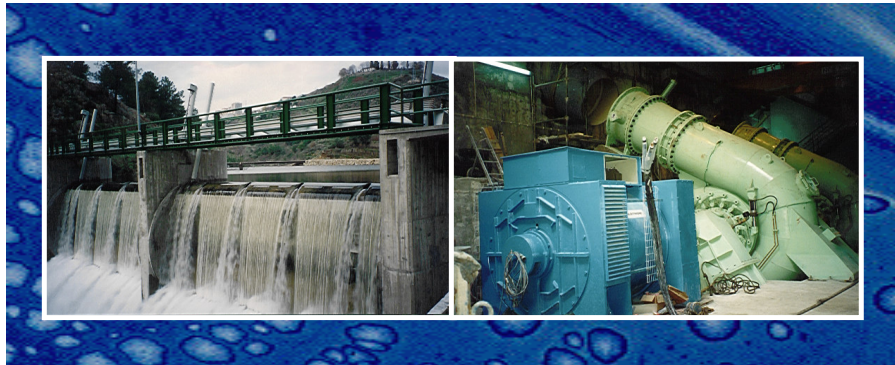


GUIDELINES FOR DESIGN OF
SMALL HYDROPOWER PLANTS



Editor
Helena RAMOS

TECHNICAL CARD

Copyright © 2000.

Published in 2000 through WREAN (Western Regional Energy Agency & Network) and DED (Department of Economic Development), Belfast, North Ireland.

All rights reserved. No part of this publication may be reproduced, stored in a retrieval system, or transmitted, in any form or by any means except under terms of the Copyright, Designs and Patents Act 1988 without the permission in writing of the publisher, with the exception of any material supplied specifically for the purpose of being entered and executed on a computer system for exclusive use by the purchaser of the publication.

Title:

*Guideline for Design of
SMALL HYDROPOWER PLANTS*

Helena Ramos	(Chapters: 1, 2, 3, 5, 6, 7 and 9)
A. Betâmio de Almeida	(Chapters 1, 2, 3, 5, 7, 9 and reviser)
M. Manuela Portela	(Chapters: 4 and 10)
H. Pires de Almeida	(Chapter 8)
A. Carvalho Quintela	(Foreword and reviser)

Legal deposit: July 1999.

Contents production: Helena Ramos.

Cover production - Designer: Joana Cruz.

This book has the support of CEHIDRO, WREAN and DED.
Typeset in 11/14 pt Times New Roman MS Word 97.

FOREWORD

The book Guidelines for design of SMALL HYDROPOWER PLANTS comprises the following ten chapters covering the main subjects connected with the design of such plants:

- 1 – Hydroenergy.
- 2 – General types of small hydropower plants.
- 3 – Hydropower and water uses.
- 4 – Hydrology.
- 5 – Hydraulic design of small power plants.
- 6 – Small hydraulic turbines.
- 7 – Hydraulic transients and dynamic effects.
- 8 – Electrical equipment.
- 9 – Environment.
- 10 – Economic analysis.

The first three chapters contain an introduction to the problems involved in the subject under consideration.

Chapter 4 deals with hydrologic studies for small hydroelectric schemes with a run-of-the-river exploitation in order to define the design discharges of water conveyance systems and of hydraulic turbines, and power productions. The proposed methodologies may overcome the non-existence of basic flow information at the river section of the scheme intakes.

Chapter 5 contains relevant information about the hydraulic design of intakes, bottom outlets, hydraulic conveyance systems (including canals and penstocks) and powerhouses.

The object of Chapter 6 is to characterise the differences among the types of small turbines and their behaviour, as well as turbines application ranges and to estimate main dimensions of the turbines.

Chapter 7 is the longest one (it fulfils with chapters 5 and 6 half of the book) and presents the main problems concerning hydraulic transients and dynamic effects.

Chapter 8 deals with synchronous and asynchronous generators, electrical facilities and control systems.

Environment impact assessment is the object of chapter 9.

Chapter 10, after considering capital costs, annual operations costs and benefits, presents an economic analysis according to two different concepts and defines economic indexes. It includes an application example of economic analysis.

The chapters authors are as following:

- Helena Ramos
 - author of chapters 1, 2, 3, 5, 9 (being second author A. Betâmio de Almeida), and 6, and second author of chapter 7 (being first author A. Betâmio de Almeida).
- A. Betâmio de Almeida
 - author of chapter 7 (being second co-author Helena Ramos) and second author of chapters 1, 2, 3, 5 and 9 (being Helena Ramos the first author).
- M. Manuela Portela
 - author of chapters 4 and 10.
- Pires de Almeida.
 - author of chapter 8.

Each author has a good professional experience as consulting engineer in the field of his participation in this book. The first three mentioned authors also have a significant teaching and research experience in such field. The very special background of Prof. A. Betâmio de Almeida must be emphasised.

The book identifies the more important subjects of interest for the design of small hydropower plants, which are treated under a rigorous, synthetic and understandable way.

The text is completed by appropriated illustrations.

Thus, I think that the book will be a very useful tool for those who have a participation in one or more of the multi-disciplinary aspects involved in the design of small power plants and a favourable acceptance will be expected.

António de Carvalho Quintela

ACKNOWLEDGEMENTS

This book is the result of the applied experience in several small hydropower base and final type projects, since I joined the HIDRO4 (now a part of SOMAGUE Engineering Group). A research program in this domain was developed, since the eighties, at Instituto Superior Técnico (IST) of Technical University of Lisbon, and was the basis of my PhD. The book also benefited from the contribution and support from different parts that I would like to acknowledge hereafter.

Firstly, I would like to give special thanks to my adviser Professor A. Betâmio de Almeida (Full Professor of IST and an expert in hydraulic transients), whose enthusiasm and scientific revision contributed substantially with their suggestions. Moreover, the colleague Prof. M. Manuela Portela for chapters 4 and 9 and Eng. H. Pires de Almeida, Manager of ENERSIS a mini-hydro power company, with his contribution for chapter 7, are greatly appreciated.

I also want to express my gratitude to Professor A. Carvalho Quintela (Full Professor of IST and a very experienced designer of hydroelectric schemes) for taking the time out to do relevant remarks to the original and to write the foreword of this book.

Special thanks go to Dr. Robert Gibson, manager from WREAN (Western Regional Energy Agency and Network, North Ireland) and Dr. Lawrence Foy from DED (Department of Economic Development – Energy Division) through

their invitation, confidence and interest always proved along the development of this book.

I am thankful for the co-operation of several companies, such as ALSTHOM, BOUVIER, CEHIDRO, DED - Energy Division, DUMONT, ENERSIS, EDP, HIDRO4/SOMAGUE, HIDROCENTRAIS REUNIDAS, LNEC, NEYRPIC, VOITH and WREAN, which allowed to support the publishing of this document including some knowledge, photos, project elements and data information.

Helena Ramos

PhD in Hydraulic/Civil Engineering
I S T - Technical University of Lisbon
hr@civil.ist.utl.pt

CONTENT

1- HYDROENERGY	1
1.1- Renewable energy concept	1
1.2- Hydroelectricity: energy conversion and hydropower principles	3
1.3- Environmental considerations	7
2- GENERAL TYPES OF SMALL HYDROPOWER SCHEMES	9
2.1- Classification of hydropower plants	9
2.2- Hydropower schemes	10
2.3- Site location	13
3- HYDROPOWER AND WATER USES	15
3.1- Power generation and different sector-users	15
3.2- The benefit of small hydropower plants	19
4- HYDROLOGY	21
4.1- Introduction. Scope of the studies	21
4.2- Basic data required for the hydrologic study	22
4.3- Mean annual And mean daily flow series	25

4.3.1- Introduction	25
4.3.2- Annual precipitation and annual flow. Mean daily flow series	26
4.3.3- Mean annual flow duration curve	31
4.4- Energy Evaluation	34

5- HYDRAULIC DESIGN OF SMALL POWER PLANTS

39

5.1- Introduction	39
5.2- Layout schemes	40
5.2.1- Intakes	40
5.2.1.2- Minimum submergence	40
5.2.1.3- Bottom intake	44
5.2.2- Protection rack	48
5.2.3- Sedimentation or desilting basin	51
5.2.4- Weir	54
5.3- Sluice bottom outlet	57
5.4- Conveyance system	58
5.4.1- General layout	58
5.4.2- Head losses and net head	60
5.4.3- Canals	66
5.4.3.1- Uniform and steady state hydraulic regimes	67
5.4.3.2- Boundary conditions	69
5.4.3.3- Forebays	69
5.4.4- General remarks about mixed circuit	71
5.4.5- Penstocks	72
5.5- Powerhouses	75
5.6- Analysis of hydropower schemes	78

6- SMALL HYDRAULIC TURBINES

79

6.1- Types of turbines	79
6.2- Turbine similarity laws and specific speed (N_s)	84
6.3- Turbine efficiency	89

6.4- Dimensions of turbines	90
-----------------------------	----

7- HYDRAULIC TRANSIENTS AND DYNAMIC EFFECTS	101
--	------------

7.1- Introduction	101
7.2- Canal systems	104
7.3- Pressurised systems	108
7.3.1- Typical transient regimes	108
7.3.2- Preliminary analysis	110
7.3.3- Governing equations	113
7.4- Overspeed dynamic effects	115
7.4.1- Overspeed runner control	115
7.4.2- Overspeed effect on turbine discharge	119
7.4.3- Turbine overspeed effects on waterhammer	122
7.5- Special protection devices	123
7.5.1- Introduction	123
7.5.2- Surge tanks	124
7.5.3- Differential surge tank	125
7.5.4- Air vessel	127
7.5.5- Synchronised valve or relief valve	129
7.5.6- Flywheel	130
7.5.7- Protection devices behaviour	131
7.5.7.1- Analysis of a surge tank	131
7.5.7.2- Analysis of an air vessel	136
7.6- Examples	138
7.7- Other protection devices and procedures	140
7.8- Integrated analysis and design	142
7.9- Case studies	143

8- ELECTRICAL EQUIPMENT	145
--------------------------------	------------

8.1- Generators	145
8.1.1- Synchronous generators	145
8.1.2- Asynchronous generators	148
8.2- Electrical installations	149
8.2.1- Main transformer	149
8.2.2- Switchgear	150
8.2.3- Control equipment	150
8.2.4- Electrical protection	150

8.3- Control system considerations	151
8.3.1- Introduction	151
8.3.2- Speed control	152
8.3.3- Water level regulation	152
8.3.4- Generator output power control	153
8.3.5- Voltage and power factor regulation	153
8.3.6- Flood level control	153
8.3.7- Blade control of Kaplan turbines	153
8.3.8- Analytical representation	154
8.3.9- Switchyard	154

9- ENVIRONMENT 155

9.1- Environmental impact assessment	155
9.2- Identification of impacts	157
9.3- Activities associated with hydropower plants	161
9.4- Reference criteria to support environmental studies	163
9.5- Safety	163

10- ECONOMIC ANALYSIS 167

10.1- Introduction. Costs and benefits	167
10.2- Economic analysis	172
10.2.1- Introduction	172
10.2.2- Constant market prices system concept	172
10.2.3- Discount rate and present value concept	173
10.2.4- Economic indexes	181
10.2.4.1- Basic considerations	181
10.2.4.2- Net Present Value (NPV)	184
10.2.4.3- Benefit/cost ratio (B/C)	184
10.2.4.4- Internal Rate of Return (IRR)	185
10.2.4.5- Average Price of the kWh (AP)	187
10.2.4.6- Payback period (T)	188
10.2.5- Sensitivity analysis	188
10.2.6- Application example	188

BIBLIOGRAPHY 191

SYMBOLOLOGY

A	flow cross section (m ²).
a	net interval between two bars of a rack of a rack (m).
AF	actualisation factor (-).
A _f	horizontal area of the forebay (m ²).
A _{GS}	area of the watershed of the stream-gauging station (km ²).
A _{HP}	area of the watershed of the hydropower scheme (km ²).
AP	average price of the kWh (cost unit).
A _{RAC}	cross-section of the air vessel (m ²).
A _s	superficial area (m ²).
B/C	benefit/cost ratio (-).
B _f	forebay width (m).
BH	hydraulic torque (N.m).
BR	resistant or electrical torque (N.m).
B _s	superficial width (m).
b	length bar of a rack in the flow direction (m).
b _o	inlet runner height (m).
C	discharge coefficient (-); runner speed (m/s); parameter of flow approximation (-); Hazen-Williams coefficient (m ^{0.37} /s); present value of the capital costs (cost unit).

c	thickness or diameter of a rack bar (m); wave celerity in open channel or in a pressure pipe (m/s); specific runner speed (-).
C_d	discharge coefficient of a rack (-).
C_E	equivalent tariff of energy (cost unit).
CEC	coefficient of energy cost (-).
C_i	capital costs in year i (cost unit).
CMP	pipe material coefficient (-).
C_p	pipe material cost per unit weight (cost unit).
C_r	ratio area between free and total or gross rack area (-).
C_{vd}	weir discharge coefficient (-).
C_{valv}	valve discharge coefficient (-).
D	pipe diameter (m); average number of days per year during which mean daily flows are equal or greater than a given flow (days/year).
d	solid particle density (ratio between specific weight of a particle γ_p and of the water γ , $d = \frac{\gamma_p}{\gamma}$ (-); opening of the intake (m).
$D_1; D_2$	inlet and outlet diameter of a turbine runner (m).
D_{50}	reference dimension of a settling solid particle (m).
D_N	nozzle diameter (m).
D_o	optimum diameter or economic diameter (m).
E	Young's modulus of elasticity of the conduit walls (N/m^2); Euler number (-).
E_b	modulus of elasticity of rack bars (N/m^2).
\bar{E}	mean annual energy production (GWh).
f	Darcy-Weisbach friction factor (-); frequency of the electric grid (Hz).
fb	rack bar oscillation frequency (Hz).
F_{r0}	Froude number of the flow (-).
F_{r1}	Froude number related to the wave propagation of a bore (-).
fs	swirl frequency (Hz).
g	gravitational acceleration (m/s^2).
H	annual flow depth (mm); specific energy flow (m); head (m); head over the rack (m); piezometric head (m).
h	water depth (canal) (m).
H_a	atmospheric piezometric head (m).
H_d	downstream water level or outlet level added by kinetic head (m).
H_F	final head (energy) (m).
h_G	depth of the gravity centre (m).

H_g	available gravity potential head or gross head (m).
H_{GS}^i	annual flow depth over the watershed of the stream-gauging station in year i (mm).
$H_{g,c}$	piezometric head in the junction section of the surge tank (m).
H_{HP}^i	annual flow depth over the watershed of the hydropower scheme in year i (mm).
H_o	net head (m); head over the spillway (m).
H_{RAC}	piezometric head in the junction section of the air vessel (m).
H_p	piezometric head at any conveyance system section (m).
H_r	upstream level variation (m).
H_s	freeboard between the N_{river} and the nozzle axe for turbine with vertical shaft, and between water level of the river or rejection canal and the lowest runner point for turbines with horizontal shaft (m).
I	rotating mass inertia ($kg\ m^2$).
i	time step index (-).
I_i	inflow to the scheme water intake during time step i (m^3/s).
IRR	internal rate of return (%).
β	angle between runner blades and rotational speed C direction (degree).
α	angle between the velocity vector (V) and the rotational speed (C) (degree).
J	hydraulic headloss gradient (-).
J_{ch}	hydraulic grade line of the interior pipe of the differential surge tank (-).
K	fluid bulk modulus of elasticity (N/m^2); pipe Gauckler-Manning-Strickler coefficient ($m^{1/3}/s$).
k	absolute pipe roughness (m); trash cleaning coefficient (-); index day (-); overspeed coefficient (-).
K_{ch}	singular head loss coefficient in junction section of the surge tank (-).
K_{or}	singular orifice' head loss coefficient (-).
K_{RAC}	inlet or outlet singular head loss coefficient of the air vessel (-).
k_u	peripheral speed coefficient (-).
K_{u1}, K_{u2}	numeric factors to unit conversion (-).
K_α	coefficient that depends upon the angle α of the conical needle end (-).
L	distance between rack bar supports (m); pipe length (m); length of the hydraulic circuit (m); weir length (m); length of the sedimentation basin (m); field inductance (-).
L_{desc}	perimeter of the internal weir in a differential surge tank (m).
L_f	forebay length (m).

n	rotational speed (r.p.m.); period with simultaneous annual records of precipitation depths and of flows depths over the watershed of the stream-gauging station (-); time step index (-); polytropic coefficient for the air behaviour inside a vessel (-); number of the project lifetime periods (generally measured in years).
N	number of nozzles of a Pelton turbine(-).
N_f	free surface level in a forebay (m).
n_{GS}	period with records either of mean daily flows at the stream-gauging station or of annual precipitation depths in rain gauges located inside or near the station watershed (years).
n_{HS}	period with records of annual precipitation depths in rain gauges located inside or near the hydropower scheme watershed (years).
N_{noz}	nozzle level (m).
n_o	nominal rotational speed (r.p.m.).
NPV	net present value (cost unit).
N_{res}	reservoir water level (m).
N_{river}	water level at downstream powerhouse (m).
N_s	specific speed (r.p.m.).
O	present value of the annual operation costs (cost unit).
O_i	discharge over the spillway during time step i (m ³ /s).
O_j	annual operation cost for year j (cost unit).
P	annual precipitation depth (mm); installed power (kW); present value of the reposition costs (cost unit).
p	number of alternator pairs of polos (-).
$\frac{P_{atm}}{\gamma}$	atmospheric local head (m).
P_{GS}^i	precipitation depth over the watershed of the stream-gauging station in year i (mm).
\bar{P}_{GS}	mean annual precipitation depth in the watershed of the stream-gauging station (mm).
P_{HP}^i	precipitation depth over the watershed of the hydropower scheme in year i (mm).
\bar{P}_{HP}	mean annual precipitation depth in the watershed of the hydropower scheme (mm).
P_m	reposition cost in year m (cost unit).
P_o	hydraulic linear growing power (kW).
P_t	installed turbine power (kW).
PV	present value (cost unit).

Q	turbine discharge (m ³ /s); mean daily flow (m ³ /s); spillway discharge (m ³ /s).
q	unit discharge over an inlet bottom rack (m ² /s).
Q _d	design discharge (m ³ /s).
q _d	unit discharge through the rack (m ² /s).
q _{GSK} ⁱ	mean daily flow measured in the stream-gauging station during day k of the year i (m ³ /s).
q _{HPK} ⁱ	estimate, for the water intake of the hydropower scheme, of the mean daily flow during day k of year i (m ³ /s).
Q _{in}	forebay inflow (m ³ /s).
Q _{inlet}	inlet discharge (m ³ /s).
Q _{max}	design discharge of the hydropower scheme (m ³ /s).
Q _{min}	minimum discharge compatible with turbine operation (m ³ /s).
Q _{mod}	modulus discharge or average of the mean daily flows (m ³ /s).
Q _{modHP}	modulus discharge at the intake of the hydropower scheme (m ³ /s).
Q _{noz}	maximum discharge of each nozzle (m ³ /s).
Q _{out}	forebay outflow, over the weir and for penstock (m ³ /s).
Q _{outlet}	outlet discharge from the reservoir (m ³ /s).
Q _p	discharge at any conveyance system section (m ³ /s);
Q _{RAC}	discharge into or out of the air vessel (m ³ /s).
Q _{res}	reserved discharge (m ³ /s).
Q _{resi}	reserved discharge during time step i (m ³ /s).
Q _{RW}	discharge at runaway conditions (m ³ /s).
Q _{T+V}	turbine discharge and the valve discharge (m ³ /s).
R	present value of the revenues (cost unit).
\bar{R}	mean annual benefit (cost unit).
r	radial distance (m); discount rate (%).
R _e	Reynolds number (for sedimentation: $R_e = \frac{V_s D_{50}}{\nu}$; for pressurised flow $R_e = \frac{U D}{\nu}$) (-).
R _H	hydraulic radius (m).
R _j	revenues in year j (cost unit).
S	flow cross section of the nozzle (m ²); cross-section of the rack (m ²); submergence (m); elastic wave transmission (%).
s	bottom slope (-).
St	Strouhal number (-).

T	payback period (years).
t	time (s); pipe wall thickness (m).
T _C	guide vane closing discharge time (s).
T _E	elastic time constant (s).
t _i	instant i Δt after instant t ₀ (s).
T _m	machine starting time (s).
T _o	average annual time operation (h).
t ₀	initial instant of numerical simulation (s).
T _s	sedimentation time (s).
T _{scour}	scour time of a particle (s).
$\frac{t_v}{\gamma}$	vapour head (m).
T _W	hydraulic inertia time constant (s).
U	mean velocity of the flow (m/s). V absolute flow velocity in the turbine runner (m/s); flow velocity (m/s).
v	unit absolute flow velocity (-).
V _o	flow velocity (m/s).
V _s	sedimentation velocity (m/s).
V ² /2g	kinetic energy head (m).
\bar{V}	reservoir volume (m ³).
\bar{V}	maximum mean annual volume utilised in the energy production (hm ³).
\bar{V}_{GS}	mean annual flow volume at the stream-gauging station (hm ³).
\bar{V}_{HP}	mean annual flow volumes at the water intake of the hydropower scheme (hm ³).
\bar{V}_{RAC}	air volume inside the air vessel (m ³).
W	relative flow (or meridian) velocity in the turbine runner (m/s).
w	unit relative velocity (-).
x	distance along the canal bottom or the pipe axis (m); flow and rack direction (-).
Z _d	water level at the tailrace (m).
Z _u	water level at the intake (m).
z _{ch}	water level in the surge tank (m).
Z _{RAC}	water level in the air vessel (m).
α	angle with horizontal (degree); Coriolis coefficient (-).
β	coefficient of the rack bar (-).

ε	relative roughness (-).
ω	angular rotating speed (rad/s).
ν	kinematic viscosity of the water (m^2/s) ($\nu=10^{-6} \text{ m}^2/\text{s}$).
ρ	specific mass of the fluid (kg/m^3).
γ	specific weight of fluid (N/m^3) (e.g. for water = $9800 \text{ N}/\text{m}^3$).
γ_b	specific weight (N/m^3) of the bar material (e.g for steel $\gamma_b=78000 \text{ N}/\text{m}^3$).
γ_p	specific weight of the pipe material (N/m^3).
γ_w	specific weight of the water (N/m^3).
η	turbine-generator efficiency (-).
η_i	efficiency of the powerhouse during time step i (-).
ΔE_i	energy production during time step i (GWh).
ΔH	head loss (m).
Δh	constant net head (m).
ΔH_f	pipe or canal friction loss (m).
Δh_i	constant net head during time step i (m).
ΔH_M	maximum transient overpressure (m).
ΔH_T	total head losses in the hydraulic circuit (m).
Δp	pressure difference between two sections (N/m^2).
ΔS	variation of the volume of water stored in the reservoir during a time step of the simulation program (hm^3).
Δt	time step of the simulation program (s).
$\Delta \nabla_i$	turbined volume during time step i (hm^3).
δ	output power angular non-phase (degree).
θ	open degree of the nozzle (degree); angle of the canal with horizontal (degree).
ψ	non-dimensional parameter that depends upon the elastic properties of the conduit (-).
ξ	singular loss coefficient that depends on the geometry of the singularity (-).
σ	dynamic depression coefficient or Thoma coefficient (-); allowable stress of pipe wall material (N/cm^2).

Subscripts

1 and 2	upstream and downstream of wave front in a canal; inlet and outlet of a runner.
m; p	model and prototype.

1

HYDROENERGY

**Helena Ramos
A. Betâmio de Almeida**

1.1- Renewable energy concept

The renewable energy concept is basically associated to the following remarks:

- Inexhaustible energetic sources, in spite of being limited or conditioned;
- Low polluted energy with small environmental impacts;
- Relevant component of a sustainable development.

Nowadays, the policy in most of the countries is devoted to assure additional generating energy from renewable, in particular with small hydropower schemes, which can contribute with a cheap source, as well as to encourage internationally competitive small industries across a wide range of new energy sources options and technologies.

The hydraulic power is one of the oldest energy sources of the mankind, namely for irrigation and industry. Nowadays, small hydro is one of the most valuable answers to the question of how to offer to isolated rural communities the benefits of electrification and the progress associated with it, as well as to improve the quality of life. The hydroelectric power plant utilises a natural or artificial fall of

a river. The water flow energy is used to turn the wheel of a turbine and returns again to the river. This type of electricity production does not consume water, thus it is usually considered a renewable energy source. The flow will continue to fall downhill and the water will continue to be available as a resource for men and environment needs, thanks to the natural hydrologic cycle.

The economic utilisation of renewable energies is now based on new technologies and on environmental protection techniques. Small hydropower, with its multiple advantages, as a decentralised, low-cost and reliable form of energy, is in the forefront of many developing countries to achieve energy self-sufficiency.

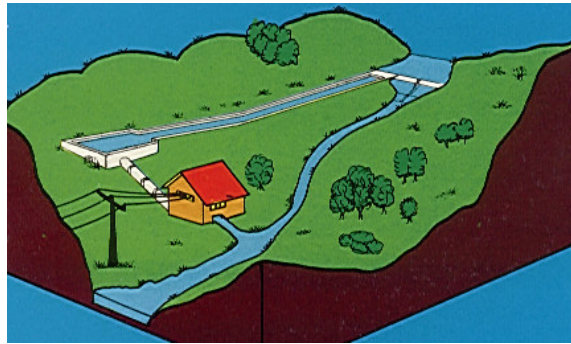


Fig. 1.1 - Typical scheme of a renewable energy source based on the waterpower.

For environmental protection it must be considered, in each small hydro project, the ecological or reserved flow in order to protect downstream the wildlife habitats and to encourage or maintain the migration through fish-passages. It will be enhanced the main advantages to develop small hydro comparing with other electricity sources:

- It saves consumption of fossil, fuel, and firewood.
- It is self-sufficient without the need of fuel importation.
- It does not contribute for environment damages by resettlement, as it occurs with large dams and reservoirs.
- It can be a good private capital investment in developing or developed countries.

- It offers a decentralised electrification at a low running cost and with long life.

A small-scale project can also induce tourist activities and can benefit both rural and small urban areas with a friendly water scenario.

1.2- Hydroelectricity: energy conversion and hydropower principles

The theory of hydropower generation is based on the conversion of the hydraulic potential energy of a flow into electric energy, which corresponds to a differential net head. The energy of the flow is associated to the gravity energy through natural or artificially created topographic water falls in rivers or through hydraulic conveyance systems, composed by pressurised pipes or penstocks or by mixed hydraulic conveyance system composed by canal and penstocks.

According to the principle of conservation of energy, the energy balance of a steady flow from A to B will obey to the following relationship:

$$\underbrace{Z_A + \frac{p_A}{\rho g} + \frac{\alpha_A U_A^2}{2g}}_{H_A} = \underbrace{Z_B + \frac{p_B}{\rho g} + \frac{\alpha_B U_B^2}{2g}}_{H_B} + \Delta H_{AB} \quad (1.1)$$

where Z_A and Z_B (m) are the elevations above a datum plane, p_A and p_B (Pa) are the pressures at the centres of gravity of the flow cross sections at A and B, U_A and U_B (m/s) are the average flow velocities respectively at A and B, ρ (kg/m^3) is the water density, g (m/s^2) is the gravity acceleration and α_A and α_B are numerical coefficients accounting for the non-uniform velocity distribution. Equation (1.1) express that the difference between total heads at A (H_A) and B (H_B) equals the headloss ΔH_{AB} between the two flow cross sections, where the head is the total flow energy by the weight of the flowing water.

For free surface flow, equation (1.1) simplifies to the following form:

$$\Delta H_{AB} = N_A - N_B + (\alpha_A U_A^2 - \alpha_B U_B^2) / 2g \quad (1.2)$$

where N_A and N_B (m) are respectively the elevations of the free surface of cross sections at A and B. Equation (1.2) express that the dissipated head equals the head difference between A and B. Should the difference $\alpha_A U_A^2 - \alpha_B U_B^2$ be very small or equal and the dissipated head equals the difference between elevations N_A and N_B .

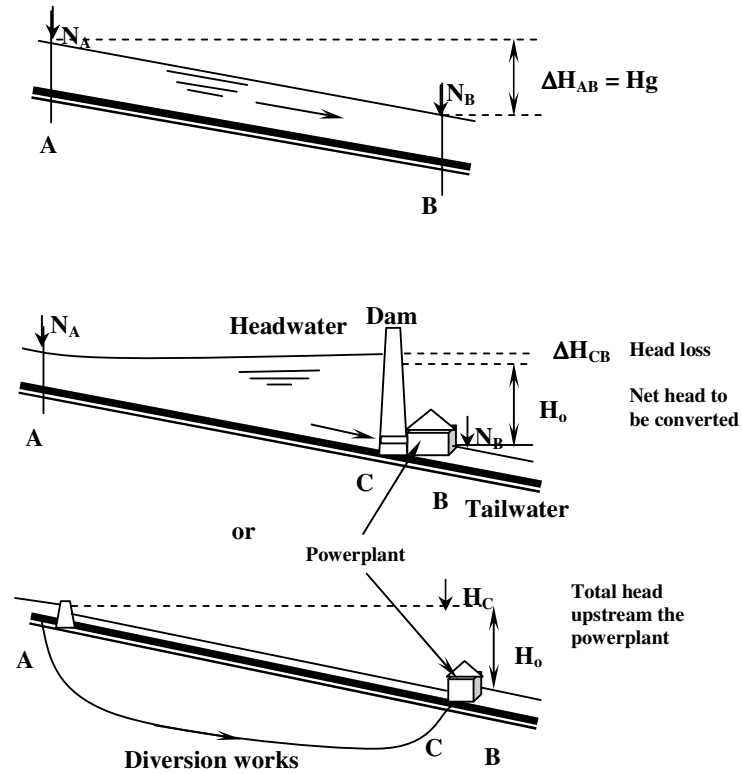


Fig. 1.2 – Different types of hydropower schemes.

The basic hydropower principle is based on the conversion of H_o or net head, the large part of the naturally dissipated head along a watercourse into mechanical and electrical energy (Figure 1.2):

$$H_o = H_g - \Delta H_{CB} \quad (1.3)$$

implying that the headloss ΔH_{CB} along the hydraulic conveyance circuit of the scheme will be much less than H_g . The headloss between A and C are artificially reduced in order to convert H_o into electricity.

The net head of a small hydropower plant built at section B can be created in quite a number of ways. Two fundamental ways are the following ones:

- to built a dam across a stream to increase the water level just above the powerplant;
- to divert part of the stream, with a minimum of headloss, to just above the powerplant built far away the dam.

The net flow power P_o and the corresponding energy E_o over an interval time Δt (s) of the hydropower plant will be respectively:

$$\begin{aligned} P_o &= \rho g Q H_o \\ E_o &= \rho g Q H_o \Delta t \end{aligned} \quad (1.4)$$

where Q (m^3/s) is the constant discharge diverted to the powerplant.

The final useful head delivered to the electrical network is smaller than the available gross head:

$$H_F = \eta H_g \quad (1.5)$$

where η is the global efficiency, resulting of the multiplication of partial efficiencies from the successive transport and conversion phases ($\eta < 1$).

Along the same watercourse the total gross head can be profited by a multistage scheme with several powerplants (i.e. cascade system).

The hydroelectricity production is an energy conversion process, in which the water is the vehicle of transmission and transformation of the gravity potential energy into mechanical and electric energy by the turbine-generator set installed in the powerhouse. The water is led through pipes and/or canals to the turbine, which turn the shaft of the generator to produce electric energy. From the powerhouse, after a convenient voltage transformation, transmission lines carry electricity out to communities or to the national grid.

In a river, the available potential energy or gross head (H_g) will be converted in a system through the following components:

- Reservoir: constitute a storage form of the available potential energy and creates the conditions for water diversion through the intake.
- Conveyance system, including the intake, conveyance canal, penstock, galleries and tailrace or outlet where part of the available energy is converted into kinetic energy; another part is transformed into reversible flow work capacity (pressure head) and another part is dissipated in heat (by fluid viscosity) resulting in the net or useful head.
- Hydraulic turbine: where the net head is converted into rotor speed of the turbomachine.
- Generator rotor: the mechanical energy on the shaft maintain the speed of the rotor and it is transformed in electric energy according to electromagnetic laws.
- Line to link to the grid: the electric energy is driven and transformed in order to connect to the grid for transportation to long distances and distribution.

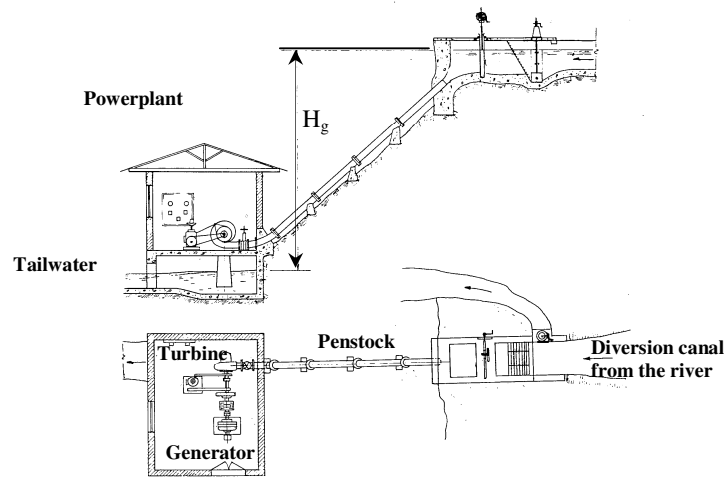


Fig.1.3 – Small old hydroelectric scheme
(adapted from MACINTYRE, 1983).

The hydraulic engineering intervention has a major contribution in these types of projects, namely in the planning, conception, study, design, building and

exploitation phases of a project that involves multidisciplinary teams, with technicians and experts in several domains: civil, mechanical, electric engineering and specialists in geologic and environmental sciences, among others.

Adequate head and flow are necessary requirements for hydro generation. For the head characterisation is necessary to consult available maps and confirm “in situ” through field visits and surveys about the potential sites. Typically, maps information need to be complemented by field surveying along the hydraulic circuit. Such maps are particularly important for positioning the structural components of the system. In what concerns the flow, a hydrologic study must be carried out. After a first approach to power potential calculation, estimation of energy output, identification of civil works and other critical issues (e.g. environmental and social constraints), a technical-economic feasibility study must be performed.

In small hydroelectricity, the hydraulic structures will be much less complex than in large hydroelectricity. The hydraulic conveyance circuit can be integrated in other components for multiple purposes (e.g. irrigation or water supply schemes). Sometimes, small hydropower plants can be very unconventional both in design concept and in components (e.g. the turbines can replace pressure reducing valves or other types of localised dissipation of excess head). The small hydroelectricity depends mainly upon the local and regional characteristics. It has low environmental impacts but also a relatively less guaranteed energy production due to the very small storage volume in the upstream reservoir.

1.3- Environmental considerations

Hydropower plants produce no carbon dioxide, sulphur oxides or nitrous oxides, no air emissions and no solid or liquid wastes. Nevertheless there are impacts by retaining water and inducing sediment to settle down, as well as by obstructing the fish passage and upsetting the wildlife habitats.

The world concern about global planet warming phenomenon essentially due to CO₂, SO₂, NO_x emissions in energy generating process with fossil fuels and the problem for the future of nuclear wastes, more and more will be emphasised the advantages of energy production through renewable sources. Hydropower represents an important environmental benefit to aid sustainable development

because there is no release of carbon dioxide that contributes to ozone depletion and global warming. In the future a penalty cost by combustion of fossil sources should be applied. Being hydroelectric energy 11.7% from total generated energy in European Union, the small hydropower plants can still occupy a relevant contribution around the world, being estimated that the energy production through renewable sources can increase three more times than the values obtained in ninety decade.

However the abstraction of water from a watercourse must be controlled in order to avoid serious damages to aquatic biota between the intake and the tailrace. In fact, a variable residual flow should remain along the year between the diversion dam and the powerplant. This residual flow, also known by reserved flow, compensation or ecological flow, must be environmentally acceptable. In particular, in seasons of low flow the residual flow is very important in order to keep a steady regime to warrant the aquatic natural development and the water quality. In this way, a compromise has therefore to be sought to ensure the maximum energy production with maintenance of the equilibrium of the aquatic system.

The small hydropower does not require high dams because the majority of them are run-of-river schemes, meaning simply that the turbine only generates when there is available water. A minimum daily storage and flow regulation is typically guaranteed. When the river dries up the generation ceases.

2

GENERAL TYPES OF SMALL HYDROPOWER PLANTS

**Helena Ramos
A. Betâmio de Almeida**

2.1- Classification of hydropower plants

The classification of hydropower plants can be based on different factors:

- head: low (less than 50 m); medium (between 50 and 250 m); high (greater than 250 m);
- exploitation and storage: with daily (or seasonal) flow regulation (reservoir type); without flow regulation (run-of-the-river type);
- conveyance system: pressurised (penstock); mixed circuit (canal and penstock);
- powerhouse site: dam or diversion scheme;
- energy conversion mode: turbinning or reversible pumping-turbinning;
- type of turbines: impulse, reaction and reversible;
- installed power: micro ($P_t < 100$ kW); mini (100 kW $< P_t < 500$ kW); small (500 kW $< P_t < 10$ MW).

The classification based on the power is very important because is an institutional and legislative reference (ESHA, 1994).

The present concept of small-scale hydropower is neither a miniature of large hydro nor a simple repetition of old techniques, but it requires advanced and modern studies and properly adaptations for different solutions.

2.2- Hydropower schemes

In Europe, the development of small hydroelectricity has known a new growing since the seventies, due to the world energy crisis and concerns of negative environmental impacts, as well as due to the development of automation and remote control (i.e. abandoned exploitation) and the standardisation of the equipment (e.g. for turbines and generators). The small hydropower can fill the gap of the decentralised production, for instance for private or municipal activity production for sale to national electric grid or for supply industries, rural or isolated zones, improving their development.

Small hydropower is typically associated with rivers with catchement areas of less than 200 km². The net head and the plant discharge are two important parameters to be considered in the small hydropower design. The reservoir for discharge regulation allows, under certain limits, to make independent the energy production from river flow variations. With an adequate reservoir volume it is possible to program the power generation in order to be able to satisfy the demand or to sale energy in rush hours, when its price is valorised. Nevertheless, a small hydropower plant is rarely compatible with a large reservoir due to economic constrains. However, it can be shared with other types of water uses (e.g. irrigation systems, water supply systems, flood protection structures or discharge regulation structures), and can generate energy whenever excess discharge exists.

According to the mode of net head characterisation, the following main hydropower schemes can be identified:

Dam scheme - The dam is used to concentrate the head, which raises the upstream water level. In this way, the powerhouse can be placed either at downstream incorporated inside the dam.



Fig. 2.1 – Typical scheme of dam used to raise head for intake diversion and small water storage.

Diversion scheme – The utilisation of diversion structures, such as canals, tunnels or galleries or low-pressure conduits allows the head gain. A small dam can be used with long hydraulic circuit in order to obtain the net head at downstream end.

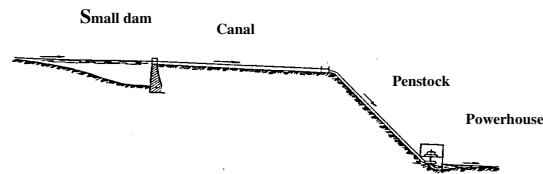


Fig. 2.2 – Typical scheme with a mixed hydraulic conveyance circuit with a canal and a penstock.

Mixed scheme - A dam can partly raise the net head and a long hydraulic conveyance circuit will raise the other part.

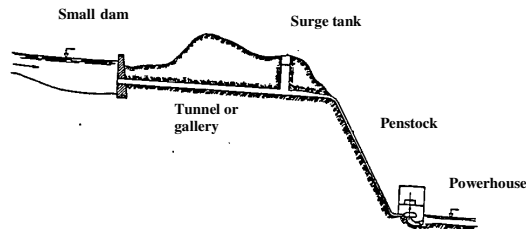


Fig. 2.3 – Typical scheme of a low pressure tunnel and a penstock as a diversion structure.

According to the mode of discharge exploitation, the following hydropower scheme types can be identified:

Run-of-the-river scheme – Power is generated without inflow regulation. It is a common scheme applied to mini or micro hydropower plants.

Daily regulation scheme – Power is generated according to the natural fluctuation of the daily demand, the water being stored in a regulating pond or small reservoir at off-peak times and discharging it at peak hours, resulting a bigger energy output comparatively to without regulation capability.

Seasonal regulation scheme – This scheme is commonly applied in larger power plants, which needs a reservoir to store water in rainy season and discharge it in the dry season, enhancing a constant energy all year (normally it is not a common scheme in small power plants).

Cascade scheme – The cascade scheme is a typical exploitation of the river, in order to make the best use of the river falls.



Fig. 2.4 – Example of a cascade scheme of large dams (courtesy of EDP).

In case of existing water intake in differential canal (e.g. irrigation system) it can be used a siphon, which penstock pass over the dam without affecting it. Nevertheless, for the turbine start-up it is necessary to use a vacuum pump and for the stoppage, the system must have installed an air valve. The gross head is equal to the difference between upstream and downstream water levels (normally for schemes with heads smaller than 10 m and discharges between 1 to 50 m³/s). These solutions can avoid onerous civil works.

2.3- Site location

A hydro generation site is mainly conditioned by head and flow requirements. In a preliminary stage, the power potential and energy output, the estimation of the need works, and the economic feasibility, must be studied.

Depending upon the geographical characteristics of the available site, the hydropower scheme can have high or low head. High head schemes can be less expensive because, the required hydraulic equipment (e.g. dam, turbines and valves) will be smaller. Nevertheless, the topographical and geotechnical characteristics could constrain the type and the alignment for the conveyance system. The nature of the streambed, bends and the access, stability of the soil, are important condition factors to take into account in the site choice.

3

HYDROPOWER AND WATER USES

Helena Ramos
A. Betâmio de Almeida

3.1- Power generation and different sector-users

Water resources can be used in different ways to serve the society, taking into consideration all demands arising from different social and economic sector-users. Meanwhile, the exploitation and the utilisation of water resources aim to obtain the maximum benefit that should be controlled in order to reduce natural hazards and environmental impacts. Small hydropower can be associated with different water uses:

Power generation and water supply - Water conveyance systems to feed a water supply of a town through a pressure pipe, from a reservoir to a treatment plant are, e.g., equipped by a valve system to dissipate the excess energy. In that case, a turbine can substitute this dissipation system. However, this solution demands the installation of a by-pass in order to guarantee the flow continuity in case of turbine stoppage. The turbo-generator units will work according to special regulation and control systems under the command of a water treatment plant. They can be built in canals or on water supply high pressure pipes (e.g. micro turbines replacing pressure-reducing valves to take the advantages of excess head – Figure 3.1).

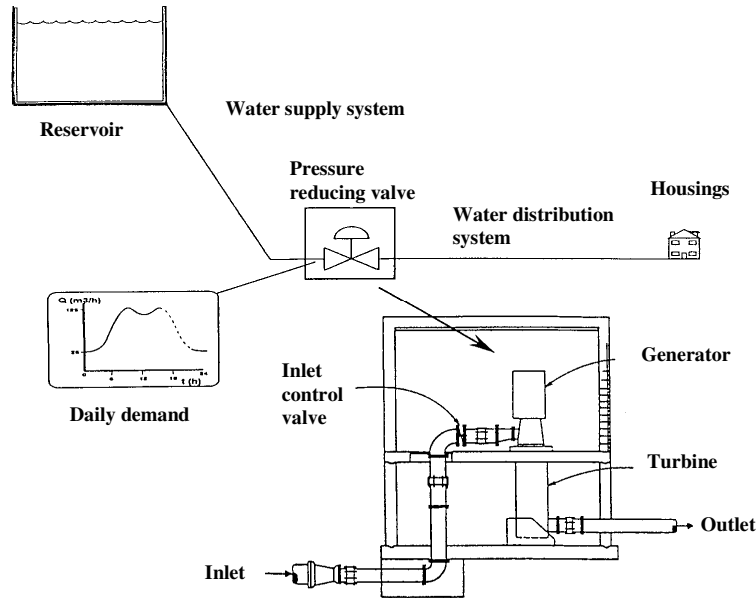


Fig. 3.1 – Example of application a micro-turbine as a pressure reducing valve in a water supply system to yield power.

Power generation and irrigation – It is possible to install a powerplant in irrigation canal or in a lateral by-pass canal, with a small penstock, in order to take the advantage of the head created by an upstream dam. In several irrigation schemes there is the opportunity to insert a small powerplant and to convert, permanently or seasonally, the excess head into electric energy:

- should an upstream dam imposes an excess head relatively to the downstream delivery canal, a turbine scheme can be envisaged in order to replace, totally or partially, the dissipation structure;
- along an irrigation canal, significant topographic uneven can be used by a diversion scheme out of irrigation period (e.g. whenever exists uneven between canal branches, with different water levels).



Fig. 3.2 – Diversion canal for a powerplant.

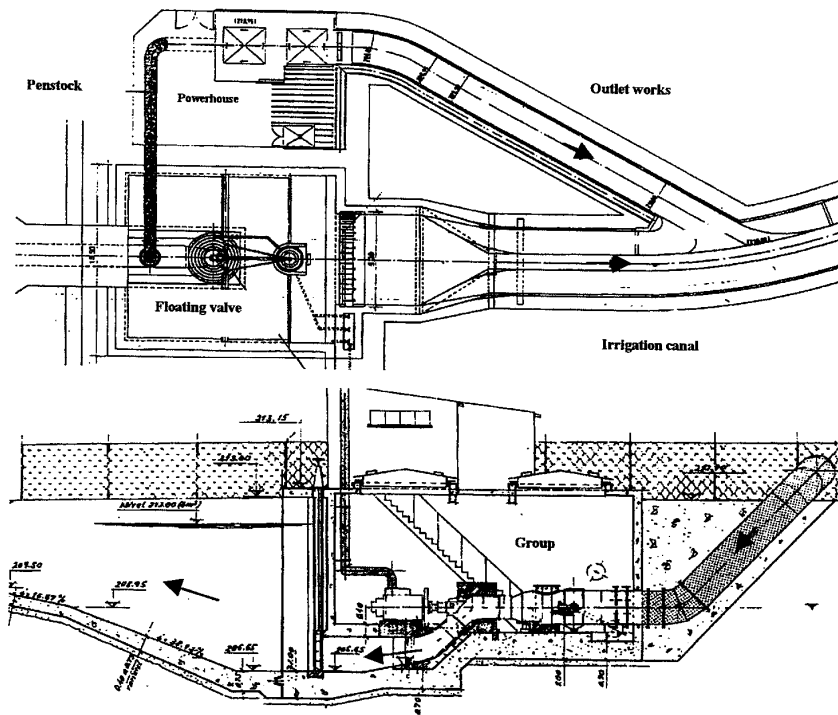


Fig. 3.3 – Scheme of a power generation in an irrigation system.
Diversion works for the powerhouse.

Power generation and flood prevention - Dams can be used to prevent floods through the creation of reservoirs that should be emptied ahead of a rainy season, although allocating a certain volume in the reservoir for power generation. An integrated management of reservoirs operation tends, nowadays, to reduce the energy loss to a minimum and to benefit water users. Nevertheless in small hydropower plants this is not enough significant.



Fig. 3.4 – Visualisation of the dry zone to absorb floods.

Power generation and fish protection - Damming a river impedes fish migration and the passage between downstream to upstream sides and vice-versa, when it is applied, being necessary to built fish-passes and to assure the navigation, in case of it is applied. However, the reservoir would create a special spot for recreation and breeding conditions for aqua-culture.



Fig. 3.5 – Fish-ladders of pool-type with bottom orifice and top weir.

3.2- The benefit of small hydropower plant

Automatic isolated or autonomous small hydro plants provide an alternative solution to electric grid extension that serves widely scattered communities, as a high efficiency power supplement in urban areas, small industries and for domestic purposes. Namely, there is a great disparity between urban and isolated rural zones, with a consequent imbalance in the accessibility of energy.

Local conditions limit the adequate alternatives where is feasible to invest. The global profit of watercourse management based on integrated programs related to the water uses can indicate the best sites to install these types of plants. It is convenient to know about the existing or future water uses and the needs whether for water supply, either irrigation, or energy and for tourist purposes in order to avoid incompatibilities between different needs.

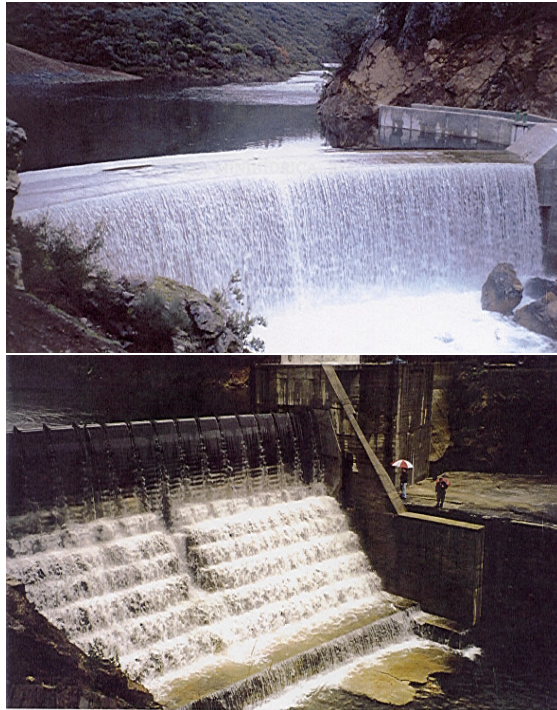


Fig. 3.6 – The effect on the landscape of low dams.

4

HYDROLOGY

M. Manuela Portela

4.1- Introduction. Scope of the studies

The main objectives of the hydrologic study of a small hydropower scheme are the characterisation of:

- the runoff at the water intake of the scheme in order to allow the determination of the design discharge, and, thus, the design of the water intake, of the diversion circuit and of the powerhouse, as well as the evaluation of the energy production;
- the floods or, more precisely, the peak flows, to consider in the design of the weir, of some of the diversion works and of the powerhouse (for instance, if the turbines are of the Pelton type they should be located above the water surface elevation in flood conditions, at the powerhouse outlet.

The flood characterisation involves a domain, which is not specific of small hydropower schemes. For this reason, this subject will not be treated herein. However the dam works built along the river need to be prepared for typical floods according to an accepted risk or design flood criteria based on a return period.

Most of the time it is not easy to carry out the fulfilment of the previous objectives, as there is not the required basic hydrologic data in the watershed of the hydropower scheme.

In this chapter some simple methodologies that can be applied to the hydrologic studies of small hydropower schemes when the available hydrologic data is scarce are briefly and systematically presented. The proposed methodologies partially overcome the non-existence of basic hydrologic information and with some minor approximations allow the hydrologic and energetic characterisation of the schemes under consideration.

More detailed analysis in the field either of the hydrologic studies or of their application to small hydropower schemes can be found in books of hydrology (LINSLEY and *al.*, 1985, CHOW and *al.*, 1988, MAIDMENT, 1992) and in books specially dealing with the design of small hydropower schemes (ESHA, 1994, and JIANDONG and *al.*, 1997)

4.2- Basic data required for the hydrologic study

The basic hydrologic data required for the evaluation of the energy production in a small hydropower scheme is the mean daily flow series at the scheme water intake in a period that has to be long enough in order to represent, in average, the natural flow regime. By this way, it is reasonable to assume that the errors of the estimates that result from the variability of the natural flows are minimised.

However, the small hydropower schemes, being frequently located in the upper zones of the streams, have small drainage areas for which recorded stream flow series are seldom available. For this reason, the inflows at the hydropower scheme are usually evaluated by indirect procedures. Some of these procedures utilise the transposition to the scheme water intake of the flow records in other watershed, namely in the watershed of a stream-gauging station¹.

For this last scenario, the required basic hydrologic data is next systematised. In the next items, the methodology that allows the evaluation of the energy production based on the previous data is presented. If the natural stream-flow

¹ A gauging station where records of the discharges of a stream are obtained, LANGBEIN and ISERI, 1960, p. 19.

series at the section of scheme water intake is known, some of the procedures presented in the next items become less important or, even, not applicable.

It should be pointed out that the decision of installing a stream-gauging station in an ungauged watershed is mainly a long term decision that will only produce *usable results* after several years of measures and, thereby, it is not a suitable decision for design of any small hydropower scheme. In fact, only after a long period of measurements it will be possible to achieve a series of flow records having statistics characteristics that can be considered to represent the variability of the natural flow regime. Once the small hydropower scheme is built and started to operate it is advisable to measure the stream-flows in order to confirm the design assumptions.

As mentioned, when the inflows to the hydropower scheme are not known, the evaluation of the energy production can be based on the transposition to the scheme water intake section of a mean daily flows series measured in another site, namely in a stream-gauging station having a natural flow regime similar to the one expected in the hydropower scheme.

When selecting the previous station one should guarantee that the corresponding watershed and the one of the hydropower scheme have similar areas and are close enough or both located in regions with similar hydrological behaviour, in terms of similar mean annual values of precipitation and of runoff. The hydrological comparison between the drainage areas of the stream-gauging station and of the hydropower scheme can be based on maps of annual average values of precipitation and runoff, which are published in most countries by the appropriate water authorities.

The climate and the geological constitution, as well as the vegetal cover and the human occupation and activity in the watersheds of the hydropower scheme and of the stream-gauging station should also be similar. The period with stream-flow records at the station has to be long enough, for saying, at least, twenty years.

In this conditions, it is necessary the collect the following data:

- For the stream-gauging station:
 - series of mean daily flows (in m^3/s) in a period that will be denoted by n_{GS} years;

- in each year with known mean daily flows, the annual precipitation in rain gauges located inside and near the watershed of the station.
- For the hydropower scheme:
 - series of annual precipitation in rain gauges located inside or near the watershed of the hydropower scheme. These records have to be collected for a period as long as possible and during which all the rain gauges have simultaneous data. This period will be denoted by n_{HS} .

The available precipitation data usually allows a detailed description of the spatial variability of the precipitation as the number of sample points of the corresponding gauging network is non-comparatively higher than the number of stream-flow sample points. The annual values of precipitation are generally related to hydrological years or water years² and are expressed as the uniform vertical depths of water that would accumulate on the level surface of the corresponding watershed if the precipitation remained where it fell (LINSLEY and *al.*, 1985, p. 55) – annual precipitation depths (in mm).

The quality of the collected data should be tested, at least in terms of the general consistency of the records. The most common test to evaluate the consistency of the annual precipitation data at each rain gauge is the double-mass analysis which compares the accumulated annual precipitation in a station with the concurrent accumulated values of mean precipitation for a group of surroundings stations (LINSLEY and *al.*, 1985, p. 70). If the records are consistent, the points thus achieved are displayed along a straight line (Figure 4.1).

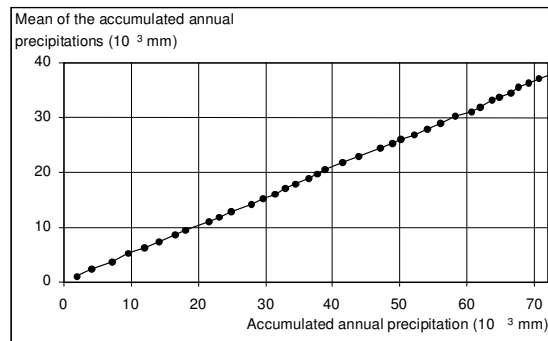


Fig. 4.1 – Double-mass test.

² Generally from October 1 to September 30 (LANGBEIN and ISERI, 1960, p. 21 and LINSLEY and *al.*, 1985, p. 116)

The application of the double-mass test to stream-flow records is, most of the time, impossible, as there are not stream-gauging stations close enough in order to allow the comparison of their records. So, the analysis of the consistency of the annual flow records is usually based on a simple-mass curve analysis: graph of the cumulative annual flows, generally as ordinates, plotted against time (namely years), as abscissa. Once more, if the representation thus achieved follows a straight line, the records are considered to be consistent (Figure 4.2).

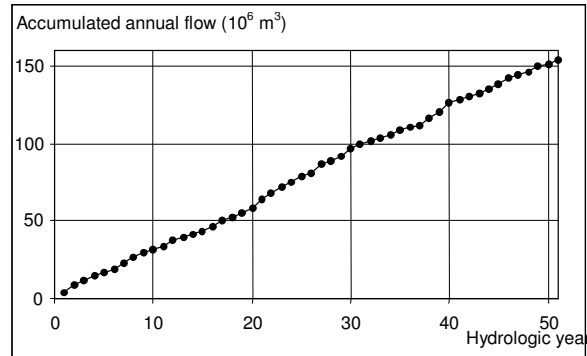


Fig. 4.2 – Simple-mass test.

There are several other procedures, most of them of statistical nature, to evaluate, in terms apart from the general consistency, the quality of the recorded series. These procedures are, however, far beyond the scope of this book.

4.3- Mean annual and mean daily flow series

4.3.1 Introduction

Based on the hydrological basic data systematised in item 4.2, either for the stream-gauging station or for the hydropower scheme, the mean daily flows series at the scheme water intake can be performed according to the following main steps:

- Evaluation of the mean annual flow depth in the watershed of the hydropower scheme, based on the mean annual precipitation depth over the same watershed and in a correlation between annual flow and annual

precipitation. This correlation has to be previously established for the watershed of the selected stream-gauging station.

- By using the previous estimate of the mean annual flow depth, transposition of the mean daily flow series measured in the stream-gauging station to the water intake of the hydropower scheme.

4.3.2 Annual precipitation and annual flow. Mean daily flow series

Once the annual records of the rain gauges located inside or near the watersheds of the stream-gauging station and of the small hydropower scheme are collected, the annual areal average depths of precipitation over the corresponding areas can be evaluated. For this purpose the Thiessen method can be applied (LINSLEY and *al.*, 1985, p. 71, SMITH, 1992, p. 3.20).

In this method the stations are plotted on a map, and connecting lines are drawn. Perpendicular bisectors of these connecting lines form polygons around each station. The sides of each polygon are the boundaries of the effective area assumed for the station. Over this area the precipitation is considered to be constant and equal to the one measured in the station. The area of each polygon within the basin boundary is determined by planimetry and is expressed as a percentage of the total area – Figure 4.3.

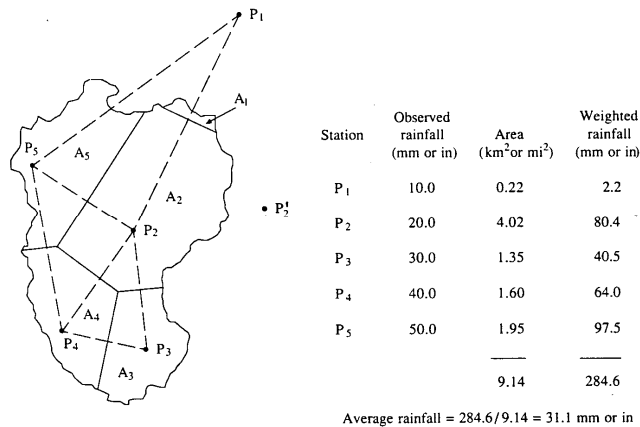


Fig. 4.3 – Areal averaging of precipitation by the Thiessen method (from CHOW *et al.*, 1988, p. 79).

Weighted average rainfall for the total area is computed by multiplying the precipitation at each station by its assigned percentage of area and totalling.

Besides the Thiessen method, other methods, as the isohyetal one, can be applied to evaluate the precipitation over each watershed. The presentation of these methods and the discussion of their comparative advantages can be found in SHAW, 1984, 209-216, LINSLEY and *al.*, 1985, p. 71, and CHOW *et al.*, 1988, p. 78-80.

If the digital elevation models of the watersheds are available, the previous work can be significantly simplified taking advantages of a GIS capabilities, where other area averaging methods, besides the Thiessen method, are implemented and easily manipulated.

Once the series of annual precipitation depth over each watershed is known, the corresponding mean annual value is obtained: \bar{P}_{GS} (mm), for the stream-gauging station, and \bar{P}_{HP} (mm), for the hydropower scheme

$$\bar{P}_{GS} = \frac{P_{GS}^i}{n_{GS}} \quad \text{and} \quad \bar{P}_{HP} = \frac{P_{HP}^i}{n_{HP}} \quad (4.1)$$

where P^i (mm) denotes the precipitation depth in year i over the watershed identified by the sub index.

According to the methodology presented herein, the evaluation of the mean annual flow depth in the watershed of the hydropower scheme requires the establishment of a relation between annual precipitation and annual flow for the watershed of the stream-gauging station.

Let P_{GS}^i (mm) denote the precipitation depth over the watershed of the stream-gauging station in year i ($i=1, \dots, n$) and H_{GS}^i (mm) the corresponding annual flow depth obtained from the records of mean daily flows according to

$$H_{GS}^i = \frac{\sum_{k=1}^{365} q_{GSk}^i}{A_{GS}} \times 86,4 \quad (4.2)$$

where A_{GS} (km^2) is the watershed area and q_{GSk}^i (m^3/s), the mean daily flow that was measured during day k of the year i .

If the plotting of the annual precipitation, P_{GS}^i , versus annual runoff, H_{GS}^i , for a common period of n years ($i = 1, \dots, n$, n being as great as possible) displays a good correlation between those two variables it will be acceptable to establish a relation expressing, by means of statistics, the observed dependency (Figure 4.4).

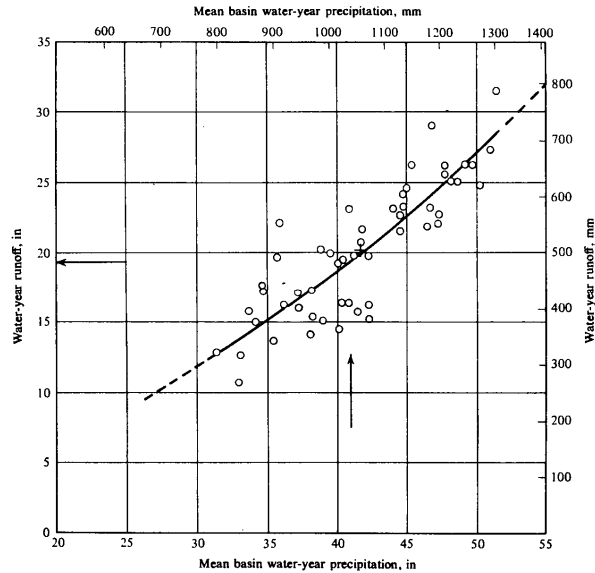


Fig. 4.4 – Relation between annual precipitation and runoff
(from LINSLEY *et al.*, 1985, p. 255).

If a linear relation is assumed (which provides the simplest relation between annual precipitation and annual runoff) the following expression can be obtained by means of a regression analysis:

$$H = \alpha P - \beta \tag{4.3}$$

where α and β are positive parameters that can be estimated from the sample of annual values of precipitation and flow by the least square method (DRAPPER

and SMITH, 1981). The parameter α should be less than one. The sub indexes of H (annual flow depth) and P (annual precipitation depth) have been omitted as (4.3) can be applied to a watershed different from the one to which the relation was established (although both watersheds should have the similarities previous pointed out).

Other types of relation between P and H can be assumed, for instances a parabolic relation

$$H = \alpha P^2 - \beta \quad (4.4)$$

where α and β should also be positive.

The choice of the most suitable relation between annual values of precipitation and flow should result from the graphical analysis of the behaviour of those two variables (Figure 4.4) and from the hydrologic specific knowledge of the region where the small hydropower scheme is going to be built.

Applying (4.3) or (4.4), or an equivalent expression, to the mean annual precipitation depth over the watershed of the hydropower scheme, \bar{P}_{HP} (mm), the corresponding mean annual flow depth, \bar{H}_{HP} (mm), is obtained.

The mean annual flow depth over the watershed of the stream-gauging station, \bar{H}_{GS} (mm), can be evaluated from the series either of the mean daily flows, q_{GSk}^i (m^3/s), or of the values of H_{GS}^i (mm) given by (4.2)

$$\bar{H}_{GS} = \frac{86,4 \times \sum_{i=1}^{n_{GS}} \left(\sum_{k=1}^{365} q_{GSk}^i \right)}{A_{GS} \times n_{GS}} = \frac{\sum_{i=1}^{n_{GS}} H_{GS}^i}{n_{GS}} \quad (4.5)$$

Let \bar{V}_{GS} (hm^3) and \bar{V}_{HP} (hm^3) denote the mean annual flow volumes at the stream-gauging station and at the water intake of the hydropower scheme, respectively

$$\bar{V}_{GS} = \frac{\bar{H}_{GS} \times A_{GS}}{1000} \quad (4.6)$$

$$\bar{V}_{HP} = \frac{\bar{H}_{HP} \times A_{HP}}{1000} \quad (4.7)$$

The transposition to the water intake of the hydropower scheme of the mean daily flow series measured at the stream-gauging station can be finally accomplished by applying the following relation:

$$q_{HPk}^i = q_{GSk}^i \frac{\bar{V}_{HP}}{\bar{V}_{GS}} \quad (4.8)$$

where q_{HPk}^i (m³/s) is the estimate, for the water intake of the hydropower scheme, of the mean daily flow during day k of year i.

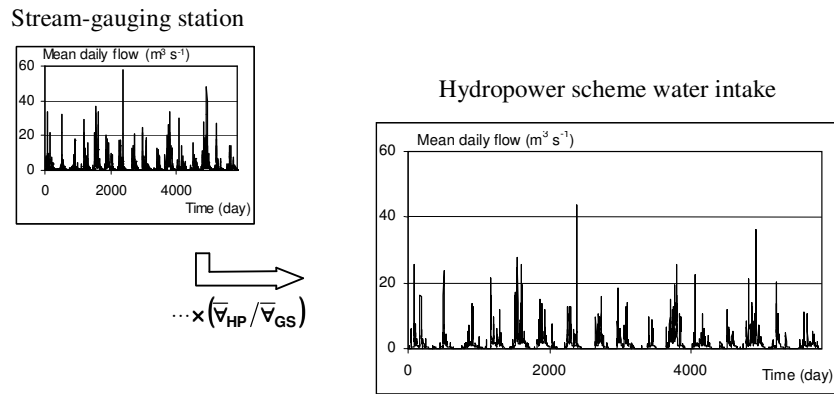


Fig. 4.5 – Obtainment of the chronological diagram of the mean daily flows at the hydropower scheme water intake.

Applying (4.8) to the flow series measured at the stream-gauging station the mean daily flow series for the water intake of the small hydropower scheme is obtained. Once this series is known the correspondent chronological diagram can be established – Figure 4.5.

The application of relation (4.8) leads to an approximated idea of the mean daily flow series at the water intake of the scheme. However, this idea is frequently the best one, as alternative daily models require much more data and are often highly complex, as the deterministic models that perform the transformation of

precipitation into runoff (rainfall-runoff process models). It should be noticed that assuming relation (4.8) is equivalent to consider that the shapes of the non-dimension chronological diagrams of the mean daily flow series³ at the stream-gauging station and at the scheme water intake are equal.

4.3.3- Mean annual flow duration curve

Two techniques are available to determine the energy potential of a hydropower site, namely, the flow duration curve method and the sequential stream-flow routing.

The evaluation of the mean annual energy production in a small hydropower scheme with a run-of-river exploitation was traditionally based on a mean annual daily flow duration curve. This curve gives for each value of the mean daily flow, Q , the average number of days per year, D , during which occurred mean daily flows equal or greater than Q (MOSLEY and McKERCHAR, 1992, p. 8.27) – Figure 4.6.

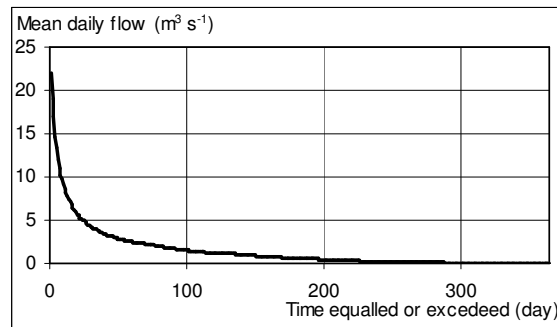


Fig.4.6- Mean annual daily flow duration curve.

Once the mean daily flows at the hydropower scheme are evaluated, the corresponding mean annual daily flow duration curve can be obtained by organising those flows by magnitude instead of chronological.

The flow duration curve can also be represented in a dimensionless form in what concerns the mean daily flow series or both these flows and the time – Figure 4.7.

³ Chronological diagram of the mean daily flows divided by the corresponding average.

In the Figure 4.6, Q_{mod} represents the modulus or the average mean daily flow derived from the known mean daily flows by summation and averaging. For the hydropower scheme $Q_{mod_{HP}}$ ($m^3 s^{-1}$) can be obtained by one of the following expressions:

$$Q_{mod_{HP}} = \frac{\sum_{i=1}^n \left(\sum_{k=1}^{365} q_{HPk}^i \right)}{365 \times n} = \frac{\bar{H}_{HP} \times A_{HP}}{0.365 \times 24 \times 3.6} = \frac{\bar{V}_{HP}}{0.365 \times 24 \times 3.6} \quad (4.9)$$

where \bar{H}_{HP} (mm) is evaluated by applying (4.3) or (4.4) or an equivalent relation, \bar{V}_{HP} (hm^3) is given by (4.7) and the meaning and units of the other variables have already been presented.

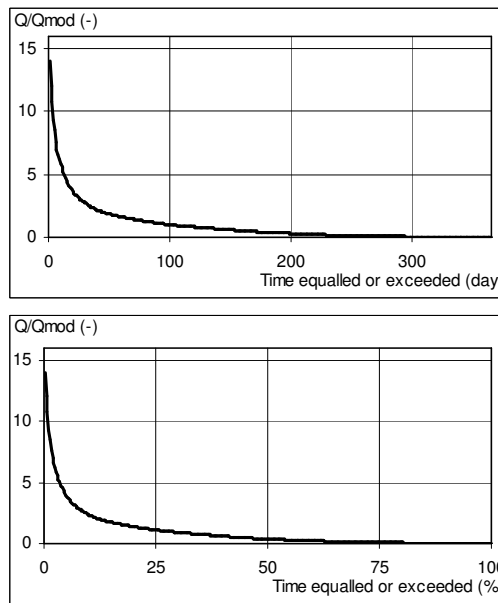


Figure 4.7 – Dimensionless forms of the mean annual flow duration curve.

Taking into account the procedures that led to the mean daily flow series at the hydropower scheme water intake, q_{HPk}^i (m^3/s), it is easy to conclude that the

dimensionless forms of the mean annual daily flow duration curves thus reached for the scheme and the one relative to the stream-gauging are equal.

Once a dimensionless mean annual daily flow duration curve is put in a dimension form by multiplying its ordinates by a modulus, Q_{mod} , the curve thus obtained will become exclusively representative of the watershed to which that modulus is referred.

The mean annual daily flow duration curve at an ungauged site was traditionally based on the adoption of a standardised regional non-dimensional duration curve. The curve selected was next synthesised by multiplying its ordinates by the hydropower scheme modulus, Q_{mod} , in order to provide its particular flow duration curve with numerical discharges.

However, the previous traditional methodology only leads to reasonable results with respect to the energy evaluation when no storage capacity is available in the hydropower scheme, that is to say, when the scheme has a pure run-of-river exploitation. At the same time and even for a run-of-river scheme, this methodology only gives the estimate of the mean annual energy production and it does not allow any simulation study in order to analyse the variability of the energy production due to the natural variability of the flows.

Regional standardised flow duration curves are generally available in all European countries. The criteria that lead to the establishment of those curves depend, however, on each specific country. When selecting a standardised curve care should be taken in order to ensure that the watershed of the hydropower scheme and the one inherent to the regional curve are similar (in terms of hydrological behaviour, climate, area, occupation and geological constitution).

The procedures proposed in this chapter, namely those that result from relation (4.8) are also regional transposition methodologies. The main advantages of these procedures comparatively to the traditional ones based on the adoption of a regional standardised mean annual flow duration curve can be systematised by:

- In what concerns the factors that determine the similarity of the flow regime at the stream-gauging station and at hydropower scheme, the methodologies presented herein allow a more rigorous selection of the daily data to be transposed.

- They provide not only a mean annual flow duration curve but also a mean daily flow series allowing the evaluation of the energy production either in a pure run-of-river scheme or in a scheme having some storage capacity, in this last case, by means of simulations algorithms.
- In a general sense, whenever mean daily flows exist it is possible to perform any simulation study. By this way, either the design of the small hydropower scheme, or the comparison of different solutions for the same become much easier.

4.4- Energy Evaluation

The evaluation of the mean annual production in a small hydropower scheme with a pure run-of-river exploitation can be accomplished only by using the mean annual flow duration curve at the scheme water intake. The simple procedures that can be adopted in this situation are next briefly described.

For this purpose, let Q_{max} denote the design discharge of the hydropower scheme. In this situation, the maximum mean annual volume, \bar{V} , that can be utilised in the energy production is represented by the dashed area of in Figure 4.8 a).

If the efficiency of the powerhouse, η , and the net head, Δh (m), are considered constants, the mean annual energy production, \bar{E} (GWh), in the scheme having the flow duration curve of Figure 4.8 a) is given by

$$\bar{E} = \frac{\bar{V} \times \Delta h}{3600 \times g \times \eta} \quad (4.10)$$

where g is the gravity acceleration ($m\ s^{-2}$) and \bar{V} is expressed in hm^3 .

Most of the time, the average turbined volume will be less than \bar{V} as no turbine can operate from zero flow to its rated discharge and, so, a minimum discharge, Q_{min} , compatible with the energy production has to be considered.

A reserved discharge (for ecological purpose but also for others purposes as irrigation or water supply) has also to be taken into account. In this situation the

mean annual turbined volume to be considered in the application of (4.10) will be the one represented by the dashed area of Figure 4.8 b).

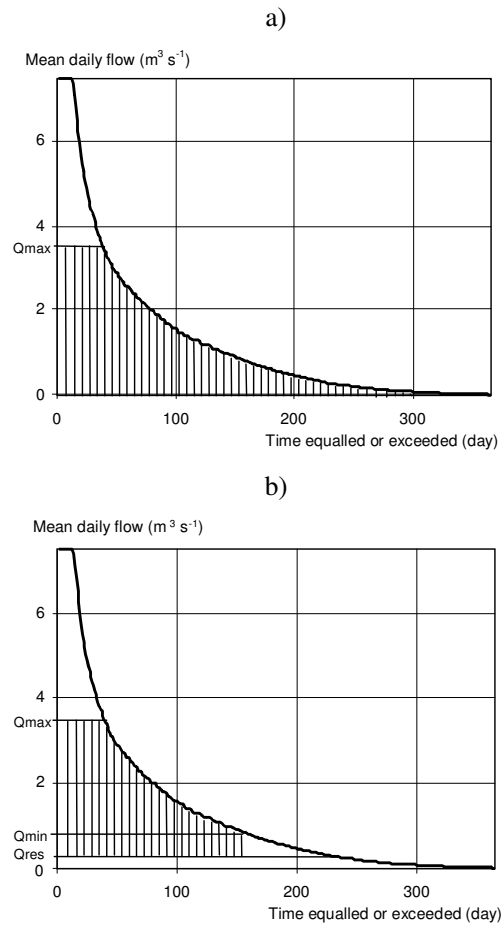


Fig. 4.8 – Maximum turbined volume in a pure run-of-river scheme: a) without considering and b) considering a minimum turbined flow, Q_{min} , and a reserved flow, Q_{res} .

Usually neither η nor Δh will be constant but, instead, will exhibit a non-negligible variation. In this situation, as well as when there is a storage capacity, the mean annual flow duration curve does not provide a correct way of estimate the energy production and other procedures have to be implemented.

These procedures generally simulate the exploitation of the powerhouse by means of a sequential stream-flow routing methodology. The simulation studies can be developed according to different points of view, namely:

- by adopting the mean daily flow series previously established for the scheme, that is to say, assuming that the future flows will be equal to the flows that occurred in the pass;
- by adopting a synthetic mean daily flow series obtained from the series previously established for the scheme water intake by a model, generally a stochastic model (for instances, a disaggregation model).

As this last option involves models far beyond the scope of this work, only the first option will be considered.

The simulation studies have to be performed by means of a computational program. The main data generally required for this program is the following one:

- mean daily flow series at the water intake of the scheme;
- design discharge, Q_{max} (m^3/s), and minimum discharge, Q_{min} (m^3/s), compatible with the operation of the powerhouse;
- ecological discharge and any other discharge required for the consumption between the sections of the weir and the outlet of the powerhouse (reserved flow, Q_{res} , in m^3/s).
- head losses in the diversion circuit as a function of the diverted flow;
- rating curve at outlet section of the powerhouse;
- efficiency curve of the equipment of the powerhouse.

If the weir of the scheme creates a lagoon providing a storage capacity and if the operation of the schemes foresees the use of this capacity in order to improve the conditions of the energy production, the following additional data is also required:

- the reservoir stage-capacity curve⁴;
- the exploitation rules, that is to say, the rules that translate the way the stored volume of water is going to be exploited, for instances, in order

⁴ A graph showing the relation between the surface elevation of the water in the reservoir, usually plotted as ordinates, against the volume of water stored below that elevation, plotted as abscissa.

to concentrate the energy production during the periods where the sale tariffs are higher (or when the energy demands are higher).

The simulation algorithm is almost exclusively based on the application of the continuity equation.

Let t_0 (usually $t_0 = 0$ s) denote the initial instant and t_i (s) the instant $i \Delta t$ seconds after t_0 , that is to say

$$t_i = t_0 + i \Delta t \quad (4.11)$$

where Δt is the computational time step (s).

In these conditions the volume utilised in the energy production during the time step i , $\Delta \forall_i$ (hm^3), will result from the “net” inflow also during the time step – inflow to the water intake, I_i (m^3/s), minus the discharge over the spillway, O_i (m^3/s), minus the reserved flow, Q_{res_i} , (m^3/s) – and from the variation of the volume of water stored in the reservoir, ΔS_i (hm^3)

$$\Delta \forall_i = (I_i - Q_i - Q_{\text{res}_i}) \Delta t + \Delta S_i \quad (4.12)$$

The variation of volume ΔS_i is a consequence of the exploitation rules which, generally, results into the two following different actions:

- priority to the energy production;
- priority to the storage.

The former of the previous rules is generally applicable to the peak hour period and the latter, to the low hour period.

When priority to the energy production prevails, the turbined flow should be as great as possible. For this purpose, the inflow to the scheme water intake is increased (if possible, until the limit of the design flow, Q_{max}), by emptying totally or partially the reservoir.

If priority to the storage prevails, as much water as possible should be stored. In this situation the powerhouse will work only if discharges over the spillway are foreseen.

The computation of the energy produced during the time step i , ΔE_i (GWh), is accomplished by an expression equivalent to (4.10) but where \bar{V} is replaced by the volume turbinéd during the time step, ΔV_i (hm^3) and the variables Δh and η are replaced by Δh_i and η_i in order to denote the specific conditions under which the production occurred

$$\Delta \bar{E}_i = \frac{\Delta \bar{V}_i \times \Delta h_i}{\frac{3600}{g \times \eta_i}} \quad (4.13)$$

The estimate of the energy produced in the hydropower scheme, \bar{E} (GWh), during the interval between instants t_0 and t_n , such as $t_n = t_0 + n \Delta t$, is obtained by totalling the production during the successive time steps

$$\bar{E} = \sum_{i=1}^n \Delta \bar{E}_i \quad (4.14)$$

By omitting the term ΔS_i of equation (4.12), the previous methodology is also applicable to the evaluation of the energy production in a hydropower scheme with no storage capacity. In this situation, the values of the remaining variables that appear in (4.12) will only reflect the particular conditions – in what concerns the net inflow, the net head and the efficiency of the equipment – under which the production occurred during each time step.

5

HYDRAULIC DESIGN OF SMALL POWER PLANTS

**Helena Ramos
A. Betâmio de Almeida**

5.1- Introduction

Civil works are quite significant in Small Hydropower Plants (SHP). The investment is generally limited and there is not capability to spend so much on geological or survey exploration, hydraulic and structural design. These undertakings have the main advantage in the use of local materials and attracting local people for the construction. They can not be scaled down from large projects. A SHP design should be the result of the work of a multi-disciplinary engineering or multi-specialist team including hydrologic, hydraulic, structures, electric, mechanical, geologic and environmental experts.

Several types of SHP layout schemes are, essentially, characterised by different intakes and diversion structures depending on the type of the conveyance system. The powerhouse depends on the type of turbines. The headwork typically includes a low dam or barrage belonging to low dam risk category (e.g. depending on the specific legislation of each country).

5.2- Layout schemes

5.2.1- Intakes

There are different types of intakes with diversion flow to the turbine through the penstock: frontal, lateral, bottom drop and siphon type. For the two first types it is necessary to design the entrance shape in order to avoid separated zones of the flow and excessive head loss through wing walls. It is also necessary for all types to verify the minimum submergence in order to avoid vortex formation and, consequently, air entrance. The discharge control can be improved by special structures (e.g. narrow sections, weirs and gates or valves) depending upon if the flow is pressurised or of free surface type.

The elevation of the water level can be controlled by a weir or by an inflatable or rubber dam or gate in order to create enough available head (H_o) to divert the desired discharge (Q_o).

5.2.1.2- Minimum submergence

The vortex formation by insufficient intake submergence can induce air dragging or even solid material to the intake, reducing the turbine efficiency. The vortex development will depend on the geometry of the intake, the submergence and the approach flow velocity.

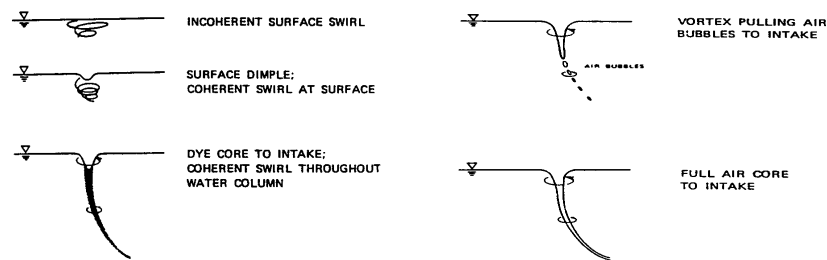


Fig. 5.1 – Development vortex phenomenon (ASCE/EPRI, 1989).

The intake' design criteria is based on the definition of minimum submergence in order to enable the guarantee of non-vortex formation, with air dragging into the hydraulic conveyance system. Several models and field studies were

developed in order to obtain design criteria like those based on Gordon (ASCE/EPRI, 1989) formula (Figure 5.2).

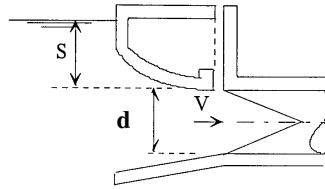


Fig. 5.2 – Intake scheme with submergence. Gordon symbols.

Gordon (in ASCE/EPRI, 1989) developed a criterion for intake design in order to avoid the vortex formation (Figure 5.3). Two different types of flow approximation (i.e. symmetric and asymmetric) were considered and the following dimensionless equation was proposed:

$$\frac{S}{d} = C \frac{V}{\sqrt{gd}} \quad (5.1)$$

where:

S is the submergence (m); d is the intake opening (m); V is the mean flow velocity at the inlet (m/s); g is the gravity acceleration (9.8 m/s²) and C=1.7 (symmetric) or C=2.3 (asymmetric).

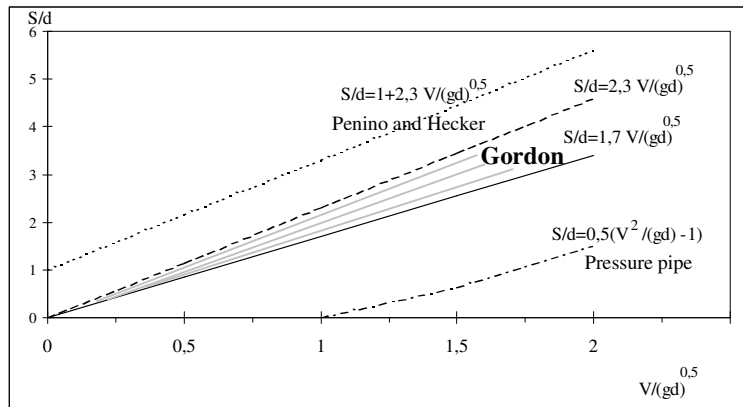


Fig. 5.3 – Different criteria for minimum submergence.

Figure 5.3 shows that formula deduced by Pennino and Hecker (in ASCE/EPRI, 1989) is a conservative criterion in what concerns the vortex formation.

Different types of vortices can occur in the water intakes. The following classification of situations is proposed (adapted from ASCE/EPRI 1989), considering main types (Figure 5.4): Type 1 – developed vortex with a deep nucleus and with drap air; Type 2 - superficial depression, without drag bulb air but with well defined nucleus; Type 3 – depression quite negligible with unstable nucleus; Type 4 – rotational movement without free-surface depression but with superficial circulation.

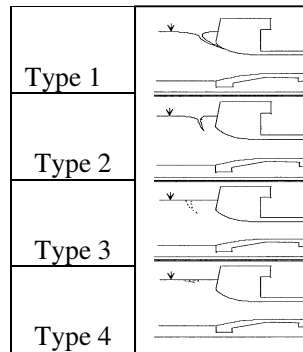


Fig. 5.4 –Scheme of different types of vortex.

Some of the procedures to consider, in order to avoid vortex formation beyond the minimum submergence, consist in creating a good inflow approximation, with a canal or by a convergent. If there is any singularity that provokes flow circulation the minimum submergence criteria could be not enough. These types of vortices can occur close to partial opening gates, sluice valves or bottom outlet.

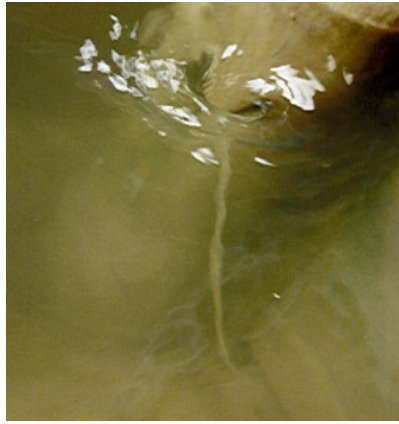


Fig. 5.5 – A vortex in a model facility with a visible air dragging.

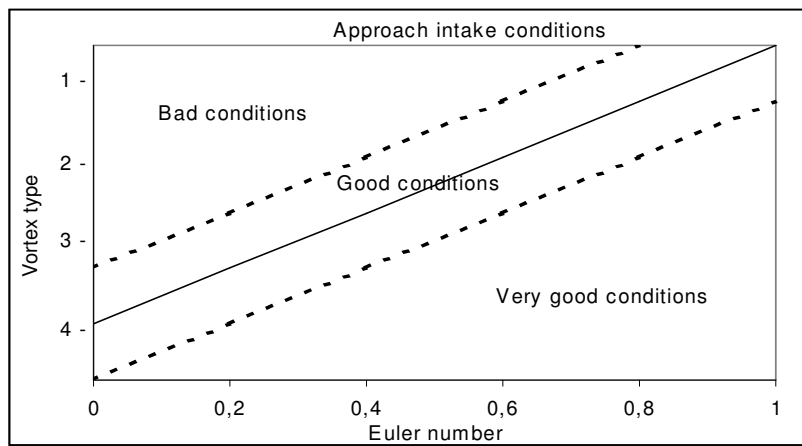


Fig. 5.6 – Relation between Euler number and the type of vortex (adapted from NEIDERT et al., 1991).

Based on experimental tests and for the intake without type 1 vortex the following formulation for the minimum submergence should be applied:

$$\frac{S}{d} = \frac{1}{2} \left(\frac{V^2/gd}{E^2} - 1 \right) \quad (5.2)$$

with E, the Euler Number, that is defined as

$$E = \frac{\Delta p}{\rho V^2} \quad (5.3)$$

where

Δp - differential pressure between two sections (upstream and downstream of the vortex);

ρ - specific mass of the water;

V - mean flow velocity at the inlet.

The Euler number (E) is a dimensionless parameter that represents physically the pressure drop by the velocity arising, which can influence the appearance of vortex (Figure 5.6). For instance, if an intake has good approach conditions it can appear type 1 vortex (with air drag) for $E > 0.85$ and it can be avoided, for new operational conditions, characterised by $E < 0.60$. Very good approach conditions are characterised by the non-existence of separated flow zone or any type of singularity near the intake. When there are bad conditions (e.g. with swirl conditions or presence of singularities near the intakes) the conservative formulation of Penino and Hecker (in ASCE/EPRI, 1989) can be insufficient requiring lab tests for a complementary analysis.

5.2.1.3- Bottom drop intake

The bottom drop intake is largely used in SHP (Figure 5.7). Due to its simplicity it can be easily adapted to a low dam. The diverted discharge by this type of intake structure is a function of the rack characteristics, namely the opening degree or free area under non-submerged operational conditions. The design criteria must have into consideration the removing capacity of solid material by dragging.

The rack is normally located at the top of the low dam, allowing the absorption of a discharge less or equal to the design one.

Should the turbulence be not significant over the weir where is placed the bottom intake, the total head of the flow can be considered approximately constant along the crest.

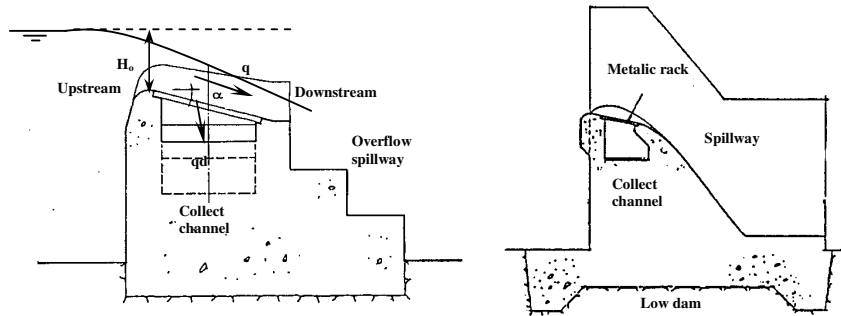


Fig. 5.7 – Bottom drop intake schemes.

The minimum length of the rack for diversion of the turbine design discharge is determined by the integration of the following equations (NOSEDA, 1956):

$$\frac{dq_d}{dx} = C_d C_r \sqrt{2gh} \quad (5.4)$$

$$\frac{dh}{dx} = \frac{2C_d C_r \sqrt{h(H_o - h)}}{2H_o - 3h} \quad (5.5)$$

with

$$H_o = h + \frac{q^2}{2g h^2}$$

being

H_o = specific energy over the rack;

h = head flow over the rack;

q = unit discharge over the rack (Q/L with L the total transversal length of the rack);

q_d = unit discharge absorbed by the rack;

C_d = discharge coefficient of the rack;

C_r = ratio area between free and total rack area;

x = flow and rack direction.

The combined integration of equation (5.4) and (5.5) results in the following solutions:

$$L = \frac{H_o}{C_d C_r} [\phi(y_2) - \phi(y_1)] \quad \text{and} \quad L = \frac{H_o}{C_d C_r} [\beta(u_2) - \beta(u_1)] \quad (5.6)$$

in which

$$\phi(y) = \frac{1}{2} \arccos \sqrt{y(1-y)} \quad \text{and}$$

$$\beta(u) = \frac{1}{2} \arccos \frac{1}{\sqrt{3}} \sqrt{2 \cos \theta + 1} - \frac{\sqrt{2}}{2} \sqrt{(2 \cos \theta + 1)(1 - \cos \theta)}$$

with

$$y = \frac{h}{H_o} \quad (\text{for head variation}); \quad u = \frac{q}{q_{\text{m\acute{a}x}}} \quad (\text{for discharge variation}).$$

Equations (5.6) have the following positive roots:

- $y = \frac{2}{3} \cos \left[\frac{1}{3} \arccos(1 - 2u^2) \right] + \frac{1}{3}$ - for subcritical flow
- $y = \frac{2}{3} \cos \left[\frac{1}{3} \arccos(1 - 2u^2) + 240^\circ \right] + \frac{1}{3}$ - for supercritical flow
- $\theta = \frac{1}{3} \arccos(1 - u^2)$ - for subcritical flow
- $\theta = \frac{1}{3} \arccos(1 - u^2) + 240^\circ$ - for supercritical flow

These equations, based on NOSEDA (1956) study, are only valid for a free or not submerged flow through the rack. This method allows the calculation of head and discharge at upstream and downstream of the rack and the rack absorbed discharge (Figure 5.7). Based on computational simulations, Figure 5.8 shows the influence of the discharge design on the rack length, for different values of rack width.

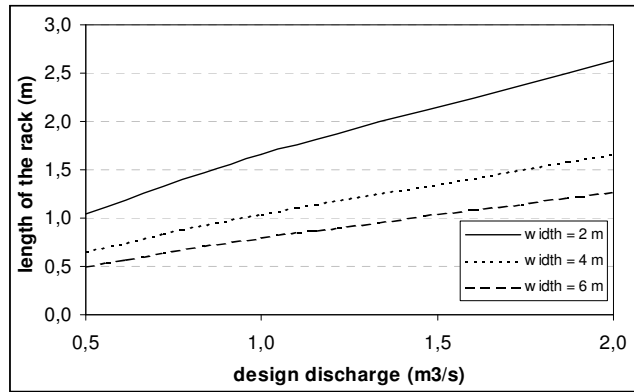


Fig. 5.8 – Estimation of the rack length as a function of the design discharge.

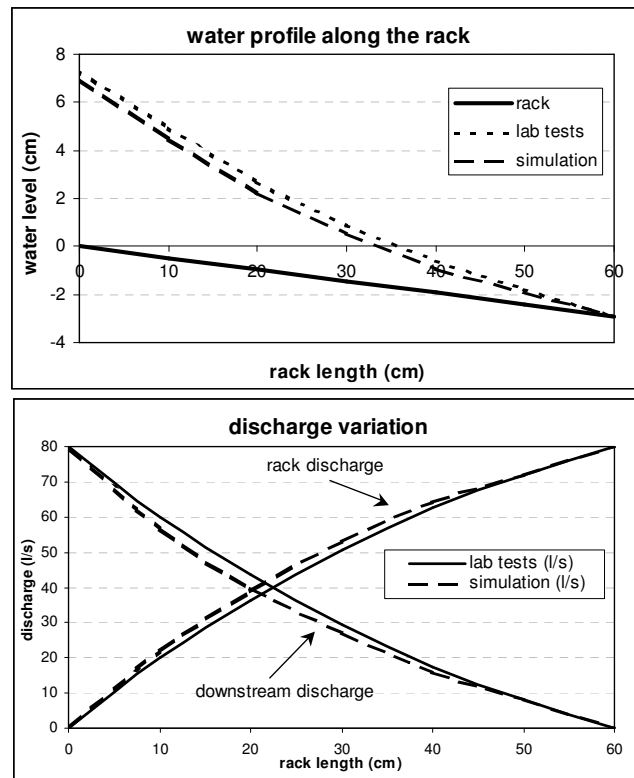


Fig. 5.9 – Comparisons between lab tests and computer simulations (RAMOS and ALMEIDA, 1991).

In the calculations it was supposed a rack discharge coefficient of 0.63, a net rack area coefficient of 0.60, a rack slope of 20% inclined to downstream and a 20% of obstruction by solid materials. This assumption is currently applied in design phase for bottom drop intake type.

Another formulation based on CHARDONNET, MEYNARDY and OTH, 1967, and posterior developed by a computer code (RAMOS, H. and ALMEIDA, A. B., 1991) to calculate the discharge and head variation along the rack was compared with experimental tests (Figure 5.9).

This type of intake (also known by Tyrolian, bottom-grade, streambed and drop intake) has been applied in Alpine areas, in Europe. Worldwide practice shows it is applicable to small rivers, in particular, in mountainous or hilly regions with flows transporting a large amount of debris and big stones, where there are steep gradients or falls provoking rapids or flash floods to facilitate the automatic rack cleaning.

5.2.2- Protection rack

The rack is an important protection element against the inflow of solid material that can provoke damage in turbines. Longitudinal bars lean against crossbars with or not transversal reinforcement beams characterise each rack element. The rack is located at inlet entrance and can be of fixed or movable type. The rack can be defined by its spacebar, a , length in flow direction, b , thickness, c , and the total cross-section, S .

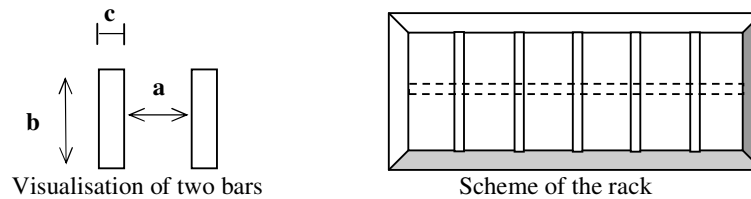


Figure 5.10 – Characterisation of the rack.

In order to avoid rack obstructions by solid materials, namely when installed in an approach canal, it will be appropriate to provide it with an auto-trash-rack.

The rack must be specified in order to avoid excessive head loss by grid obstruction if the spacebars are too small, neither to allow driving solid material

into the turbine flow if the spacebars are very large. According to LENCASTRE, 1991, the spacebar dimension, a, is a function of the turbine type (Table 5.1).

**Table 5.1 – Spacebar for different turbine types
(LENCASTRE, 1991)**

Turbine type	Kaplan	Fast Francis	Slow Francis	Pelton	Pumps as Turbines
a (m)	0.10 - 0.15	0.08 - 0.10	0.06 – 0.09	0.03 – 0.05	0.02

For a submerged rack with an automatic trash rack system, the maximum flow approach velocity is about 0.80 to 1.00 m/s. This velocity is based on the kinematic law:

$$v = \frac{Q}{S} \tag{5.7}$$

where Q is the turbine discharge value (m³/s); S is the cross-section of the rack (m²).

The flow through the rack can induce severe vibrations due to detached swirls with a frequency, fs, which should be different of the bars frequency, fb, in order to avoid resonance phenomena and the collapse of the rack. According to the stability criteria, fb must obey to the following condition:

$$fb \geq 1.5 fs \tag{5.8}$$

The swirl frequency, fs (Hz) is given by

$$fs = \frac{St V}{c} \tag{5.9}$$

where

St is the Strouhal number (Table 5.2) that depends upon the cross-section of the bar, whose value must be majored by the safety factor F, according to Table 5.3.

Table 5.2 – Strouhal number for different types of bars (LENCASTRE, 1987).







Type of bar	b=c	b=2.8 c	Diameter=c	b=2.8 c	b=2.8 c	b=5 c
						
S_t	0.130	0.155	0.200	0.255	0.265	0.275

Table 5.3 – Safety coefficient for Strouhal number (LENCASTRE, 1987).

(a+c)/c	1.50	2.00	2.50	3.00	4.00	5.00
F	2.15	1.70	1.40	1.20	1.05	1.01

The calculation of the structural frequency of the bar of a rack with fixed extremities, is based on the following equation:

$$f_b = 3.6 \frac{c}{3.46L} \sqrt{\frac{gE_b}{\gamma_b + \frac{a}{c}\gamma}} \quad (5.10)$$

being L the distance between bar supports (m); g the gravity acceleration (9.8 m/s²); E_b is the elasticity modulus (N/m²); γ_b the specific weight (N/m³) of the bar material (for steel bars E_b = 2.1x10¹¹ N/m² and γ_b = 78000 N/m³); γ is the water specific weight (9800 N/m³).

In order to guarantee the rack structural stability, the dimension L must be reduced until equation (5.8) be verified. Sometimes the solution leads to consider a transversal bar in order to obtain a lower bar length.

5.2.3- Sedimentation or desilting basin

A sedimentation basin allows settling fine particles in suspension that may wear parts of the turbine and to decrease its efficiency and lifetime. The settling is due to gravity action through the reduction of the flow velocity by increasing the sedimentation basin cross-section. Normally these basins are prismatic tanks, where the flow has a main horizontal direction. The tank can be considered divided in four zones:

- 1- inlet zone;
- 2- settling zone;
- 3- outlet zone;
- 4- sludge storage zone.

The inlet zone should assure a uniform flow velocity distribution by imposing the flow through a perforated baffle. The settling zone allows the sedimentation of the solid particles. Clean water is driven into the hydraulic conveyance system through an outflow weir or orifice (Figure 5.11).

For design purposes, the solid particles are assumed as spherical shape, with a uniform specific weight. The typical dimensions of the sedimentation (or a desilting) basin are represented in Figure 5.12, for an inflow discharge Q at the inlet of the basin with a depth D , a width W and a length L .

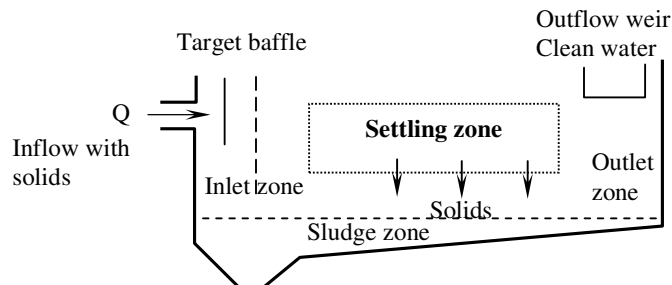


Fig. 5.11 – Different zones in a sedimentation basin.

Through each basin cross-section, the free-surface flow is supposed to be horizontal and with a uniform velocity. Particles tend to settle to the bottom with a fall velocity V_s . Normally, the median diameter of the particles (D_{50}) considered admissible for small turbines is about 0.2 mm. This value will depend

on each turbine type (the turbine manufacturer should indicate the maximum allowable particle dimension).

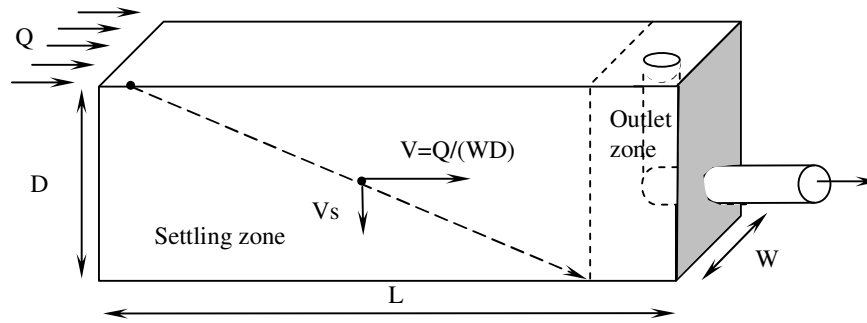


Fig. 5.12 – Schematic sedimentation basin.

The flow velocity is dependent of the basin main dimensions:

$$V = \frac{Q}{W \cdot D} \quad (5.11)$$

The sedimentation velocity can be obtained by the following equation:

$$V_s = 3.61 \sqrt{\frac{D_{50} (d - 1)}{C_R}} \quad (5.12)$$

where d is the solid particle density (ratio between specific weight of the particle and the water, $d = \frac{\gamma_p}{\gamma}$, normally for silt type $d \approx 2.5$); R_e is the Reynolds number, $R_e = \frac{V_s D_{50}}{\nu}$, being ν the water viscosity $\nu = 10^{-6} \text{ m}^2/\text{s}$ and C_R the shape particle coefficient.

Based on the design discharge (Q) and the sedimentation velocity (V_s) the superficial area, A_s , necessary for settling particles of dimension greater than D_{50} is obtained by

$$A_s = \frac{Q}{V_s} \quad (5.13)$$

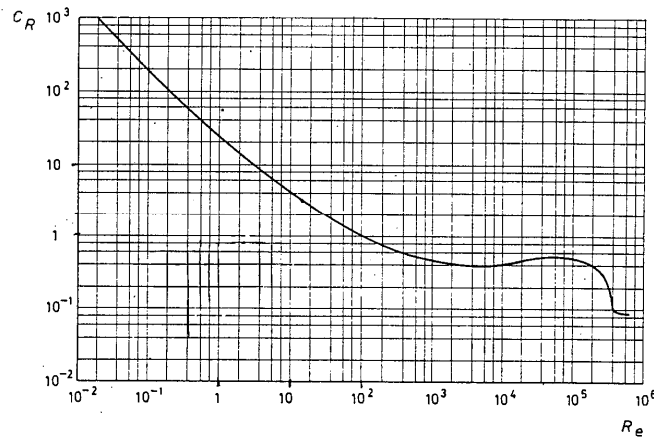


Fig. 5.13 – Spherical shape coefficient, C_R , as a function of Reynolds number, R_e (adapted from QUINTELA, 1981).

The sedimentation criterion requires that the scour time T_{scour} of a particle must be greater than the sedimentation time T_s , ($T_{scour} \geq T_s$) in order to the particle be settled.

$$T_{scour} = \frac{L}{V} \quad (5.14)$$

$$T_s = \frac{D}{V_s} \quad (5.15)$$

In practice the basin behaviour will be different from the one predicted by the theoretical calculation. Main factors that influence the basin behaviour are the flow turbulence and the non-uniform horizontal velocity distribution. One way to obviate this problem consists in increasing the superficial area, A_s , of the basin or the width, W , and removing all angles or unnecessary singularities.

The width and the flow velocity must be controlled in order to avoid the lift up of settling particles (i.e. verification of the non-drag criterion). The equation (5.16) gives the relation between the scour velocity and the geometry of the sedimentation basin (ASCE/EPRI, 1989):

$$V_{\text{scour}} = 0.5 K \left[R_H^{1/6} C_R (d-1) \frac{D_{50}}{304.8} \right]^{1/2} \quad (5.16)$$

with R_H = hydraulic radius of the sedimentation basin, $R_H = \frac{W \cdot D}{W + 2D}$, and K the Gauckler-Manning-Strickler coefficient, for concrete basin $K = 75 \text{ m}^{1/3}/\text{s}$.

The methodology consists in a set value for W , based on A_s , and the considered L . Then D values will be estimated in order to verify that V_{scour} is greater than the mean flow velocity, V .

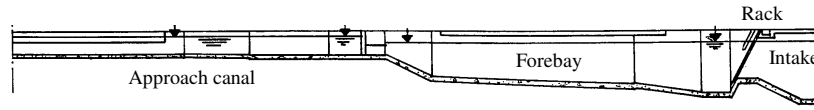


Fig. 5.14. –Scheme of a forebay.

The depth, D , can vary along the length of the basin, as showed in Figure 5.14. Nevertheless the obtained D will be an average depth.

5.2.4- Weir

The weir crest (in this case it will also be the spillway) is, normally, located in the middle of the low dam. Sometimes it is convenient to install a gate in order to increase the storage capacity. The full storage water level (FSWL) coincides with the spill crest because it is the maximum capacity under normal operating conditions. The weir will be designed for a discharge flow with a chosen return period (e.g. 100 years) and should also be verified for greater discharges, corresponding to the maximum flood water level (MFWL).

The typical spillway profile is defined in order to have atmospheric pressure along its surface for design conditions. For discharge verification ($Q_{\text{max}} > Q$) under-pressures along the spillway profile will be accepted, since the head is greater than the design head (according to criteria based on Waterways Experimental Station (WES) – Corps of Engineers, US).

The general free-surface weir discharge law is given by

$$Q = CL\sqrt{2g}H^{3/2} \quad (5.17)$$

being:

Q = weir discharge (m^3/s);

C = discharge coefficient (-);

L = weir width (m);

H_o = head over the spillway crest (m);

g = gravity acceleration (m/s^2).

The discharge coefficient for a WES type weir is around $C = 0.50$, depending on the head value (e.g. is superior to 0.50 for heads greater than design one and conversely for lower heads).

Figure 5.15 shows the variation of discharge coefficient with the head if lateral entrance wing walls exist in order to avoid flow lateral contraction.

The design head is obtained by equation (5.17) based on the design flood discharge. The analytic definition of the spillway profile is obtained by the following equation (WES profile):

$$y = \frac{x^{1.85}}{2H_o^{0.85}} \quad (5.18)$$

The connection curve between the vertical upstream wall and the spillway profile is obtained by an ellipse or by several arcs of curves according to Figure 5.16.

For verification discharge ($Q_{\max} > Q$), the spillway profile will operate with a pressure lower than atmospheric one and the discharge coefficient will increase. For a smaller discharge ($Q_{\min} < Q$) over-pressures (greater than atmospheric pressure) will occur along the spillway profile with a smaller discharge coefficient.

In order to avoid flow separation and cavitation risk for higher heads, the following design criteria should be obeyed (LENCASTRE, 1987):

$$H_{\max}/H_o \leq 1.4$$

According to the value accepted for the maximum or verification head (H_{\max}) the wing lateral walls are fixed with a certain freeboard (e.g. 1 m at least) above the MFWL.

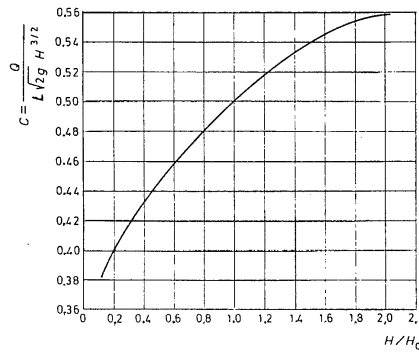


Fig. 5.15 - Discharge variation of a WES spillway as a function of H/H_0 (ABECASIS, 1977).

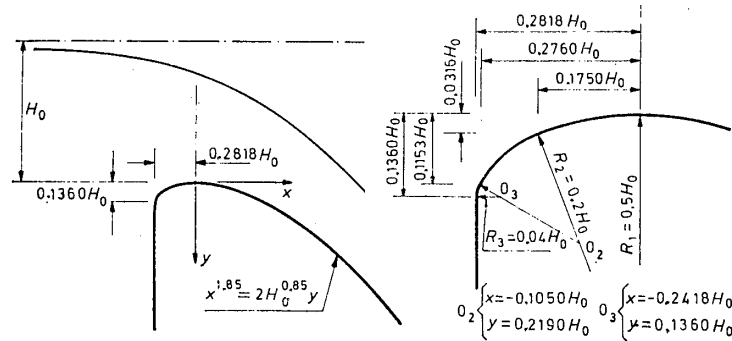


Fig. 5.16 - Spillway profile for the design head (H_0) (CORPS OF ENGINEERS, WES referred in QUINTELA, 1981).

In small dams it is very acceptable to choose a stepped spillway where the downstream side is built with steps, where the flow can dissipate a certain amount of energy (see Figure 3.6 - at bottom). When the available space for locating the spillway is limited, labyrinth or semicircular solutions should be considered, in order to obtain higher discharge.

A dissipation structure should be provided downstream the dam if the geotechnical conditions are not strong enough.

5.3- Sluice bottom outlet

To empty the reservoir the dam should be provided with a sluice bottom discharge. To obtain the emptying law of a dam sluice bottom outlet is necessary to define the hydraulic conveyance system characteristics, namely singular loss coefficients, the roughness and the cross section. Through the energy equation between reservoir level, H_r , and the axe of the outlet works, H_d , (flow into the atmosphere):

$$H_r - \Delta H = H_d \quad (5.19)$$

where

H_r = is the upstream water level;

ΔH = is the total head losses (as a function of Q - see 5.4 - Conveyance System);

H_{ref} = outlet water level plus kinetic outlet head $\left(= \frac{Q^2}{2gA^2} \right)$, where A is the

outlet cross-section area.

It is important to know the velocity value at outlet, in order to prevent energy dissipation structure (e.g. impact blocks), as well as to predict the safe height of wing walls to direct the flow towards the river.

The bottom discharge will allow the total or partial reservoir emptying. To estimate the emptying duration, it is common to consider a null inflow discharge and the full storage water level (FSWL). Based on the discharge law for sluice bottom outlet and on mass flow balance equation:

$$Q_{outlet} = \xi A \sqrt{2g(H_r - H_{ref})}$$

$$\frac{d\forall}{dt} = Q_{inlet} - Q_{outlet} \quad (5.20)$$

where Q_{inlet} is the inlet discharge into the reservoir ($= 0 \text{ m}^3/\text{s}$), Q_{outlet} is the outlet discharge from the reservoir (m^3/s), ξ is the discharge coefficient and \forall the volume of the reservoir (m^3).

The differential equation can be integrated by a finite difference technique in order to obtain the total duration of the reservoir emptying.

5.4- Conveyance system

5.4.1- General layout

The conveyance system includes all elements designed to water transport from the intake to the powerhouse. The conveyance layout can be composed either by pressure galleries or pipes or by a mixed system composed by free-surface canals and pressurised pipes. The layout of the conveyance system depends on several factors according to the following remarks:

- Analysis of different sites for low dam and powerhouse installation and conveyance layout alternatives to seek for the economic net head, the minimum length that involves smaller volumes of excavation, and land volume to replace. All alternatives should have good access and the concern about minimisation of expropriating, must be performed. The selection of the more adequate final proposes will be based on technical, economic and social considerations.
- The cross-section of the hydraulic system (e.g. characterised by a diameter) should be the most economic in order to attend the annual average energy production and the costs for conveyance system installation.
- Definition of all system accessories and components along the total conveyance system and the more correct place to install them.

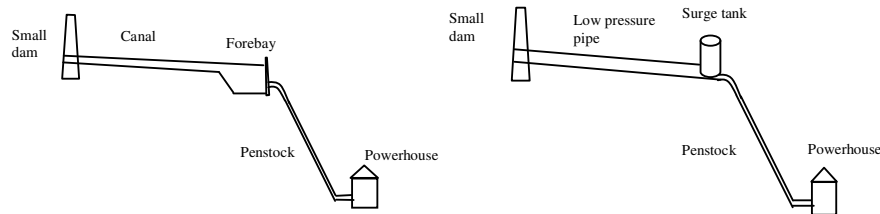


Fig. 5.17 – Schematic layouts for the hydraulic system.

Site selection for the headwork (e.g. a small dam and the intake structure) and the powerhouse should be carried out on the basis of topographical and geological conditions and on site visits. The main factors to be considered in the alignment of canals or low pressure pipes are:

- If a reach of a pipe can gain at least 2% of head, H, the experience shows it is economically feasible.

- A long canal could be unfeasible geomorphologically because of the need to overcome side streams and gullies that imposes the construction of crossing structures, such as inverted siphon. Potential slides should be properly considered.
- River bends associated to enclose geological conditions are generally favourable to a straight tunnel or a combined system with canals and low-pressure pipes.
- Plastic pipes could be considered but there are limitations of available diameters, or sunlight effects, or overpressures, or temperature variations. These factors impose that the plastic pipes should be buried or covered.

These constraints should be studied comparatively in order to obtain the best solution.

For a total pressurised circuit with pipes following the land profile, blow-off valves should be installed at the lowest points, allowing the pipe emptying, and air-valves should be installed at higher points of the pipeline to avoid it collapsing or buckling associated to air pressure problems. The highest point of the conduit should be positioned below the lowest hydraulic gradient line under extreme operating conditions (Figure 5.18).

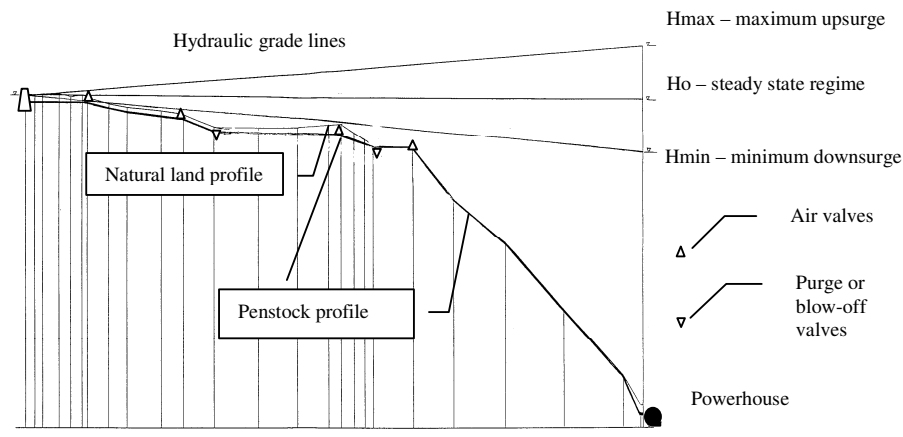


Fig. 5.18 – Example of the hydraulic grade line and penstock profile.

Due to turbine discharge variations or turbine guide vane manoeuvres the pressure will vary, along the penstock, with the time. The maximum and

minimum piezometric head envelopes should be indicated and compared with the pipe resistance pressure classes and with the penstock profile, respectively. The control of both the maximum and minimum transient pressure variations is one of design objectives.

5.4.2- Head losses and net head

The net head, H_o , is one of the most important parameters for a feasibility analysis and design of a hydroelectric system. The material, the cross-section and the length of the penstock will influence the headloss. Net head calculation will be based on the knowledge of the gross head (H_g - geometric water level or the difference between water level at the intake (Z_u) and the water level at the tailrace (Z_d)- for reaction turbines or the nozzle axe – for action turbines (in particular for Pelton)) and all head losses (friction and all singularities losses, ΔH_i , along the hydraulic conveyance system). The headloss will be a function of the diverted discharge.

$$H_o = Z_u - Z_d - \sum_i \Delta H_i \quad (5.21)$$

The design head is used to define the design power output of a turbine (typically the maximum power output for the best efficiency head). For a hydropower station with a high head scheme (e.g. with long penstock), the variation in the water level can have less effect on power output than, perhaps, the variation of the head losses with the discharge.

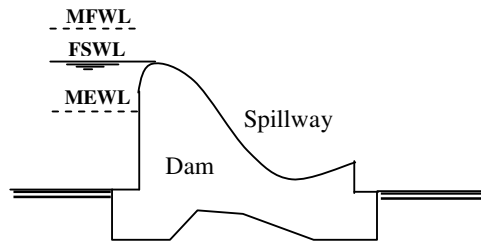


Fig. 5.19 – Water levels at the reservoir.

When the reservoir water level changes during the powerplant operation, a maximum water level may appear when the weir discharges the maximum flood (MFWL - maximum flood water level). The crest of the weir gives the full

storage water level (FSWL) and the minimum exploitation water level (MEWL) is determined by requirements of the intake operation (e.g. minimum submergence criteria).

Friction head loss

The friction loss along a canal or a penstock is expressed by

$$\Delta H_f = J L \tag{5.22}$$

where

J = hydraulic gradient;

L = length of the canal or the penstock (m).

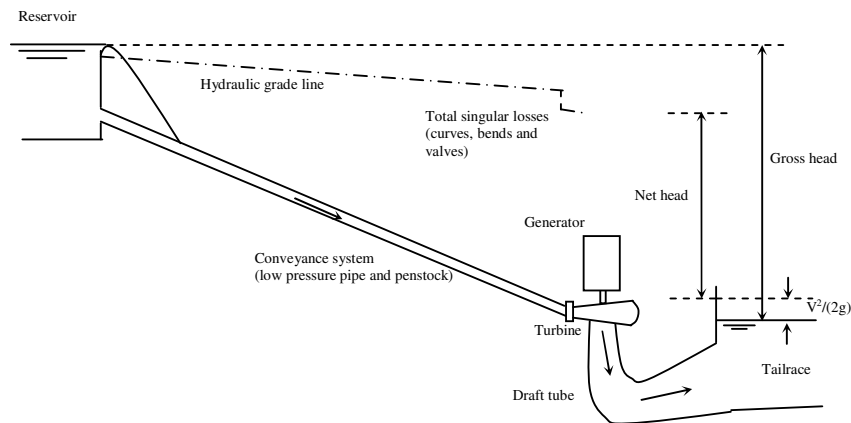


Fig. 5.20 – Hydraulic grade line of a hydro scheme equipped with a reaction turbine.

By assuming that the flow velocity is negligible in the reservoir, the gross and net heads are schematically defined in Figure 5.20, for a reaction turbine with downstream draft tube. In this case the internal draft headlosses are considered in the turbine efficiency.

- Pressure flow

In closed pressurised pipes the flow is typically turbulent ($Re > 2000$), thus the Colebrook-White formula is advisable for friction head loss calculation. This formula requires an iterative method to solve the friction factor for each discharge value. The Moody diagram allows a graphical calculation. For this calculation it is necessary to know the mean velocity of the flow (U), the

diameter of the pipe (D), the absolute roughness (k) and the kinematic viscosity of the water (ν).

$$\frac{1}{\sqrt{f}} = -2 \log \left(\frac{\epsilon}{3.7} + \frac{2.51}{R_e \sqrt{f}} \right) \quad (5.23)$$

where

$$f = \frac{JD}{U^2/2g} \quad \epsilon = \frac{k}{D} \quad R_e = \frac{UD}{\nu}$$

being f = Darcy-Weisbach factor; ϵ = relative roughness; R_e = Reynolds number.

In order to obtain the hydraulic gradient (J), it can be possible to establish the following iterative process based on any initial arbitrary value J_n :

$$J_{n+1} = \frac{U^2}{8gD} \left(\log \left(\frac{k}{3.7D} + \frac{2.51\nu}{D\sqrt{2gDJ_n}} \right) \right)^{-2} \quad (5.24)$$

The value of the absolute roughness (k) depends on the material of the conduit. In Table 5.4 some k values are presented.

Table 5.4 – Typical values of absolute roughness, k, for different type of materials (BUREAU OF RECLAMATION, 1977)

material	k (mm)
cast iron	
- new	0.25
- strongly rusty	1.50
soldier steel	
- new	0.10
- rusty	0.40
concrete	
- rough	0.60
- smooth	0.18

The friction head loss calculation can also be based on empirical formulae. In fact there are several empirical equations to estimate the hydraulic gradient (J), but they are only valid for some flow conditions. Two of the most popular empirical formulations are the Hazen-Williams (5.25) and Gauckler-Manning-Strickler (5.26), which when expressed in SI units, are presented by the following equations:

$$Q = 0.849 C S R^{0.63} J^{0.54} \quad (5.25)$$

$$Q = K S R^{2/3} J^{1/2} \quad (5.26)$$

where K and C depend on the pipe wall material; R is the hydraulic radius ($R = D/4$) and S is the pipe cross-section area.

Table 5.5 – Typically coefficients of Gauckler-Manning-Strickler and Hazen-Williams formulas

Materials	K (m ^{1/3} /s)	C (m ^{0.37} /s)
steel	90	130
cast iron	80	120
concrete	75	120
PVC	110	140

- Free-surface flow

The Gauckler-Manning-Strickler formula (see equation 5.26) is currently applied in open channel flows. This formulae due to its simplified application is often chosen to obtain a quick estimation of the hydraulic gradient or unit headloss (J). The following relation defines the hydraulic radius

$$R = \frac{A}{P} \quad (5.27)$$

where A is the cross-section area of the canal and P is the wet perimeter of the cross-section.

Singular or local losses

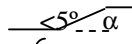
Local losses are expressed by a general equation of the following type:

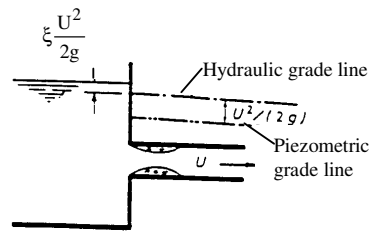
$$\Delta H_L = \xi \frac{U^2}{2g} \tag{5.28}$$

where ξ = coefficient of singular loss that depends mainly on the geometry of the singularity and Reynolds number.

Typical values of ξ are next presented (QUINTELA, 1981 and LENCASTRE, 1987):

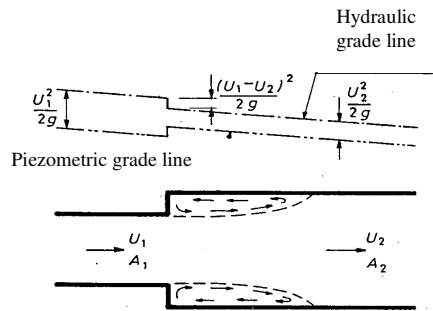
- Contractions

Type of contraction	ξ
sharp edge	0.50
rounded edge	0.25
conical horn	0.10
gradual	
	0.06
α { 20°	0.20
45°	0.30
60°	0.32
75°	0.34



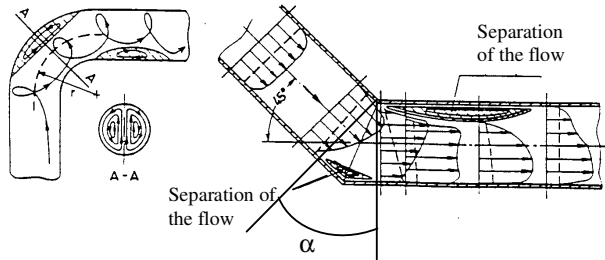
- Expansions

$$\xi = \left(1 - \frac{A_1}{A_2}\right)^2 \tag{5.29}$$



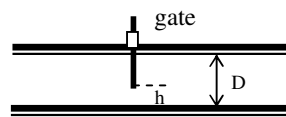
• Bends

α	ξ
30°	0.20
40°	0.30
60°	0.55
80°	0.99
90°	1.10

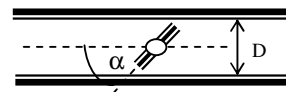


• Gates and valves

h/D	ξ
0.20	31.4
0.50	3.3
0.70	0.8






α	ξ
5°	0.24
20°	1.54
40°	10.8
60°	118

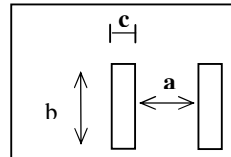


• Racks

The flow through a rack induces a singular head loss that can be calculated as follows:

$$\xi = \beta \left(\frac{c}{a} \right)^{\frac{4}{3}} k \sin \alpha \quad (5.30)$$

Types of bars				
β	2.42	1.79	0.76	0.92



where

β = shape coefficient of a rack bar;

c = thickness or diameter of a bar (mm);

a = distance between bars (mm);

α = angle of the bar with the horizontal;

k = coefficient rack obstruction.

5.4.3- Canals

The economic feasibility of small powerplants is based, among other factors, on the available flow energy. The construction of long circuits in order to raise the total net head is a solution depending on the topographic and geologic conditions. Typically, a diversion scheme composed by a canal has a reservoir or a forebay at downstream, where it is located the intake to the penstock. This forebay will benefit the turbine flow changes.

The length of a canal is an important constraint that depends on the technical/economic study. The height of the lateral walls of a canal must be specified in order to avoid the overtopping by the flow including both the unsteady and hydrostatic conditions.

A lateral weir is a way to control the canal water level (Figure 5.21). The location of a weir depends on the terrain characteristics and strength against the erosion provoked by the outflow. The weir will control the hydrostatic water level and the height of the canal lateral walls. The implementation of automatic control systems for water level regulation and discharge by gates are also important solutions for the exploitation conditions in order to avoid an excessive height of lateral walls.

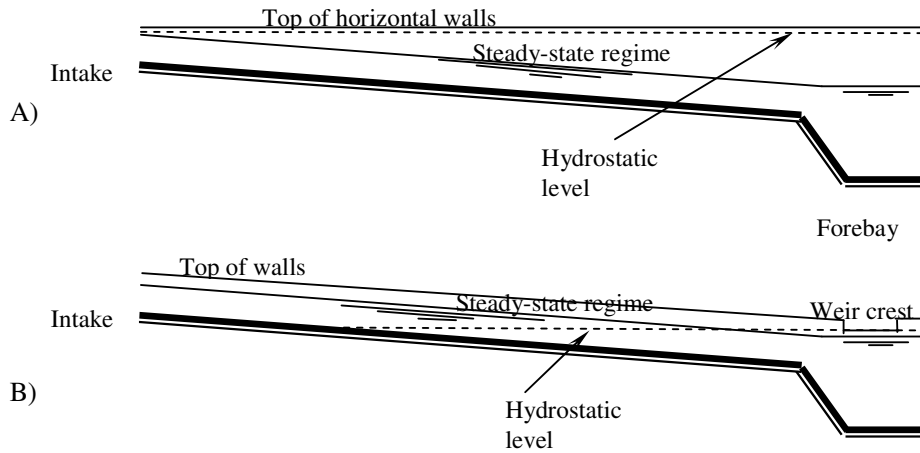


Fig. 5.21 – Canal layouts: A) with top of horizontal walls; B) with a lateral weir.

5.4.3.1- Uniform and steady state hydraulic regimes

The steady state uniform flow regime is a state of equilibrium between the weight and the resistance force components along a prismatic canal with constant roughness. Under uniform conditions, the free-surface profile, the hydraulic grade line and the longitudinal bottom profile are straight and parallel lines.

In uniform regime the discharge, flow velocity, roughness and α coefficient are constants along the canal. For canals with small slope (θ), the hydraulic gradient, J , is equal to the bottom slope:

$$J = \sin \theta \approx \tan \theta = s \quad (5.31)$$

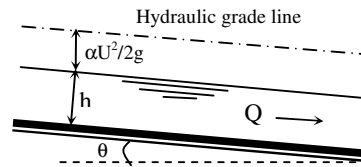


Fig. 5.22 – Scheme of uniform hydraulic regime

By assuming that the flow is a rough turbulent flow, the Gauckler-Manning-Strickler formula is currently applied due to its simplicity, with enough accuracy.

For non-uniform steady flows the free-surface profile or backwater curve is obtained in engineering by the following formula:

$$\frac{dH}{dx} = \sin \theta - J \quad (5.32)$$

with

$$H = h \cos \theta + \alpha \frac{Q^2}{2gA^2} \quad (5.33)$$

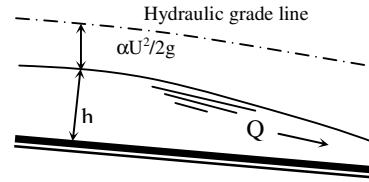


Fig. 5.23 - Scheme of free surface steady state flow

For small slopes ($\cos \theta \approx 1$), turbulent flow ($\alpha \approx 1$) and considering no lateral discharge and a prismatic canal with a slope $s = \tan \theta \approx \sin \theta$, the following equation can be obtained by using a finite difference technique (“standard step method”):

$$h_{n+1} - h_n = \frac{s - \left(\frac{J_n + J_{n+1}}{2} \right)}{1 - \frac{Q^2}{2g} \left(\frac{B_{sn}}{A_n^3} + \frac{B_{sn+1}}{A_{n+1}^3} \right)} \Delta x \quad (5.34)$$

where

A = cross-section area;

B_s = surface flow width;

g = gravity acceleration;

H = specific energy flow;

h = water depth;

J = hydraulic gradient;

s = bottom slope;

θ = angle of the canal bottom with the horizontal;

dx = space step discretisation along the canal.

The flow regime in a small powerplant canal is typically controlled by downstream conditions (the forebay water level), being Q the turbine discharge.

5.4.3.2- Boundary conditions

The analysis of typical boundary conditions of diversion canals in small hydropower plants is based on the following fundamental basic concepts:

- Upstream

Specific energy law – from a reservoir to a subcritical flow canal;

Fixed water level –downstream control gate with float;

Gate or valve that imposes a discharge variation law – canal inflow law.

- Downstream

Turbine discharge law – a discharge demand law;

Lateral weir and water level regulation – control of the forebay water level, including an automatic control action through the turbine guide vane or the control by the outflow through the weir.

5.4.3.3- Forebays

Sudden turbine discharge variations will provoke water level oscillations along the diversion canal. A forebay can be considered as a regulation reservoir (PINHEIRO, 1989), in order to reduce the water level variations and to improve the canal response to turbine discharge variations and, can also operate as a protection against silt or floating particles (see Figure 5.14).

When the plant demands a greater discharge, the water level quickly draws down while the canal cannot supply enough flow. Otherwise, when the plant shutdown a hydraulic bore will propagate upstream while the canal is still supplying the forebay. This last event can induce secondary oscillatory waves and the canal wall overflow.

A forebay (Figure 5.24) positioned at downstream end of a canal has its size conditioned by the following factors (PINHEIRO, 1989):

- 1) To assure conditions to install the penstock intake with its equipment (e.g. trash-rack, level detectors, sluices, gates, and weirs) always with the minimum submergence criteria verified.
- 2) To limit the flow oscillations along the canal by turbine discharge variations.
- 3) To assure the regulation function (e.g. to allow the transient turbine demand satisfaction independent of the flow regime).

In a simple way, the modelling of the flow inside a forebay can be considered as a reservoir with horizontal free surface in each time step, because the forebay is, normally, much deeper than the canal.

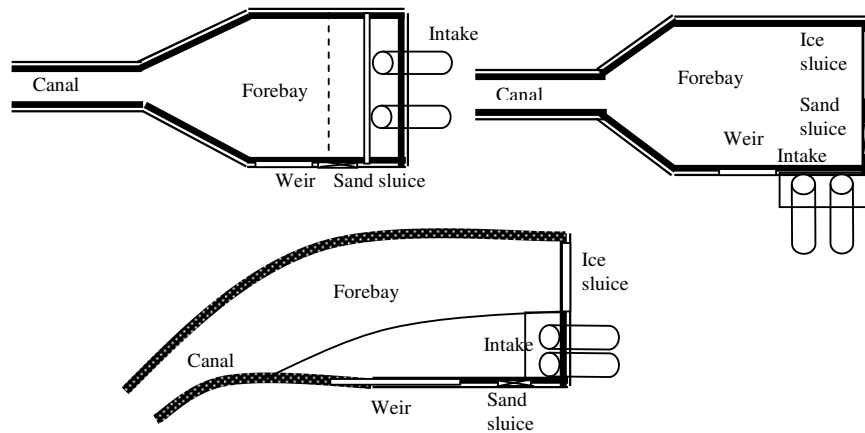


Fig. 5.24 – General forebay layouts.

Applying the continuity equation at the forebay, the level oscillations can be obtained by the following relationship:

$$\frac{dN_f}{dt} = \frac{(Q_{in} - Q_{out})}{A_f} \quad (5.35)$$

where

N_f = free surface level in the forebay;

Q_{in} = forebay inflow, from the canal;

Q_{out} = forebay outflow, over the weir and for penstock;

A_f = horizontal area of the forebay.

Typically $L_f > 2.5 B_f$ (being L_f the forebay length and B_f the forebay width) and the velocity in the forebay is less than 0.5 m/s, in order to induce settling of the harmful solid particles.

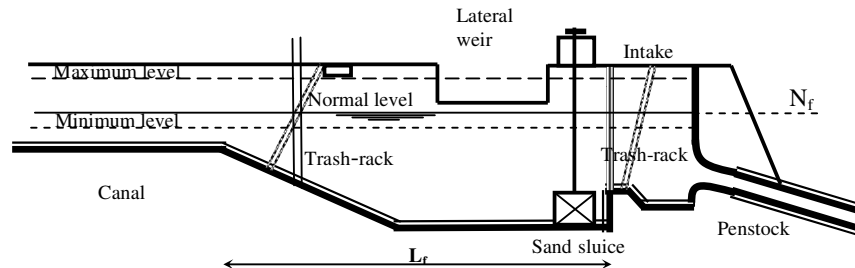


Fig. 5.25 – A typical profile of a forebay.

5.4.4- General remarks about mixed circuit

Depending on local conditions, a mixed circuit solution can be chosen. The flow velocity and the shape of its cross section can be defined through practical considerations. The canal is specified for operation with the maximum turbine discharge, although it can also operate under much smaller discharges (during dry seasons), which can induce serious silting problems along the canal or in the forebay.

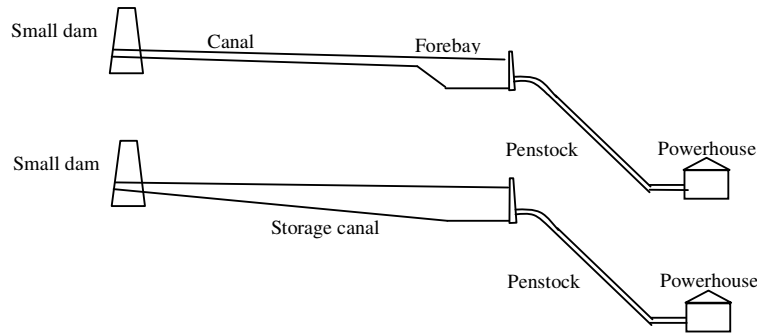


Fig. 5.26 – Two different types of mixed circuits.

The main disadvantages of a canal, comparing with a total pressurised circuit, are the following factors:

- The canal length will be greater than a straight pressurised tunnel (unless a free surface flow tunnel be considered).
- Potential construction troubles of the canal along the river border.

- Crossing structures to overcome side streams and gullies.
- People and animal passages and the danger to fall into the canal.
- Ice troubles in cold regions.
- Efficient drainage systems for sediments.
- Environmental impacts due to excavations.
- Potential slope slides along the canal.
- Maintenance costs.

Alternatively, in many cases the tunnelling is preferable. However, tunnels could present, normally, difficulties during construction, especially caused by geologic fault or shear zones.

5.4.5- Penstocks

The penstock must be designed to bear the maximum internal pressure due to water hammer phenomenon during normal and abnormal operational conditions. It is always laid on a stable site and towards the hill-slope. For small slopes, the pipe must be buried to avoid temperature effects, or these effects must be considered in the pipe design.

There are two suitable layout schemes, namely separate penstocks for each unit, or a common penstock, with branching pipes only close to the powerhouse. The principle of maximum economy and safe constructions must be taken into account in the more suitable layout selection.

Economic diameter

The economic diameter is obtained considering the incremental energy benefits from a lower energy loss associated to a larger diameter, as well as stability of operational conditions (i.e. water hammer and wear) with the total investment cost increase.

The selection of the more suitable diameter for each pipe branch can be based on simplified economic study, with the main objective the minimisation of total cost (e.g. dependent of unit pipe cost and of the loss energy by friction loss):

$$\frac{dC}{dD} = K_{u1} \gamma_p \gamma_w \frac{H_o}{\sigma} C_p D_o - K_{u2} C_E \frac{T_o}{K^2} Q^3 \frac{AF}{D_o^{19/3}} = 0 \quad (5.36)$$

where

AF = actualization factor;

C_E = equivalent tariff of energy (cost unit);
 C_p = pipe material cost per unit weight (cost unit);
 D_o = optimum diameter or economic diameter (m);
 H_o = net head (m);
 K = Gauckler-Manning-Strickler coefficient for the pipe material ($m^{1/3}/s$);
 K_{u1}, K_{u2} = numeric factors to unit conversion;
 Q = equivalent turbine discharge (m^3/s);
 T_o = average annual time operation (h).
 σ = admissible pressure for pipe material (N/cm^2);
 γ_p = specific weight of the pipe material (N/m^3);
 γ = specific weight of the water (N/m^3).

Nevertheless, empirical formulations are recommended to economic diameter estimation for small hydropower plants by several authors, on the basis of data from existing penstocks (JIANDONG et al., 1997):

$$D_o = C_{EC} C_{MP} Q^{0.43} H_o^{-0.24} \quad (5.37)$$

where

C_{EC} = coefficient of energy cost (zones where the energy cost is low = 1.2, medium = 1.3 and high or no alternative source = 1.4);
 C_{MP} = coefficient for the pipe materials (for steel = 1; or plastic = 0.9);
 H_o = net head (m);
 Q = design discharge (m^3/s).

Another criteria based on the maximum velocity flow in the penstock can give a first estimation for the diameter: 2-3 m/s low head plants; 3-4 m/s for medium head and 4-5 m/s for high head plants. These are typical values based on real cases.

For penstocks different types of materials can be used:

- 1) *steel* and *cast iron* pipe is widely used in hydropower design and is preferable for very high head plants that allow a long service life. Expansion joints should be provided at certain intervals, depending on local climatic conditions;
- 2) *prestressed concrete* pipe presents a reduce cost but greater difficulty in installation;
- 3) *plastic* and *glassfibre-reinforced plastic* pipe induces small friction loss, but must be buried or wrapped to protect it from sunlight effect;
- 4) *reinforced concrete* pipe, should sewage pipe producers exist and it has a long service life, little maintenance, low cost, but high difficulty in installation and resistance problems.

Minimum thickness of steel pipe

The critical condition of pipe minimum thickness corresponds to the pipe crush condition due to sub-atmospheric-pressure by vacuum occurrence because it is the much warning condition. For this situation, when a long pipe is forced by an external pressure p_2 greater than internal pressure p_1 , creates a differential critical pressure Δp_{cr} that can be obtained by the following equation:

$$\Delta p_{cr} = (p_2 - p_1)_{cr} = \frac{2E}{1-\nu^2} \left(\frac{t}{D} \right)^3 \tag{5.38}$$

being E the steel elasticity modulus ($E = 2 \times 10^{11} \text{ N/m}^2$), ν the steel Poisson coefficient ($\nu = 0.3$), t the pipe thickness and D the pipe diameter. By substitution of the former values on equation (5.38), yields the following result:

$$\Delta p_{cr} = 4.53 \times 10^{11} \left(\frac{t}{D} \right)^3 \tag{5.39}$$

According to European Convention for Constructional Steelwork (ECCS) in order to take into account eventual geometric imperfections, it is proposed the following relation:

$$p_{atm} = \frac{7.35}{F} \Delta p_{cr} \tag{5.40}$$

where p_{atm} is the atmospheric pressure ($p_{atm} = 1.012 \times 10^5 \text{ N/m}^2$) and F a safety factor (e.g. $F = 2$).

Based on equations (5.39) and (5.40) the minimum steel pipe thickness for a given diameter (D) will be obtained:

$$\frac{t}{D} \geq 8.4 \times 10^{-3} \tag{5.41}$$

In order to consider corrosion effects it is adopted more 1 mm to the pipe thickness.



Fig. 5.27 – Resultant forces produced by weight, pressure and change in momentum.

Net forces due to pressure and momentum change are computed as follows:

$$\begin{aligned} F_{px} &= p A (\cos \theta_1 - \cos \theta_2) & F_{py} &= p A (\sin \theta_1 - \sin \theta_2) \\ F_{mx} &= \rho Q U (\cos \theta_1 - \cos \theta_2) & F_{my} &= \rho Q U (\sin \theta_1 - \sin \theta_2) \end{aligned} \quad (5.42)$$

where p = pressure in penstock; A = cross-section area of penstock; Q = discharge; θ_1 and θ_2 = angles as shown in Figure 5.27.

At each pipe change direction, the penstock and its supporting structures must be designed to resist the forces resulting from changes in direction (Figure 5.27).

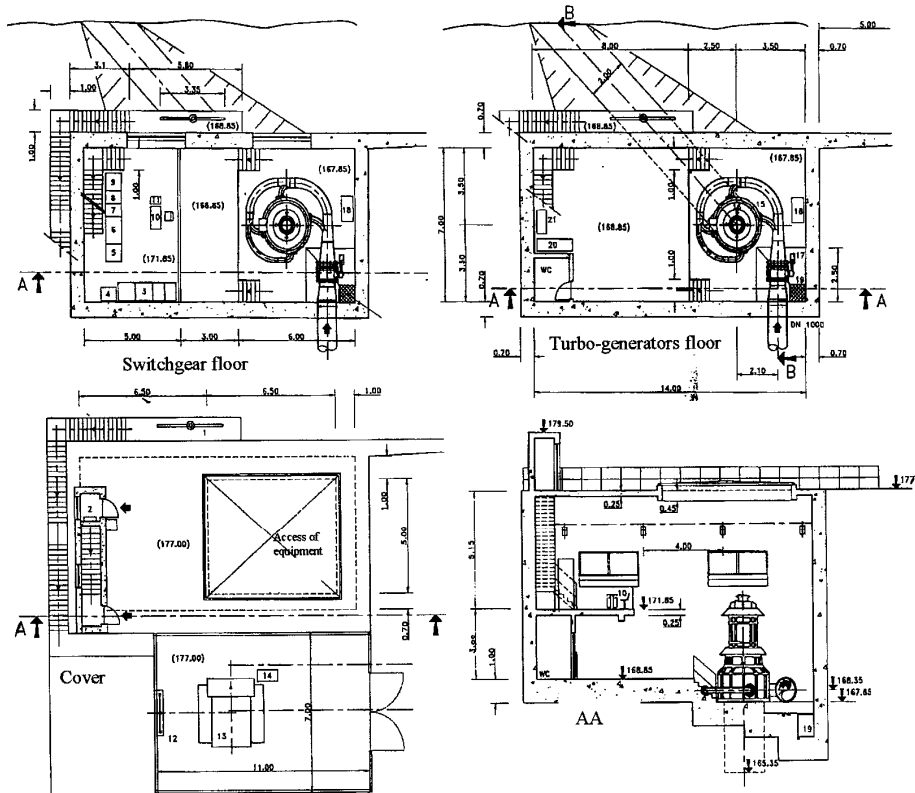
5.5- Powerhouses

The function of powerhouses consists in housing and protecting turbo-generator groups and the auxiliary equipment (e.g. safety and protection valves, electric boards, control equipment, remote controller, switchgear panel and protection equipment). The powerhouse layouts need to allow an easy installation of the equipment as well as access for inspection and maintenance of the turbines and all other equipment. Generally speaking, the dimensions of powerhouses are mainly determined by the size of the generating unit(s) and equipment (Figure 5.28).

For small power plants with long hydraulic conveyance circuit (e.g. of diversion-type) the head is generally greater than 40 m, even up to several hundred meters and the discharge is, normally, smaller than 10 m³/s. As a result, open-flume Francis, propeller or Kaplan and S-type turbines are not suitable. Experience shows that horizontal Francis and impulse (e.g. Pelton, Turgo and crossflow types) type of turbines are the appropriate. For horizontal shafts, the civil work costs can be reduced 20% due to smaller height, easier inspection and installation of protection devices, such as flywheels. A hand-operated travelling bridge crane, with a maximum capacity to move the heaviest part of the equipment (e.g. 5-15 tons) is used to assemble generators, turbines, protection valves and other components. The crane type should be considered during the design through the definition of the layout and dimensions of the powerhouse. In small hydro an alternative layout should be considered based on the use of mobile cranes and large opening on the building roof (Figure 5.28). For economic and environmental reasons the powerhouse should be as compact as possible, in order to minimize the landscape disturbance.

One of the most important construction constraints is the noise prevention outside the powerhouse. This is a crucial aspect should the powerhouse be installed near building or urban areas. To avoid the external noise all the building openings need to be prepared with special isolation devices and noise sources should be carefully identified (e.g. generator ventilation or flow through the turbine wheel).

The generator can be ventilated by natural air. The control panel is housed in an isolated and easy accessible part of the powerhouse.



- 1 – gate isolation; 2 – counting board; 3 – 6 kV equipment; 4 – battery and loader; 5 – auxiliary services board; 6 – command and control switchgear; 7 – automation switchgear; 8 – protection switchgear; 9 – tension regulation board; 10 – support table; 11 – bookcase for technical documents; 12 – 60 kV equipment; 13 – main transformer 6/60 kV – 5500 kVA; 14 – auxiliary transformer 6/0.4 kV – 50 kVA; 15 – Pelton turbine with 4 nozzles; 16 – synchronous generator of 5500 kVA – 6 kV; 17 – valve of isolation DN 800; 18 – oil hydraulic central; 19 – well of drainage and pumping; 20 – workbench; 21 – tools-case; water reservoir for WC

Fig. 5.28 – A typical example of a small powerhouse with a Pelton turbine installed.

Different characteristics of the units will impose different powerhouse layouts and sizes (see Figure 5.29 and 5.30).

The design of a powerhouse needs to take into account the hydraulic constraints for a good turbine inflow, the weight of the equipment and hydrodynamic forces that will be transmitted to the structure including the massive concrete (anchor blocks).

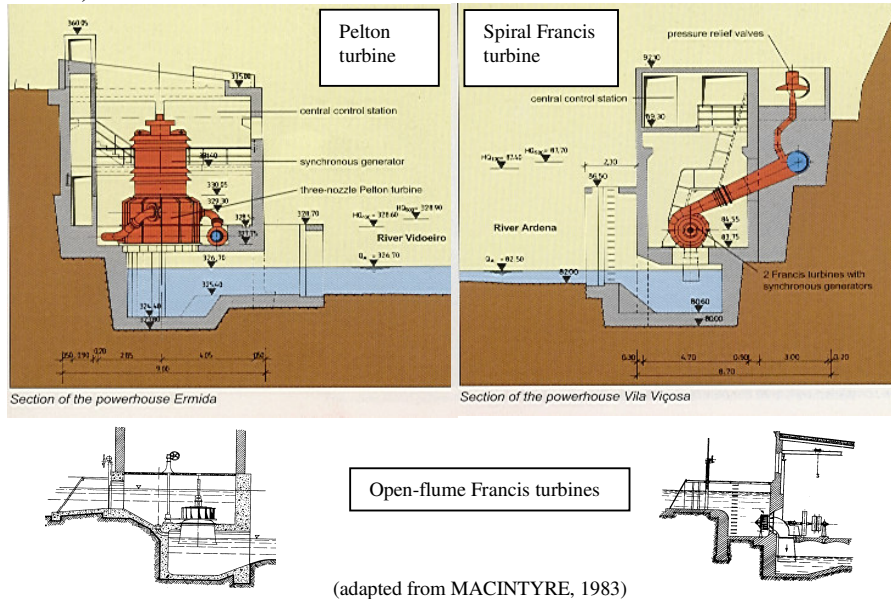


Fig. 5.29 – Different types of powerhouse layouts.

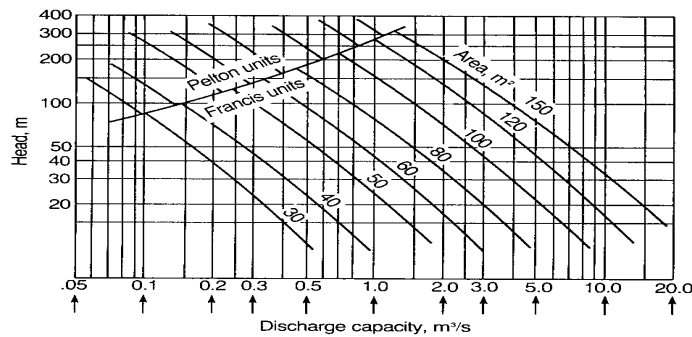


Fig. 5.30 – Powerhouse floor area required for Pelton and high head Francis turbine installations (adapted from FRITZ, 1984).

In some powerhouses the control panel and the switchboard need to be properly arranged at a level above the maximum design flood in the river, in order to be protected against at least 500-year flood. For impulse turbines it must be provided that the runner is non-submerged during operation.

5.6- Analysis of hydropower schemes

After the definition of the total characteristic of each component of a hydropower scheme (see following chapters 6, 7 and 8), it is finally possible to obtain the final design and the best solution. Figure 5.31 shows the importance of the identification of different elements to input in a complete computational model in order to obtain the dynamic response of the system.

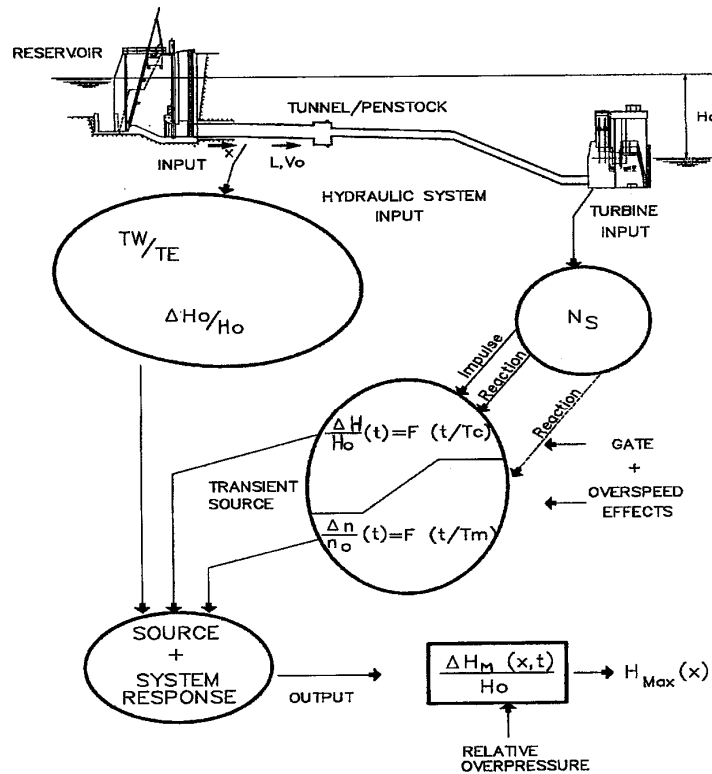


Fig. 5.31 – Typical inputs for hydropower scheme analysis.

6

SMALL HYDRAULIC TURBINES

Helena Ramos

6.1- Type of turbines

The choice of standardised turbines for small hydro schemes depends upon the main system characteristics: net head, unit discharge and unit power. Hydraulic turbines convert hydropower energy into rotating mechanical energy. The main different types of turbines depend upon the way the water acts in the runner:

- a free jet at atmospheric pressure - impulse turbines;
- a pressurised flow - reaction turbines.

Impulse turbines are more efficient for high heads. The Pelton turbine is the most known model of this type and is composed by a runner and one or more nozzles. The runner has blades with the shape of a double spoon (Figure 6.1). The jet coming from the nozzle hits the blades of the runner, transforming the flow kinetic energy into rotational mechanical energy. Each nozzle has a movable needle to control the discharge. The maximum number of nozzles is two, for horizontal shaft, or six for vertical shaft. The nozzle has a deflector, which is a device to control the flow whenever a load rejection occurs, provoking a deviation of the jet enabling its slow closing, controlling the overpressure in the penstock and avoiding the overspeed of the runner.

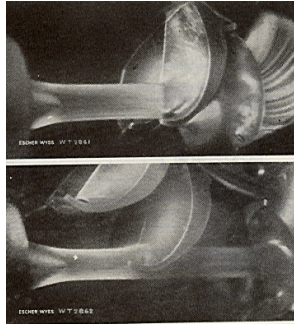


Fig. 6.1 - The trajectory of a jet from a nozzle in blades direction of a Pelton (adapted from MACINTYRE, 1983).

The main advantages of these turbines are:

- They can be easily adapted to power variation with almost constant efficiency.
- The penstock overpressure and the runner overspeed control are easier.
- The turbine enables an easier maintenance.
- Due to the jet, manufacturers of these turbines impose a better solid particles' control inducing, consequently, a lower abrasion effect.

The Pelton turbine wheel diameter is usually 10 – 20 times the nozzle jet diameter, depending on the spacing of the buckets. The net head in this type of impulse turbine is measured between the total head in the penstock just upstream the nozzle and the axis level of the water jet. Often runners are installed on both sides of the generator (double- overhang installation). Impulse turbines are provided with housings to prevent splashing, but the air within this housing is substantially at atmospheric pressure.

Reaction turbines have, normally, a closed chamber (spiral case), where the flow takes place in transforming part of pressure energy into rotational mechanical energy of the runner. A movable guide vane (or wicket gate) guides the flow around the runner, making, simultaneously, the regulation of the turbine discharge.

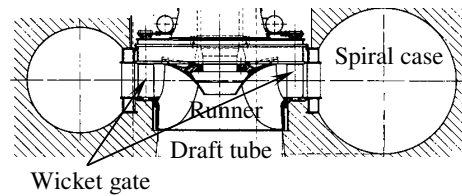


Fig. 6.2 – A scheme of a reaction turbine.

A shifting ring to which each gate is attached moves the guide vane. At downstream of the runner follows the draft tube that due to its shape (section progressively rising) allows the partial recuperation of the kinetic energy of the runner. The main advantages of this type of turbine are:

- It needs lesser installation space (e.g. the runners are smaller than Pelton runners).
- It provides a greater net head and a better protection against downstream high flood levels (can run submerged).
- It can have greater runner speed.
- It can attain higher efficiencies for higher power values.

The arrangement of the turbine shaft as vertical or horizontal and with fixed (Francis) or adjustable blades (Kaplan) are important factors to take into account in the classification. Francis turbines can be of single or double effect. Thus, depending upon the type and the main conditioning factors, a particular turbine can be classified as a function of:

- installation type;
- number of runners;
- position of the runner shaft;
- net head available.

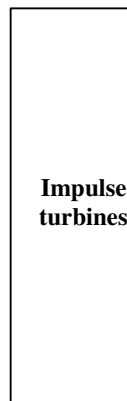
The turbine types are typically characterised by a well-known parameter: the specific speed N_s defined in 6.2 section.

In Table 6.1 it is shown a general classification of turbines with typical nominal head, discharge power and N_s .

**Table 6.1 – Resume of application range of standard turbines
(based on several manufacturers data)**

Hydraulic Turbines		H (m)	Q (m ³ /s)	P (kW)	N _s (r.p.m.) (kW, m)
Reaction	Bulb	2 - 10	3 - 40	100 - 2500	200 - 450
	Kaplan and propeller – axial flow	2 - 20	3 - 50	50 - 5000	250 - 700
	Francis with high specific speed - diagonal flow	10 - 40	0.7 - 10	100 - 5000	100 - 250
	Francis with low specific speed – radial flow	40 - 200	1 - 20	500 - 15000	30 - 100
Impulse	Pelton	60 - 1000	0.2 - 5	200 - 15000	<30
	Turgo	30 - 200		100 - 6000	
	Cross-flow	2 - 50	0.01 – 0.12	2 - 15	

Main differences between turbines (see Figure 6.3)



Pelton turbine can have single or multiple jets and is used for medium or high heads. The flow incises on the blades and falls into the tailrace canal at atmospheric pressure. It can not work submerged.

Turgo turbine is also an impulse turbine type but with different buckets, when compared with Pelton. The water enters into the runner through one side of the runner disk, through it and then emerges from the other side.

Cross-flow turbine (also designated by Ossberger, Banki or Mitchell) is used for a wide range of heads. The water passes through a guide-vane located at upstream of the runner and has a double action on the blades of the runner.

Reaction turbines

Reverse-pump operates as a reaction turbine with reverse flow, from the outlet to inlet. Since it has no flow regulation (guide vane) it can only operate under approximate constant head and discharge.

Francis turbine is of radial or mixed flow with adjustable wicket gate. It is used for medium heads. In these turbines the flow is uniformly distributed through the spiral case, that is connected to the penstock. The turbine may have vertical or horizontal shaft and it can be arranged in open flumes too.

Kaplan and **propeller** turbines are axial flow reaction types, generally used for low heads. The Kaplan has adjustable runner blades with adjustable (double regulated) or not guide vane (or single regulated). Propeller is chosen when both flow and head remain practically constants. Both types can be arranged in an open flume or with a spiral case of concrete or cast iron, similarly to Francis turbines. When the head is low and the flow is high a **Bulb** unit and S conduit can be used.

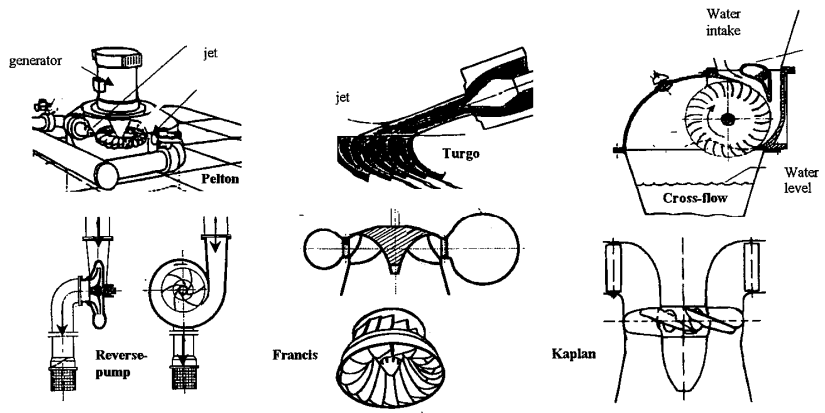


Fig. 6.3- Typical turbines for small hydropower plants.

In engineering practice, a standard type of turbine can be selected in an early design stage from charts prepared by the manufacturers like the one presented in Figure 6.4.

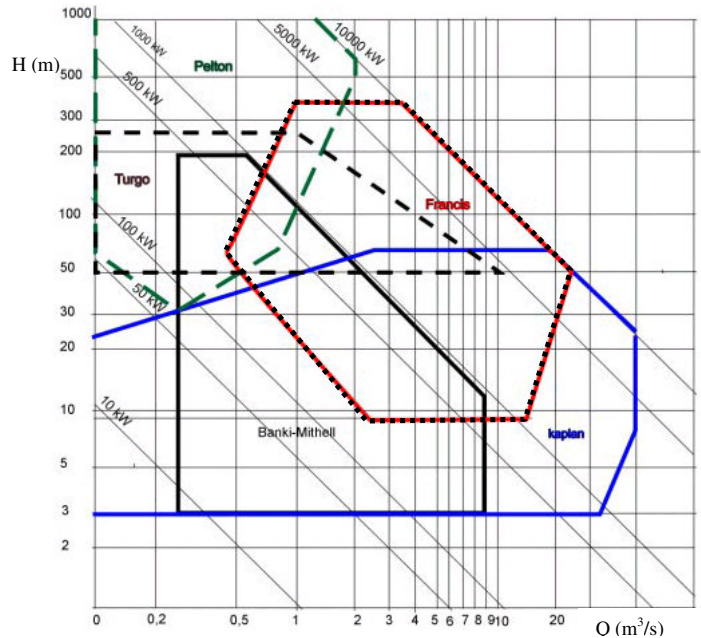


Fig. 6.4 – Example of hydro turbine range application (adapted from ALTENER, 1997).

6.2- Turbine similarity laws and specific speed (N_s)

The application of similarity laws are required in order to predict the behaviour of a full size prototype through the interpretation of model tests. The similitude theory of turbomachines is used in different applications. This theory requires the verification of three similarity conditions: geometric, kinematics and dynamic conditions. For geometric similarity, the turbine dimension and the flow passage must obey to one geometrical scale. The kinematic similarity implies equivalent velocity triangles at inlet and outlet of the runner. For the dynamic similarity there are analogous action forces (e.g. equivalent force polygon). A complete similarity between runners of different dimensions is always difficult and “scale effects” can occur. Reynolds similarity is not valid

(due to scale effects of internal loss and efficiency), because Reynolds number has lower value in laboratory model than in prototype (MATAIX, 1975 and JACOB, 1994). JACOB, 1994 and RAMOS, 1995 verified that the use of Froude similarity to guarantee the relation between the inertia forces and the gravity forces is the same for the model and the prototype. It can also guarantee the pressure gradient similarity for a given average velocity.

Any similarity is related with homologous relationships in model and in prototype, in particular, to allow the definition of the specific speed of turbines, as an important parameter of each set of similar turbines that characterises its dynamic behaviour. Based on similarity laws, a full description of the external and internal (inertia) forces balance acting on a control volume defined between inlet and outlet runner sections, through momentum equation, will provide the discharge variation.

Under similarity operational conditions, the turbine speed, head and power, both in model and prototype, follow the general equation:

$$\frac{n_{op}}{n_{om}} = \left(\frac{P_m}{P_p} \right)^{1/2} \left(\frac{H_{op}}{H_{om}} \right)^{5/4} \quad (6.1)$$

that yields in the following specific speed definition:

$$N_s = n_o \frac{\sqrt{P}}{H_o^{1.25}} \quad (6.2)$$

where n_o (rev/min) is the nominal rotational speed and N_s ((rev/min) in m, kW) is the speed of a similar turbine with unit head and unit output power during similar operating conditions. Thus, under the same conditions, the N_s value is considered as constant for similar turbines.

Impulse turbines have low specific speeds, Francis turbines have medium values of N_s and propeller or Kaplan turbines have high values.

It is important to analyse the influence of multiple turbines in the specific speed value. In small hydropower, double Francis wheel, reaction turbines with more than one-stage and Pelton turbines with several nozzles can be selected. For the case of wheels installed in parallel the discharge will increase ($Q = n \cdot Q$ and H

= cte, with n the number of wheels or nozzles) and the specific speed ($N_{s,n} = N_s \sqrt{n}$) will increase too. For different stages of wheels the head will increase ($Q = \text{cte}$ and $H = n H$) yielding in a decreasing of N_s ($N_{s,n} = \frac{N_s}{n^{3/4}}$).

The relation between real velocity components, absolute (V) and relative (or meridian) (W) of a water particle and the blade speed (C), at the inlet and outlet of the runner will depend on N_s value of each turbine:

$$\bar{V} \equiv \bar{W} + \bar{C} \quad (6.3)$$

The ratio between absolute velocities and Torricelli velocity defines the specific or unit speeds.

$$v = \frac{V}{\sqrt{2gH_0}}; \quad c = \frac{C}{\sqrt{2gH_0}}; \quad w = \frac{W}{\sqrt{2gH_0}} \quad (6.4)$$

The increase of the specific speed (N_s) implies variations on unit speeds (absolute, relative and tangential), on velocity triangles (Figures 6.5 and 6.6) and on turbine discharge. Two geometrically similar turbines have same unit speeds and analogous velocity triangles.

The N_s parameter can also be a complement parameter for turbine selection in an early design stage (Figure 6.7).

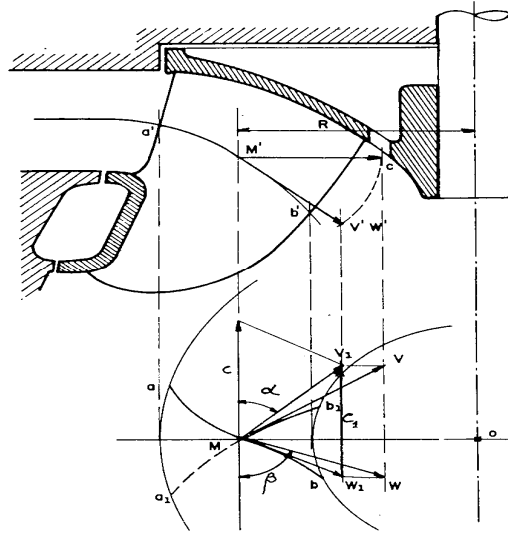
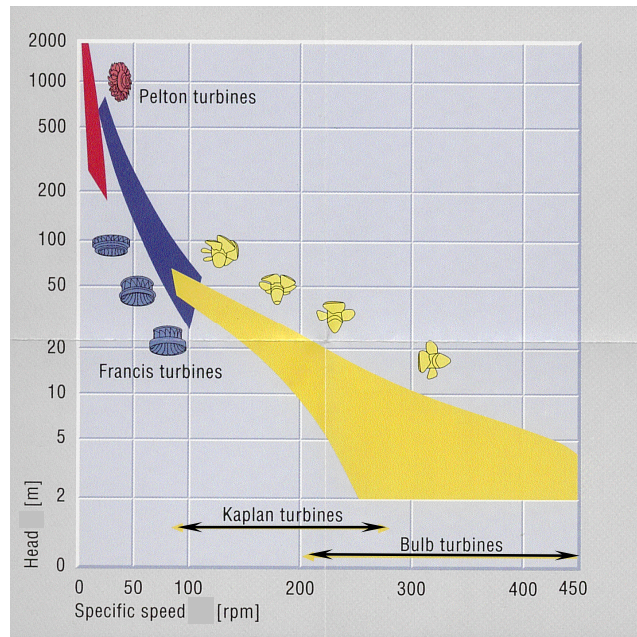


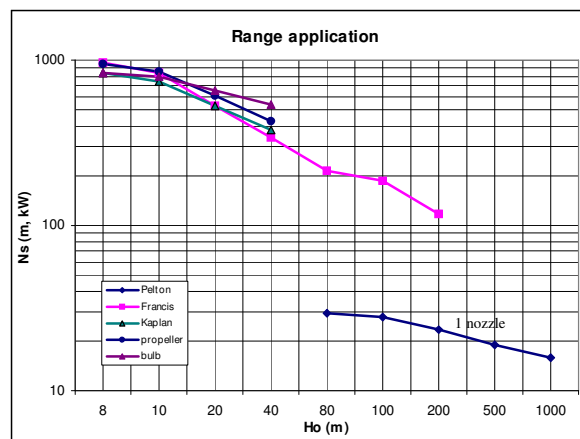
Fig. 6.5 - Velocity diagrams through a Francis runner.

Pelton	N_s (kW, m)		
	28		
Francis	80		
	130		
	210		
	250		
	300		
Kaplan	300 - 700		

Fig. 6.6 – Variation of inlet triangle velocity with the specific speed (or runner shape) (adapted from MATAIX, 1975).



A - Courtesy of VOITH HYDRO Power Generation Group, 1997.



B - Based on ALTENER, 1997.

Fig. 6.7- Example of turbine selection charts based on N_s parameter.

6.3- Turbine efficiency

The turbine efficiency is defined as the ratio of power delivered to the shaft to the power taken from the flow. Hence, in general, the efficiency of turbines is defined by:

$$\eta = \frac{BH \omega}{\gamma Q H_0} = \frac{P}{\gamma Q H_0} \quad (6.5)$$

where BH (N.m) is the torque (and P (W) the power) delivered to the shaft by the runner, ω (rad/s) is the angular velocity, Q (m^3/s) is the flow rate, H_0 is the net head on the turbine. The efficiency of various types of turbine change with the discharge, as shown in Figure 6.8. As can be seen the impulse turbine maintains high efficiency over a wide range of discharges. The efficiency of propeller turbines is very sensitive these values. The Kaplan turbine (with movable blades) maintains high efficiency over a wide range of discharge values.

Turbines can not economically operate from zero flow to rated discharge. The efficiency decreases rapidly below a certain percentage of the rated discharge. Many turbines can only operate upward 40% of rated discharge.

The total unit and “powerhouse” efficiency will be obtained by multiplication of other efficiencies (e.g. generator efficiency).

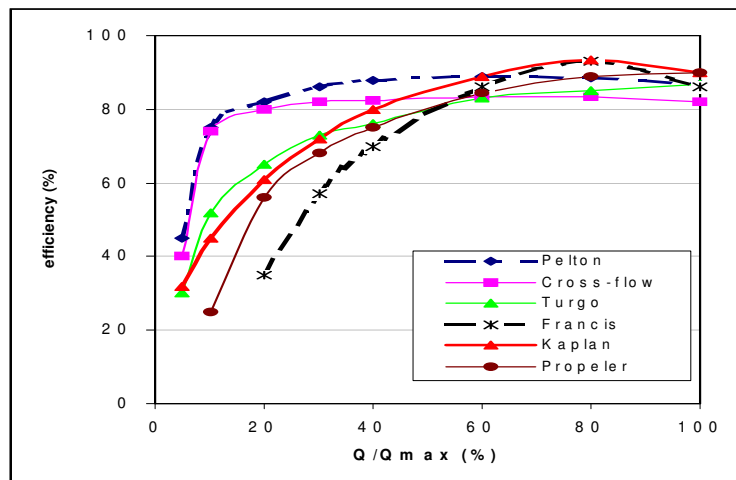


Fig. 6.8- Typical efficiency as a function of a percentage of the rated discharge for several types of turbines (adapted from ATERNER, 1997).

6.4- Dimensions of turbines

Dimensions of turbines can be obtained by manufacturer information or estimated through empirical formulas.

- **Impulse turbines**

The main criteria for design of impulse turbines, in particular, for Pelton turbines, consists in the calculation of the number of nozzles, turbine speed and the dimensions of the runner and, finally an economic comparison between the head benefit vs. powerhouse costs.

One of the most important parameter that characterises the turbo-generator group, allowing the comparison of its behaviour with other machine, is the specific speed (defined in 6.2). It is important to remember that if a turbine has more than one nozzle (N) the actual specific speed ($N_{s,N}$) relates with the specific speed of a turbine (N_s) with only one nozzle, through the following equation:

$$N_{s,N} = N_s \sqrt{N} \quad (6.6)$$

The generator is an element coupled to the turbine that transforms the mechanical energy into electric energy. For synchronous generators, its speed depends upon the frequency of the electric grid (f(Hz) in EU f=50 Hz) and the pair of poles of the generator (p):

$$p n = 60 f \quad (6.7)$$

The net head is obtained by

$$H_o = N_{res} - N_{noz} - \Delta H_T \quad (6.8)$$

where

N_{res} = reservoir (upstream) water level;

N_{noz} = nozzle axis level ($N_{noz} = N_{river} + H_s$);

N_{river} = water level at outlet powerhouse (e.g. for 10 years of return period);

H_s = freeboard between the N_{river} and the nozzle axis for turbine with vertical shaft, and between river and the lowest runner point for turbines with horizontal shaft (SIERVO and LUGARESÌ, 1978):

$$H_s = 1.87 + 2.24 \frac{Q}{N_s} \quad (\text{e.g. } H_s \approx 2\text{m}) ;$$

Q = turbine rated flow;

ΔH_T = total head losses in the hydraulic circuit.

and the turbine output capacity is given by

$$P = \gamma \eta Q H_o \quad (6.9)$$

In order to avoid turbines with low efficiency or bad designed characteristics it is convenient to adopt N_s according to manufacturer data information (or by available literature) based, normally, on the net head and results of turbines already tested.

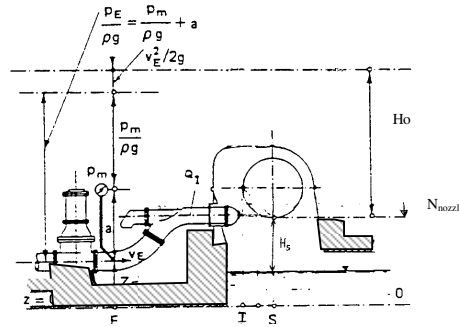


Fig. 6.9 - Definition of the net head for a Pelton with one nozzle.

The nozzle can have different open/close positions and it can be defined by a displacement X (of the needle). When the nozzle is completely closed X is null.

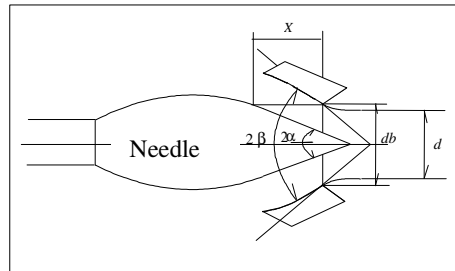


Fig. 6.9 – Scheme of a Pelton nozzle.

According to experimental tests (VIVIER, 1966 and MATAIX, 1975) the discharge coefficient can be considered constant (except for very small open degrees) and the discharge will be proportional to the free section of the nozzle. Before the simulation of nozzle manoeuvres it is necessary to calculate the initial conditions (steady state) and the nozzle diameter. Based on the maximum discharge and the free cross section defined by:

$$S = \pi \text{sen}\alpha \left(D_N X - \frac{\text{sen } 2\alpha}{2} X^2 \right)$$

$$Q_{\text{noz}} = CS \sqrt{2gH_0} \quad (6.10)$$

$$X = \frac{\theta D_N}{1.2}$$

where X is represented in Figure 6.9.

For each nozzle it yields a diameter given by the following equation

$$D_N = K_\alpha \sqrt{\frac{Q_{\text{noz}}}{\sqrt{H_0}}} \quad (6.11)$$

where

Q_{noz} = maximum discharge of each nozzle;

θ = the open degree of the nozzle ($\theta = 1$ completely open; $\theta = 0$ completely closed);

C = discharge coefficient (≈ 0.97 - VIVIER, 1966 and MATAIX, 1975);

S = flow cross-section area of the nozzle;

K_α = coefficient that depends upon the angle α of the conical needle end (e.g. $\theta = 1$ $K_\alpha = 0.5445$).

For constructive proposes, according to SIERVO and LUGARESI, 1978, the external diameter D_2 and the diameter related to the centreline of the blades D_1 are

$$D_1 = \frac{60 K_u \sqrt{2gH_0}}{\pi n} \quad (6.12)$$

$$D_2 = D_1 (1.028 + 0.0137 N_{s,N}) \quad (6.13)$$

and the blade dimensions are a function of nozzle diameter (D_N)

$$D_N = \frac{D_1 N_{s,N}}{250.74 - 1.796 N_{s,N}} \quad (6.14)$$

$$H_1 = 3.20 D_N^{0.96} \quad \text{and} \quad H_2 = 3.23 D_N^{1.02} \quad (6.15)$$

where

$K_u = 0.5445 - 0.0039 N_{s,N}$ is the periphery speed coefficient;

$N_{s,N} = \frac{n}{H_o^{5/4}} \sqrt{\frac{P}{N}}$ is the specific speed for each nozzle (N = number of nozzles);

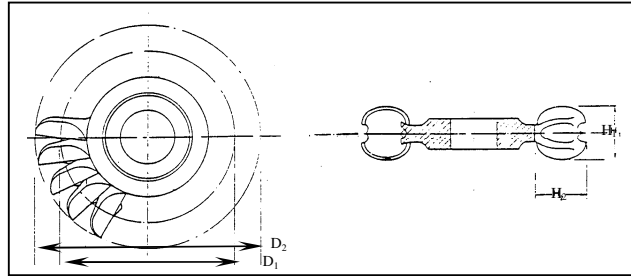


Fig. 6.10 – A typical Pelton runner
(adapted from SIERVO and LUGARESI, 1978).

Theoretically the choice falls on the greater specific speed turbine solution, because it corresponds to smaller dimensions. However, on the one hand it must be based on economic assessment, that includes civil works and equipment costs, on the other hand the mechanical disposition of the group, maximum peripheral speed of the rotor, and the minimum dimension for powerhouse, could help to find the best choice. It is important to be aware that turbines with greater specific speed lead a smaller runner diameter. However, for Pelton turbines the number of nozzles can constrain powerhouse dimensions.

The dimensions of a Pelton turbine casing depend on the outlet diameter D_2 of the wheel, where L gives the horizontal size of the casing. For prismatic casings this value has been assumed equal to the average diameter of the circle inscribed and circumscribed on the casing.

$$L = 0.78 + 2.06 D_2 \quad (6.16)$$

The distance G between the wheel centreline and the top of the casing and the other dimensions for casing and for spiral case are defined by

$$\begin{aligned}
 G &= 0.196 + 0.376D_2 \\
 F &= 1.09 + 0.71L \\
 H &= 0.62 + 0.513L \\
 I &= 1.28 + 0.37L \\
 B &= 0.595 + 0.694L \\
 C &= 0.362 + 0.68L \\
 D &= -0.219 + 0.7L \\
 E &= 0.43 + 0.70L
 \end{aligned}
 \tag{6.17}$$

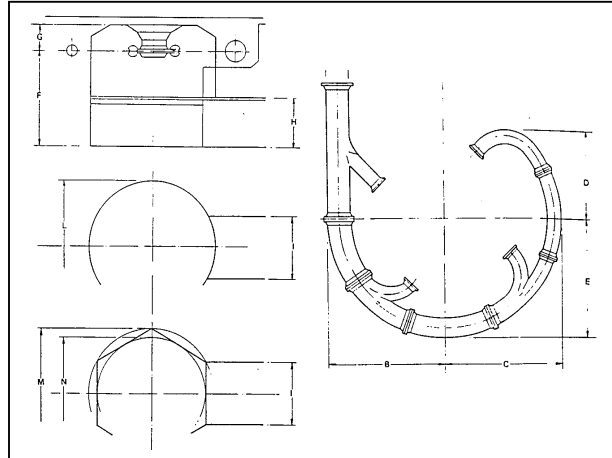


Fig 6.11 – Casing dimensions (adapted from SIERVO and LUGARESI, 1978).

- **Reaction turbines**

Any reaction turbine is composed by the following elements:

Spiral case - with decreasing cross section to downward direction to transform the pressure energy into kinetic one.

Wicket gate (or guide vane) - guides the inlet of the flow into the runner, delivers it uniformly and controls the turbine discharge.

Runner - radial or axial with or without movable blades.

Draft tube - pipe with increasing cross section to downward direction.

For reaction turbines, the flow is totally pressurised and the net head is defined by

$$H_o = N_{res} - N_{river} - \Delta H \tag{6.18}$$

and the rotor speed is similarly calculated as for impulse turbines. The estimation of specific speed is obtained from manufacturer data as a function of head (see 6.2)

Defined the turbine speed, the next step consists in calculation the runner position in order to avoid cavitation. The admissible suction head (h_{smax}) is defined by the difference between the characteristic runner section and the tailrace level. When the suction head is negative, the turbine operates in “back-pressure”.

$$h_{smax} = \frac{p_{atm}}{\gamma} - \frac{t_v}{\gamma} - \sigma H_o \quad (6.19)$$

where

$\frac{p_{atm}}{\gamma}$ = local barometric head (m);

$\frac{t_v}{\gamma}$ = vapour pressure (m);

σ = dynamic depression coefficient or Thoma coefficient;

H_o = net head (m).

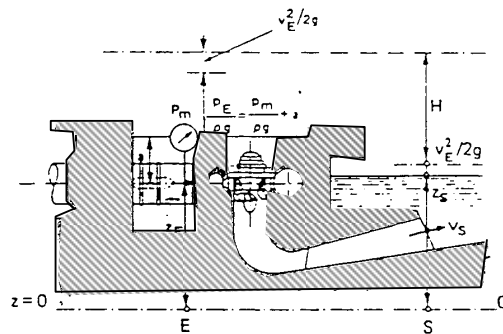


Fig. 6.12 - Definition of the net head for a reaction turbine.

The barometric head depends on the local altitude and the vapour pressure from the local temperature (Table 6.1).

Table 6.1 - Barometric head and vapour pressure

Altitude (m)	$\frac{P_{atm}}{\gamma}$ (m)	Temperature (°C)	$\frac{t_v}{\gamma}$ (m)
0	10.35	5	0.089
500	9.75	10	0.125
1000	9.18	15	0.174
1500	8.64	20	0.239
2000	8.12	25	0.324

(adapted from Almeida, 1995)

Thoma coefficient (σ) gives the susceptibility to the cavitation occurrence of a reaction turbine. This parameter depends upon N_s . Several curves have been proposed being one of them presented by Bureau of Reclamation, 1976 for reaction turbines with vertical shaft:

$$\sigma = \frac{N_s^{1.64}}{38652} \quad (6.20)$$

with N_s in (m, kW)

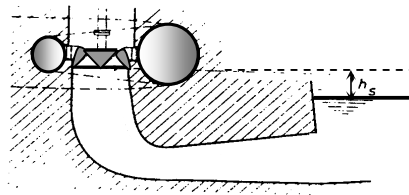


Fig. 6.13 – Schematic representation of suction turbine head (h_s).

Out of normal operating conditions induce the cavitation phenomena occurrence. As a result of cavitation development, the performance of the turbine will be affected, since efficiency, appearance of vibrations and noise or blades erosion:

- whenever exists a discontinuity of the flow;
- the surface of a blade with different stages of vortex development: initially by separating zones, after by bulb formation and consequent rupture that provokes vibrations and serious noise;
- vortex band cavitation at the inlet of a draft tube during partial load operating conditions, especially for Francis, inducing periodical pressure fluctuation, noise and hard vibrations in the turbine.

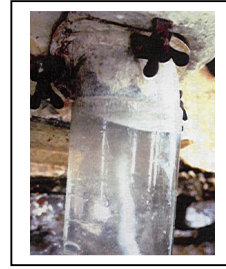


Fig 6.14 – Vortex in the draft tube of an experimental facility (at LNEC, RAMOS, 1995).

In order to predict turbine dimensions and the necessary excavation for the powerhouse in an early phase of design, SIERVO and LEVA, 1976 present a practical formula based on several parameters:

$$D_3 = 84.5 K_u \frac{\sqrt{H_o}}{n}$$

$$K_u = 0.31 + 2.5 \times 10^{-3} N_s$$

$$D_1 = D_3 \left(0.4 + \frac{94.5}{N_s} \right)$$

$$D_2 = \frac{D_3}{0.96 + 0.00038 N_s}$$

$$H_1 = D_3 (0.094 + 0.00025 N_s)$$

$$H_2 = D_3 \left(-0.05 + \frac{42}{N_s} \right)$$

(6.21)

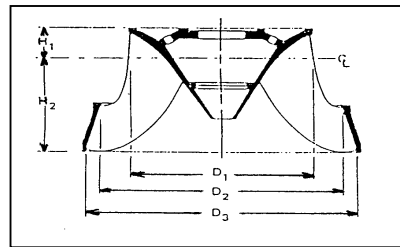


Fig. 6.15 – Reaction turbine runner (adapted from SIERVO and LUGARESI, 1976).

In the same way, the steel spiral case and the draft tube dimensions are estimated by the following relationships (SIERVO and LUGARESI, 1976):

- Francis turbine

$$A = D_3 \left(1.2 - \frac{19.56}{N_s} \right)$$

$$B = D_3 \left(1.1 + \frac{54.8}{N_s} \right)$$

$$C = D_3 \left(1.32 + \frac{49.25}{N_s} \right)$$

$$D = D_3 \left(1.5 + \frac{48.8}{N_s} \right)$$

$$E = D_3 \left(0.98 + \frac{63.6}{N_s} \right)$$

$$F = D_3 \left(1 + \frac{131.4}{N_s} \right)$$

$$G = D_3 \left(0.89 + \frac{96.5}{N_s} \right)$$

$$H = D_3 \left(0.79 + \frac{81.75}{N_s} \right)$$

$$I = D_3 (0.1 + 6.5 \times 10^{-4} N_s)$$

$$L = D_3 (0.88 + 4.9 \times 10^{-4} N_s)$$

$$M = D_3 (0.60 + 1.5 \times 10^{-5} N_s)$$

$$N = D_3 \left(1.54 + \frac{203.5}{N_s} \right)$$

$$O = D_3 \left(0.83 + \frac{140.7}{N_s} \right)$$

$$P = D_3 (1.37 - 5.6 \times 10^{-4} N_s)$$

- Kaplan turbine

$$A = D_M 0.40 N_s^{0.2}$$

$$B = D_M (1.26 + 3.79 \times 10^{-4} N_s)$$

$$C = D_M (1.46 + 3.24 \times 10^{-4} N_s)$$

$$D = D_M (1.59 + 5.74 \times 10^{-4} N_s)$$

$$E = D_M (1.21 + 2.71 \times 10^{-4} N_s)$$

$$F = D_M \left(1.45 + \frac{72.17}{N_s} \right)$$

$$G = D_M \left(1.29 + \frac{41.63}{N_s} \right)$$

$$H = D_M \left(1.13 + \frac{31.86}{N_s} \right)$$

$$I = D_M \left(0.45 - \frac{31.80}{N_s} \right)$$

$$L = D_M (0.74 + 8.7 \times 10^{-4} N_s)$$

$$M = D_M / (2.06 - 1.2 \times 10^{-3} N_s)$$

$$N = D_M (2.0 - 2.14 \times 10^{-6} N_s)$$

$$O = D_M (1.4 - 1.67 \times 10^{-5} N_s)$$

$$P = D_M \left(1.26 - \frac{16.35}{N_s} \right)$$

$$\begin{aligned}
 Q &= D_3 \left(0.58 + \frac{22.6}{N_s} \right) & Q &= D_M \left(0.66 - \frac{18.40}{N_s} \right) \\
 R &= D_3 (1.6 - 0.0013 N_s) & R &= D_M (1.25 - 7.98 \times 10^{-5} N_s) \\
 S &= N_s / (-9.28 + 0.25 N_s) & S &= D_M \left(4.26 + \frac{201.51}{N_s} \right) \\
 T &= D_3 (1.50 + 1.9 \times 10^{-4} N_s) & T &= D_M (1.20 + 5.12 \times 10^{-4} N_s) \\
 U &= D_3 (0.51 - 7 \times 10^{-4} N_s) \\
 V &= D_3 \left(1.10 + \frac{53.7}{N_s} \right) \\
 Z &= D_3 \left(2.63 + \frac{33.8}{N_s} \right) & Z &= D_M \left(2.58 + \frac{102.66}{N_s} \right)
 \end{aligned}
 \tag{6.56}$$

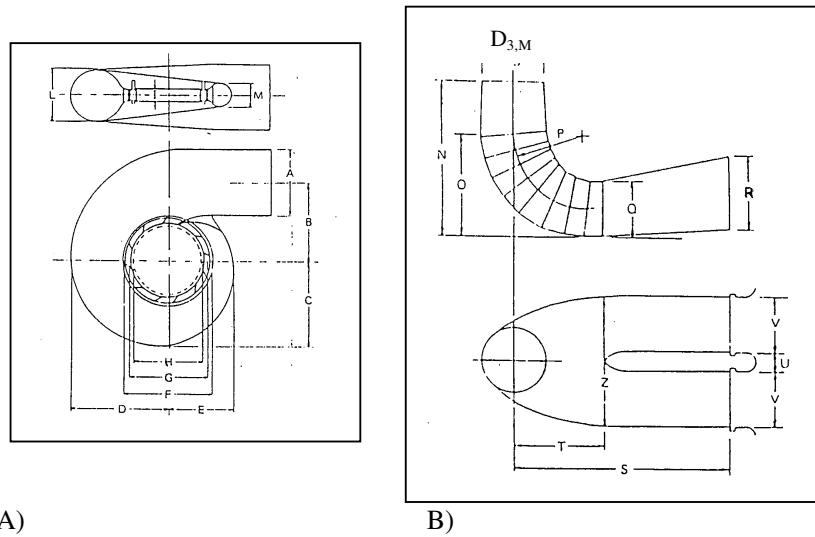


Fig 6.34 – Spiral case dimensions (A) and draft tube dimensions (B)
(adapted from SIERVO and LUGARESI, 1978).

Through these interpolation functions the dimension of Francis and Kaplan turbines can be characterised, allowing a better estimate of economic costs for powerhouses and the definition of the relative position of each component.

7

HYDRAULIC TRANSIENTS AND DYNAMIC EFFECTS

A. Betâmio de Almeida
Helena Ramos

7.1- Introduction

When the turbined flow changes during the hydropower operation disturbance will occur along the hydraulic conveyance system. Hydraulic transients are the regimes caused by these types of disturbances during a change from one steady state to another. Physically, the hydraulic transients provoke surface gravity waves along the diversion canals and pressure elastic waves along the pressurised pipes.

Since the early design phases of the hydropower scheme, the hydraulic transients should be considered in order to find the technical specifications corresponding to a more economic and safer layout. The flow changes are inevitable: any turbo-generator unit must, at sometimes, to start-up, to undergo changes of load or to be switched off. Unpredictable events, like human errors, equipment failures or environmental hazards, can also cause severe unsteady regimes.

In canal systems a flow stoppage at the powerhouse will induce a transient water depth wave that propagates upstream. The canal freeboard and all hydraulic components along the canal need to be prepared for this situation.

In pressurised systems all the components need to support the maximum transient overpressures and under pressures due to flow changes.

The final optimum solution need to conciliate the hydraulic transients with all collateral dynamic effects, as well as the interaction with the hydro-electric equipment:

- hydrotransients will be the major factor in definition of operational safety conditions, in order to avoid pipe rupture, water column separation or air entrance into the system and the overtopping of canal or forebay walls;
- hydrotransients and flow control manoeuvres will influence the overall system response, in what concerns the turbine runner overspeed, the automatic control system (e.g. regulators and governors) and the stability conditions as well as the structural efforts induced by the hydrodynamic forces.

The type of methods of analysis to be applied will depend on the design phase and on the characteristics of each hydrosystem.

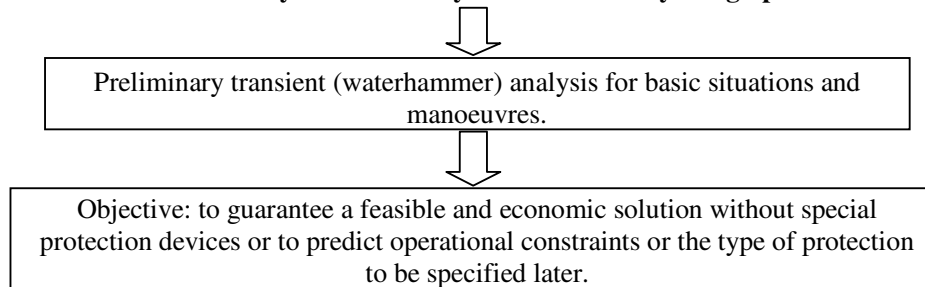
Accidents due to hydraulic transients can represent a very important risk in what concerns both economic and life losses and of the powerplant operation, reliability, as well as the production quality.

Since the sixties, computer methods for hydraulic analysis and simulation were developed and it is now possible to have a large number of techniques to deal with hydraulic transients and the global dynamic behaviour of the hydrosystems.

In some cases the diversion system and the hydraulic conveyance circuit is a mixed one (free-surface diversion canals and pressure circuits or penstocks) but in some cases it is more economic and environmentally more acceptable to select a full pressurised hydraulic system.

For small hydro-systems the general methodology of transient analysis can be the following one:

A- Preliminary and feasibility studies and early design phases.



B- Detailed design studies for the all for tenders.



Detailed transient analysis and studies, including the selected protection systems, in order to obtain the hydraulic response to normal and abnormal turbine operational conditions and select the main parameters of the equipment.



Objective: to specify the main component characteristics, namely in what concerns the hydraulic conveyance circuit (canal, forebay and penstocks or tunnels and conduits) and the flow control equipment (safety and control valves, time of manoeuvres, unit inertia, among others), based on estimated equipment characteristics.

C- Final studies for construction and operation.



Detailed transient analysis and computer simulations including the characteristics of the selected equipment and final specifications of the civil works.



Objective: to verify the safety level of the hydro-system and to specify operation rules and to support the software development for special automation systems.

The complete hydraulic analysis, including the transient regimes and the interactive dynamic effects, is a complex topic that justifies, in large hydroelectric schemes, research activities related to special phenomena and the development of advanced computer codes. For small hydropower plants, the restricted design budget will certainly impose the application of well-known criteria and already existing operational methods of analysis. However, the computer codes based on the complete unsteady and transient hydrodynamic equations are, now, relatively easy to use and the old approximate waterhammer analysis methods are not justifiable and can be dangerous from both the economic and the safety point of views.

The hydraulic analysis will strongly depend on the type of electric grid to which the small hydroplant is connected: 1) a connection with an isolated (or island) grid will impose more severe constraints, especially in what concerns the dynamic effects and the stability of regulation of the turbine speed; 2) a

connection to a large national electric grid with a much greater power production will make easier the powerplant operation in what concerns the dynamic effects.

7.2- Canal systems

In a diversion canal, the water levels will vary during the transient regime according to the flow changes and the system characteristics.

Normal operations will include the complete powerplant shutdown: for this condition the water level will increase and after a period of time the wave fluctuations will dampen and the water level will align itself with the static headwater level or other situation compatible with the control gates and safety spillways.

As soon as the flow through the turbines is locked a pressure flow will travel upstream along the penstock and when it arrives at the pipe intake, the water level at the canal system begins to rise until the depth, the velocity and, hence, the discharge reach new values. The sudden increase in depth gives rise to a gravity wave (or bore) which propagates with a relative velocity V_p :

$$V_p = \sqrt{\frac{gA_2(A_2h_{G2} - A_1h_{G1})}{A_1(A_2 - A_1)}} \quad (7.1)$$

where 1 and 2 correspond to upstream and downstream of the wave front, respectively; A is the flow cross section area; h_G - depth of the gravity centre of the area A .

In free-surface flow, the complete dynamic model is based on the Saint-Venant equations, which must be written under the conservative form, in order to be able to simulate the bore propagation (RAMOS, 1995):

$$\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} = D(U) \quad (7.2)$$

where U , $F(U)$ and $D(U)$ are the following vectors:

$$U = \begin{bmatrix} A \\ Q \end{bmatrix}; \quad F(U) = \begin{bmatrix} Q \\ \frac{Q^2}{A} + gAh \end{bmatrix}; \quad D(U) = \begin{bmatrix} 0 \\ gA(s - J) \end{bmatrix} \quad (7.3)$$

and x = distance along the canal axis (m); t = time (s); A = cross-section flow area (m^2); Q = discharge (m^3/s); h = water depth (m); s = canal bottom slope (-); J = slope of the energy grade line (-); and g = gravity acceleration (m/s^2).

A true bore will be formed should the surface elevation of the surge above the initial level be more than 20% of the average initial depth. When the wave height is smaller, a train of short waves (or movable undulate jump) will be created. The computer solution of the system of equation (7.3) based on adequate numerical techniques (method of characteristics or finite difference techniques) and on the boundary conditions will give the depth variation along the canal (see ALMEIDA and KOELLE, 1992).

In a detailed transient analysis and canal design, the secondary oscillations, known by Favre waves, need to be added to the surge waves, or bores obtained through the integration of Saint-Venant equations (7.2). PREISSMAN AND CUNGE, 1967 present a methodology to calculate the amplitude of these waves (Figure 7.2), based on Froude number for the propagation of a bore:

$$F_{r1} = F_{r0} + \sqrt{\frac{1}{2} \left(1 + \frac{h_1}{h_2} \right)} \quad (7.4)$$

where

$$F_{r0} = \frac{V_1}{\sqrt{g h_1}}$$

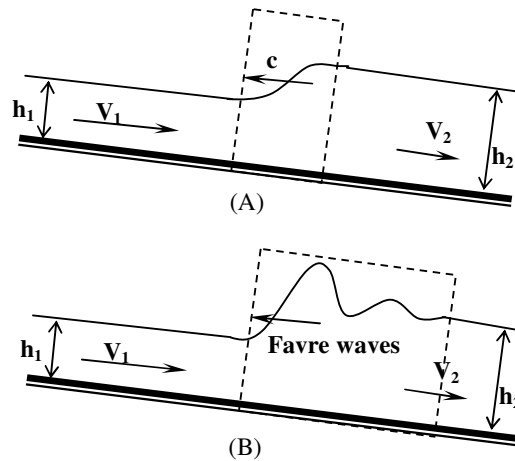


Fig. 7.1 – Propagation of bores A) based on Saint-Venant equations; B) considering Favre waves.

The limits of different types of configurations were obtained: in case of $1 < F_{r1} < 1.3$ will appear secondary waves overlapping the main wave (Figure 7.1); in case of $F_{r1} > 1.7$ breaking waves or moving hydraulic jump will appear.

Favre waves constrain the height of the canal walls, especially along the downstream 2/3 length of the canal. However, it is necessary to consider other factors that can influence the water level along the canal: sedimentation of solid particles; wind induced waves; lateral discharge from hillside; variation of canal wall roughness and a variation on the turbine maximum discharge. Considering these factors, US Bureau of Reclamation (AISENBREY et al., 1978) and INVERSIN, 1978 propose a minimum limit of ≈ 0.15 m for the wall freeboard.

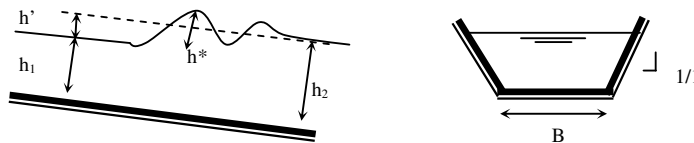
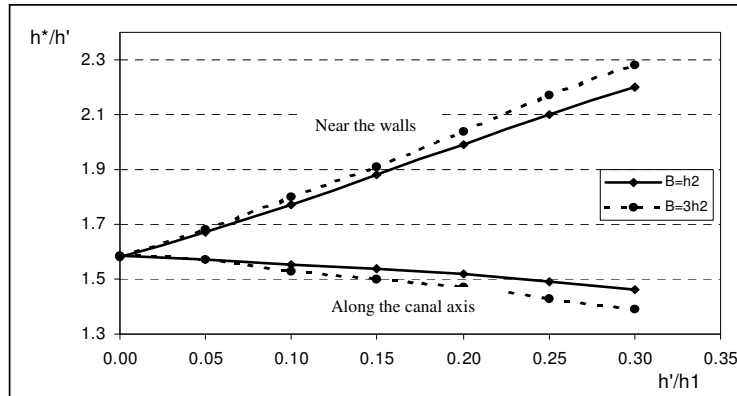


Fig. 7.2 – Maximum amplitude of Favre waves in trapezoidal canal with 1/1 lateral wall slope (adapted from PREISSMAN AND CUNGE, 1967).

For a preliminary analysis and canal freeboard evaluation, the Feifel formula can be applied in order to calculate the surge depth induced by the complete downstream flow stoppage:

$$\Delta h = \frac{V_1^2}{2g} + \sqrt{\left(\frac{V_1^2}{2g}\right)^2 + 2 \frac{V_1^2}{2g} \frac{A_1}{B_2}} \quad (7.5)$$

where:

Δh = surge depth above the initial canal water level;

V_1 = initial upstream flow velocity (before the flow stoppage in the powerhouse);

A_1 = flow cross section for the initial discharge and flow velocity V_1 ;

B_2 = surface width induced by the bore (this value needs to be evaluated by iteration).

In powerplants equipped with action or impulse type turbines (e.g. Pelton turbines), the turbined water will flow downstream turbines by a tailrace canal. Should this canal be long enough and the transient free surface flow due to turbine discharge variations should also be considered. For this type of powerplant unit the turbine wheel should be always placed above the highest downstream level, including the selected design flood level at the downstream river, where the turbined flow will be conducted. The canal freeboard or the ceiling level, in case of a closed canal or tunnel, need to consider these aspects as well as the need for an air flow circulation during normal turbine operation.

If a forebay is placed between the diversion canal and penstock intake, their geometric characteristics will modify the canal surge.

The canal-forebay characteristic parameter PCC is defined by:

$$PCC = \frac{A_F \Delta H_c}{Q_0 \left(\frac{L}{\sqrt{gh}} + \frac{Q_0}{gA \frac{\Delta H_c}{L}} \right)}$$

being

A – flow cross section area;
 A_F – horizontal forebay area;
 h – flow depth;
 L – canal length;
 Q_0 – initial discharge;
 ΔH_c – water level difference between upstream and downstream end sections of the diversion canal.

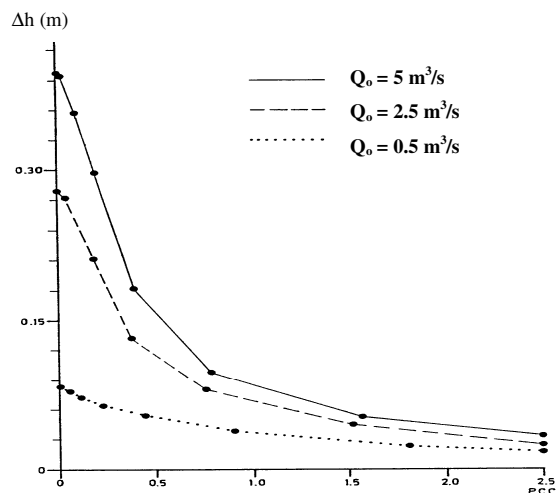


Fig. 7.3 – Example of upsurge (Δh) wave attenuation due to forebay effect after an instantaneous flow stoppage at the powerplant ($L = 1000 \text{ m}$ and $s = 0.0005$ – adapted from PINHEIRO, 1989)

For $PCC=0$ (no forebay) the Δh values will correspond to the no modified surge or bore height. The increase in the horizontal forebay surface area will attenuate the bore height Δh that will propagate upstream the diversion canal.

In order to guarantee the maximum powerplant head and generating power, an automatic water level is typically placed in the forebay. At upstream side of the diversion canal the flow changes can be caused either by natural river discharge variation or by a gate action. At downstream side, the main source of level variation is the variation of the turbine discharge. In a simplified way, for high head plants with impulse turbines it can be admitted that penstock discharge is imposed by the regulator through the turbine gate (nozzle gate) position. In this case, the water level regulating can be based on a P.I.D. (Proportional, Integral and Derivative) regulator for a stable and efficient control (ALMEIDA and KOELLE, 1992 and RAMOS, 1995).

7.3- Pressurised systems

7.3.1- Typical transient regimes

When the flow through a penstock diminishes too rapidly, the pressure along the pipe upstream the flow control device will rise above the initial level and might cause the penstock to burst. At downstream side of the control device the pressure variation will follow the opposite way, when the flow diminishes the pressure will tend to lower. This phenomenon is typically known as the waterhammer and is one of the most dramatic aspects of the hydraulic transients (RAMOS, 1995).

In any pressure transient analysis the following steps need to be considered:

- 1- The physical origin of the phenomenon and its mathematical characterisation.
- 2- The selection of scenarios compatible with the hydropower characteristics and the evaluation of the transient pressure variations, as accurate as possible according to the design stage.
- 3- The selection, analysis and specification of special protection operational procedures or/and devices or components to control the transient pressure variations and other harmful dynamic effects on the plant operation.

In what concerns the operational conditions to be considered in small hydropower schemes the following regimes can be selected:

- Normal operating conditions, as expected or specified, that should not induce any difficulty or problem (maximum safety factors).

- Emergency operating conditions, probable but unexpected they may cause some inconveniences but should not strongly damage the hydrosystem (average safety factors).
- Exceptional operating conditions, very unexpected and highly improbable, they may cause severe damage to the hydrosystem (minimum safety factors).

In Table 7.1 some examples of operating conditions are shown.

Table 7.1 - Examples of operational conditions for transient analysis and simulation (adapted from PEJOVIC et al., 1987).

Normal	<ul style="list-style-type: none"> • Steady-state flow regimes for different turbine discharges and load elements. • Steady-state flow regimes for turbinning no-load conditions. • Transient operation to synchronous regime and loading conditions. • Load rejection during turbine operation followed by closure of the flow control equipment. • Penstock filling and emptying situations.
Emergency	<ul style="list-style-type: none"> • A failure on the closure mechanism of a turbine wicket. • A failure on a protection device against waterhammer. • The closure of the safety guard valve of one unit after a complete load rejection of the plant. • Flow control under turbine runaway regime. • Rapid start-up of the plant followed by a failure of one unit. • Unfavourable succession of operations with overlapping of transient regimes.
Exceptional	<ul style="list-style-type: none"> • Complete failure of the turbine wicket closure mechanisms with a very fast flow stoppage (in a time less than $2L/c$). • More than one protection or safety device fails. • Complete blockage of all turbine wickets and closure of all penstock safety valves. • Downstream or draft column separation and closure due to the reverse flow. • Resonance or oscillation phenomena. • Breakdown of the penstock. • Seismic induced hydrodynamic actions.

The hydraulic transient effects will strongly depend on the overall hydrosystem characteristics, including the number of units and type of turbines. In all cases

the main objective is to prevent any serious damage to the penstocks or other pressure conduits, as well as to any other component of the hydrosystem. In what concerns the structural penstock protection both the maximum and minimum transient pressures need to be controlled in order to avoid that: 1) the allowable maximum pressure be exceeded, to prevent a breakdown or pipe burst; and 2) the sub-atmospheric pressure in order to avoid cavitation phenomena and water column separation effects, as well as a potential pipe wall buckling event. Typically, the allowable maximum relative transient head variations $\Delta H/H_0$ will depend on the design head:

H₀ (m)	Allowable maximum $\Delta H/H_0$
500 - 200	0.15 - 0.20
200 - 50	0.25 - 0.35
<50	0.50

In all cases the downsurge or minimum transient pressure should be properly considered in order the penstocks or conduits be protected against the formation of under atmospheric pressures and vacuum formation (water column separation).

7.3.2- Preliminary analysis

Any disturbance induced to the flow will propagate as a wave with a finite relative velocity known as the elastic wave celerity whose value will depend on the elastic properties of both the fluid and the pipe or tunnel wall according to this general formula:

$$c = \sqrt{\frac{K}{\rho[1 + (K/E)\psi]}} \quad (7.6)$$

where

E - Young's modulus of elasticity of the pipe wall;

K – fluid bulk modulus of elasticity (2×10^3 MPa for water);

ψ - parameter that depends upon the pipe structural constraints;

ρ – fluid specific mass (10^3 kg/m³ for water).

For thin wall pipes $\psi = D/t$ (with D the pipe diameter and t the wall pipe thickness) and the celerity formula will be for water and S.I. units:

$$c = \frac{1425}{\sqrt{1 + \frac{KD}{Et}}} \quad (7.6a)$$

Should the pipe or tunnel thickness be very large and the elastic wave celerity will be 1425 m/s. The pipe or tunnel elasticity will make to lower the celerity. The pressure fluctuations related to these elastic waves will have, in a uniform pipe of length L, the period $4L/c$.

The basic waterhammer theory (ALMEIDA and KOELLE, 1992) teaches us that the maximum transient pressure variation ΔP_J , due to a flow disturbance, follows the Joukowsky formula:

$$\Delta p_J = \pm \rho c \Delta V \quad (7.7)$$

where ΔV is the flow velocity variation.

The negative signal is for upstream side of the flow control device and the positive signal is for downstream side. Thus, for a flow velocity reduction $\Delta p > 0$ (the pressure will increase) at upstream side and will be $\Delta p < 0$ at downstream side. For very long pipes with high friction headloss formula (7.7) for upstream pressure variation needs to be modified:

$$\Delta p_J^* = \Delta p_J \left[1 + \frac{\Delta H_o}{\Delta H_J} - \frac{1}{12} \left(\frac{\Delta H_o}{\Delta H_J} \right)^3 \right] \quad (7.8)$$

where ΔH_o is the initial pipe head loss and ΔH_J the head variation corresponding to Δp_J (ALMEIDA, 1985).

The Joukowsky formula will be only valid for instantaneous or fast manoeuvres, when the duration of flow change T_C is less or equal to T_E , the time of one round trip of the first pressure elastic wave or the time for the elastic wave to propagate from the source of the disturbance to the penstock intake, where it will be reflected back with the same velocity, and returns to the disturbance source.

This situation can be avoided if T_C is carefully chosen for normal operation conditions in order that $T_C \gg T_E$ (slow manoeuvre). This could be easily obtained by closing or opening the turbine flow control devices (nozzle or guide vane) any safety valve or gate very slowly. For slow manoeuvres of complete flow stoppage or start up), in real and normal design situations the approximate relative head variation $\Delta H_M/H_o$ can be given by

$$\frac{\Delta H_M}{H_o} = K_T \frac{T_w}{T_C} \quad (7.9)$$

where K_T is a factor that depends on the turbine type and operation (LEIN, 1965) and T_w is the hydraulic inertia time constant defined by

$$T_w = \frac{L V_o}{g H_o} \quad (7.10)$$

where

L = pipe length (m);

V_o = initial or final flow velocity (m/s);

H_o = reference net head (m).

For Pelton turbines K_T will vary between 3.7 (closure) to 3.3 (opening) and for Francis turbines K_T will vary for both manoeuvres from 1.2 to 2.0 (being the majored K_T factor between 1.8 and 2.2). For certain theoretically conditions $K_T=2.0$ (Michaud' formula mentioned in RAMOS, 1995).

A similar equation can be applied for the calculation of minimum heads or pressures near the turbine gate. For pipe design, the transient maximum and minimum heads need to be obtained along the axis profile (see Figure 5.18). For a computer this information is easily obtained in each computational section. In most of the approximate methods, a criterion for the head envelops needs to be followed: typically, for slow manoeuvres, a straight line is adopted for each envelope, from the upstream or the downstream side of the turbine to the fixed reservoir or tailrace levels, respectively.

Based on the two extreme enveloped lines, the maximum and minimum pressures can be easily found by comparing those lines with the pipe or penstock axis profile along terrain.

For both normal and abnormal turbine operations it is necessary to be very careful with the minimum transient pressures downstream the turbine runner (in the draft tube) in order to avoid the water column separation, specially when there is a long tailrace pressurised tunnel or pipe. An approximate criterion was proposed by LEIN, 1965:

$$\frac{P_{m}}{\gamma} = Hd - \Delta H_m - \frac{V_2^2}{2g} - Z_r \quad (7.11)$$

where

Hd – downstream level (e.g. downstream river level);

ΔH_m – head variation giving the minimum transient head downstream the draft tube;
 V_2 – flow velocity at runner outlet;
 Z_r – turbine runner outlet section level;
 p_{m} – minimum transient pressure at runner outlet section;

Water column separation will be avoided if $p_m/\gamma > -6$ m w.c.. This means that a reaction type turbine runner elevation and the powerhouse elevation should be fixed according, among other factors, to the downstream pressure transient analysis.

7.3.3- Governing equations

The basic differential equations of the unsteady and transient pressure flows can be written as follows (RAMOS, 1995):

$$\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} = D(U) \tag{7.12}$$

where U, F(U) and D(U) are the following vectors:

$$U = \begin{bmatrix} H \\ Q \end{bmatrix}; \quad F(U) = \begin{bmatrix} \frac{c^2}{gA} Q \\ gAH \end{bmatrix}; \quad D(U) = \begin{bmatrix} 0 \\ -\frac{JgA}{Q^2} Q|Q| \end{bmatrix} \tag{7.13}$$

in which x = distance along the pipe axis (m); t = time (s); A = cross-section flow area (m²); Q = discharge(m³/s; H = piezometric head (m); J = slope of the energy grade line; g = gravity acceleration (m/s²); and c = elastic wave celerity (m/s).

Equations (7.12) can be solved by computer codes based on numerical techniques (e.g. method of characteristics) and on the specified boundary conditions. Nowadays, the computer modelling is based on the discretisation of the hydrosystem and the computational model will be composed by several components (tube and non-tube elements) and nodes (ALMEIDA and KOELLE, 1992).

With these kind of models it is possible to simulate different operation conditions and study the hydraulic behaviour of complex systems, including pipe networks and all major hydro-mechanical equipment. Among these ones, the turbines are the most important source of flow disturbances and their modelling is crucial for a reliable computational model as a design aid tool.

The turbine modelling, especially in what concerns the reaction turbines (e.g. Francis and Kaplan turbines), the turbine couples the upstream and downstream sides of the pressurised hydraulic circuit. In these cases, the turbine modelling implies the knowledge of the complete characteristic curves based on the manufactures' tests. These curves are very difficult to obtain for small hydroplants.

In what concerns the action turbines (e.g. Pelton turbines), the pressure transients will need to be considered along the upstream penstock. The disturbance source will be the turbine nozzles, a special type valve that can control the flow by changing the position of an internal part known by the needle. In this case, as in all pressure transients induced by a flow control device placed downstream a long pipe, it is very important to know the characteristic equation of such control device, or the controlled flow for each nozzle (or valve) opening and each head variation across the upstream and downstream sides or nodes of this specific component.

Small hydropower schemes when installed in mountainous regions are typically associated to long penstocks, in order to increase the available head, and to action or impulse turbines (e.g. Pelton turbines).

With computational models it is possible to include the real nozzle characteristics and the headloss variation along the penstock during the transient regimes. The friction headloss can be characterised by the following pipe parameter (RAMOS, 1995):

$$F_p = \frac{\Delta H}{V^2/2g} = \frac{fL}{D} \quad (7.14)$$

where ΔH is the flow total head loss; $V^2/2g$ is the kinetic head; f is Darcy-Weisbach friction factor; L the pipe or penstock length; and D the pipe or penstock diameter.

The computer simulations indicate that for high F_p values the flow control can be very difficult to obtain, because the time duration of the physical manoeuvre of the nozzle or valve differs very much of the real flow change duration T_C . In these cases, for a nozzle linear time closure, the discharge variation will be only effective near the full closed position: the duration of the nozzle mechanical manoeuvre is very different from the effective time of flow discharge variation and a theoretically slow manoeuvre ($T_C > T_E$) can be, in fact, a fast one ($T_C < T_E$) as

can be seen in Figure 7.4 (where Q_0 is the initial turbine discharge for full open nozzle or turbine maximum discharge):

- the decrease of fL/D allows the discharge variation to be more favourable;
- for large values of fL/D most of the discharge variation only occurs for small values of nozzle opening, at the end of the nozzle manoeuvre.

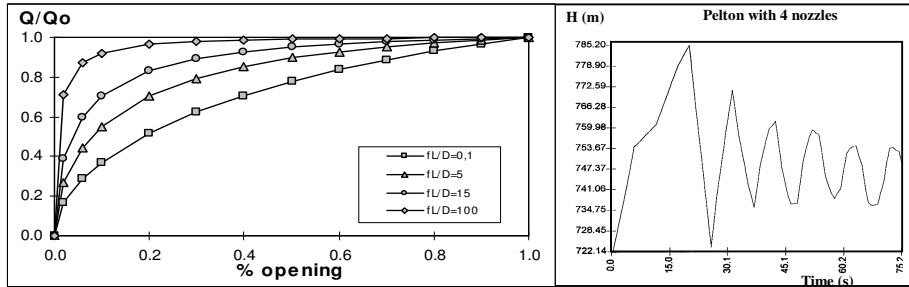


Fig. 7.4- Discharge variation for different penstock characteristics and nozzle closures. Head variation in the powerhouse for a typical manoeuvre (RAMOS, 1995).

The hydraulic conveyance circuit at upstream side of impulse turbines will be influenced by the nozzle control system of the discharge variation imposed through the closure law. This law and the hydraulic penstock characteristics will be very important on the transient pressure analysis.

It is very important to note that it is *no sufficient* to define the total manoeuvre time duration without knowing the conveying system characteristics.

7.4- Overspeed dynamic effects

7.4.1- Overspeed runner control

During normal steady-state operating conditions the speed of a turbine is constant, but it will rise quickly when the load of the turbine is rejected. During the closure of the nozzle or the turbine guide vane, the speed of the turbine will rise to a maximum value that can not surpass the allowable one fixed by the turbine manufacturer.

For impulse turbines the transients overspeed does not influence the pressure along the hydraulic circuit. With reaction turbines the runner speed will change the turbine discharge and the overspeed conditions will also influence the hydraulic circuit (RAMOS, 1995). The transient overspeed can be a waterhammer additional source, especially in what concerns the slow Francis turbines (with small N_s values). A complete computer analysis should be performed based on the turbine characteristic equations, the complete hydraulic equations (7.12) and the equation of the rotating masses of each unit:

$$BH - BR = I \frac{d\omega}{dt} \quad (7.15)$$

in which BH is the transient hydraulic torque (N m), BR is the resistant or electrical torque (N m), I is the rotating masses inertia (kg m^2) and ω is the angular rotating speed (rad/s).

An inertia machine time constant (or start-up time of rotating masses), T_m , can be defined based on equation (7.16):

$$T_m = \frac{WD^2 n_o^2}{3575 P_o} \times 10^{-3} \quad (7.16)$$

where n_o is the nominal runner speed (r.p.m.), P_o the reference power or full load turbine power (kW) and $WD^2 = 4gI$ (N m^2). T_m has the order of magnitude of the time for the unit to attain the speed n_o when submitted to a linear increasing hydraulic power from 0 to P_o .

For impulse turbines, and especially in what concerns the Pelton type turbine, the runner overspeed control can be obtained by a deflector that deviates the flow of the runner. This means that a slower closure manoeuvre can be specified in order to better control the transient pressures along the penstock. For reaction turbines the simultaneous transient pressure and overspeed control is much more difficult (RAMOS, 1995). In order to evaluate, in a preliminary analysis, what is the turbine maximum relative overspeed after a full load rejection some approximate formula based on equation (7.16) can be used (LEIN, 1965 and HADLEY, 1970):

- *Lein* formula

$$\frac{\Delta n}{n_o} = \sqrt{1 + \frac{kT_C}{T_m} \left(1 + \frac{\Delta H_C}{H_o}\right)} - 1 \quad (7.17)$$

valid for $\Delta n < 0.5 n_o$. For Pelton turbines with jet deflectors, $k=0.9$, ΔH is the upstream waterhammer or transient head variation due to the manoeuvre and T_C is the time required by the deflector to change the flow direction, including the dead time ($T_C \geq 1.5$ s). For Francis turbines, $k=0.8$ and T_C is the guide vane closure time.

- *Hadley formula*

$$\frac{\Delta n}{n_o} = \sqrt{1 + \frac{kT_C}{T_m} \left(1 + \frac{\Delta H_C}{H_o}\right)^{3/2}} - 1 \quad (7.18)$$

where k is now related to turbine characteristics as a function of the specific speed N_s , as indicated in the following table, and T_C is the guide vane time of closure:

N_s (r.p.m.) (kW, m)	k
76	0.98
134	0.96
286	0.84
381	0.77
573	0.66
672	0.61
763	0.57

Typically $\Delta n/n_o$ should not exceed 0.6. For normal runner speed governing, Hadley estimated that the following relationship between the minimum T_m and T_w (see eq. (7.10)) should be obeyed:

- Pelton turbines – $T_m = 2.5 T_w$
- Francis and Kaplan turbines – $T_m = 3.0 T_w$

The overspeed control and speed regulation stability in small hydroplants, during normal operation with load demand changes, can be improved by increasing the T_m value through the WD^2 value (flywheel effect) or by decreasing T_w value (e.g. by inserting a surge tank or by selecting a larger penstock diameter). The advanced electric speed-load regulators can now guarantee a better regulation stability criterion.

Values of WD^2 and T_C should be guaranteed by the turbine and generator manufacturers and are agreed upon at design stage. Another effective way to control the transient overspeed and pressure in hydrosystems equipped with Francis turbines is to equip the spiral case with special pressure regulators or water outlets which automatically open and discharge water (as a relief valve) into the tailwater pool when guide vanes are closed too rapidly (synchronous relief valves). If none of these procedures is enough to control the transient overspeed and pressure then another special protection device should be considered like a surge tank as a last resort.

In small hydropower plants connected to a large national electric grid, the stability of speed regulation does not present any special problem, because the grid has the capacity to stabilise any unstable behaviour.

In exceptional conditions the turbine runner can be forced to accelerate under the hydraulic power until a maximum limit steady state speed or runaway speed n_{rw} is attained in 3 or 5 seconds. This maximum speed will depend on turbine N_s and the head values and varies typically between $1.5 n_o$ (propeller turbines) and $2.0 n_o$ (Pelton turbines) to $3.5 n_o$ (Kaplan turbines) for design head H_o (see Table 7.2). For other values of H the runaway speed can be estimated as follows:

$$n_{rw}^* = n_{rw} \sqrt{\frac{H^*}{H_o}} \quad (7.19)$$

According to the BUREAU OF RECLAMATION, 1976, the runaway speed can be estimated by this empirical formula:

$$n_{rw} = 0.63 N_s^{1/5} n_o \sqrt{\frac{H^*}{H_o}} \quad (7.20)$$

where H^* is the turbine head, H_o is the net head for the best efficiency regime and n_o rotational speed.

Approximate formula must be used with care, as well as the estimation of the WD^2 or I values for turbine and generator. For a detailed analysis and final design stage, computer simulations should be made based on the real selected machine characteristics in order to be possible to analyse the complete dynamic behaviour and interaction between the system components.

7.4.2- Overspeed effects on turbine discharge

For impulse turbines (e.g. Pelton turbines) at a constant needle position, the flow is independent of the runner speed. It is usual to operate the nozzle and the deflector simultaneously (double regulation turbine) during a closure operation. The deflector moves the jet away from the runner and the nozzle closure law has the main influence in the discharge regulation and penstock flow variation. For Francis turbines with low specific speed, the flow drops with the transient overspeed a constant gate position. Conversely, for Francis turbines with high specific speed and for Kaplan turbines the transient discharge tends to increase (RAMOS, 1995).

On Table 7.2 and Figure 7.5 is shown the turbine discharge reduction for low specific speed Francis turbines.

Table 7.2 – Runaway conditions for different types of turbines

Turbine Type	N_s (m, kW)	Normal speed n_o (r.p.m.)	Runaway speed n_{RW}/n_o	Runaway discharge Q_{RW}/Q_o
Pelton	15-60	500-1500	1.8-2.0	-
Turgo	20-70	600-1000	2.0	-
Cross-flow	20-80	60-1000	1.8-2.0	-
Francis	80-300	500-1500	1.8-2.2	0.45-1.10
Kaplan	200-800	75-150	2.0-3.2	0.80-1.90

The flow across a runner, as former mentioned, is characterised by three types of velocities: absolute velocity of the water V , with the direction imposed by the guide vane blade, relative velocity W and tangential velocity C of the runner. For uniform velocity distribution assumed at inlet (section 1) and outlet (section 2) of the runner and by application of Euler's theorem it enables obtaining the equation, which relates the motor binary and the momentum moment between these two sections:

$$BH = \rho Q (r_1 V_1 \cos \alpha_1 - r_2 V_2 \cos \alpha_2) \quad (7.21)$$

being α and r are the angle and radius (see Figure 6.6).

The output power in the turbine shaft is a result of multiplication the binary by the angular speed ω .

$$P = BH \cdot \omega \quad (7.22)$$

where

BH is the hydraulic torque and P the output power.

It yields in the following equation, after some transformations (RAMOS, 1995),

$$Q = A \frac{\eta}{n} + B n \quad (7.23)$$

with

$$A = \frac{\left(\frac{60gH_o}{2\pi r_2} \right)}{\left(\frac{1}{2\pi b_o r_2 \tan \alpha_o} + \frac{1}{A_2 \tan \beta_2} \right)} \quad \text{and} \quad B = \frac{\left(\frac{2\pi r_2}{60} \right)}{\left(\frac{1}{2\pi b_o r_2 \tan \alpha_o} + \frac{1}{A_2 \tan \beta_2} \right)} \quad (7.24)$$

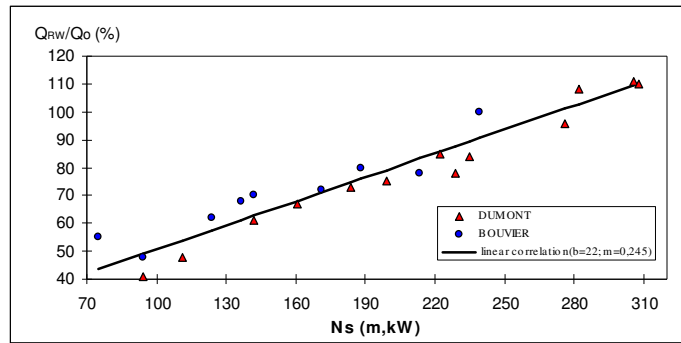
that gives a relation for turbine discharge depending upon the rotational speed value and the characteristic of the runner.

The subscript *o* denotes outlet from the wicket gate, *1* and *2* denote, respectively, inlet to and outlet from the runner, *b_o* is the runner height (or free-area), *r* is the radial distance, *α* is the angle that the velocity vector (*V*) makes with the rotational velocity (*C*), *β* is the angle that runner blades makes with the *C* direction and *A₂* is the exit flow cross-section area.

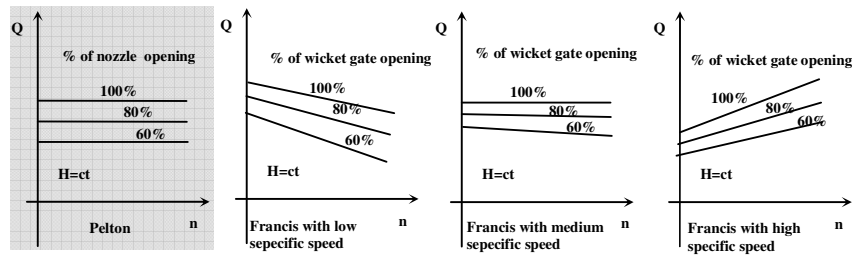
Looking to this equation, the discharge regulation can be obtained by variation of *b_o*, *α_o* or *β₂*. The *b_o* variation is not easy to obtain, because this parameter is a fixed characteristic of a runner (related to the height of the runner). Thus, for a constant rotational speed, the discharge variation can be obtained by the following procedures:

- 1 - variation of *α_o*;
- 2 - simultaneous variation of *α_o* and *β₂*;
- 3 - variation of *β₂*.

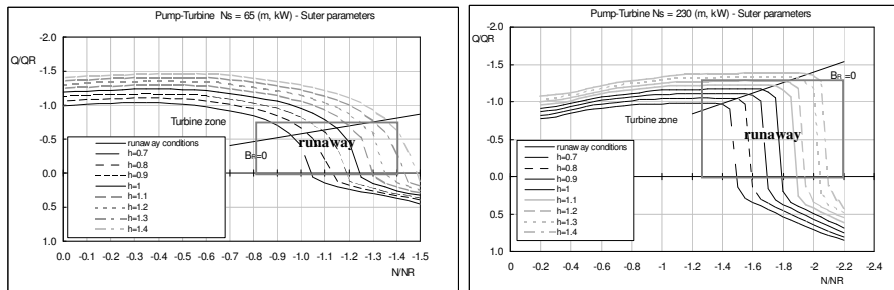
Procedure 1 is used for discharge regulation of Francis and propeller turbines through the guide vane; procedure 2 is used for Kaplan turbines and procedure 3 is used for Kaplan turbines with a fixed wicket gate.



A – Overspeed effect on discharge variation of reaction turbines (RAMOS, 1995).



B – Discharge variation with the percentage of guide vane opening.



C – Example of a low and medium specific pump-turbine speeds.

Fig. 7.5 – Turbine discharge variation analysis as a function of the runner speed.

7.4.3- Turbine overspeed effects on waterhammer

For low specific speed reaction turbines (Francis turbines) or $Q_{RW}/Q_0 < 1$ (where Q_{RW} is the turbine discharge at runaway conditions and Q_0 the nominal turbine discharge), the overspeed effect provoked by runaway conditions will potentially induce greater overpressures than those induced by just the guide vane closure effect and it can be obtained through chart of Figure 7.6 based on systematic computer simulations (RAMOS, 1995 and RAMOS; ALMEIDA, 1996).

In hydropower plants with long penstocks (or large $T_W / ((Q_{RW} / Q_0) T_m)$ values) severe waterhammer troubles can occur. As presented on Figure 7.6, the maximum upstream transient head variation will depend on N_s and two other parameters: T_W/T_m and T_C/T_E .

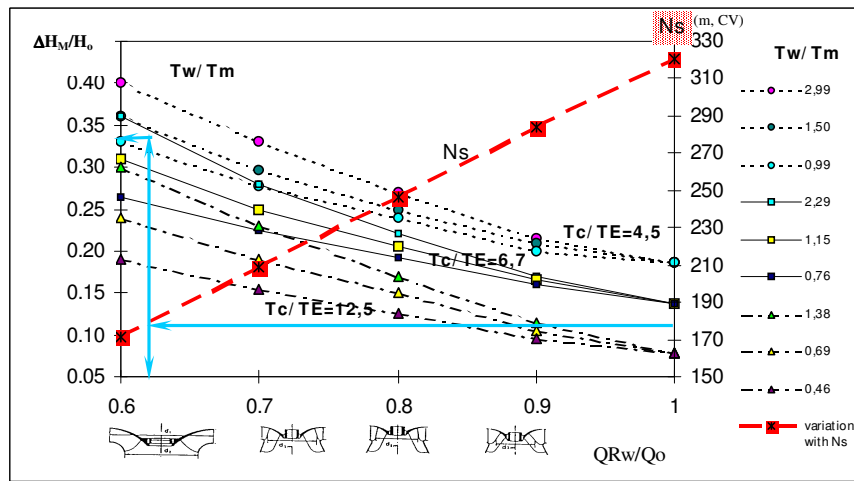


Fig. 7.6 – Maximum overpressure induced by both the overspeed and gate effects of low specific speed reaction turbines on upstream penstock (RAMOS, 1995).

As indicated in the figure (follow the example through the arrows), the calculation begins with the N_s turbine value. Moving horizontally the N_s dashed line is reached and the Q_{RW}/Q_0 can be known. Knowing the T_W/T_m value (the relative water and turbine inertia time constants) and selecting the relative wicket closure time T_C/T_E , the relative maximum upsurge variation can be obtained. For $Q_{RW} = Q_0$, the overpressure will only depend on the gate effect. These results allow an approximate prediction of the maximum overpressure due to a sudden load rejection in Francis turbines of a small powerplant.

When the powerhouse is equipped with reaction turbines, both the upstream and downstream water columns must be analysed, in order to avoid excessive overpressure at penstock and at powerhouse or even water column rupture at draft tubes. A classic solution to minimise hydraulic transients or to reduce the total length of the hydraulic conveyance system under waterhammer action consists to insert a surge tank as near the powerplant as possible. However, this solution can not be advisable, because it can be a very costly structure and can cause significant environmental impacts for a small hydropower plant. Thus, a good estimation of the maximum transient pressure without any special protection device is very important.

7.5- Special protection devices

7.5.1- Introduction

The protection devices to control transient pressures due to turbine operation can act by different ways: by supplying or removing water, or by flow energy accumulation or dissipation. Typical protection devices for small hydropower plants are among others (RAMOS, 1995):

- surge tanks;
- air vessels;
- synchronised valves, automatic discharge valves or pressure relief valves;
- flywheels.

The mathematical formulation applied in computational modelling and in the analysis of protection components of a hydropower scheme are presented. The selection of the best solution for the protection devices depends upon the hydraulic characteristics and topography that can influence the conveyance system profile of the hydraulic circuit. Also important is the simulation of the integrated system in order to know the best and correct dynamic response and the influence of different devices always kept up with an economical comparison study. Suitable protection devices to small hydropower plants are next presented based on a simplified mathematical formulation and a brief explanation of its performance.

For abnormal conditions (emergency or exceptional operating conditions) creating potential excessive overpressures during a full load rejection of very fast flow stoppage due to a safety valve closure, a pressure fusible section can be

used. This protection device is based on safety discs or membranes that will be specified to rupture whenever pressure rises above a pre-set value. A battery of discs can be selected for different pre-set pressures. The disc burst will allow an outflow as a relief valve.

7.5.2- Surge tanks

Surge tanks allow the attenuation and control of rapid variations of discharge and pressure, through storing excess energy (water volume) in an open reservoir. During a transient, a surge tank will act as a large reservoir, where the elastic pressure waves are supposed to be totally or partially reflected. Thus, the penstock length submitted to waterhammer can be reduced to the length between the surge tank and the powerplant. In this way T_E and T_W values can be drastically reduced.

The unsteady flow between the upstream intake and the surge tank will have a mass flow oscillation type, without elastic effects, as a rigid liquid column oscillation between two open reservoirs. The singular headloss at the surge tank connection side will dampen the water level oscillations.

Surge tanks can have different shapes and be of different types (Figure 7.7).

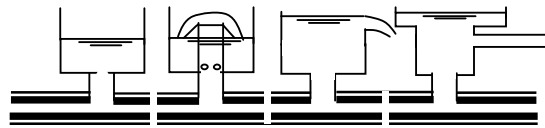


Fig. 7.7 – Different types of surge tanks.

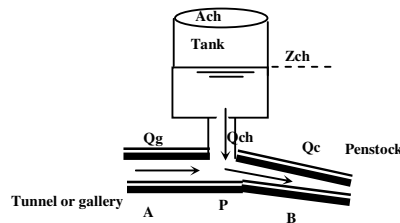


Fig. 7.8 – Typical scheme of a surge tank.

Equations used in a computational modelling of a simple surge tank (Figure 7.8) are presented as follow:

- Continuity equation at the surge tank connection section or node

$$Q_g + Q_{ch} = Q_c \quad (7.25)$$

- Kinematic equation (rigid model without considering inertia of the surge tank water)

$$A_{ch} \frac{dZ_{ch}}{dt} = -Q_{ch} \quad (7.26)$$

- Head loss equation in the surge tank connection section

$$H_{g,c} = Z_{ch} - K_{ch} Q_{ch} |Q_{ch}| \quad (7.27)$$

- Hydrodynamic characteristic equations

$$C^+: H_p - H_A + B(Q_p - Q_A) + RQ_A |Q_A| = 0 \quad (\text{gallery}) \quad (7.28)$$

$$C^-: H_p - H_B - B(Q_p - Q_B) - RQ_B |Q_B| = 0 \quad (\text{penstock}) \quad (7.29)$$

being:

B = parameter of characteristic lines of method of characteristics (MOC)

$\left(B = \frac{c}{gA} \right)$, in which c = wave celerity (m/s) and A = pipe cross-section area

(m²);

H_{g,c} and H_p = piezometric head at surge tank insertion and any conveyance system section (m);

K_{ch} = singular head loss coefficient at surge tank insertion;

Q_c = discharge in the penstock (m³/s);

Q_{ch} = discharge in the surge tank (m³/s);

Q_g = discharge in the gallery (m³/s);

Q_p = discharge at any conveyance system section (m³/s);

R = friction coefficient of the gallery and penstock ($R = J\Delta x/Q^2$);

Z_{ch} = water level in the surge tank (m);

A, B, P – left, right and device section, respectively.

7.5.3- Differential surge tank

A differential surge tank is similar to a simple one, but has a much better performance in what concerns the overpressure attenuation capacity (Figure 7.9).

It is provided by an eccentric or symmetric orifice connected to the main pipe (i.e. gallery or penstock) and an interior pipe or riser that discharges into the external or outer tank (RAMOS, 1995).

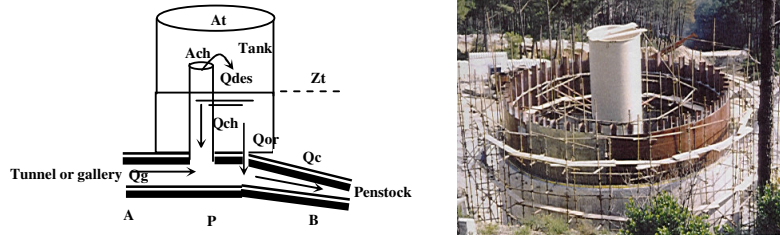


Fig. 7.9 –Differential surge tank. A scheme with notation at left and real example under construction.

The computational modelling of this element is based on the following set of equations (RAMOS, 1995):

- Continuity equation at the surge tank connection section or node

$$Q_g + Q_{ch} + Q_{or} = Q_c \quad (7.30)$$

- Continuity equation of the interior pipe or riser

$$A_{ch} \frac{dZ_{ch}}{dt} = Q_{ch} + Q_{desc} \quad (7.31)$$

- Discharge law of the weir at the top river section

$$Q_{desc} = C_{vd} L_{desc} \sqrt{2g} H_{desc}^{\frac{3}{2}} \quad (7.32)$$

- Continuity equation of the surge tank

$$A_t \frac{dZ_t}{dt} = -(Q_{or} - Q_{desc}) \quad (7.33)$$

- Head equation of the orifice

$$H_{g,c} = Z_t - K_{or} Q_{or} |Q_{or}| \quad (7.34)$$

- Head equation at the insertion node of the differential surge tank

$$H_{g,c} = Z_{ch} - \left[\frac{1}{gA_{ch}} \frac{dQ_{ch}}{dt} + J_{ch} \right] (Z_{ch} - Z_{bch}) - K_{ch} Q_{ch} |Q_{ch}| \quad (7.35)$$

- Hydrodynamic characteristic equations

$$C^+: H_p - H_A + B(Q_p - Q_A) + RQ_A |Q_A| = 0 \quad (\text{gallery}) \quad (7.36)$$

$$C^-: H_p - H_B - B(Q_p - Q_B) - RQ_B |Q_B| = 0 \quad (\text{penstock}) \quad (7.37)$$

being:

B = parameter of characteristic lines of the method of characteristics (MOC)

$$\left(B = \frac{c}{gA} \right) \text{ which } c = \text{wave celerity (m/s) and } A = \text{pipe cross-section (m}^2\text{);}$$

C_{vd} = discharge coefficient of the weir;

g = gravity acceleration (m/s^2);

$H_{g,c}$ = piezometric head at the surge tank connection section (m);

H_p = piezometric head at any conveyance system section (m);

J_{ch} = hydraulic grade line of interior pipe of differential surge tank;

K_{or} = singular head loss coefficient in the orifice;

K_{ch} = singular head loss coefficient at the tank connection section;

L_{desc} = perimeter of the riser pipe of the differential surge tank (it works as a weir) (m);

Q_c = discharge in the penstock (m^3/s);

Q_{ch} = discharge in the interior pipe (m^3/s);

Q_{desc} = discharge over the weir (m^3/s);

Q_g = gallery discharge (m^3/s);

Q_{or} = discharge through the orifice (m^3/s);

Q_p = discharge at any conveyance system section (m^3/s);

R = friction coefficient of the gallery and penstock ($R = J\Delta x/Q^2$);

Z_{ch} = water level in the riser pipe (m);

Z_t = water level in the outer tank (m);

A, B, P – left, right and device section.

7.5.4- Air vessel

An air vessel has a similar function to a surge tank, but it is closed and a smaller reservoir, with entrapped air inside avoiding, thus, too large reservoir dimensions due to the air compressibility absorption (Figure 7.10).

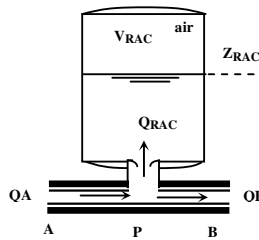


Fig. 7.10 – Air vessel scheme.

The volume of air due to its compressibility effect will contribute for the overpressure attenuation. An energy dissipater can be inserted in the connecting pipe in order to better control the maximum overpressures and to dampen their oscillations.

The computational modelling of this element can be based on the following set of equations:

- Continuity equation at the air vessel connection section or node

$$Q_A = Q_{RAC} + Q_B \quad (7.38)$$

- Head equation at the air vessel connection section

$$H_{RAC} = H_A + H_a - Z_{RAC} - K_{RAC} Q_{RAC} |Q_{RAC}| \quad (7.39)$$

- Polytropic equation for perfect gas behaviour

$$H_{RAC} \forall_{RAC}^n = C \quad (7.40)$$

- Continuity equation inside the air vessel

$$A_{RAC} \frac{dZ_{rac}}{dt} = Q_{RAC} \quad (7.41)$$

- Hydrodynamic characteristic equations

$$C^+: H_p - H_A + B(Q_p - Q_A) + RQ_A |Q_A| = 0 \quad (\text{gallery}) \quad (7.42)$$

$$C^-: H_p - H_B - B(Q_p - Q_B) - RQ_B |Q_B| = 0 \quad (\text{penstock}) \quad (7.43)$$

being:

A_{RAC} = cross-section area of the air vessel RAC (m^2);

B = parameter of characteristic lines of the method of characteristics (MOC)

$\left(B = \frac{c}{gA} \right)$ which c = wave celerity (m/s) and A = pipe cross-section (m^2);

H_a = atmospheric pressure (= 10.33 m);

H_{RAC} = piezometric head at the air vessel insertion (m);

H_p = piezometric head at any conveyance system section (m);

K_{RAC} = inlet and outlet singular head loss coefficient of the air vessel (RAC);

n = polytropic coefficient for the gas inside the air vessel;

Q_p = discharge at any conveyance system section (m^3/s);

Q_{RAC} = discharge in the interior of air vessel (m^3/s);

R = friction coefficient of the penstock ($R = J\Delta x/Q^2$);

Z_{RAC} = water level in the air vessel (m);

A^{KVC} = air volume inside the air vessel RAC (m^3);
 A, B, P – left, right and device section.

7.5.5- Synchronised valve or relief valve

These valves allow the exit of the flow, within a control discharge law and, consequently, the attenuation of maximum upsurges by increasing the effective flow stoppage time (Figure 7.11).

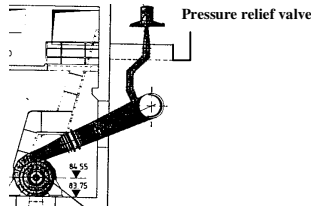


Fig. 7.11 –Typical installation of a pressure relief valve upstream a turbine.

These valves will be placed near each unit of the powerplant and will automatically open to the atmosphere or outlet pipe if a fast closure of the wicket gate occurs. The relief valve will open during the turbine fast gate operation (synchronous valve – RAMOS, 1995). After the end of the wicket gate closure, the relief valve will slowly close. Other types of relief valves will open as soon as the penstock pressure exceeds a limit value. A relief valve is an adequate device for high heads (typically $H_o > 50$ m).

The computational modelling of this element is based on the following set of equations:

- Turbine characteristic equations

- Characteristic equation of the turbine and valve (when the valve is coupled to the turbine)

$$Q_v = (C_{valv})\sqrt{H_o} \quad (7.44)$$

- Hydrodynamic characteristic equation C^+ of the penstock

$$H_p = C_1 - BQ \quad (7.45)$$

and a tailrace at constant water level Z_J :

$$H_p = H_o + Z_J \quad (7.46)$$

where

B = parameter of characteristic lines of the method of characteristics (MOC)

$$\left(B = \frac{c}{gA} \right) \text{ which } c = \text{wave celerity (m/s) and } A = \text{pipe cross-section area (m}^2\text{);}$$

C_1 = parameter of characteristic lines (MOC) ($C_1 = H_A + (B - R|Q_A|)Q_A$);

C_{valv} = valve discharge coefficient that will change as a function of the valve closure or opening laws (-);

H_o = net valve head (m);

H_p = piezometric head at any conveyance system section (m);

Q_p = discharge at any conveyance system section (m³/s);

Q_v = valve discharge (m³/s);

R = friction coefficient of the hydraulic circuit ($R = J\Delta x/Q^2$).

7.5.6- Flywheel

The flywheel, whose function is to accumulate energy by increasing the rotating mass energy of turbo-generators, allows the increase of the stoppage time T_C of the units as well as the time to attain the runaway speed (Figure 7.12).

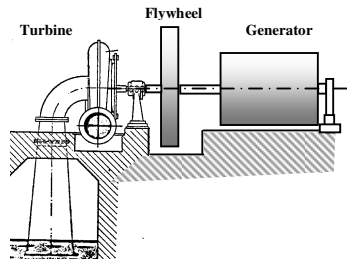


Fig. 7.12 – Typical turbine flywheel installation.

With this device, the turbine runner overspeed and the transient pressure will be both controlled.

The computational modelling of this element is based on the following equation that should be solved with the turbine characteristics and the hydraulic equations:

-Rotating mass equation

$$B_H - B_R = I \frac{2\pi}{60} \frac{dn}{dt} \quad (7.47)$$

in which

$$I = I_{\text{tur}} + I_{\text{ge}} + I_{\text{fw}} \quad (7.48)$$

where

B_H = hydraulic torque (N m);

B_R = resistive electromagnetic torque (N m);

I = total rotating mass inertia (kg m^2);

I_{tur} = rotating mass inertia of the turbine (kg m^2);

I_{ge} = rotating mass inertia of the generator (kg m^2);

I_{fw} = rotating mass inertia of the flywheel (kg m^2);

n = rotating speed of the unit (r.p.m.).

It can be concluded that the flywheel computational analysis just implies an increase of the total rotating mass inertia, I , of each unit in the model. With this increase T_m will also be increased, as well as the starting up time and the regulation stability conditions will be improved, especially in what concerns the isolated grid operation. On the other hand the increase in T_m will diminish the transient maximum runner overspeed.

7.5.7- Protection devices behaviour

7.5.7.1- Analysis of a surge tank

Generally, surge tanks are installed at downstream of a diversion gallery (or tunnel) or at upstream of a long tailrace tunnel, in order to reduce the total length of the hydraulic conveyance circuit under waterhammer effects (due to elastic waves) and to improve the turbine speed regulation stability should the plant be connected to an isolated electric grid.

In a surge tank analysis (Figure 7.13), it is usual to consider the mass oscillation (or rigid water column) regime between the upstream reservoir and the surge tank or between the surge tank and the tailrace when the surge tank is placed downstream the powerplant (RAMOS, 1995).

In some cases, due to construction and economic constraints, protection devices are not directly connected to the gallery or pipe and, sometimes, it is necessary to have a long connection pipe. In practice, at small power plants this pipe does not have a significant influence, since the pipe length is not too much long. However

a surge tank may be not able to reflect the most severe incident elastic waves coming from the penstock. In this situation a transmitted wave with reduced intensity will propagate along the protected tunnel or gallery. To avoid this wave transmission the surge tank connection should not have a very strong restricted linking pipe.

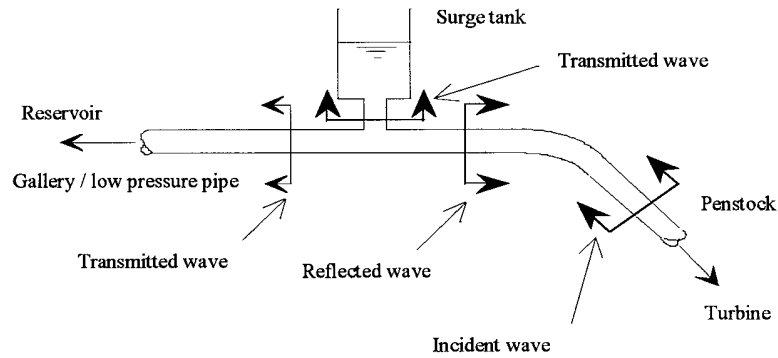


Fig. 7.13. – Surge tank effect on the reflection and transmission of the pressure elastic waves induced by turbine discharge variation.

Some sensitivity analysis carried out concerning the gallery and the penstock cross sections allow concluding that the main influence is obviously imposed by the gallery cross section area that as smaller as greater is the water level attained inside the surge tank (RAMOS, 1995).

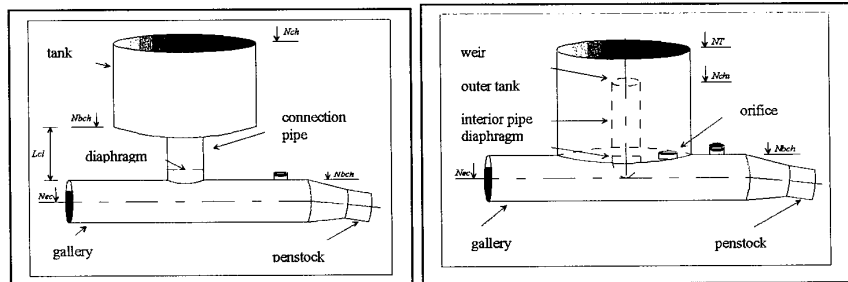


Fig. 7.14 – Two typical surge tank schemes: simple cylindrical tank with a restricted connection and a differential tank.

Based on the main oscillation theory, analytical solutions can be obtained for the calculation of the extreme surge tank water levels for simplified conditions

as, by example, for simple (cylindrical) tanks and the complete and instantaneous flow stoppage at the powerplant. Neglecting the friction and singular head losses, the maximum water level oscillation ΔZ^* in the surge tank will be:

$$\Delta Z^* = Q_o \sqrt{\frac{L}{gA A_{ch}}} = V_o \sqrt{\frac{LA}{g A_{ch}}} \quad (7.49)$$

where

Q_o = initial gallery or tunnel discharge (m³/s);
 V_o = initial flow velocity corresponding to Q_o (m/s);
 L = gallery length (m);
 A = gallery cross section area (m²);
 A_{ch} = surge tank cross section area (m²);
 g = gravity acceleration (m/s²).

The water level period of oscillations T^* , in these simplified condition, may be obtained by

$$T^* = 2\pi \sqrt{\frac{LA_{ch}}{g A}} \quad (7.50)$$

It can be easily concluded that this period is much longer than the pressure elastic fluctuations ($4L/c$). The water level oscillations will be damped by the friction and singular headlosses along the gallery. The initial headlosses will also modify the maximum water level fluctuation ΔZ^* . This influence can be evaluated by the following approximate formula deduced by Jaeger:

$$\Delta Z_1 = \Delta Z^* - \frac{2}{3} \Delta H_o + \frac{1}{9} \frac{\Delta H_o^2}{\Delta Z^*} \quad (7.51)$$

where

ΔZ_1 - deduced water level oscillation;
 ΔZ^* - theoretical water level variation without headloss effect;
 ΔH_o - gallery total headlosses for the initial discharge Q_o .

Another water level oscillation reduction can still be considered as a function of the time of the turbine discharge change or time of gate closure (e.g. for $T_C = 0.4 T^*$ and $\Delta Z \approx 0.75 \Delta Z^*$).

A longer closure time for the wicket turbine can be feasible for Francis turbines if a synchronous relief valve is installed. For Pelton turbines the deflector will deviate the jet and the transient overspeed will be controlled.

In order to reduce the water level variation in the surge tank some special energy dissipative devices or flow restrictions are sometimes placed in connecting pipe (e.g. an orifice or diaphragm). However, a strong flow restriction will increase the transmission of the waterhammer waves for the gallery.

One of the classic problems to be considered in a surge tank design and specification is the stability of the water level oscillations, should the hydroplant be under the action of an automatic regulator with feedback imposing a turbine discharge variation through the turbine gate position, as a function of the continuous variation of the load power demand and the net head. This is a typical situation found in an isolated electric grid.

The surge tank stability analysis depends on the overall hydro and electric (demand) system behaviour. An approximate analysis can be based on very simplified conditions and small disturbances and a linear method of analysis of the set of equations. A minimum cross section area, or Thoma section, is theoretically obtained for simple cylindrical tanks in order to guarantee the stability of the water level oscillations:

$$A_{th} = \frac{L A}{H_o} \frac{V^2}{\Delta H_o 2g} \quad (7.52)$$

where

A_{th} = Thoma cross section area;

A = gallery cross section area;

L = gallery length;

V = gallery flow velocity;

H_o = net head;

ΔH_o = gallery headloss.

This limit area can impose a very costly surge chamber. Formula (7.52) can be improved with minimum cross section areas reduced to 50 - 60%, should the dimensions and cost, as well as the powerplant importance, justify it by taking into consideration several additional factors (GARDEL, 1956).

Computer simulations with the complete non-linear equations will allow to consider the influence of the entire main factors, as well the large water level oscillations, in the stability and operational analysis.

There are a lot of types of special surge tanks. Each one has some advantage in what concerns the hydraulic response to transient regimes, the local technical constraints (e.g. geological and topographical), the environmental impacts or the construction costs. Most of the types of surge tanks are not adequate to small hydropower systems because they are too complex and costly.

One exception is the differential surge tank (e.g. the Johnson type) with an internal pipe or riser and an outer tank. When the flow reaches the top of the internal pipe, after a turbine discharge stoppage, it will spill into the outer tank. The rapid head increase in the riser will create a faster deceleration head applied to the gallery flow. In order to prevent excessive surge amplitude, the height of the riser is limited and the outer tank will store the spilled water.

A load demand increase will induce a rapid drop in the riser with a forcing acceleration effect upon the gallery flow.

The water required to supply the increased turbine demand is drawn from the storage outer tank through one or more openings.

The savings in volume of this type of surge tank over that of a single chamber with a restricted - orifice can be obtained if the critical turbine flow demand is smaller than the critical rejected flow (e.g. when the powerplant has multiple units). With a differential surge tank including an eccentric extra orifice (see Figure 7.14), the elastic wave transmission (S) can considerably be reduced. Relatively to the Johnson type surge tank (Figure 7.15), especially in what concerns the wave transmission a reduction of $\Delta S = 5\%$ can be obtained depending on the relative dimensions of the orifice diameter (D_{orif}) and of the tank diameter (D_{tank}).

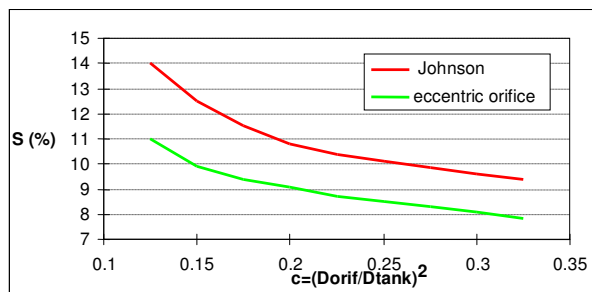


Fig. 7.15 – Example of comparison between a Johnson surge tank and eccentric orifice surge tank (RAMOS, 1995).

Some conclusions can be presented according to this analysis:

- 1 - the differential surge tank with eccentric orifice seems to be more efficient in wave transmission;
- 2 - the internal tank cross-section must be carefully studied because it can reduce or induce variations in the gallery wave transmission factor;
- 3 - in general, there is an increasing in the gallery wave transmission when the outer tank cross-section is reduced;
- 5 - both surge tank types have less environmental impact than a normal surge tank due to have smaller global volumes.

More information about methods of analysis and the behaviour of different types of surge tanks can be found in JAEGER, 1977 and MOSONYI, 1991.

7.5.7.2- Analysis of an air vessel

The use of air vessels or air-cushion surge chambers is well known as an efficient protection device in pumping systems. Air vessel has also been used in large high head hydroelectric schemes through rock caverns (e.g. in Norway). In small hydroplants this device is not frequently selected. However, the air vessel can theoretically replace an open surge tank with much less volume (Figure 7.16).

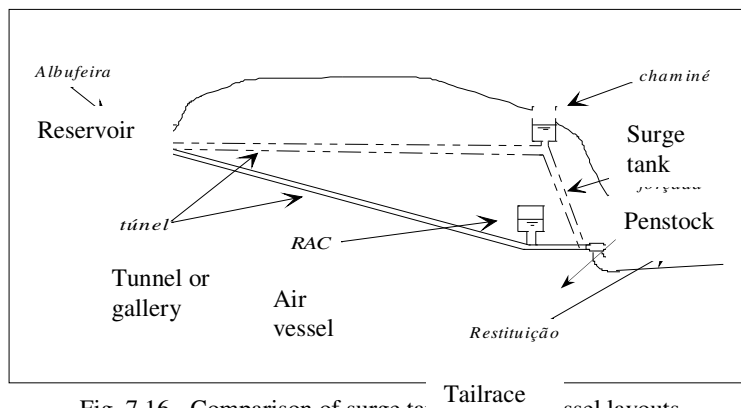


Fig. 7.16 - Comparison of surge tank and air vessel layouts.

The range of water level oscillations depends on selection of the air-filled volume of the vessel or chamber, as well as on the singular headlosses created at the connecting pipe.

Several simulations were made for a specific system to enable the analysis of an air vessel behaviour based on the analysis of the singular head loss coefficient at the inlet (K_e) and outlet (K_s) of the vessel, and its influence in the surge attenuation (Fig. 7.17). Sensitivity analyses were also developed, in what concerns the air volume, the coefficients $K_{s,e}$ and the gallery length variations (Fig. 7.18). Longer circuits are unfavourable for the overpressures, as well as it is known, and K_s has more influence on minimum pressure values and K_e on the maximum overpressure values. It is necessary to have some care with these values because they can also increase the overpressures when the constraining is too high.

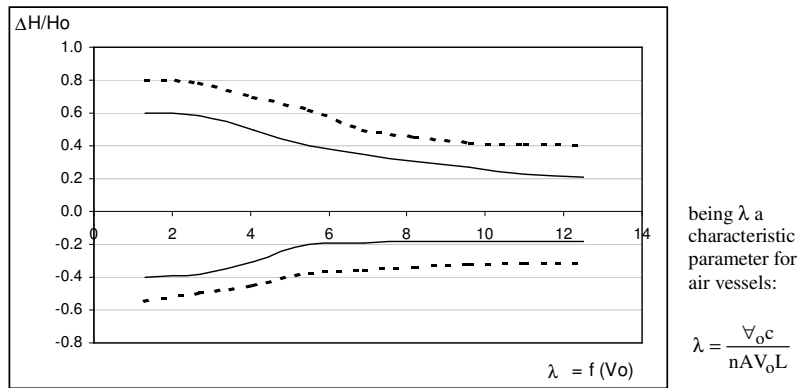


Fig. 7.17 –Extreme variation of the piezometric head by increasing the air volume inside the air vessel (RAMOS, 1995)

The air vessel dimensions or volume should also satisfy the stability constraints similar to open surge tanks. A modified Thoma critical section A_{Tha} was presented by Mosonyi for preliminary studies (Svee formulae):

$$A_{Tha} = A_{Th} \left(1 + n \frac{P_o}{\gamma h_o} \right) \quad (7.53)$$

where

n - polytropic coefficient for air thermal process;

p_o - reference air pressure inside the vessel; h_o - air height inside the vessel;
 γ - water volumetric weight.

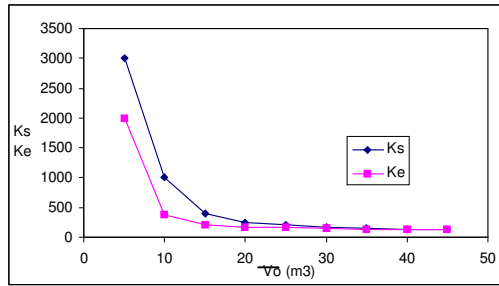


Fig. 7.18 – Optimisation of the head loss coefficient value as a function of air vessel volume

It is always important to determine the maximum downsurge in consequence of a sudden shut-down of the electric load demand in a standstill period.

7.6- Examples

In this section a graphically transient comparative analysis will be presented, for a total pressurised penstock. Different alternative types of protection devices are considered. The system dynamic response varies with the hydraulic and equipment characteristics.

Figure 7.19 shows the more or less capability of each protection device to attenuate the pressure variation along the conveyance system, and the time duration to dampen the transient regimes and to establish again the operational capacity system. Only through a technical and economic comparative analysis will be possible to select the best solution for each case.

A sensitivity analysis was carried out in order to obtain the dynamic response of a canal connected to a forebay, during the fill up followed by start-up and stoppage of the turbo-generator units and manoeuvres in valves (Figure 7.20). The knowledge of the dynamic behaviour of the complete integrated system is fundamental to establish criteria to specify the walls height, the dimensions of the canal, forebay and weir, the need of a PID type (or equivalent) regulator, the characteristic time constants and automation parameters.

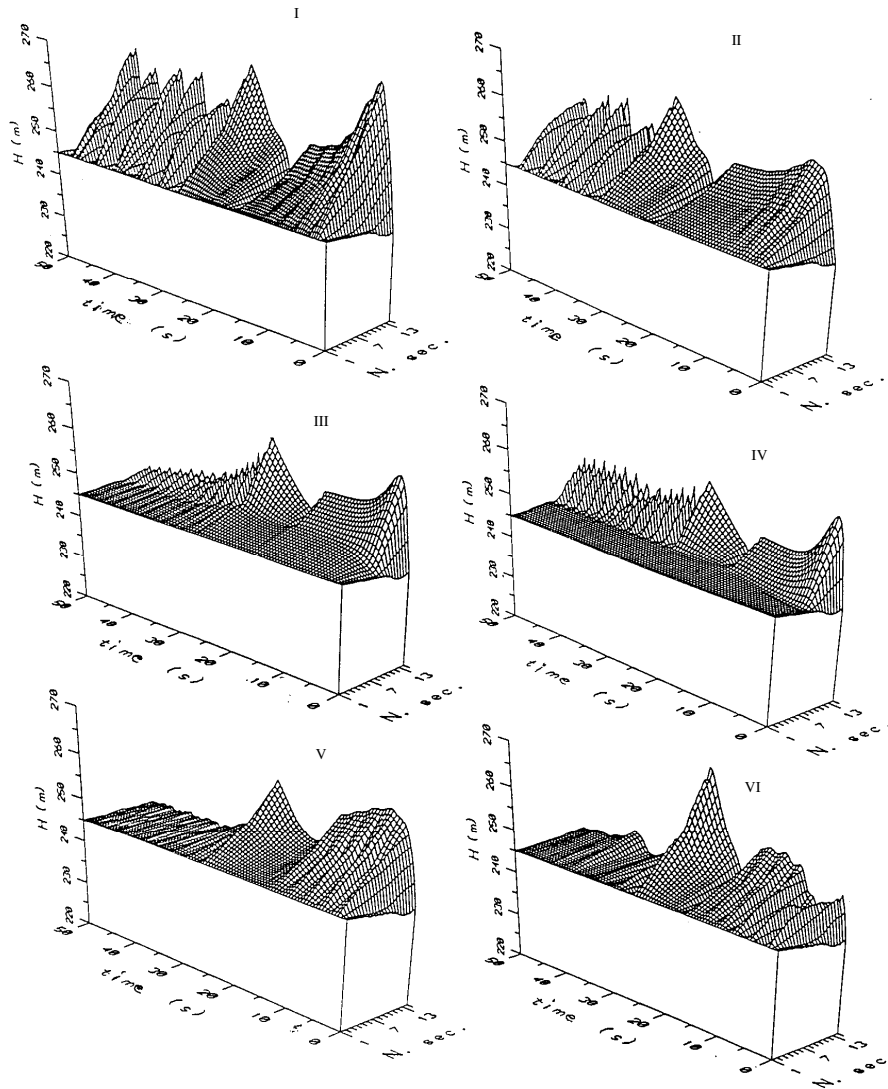


Fig. 7.19 - Sensitivity analysis regarding different protection devices for overpressure control due to overspeed effect induced by a Francis turbine: I) without protection device; II) with flywheel; III) with elastic pipe and expansion chamber; IV) with differential surge tank; V) with relief valve by pressure control; VI) synchronised relief valve by overspeed effect (RAMOS, 1995).

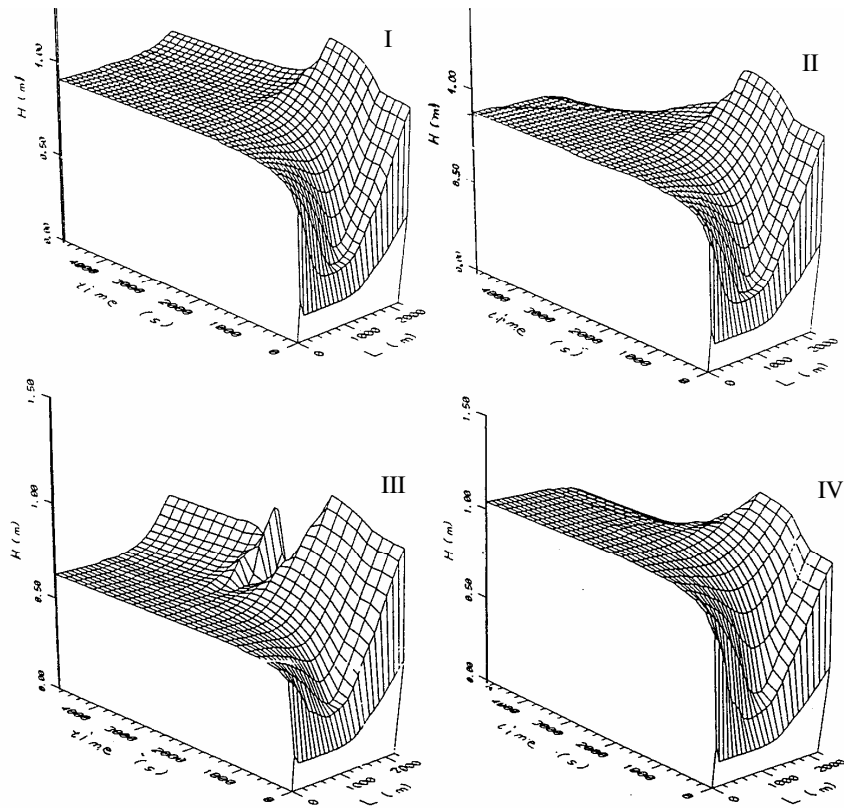


Fig. 7.20 - Dynamic behaviour induced by the filling of a canal through the opening of an upstream gate (with a forebay at downstream): I) without regulation and the turbine discharge equal to canal discharge; II) without regulation and the turbine discharge greater than canal discharge inducing the emptiness of the canal; III) with the actuation of a PID regulator to control downstream level at 0.90 m although the turbine discharge is greater than canal discharge; IV) with a PID regulator to control the water level in 0.90 m although turbine discharge is smaller than canal discharge (RAMOS, 1995).

7.7- Other protection devices and procedures

In small hydroplants with long hydraulic conveyance systems upstream or downstream the turbines, or in both sides, the most feasible, reliable and economic solution should be selected and specified.

For normal operations the maximum and minimum transient pressures along the tunnels and penstocks should be compatible with specific safety factor in order to avoid any pipe bursting, buckling effect or cavitation or vacuum formation phenomena.

The designer should always consider the most adverse conditions. Some of these situations correspond to the maximum turbine flow, others correspond to smaller flow velocities as, by example, the full closing of the turbine gate with speed no load condition: the minimum flow rate for nominal speed can be nullified by a fast manoeuvre and the overpressure will be the one given by Joukowski formula.

To guarantee the global safety condition several alternative solutions are typically considered during the design phases. The most common solution involves the specification of the adequate laws of closure and opening of the turbine flow control devices (e.g nozzles and guide vanes (or wicket gates)) and of any safety valve (e.g. pressure relief valves).

The filling and emptying of pressurised pipes need also to be considered, including the position of bottom (or purge) valves and strategically placed air vents for air inlet and outlet.

In each selected case the extreme transient head envelopes along the hydraulic circuit should be obtained and compared with the profile of the pipe system.

Abnormal conditions should also be envisaged according to the hydrosystem characteristics, including the malfunction of any vital device or an operational failure due to human error.

A large number of these situations involve the operation of safety valves as

- the fast closing of the safety valve placed just upstream a turbine should the normal flow control fail and the turbine runner velocity increase;
- the fast closing of a safety valve due to an abnormal flow velocity increase due to a pipe burst.

In these cases, a convenient closure law needs to be specified in order to control the transient maximum and minimum pressures. This specification will mainly depend on the following factors:

- the characteristics of the pipe system to be protected, especially the pipe factor $F_p = fL/D$ as was already explained; in fact, the relative pipe headloss can adversely modify the system behaviour and the same valve closure time can induce a slow or a rapid flow change according to F_p value;
- the intrinsic characteristics of the valve; a butterfly valve (for medium heads) and a spherical valve (for high heads) have different effects on the flow for the same closure law;
- the type of valve actuator.

Most of the fast closing hydroplant safety valves close under an external arm action with a weight that is liberated by a mechanism as soon as the flow velocity at a nearby pipe section exceeds a pre-fixed value. Under the weight action the valve will tend to close very fast. A hydraulic damper will slow the movement in the closing final phase. A valve closure law with two speeds is obtained that can be considered as a bi-linear closure law with two fundamental time parameters: the total time T_F and the intermediate time, where the closing gradient changes, T_I .

For these types of valves this bi-linear closure law is also very adequate and easy to model should the intrinsic valve characteristics or the valve discharge equation as a function of the valve relative opening be known.

7.8- Integrated analysis and design

The integrated system response with the different components depends on the disturbance type or excitation induced (Figure 7.21). The interaction of the different components can induce potential accidents and, at limit, resonance phenomena. As mentioned before, the type of turbines will strongly influence the system response.

The hydraulic conveyance system behaviour will depend on several components and interactions. In each case, especially in what concerns a small hydroplant the design needs to select the most important disturbance sources to be considered. A small powerplant connected to the national grid can justify not to consider most of the stability criteria applied to isolated electric grids. In all cases the integrated analysis should not forget some special situations that can be modelled with less components, as by example:

- penstock emptying or drainage by bottom valves and selection of air inlet or outlet valves if it is considered necessary;
- stability analysis of pipe anchors based on the extreme hydrodynamic forces obtained from the transient analysis and the extreme pressure envelope along the pressurised penstock.

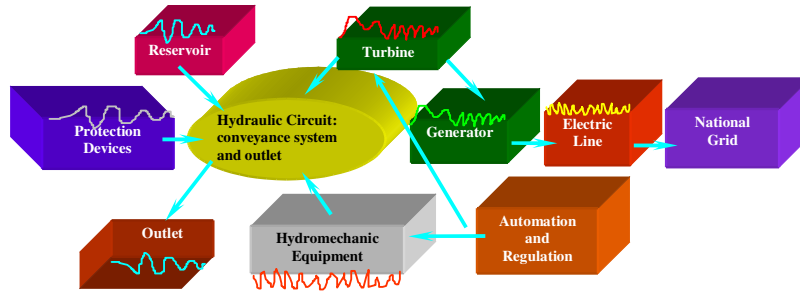


Fig. 7.21 - Oscillatory characteristics of different components of an hydropower scheme.

The control of hydrotransients and the dynamic behaviour of hydroelectric systems are fundamental for the design and the exploitation, in order to guarantee a safe and reliable solution. The effectiveness of new design criteria based upon computational techniques should allow the analysis of the global behaviour in order to identify eventual operational constraints, since the beginning of the exploitation, as well as particular local and partial situations that are vital for the system safety.

7.9- Case studies

In the Table 7.3 are presented the main characteristics of eight small hydropower systems, in the North of Portugal. All schemes have long hydraulic conveyance systems. One of them has a differential surge tank and two are equipped with relief valves. The schemes equipped with Pelton have no special protection devices against transient pressures. Two schemes with Francis turbines have not also any special protection devices. The S. Pedro do Sul powerplant is a very special case with $T_W = 8.5$ s and $Q_{Rw}/Q_o = 0.9$. In the Torga scheme, field tests (load rejection) were performed. The Francis turbines

have $N_s = 130$ (m, kW) and estimated $Q_{Rw}/Q_o = 0.6$. Comparing with S. Pedro do Sul this scheme has smaller T_w value but greater discharge reduction by overspeed (runaway condition). In fact, the overpressure at S. Pedro do Sul is due to the wicket gate (or guide vane) closure and at Torga is due to transient turbine overspeed.

Table 7.3 – Case studies. Comparison between eight small hydropower plants with different characteristics

Identification	L (m)	Hg (m)	Q _o (m ³ /s)	P _o (MW)	Turbines	Prot. device	T _w
Ermida	3900	395	2.35	7.7	Pelton (1)	None	2.5
Ovadas	3025	334	2.15	5.9	Pelton (1)	None	2.5
Sordo	3600	321	3.6	9.8	Pelton (2)	None	2.6
Torga	1200	60	20	9.1	Francis (2)	None	3.2
Nunes	2500	107	12	9.9	Francis (2)	Relief valve	3.2
V. Viçosa	1935	123	3.59	3.6	Francis (2)	Relief valve	4.3
S. Pedro Sul	4000	74	14	8.2	Francis (2)	None	8.5
Terragido	1575	125	10	10	Francis (3)	Dif. sur. tank	2.0

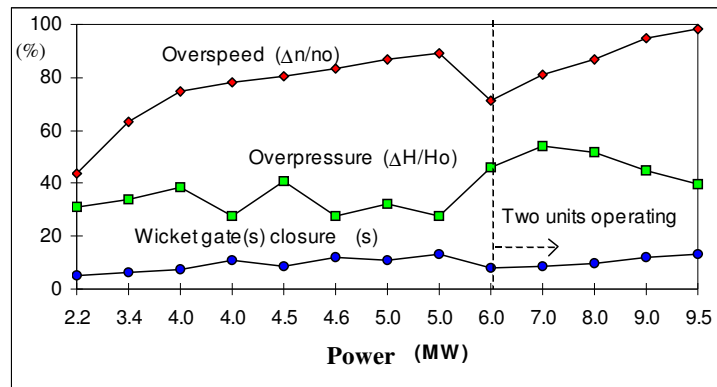


Fig. 7.22 – Full load rejection of Torga units. Field tests of overpressures, overspeeds and wicket gate(s) time closures (RAMOS, 1995).

8

ELECTRICAL EQUIPMENT H. Pires de Almeida

8.1- Generators

8.1.1. Synchronous generators

The generator is a rotating machine, with its shaft coupled to the turbine, providing the conversion from mechanical to electrical power.

There are two main types of generators used for this purpose, the synchronous and the asynchronous.

The synchronous generators have alternating current in the three-phase armature windings (normally in the stator) and direct current in the field winding (normally in the rotor). With the rotor at synchronous speed, its direct current field flux establishes a rotating field on the stator travelling at equivalent speed, hence at the nominal system frequency (50 Hz).

Its speed and the number of poles define the generator operating frequency.

$$n = f/p \quad (8.1)$$

with

n - speed in revolutions per second;

f - frequency in hertz;

p - number of pole pairs.

The equivalent circuit shown in Figure 8.1 represents these generators. The airgap flux and resulting voltage V_0 depend on the field current. If an automatic voltage regulator (AVR) which establishes the voltage under no-load conditions and the power factor when the generator is connected to the grid normally controls this current.

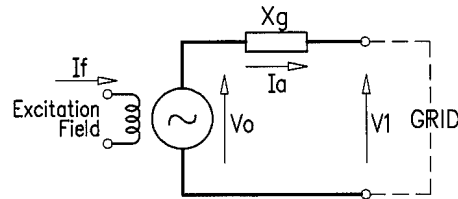


Fig. 8.1 – Synchronous generator equivalent circuit.

In approximate terms, the following expression shows the influence of the field current:

$$V_0 = d\phi / dt = \omega \phi = 2 \pi f L I_f \quad (8.2)$$

with

L - field inductance;

ω – generator angular velocity.

The generator is started-up unexcited and driven by the turbine from standstill to nominal speed. The field excitation is applied at around 95% nominal speed and V_0 and V_1 are then controlled by the AVR, through the I_f value.

Controlling speed and phase angle via the turbine wicket gate control and with the AVR providing fine voltage adjustments does the manoeuvre of synchronisation with the grid. The machine is synchronised by closing of its circuit breaker when the generator and grid voltages are in phase and with the same magnitude.

Once the generator is connected to the grid the system voltage imposes a reference to its behaviour, which can be interpreted from the P/Q diagram shown in Figure 8.2.

The voltages V_0 and V_1 are typically separated by an angle δ , called the load angle. The angle ϕ shows the phase difference between voltage and current. The $\cos \phi$ is normally referred to as power factor. The complex power components are:

Active power - $P = V_1 I_a \cos \phi$

Reactive power - $Q = V_1 I_a \sin \phi$

The load angle is an indicator of generator stability with respect to the grid. The active power supplied to the grid is given by $P = V_1 V_o \sin \delta / X_g$, reaching the steady state stability limit for $\delta = 90^\circ$ (Figure 8.2). Further increases in load angle only take place under transient stability conditions. In such cases, a suitable response of the voltage regulators is necessary to avoid pole slipping.

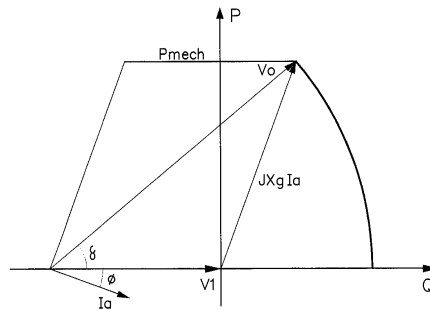


Fig. 8.2 – P/Q Diagram.

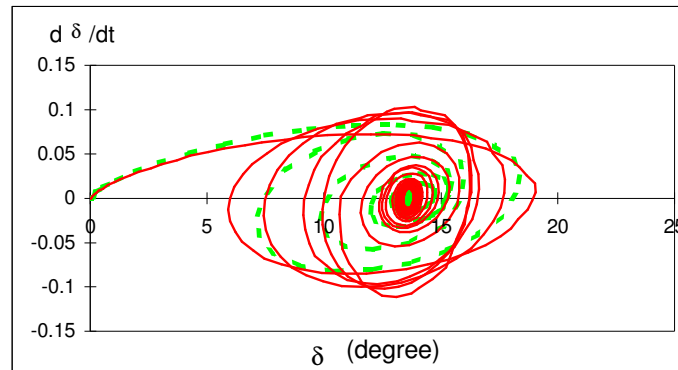


Fig. 8.3 – Typical response of a transitory stability of a generator. Linear variation of the turbine guide vane opening degree from parallel situation (RAMOS, 1995).

Increases in load angle are due to accelerating power, $P_a = P_{mec} - P_{el} = M d\omega/dt$, where P_{mec} is P, turbine power, P_{el} is generator power, with energy transfers between the electrical power and the machine rotational kinetic energy ($E_R = 1/2 I \omega^2$), where I is the polar inertia moment.

The AVR fast response provides enhanced transient stability by increasing I_f and V_o . This allows for a transient increase in output power, reducing the load angle and allowing the machine to re-gain stability (see Figure 8.2).

Figure 8.3 shows that transitory generator stability is assured for a final δ inferior than 45° . In the same figure is visible the phase plan for an increasing of opening degree of the guide vane (25% to 30%) in different time increments (dashed line in 2 s and fill line in 10 s).

Under isolated grid operation V_0 and V_1 set up the output voltage and a precise wicket gate control ensures appropriate speed control and approximately constant frequency. In these operating conditions, the generating units are provided with flywheels with sufficient polar inertia and kinetic energy to absorb and provide transient power fluctuations due to load variation, compensating for the wicket gate response times.

8.1.2 Asynchronous generators

These machines are also called induction generators, being magnetised from the three-phase winding in the stator and inducing slip currents on the rotor. The rotor revolves at a speed slightly above synchronism and the currents induced on it are of a frequency corresponding to the speed difference.

$$\text{Slip} = (\text{synchronous speed} - \text{rotating speed}) / \text{synchronous speed}$$

Torque results from the magnetic interaction between stator and rotor fluxes and varies with slip, as indicated on the torque / slip curve of Figure 8.4.

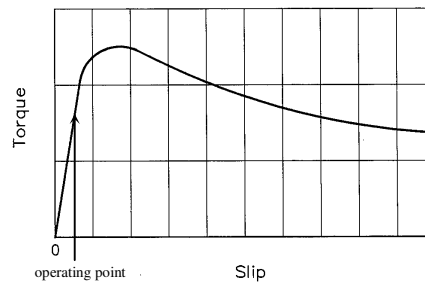


Fig. 8.4 – Torque / Slip curve

These generators can only supply active power to the grid and must import reactive power for their magnetisation.

The procedure for grid connection requires running up driven by the turbine. The generator breaker is closed when the actual speed passes over the synchronous speed, at very reduced acceleration, in order to limit circulating currents when closing. On the grid connection the generator absorbs a transient magnetising

current of short duration. The turbine power is then increased at a suitable rate by the wicket gate control, while the slip and stator current increase according to the generator characteristics.

The asynchronous generator control and handling towards the grid is simpler than the synchronous but its application in small power plants (or mini-hydro) schemes is normally limited to 2 MW machines due to transient effects of grid connection and the need to install capacitors for power factor correction.

8.2. Electrical installations

8.2.1. Main transformer

The typical single line diagram of a mini-hydro power station is shown in Figure 8.5.

The transformer is a static unit with the purpose of stepping-up the generation voltage to the grid connection voltage level.

The voltage transformation is achieved by having the suitable number of turns in the primary and secondary windings. The voltage ratio is defined by

$$N1/N2=V1/V2= I2/I1 \quad (8.3)$$

with

N1 and N2 - primary and secondary winding number of turns.

V1 and V2 - primary and secondary voltages.

I1 and I2 - primary and secondary currents (inverse ratio).

The transformers are normally of the immersed oil type, but dry types/resin impregnated are also used for the lower powers.

The high voltage winding of the power stations main transformers are normally fitted with tap changers to allow for expanded adjustment range to the grid voltage.

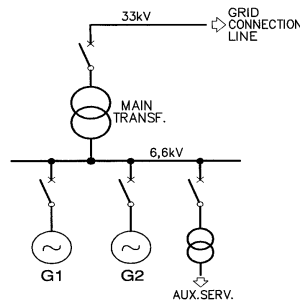


Fig. 8.5 – Typical single line diagram.

8.2.2. Switchgear

The mini-hydro power stations normally use the medium voltage switchgear, switches and circuit breakers, housed in metal cubicles, these being standard market products satisfying the European electrical regulations (CEI).

The circuit breakers operate in SF₆ or vacuum also with standard ranges of breaking capacity and open / close times.

8.2.3. Control equipment

The typical powerstation is equipped with electrical control and supply boards, housing the manual and automatic control circuits and auxiliary equipment.

Switches and push buttons support the manual controls, with the operator being informed about the state of the plant by the indicating instruments.

The automatic controls are based on a Programmable Logical Controller (PLC), which receives on line information through transducers and digital input signals and takes the necessary control actions for water utilisation, flow or level control. The PLC output is processed via suitable relays.

The PLC is also used for data processing and transmission via telephone modems or radio signalling.

8.2.4. Electrical protection

The main features of electrical protection are the generator, transformer and grid connection.

Generator protection

The amount of protection relays depends on the machine's rating, normally being installed overcurrent, under-voltage, reverse power and stator earth fault. The units above 2 MW may have also differential, loss of field and negative phase sequence relays.

Transformer protection

The overcurrent relays are always installed. The differential relay is used in units above 2 MW. Transformers are normally supplied with Buchholz, oil and winding temperature relays and pressure relief valves.

Grid protection

These protections must ensure appropriate separation from the grid in case of a fault. The minimum protection scheme is normally defined by the utility standards.

Nowadays the tendency is to install the less important protection features, on the small machines, as part of the PLC software. This option requires however some careful risk evaluation and circuitry analysis, ensuring minimum reliability levels.

8.3. Control system considerations

8.3.1. Introduction

The generating unit behaviour shows the effects of the hydraulic circuit, the turbine and the generator dynamics.

Figure 8.6 shows the interaction of the various blocks.

The hydraulic circuit and turbine features have been dealt in previous chapters.

This chapter describes the way the control actions are carried out for speed, water level, power output and voltage / power factor control.

Speed, water level regulation and generated power are controlled, as previously mentioned, by the powerstation PLC using appropriate software. This software is based on sampled data with a short (20 to 50 ms) scan cycle time. The controller is continuously acquiring data at a fixed sample rate. This system is very suitable to mini-hydro stations because the time constants involved in these processes are very long when compared to the cycle time. Furthermore, the regulation parameters are very accessible.

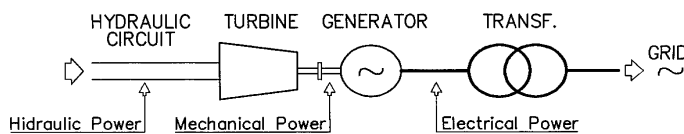


Fig. 8.6 – Power conversion scheme.

8.3.2. Speed control

Taking a Francis turbine as an example, the wicket gate is operated by a hydraulic servomotor, controlled by electro-valves activated by the PLC outputs. On start-up the wicket gate opens to a pre-set value and as the unit gains speed the wicket gate closes to the value needed to run at no-load around synchronous speed. Acceleration must be very low as speed reaches the nominal value. A percentage opening varying between 6% and 12%, depending on the turbine characteristics ensures the no-load running. The precise speed regulation for synchronisation takes place via an automatic synchroniser. The total time from standstill to synchronisation is of the order of 3 to 4 minutes.

The initial stage of speed increase is done with proportional control impulses, adding derivative and integral control when necessary during the closure manoeuvre and final regulation to reach synchronous speed.

The automatic synchroniser is an analogue device fitted with proportional and integral control loops.

8.3.3. Water level regulation

This type of regulation is used mainly on run-of-the-river schemes. For cases of small forebays, it is necessary to have proportional and derivative control and sometimes, integral controls if it is relevant to maintain a very precise water level.

In other cases, with some storage capacity, it is sufficient to have proportional control and slow actions.

Normally in this type of regulation, maximum and minimum water levels are established to back up the normal control system and avoid excessive deviations.

8.3.4. Generator output power control

This is normally done when significant storage capacity is available and consists in operating a powerstation at pre-established fixed power / flow values during the more relevant hours of the day and making use of the best combination of turbine and hydraulic efficiencies.

The wicket gate control is done by impulses until the pre-set generator output power is reached.

As in the previous case, upper and lower limits are established to the water level.

In this type of water utilisation, the volumes of water to be used are calculated daily to establish the operating power values.

8.3.5. Voltage and power factor regulation

This control is done over the generator excitation system.

Under no-load conditions the AVR controls voltage and assists the automatic synchroniser, providing the correct voltage for synchronisation. When on-load, connected to the grid, it changes to power factor regulation and controls the reactive power exchange with the grid, according to the regulations.

The voltage regulation is still typically done by dedicated AVR's, fitted with Proportional, Derivative and Integral control loops (PID).

The parameters on this equipment are available for adjustment in terms of the gains and time constants of each loop, as well as the set points.

The AVR settings are normally pre-set at the factory and adjusted experimentally on site.

8.3.6. Flood level control

When flood control gates are installed at the dam, these should have a control system of the water level regulation type. The gates open by the appropriate amounts, in order to regulate the upstream level. The flood gate manoeuvres are very slow, avoiding sudden changes of flow downstream.

8.3.7. Blade control of Kaplan turbines

This is also an example of application of Proportional and Integral control since precision is necessary as it affects the turbine efficiency. For specific cases derivative control may also be added.

8.3.8. Analytical representation

The PID control may be represented (Figure 8.7) by the following basic expression:

$$e(t) = K_p Z(t) + K_i \int_0^t Z(t) dt + K_d \frac{dZ(t)}{dt} \quad (8.4)$$

where

$e(t)$ – is the relative variation of any parameter (ex: voltage deviation: $V_{ref} - V_g$) with respect to the reference;

$Z(t)$ – is the relative variation of other associated variable (ex: exciter field current – I_{fe}) with respect to the regulation variable;

$K_p; K_i; K_d$ – are the proportional, integral and differential gains of the regulator.

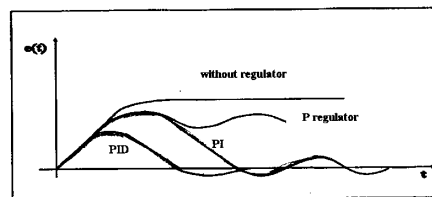


Fig. 8.7 – Typical response of a PID regulator (RAMOS, 1995).

Applying Laplace Transforms this may be represented under the block diagram transfer function form, as indicated in Figure 8.8.

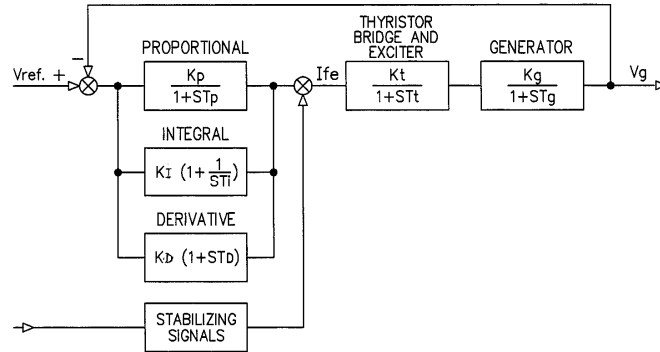


Fig. 8.8 – PID control system. Example of AVR block diagram.

8.3.9- Switchyard

For plants with capacities greater than 600 kW the transformers and switching gear should be located in a yard outside. This switchyard should be as close to the powerhouse as possible and should be surrounded by a sturdy, durable fence for safety (Figure 8.9).



Fig. 8.9 – Grid substation to transform the energy produced from 6kV to 60kV and fed it into the supply network of the national electric grid.

9

ENVIRONMENT

Helena Ramos

9.1- Environmental impact assessment

The purpose of an environment impact assessment (EIA) applied to small hydro power plant consists in the evaluation of the favourable and unfavourable impacts, in what concerns the two categories of impacts, in natural and social environmental context. Natural impacts include hydrology and sediment effects, as well as the water temperature and quality, ecology, engineering construction, biology, landscape and effects on archaeology and cultural assets, soils and geology, air, noise and, eventually, climate local change (in case of large reservoirs). Social impacts involve social, cultural and economic development inducing local industrialisation and changes in citizens' life quality as well as potential people displacement due to submersion by the reservoir (nevertheless rarely applied at small hydro plants - Figure 9.1).

For an EIA, firstly, it is necessary to identify potential impacts based on intrinsic characteristics of the small hydro plant and on the site, taking into consideration the experience and the knowledge of impacts provoked by similar projects and contributions from the technical team. According to the magnitude and the importance of impacts, the prediction specific techniques, based on quantitative and qualitative analyses, must be carried out, in order to classify them as

positive, significant or not, whenever occurs a violation of standards or rules legally approved or accepted by the scientific community.

Being the water an essential resource that sustains all life on Earth, it is important to plan its correct use. Nowadays it is obligatory to provide all information about watercourses through suitable catchement management plans, which includes several aspects, such as water abstraction, effluent disposal, flood defence, amenity and recreation, pollution, land uses, flow variation and wildlife conservation.

The flow in streams and rivers is, normally, the main outflow component from a catchement, and the river damming even by a low dam constitutes a major concern of populations (when not well informed).

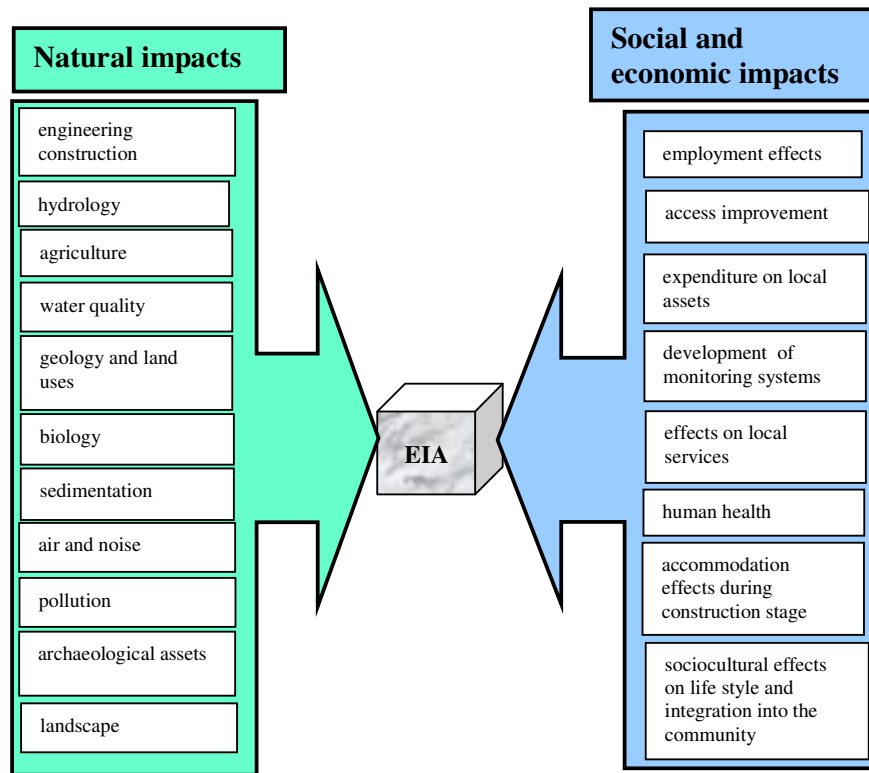


Fig. 9.1 - Linkage between natural and socio-economic impacts in an EIA study for a small hydropower plant (SHP).

9.2- Identification of impacts

Water quality - Water in natural rivers is never completely pure, varying, considerably, in the range and in concentrations of dissolved substances present, such as suspended particles, pH and temperature. Fast-flowing streams and rivers, normally, have high oxygen levels absorbed from the atmosphere due to natural turbulence. Still waters, such as reservoirs or forebays have highly variable levels that may range from a minimum during groups' operation to a maximum during non-operation period. Stratification can occur that in upper layer is well oxygenated and in the lower layer there is no any contact with the atmosphere suffering oxygen depletion.

The aquatic life is adapted to temperature regimes presented in freshwater systems. Changes in temperature can eventually promote oxygen consumption. Nevertheless, in these type of systems, there is no significant change in water temperature.

Sedimentation – Dams will retain solid material transported by the river. The reduction of solid flow for the downstream river reach may cause concern due to the increased potential erosion capacity. Small reservoirs and sediment discharge outlets provided in the dam will mitigate this impact..

Air - There are no emissions of toxic gaseous, such as oxides of sulphur and nitrogen that could affect the natural pH and, consequently, the water quality. Although, during the construction is necessary the installation of a concrete central and due to excavations and the circulation of heavy vehicles these can induce particle emissions and gases (e.g. CO, CO₂ and SO_x).

Pollution – During exploitation, in powerhouses the oil of generator' lubrication and transformers must be caught in appropriate site, in order to avoid the water or land contamination with complex toxicity. Residual materials from the construction must be removed and located in appropriated sites. Gas emissions by vehicles and machines in working and siltation by excavations are a matter of concern.

Soils – The implementation of the hydro scheme should take into account the soil ability, in particular for agricultural purposes. During the construction phase, soils occupation by civil works and reservoir (in case of existing) can affect the

soils in a non-reversible way. Excavation, land moving and superficial deforestation are relevant actions in soil impacts.

Noise - The noise produced during hydropower operation caused by the turbine(s), the generator(s) and the cooling ventilation, is transmitted to the powerhouse structure and can be harmful for the external or outdoor environment. The external noise level can be minimised by improving the acoustic isolation of the powerhouse and turbine, controlling vibrations of the ventilation system, improving hydrodynamic design of hydraulic structures and by using non-reflecting and sound absorbing materials such as fibreglass mat, false ceiling and heavy mass trap doors. The sound produced in a powerhouse through the generator is about 82 dB(A) at 7 m of distance. A covered wall can decrease it about 25 – 50 dB(A) using specific soundproofing materials. The World Health Organisation (WHO) suggests that daytime outdoor noise levels should be below 50 dB(A) (i.e. Leq levels must be lesser than 55 dB(A) in daytime and 45 dB(A) at night, where Leq is the average notorious steady noise sounds for a given period of time). During the construction, a temporary increasing of sound levels caused by working machines will be verified. The noise level study should include also the flowing water, especially at the tailrace (as it can be higher than 60 – 70 dB(A)).

Biota – Civil works will provoke several disturbances in fish habitats. Any turbined flow variation due to power generation change can affect these habitats. It is required that any engineering works provide a minimum flow (or reserved flow), in order to ensure the life and the free passage of migrating fishes. This happens more frequently whenever there are diversion works with reduction of flow between the intake and the powerhouse. The reserved flow depends on the guarantee of aquatic biota protection and the physical characteristics of the catchment of the hydropower schemes. Nevertheless, as an approximate value, this flow can be about 5 to 10% of the long-term average flow. Fish-passages or fish ladders should be installed to allow migratory fish. A fish-ladder should allow the movement of the existing species in the ecosystem by providing a well designed access and flow rates whether by swimming or jumping.



Fig. 9.2 – A fish-ladder of a SHP.

Landscape – Civil works and exploration of stone quarries, lend areas, the transport of materials and sites for waste deposition should not affect areas with high landscape quality. Deforestation, land removal and the installation of long conveyance systems (e.g. canal or penstock) may have a harmful visual impact in higher slope zones or mountainsides. The constructions of the dam and the powerhouse, as well as the access imply local deforestation, creating a clearing in the woods with potential significant morphologic alterations.



Fig. 9.3 – A good example of SHP dam in landscape integration.



Fig. 9.4 – An example of a SHP powerhouse in landscape integration.



Fig. 9.5 – Deforestation for pipe installation.

Social-economics, land arrangement and municipal planning – Whenever plans land arrangement policy and life population standards are affected or modified special concerns get higher. During the construction will happen an increase of the local traffic that can disturb the quotidian life of a rural isolated

region. The construction of a small hydropower plant or mini-hydro can affect zones that can belong to ecological protected areas (EPA) or agricultural protected areas (APA). Disturbances related with natural flow river interruption, dragging solid materials to downstream and excavations are predictable in EPA. Reservoir filling and vehicles motion can affect APA.

Geomorphology - Excavation and land depositions normally associated to the dam and powerhouse construction and conveyance system installation can affect the ground and geologic characteristics. The retention of water and solid particles by a dam will modify the geomorphologic equilibrium of a river inducing the erosion phenomenon.

Climate local change – During the construction stage a small alteration of the micro-clime conditions close to the soil can occur, through an increasing of the temperature and wind, and a decreasing of relative air humidity due to destruction of natural vegetation in the dam site, powerhouse access and hydraulic conveyance installation. During exploitation it will occur more frequently smog at the reservoir zone (if it is large enough).

Water resources – The deforestation occurred during the construction induces the production of organic material that induces the degradation of the water resources. Land removing also becomes water turbidity.

Archaeological and historic heritage – It is advisable to develop surveys in situ and to have necessary caution for special local heritage materials, namely old mills and barrages or aqueducts or cultural heritage monuments.

Developers and engineers must be aware of what it is required to be taken into consideration during the different stages of a project (e.g. during the planning, construction and exploitation).

9.3- Activities associated with hydropower plants

The use of an area for construction will affect the soil structure, reducing the porosity and the aeration, diminishing the diversity of the soil biota, and damaging vegetation through the destruction, reducing plant growth and ability land to recover. For a low dam and powerhouse these effects are not especially relevant.

Table 9.1- Activities and impacts due to a small hydropower plant project

Activities	Potential Impacts
- River engineering - Pipe installations - Channelization - Dam and Powerhouse construction	Enhanced erosion and silt production Destruction of some vegetation Damage in habitats and biota Land expropriation and access affectation Wastes creation and noise production Different land uses affected
- Reservoir (even small)	Localised changes in the water level Loss of terrestrial habitats (at small scale) Changes in flow regimes, detention time, siltation, depth and potential stratification Changes in groundwater recharge areas and in the water quality
- Access - roads	Changes in drainage systems due to landscaping (e.g. gradient changes and embankments)
- Powerhouse	Noise production (noise control will be possible)
- Excavations	Increase runoff, affect stream flows and stream sediment loads and modify soil compaction
- Deforestation	Reduced interception with consequent channel erosion, flood risk and runoff (can be minimised)

A small hydro scheme that comprises different components (i.e. a low dam, weir, spillway, intake, channel/penstock, powerhouse, tailrace, substation and transmission electric lines) changes the visual aspect of the valley. Nevertheless, the most significant impacts occur during the construction phase. Some architectural creativity for the powerhouse external aspect and integration can avoid contrasting with the background and landscape colours resulting in an acceptable visual project (e.g. colours of penstock and powerhouse).

Buried pipes in order to avoid a barrier for wildlife or people passage or agricultural works, utilising local materials (e.g. stones or rocks to adornment channel walls and powerhouses or small dams and weirs are appropriated), reforesting excavation areas, according to local existent vegetation, are examples to improve the compatibility of artificial works with the environment.

9.4- Reference criteria to support environmental studies

University of Manchester Centre developed a methodology based on a list of revision criteria hierarchically organised that includes the following stages: 1) project description and reference situation; 2) identification and assessment of main impacts; 3) alternative solutions and mitigation; 4) communication of final results (see Figure 9.6).

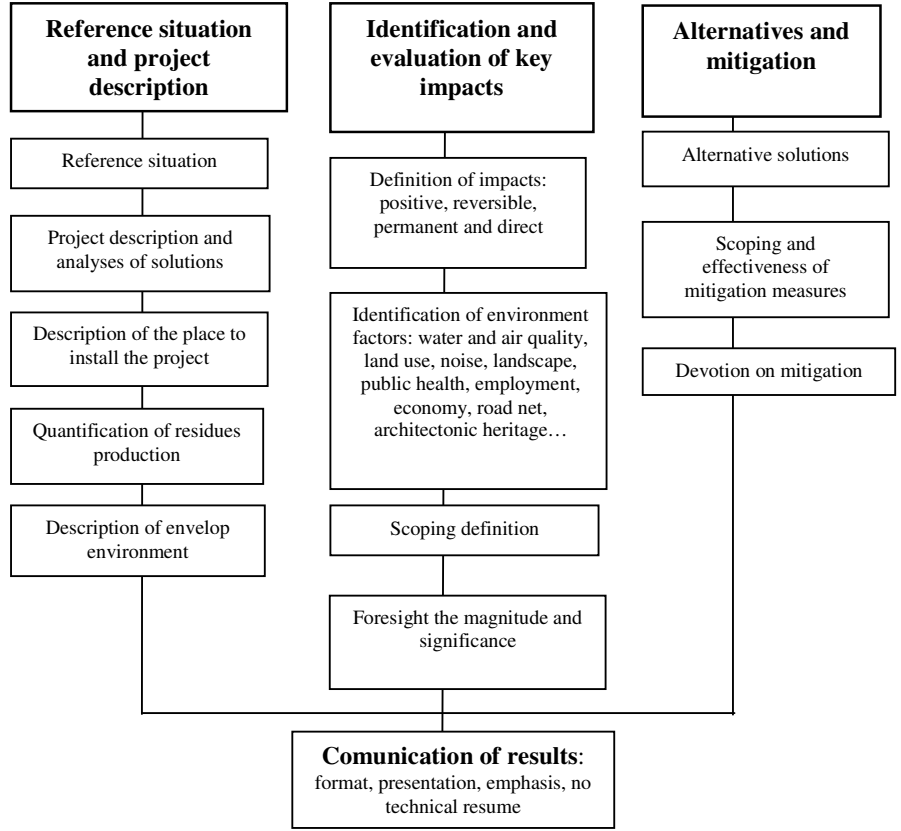


Fig. 9.6 - Environmental impact assessment criteria.

Mitigation strategies are based on elimination, reduction or compensation measurements. The development of an EIA should attend the existing

legislation, the decision of the project proponent or by imposition of responsible organisms or even for licence purposes.

The content of an EIA should characterise the environmental system (scoping analysis and definition), the relation between project components and environment changes (screening), identification of affected environmental factors, impacts quantification through specific models, environmental effect assessment and consequences and, finally, a correct and sufficiently clear presentation of results. This analysis summarises three important components: a) identification of the type and the cause; b) foresight the quantity resulting from relationship between cause-effect of each impact; c) consequence assessment.

9.5- Safety

The exploitation of a small hydropower scheme can be dangerous. In fact, the water volume stored by the dam and the flow towards the intake can be danger for children and others that fall into the water or want to enjoy the benefits of an artificial lake. Protection devices should be installed to safe anyone and to prevent the suction towards the intake. Downstream the powerhouse there is always the danger associated to a sudden flow variation during a turbine operation. This hazard should be clearly noticed and sound signals should be considered, in order to warn fishermen and other people crossing the river.

Low dams do not constitute a great risk for downstream valley, regarding a failure scenario due to an abnormal hazard. However, the potential damages and losses along the valley due to a dam-break flood need to be evaluated. Emergency plans and risk zoning should be considered in certain special cases.



Fig. 9.7 – A life guard grid with float balls.

10

ECONOMIC ANALYSIS

M. Manuela Portela

10.1-Introduction. Costs and benefits

The final decision on whether or not a small hydropower scheme should be constructed, or the selection among alternative design solutions for the same is generally based on the comparison of the expected costs and benefits for the useful life of the project, by means of economic analysis criteria. This analysis should be performed in the first stages of the design (along with the feasibility study) as nothing ensures that a project suitable from a technical point of view is also advantageous from an economic point of view.

It should be pointed out that the choice among alternative solutions having identical benefits is simply accomplished by identifying which one has the lower cost or the less negative environmental impact. Only the comparison of alternative projects or design solutions with different costs and benefits requires the application of economic criteria in order to identify the most desirable alternative.

The effectiveness of the economic analysis as a decision tool of a small hydro investor depends on the accuracy of the project cost and benefits estimates. These estimates are not easy to reach, especially in the early stages of design where some of the scheme characteristics are only preliminary defined.

The costs of a small hydropower scheme can be grouped in the three following categories, also systematised in Table 10.1:

- The capital cost that may be defined as the sum of all expenditures required to bring the project to completion. These costs occur during the construction period (generally, from one to three years).
- The annual operation costs resulting from the exploitation and maintenance of the scheme during its useful live.
- The reposition costs concerning to the substitution of the equipment having a useful live lesser than the one of the scheme.

Table 10.1 – Costs of a small hydropower scheme

Capital costs	Studies and design Supervision during the execution Civil works Equipment Land acquisition Contingencies or unforeseen cost
Annual operation costs	Exploitation Maintenance Spare parts Grant of permission
Reposition costs	

The following items should generally be considered within the capital costs – Table 10.1:

- Studies and design.
- Supervision during the execution.
- Civil works (including access roads and building site facilities).
- Equipment (including hydromechanical, electromechanical and electrical equipment and the interconnection to the electrical grid).
- Land acquisition.

It should also be considered an item for contingencies or unforeseen cost. This item represents expenditures that are possible but not certain or yet foreseen. For instance, this item intends to overcome the uncertainty that result from the fact

that the characteristics of the site where the scheme is going to be constructed are not totally known, especially in the first design phases and, so, they may be incompletely expressed in the scheme conception. The uncertainty under consideration is mainly related with the civil works and can result, for instance, from undervalued excavation or landing works or from bedrock having characteristics worse than those foreseen in the design.

The studies and design costs and the supervision costs during the scheme execution result from agreements between the investor and consulting firms. If the investor has previous experience in the execution of small hydropower schemes, he should be able to express the costs under consideration as a percentage of the civil works and equipment costs.

The civil work costs are evaluated from the design, by measuring the work quantities relative to the different components of the scheme and affecting them of unit prices, which generally are not difficult to obtain from each country civil work contractors market. To evaluate the equipment costs, budget prices from the suppliers should also be obtained.

The cost of the acquisition (or renting) of the land that will be occupied by the scheme (including accesses and the area that will be submerged by the scheme reservoir) depends strictly on the land valorisation in each country or region.

If possible, the capital costs estimated for the small hydropower scheme under study should be compared with the ones of similar schemes, already built or previously characterised. This procedure is more important in what concerns the evaluation of the equipment cost as frequently it is not possible to get the budget prices from the suppliers in a period as short as the design one.

The annual operation costs include the following main components – Table 10.1:

- Exploitation.
- Maintenance.
- Spare parts.
- Grant of permission or legal permit.

The exploitation costs represent the charges with the staff responsible for the scheme operation. To reduce these costs the scheme should be made fully

automatic, that is to say, explored in an abandon mode. This kind of exploitation requires additional equipment (for instance, automation systems, telephone or satellite lines for telemetry and alarm signals).

The maintenance costs include two parcels, related one with the civil works and the other, with the equipment. The former is generally the smallest representing from 0,25% to 0,50% of the capital costs in civil works, while the latter can reach about 2,50% of the respective capital costs.

The spare parts costs are the costs with the reposition of the material that is necessary to keep in stock in order to perform the maintenance of the hydropower scheme or to execute small repairs in the same. These costs can be assigned to annual costs or to costs without periodicity, occurring whenever it is necessary to restore the stock of the material.

The grant of permission costs occurs once the scheme starts to operate and represent the annual payments due for the scheme license and for the water utilisation. Although these costs result from the legislation in force in each country, in terms of economic appraisal they can be treated as fixed percentages of the energy incomes.

From the investor point of view, the only tangible revenue or benefit in a small hydropower scheme is the annual income with the energy production sale. This income depends on the amount of energy produced during the scheme lifetime and on the specific conditions that rule the hydroelectric sector, namely the energy sale contract conditions and the tariffs policy, which are specific in each country.

In the further development of this chapter it will be assumed that the purchase of the energy produced in a small hydropower scheme is ensured, no matter the amount and the characteristics of the production. This scenario should be real as it translates the special rule that the renewable energies are expected to play in the energetic policy, not only of Europe, but also of the world.

The evaluation of the energy production requires accurate hydrological studies based on specific methodologies that overcome the non-existence of basic hydrological data that characterises most of the small hydropower schemes, generally located in small and ungauged watersheds. The results of the

hydrological study have to be interpreted taking into account the characteristics of the watershed that condition the water availability and regime at the water intake of the scheme: watershed geology, vegetation covering, land use, water consumption, existing upstream storage reservoirs, ecological requirements.

As the energy production that will result from the future inflows at the scheme water intake can not be specified, the revenues in the economic analysis are considered to be fixed during the lifetime of the project and based on a annual long-term average discharge of the river. By other words, the revenues are generally defined on the basis of an annual production constant and equal to the expected mean one, \bar{E} .

The uncertainty concerning the future inflows and, thus, concerning the future energy productions, can be regard as a hydrologic risk. Based on the statistic analysis of the historic flow series, this risk can be expressed, for instances, in terms of the probability of occurring future periods of dry years or of years with annual flows bellow a limit. This kind of analysis is most important and should be performed in countries, as those of south Europe, characterised by an irregular flow regime where dry periods of several years can occur.

Based on simulation studies, PORTELA and ALMEIDA, 1995, carried out hydrologic risk analysis for several hypothetical small hydropower schemes located in the North of Portugal. This analysis clearly shows that the profitability of those schemes could be drastically reduced if periods of dry years occur, especially in the first years of exploitation, when the incomes are crucial for the capital recovery.

Figure 10.1 represents some of the results achieved by those authors. The cost factor, λ , was defined as the ratio between the income under real flow conditions and the income evaluated on the basis of an annual production constant and equal to the mean annual production. Two exploitation periods where considered: one with 33 years and the other with only 10 years. Based on real flow data, simulation studies were performed in order to evaluate the maximum (best series) and the minimum (worst series) incomes during each of the previous periods. The results thus achieved were represented as a function of the ratio between the design flow, Q_{max} , and the average mean daily flow, Q_{ave} .

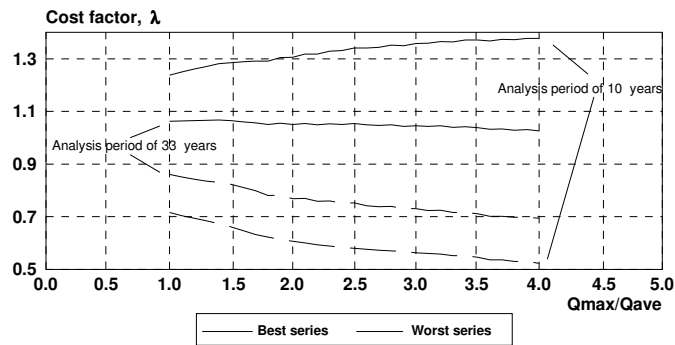


Figure 10.1 – Variation of the income in a hypothetical small hydropower scheme, located in the North of Portugal, due to the natural variability of the flow regime (PORTELA and ALMEIDA, 1995).

10.2-Economic analysis

10.2.1-Introduction

In this item some concepts necessary to the economic analysis or appraisal of small hydropower schemes are presented in a simple and straight way.

Several literature references are available either in general terms of economic and financial analysis principles and applications (which are far beyond the scope of this Guide) or in specific terms for small hydropower schemes. Among these last ones KUIPER, 1981, ESHA, 1994 and 1997, and JIANDONG *et al.*, 1996, may also have interest in the domain of the present chapter.

10.2.2- Constant market prices system concept

Although the design of a small hydropower scheme generally foresees a schedule for the investment costs and revenues, the respective estimates are based on market prices referred to a given year, the one during which the studies were performed. The evolution of the inflation during the project lifetime and its effect on each component of the previous cost estimates is practically impossible to establish.

So, one could say that one of the main problems of the economic analysis is to conceive a scenario for the future evolution of the inflation. To overcome this problem a common and simple economic approach based on a constant market prices system referred to a given year is generally applied in the comparison of costs and benefits either of a project or of alternative design solutions for the same. This approach, which will be adopted in the present chapter, assumes that it is not necessary to account for the inflation, as it will have the same effect in any monetary flux. The future costs and benefits are, then, evaluated at present market prices.

10.2.3-Discout rate and present value concept

A given monetary unit (for instances, a pound or a dollar) is more worthy in the present than in the future. By other words, the future value of the monetary flux that has a present value of one unit will be great than one. At the same time, the present value of a future unitary monetary flux will be lesser than one.

Through the years, this situation generates different “appetencies” to transfer money from the present to the future and vice-verse. Theses “appetencies” can be expressed in terms of different discount rates, r . The values of these rates depend, among other factors, on the state of the economy, on the risk that involves the investment, the capital availability and on the expected future rate of inflation.

Let n denote a period of n years, from year 1 to year n . According to the discount rate concept and as represented in Figure 10.2, one monetary unit of today will be changed in year n by $(1+r)^n$ monetary units and one monetary unit of year n will be changed today by $1/(1+r)^n$ units. If r is defined, not on a yearly basis, but for a generic period of time α , n will denote the total number of α periods between the two monetary fluxes of Figure 10.2.

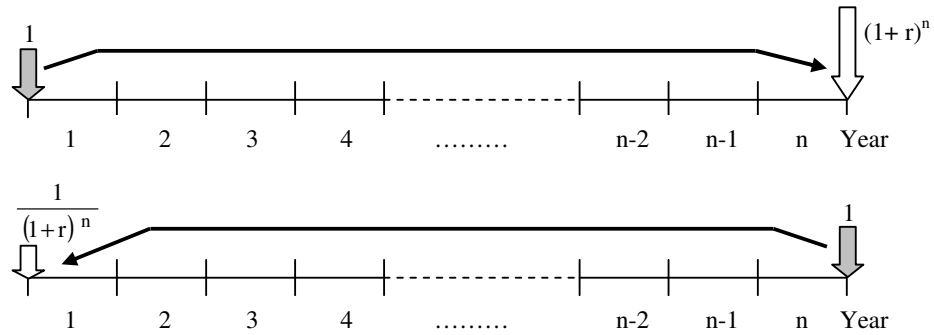


Fig. 10.2 – Transference of unitary monetary fluxes by means of the discount rate, r .

The factor $\frac{1}{(1+r)^n}$ that expresses the depreciation suffered by future monetary fluxes when transferred to the present is called present worth or value factor. According to this concept, the present value, PV, of a single generic monetary flux occurring in future year i , C_i , is given by

$$PV = \frac{1}{(1+r)^i} C_i \quad (10.1)$$

The present value for the beginning of year 1 of the continuous sequence of annual monetary fluxes represented in Figure 10.3 is given by

$$PV = \frac{1}{(1+r)} C_1 + \frac{1}{(1+r)^2} C_2 + \dots + \frac{1}{(1+r)^i} C_i + \dots + \dots + \frac{1}{(1+r)^{(n-1)}} C_{n-1} + \frac{1}{(1+r)^n} C_n \quad (10.2)$$

$$PV = \sum_{i=1}^n \frac{1}{(1+r)^i} C_i \quad (10.3)$$

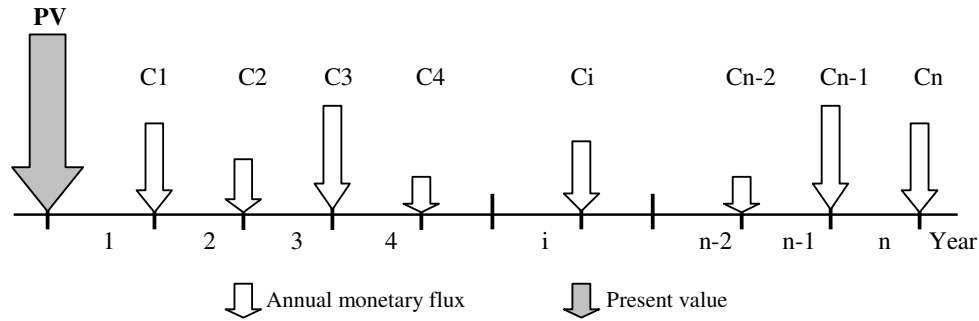


Fig. 10.3 – Continuous sequence of monetary fluxes.

For economical analysis purpose the monetary fluxes are grouped in periods (generally years) and are considered to occur as represented in Figure 10.3, that is to say, concentrated in the end of each of those periods. The present value operations are performed for the beginning of the year adopted as reference (year one in the previous figure).

If the monetary fluxes of Figure 10.3 are constant and equal to an annuity, C , – uniform series of annual monetary fluxes – the following relation is achieved for the respective present value referred to the beginning of year 1:

$$PV = C \sum_{i=1}^n \frac{1}{(1+r)^i} = C \frac{(1+r)^n - 1}{(1+r)^n r} \quad (10.4)$$

where $\frac{(1+r)^n - 1}{(1+r)^n r}$ is the present worth factor for uniform series. The inverse of this factor is denoted by capital-recovery factor.

The present worth factor either for a single monetary flux or for a series of uniform monetary fluxes is generally provided by tables as the ones presented in next pages, which can be found in most books dealing with economic analysis concepts.

The factor $(1+r)^n$ represented in the first part of Figure 10.2 is the capitalisation factor or the future value factor. Table 10.2 also includes its value either for a single monetary flux or for an uniform series of monetary fluxes. The formula that provides the future value factor for an uniform series of monetary fluxes is given by $\left[\frac{(1+r)^n - 1}{r} \right]$. The inverse of the previous ratio is the sinking-fund factor.

Although the computation of the previous factors does not offer special difficulty it is useful to provide tables as those included herein in order to allow expedite evaluations or to control results of the economic appraisal.

It should be pointed out that the expressions and concepts presented so far, as well as their development and application in the rest of this chapter, assume that the discount rate, r , is constant during the n years period under analysis. If this is not the situation, different sub periods having different discount rates should be considered. However, this is not a common procedure in the economic analysis of small hydropower schemes.

Table 10.2 (1/4) – Values of the present worth factor and of the capitalisation factor for different time periods, n, and discount rates, r

a1) Present worth factor for single monetary fluxes $\left[\frac{1}{(1+r)^n} \right]$

Time Period, n	Discount rate, r, of:						
	2%	4%	6%	8%	10%	12%	14%
1	0,98039	0,96154	0,94340	0,92593	0,90909	0,89286	0,87719
2	0,96117	0,92456	0,89000	0,85734	0,82645	0,79719	0,76947
3	0,94232	0,88900	0,83962	0,79383	0,75131	0,71178	0,67497
4	0,92385	0,85480	0,79209	0,73503	0,68301	0,63552	0,59208
5	0,90573	0,82193	0,74726	0,68058	0,62092	0,56743	0,51937
6	0,88797	0,79031	0,70496	0,63017	0,56447	0,50663	0,45559
7	0,87056	0,75992	0,66506	0,58349	0,51316	0,45235	0,39964
8	0,85349	0,73069	0,62741	0,54027	0,46651	0,40388	0,35056
9	0,83676	0,70259	0,59190	0,50025	0,42410	0,36061	0,30751
10	0,82035	0,67556	0,55839	0,46319	0,38554	0,32197	0,26974
11	0,80426	0,64958	0,52679	0,42888	0,35049	0,28748	0,23662
12	0,78849	0,62460	0,49697	0,39711	0,31863	0,25668	0,20756
13	0,77303	0,60057	0,46884	0,36770	0,28966	0,22917	0,18207
14	0,75788	0,57748	0,44230	0,34046	0,26333	0,20462	0,15971
15	0,74301	0,55526	0,41727	0,31524	0,23939	0,18270	0,14010
16	0,72845	0,53391	0,39365	0,29189	0,21763	0,16312	0,12289
17	0,71416	0,51337	0,37136	0,27027	0,19784	0,14564	0,10780
18	0,70016	0,49363	0,35034	0,25025	0,17986	0,13004	0,09456
19	0,68643	0,47464	0,33051	0,23171	0,16351	0,11611	0,08295
20	0,67297	0,45639	0,31180	0,21455	0,14864	0,10367	0,07276
21	0,65978	0,43883	0,29416	0,19866	0,13513	0,09256	0,06383
22	0,64684	0,42196	0,27751	0,18394	0,12285	0,08264	0,05599
23	0,63416	0,40573	0,26180	0,17032	0,11168	0,07379	0,04911
24	0,62172	0,39012	0,24698	0,15770	0,10153	0,06588	0,04308
25	0,60953	0,37512	0,23300	0,14602	0,09230	0,05882	0,03779
26	0,59758	0,36069	0,21981	0,13520	0,08391	0,05252	0,03315
27	0,58586	0,34682	0,20737	0,12519	0,07628	0,04689	0,02908
28	0,57437	0,33348	0,19563	0,11591	0,06934	0,04187	0,02551
29	0,56311	0,32065	0,18456	0,10733	0,06304	0,03738	0,02237
30	0,55207	0,30832	0,17411	0,09938	0,05731	0,03338	0,01963
35	0,50003	0,25342	0,13011	0,06763	0,03558	0,01894	0,01019
40	0,45289	0,20829	0,09722	0,04603	0,02209	0,01075	0,00529
45	0,41020	0,17120	0,07265	0,03133	0,01372	0,00610	0,00275
50	0,37153	0,14071	0,05429	0,02132	0,00852	0,00346	0,00143
60	0,30478	0,09506	0,03031	0,00988	0,00328	0,00111	0,00039
70	0,25003	0,06422	0,01693	0,00457	0,00127	0,00036	0,00010
80	0,20511	0,04338	0,00945	0,00212	0,00049	0,00012	0,00003
90	0,16826	0,02931	0,00528	0,00098	0,00019	0,00004	0,00001
100	0,13803	0,01980	0,00295	0,00045	0,00007	0,00001	0,00000

Table 10.2 (2/4) – Values of the present worth factor and of the capitalisation factor for different time periods, n, and discount rates, r

a2) Present worth factor for uniform monetary fluxes $\left[\frac{(1+r)^n - 1}{(1+r)^n r} \right]$

Time period, n	Uniform series of monetary fluxes and discount rate, r, of:						
	2%	4%	6%	8%	10%	12%	14%
1	0,98039	0,96154	0,94340	0,92593	0,90909	0,89286	0,87719
2	1,94156	1,88609	1,83339	1,78326	1,73554	1,69005	1,64666
3	2,88388	2,77509	2,67301	2,57710	2,48685	2,40183	2,32163
4	3,80773	3,62990	3,46511	3,31213	3,16987	3,03735	2,91371
5	4,71346	4,45182	4,21236	3,99271	3,79079	3,60478	3,43308
6	5,60143	5,24214	4,91732	4,62288	4,35526	4,11141	3,88867
7	6,47199	6,00205	5,58238	5,20637	4,86842	4,56376	4,28830
8	7,32548	6,73274	6,20979	5,74664	5,33493	4,96764	4,63886
9	8,16224	7,43533	6,80169	6,24689	5,75902	5,32825	4,94637
10	8,98259	8,11090	7,36009	6,71008	6,14457	5,65022	5,21612
11	9,78685	8,76048	7,88687	7,13896	6,49506	5,93770	5,45273
12	10,57534	9,38507	8,38384	7,53608	6,81369	6,19437	5,66029
13	11,34837	9,98565	8,85268	7,90378	7,10336	6,42355	5,84236
14	12,10625	10,56312	9,29498	8,24424	7,36669	6,62817	6,00207
15	12,84926	11,11839	9,71225	8,55948	7,60608	6,81086	6,14217
16	13,57771	11,65230	10,10590	8,85137	7,82371	6,97399	6,26506
17	14,29187	12,16567	10,47726	9,12164	8,02155	7,11963	6,37286
18	14,99203	12,65930	10,82760	9,37189	8,20141	7,24967	6,46742
19	15,67846	13,13394	11,15812	9,60360	8,36492	7,36578	6,55037
20	16,35143	13,59033	11,46992	9,81815	8,51356	7,46944	6,62313
21	17,01121	14,02916	11,76408	10,01680	8,64869	7,56200	6,68696
22	17,65805	14,45112	12,04158	10,20074	8,77154	7,64465	6,74294
23	18,29220	14,85684	12,30338	10,37106	8,88322	7,71843	6,79206
24	18,91393	15,24696	12,55036	10,52876	8,98474	7,78432	6,83514
25	19,52346	15,62208	12,78336	10,67478	9,07704	7,84314	6,87293
26	20,12104	15,98277	13,00317	10,80998	9,16095	7,89566	6,90608
27	20,70690	16,32959	13,21053	10,93516	9,23722	7,94255	6,93515
28	21,28127	16,66306	13,40616	11,05108	9,30657	7,98442	6,96066
29	21,84438	16,98371	13,59072	11,15841	9,36961	8,02181	6,98304
30	22,39646	17,29203	13,76483	11,25778	9,42691	8,05518	7,00266
35	24,99862	18,66461	14,49825	11,65457	9,64416	8,17550	7,07005
40	27,35548	19,79277	15,04630	11,92461	9,77905	8,24378	7,10504
45	29,49016	20,72004	15,45583	12,10840	9,86281	8,28252	7,12322
50	31,42361	21,48218	15,76186	12,23348	9,91481	8,30450	7,13266
60	31,72839	21,57725	15,79217	12,24336	9,91810	8,30561	7,13304
70	31,97842	21,64146	15,80910	12,24793	9,91936	8,30597	7,13315
80	32,18353	21,68485	15,81855	12,25005	9,91985	8,30609	7,13317
90	32,35179	21,71416	15,82383	12,25104	9,92004	8,30612	7,13318
100	32,48982	21,73396	15,82678	12,25149	9,92011	8,30614	7,13318

Table 10.2 (3/4) – Values of the present worth factor and of the capitalisation factor for different time periods, n, and discount rates, r

b1) Capitalisation factor for single monetary fluxes $\left[(1+r)^n \right]$

Time Period, n	Discount rate, r, of:						
	2%	4%	6%	8%	10%	12%	14%
1	1,02000	1,04000	1,06000	1,08000	1,10000	1,12000	1,14000
2	1,04040	1,08160	1,12360	1,16640	1,21000	1,25440	1,29960
3	1,06121	1,12486	1,19102	1,25971	1,33100	1,40493	1,48154
4	1,08243	1,16986	1,26248	1,36049	1,46410	1,57352	1,68896
5	1,10408	1,21665	1,33823	1,46933	1,61051	1,76234	1,92541
6	1,12616	1,26532	1,41852	1,58687	1,77156	1,97382	2,19497
7	1,14869	1,31593	1,50363	1,71382	1,94872	2,21068	2,50227
8	1,17166	1,36857	1,59385	1,85093	2,14359	2,47596	2,85259
9	1,19509	1,42331	1,68948	1,99900	2,35795	2,77308	3,25195
10	1,21899	1,48024	1,79085	2,15892	2,59374	3,10585	3,70722
11	1,24337	1,53945	1,89830	2,33164	2,85312	3,47855	4,22623
12	1,26824	1,60103	2,01220	2,51817	3,13843	3,89598	4,81790
13	1,29361	1,66507	2,13293	2,71962	3,45227	4,36349	5,49241
14	1,31948	1,73168	2,26090	2,93719	3,79750	4,88711	6,26135
15	1,34587	1,80094	2,39656	3,17217	4,17725	5,47357	7,13794
16	1,37279	1,87298	2,54035	3,42594	4,59497	6,13039	8,13725
17	1,40024	1,94790	2,69277	3,70002	5,05447	6,86604	9,27646
18	1,42825	2,02582	2,85434	3,99602	5,55992	7,68997	10,57517
19	1,45681	2,10685	3,02560	4,31570	6,11591	8,61276	12,05569
20	1,48595	2,19112	3,20714	4,66096	6,72750	9,64629	13,74349
21	1,51567	2,27877	3,39956	5,03383	7,40025	10,80385	15,66758
22	1,54598	2,36992	3,60354	5,43654	8,14027	12,10031	17,86104
23	1,57690	2,46472	3,81975	5,87146	8,95430	13,55235	20,36158
24	1,60844	2,56330	4,04893	6,34118	9,84973	15,17863	23,21221
25	1,64061	2,66584	4,29187	6,84848	10,83471	17,00006	26,46192
26	1,67342	2,77247	4,54938	7,39635	11,91818	19,04007	30,16658
27	1,70689	2,88337	4,82235	7,98806	13,10999	21,32488	34,38991
28	1,74102	2,99870	5,11169	8,62711	14,42099	23,88387	39,20449
29	1,77584	3,11865	5,41839	9,31727	15,86309	26,74993	44,69312
30	1,81136	3,24340	5,74349	10,06266	17,44940	29,95992	50,95016
35	1,99989	3,94609	7,68609	14,78534	28,10244	52,79962	98,10018
40	2,20804	4,80102	10,28572	21,72452	45,25926	93,05097	188,88351
45	2,43785	5,84118	13,76461	31,92045	72,89048	163,98760	363,67907
50	2,69159	7,10668	18,42015	46,90161	117,39085	289,00219	700,23299
60	3,28103	10,51963	32,98769	101,25706	304,48164	897,59693	2595,91866
70	3,99956	15,57162	59,07593	218,60641	789,74696	2787,79983	9623,64498
80	4,87544	23,04980	105,79599	471,95483	2048,40021	8658,48310	35676,9818
90	5,94313	34,11933	189,46451	1018,91509	5313,02261	26891,9342	132262,467
100	7,24465	50,50495	339,30208	2199,76126	13780,6123	83522,2657	490326,238

Table 10.2 (4/4) – Values of the present worth factor and of the capitalisation factor for different time periods, n, and discount rates, r

b2) Capitalisation factor for uniform monetary fluxes $\left[\frac{(1+r)^n - 1}{r} \right]$

Time period, n	Uniform series of monetary fluxes and discount rate, r, of:						
	2%	4%	6%	8%	10%	12%	14%
1	1,00000	1,00000	1,00000	1,00000	1,00000	1,00000	1,00000
2	2,02000	2,04000	2,06000	2,08000	2,10000	2,12000	2,14000
3	3,06040	3,12160	3,18360	3,24640	3,31000	3,37440	3,43960
4	4,12161	4,24646	4,37462	4,50611	4,64100	4,77933	4,92114
5	5,20404	5,41632	5,63709	5,86660	6,10510	6,35285	6,61010
6	6,30812	6,63298	6,97532	7,33593	7,71561	8,11519	8,53552
7	7,43428	7,89829	8,39384	8,92280	9,48717	10,08901	10,73049
8	8,58297	9,21423	9,89747	10,63663	11,43589	12,29969	13,23276
9	9,75463	10,58280	11,49132	12,48756	13,57948	14,77566	16,08535
10	10,94972	12,00611	13,18079	14,48656	15,93742	17,54874	19,33730
11	12,16872	13,48635	14,97164	16,64549	18,53117	20,65458	23,04452
12	13,41209	15,02581	16,86994	18,97713	21,38428	24,13313	27,27075
13	14,68033	16,62684	18,88214	21,49530	24,52271	28,02911	32,08865
14	15,97394	18,29191	21,01507	24,21492	27,97498	32,39260	37,58107
15	17,29342	20,02359	23,27597	27,15211	31,77248	37,27971	43,84241
16	18,63929	21,82453	25,67253	30,32428	35,94973	42,75328	50,98035
17	20,01207	23,69751	28,21288	33,75023	40,54470	48,88367	59,11760
18	21,41231	25,64541	30,90565	37,45024	45,59917	55,74971	68,39407
19	22,84056	27,67123	33,75999	41,44626	51,15909	63,43968	78,96923
20	24,29737	29,77808	36,78559	45,76196	57,27500	72,05244	91,02493
21	25,78332	31,96920	39,99273	50,42292	64,00250	81,69874	104,76842
22	27,29898	34,24797	43,39229	55,45676	71,40275	92,50258	120,43600
23	28,84496	36,61789	46,99583	60,89330	79,54302	104,60289	138,29704
24	30,42186	39,08260	50,81558	66,76476	88,49733	118,15524	158,65862
25	32,03030	41,64591	54,86451	73,10594	98,34706	133,33387	181,87083
26	33,67091	44,31174	59,15638	79,95442	109,18177	150,33393	208,33274
27	35,34432	47,08421	63,70577	87,35077	121,09994	169,37401	238,49933
28	37,05121	49,96758	68,52811	95,33883	134,20994	190,69889	272,88923
29	38,79223	52,96629	73,63980	103,96594	148,63093	214,58275	312,09373
30	40,56808	56,08494	79,05819	113,28321	164,49402	241,33268	356,78685
35	49,99448	73,65222	111,43478	172,31680	271,02437	431,66350	693,57270
40	60,40198	95,02552	154,76197	259,05652	442,59256	767,09142	1342,02510
45	71,89271	121,02939	212,74351	386,50562	718,90484	1358,23003	2590,56480
50	84,57940	152,66708	290,33590	573,77016	1163,90853	2400,01825	4994,52135
60	114,05154	237,99069	533,12818	1253,21330	3034,81640	7471,64111	18535,1332
70	149,97791	364,29046	967,93217	2720,08007	7887,46957	23223,3319	68733,1784
80	193,77196	551,24498	1746,59989	5886,93543	20474,0021	72145,6925	254828,441
90	247,15666	827,98333	3141,07519	12723,9386	53120,2261	224091,118	944724,767
100	312,23231	1237,62370	5638,36806	27484,5157	137796,123	696010,547	3502323,12

10.2.4- Economic indexes

10.2.4.1-Basic considerations

The evaluation of the profitability of a small hydropower scheme or the comparison of alternative design solutions for the same can be based on economic indexes or parameters. Among these parameters the next five ones will be treated herein: net present value (NPV), benefit/cost ratio (B/C), internal rate of return (IRR), average price of the kWh (AP) and payback period (T).

The economic parameters do not always identify the same project as being the most economic amongst a number of alternatives. At the same time, the preference of one particular parameter may alter with a change, even small, in the discount rate.

Being the results of the economic analysis so sensitive to the value considered for the discount rate much attention should be paid in the establishment of this value. However, in most situations, there is not a well-defined and unique discount rate but, instead, a range of possible discount rates. Although this range is often narrow, it is advisable to perform a sensitivity analysis of the project response to different discount rates. By this way it will be possible to partially overcome the uncertainty related with this rate and, at the same time, to conclude if there is any change in the relative preferences indicated by the economic parameters.

Specially to compare alternative design solutions for a small hydropower it is usual to evaluate, for each of these solutions, the ratio of the total capital cost to the installed capacity, that is to say, the cost per installed kW. Although this index is quite easy to obtain it is poorly informative from an economic point of view, as it does not take into account neither other costs of the project, nor its benefits. However it may be useful in the systematisation of the capital costs required for small hydropower projects or in comparison of alternative projects having similar benefits.

In the next items the following symbols will be utilised:

- n - number of the project lifetime periods (generally measured in years and equal to the duration of the legal permits);
- r - discount rate (constant along the project useful live);

- C_i - capital costs in year i ;
- O_j - annual operation cost for year j ;
- R_j - revenues in year j ;
- P_m - reposition cost foreseen for year m ($n/2 < m < n$);
- \bar{E} - mean annual production.

The abbreviations C , O , R and P will respectively denote the present values of the costs, C_i , O_j , P_m and revenues R_j . If

- the capital costs occur in the first k years,
- the energy production takes place from year $(k+1)$ until year n ,
- the equipment reposition is foreseen for year m ,

the following relations are obtained for the beginning of the n years period:

$$C = \sum_{i=1}^k \frac{C_i}{(1+r)^i} \quad (10.5)$$

$$O = \frac{\sum_{j=k+1}^n \frac{O_j}{(1+r)^j}}{(1+r)^k} \quad (10.6)$$

$$R = \frac{\sum_{j=k+1}^n \frac{R_j}{(1+r)^j}}{(1+r)^k} \quad (10.7)$$

$$P = \frac{P_m}{(1+r)^m} \quad (10.8)$$

The numerator of expressions (10.6) and (10.7) provides the present value (for the beginning of year $k+1$) of an annuity that will occur during $n-k$ years, from year $k+1$ until year n . The denominator performs the transference of the previous value from year k to the beginning of year 1 – Figure 10.4.

Expression (10.6) assumes that the parcel of the annual operation costs related with the legal permits will occur only when the scheme starts to operate which is generally a far enough realistic scenario.

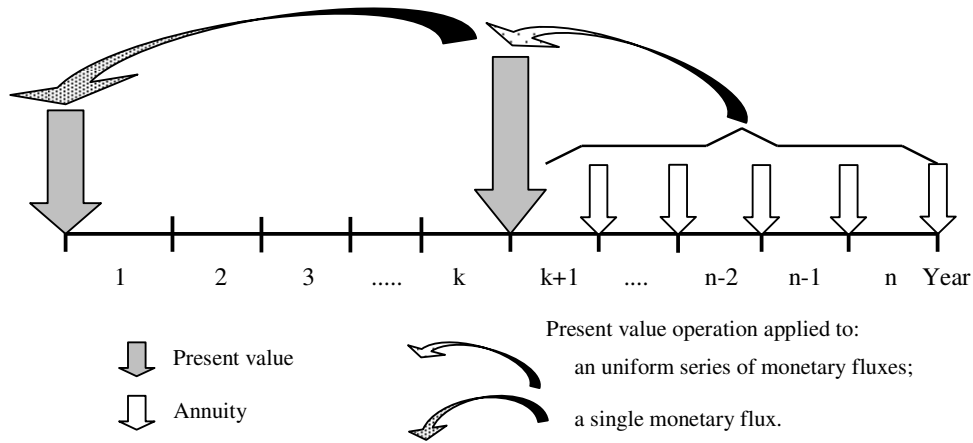


Fig. 10.4 – Compound present value operations.

As the annual benefits in a small hydropower scheme are generally evaluated on a constant annual production basis (the mean annual production, \bar{E}), expression (10.7) becomes

$$R = \frac{\bar{R} \frac{(1+r)^{(n-k)} - 1}{(1+r)^{(n-k)} r}}{(1+r)^k} \quad (10.9)$$

where \bar{R} is the mean annual benefit obtained by multiplying the \bar{E} by unitary sale price of the energy in the first year of the period under analysis (year 1).

The period for which the economical analysis is performed should be equal to the period foreseen in the legal permit of the hydropower exploitation. However, it is not reasonable to adopt periods greater than twenty five to thirty years, as the present values of the expected costs and benefits in the last years become so small that their contribution to the economic indexes is negligible.

If the lifetime of the equipment is greater than the assumed useful life of the hydropower scheme a residual value of the investment over its lifetime can be considered and jointed to the benefits. As the estimate of this residual quantity is quite subjective and fallible and its present value contribution for the scheme profitability will certainly be very modest, it will not be considered in the further development of this chapter.

10.2.4.2-Net Present Value (NPV)

The net present value, NPV, represents the cumulative sum of all expected benefits during the lifetime of the project minus the sum of all its cost during the same period, both expressed in terms of present values

$$NPV = R - C - O - P \quad (10.10)$$

If NPV is negative, the project should be rejected as it is expected that the benefits during its lifetime will be insufficient to cover the project costs. Assuming that there are not any restrictions with respect to the required initial capital availability, among projects or alternative design solutions with positive NPV the best ones will be those with greater NPV.

The net present value can also be evaluated by computing the discount cumulative cash flow. This cash flow gives, through each year of the economic analysis period, the value of the cumulative sum of the present values of the costs minus the present values of the benefits. The NPV equals the result provided by the cash flow for the last year of the period under analysis (year n).

10.2.4.3-Benefit/cost ratio (B/C)

The benefit/cost ratio, B/C, compares the present values of the benefits and costs of the hydropower scheme on a ratio basis. This index can be defined as the ratio between present values of the net annual benefits and of the capital and reposition costs or as the ratio between present values of the benefits and of the total costs, that is to say

$$B/C = \frac{R - O}{C + P} \quad (10.11)$$

$$B/C = \frac{R}{C+O+P} \quad (10.12)$$

The first definition seems more coherent as it combines together the annual monetary fluxes that occur during the project lifetime.

The B/C parameter has much popular appeal since it gives an immediate indication of the “degree” of desirability of a project. If the benefit/cost is less than one, the project is evidently undesirable. If it is exactly one the project has a marginal interest and if it is greater than one, its implementation would seem justified and as much as B/C is higher.

It should be pointed out that the B/C ratio evaluated by expression (10.12) is always equal or greater than the B/C ratio that results from expression (10.11). However, if (10.11) provides a unitary value, the B/C ratio given by (10.12) will also be one. An unitary B/C ratio implies a NPV equal to zero.

10.2.4.4-Internal Rate of Return (IRR)

The internal rate of return, IRR, is defined as the discount rate that makes the net present value, NPV, equal to zero.

Let us consider a project having capital costs, C_i , during the first k years and energy revenues, R_j , and annual operation costs, O_j , from year $j=(k+1)$ until year n . A reposition cost is foreseen for year m . In this condition, the project NPV for the beginning of year 1 can be evaluated according to

$$NPV = R - C - O - P = \frac{\sum_{j=k+1}^n \frac{1}{(1+r)^j} R_j}{(1+r)^k} - \sum_{i=1}^k \frac{1}{(1+r)^i} C_i - \frac{\sum_{j=k+1}^n \frac{1}{(1+r)^j} O_j}{(1+r)^k} - \frac{P_m}{(1+r)^m} \quad (10.13)$$

The respective IRR will be obtained by

$$NPV = \frac{\sum_{j=k+1}^n \frac{1}{(1+IRR)^j} (R_j - O_j)}{(1+IRR)^k} - \sum_{i=1}^k \frac{1}{(1+IRR)^i} C_i - \frac{P_m}{(1+IRR)^m} = 0 \quad (10.14)$$

A discount rate equal to IRR will imply an unitary B/C ratio and a null NPV.

A process of trial and error can provide the solution of expression (10.14). This process, which is generally simple to carry out with the software available nowadays, begins with an arbitrary discount rate. If the NPV thus obtained is positive, a higher discount rate must be tested; if NPV is negative, the search must proceed with a lower discount rate – Figure 10.5.

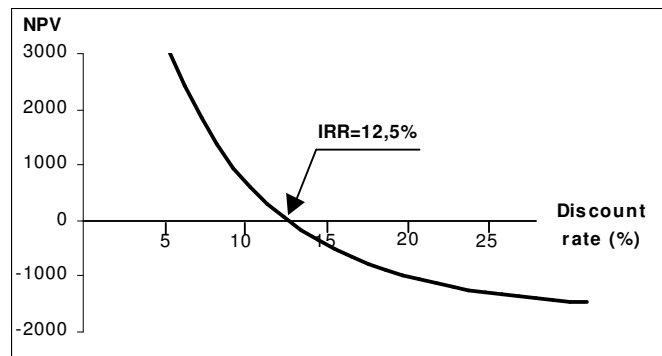


Figure 10.5 – Internal rate of return, IRR.

The previous figure assumes that the project is well behaved from an economic point of view. This is a correct assumption for small hydropower projects in which the capital costs occur during a very short period of time (generally, less than three years) while the incomes last during the useful life of the scheme (for saying, more than twenty years).

Among projects or alternative design solutions with different internal rates of return the best ones will be those with greater IRR. If the rates thus achieved are

greater than the opportunity costs of the capital, the projects are advantageous from an economic point of view.

10.2.4.5-Average price of the kWh (AP)

Although this parameter is not generally referred in expert literature, it is an interesting index as it provides economic information in a way very simple to understand. It is also useful in the comparison of alternative design solutions. According to the abbreviations established, the average price of the kWh, AP, is defined by

$$\begin{aligned}
 AP &= \frac{\sum_{i=1}^k \frac{1}{(1+r)^i} C_i + \frac{\sum_{j=k+1}^n \frac{1}{(1+r)^j} O_j}{(1+r)^k} + \frac{P_m}{(1+r)^m}}{\frac{(1+r)^{(n-k)} - 1}{(1+r)^{(n-k)} r} \bar{E}} = \\
 &= \frac{C + O + P}{\frac{(1+r)^{(n-k)} - 1}{(1+r)^{(n-k)} r} \bar{E}} \quad (10.15)
 \end{aligned}$$

This parameter represents the ratio between the present value of all costs and an “equivalent energy present amount”. This last amount is obtained by applying the present worth factor to the mean annual production. By other words, the AP index represents the unitary energy sale price that makes the NPV equal to zero and the B/C ratio equal to one.

If the AP price is lower than the nominal unitary energy-selling price the project is justified from an economic point of view. The best project will be the one with lower AP.

10.2.4.6-Payback period (T)

The payback period or recovery period, T, can be defined either on a discounted or without discount basis and represents the number of years it takes before

cumulative forecasted cash flows equal the initial investment. Its value is provided by the year when the cumulative cash flow changes from a negative value to a positive value.

10.2.5-Sensitivity analysis

A small hydropower scheme project is usually characterised by a lack of certainty about the capital cost estimates, future annual costs and future value of the energy. So a sensitivity analysis should be performed in order to analyse the project response capability to different scenarios. These scenarios can be obtained by increasing the costs and maintaining or decreasing the benefits or, if the cost estimates are considered to be accurate enough, maintaining the costs and decreasing the benefits. Variations from $\pm 10\%$ to $\pm 20\%$ are usually adopted in the previous analysis.

In what concerns the discount rate, at least two values should be considered around the most probable discount rate: a pessimist value higher than the expected one and a optimist value, lower than the expected discount rate.

10.2.6-Application example

Table 10.3 provides an application example for a hypothetical hydropower scheme having an installed capacity of 2,2 MW and an average energy production of 8,5 GWh.

The costs were considered to occur during the first two years (execution period) and the benefits during the next twenty-eight years (total duration of the legal licence period of 30 years). No residual costs, reposition costs and spare parts costs were considered. The economic analysis was developed for three discount rates: 10%, 8% and 6%. The first year is adopted for reference year.

The annual operation costs were estimate on the basis of fixed percentages either of the capital costs (exploitation and maintenance costs), or of the incomes (legal permit costs).

As one can conclude from the previous table, if the discount rate is set equal to the internal rate return then the net present value becomes zero, the benefit/cost ratio one, the average price of the kWh the nominal unitary sale price of the energy and the payback period the analysis period.

Table 10.3 – Application example.

ECONOMICAL ANALYSIS						
GENERAL CHARACTERISTICS						
Installed capacity (MW)	2,200					
Mean annual production (GWh)	8,500					
Cost per kW (EURO/kW)	1492273					
ECONOMIC ANALYSIS						
Analysis period (years)	30					
IRR (%)	13,369					
Discount rate (%)	0,100	0,080	0,060	0,050	0,13589	
NPV (EURO)	880175,3	1644455,4	2729947,8	0,0		
B/C (C/1 000 kW)	4,376	4,286	4,186	4,086	1,000	
Payback period (years)	49,039	42,086	35,621	62,500		
	(See discount cumulative cash flow)					
Year	Discount cumulative cash flow (K€)					
1	-89227,3	-908259,3	-826415,1	-86418,8		
2	-794380,2	-2881995,9	-2974296,9	-2672952,2		
3	-2435495,5	-2502742,5	-2573168,8	-2321914,2		
4	-2109129,8	-2151582,0	-2194746,0	-2034933,7		
5	-1812484,7	-1826433,4	-1837743,5	-1782285,9		
6	-152626,3	-1246607,9	-1246607,9	-1246607,9		
7	-129206,3	-988493,9	-883471,3	-1191663,7		
8	-107477,1	-749499,9	-600691,8	-1039500,2		
9	-872159,4	-528209,2	-33918,7	-906292,9		
10	-687966,1	-323310,4	-82245,9	-786669,6		
11	-520517,7	-133589,3	153184,2	-68518,2		
12	-368291,8	204733,6	379169,0	-59355,1		
13	-229904,6	42078,3	590478,3	-513698,2		
14	-104098,1	353540,3	78926,7	-43042,9		
15	11243,7	45391,0	11531,2	-35079,4		
16	11243,7	45391,0	11531,2	-35079,4		
17	204691,6	743468,5	1322667,3	-277869,8		
18	204691,6	956669,3	1480569,9	-236027,7		
19	373807,5	1051577,1	1629554,6	-198063,2		
20	443822,0	1139454,7	1702665,4	-165168,7		
21	508350,6	1220822,8	202739,1	-110714,9		
22	567070,3	1296163,7	2145733,1	-88701,9		
23	620424,5	1369923,7	2257048,2	-68101,9		
24	668928,4	1438016,3	236062,4	-5401,0		
25	713022,8	1491674,3	245459,7	3581,0		
26	753108,6	1536702,4	254154,4	22318,7		
27	789206,3	1574570,1	262456,7	10449,4		
28	822396,2	1604455,4	2729947,8	0,0		
29	852736,2	1627295,2				
30	880175,3	1644455,4				

COST AND BENEFITS						
	YEAR 1	YEAR 2	YEAR 3	YEAR 4	YEAR 5	YEAR 30
	CAPITAL COSTS (10 ⁶ EURO)					
1. Studies and design	74000	38000				
2. Site preparation during execution	25000	60000				
3. Civil works						
3.1. Weir and water intake	70000	140000				
3.2. Channel	69000	269000				
3.3. Forebay	60000	70000				
3.4. Penstock	18000	18000				
3.5. Powerhouse	17000	17000				
3.6. Access road	42500	17800				
4. Equipment	30000	60000				
4.1. Weir and water intake	25000	25000				
4.2. Forebay	270000	300000				
4.3. Penstock	200000	750000				
4.4. Powerhouse		400000				
5. Connection to the electrical grid	99000	50000				
6. B. site facilities, Contingencies	55000	55000				
7. Land acquisition	982000	2301000				
TOTAL (10⁶ EURO)						
ANNUAL OPERATION COSTS (10 ⁶ EURO)						
1. Exploitation			14000	14000	14000	14000
2. Maintenance			6500	6500	6500	6500
2.1. Civil works			25500	25500	25500	25500
2.2. Equipment			7500	7500	7500	7500
3. Legal permission						
TOTAL (10⁶ EURO)			53500	53500	53500	53500
ANNUAL BENEFITS						
1. Mean annual production (GWh)			8,5	8,5	8,5	8,5
2. Unitary sale price (EURO/1 000 kWh)			62,5	62,5	62,5	62,5
TOTAL (10⁶ EURO)			531250	531250	531250	531250

