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VICEPRESIDENCIA TERCERA DEL GOBIERNO
MINISTERIO PARA LA TRANSICIÓN ECOLÓGICA Y EL RETO DEMOGRÁFICO
CONFEDERACIÓN HIDROGRÁFICA DEL DUERO, O.A.

Montes

UNIVERSIDAD POLITÉCNICA DE MADRID

UPHILL FLOW **ROCK RAMP** DESIGN

José Anastasio Fernández Yuste & Carolina Martínez Santa-María
***Forest Engineering College.** Technical University of Madrid*

English version:

Roberto Martínez Romero
Senior Hydromorphologist
Scottish Environment Protection Agency

Fernando Magdaleno Mas
Technical University of Madrid

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ACTUACIONES PARA LA PROTECCIÓN Y CONSERVACIÓN DE CIPRÍNIDOS IBÉRICOS DE INTERÉS COMUNITARIO

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Junta de Castilla y León

patrimonio natural de castilla y león

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AUTHORS: José Anastasio Fernández Yuste and Carolina Martínez Santa-María
E.T.S. INGENIERÍA DE MONTES, FORESTAL Y DEL MEDIO NATURAL
UNIVERSIDAD POLITÉCNICA DE MADRID

COLLABORATORS: Gustavo González, ICTHIOS GESTIÓN AMBIENTAL S.L.

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DESIGN AND LAYOUT: Carolina Gómez, www.estudioplanc.com
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PRESENTATION

The fluvial continuity is the capacity of the river to transport water, sediment and biota. It is measured longitudinally and laterally, this last in relation with the floodplain, and it has become one key element in the public management of water. The Water Framework Directive has played, as in many other questions, an essential role because it points out to the relevant institutions/public bodies from the different States that we must include in our worries and our day-to-day work measures of planning, action, control, monitoring and improvement of the fluvial continuity of rivers.

In the topic of the improvement of fluvial continuity, the Confederación Hidrográfica del Duero (River Basin Management, Duero Basin) has been pioneer, having carried out many projects to remove redundant transversal barriers, also promoting with the owners/managers of active barriers, measures to mitigate their negative effects. Among those measures, we could highlight those aimed at those aimed to letting fish species migrate, known as "fish passes".

*The traditional and most common model is the technical fish pass. They are concrete structures, fixed to the main body of the dam/weir that is aimed to be passable, and that brings many design and exploitation issues. The best of the fish passes, with the best design, and with a detailed construction, requires permanent monitoring and maintenance. And even this way, the effectiveness is still partial. In order to know and improve this type of fish pass devices, the Confederación Hidrográfica del Duero, as part of a collaboration agreement with the Group of Ecohydraulics of the University of Valladolid (Palencia Campus), developed a "**Manual para la evaluación de la funcionalidad de pasos para peces de estanques sucesivos. Metodologia AEPS** (1.0)" (Handbook to assess the functioning of technical fish passes. Methodology AEPS 1.0).*

*The experience developed throughout more than 10 years, has shown that the fishways "ramp" type are the most efficient and the ones that are better integrated. As a consequence of the works undertaken in many river restoration projects, we have acquired a knowledge that we want to share. The "**Uphill flow rock ramps. Handbook**" presented here, with a solid foundation thanks to its empirical base, follows the track of the previous works, and explains the functioning and help to design and build this type of fish pass devices. This handbook comes out from a collaboration between ICTHIOS, and Technical University of Madrid, who have walked and measured many rivers and ramps to gather a high level knowledge that they share with all of us here.*

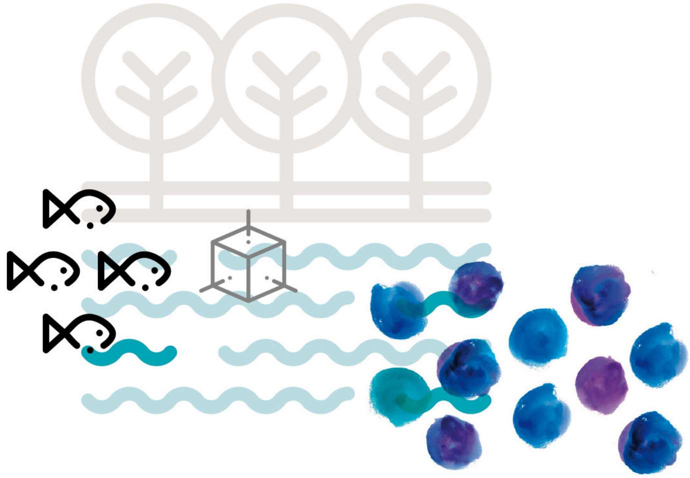
Ignacio Rodríguez Muñoz
Water Commissioner of the Duero Hydrographic Confederation

ACKNOWLEDGMENTS

Our names –Carolina and Tasio- appear as authors of this Handbook, and although being right, we have written it is not entirely truth. This is so because in order for this Handbook to become a reality, it was required, indeed, the collaboration, the enthusiasm, the trust, the encouragement, the knowledge and the implication of many others.

Without the encouragement and trust of Carlos Marcos –CH Duero-, without the knowledge of Gustavo González, Anna Pedescoll and Rafael Aguado –Icthios-, without the implication and enthusiasm of Luis Carlos Arias, Manuel Oliva –Tragsa-, Javier Carpio –Serbaikal- and David Martinez –Alida-, without the collaboration of Lorenzo Aguilera, Montaña Cepa, Lidia Arenillas –CH Tajo-, we could not have written any single line.

Thanks to this intangible and fertile background that we took from you, which has borne fruit through this text, with the hope that it can help improving the bad condition of our fish populations.



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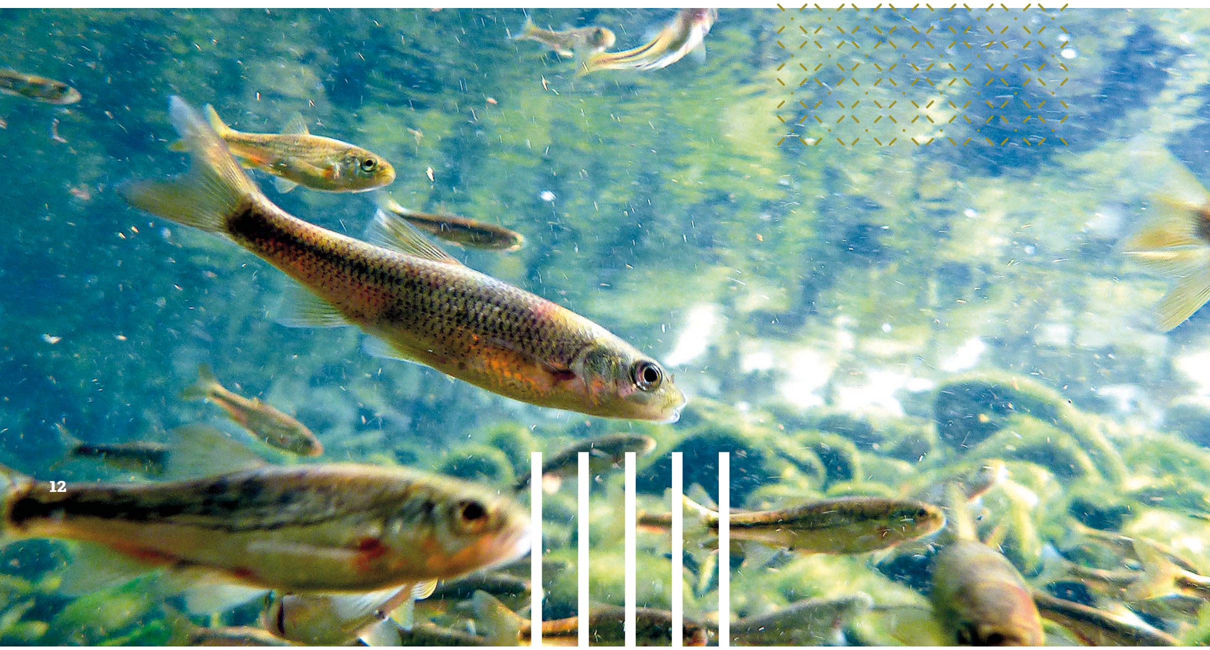
INTRODUCTION



01

The problem_

Many fish species in our rivers migrate at some point of their life stage. The aim of these migrations is getting access to certain habitats looking for proper environmental conditions (water temperature, oxygen…) and suitable (depth, velocity and substrate) that can guarantee, -or can make suitable - , spawning, feeding, shelter, etc. (McIntyre *et al.*, 2015).

The transversal infrastructures that are spread out throughout the river network (dams, weirs, bed reinforcements, drainage…), when no built with a correctly designed fishway, can be a real obstacle for fish migration. As a consequence, the distribution and abundance of fish populations in rivers are completely altered (Cooper *et al.*, 2017; Fuller, Doyle and Strayer, 2015).

The solution _

The existing typology of fish pass devices to mitigate the impact of these barriers is very diverse (Franklin *et al.*, 2018; U.S. Fish and Wildlife Service, 2017; Sanz-Ronda *et al.*, 2013).

As a first approximation, two main groups can be distinguished: assisted fish passages and non-assisted fish passages, depending on if fish is supported in progressing the obstacle.

The first group includes lifts, sluices and capture-transport-release. The second group includes technical fish passes (baffles of different typologies), nature-like channels (rock ramps and bypasses) and other devices with more limited uses like pre-barrages or angular ramps, among others.

Rock ramps _

Rock ramps constitute due to its nature-like character, versatility, and its high degree of fish passability, one of the preferable devices for low height obstacles. They consist of channel with nature-like substrate and smooth gradients, designed to keep suitable water depth and velocities during the representative flows for the migratory window season for a target species. Other design elements provide the discharge of higher flows without compromising the stability of the structure.

The flow conditions through the ramp are determined, apart from the gradient, by the distribution and size of the boulders that are part of it. Regarding the distribution of the boulders, rows of boulders lined-up perpendicular to the main flow or random distribution of boulders are the two most frequent distributions, keeping certain patterns in relation to relative distances (FAO/DVWK, 2002).

In most ramps, the flow goes through the spillways or existing notches/gaps between two consecutive boulders. The critical conditions of fish passability are given by the velocity in the flow gap, minimum depth, the jump in the case there is not submerged pass, and the power generated in the water drop and its dissipation to fulfil suitable thresholds.

Uphill flow rock ramps _

The uphill flow rock ramps presented in this handbook keep many of the elements described in the previous rock ramps, but they introduce a particular distribution of boulders within the row.

This new design generates a very particular hydraulic functioning as it induces uphill flows opposite to the main flow on the ramp.

The biologic importance of these new flows is unquestionable; they support the fish progressing upstream, to the point where fish could progress throughout the ramp, being pushed by these uphill flows and only would require swimming to pass the gap between two boulders.

The design and the dimensioning protocol in this handbook, not only guarantee the generation of these uphill flows, but also provide the required velocities in the different gaps and passes, minimum depths in the pools and energy dissipation, in all cases considering the swimming capabilities of the fish species. In addition, it considers elements that allow the designer dimensioning the ramp for a wide range of flows, guaranteeing fish passability for the whole range of flows.

What are you going to find in this handbook? _

The aim is to provide a very clear and practical handbook for the managers and practitioners to support the design and dimensioning of uphill flow rock ramps.

The handbook provides a detailed list of the different variables, calculation process and the required checks to guarantee a good functioning of the device including determination of operative range of flows.

This handbook also takes into account special circumstances as (i) minimum water surface level to guarantee the flow diversion corresponding to the weir/dam, (ii) environmental flows, (iii) effectiveness of the attraction flow and (iv) the design of the meeting pool between the ramp and the river, considering the influence of the variation of depths downstream of the ramp on the hydraulic functioning of the ramp.

The uphill flow rock ramps presented in this handbook are innovative; however, the content of this handbook has made use of (i) hydraulic and hydro-biologic foundations available in the literature (Baki *et al.*, 2017a; Baki *et al.*, 2017b; Muraoka, Nakanishi and Kayaba, 2017; Cassan and Laurens, 2016; Tran *et al.*, 2016; Baudoin *et al.*, 2015; Bretón *et al.*, 2013; Santos *et al.*, 2012; Wang, 2008; Mooney, Holmquist-Johnson and Broderick, 2007; United States Department of Agriculture, 2007), (ii) reflections and contributions from the professional

team working for the CIPRIBER project (https://cipriber.eu/), and (iii) the acquired experience through the different stages (design, calculations, construction and monitoring) of the first uphill flow rock ramp built recently (2019) in El Pardo (Madrid, Spain), as part of the project "Environmental restoration of the Manzanares River, Madrid-Spain".
(http://restauracionfluvialriomanzanares.es/)

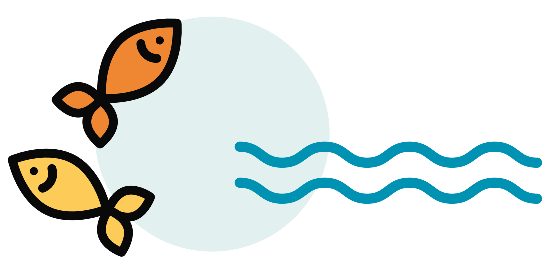
Finally, to emphasise there is a piece of software available for the user to apply the sequence of designing the uphill flow rock ramp.

Limitations _

The diversity of the fluvial network, both natural (hydrology, topography, biology, etc.) and anthropogenic (accesses, types of obstacles for fish migration, water supplies, environmental constraints, etc.) is of such complexity that it makes impossible to produce a handbook able to cover every single design constraint. Therefore, the recommendations compiled in this guidance must be adjusted in a case-by-case basis to the particular constraints of the project.

It is important to highlight that the application of this handbook should exclusively not be approached either from a hydraulic perspective or from a hydro-biologic point of view. Both aspects – hydraulic and hydro-biologic – are essential and indispensable to achieve a suitable design for this type of ramps and, in general, every type of fishway device (Valbuena Castro *et al.*, 2016).

Lastly, to point out that a correct design and dimensioning of the ramp is important, but even though; it is not guarantee of the functionality of the device. Together with that design phase and dimensioning is required to consider, because of their relevance, the construction, maintenance and monitoring phases (Pedescoll *et al.*, 2019; Dodd, Cowx and Bolland, 2017; Baudoin *et al.*, 2015; O'Connor, Mallen-Cooper and Stuart, 2015; BAW/BfG, 2015), those stages are out of the scope of this handbook, but the user must consider them rigorously in order to propose an effective solution to tackle the problems related with the obstacles for fish migration.



02

HYDRAULIC FUNCTIONING

02

The most characteristic component of this type of device is the boulder (*FIGURE 1*), natural or artificial, defined by its three spatial dimensions (*Db, Wb y Hb*).

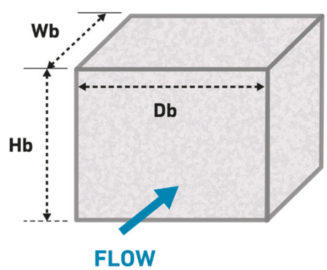


FIGURE 1
Diagram showing the geometry of the boulders, natural or artificial: visible height (Hb), diameter (Db) and width (Wb).

The references for a correct dimensioning of these three variables will be given in the following chapters of this handbook.

The boulders are placed in rows, with the dimension *Db* transversal to the main flow. Although the main axis of the row is transversal to the main flow, in each row the distribution of the boulders is following a zigzag line, keeping an angle (α) between two consecutive boulders (*FIGURE 2*).

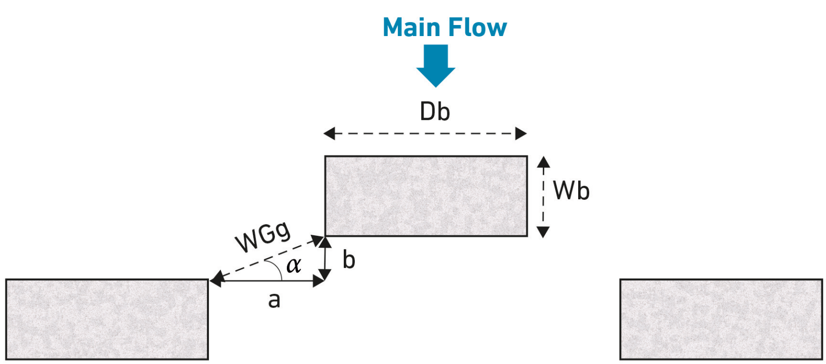


FIGURE 2
Plan view of the distribution of boulders in a row (annex 1 provides a description of all the variables included in this handbook).

Angle α is an independent variable, but it must be verified between: $30° \leq \alpha \leq 45°$

The gap between boulders is called "width of the geometric gap" (WGg), with a projection on the main flow direction (*b*) and a projection on the transversal direction (a) (*FIGURE 2*).

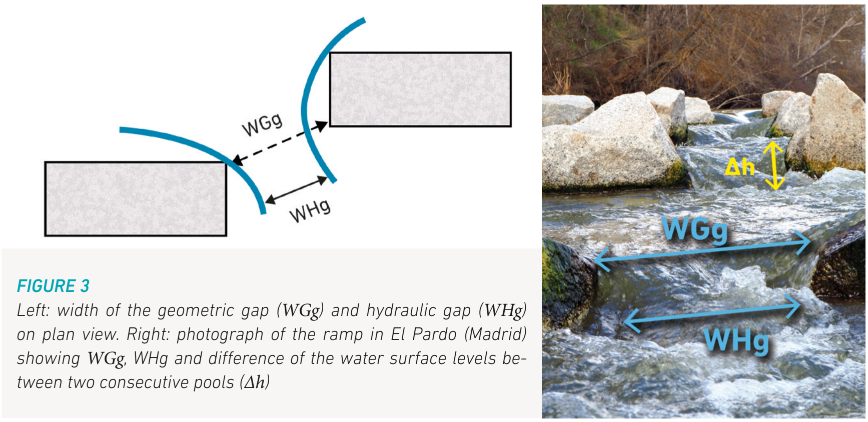


FIGURE 3
Left: width of the geometric gap (WGg) and hydraulic gap (WHg) on plan view. Right: photograph of the ramp in El Pardo (Madrid) showing WGg, WHg and difference of the water surface levels between two consecutive pools (Δh)

Because of the contraction of the water flow through the gap, the width of the hydraulic gap (WHg) is always narrower than the geometric width (WGg) (*FIGURE 3*). This reduction is a function of the velocity of the water drop and therefore of Δh (difference of the water surface level between two consecutive pools), thus, the relationship between them is:

$$\text{Eq. 1} \quad WHg = WGg\text{-}Cc * \Delta h$$

where[1]:

WHg= width of the hydraulic gap
WGg= width of the geometric gap
Δh= difference of the water surface height between two consecutive pools
Cc= contraction coefficient

The flow through the gap can be estimated with the equation (Fuentes-Pérez *et al.*, 2017):

$$\text{Eq. 2} \quad Qg = {}^{2}/_{3} * \sqrt{2g} * Cd_{gap} * WHg * (h_1)^{1.5}$$

$$\text{Eq. 3} \quad Cd_{gap} = \beta_0 * \left[1 - \left(\frac{h_2}{h_1}\right)^{1.5}\right]^{\beta_1}$$

where:

Qg= flow throughout the flow gap
Cd_{gap} = gap discharge coefficient

[1] *All the variables in this handbook are in the International System (IS)*

WHg = width of the hydraulic gap

h_1= depth upstream of the boulder

h_2= depth downstream of the boulder

β_0= coefficient (0.812)

β_1= coefficient (0.335)

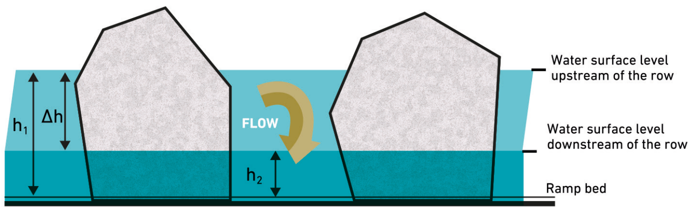


FIGURE 4
Front view of the flow through a row. See the water surfaces upstream and downstream of the row and the variables: depth upstream of the boulder (h_1), depth downstream of the boulder (h_2), and difference of the water level between two consecutive pools (Δh)

Knowing that the water spill goes always perpendicular to the flow gap, the zigzag distribution of the boulders in the row generates that water spill of two consecutive gaps interact one against the other, slowing down each other and dissipating the power of the vertical drop (*FIGURE 5*).

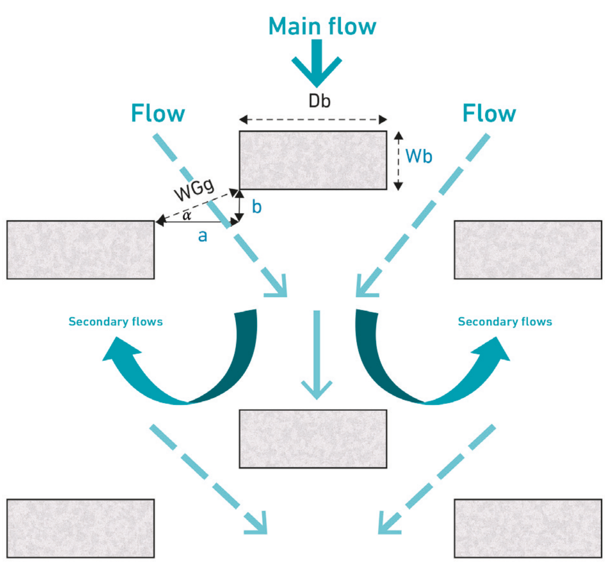


FIGURE 5
Simplified diagram showing the flow through two consecutive gaps and between two consecutive rows.

This slowed down flow would start its acceleration downstream on the ramp, but the flat face of the boulder of the following row is slowing it down again. As a consequence of this process, uphill secondary flows are generated (opposite to the main flow), that can be easily observed on site, where, very low, nulls, and even negative velocities have been measured in many occasions (*FIGURE 6*). This fact is the reason to name these ramps as "uphill flow ramps".

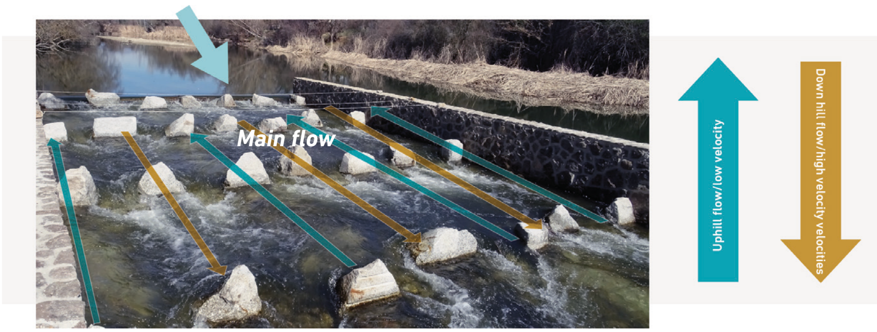


FIGURE 6
Ramp in El Pardo (Madrid). See the two zones of fast velocities (yellow) and the zones of uphill secondary flows (blue).

The generation of these uphill flows is guaranteed as long as, in the row, the boulders are distributed with the angle (α). This way, the flow through the ramp alternates between zones of downhill flow with high velocities and zones of uphill flow with low velocities. This fact allows defining a fundamental element of the design in this type of ramps: the "module".

A module is the minimum unit required to generate uphill flows and is formed by three consecutive boulders belonging to one particular row and placed in the correct distribution.

FIGURE 7 shows, in a schematic way, the flow behaviour in a ramp with three modules, this ramp generates four resting zones (or uphill flow) and three fast zones of downhill flow.

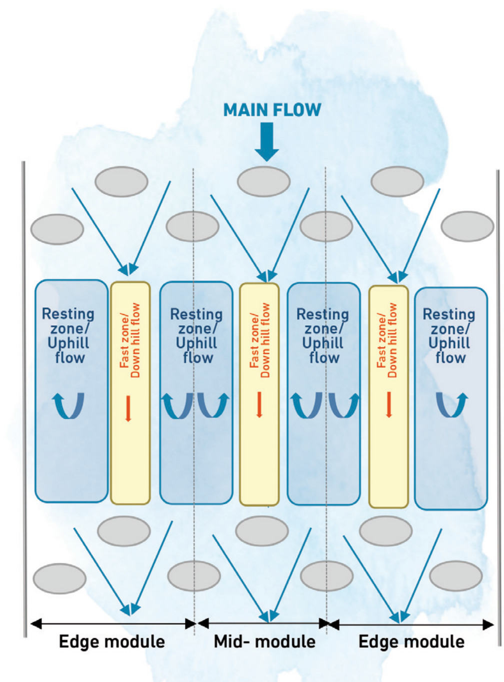


FIGURE 7
Diagram of the zonation of flows in uphill flow ramps showing the two types of modules.

The modules present two different typologies: edge modules (corresponding to the modules placed on the sides of the ramp) and mid-modules (the remaining ones). (*FIGURE 8*).

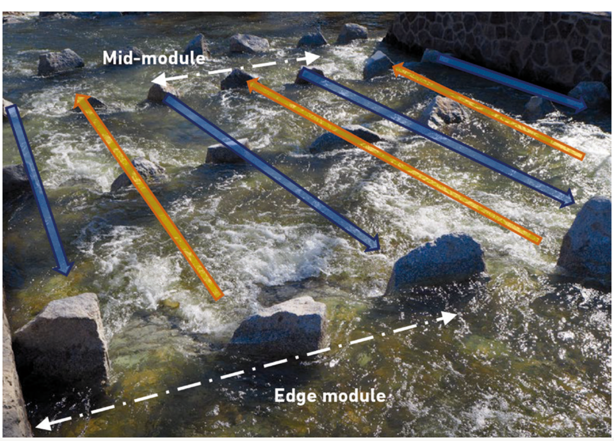
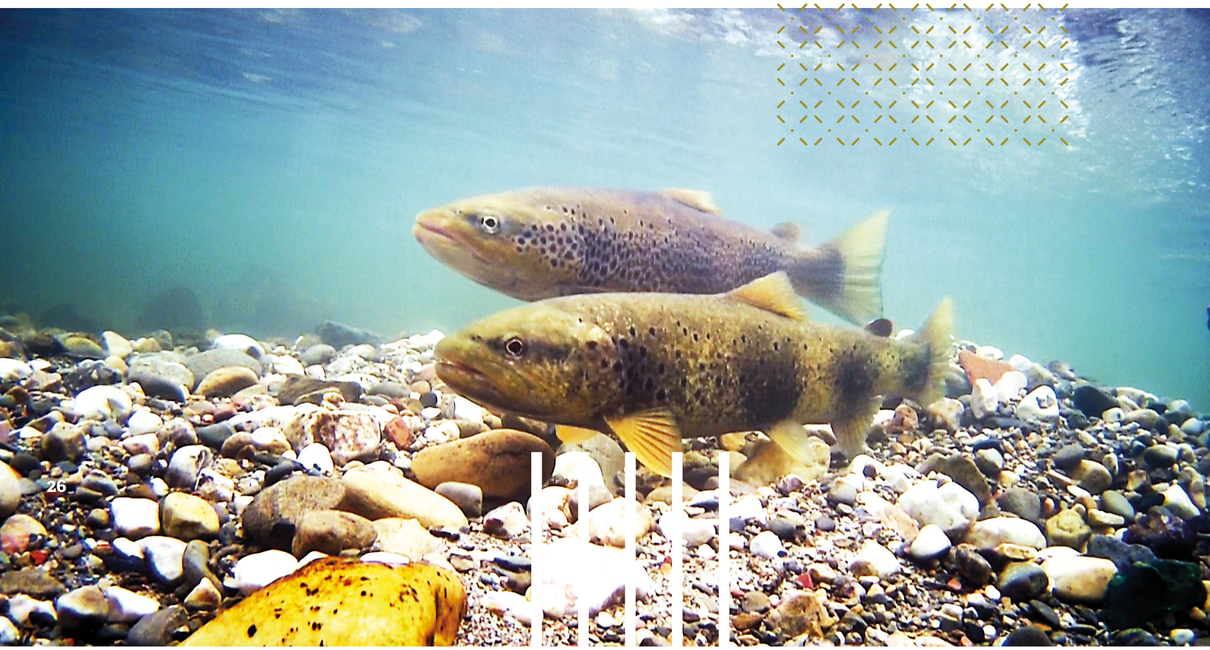


FIGURE 8
Photograph of the ramp in El Pardo (Madrid) showing the two types of modules.

03

FISH PASSABILITY



03

The behaviour of the flow described in this type of ramps, allows predicting the way fish can pass through with a minimal use of energy.

The difference of height between two pools (Δh) does not require to be passed by jumping it, as the water spill is submerged, generating a “submerged gap” (*FIGURE 9*) that allows passing to the upper pool without the need of jumping.

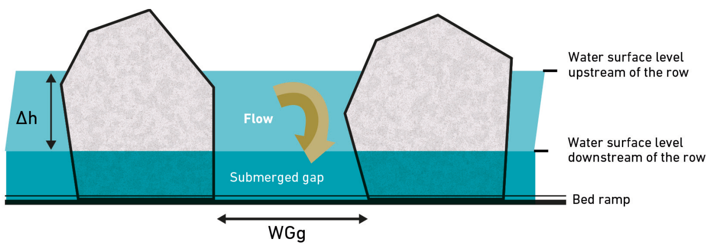


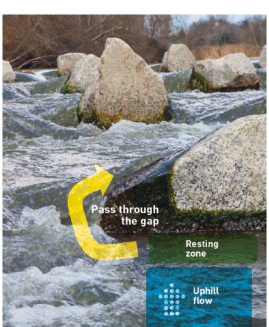
FIGURE 9

Front view of a row showing the submerged gap and the variables: width of the geometric gap (WGg) and difference of the water surface level between two consecutive pools (Δh).

In addition, in order to guarantee the biological functionality of this gap, the value of WGg –as it determines the pass width- must be subject to an environmental condition, i.e. it must guarantee an easy transit for the fish.

In the following sections, recommended values for WGg are presented. “Big” values are recommended in this gap to (i) reduce the risk of blockage –that would alter the correct hydraulic functioning- and (ii) minimise maintenance.

Regarding the transit of fish through the ramp, the hypothesis is the following:



The fish will make use of the zones of uphill flow to progress through the pool (blue zone in *FIGURE 10*).

- Sheltered beside the boulder fish will find a resting area (green zone).
- With a quick “sprint” the fish will pass the gap (yellow zone).
- This process is repeated again up to the next row and so on.

FIGURE 10

Route followed by the fish progressing through uphill flow ramps.

FIGURE 11 shows the results of one monitoring campaign for the ramp in El Pardo (Madrid), where depths and velocities were measured in the flow gaps (gap between two boulders) and in the intermediate points of the modules, both in the uphill flow zone (light blue) and in the downhill flow zone (yellow).

The measurements were taken on the 25th of June 2019, flow on the ramp was 2.15 m^3/s, total flow in the river 2.37 m^3/s and 0.23 m^3/s through the spillway of the weir. The value Δh was approximately 18 cm.

The velocity was estimated with a Global Water FP101, accuracy ±0.03m/s and depth with a deeping-bar, accuracy ±0.5 cm.

Two text boxes are shown for each of the measurement points: left box shows depth in cm, and right box velocity in m/s.

When the text in the right box is highlighted in bright blue means there is an uphill flow or zero velocity.

The values measured on site allow confirm:

- Predominance of zero or negative velocities in the resting zones or uphill flow zones.
- Higher velocities in the fast flow zones but always keeping suitable values for fish swimming capabilities (<2m/s).
- The highest velocities are obviously located in the flow gaps (between two boulders) but always with values under 2 m/s.
- The depths for those three different locations always showed suitable values for fish passability (≥ 20 cm).

In addition, in order to guarantee the biologic effectiveness of the ramp, the turbulence generated by the water drop Δh, must be properly dissipated in the volume of the module:

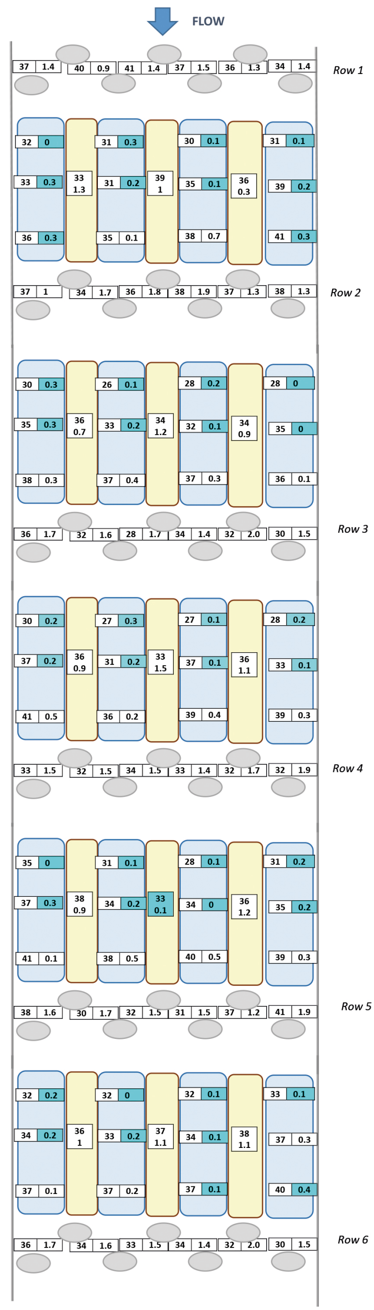


FIGURE 11. Results of the monitoring campaign in El Pardo (Madrid). Depths and velocities for flow 2.5 m^3/s. (Row 4 is located downstream of row 3; diagram is split into two to help visualising in one page).

$$Eq.\ 4 \quad Pdm = \frac{\gamma * Qmodule * \Delta h}{Vol\ m}$$ (Towler, Mulligan and Haro, 2015)

where:

Pdm= power dissipation in the module

γ= specific weight of the water (9810 N/m^3)

$Q\ module$= flow through the module, estimated 2*Qg (Qg= Flow throughout the gap)

Δh = height difference between two consecutive pools

$Vol\ m$= volume of the module

The designer must guarantee that the dissipated energy is below the acceptable threshold for each species. Following sections (see 5.3.3) describe options to modify the dissipated power playing with the angle between the ramp and the horizontal axis (β).

TABLE 1. *References for maximum acceptable dissipated power (Pd).*

VARIABLE	Valbuena Castro *et al.*, 2016		BAW/BfG, 2015	
	Optimum	**Acceptable**	**"Barbel zone"** Gradient (%): 0.3-0.025	**Salmonids** Gradient (%): 10-0.45
Pd(W/m^3)	≤150	150<Pd≤250	**Pd≤200** (in rock ramps)	**Pd≤300** (in rock ramps)

04

VARIABLES

04

4. 1 Introduction _

The set of variables used for the design of this type of ramps is summarised in Annex 1, where the following attributes are defined for each:

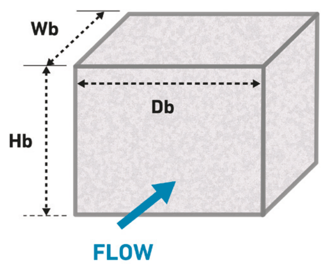
- Symbol
- Units: this Manual makes use, in all cases, of the International System of Units (IS).
- Definition: short description of the variable.
- Character: it refers to the variable type: input datum, independent variable defined by the designer, dependent variable resulting from previous calculations, threshold reference value, or coefficient.
- Calculation phase: it indicates the phase in which the variable is applied (Phase 1: hydraulic dimensioning; Phase 2: geometric dimensioning; Phase 3: dimensioning the power value to be dissipated in each pool; Phase 4: definition of the functional range of flows; Phase 5: assessment of behaviour in non-uniform regime).
- Conditioning: it shows if the variable is subject to any external environmental, hydraulic and/or geometric condition.
- Design recommendations: it indicates if there are available (minimum, maximum or optimum) value references for each variable.

In the following sections, variables are detailed according to their location: in planform, in longitudinal profile between two consecutive rows, in longitudinal profile for the entire ramp. Finally, other variables are defined which apply specifically for particular conditions.

4.2 Planform variables

Boulders, their dimensions and location in the ramp play an essential role, as aforementioned, in the correct functioning of the ramp.

Boulder is defined by its three dimensions: Db, Wb and Hb. Boulder typology can be very different. In upper and middle reaches, it is relatively common to use boulders from the riverbed or the riverbanks. If boulders are non-existent in the vicinity of the project area, or despite existing, their dimensions do not match with the required size/shape, they can be brought from quarries. Less frequent is, up to this date, using prefabricated elements which are later conveniently disposed, filled and anchored, in order to face flow forces. This last alternative is quite recommendable, because: i. makes construction easier; ii. allows committing initial project requirements, something not feasible in all cases when boulders are taken from the river or from a quarry; iii. provides elements which may be closer in size and shape to those naturally present in the ramp environment.



Db:

- Boulder diameter, or transversal dimension to the ramp flow. It means the most representative dimension. It is an independent variable, which must be determined by the designer. Anyway, Db must be high enough to allow pool volume to dissipate power in the required amount.

- Db is the boulder dimension which transversally brakes flow; as such, plain faces and angular edges are preferential.

Wb:

- Boulder width or longitudinal dimension to the ramp flow. Like Db, this is again an independent variable which must, logically, keep proportion with the other two boulder dimensions

Hb:

- Boulder height. Its relevance will be explained in the following chapter, where the longitudinal profile of the ramp is detailed. Nonetheless, it is explained in this section to allow a more general understanding of the boulder dimensioning. It is a dependent variable, defined during the geometric design, once the hydraulic dimensioning is entirely fulfilled.

Eq. 5 $$Hb = h_1 + y + h_{weir}$$

Where:

- h_1= depth downstream of the boulder
- y= height difference between the height of the weir crest ($Hcweir$) and $HWs(Q_{RMIN})$, being $HWs(Q_{RMIN})$ the height of the water surface upstream of the boulder of the top row for Q_{RMIN} (minimum flow for which the ramp must be functional) (see section 5.2).
- h_{weir}= maximum height of water surface over the weird which maintain ramp functionality without boulders becoming drowned. It identifies height difference between the upper end of boulders in the top row and $Hcweir$.

- A **recommendation** is given for the minimum value of Hb (Hb_{min}):

Eq. 6 $$Hb \geq Hb_{min} = h_1 + y + 0.15$$

  • This minimum value guarantees, as later shown, the hydraulic functioning of the ramp for a wide range of flow values with no drowning of boulders

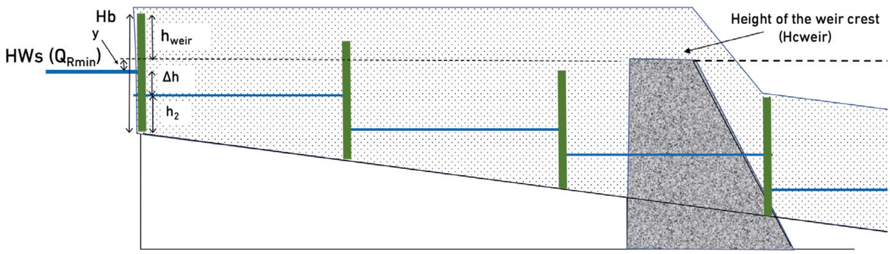


FIGURE 12
Longitudinal profile of ramp, with indication of variables which define boulder height (Hb)

Once the boulder has been dimensioned, the following step in the ramp design would be the characterisation of the planform pattern of boulders: angle between two consecutive boulders (α), width of geometric gap (WGg), transversal projection of geometric gap to the flow (b), width of hydraulic or effective gap (WHg), module width (Wm, Wmm, Wem), number of modules (Nm), number of rows ($Nrow$), pool length (Lp), number of pools (Np) and ramp length (L_R).

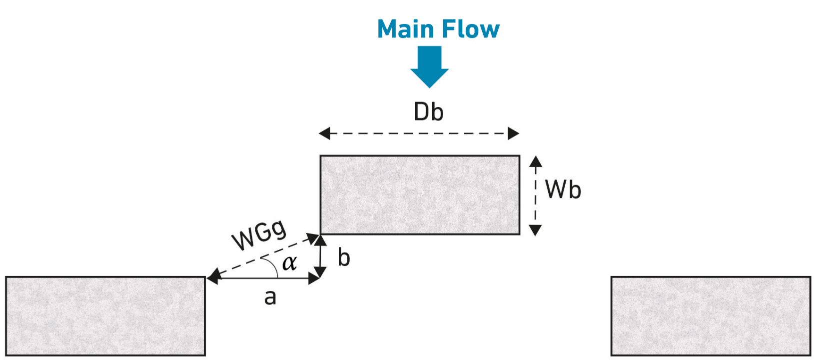


FIGURE 13
Relative location of boulders, in planform, with indication of angle α and projections a and b of the geometric gap width (WGg)

Angle α, is an independent variable, to be determined by the designer, but conditioned to the interval of uphill flows: $30° \leq \alpha \leq 45°$ (FIGURE 13).

The orthogonal line to the axis of the flow gap creates an angle α with the flow direction. This fact suggests the necessity of including, in equation 2, an additional coefficient which considers the effect of this angularity on flows. It was not finally incorporated since, following the recommendations of Ven te Chow, the angle coefficient has a value close to 1, except when $\alpha \geq 45°$, and $WGg > 0.6$m.

Distance between boulders, or width of geometric gap (WGg), with its two projections: transversal (a) and longitudinal (b) to flow:

Eq. 7 $$a = WGg * cos\alpha$$

Eq. 8 $$b = WGg * sen\alpha$$

WGg is a variable which depends on hydraulic variables previously calculated, such as WHg (width of hydraulic gap) and Δh (height difference of water surface between two consecutive pools). Relation between both was formerly mentioned, when the hydraulic functioning of the ramp was described (Eq. 1):

$$WHg = WGg\text{-}Cc * \Delta h$$

where Cc is the contraction coefficient of the flow sheet – for which Marbello Pérez (2005) suggests a value of 0.2–.

At the time, since WGg determines the width of the flow gap, an environmental condition –fish passing- should be addressed. Some authors (Valbuena Castro *et al.*, 2016; BAW/BfG, 2015) make the following recommendations:

TABLE 2. References for dimensioning the width of the geometric gap (WGg).

VARIABLE	Valbuena Castro *et al.*, 2016		BAW/BfG, 2015	
	Optimum	Acceptable	Barbel zone Gradient (%): 0.3-0.025	Salmonids Gradient (%): 10-0.45
WGg (m)	≥0.2	0.1≤ WGg< 0.2	**WGg ≥3·Fish height** (for Barbus, 0.25m)	

On the other hand, high values of WGg are recommendable, so as to avoid obstruction by litter, and make maintenance works easier.

Once boulder dimensions are defined, module width (Wm) varies according with the type of module (edge or central):

Eq. 9 $$Wem = 2.5 * \mathrm{Db} + 2\mathrm{a}$$

Eq. 10 $$Wmm = 2 * (Db + \mathrm{a})$$

Where:

Wem= width of edge module

Wmm= width of mid-module

Db= boulder diameter or boulder dimension in the transversal direction to flow

a= gap in the transversal direction to flow

If modules are wide enough, water jump power will be adequately dissipated. As reflected in the previous equations, this width largely depends on the boulder diameter. Thus, correct selection of Db becomes essential to ensure a correct behaviour of the ramp.

The number of modules (Nm) is also an independent variable defined by the ramp designer. Obviously, the minimum number of modules is 1, but it would be recommendable to include 2 or more modules.

Selection of the number of modules should consider that, remaining all other parameters unaltered (boulder dimensions, α, flow, other hydraulic variables), an increase in the number of modules means:

- An increment of the ramp width
- A decrement of flow crossing the flow gap. This implies a reduction of the hydraulic gap width (WGg), and thus creating more difficulties for fish and debris passing.

Once the number of modules is defined, the ramp width is immediately calculated *(TABLE 3)*.

TABLE 3. Definition of ramp width according to the number of modules

NUMBER OF MODULES	TYPE OF MODULE		RAMP WIDTH (M)
	Edge	Centre	
1	Unique		3*Db+2a
2	2		2*(2.5*Db+2a)
3	2	1	2*(2.5*Db+2a) + 2*(Db + a)
4	2	2	2*(2.5*Db+2a) + 2*(2*(Db + a))
...			
n	2	n-2	2*(2.5*Db+2a) +(n-2)*2*(2*(Db +a))

The number of rows (*Nrow*) is a dependent variable, calculated by means of equation 11:

$$\text{Eq. 11} \quad Nrow = integer\left(\frac{H_T - h_1}{\Delta h}\right) + 1$$

where:

H_T=total height to surpass (see section 5.3.2)

h_1=depth upstream of the boulder (see *FIGURE 12*)

Δh=difference of water surface level between two consecutive pools (see *FIGURE 12*)

Rows are disposed that way along the ramp, with a distance between them called Lp or Pool length. The value of Lp is calculated as:

$$\text{Eq. 12} \quad Lp = {}^{\Delta h}/_{tg\beta}$$

where:

Δh= difference of water surface level between two consecutive pools

β (degrees), angle between the ramp plane and the horizontal plane (independent variable)

Number of pools (Np) is defined as:

$$\text{Eq. 13} \quad Np = Nrow - 1$$

Finally, ramp length (L_R) is a dependent variable:

$$Eq.\ 14\quad L_R = {H_R}/{tg\beta}$$

where:

H_R= ramp height (see section 5.3.2)

β (degrees)= angle between the ramp plane and the horizontal plane (independent variable)

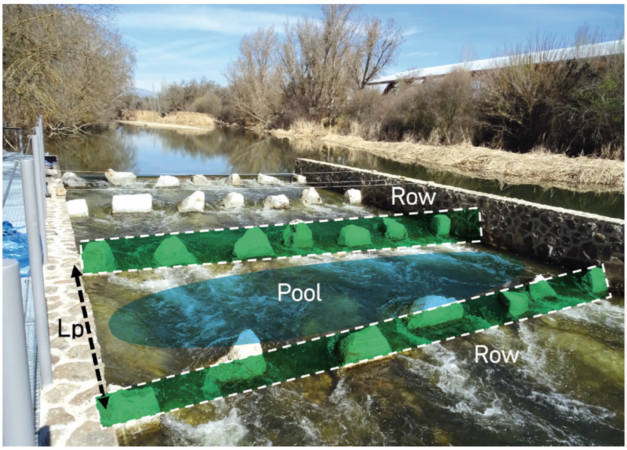


FIGURE 14
El Pardo ramp, where variables Nrow and Lp can be seen

4. 3 Variables of longitudinal profile between two rows _

The six variables shown in *FIGURE 15* and classified in *TABLE 4* can be differentiated:

TABLE 4. Variables considered in the longitudinal profile of the ramp, between rows, with indication of its character

VARIABLES OF THE LONGITUDINAL PROFILE	
Independent	**Dependent**
• Depth downstream of the boulder (h_2) • Height difference of water surfaces between two consecutive pools (Δh) • Angle between ramp plane and horizontal plane (β)	• Depth upstream of the boulder (h_1) • Boulder height (Hb) • Pool length (Lp)

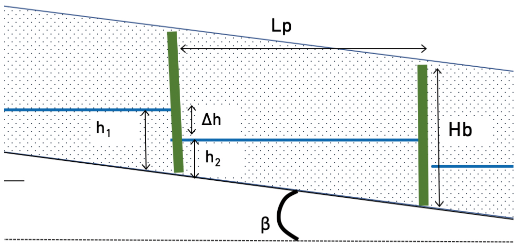


FIGURE 15
Longitudinal profile of ramp between two rows, and relevant variables associated

Depth downstream of the boulder (h_2), height difference of water surfaces between two consecutive pools (Δh) and the angle between ramp plane and horizontal plane (β) are independent variables which must be defined by the designer in the first phase of the hydraulic dimensioning.

Tangent of angle β determines the ramp slope.

Nonetheless, those variables are subject to environmental and/or geometric constraints, and some recommendations may be done before they are calculated (*TABLES 5, 6 Y 7*).

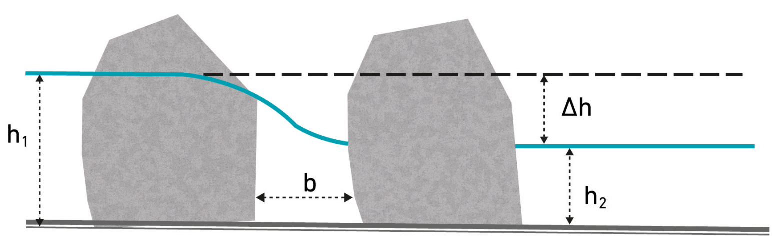


FIGURE 16
Longitudinal section between two consecutive boulders in a row, with indication of variables associated

TABLE 5. Biological significance and environmental constraints of Δh and h_2

VARIABLE	BIOLOGICAL EFFECT	RECOMMENDED VALUE
Δh(m)	It determines velocity in the flow gap: $v=(2g\Delta h)^{1/2}$ It must be lower than sprint velocity of species and stage	Optimum ≤0.2 Acceptable 0.20-0.35
h_2(m)	It determines minimum depth in the ramp It should allow a comfortable movement of fishes through the gap, and reduce exposure to predators	Optimum ≥0.2 Acceptable 0.10-0.20

TABLE 6. Significance and geometric conditioning of Δh and tgβ

VARIABLE	BIOLOGICAL EFFECT	RECOMMENDED VALUE
Δh(m)	It determines the number of pools per meter ($Np=H_R/\Delta h$), where H_R is the height of the ramp. Low scores imply a high number of pools, for a similar height difference. For instance, for Δh=0.1m, 10 pools would be needed for each meter of height difference	≥0.1
tgβ(m)	It determines pools length (Lp) and, thus, ramp length (L_R). Pools length affects their volume (Vol p) and power to be dissipated (Pd): tgβ⬇ ⇨ Lp⬆ ⇨ Vol p⬆⇨ Pd⬇ Lp⬆ ⇨ L_R⬆ For instance, tgβ=0.03 means that the ramp must be 33.3 meters long for each meter of height difference	For rock ramps, the most frequent threshold value is 0.05 (5%), and the variable which more strongly affects that value is normally Pd (due to the small volume of ponds).

TABLE 7. Other recommendations

VARIABLE	Valbuena Castro *et al.*, 2016		BAW/BfG, 2015	
	Optimum	**Acceptable**	**Barbel zone** Gradient (%): 0.3-0.025	**Salmonids** Gradient (%): 10-0.45
Δh(m)	≤0.2	0.2<Δh≤0.35 (v=2.6m/s)	≤0.17 (v=1.8m/s)	≤0.25 (v=2.2m/s)
h_2 (m)	≥0.2	0.1≤ h_2<0.2	h_2≥2*dorsal-ventral length (for *Barbus*, 0.26m)	

Associated variables are: depth upstream of the boulder (h_1), boulder height (Hb) and pool length (Lp) –these last two variables have been described afore- which are calculated as shown in *equation 15*:

Eq. 15

$$h_1 = h_2 + \Delta h$$

$$Hb = h_1 + y + h_{weir}$$

$$Lp = \Delta h / tg\beta$$

where:

h_1= depth upstream of the boulder

h_2= depth downstream of the boulder

Δh= height difference of water surface between two consecutive pools

Hb= boulder height

y= height difference between weir crest ($Hcweir$) and water surface upstream of the boulder of the top row for Q_{RMIN}

h_{weir} =maximum height of water surface on the weir which is compatible with the ramp without drowning of boulders: difference between the upper level of boulders in the top row and height of weir crest

β (degrees)= angle between ramp plane and horizontal plane

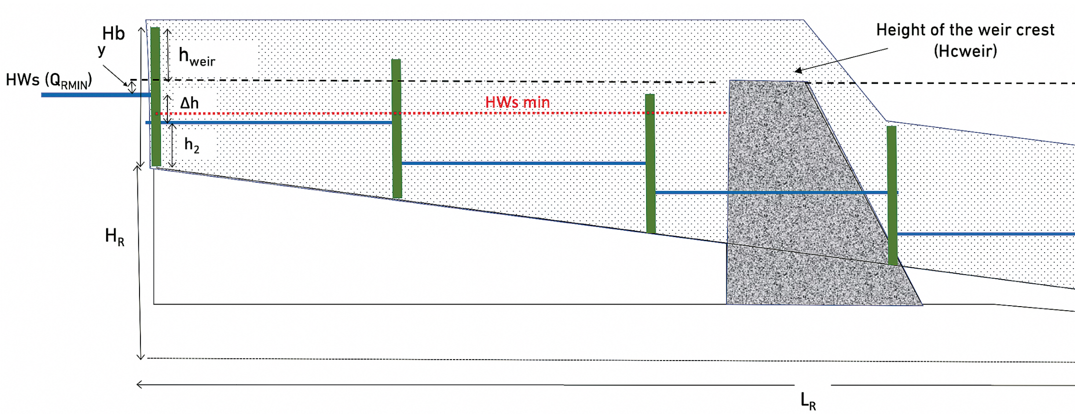
The **recommendation** for the minimum value of Hb (Hb_{min}) is reminded here:

$$Hb \geq Hb_{min} = h_1 + y + 0.15$$

4. 4 Variables of longitudinal profile in the entire ramp _

FIGURE 17 presents together the variables of the longitudinal profile between rows –already defined- and those linked to the entire ramp. In *TABLA 8* all these variables are shown and classified, according to their features.

Figures included in this manual, aimed at the identification of all relevant variables, present the ramp partially relocated from the weir. This design must not be taken as absolute refe-



rence. In each situation, the user must adjust the design to the particular conditions of the weir and the surrounding areas. And allowing that fish entrance is as close as possible to the toe of the weir, in order to optimise the call effect of the attraction flow.

TABLE 8. Variables considered in the longitudinal profile of the entire ramp, with indication of their character.

VARIABLES OF THE LONGITUDINAL PROFILE		
Preliminary	**Independent**	**Dependent**
• Height of the weir crest (*Hcweir*) • Bed height at the ramp end (H_0) • Minimum depth upstream of the weir, which is compatible with the existing abstraction ($y_{min\ diversion}$)	• Maximum height of water surface on the weir, which is compatible with the ramp functioning, without drowning of boulders (y_{weir}) • Total height to pass (H_T)	• Height of the ramp (H_R) • Height difference between the weir crest and the water surface upstream of the boulder of the top row for Q_{RMIN} (y)

A preliminary topographic survey is required for the design, the following items can be listed: height of weir crest (*Hcweir*) –or, if the case, spillway height-, and bed height at the ramp end (H_0). Should the weir is associated to any water abstraction, it would be necessary to know –in order to guarantee its adequate functioning once the ramp is constructed- the value of the minimum depth upstream of the weir which is compatible with the existing abstraction ($y_{min\ diversion}$). Or, alternatively, *HWs min* (water surface in the upper wall of the weir which guarantees the requisites of the abstraction).

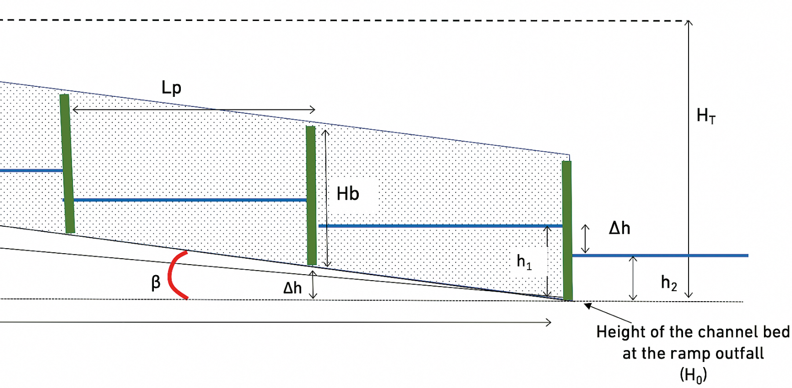


FIGURE 17
Longitudinal profile of the ramp, with indication of the most representative variables

Regarding to the independent variables, one of the most relevant is the maximum height of water surface over the weir which allows the ramp functioning with no drowning of the boulders (h_{weir}). It may be calculated as the height difference between the higher point of boulders in the top row and the height of weir crest ($Hcweir$).

h_{weir} influences boulder height (Hb) and, thus, the maximum depth upstream of the boulder ($h_{1\ max}$) so that the ramp may work with no drowning. It is recommended that the ramp designer considers values of h_{weir}≥0.15m.

Total height (H_T) may be calculated by means of the initial data:

Eq. 16 $$H_T = Hcweir - H_0$$

For calculating the dependent variables ramp height and height difference between the depth upstream of the ramp and the height of the weir crest, the following expressions are used:

Eq. 17 $$H_R = (Nrow - 1) * \Delta h$$

Eq. 18 $$y = H_T - H_R - h_1$$

where:

H_R= ramp height (height difference in the ramp bed between inlet and outlet sections).

$Nrow$= number of rows in the ramp.

Δh= height difference in the water surface between two consecutive pools.

y= height difference between weir crest ($Hcweir$) and water surface level upstream of the boulder in the top row for Q_{RMIN}

H_T= total height to pass

h_1= depth upstream of the boulder

It is advisable that y≥0.1m. The higher the value of y, the wider the range of flows which strictly run over the ramp (see section 5.2). The lower range of y is defined by the minimum depth of water in the upstream side of the weir ($HWs\ min$; see *FIGURE 17*). This depth is associated to the water outtake for which the weir was built. If necessary, the value of y may be adjusted modifying the values of h_2 and Δh.

4.5 Other design variables _

Adequation of the ramp functionality for low flows may be supported by constructing a bottom crest or step (Pena *et al.*, 2018).

FIGURE 18 and *FIGURE 19* show, in each case, the planform and longitudinal profile of the row in a module without and with step. In the last case, with height (p) over the ramp bed in the flow gap.

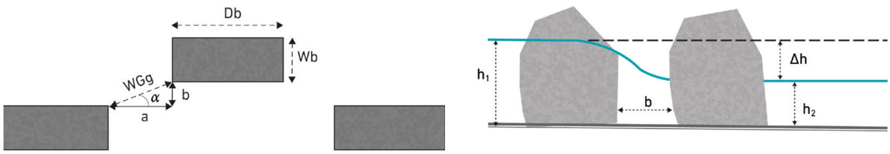


FIGURE 18
Row for a module without step: planform (left) and longitudinal profile (right)

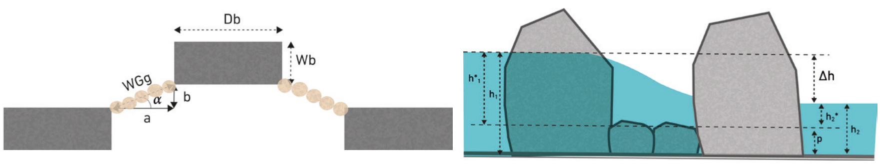


FIGURE 19
Row for a module with step: planform (left) and longitudinal profile (right)

For a given gap, the step allows

- for a given flow, higher depths in the pool
- for a given flow, increasing the pool volume ($Vol\,p$) and thus, the ability to dissipate energy
- maintenance of a given depth in the pool with lower flow
- maintenance of a given Δh with lower flow

The step becomes particularly useful when the minimum flow in the ramp is very low.

A major disadvantage is that the maximum flow which will be able to run with no drowning of boulders ($h_1=Hb$) will be lower than that which could flow without the step.

The flow-depth equation (gap flow) used in the design (Fuentes-Pérez *et al.*, 2016) includes the step height (p), in order to strictly consider the effective hydraulic heights: h_1* and h_2*.

Eq. 19 $$h_1^* = h_1 - p$$

Eq. 20 $$h_2^* = h_2 - p$$

Eq. 21 $$Qg(p) = \frac{2}{3} * \sqrt{2g} * Cd_{gap} * WHg * (h_1 - p)^{1.5}$$

Eq. 22 $$Cd_{gap}(p) = \beta_0 * \left[1 - \left(\frac{h_2 - p}{h_1 - p}\right)^{1.5}\right]^{\beta_1}$$

where:

h_1*= depth, measured over the step, upstream of the boulder
h_1= depth upstream of the boulder
h_2*= depth, measured over the step, downstream of the boulder
h_2= depth downstream of the boulder
p= step height
$Qg(p)$= flow running through the flow gap, with step
$Cd_{gap}(p)$= flow-depth coefficient in the flow gap, with step
WHg= width of hydraulic gap
β_0= coefficient of calculation (0.812)
β_1= coefficient of calculation (0.335)

TABLE 9. Hydraulic relevance and recommendations for step height (p)

VARIABLE	HYDRAULIC EFFECT	RECOMMENDED VALUE
p	• The step allows an increment of h_2, h_1 and pool volume ($Vol\ p$), for a given flow and Δh • It conditions the value of the measured gap flow (h_2*)	**h_2*= h_2-p≥ MAX (0.1m; dorsal-ventral length)**

05

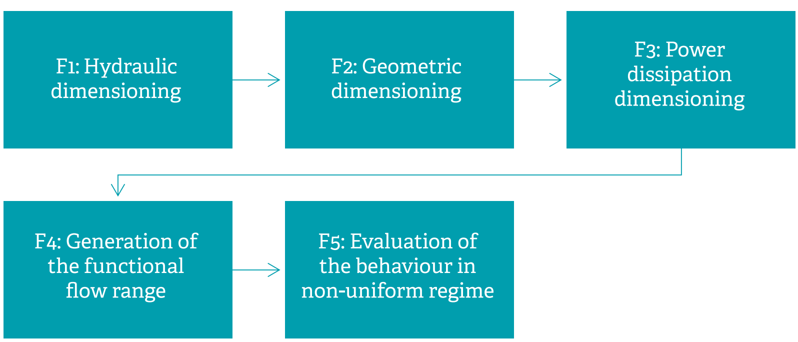
DIMENSIONING

05

5.1 Introduction _

This chapter presents an initial section (5.2) where all different flow scenarios are described. Each of them requires different considerations (geometric and hydraulic) that are detailed in subsequent sections.

Section 5.3 details the protocol to calculate the different elements, this protocol consist of five phases that must be followed consecutively.



For each of the phases the following elements are indicated: required inputs, calculation sequence and recommendations to select the design values. In order to help understanding this protocol, the different phases are resolved through a case study.

CASE STUDY

- Environmental flow: *0.25* m^3/s
- Maximum velocity: *2.2 m/s* ⇨ $\Delta h \leq 0.25m$
- Minimum depth: *0.2m* ⇨ $h_2 \geq 0.2m$
- Minimum pass width: *0.2m* ⇨ $WGg \geq 0.2m$
- Maximum dissipated energy: $200W/m^3$ ⇨ $Pd \leq 200W/m^3$
- Length of the weir *(Lweir): 30m*
- Crest width *(Wcweir): 0.5m*
- Height of the channel bed at the ramp outfall (H_0): *1000.8 m.a.s.l.*
- Height of the weir crest *(Hcweir): 1002.9 m.a.s.l.*
- Height difference between the highest point of the boulders in the top row and *Hcweir* (h_{weir}): *0.15m*

5.2 Flow scenarios_

Before describing the dimensioning of the ramp, it is convenient to present the different scenarios. These scenarios correspond to the different flow ranges that determine specific flow characteristics for the system ramp-weir.

Firstly, the relevant flow to consider is the river flow upstream of the weir [*Q river*] once subtracted the diverted flow to the lade [*Q diverted*]. This flow will be the one that will be considered through the process of design and dimensioning, [Available flow: Qa=*Q river – Q diverted*].

From the different available flows, the most relevant for the dimensioning is the minimum flow for which the ramp must be functional [Q_{RMIN}]. Once fixed the period for which the ramp must be passable, normally the pre-reproductive period of the target fish species, the user must set Q_{RMIN} considering:

- Environmental flows.
- The regime for Qa corresponding for the period of functionality of the ramp. When enough years within the flow series for Qa are available, Q_{RMIN} can be estimated as follows:
    - Generate the flow duration curve of the daily Qa for the period of functionality of the ramp.
    - Extract Q90 (flow equal or greater than 90% of the days of the period).
    - This flow is a good initial reference to fix Q_{RMIN}.

It is important to know that Q_{RMIN} is the reference for the basic dimensioning of the ramp, for this reason the user must be careful determining this value.

In relation to the weir functioning, the user must consider the minimum height of the water surface on the bed upstream of the weir to guarantee that the requirements of the flow diversion are fulfilled. This water surface level [*HWs min*] has a relevant role in the dimensioning of the ramp.

FIGURE 20 shows the minimum height of the water surface upstream of the ramp when $Qa = Q_{RMIN}$, [$HWs\ (Q_{RMIN})$]. It can be seen:

- All the flow goes through the ramp, as $HWs\ (Q_{RMIN})$, and therefore the height of the water surface on the weir bed, is less than the height of the weir crest (*Hcweir*).
- $HWs\ (Q_{RMIN})$ must be higher than *HWs min* to guarantee the correct functioning of the weir.

The difference between *Hcweir* and $HWs\ (Q_{RMIN})$ [$y = Hcweir - HWs\ (Q_{RMIN})$] determines the flow range for which all the available flow will discharge through the ramp and, therefore, there will not be weir spill. This value y plays an important role in the dimensioning.

SCENARIO 1: All Qa discharges through the ramp. No weir spill.

This scenario occurs for a range of flows that goes from Q_{RMIN} up to the flow over which the weir starts spilling [$Q_R(y=0)$]. *FIGURE 20* corresponds to Q_{RMIN} and *FIGURE 21* to $Q_R(y=0)$, flow for which $HWs\ (Q_R(y=0)) = Hcweir$.

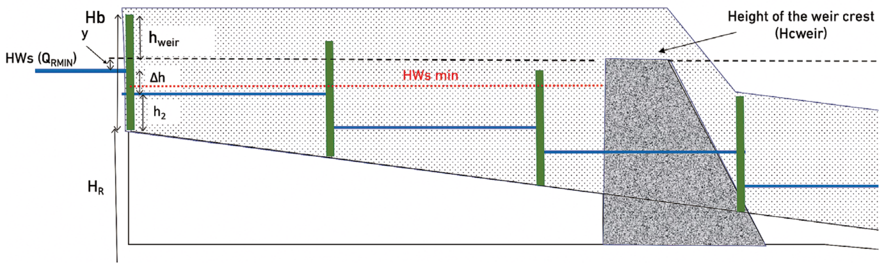


FIGURE 20
Longitudinal profile of the water surface for $Qa=Q_{RMIN}$.

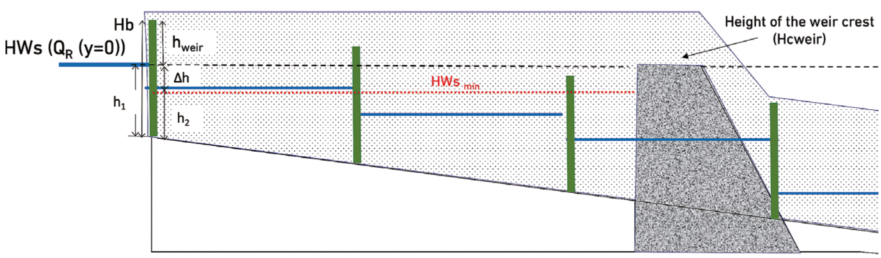


FIGURE 21
Longitudinal profile of the water surface for $Qa=Q_R\ (y=0)$.

In terms of the functionality of the ramp, it is important to bear in mind that this scenario generates the maximum call effect of the attraction flow because all Qa discharges through the ramp. Consequently, it is recommended that the range of Qa for this scenario [$Q_{RMIN} \leq Qa \leq Q_R(y=0)$] is as large as possible.

This range depends on the value of y, where

$$\text{Eq. 23} \quad y = \begin{cases} = Hcweir - HWs(Q_{RMIN}) \\ \text{with } HWs\,(Q_{RMIN}) \geq HWs\ min \end{cases}$$

As seen in section 4.4, $y=H_T - H_R - h_1$. Taking into account that H_T=$Hcweir$-H_0 is a given value, that $H_R=(Nrow-1)*\Delta h$ and that $h_1=h_2+\Delta h$, is obvious that $y=f(h_2; \Delta h)$. Therefore, the flow range for this scenario [$Q_{RMIN} \leq Qa \leq Q_R(y=0)$] will be function of h_2 and Δh.

With a given Q_{RMIN}, in order for the flow range to increase, y must increase. As y is a decreasing function of h_2 and Δh, decreasing the values of these variables increases the flow ranges. However, this reduction has some important limits: on one side, h_2 determines the minimum depth for the fish progressing the ramp, thus this value cannot be fixed under the corresponding threshold of the target species; on the other side, a reduction of Δh implies an increase of the number or rows and with it, the number of pools that, for a given length of ramp, implies the decrease of the pool length and therefore, it can increase the dissipated power setting this value over the maximum acceptable value for the target species.

In order to the user to select the most appropriate value of y for each case, the dimensioning protocol contemplates its specific calculation.

It is important for remark that for this scenario to happen, the visual height of the boulder [Hb] fulfils: $Hb \geq h_2(Q_{RMIN})+\Delta h+y$.

If $Hb= h_2(Q_{RMIN})+\Delta h+y$, then for $Qa>Q_R(y=0)$, the boulders are drowned and the uphill flows typical of this design are jeopardised. For this reason, to avoid the uphill flows disappear with $Qa>Q_R(y=0)$, it is recommended that $Hb=h_2(Q_{RMIN})+\Delta h+y+h_{weir}$, con $h_{weir}\geq 0.15$m.

SCENARIO 2: There is weir spill and the flow through the ramp does not drown the boulders.

This scenario starts when $Qa>Q_R(y=0)$. To work out the top end [Qa_{MAX}] of Qa is needed to present two new flows Q_{RMAX} y Q_{weir} (h_{weir}) whose sum will determine the value of Qa_{MAX}.

Considering the situation for which the water surface reaches the top edge of the boulder (*FIGURE 22*), this is, when $h_1=Hb$.

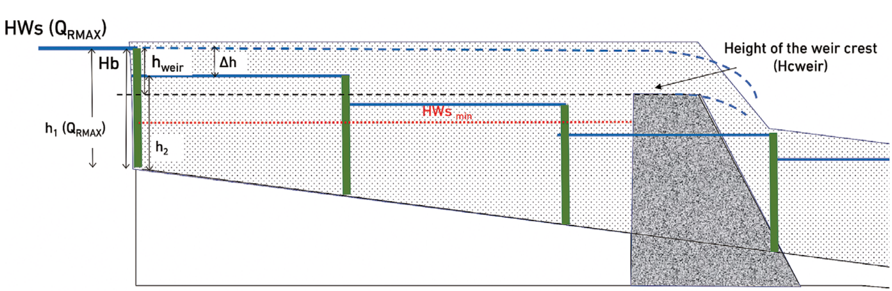


FIGURE 22

Longitudinal profile of the water surface when Qa= Q_{RMAX}+ Q_{weir} (h_{weir})

For that hypothesis:

- The flow circulating on the ramp Q_{RMAX} is the maximum flow before the rows of boulders get drowned [$h_1(Q_{RMAX})=Hb$] and the uphill flows provided by this type of devices are jeopardised[2].
- The flow spilling over the weir crest, determined by its discharge equation [$Q_{weir}\ (y_{weir})$], will be $Q_{weir}\ (h_{weir})$, due to when Q_{RMAX} circulates on the ramp, then $y_{weir}=h_{weir}$
- Weir discharge equation[3]:

$$\text{Eq. 24} \quad Q_{weir}\left(y_{weir}\right) = 1.7 * Cd_{weir} * y_{weir}^{1.5} * Lweir$$

$$\text{Eq. 25} \quad Cd_{weir} = \begin{cases} 0.75 + \dfrac{0.1}{Wcweir/y_{weir}} & \text{if } \dfrac{Wcweir}{y_{weir}} > 3 \\ 0.7 + \dfrac{0.185}{Wcweir/y_{weir}} & \text{if } \dfrac{Wcweir}{y_{weir}} \leq 3 \end{cases}$$

Where:

y_{weir} = height difference between the water surface upstream of the weir and the weir crest.

$Wcweir$= crest width of the weir

$Lweir$= length of the weir

Therefore, the maximum value of Qa for which there is not boulder drowning is:

$$\text{Eq. 26} \quad Qa_{MAX} = Q_{RMAX} + Q_{weir}\left(h_{weir}\right)$$

Unlike SCENARIO 1, for the flow range of this scenario [$Q_{RMAX} \leq Qa \leq Qa_{MAX}$], the flow circulates both through the ramp and over the weir. The call effect of the attraction flow, absolute in the previous scenario, is decreased in this scenario, thus the user must consider the convenience of adding a notch on the weir, close to the toe of the ramp, to improve the call effect of the attraction flow.

In order to ease the fish passage in the ramp, it is recommended that the access is placed as close as possible to the toe of the weir. The ramp shown in this handbook´s photographs was designed with this criterion.

---

[2] *It is possible that over that flow the ramp is still functional for fish pass, but its hydraulic behaviour must be assessed, for that new situation, with the given equations. This scenario is not assessed in this handbook.*

[3] *It is proposed the use of the equation that, using as a reference the Bazin formulation for a rectangular spillway (thin-walled) without lateral contraction, applies a reduction coefficient of Wcweir/y. The users can consider any other equation that, to their knowledge, better suits the characteristics of the spill.*

In the figures supporting the description of the variables, the ramp has been set back from the weir, but only partially. This design must not be considered as a reference. The user must adjust the design based on the specific constraints of the weir and its surroundings in a case-by-case basis, always placing the fish access as close as possible to the toe of the weir.

SCENARIO 3: The flow circulating through the ramp drowns the boulders.

This scenario appears when $Qa > Qa_{MAX}$. As said before, under these circumstances the secondary flows generating the uphill flow decrease or disappear. The fact that these uphill flows are not present, does not imply that the ramp stops being functional as this functionality is linked to satisfying the passage conditions for the target species based on the thresholds set for maximum velocity, minimum depth, dissipated power, etc.

However, the hydraulic assessment of these variables requires hypothesis and equations that go beyond this handbook. This scenario is not analysed in this handbook.

TABLE 10 shows a summary of the characteristics of the different scenarios.

TABLE 10. Flow range for each of the scenarios considered for the dimensioning of the ramp.

SCENARIO	LOCATION OF THE DISCHARGE	FLOW RANGE Qa	COMMENTS
1	Only through ramp	$Q_{RMIN} \le Qa \le Q_R(y=0)$	The call effect of the flow is maximum. The flow range increases with the value of y, which is function of h_2 and Δh. Also, the value of y influences the visual height of the boulder.
2	Ramp and weir No drowning of boulders	$\underset{RAMP}{Q_R(y=0)} < Qa \le \overbrace{\underset{RAMP}{Q_{RMAX}} + \underset{WEIR}{Q_{weir}(h_{weir})}}^{Qa_{MAX}}$	The attraction flow is impacted by the weir spill. It is convenient to consider creating a notch next to the ramp to increase the attraction flow. The flow range increases with the value of h_{weir}. The value of h_{weir} also determines the visual height of the boulder.
3	Ramp and weir With drowning of boulders	$Qa_{MAX} = Q_{RMAX} + Q_{weir}(h_{weir}) < Qa$	It is not possible to guarantee the uphill flow, although the ramp can be functional for the fish to progress swimming uphill. This scenario is not assessed in this handbook.

5.3 Calculation protocol _

Before introducing the protocol is convenient to make the following considerations:

- The dimensioning is done in uniform regime, assuming that for a given flow – constant-, the depths upstream and downstream of the boulders [h_1; h_2] are the same and constant in all rows (*FIGURE 21*).
- Later, once the ramp has been dimensioned, particular cases are assessed under non-uniform regime.
- In order for the dimensioning to be more versatile, the procedure here presented contemplates a wide acceptable range[4], both for minimum depths and maximum velocities in the gaps:

  - The minimum depths are shown just downstream of each row -h_2-, considering values between 0.1m and 0.4m
  - For the estimation of max velocities:

$$\text{Eq. 27} \quad V_{MAX} = \sqrt{2g\Delta h}$$

  Considering values of Δh between 0.1m and 0.35m, corresponding, respectively, to V_{MAX} =1.4m/s and 2.6m/s

This protocol has 5 phases:

Phase 1	**Hydraulic dimensioning:** given Q_{RMIN}, WGg_{min}, and assuming a number of modules (Nm), pairs of values (h_2; Δh) [$0.1 \leq h_2 \leq 0.4$; $0.1 \leq \Delta h \leq 0.35$] are generated in order for the corresponding WGg[5] is $\geq WGg_{min}$
Phase 2	**Geometric dimensioning:** using the previous pairs of values (h_2;Δh), the following values are generated: y (height difference between the weir crest and the water surface upstream of the boulder of the top row for Q_{RMIN}), H_R (height difference, on the ramp bed, between inlet and outfall), number rows -$Nrow$-, number of pools -Np- and visible height of the boulder -Hb-.

[4] *The range of values h_2 and Δh [$0.1 \leq h_2 \leq 0.4$; $0.1 \leq \Delta h \leq 0.35$] considered here is wider than the one set as recommended values. This offers to the user more options to deal with particular fish species or circumstances. .*

[5] *It is recommended to assign large values of WGg to reduce blockage and maintenance tasks. Values under 0.1m should not be acceptable, suggesting WGg ≥0.2m.*

Phase 3	**Dimensioning to control the dissipated power in each pool:** considering the range of slopes set by the user, for each of them, the values of pool length and ramp length will be generated, and also the pool volume and the dissipated power (checking if the dissipated power fulfils the threshold of the target species).
Phase 4	**Generation of the functional flow range:** with the dimensions set in the previous phases, flows $Q(y=0)$, Q_{RMAX} y Q_{weir} (h_{weir}) are determined, in order to determine the functional range of flows: - Ramp discharge only: $Q_{RMIN} \leq Qa \leq Q_R(y=0)$ - Ramp and weir discharge: $Q_R(y=0) < Qa \leq Qa_{MAX}$
Phase 5	**Assessment of behaviour in non-uniform regime:** The non-uniform case that can jeopardise the ramp functionality is when Q_{RMAX} is circulating on the ramp. For this case, the velocities in the pass gap will be assessed to verify if the values are under the threshold of the target species.

In the three first phases of the dimensioning, the considered flow is Q_{RMIN}. In phase 4 $Q_R(y=0)$, Q_{RMAX} and Q_{weir} (h_{weir}), and for the fifth phase Q_{RMAX}.

If the user cannot find values satisfying the requirements in one phase/s, he will have to come back to previous phase/s to re-adjust some variables.

In order to simplify the application of this protocol, a piece of software has been developed that allows solving this sequence easily.

Below, the details of the calculations in each phase are presented.

5.3.1 PHASE 1. HYDRAULIC DIMENSIONING.

What is needed?

Q_{RMIN}, WGg_{min}, and to assume the number of modules *(Nm)*

What is generated?

Pairs of values $(h_2;\ \Delta h)$ $[0.1 \leq h_2 \leq 0.4;\ 0.1 \leq \Delta h \leq 0.35]$ for which the corresponding WGg is $\geq WGg_{min}$

What are these values for?

Alternatives used in phase 2 to select the most convenient one for the values that determine the geometric caracteristics of the ramp.

5.3.1.a Calculation sequence:

a.1.- Given Q_{RMIN} –set based on the hydrologic analysis of flows -, and assuming the number of modules -Nm-, the following values are calculated::

- The number of gaps in each row -2^*Nm-
- The flow circulating through each gap: $Qg = \frac{Q_{RMIN}}{2*Nm}$

a.2.- For each combination (h_2; Δh) the following values are calculated:

- h_1 [$h_1 = h_2 + \Delta h$] and the gap discharge coefficient Cd_{gap}:

$$Cd_{gap}\,(h_2; \Delta h) = \ \beta_0 * \left[1 - \left(\frac{h_2}{h_1}\right)^{1.5}\right]^{\beta_1} \quad \beta_0 = 0.812;\ \ \beta_1 = 0.335$$

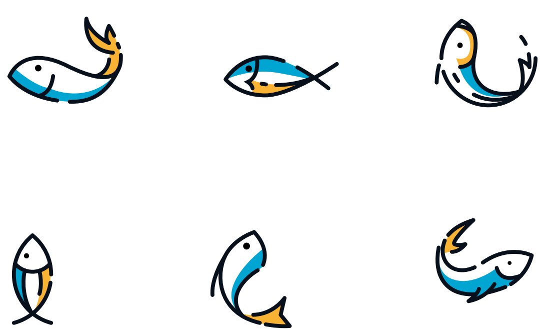
- With the corresponding values h_1 y Cd_{gap}, WHg is calculated from *Eq. 2*:

$$WHg(h_2;\Delta h) = \frac{3}{2} * \frac{Qg}{Cd_{gap} * h_1^{1.5} * \sqrt{2g}}$$

a.3.- The following values are calculated WGg (h_2; Δh)= WHg (h_2; Δh) + $0.2 * \Delta h$. Note: the flow contraction in the gap is considered as 20% of Δh less than the geometric width.

a.4.- WGg (h_2; Δh) values are only accepted when fulfilling WGg (h_2; Δh)≥WGg_{min}

a.5.- If in a.4, acceptable values (h_2; Δh) are not generated, or the user does not consider them as appropriate , the number of modules is adjusted and the calculation sequence is repeated. If with only one module there are no acceptable values of WGg (h_2; Δh) , or if they are acceptable but the designer does not consider appropriate, there is an alternative of design developed in section 5.3.6.



[6] *Δh determines the number of pools per height (meters) to pass (number of pools =$H_R/\Delta h$). Low values of Δh require a high number of pools for a same height to pass; high values imply high velocities through the gap. h_2 determines the min depth on the ramp and must allow the fish to progress easily through the gap; high values imply high visible height of the boulder.*

[7] *If the acceptable values of h_2 (in a.4) are low, those values can be increased by decreasing the number of modules and, if they are high, they can be decreased by increasing the number of modules.*

5.3.1.b Example:

CASE STUDY

- Environmental flow: 0.25 m^3/s
- Maximum velocity: 2.2 m/s ⇨ $\Delta h \leq 0.25$m
- Minimum depth: 0.2m ⇨ $h_2 \geq 0.2$m
- Minimum gap: 0.2m ⇨ $WGg \geq 0.2$m
- Dissipated power: 200W/m^3 ⇨ $Pd \leq 200$W/m^3
- Length of the weir ($Lweir$): 30m
- Width of the crest of the weir ($Wcweir$): 0.5m
- Height at the toe of the ramp (H_0): 1000.8 m.a.s.l.
- Height of the weir crest ($Hcweir$): 1002.9 m.a.s.l.
- Height difference between the top end of the boulder in the top row and $Hcweir$ (h_{weir}): 0.15m

For phase 1:

INPUTS:

- Q_{RMIN}=0.25m^3/s
- WGg_{min}=0.2m
- First starting this phase with 2 MODULES and gaps without steps (p=0)

RESULTS:

TABLE 11 is generated listing combinations of (h_2; Δh) fulfilling $WGg \geq WGg_{min}$

	Width of the geometric gap *WGg(m)*				
	h_2(m)				
$\Delta h(m)$	**0.2**	**0.25**	**0.3**	**0.35**	**0.4**
0.1	0,23				
0,12	0.20				
0.15					
0.17					
0.2					
0.22					
0.25					
0.3					
0.32					
0.35					

TABLE 11. Combinations of (h_2; Δh) that, with two modules and for Q_{RMIN}= 0.25m^3/s, fulfil $WGg \geq WGg_{min} \geq 0.2m$

The highest acceptable value of Δh (0.12m), is very low regarding to the maximum suitable one compatible with the max velocity for the target species (Δh_{max}=0.25m ⇨ V_{MAX}=2.2m/s). To assume this value would imply 8 rows per vertical meter to pass.

This value is ruled out and the calculations are repeated once again but with a single module.

RESULTS FOR 1 MODULE:

TABLE 12 summarises the results for this hypothesis.

	Width of the geometric gap *WGg(m)*				
	h_2(m)				
Δh(m)	**0.2**	**0.25**	**0.3**	**0.35**	**0.4**
0.1	0.43	0.36	0.31	0.27	0.24
0,12	0.39	0.33	0.29	0.25	0.23
0.15	0.33	0.29	0.25	0.23	0.21
0.17	0.31	0.27	0.24	0.22	
0.2	0.28	0.25	0.22	0.20	
0.22	0.26	0.23	0.21		
0.25	0.24	0.22	0.20		
0.3	0.22	0.20			
0.32	0.22				
0.35	0.21				

TABLE 12. Combinations of (h_2; Δh) that, with 1 module for $Q_{RMIN}=0.25m^3/s$, fulfil $WGg \geq WGg_{min} \geq 0.2m$

It is clear the range of combinations (h_2; Δh) fulfilling $WGg \geq WGg_{min}$ is much wider. The blue square shows those that also fulfil the requirements of velocity ($\Delta h \leq 0.25m$) and depth ($h_2 \geq 0.2m$) for the target species of this example.

5.3.2 PHASE 2. GEOMETRIC DIMENSIONING.

What is needed?

Height at the weir outfall (H_0), height of the weir crest ($Hcweir$), that determines the total height to pass ($H_T=Hcweir - H_0$), and the difference of height between the highest point of the boulders in the top row and Hcweir (h_{weir}).

What is generated?

Selecting (h_2; Δh) from the previous phase, then it is generated y -height difference between the weir crest and the water surface upstream of the ramp-, H_R -height of the ramp-, number of rows -*Nrow*-, numbe of pools -*Np*- and visual height of the boulder -*Hb*-.

What are these values for?

To define gemetric characteristics of the ramp.

5.3.2.a Calculation sequence:

a.1.- Once selected (h_2; Δh), with the corresponding value of WGg, and give the values of $H_T=H_0\text{-}Hcweir$ and h_{weir}[8], the following values are generated:

- Width of the hydraulic gap: $WHg = WGg\text{-}02 * \Delta h$
- Number of rows: $Nrow = integer\left(\frac{H_T - h_1}{\Delta h}\right) + 1$

[8] *h_{weir} determines the visible height of the boulder (Hb) and, therefore, the max depth upstream of the boulder ($h_{1\ max}$) for which the ramp can work without drowning, this depth corresponds to the Q_{RMAX}. The greater h_{weir}, the greater the functional range of flows of the ramp, but high values of h_{weir} imply high values of Hb. Recommending $h_{weir} \geq 0.15m$*

- Number of pools $Np = Nrow - 1$
- Height of the ramp: $H_R = (Nrow - 1) * \Delta h$
- Height difference between the water surface upstream of the ramp and the weir crest[9]: $y = H_T - H_R - h_1$, fulfilling $Hcweir\text{-}y \geq HWs\ min$
- Boulder height: : $Hb = h_1 + y + h_{weir}$

a.2.- If the values of Hb generated in the previous step are not suitable, step a.1 is repeated with other acceptable values of h_2; Δh; WGg.

5.3.2.b Example:

INPUTS:

- Height of the weir outfall: H_0=1000.8 m.a.s.l.
- Height of the weir crest: $Hcweir$= 1002.9 m.a.s.l.
- Height difference between the highest point of the boulders in the top row and $Hcweir$: h_{weir}=0.15m
- From the results table in the previous phase, the combination h_2=0.2m Δh=0.25m (implying V_{MAX} =2.2 m/s) is selected, those values correspond to the extreme acceptable values for each of those variables. These results will give an initial idea of the characteristics of the ramp for those extreme values.

RESULTS:

- ✓ Height difference between the crest of the weir and the water surface upstream of the ramp: y= 0.15m. Acceptable value for which the flow range discharged only through the ramp is significant.
- ✓ Width of the hydraulic gap: WHg= 0.19m
- ✓ Depth upstream of the boulder: h_1=0.45m
- ✓ Total height to pass: H_T= 2.1m
- ✓ Height of the ramp: H_R= 1.5m
- ✓ Number of rows: $Nrow$= 7
- ✓ Number of pools: Np= 6
- ✓ Visible height of the boulder: Hb= 0.75m

5.3.3 PHASE 3. DIMENSIONING TO CONTROL THE DISSIPATED POWER IN EACH POOL.

What is needed?

The values of the diameter of the boulder or dimension of the boulder perpedicular to the main flow (Db), width of the boulder or dimension of the boulder paralel to the main

[9] The greater y, the greater the range of flows discharging only through the ramp [$Q_{RMIN} \leq Qa \leq Q_R(y=0)$], range for which the call effect of the attraction flow is absolute. Recommending $y \geq 0.1m$

flow (Wb) and the angle between two consecutive boulders (α). With these values the width of the module can be determined, both edge (Wem) and (Wmm).

The range of gradients ($tg\beta$) to study the dissipated power.

What is generated?

For each of the considered gradients, the values of the pool lenghts (Lp) and ramp length (L_R), and also the module volumes ($Vol\ m$) and dissipated power (Pd), both for edge module and mid module.

What are these values for?

To select the gradient of the ramp that fulfils the condition of max dissipated power (determined by the user).

5.3.3.a Calculation sequence:

a.1.- Given Db[10], Wb[11]y α[12] , the following are calculated:

- Dimension of the gap (perpendicular to the main flow):

$$\mathrm{a} = WGg * cos\alpha$$

- Dimension of the gap (parallel to the main flow):

$$b = WGg * sen\alpha$$

- Width of modules (edge and mid):

$$Wem = 2.5 * \mathrm{Db} + 2\mathrm{a};\ Wmm = 2 * (Db + \mathrm{a})$$

a.2.- For a range of gradients of the ramp[13] [0.03≤$tg\beta$≤0.07], the following values are generated:

- Length of the pool: $Lp = \Delta h / tg\beta$
- Length of the ramp: $L_R = H_R / tg\beta$
- Volumes of the modules (edge and mid):

$$\text{Eq. 28} \quad Vol\ em = Wem * Lp * h_0$$

$$\text{Eq. 29} \quad Vol\ mm = Wmm * Lp * h_0$$

[10] *The value of Db determines the pool width and pool volume, playing a significant role in the dissipated power. It must be a value large enough –although proportional to Hb-, in order for the pool volume to allow fulfil the condition of dissipated power.*

[11] *The value Wb must be set proportionally to the values of and Hb.*

[12] *The recommended values of α [30°≤α≤45°] must be respected to generate the secondary flows needed to create uphill flows.*

[13] *The ramp gradient determines the length of the pools and, therefore, the ramp length. The length of the pools influences the volume and therefore the dissipated power:* $tg\beta\downarrow \Rightarrow Lp\uparrow \Rightarrow Vol\ m\uparrow \Rightarrow Pd\downarrow$

$Lp\uparrow \Rightarrow L_R\uparrow$

- Where the mean depth of the pool is h_0

$$\text{Eq. 30} \quad h_0 = \frac{h_1+h_2}{2}$$

- Dissipated power in the edge and mid modules:

$$\text{Eq. 31} \quad Pd_{em} = \frac{\gamma * Q\ module\ * \Delta h}{Vol\ em}$$

$$\text{Eq. 32} \quad Pd_{mm} = \frac{\gamma * Q\ module\ * \Delta h}{Vol\ mm}$$

where $Qmodule=2*Qg$

a.3.- The user selects the gradient depending on the requirements of max dissipated power suitable for the target species[14] and, taking into account the length of the ramp associated to that gradient.

5.3.3.b Example:

INPUTS:

- Dimension of the boulder (perpendicular to the main flow): Db **=0.55m**

- Width of the boulder: : Wb =0.45m

- α=40°

RESULTS:

For a single module, the width is generated by: $Wm= 3Db + 2a$

✓ Wm= 2.02m

TABLE 13 shows the results of the pool lengths (Lp), ramp length (L_R), volume of the module ($Vol\ m$) and dissipated power (Pd), for the assessed range of slopes [$0.03 \le tg\beta \le 0.07$].

			Module	
tgβ	L_R(m)	Lp(m)	Vol m (m^3)	Pd (W/m^3)
0.07	21.4	3.57	2.3	261.0
0.06	25.0	4.17	2.7	223.7
0.05	30.0	5.00	3.3	186.4
0.04	37.5	6.25	4.1	149.1
0.03	50.0	8.33	5.5	111.8

TABLE 13. For Db=0.55m, pool lengths (Lp), ramp length (L_R), volume of the module (Vol m) and dissipated power (Pd), for the range of slopes $0.03 \le tg\beta \le 0.07$.

[14] The dissipated power is a proxy for the turbulences and the incorporation of air that it brings – the literature normally refers as ´white waters´-. The presence of ´white waters´ is a factor that limits the passability of fishway devices. In the calculation protocol, the calculation of dissipated power refers to the whole module. However, for the case of uphill flow ramps, is obvious that the ´white waters´ are focused on the zones of high velocities, and their presence in the zones of uphill flows is irrelevant, therefore these ones will be the zones chosen by the fish to progress upstream. It is reasonable then, to assume that the value of max dissipated power (TABLE 1) can be, for this type of ramps, greater than the recommended values for devices where the turbulence is present in a homogenous way.

Taking into account that for the target species $Pd(W/m^3) \leq 200$, the greater possible gradient would be 5%, with a ramp length of 30m.

To assess other options, the calculations are repeated for a larger Db:

INPUTS:

- Dimension of the boulder (perpendicular to the main flow): Db **=0.65m**
- Width of the boulder Wb =0.45m
- α=40°

RESULTS:

✓ Wm= 2.32m

For the new value of Db, TABLE 14 shows the results of the pool lengths (Lp), ramp length (L_R), volume of the module ($Vol\ m$) and dissipated power (Pd), for the considered range of the gradients [$0.03 \leq tg\beta \leq 0.07$].

			Module	
$tg\beta$	L_R(m)	Lp(m)	Vol m (m^3)	Pd (W/m^3)
0.07	21.4	3.57	2.7	227.3
0.06	25.0	4.17	3.1	194.8
0.05	30.0	5.00	3.8	162.3
0.04	37.5	6.25	4.7	129.9
0.03	50.0	8.33	6.3	97.4

TABLE 14. For Db=0.65m, pool lengths (Lp), ramp length (L_R), volume of the module (Vol m) and dissipated power (Pd), for the range of gradients $0.03 \leq tg\beta \leq 0.07$

For Db=0.65m, the gradient can be 6%, with a ramp length of 25m.

These are the values accepted for the design.

5.3.4 PHASE 4. GENERATION OF THE FUNCTIONAL FLOW RANGE.

What is needed?

The values of Hb, $hweir$, Δh, Nm y $tg\beta$ generated in previous phases for Q_{RMIN} and the width of the weir crest ($Wcweir$) and its length ($Lweir$).

What is generated?

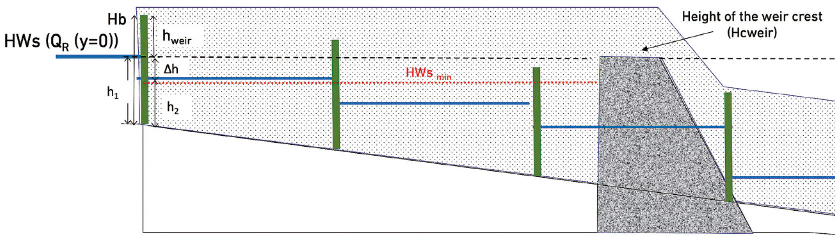
Keeping the hypothesis of uniform regime, for the dimensioned ramp, to generate the flows $Q_R(y=0)$, Q_{RMAX} y $Q_{weir}(h_{weir})$

What are theses values for?

To determine the functional range of flows of the ramp. [$Q_{RMIN} \leq Q_{RAMP} \leq Q_{RMAX}$], the range for which Qa is only dischargued thfough the ramp [$Q_{RMIN} \leq Qa \leq Q_R(y=0)$] and the maximum Qa for which the ramp is functional [$Qa_{MAX} = Q_{RMAX} + Q_{weir}(h_{weir})$]

5.3.4.a Calculation sequence for $Q_R(y=0)$

The *FIGURE 21* showed the location of the water surface for this case. The figure is shown here again to help interpreting the calculation sequence.



a.1.- For the values Hb, h_{weir}, the new h_1 and h_2 are generated. Δh does not require being re-calculated as it does not vary in uniform regime.

$$h_1(y=0)=Hb-h_{weir}$$
$$h_2(y=0)=h_1(y=0)-\Delta h$$

a.2.- With the equations 2 and 3, Qg y Cd_{gap} can be calculated, and known the number of modules and gaps, the flow in the ramp can be generated $Q_R(y=0)= Qg(y=0)*2Nm$

a.3.- For the gradient selected in phase 3, the dissipated power (both edge and mid modules) is calculated using equations 31 and 32.

a.4.- If the value of the dissipated power is greater than the max acceptable for the target species, the user can come back to the step a.3 (phase 3) to select a different gradient to dimension the ramp.

5.3.4.b Example:

INPUTS:

- Hb= 0.75m
- h_{weir}= 0.15
- Δh =0.25m
- Nm =1
- $tg\beta$ =0.06

RESULTS:

✓ $h_1[Q_R(y=0)]$= 0.6m
✓ $h_2[Q_R(y=0)]$=0.35m
✓ $Q_R(y=0)$=0.355 m^3/s

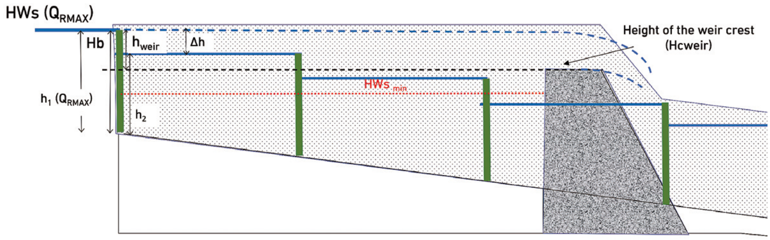
			Module	
$tg\beta$	L_R(m)	Le(m)	Vol m (m^3)	Pd (W/m^3)
0.07	21.4	3.57	3.9	221.0
0.06	25.0	4.17	4.6	189.5
0.05	30.0	5.00	5.5	157.9
0.04	37.5	6.25	6.9	126.3
0.03	50.0	8.33	9.2	94.7

TABLE 15. For $Q_R(y=0)$=0.355 m^3/s, volume of the module (Vol m) and dissipated power (Pd), for the range of gradients $0.03 \le tg\beta \le 0.07$

Because for the selected gradient (6%), the dissipated power is still under the maximum dissipated power set for the target species, it is not required to re-dimension.

5.3.4.C Calculation sequence for Q_{RMAX}, $Q_{weir}(h_{weir})$ y Qa_{MAX}

FIGURE 22 showed the situation of the water surface for this case. The figure is shown here below again.



c.1.- To calculate again the values of h_1 and h_2. In uniform regime Δh does not vary.

$$h_1(Q_{RMAX})=Hb$$

$$h_2(Q_{RMAX})=h_1(Q_{RMAX})-\Delta h$$

c.2.- To calculate Cd_{gap} and Qg with equations 2 and 3, knowing the number of modules (and gaps), the flow in the ramp is generated $Q_{RMAX}= Qg(Q_{RMAX})*2Nm$

c.3.- For the selected gradient selected, the dissipated power (both edge and mid modules) is calculated using equations 31 and 32.

c.4.- If the value of the dissipated power is greater than the max acceptable for the target species, the user can come back to the step a.3 (phase 3) to select a different gradient to dimension the ramp.

c.5.- Known the width of the weir crest ($Wcweir$) and its length ($Lweir$), to calculate the flow spilling over the weir for a hydraulic head h_{weir}:

$$Q_{weir}\left(y_{weir}\right) = 1.7 * Cd_{weir} * y_{weir}^{1.5} * Lweir$$

where: $$Cd_{weir} = \begin{cases} 0.75 + \dfrac{0.1}{Wcweir/y_{weir}} & \text{if } \dfrac{Wcweir}{y_{weir}} > 3 \\ 0.7 + \dfrac{0.185}{Wcweir/y_{weir}} & \text{if } \dfrac{Wcweir}{y_{weir}} \leq 3 \end{cases}$$

c.6.- Generating $Qa_{MAX}= Q_{RMAX} + Q_{weir}\left(h_{weir}\right)$

5.3.4.d Example:

<u>**INPUTS**</u>:

- For the ramp, same data that those previously used to calculate $Q_R(y=0)$ in the former phase
- For the weir:
  - Crest width: : $Wcweir$=0.5m
  - Weir length: $Lweir$=30m

RESULTS:

- ✓ $h_1(Q_{RMAX})$= 0.75m
- ✓ $h_2(Q_{RMAX})$= 0.5m
- ✓ Q_{RMAX}= 0.46 m³/s
- ✓ $Q_{weir}\ (h_{weir})$ = 2.31 m³/s
- ✓ Qa_{MAX} = 2.77 m³/s

			Module	
$tg\beta$	L_R(m)	L_P(m)	Vol m (m³)	Pd (W/m³)
0.07	21.4	3.57	5.2	219.8
0.06	25.0	4.17	6.1	188.4
0.05	30.0	5.00	7.3	157.0
0.04	37.5	6.25	9.1	125.6
0.03	50.0	8.33	12.1	94.2

TABLE 16. For Q_{RMAX}=0.46 m³/s, volume of the module (Vol m) and dissipated power (Pd), for the range of gradients $0.03 \leq tg\beta \leq 0.07$

Because for the selected gradient (6%), the dissipated power is still under the maximum dissipated power set for the target species, it is not required to re-dimension.

The range of flows for which the designed ramp is functional are summarised in TABLE 17.

TABLE 17. Range of flows for which the design ramp is functional

SCENARIO	LOCATION OF THE DISCHARGE	RANGE OF FLOWS Qa	VALUES GENERATED (m³/s)
1	Ramp only	$Q_{RMIN} \leq Qa \leq Q_R(y=0)$	Q_{RMIN}= 0.25 $Q_R(y=0)$=0.35
2	Ramp and weir No drowning of boulders	$\underset{\text{RAMP}}{Q_R(y=0)} < Qa \leq \overbrace{\underset{\text{RAMP}}{Q_{RMAX}} + \underset{\text{WEIR}}{Q_{weir}(h_{weir})}}^{Qa_{MAX}}$	Q_{RMAX}= 0.46 $Q_{weir}(h_{weir})$=2.31 Qa_{MAX}=2.77

If the user has the flow duration curves for the pre-reproductive period –or a different period to assess the functionality of the ramp- the user can generate the percentiles of exceedance corresponding to the flows Q_{RMIN} y Qa_{MAX}.

Being A% the exceedance percentile[15] corresponding to Q_{RMIN} and B% the corresponding to Qa_{MAX}.

The percentage of time that, as average, the ramp will be functional within the considered period is: A%-B%.

The number of days that, as average, the ramp will be functional within the period will be: [(A%-B%) *number of days of the period]/100.

[15] *The flow Q_{RMIN} is equal or over (average), the A% of the days for that period.*

5.3.5 PHASE 5. ASSESSMENT OF THE BEHAVIOUR IN NON-UNIFORM REGIME.

What is needed?

The design of the ramp from previous phases.

Q_{RMAX}

What is generated?

The velocity values in the gaps of each row, assuming the hypothesis of non-uniform regime.

What are these values for?

To assess the behaviour of the ramp in non-uniform regime and to assess its functionality for this condition.

5.3.5.a Introduction

In previous phases, under the hypothesis of uniform regime, and for each of the three considered flows [Q_{RMIN}; $Q_R(y=0)$; Q_{RMAX}], the value h_2 has been generated for each of them; and because of the uniform regime, it is assumed constant in all the rows.

But, what would happen if that value changes at the bottom row?

This situation is reasonable, because, although there is one additional pool that allows some degree of control between the last row and the river, the depths in that pool will be determined by the flow and boundary conditions in the river reach downstream of the ramp, conditions that are not easy to control and subject to the changes of fluvial dynamic.

If h_2 takes a different value to the corresponding to uniform regime in the bottom row (h_2^U), the values h_2 change in the rest of rows, and with them, there are changes in Δh. Therefore, the values of min depths and max velocities in the ramp change in relation to the values in uniform regime. Note that minimum depth is determined by h_2 and that $V_{MAX} = \sqrt{2g\Delta h}$, which can compromise the functionality of the device.

Therefore, in the dimensioning of the ramp, it is important to consider the non-uniform regime (Fuentes-Pérez *et al.*, 2016; Fuentes-Pérez *et al.*, 2018).

FIGURE 23 shows the situation in uniform regime (U) and the two cases (for that same flow) in non-uniform regime. The numbering of the rows goes from upstream to downstream.

- M1: when the value of h_2 in row n is larger than the one corresponding to uniform regime [$h_{2,n}^{no\,U} > h_2^U$].
- M2: when the value of h2 in the row n is lesser than the one corresponding to uniform regime [$h_{2,n}^{no\,U} < h_2^U$].

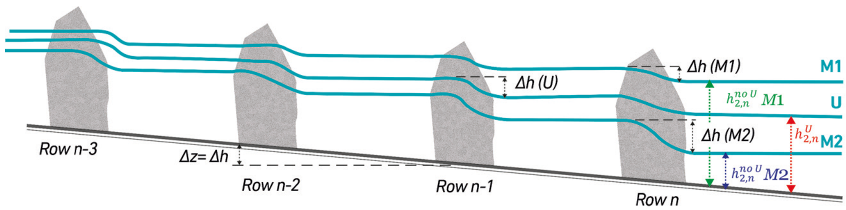


FIGURE 23

Profiles of the water surface in uniform regime (U), [Δh(U)=Δz], and non-uniform with backwater type M2 [Δh(M2)>Δz], and backwater type M1 [Δh(M1)<Δz]

In both cases, the system tend to the upstream value h_2 in uniform regime, that will reach depending on the number of rows and the absolute value of the difference between $h_{2,n}^{no\,U}$ and h_2^U: $[ABS\,(h_{2,n}^{no\,U} - h_2^U)]$

Considering the functionality of the ramp, the most difficult situation is M2, because the values of Δh generated in the bottom rows are larger than the value Δh in uniform regime $[\Delta h(M2) > \Delta h(U) > \Delta h(M1)]$. Note that Δh determines the max velocity in a gap, therefore, $V_{MAX}(M2) > V_{MAX}(U) > V_{MAX}(M1)$, at least in the bottom rows.

It could happen that the ramp stopped being functional: $V_{MAX}(M2) > V_{MAX}$ (target species).

5.3.5.b Recommendations for the outfall pool

To reduce the likelihood of this issue happening it is important to design a final pool –outfall pool- that makes the transition between the ramp and the river, controlling the value of $h_{2,n}$.

This link must be set in a way that, when the situation M2 appears, promotes that $ABS\,(h_{2,n}^{no\,U} - h_2^U)$ is not too large.

For the design of the outfall pool, it is recommended:

- A pool length 1.5 or 2 times longer than the ramp pools –to provide enough capacity to dissipate energy-, offering a comfortable starting point to the fish to progress through up the ramp.
- Horizontal pool bed at least 0.3 meters below the riverbed. This provides a volume for depositing sediment to reduce maintenance and give additional depth to improve shelter for fish.
- To create boundaries, to use boulders with a top height at least equal to the water surface $h_{2,n}^{no\,U}$ of the top row corresponding to flow $Q_R(y=0)$.
- In the 50% of the flow gaps of the outfall pool, its capacity of draining off must be decreased between 20 and 30%. This reduction can be attained by decreasing the width of the gap in that same percentage compared to the width of ramp gaps.
- If this option is not advisable – because it could make more difficult the access of

fish and/or limit the self-cleaning capacity of the pool-, a step could be set (see section 5.3.6) that decreases the drain capacity in that same percentage. This way, it is guaranteed that for all the range [$Q_{RMIN} \leq Qa \leq Q_R(y=0)$], the values of $h_{2,n}^{no\,U}$ determined by the outfall pool, will always be greater or equal than the corresponding value in uniform regime, and therefore a situation M1 or U will occur.

5.3.5.c Scenarios for the assessment

The assessment of the ramp functionality in non-uniform regime requires that the designer defines a range of assessment scenarios.

If the outfall pool was dimensioned following the aforementioned guidelines, we could assume that a non-uniform regime for the flow range $Q_{RMIN} \leq Qa \leq Q_R(y=0)$ would occur for M1 condition, without obstacles for fish passability.

The most critical situation could arise for flows in the range $Q_R(y=0) \leq Qa \leq Qa_{MAX}$. Along this range, even existing net flow through the spillway, depth in the channel does not guarantee in the outfall pool the presence of depths which exclude the possibility of occurrence of a M2 non-uniform regime (see *FIGURE 23*).

The least favourable situation –that is, a maximum value of Δh in the bottom row- is associated to the following hypothesis:

- The maximum potential flow is crossing the ramp $Q=Q_{RMAX}$.
- In the bottom row, $h_{2,n}^{no\,U}$ will be equal to the value linked, in uniform regime, to the flow $Q_R(y=0)$: $h_{2,n}^{no\,U}(Q_{RMAX}) = h_2^U Q_R(y=0)$

This value of $h_{2,n}^{no\,U}$ in the bottom row [$h_2^U Q_R(y=0)$] is guaranteed if the recommendations for defining the size of boulders which delineate the outfall pool have been followed. It is a conservative value, because it implies assuming that, for $Qa = Qa_{MAX}$, depth in the last row is equal to the depth for $Qa=Q_R(y=0)$. Consequently, results obtained under this hypothesis place the assessment clearly on the conservative side.

A less strict hypothesis would be assuming $h_{2,n}^{no\,U}(Q_{RMAX}) > h_2^U Q_R(y=0)$ We could use:

$$- h_{2,n}^{no\,U}(Q_{RMAX}) = 0.6 - 0.8 * h_2^U(Q_{RMAX})$$

Always that, logically, the value of $h_{2,n}^{no\,U}(Q_{RMAX})$ obtained by that procedure is higher than $h_2^U Q_R(y=0)$.

Values of $h_2^n(Q_{RMAX})$ and $h_2^U Q_R(y=0)$ are obtained in phase 4.

5.3.5.d Calculation of Q_{RMAX} in non-uniform regime

The procedure requires applying some expressions already presented:

Eq. 2 $$Qg = {}^2/_3 * \sqrt{2g} * Cd_{gap} * WHg * (h_1)^{1.5}$$

with $$h_1 = h_2 + \Delta h;\ Cd_{gap} = \beta_0 * \left[1 - \left(\frac{h_2}{h_1}\right)^{1.5}\right]^{\beta_1} \quad \beta_0 = 0.812 \quad \beta_1 = 0.335$$

Gap flow must be a fixed value: $Qg_{MAX} = Q_{RMAX}$/*number of gaps*

In each row i (i=number of row, being 1 the row located more upstream, and n more downstream), the value of $h_{2,i}$ will be withdrawn from the calculations in row i+1. Boulders delineating the outfall pool are not considered as ramp rows.

Step 1: For the bottom row (row n), the value of $h_{2,n}^{no\,U}(Q_{RMAX})$ must be estimated by the designer, considering the alternatives shown in the former section:

$$\text{Eq. 33} \quad h_{2,n}^{no\,U}(Q_{RMAX}) = \begin{cases} 0.6 - 0.8 * h_2^U(Q_{RMAX}) \\ h_2^U Q_R(y=0) \end{cases}$$

Step 2: The value of $\Delta h_n^{no\,U}$ is calculated by iterating equation 2:

ABS[Qg($h_{2,n}^{no\,U}$; $\Delta h_n^{no\,U}$) - Qg_{MAX}] ≤threshold (fixed by designer).

Once $\Delta h_n^{no\,U}$ is obtained, the maximum velocity in the gap of row n is estimated $V_{MAX,n} = \sqrt{2g * \Delta h_n^{no\,U}}$

Step 3: Later, the value of Δh is estimated in the following row ($\Delta h_{n-1}^{no\,U}$):

- $h_{2,n-1}^{no\,U}$ is calculated as ($h_{2,n}^{no\,U}$+$\Delta h_n^{no\,U}$)-Δh^U, being Δh^U the value selected in phase 1, and associated to Δz (*FIGURE 23*).
  - Equation 2 is solved for $h_{2,n-1}^{no\,U}$ giving values of $\Delta h_{n-1}^{no\,U}$ until reaching *ABS[Qg($h_{2,n-1}^{no\,U}$; $\Delta h_{n-1}^{no\,U}$) - Qg_{MAX}] ≤threshold.*
  - After calculating $\Delta h_{n-1}^{no\,U}$ the maximum velocity in the gap of row n-1 is estimated: $V_{MAX,n-1} = \sqrt{2g * \Delta h_{n-1}^{no\,U}}$

Step 4: The sequence is repeated in the upstream direction, until reaching the top row (row 1).

This way, the designer may obtain all values of $\Delta h_i^{no\,U}$ (i=1...n), and the associated velocities $V_{MAX,i}$ (i=1...n). And assess the ramp functionality in non-uniform regime, considering the natation capacities of the target species versus $V_{MAX,i}$ (i=1...n).

5.3.5.e Example:

INPUTS:

- The least favourable hypothesis is used for the depth downstream of the last row: $h_{2,n}^{no\,U}(Q_{RMAX}) = h_2^U Q_R(y=0)$ =0.35m

RESULTS:

TABLE 18 presents the results of $h_{2,i}^{no\,U}(Q_{RMAX})$, $h_{1,i}^{no\,U}(Q_{RMAX})$, Δh_i and $V_{MAX,i}$ (i=1...n). Values of $V_{MAX,i}$ which do not fit the conditioning for the target species are highlighted in red.

FIGURE 24 shows the values of Δh_i.

ROW (i)	X	h_2(m)	h_1(m)	Δh(m)	v(m/s)
n=7	25.0	0.35	0.70	0.35	2.6
6	20.9	0.45	0.73	0.28	2.4
5	12.5	0.49	0.75	0.26	2.3
4	4.2	0.50	0.75	0.25	2.3
3	0.0	0.50	0.75	0.25	2.2
2	4.2	0.50	0.21	0.25	2.2
1	0.0	0.50	0.20	0.25	2.2

TABLE 18. Depth values downstream (h_2) and upstream (h_1) of the boulder, Δh and velocity, for non-uniform regime, with Q_{RMAX} and $h_{2,n}^{no\,U}(Q_{RMAX}) = h_2^U Q_R(y=0)$ = 0.35m

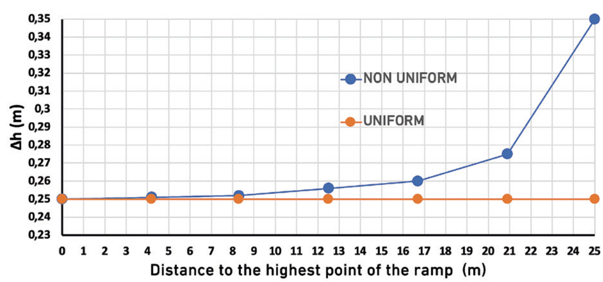


FIGURE 24
Evolution of Δh values along the ramp for non-uniform regime, with Q_{RMAX} and $h_{2,n}^{no\,U}(Q_{RMAX}) = h_2^U Q_R(y=0) =$ 0.35m (Dots indicate position of the boulder rows).

Calculations for a less conservative hypothesis are repeated, assuming that, the flow discharged through the weir contributes to the elevation of the water surface in the outfall pool.

INPUTS:

- $h_{2,n}^{no\,U}(Q_{RMAX}) = 0.8 * h_2^U(Q_{RMAX})$ = 0.4m
- For the condition $ABS[Qg(h_{2,n}^{no\,U};\ \Delta h_n^{no\,U}) - Qg_{MAX}] \leq threshold$, the threshold value considered has been 0.003m.

RESULTS:

TABLE 19 shows results for $h_{2,n}^{no\,U}(Q_{RMAX})$, $h_{1,n}^{no\,U}(Q_{RMAX})$, Δh_i and $V_{MAX,i}$ $(i=1\ldots n)$. Values of $V_{MAX,i}$, which do not fit the conditioning for the target species are highlighted in red.

ROW (i)	X	h_2(m)	h_1(m)	Δh(m)	v(m/s)
n=7	25.0	0.4	0.71	0.31	2.5
6	20.9	0.46	0.74	0.28	2.3
5	16.7	0.49	0.74	0.26	2.3
4	12.5	0.49	0.75	0.25	2.2
3	8.3	0.50	0.75	0.25	2.2
2	4.2	0.50	0.75	0.25	2.2
1	0.0	0.50	0.75	0.25	2.2

TABLE 19. Depth values downstream (h_2) and upstream (h_1) of the boulder, Δh and velocity, for non-uniform regime, with Q_{RMAX} and $h_{2,n}^{no\,U}(Q_{RMAX}) = 0.8 * h_2^U(Q_{RMAX})$ = 0.4m.

FIGURE 25 shows the values of Δh_i.

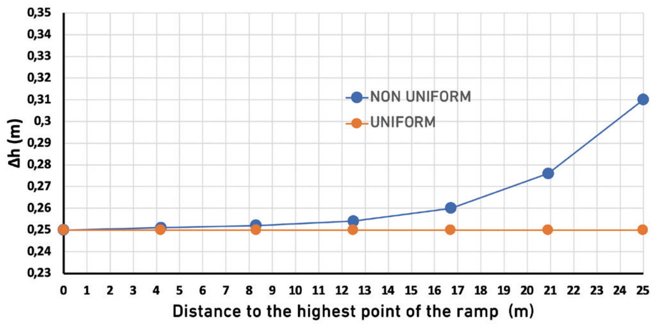


FIGURE 25
Evolution of Δh values along the ramp for non-uniform regime, with Q_{RMAX} and $h_{2,n}^{no\,U}(Q_{RMAX}) = 0.8 * h_2^U Q_R(y = 0) = 0.4m$ (Dots indicate position of the boulder rows).

V_{MAX} is also overcome in the last three rows for this hypothesis. But, differently from the former case, just in the last row velocity is over 10% of the maximum value. Consequently, the proposed design could be accepted.

5.3.6 DIMENSIONING THE RAMP FOR FLOW GAPS WITH STEP

When Q_{RMIN} is low, it may happen that, even with an only module, design values for h_2 are not high enough to pass the threshold fixed by the target species.

In those cases, for a given WGg, the construction of a step (see *FIGURE 26*) may allow:

- For a given flow, generation of higher depths.
- Maintenance of a given depth with lower flow.

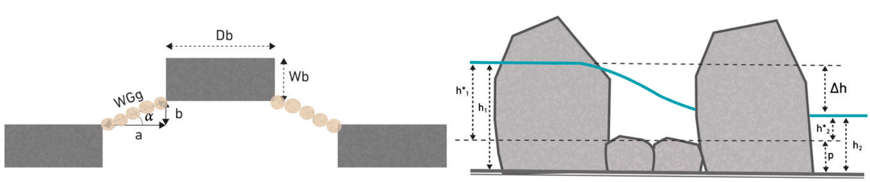


FIGURE 26
Planform and lateral view of a module with step between boulders

However, if the step is added to the ramp, flow Q_{RMAX} will be lower than the flow which could cross the ramp without it. Thus, the range of flows which make the ramp functional would be smaller.

It also conditions the value of the passing depth in the flow gap: $h_2^* = h_2 - p$, which must be established in a way that $h_2^* \geq max(0.1\text{m}; \text{dorsal-ventral length})$.

5.3.6.a Calculation sequence

The design protocol is equal to that previously presented. The only difference is that the equation which allow quantifying gap flow must include the value of the step height (p) (Fuentes-Pérez *et al.*, 2016):

$$h_2^* = h_2 - p$$

$$Qg(p) = \frac{2}{3} * \sqrt{2g} * Cd_{gap} * WHg * (\mathrm{h}_1 - \mathrm{p})^{1.5}$$

5.3.5.e Example:

Phase 1:

In phase 1, the alternative of constructing two modules in the ramp was discarded. TABLE 11, gathers the combinations of (h_2;Δh) offering values of $WGg \geq WG_{gmin}$. The highest suitable value of Δh (Δh=0.12m) was too low when compared to the maximum value coherent with the maximum velocity for the target species (Δh_{max}=0.25m, then V_{MAX}=2.2 m/s). Assumption of that value implied 8 rows for each meter to pass.

At this stage, the alternative of constructing two modules will be assessed, adding a step in the flow gaps.

Considering a step height (p) of 0.1 m, and for two modules, the range of combinations (h_2; Δh) which offer values of $WGg \geq WGg_{min}$ becomes much wider (TABLE 20).

	Values of WGg (m)				
	h₂(m)				
$\Delta h(m)$	0.2	0.25	0.3	0.35	0.4
0.1	0.36	0.28	0.23		
0,12	0.31	0.25	0.20		
0.15	0.26	0.21			
0.17	0.24				
0.2	0.21				
0.22					
0.25					
0.3					
0.32					
0.35					

TABLE 20. Combinations of (h_2;Δh) that, with two modules, step height of 0.1m and Q_{RMIN}=0.25m^3/s, offer values of $WGg \geq WGg_{min} \geq 0.2$m

In this section, values of variables which turn different to the one-module alternative will be highlighted (in bold).

Phase 2:

INPUTS:

- H_0=1000.8 m.a.s.l.
- *Hcweir*= 1002.9 m.a.s.l.
- h_{weir}=0.15m
- From *TABLE 20*, combination h_2=0.2m; **Δh=0.2m** is selected (which means V_{MAX}= 2m/s)

RESULTS:

- ✓ Height difference between weir threshold and water surface upstream of the ramp: ***y*= 0.1m**. Acceptable value for a significant range of flows strictly discharged through the ramp
- ✓ Depth upstream of the boulder **h_1=0.4m**
- ✓ Total height to pass H_T= 2.1m
- ✓ Ramp height **H_R**= 1.6m
- ✓ Number of rows ***Nrow*= 9**
- ✓ Number of pools ***Np*= 8**
- ✓ Boulder height ***Hb*= 0.65m**

Phase 3:

INPUTS:

- Boulder dimension in the transversal direction to the flow: *Db* **=0.55m**
- Boulder width: *Wb*=0.45m
- α=40°

RESULTS:

- ✓ ***Wem*=1.7m**

TABLE 21 shows the results of pool length (*Lp*), ramp length (L_R), module volume (*Vol m*) and dissipated power (*Pd*), for the range of slopes considered [[0.04≤*tg*β≤0.08].

			Edge Module	
tgβ	L_R(m)	L_p(m)	Vol m (m^3)	Pd(W/m^3)
0.08	20.0	2.50	1.3	192.7
0.07	22.9	2.86	1.5	168.6
0.06	26.7	3.33	1.7	144.5
0.05	32.0	4.00	2.0	120.4
0.04	40.0	5.00	2.5	96.3

TABLE 21. For Db=0.55m, pool length (Lp), ramp length (L_R), module volume (Vol m) and dissipated power (Pd), for the slopes range 0.04≤tgβ≤0.08

Considering that, for the target species Pd(W/m^3)≤200, maximum possible slope would be **8%**, with a ramp length of **20m**.

Phase 4:

a) Calculation sequence for $Q_R(y=0)$

INPUTS:

- ***Hb*= 0.65m**
- h_{weir}= 0.15m
- ***Δh* =0.2m**
- ***Nm* =2**
- ***tgβ* =0.08**

RESULTS:

✓ **$h_1[Q_R(y=0)]$= 0.5m**

✓ **$h_2[Q_R(y=0)]$= 0.3m**

✓ **$Q_R(y=0)$= 0.357 m³/s**

			Edge Module	
tgβ	L_R(m)	L_p(m)	Vol m (m^3)	Pd (W/m^3)
0.08	20.0	2.50	1.7	206.5
0.07	22.9	2.86	1.9	180.7
0.06	26.7	3.33	2.3	154.9
0.05	32.0	4.00	2.7	129.0
0.04	40.0	5.00	3.4	103.2

TABLA 22. For $Q_R(y=0)$=0.357 m^3/s, module volume (Vol m) and dissipated power (Pd), for the slopes range 0.04≤tgβ≤0.08

For the selected slope (8%), dissipated power is only 3% higher than the maximum assumable value. As such, it is assumed that re-dimensioning is not necessary.

b) Calculation sequence for Q_{RMAX}, Q_{weir} (h_{weir}) y Qa_{MAX}

INPUTS:

- For the ramp, same data that those previously used to calculate $Q_R(y=0)$ in the former phase
- For the weir:
  - Crest width: *Wcweir*=0.5m
  - Weir length: *Lweir*=30m

RESULTS:

✓ **$h_1(Q_{RMAX})$= 0.65m**

✓ **$h_2(Q_{RMAX})$= 0.45m**

✓ **Q_{RMAX}= 0.53 m³/s**

✓ **Q_{weir} (h_{weir})= 2.31 m³/s**

✓ **Qa_{MAX}= 2.84 m³/s**

tgβ	L_R(m)	L_p(m)	Edge Module Vol m (m³)	Edge Module Pd (W/m³)
0.08	20.0	2.50	2.3	221.0
0.07	22.9	2.86	2.7	193.4
0.06	26.7	3.33	3.1	165.8
0.05	32.0	4.00	3.7	138.1
0.04	40.0	5.00	4.7	110.5

TABLE 23. For Q_{RMAX}=0.53 m³/s, module volume (Vol m) and dissipated power (Pd), for the slopes range 0.04≤tgβ≤0.08

For the selected slope (8%), dissipated power is much higher than the maximum assumable for the target species. It is, thus, necessary to reduce ramp slope, increase pool width, or introduce both changes.

A 7% slope is assumed, with no further modifications. The condition of maximum assumable dissipated power is suited for all scenarios.

Phase 5:

INPUTS:

- The least favourable hypothesis is considered for the depth downstream of the bottom row: $\boldsymbol{h_{2,n}^{no\,U}(Q_{RMAX}) = h_2^U Q_R(y=0)}$ **= 0.3m**

RESULTS:

TABLE 24 shows the results for $h_{2,i}^{no\,U}(Q_{RMAX})$, $h_{1,i}^{no\,U}(Q_{RMAX})$, Δh_i and $V_{MAX,i}$ $(i=1\dots n)$.

FIGURE 27 includes the values of Δh_i .

ROW (i)	X	h_2(m)	h_1(m)	Δh(m)	V(m/s)
n=9	33.4	0.3	0.53	0.23	2.1
8	29.2	0.33	0.54	0.21	2.0
7	25.0	0.34	0.55	0.21	2.0
6	20.9	0.35	0.55	0.20	2.0
5	16.7	0.35	0.55	0.20	2.0
4	12.5	0.35	0.55	0.20	2.0
3	8.3	0.35	0.55	0.20	2.0
2	4.2	0.35	0.55	0.20	2.0
1	0.0	0.35	0.55	0.20	2.0

TABLE 24. For non-uniform regime, two modules with step, Q_{RMAX} and $h_{2,n}^{no\,U}(Q_{RMAX}) > h_2^U Q_R(y=0)$, , the values of depth downstream (h_2) and upstream (h_1) of the boulder, Δh and the associated velocity are presented, for each row.

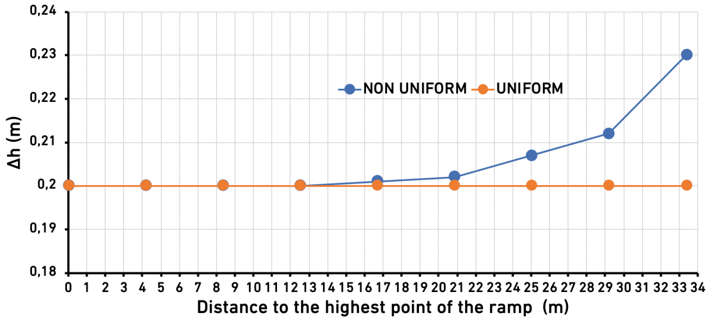


FIGURE 27

For non-uniform regime, two modules with step, Q_{RMAX} and $h_{2,n}^{no\,U}(Q_{RMAX}) > h_2^U Q_R(y=0)$= 0.3m, evolution of Δh values along the ramp. (Dots indicate position of the boulder rows).

V_{MAX} (2.2m/s) is not overcome in any row: the design may thus be accepted.

5.3.7 DESIGN SUMMARY

It follows an update of the case data, and the values of the variables obtained for the two alternatives analysed in the dimensioning:

CASE STUDY

- Minimum environmental flow: 0.25m³/s
- Maximum velocity: 2.2 m/⇨$\Delta h \leq 0.25$m
- Minimum depth: 0.2m⇨$h_2 \geq 0.2$m
- Minimum gap width: 0.2m⇨$WGg \geq 0.2$m
- Maximum dissipated power: 200 W/m³⇨$Pd \leq 200$ W/m³
- Weir length ($Lweir$): 30m
- Crest width ($Wcweir$): 0.5m
- Height of ramp outfall (H_0): 1000.8 m.a.s.l.
- Height of weir crest ($Hcweir$): 1002.9 m.a.s.l.
- Height difference between the highest point of the boulders on the top row and the height on the weir crest (h_{weir}): 0.15m

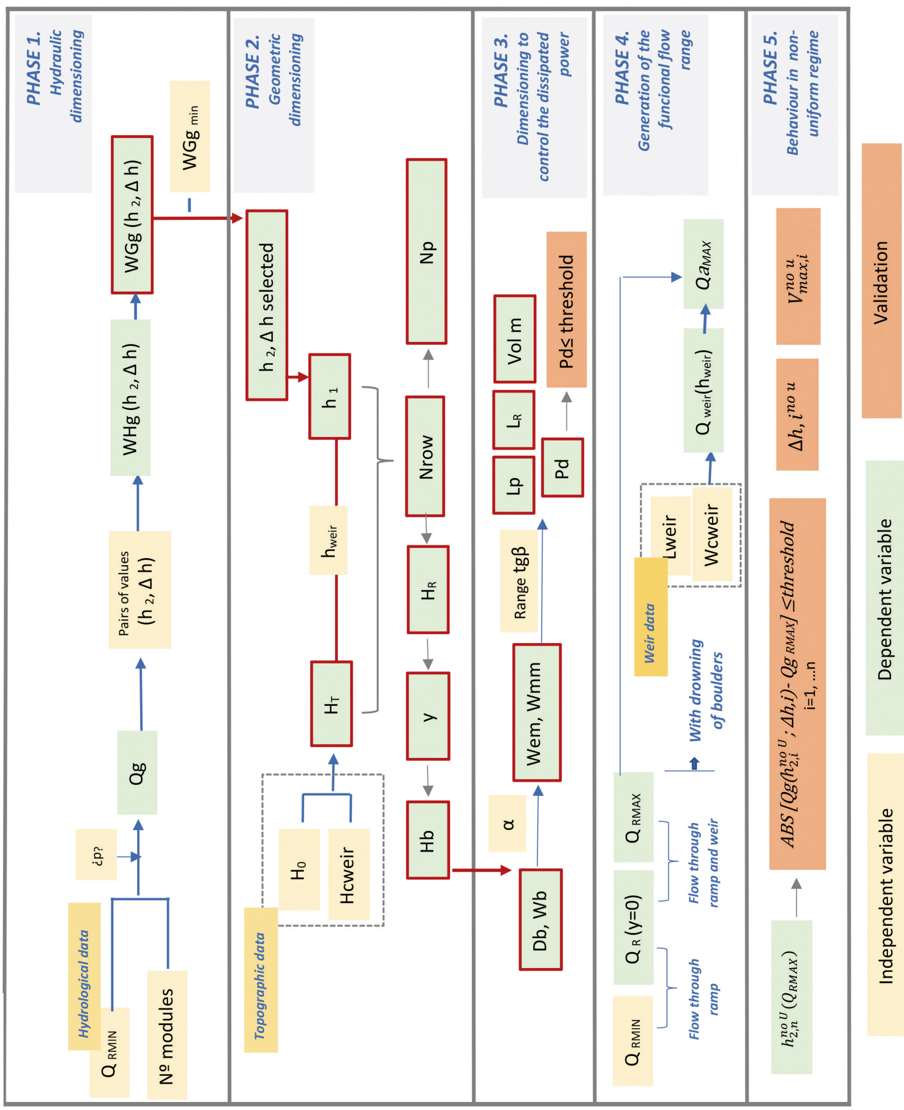
	Number of modules	
VARIABLE	1 without step	2 with step
$p(m)$	0	0.1
$h_2(m)$	0.2	0.2
$\Delta h(m)$	0.25	0.2
$h_{weir}(m)$	0.15	0.15
$V_{MAX}(m/s)$	2.2	2
$WGg(m)$	0.24	0.21
$WHg(m)$	0.19	0.17
$h_1(m)$	0.45	0.4
$y(m)$	0.15	0.1
$H_T(m)$	2.1	2.1
$H_R(m)$	1.5	1.6
$Nrow$	7	9
Np	6	8
$Hb(m)$	0.75	0.65
$Db(m)$	0.65	0.55
$Wb(m)$	0.45	0.45
$\alpha(°)$	40	40
$tg\beta$	0.06	0.07
$Wm(m)$	2.32	
$Wem(m)$		1.7
$Wramp(m)$	2.32	3.4
$Lp(m)$	4.2	2.9
$L_R(m)$	25	23
$Pd(W/m^3)$	195	169

TABLE 25. Results obtained for Q_{RMIN}=0.25m^3/s and the two alternatives considered in the design (1 module without step and 2 modules with step).

	Number of modules	
VARIABLE	1 without step	2 with step
$Q_{RMIN}(m^3/s)$	0.25	0.25
$Q_R(y=0)(m^3/s)$	0.35	0.36
$Q_{RMAX}(m^3/s)$	0.46	0.53
$Q_{weir}(h_{weir})(m)$	2.31	2.31
$Qa_{MAX}(m^3/s)$	2.77	2.84

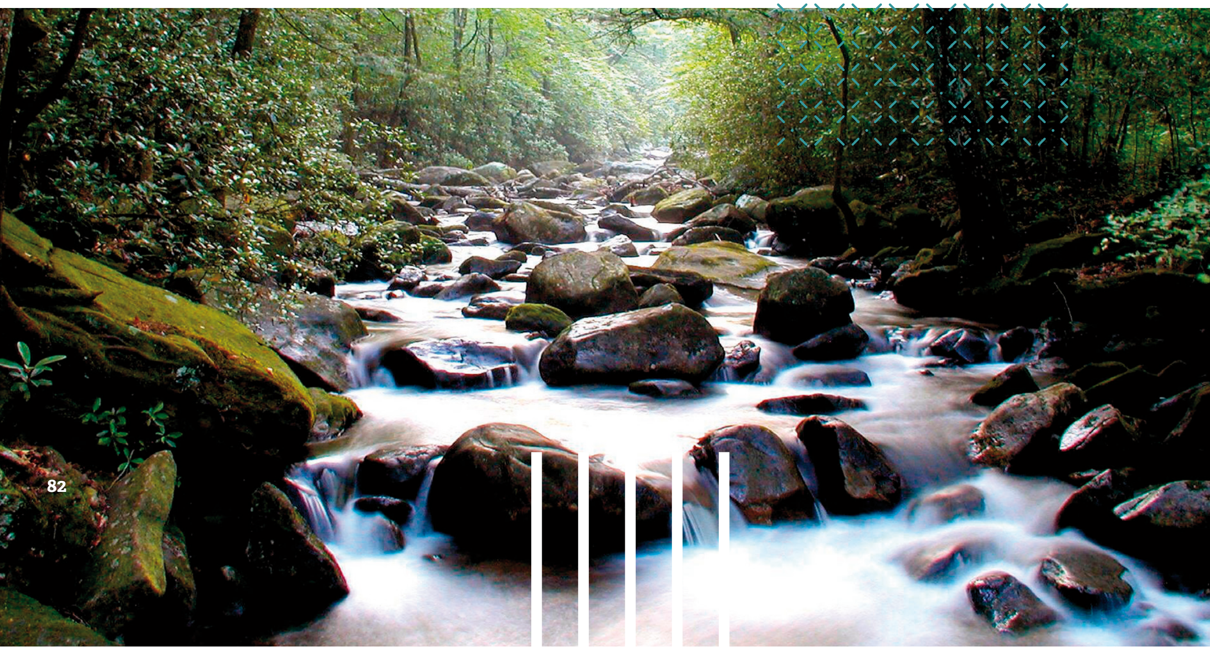
TABLE 26. Flow values which define the different ranges of functionality of the ramp for the two alternatives considered in the design (1 module without step and 2 modules with step).

5.3.8 FLOWCHART



06

PROJECT MANAGEMENT



06

Recommendations included in this Manual strictly provide support for the design and calculation phase. The project requires devising many other aspects which are out of scope of this text.

We do not intend to make a detailed relation of each single relevant aspect for preparing the ramp project, but we aim at spotting some of the most significant:

I. Boundary conditions of the work

As in any other project, it becomes essential to analyse the boundary conditions of the work, in order to consider them in all project phases –design, construction, control and maintenance. In the particular case of ramps, it is important to study with further detail those constraints related to:

- Accessibility and stocking areas.
- Flow regime.
- Phenology of target species.
- Environmental constraints (terrestrial or aquatic).
- Uses associated to the weir.

II. Civil engineering

Tasks related to demolitions, cofferdams, earth movements, reinforcing, concreting, boulder and bed disposition, side walls, stabilization of river margins, ..., must be redacted under the context of environmental issues and the singularities of the working area.

Definition of the work plan will additionally consider flow regime and weir uses.

Special attention must be given to all single elements which ensure the stability and functionality of the ramp:

- Geotextiles or gravel filters for contact between natural riverbed and the foundations or base of the ramp.
- Piping risk under the ramp, and to the elements designed to avoid it.
- Stability of boulders or prefabricated elements in the rows, considering static forces and hydrodynamic actions they will suffer.

- Stability of ramp bed versus shearing forces of design flows
- Stability of lateral walls or margins, considering thrusts of water and earth, and shearing forces exerted by water.

III. Environmental assessment

Independently of what legislation requires, the project should include an environmental assessment in which:

- Potential environmental impacts are identified, both in quantity and quality.
- The necessary preventive and corrective measures are determined.
- An environmental monitoring plan is developed, which allows verifying the application of the aforementioned measures, and their effects, and, includes protocols to detect non-expected potential impacts and to define measures to avoid or mitigate them.

IV. Monitoring

It is of higher significance that the project incorporates all necessary installations to monitor ramp effects on fish fauna. Be them elements for the continuous monitoring of fish passing –collection, reception and storage of signals for PIT tagged fishes-, or the actions required for their punctual monitoring –e.g., electrofishing-.

V. Maintenance

Ramp functionality may be reduced by obstructions (woody debris, litter) in flow gaps. Thus, it is crucial that the project includes technical and financial considerations for the ramp maintenance (periodic and punctual maintenance – this last associated to extreme hydrological events-).

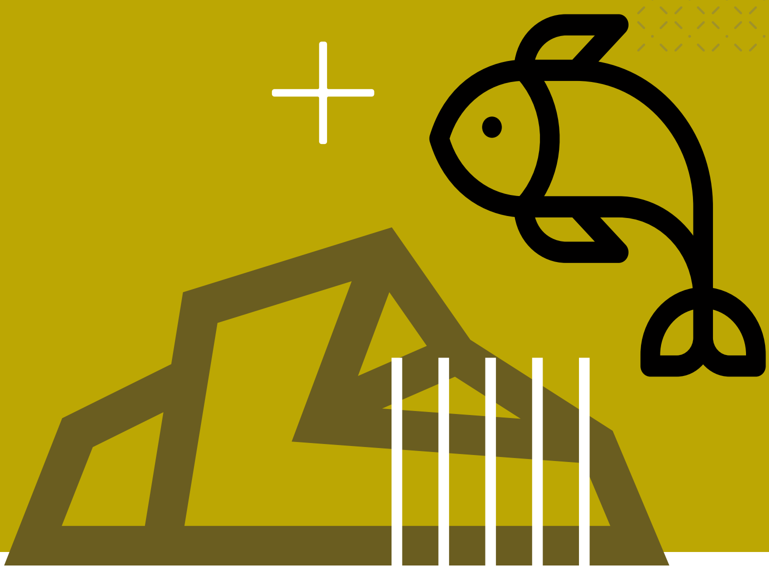
Maintenance must also be fulfilled in the outfall pool. Its filling with sediments or the obstruction of the flow gaps which connect it with the channel may lead to the modification of water depths, and then to unfavourable conditions in the ramp (see non-uniform regime section).

VI. Social awareness and communication

It is important that, before works are initiated, informative meetings are held with social stakeholders (neighbours' associations, environmental groups, schools…) in the nearby municipalities. The target of those meeting would be informing and increasing awareness about the environmental relevance of the measure, and about the manners in which the continuity of the weir uses have been devised. The project should also include informative panels, capable of explaining –*in situ*- which are the objectives of the ramp construction, and which its environmental ben.

07

PROS AND LIMITATIONS



07

PROS

DESIGN	CONSTRUCTION AND MAINTENANCE	FISH PASSAGE AND BIOTA
The handbook provides the design protocol based on solid hydraulic equations/ foundations, hydro-biological parameters and practical applications.	Modular design.	Design allows for very different requirements of depth, velocities and dissipated power, depending on the fish species.
Software is provided to test different scenarios.	Outfall pool allows smooth transition between ramp and river, also functional in non-uniform regime.	Ramp can be designed for a wide range of flows.
The tool provides different design options for the same initial situation.	Ramp can be designed to reduce the risk of blockage.	The method allows designing a ramp with a range of flows with absolute call effect (attraction flows).
Clear requirements to generate uphill flows.	Ramp can be constructed fixed to the weir body, bank or as a bypass.	Ramp is permeable to sediment transport and fauna (macroinvertebrates, etc.) and flora.
It considers design for very low flows (using steps in the flow gap).	Construction can be done using natural materials, pre-manufactured (concrete) or imported that can be naturalised in-situ.	Upstream migration for fish species with and without jumping capabilities.

It allows to work with steeper slopes compared to conventional rock ramps.	It allows to be integrated in the landscape.	Upstream migration, fish can make use of shelters and pools to rest.
Design considers environmental flows.	The functional ramp doesn't require full channel width to be implemented.	Downstream migration guaranteed.
Design considers diverted flows for different active uses.		Migration (upstream and downstream) is possible for different life stages and seasons.
		Ramp bed can be natural and used by other communities (e.g. macroinvertebrates.).

LIMITATIONS

DESIGN	CONSTRUCTION AND MAINTENANCE	FISH PASSAGE AND BIOTA
It requires a minimal width to accommodate a functional module, but no wider than a conventional rock ramp		
As conventional rock ramps it requires longitudinal space to implement the design.		
	Flows over the drowning limit do not guarantee uphill flows (but the ramp can still be functional).	The construction phase <u>must</u> follow the design rigorously, in particular placements of boulders to guarantee uphill flows.
		Special attention should be given to the selection of material, particularly if coming from quarry. Placement of each element and visual faces is very relevant.

08

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08

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A1

ANNEXES

ANNEX I
VARIABLES AND EQUATIONS

ANNEX 1: Variables and equations

Variables

SYMBOL	UNITS	DESCRIPTION	TYPE
a	m	Flow gap dimension (transversal to the flow on the ramp)	Dependent
b	m	Dimension of the flow gap (parallel to the flow on the ramp)	Dependent
Cc		Contraction coefficient	Coefficient
Cd_{gap}		Gap discharge coefficient	Coefficient
Cd_{weir}		Weir discharge coefficient	Coefficient
Db	m	Diameter of the boulder or dimension of the boulder (transversal to the ramp flow)	Independent
h_0	m	Mean depth in the pool	Dependent
h_1	m	Depth upstream of the boulder	Dependent
h_1 (Q_{RMIN})	m	Depth upstream of the boulder for a given Q RMIN	Dependent
$h_{1\ max}$ = h_1 (Q_{RMAX})	m	Max depth upstream of the boulder to guarantee the ramp is working without drowning – it corresponds to a ramp flow Q_{RMAX}-	Dependent
h_1*	m	Depth, over the step upstream of the boulder	Dependent
h_2	m	Depth downstream of the boulder	Independent
h_2^U	m	Depth downstream of the boulder in the row n corresponding to a uniform regimen for a given Q_{RMIN}	
$h_{2,n}^{no\ U}$	m	Depth downstream of the boulder in the row n corresponding to a non-uniform regime	
h_2*	m	Depth, over the step, downstream of the boulder	Dependent
h_{weir}	m	Height difference between the highest point of the boulders on the top row and the height on the weir crest	Independent
Hb	m	Boulder height (visible height)	Dependent
Hb_{min}	m	Min boulder height	Threshold reference
$Hcweir$	m	Height of the weir crest	Initial data
H_0	m	Height of the channel bed at the ramp outfall	Initial data
H_R	m	Height of the ramp. Corresponding to the height difference between the height at the inlet and outfall of the ramp bed	Dependent
H_T	m	Total height to pass	Dependent
$HWs\ min$	m	Min height of the water surface, which must be present on the upstream face of the weir, to provide the requirements of water supply in the diversion associated to the weir	Initial data

	CALCULATION PHASE					CONSTRAINTS	RECOMMENDATION
	1 Hydraulic dimensioning	2 Geometric dimensioning	3 Dimensioning to control the dissipated power	4 Generation of the functional flow range	5 Behaviour in non-uniform regime		
			x				
			x				
	x						Usual value 0.2
	x						
				x			
		x					
			x				
	x						
					x		
				x			
	x						
	x					Environment: fish passability Geometric: no blockage	Optimum: $h_2 \geq 0.2$ Acceptable: $0.1 \leq h_2 < 0.2$
					x		
					x		
	x						
		x		x		Hydraulic: no drowning of the boulder	$h_{weir} \geq 0.15$
		x					$Hb \geq Hb_{min}$
		x					$Hb_{min} = h_1 + y + 0.15$
		x					
		x					
		x					
		x					
				x			

SYMBOL	UNITS	DESCRIPTION	TYPE
HWs (Q_{RMIN})	m	Height of the water surface upstream of the ramp when circulating Q_{RMIN}	Dependent
HWs (Q_{RMAX})	m	Height of the water surface upstream of the ramp when circulating Q_{RMAX}	Dependent
HWs (Q_R(y=0))	m	Height of the water surface upstream of the ramp when circulating Q_R(y=0)	Dependent
Lp	m	Pool length	Dependent
L_R	m	Ramp length	Dependent
Lweir	m	Weir length	Initial data
Nm		Number of modules	Independent
Np		Number of pools	Dependent
Nrow		Number of rows	Dependent
p	m	Height of the step	Independent
Pd	W/m^3	Dissipated power	Dependent
Pd_{em}	W/m^3	Dissipated power through the edge module	Dependent
Pdm	W/m^3	Dissipated power through the module	Dependent
Pd_{mm}	W/m^3	Dissipated power through the mid module	Dependent
Qa	m^3/s	Available flow in the river. Corresponding to the difference between Qriver and Qdiverted	Initial data
Qa_{MAX}	m^3/s	Available flow in the river when Q_{RMAX} circulating on the ramp	Dependent
Q diverted	m^3/s	Diverted flow to attend existing uses	Initial data
Qg	m^3/s	Flow throughout the flow gap	Dependent
Q module	m^3/s	Flow throughout the module	Dependent
Q river	m^3/s	River flow upstream of the ramp	Initial data
Q_{RMIN}	m^3/s	Min flow for which the ramp is functional. This is the flow for the three first phases of dimensioning	Independent
Q_{RMAX}	m^3/s	Max flow through the ramp over which the boulders are drowned	Dependent
Q_R(y=0)	m^3/s	Flow through the ramp for y=0, or flow over which the weir starts spilling	Dependent
Q_{weir}	m^3/s	Flow discharged by the weir	Dependent
Q_{weir} (h_{weir})	m^3/s	Flow discharged by the weir when Q_{RMAX} is circulating on the ramp	Dependent
Q_{weir}(y_{weir})	m^3/s	Flow discharged by the weir when the depth upstream the crest is y_{weir}	Dependent
V_{MAX}	m/s	Max velocity throughout the flow gap	Dependent

	CALCULATION PHASE					CONSTRAINTS	RECOMMENDATION
	1 Hydraulic dimensioning	2 Geometric dimensioning	3 Dimensioning to control the dissipated power	4 Generation of the functional flow range	5 Behaviour in non-uniform regime		
				x			
				x			
				x			
			x				
			x				
				x			
	x						
		x					
		x					
		x				Hydraulic Biologic	$h_2^* = h_2 - p \geq$ Max (0.1; dorsal-ventral length)
			x				$Pd \leq 150$
			x				$Pd_{em} \leq 150$
			x				$Pdm \leq 150$
			x				$Pd_{mm} \leq 150$
				x			
				x			
				x			
	x						
			x				
				x			
	x	x	x	x			
				x			
				x			
				x			
				x			
				x			
	x			x			

SYMBOL	UNITS	DESCRIPTION	TYPE
$V^{no\,U}_{max,i}$	m/s	Max velocity throughout the flow gap corresponding to row i, for non-uniform regime and flow Q_{RMAX}.	Dependent
Vol em	m^3	Volume of the edge module	Dependent
Vol p	m^3	Pool volume	Dependent
Vol m	m^3	Module volume	Dependent
Vol mm	m^3	Volume of the mid module	Dependent
Wb	m	Width of the boulder (parallel to the flow on the ramp)	Independent
Wcweir	m	Crest width	Initial data
Wem	m	Edge module width	Dependent
Wm	m	Module width	Dependent
Wmm	m	Mid- module width	Dependent
Wramp	m	Ramp width	Dependent
WGg	m	Width of the geometric gap	Dependent
WHg	m	Width of the hydraulic gap	Dependent
y	m	Height difference between the weir crest and the water surface upstream of the boulder of the top row for Q_{RMIN}	Dependent
$y_{min\ diversion}$	m	Min depth upstream of the weir compatible with the existing diversion	Initial data
y_{weir}	m	Height difference between the water surface in the river for $Q > Q_R(y=0)$ and the weir crest	Dependent
α	degrees	Angle defining the alignment between two consecutive boulders	Independent
β	degrees	Angle (slope) of the ramp and the horizontal axis	Independent
β_0		Dimensionless coefficient for Eq. 3	Coefficient
β_1		Dimensionless coefficient for Eq. 3	Coefficient
γ	N/m^3	Specific weight of the water	
Δh	m	Water surface drop between two consecutive pools	Independent
$\Delta h,i^{no\,U}_{n}$	m	Water surface drop between two consecutive pools corresponding to the row i, for non-uniform regime and flow Q_{RMAX}	Dependent
Δz	m	Height difference of the bed between two consecutive rows	Dependent

	CALCULATION PHASE					CONSTRAINTS	RECOMMENDATION
	1 Hydraulic dimensioning	2 Geometric dimensioning	3 Dimensioning to control the dissipated power	4 Generation of the functional flow range	5 Behaviour in non-uniform regime		
					x		
			x				
			x				
			x				
			x				
		x					
				x			
			x				
			x				
			x				
			x				
	x					Environment: fish passability Geometric: no blockage	Optimum: WGg≥0.2 Acceptable: 0.1≤WGg<0.2
	x						
				x			y≥0.1
				x			
				x			α
			x				30º≤ α ≤45°
			x				
	x						β_0=0,812
	x						β_1=0,335
			x				γ=9810
	x					Biologic Geometric	Optimum: Δh ≤0.2 Acceptable: 0.2≤Δh <0.35
					x		
					x		

Equations

A2

ANNEXES

ANNEX II
"EL PARDO" RAMP

A2

This Annex details the process followed and the variables calculated during the design of "El Pardo" ramp in Madrid. This is the first uphill flow rock ramp constructed worldwide.

1 Introduction _

El Pardo ramp is one of the set of measures included in the project of restoration of the Manzanares River in The Royal Site of El Pardo (http://restauracionfluvialriomanzanares.es/).

This project was approved in 2016, and was conducted by the General-Directorate of Water of the Spanish Ministry for the Ecological Transition, and the Tagus Basin Agency. The project was fulfilled in the context of the PIMA Adapta Plan (Plan of Enhancement of the Environment for the Adaptation to Climatic Change in Spain).

The project area comprised the river channel, from the toe of El Pardo dam until its entrance in Madrid (crossing with the M40 highway). The total length of the project area was 8.4 km in the Manzanares mainstem, and an additional 7.0 km in La Trofa stream, main tributary of Manzanares from the right margin of the river.

The initiative included many different types of measures (rehabilitation of river habitats, improvement of water quality, recovery of riparian ecosystem, hydromorphological processes, and public use, etc.). One major milestone was the construction of a fish ramp in a weir known as "Golf pitch weir" or "El Pardo weir". The aforementioned ramp was designed by the authors of this Manual, and constructed by early 2019 by the firm TRAGSA.

FIGURE 28 shows the location of El Pardo weir, whose coordinates are ETRS89 UTM H30 N (X: 433,856.86 Y: 4,486,220.33).

FIGURE 28
Location of El Pardo weir: general view of the Manzanares river reach where the weir was constructed (left). Location detail (right).

The weir was designed to supply water to a fire water tank located in El Pardo Protected Area. *FIGURE 29* shows an image of the weir in its original condition. The intake gate and the building with the pumping system can be seen in the left margin of the photograph.

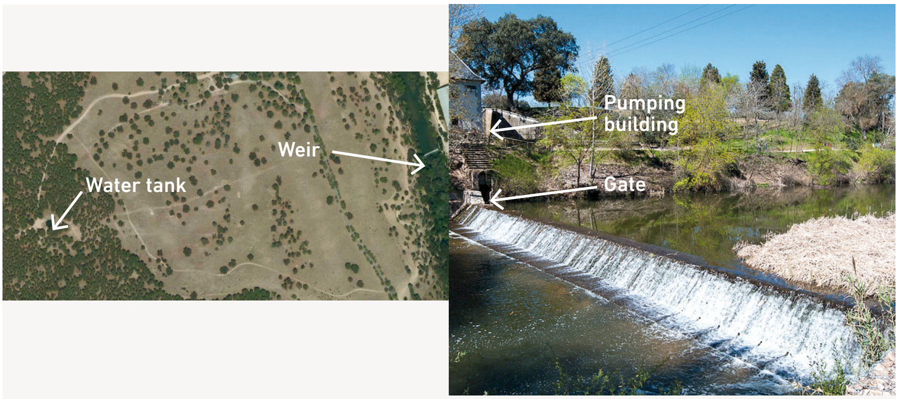


FIGURE 29
Previous condition of El Pardo weir (right) and relative location of fire water tank and pumping building (left)

2 Targets and proposed solution _

The restoration project of the Manzanares River in the surroundings of the Royal Residence of El Pardo (Madrid) considered, for this weir, a two-fold action and a main contextual constraint:

- Target 1: allow fish passing through the artificial barrier
- Target 2: reduce the pool generated upstream by the weir (its length was estimated in 1.1 km)
- Constraint: keep the functionality of the water outlet

The selected solution was also two-fold, and able to give answer to the aforementioned constraint:

- Solution 1: construct **a rock ramp to reduce the barrier effect**. Its design is explained along the following sections.
- Solution 2: lower the height of the weir crest, in order to mitigate the pool effect of the weir. As a result of the hydromorphological analysis, the required height **reduction of the crest was estimated in 0.5 m**, over a length of 7.2 m.
- Constraint: the functionality of the outlet was ensured, improving the existing installation. The reduction of the crest height enhanced the modification of the height of the diversion outlet, aimed at the maintenance of its full functionality.

  HWs min = Hcweir – 0.37m

  where *Hcweir* is the height of the weir crest, once this is lowered.

FIGURE 30 shows the present location of the weir. *FIGURES 31* and *32* show the previous and present condition of the pool generated by the weir.

FIGURE 30
Present condition of the fish ramp (2019), and the lowered height of the weir crest

FIGURE 31
UAV-operated image (2019), where the pool upstream of the weir may be seen, along with the newly emerged islands.

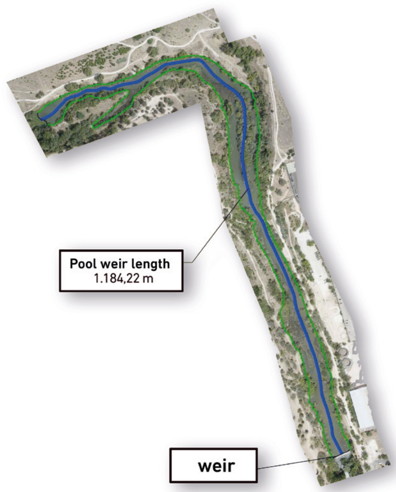


FIGURE 32
UAV-operated image (2017), where the previous condition of the weir pool may be seen.

3 Conditioning factors for the ramp design _

The main conditioning factor has already been explained:

- The existing water abstraction must be maintained for the new height of the weir crest, in a combined way with updated flow diversion and pumping facilities (lateral channel, gate, suction foot valve, ...). Nonetheless, since water diversions are very punctual and scarce (just during the fire campaign, and sometimes strictly fulfilled for the initial filling of the tank), the diverted flow will not be considered during the estimation of the available flow in the river.

Other design constraints of the adopted solution are:

a) Location of the lowering operation: the weir crest lowering must be done close to the ramp, in order to contribute to the fish call effect when the new spillway is active with flow.

b) Barbels were selected as target species in the ramp[16].

c) Functional period: the ramp was designed to be functional during the pre-reproductive period for cyprinids (April - June).

d) The river reach has a regime of minimum flows according to the present RBMP- River Basin Management Plan (see *TABLE 27*).

TABLE 27. Minimum flows defined –each trimester-, in m^3/s, for the planning cycle 2015-2021 in the water body "Manzanares River, from El Pardo dam to La Trofa stream" (BOE 89, 2014)

Minimum flows (m^3/s)			
Oct-Dic	Jan-Mar	Apr-Jun	Jul-Sep
0.82	0.93	0.97	0.49

e) The range of available flows is very limited, since it is a heavily regulated river reach, located downstream of Santillana and El Pardo reservoirs, and immediately upstream of downtown Madrid. These facts sustain the heavy regulation of high flows in the river.

f) Ramp location: access limitations of machinery during the works determined ramp location in the right margin, in the vicinity of the new diversion lade for the pumping outlet.

g) It was also considered necessary to relocate the ramp in the weir, due to the morphological singularities of the reach downstream of the fish passage. This circumstance reinforces the call effect of the attraction flow to fishes, since the ramp outfall is close to the weir spillway.

h) The existence of an apron downstream of the weir required the installation of a line of boulders outside it (known as control row), which could redirect the flow crossing the weir spillway, and improved the call effect of the attraction flow to fishes.

i) The intense public use of the river reach confers an educational and social awareness dimension to the ramp. As such, certain additional actions could be devised and carried out (gazer, informative panels, etc.).

[16] *The preliminary analysis of fish communities conducted for the restoration project highlighted, as dominant native species, a range of cyprinids, such as common barbel (Luciobarbus bocagei) and gudgeon (Gobio lozanoi). Also different alien species were identified, such as the cyprinid common bleak (Alburnus alburnus), the centrarchids pumpkinseed (Lepomis gibbosus) and black-bass (Micropterus salmoides), the ictalurid cat fish (Ameiurus melas), and the poeciliid mosquitofish (Gambusia holbrooki).*

FIGURE 33 shows the planform view of the constructed fish ramp.

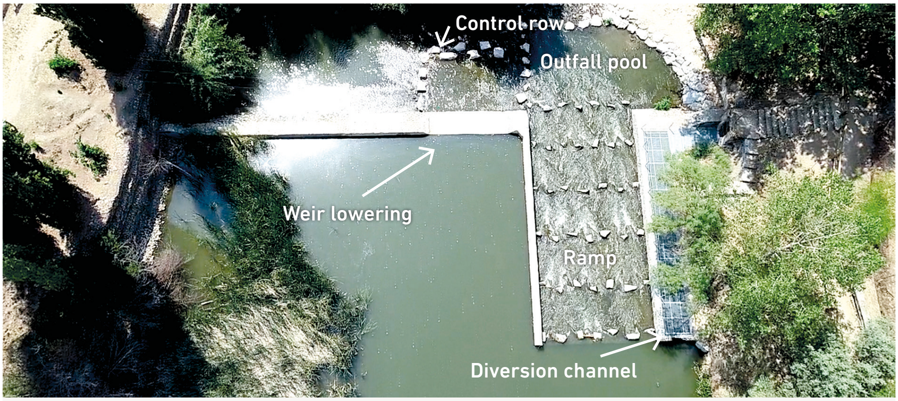


FIGURE 33
Aerial view of the ramp in operation (UAV image, 2019)

4 Ramp design _

PRELIMINARY PHASE: ESTIMATION OF Q_{RMIN}

Estimation of design flow Q_{RMIN} –following the requirements of this Manual- was undertaken by analyzing the range of flows during the functional period of the ramp.

The river reach is located in a water body in which a minimum flow regime is defined by the RBMP, and is compulsory in all senses. Information provided by the Tagus Basin Agency allowed the characterisation of the present flow pattern. For the functional period of the ramp, flows released by El Pardo dam are quite bigger than the minimum flows shown in *TABLE 27*. On this basis, the design flow was defined as **1.15 m³/s.**

All other input data are summarised in the following chart:

Original data	• Maximum velocity: 2.2 m/s⇨$\Delta h \leq 0.25$m • Minimum depth: 0.2m⇨$h_2 \geq 0.2$m • Width of the geometric gap: 0.2m⇨$WGg \geq 0.2$m • Maximum dissipated power: 200w/m³⇨$Pd \leq 200$W/m³ • Weir length (***Lweir***): 7.2m (lowered portion) • Crest width (*Wcweir*): 1m • Height of the channel bed at the ramp outfall (H_0): 599.00 m.a.s.l. • Height of weir crest (***Hcweir***): 600.15 m.a.s.l. • Height difference between the highest point of the top row of boulders and ***Hcweir:*** h_{weir}= 0.25m

PHASE 1. HYDRAULIC DIMENSIONING

INPUTS:

- Q_{RMIN}=1.15m^3/s
- WGg_{min}=0.2m
- This phase is initiated by considering the construction of 3 modules, and flow gaps without step (*p=0*)

RESULTS:

Combinations of (h_2; Δh) are calculated, which offer values of $WGg \geq WGg_{min}$, selecting Δh=0.15m, h_2= 0.2m and WGg=0.5 m

PHASE 2. GEOMETRIC DIMENSIONING

INPUTS:

- Height of the channel bed at the ramp outfall: H_0=599 m.a.s.l.
- Height of the lowered weir crest: *Hcweir*= 600.15 m.a.s.l.
- Height difference between the highest point of the boulders on the top row and the height on the weir crest: h_{weir}=0.25m

RESULTS:

✓ Hydraulic gap width: *WHg*= 0.47 m

✓ Height difference between weir crest and height of water surface upstream of the ramp:

y= 0.05m

✓ Depth upstream of the boulder: h_1=0.35m

✓ Total height to pass: H_T= 1.15m

✓ Ramp height: H_R= 0.75m

✓ Number of rows: *Nrow*= 6

✓ Number of pools: *Np*= 5

✓ Minimum boulder height: Hb_{min} = 0.65m

PHASE 3. DIMENSIONING OF DISSIPATED POWER

INPUTS:

- *Hb* is selected = 0.80 m
- Boulder diameter in the transversal dimension to the flow: *Db* =0.80m
- Boulder width: *Wb* =0.60m
- α=30°

RESULTS:

- ✓ *Wem*= 2.8m
- ✓ *Wmm*= 2.4m
- ✓ *Wramp*= 8.2m
- ✓ $tg\beta$= 0.04
- ✓ L_R= 19 m
- ✓ *Lp*= 3.7 m
- ✓ *Vol em*=2.9 m^3
- ✓ *Vol mm*= 2.5 m^3
- ✓ Pd_{em}= 191 W/m^3
- ✓ Pd_{mm}= 222 W/m^3 (see footnote[17])
- ✓ V_{MAX}= 1.72 m/s

PHASE 4. FUNCTIONAL RANGE OF FLOWS

a) **Calculation for Q_R(y=0)**

INPUTS:

- *Hb*= 0.8m
- y_{weir}= 0.25m
- Δh =0.15m
- *Nm* =3
- $tg\beta$ =0.04

RESULTS:

- ✓ $h_1[Q_R(y=0)]$= 0.40m
- ✓ $h_2[Q_R(y=0)]$=0.25m
- ✓ $Q_R(y=0)$ =1.35 m^3/s
- ✓ Pd_{em}= 190 W/m^3
- ✓ Pd_{mm}= 221 W/m^3 (See footnote [17])

[17] *Dissipated power means an indirect estimation of turbulences and of air incorporation they generate – normally referred in specialised literature as “white waters”-. Presence of those “white waters” is a limiting factor for the passability of fish passages. In the protocol of calculation presented in this document, dissipated power is quantified for the whole module. However, for uphill flow ramps, “white waters” are logically concentrated on those areas of higher velocity, while in backflow areas their presence is much scarcer; those last zones will be selected by fishes for upstream passing. Thus, it is reasonable to assume that the value of the maximum dissipated power (Table 1) will be, for this type of ramps, higher than that present in fish passages where turbulences appear more homogeneously.*

b) **Calculation for Q_{RMAX}, $Q_{weir}(h_{weir})$ y Qa_{MAX}**

INPUTS:

– For the ramp, same than previously used for $Q_R(y=0)$

– For the weir:

- Crest width: *Wcweir*=1m
- Weir length: *Lweir*=7.2m

RESULTS:

- ✓ $h_1(Q_{RMAX})$= 0.8m
- ✓ $h_2(Q_{RMAX})$= 0.65m
- ✓ Q_{RMAX}= 3.1 m^3/s
- ✓ Q_{weir} (h_{weir}) =2.4 m^3/s
- ✓ Qa_{MAX}= 5.5m^3/s
- ✓ Pd_{em}= 195 W/m^3
- ✓ Pd_{mm}= 226 W/m^3 (See footnote [17])

Flow range for a functional ramp is summarised in *TABLE 28*.

TABLE 28. Flow range for which the ramp is functional

SCENARIO	LOCATION OF THE DISCHARGE	RANGE OF FLOWS *Qa*	VALUES GENERATED (m^3/s)
1	Ramp only	$Q_{RMIN}\le Qa\le Q_R(y=0)$	Q_{RMIN}= 1.15 $Q_R(y=0)$ =1.35
2	Ramp and weir No drowning of boulders	$\underbrace{Q_R(y=0)}_{RAMP}<Qa\le \overbrace{\underbrace{Q_{RMAX}}_{RAMP}+\underbrace{Q_{weir}(h_{weir})}_{WEIR}}^{Qa_{MAX}}$	Q_{RMAX}= 3.1 $Q_{weir}(h_{weir})$=2.4 Qa_{MAX}=5.5

PHASE 5. VERIFICATION IN NON-UNIFORM REGIME

INPUTS:

– The following hypothesis is considered for the depth downstream of the bottom row:

$$h_{2,n}^{no\,U}(Q_{RMAX}) = 0.6 - \ 0.8 * \ h_2^U(Q_{RMAX})$$

where $h_2^U(Q_{RMAX})$ = 0.65 m and defining a 70% reduction, its value would be

$$h_{2,n}^{no\,U}(Q_{RMAX}) = 0.455\ m$$

RESULTS:

TABLE 29 shows the resulting values for $h_{2,i}^{no\,U}(Q_{RMAX})$, $h_{1,i}^{no\,U}(Q_{RMAX})$, Δh_i y $V_{MAX,i}$ (i=1...n). All estimated velocities are within the required range for the target species.

TABLE 29. Results of verification for the non-uniform regime

Row	h_2 (m)	h_1(m)	Δh(m)	v (m/s)
6	0.455	0.705	0.250	1.95
5	0.555	0.748	0.193	1.83
4	0.598	0.769	0.171	1.78
3	0.619	0.781	0.162	1.76
2	0.631	0.789	0.158	1.76
1	0.639	0.793	0.154	1.74

SUMMARY OF VARIABLES

TABLE 30 summarised the range of variables defined for the study ramp.

TABLE 30. Summary of design variables

VARIABLES	VALUE
Number of modules	3
p (m)	0
h_2 (m)	0.2
Δh (m)	0.15
h_{weir} (m)	0.25
V_{MAX} (m/s)	1.7
WGg (m)	0.5
WHg (m)	0.47
h_1 (m)	0.35
y (m)	0.05
H_T (m)	1.15
H_R (m)	0.75
Nrow	6
Np	5
Hb (m)	0.8
Db (m)	0.8
Wb (m)	0.60
α (°)	30
tgβ	0.04
Wem (m)	2.8
Wmm (m)	2.4

Wramp (m)	8.2
Lp (m)	3.7
L_R (m)	19
Pd_{em} (W/m^3)	191
Pd_{mm}(W/m^3)	222
Q_{RMIN} (m^3/s)	1.15
Q_R (y=0)(m^3/s)	1.35
Q_{RMAX} (m^3/s)	3.1
Q_{weir} (h_{weir}) (m^3/s)	2.4
Qa_{MAX} (m^3/s)	5.5

5 Other recommendations _

a) Outfall pool

The design of the outfall pool (*FIGURE 34*) was based on the following recommendations – compiled in this Manual:

- The length was 1.5-2 times the length of the ramp pools (*Lp*=3.7m): it was established as 6 m.
- Horizontal bed was set 0.3 meters below the riverbed.
- The pool perimeter was delineated with boulders which, in their top height reach, at least, the height of the water surface h_2^U of the bottom row for $Q_R(y=0)$= 0.25m. In this case, boulders with a height of 0.5 m were used.
- In 50% of flow gaps in this pool, flow capacity was reduced between 20 and 30%. Reduction was reached by diminishing gap width in that percentage, in relation with the width of the ramp gaps. In this case, gap widths of 0.45m were defined in the pool edges closer to the spillway, in order to enhance the call effect of attraction flows in this zone.

FIGURE 34
Outfall pool in the ramp

b) Control row

As previously indicated, a control row was required in the ramp (*FIGURE 35*). And, more particularly, in the concrete stab located downstream of the weir, in the spillway zone. The objective of the control row was redirecting the spillway flow, and contributing to the call effect of attraction flows. Boulders of quite larger size to those used in the outfall pool were used, and with wider gaps.

FIGURE 36 shows an UAV image of the ramp, where outfall pool and control row may be easily identified.

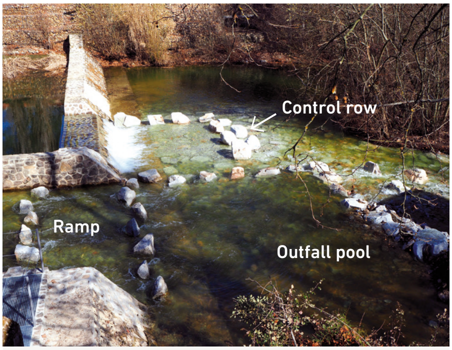


FIGURE 35.
View of control row

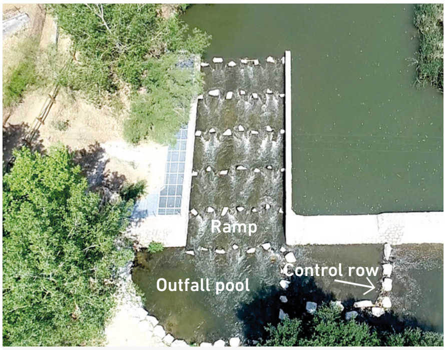


FIGURE 36
Aerial view of ramp and weir in their present condition

c) Water intake construction

A lateral channel was constructed for the water intake, with a regulatory gate at the entrance, and a protective grating (*FIGURE 37*).

The suction pipe was also replaced, increasing its length at the new diversion height.

FIGURE 37
View of diversion channel (left) and regulatory gate (right)

d) Social relevance

Public use was favoured by placing, in the right margin of the river, a gazer with informative panels, which could maximise social, educational and awareness services of the fish ramp.

e) Environmental integration

The ramp bed (*FIGURE 38*) was naturalised with sediments of heterogeneous size. Lateral walls were covered with rocks (*FIGURE 39*).

FIGURE 38. Naturalised bed of the ramp

FIGURE 39. Ramp view, with the lateral walls covered with rocks

f) **Monitoring**

Finally, it is relevant to mention that the ramp functioning is being monitored for different flow values, in order to increase knowledge about its success, verify calculations done during the design, and extrapolate results to other future fish ramps (*FIGURE 40*).

FIGURE 40
Monitoring campaign for the estimation of depths and velocities

6 Photographic report during construction phase _

FIGURE 41. Construction of ramp access in the right margin of the Manzanares River

FIGURE 42
Partial demolition of the weir, aimed at pool emptying and cofferdam construction

FIGURE 43
Detail image of ramp base construction

FIGURE 44
Placement of boulders in the ramp

FIGURE 45
Verification of relative location of boulders within the row

FIGURE 46
Ramp bed concreting

FIGURE 47
Covering ramp bed with rocks

FIGURE 48
Ramp opening with the technical and construction team (left to right, J. Carpio, M. Oliva and L.C. Arias)

FIGURE 49
Initial phases of construction of outfall pool

FIGURE 50
Placement of elements in control row

FIGURE 51
Ramp view for Q_{RMIN} scenario

FIGURE 52
Ramp view for (Q_{RMIN} <Q<Q_R (y=0)) scenario

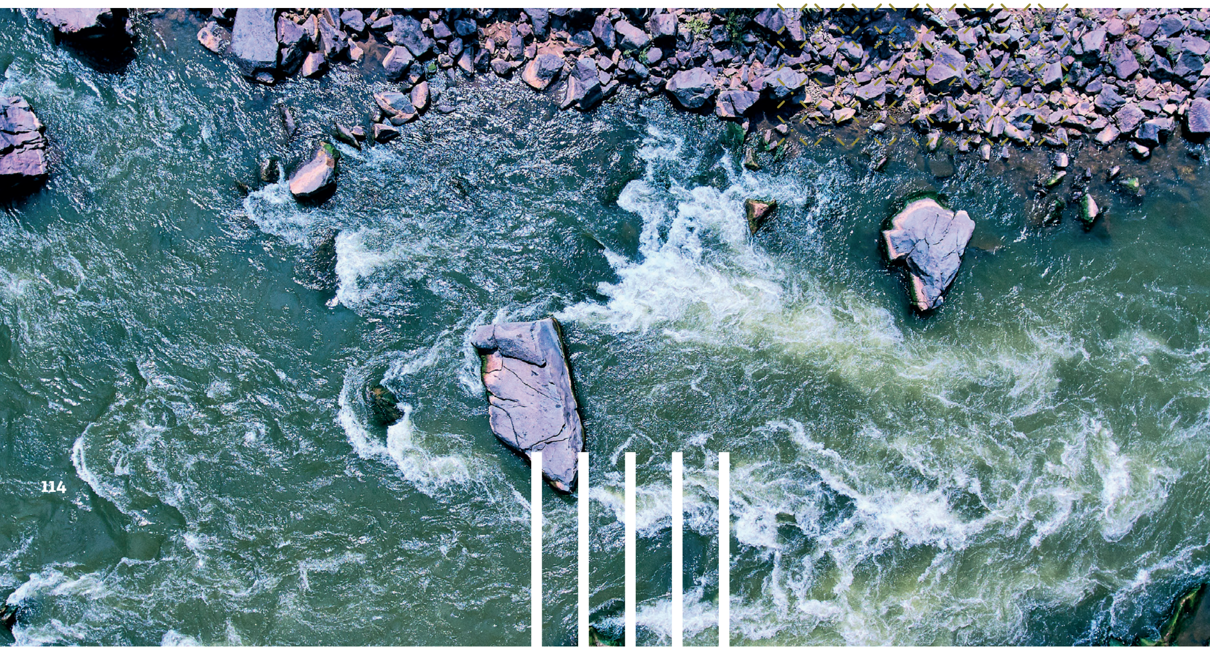
FIGURE 53
Ramp view for Q> Q_R (y=0) scenario

A3

ANNEXES

ANNEX III

ASSISSTANT FOR THE DESIGN OF **UPHILL FLOW ROCK RAMPS**
BRIEF GUIDE Versión 1.2



Upflow Ramps

A3

What is RAMPS? _

RAMPS is a **software** which allows the calculation of uphill flow rock ramps, and more specifically:

- Characterize their hydraulic functioning, assessing those variables which more critically affect fish connectivity (depth upstream and downstream of the boulder, max velocity throughout the flow gap and mean depth in the pool).
- Fulfill the geometric design of the ramp, defining base variables such as ramp width, ramp length, pool length, number of rows, number of pools and number of modules).
- Select optimum ramp slopes according to the power to be dissipated in the pool.
- Fulfill the geometric design of the boulder (size and angle between contiguous boulders).
- Define ramp features with no limitation of other weir uses or functionalities.
- Analize their behaviour in non-uniform regime. It allows determining the effect of the water surface level at the ramp outfall on water depths and velocities in the ramp.
- Determine the functional range of flows by assessing three values: i) flow for which the ramp is functional. -Q_{RMIN}-; ii) Flow through the ramp for y=0, or flow over which the weir starts spilling -Q_R(y=0)-; y iii) Max flow through the ramp over which the boulders are drowned - Q_{RMAX} -.

Other tips:

- To guarantee fish connectivity, RAMPS defines threshold scores for some variables (min depth downstream of the boulder h_{2min}= 0.1 m , and max velocity throughout the flow gap v_{MAX}= 2.6 m/s corresponding to a water surface drop between two consecutive pools Δh= 0.3m).
- The user may also set the minimum value of the width of the geometric gap (WGgmin).

How may RAMPS be of help?

- By offering managers and ecologists a support software for the early design of uphill flow rock ramps.
- By optimizing the design of such fish ramps throughout the simulation of different geometries (width, length, slope, size and location of boulders, etc.), which allows their adaptation to a wide array of surrounding conditions.
- By providing scientists with a new tool based on further knowledge about the behaviour of uphill flow ramps; particularly regarding the links between hydraulic and geometric variables and their relevance for fish connectivity.

How can RAMPS be downloaded?

- The software is available at https://ramps.insolubilia.xyz/dashboard/

For further information about RAMPS

- RAMPS follows the methodology included in the "Manual for the design and calculation of uphill flow rock ramps" available at: https://www.chduero.es/documents/20126/427605/ManualDisenoCalculoRampas.pdf

- RAMPS users are encouraged to previously consult the Manual to understand the calculation sequence and variables. Annex 1 of the Manual offers a table where variables are listed accordingly with their symbols, units, definition and phase of utilization.

How should RAMPS be used?

REQUESTED DATA

The user will need to include the following data in the software –all data must be added in the International System and using "." as decimal separator -:

	GEOMETRIC DATA OF THE RAMP	See the Manual
WGgmin(m)	Min width of the geometric gap	Chapter 4.2
α (°)	Angle defining the alignment between two consecutive boulders	Fig. 2 and 13
Nm	Number of modules	Chapter 2 Fig.7
p(m)	Height of the step	Chapter 4.5 Fig.19
H_0(m)	Height of the channel bed at the ramp outfall	Chapter 4.4 Fig.17
	GEOMETRIC DATA OF THE BOULDER	**See the Manual**
Db(m)	Diameter of the boulder or dimension of the boulder transversal to the ramp flow	Chapters 2 and 4.2 Fig.1
Wb(m)	Width of the boulder (parallel to the flow on the ramp)	
	GEOMETRIC DATA OF THE WEIR	**See the Manual**
Wcweir (m)	Crest width	
Lweir (m)	Weir length	
Hcweir(m	Height of the weir crest	Chapter 4.4 Fig.17
h_{weir}(m)	Height difference between the highest point of the boulders on the top row and the height on the weir crest	

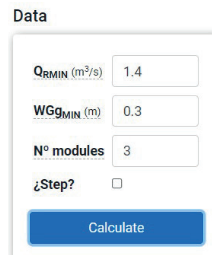
HYDRAULIC DATA		See the Manual
$Q_{RMIN}(m^3/s)$	Min flow for which the ramp is functional	Chapter 5.2 Fig. 20
$h_{2,n}^{no\ U}$ user(m) (optional)	Depth downstream of the boulder in the row n corresponding to a non-uniform regime	Chapter 5.3.5 Fig.23

CREATE PROJECT

- Give a name and description to the project. They both will show up in the heading of the report downloaded by the user when the software ends the calculation sequence.
- Create a project

PHASE 1

- Add the requested **DATA** and click **Calculate** .
- The software shows in a table the values [**Δh; h_2; WGg**] associated to the abovementioned data.
- The user may modify **DATA**, and the table updates after pressing **Calculate**.
- The user must select a value of **WGg** in the table. The software shows the selected values [**Δh; h_2; WGg**].

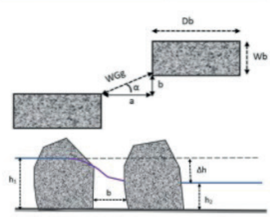


Results nº1a Values of [Δh (m); h_2 (m); WGg (m)] compatible with data

	h_2 (m)				
Δh (m)	0.2	0.25	0.3	0.35	0.4
0.1	0.77	0.64	0.55	0.48	0.42
0.12	0.68	0.57	0.49	0.43	0.38
0.15	0.57	0.48	0.42	0.37	0.33
0.17	0.51	0.44	0.38	0.34	0.3
0.2	0.45	0.39	0.34	0.3	
0.22	0.41	0.36	0.31		
0.25	0.36	0.32			
0.3	0.3				
0.32					
0.35					

Results nº 1b: Select values [Δh (m); h_2 (m); WGg (m)]

h_2 (m)	0.3
Δh (m)	0.2
WGg (m)	0.34



PHASE 2

- Add the requested **DATA** and click **Calculate** .
- The software offers the results obtained.
- If the user wishes to modify the hydraulic results of Phase 1, this phase may be revisited to select another value of **WGg**. The hydraulic data for phase 2 are then automatically updated.

- Geometric data may be modified; after pressing **Calculate** the results become updated.
- If due to the availability of materials or due to constructive reasons the user wishes a specific value of **Hb**, he would only have to repeat the process, entering as $\mathbf{h_{weir}}$ the value corresponding to the expression: $\mathbf{h_{weir} = Hb-(h_1+y)}$, where $\mathbf{h_1}$ and **y** are the values obtained above. Once **Hb** is introduced, and after pressing **Calculate** again, the results of this Phase will be obtained.
- If the weir has other functionalities, for instance water abstraction, it must be checked that the water surface level for $\mathbf{Q_{RMIN}}$ extracted from this phase $\mathbf{[HWs(Q_{RMIN})]}$ is higher that the minimum water surface level required for the abstraction.

Data

Hydraulic

h_2 (m)	0.3
Δh (m)	0.2
WGg (m)	0.34

Geometric

Ce (m)	599
Hcweir (m)	602.25
h_{weir} (m)*	0.15

Calculate

Results nº2

Hydraulic

h_1 (m)	0.50
y (m)	0.15
V_{MAX} (m/s)	1.98
HWs(Q_{RMIN}) (m)	602.1

Geometric

H_T (m)	3.25
H_R (m)	2.60
nº rows	14
nº pools	13
Hb (m)*	0.80
WHg (m)	0.34

PHASE 3

- Add the requested **DATA** and click **Calculate**.
- Data may be modified. Results are renewed by pressing **Calculate**.
- The user must pre-select a slope for the ramp –the most frequent criterion takes into account the values of dissipated power -**Pd**-.
- The user will have to re-assess the ramp length with the slope value calculated in this phase, on the basis of the height of the channel bed at the ramp outfall $\mathbf{(H_0)}$ –added as geometric data in Phase 2-. Considering the results, the user can confirm or modify the value of the variables.

Data

Db (m)	0.75
Wb (m)	0.65
α (°)	40

Calculate

Results nº 3a:

a (m)	0.26	Width of modules	
b (m)	0.22	Wm_{edge} (m)	2.39
W_{ramp} (m)	6.81	Wm_{mid} (m)	2.02

Results nº 3b

			Edge module		Mid module	
tgβ	L_R (m)	Lp (m)	Vol m (m³)	Pd (W/m³)	Vol m (m³)	Pd (W/m³)
0.1	26	2	1.92	478	1.62	567
0.09	28.89	2.22	2.13	431	1.79	510
0.08	32.5	2.5	2.39	382	2.02	453
0.07	37.14	2.86	2.74	334	2.31	396
0.06	43.33	3.33	3.19	287	2.69	340
0.05	52	4	3.83	239	3.23	283
0.04	65	5	4.79	191	4.04	227

PHASE 4a

- It is not necessary to add data.
- Firstly -4a.1-, the resulting values for flow and depth can be visualized, for **Q_R(y=0)** - Flow through the ramp for y=0, or flow over which the weir starts spilling -.
- For that flow value, the results 4a.2 show the ramp features for different slopes. The slope selected in the previous phase is highlighted. If the user does not accept the results for that slope value and **Q_R(y=0)**, Phase 3 must be revisited in order to select other slope value. Automatically, the new slope value becomes highlighted as part of the results of this Phase.
- Once the results are accepted, the following Phase becomes activated.

Hydraulic data

Hb (m)	0.80
h_{weir} (m)	0.15
Nº modules	3
tgβ	0.06

Geometric data

Δh (m)	0.2

* These values have been set in previous phases. If you want to change them, you must go to the corresponding phase and update the results.

Results nº 4a.1

Q_R(y=0) (m^3/s)	1.92
$h_2[Q_R(y=0)]$ (m)	0.45
$h_1[Q_R(y=0)]$ (m)	0.65
$h_0[Q_R(y=0)]$ (m)	0.55

Results nº 4a.2

			Edge module		Mid module	
tgβ	**L_R** (m)	**Lp** (m)	**Vol m** (m^3)	**Pd** (W/m^3)	**Vol m** (m^3)	**Pd** (W/m^3)
0.1	26	2	2.63	476	2.22	565
0.09	28.89	2.22	2.92	429	2.47	509
0.08	32.5	2.5	3.29	381	2.78	452
0.07	37.14	2.86	3.77	333	3.18	395
0.06	43.33	3.33	4.39	286	3.7	339
0.05	52	4	5.27	238	4.44	282
0.04	65	5	6.59	191	5.55	226

PHASE 4b

- Add the requested **DATA** and click **Calculate** .
- The software shows the results for flows and depths -4b.1- when the flow in the ramp is **Q_{RMAX}** -Max flow through the ramp over which the boulders are drowned - and the flow through the weir spillway is **$Q_{weir}(h_{weir})$**.
- The table with the **DATA** can be modified. Results become renewed after pressing **Calculate** .
- For **Q_{RMAX}**, the results 4a.2 show ramp features for a range of slope values. The slope value selected in Phase 3 is higligthted. If the user does not accept the results for that slope value and **Q_{RMAX}**, Phase 3 must be revisited for selecting another slope. Automatically, the new slope value turns highlighted as part of the results 4a.2 and 4b.2.

Weir data

Crest width (m)	0.75
Weir length (m)	23.5

Calculate

Results nº 4b.1

Q_{RMAX} (m^3/s)	2.46
h_2 (Q_{RMAX}) (m)	0.60
h_1 (Q_{RMAX}) (m)	0.80
h_0 (Q_{RMAX}) (m)	0.70
Q_{weir}(h_{weir}) (m^3/s)	1.79
Qa_{MAX} (m^3/s)	4.25

Results nº 4b.2

			Edge module		Mid module	
tgβ	**L_R** (m)	**Lp** (m)	**Vol m** (m^3)	**Pd** (W/m^3)	**Vol m** (m^3)	**Pd** (W/m^3)
0.1	26	2	3.35	480	2.83	569
0.09	28.89	2.22	3.72	432	3.14	512
0.08	32.5	2.5	4.19	384	3.53	455
0.07	37.14	2.86	4.79	335	4.04	398
0.06	43.33	3.33	5.58	288	4.71	342
0.05	52	4	6.71	240	5.66	284
0.04	65	5	8.38	192	7.07	227

PHASE 5

- Add the requested **DATA** and click **Calculate** .

Calculation hypothesis

Lp (m)	3.33
Standard hypothesis 1: $h_{2,n}$ (m) **= h_2[Q_R (y=0)]**	0.45
Standard hypothesis 2: $h_{2,n}$ (m) **= 0.8 * h_2(Q_{RMAX})**	0.48
User hypothesis: $h_{2,n}$ (m) **user**	optional Must be lower than h_2(Q_{RMAX})

Calculate

Results nº 5

Standard hypothesis 1: h_2 = 0.45 m

n	**X** (m)	**h_2** (m)	**h_1** (m)	**Δh** (m)	**v_{MAX}** (m/s)
14	43.29	0.45	0.73	0.28	2.36
13	39.96	0.53	0.77	0.23	2.14
12	36.63	0.57	0.78	0.22	2.06
11	33.3	0.58	0.79	0.21	2.02
10	29.97	0.59	0.79	0.2	1.98
9	26.64	0.59	0.79	0.2	1.98
8	23.31	0.59	0.79	0.2	1.98
7	19.98	0.59	0.79	0.2	1.98
6	16.65	0.59	0.79	0.2	1.98
5	13.32	0.59	0.79	0.2	1.98
4	9.99	0.59	0.79	0.2	1.98
3	6.66	0.59	0.79	0.2	1.98
2	3.33	0.59	0.79	0.2	1.98
1	0	0.59	0.79	0.2	1.98

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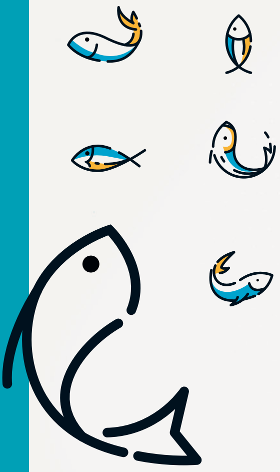
UPHILL FLOW
ROCK RAMPS
DESIGN - HANDBOOK

The reinstatement of the longitudinal continuity of our rivers must be a compulsory commitment that the society has to face, and the engineering must contribute providing more efficient solutions.

With this aim and within the framework of the CIPRIBER Project, this handbook is a technical tool for fluvial practitioners, professionals and river management planners to design a new type of device, different to conventional rock ramps: “uphill flow rock ramps”.



About the authors:

José Anastasio Fernández Yuste and Carolina Martinez Santa-María are Forest Engineers professors and lecturers in the Technical University of Madrid. They have been contributing to the hydraulic and fluvial training of many generations of engineers, they have collaborated with river managers and ecologists, taking part in projects, conferences, scientific workshops, and providing the results of those collaborations in a wide number of papers and books. Roberto Martínez and Fernando Magdaleno have extensive experience working with fluvial processes and river restoration projects.

This handbook reflects the permanent learning spirit of the authors; this text comes to life with the hope of incorporating in a systematic way, the lessons learned in our rivers.

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