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Non-uniform hydraulic behavior of pool-weir fishways: A tool to optimize its design and performance

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Abstract

Fishways are structures that aim to achieve the free movement of fish through transversal obstacles in rivers. Despite the wide research about their performance, their hydraulic study and characterization has been so far limited to uniform hydraulic conditions which are usually difficult to reach in natural scenarios, either because inaccurate building or simply because the studied situations during the design of the prototypes are never encountered. This study aims to model pool-type fishways with submerged notches and orifices under different regimens, and uniform and non-uniform performances. For this purpose, the classical formulation used in their design has been modified by studying a real-scale fishway under 29 different hydraulic conditions. The proposed new formulation together with a logical bottom-up iterative calculation is able to predict the observed water level distributions. This study demonstrates that orifices and notches can be considered independently when estimating the water level distribution and discharge through the fishway, and the need to modify the classical formulation. The modelling under non-uniform scenarios will allow to enhance and adapt fishways to achieve a greater fish passage during longer time periods.

Keywords: Pool-weir fishways; Water levels; Flow discharges; Hydraulic design; Non-uniform performance

1. Introduction

Current society needs a large volume of fresh water to keep its present lifestyle, whether for irrigation, to generate electricity or to fulfill industrial, domestic and recreational needs. This, coupled with the exponential population growth, has caused the installation of a great number of infrastructures to collect and use this resource (Nilsson et al., 2005).

These structures are usually cross-sectional to the river, breaking its longitudinal connectivity and blocking the movement of some animals such as fish, which require different environments for some of the most important stages of their life cycle (Porcher and Travade, 2002; Branco et al., 2013). In the best case scenario, the impact of these barriers will cause the diminution in abundance of some species and, in the worst case scenario, their disappearance (Larinier, 2001; Lucas et al., 2001; Branco et al., 2012). It is in this context that fish passes or fishways arise to facilitate the free movement of fish fauna through these obstacles (Clay, 1995; FAO/DVWK, 2002; Larinier, 2002a).

Fish passes are a clear example of ecological engineering, since they are civil engineering devices, which can be natural-like according to their type, with a efficiency, understood as the proportion of fish from a given population that attempt and succeed in surpassing the obstacle, associated to their hydrodynamic variables (discharge, velocity, depth, power, turbulence fields, etc.) and a combination of swimming capacity, behavior, and motivation of fish (Bermúdez et al., 2010; Sanz-Ronda et al., 2015a). In addition, these hydrodynamic variables depend on environmental parameters, such as fluctuation in water levels upstream and downstream of the structure (Fuentes-Pérez et al., 2014).

In Europe, the installation of fishways has increased since the implementation of the Water Framework Directive (European Commission, 2000). However, the efficiency of these structures has been questioned due to the wide diversity of fish species and the great unknowns regarding their swimming abilities, migration periods, and motivation (Bunt et al., 2012; Williams et al., 2012). Therefore, biological and ecological studies are essential, particularly in less studied species, such as potamodromous species (Roscoe and Hinch, 2010; Bunt et al., 2012; Katopodis and Williams, 2012; Silva et al., 2012). In recent years, in the Iberian Peninsula, this knowledge gap has been addressed by a number of studies (Santos et al., 2012; Silva et al., 2012; Alexandre et al., 2013; Branco et al., 2013; Sanz-Ronda et al., 2015a, among others). From a practical viewpoint, all of these studies should show a correct hydraulic characterization in order to enable the application of the collected knowledge to new designs.

The most common fishways are pool-type fishways (Clay, 1995; Martínez de Azagra, 1999; FAO/DVWK, 2002; Puertas et al., 2012). They consist of a sloping-floor channel divided by weirs, cross-walls, or baffles into a series of pools, distributing the height to be crossed by the fish (H) into several smaller water drops (ΔH) (Larinier, 2002a). A further classification of pool-type fishways is possible according to the type of connection between pools, being one of the most popular those composed by submerged notch and orifices (SNOF) (Larinier, 2008) (Fig. 1).

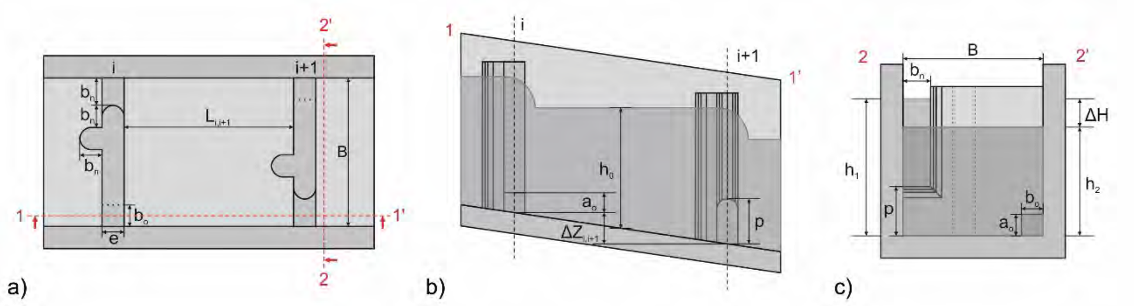

Fig. 1. Schematic representation of a submerged notch and orifice fishway (SNOF) pool, used in the present study: (a) plant, (b) longitudinal section, (c) cross section. Symbols are defined in the notation section.

This type of fishways can have two different performances or regimes, streaming or plunging, depending on whether the downstream water level (h_2) influences or not, respectively, upstream water level (h_1) (Rajaratnam et al., 1988; Larinier, 2002a). Likewise, it is also possible to define different sub-regimes within these two main performances (Ead et al., 2004).

Pool and weir type fishways have been commonly designed with notches working under plunging conditions (Kim, 2001; Yagci, 2010; Santos et al., 2012). However, in SNOF, the notch is designed to operate in a streaming regimen, which has been shown to enhance the upstream movements of species like Iberian barbel (*Luciobarbus bocagei*), Iberian chub (*Squalius pyrenaicus*) or Iberian nase (*Pseudochondrostoma duriense*), and seems to be more suitable for rivers with fish with wide morpho-ecological traits (Silva et al., 2009; Branco et al., 2013; Sanz-Ronda et al., 2015b). Furthermore, SNOF shows additional benefits such as alternating submerged orifices from side to side. This orifice configuration has shown higher rates of passages than other configurations for the Iberian barbel (Silva et al., 2012).

Previous reports have widely studied similar type of designs using either classical weir-discharge equations (Martínez de Azagra, 1999; Kim, 2001; FAO/DVWK, 2002; Larinier, 2002a; Boiten and Dommerholt, 2006; Santos et al., 2012) or dimensionless relationships (Rajaratnam et al., 1989; Ead et al., 2004; Yagci, 2010); however, these studies were always performed under uniform operational conditions (same mean water level (h_0) and ΔH in all pools). This simplification limits the interpretation of their behavior once they are installed because fishways will work under changing non-uniform conditions which may decrease their efficiency (Fuentes-Pérez et al., 2014), due to large variations in the hydrological regime of rivers, as it is the case in the Mediterranean regions (Gasith and Resh, 1999), or due to an inaccurate execution. That is to say, the boundary conditions and its geometry will determine not only the regimen of the fishway (plunging, streaming or mixed) but also the water level distributions (non-uniform or uniform profiles), which may modify the observed efficiency in laboratory models.

In order to solve the above mentioned issues, this study aims to define the operational conditions of submerged notch and orifice fishways both under different regimens and uniform and non-uniform performances, modifying the classical formulation that has widely been used to describe their behavior. This will allow to describe and predict their functioning, i.e. the hydraulic variables, under natural scenarios, and evaluate the influence of modifications in the design of the fishways regarding the necessities of the target species, making possible the improvement of fishways efficiency. The above is summarized in the main following contributions: (i) new definition of calculus equations for SNOF under uniform and non-uniform performance; (ii) evaluation and validation of predictability of water levels of the proposed equations; (iii) theoretical demonstration of the use of the defined equations and algorithm to improve the fishway efficiency.

2. Materials and methods

2.1. Fishway description

The experiments were conducted in a real-scale SNOF with a design discharge of 0.278 m^3/s (Fig. 1). This structure is located in the Tormes River near the village of La Flecha (Salamanca, Spain) and it is characterized by small deviances from design parameters (± 0.010 m) (Sanz-Ronda et al., 2010). The average topographic difference between cross-walls ($\Delta\overline{Z}$) is 0.247 m, that is to say, it has an average slope [$\overline{S}$ = $\Delta\overline{Z}/(\overline{L}+\overline{e})$, where $\overline{L}$ stands for the length of the pool and $\overline{e}$ for the thickness of the cross-wall] of 0.088 m/m (Fig. 2). Cross-walls consist on an alternative succession of submerged hydrodynamic notches [mean width ($\overline{b}_n$) of 0.310 m and mean height of the sill ($\overline{p}$) of 0.917 m] and bottom orifices [mean width $\overline{b}_o$ of 0.197 m and mean height ($\overline{a}_o$) of 0.191 m].

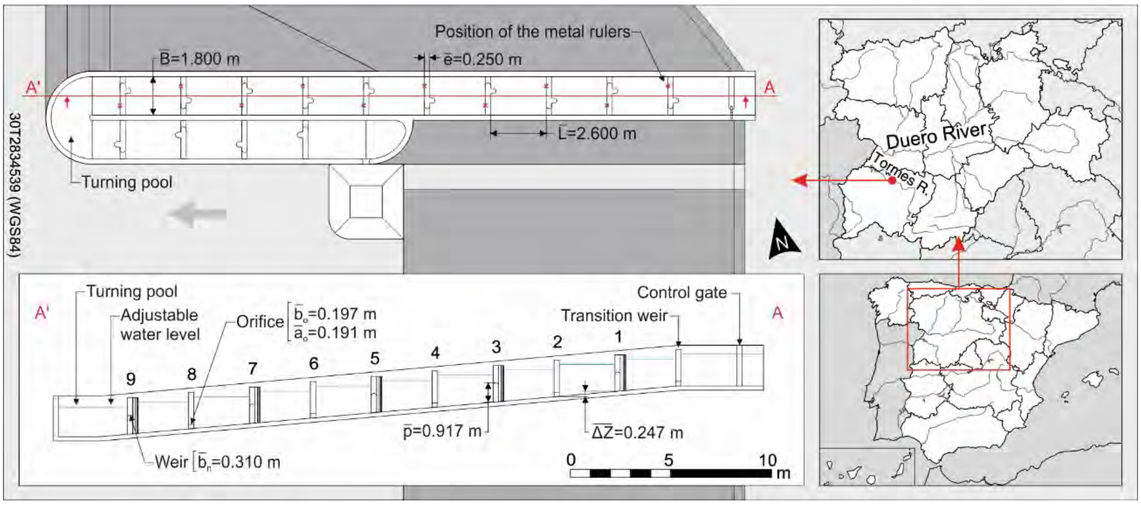

Fig. 2. Location and schematic representation of the studied SNOF in La Flecha (Salamanca, Spain). Numbers 1 to 9 indicate the nine cross-walls used in this study.

The geometrical parameters were measured by topographic surveying with a total station Leica TC307 to a precision of 0.001 m. Smaller details, such as orifice and notch dimensions, were measured by means of a metal ruler to the same precision level. The measurement of all individual characteristics was necessary in order to discriminate between the non-uniform performance due to geometrical differences and the non-uniform performance due to water level changes.

2.2. Experimental arrangement and experiments

Table 1 summarizes all the experiments performed. To determine flow interdependencies between orifices (O) and notches (N), both were studied separately and in combination (NO). In the experimental scenarios where notches or orifices had to be closed, wood covers were used and the junctions with the concrete were completely sealed by means of insulation foam. In those experimental scenarios involving the notch, streaming (S) and plunging (P) regimens were studied. For each combination above (O, N.S., N.P., NO.S. and NO.P.) three different discharge regimes were studied: low (L), high (H), and medium (M) (Table 1). The discharge was controlled through the gate situated in the upper slot (Fig. 2) and was measured by chemical gaging using Rhodamine WT as tracer (Martínez, 2001). This gate was installed for the maintenance and cleaning of the fishway, and provided the opportunity to achieve different hydrodynamic scenarios in a single season.

Table 1. Description of the studied scenarios. O: Orifice alone; N: Notch alone; NO: Notch and orifice together; L: Low discharge; M: Medium discharge; H: High discharge; P: Plunging performance; S: Streaming performance; P/S: Partly plunging and partly streaming; 1: Backwater profile; 2: Drawdown profile; U: Uniform profile.

Scenario	Discharge (m^3/s)	$h_{1,1}$ (m)	$h_{2,9}$ (m)
O.L.2	0.078	1.483	0.860
O.L.U	0.072	1.120	0.861
O.L.1	0.060	1.110	1.447
O.M.2	0.080	1.514	0.987
O.M.U	0.065	1.339	1.090
O.M.1	0.063	1.282	1.481
O.H.U	0.073	1.531	1.298
O.H.1	0.070	1.502	1.485
N.P.L.U	0.077	1.173	0.881
N.P/S.L.1	0.077	1.172	1.503
N.S.M.2	0.135	1.297	0.862
N.S.M.U	0.135	1.292	1.055
N.S.M.1	0.135	1.294	1.467
N.S.H.2	0.242	1.510	1.083
N.S.H.U	0.242	1.507	1.256
N.S.H.1	0.242	1.509	1.440
NO.P.L.U	0.078	0.992	0.795
NO.P/S.L.1	0.078	0.994	1.472
NO.P.M.U	0.095	1.065	0.827
NO.P/S.M.1	0.095	1.066	1.466
NO.P.H.U	0.131	1.141	0.912
NO.P/S.H.1	0.131	1.141	1.475
NO.S.L.U	0.151	1.195	1.040
NO.S.L.1	0.151	1.190	1.488
NO.S.M.U	0.195	1.306	1.097
NO.S.M.1	0.195	1.299	1.391
NO.S.H.U	0.271	1.455	1.230
NO.S.H.1	0.271	1.458	1.479
NO.S.H.U*	0.337	1.519	1.255

*Extra scenario taking advantage of the high river flow recorded while the experiments were carried out.

The study was conducted in nine cross-walls located upstream, after the control gate and a transition weir (Fig. 2). The turning pool was adapted to allow the artificial modification of the water level to achieve uniform profiles (U) (same depth in all pools), as well as non-uniform profiles which include conceptual backwater profiles (1) (higher depths upstream) and drawdown profiles (2) (higher depths downstream) (Rajaratnam et al., 1986; Chow, 2004; Fuentes-Pérez et al., 2014).

After excluding nearly impossible scenarios, due to the complexity required to reach them in real fishways (e.g. backwater profiles in NO experiments), twenty nine scenarios were studied (Table 1). The water level was measured to a millimetric precision in each cross-wall by means of metal rulers installed downstream and in the opposite site of the notches, where the water surface is more stable (Fig. 2). The water level oscillations were recorded for 8 seconds using a camera (Cannon EOS 600D) with a sampling rate of 25 Hz; in all the cases a stable mean value was obtained after two seconds (50 samples).

2.3. Discharge equations

There are different ways to interpret the operation of SNOFs. Rajaratnam et al. (1988, 1989), following earlier works on vertical slot fishways (Rajaratnam et al., 1986) and baffle fishways (Rajaratnam and Katopodis, 1984), proposed the use of dimensionless relationships to describe their performance. These equations have been widely proven and reaffirmed (Ead et al., 2004; Yagci, 2010). However, according to the formula employed, they are independent of the h_2 of each cross-wall and, therefore, they are only valid for uniform operational conditions. It is also possible to use the classical equations for weirs to interpret the operation of SNOFs (Martínez de Azagra, 1999; FAO/DVWK, 2002; Larinier, 2002a; Santos et al., 2012). In this case, discharge through a notch (Q_n) is described using the equation for weirs proposed by Poleni (1717) together with a discharge coefficient that describes the working conditions under plunging performance (C_p), and another coefficient that reflects the effect (contractions and upstream discharge influence) of the streaming performance (C_s) (Eq. 1). In addition, the discharge through submerged orifices (Q_o) is described by the equation derived from Bernoulli's principle together with a discharge coefficient that takes into account the effect of contractions and expansions (C_o) (Eq. 2). In both equations g stands for gravity.

$$Q_n = \frac{2}{3} \cdot C_p \cdot C_s \cdot b_n \cdot \left(h_1 - p\right)^{1.5} \cdot \sqrt{2 \cdot g} \tag{1}$$

$$Q_o = C_o \cdot b_o \cdot a_o \cdot \sqrt{2 \cdot g \cdot \Delta H} \tag{2}$$

C_p is usually defined by Rehbock's equation for free discharge sharp-crested rectangular weirs without contractions [application range: $0.05 \leq (h_1\text{-}p) \leq 0.80$ and $0.01 \leq p \leq 1.00$] (Eq. 3) (Rehbock, 1929; Kim, 2001; Ead et al., 2004) or by a constant coefficient (ranging from 0.495 to 0.750, where usually a value of 0.600 in selected for design purpose) (Rajaratnam et al., 1988; Martínez de Azagra, 1999; FAO/DVWK, 2002; Larinier, 2002a). C_s takes into account the performance of the notches in submerged conditions, thus, generally the equation for sharp-crested rectangular weir proposed by Villemonte (1947) is used (Eq. 4, $\beta_0 = 1.000$ and $\beta_1 = 0.385$) [application range: $0 \leq (h_2\text{-}p)/(h_1\text{-}p) \leq 0.90$] (Martínez de Azagra, 1999; FAO/DVWK, 2002; Larinier, 2002a). C_s will be equal to 1 when the notch works under plunging performance.

$$C_p = 0.605 + \frac{1}{1000 \cdot \left(h_1 - p\right)} + 0.08 \cdot \left(\frac{h_1 - p}{p}\right) \tag{3}$$

$$C_s = \beta_0 \cdot \left[1 - \left(\frac{h_2 - p}{h_1 - p}\right)^{1.5}\right]^{\beta_1} \tag{4}$$

2.4. Water level calculation

To simulate the water level distributions in SNOFs under different scenarios, it is necessary to take into account the specific geometrical characteristics of each pool and cross-wall. After defining these characteristics, an iterative bottom-up calculation can be carried out considering boundary conditions: the discharge through the fishway or the headwater level (upstream water level in the first cross-wall considered, $h_{1,1}$), and $h_{2,n}$ (where n is the number of cross-walls studied or in the fishway). A complete definition of this algorithm is explained in Fuentes-Pérez et al. (2014).

2.5. Validation

The fit of the proposed discharge equations has been evaluated using r-squared (R^2), as well as variance of observed and predicted values [σ^2 = RSS/(n-2), where RSS is the residual sum of squares] and graphical validation. The comparison of the predicted water level profiles has been carried out by comparing the mean relative errors (MRE) for each scenario as well as by checking water level distributions.

3. Experimental results and discussion

3.1. Discharge coefficients

Traditionally, the same discharge coefficients have been used for the design of both fishways and weirs. However, the performance of fishways is different mainly due to the slope, thickness of cross-walls, contractions and the dependency between cross-walls, which will probably cause a different water level distribution. Thus, the classical formulation must be either confirmed or modified.

3.1.1. Orifices

The study of submerged orifice fishways has demonstrated, for wide a range of operation conditions, that ΔH in all cross-walls remains constant independently of the water levels upstream and downstream the fishway (Boiten and Dommerholt, 2006). This property is derived from Eq. 2 and from the fact that the discharge coefficients only depend on the shape of the submerged orifice and the thickness of the cross-walls, remaining constant for different water levels.

The above described performance has also been observed in the fishway studied here. For all orifice experiments (ΔH from 0.176 m to 0.324 m), the discharge coefficient ($C_o \pm SD$ = 0.876 ± 0.050) was independent from the water levels of each cross-wall (h_1, h_2, and ΔH). The values obtained agree with the values observed by other authors (Larinier, 2002a; Boiten and Dommerholt, 2006). Likewise, C_o shows a small correlation with the dimension of the orifice ($a_o \cdot b_o$) and Q_o. This is due to the fact that, since C_o is independent to the water level variables, the simplifications from orifice dimension measuring and fishway discharges (chemical gaging) are transmitted directly to C_o. The cross-walls were built *in situ* with concrete, which produced geometrical irregularities that cannot be characterized with the geometrical variables used in the calculation (a_o and b_o). These irregularities are greater in orifices than in notches, due to their position near the rough bed and their small dimensions. Since these deviations are not considered in a_o and b_o measures, they are translated to variance in C_o.

The observed properties in submerged orifices make them of particular interest to design fishways with autonomous water drop compensation. However, they can suffer from inappropriate attractions or locating difficulty, obstructions, and the drawbacks of trapping floating debris (FAO/DVWK, 2002; Larinier, 2002a). Likewise, some fish species may be reluctant to use them [e.g. American shad (*Alosa sapidissima*) (Larinier, 2002a)] or may prefer other types of connections between pools [e.g. Iberian chub (Branco et al., 2013) or Iberian nase (Sanz-Ronda et al., 2015b)].

3.1.2. Notches

Although orifices are important to maintain the fishways clean of sediments and to allow the pass of certain fish species (Larinier, 2002a), generally the most important fraction of discharge will flow through the notch. To differentiate between plunging and streaming effects, and to determine the independency between notches and orifices, first, taking into account the calculated C_o, Q_o was deducted from combined notch and orifice experiments. Then, the coefficients for the submerged equation (β_0 and β_1 in Eq. 4) were determined from observed data, as recommended by Villemonte (1947) [Fig. 3(b)]. In order to do this, only the data from submerged notches were used, obtaining a first approximation for the coefficients, where β_0 can be considered a constant approximation of C_p. To evaluate the distribution of C_p, the first approximation for C_s was deducted from all data (dividing the data by the estimated C_s). This demonstrated that the distribution of C_p depends on h_1, and describes an exponential model with a decelerated increase of the coefficient that approaches a horizontal asymptote [Fig. 3(a)]. Finally, the observed models for discharge coefficients were fitted together, using all the data, to obtain the final expressions (Fig. 3, Eq. 5 and Eq. 6).

$$C_p = 0.689 \cdot \left[1 - e^{-8.889 \cdot (h_1 - p)} \right] \qquad (5)$$

$$C_s = \left[1 - \left(\frac{h_2 - p}{h_1 - p}\right)^{1.5}\right]^{0.331} \qquad (6)$$

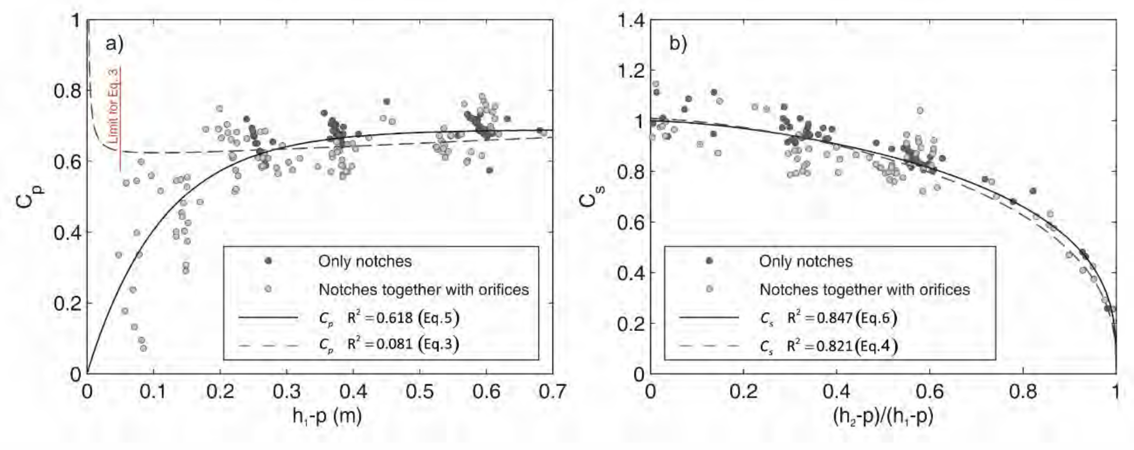

Fig. 3. Fits of the discharge coefficients for the notches. a) Discharge coefficient for the plunging regimen (C_p). b) Discharge coefficient for the streaming regimen (C_s).

Fig. 3 shows that notch data for both set of experiments (notches together with orifices and notches alone) can be explained by the same equations, demonstrating that the performance of orifices and notches can be considered independent for the estimation of water level distribution. Likewise, even if independent fits for each set are considered, the same mean variance is obtained (0.003).

The C_p model differs from the values and equations proposed by other authors (Rehbock, 1929; Rajaratnam et al., 1989; Martínez de Azagra, 1999; FAO/DVWK, 2002; Larinier, 2002a; Ead et al., 2004); however, its performance has a logical explanation. The coefficient must be 0 when h_1-$p \leq 0$, then, when the discharge starts, the influence of the sill is big producing a small C_p. As h_1 increases, sill influence remains almost constant, producing a decelerated progressive increase of C_p. Likewise, the kinetic influence of the cross-wall situated upstream, for first stages of the discharge (low h_1), is negligible. It is worth mentioning that the measuring error in the experiments increases at lower discharges (lower C_p values) due to a greater influence of the geometrical measure precision, as well as the nature of the chemical gaging and the lower mixing power in pools. As the water height increases, C_p will approach a horizontal asymptote with a value in the range suggested by other specialized references (Martínez de Azagra, 1999; FAO/DVWK, 2002; Larinier, 2002a).

Regarding C_s, most of the guides for fishway design recommend Eq. 4, which uses the coefficients proposed by Villemonte (1947) for rectangular weirs (β_0 = 1.000 and β_1 = 0.385) (Martínez de Azagra, 1999; Larinier, 2002a; FAO/DVWK, 2002). However, despite the fit being considerably good [Fig 3(b), R^2=0.821], as recommended by Villemonte, these coefficients should be calculated or evaluated for each configuration. The proposed new relations suggest that for the same scenario, there is a smaller influence of C_s (a greater value) than in Villemonte's experiments due to the different conditions in the fishway (possibly due to a different slope or smaller water path than the one used by the author).

The observed difference between the data and the proposed model, as argued in the previous section, seems to be related mainly to the simplification of geometrical variables and chemical gagging. This becomes more obvious when comparing experimental data from notches, with data from notches together with orifices (Fig. 3). The variance of NO experiments is greater (0.005 vs 0.001) because the geometrical simplification of the orifices is also involved in the estimation of C_p and C_s. However, as described in the next section, the deviations in the estimation of water level distribution as a result of this variance will be rather small.

3.2. Depth profiles and applications

With equations able to predict the water level distribution in non-uniform scenarios, it will be possible to model the performance of fishways under dynamic situations and adapt their operation to new conditions, improving, when necessary, the attraction efficiency [number of fish that are able to find the entrance (Bunt et al., 2012)] and passage efficiency [number of fish exited of those that entered in the fishway (Bunt et al., 2012)] and, thus, achieving a higher fishway efficiency during longer periods of time.

The proposed equations (Eq. 5 and Eq. 6) together with the algorithm described in Fuentes-Pérez et al. (2014) are able to estimate water level distributions with a high degree of precision in all the studied scenarios (Table 2 and Fig. 4). In all cases, the maximum MRE was registered in orifices when working alone (Table 2). This, as it was discussed previously, is explained by the observed variance in the discharge coefficient. Despite this, the deviations are small, higher in ΔH distributions because of the normalization of the error [Fig. 4(b)]

Table 2. MRE (%) for the studied scenarios with the proposed equations.

Scenario	**Notch and orifices**			**Notch**			**Orifices**			**Mean**		
	h_1	h_2	ΔH	h_1	h_2	ΔH	h_1	h_2	ΔH	h_1	h_2	ΔH
Uniform	1.74	1.93	3.79	0.84	0.84	3.70	4.40	4.97	9.36	1.96	2.17	4.69
Backwater	1.36	1.52	7.17	0.66	0.65	4.89	4.12	3.85	9.30	1.88	1.88	7.13
Drawdown	-	-	-	0.68	0.66	4.23	8.25	9.13	10.03	4.46	4.89	7.13
Mean	1.57	1.74	5.35	0.73	0.72	4.28	5.38	5.68	9.53	2.28	2.43	6.09

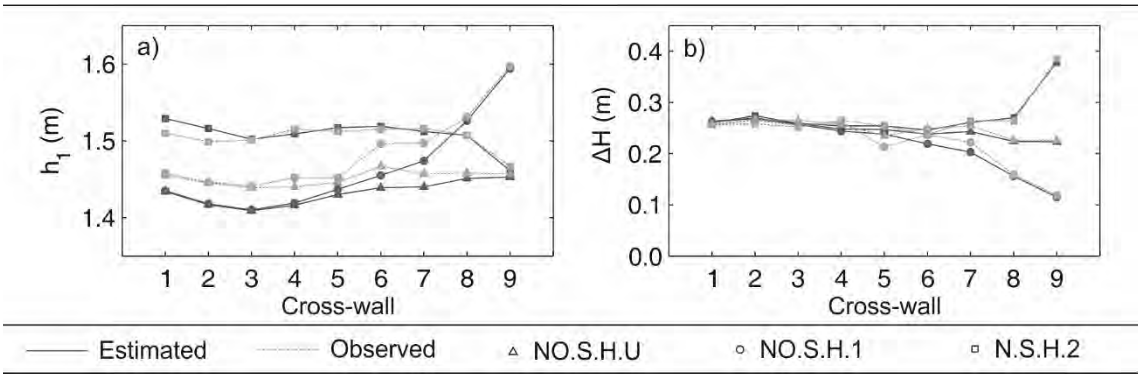

Fig. 4. Water level distributions in the 9 studied cross-walls of the fishway. a) Observed and estimated h_1 profiles for 3 of the 29 scenarios. b) Observed and estimated ΔH profiles for 3 of the 29 scenarios.

When the fishway is placed correctly [i.e. in the attractive areas for fish (Larinier, 2002b)], first, the fish will need to find the fishway entrance in order to pass it. To accomplish this, the entrance needs to strike a compromise between attracting the fish and enabling them to enter (Bunt, 2001; Larinier, 2002b; Williams et al., 2012). Non-uniform performances, produced by changes in headwater or tailwater levels, will modify the hydraulic conditions in the entrance from the ones defined during the design process, causing backwater or drawdown profiles. Backwater profiles are produced by decreasing tailwater or increasing headwater levels. These can generate excessive ΔH in most downstream cross-walls, increasing velocities [the expected maximum velocity is $\sqrt{2\cdot\Delta H\cdot g}$ (Rajaratnam et al., 1986; Larinier, 2002a; Liu et al., 2006)], turbulence, noise or oxygenation. Although in a first instant this can increase the attraction (Williams et al., 2012), entrance can be limited according to the swimming speed of the migrating species involved or the produced turbulence, and sometimes it will require the fish to jump to enter, reducing or impeding its use for some species (Bunt et al., 2012). For instance, Branco et al. (2013) and Sanz-Ronda et al. (2015b) observed a reduced use of notches for Iberian chub, Iberian barbel and Iberian nase, when the fishway entrance was working in plunging regimens. Regarding velocity limits at the entrance, Larinier (2002b) defines an optimal water speed for salmonids and large migrants in the order of 2.0 m/s to 2.4 m/s (ΔH = 0.2-0.3 m). Similar values can be considered for species with comparable swimming capacities such as Iberian barbel and Iberian nase (Sanz-Ronda et al., 2015a).

The backwater profile created by the increase in headwater level can be managed, for instance by designing a first cross-wall (the most upstream one) with a gate and, after this, some cross-walls without ΔZ between them. Likewise, backwater profile created by the decreases of the tailwater level can be managed by decreasing the sill of the downstream notches or installing submerged pre-barrages that will absorb the reduction of the water level. However, the latter is not a probable scenario as the fishway should be designed to work under the reasonably highest difference between the headwater and tailwater levels of the obstacles (Wang, 2008).

Regarding drawdown profiles, they occur when the tailwater level increases or headwater level decreases. However, in most cases, the headwater level will remain more or less constant (Larinier, 2002a). These profiles decrease the ΔH in most downstream cross-walls, reducing, among others, the velocity at the entrance, which, in turn, can produce a diminution on the attraction efficiency. Larinier (2002b) recommends at minimum velocity of 1 m/s at the entrance. In both cases, these issues can be solved by increasing the sill elevation of the most downstream notch or increasing the discharge input in the most downstream pool.

It is possible to use the proposed equations and calculation process to model all defined performances, and design specific solutions (as mentioned above). However, as the fishway should be designed to work under the reasonably highest difference between water levels, and as the headwater level in most cases will not change significantly, the most probable scenario will be the drawdown profile where tailwater level increases. Fig. 5(a) shows the simulation of the defined two options to improve the attraction of a drawdown profile for the studied fishway, that is, the elevation of the sill of the most downstream cross-wall (*p* + 0.550 m) and the increase of the discharge input in the last pool (Q + 0.250 m^3/s). Both solutions will increase the final ΔH to reach to the desired value.

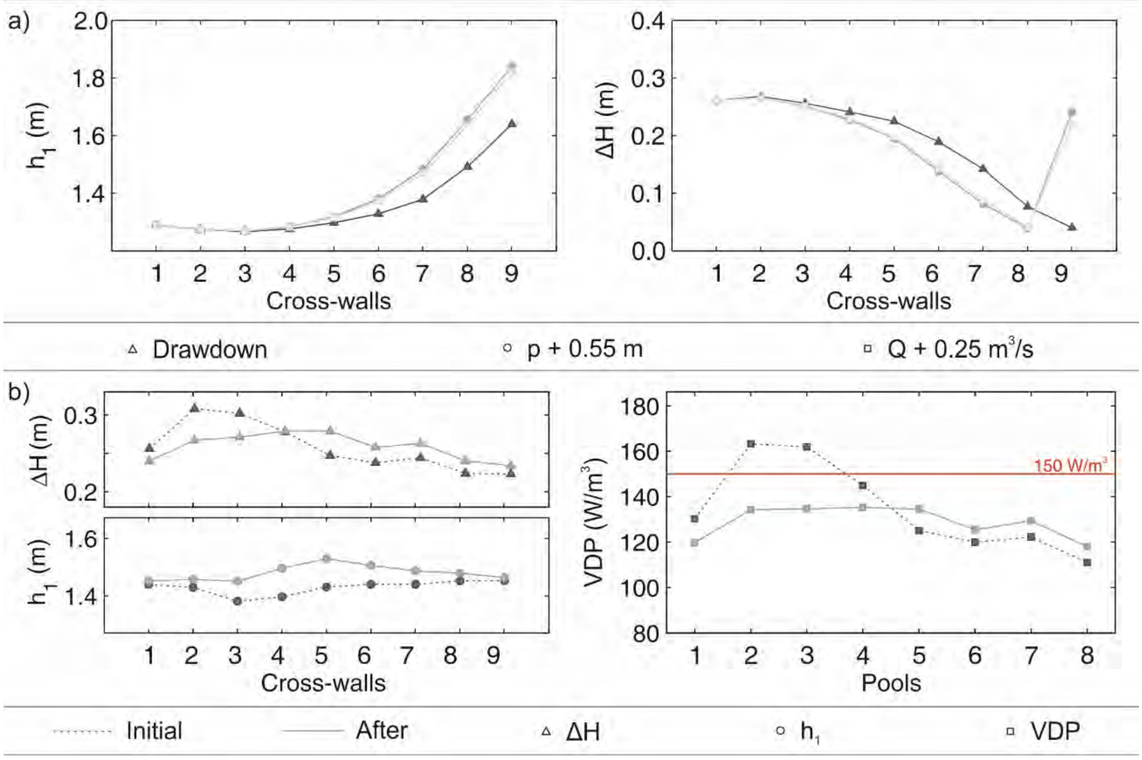

Figure 5. Examples of use of the proposed equations to improve efficiency of the fishways. a) Distribution of h_1 and ΔH in an attraction optimization example with 2 options to improve the use of a fishway with drawdown profile (boundary conditions: Q = 0.200 m^3/s and $h_{2,9}$ = 1.600 m). b) modification of h_1, ΔH and *VDP*, in a fishway with higher ΔZ (≈0.30 m) between cross-

walls 3 and 4, and 4 and 5 after the increase of sill height in downstream cross-walls (from p_3 to p_9 +0.05, +0.10, +0.12, +0.08, +0.06, +0.04, +0.02 m, respectively) (boundary conditions: Q = 0.271 m^3/s and $h_{2,9}$ = 1.230 m).

Once fish have entered to the fishway, its internal hydraulic performance will determinate the passage efficiency. Usually, at practical and design level, two main factors should be considered when evaluating the internal performance: ΔH between adjacent pools and the volumetric dissipation power (VDP). It is possible to estimate both variables with the proposed equations, as VDP only depends on the pool geometry, and the calculated variables [$VDP = Q \cdot \Delta H \cdot g \cdot \rho / \left(h_0 \cdot B \cdot L \right)$ where ρ is the water density (kg/m^3)] (FAO/DVWK, 2002; Larinier, 2002a; Towler et al., 2015). VDP will provide an indication of average pool turbulence and ΔH can be considered as an indicative of the maximum velocity that the fish will need to overcome.

VDP should be maintained under certain levels according to the target species, fishway type and type of pools (step pool, resting pool or turning pool) (Towler et al., 2015). It is roughly correlated with more complex parameters (such as velocity field, turbulence or shear stress levels within the pool), which, in turn, are strongly correlated with fish preferences. For instance, several studies have observed that, within a pool, the Iberian barbel has preference for areas with lower velocities, turbulence and shear stress (Silva et al., 2011; Silva et al., 2012; Alexandre et al., 2013).

The maximum velocity to be overcome by the fish, directly related with ΔH, will occur in the cross-walls. This fact has been shown for example in electromyogram telemetry studies that revealed that Iberian barbels reached the maximum swimming speed during the orifice passage within a pool-weir fishway (Alexandre et al., 2013). After surpassing the cross-wall is believed that the fish rest, if necessary, within the recirculation areas of the pools before facing to the next cross-wall (Silva et al., 2011; Alexandre et al., 2013).

Thus, each cross-wall can be seen as a small obstacle that fish will need to surpass taking advantage of its abilities and the resting area. In this sense, local design or constructing failures inside the fishway could reduce fish passage. By modeling the internal performance of a fishway, it is possible to compensate for any possible drawbacks. For instance, Fig. 5(b) simulates a deviation in ΔZ (real ΔZ + 0.05 m) between cross-walls 3 and 4, and 4 and 5, and shows one of the possible solutions. The deviation of ΔZ will produce the increment of *VDP* and ΔH from the recommended ones for the target species in the upstream pools, which could be a limiting factor for passage. However, by using the proposed equations and bottom-up calculations, it is possible to design a solution (in this case the increase of downstream notches sill height) to compensate for these errors, reducing both, ΔH and *VDP*.

4. Summary and conclusions

In this article, a modification of the discharge equations for submerged notch and orifice fishways is proposed. Its formulation differs from the classical method because (a) the equations have been specifically adapted to fishways and (b) non-uniform scenarios have also been studied. The equations fit the observed data and, for most common design conditions, suggest higher discharge coefficients than the traditional values and equations used. Likewise, a new logical distribution pattern for C_p has been detected, observed, and modeled.

The discharge equations together with a logical bottom-up iterative calculation are able to correctly model the performance of fishways under different regimens, and uniform and non-uniform scenarios. This will allow to create specific solutions for changing scenarios or when building errors are detected.

This work also exposes the necessity to specifically adapt the classical design equations to fishways in order to model correctly the hydraulic parameters (ΔH, h_0, VDP, etc.) that might limit their use. The correct modeling and interpretation could be used to design more accurate and better adapted solutions, to determine whether a fishway has hydraulic constraints which could compromise its efficiency, and to adapt or correct fishways when necessary.

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6. Notation

The following symbols are used in this paper:

a_o = height of the orifice (m)

B = pool width (m)

b_n = notch width (m)

b_o = orifice width (m)

C_o = discharge coefficient for orifices

C_p = discharge coefficient for the plunging regimen

C_s = discharge coefficient for the streaming regimen

e = thickness of the cross-wall (m)

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

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

h_1 = mean water level of the flow in the pool in relation to the upstream of the notch (m)

$h_{1,i}$ = mean water level of the flow in the pool in relation to the upstream of the notch in the cross-wall number i (m)

h_2 = mean water level of the flow in the pool in relation to the downstream of the notch (m)

$h_{2,i}$ = mean water level of the flow in the pool in relation to the downstream of the notch in the cross-wall number i (m)

i = cross-wall number

L = pool length (m)

n = total number of cross-walls

p = sill height (m)

Q = discharge or flow rate ($Q = Q_n+Q_o$ for combined scenarios) (m^3/s)

Q_n = discharge through notches (m^3/s)

Q_o = discharge through orifices (m^3/s)

R^2 = determination coefficient

S = slope of the fishway (m/m)

VDP = volumetric dissipation power (W/m^3)

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

ΔH = difference in water level between pools or head drop ($\Delta H = h_1 - h_2$) (m)

ΔZ = topographic difference between cross-walls (m)

ρ = density of water (kg/m^3)

σ^2 = variance

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