APPLICABILITY OF THE COSH-TOOL TO ADDRESS THE HYDROPEAKING IN SMALL HYDROPOWER SCHEMES LOCATED MAINLAND PORTUGAL

Marta de Sousa Paiva e Trincão

October 2018

Abstract: Hydropower plants can cause unnatural rapidly fluctuations of flow/water level downstream tailrace as a result of the adaptation to the electric energy market demand. This phenomenon is known by hydropeaking, which can disturb fish species and macroinvertebrates activities. In order to minimize the negative impact of this phenomenon, the magnitude, the frequency and the duration of those peak events need to be analysed.

The present study intends to analyse Bragado hydropower plant (HPP) (small hydropower plant) operation scheme in order to analyse the hydropeaking, taking into account the existing literature. Due to the inexistence of hydrological data at the Avelames River, a regionalization model for mainland Portugal was applied to obtain the natural inflows to HPP. Afterwards, a model that simulates the operational rules of Bragado HPP was developed in order to obtain a continuous subdaily flow series in the downstream section. The COSH-tool was applied to the 15 minutes flow series in order to quantify the peak events. In addition to this analysis, the two indicators suggested by Carolli *et al.* (2015) were calculated, so that the impact level of hydropeaking was classified. After that, the limitations of both methodologies were analysed when applied to a Mediterranean river and the differences between natural and modified regimes were also analysed by daily and monthly Pardé coefficients and mean annual flow duration curve of mean daily flow.

Key-words: hydropeaking, COSH-tool, regionalization model, small hydropower plants

1. Introduction & theoretical concepts

The main goal of hydropower plants (HPP) is to maximize revenues in order to respond to market demands, despite the negative effects that it may have in the aquatic and terrestrial surrounding (Jager & Smith, 2008). One of the effects on aquatic ecosystems that have been largely studied in the scientific community is the flow variation downstream hydropower plants. Hydropeaking refers to that flow variation, *i.e.* the release of water stored in a reservoir in order to produce energy in response to market demands (Moog, 1993). These artificial rapid fluctuations of flow are caused by turn on and off in short-terms of the turbines (Carolli *et al.*, 2015). Most of the negative effects are detected downstream of the hydropower plants, such as, alteration of flow velocity and water level; decrease of fish habitats abundance, as well as, decrease of macroinvertebrates biomass (Meile *et al.*, 2011).

Harby & Noak (2013) refer that rapid flow fluctuations can be characterised by the following parameters: (1) magnitude, *i.e.* flow ratio; (2) rate of flow change, *i.e.* water level change divided by time of the change; (3) frequency, *i.e.* how often peaking occurs; (4) duration, *i.e.* time between peaks and (5) timing duration of a peak event. Additional to these parameters, Bauman & Klaus (2003) *in* Charmasson & Zinke (2011) suggest determining the biotic and abiotic parameters to improve the hydropeaking characterization. The biotic parameters are related with aquatic organisms activity during their life cycle (*e.g.* reproduction, feeding) and the abiotic parameters are related with hydraulic parameters, water quality and river morphology.

One of the first studies about alterations of river flow regime was made by Richter *et al.* (1996). In this study, the authors proposed 32 Indicators of Hydrologic Alteration (IHA) organized into 5 groups named by magnitude, time, frequency, duration and rate of change. These indicators were calculated based on

mean daily flows. After this study, Older & Poff (2003) analysed 171 hydrologic indices from different authors, including the 32 IHA. Here, the authors concluded that most of the indices were strongly correlated. Therefore, they suggested a number of indices to be in the future for studies regarding flow alterations. However daily time scale cannot detect peak flows from hydropeaking, since these events occur at subdaily scale (Bevelhimer *et al.*, 2015). Therefore, some authors proposed new studies in order to identify and quantify rapid fluctuations of flow and/or water level induced by hydropeaking on a subdaily scale. One of the studies made by Meile *et al.* (2011) suggested 3 indicators: seasonal distribution and transfer of water; subdaily flow fluctuations and, the intensity and frequency of flow fluctuations. Other study from Sauterleute & Charmasson (2014) identifies and separates the peak events in rapid increases and rapid decreases. These different phases are related to different of impacts: rapid increases may lead to catastrophic drift of invertebrates (Moog, 1993) and rapid decreases may cause stranding of juvenile fish (Saltveit *et al.*, 2011). In this study 18 parameters were calculated, organized into 3 groups, named by magnitude, time and frequency [same as Ritcher *et al.* (1996) study]. Carolli *et al.* (2015) suggested 2 indicators, based on Meile *et al.* (2011) work, capable of detecting different levels of hydropeaking impact and compared those impacts between natural and modified regime from rivers of the same region.

To minimize the hydropeaking negative impact, 2 types of mitigation measures exist: operational and structural. The first one is related to hydropower plant operation. An example of this kind of measure is decreasing and increasing the flow (*i.e.* when turbine starts to work) in a way that fishes can move to a better habitat before low flow or a peak event occurs. The second one is related to the construction of structures or river channel and floodplain alterations, *e.g.* compensation basins or lateral channel.

In Portugal, HPP can be classified as small hydropower plants (SHP) when their installed capacity is 10 MW or lower and have almost non storage capacity. On the other hand, big hydropower plants have more than 10MW of installed capacity and are often associated to large reservoirs to store water. In this last case, the HPP can produce energy at any time and, therefore, are not depended on rainfall.

To simulate hydropower plants operation schemes, their reservoirs have to obey to 2 equations in each time step:

$$\mathrm{Va} - \mathrm{Ve} - \mathrm{L} = \Delta S \quad (1)$$

$$\mathrm{S} \leq \mathrm{C} \quad (2)$$

where Va is the inflow water volume, Ve is the outflow water volume, L is the infiltration/evaporation losses volume, ΔS is the change of the volume stored in the reservoir and C is the reservoir maximum volume.

The first equation represents the mass balance equation and the second one is a limitation of the HPP.

In mainland Portugal, there are many watercourses without gauging stations to monitor the river flows. In order to obtain a reliable forecast for non-monitored watershed, a regionalization model can be applied. The model applied was developed for Portugal by Portela (2014) and uses the mean annual flows depths (H) exhibit similar dimensionless flows.

The main goal of this dissertation was to analyse the hydropeaking phenomenon in a downstream section of Bragado HPP (Avelames river) by using Sauterleute & Charmasson (2014) and Carolli *et al.* (2015) methodology.

2. Methodology

Regionalization model

Because Avelames River is an ungauged watershed, to obtain the inflows under natural conditions along the river reach were Bragado HPP is installed a regionalization model for mainland Portugal (Portela, 2014), based on the following equation:

$$\mathrm{Q}_2 = \mathrm{Q}_1 \times \mathrm{Qmod}_2 / \mathrm{Qmod}_1 \quad (3)$$

where Q is mean flow and Qmod is modular flow. Index 1 is related to gauged watershed section and 2 is related to ungauged.

Simulation model

In order to obtain a subdaily continuous flow series (time step of 15min) at the downstream section of Bragado HPP, the regionalization model was applied.

Such section was named key-section, Figure 1. The corresponding watershed area is 85 km^2.

But prior, it was necessary to know how the different components of the small hydropower plant (SHP) are interlinked - Figure 1.

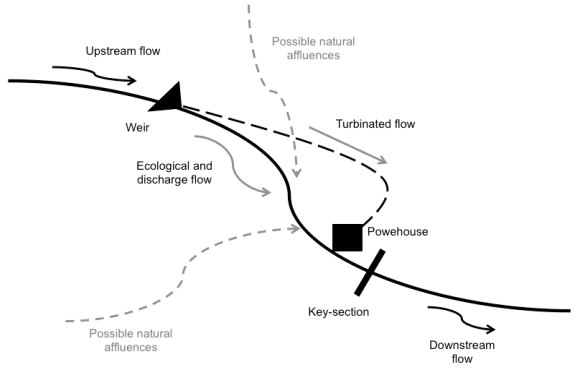


Figure 1 Small hydropower plant (SHP) schemes.

The mass balance equation (1) enables to describe the behaviour of reservoir created by the SHP. Downstream flows over the weir occur whenever the storage capacity is exceed [equation (2)]. At the key-section (SHP downstream section – Figure 1), the flow is the sum of the environmental flow, the hypothetical flow over the weir, the turbined flow and the contribution of the intermediate watershed, between the weir and the tailrace of the powerhouse.

Each HPP has its own characteristics and operation rules and each simulation model has to be adapted to them. In the model developed for the case study we assumed that the environmental flows followed by the power production have priority over the filling of the reservoir. This means that only when the natural inflows subtracted of the environmental flows exceed the design flow or are smaller than the minimum flow the water will be stored in the reservoir, provided that there is still available capacity. Along with the previous operation rule the following conditions were considered: (1) turbine working from a maximum (design flow) and to a minimum flows (30% of the design flow); (2) reservoir with a fixed maximum capacity; and (3) at the start of the simulation, the reservoir is full (maximum capacity).

The model was validated by comparing the turbined flows and energy production given by the model with the corresponding real values. The energy produced was calculated based on the following equation:

$$E = V_{turb} \times H_d / (3600/9{,}8\eta) \quad (4)$$

where E is the energy in GWh; V_{turb} is the turbined volume in hm^3; H_d is the design net head in m and η is a global efficiency of the powerhouse.

Water levels

The hydropeaking analysis with the COSH-tool uses flows and water levels series to calculate the hydropeaking parameters. The rating curve is the curve that relates flow and water level at a specific section of a river (Quintela, 1967). One way to obtain the water level for each flow uses the Manning-Strickler equation defined by:

$$Q = K_s \times R^{2/3} \times A_m \times i^{1/2} \quad (5)$$

where Q is the flow in m^3/s; K_s is the Strickler coefficient in m$^{1/3}$/s; R is the hydraulic radius in m; A_m is the wet area of the cross-section in m^2 and i is the medium slope of the river reach to which the previous equation is applied in m/m.

The hydraulic radius (R) is defined by:

$$R = A_m / P_m \quad (6)$$

where P_m is the wet perimeter in m.

In this work, the Manning-Strickler equation was solved by an iterative step. In each step, the process adds a flow increment in order to calculate the water level for each flow.

The input variables are: the flow range (maximum and minimum flow) to which the rating curve refers; the Strickler coefficient (K_s); the flow increment from the minimum flow to the maximum one (number of points of the rating curve); average slope of the river reach (i) and the cross key-section geometry.

The output variables are: flow in m^3/s; water level in cm; water free surface elevation in m; wet area in m^2; wet perimeter in m; Froude number (Fr) and average velocity in m/s.

An equation can be adjusted to the paired values of flows; flow depth thus obtained in order to define the rating curve.

COSH-tool

In this work, the COSH-tool program, developed by Sauterleute & Charmasson (2014), was applied. This software can identify and quantify rapid fluctuation in flow (Q) and water level (h) with a subdaily resolution.

The peak events can be separated in rapid increases and rapid decreases.

To identify the peaking events, the software starts to calculate the first derivative of Q/h series that gives the rate of change of Q or h ($\dot{X} = dX/dt$). If the 1st derivative is positive, it is a rapid increase and if is negative, it is a rapid decrease. After that, the program creates a threshold for the rate of change ($\dot{X}_{th}$) by using an iterative process. The magnitude of the threshold of increase/decrease is determined by the absolute maximum value of the rate of change occurring in the time series by the factors c_{inc} and c_{dec}, respectively: $\dot{X}_{th,inc} = c_{inc}\max\{\dot{X}\}$ and $\dot{X}_{th,dec} = c_{dec}\min\{\dot{X}\}$. If the rate of change is positive, it will be compared to the threshold for rapid increases. If the rate of change is negative, it will be compared to the threshold for rapid decrease. For both cases, if the absolute value of the rate of change is larger than the threshold, the data point will be defined as a part of a peaking event. The factors c_{inc} and c_{dec} are usually defined between 0.05 and 0.2.

After COSH-tool identified the peaking events, the program identifies multiple peaking events. The succeeding peaking events may occurs as a result of successive starting and stopping of two turbines in a HPP.

Additionally to these identifications, COSH-tool also identifies peaking events according to daylight conditions (daylight, darkness and twilight), because daylight conditions and water temperature have a strong influence in fishes daily activity and behaviour (Linnansaarin *et al.*, 2008).

Carolli *et al.* indicators

The methodology suggested by Carolli *et al.* (2015) was also applied to this case study. This methodology consists of 2 indicators in order to characterize three different levels of physical alteration caused by hydropeaking

The two indicators are: HP1, a dimensionless measure of the magnitude of hydropeaking and, HP2, which measures the temporal rate of flow changes in $m^3/s/h$.

The first parameter is defined as:

$$HP1_i = \frac{Q_{max,i} - Q_{min,i}}{Q_{mean,i}} \qquad i \in [1,365] \qquad (7)$$

$$HP1 = \text{median}|HP1_i| \qquad (8)$$

where index i denotes the day of the year. HP1 is the anual median of daily values of $HP1_i$, calculated as the difference between the maximum and the minimum flow value ($Q_{max,i}$ and $Q_{min,i}$, respectively) over the i-th day, divided by the flow daily mean value ($Q_{mean,i}$).

The second parameter is defined as:

$$(HP2_k)_i = \left(\frac{\Delta Q_k}{\Delta t_k}\right)_i = \left(\frac{Q_k - Q_{k-1}}{t_k - t_{k-1}}\right)_i \qquad i \in [1,365] \qquad (9)$$

$$HP2_i = P_{90}|(HP2_k)_i| \qquad (10)$$

$$HP2 = \text{median}|HP2_i| \qquad (11)$$

where Q_k refers to which subdaily flow of the data series. HP2 is calculated as the annual median of daily values of $HP2_i$ in $m^3/s/h$ and $HP2_i$ is the 90th percentile (P_{90}) of the discretized time derivative of the subdaily flow series.

To classify the level of impact of hydropeaking is necessary to calculate a threshold for each indicator: TR_{HP1} and TR_{HP2} which are defined by:

$$TR_{HP1} = P_{75}(HP1_i^{unp}) + 1{,}5(P_{75} - P_{25})(HP1_i^{unp}) \qquad (12)$$

$$TR_{HP2} = P_{75}(HP2_i^{unp}) + 1{,}5(P_{75} - P_{25})(HP2_i^{unp}) \qquad (13)$$

where $HP1_i^{unp}$ and $HP2_i^{unp}$ are the daily values of the 2 indicators for watercourses in natural regime and P75 and P25 are the 75th and 25th percentile of the distribution, respectively.

Classification of the impact level of hydropeaking in watercourses in modified regime is:

Class 1: non/low impact. $HP1<TR_{HP1}$ and $HP2< TR_{HP2}$

Class 2a: medium impact. $HP1>TR_{HP1}$ and $HP2<TR_{HP2}$

Class 2b: medium impact. $HP1<TR_{HP1}$ and $HP2>TR_{HP2}$

Class 3: high impact. $HP1>TR_{HP1}$ and $HP2>TR_{HP2}$

Pardé coefficients

The Pardé coefficients describe temporal pattern of the flow in a river section (Pardé, 1933 *in* Meile *et al.*, 2011) by means of coefficients, usually dimensionless, to allow comparisons between different rivers.

In this dissertation, the daily and monthly Pardé coefficient ($PC_{d,i}$ and $PC_{m,j}$, respectively) were calculated according to:

$$PC_{d,i} = Q_{md,i}/Qmod \quad (14)$$

$$PC_{m,j} = Q_{mm,j}/Qmod \quad (15)$$

where $Q_{md,i}$ is the mean flow of the day i in m^3/s; $Q_{mm,j}$ is the mean flow of the month j in m^3/s and Qmod is the modular flow of the total flow series in m^3/s.

Mean annual flow duration (MAFD) curve of mean daily flow

MAFD curve of mean daily flow is nothing but a representation, for the average of the years, of the ranked daily flows. Each y-ordinate of the curve refers to a mean daily flow (made dimensionless by dividing by Qmod) and the corresponding x-ordinate to the mean annual number of days with flows higher or equal than the considered one.

Because the MAFD curve utilized mean hourly flows (although constant along each day), its x-ordinates were obtained by:

$$y = \frac{\text{nr of order}}{\text{nr of total hours}} \times 365 \quad (16)$$

3. Case study & results

Bragado hydropower plant – description

The case study, Bragado HPP is located at Avelames River, 2 km upstream the confluence of Avelames and Tâmega River, in Vila Pouca de Aguiar municipality, north of Portugal. The Avelames' watershed has an area of 78.8 km^2 (upstream of the HPP) and a mean annual flow of 44.1 hm^3. The Bragado HPP reservoir has a maximum capacity of 25 000 m^3. The environmental flow is 0.064 m^3/s. Bragado HPP is equipped with a Francis turbine for the design flow of 2.2 m^3/s and the minimum of 0.66 m^3/s (30% of the maximum). In Figure 2 is represented the efficiency (η) curve of the Bragado's Francis turbine.

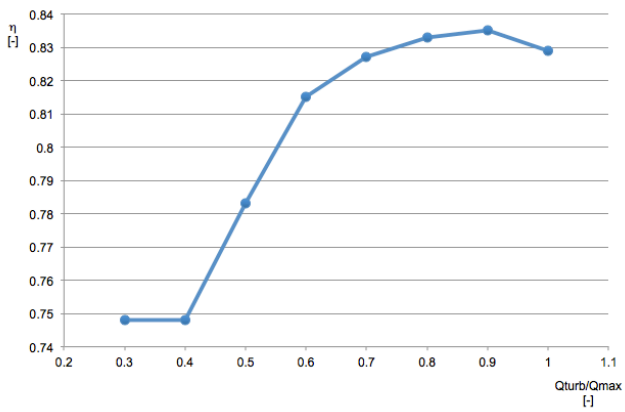


Figure 2 Efficiency curve of Francis turbine of Bragado HPP.

The HPP has a design gross head (H_d) of 155.2 m.

The intermediate watershed, *i.e.*, the watershed between of the weir and the powerhouse, has an area of 6.2 km^2 - Figure 3.

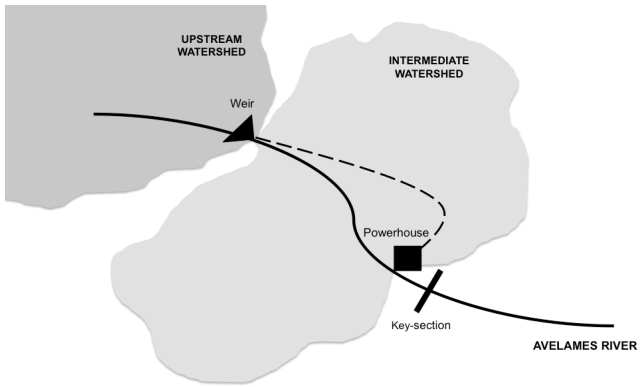


Figure 3 Identification of upstream and intermediate watershed in Bragado HPP.

Simulation model validations

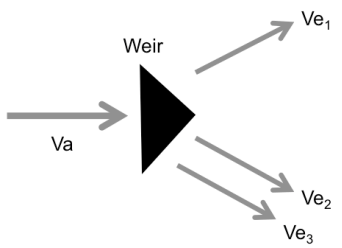
The mean daily flow series used in the regionalization model goes from October 1st 2006 until September 30th 2011 (5 hydrologic years). The daily records relate to Santa Marta do Alvão gauging station and were acquired via the public database SNIRH (*Sistema Nacional de Informação de Recursos Hídricos, Agência Portuguesa do Ambiente*). This gauging station is located at Louredo River which also belongs to Tâmega River watershed. The watershed area at the gauging station is 48.76 km^2. Because Louredo and Avelames Rivers are close, besides having similar mean annual flow depths they are expected to present similar morphological characteristics, thus supporting the application of the regionalization model.

The modules relevant in terms of the regionalization model are as follows: for Bragado HPP at the section of the weir 1.40 m^3/s and of the intermediate watershed 0.11 m^3/s; for Santa Marta do Alvão gauging station 1.45 m^3/s.

The two flow series (at the section of the weir and of the intermediate watershed) given by the regionalization model refer to daily flow. In order to have subdaily flow, it was admitted that each mean subdaily flow has was constant along the day it refers to. After that, the simulation model was applied and the validation procedure started.

Regarding the application of the mass equation, the direct precipitation over the surface of the reservoir was

neglected due to its small area; the infiltration/evaporation losses from the reservoir were also neglected; Va represents the inflow volume at the section of the weir; and Ve refers to the volume of water released downstream, comprehending the water diverted through the conveyance system (turbined volume, Ve_1), the environmental volume (Ve_2) and the volume of water over the weir (Ve_3) – see Figure 4.



Note: Ve_1 – turbined water volume; Ve_2 – environmental volume; Ve_3 – volume of water over the weir

Figure 4 Flow at the section of the weir.

In Table 1 are represented the inflow and outflow volumes taken from the flow series from 2006/07 until 2010/11.

Table 1 Inflow and outflow volumes at the section of the weir from 2006/07 until 2010/11

Affluent volume at weir [m^3]		Effluent volume at weir [m^3]	
Va	220 500 000	Ve_1	87 861 010
		Ve_2	9 030 335
		Ve_3	123 633 418

Knowing that:

$$\mathrm{Va} - (\mathrm{Ve}_1 + \mathrm{Ve}_2 + \mathrm{Ve}_3) = \Delta S$$

And according to table 1, ΔS is equal to – 24 763 m^3 during 5 years. Taking into account that the reservoir was considered to be full at the initial instant, the error in verifying the mass equation due to rounding (mainly because all the computations were done at a subdaily scale) is only of 237 – 25 000 = – 24 763 m^3 which means that the mass equation is verified.

The distribution of the simulated and real turbined flows from 2006/07 to 2010/11 is represented in Figure 5.

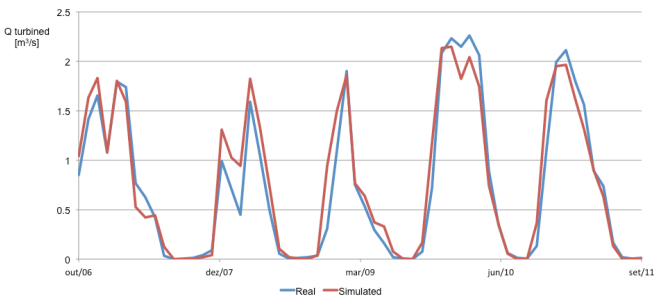


Figure 5 Distribution of mean monthly turbined flows during 2006/07 until 2010/11.

Although the model did not consider the concentration of the energy production out of the empty hours, there is a very agreement between simulated and real values which indicates the suitability of the adopted operation rules and of the models applied. It should be stressed that, due to its small capacity, the ability of the reservoir to store water during the empty hours and to turbine during the remaining periods is, in fact, very small. The comparison between the simulated and the real energy productions revealed equivalent good adjustment.

Rapid fluctuations in flow and water level analysis

To analyse the rapid fluctuations due to the powerhouse operation, a period form October 1st 2010 to September 30th 2016 (6 hydrologic years) was used. For that period the records at Santa Marta do Alvão gauging station at a time scale of 30 min were given by EDP (*Energias de Portugal*) (Qmod = 1.59 m^3/s). Based on this information, the regionalization model was applied to the watershed at the weir of Bragado HPP (Qmod = 1.40 m^3/s) and to the intermediate watershed (Qmod = 0.11 m^3/s) aiming at obtaining the natural and the modified river flows at the key-section. The area of the natural flow regime river watershed at key-section results from the sum of the upstream and intermediate watershed area and equals to 85 km^2 (Qmod = 1.51 m^3/s).

After the regionalization model was applied, all mean subdaily flow series were transformed in 15 minutes flow series. This transformation consisted on:

$$\mathrm{Q}(\mathrm{t} = 15) = \frac{\mathrm{Q}(\mathrm{t} = 30) - \mathrm{Q}(\mathrm{t} = 0)}{2} \qquad (17)$$

After the application of the simulation model, the turbined flows were replaced by the real ones, in order to minimize the errors from the model at the key-section. All null values at this section were replaced by 0.01 m^3/s, because COSH-tool would assume a zero value as a missing value.

The limits adopted when computing the rating curve vary from 0.01 m^3/s to 131.59 m^3/s, with an increment of 0.05 m^3/s. The values assumed for K_s and for the slope of the river bed were 15 m$^{1/3}$/s (slow flow with deep zones and vegetation, Lencastre, 1993) and 0.0332 m/m, respectively. Figure 6 shows the cross section of the river at the key-section.

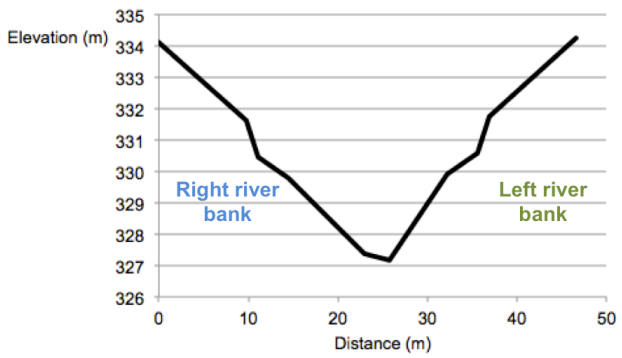


Figure 6 Avelames River cross key section profile.

The rating curve obtained is given by:

$$Q = 9.940 \times (h - 0.024)^{2.239} \quad (18)$$

The COSH-tool program was applied to flow (Q) and water level (h) series. The software starts to smooth the input data (Q and h) by applying a moving average (w). For the identification of rapid increases and decreases different thresholds were set (Table 2).

Table 2 Threshold values for analysis of the flow and water level time series for Avelames River.

w [-]	c_{inc} [-]	c_{dec} [-]	p [-]	T [min]	d [min]
5	0.13	0.13	0.2	120	45

Those threshold values are the same used by Sauterleute & Charmasson (2014) in their case study. We opted to use the same values, because the authors used a time series with the same resolution (15minutes) as this case study. For the daylight condition, the geographic characteristics were set (Table 3).

Table 3 Geographic characteristics of Avelames River.

Latitude [º]	Longitude [º]	Time zone [-]	Twilight [-]
41.57 N	-7.67 E	GMT 00:00	Civil

The number of rapid increases and decreases per year is represented in Figure 7.

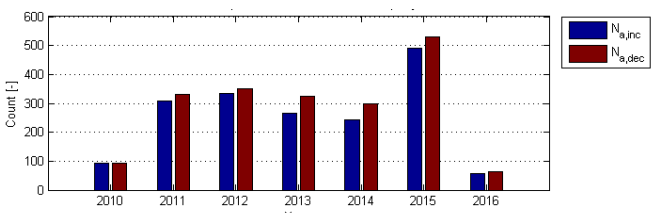


Figure 7 Number of rapid increases ($N_{a,inc}$) and decreases ($N_{a,dec}$) per year for Avelames River.

The number of the peaking events is displaying in civil years, but time series are in hydrologic years. Consequently, the information from 2010 and 2016 is irrelevant to analyse, because they are incomplete (loss of information). The years with more peaking events are the ones with less water availability (Figure 8). From here we could assume that Avelames River is very sensitive to rapid fluctuation of flow/water level when the water in the river scarce. Hence the flow difference between turbined flow and flow in the river is very high and COSH-tool detects the peaking events immediately.

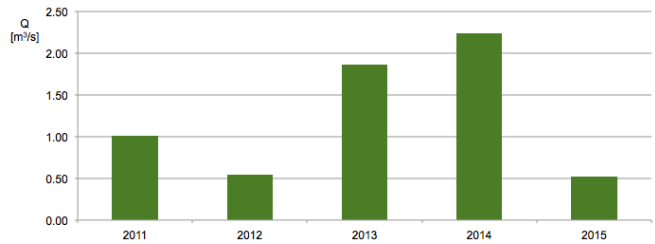


Figure 8 Distribution of mean annual flow at key-section of Avelames River during 2011 until 2015.

According to Figure 9, most of the peaking events occur during daylight, 54.5% of rapid increases and 49.9% of rapid decreases.

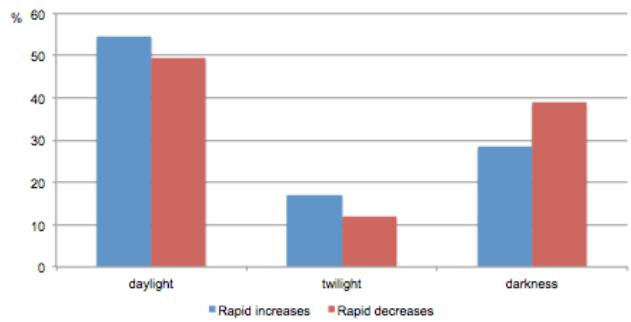


Figure 9 Percentage of rapid increases and decreases according daylight conditions (daylight, twilight and darkness).

By applying the Carolli *et al.* (2015) methodology, the modified regime of Avelames River was classified at class 3, except for the 2015/16 year when it was classified by class 2a. Concerning the natural flow regime, the classification of Carolli *et al.* (2015) returned class 1 for all the years analysed.

Natural *versus* modified regime

The monthly Pardé coefficient at the key-section is represented in Figure 10.

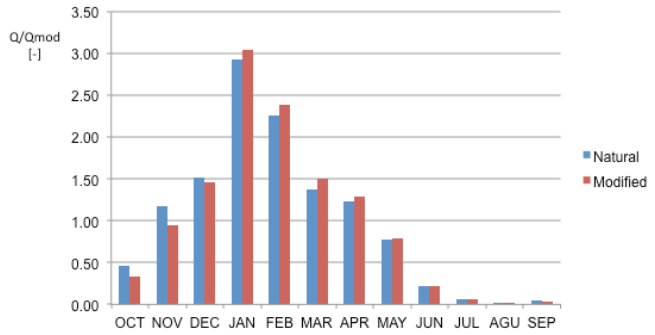


Figure 10 Monthly Pardé coefficient for 2010/11 until 2015/16.

No marked differences were observed between the natural and modified regime at Avelames River key-section. This is probably due to the very small storage capacity of Bragado reservoir.

In this analysis, the effect of the hydropeaking is not remarkable because of the time scale adopted (month).

4. Discussion of the results

This study shows the application of different methodologies to evaluate the hydropeaking impact in a SHP by using a subdaily (15 min) flow/water level series.

According to the results of COSH-tool, Figure 7 and 8, we concluded that Avelames River is sensitive to rapid variations of flow/water level, in order to minimize this effect the powerhouse should introduce smooth changes in flow when water in the river is low to reduce the stranding risk (Harby & Noak, 2013).

Since most of the peaking events occur at daylight (Figure 9) and in this time of the day, fishes are less mobile and hiding in the substrate, in order to reduce the standing risk, the peaking operation should occurs after dark (Harby & Noak, 2013).

A few percentages of the peaking events occurred in twilight phase (17% for rapid increases and 12% for rapid decreases). The analysis of the twilight phase in Portugal is not very important, because in Portugal the twilight phase lasts for 30min to 1h; but in Northern countries (where COSH-tool was developed), the twilight lasts longer and has a significant impact on the aquatic organisms activities.

The COSH-tool cannot read null data. Once again, because this tool was developed in Northern countries where null flows are not experienced. During summer, Northern rivers are influenced by snowmelt and consequently, they do not have null flows during this period of the year.

The input data used in this study was represented by hydrologic years (beginning at October 1st and end at September 30th). The COSH-tool outputs use the civil years (beginning at January 1st and end at December 31st) to display yearly parameters. Therefore, the information about the first and last year of the used time series has to be thrown away.

The COSH-tool returns much of the calculations in figures which difficult interpretations. Visual analyses can induce wrong interpretations because they depend on the observation capacity of the reader.

The river morphology and cross-section profile can reduce the impact of rapid fluctuations in flow/water level minimizing the effects downstream the HPP. Therefore, the river morphology should be also assessed together with COSH-tool results. The time series resolution should be 15 to 30 min in order to detect the peak events.

According to the results of the Carolli *et al.* (2015) methodology, the Avelames River suffered from a strong impact of the hydropeaking phenomenon.

5. Conclusions

This dissertation focused on the study of hydropeaking on a small hydropower plant at an ungauged watershed.

The regionalization and the simulation models applied proved to be robust and, thus, can be used in other cases studies.

According to the hydropeaking analyses made in this work, it is concluded that, even though the modified regime by Bragado HPP of Avelames River does not differ much from the natural regime, due to the fact of the hydropower scheme is a run-of-river, according to Carolli *et al.* (2015) the hydropeaking impact is high.

In spite of this conclusion, COSH-tool and Carolli *et al.* (2015) methodologies have to be improved if they are to be used to other case study located in a different region from where they were developed.

However, the hydropeaking analysis made in this study is a good way to help in the restoration of rivers that are affected by hydropower plants.

Acknowledgements To Silva Dias from EDP, Electricidade de Portugal, for providing hydrological data, and to Hidroerg and Lusiterg for the data regarding the exploitation of Bragado Hydropower Plant. To CERIS, Civil Engineering Research and Innovation for Sustainability and FCT, National Foundation for Science for the support. This project has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No 727830.

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