale experiments.

Flash floods are often associated to short duration and intrinsically transient flood events. This is however not always the case, as stated correctly by the Referee and also, for instance, by Gaume *et al.* (2009):

"*The duration* ... (of flash) *floods depends on the causative storm and hence on the climatic setting. Most generally, the storms inducing flash floods lead to local rainfall accumulations exceeding 100 mm over a few hours ... longer lasting stationary storm events may, however, occur in some meteorological contexts, especially in the Mediterranean region*."

Therefore, we have removed from the Introduction the sentence "*Flash floods are therefore out of the scope of the present study*". Instead, we have elaborated the following discussion.

We undertook extra simulations in unsteady mode, with the purpose of identifying a characteristic time scale of the experimental model. The initial condition corresponds to virtually no water in the model (initial water depth = 1 mm) and no flow. At the initial time, the inflow was suddenly raised to its maximum value upstream of each street. We considered inflow discharges of 20, 60, 80 and 100 m³/h, as well as the following parameters: $\phi_{\text{west}} = 50$ %, no turbulence model, smooth bottom ($k = 0$ m) and $\Delta x = 1$ cm.

Time series of computed water depths in the centre of the most upstream intersection (i.e. between streets 1 and A) and between the two wide streets 4 and C are displayed in Figure 1. They reveal that the time necessary for reaching a steady state is of the order of 30 ÷ 60 s at the scale of the laboratory model.

The scale factor for time is given by the ratio between the scale factor for horizontal lengths ($1 / e_H$) and the scale factor for velocity. The latter is the square root of the scale factor for vertical dimensions ($1 / e_V$), consistently with the Froude similarity adopted here. Hence, the characteristic times obtained at the scale of the laboratory model must be magnified by $e_H / (e_V)^{0.5}$ to obtain the corresponding characteristic times at the prototype scale.

- For Prototype 1, this leads to a magnification factor of $200 / 200^{0.5}$, hence to a characteristic time of the order of 7 ÷ 14 min.
- For the more realistic Prototypes 2 and 3, a magnification factor of $200 / 20^{0.5}$ is obtained, which leads to a characteristic time of the order of 20 ÷ 45 min.

In conclusion, the observations of the present research remain valid provided that the considered flood events remain sufficiently gradual, i.e. that the characteristic time scales of the flood waves remain above 20 ÷ 45 min. This will be explained in the revised manuscript and, by the way, is consistent with the title of the manuscript ("long-duration urban floods").

> Q3/ About boundary conditions: I understand from the text that the experiments were conducted with a horizontal bottom but this should be stated more precisely in the manuscript. Could the authors confirm, as it seems to be stated L26-27 page 4, that the linear (q=Q/b) inlet flow rate was constant within all the inlet streets? Finally, are the free-flow conditions performed thanks to chutes?

The use of a horizontal bottom will be clearly stated at the beginning of section 2.1.1 in the revised manuscript.

We also confirm that the specific discharge (i.e. discharge per unit width of the street at inlet) was the same at the inlet of all streets of a given face (west or north). The specific discharge differs from one face to the other because the total inlet width is different (one wide street along the west face *vs.* two wide streets along the north face). Moreover, in tests with $\phi_{west} \neq 50$ %, the specific discharge obviously differs from one face to the other.

The free-flow conditions at the outlet of each street are realized experimentally thanks to chutes, as shown in Figure 2.

Q4/ Can some details be provided about the "optical gauge" P5L7? The uncertainty seems quite high (+-1mm) compared to classical devices.

Water level measurements were conducted using an optical gauge fixed on an automatic traverse system. The gauge detects the phase (air *vs.* water) in which it is located. The measurement uncertainty results therefore mainly from the accuracy of the motor, which was estimated at ± 1 mm. This will be explained in the revised version of the manuscript (section 2.1.2).

This optical device was preferred here compared to more standard acoustic techniques due to the narrow character of the streets which would have induced reflections of the sound waves on the walls and disruptions in the measurements if an acoustic device had been used.

Q5/ I completely agree with authors about the use of a Darcy-Weisbach coefficient instead of a Manning coefficient (p6L11). Nevertheless, the reason could be stated more clearly. Both of these coefficients are "process oriented": one for the channel, one for the pipes. As far as I am concerned, the use of a Darcy-Weisbach coefficient is required here due to the limited values of the Reynolds numbers in the laboratory experiments. The "fully rough regime" is not guaranteed, which prevent from using safely a Manning coefficient depending only on the wall roughness.

The Referee is right. This will be stated as follows in section 2.2 of the revised manuscript:

"*Also, the experimental conditions do not a guarantee a hydraulic rough flow regime, which is necessary for applying Manning formula*."

Q6/ P7 L8-15 : the downstream discharge distribution is about 60% in the streamwise (inlet) direction and 40% in the crosswise direction. This is not 50%-50% and can some reasons be proposed for this ratio: a slope (but I understand the slope was nil, see Q3), a reference with a single crossroad?

The bottom is indeed flat, as will be mentioned in section 2.1.1 of the revised manuscript. Nonetheless, for $\phi_{west} = 50$ %, the experimental observations indicate that the outflow through the east face is about 40 % of the total inflow, while the outflow through the south face is 60 % of the total inflow. The numerical computations also confirm these observations (see Figure 2 of the original manuscript).

Surely, this is partly explained by the total flow width available along the north-south direction compared to the west-east direction. Indeed, only one "wide" street (street 4) is aligned along the west-east direction, whereas two of them (streets C and F) convey the flow in the north-south direction. As a result, the total flow width in the west-east direction is 42.5 cm, which is lower than the total flow width along the north-south direction (50 cm). Consequently, the available flow width along the east face is 46 % of the total outflow width, whereas it is 54 % for the south face. This difference goes in the same direction as the difference in outflows (40 % *vs.* 60 %).

[See also our response to Anonymous Referee #2.]

To test this hypothesis of “attraction” effect of the wider streets, we undertook additional simulations corresponding to a single “equivalent” 4-branch intersection, with the north-south and west-east streets widths respectively equal to 0.5 m and 0.425 m. These widths mimic the cumulative street widths along the north-south and the west-east directions in the experimental model (respectively equal to 0.5 m and 0.425 m). We performed the simulations for the two extreme discharges (20 m^3/h and 100 m^3/h), with equal inflow partition between the west and north faces (ϕ_{west} = 50 %) and assuming a smooth bottom (k = 0 m). Free flow boundary conditions were prescribed at the downstream end of each street, located at a long distance downstream of the crossroad (8 times the street width). We used the finest grid spacing considered in the paper (Δx = 2.5 mm) and we tested the computations with and without activation of the turbulence model.

For both inflow discharges (20 m^3/h and 100 m^3/h), the computed results reveal a partition of the outflow discharge proportional to the street widths (54 % *vs.* 46 %). The same results were obtained with and without activation of the turbulence model. Nonetheless, this geometric effect explains only partly the difference in the experimentally observed outflow discharges (60 % *vs*. 40 %). We attribute the remaining difference to features which are not properly reflected in the single “equivalent” intersection considered here (e.g., the spatial distribution of the wider streets within the scale model); but which are expected to further amplify the difference in the outflow discharges between the north-south and the west-east directions. Particularly, the downstream parts of the streets aligned along the west-east direction (streets 1 to 7) are all inclined towards the north, i.e. towards upstream as far as the north-south direction is concerned. This surely contributes to further reduce the outflow through the east face. This is will be explained in section 3.1.1 of the revised manuscript.

Q7/ P9 L4-8. The location of the water depth profiles drawn should be specified: middle (centreline) of the street, average on a section, : : : Notably, "significant variations" are commented but the comments should account for a possible crossing of the recirculation zones or, instead, of the vena contracta. I expect slightly different comments regarding one case or the other.

The water depth profiles are drawn along the centreline of the streets because experimental data have not been collected elsewhere. This will be explicitly stated in section 3.1.4 of the revised manuscript. For the sake of consistency, the displayed computed water depths were also taken along the centreline of the streets.

So far, water depths have not been measured beyond the streets centreline because the experimental procedure for conducting water level measurements was particularly slow. For each test, water levels were measured at about 600 locations along the centreline of the streets using the automatic traverse system (see also Q. 4). A single survey of this type (600 points) took almost one whole day. This will be detailed in section 2.1.2 of the revised manuscript. In the future, more detailed water level measurements will be performed in the near-field of the street intersections.

In section 3.1.4, the wording “most significant variations” refers to variations in the *streamwise* direction. This will be clarified in the text of the revised manuscript. However, we have no experimental information available to state whether the observed profiles along centreline of the streets cross recirculations and/or the *vena contracta* in the experiments.

As detailed in our response to Anonymous Referee #2, we have revised Figures 5 and 6 of the original manuscript to examine the crosswise distribution of water depths in the numerical results. This is displayed by the shaded area ( ) in Figures 1 and 2 of our response to Anonymous Referee #2. These additional data confirm that significant crosswise variations in the water depths are located immediately downstream of the street intersections, which is consistent with the location of recirculation zones and *vena contracta*.

Q8/ P10L30 : multiplying the cells by a factor 4 increases the computational cost by 8. Can this be commented?

This results from the *Courant-Friedrichs-Lewy* (CFL) condition for explicit time integration (e.g., Bates et al., 2010), which states that the time step Δt must scale with the space step Δx to preserve the stability of the numerical scheme: $\Delta t \sim \Delta x$. Hence, when the grid spacing Δx is reduced by a factor two, the number of cells in 2D increases by a factor four and the time step is reduced by a factor two. As a result, the overall computational cost increases by a factor eight. This will be detailed in the revised version of the manuscript.

Q9/ Typical values of the Froude number should be added in table 5

The typical values of the Froude number remain the same for the laboratory model and for the three prototypes, since the Froude similarity was used for upscaling the experimental results in all cases (whether distorted or non-distorted). Therefore, we prefer not to include the Froude number in Table 5; but to report its typical values (0.15 - 0.4 close to the inlets) in the main text (at the end of section 2.1.3, Test program) of the revised manuscript.

Q10/ The Reynolds number is defined using the water depth, i.e. assuming that the hydraulic diameter can be assimilated to 4h. This assumption should be valid for prototypes 2 and 3 but is more questionable for the laboratory model and the prototype 1. Was it taken into account to compute the values of the Darcy-Weisbach coefficient reported in section 5.1?

The Referee is right that the hydraulic radius may be much smaller than the water depth in the laboratory model and in Prototype 1, particularly in the case of high inflow discharge in the narrow streets. However, the definition of the Reynolds number used in the manuscript ($\mathsf{R} = 4\,h\,u\,/\,\nu$) is consistent with the standard formulation used in 2D-horizontal flow models, while the general definition based on the hydraulic radius ($\mathsf{R}_{1D} = 4\,R_h\,u\,/\,\nu$, with $R_h = b\,h\,/\,(\,b + 2\,h\,)$ and b = street width) is mostly used in the context of 1D flow modelling.

Nonetheless, we took the Referee’s remark into consideration and we display below a modified version of Table 5, in which we also include the values obtained by using R_{1D} instead of R. By the way, we also corrected the values of R for a missing factor 1/2 (in the original manuscript, we evaluated R in Table 5 considering that the *total* inflow was supplied to *each* face, which is obviously not the case).

In the end, the only consequences of using the "water depth-based" Reynolds number R instead of the "hydraulic radius-based" Reynolds number R_{1D}, are the following:

- the range of f is $2 \times 10^{-2} \div \underline{3} \times 10^{-2}$, instead of $2 \times 10^{-2} \div \underline{4} \times 10^{-2}$ for the laboratory model,
- f is estimated equal to $\sim 2 \times 10^{-2}$, instead of being in the range $2 \times 10^{-2} \div \underline{3} \times 10^{-2}$ for prototype 1,
- and the range of f is $4 \times 10^{-2} \div \underline{7} \times 10^{-2}$, instead of $4 \times 10^{-2} \div \underline{6} \times 10^{-2}$ for prototype 3.

Those slight variations in the values of f do not result in any significant change in the discussion of section 5.1 in the manuscript. Therefore, our suggestion is not to modify the definition of the Reynolds number in the manuscript as this would not add to the main message we want to convey.

3 Typing errors

> - Page 1: the third address in the authors' affiliations is not complete
>
> - P6 L25: "a first test series of tests"

This will be corrected in the revised manuscript.

> - P8 L27-30: What is the "supplement" cited twice? In case it is on the website of the journal, please do not account for this comment.

Supplements are indeed available on the website of the journal.

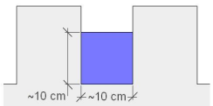
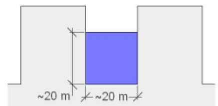
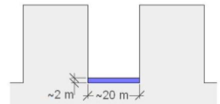
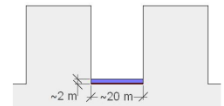
References cited here but not in the original manuscript

Gaume, E., Bain, V., Bernardara, P., Newinger, O., Barbuc, M., Bateman, A., Blaškovičová, L., Blöschl, G., Borga, M., Dumitrescu, A., Daliakopoulos, I., Garcia, J., Irimescu, A., Kohnova, S., Koutroulis, A., Marchi, L., Matreata, S., Medina, V., Preciso, E., Sempere-Torres, D., Stancalie, G., Szolgay, J., Tsanis, I., Velasco, D., Viglione, A. (2009). A compilation of data on European flash floods. *Journal of Hydrology*, **367**(1-2), 70-78.

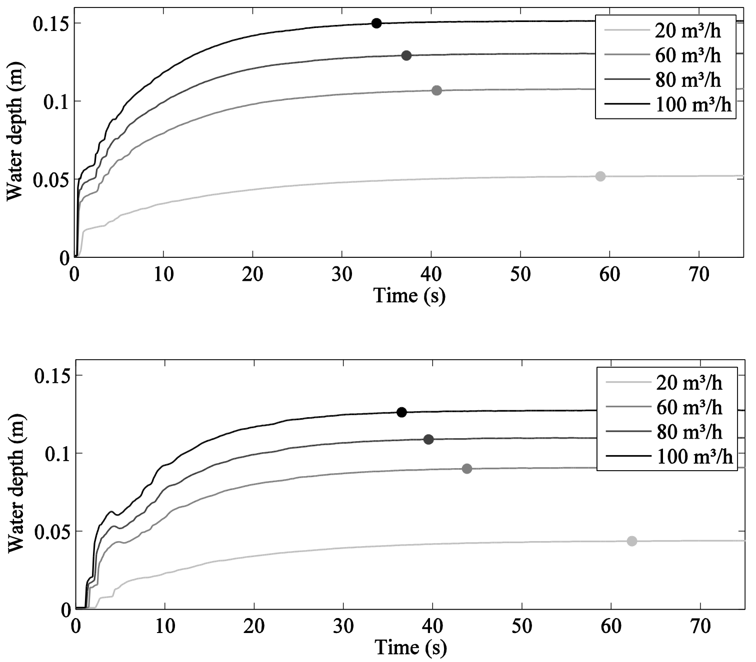
Bates, P.D., Horritt, M.S., Fewtrell, T.J. (2010). A simple inertial formulation of the shallow water equations for efficient two-dimensional flood inundation modelling. *Journal of Hydrology*, **387** (1-2), 33-45.

Tables

Table 1 (expanded version of Table 5 in the original manuscript): Characteristic Reynolds numbers R and R_{1D}, roughness height k and corresponding Darcy-Weisbach coefficient f at the laboratory model scale and at the prototype scale (real-world) as a function of the horizontal and vertical magnification factors e_H and e_V.

	Laboratory model	Prototype 1	Prototype 2	Prototype 3
				
e_H	-	200	200	200
e_V	-	200	20	20
k_s	$< 10^{-5}$ m	$\sim 5 \times 10^{-2}$ m	$\sim 5 \times 10^{-3}$ m	~ 0.1 m
Reynolds number based on the water depth: $\mathsf{R} = 4\, h\, u\, /\, \nu$				
R	$1 \times 10^{4} \div 1 \times 10^{5}$	$3 \times 10^{7} \div 4 \times 10^{8}$	$1 \times 10^{6} \div 1 \times 10^{7}$	$1 \times 10^{6} \div 1 \times 10^{7}$
$k / 4\, h$	$3 \times 10^{-5} \div 7 \times 10^{-5}$	$7 \times 10^{-4} \div 2 \times 10^{-3}$	$7 \times 10^{-4} \div 2 \times 10^{-3}$	$1 \times 10^{-2} \div 4 \times 10^{-2}$
f	$2 \times 10^{-2} \div 3 \times 10^{-2}$	$\sim 2 \times 10^{-2}$	$\sim 2 \times 10^{-2}$	$4 \times 10^{-2} \div 6 \times 10^{-2}$
Reynolds number based on the hydraulic radius : $\mathsf{R}_{1D} = 4\, R_h\, u\, /\, \nu$, with $R_h = b\, h\, /\, (\, b + 2\, h\,)$ and b = street width				
R_{1D}	$5 \times 10^{3} \div 5 \times 10^{4}$	$1 \times 10^{7} \div 2 \times 10^{8}$	$9 \times 10^{5} \div 1 \times 10^{7}$	$9 \times 10^{5} \div 1 \times 10^{7}$
$k / 4\, R_h$	$7 \times 10^{-5} \div 2 \times 10^{-4}$	$2 \times 10^{-3} \div 4 \times 10^{-3}$	$8 \times 10^{-4} \div 2 \times 10^{-3}$	$2 \times 10^{-2} \div 4 \times 10^{-2}$
f	$2 \times 10^{-2} \div 4 \times 10^{-2}$	$2 \times 10^{-2} \div 3 \times 10^{-2}$	$\sim 2 \times 10^{-2}$	$4 \times 10^{-2} \div 7 \times 10^{-2}$

Figures



(a)

(b)

Figure 1: Computed water depths in the unsteady simulations, at the intersections (a) between streets 1 and A, and (b) between streets 4 and C. The markers (•) indicate the moment when the water depth reaches 99 % of its ultimate value.

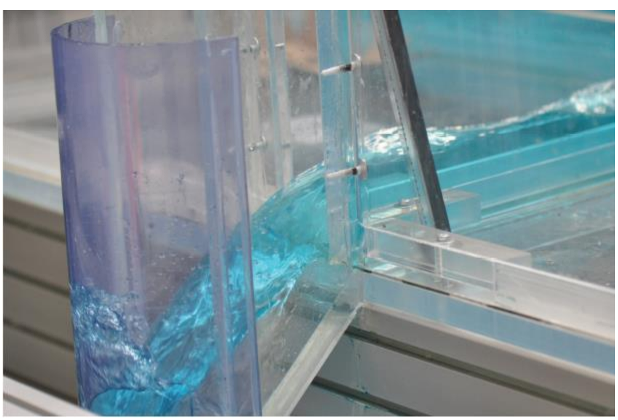


Figure 2: Outlet of a street in the experimental model.