Energy production based on small hydropower schemes. A case study

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Abstract

The ambit of this thesis is to design a small hydropower plant (SHP) and to summarize the environmental impacts that such a power plant might have. The objective is to develop the necessary skills behind every stage of the process, mainly the necessary hydrological studies, the hydraulic design and the economical analysis. The work is based on a case study of the Bestança river, an affluent of the Douro river. The text structure is such that first the applied methodology is detailed without the specifics of the Bestança river, and then the results of the application on the case study are displayed and analysed. The study begins with the preliminary design of several SHPs, which includes the necessary hydrological studies and simplified design of some of the elements of the plants. These pre-designed alternatives are used as the base of the economical analysis, depending on estimated costs and return of the project, with the purpose of determining the most favourable alternative. Once the alternative is chosen, the exact design of the system is carried out, optimizing the elements from the pre-design phase and designing the missing ones. One great advantage of SHPs is the reduced environmental impacts, when compared to traditional large scale hydropower, however they are not negligible. Therefore, once the design of the plant is completed, the environmental impacts of an SHP of the type that was designed are summed up along with the relevant mitigation measures.

Keywords: Hydropower, small hydropower plant, economical analysis, environmental impacts

Introduction

The term hydropower is defined as: "of or relating to, the production of electricity by waterpower"; the concept is often associated with large dams, huge reservoirs and a massive intervention in the rivers. This text will touch on another side of hydroelectric power, small hydropower plants (SHP), defined as a hydropower plant with an installed capacity below 10 MW by the European Small Hydropower Association (ESHA, 2004), which is the definition adopted in Portugal. The common misconception of SHP is that it mimics large dams and on a smaller scale, and although the underlying hydraulic principles are the same, that energy production is proportional to flow and head, this text will show that it in fact has a separate set of techniques.

The objective of this dissertation was to completely design a small hydropower plant, so as to acquire a broader and deeper understanding of the various aspects of SHPs and their design The work was done based on a case study, the Bestança small hydropower plant.

1 SHP overview

1.1 History

The first SHPs in Portugal, which were also the first hydropower plants in general, were built around the end of the XIXth century, initially to power the illumination of cities and later to support local

industry. However, after 1930 there is a shift towards nationwide large scale projects. (Madureira and Baptista, 2012; REN, 2006)

It was not until the 1980's that the interest in SHP grew once again, this was due to the rise in energy prices, the exhaustion of locations for implementation of large scale hydropower stations and the reduced operational costs from scientific advances. In 1988 legislation (DL 189/88) was passed which permitted natural and legal persons, public or private, to exploit renewable energy sources as long as the station's apparent power was under 10 MVA. This legislation coupled with funding from the EEC through the VALOREN programme made the construction of SHPs attractive (Castro, 2002).

1.2 Types of SHP

SHPs can be divided according to their classification, the specific properties of the plant, or to their configurations, the general layout of the plant.

They can be classified according to:

- head
- exploitation and storage
- the mode of obstruction of the river
- the installed power

The main SHP configurations are:

- run-of-river schemes
- powerhouse at the foot of a dam
- integrated in an irrigation canal
- integrated in a water abstraction system

2 Case study

The project that was undertaken is based on the analysis of four distinct alternatives for the Bestança small hydropower plant, located on different sections of the Bestança river, a Douro affluent, and the Tendais river, a Bestança affluent. The base material for the work was cartography, precipitation data and flow data.

Based on the cartography (Military 1:25000), the general outline of each alternative was designed - section of the river where the water intake/weir is installed and resulting watershed, layout of the channel and penstock. The criteria for choosing the layouts were with the thought of maximizing potential hydroelectric power production, which is proportional to head and discharge. To increase the head while containing costs, a water conveyance system that will result in a large difference between altitudes at the water intake and tail race of the powerhouse is delimited, but while trying to reduce the length. To obtain higher flows at the intake, which results in higher discharges used for energy production, it is placed at the outlet section of larger watersheds. The four alternatives are defined in Table 1, where: A_W is the watershed area; Z_{intake} is the altitude of the water intake; Z_{TR} is the altitude of the tail race; H_b is the gross head; L_{DC} is the length of the channel; L_P is the length of the penstock.

Table 1: General characteristics of the 4 initial alternatives

Alternative	1	2	3	4
A_W (km^2)	53.7	10.9	14.0	34.5
Z_{intake} (m)	240	585	585	500
Z_{TR} (m)	120	355	340	355
H_b (m)	120	230	245	145
L_{DC} (m)	1730	713	1585	2011
L_P (m)	229	567	539	320
Nº weirs	1	1	2	2

Having delimited the watersheds of the different alternatives and aiming at evaluating the available water resources that can be used in the energy production, precipitation data from all the rain gauges that influence them was collected. Discharge data was also collected from a nearby river gauge station along with the precipitation data from the rain gages that influence its catchment area. The precipitation data can only be used after being subjected to a test to confirm its reliability, in this case the simple mass curve test was chosen and the data proved satisfactory.

3 Preliminary design of the alternatives. Economic analysis for selection of the optimal solution

3.1 Hydrological studies

3.1.1 Surface water availability. Mean annual energy production

To evaluate the surface water availability at each of the alternatives, a relationship between mean annual precipitation, P, and annual runoff represented by the flow depth, H, was applied. In order to establish such a relationship the river discharges at the river gage station of Cabriz (07I/04H) were utilized to calculate the annual surface runoff and the precipitation records in the rain gages with influence in the watershed of said river gage station were used to calculate the annual precipitation, according to the Thiessen method. The precipitation is plotted against the runoff for every year with valid data and

the H/P relation is determined, assuming it is linear (Equation 1).

$$H = 0.9261\ P - 572.54 \quad (1)$$

Once more resorting to the Thiessen method, the annual precipitation was determined in each of the alternatives' watersheds for as many years as there were records for all the rain gauges that influence each of them. Resorting to the previously determined H/P relation, the mean annual flow depth of each watershed can be calculated. With the flow depth and watershed area, the annual runoff volume is easily calculated which is in turn used to calculate the average mean daily flow, Q_{mod}, which is the base for the design discharge, Q_d, possibilities. For each alternative 3 possible design discharges were considered: 1.6 Q_{mod}, 1.8 Q_{mod} and 2 Q_{mod}. All the calculated values are in Table 2.

Table 2: Water availability and design flows of each of the alternatives

Alternative	1	2	3	4
Area (km^2)	53.7	14	10.9	34.5
P (mm)	2090	1681	1684	2054
H (mm)	1363	984	987	1330
V (hm^3)	73.2	13.8	10.8	45.9
$Q_{mod}(m^3/s)$	2.32	0.34	0.44	1.46
1.6 $Q_{mod}(m^3/s)$	3.71	0.55	0.70	2.33
1.8 $Q_{mod}(m^3/s)$	4.18	0.62	0.79	2.62
2 $Q_{mod}(m^3/s)$	4.64	0.69	0.88	2.91

To determine the mean annual energy production the daily exploitation of each power plant was simulated based on a series of mean daily discharges at the corresponding water intake. For that purpose the data required was:

- A mean daily flow series at the water intake
- The available head
- The ecological discharge
- The turbine design discharge and minimum discharge compatible with the operation of the powerhouse
- The global efficiency of the powerhouse

To create a realistic mean daily flow series for each of the alternatives, a transposition procedure was applied based on the mean daily discharges at the river gage station of Cabriz. To mimic the natural river regime, a varying ecological discharge, Q_e, is chosen, varying each month according to the mean monthly flow of the river in its natural state. The channel will be designed to have a small storage capacity that will allow it to store the diverted flows smaller than the minimum discharge compatible with the operation of the turbine. Once that storage capacity is filled the turbine will operate for a short period, emptying it. Because of this volume storing system, known as pondage, all flows below the turbine minimum operational discharge are considered energy production, and considered in the simulation.

Once the simulation is complete the average annual volume utilized for energy production, V_E, can be calculated, along with the average annual energy production according to equation 2, where the net head, H_N = 97.5% H_b, and the global efficiency η = 0.86. The results of the simulation for the various design flows of the alternatives are in in Table 3.

$$E = \frac{9.8\ \eta\ \ H_N\ V_E}{3600} \quad (2)$$

Table 3: Average annual energy production

	Q_d (m^3/s)	V_E (hm^3)	E (GWh)
Alternative 1	3.71	44.7	12.25
	4.18	46.9	12.83
	4.64	48.7	13.34
Alternative 2	0.55	6.6	3.46
	0.62	6.9	3.63
	0.69	7.2	3.77
Alternative 3	0.70	8.5	4.73
	0.79	8.9	4.95
	0.88	9.2	5.15
Alternative 4	2.33	28.0	9.28
	2.62	29.4	9.72
	2.91	30.5	10.10

3.1.2 Flood characterisation

In order to design the weir, the one hundred year peak flood discharge was adopted. To compute it four different methods were applied and their results analysed prior to a decision regarding the design value: the rational formula, the Loureiro method, transposition with Loureiro's coefficients and the HEC-HMS program. The weirs that correspond to each alternative are on Table 4[1].

Table 4: Weirs belonging to each alternative

	Weir name(s)
Alternative 1	Bestança 1 (B1)
Alternative 2	Tendais 2 (T2)
Alternative 3	Tendais 2 (T2) Tendais 1 (T1)
Alternative 4	Tendais 3 (T3) Bestança 2 (B2)

The rational method is one of the most common formulas for calculating peak flood discharge and uses equation 3 where Q is the peak flood discharge

[1] Weirs were named according to the river they are inserted on and numbered from West to East

with a given return period, T (m^3/s); C is a kind of runoff coefficient that accounts for the precipitation losses according to the return period; i is the intensity of the design rainfall with the same return period, T, and a duration equal to the time of concentration of the watershed (m/s); and A is the area of the watershed. (m^2)

$$Q = C\ i\ A \tag{3}$$

For a return period of T=100 years and watersheds with an area under 500 km^2 the value of 0.8 for C has proven to give satisfactory results (Quintela, 1984) and the area of each watershed is obviously known, so the only parameter that had to be determined was rainfall intensity. It is known that peak flood discharge at a given river cross section occurs for rainfall with a duration equal to the time of concentration, t_c, of the corresponding catchment area, so t_c had to be determined and then the rainfall intensity for precipitations with that duration, i_{tc}. To determine t_c three empirical equations are used: Giandotti, Témez and NERC. The results for each and the average time that was considered can be seen in Table 5, the watersheds are named according to the weir at their outlet section.

Table 5: Times of concentration of the watersheds

	B1	B2	T1	T2	T3
t_c Giandotti (h)	2.33	1.69	0.85	1.52	1.62
t_c NERC (h)	3.24	2.38	1.55	2.27	2.37
t_c Témez (h)	3.15	2.15	1.10	1.78	1.84
Average t_c (h)	2.91	2.07	1.17	1.85	1.94

As there is no information about short durations precipitations in the watersheds under consideration but only for the duration of 24 hours. However, authors have shown that the ratio between two maximum annual precipitations of fixed duration with the same return period has an extremely smooth spatial variability (Portela, 2006). This ratio is known as the partition coefficient.

The rainfall intensity equations (IDF) determined by Brandão et al. (2001) from the nearest precipitation gauge that possesses them (Caramulo 10H/01) were used to to determine rainfall for the time of concentration, P_{tc}^{IDF}, and for 24 hours, P_{24}^{IDF}, for a return period of T=100 years. Precipitation data from the rain gages that affect the watersheds and statistical analysis is used to determine rainfall in 24 hours, P_{24}^{W}, for T=100 years. In order to determine rainfall in the time of concentration for the watershed, P_{tc}^{W}, with the same return period, equation 4 is used. In this case the partition coefficient is $P_{tc}^{IDF}/P_{24}^{IDF}$). Once P_{tc}^{W} is known, i_{tc} is calculated by dividing it by t_c and the rational formula can be applied, to obtain the peak flood discharge.

$$P_{tc}^{W} = P_{24}^{W}\ \frac{P_{tc}^{IDF}}{P_{24}^{IDF}} \tag{4}$$

The formula developed by Loureiro (1984) (Equation 5) returns the peak flood discharge, where Z and C were determined empirically and vary according to the zone in the country and according to the zone and return period, respectfully. It is not used directly any more but it is useful for transposing from a watershed with a known peak flood discharge that was determined with more precise methods like statistical analysis, as long as both watersheds are similar, i.e. have the same C. Because the watersheds appear to be on the intersection of zones 3 and 4 calculations for both scenarios were made, by direct application of the formula and by transposition using Loureiro's coefficients.

$$Q = C\ A_W^Z \tag{5}$$

The method from the *HEC-HMS* program implemented used the SCS unit hydrograph model that computes the peak discharge from a hyetograph, based on the watershed area and on the lag time, t_{lag}. Two hyetographs were considered, one uniform and one varied, both with duration equal to to t_c.

The peak flood discharges determined by all the methods presented can be found in Table 6, along with the chosen design value, Q. The most trustworthy method used was the rational method, and its roughly coinciding with the HEC-RAS varied hyetograph gives us extra confidence in considering both values for the final results.

Table 6: All results for calculations of the one hundred year peak flood discharge (m^3/s)

	B1	B2	T1	T2	T3
Rational	272	159	17	48	47
Loureiro Z3	377	247	88	168	169
Loureiro Z4	184	123	46	84	85
Transposed Z3	298	195	70	132	133
Transposed Z4	314	210	78	144	145
HEC-HMS Uniform	227	133	14	40	40
HEC-HMS Varied	263	153	16	46	46
Q	265	154	17	47	47

3.2 Preliminary design of the diversion structure, conveyance system and powerhouse

Due to the amount of alternatives, and large amount of parameters that need to be calculated for each one, this section will not present any results.

Instead, once the economical analysis is complete, the chosen alternative will be generally defined and the costs and revenues considered for the the analysis will be presented.

The crest of the weir must be designed in order to safely discharge the one hundred year peak flood discharge. It should be of WES type and the discharge over it obeys equation 6 where Q is the discharge, C the discharge coefficient (C=0.48), l the length of the crest of the weir, and H the head above the crest. It must be tall enough to prevent the initial stretch of the conveyance system downstream of the weir to be overtopped in case of a one hundred year flood, and have the necessary dimensions (lateral anchoring, base plate and weir profile) for the structure to withstand the loads it will be subject to during operation. To prevent excavation a crest length approximately the size of the river is chosen and the lateral embankment walls are raised by one meter for safety.

$$Q = C\ l\ \sqrt{2g}\ H^{3/2} \tag{6}$$

The design of the channel utilized the Manning Strickler equation for a bed slope of 0.001 and for the ratio width/height, b/h, of approximately 1.5. The b/h ratio was blocked so as to avoid having too many different options at this stage, as they are still for comparison purposes. The construction of the channel implies the excavation of a horizontal ledge on the hillside where it will sit, which must wide enough to allow a maintenance vehicle to drive along it, and have a safety gap on the uphill side of the channel to allow for falling rocks and for adequate drainage of water flowing from uphill. In order for the channel to have storage capacity for the pondage system, the channel walls at the end are raised, $\Delta\ h$, and extended horizontally upstream until they join the normal channel walls. The length and height of these walls will depend on the minimum turbine discharge, Q_{min}, and limitations regarding turbine operation.

The flow velocity in the penstock must be kept below certain thresholds in order to prevent vibrations that might cause damage to the cement holding blocks or the penstock itself and also to avoid excessive head losses, a limit of 3 m/s was preliminarily adopted. Once the diameter was calculated using the Manning Strickler equation equation 7 was used to determine the necessary thickness, which must be enough to withstand the maximum pressure of when the turbine interrupts its operation and an additional thickness due to corrosion also needs to be considered.

$$t = \frac{1.5 * \frac{H_b}{10} * D * 100}{2400} + t_{corr} \tag{7}$$

The turbine installed capacity, P, is given by equation 8. Where, at this preliminary stage, the net head, H_N, was considered 95% of the gross head, H_b, and the average efficiency of the powerhouse, η was set equal to 86%.

$$P = g\ H_N\ Q_d\ \eta \tag{8}$$

The choice of the turbine type depends on the net head and design discharge, as different types are better suited to different combined conditions, a turbine application chart was consulted to confirm the turbine choice.

3.3 Economic analysis

To calculate the cost of the weir, channel and penstock the amount of materials needed for construction are multiplied by their relevant unit price, and the volume of excavation by its unit cost. The powerhouse cost is given by equation 9, where P is the turbine installed capacity (MW); the value of K_P varies depending on which type of turbine has been chosen (the adopted values were $K_{P\ Francis}$ = 5 500 000 and $K_{P\ Pelton}$ = 6 000 000).

$$C = K_P\ H_N^{-0.35}\ P^{0.7} \tag{9}$$

There are other costs that need to be considered: the forebay, considered to be equal to the cost of 100 m of channel; connecting to the grid, fixed at €200 000 which is equivalent to the cost of the control panel and around 2 km of electrical line; the construction site, which is set at 10% of the cost of civil works; the replacement of equipment, set at 50% of the cost of equipment; and annual operational costs, which are the salary of one employee, licensing costs equal to 1% of revenue, administrative costs considered to be €3500/MW of installed power, maintenance of the civil works set at 1% of their initial cost and maintenance of the equipment at 2.5% of its initial cost.

The economic analysis is done considering the present value, or discount value, of the investment in the beginning of the plant's first year of operation. The analysis was carried out considering discount rates, t_a, of 3, 6 and 8%, cover the likely economic scenarios during the lifespan of the hydropower scheme. The assumed lifespan of the SHP is 35 years and that all the initial works are done in the two years preceding the start of the operation, known as the construction period, 30% in the first year and 70% in the second. The three economic indicators considered were the net present value (NPV), the benefit cost ratio (B/C) and the internal rate of return (IRR), the latter being the most important.

The chosen alternative was number 1, with Q_d=3.71. The pre-designed element characteristics

are in Table 7 and the costs and revenues considered in the economical analysis are in Table 8.

Table 7: Characteristics of the elements of Alternative 1, with $Q_d = 3.7\ (m^3/s)$

Weir	l = 19.0 m H = 3.5 m h_{body} = 3.5 m h_{total} = 10.0 m
Channel	b = 1.95 m h = 1.30 m L = 1730 m
Penstcock	D = 1250 mm L= 229 m
Powerhouse	Francis Turbine P = 3569 kW
Annual energy production	E = 12.25 GWh

Table 8: Costs and revenues considered in the choosing Alternative 1, with $Q_d = 3.71\ (m^3/s)$ (thousands of €)

Singular	Weir	137
	Channel	1374
	Forebay	78
	Penstock	507
	Powerhouse	2556
	Construction site	228
	Grid connection	200
	50% equipment replacement	1232
Operational	Energy Sales	796
	Civil works maintenance	23
	Equipment maintenance	62
	Other operational costs	36

4 Hydraulic design of the water intake, conveyance system and powerhouse. Final economic analysis

4.1 Design

As the weir, water intake and lateral spillway are dependant on each other, they are designed simultaneously. Before the exact outline of the weir is designed, it is necessary to check if it is tall enough so that the one hundred year peak flood discharge would not overtop the initial reach of the channel downstream of the weir. The cartography is used to determine the downstream section and slope of the river reach, and then the Manning Strickler equation is applied to determine the free surface level. When the lateral spillway is chosen, it is necessary to guarantee that its crest is not below this level. Table 9 shows the free surface water level downstream of the 100 year flood.

Table 9: Free surface level and flow volume downstream of the weir

FSWL (z)	$Q\ (m^3/s)$
260 (Weir crest)	1406
257.5	**261**

A Tyrolean style intake was chosen, as was mentioned during the annual energy calculations of the preliminary design. The parameters that need to be determined are the width and length of the rack and the head over the rack, so that when the upstream surface is at full storage water level the design flow is diverted to the channel. The design of the intake is done assuming that all the flow over the rack up to the design discharge is diverted, and consists of two parts. Firstly, a width, W, is chosen and the necessary head over the rack, H_0, that results in the design discharge flowing over the rack is calculated by solving equations 10, where the reduction factor, χ depends on the slope of the rack.

$$\begin{cases} H_0 = h + \dfrac{q^2}{2g\,h^2} \\ h = \chi h_c = \chi\,\dfrac{2}{3}H_0 \end{cases} \tag{10}$$

Once H_0 is known, the rack length is determined to guarantee that all the discharge over the rack is diverted with equation 11 (Ramos and de Almeida, n.d.), as per the initial assumption.

$$L = \frac{1.1848\ H_0}{(1-\alpha)\ C_v\ C_a} \tag{11}$$

Table 10 shows the options that were considered for the water intake, in bold is the chosen one. The rack length was rounded up to 1.60 m and H_0 = 0.40 m was chosen.

Table 10: Selection of Tyrolean water intake dimensions

$W(m)$	7.0	8.0	**9.0**	10.0
$q_d\ (m^3/s/m)$	0.53	0.46	**0.41**	0.37
$H_0\ (m)$	0.46	0.42	**0.39**	0.36
$h\ (m)$	0.28	0.25	**0.23**	0.22
$L\ (m)$	1.81	1.65	**1.53**	1.43

In order to determine the length and height of the lateral spillway the design discharge, i.e. how much water will flow into the Tyrolean intake in case of a one hundred year flood, must be known.

As a simplification, the diverted water was calculated considering the intake to be a bottom orifice (equation 12), taking into account the net area, A_u and considering the discharge coefficient, C = 0.6

$$Q = C\,A_u\,\sqrt{2g\,h} \tag{12}$$

Once the side weir design discharge has been determined various lengths and heights are tested, using equation 13, where C=0.4 due to the flow and discharge direction being perpendicular. The chosen intake resulted in Q = 33.3 (m^3/s) for the lateral spillway, the chosen height/length is in bold on Table 11, along with the resulting crest elevation, Z_{crest}, which is above the FSWL for the one hundred year flood, meaning the weir height is satisfactory.

$$Q = C\,L\sqrt{2g}H^{3/2} \tag{13}$$

Table 11: Heads and corresponding lateral spillway crest lengths

H (m)	L (m)	Z_{crest}
0.80	26.3	258.7
0.90	**22.0**	**258.6**
1.00	18.8	258.5

The settling basin serves the purpose of removing the particles that are undesirable downstream, that might cause damage to the turbine. It is designed so that the undesirable particles spend more time in the basin, scouring time T_s, than the time it takes for them to settle in the bottom, settling time T_{sed}. This is achieved by increasing the section to slow down the flow and prolonging the length of the basin.

Table 12: Characteristics of the settling basin

w_s (m/s)	0.025
T_{sed} (s)	61
$D(m)$	2.5
$W(m)$	3.2
$u(m/s)$	0.46
L (m)	28.1
T_{scour} (s)	61

The channel height/width is optimized at this point, releasing the b/h=1.5 constraint so as to find the dimensions that result in the lowest construction cost. Table 13 shows the pre-designed and final dimensions, along with the cost.

Table 13: Pre-designed and final channel

	Pre design	Final
b (m)	1.95	1.60
h (m)	1.30	1.50
Cost (€)	137972	1252456

The forebay must have sufficient volume to assure the regulatory function in turbine transient regimes, have a slow enough flow to induce the settling of solid particles that might damage the turbine and must also guarantee the minimum submergence criteria for the intake. As the channel will have a large storage capacity for pondage, verifications for the transient regimes were not undertaken. The conveyance system is already equipped with a settling basin to prevent solid particles entering the turbine, but as a precaution the speed in the forebay is limited to 0.5 m/s and the length, L, is set to a minimum of 2.5 times the width, W, for that same purpose. The minimum submergence, S, is deduced from equation 14, where v is the flow speed at the inlet (m/s); d is the height of the inlet in (m); and considering an upstream flow coefficient, C = 1.7 which represents a symmetrical flow approach. Once the minimum submergence is determined a depth, D, is chosen to guarantee it and the remaining dimensions are chosen to meet the particle settling criteria.

$$\frac{S}{d} = \frac{C\,v}{\sqrt{g\,d}} \tag{14}$$

Table 14: Forebay dimensions

d (m)	1.25
S (m)	1.84
D (m)	3.20
W (m)	3.00
L (m)	10.00

To determine the ideal penstock dimensions the least costly diameter must be found, taking into consideration the cost of the energy that won't be produced due to head loss, along with the initial cost of purchase. Head loss, ΔH, is calculated resorting to the Manning-Strickler equation and the average annual loss in energy production ΔE is determined using equation 2. The result was that the initial diameter was the optimal one, and the change in cost is due to the precise layout of the penstock which resulted in a small length variation, shown in Table 15

Table 15: Preliminary and final penstock cost (€)

	Preliminary	Final
Penstock	507081	488319

The powerhouse design begins by defining the turbine speed, n, taking into account the net head and the frequency of the grid it will connect to. Once the turbine speed has been defined the runner needs to be positioned to avoid cavitation. The maximum admissible suction head, h_{smax}, is the maximum difference between the characteristic runner section level and the level of the river at which cavitation will not occur, it is calculated using equation 15. σ is the Thoma coefficient that translates a turbines susceptibility to the occurrence of cavitation, and the first two elements of the equation are the barometric head and vapour pressure which depend on the altitude and temperature respectively. Once the maximum admissible suction head has been calculated and therefore the minimum tailrace level is known, the outlet of the powerhouse must be designed to guarantee that this level is always met. Table 16 shows the properties of the turbine.

$$h_{smax} = \frac{p_{atm}}{\gamma} - \frac{t_v}{\gamma} - \sigma H_N \tag{15}$$

Table 16: Turbine properties

n (rev/min)	h_{smax} (m)
1000	-2.30

4.2 Annual energy production and final economic analysis

The annual energy production depends on the head losses in the system, ΔH, of which the following are considered: losses in the trashrack; losses in the intake; losses in the square to round valve; and losses in the penstock, both continuous and in the bends. The singular head losses are calculated by Borda's equation 16 where K is a singular loss coefficient that depends on the geometry of the singularity. The continuous head loss in the penstock is determined by the Manning-Strickler equation.The total head losses in the system are shown on Table 17.

$$\Delta H = K \frac{U^2}{2\,g} \tag{16}$$

Table 17: Head losses in the system

	ΔH (m)
Trashrack	0.030
Intake	0.019
Transition Valve	0.018
Penstock	1.90
Total	1.96

Once ΔH for the design discharge is known, a head loss coefficient, C = 0.143, is calculated according to equation 17.

$$\Delta H = C\,Q^2 \tag{17}$$

With this coefficient and using the same equation, it is now possible to calculate the head losses according to the flow, which is used in the final energy simulation along with the real gross head to determine the real net head (equation 18

$$H_N = H_B - 0.143\,Q^2 \tag{18}$$

The final economic analysis can at this point be undertaken, using the same methodology as in the alternative selection stage, taking into account the final design and the precise energy calculation. The results can be found on Table 18

Table 18: Results for the final economical analysis

IRR	12.14%	
	NPV (€)	B/C
$t_a = 3\%$	8581678	2.48
$t_a = 6\%$	4163523	1.75
$t_a = 8\%$	2338313	1.43

5 Environmental impact

5.1 Impacts

The interaction between flow and physical attributes of the riverbed result in the creation of different habitats such as ripples, pools or backwaters, for various aquatic species to inhabit, that have adapted to the river's natural flow conditions. By placing a diversion scheme, the flow downstream of the intake will be reduced which will result in the transformation of theses habitats with loss or reduction of indigenous fauna and flora (Bunn and Arlington, n.d.). Terrestrial animals are also dependant on the river for drinking water, so the habitat of the affected fauna extends parallelly to the watercourse beyond the boundaries of the riverbanks (Bergkamp et al., 2000).

Migratory fish require different ecosystems for different phases of their life - growth, sexual maturation, reproduction and production of juveniles. Different types of fish will spend different phases of their life in freshwater or sea water, needing to migrate up or downstream accordingly. The impact is felt by all aquatic fauna depending on each site, although fish are the most significantly affected species on the whole (ESHA, 2009).

The presence of a weir makes it near impossible for fish to swim upstream, which results in the extinction of species that cannot reach or return from the grounds needed for the development phase they

find themselves in. The ability to surpass the weir is strongly related to its height, but also to the hydraulic conditions over and at the base of the weir and the species trying to migrate upstream. (Larinier, 2001)

It can be potentially deadly for fish to cross a weir downstream, as they pass over spillways or through the various stages of the diverted flow's hydraulic circuit. Fish death due to passage through the turbine is related to impact, sudden changes in speed and pressure and cavitation, survival chances are strongly affected by the size of the fish, the larger ones suffering the most. Fish death from passing over a spillway can be direct or indirect, by injuring or disorienting the fish making him more susceptible to predators. Even without crossing the weir it's presence can be harmful, the large concentration of fish around the barrier makes fish more susceptible to predation. (Larinier, 2001)

The presence of a weir will also prevent suspended solid transport to occur naturally, causing an accumulation of sediments upstream. The change in suspended solids may have an effect on fish during their spawning period (Abbasi and Abbassi, 2010). The physical barrier prevents transport of organisms, organic matter, sediments and seeds. Therefore, the stretch of river downstream of the weir will be deprived of the base for a diverse and fertile ecosystem (Jansson, 2002)

There are other minor impacts that will not be detailed in this section or the next, related to the small reservoir that is created by the weir, change in water quality, the sonic impact of the turbine operation, the visual effect on the landscape and the risk of bird collision from the electrical wires.

5.2 Mitigation

To mitigate the effects of the reduced flow in the reach between the water intake and the tail race, common practice is to define an ecological flow or reserved flow, that is not diverted for energy purposes with the goal of trying to mimic the river's natural state. There are four main categories of methods for calculating the ecological flow: hydrological, hydraulic rating, habitat simulation and holistic methodologies. (Gordon et al., 1992)

The upstream passing over a weir is achieved with upstream fish migration systems, also known as fishways, fish passes and fish ladders. The choice for which passage is site specific, depending mostly on the species of fish that will use the passage. (Penche, 1998; Therrien and Bourgeois, 2000)

There are several ways to deal with downstream migration, grouped into three categories: bypassing the conveyance system, bypassing the powerhouse and complete passage through the hydraulic circuit. (Penche, 1998; Therrien and Bourgeois, 2000)

The passage of sediment and other desirable solid matter downstream is achieved by designing structures that will allow it to pass naturally, operating scour valves or dredging upstream and dumping downstream. The structures can be different designs of the intake, such as a scoop intake that flushes sediment over it during floods, or fish ladders that allows sediment passage like vertical slot and pool and weir ladders with bottom orifices (Therrien and Bourgeois, 2000; SEPA, 2015).

6 Considerations from the case study

6.1 Optimization of the alternative selection phase

During the pre-design and alternative selection some discrepancy was felt between the complexity of the cost estimation of some of the elements and their economic significance. Specifically, the process behind the calculation of the cost of the weir, i.e. its design, was extremely complex when considering that it is the second least economically significant element of the plant. Also, concerning the channel, once the economic analysis was complete and an alternative had been selected a far more precise methodology was used to determine its final design, with only the slight increase in complexity of taking onto account the hillside natural gradient, and a large change in cost due to its large economic significance. Taking these points into account, the proposals to optimize the alternative selection phase are:

- Use a fixed value for the weir cost
- Design the channel with greater precision, taking the hillside gradient into account from the start

6.2 Alternative intake structures

A significant amount of the environmental impacts were directly related to the barrier effect of the weir. Although a number of mitigation measures exist, the ones presented in the previous section among others, there are very few intake structures that avoid the problem in and of themselves.

One solution is the use of a lateral intake without damming, but this method has certain limitations namely a low maximum diverted water capacity (Lauterjung and Schmidt, 1989). As environmental impact mitigation gains more importance and technology progresses, these structures can be improved or new ones can emerge to compete with the hy-

draulic performance of the systems presently available, with no or a greatly reduced barrier effect.

7 Final note

As this is a more practical learning thesis there are no possible future conceptual or theoretical developments to be suggested, although one should note the possible technical advances that can have a positive impact on the design of SHPs, namely, enabling alternative solutions by making them equally economically feasible or by improving the performance of the SHP, related, for example, with:

- An economically viable completely pressurized conveyance system .
- The installation of more than one turbine under equally economically feasible conditions aiming at a more versatile exploitation.
- Better understanding of Tyrolean style intakes

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