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Experimental assessment of level pool routing in preliminary design of floodplain storage

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ABSTRACT

Among control structures in flood management, floodplain storage represents one of the most effective measures, since it holds part of flood volume in a delimited area thus reducing the peak discharge. Sizing of floodplain storage, both on-stream and off-stream, is complex and several methodologies for preliminary design are available in literature, almost all assuming level pool reservoir routing, i.e. the water level in the floodplain is horizontal during the storage filling. Few studies examine the accuracy of that assumption. The present paper work reports an extensive experimental investigation to assess the reliability of level pool routing in the design of on-stream floodplain storages. The good agreement between numerical and experimental values during the filling phase confirmed the reliability of the hypothesis in the preliminary sizing of on-stream floodplain storage. In contrast, even significant differences can be shown during the floodplain draining, due to vegetation and bottom irregularities.

1. Introduction

In lowland river reaches floods can be extremely dangerous, especially if floodplains are intensively developed. In such areas floods can cause considerable economic losses and even risk of deaths (Roux and Dartus, 2008; Julien et al., 2009). Among structural measures for flood risk reduction, an effective method is to construct floodplain storage. Floodplain storage allows a part of the flood volume to be temporarily stored, thus reducing the outflow discharge. When the discharge falls below the maximum allowable flow, the flood volume is released back to the river.

On-stream floodplain storages are often used, since they do not interfere with the natural drainage pattern between the stream and the floodplain. Only an outlet structure is needed to regulate the outflow discharge. In addition, off-stream storages need a lateral embankment to be built adjacent to the river and a weir structure to regulate the discharge entering the floodplain (Ackers and Bartlett, 2009). Design of floodplain storage for reducing flood risk has often been discussed in the literature (CDWR, 1984; CALFED, 1998). Since storage areas are generally relatively flat, two- and three-dimensional numerical models can be adopted to simulate accurately the floodplain storage.

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2. 2D models

Several two-dimensional flood models are available in literature; among these, Jaffe and Sanders (2001) proposed a 2D backwater model reproducing the filling of off-stream floodplain storage after levee breach occurrence. Beffa and Connell (2001) describe a 2D finite element model, Hydro2de, later applied by Connell et al. (2001) to simulate two floods on the Waihao River in New Zeland. Starting from the Monoclinal Flood Wave theory, Shome and Steffler (2006) deduced a theoretical 2D model to estimate the flood wave velocity and the volume filling the storage. The authors applied the model to the simple case of a rectangular channel and correlated the discharge filling the storage with its geometric characteristics and bottom roughness.

Adopting two-dimensional models can generate difficulties (Freeman et al., 2003) in preliminary sizing of floodplain storages in relatively flat areas since they are data intensive and require advanced modeling capabilities. Consequently, it is often suggested to adopt simpler one-dimensional models, based on level pool reservoir (or uniform storage) hypothesis (McEnroe, 1992; Basha, 1994, 1995).

3. Level-pool reservoir routing models

Assuming uniform storage, Marone (1971) dealt with the peak discharge reduction for artificial lakes with spillways by assuming fixed hydrographs shapes (rectangular, symmetric triangular and asymmetrical). A simple equation was inferred for the peak discharge reduction ratio $\eta = Q_{o,max}/Q_{i,max}$, where $Q_{o,max}$ is the maximum outflow discharge and $Q_{i,max}$ is the peak inflow discharge, as the storage ratio $w = W_s/W_f$ varies, where W_s is the storage volume and W_f the flood volume (Marone, 1971). Horn (1987) provided charts for a direct estimation of the peak outflow discharge varying input hydrograph shape, storage and outlets. The proposed approach assumed an exponential dimensionless inflow hydrograph and a power law for the storage–outflow discharge relation. McEnroe (1992) showed that a bottom outlet is more efficient than a spillway in flood storage management and proposed approximate formulas for the preliminary sizing of detention reservoirs.

Basha (1994) derived an analytical solution for non linear reservoir routing by introducing simplifications on storage geometry and inflow hydrograph, showing that the peak reduction ratio is a quadratic function of the storage ratio. Afterwards he obtained an approximate solution by a two-term perturbation expansion with a zeroth-order linear solution (Basha, 1995).

Other studies were carried out calculating inflow and outflow discharges by coupling runoff hydrograph derived from a design storm model and reservoir routing (e.g. Akan, 1989, 1990; De Martino et al., 2000). All the above methodologies assume level pool reservoir routing when the floodplain fills and drains. Although level pool routing procedures are generally accepted in reservoir and storage design, experimental support of uniform water level hypothesis is needed. This work examines the reliability of uniform storage hypothesis by means of extensive experimental investigation at Department of Hydraulic, Geotechnical and Environmental Engineering of "Federico II" Naples University. The experimental setup reproduces filling and emptying of on-stream floodplain storages. Experiments were performed to examine the effects of varying floodplain geometry and roughness, the size of the outlet bottom, the peak inflow discharge and the shape of the input hydrograph.

4. Experimental setup

The experimental setup was extensively described in De Paola et al. (2006) and De Martino et al. (2007) although a schematic view of the installation is shown in Fig. 1.

A gate valve allows control of the inflow discharge and generated outflow hydrograph. The discharge flows into the stilling basin to dissipate energy and provides for accurate inflow discharge measurement using a rectangular weir, a 0.4% sloped rectangular channel with the dimensions of $h_c = 0.30$ m high and 0.45 m wide. In the channel, a flood gate was arranged in order to produce flooding into

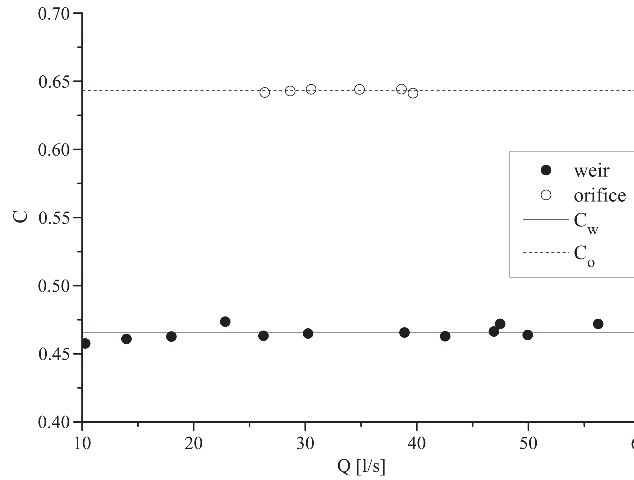

Fig. 2. Calibration of weir discharge coefficient and orifice discharge coefficient.

the storage embankment area when orifice flow was established; and an outflow tank was utilized for providing the flow intake into the laboratory hydraulic circuit.

The inlet structure is a vertical concrete wall supporting a plexiglass sharp crested weir. Inflow discharge Q_i was calculated from h' according to:

$$Q_i = C_w l_w h' \sqrt{2gh'} \tag{1}$$

in which C_w = weir discharge coefficient, l_w = notch width, h' = head on the weir and g = acceleration due to gravity. To accurately predict the flow rate over the inlet weir, a volumetric method was used to calibrate C_w. Stilling basin filling time was measured and, knowing the basin volume, flow rate was derived as ratio between volume and time. Due to the small deviations in discharge coefficients calculated at several discharges ranging between 10 and 60 l/s, C_w was assumed constant over the investigated range and equal to $C_w = 0.465$ (Fig. 2).

The outlet flood gate was used to control discharge. When high flows occur, water surfaces come in contact with flood gate low chord and orifice flow is established. Backwater effects begin and the storage area is flooded, thus reducing the downstream peak discharge. Since

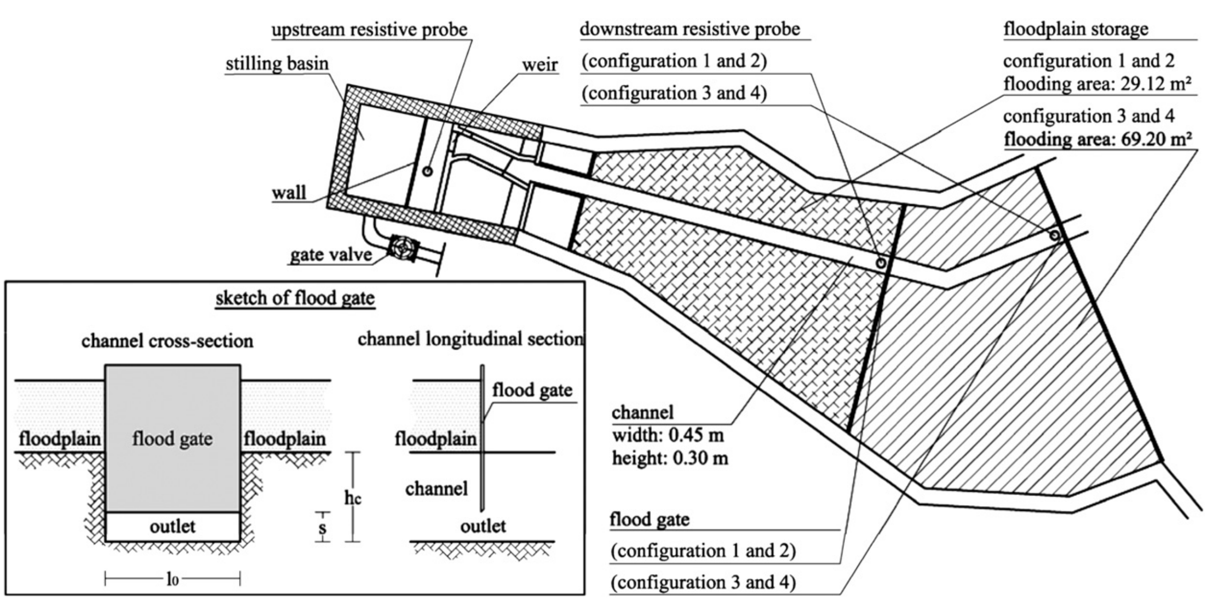

Fig. 1. Plan of experimental setup.

Table 1
Characteristics of analyzed configurations.

Configuration	S [m^2]	bottom	L_c [m]	C_w [−]	C_o [−]	No. tests
1a	29.12	concrete	11.21	0.465	0.640	64
2a	29.12	grass	11.21	0.465	0.640	39
3a	69.20	concrete	15.96	0.465	0.612	41
4a	69.20	grass	15.96	0.465	0.612	115
4b	69.20	grass	15.96	0.465	0.621	37

no backwater occurred in the downstream channel and the orifice was never submerged during the experiments, the outflow discharge Q_o was calculated from channel water level h according to:

$$Q_o = C_o A_o \sqrt{2g(h-s/2)} \quad (2)$$

in which C_o = orifice discharge coefficient, A_o = area of the floodgate outlet ($A_o = l_o s$, where l_o and s are respectively the width and the height of the outlet). Although the effective head on the orifice is the difference in elevation between the water surfaces upstream and downstream from the gate, the approximate expression (Eq. (2)) was used, since only the upstream water level had to be measured. The orifice discharge coefficient was calibrated using Eq. (2) by flowing known discharges through the orifice. When steady state flow was achieved (so that outflow discharge equals the inflow rate), the water level upstream of the gate was measured and the discharge coefficient was calculated. Since discharge coefficients exhibit small deviations, a constant value was assumed for each configuration (Table 1). As an example, Fig. 2 exhibits the orifice discharge coefficient for configuration 1a.

Heads on the weir and orifice were measured via two resistive level probes in the stilling basin and upstream of the floodgate (Fig. 1). The probes were made by Edif, model Level3 with 400 mm measurement range and sensitivity ranging from 1 V/100 mm to 2.5 V/100 mm. The probes were calibrated before each test to correlate the measured difference in electric potential ΔV to the water level, exhibiting an accurately linear relationship.

Five configurations were analyzed in order to investigate the influence of storage area, floodplain bottom roughness and floodgate outlet size on the peak discharge reduction. When different storage areas were analyzed, all the related parameters, such as upstream channel length and outlet discharge coefficient, were recalculated.

In configuration 1 the floodplain area was set to approximately 29 m^2 and a concrete bottom surface was considered. To account for greater bottom roughness, in configuration 2 a vegetated floodplain was considered. Floodplain vegetation was simulated by means of polyolefin synthetic grass type B-SOFT. Grass is 55 mm high, with a total height of 57 mm and a total unit weight of 2665 g/m^2. A picture of the installation with synthetic grass is shown in Fig. 5. Configurations 3 and 4 were similar to configurations 1 and 2 respectively, except for the floodplain area, which was increased to approximately 69 m^2 (Fig. 1). In most experiments the outlet height was set to s = 0.05 m. In order to examine the influence of the floodgate outlet size and obtain more general results, a different outlet height was analyzed (s = 0.06 m) in several tests. As shown by stage-area curves (Fig. 3), the investigated floodplains were substantially flat, and bottom irregularities were present. Full details of analyzed configurations are given in Table 1. Subscripts a and b refer to outlet height s = 0.05 m and s = 0.06 m respectively.

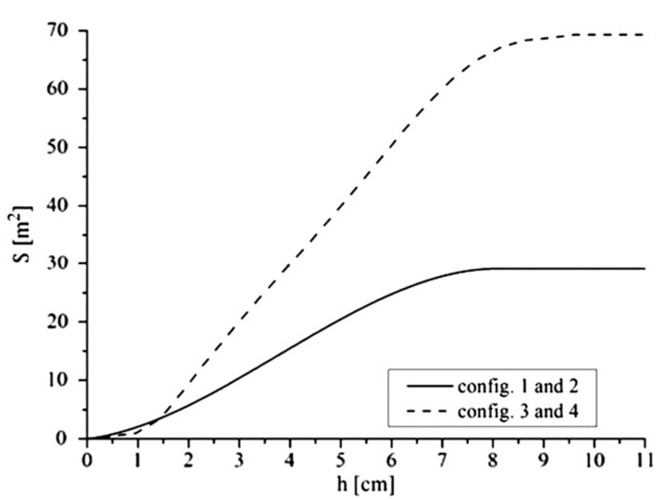

Fig. 3. Stage-area curves.

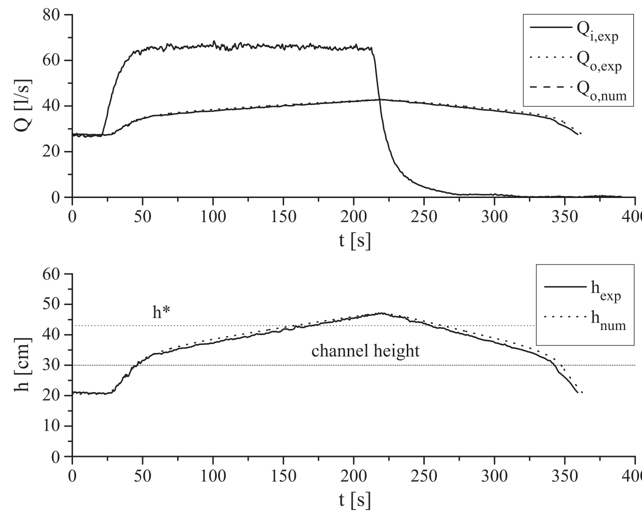

Fig. 4. Inflow and outflow hydrographs and related water levels upstream of the floodgate for configuration 1 experiment.

5. Comparison between theoretical and experimental results

Assuming horizontal free surface during the flooding of the area, the channel water levels h can be estimated by numerical integration of the following differential equation:

$$\frac{dh}{dt} = \frac{1}{S_c + S(z)}\left[Q_i(t) - C_o A_o \sqrt{2g(h-s/2)}\right] \quad (3)$$

in which S_c is the channel area, $S(z)$ is the floodplain flooded area, which was assumed to vary as the floodplain water level $z = h\text{-}h_c$ increases (stage-area curves, Fig. 3) to take into account the bottom irregularities. Eq. (3) holds even during recession phase, in which $Q_i = 0$.

Fig. 5. Picture of the installation with artificial vegetation (configuration 2).

We don't need measuring water surface slope that is very hard and uncertain. The whole point of the paper is to determine the applicability of level pool routing, so we have no interest in checking if the water level is horizontal but we have great interest in checking if the results given by equations (written in level pool routing hypothesis) in terms of flow and water level, conform or not to experimental measures.

Fig. 4 depicts results of a configuration 1 experiment, showing hydrographs and channel water levels. In Fig. 4 the experimental channel water levels measured upstream of the floodgate (h_{exp}) were compared with those predicted by Eq. (3) (h_{num}). Fig. 4 also shows experimental inflow discharge ($Q_{i,exp}$), experimental outflow discharge inferred from measured h ($Q_{o,exp}$) and outflow discharge ($Q_{o,num}$) calculated from Eq. (2) with the predicted h.

Experiments show the fairly good agreement between measured and theoretical values both in terms of hydrographs and water level thus verifying the reliability of uniform storage hypothesis.

Similar tests were carried out for configuration 2 to better explain the influence due to bottom vegetation when the floodplain is inundated. Bottom vegetation changes the flow field, as experimentally well known (Armanini and Righetti, 2002; Armanini et al., 2005; Pulci Doria et al., 2007). By means of laboratory experiments, Mushle and Cruise (2006) analyzed the effect of rigid non-submerged vegetation on flow resistance in wide floodplain and obtained some relationships between hydraulic parameters and vegetation density. Adopting stochastic criteria, Yang et al. (2007) established velocity, turbulence and Reynolds stress distributions in a vegetated floodplain in a small-scale laboratory setup. Nevertheless, the literature gives no suggestion on reliability of level pool routing hypothesis in design of vegetated floodplain storage.

To this aim, floodplain vegetation was simulated by means of synthetic grass as said before. Water levels and hydrographs for a test on the vegetated floodplain (configuration 2) are given in Fig. 6, which shows the experimental water levels measured upstream of the floodgate (h_{exp}) and those predicted by Eq. (3) (h_{num}). Fig. 5 also shows experimental inflow discharge ($Q_{i,exp}$), experimental outflow discharge inferred from measured h ($Q_{o,exp}$) and outflow discharge ($Q_{o,num}$) calculated from Eq. (2) with the predicted h. During the filling phase, water levels tended to increase compared to the experiments without vegetation, since water passed through the grass with more difficulty. That caused a small discrepancy between the experimental and numerical values which did not account for the effect of vegetation. Deviations decreased as water level increased and tended to disappear when the floodplain was filled and the vegetation completely submerged.

In contrast, when water levels decreased, a more marked difference between numerical and experimental values can be observed. Numerical simulations always overestimate measured water levels and duration of recession phase. Vegetation causes a slower emptying, not only as a consequence of the barrier due to the grass, but also because vegetation holds part of the water in the floodplain, thus reducing water volume flowing to the channel. This causes a faster water level decrease in the channel, where the free surface does not depend on the storage water level as confirmed by Lai et al. (2000).

Experimental results of configuration 3 (without vegetation and greater storage area) and 4 (with bottom vegetation and greater storage area) are similar to configuration 2. Good agreement between numerical and experimental water levels can be shown during floodplain inundation whereas the recession phase exhibits again significant differences, caused by flow resistance due to vegetation and bottom irregularities.

Differences progressively increase in configurations 3 and 4, showing the major effect due to vegetation opposing the water movement and the less important (but not negligible) effect of the bottom irregularities in the recessing phase. Configuration 4 exhibits the most significant differences, since the abovementioned factors are present together.

Although preliminary design of floodplain storage does not depend on the recession phase, we tried to explain strong deviations between numerical and experimental water levels, taking into account the flow resistance caused by vegetation and/or greater storage area.

Experiments showed that after the outflow discharge attained the peak value and the floodplain started draining, the water surface was horizontal over the model, whereas at a certain time water level in the channel decreased more rapidly than in the floodplain (Fig. 7). On the basis of such observation, we have assumed that when the channel water level falls below a certain level, denoted as h^* the floodplain draining model can be decoupled from the water levels in the channel. The storage emptying was simulated assuming a linear reservoir model:

$$Q_{o,f}(t) = W(t)/k; \qquad \frac{dW(t)}{dt} = -Q_{o,f}(t) \tag{4}$$

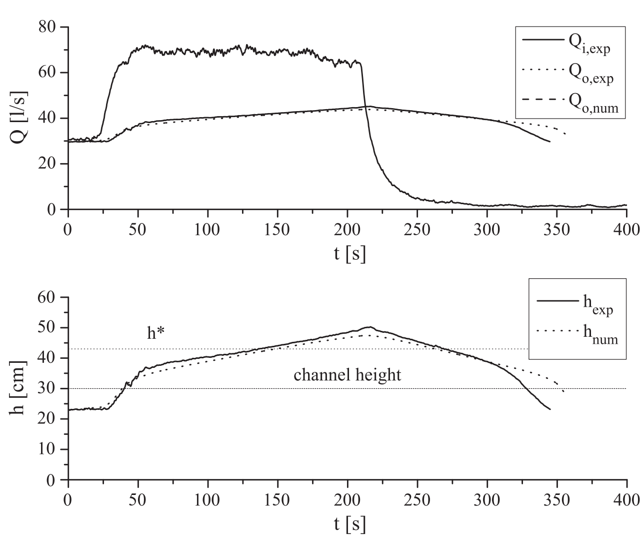

Fig. 6. Inflow and outflow hydrographs and related water levels upstream of the floodgate for configuration 2 experiment.

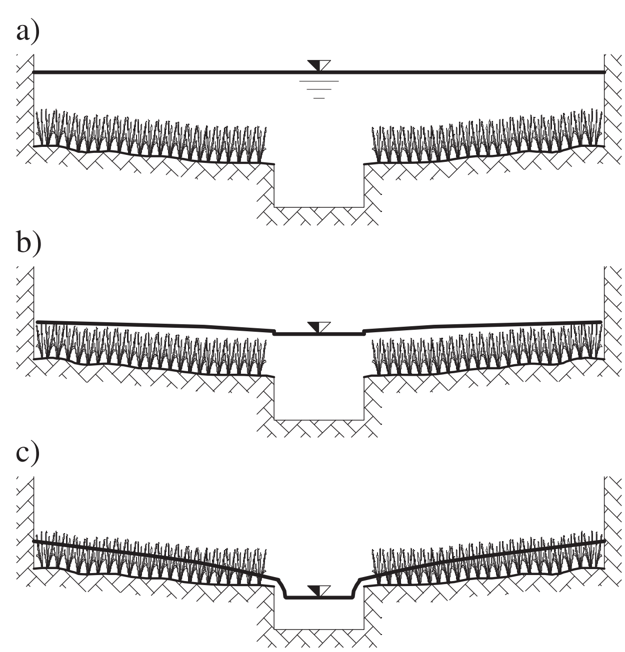

Fig. 7. Sketch of falling water surface during recession phase.

where $Q_{o,f}$ is the outflow discharge flowing from floodplain to the channel and k is a storage constant, calculated as the ratio between stored water volume W^* when h equals h^* and the outflow discharge Q^* at the same time. The inflow discharge calculated by (4) causes a water level variation in the channel, that we can calculate assuming uniform storage:

$$\frac{dh}{dt}=\frac{1}{S_c}\left[Q_{o,f}-C_o l_o\sqrt{2g(h-s/2)}\right] \tag{5}$$

The proposed model allows a more accurate simulation of the floodplain draining. Water levels in the emptying phase for the configuration 2 experiment (shown in Fig. 6) were enlarged in Fig. 8, in which experimental water levels and numerical values calculated by Eq. (5) are given. Water levels given by Eq. (5) were calculated assuming $h^*=43$ cm and fit experimental data better than a coupled model. Value of h^* was fixed in order to minimize deviations between experimental and numerical values and it has a clear physical meaning, since it represents elevation at which falling water surface exposes submerged vegetation.

For the sake of brevity, results obtained for configurations 3 and 4 were not given. For such configurations different h^* were found. Nevertheless, they have the same physical meaning of elevation at which water surface exposes highest bottom irregularities (configuration 3) or vegetation (configuration 4). For all the investigated configurations, the storage ratio $w=W_s/W_f$ was calculated, where W_f is the flood volume, calculated as the area under inflow hydrograph limited by intersection with outflow hydrograph, and W_s the stored volume, bounded by inflow and outflow hydrograph. Data fit well a model $\eta=1\text{-}w$ proposed by Marone (1971) to represent a suitable relationship between the peak discharge reduction ratio and the storage ratio. Fig. 9 shows a good correlation between experimental points and linear model with a maximum deviation almost always smaller than 0.05. That means that if one calculates the value of w using linear model and fixing the value of η, the results may be different from the real ones for a maximum of ± 0.05. To test the agreement three validation indexes (Krause et al., 2005) were calculated: coefficient of determination (r^2), the Nash–Sutcliffe efficiency (E) and the index of agreement (ia); results obtained are the following: $r^2=0.902$, $E=0.894$, $ia=0.972$, the values of all indexes confirm that the linear model well represents the experimental results.

6. Conclusions

This paper reports the results of extensive experimental investigation aimed at assessing the reliability of the uniform storage hypothesis in floodplain storage sizing. Experiments were carried out on laboratory installation to simulate the filling and draining of floodplain storages varying storage area and roughness, floodgate outlet size and inflow hydrographs (shape and peak discharge).

A simple numerical model was proposed to estimate the outflow hydrograph, which reproduces well water levels in the floodplain and the peak discharge reduction ratio.

The good agreement between numerical and experimental values verified the reliability of the uniform storage hypothesis in the preliminary sizing of on-stream floodplain storage.

Vegetation or significant bottom irregularities can hold a fraction of the flood volume during the floodplain emptying, so the water level in the channel becomes independent from water levels in the storage area. A simple model was proposed to simulate floodplain emptying, assuming the uniform storage model for the channel coupled to a linear reservoir model to simulate the floodplain draining. The model considerably reduces differences between experimental water levels and numerical values inferred from Eq. (4). Finally it has been shown that a simple linear model can represent a suitable relationship between the peak discharge reduction ratio and the storage ratio because it gives results very close to experimental data.

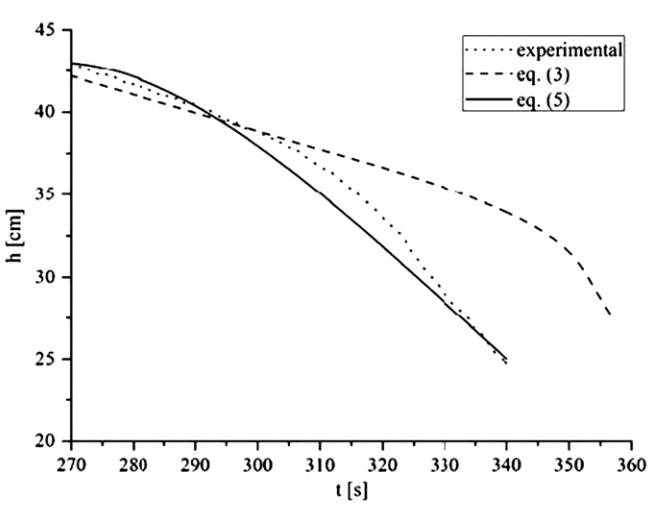

Fig. 8. Channel water level during floodplain draining (configuration 2).

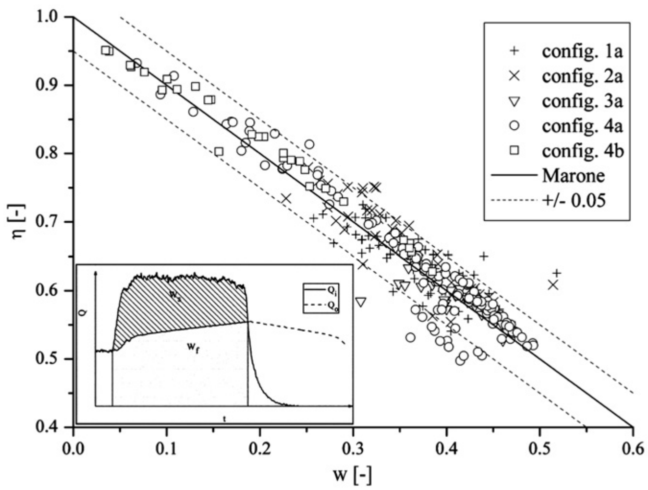

Fig. 9. w-η experimental data.

Notation

A_o	area of floodgate outlet;
C_o	discharge coefficient of floodgate outlet
C_w	discharge coefficient of weir
g	gravitational acceleration
h	channel water level
h	water level in the channel
h_c	channel height
h'	head on the inlet weir
h^*	channel water level to uncouple the storage draining from water levels in the channel
k	storage constant: W^*/Q^*
l_o	width of floodgate outlet
l_w	length of inlet weir
L_c	channel length
Q	discharge
Q^*	outflow discharge at h^*
s	height of floodgate outlet
S	storage area
S_c	channel area
$S(z)$	storage area, as function of water level in the storage
t	time
w	storage ratio
W	stored volume at time t

W_s	stored flood volume
W_f	flood volume
W^*	stored volume at h^*
z	water level in the floodplain storage
ΔV	difference in electric potential
η	peak reduction ratio

Suffixes

exp	measured
f	flowing from floodplain to the channel
i	inflow
max	peak
num	numerical
o	outflow

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