Experimental Scaled-model as a benchmark for validation of Urban Flood models

Utilisation d'un modèle réduit expérimental comme référence pour la validation des modèles d'inondations en zone urbaine

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RÉSUMÉ

L'évolution actuelle de la modélisation des inondations urbaines a conduit au développement d´améliorations importantes des modèles d'inondations en milieu urbain. Ces modèles de programmation mathématique simulent des inondations en tenant compte de l'interaction entre l'écoulement de surface et l'écoulement dans le réseau d'assainissement. Toutefois, il est difficile d'obtenir des données de qualité sur ces inondations. Ce travail vise à présenter des données de qualité mesurées à l'échelle expérimentale (déversements et niveaux d'eau en divers points d'un canal), pour être utilisé comme test de référence et donc comme benchmark pour la validation des modèles d'inondations urbaines. De plus, les coefficients de déversement présentés sont calculés expérimentalement, permettant ainsi la validation des modèles d'assainissement. Le modèle à l'échelle est un canal rectangulaire avec renvoi du déversoir en aval et six arrivées de diamètre identique. Les résultats de l'expérience indiquent: (1) la cohérence entre les déversements mesurés par les sondes ADV/WRP et les résultats de la méthode volumétrique; (2) la cohérence des niveaux d'eau obtenus pour chaque sonde WRP; (3) une bonne corrélation entre le coefficient de déversement d'arrivée et le coefficient de déversement du déversoir en aval; (4) les arrivées au centre présentent des coefficients de déversement plus grands, de même que les arrivées en aval.

ABSTRACT

Recent advances in flood modelling of urban areas have lead to the development of more sophisticated urban flood models. These models simulate flooding events by coupling the surface flow and the sewer system with an internal boundary condition (IBC). However, good quality data of flood events are hazardous to obtain. In this work we aim to present quality data (i.e. discharges and water levels at several locations) collected from an experimental scaled-model for enabling a benchmark test for urban flood models. In addition the IBC discharge coefficients provided herein are determined experimentally enabling a consistent validation of drainage models. The scaled model comprises of a rectangular surface-channel with a downstream weir discharge, and with six inlets with identical diameter. The experimental results indicate: (1) consistency between the discharges measured by the ADV/WRP and the volumetric method (2) consistency of the water levels obtained in each WRPs; (2) Good correlation between the inlet discharge coefficients, and the weir discharge coefficient (3) The centre inlets show a higher discharge than the side inlets and the downstream inlets show a higher discharge than the upstream ones.

KEYWORDS

Scaled-model; Urban flood models; Inlet location; Surface system; Sewer system

INTRODUCTION

Recent flooding events across continents have pushed the development of more sophisticated urban flood models for the simulation of flooding (Lhomme et al. 2006; Paquier et al. 2003). These models enable simultaneous simulation of the Sewer and the Surface systems in an integrated manner. Given the lack of data for validation of drainage models, benchmarking using simulation data from numerical models has been done (Hunter et al. 2008), however, that data is not without models' original assumptions and simplifications which constrain their application (Leandro et al. 2009b). We aim to provide quality data from an experimental scaled-model.

Urban flood models simulate the connection between surface and sewer (inlet) with an internal boundary condition (IBC). The inlets IBC used can vary from model to model and they are not equivalent nor do they provide the same results (Leandro et al. 2009a); these can be an orifice, a weir, a combination of both (Kawaike and Nakagawa 2007; Mark et al. 2004), or a combination of different control sections depending on the geometry of the inlet (Leandro et al. 2007).

The study of orifice and weir discharges is based on theoretical simplifications, which cannot replicate all variables. Hence the need for complex CFD and experimental studies (Anh and Hosoda 2007; Sarker and Rhodes 2004; Yildirim et al. 2009). The best example is the flow discharge coefficient in the orifice equation that accounts for the vena contraction and the velocity reduction:

$$Q = C_d * A * \sqrt{2 * g * H} \tag{1}$$

where A = area of the orifice, and:

$$C_d = C_v * C_c \tag{2}$$

being C_v = coefficient of velocity reduction and C_c = coefficient of contraction, and C_d the orifice discharge coefficient. Although classical literature presents standard values, the discharge coefficient is known to be variable (Chanson et al. 2002).

In case of a weir the expression (1) is changed to:

$$Q = C_w * b * H * \sqrt{2 * g * H} \tag{3}$$

where C_W= weir discharge coefficient. The "Societé des Ingénieurs et Arquitectes Suisses (SIAS)" presented in 1947 the following discharge coefficient for a Bazin weir (Quintela 2007):

$$C_w = 0.410\left(1 + \frac{1}{1000\ h + 1.6}\right) * \left[1 + 0.5\left(\frac{h}{h + p}\right)^2\right] \tag{4}$$

where p = height of the weir and, h = water depth atop the weir level, and C_w= weir discharge coefficient. Equation (3) is valid for 0.8 m > h > 0.025 m ; p > 0.3 m ; p > h. The "Belgian Society of Mechanical's" (Lencastre 1996) replaced the previous weir crest with a beam and presented another weir formula for the discharge coefficient:

$$C_W = 0.41067\ \left(1 + \frac{1.8}{1000\ h}\right) * \left[1 + 0.55\left(\frac{h}{h + p}\right)^2\right] * \left(0.70 + 0.185\frac{h}{t}\right) \tag{5}$$

where t is the weir thickness. Equation (7) is valid for: 0.1m ≤ h ≤ 0.8 m ; 0.3 m ≤ p ≤1.5 m; 0.03 m ≤ t ≤ 0.23 m ; h ≤ p.

Existing experimental work fall outside the geometry conditions of our experiment. In addition, it is necessary to obtain the specific discharge coefficients to be used within the IBC of urban-flood-models. Hence, in this paper we aim to present data collected from an experimental scaled-model for building a benchmark study along with the correspondent discharge coefficients, for enabling a thorough validation of urban flood models.

EXPERIMENTS

The experimental scaled model is built inside the IMAR (Institute of Marine Research) flume channel. The IMAR flume is a rectangular channel 36.0x1.25x1.0 m prepared for multipurpose applications. A control room equipped with a “Supervision, Control and Data Acquisition” system allows management of the different parts of the system, e.g. monitoring flow rates and water levels (Santos et al. 2008).

The experimental scaled model comprises a channel of 8.0m x 0.5m x 0.5m with 1% slope, a collection chamber upstream with an adjustable gate for controlling the flow regime, six identical circular inlets at the bottom with 21mm each, and a weir outlet downstream of the main channel with 0.10m height and 0.010m thick. The gate opening is set fix with 2mm (Figure 1).

The gate opening and the downstream weir height were selected in order to keep the flow regime in the channel sub-critical and the discharge trough the inlets vortex free. Sub-critical is the typical flow regime that occurs in most critical flooding areas, e.g. low lying areas with mild slopes. Vortexes can happen in scaled-models due to scale effect, in order to minimise such effect the weir outlet level was raised to 0.10m.

For collecting data, 6 Water Resistive Probes (WRP) were placed along the channel, and one Acoustic Doppler Velocimeter (ADV) close to the first WRP1. Hence, the flow in the main channel could be verified by the volumetric method and continuity equation using the ADV and WRP1.

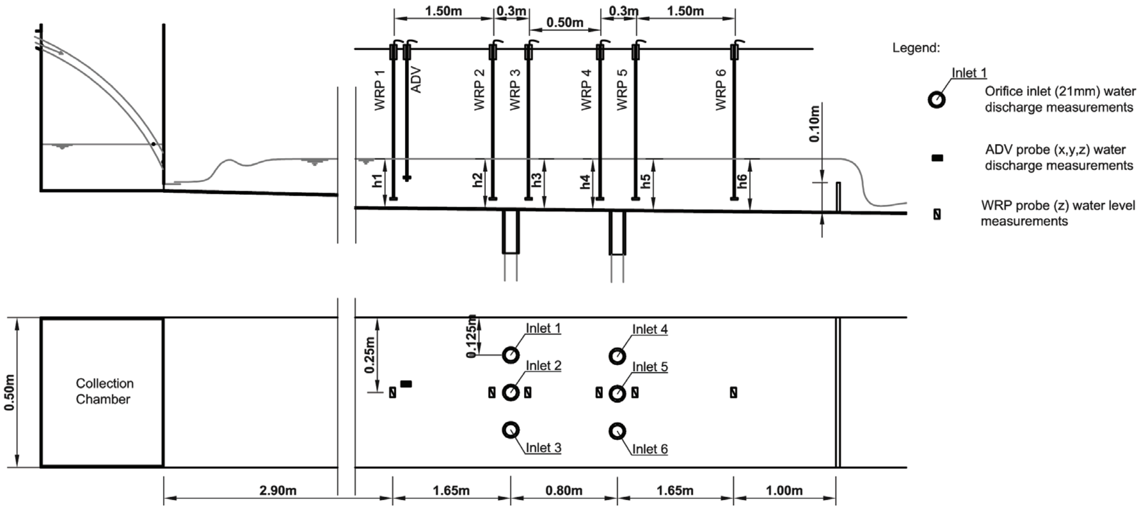


Figure 1. Profile and plant view of the experiment scaled-model.

EXPERIMENTAL RESULTS

Three different configurations of open inlets were tested for flow rate between 1 and 12 l/s: the 4 exterior inlets opened (1, 3, 4 and 6), simulating the normal positioning of the gutters, both the central inlets opened (2 and 5), and all six inlets opened (1 to 6). The selection of these specific configurations will be discussed later. Figure 1 shows the location of all six inlets.

In order to check the consistency of the experimental data collected, two types of flow measurements methods were used. The first data was collected by adding the volumetric measurements of flow through the inlets and the main channel. The second data used the water depth collected at WRP1 and the velocity in the main channel measured using the ADV (Figure 2).

Table 1 shows the main channel flow and the water-depth mean with the upper and lower 95% confidence intervals (CI) of a normal distribution for each probe and for each of three different configurations. The main discharge channel should be used as the upstream boundary condition of the drainage model.

The scaled-model specific discharge coefficients of each orifice are presented in Figure 3. As mentioned earlier these are the coefficients that need to be specified in the urban flood models internal boundary conditions (Inlets). For each inlet, there is one graph where the vertical axis represents the hydraulic head over the inlet, and the horizontal axis represents the discharge through each inlet. The three configurations selected guaranty that each inlet coefficient is calibrated with at least two independent data sets, i.e. from different configurations. For example in Figure 1 a) the first inlet (1) is obtained from the 6 opened inlets (I 6.1) and the 4 opened inlets (I 4.1). Where in (I a.b), a=the configuration ordering, b= the inlet number. The gray circle points to the correspondent inlet.

The discharge coefficient of the weir downstream is found to be a function of the water depth above the weir crest level (Figure 4). This is in agreement with previous work already discussed in the introduction. The following expression is proposed:

$$C_w = 0.150 * \ln(h) + 0.591 \qquad (6)$$

Figure 4 shows on the horizontal axis water depth above weir crest level (h), and on the vertical axis the values of C_w as a function of h.

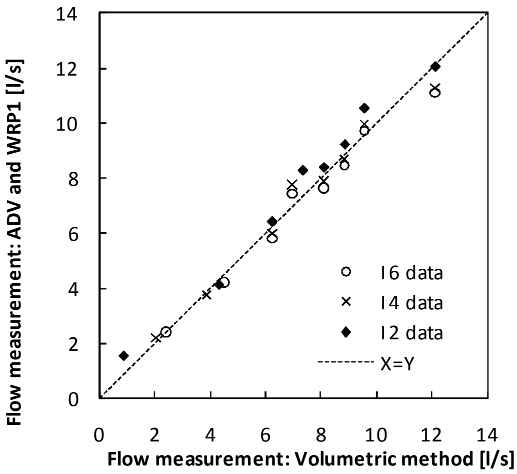


Figure 2. Consistency check of the main channel flow using volumetric method and ADV and WRP1 probes.

Table 1. Mean water-depths (h) with 95% confidence intervals (CI) at each WRP and for the three configurations studied.

Configuration 1: 4 inlets open - 1, 3, 4 and 6

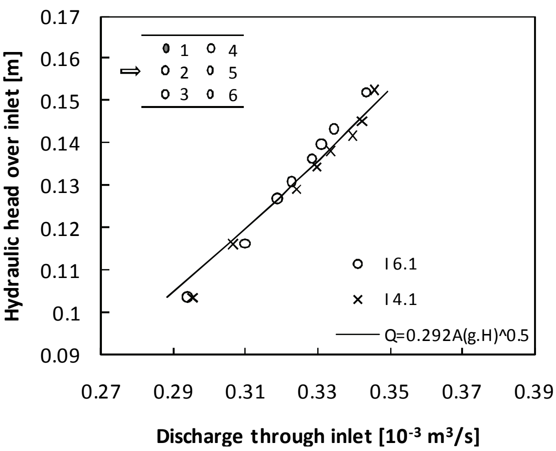
Main Channel flow	WPR1		WPR2		WPR3		WPR4		WPR5		WPR6	
[l/s]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]
12.10	0.140	0.001	0.152	0.001	0.154	0.001	0.157	0.001	0.160	0.001	0.168	0.002
9.54	0.131	0.001	0.143	0.001	0.146	0.001	0.148	0.001	0.151	0.001	0.160	0.001
8.85	0.127	0.001	0.140	0.001	0.142	0.001	0.145	0.001	0.148	0.001	0.156	0.001
8.10	0.124	0.001	0.136	0.001	0.139	0.001	0.141	0.001	0.144	0.001	0.152	0.001
6.95	0.118	0.001	0.131	0.001	0.134	0.001	0.136	0.001	0.139	0.001	0.147	0.001
6.23	0.114	0.001	0.127	0.001	0.130	0.001	0.132	0.001	0.135	0.001	0.144	0.001
4.51	0.104	0.000	0.116	0.000	0.119	0.000	0.122	0.001	0.125	0.001	0.134	0.001
2.41	0.091	0.000	0.104	0.000	0.107	0.000	0.109	0.000	0.112	0.000	0.122	0.000

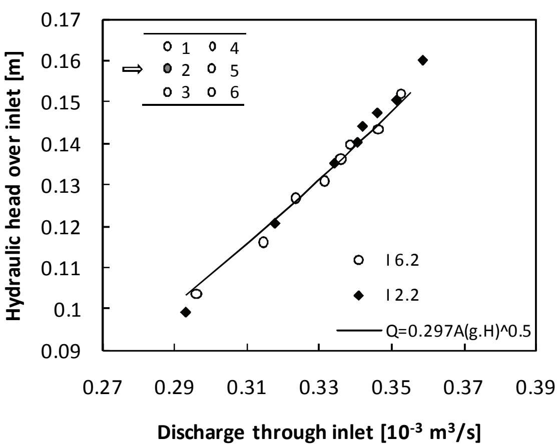
Configuration 2: 2 inlets open - 2 and 5

Main Channel flow	WPR1		WPR2		WPR3		WPR4		WPR5		WPR6	
[l/s]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]
12.10	0.142	0.002	0.153	0.002	0.155	0.002	0.158	0.002	0.161	0.002	0.169	0.002
9.54	0.134	0.001	0.145	0.001	0.148	0.001	0.151	0.001	0.153	0.001	0.162	0.001
8.85	0.131	0.001	0.142	0.001	0.145	0.001	0.147	0.001	0.150	0.001	0.159	0.001
8.10	0.127	0.001	0.138	0.001	0.141	0.001	0.144	0.001	0.147	0.001	0.156	0.001
6.95	0.123	0.001	0.134	0.001	0.137	0.001	0.140	0.001	0.143	0.001	0.152	0.001
6.23	0.118	0.001	0.129	0.001	0.132	0.001	0.135	0.001	0.138	0.001	0.147	0.001
3.86	0.105	0.000	0.116	0.000	0.119	0.000	0.122	0.000	0.125	0.000	0.135	0.000
2.05	0.093	0.000	0.103	0.000	0.107	0.000	0.109	0.000	0.112	0.000	0.123	0.000

Configuration 3: 6 inlets open - 1 to 6

Main Channel flow	WPR1		WPR2		WPR3		WPR4		WPR5		WPR6	
[l/s]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]	h [m]	± CI [m]
12.10	0.150	0.001	0.160	0.001	0.163	0.001	0.165	0.001	0.169	0.001	0.176	0.002
9.54	0.140	0.001	0.150	0.001	0.153	0.001	0.156	0.001	0.159	0.001	0.167	0.001
8.85	0.137	0.001	0.147	0.001	0.150	0.001	0.153	0.001	0.156	0.001	0.164	0.001
8.10	0.133	0.001	0.144	0.001	0.147	0.001	0.150	0.001	0.153	0.001	0.161	0.001
7.34	0.130	0.001	0.140	0.001	0.143	0.001	0.146	0.001	0.149	0.001	0.157	0.001
6.23	0.125	0.000	0.135	0.000	0.138	0.000	0.141	0.000	0.144	0.001	0.153	0.001
4.33	0.110	0.000	0.121	0.000	0.124	0.000	0.126	0.000	0.129	0.000	0.139	0.000
0.91	0.089	0.000	0.099	0.000	0.103	0.000	0.105	0.000	0.108	0.000	0.118	0.000





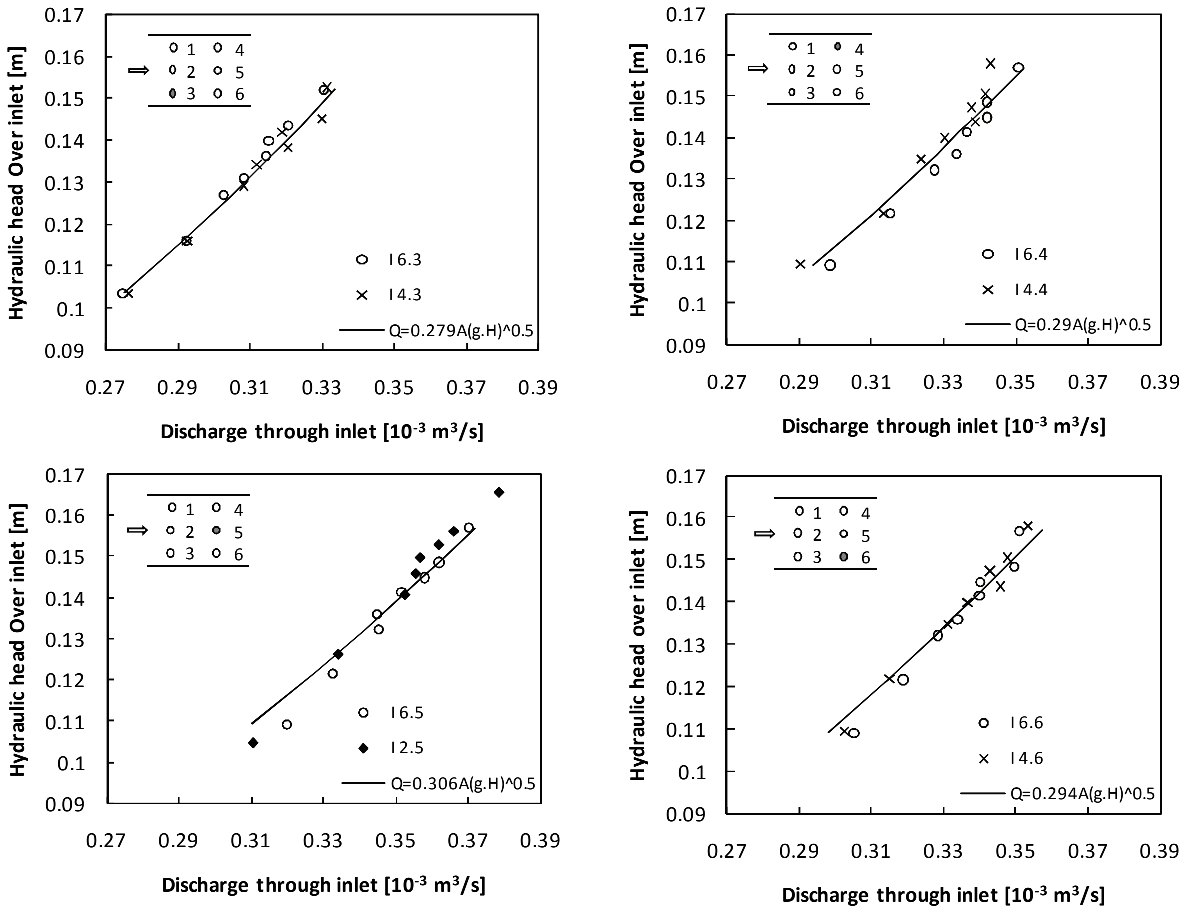


Figure 3. Scaled-model specific Inlet discharge coefficients to be used in the urban flood model internal boundary condition. Inlets graphs are ordered from the 1st to 6th inlet.

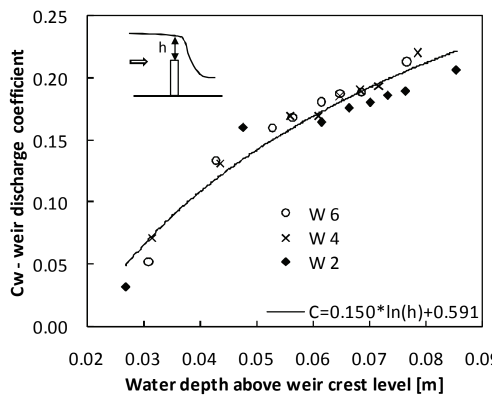


Figure 4. Scaled-model specific weir discharge coefficient $C_w = 0.150 * \ln(h) + 0.591$, to be used in the urban flood model downstream boundary condition.

DISCUSSION

The comparison between the volumetric method and the continuity equation shows that the data is consistent. Indeed the main channel flow determined by the mean velocity, collected by the ADV, and the mean water depths, obtained by the WPR1, is consistent with the total discharge flowing out of the inlets and the weir downstream.

The mean water depths obtained show a consistent increase of water depth as the water flows toward the weir downstream. As the main channel flow is gradually raised turbulence increases. This could be easily recognized visually when running the experiments and it is verified in the data with the increase of the water depths CI. Despite the flow turbulence it is still possible to guaranty 2 significant digits in the water levels readings.

The inlet discharge coefficients vary depending on the location of the inlets. The centre inlets have a higher value than the side inlets. The downstream inlets have a higher discharge than the upstream inlets. Indeed the values found could not be predicted or estimated based on classical literature, as discussed in the introduction. Despite some oscillations, all curves show a good fit to the observed data. The turbulence near the inlets becomes higher due the complex flow pattern. The flow changes from purely one-dimensional flow along the main channel length (this could be easily checked by the ADV readings), to three-dimensional flow when entering the inlets at the bottom.

The weir discharge coefficient varies depending on water depth above the weir crest level. This is in agreement with other authors. The curve shows an overall good fit to the observed data. As in inlet case, as the flow approaches the weir it will also change direction. In addition, the oscillations are also caused by the flow passing over the inlets, which contribute to an increase of the turbulence as the water flows toward the weir. This can be checked by the slight increase of the water depths CI when observing the WRPs data.

CONCLUSIONS

The experimental scaled model presented was able to replicate flooding. The model comprised of a collection chamber, and an 8.0m long channel with 6 inlets at the bottom and one weir downstream at the channel end. The set-up allowed a complete benchmark test to be built, that can be used to validate urban flood models. Three configurations were tested, for flow rates between 1 and 12 l/s. Water levels and discharges in the main channel were determined using WRP and ADV probes. The main channel discharge should be used as the upstream boundary condition. The discharge coefficients for each inlet and the weir downstream, essential to validate urban flood models, were determined. The orifice discharge coefficients should be used with equation (1) to validate the internal boundary condition that couples the surface with the sewer system. The proposed expression (6) should be used with equation (3) in the downstream boundary condition of the flood model. Consistency was shown between the discharges measured by the volumetric method and the ADV/WRP. The inlet and weir discharges coefficients showed a good fit with the observed data. It was observed that the centre inlets had a higher discharge than the side ones, and the downstream inlets had a higher discharge than the upstream ones.

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