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Modeling water depth distribution in vertical slot fishways under uniform and non-uniform scenarios

J.F. Fuentes-Pérez[1]; F.J. Sanz-Ronda[2]; A. Martínez de Azagra Paredes[3]; and A. García-Vega[4]

Abstract

Vertical slot fishways are a type of fish pass of wide operating range that allows fish to move across obstacles in rivers. This study aims to model the performance of these structures, under uniform and non-uniform flow conditions, using discharge coefficients involving the downstream water level together with a logical algorithm. This will allow to explain the hydraulic behavior of this type of fishways under tailwater levels and flow variations on rivers. Two vertical slot fishways located in Duero River (North-Central Spain) subject to different hydraulic conditions were studied for the validation of the proposed formulation. The observed values are consistent with the predicted results and, among others, demonstrate the importance of including variables which consider downstream water level. Consequently, the proposed discharge coefficients together with the algorithm have resulted in a method which enables to improve the performance of both existing and future vertical slot fishways. This will have major implications in real-life scenarios where uniform operation conditions are rarely achieved.

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[1] PhD student, GEA Ecohydraulics, Department of Hydraulics and Hydrology, ETSIIAA, University of Valladolid (UVa). Avenida de Madrid 44, Campus La Yutera, 34004 Palencia (Spain). jfuentes@iaf.uva.es

[2] Professor, GEA Ecohydraulics, Department of Hydraulics and Hydrology, ETSIIAA, University of Valladolid (UVa). Avenida de Madrid 44, Campus La Yutera, 34004 Palencia (Spain). jsanz@iaf.uva.es

[3] Professor, GEA Ecohydraulics, Department of Hydraulics and Hydrology, ETSIIAA, University of Valladolid (UVa). Avenida de Madrid 44, Campus La Yutera, 34004 Palencia (Spain). amap@iaf.uva.es

[4] Engineer, GEA Ecohydraulics, Department of Hydraulics and Hydrology, ETSIIAA, University of Valladolid (UVa). Avenida de Madrid 44, Campus La Yutera, 34004 Palencia (Spain). ana.garcia.vega@iaf.uva.es

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Introduction

Loss of longitudinal connectivity by man-made obstructions is one of the main ecological problems in regulated rivers (Nilsson et al. 2005; Branco et al. 2012). This issue particularly affects migratory fish, which require different environments for the principal stages of their life cycle (Porcher and Travade 2002). However, the social benefits of these obstacles make it impractical to remove them and often, the only way to restore longitudinal connectivity, at least partly, is by building fish passes (Wang et al. 2010; Calluaud et al. 2012).

One of the most widely used fish passes are vertical slot fishways (VSFs). These structures are widespread mainly due to their capacity to cope with different flows (Tarrade et al. 2011) and their versatility regarding the water depth available for upstream fish movement (Liu et al. 2006). VSF consists on an open channel divided into a number of pools by cross-walls equipped with vertical slots. This configuration divides the total height of the obstacle into small head drops (ΔH) and forms a jet at slots, the energy of which is dissipated by mixing in pools (Liu et al. 2006).

Based on their geometric configuration, there are many types of VSFs (Rajaratnam et al. 1986; Rajaratnam et al. 1992; Wu et al. 1999; Puertas et al. 2004). However, the most common configuration is that of the Hell’s Gate model, with double or single slots (model 1 according to Rajaratnam et al. (1986)) (Fig. 1).

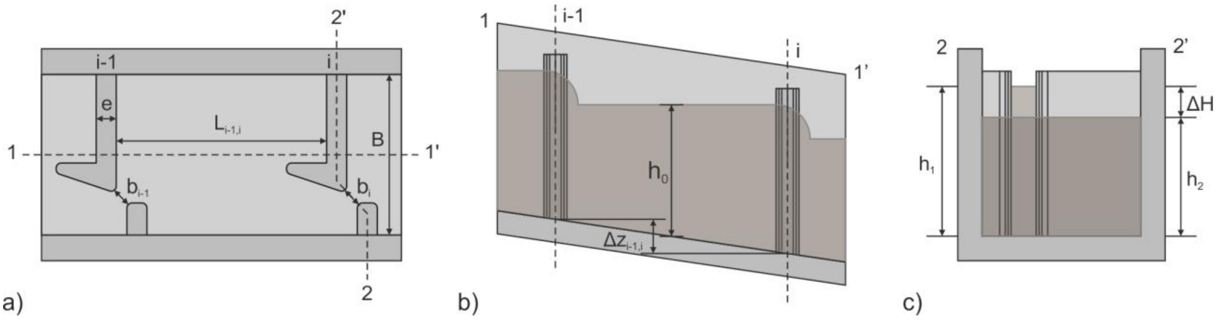

Fig. 1. Schematic representation of Hell's Gate model with a single slot (model 1 defined by Rajaratnam et al. 1996), the model under study. a) Plant. b) Longitudinal section. c) Cross section. Note: The symbols are defined in the notation section.

In some cases, the flow of VSFs is described by the equation for weirs proposed by Poleni (1717) (FAO/DVWK 2002; Krüger et al. 2010), discounting in the discharge coefficient (C_1) the effect of the lower contraction (Eq. (1)). In other cases, their flow can also be compared to that of a submerged orifice with an area equal to the product of the width (b) and the water level upstream the slot (h_1) (Eq. (2)) (Martínez de Azagra 1999; Larinier 2002; Bermúdez et al. 2010; Wang et al. 2010) and discounting in the discharge coefficient (C_2) the effect of contractions.

$$Q=\frac{2}{3}\cdot C_1\cdot b\cdot h_1^{1.5}\cdot\sqrt{2\cdot g} \qquad (1)$$

$$Q=C_2\cdot b\cdot h_1\cdot\sqrt{2\cdot g\cdot \Delta H} \qquad (2)$$

In both equations the discharge coefficients (C_1 and C_2) depend on the relative position of the water levels (upstream and downstream (h_2)) and the geometry of the VSF, while g stands for the acceleration due to gravity.

In 1986 and 1992 Rajaratnam et al., by using the geometry of the slots, proposed the use of dimensionless relationships to describe discharge in VSFs (Eq. (3) and Eq. (4)).

$$Q^*=\frac{Q}{\sqrt{g\cdot S\cdot b^5}} \qquad (3)$$

$$Q^{*}=\beta_0+\beta_1\cdot\left(h_0/b\right) \qquad (4)$$

where β_0 and β_1 depend on the geometry of the VSF, h_0 is the mean water depth (measured at the center of the pool), S is the slope and Q* is the dimensionless discharge. These relationships have widely been used (Puertas et al. 2004; Cea et al. 2007) and modified (Wu et al. 1999; Kamula 2001).

Given the variability in the factors that describe their flow, VSFs behave differently both amongst them and throughout time. Consequently, it is a common practice to simplify their study by using geometrically perfect laboratory models with uniform flow conditions, where ΔH is the same in all the slots and equal to topographic difference between slots (Δz) (Rajaratnam et al. 1986; Rajaratnam et al. 1992; Wu et al. 1999; Puertas et al. 2004; Cea et al. 2007; Bermúdez et al. 2010; Tarrade et al. 2011; Puertas et al. 2012).

These operational characteristics are difficult to achieve under laboratory conditions and, even more, in real-world conditions. In many cases, due to an inaccurate execution or simply because the ideal working situation is never encountered, fish passes will work under non-uniform flow conditions which may decrease their efficiency for fish passage.

In order to overcome these limitations, the present study aims to improve the modeling of VSFs' hydraulic performance using the equation proposed by Villemonte (1947), to evaluate the influence of downstream water level, together with a logical algorithm. This will allow to estimate the distribution of water depths in both geometrically and not geometrically perfect VSFs (i.e. different Δz between slots, different b in each slot, etc.), under different uniform or non-uniform flow states.

Materials and Methods

Experimental Arrangement and Experiments

Experiments were conducted in two VSFs of Hell's Gate type designed by the Group of

Applied Ecohydraulics of the University of Valladolid. Both VSFs are located on two weirs in the Duero River (North-Central Spain). In the first one (VSF1 – 41º37'N, 4º6'W) a succession of 27 slots were studied (n=27), while in the second one (VSF2 – 41º38'N, 3º34'W) a succession of 12 (n=12).

The geometrical parameters of the VSFs were measured by topographic surveying (Fig. 1). Both VSFs are composed by pools of a mean length of 2.100 m ($L \approx 10 \cdot b$) and a mean width of 1.600 m ($B = 8 \cdot b$). The average width of slots is 0.200 m and the mean Δz is 0.143 m for VSF1 and 0.189 m for VSF2 with an average slope ($S = \Delta z/(L+e)$, where e is the thickness of the cross-wall) of 0.062 m/m and 0.082 m/m, respectively.

During the experimental procedures the flow rate was controlled by the gates located upstream the structures and was measured by chemical gaging using Rhodamine WT as tracer (Martínez 2001). These gates are used for the maintenance and cleaning in both fishways, however they provide the opportunity to represent in the same season different hydrodynamic scenarios, that is to say different h_1 in the first slot ($h_{1,1}$) and discharges through the fishways.

This type of experiment was replicated four times to achieve in each VSF different non-uniform water depth distribution profiles (conceptual backwater profile (M1) and drawdown profile (M2) (Rajaratnam et al. 1986; Chow 2004)) (Table 1). M1 profiles were obtained by reducing the area of the slot situated downstream the last slot studied (increasing h_2 of the last slot studied ($h_{2,n}$)) and M2 and uniform (U) profiles were naturally present during the experiments.

Table 1. Results of discharge experiments in VSF-1 and VSF-2. $h_{2,n}$ is the downstream water depth in the last slot studied (when modeling the performance equal to tailwater level).

Experiment name	Estimated discharge ± CI (m^3/s)	Reached Profile	$h_{2,n}$ (m)
VSF1-1	0.247 ± 0.004	M2	0.700
VSF1-2	0.247 ± 0.004	M1	0.979
VSF1-3	0.247 ± 0.004	M1	1.029
VSF1-4	0.165 ± 0.010	M1	0.617
VSF2-1	0.232 ± 0.004	U	0.729
VSF2-2	0.232 ± 0.004	M1	0.858
VSF2-3	0.276 ± 0.007	U	0.816
VSF2-4	0.276 ± 0.007	M1	0.990

The water depth was measured in each pool by a graduate scale situated downstream the slots in the center of the cross-walls. In each pool successive measures were made to obtain a stable mean value.

Discharge Coefficient

Villemonte (1947) described the net flow over a submerged weir as the difference between the free-flow discharge due to h_1 and the free-flow discharge due to h_2. Taking into account the assumptions of this author and that under free-flow discharge Eq. (2) becomes Eq. (1) (ΔH tends to h_1), the discharge coefficient for both equations can be defined as,

$$C = \alpha_0 \cdot \left(1 - \left(\frac{h_2}{h_1}\right)^{1.5}\right)^{\alpha_1} \qquad (5)$$

where α_0 and α_1 are coefficients which depend on the geometry of the slot and the discharge equation used.

Although this coefficient was initially described by Villemonte for weirs, Krüger et al. (2010) showed the suitability of similar expressions in the description of the functioning of VSFs.

Formulation of the algorithm

To simulate the water depth distributions of the VSFs under different hydrodynamic scenarios, taking into account the specific geometrical characteristics of each slot, it is necessary to perform an iterative bottom-up calculus considering the discharge through the fishway ($Q_{fishway}$) (or the headwater level ($h_{1,1}$)) and $h_{2,n}$ (Fig. 2).

Fig. 2 represents the logical algorithm followed in order to solve a particular scenario where Eq. (6) represents each of the discharge equations proposed. Due to the iterative process, the resolution of the algorithm can be tedious; thus, its programming is highly recommended. Consequently, a computer program called “Escalas 2012” was developed (Fuentes-Pérez et al. 2012).

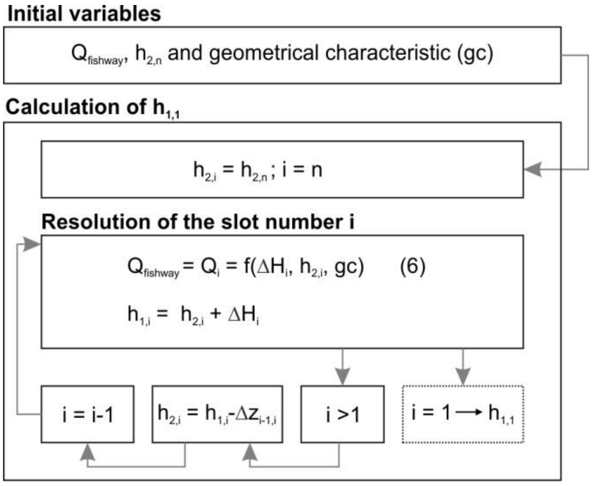

Fig. 2. Flowchart showing the steps of the proposed algorithm. Note: The symbols are defined in the notation section.

Validation

The fit of the proposed discharge equations was evaluated using r-squared (r^2) with data collected both from the specialized literature and field measurements (Fig. 3). The comparison of the predicted water depth profiles using the algorithm and each of the adjusted equations was carried out by comparing the mean relative errors (MRE) for each scenario.

Experimental Results and Discussion

Discharge Equations

Fig. 3 shows the fitted curves for the proposed equations. All of them represent part of the observed variability due to S for the different VSF models (Wang et al. 2010); either because they describe the variability of ΔH (or h_2), which in uniform settings is determined by S (Fig. 3 (a and b)) or because S is included in the equation (Fig. 3(c) and Eq. (3)). This enables the use of the equations in fishways with different slope.

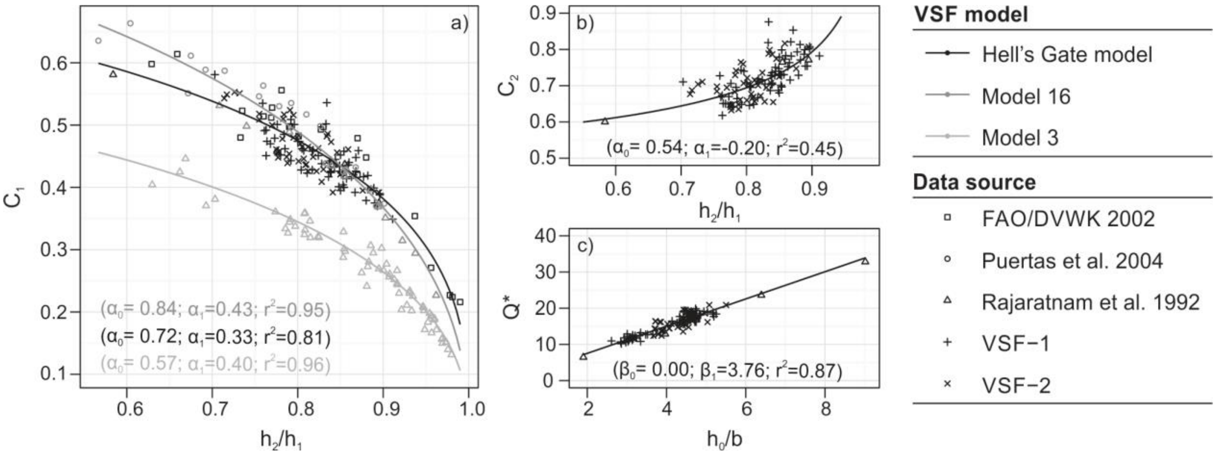

Fig. 3. Discharge equations adjustment. a) Fit of C1 for the Hell's Gate, 3 and 16 models defined by Rajaratnam et al. (1996). b) Fit of C2 for Hell's Gate model. c) Fit of Eq. 4 for Hell's Gate model.

As h_2/h_1 approaches zero (h_2 tends to 0), h_1 will reach the critical water depth while C_1 and C_2 will tend to a constant value. C_1 explains well the variability due to h_2. Regarding C_2, despite the small r^2, it provides satisfactory results when the water depth and head drop profiles of the fishway are simulated (Fig. 4). This is because Eq. (2) considers, partly, the effect of the water level distribution (by means of ΔH) providing, even when using a constant value for C_2, more satisfactory results, under non-uniform situations, than the other discharge equations.

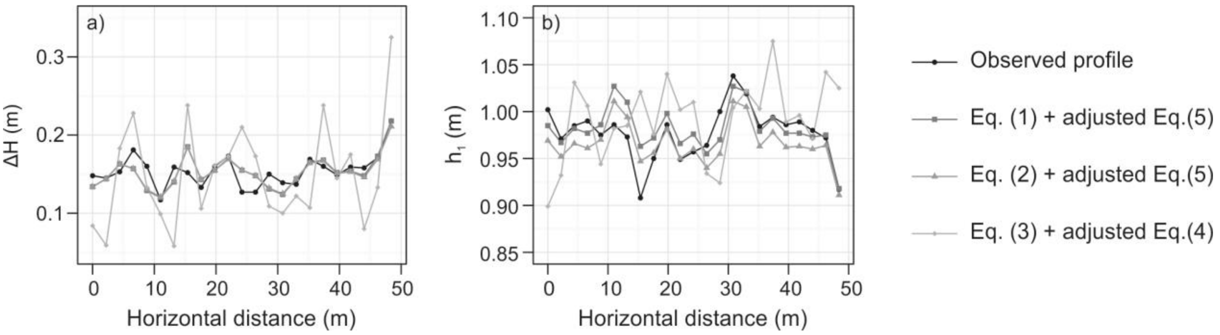

Fig. 4. Observed and predicted ΔH and h1 profiles using the algorithm for VSF1-1 according to the different equations. Horizontal distance represents the separation between slots and in 0 is situated the upper slot.

In contrast to Eq. (1) and Eq. (2), Eq. (3) does not directly consider water depth in the slot. The variability of water depth is explained by Eq. (4) by means of h_0, and thus provides a higher r^2 than the other adjustments (Fig. 3(c)). Furthermore, Eq. (4) dismisses all the variability provided by h_2, making it only possible to explain strictly uniform flow conditions (Rajaratnam et al. 1986). In order to interpret non-uniform scenarios, it is interesting to adapt data from the literature to include variables such as h_2 as shown in Fig. 3(a) (model 3 and 16).

Water depth and head drop profiles

Fig. 4 underlines the importance of considering parameters that take into account the hydrodynamic conditions of the slot, that is, either h_1 and h_2 or h_1 and ΔH. Weir and orifice equations (Eq. (1) and Eq. (2)) together with Villemonte's discharge coefficient (Eq. (5)) are able to describe well the observed ΔH profiles (Fig. 4(a)) and are capable of capturing changes in $h_{2,n}$ (MRE for all experiments of 8.88% and 8.93%, respectively). However the dimensionless equations (Eq. (3) and Eq. (4)) do not simulate properly the observed values as shown by the high MRE for ΔH (40.25%).

Regarding h_1 (Fig. 4(b)), weir and orifice equations predict a similar profile to the one observed (MRE of 1.87% and 2.17%, respectively). With the dimensionless equations, the

MRE is higher (5.84 %) and it increases as the influence of $h_{2,n}$ rises. Moreover, when using dimensionless equations the described profile is considerably different to the observed one.

Conclusions

The proposed discharge coefficients enable, using a logical algorithm, the modeling of the performance of both geometrically and not geometrically perfect VSFs under uniform and non-uniform scenarios. Furthermore, this methodology has been evaluated successfully by the experimental study of two existing structures as well as analyzing cases from the literature.

According to the results presented here, Eq. (1) and Eq. (2) together with the discharge coefficients defined by Villemonte (1947) (specific to each type of VSF) provide the best option to design and evaluate VSFs.

To get accurate water depth predictions it is essential to use equations which include a variable that considers downstream water level (h_2 or ΔH). This provides a means to incorporate both the variation in water levels as well as, given the relationship between S and ΔH in uniform stages, the different slopes used in the design.

The use of these discharge coefficients allows the simulation of the distributions of both water levels and head drops in VSFs. This will enable to evaluate the behavior of different solutions prior or after their construction and detect and correct deficiencies in fishway designs.

Finally, in order to evaluate the performance and wider applicability of the proposed formulations it would be interesting to apply it to other fishways with different hydraulic connections between pools.

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Notation

The following symbols are used in this technical note:

B = width of pools (m)

b = slot width (m)

b_i = slot i width (m)

C = generic discharge coefficient

C_1 = discharge coefficient for Eq. (1)

C_2 = discharge coefficient for Eq. (2)

e = thickness of the cross-wall (m)

g = acceleration due to gravity (m/s^2)

h_0 = mean water depth of flow in pool in relation to the center of the pool (m)

h_1 = mean water depth of flow in pool in relation to the upstream of the slot (m)

$h_{1,i}$ = mean water depth of flow in pool in relation to the upstream of the slot i (m)

h_2 = mean water depth of flow in pool in relation to the downstream of the slot (m)

$h_{2,i}$ = mean water depth of flow in pool in relation to the downstream of the slot i (m)

i = slot number

CI = 95% confidence interval

L = pool length (m)

$L_{i-1,i}$ = pool length between slot i and slot i-1 (m)

n = total number of slots

Q = discharge or flow rate (m^3/s)

Q^* = dimensionless discharge

$Q_{fishway}$ = discharge through fishway (m^3/s)

Q_i = discharge through slot i (m^3/s)

r^2 = determination coefficient

S = slope of the fishway (m/m)

α_0 = dimensionless coefficient for Eq. (5)

α_1 = dimensionless exponent for Eq. (5)

β_0, β_1 = dimensionless coefficients for Eq. (4)

ΔH = difference in water level between pools or head drop ($h_1 – h_2$) (m)

ΔH_i = difference in water level between pools or head drop in the slot i ($h_{1,i}$-$h_{2,i}$) (m)

Δz = topographic difference between slots (m)

$\Delta z_{i-1,i}$ = topographic difference between slots i-1 and i (m)

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