Guide on How to Develop a Small Hydropower Plant




The present document is an updated version developed by the Thematic Network on Small hydropower (TNSHP) of the Layman's Guidebook on how to develop a small hydro site, by Celso Penche1998.

This Guide has been translated by the TNSHP to German, French, and Swedish

European Small Hydropower Association - ESHA - esha@arcadis.be
Tel. +32-2-546.19.45 - Fax +32-2-546.19.47
ESHA is founding member of EREC, the European Renewable Energy Council

INDEX

Acknowledgements	i
Executive Summary	ii
Chapter 1. Introduction	1
Chapter 2. Fundamental of Hydraulic Engineering	12
Chapter 3. Evaluating Stream Flow	42
Chapter 4. Site Evaluation Methodologies	71
Chapter 5. Hydraulic Structures	91
Chapter 6. Electromechanical Equipment	152
Chapter 7. Environmental impact and its mitigation	199
Chapter 8. Economic analysis	236
Chapter 9. Administrative procedures	254
Glossary	290

ACKNOWLEDGEMENTS

This Guide is an updated and adapted version of the publication "Layman's Guidebook on How to Develop a Small Hydro Site", published by ESHA - the European Small Hydropower Association – in 1998 in the frame of the European Commission DG-TREN (Directorate General for Transport and Energy) ALTENER programme.

Although based on the original, this guide has been entirely updated and adapted due to significant changes in the sector in the latest years as concern environmental and administrative aspects in particular. The updated version is available in English, French, German and Swedish what has added value to the already existing Spanish and Italian versions of the original publication.

The "Guide on how to develop a Small Hydro Site" has been carried out within the EC Project "Thematic Network on Small Hydropower", financed by the Fifth RD&D Framework Programme (FP5). It has been updated and adapted by a Revision Committee under the coordination and guidelines of ESHA. Members of the Revision Committee include the project partners Francis Armand (ADEME), Anton Schleiss (EPFL-LCH), Erik Bollaert (EPFL-LCH), Pedro Manso (EPFL-LCH), Jochen Bard (ISET), Jamie O'Nians (IT Power), Vincent Denis (MHyLab), Bernhard Pelikan (ÖVFK), Jean-Pierre Corbet (SCPTH), Christer Söderberg (SERO), Jonas Rundqvist (SERO) and Luigi Papetti (Studio Frosio). The network thanks Steve Cryer (BHA) for his input.

Special thanks to Celso Penche (ESHA), author of the Layman's Guide, who has revised the contents of the current Guide guaranteeing its consistency and accuracy.

ESHA 2004

EXECUTIVE SUMMARY

Developing a small hydropower site is not a simple task. There are many aspects which have to be taken into consideration, covering many disciplines ranging from business, engineering, financial, legal and administration. These will all be necessary at the different development stages from, first choosing a site until the plant goes into operation.

The "Laymans Guide" guide brings together all of these aspects in a step-by-step approach, and will serve as a useful tool for a potential developer of a small hydropower scheme.

This guide is divided into nine chapters and covers the basic concepts, meaning of definitions and technological issues to be addressed.

Chapter 1 – Introduces basic concepts, such as the definition of small hydropower, types of schemes, ways of exploiting the water resource available and gives a general overview of the guide's contents,

Chapters 2 through to 9 – describe the essential steps to be followed to evaluate a proposed scheme before deciding whether to proceed to a detailed feasibility study.

The basic concepts considered in the guide are:

- Topography and geomorphology of the site.
- Evaluation of the water resource and its generating potential.
- Site selection and basic layout.
- Hydraulic turbines and generators and their control.
- Environmental impact assessment and mitigation measures.
- Economic evaluation of the project and financing potential.
- Institutional framework and administrative procedures to obtain the necessary consents

Reading this guide will inform the potential small hydropower developer and give a better understanding of the different issues, phases and procedures that need be followed to develop and run a small hydropower operation.

Bernhard Pelikan

President ESHA

CHAPTER 1: INTRODUCTION

CONTENTS

1 INTRODUCTION 2
1.1 A free fuel resource potentially everlasting. 2
1.2 Definition of small hydropower 3
1.3 Site configurations 3
1.3.1 Run-of-river schemes 3
1.3.2 Schemes with the powerhouse at the base of a dam 5
1.3.3 Schemes integrated within an irrigation canal 7
1.3.4 Schemes integrated in a water abstraction system 8
1.4 Planning a small hydropower scheme 8

LIST OF FIGURES

Figure 1-1 High head scheme 4
Figure 1-2 Low head scheme with penstock 4
Figure 1-3 Low head scheme integrated in the dam 5
Figure 1-4 Low head scheme using an existing dam 6
Figure 1-5 Low head scheme – siphon intake 6
Figure 1-6 Integrated scheme using an irrigation canal 7
Figure 1-7 Elongated spillway scheme using an irrigation canal 7
Figure 1-8 Scheme integrated in a water supply system 8

1 INTRODUCTION[i]

1.1 A free fuel resource potentially everlasting.

Following the "Third Conference of the Parties to the United Nations Framework Convention on Climate Change" held in Kyoto in December 1997, the European Union has recognized the urgent need to tackle the climate change issue. It has also adopted a target to reduce greenhouse gas emissions by 8 % by 2010 from 1990 levels, whereas for other industrialised countries the target is 5 %.

To facilitate the Member States achieving this objective, the Commission identified a series of actions, focusing on reducing energy consumption and carbon emissions (CO2).

The development of energy from renewable resources is a very important step in the reduction of CO2 emissions. Therefore the EU Council and Parliament has brought forward Directive 2001/77/EC for the promotion of electricity produced from renewable energy resources

Electricity production from hydropower has been, and still is today, the first renewable source used to generate electricity. Nowadays hydropower electricity in the European
Union - both large and small scale – represents, according to the White Paper, 13% of the total electricity generated, so reducing the CO2 emissions by more than 67 million tons a year. But whereas the conventional hydro requires the flooding of large areas of land, with its consequential environmental and social issues, the properly designed small hydro schemes are easily integrated into local ecosystems.

In 2001, approximately 365 TWh of hydro energy was produced in the European Union from an overall capacity of 118 GW. Small hydro plants accounted for 8.4% of installed capacity (9.9 GW) and produced 39 TWh (about 11% of Hydropower generation). Given a more favorable regulatory environment, the European Commission objective of 14000 MW by 2010 should be achievable and that small hydro would be the second largest contributor behind windpower.

The large majority of small hydro plants are "run-of-river" schemes, meaning that they have no or relatively small water storage capability. The turbine only produces power when the water is available and provided by the river.

When the river flow falls below some predetermined value, the generation ceases. Some plants are stand alone systems used in isolated sites, but in most cases in Europe, the electricity generated is connected to the grid. Stand-alone, small, independent schemes may not always be able to supply energy, unless their size is such that they can operate whatever the flow in the river is. In some cases, this problem can be overcome by using any existing lakes or reservoir storage that exists upstream, of the plant.

The connection to the grid has the advantage of easier control of the electrical system frequency of the electricity, but has the disadvantage of being tripped off the system due to problems outside of the plant operator's control.

It is possible for grid connected systems to sell either all or some of their energy to supply company. (Note: this may not necessarily be the grid operator). However, the price paid for this energy is generally, in Europe particularly, fairly low. In recent years, supported by the RES-e Directive an in some cases National Government legislation enhanced payments are available for trading renewable energy states. This has helped small scale developments obtain a reasonable rate of return on the investment. It has also led to an increase in small scale hydro schemes being developed.

1.2 Definition of small hydropower

There is no consensus in EU member states on the definition of small hydropower: Some countries like Portugal, Spain, Ireland and now, Greece and Belgium, accept 10 MW as the upper limit for installed capacity. In Italy the limit is fixed at 3 MW (plants with larger installed power should sell their electricity at lower prices) and in Sweden 1.5 MW. In France the limit has been recently established at 12 MW, not as an explicit limit of SHP, but as the maximum value of installed power for which the grid has the obligation to buy electricity from renewable energy sources. In the UK 20MW is generally accepted as the threshold for small hydro. For the purposes of this text any scheme with an installed capacity of 10 MW or less will be considered as small. This figure is adopted by five member states, ESHA, the European Commission and UNIPEDE (International Union of Producers and Distributors of Electricity).

1.3 Site configurations

The objective of a hydropower scheme is to convert the potential energy of a mass of water, flowing in a stream with a certain fall to the turbine (termed the "head"), into electric energy at the lower end of the scheme, where the powerhouse is located. The power output from the scheme is proportional to the flow and to the head.

Schemes are generally classified according to the “Head”:-

- High head: 100-m and above
- Medium head: 30 - 100 m
- Low head: 2 - 30 m

These ranges are not rigid but are merely means of categorizing sites.

Schemes can also be defined as:-

- Run-of-river schemes
- Schemes with the powerhouse located at the base of a dam
- Schemes integrated on a canal or in a water supply pipe

1.3.1 Run-of-river schemes

Run-of-river schemes are where the turbine generates electricity as and when the water is available and provided by the river. When the river dries up and the flow falls below some predetermined amount or the minimum technical flow for the turbine, generation ceases.

Medium and high head schemes use weirs to divert water to the intake, it is then conveyed to the turbines via a pressure pipe or penstock. Penstocks are expensive and consequently this design is usually uneconomic. An alternative (figure 1.1) is to convey the water by a low-slope canal, running alongside the river to the pressure intake or forebay and then in a short penstock to the turbines. If the topography and morphology of the terrain does not permit the easy layout of a canal a low pressure pipe, can be an economical option. At the outlet of the turbines, the water is discharged to the river via a tailrace.

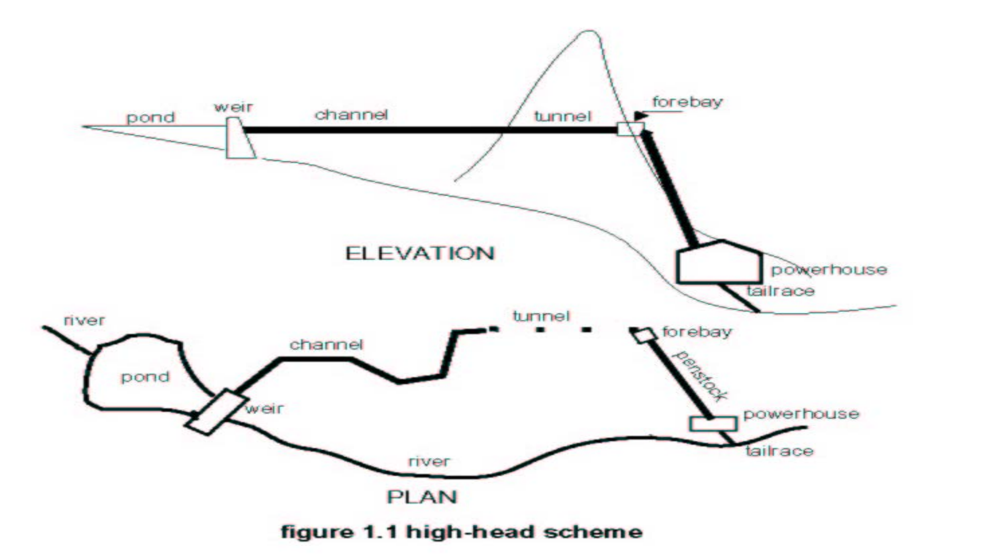


Figure 1-1 High head scheme

Occasionally a small reservoir, storing enough water to operate only on peak hours, when prices for electricity are higher, can be created by the weir, or a similarly sized pond can be built in the forebay.

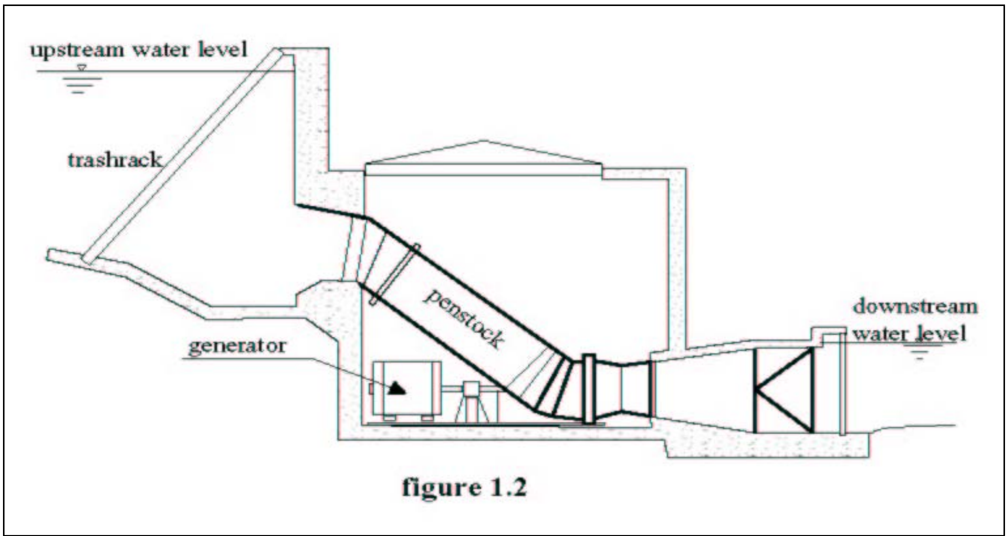


Figure 1-2 Low head scheme with penstock

Low head schemes are typically built in river valleys. Two technological options can be selected. Either the water is diverted to a power intake with a short penstock (figure 1.2), as in the high head schemes, or the head is created by a small dam, provided with sector gates and an integrated intake (figure 1.3), powerhouse and fish ladder.

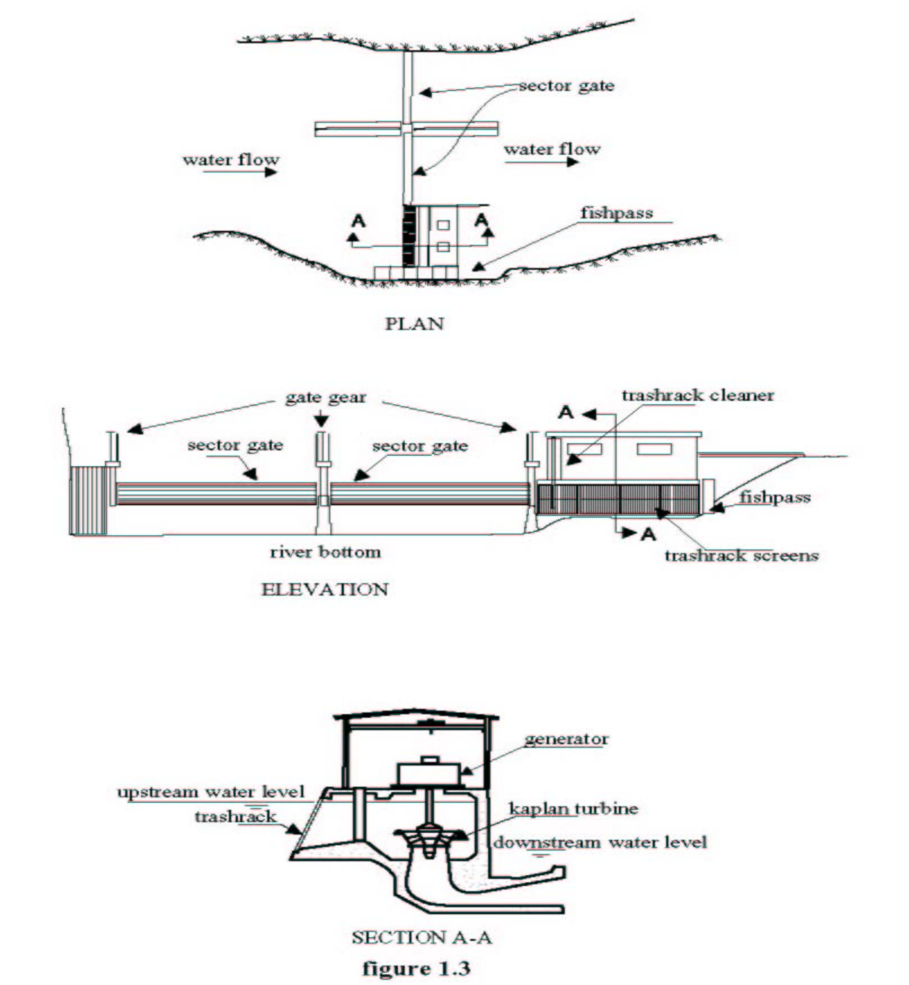


Figure 1-3 Low head scheme integrated in the dam

1.3.2 Schemes with the powerhouse at the base of a dam

A small hydropower scheme cannot afford a large reservoir to operate the plant when it is most convenient, the cost of a relatively large dam and its hydraulic appurtenances would be too high to make it economically viable. But if the reservoir has already been built for other purposes, such as flood control, irrigation, water abstraction for a big city, recreation area, etc, - it may be possible to generate electricity using the discharge compatible with its fundamental use or the ecological flow of the reservoir. The main issue is how to link headwater and tail water by a waterway and how to fit the turbine in this waterway. If the dam already has a bottom outlet, see figure 1.4, for a possible solution.

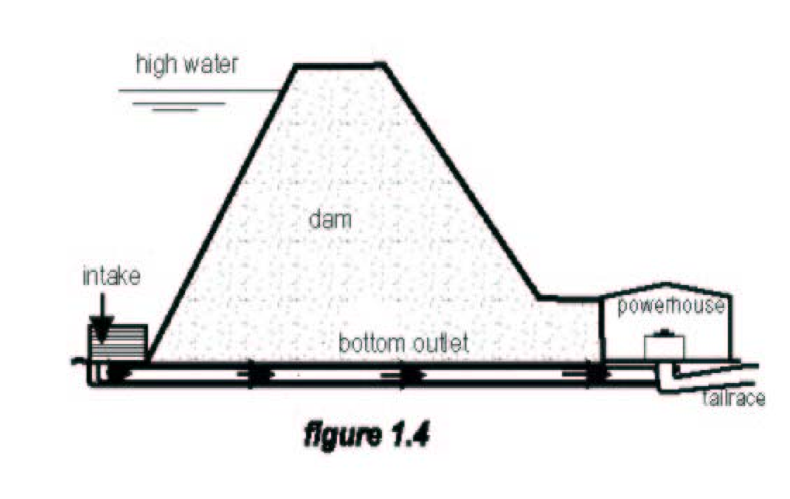


Figure 1-4 Low head scheme using an existing dam

Provided the dam is not too high, a siphon intake can be installed. Integral siphon intakes (figure 1.5) provide an elegant solution in schemes, generally, with heads up to 10 metres and for units up to about 1000 kW, although there are examples of siphon intakes with an installed power up to 11 MW (Sweden) and heads up to 30.5 meters (USA). The turbine can be located either on top of the dam or on the downstream side. The unit can be delivered pre-packaged from the works, and installed without major modifications to the dam.

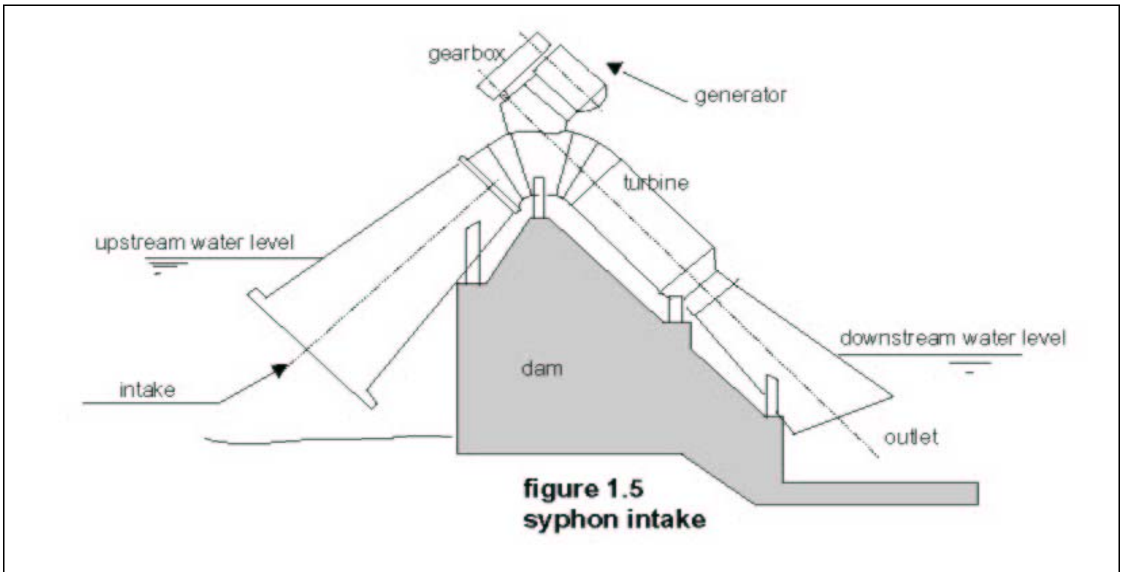


Figure 1-5 Low head scheme – siphon intake

1.3.3 Schemes integrated within an irrigation canal

Two types of schemes can be designed to exploit irrigation canal:-

- The canal is enlarged to accommodate the intake, the power station, thetailrace and the lateral bypass. Figure 1.6 shows a scheme of this kind, with a submerged powerhouse equipped with a right angle drive Kaplan turbine. To safeguard the water supply for irrigation, the scheme should include a lateral bypass, as in the figure, in case of shutdown of the turbine. This kind of scheme must be designed at the same time as the canal, as additional works whilst the canal is in full operation can be a very expensive option

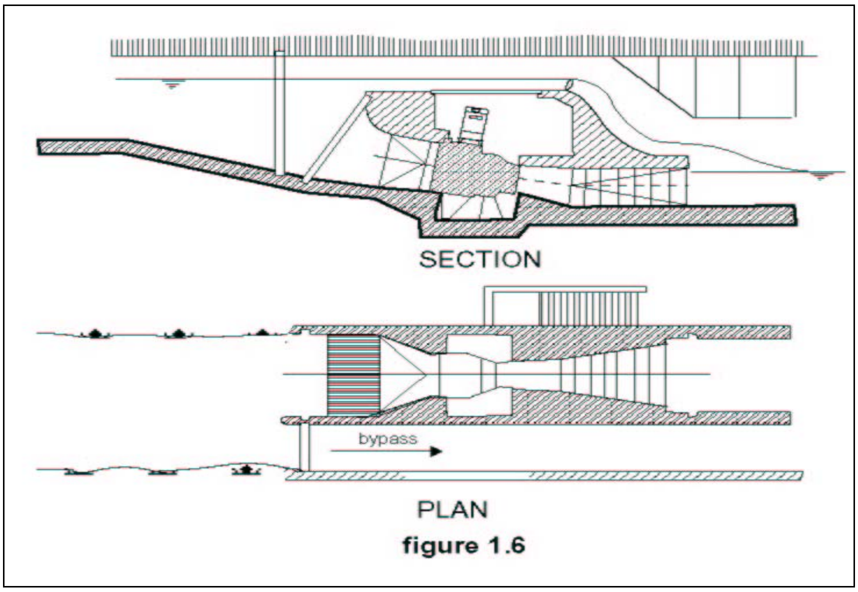


Figure 1-6 Integrated scheme using an irrigation canal

- If the canal already exists, a scheme like the one shown in figure 1.7 is a suitable option. The canal should be slightly enlarged to include the intake and the spillway. To reduce the width of the intake to a minimum, an elongated spillway should be installed. From the intake, a penstock running along the canal brings the water under pressure to the turbine. The water passes through the turbine and is returned to the river via a short tailrace.

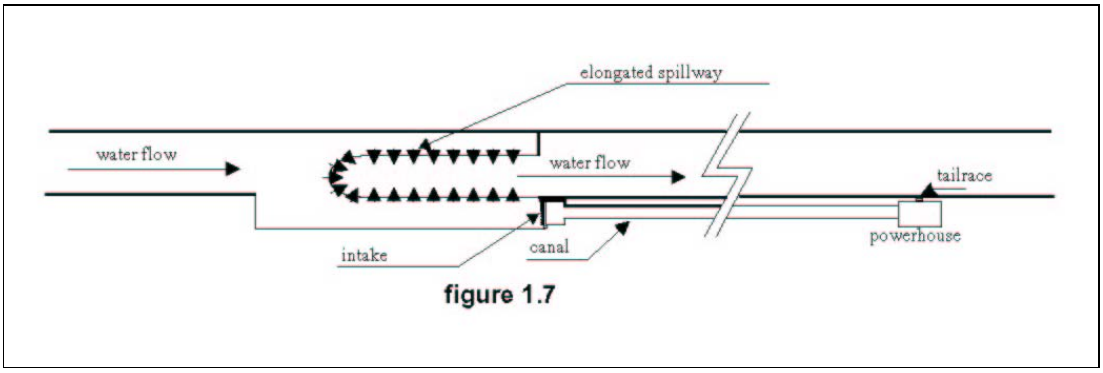


Figure 1-7 Elongated spillway scheme using an irrigation canal

Generally, migratory fish are not present in canals, fish passes are unnecessary.

1.3.4 Schemes integrated in a water abstraction system

The drinking water is supplied to a city by conveying the water from a headwater reservoir via a pressure pipe. Usually in this type of installation, the dissipation of energy at the lower end of the pipe at the entrance to the Water Treatment Plant is achieved through the use of special valves. The fitting of a turbine at the end of the pipe, to convert this otherwise lost energy to electricity, is an attractive option, provided that the water hammer phenomenon is avoided. Water hammer overpressures are especially critical when the turbine is fitted on an old pressure pipe.
To ensure the water supply at all times, a system of bypass valves should be installed. In some water supply systems the turbine discharges to an open-air pond. The control system maintains the level of the pond. In case mechanical shutdown or turbine failure, the bypass valve system can also maintain the level of the pond. Occasionally if the main bypass valve is out-of-operation and overpressure occurs, an ancillary bypass valve is rapidly opened by a counterweight. All the opening and closing of these valves must be slow enough to keep pressure variations within acceptable limits.
The control system has to be more complex in those systems where the turbine outlet is subject to the counter-pressure of the network, as is shown in figure 1.8.

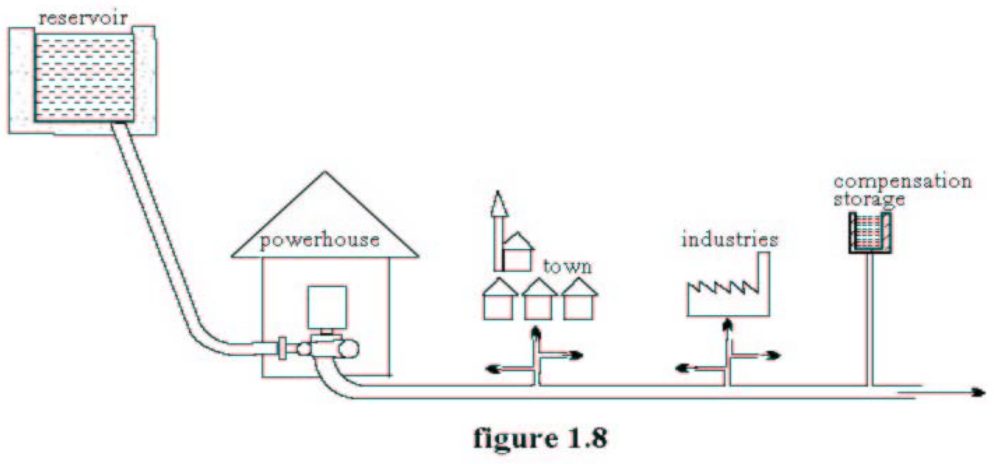


Figure 1-8 Scheme integrated in a water supply system

1.4 Planning a small hydropower scheme

The definitive project or scheme comes as the result of a complex and iterative process, where consideration is given to the environmental impact and different technological options. These are then costed and an economic evaluation carried out.

Although it is not easy to provide a detailed guide on how to evaluate a scheme, it is possible to describe the fundamental steps to be followed, before deciding if one should proceed to a detailed feasibility study or not. A list of the studies that should be undertaken:-

- Topography and geomorphology of the site.
- Evaluation of the water resource and its generating potential
- Site selection and basic layout
- Hydraulic turbines and generators and their control
- Environmental impact assessment and mitigation measures
- Economic evaluation of the project and financing potential
- Institutional framework and administrative procedures to attain the necessary consents

The water flowing along natural and man-made canals, conducted by low and high-pressure pipes, spilling over weir crests and moving the turbines involves the application of fundamental engineering principles in fluid mechanics. In Chapter 2 those principles are reviewed together with shortcuts arising from the experience accumulated from centuries of hydraulic systems construction.

To decide if a scheme will be viable it is necessary to begin by evaluating the water resource existing at the site. The energy potential of the scheme is proportional to the product of the flow and the head. Except for very low heads, the gross head can usually be considered as constant, but the flow varies over the year. To select the most appropriate hydraulic equipment and estimate the sites potential with calculations of the annual energy output, a flow-duration curve is most useful. A single measurement of instantaneous flow in a stream has little value.

Measuring the gross head requires a topographical survey. The results obtained, by using a surveyor's level and staff is accurate enough, but the recent advances in electronic surveying equipment make the topographical surveying work much simpler and faster. To produce a flow-duration curve on a gauged site is easier than producing a curve at an ungauged site. This requires a deeper understanding of hydrology. In Chapter 3 various methods for measuring the quantity of water flowing in a stream are analysed and hydrological models to calculate the flow regime at ungauged sites are discussed.

Chapter 4 presents techniques such as orthophotography, RES, GIS, geomorphology, geotectonics, etc - used nowadays for site evaluation. Some failures are also analysed and conclusions about how they might have been avoided are explained.

In Chapter 5 the basic layouts are explained and the hydraulic structures, such as weirs, canals, spillways, intakes and penstocks, studied in detail.

Chapter 6 deals with the electromechanical equipment used to convert the potential energy of the mass of water to electricity. Turbines themselves are not studied in detail, but attention is focused on turbine configurations, specifically for low head schemes, and on the process of turbine selection, with emphasis on specific speed criteria. Since small hydro schemes are usually operated unattended, the control systems, based on personal computers, are also reviewed.

An Environmental Impact Assessment may be required to obtain the necessary consents to build the scheme and utilize the water available. Although several recent studies have shown that small hydropower produce no emissions to atmosphere, nor do they produce toxic wastes, does not contribute to climatic change, designers should implement all necessary measures to mitigate local ecological impacts. Chapter 7 analyses those impacts and mitigating measures.

Chapter 8 reviews techniques for an economical evaluation of a scheme. Various methodologies of economic analysis are described and illustrated with tables showing the cash flows generated by the schemes.

In Chapter 9, the administrative procedures a developer will have to go through are presented. Unfortunately the recent deregulation of much of the electricity industry in the EU has made it difficult to establish a common procedure to follow.

A few years ago ESHA produced (December 1994) on behalf of the E.C. DGXVII, a report "Small Hydropower. General Framework for Legislation and Authorisation Procedures in the European Union", and though it is not current it still has many valid aspects. The report can be found in www.esha.be, the ESHA web page.

Further important considerations for the developer to take into account are trading tariffs for green and base energy and administrative procedures, for grid connection. These depend on the energy policy and the institutional framework of each country. An overview has been provided in the Appendix A of Chapter 9.

---

[i] By Celso Penche (ESHA), Francis Armand (ADEME), Vincent Dennis (MhyLab) and Christer Söderberg (SERO)

CHAPTER 2: FUNDAMENTALS OF HYDRAULIC ENGINEERING

CONTENTS

2. FUNDAMENTALS OF HYDRAULIC ENGINEERING 13
2.1. Introduction 13
2.2. Water flow in pipes 13
2.2.1. Head losses due to friction 16
2.2.2. Local head losses 23
2.2.2 Transient flow 28
2.3. Water flow in open channels 31
2.3.1. Classification of open channel flows 31
2.3.2. Uniform flow in open channels 32
2.3.3. Efficient cross-section in open channels 33
2.3.4. Principles of energy in open channel flows 33
Bibliography 40

LIST OF FIGURES

Figure 2-1Velocity distribution for laminar and turbulent flow 14
Figure 2-2 Hydraulic gradient and energy gradient 16
Figure 2-3 μ as a function of Reynolds number 20
Figure 2-4 Loss coefficients for trash racks 24
Figure 2-5 Kc and Kex values as a function of d/D 25
Figure 2-6 Diffuser coefficients 26
Figure 2-7 Entrance loss coefficients 27
Figure 2-8 Loss coefficients for flow in bends 27
Figure 2-9 Typical loss coefficients for flow through valves 28
Figure 2-10 Typical velocity distributions for open channel flow 31
Figure 2-11 Illustration of various types of varied flow 32
Figure 2-12 Pressure distribution for channels with vertically curved bed 34
Figure 2-13 Specific energy as a function of flow depth 36
Figure 2-14 Moody's Chart: Friction factors for pipe flow 39
Figure 2-15 Illustration of pressure wave in pipes 39

LIST OF TABLES

Table 2-1 Roughness height "e", for various commercial pipes 17
Table 2-2 Manning coefficient n for several commercial pipes 21
Table 2-3 Hazen-Williams coefficients 23
Table 2-4 Additional trash rack losses for non-perpendicular approach flows 24
Table 2-5 Geometrical characteristics of different channel profiles 36
Table 2-6 Empirical formulae used to estimate y_c, in typical channel. 37

2. FUNDAMENTALS OF HYDRAULIC ENGINEERING[i]

2.1. Introduction

Hydraulic engineering is based on the principles of fluid mechanics, although many empirical relationships are applied to achieve practical engineering solutions. Until now there does not exist and probably never will, a general methodology for the mathematical analysis of the movement of fluids. Based on the experience accumulated, over many years of study and practice, there are particular solutions to specific problems. Experience that goes back 2500 years, when a massive irrigation system, that is still operative, was built in Sichuan, China, and to the many aqueducts built during the period of the Roman Empire

In hydropower, hydraulic engineering is applied to:

- .Optimise the performance of waterways to reduce energy losses
- .Design spillways and structure for floods prevention
- .Design adequate energy dissipation works downstream of spillways
- .Control erosion and manage silt transportation

Control phenomena such as:

- Instability in waterways due to dynamic effects
- Air entrance into closed conduits
- Surges in long waterways
- Surge pressures in closed conduits
- Cavitation of structures and equipment
- Prevent reservoir sedimentation, intake obstruction and sediment related damage to the hydraulic circuit and the equipment

In order to successfully develop small hydropower a thorough understanding of the principles of hydraulics is required.

In this chapter, the fundamentals of hydraulic engineering are explained together with an explanation of some of the phenomena mentioned above.

2.2. Water flow in pipes

A body of water will have a potential energy by virtue of its velocity and the vertical height through which it drops, (as a difference in water levels is what drives the flow of water), which is known as its “head”. This energy is its “Gravitational Potential Energy” which is product of mass, acceleration due to the effects of gravity and head m.g.h and is generally expressed in Joules (J)

The energy head in the water flowing in a closed conduit of circular cross section, under a certain pressure, is given by Bernoulli's equation:

$$H_1 = h_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} \qquad (2.1)$$

Where:

H_1 is the total energy head

h_1 is the elevation above some specified datum plane,

P_1 the pressure

γ the specific weight of water

V_1 the velocity of the water, and

g the gravitational acceleration.

The total energy head at point 1 is then the algebraic sum of the potential energy h_1, the pressure energy P_1/γ, and the kinetic energy $V_1^2/2g$, commonly known as the "Velocity head".

For an open channel, the same equation applies, but with the term P_1/γ replaced by d_1, the water depth.

If water is allowed to flow very slowly in a long, straight, glass pipe of small bore into which a fine stream of coloured water is introduced at the entrance to the pipe, the coloured water would appear as a straight line all along the pipe. This effect is known as laminar flow. The water flows in lamina (layers), like a series of thin walled concentric pipes. The outer virtual pipe adheres to the wall of the real pipe, while each of the inner ones moves at a slightly higher speed, which reaches a maximum value near the centre of the pipe. The velocity distribution has the form of a parabola and the average velocity (figure 2.1) is 50% of the maximum centre line velocity.

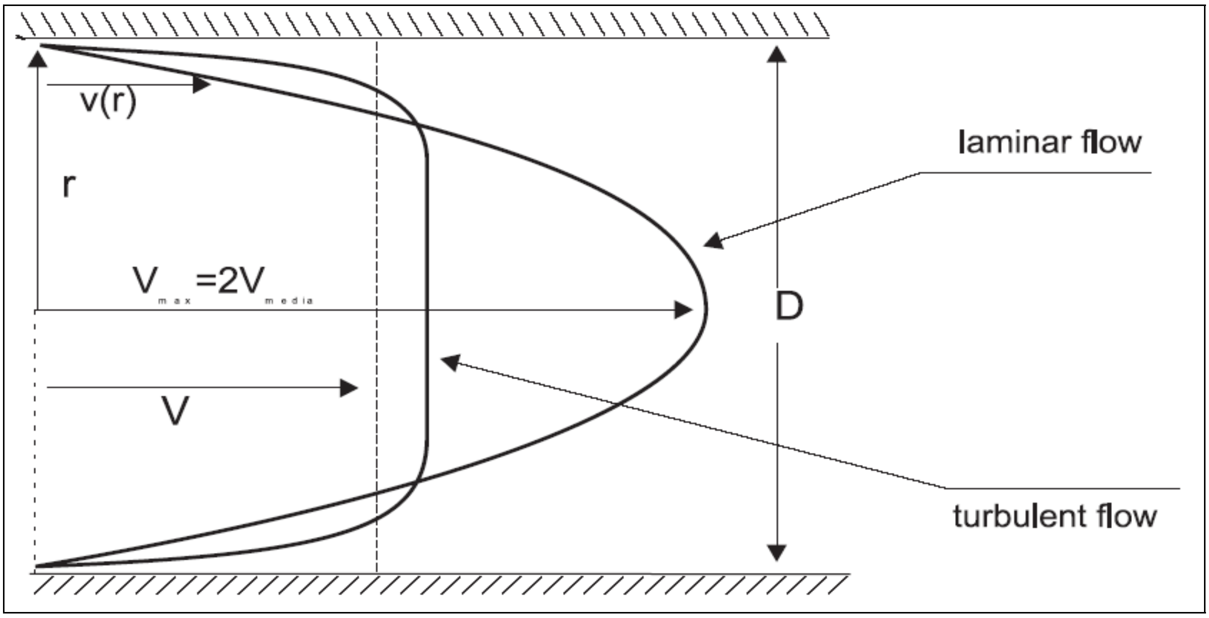


Figure 2-1Velocity distribution for laminar and turbulent flow

If the flow rate is gradually increased, a point is reached when the lamina flow suddenly breaks up and mixes with the surrounding water. The particles close to the wall mix up with the ones in the midstream, moving at a higher speed, and slow them. At that moment the flow becomes turbulent, and the velocity distribution curve is much flatter. Experiments carried out by Osborne Reynolds, near the end of the 19th century, found that the transition from laminar flow to turbulent flow depends, not only on the velocity, but also on the pipe diameter and on the viscosity of the fluid, and is a ratio of the inertia force to the viscous force. This ratio, is known the Reynolds number and can be expressed, in the case of a circular pipe, by the equation:-

$$R_e = \frac{D \cdot V}{\nu} \qquad (2.2)$$

where:

D (m) is the pipe diameter

V is the average water velocity (m/s), and

ν is the kinematics viscosity of the fluid (m^2/s).

From experimentation it has been found that for flows in circular pipes the critical Reynolds number is about 2000. In fact this transition does not always happen at exactly R_e=2000 but varies with the conditions. Therefore there is more than a transition point, what exists is a transition range.

Example 2.1

A 60-mm diameter circular pipe carries water at 20°C. Calculate the largest flow-rate for which the flow would be laminar.

The kinematics viscosity of water at 20°C is u = 1 x 10^{-6} m^2/s.

Assuming a conservative value for R_e = 2 000

V = 2 000 / (10^6x0.06) = 0.033 m/s

Q = AV = π /4x 0.06^2 x 0.033 = 3.73 x 10^{-4} m^3/s = 0.373 l/s

Water loses energy as it flows through a pipe, fundamentally due to:

1. friction against the pipe wall

2. viscous dissipation as a consequence of the internal friction of flow

The friction against the pipe wall depends on the wall material roughness and the velocity gradient nearest to the wall. Velocity gradient, as can be seen in figure 2.1, is higher in turbulent flow than in laminar flow. Therefore, as the Reynolds number increases, the friction loss will also increase. At the same time, at higher turbulences there is more intensive mixing of particles, and hence a higher viscous dissipation. Consequently the energy losses in flow in the pipe increase with the Reynolds number and with the wall pipe roughness. It can be verified that for water flowing between two sections, a certain amount of the head of energy h_f is lost:-

$$\frac{V_1^2}{2_g} + \frac{P_1}{\gamma} + h_1 = \frac{V_2^2}{2_g} + \frac{P_2}{\gamma} + h_2 + h_f \qquad (2.3)$$

Due firstly, to the friction of the water against the pipe wall, and secondly, to the internal friction of the flow. In figure 2.2, HGL is the hydraulic gradient line and EGL the energy gradient line. If the pipe cross-section is constant, V_1 = V_2 and both lines will be parallel. It is therefore necessary to determine the value of h_f ?

2.2.1. Head losses due to friction

Darcy and Weisbach, applying the principle of conservation of mass to a certain volume of fluid in a pipe, between two sections perpendicular to its axis - derived the following equation, valid for incompressible and steady flows, through pipes:

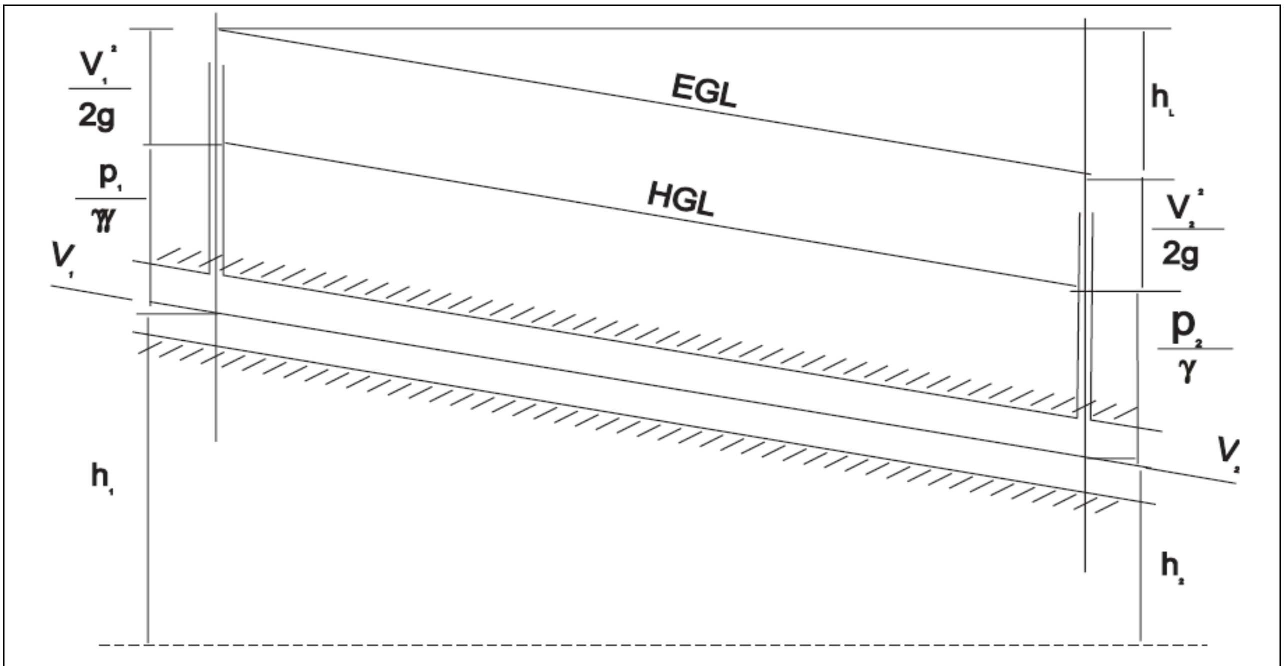


Figure 2-2 Hydraulic gradient and energy gradient

$$h_f = f \cdot \left(\frac{L}{D}\right) \cdot \frac{V^2}{2g}$$
(2.4)

where

f = friction factor, a dimensionless number
L = the length of the pipe in m
D = the pipe diameter in m
V = the average velocity in m/s, and
g = the gravitational acceleration (9.81 m/s2).

In a laminar flow f can be calculated directly by the equation:

$$f = \frac{64 \cdot \nu}{V \cdot D} = \frac{64}{R_e}$$
(2.5)

According to equation (2.5) the friction factor f in a laminar flow is independent of the wall roughness and inversely proportional to the Reynolds number. The fact that, apparently, f decreases when R_e increases, does not mean that increasing the velocity decreases the friction losses.

Substituting f in equation (2.4) by its value in (2.5), gives:-

$$h_f = \frac{64 \cdot \nu}{V \cdot D} \cdot \frac{L}{D} \cdot \frac{V^2}{2g} = \frac{32 \cdot \nu \cdot L \cdot V}{g \cdot D^2}$$
(2.6)

This shows that the specific head loss, in laminar flow, is proportional to V and inversely proportional to D^2.

When the flow is practically turbulent (R_e>2000), the friction factor become less dependent on the Reynolds number and more dependent on the relative roughness height e/D, where "e" represents the average roughness height of irregularities on the pipe wall and D the pipe diameter. Some values of the roughness height "e" are provided in table 2.1.

Table 2-1 Roughness height "e", for various commercial pipes

Pipe material	e (mm)
Polyethylene	0.003
Fiberglass with epoxy	0.003
Seamless commercial steel (new)	0.025
Seamless commercial steel (light rust)	0.250
Seamless commercial steel (galvanised)	0.150
Welded steel	0.600
Cast iron (enamel coated)	0.120
Asbestos cement	0.025
Wood stave	0.600
Concrete (steel forms, with smooth joints)	0.180

It is well known that, even in turbulent flows, immediately next to the wall pipe there exists, a very thin layer of flow referred to as the laminar sub layer. When R_e increases, the sub layer's thickness diminishes. Whenever the roughness height "e" is resolutely lower than the sub layer thickness the pipe is considered hydraulically smooth.

In a hydraulically smooth pipe flow, the friction factor f is not affected by the surface roughness of the pipe, and for this case Von Karman, developed the following equation for the friction factor f:

$$\frac{1}{\sqrt{f}} = 2 \cdot \log_{10}\left(\frac{R_e\sqrt{f}}{2.51}\right) \qquad (2.7)$$

At high Reynolds numbers, the sub layer thickness becomes very small and the friction factor f becomes independent of R_e and depends only on the relative roughness height. In this case the pipe is a hydraulically rough pipe, and Von Karman found that the friction factor f:

$$\frac{1}{\sqrt{f}} = 2 \cdot \log_{10}\left(3.7\frac{D}{e}\right) \qquad (2.8)$$

In between these two extreme cases, the pipe behaves neither completely smooth nor completely rough, for this situation, Colebrook and White devised the following equation:

$$\frac{1}{\sqrt{f}} = -2 \cdot \log_{10}\left(\frac{e/D}{3.7} + \frac{2.51}{R_e\sqrt{f}}\right) \qquad (2.9a)$$

Which can be expressed in terms of the average velocity U by:-

$$U = -2\sqrt{2gD\,{}^{h_f}\!/_{L}}\ \log\left(\frac{{}^{e}\!/_{D}}{3.7} + \frac{2.51\nu}{D\sqrt{2gD\,{}^{h_f}\!/_{L}}}\right) \qquad (2.9b)$$

Formulae 2.7 and 2.9 are difficult to solve by hand, prompting Moody to prepare his well-known chart "Friction factors for pipe flow" (figure 2.15).

Looking to the chart it shows four different flow zones:

1. A laminar flow zone (shaded area in the diagram) where f is a linear function of R (equation 2.5)
2. A badly defined critical zone (shaded area)
3. A transition zone, starting with the smooth pipes (equation 2.7) and finishing in the dashed line where, in between, f depends both of R_e and e/D (equation 2.9a)
4. A developed turbulence zone where f depends exclusively of e/D (equation 2.8)

Example 2.2

Calculate, using the Moody chart, the friction loss in a 900-mm diameter welded steel pipe along a length of 500 m, conveying a flow of 2.3 m^3/s.

The average water velocity is $4Q/(\pi D^2) = 1.886$ m/s

From the table 2.1,

e = 0.6 mm and therefore e/D = 0.6/900 = 0.000617

$R_eN_R = DV / u = (0.9 \times 1.886)/ 1.31 = 1.3 \times 10^6$ ($u = 1.31\ 10^{-6}$)
In the Moody chart for e/D = 0.00062 and $R_e = 1.3*10^6$ we find f=0.019
From equation (2.4):

$$h_f = 0.019 \cdot \frac{500}{0.9} \cdot \frac{1.886^2}{2 \cdot 9.81} = 1.91m$$

In engineering practice the Colebrook-White formula (2.9) and the Moody diagram can be used to solve the following typical problems with flows in closed pipes:

1. Given U (or Q), D and e, compute h_f;
2. Given U (or Q), h_f and e, compute D;
3. Given D, h_f and e, compute U (or Q);
4. Given U (or Q), D, h_f, compute e.

Problems in 3and 4 above can be solved directly by using formula (2.9b), whereas the remainder problems require an iterative solution. The Moody's diagram provides a direct solution for the 1st and 4th problems. Alternatively, if you want to know what the maximum water velocity flowing in a pipe of diameter D and length L, without surpassing a friction head loss hf you only need to use an independent variable μ:

$$\mu = \frac{1}{2} f R_e^2$$
(2.10)

Substituting R_e by its value in (2.2) and f by its value in (2.4) becomes:-

$$\mu = \frac{gD^3 h_f}{L\nu^2}$$
(2.11)

where all the parameters are known. Once μ is computed, f is derived from (2.10) and substituted in (2.9) to attain:

$$R_e = -2\sqrt{2\mu}\,\log_{10}\left(\frac{e/D}{3.7} + \frac{2.51}{\sqrt{2\mu}}\right)$$
(2.12)

An equation that makes it possible to plot the R_e with respect to U for different values of e/D, is shown in figure 2.3, a variation of the Moody Chart where R_e can be estimated directly.

Example 2.3

Estimate the flow rate of water at 10oC that will cause a friction head loss of 2m per km in a welded steel pipe, 1.5 m in diameter.

Substitute values in equation (2.12), with e/D=0.6/1500 = 4x104,

After computing U .

$$\mu = \frac{9.81 \cdot 1.5^3 \cdot 2}{1000 \cdot \left(1.31 \cdot 10^{-6}\right)^2} = 3.86 \cdot 10^{10}$$

$$R_e = -2\sqrt{2 \cdot 3.86 \cdot 10^{10}}\,\log_{10}\left(\frac{4 \cdot 10^{-4}}{3.7} + \frac{2.51}{\sqrt{2 \cdot 3.86 \cdot 10^{10}}}\right) = 2.19 \cdot 10^6$$

$$V = \frac{R_e \cdot \nu}{D} = \frac{2.19 \cdot 10^6 \cdot 1.31 \cdot 10^{-6}}{1.5} = 1.913 \text{ m/s; } Q=V\cdot A=3.38 \text{ m}^3/\text{s}$$

Also based on the Colebrook-White equation there exists some other monographs, to compute the friction head loss on a pipe, given a certain flow, a certain pipe diameter, with a certain roughness coefficient such as the one shown in the next page and published by courtesy of Hydraulic Research, Wallingford U.K.

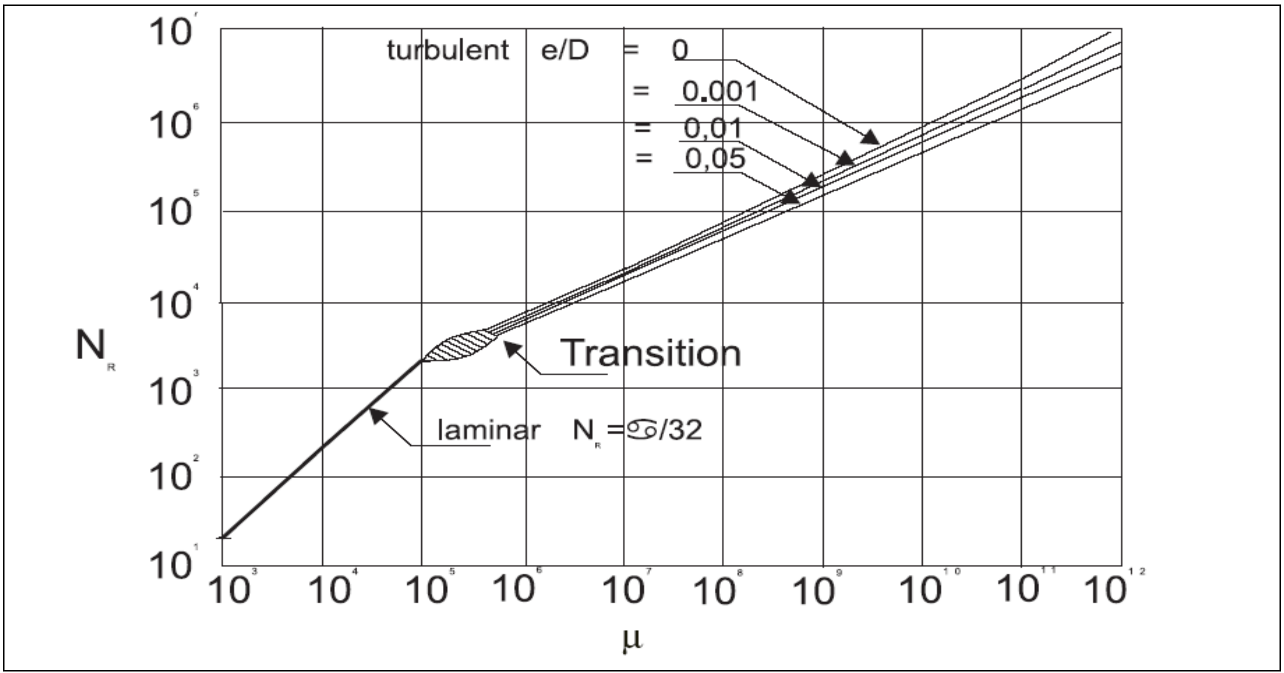


Figure 2-3 μ as a function of Reynolds number

Empirical formulae

Over the years many empirical formulae, based on accumulated experience, have been developed. They are, generally, not based on sound physics principles and even, occasionally, lack dimensional coherence, but are intuitively based on the belief that the friction on a closed full pipe is:

1. Independent of the water pressure
2. Linearly proportional to its length
3. Inversely proportional to a certain power of its diameter

4. Proportional to a certain exponent of the water velocity

In turbulent flows it is influenced by the wall roughness

One of these formulae, widely used to estimate the flow in open channels, but also applicable to closed pipes, is that developed by Manning (resp. Strickler):

$$Q = \frac{1}{n} \cdot \frac{A^{5/3} S^{1/2}}{P^{2/3}} \qquad (2.13)$$

Where: n is the Manning roughness coefficient (s/m$^{1/3}$, $K_{Strickler}$=1/n)
P is the wetted perimeter (m)
A is cross-sectional area of the pipe (m^2), and
S is the hydraulic gradient or head loss by linear meter (h_f/L).
Applying the above formulae to a full closed circular cross section pipe:

$$S = \frac{10.29 \cdot n^2 \cdot Q^2}{D^{5.333}} \qquad (2.14)$$

$$S = \frac{4^{10/3} n^2 Q^2}{\pi^2 D^{16/3}} \qquad (2.14a)$$

In Table 2.2 the Manning coefficient n for several commercial pipes is shown:

Table 2-2 Manning coefficient n for several commercial pipes

Kind of pipe	n
Welded steel	0.012
Polyethylene (PE)	0.009
PVC	0.009
Asbestos cement	0.011
Ductile iron	0.015
Cast iron	0.014
Wood-stave (new)	0.012
Concrete (steel forms smooth finish)	0.014

In example 2.4 and more specifically in example 2.5 the results obtained by applying the Colebrook-White equation and the Manning formulae can be compared.

Example 2.4

Using the parameters in example 2.2 compute the friction head loss applying the Manning formulae

Accepting n=0.012 for welded steel pipe

$$\frac{h_f}{L} = \frac{10.29 \cdot 0.012^2 \cdot 1.2^2}{0.9^{5.333}} = 0.00374$$

Whereby for L=500 m, h_f=1.87 m, slightly inferior to the value estimated with the Moody chart.

Example 2.5

Compute, using the Colebrook equation and the Manning formulae, the friction head loss on a welded pipe 500 m long, of respectively 500 mm, 800 mm, 1 200 mm, and 1 500 mm diameter respectively, under a 4 m/s average flow velocity.

D (mm)	500	800	1200	1500
Q (m*3*/s)	0.785	2.011	4.524	7.069
V (m/s)	4	4	4	4
L (m)	500	500	500	500

Applying Colebrook-White

e (mm)	0.6	0.6	0.6	0.6
h_f(m)	17.23	9.53	5.73	4.35

Applying Manning

n	0.012	0.012	0.012	0.012
h_f(m)	18.40	9.85	5.73	4.26

It can be observed that the solutions provided by the Manning formula do not differ much from those offered by the Colebrook equation, except in the smaller diameters, where the head loss provided by Manning is higher than that provided by Colebrook. In fact, both formulae agree for values of e/D=9.17E-3 and provide results within a 5 % range for values of e/D between 9E-4 and 5E-2 in the turbulent (rough) zone (Dubois, 1998). In this range of flows, the relation between the Darcy-Weisbach and Manning's coefficients is:

$$S = \tfrac{f}{D}\frac{U^2}{2g};\; f = \frac{2g.4^{4/3}n^2}{D^{1/3}} \qquad (2.14b)$$

In North America for pipes larger than 5 cm diameter and flow velocities under 3 m/s the Hazen-Williams formulae is typically used:

$$h_f = \frac{6.87 \cdot L}{D^{1.165}} \left(\frac{V}{C} \right)^{1.85} \tag{2.15}$$

Where V is the flow velocity (m/s), D the diameter (m), L the pipe length (m) and C the Hazen-Williams coefficient such as shown in Table 2.3.

Table 2-3 Hazen-Williams coefficients

Pipe type	C
Asbestos cement	140
Cast iron	
New	130
10 years	107 - 113
20 years	89 - 100
30 years	75 - 90
Concrete	
Cast on site - steel forms	140
Cast on site - wood forms	120
Centrifugal cast	135
Steel	
Brush tar and asphalt	150
New uncoated	150
Riveted	110
Wood-stave (new)	120
Plastic pipes	135 - 140

2.2.2. Local head losses

In addition to friction losses, water flowing through a pipe systems experience head losses due to geometric changes at entrances, bends, elbows, joints, racks, valves and at sudden contractions or enlargements of the pipe section. This loss also depends on the velocity and is expressed by an experimental coefficient K multiplied by the kinetic energy v2/2g.

2.2.1.1 Trash rack (or screen) losses

A screen is nearly always required at the entrance of both pressure pipes and intakes to avoid the entrance of floating debris. The flow of water through the rack also gives rise to a head loss. Though usually small, it can be calculated by a formula developed by Kirschmer:

$$h_t = Kt \left(\frac{t}{b} \right)^{4/3} \left(\frac{V_0^2}{2g} \right) sin\Phi$$

(2.16)

where the parameters are identified in figure 2.4.

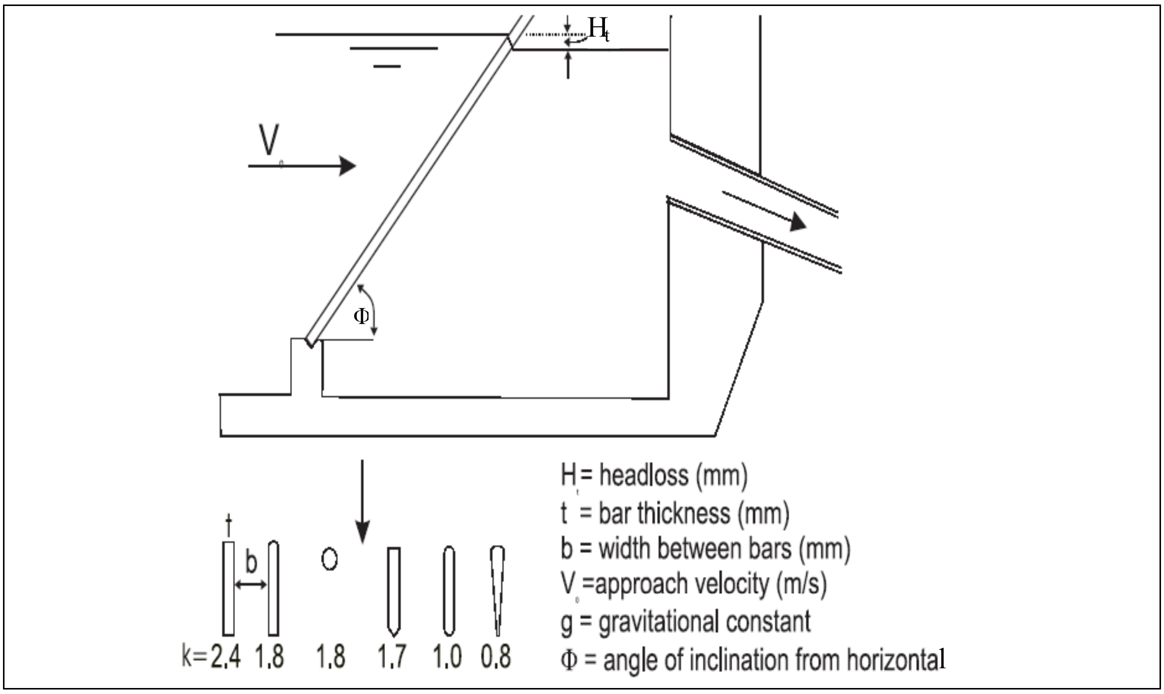


Figure 2-4 Loss coefficients for trash racks

For structural reasons, this formula is only valid if the length L of the bars is smaller than 5 times their diameter. If the grill is not perpendicular but makes an angle β with the water flow (β will have a maximum value of 90^0 for a grill located in the sidewall of a canal), there will be an additional head loss. The result of equation 2.16 should be multiplied by a correction factor κ provided in the table 2.4 (according to Mosonyi).

Table 2-4 Additional trash rack losses for non-perpendicular approach flows

t/b β	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2
0°	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
10°	1.06	1.07	1.08	1.09	1.10	1.11	1.12	1.14	1.50
20°	1.14	1.16	1.18	1.21	1.24	1.26	1.31	1.43	2.25
30°	1.25	1.28	1.31	1.35	1.44	1.50	1.64	1.90	3.60
40°	1.43	1.48	1.55	1.64	1.75	1.88	2.10	2.56	5.70
50°	1.75	1.85	1.96	2.10	2.30	2.60	3.00	3.80	…
60°	2.25	2.41	2.62	2.90	3.26	3.74	4.40	6.05	…

2.2.1.2 Loss of head by sudden contraction or expansion

When the pipe has a sudden contraction there is a loss of head due to the increase in velocity of the water flow and to the large-scale turbulence generated by the change of geometry. The flow path is so complex that, at least for the time being, it is impossible to provide a mathematical analysis of L the phenomenon. The head loss is estimated by multiplying the kinetic energy in the smaller pipe (section 2), by a coefficient K_c that varies with the ratio of contraction d/D:

$$h_c = K_c \cdot \left(\frac{V_2^2}{2g}\right)$$
(2.17)

For a ratio up to d/D = 0.76, K_c approximately follows the formula:-

$$k_c = 0,42\left(1 - \frac{d^2}{D^2}\right)$$
(2.18)

The ratio, K_c is substituted by K_{ex}, the coefficient used for a sudden expansion.

In sudden expansions, the loss of head can be derived from the momentum of flow and is given by:

$$h_{ex} = \frac{(V_1 - V_2)^2}{2g} = \left(1 - \frac{V_2}{V_1}\right)^2 \frac{V_1^2}{2g} = \left(1 - \frac{A_1}{A_2}\right)^2 \frac{V_1^2}{2g} = \left(1 - \frac{d^2}{D^2}\right)\frac{V_1^2}{2g}$$
(2.19)

where V_1 is the water velocity in the smaller pipe.

Figure 2.5 is a graphic representation of the K_c and K_{ex} values as a function o f d/D.

The head loss can be reduced by using a gradual pipe transition, known as a confuser for contraction, or diffuser for expansion.

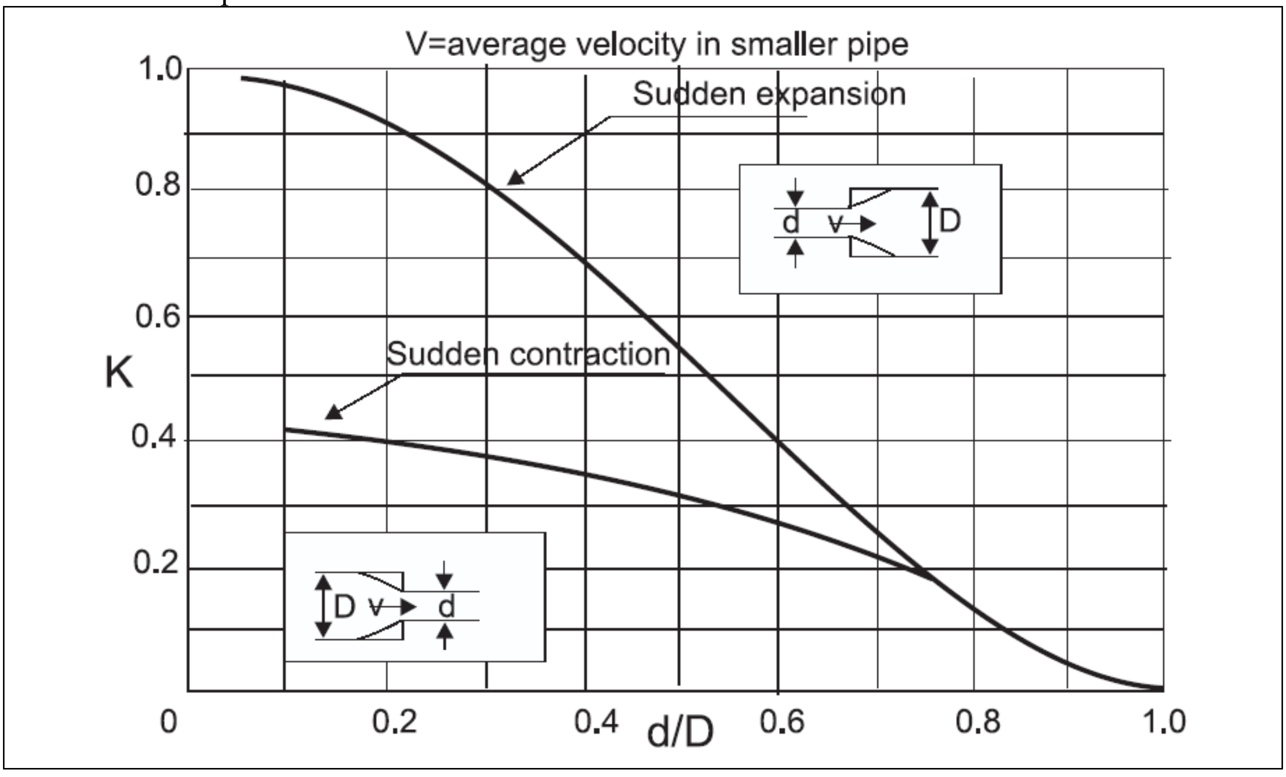


Figure 2-5 Kc and Kex values as a function of d/D

In the confuser the head loss varies with the confuser angle as it is shown in the table below where K_c values are experimental:

Angle	K_c
30^0	0.02
45^0	0.04
60^0	0.07

In the diffuser the analysis of the phenomenon is more complex. Figure 2.6 shows the experimentally found values of K_{ex} for different diffuser angles. The head loss is given by:

$$h'_{ex} = K'_{ex} \frac{V_1^2 - V_2^2}{2g} \tag{2.20}$$

A submerged pipe discharging in a reservoir is an extreme case of a sudden expansion, where V_2, given the size of the reservoir, compared with the pipe, can be considered as zero, and the loss$V_1^2/2g$.

On the other hand, the entrance from a reservoir to a pipe is an extreme case of a sudden contraction. Figure 2.7 shows the value of the K_e coefficient that multiplies the kinetic energy $V_2/2g$ in the pipe.

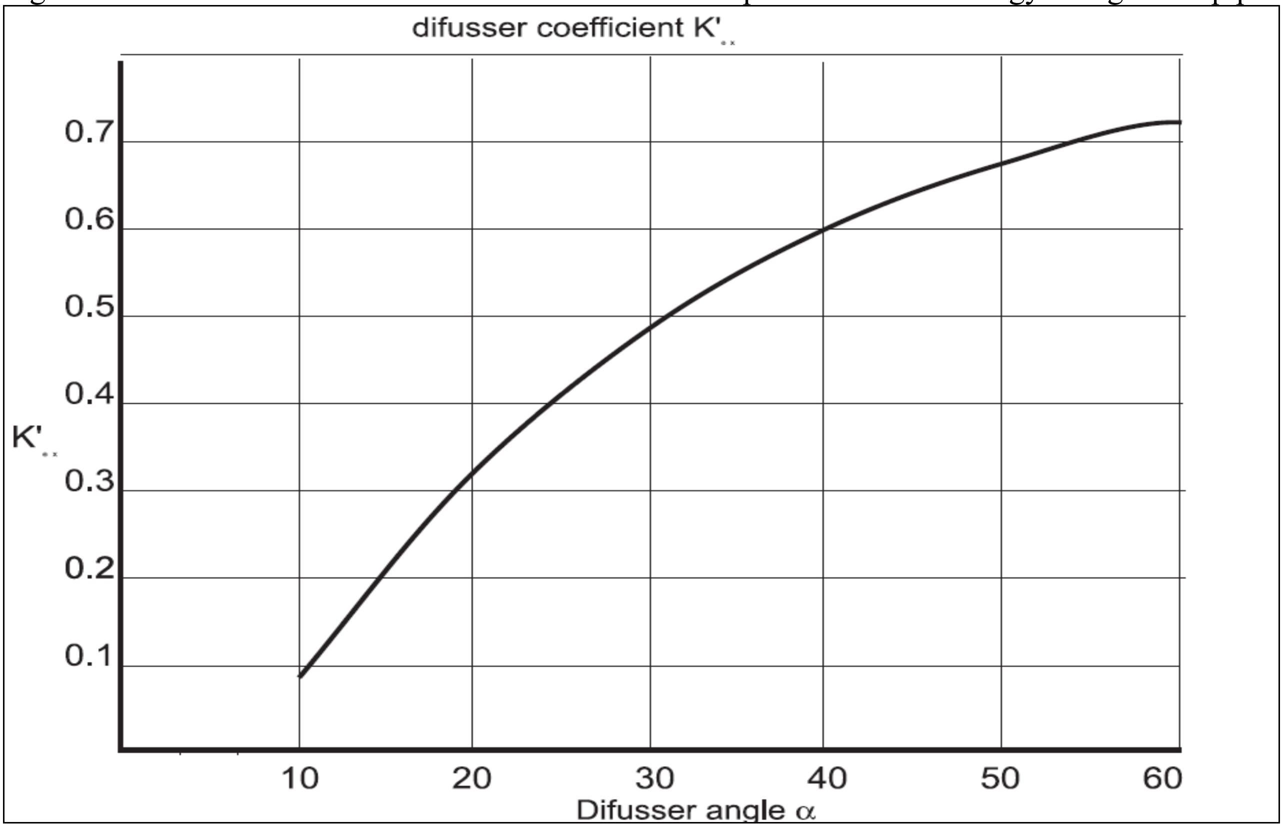


Figure 2-6 Diffuser coefficients

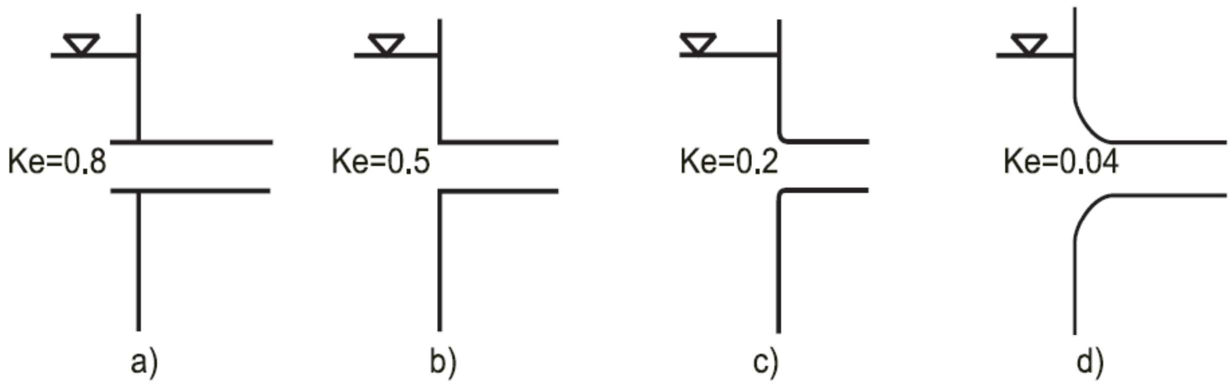


Figure 2-7 Entrance loss coefficients

2.2.1.3 Loss of head in bends

In a bend, pipe flow experiences an increase of pressure along the outer wall and a decrease of pressure along the inner wall. This pressure unbalance causes a secondary current such as shown in the figure 2.10. Both movements together - the longitudinal flow and the secondary current - produces a spiral flow that, at a length of around 100 diameters, is dissipated by viscous friction. The head loss produced in these circumstances depends on the radius of the bend and on the diameter of the pipe. Furthermore, in view of the secondary circulation, there is a secondary friction loss, dependent of the relative roughness e/D.

Figure 2.8, taken from reference 3 gives the value of K_b for different values of the ratio R/D and various relative roughness e/D. There is also a general agreement that, in seamless steel pipes, the loss in bends with angles under 90^o, is almost proportional to the bend angle. The problem is extremely complex when successive bends are placed one after another, close enough to prevent the flow from becoming stabilized at the end of the bend. Fortunately, this is hardly ever the case in a small hydro scheme.

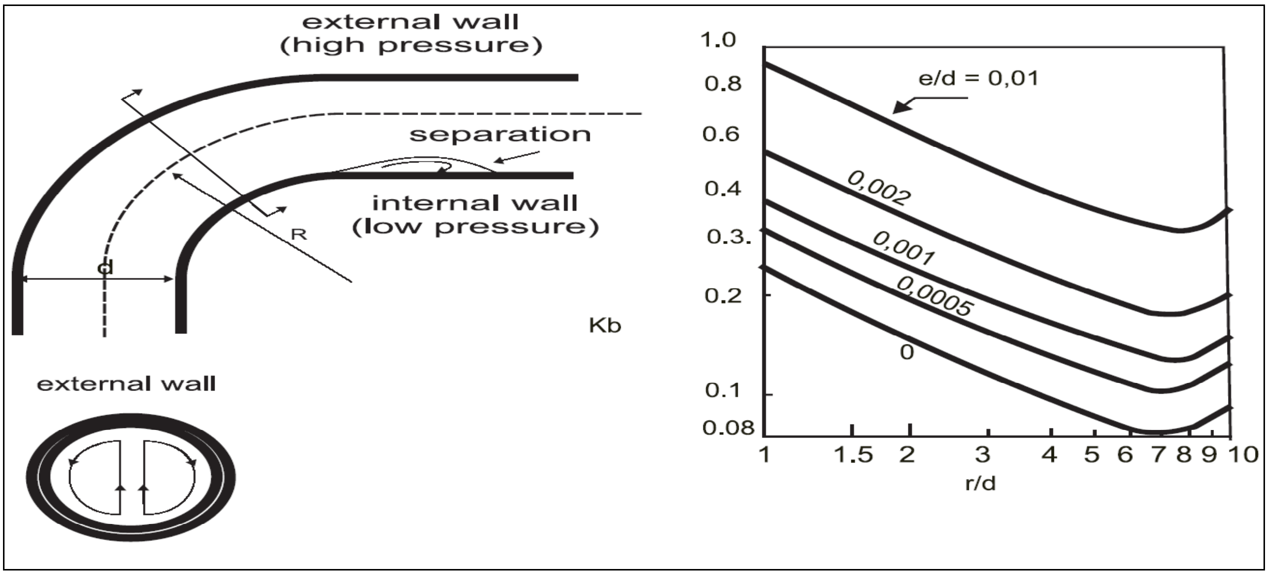


Figure 2-8 Loss coefficients for flow in bends

2.2.1.4 Loss of head through valves

Valves or gates are used in small hydro schemes to isolate a component from the rest, so they are either entirely closed or entirely open. Flow regulation is assigned to the distributor vanes or to the needle valves of the turbine. The loss of head produced by water flowing through an open valve depends of the type and manufacture of the valve. Figure 2.9 shows the value of K_v for different kind of valves.

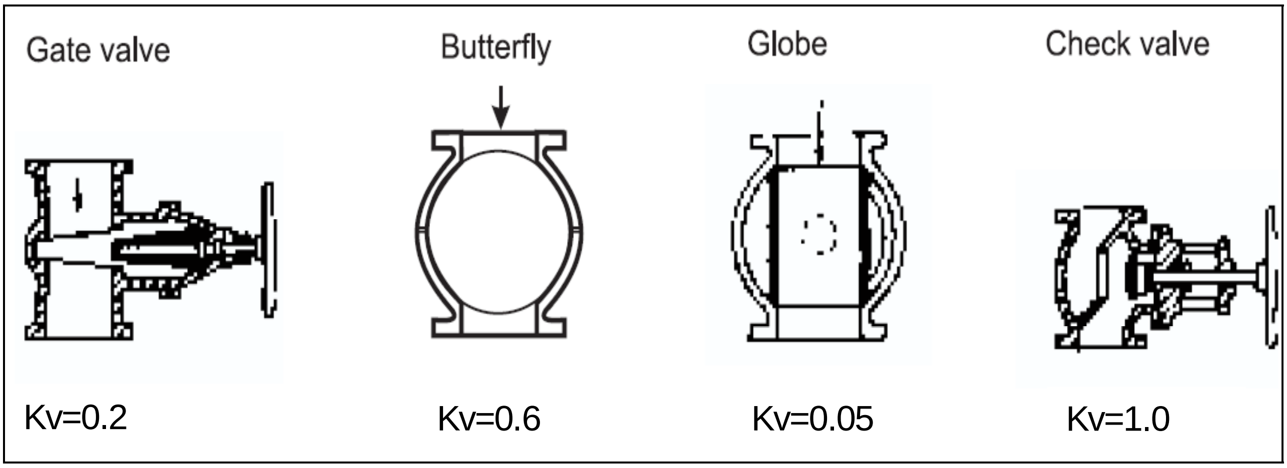


Figure 2-9 Typical loss coefficients for flow through valves

2.2.2 <u>Transient flow</u>

In steady flows where the discharge is assumed to remain constant with time, the operating pressure at any point along a penstock is equivalent to the head of water above that point. If a sudden change of flow occurs, for instance when the plant operator, or the governor system, open or close the gates too rapidly, the sudden change in the water velocity can cause dangerous high and low pressures.

This pressure wave is known as water hammer, or surge, and its effects can be dramatic. The penstock can burst from overpressure or collapse if the pressures are reduced below atmospheric. Although being transitionary the surge pressure induced by the “water hammer phenomenon” can be of a magnitude several times greater than the static pressure due to the head. According to Newton's second law of motion, the force developed in the penstock, by the sudden change in velocity, will be:

$$F = m\frac{dV}{dt} \qquad (2.21)$$

If the velocity of the water column could be reduced to zero the resulting force would become infinite. Fortunately this is not possible in practice; a mechanical valve requires some time for total closure and the pipe walls are not perfectly rigid and the water column under large pressures is not incompressible.

The following description, reproduced with the permission of the author, Allen R. Inversin from Appendix F of his "Micro-Hydropower Sourcebook", is one of the best physical explanations of this phenomenon. Figure 2.16, enclosed at the end of this chapter, illustrates how a velocity change, caused by an instantaneous closure of a gate, or valve, at the end of a pipe creates a pressure wave that travels the length of the pipe.

Initially, water flows at a velocity (Vo) as shown in (a). When the gate is closed, the water flowing within the pipe has a tendency to continue flowing due to its momentum. Because this momentum is physically stopped by the gate closing, it “piles up” behind it, the kinetic energy of the element of water nearest the gate is converted to pressure energy, which slightly compresses the water and expands the circumference of the pipe at this point (b). This action is repeated by the following elements of water (c), and the wave front of increased pressure travels the length of the pipe until the velocity of the water Vo is destroyed, the water is compressed, and the pipe is expanded over its entire length (d). At this point, the water's kinetic energy has all been converted to strain energy (under increased compression) and strain energy of the pipe (under increased tension).

Because the water in the reservoir remains under normal static pressure but the water in the pipe is now under a higher pressure, the flow reverses and is forced back into the reservoir again with velocity V_o (e). As the water under compression starts flowing back, the pressure in the pipe is reduced to normal static pressure. A pressure “unloading” wave then travels down the pipe toward the gate (f) until all the strain energy is converted back into kinetic energy (g). However, unlike case (a), the water is now flowing in the opposite direction and because of its momentum the water again tries to maintain this velocity. In so doing, it stretches the element of water nearest the gate, reducing the pressure there and contracting the pipe (h). This happens with successive elements of water and a negative pressure wave propagates back to the reservoir (i) until the entire pipe is under

compression and water under reduced pressure (j). This negative pressure wave would have the same absolute magnitude as the initial positive pressure wave if it were assumed that friction losses do not exist. The velocity then returns to zero but the lower pressure in the pipe compared to that in the reservoir forces water to flow back into the pipe (k). The pressure surge travels back toward the gate (e) until the entire cycle is complete and a second cycle commences (b). The velocity with which the pressure front moves is a function of the speed of sound in water modified by the elastic characteristics of the pipe material. In reality, the penstock pipe is usually inclined but the effect remains the same, with the surge pressure at each point along the pipe adding to or subtracting from the static pressure at that point. Also, the damping effect of friction within the pipe causes the kinetic energy of the flow to dissipate gradually and the amplitude of the pressure oscillations to decrease with time. Although some valves close almost instantaneously, closure usually takes at least several seconds. Still, if the valve is closed before the initial pressure surge returns to the gate end of the pipeline (g), the pressure peak will remain unchanged - all the kinetic energy contained in the water near the gate will eventually be converted to strain energy and result in the same peak pressure as if the gate were closed instantaneously. However, if the gate has been closed only partially, by the time the initial pressure surge returns to the gate (g), not all the kinetic energy will have been converted to strain energy and the pressure peak will be lower. If the gate then continues closing, the positive pressure surge, which it would then create, will be reduced somewhat by the negative pressure (h) surge which originated when the gate originally began closing. Consequently, if the gate opens or closes in more time than that required for the pressure surge to travel to the reservoir and back to the gate, peak surge pressures are reduced. This time is called the critical time, Tc, and is equal to:

$$T_c = 2L / c \qquad (2.22)$$

where c is the wave velocity. The wave velocity, or speed of sound, in water is approximately 1420 m/s. However, the wave velocity in a pipe - the speed with which the pressure surge travels along the pipe - is a function of both the elastic characteristics of water and the pipe material. An expression for the wave velocity is:

$$c = \sqrt{\frac{k / \rho}{1 + \frac{k \cdot D}{E \cdot t}}} \qquad (2.23)$$

where K = bulk modulus of water, $2.2x10^9$ N/m^2

ρ = density of water, 1 000 kg/m^3

D = internal pipe diameter (m)

E = modulus of elasticity of pipe material (N/m^2)

t = wall thickness (mm)

If the valve is already closed, when the pressure wave is on its way back ($t<T_c$) all the kinetic energy of the water will be converted on an overpressure, and its value in meters of water column is:-

$$\frac{\Delta P}{\rho g} = \frac{c}{g} \Delta V \qquad (2.24)$$

where .V is the change of water velocity. In practical cases, .V can be assumed equal to the initial flow velocity V_0. However, if t is greater than T_c, then the pressure wave reaches the valve before the valve is completely closed, and the overpressure will not develop fully, because the reflected negative wave arriving at the valve will compensate for the pressure rise. In this case the maximum overpressure may be calculated by the simplified Allievi formula, also known as the Michaud formula:-

$$\frac{\Delta P}{\rho g} = \frac{2L}{gt} \Delta V \qquad (2.25)$$

where L = total pipe length (m)

$\Delta P/\rho g$ = pressure difference between the initial static pressure $P_0/\rho g$ and the maximum pressure attained in the conduit (m column of water) t = closure time (s).

The total dynamic pressure experienced by the penstock will thus be:

$P = P_o +$ .P (2.26)

In chapter 5, several examples related to penstock design will clarify the above physics concepts. For a more rigorous approach it would be necessary to take into consideration not only the elasticity of the fluid and pipe material above, but also the hydraulic losses. The mathematical approach is rather cumbersome and requires the use of computers. For interested readers Chaudry, Fox and Parmakian, among others, give calculation methods, together with some worked examples.

2.3. Water flow in open channels

In closed pipes the water fills the entire pipe, in an open canal there is always a free surface. Normally, the free water surface is subject to the atmospheric pressure, commonly referred to as the zero pressure reference, and usually considered as constant along the full length of the canal. In a way this fact, by dropping the pressure term, facilitates the analysis, but at the same time introduces a new dilemma. The depth of water changes with the flow conditions, and in unsteady flows its estimation is a part of the problem. Any kind of canal, even a straight one, has a three-dimensional distribution of velocities. A well-established principle in fluid mechanics is that any particle in contact with a solid stationary border has a zero velocity. Figure 2.10 illustrates the iso-velocity lines in channels of different profile. The mathematical approach is based on the theory of the boundary layer; the engineering approach is to deal with the average velocity V.

2.3.1. Classification of open channel flows

A channel flow is considered steady when the depth at any section of the stretch does not change with time, and unsteady if it changes with time. An open channel flow is said to be uniform if the discharge and the water depth at every section of a channel length does not change with time. Accordingly, it is said to be varied whenever the discharge and/or the water depth changes along its length. Non uniform flow is a rare occurrence, and with uniform flow, steady uniform flow is understood to occur. Steady variable flow is often stated as gradual or rapid.

Figure 2.11 represents different kinds of flows: steady uniform flow, steady gradually variable flow, and steady rapidly variable flow. Unsteady flow occurs if either the flow depth, or the discharge, over the length of the canal, changes as, for instance, in the case of upstream propagation of a small perturbation wave due to closure or opening of a valve, or in the case of the discharge increase in a collector channel.

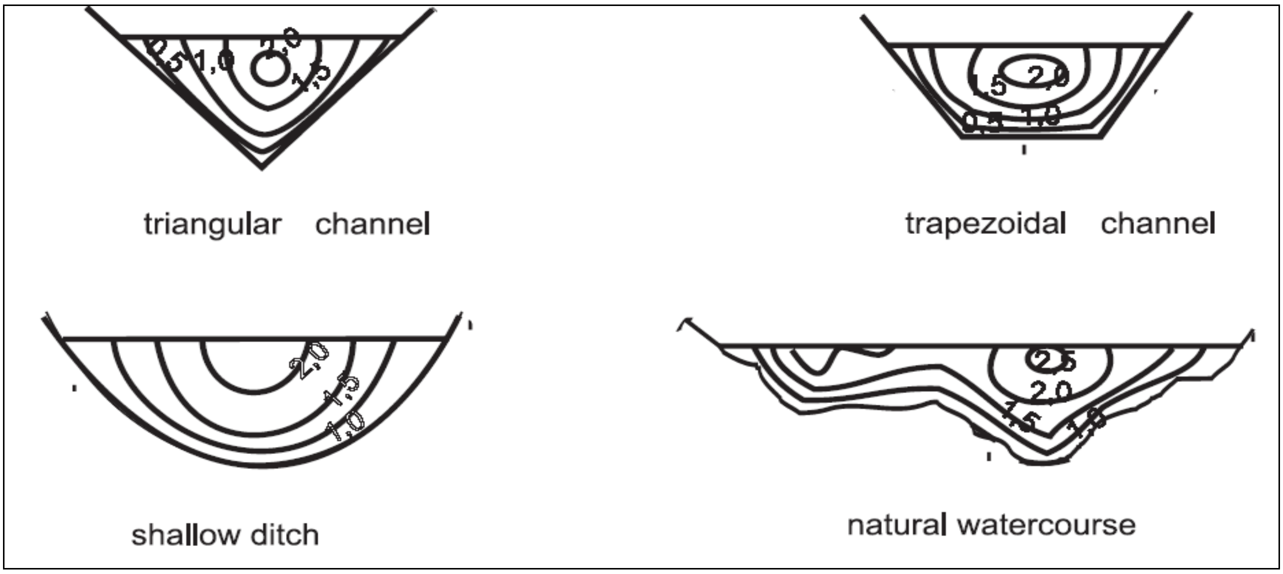


Figure 2-10 Typical velocity distributions for open channel flow

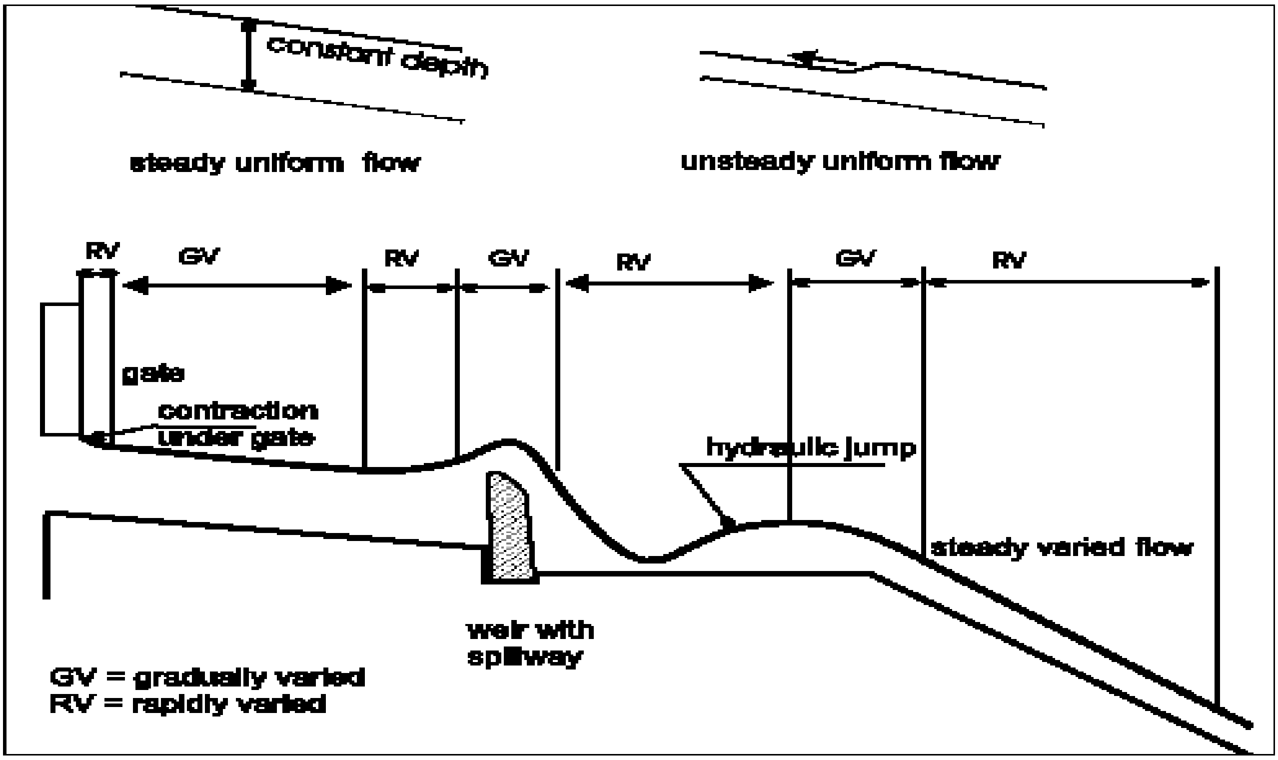


Figure 2-11 Illustration of various types of varied flow

As with the analysis of fully closed pipe flows, channel flows also follow Bernoulli's equation and consequently formula (2.1) is valid. The amount of energy loss when water flows from section 1 to section 2 is indicated by h_L.

2.3.2. Uniform flow in open channels

By definition a flow is considered uniform when:-

1. The water depth, water area, and the velocity in every cross section of the channel are constant.
2. The energy gradient line, the free surface line and the bottom channel line are parallel to each other.

Based on these concepts Chezy found that:-

$$V = C\sqrt{Ri} \tag{2.27}$$

where:-

C = Chezy's resistance factor
R_h = Hydraulic radius of the channel cross-section
S_e = Channel bottom line slope
Many attempts had been made to determine the value of C. Manning, using the results of his own experiments and those of others, derived the following empirical relation:

$$C = \frac{1}{n} R^{1/6} \tag{2.28}$$

where n is the well-known Manning's roughness coefficient (see Chapter 5, Table 5.1). Substituting C from (2.27) into (2.28) we have the Manning formula for uniform flows:

$$V = \frac{1}{n} R^{1/3} i^{1/2} \tag{2.29}$$

or alternatively:-

$$Q = \frac{1}{n} A \cdot R^{2/3} i^{1/2} \tag{2.30}$$

The parameter $ARh^{2/3}$ has been defined as the section factor and is given, for various channel sections, in table 2.5. The formula is entirely empirical and the n coefficient is not dimensionless, so the formulae given here are only valid in S.I. units. Furthermore the formulae are only applicable to channels with a flat bottom. The analysis of natural watercourses is more complex and the above formulae can only be applied for first approximations.

2.3.3. Efficient cross-section in open channels

From (2.32) it may be deduced that for a channel with a certain cross-section area A and a given slope S, the discharge increases by increasing the hydraulic radius. That means the hydraulic radius is an efficiency index. As the hydraulic radius is the quotient of the area A and the wetted perimeter P, the most efficient section will be the one with the minimum wetted perimeter. Among all cross-sectional areas, the semicircle is the one, which has the minimum wetted perimeter for a given area. Unfortunately such a channel, with a semicircular cross section is expensive to build and difficult to maintain, and so is only used in small section channels built with prefabricated elements. Putting aside the semicircular section, the most efficient trapezoidal section is a half hexagon. The most commonly used channel section in small hydro schemes is the rectangular section, easy to build, waterproof and maintain. In chapter 5 the selection of the channel section is considered from the construction viewpoint, balancing efficiency, land excavation volumes, construction methods, etc.

2.3.4. Principles of energy in open channel flows

Uniform flows in open channels are mostly steady, and unsteady uniform flows are rather rare. If the flow lines are parallel and we take the free surface of the water as the reference plane, the summation of the elevation energy "h" and the pressure energy P/γ is constant and equal to the water depth. In practice, most of the uniform flows and a large part of the steady varied flows can be considered parallel to the bottom.

On a channel with a constant slope less than 6° (figure 2.12a), the pressure head at any submerged point is equal to the vertical distance measured from the free surface to that point (depth of water). The stress distribution is typically triangular. Nevertheless if the water is flowing over a convex path, such as a spillway, the centrifugal flow acts in an opposite direction to the gravity, and the stress distribution is distorted and looks like figure 2.12b. The pressure energy is given by the difference between the depth and the centrifugal acceleration of the water mv^2/r, being r the radius of curvature of the convex path. If the path is concave, the acceleration force is added to the depth and the stress distribution looks like in figure 2.12c. Consequently the resulting pressure head, for water flows along a straight line, a convex path and a concave path is respectively:

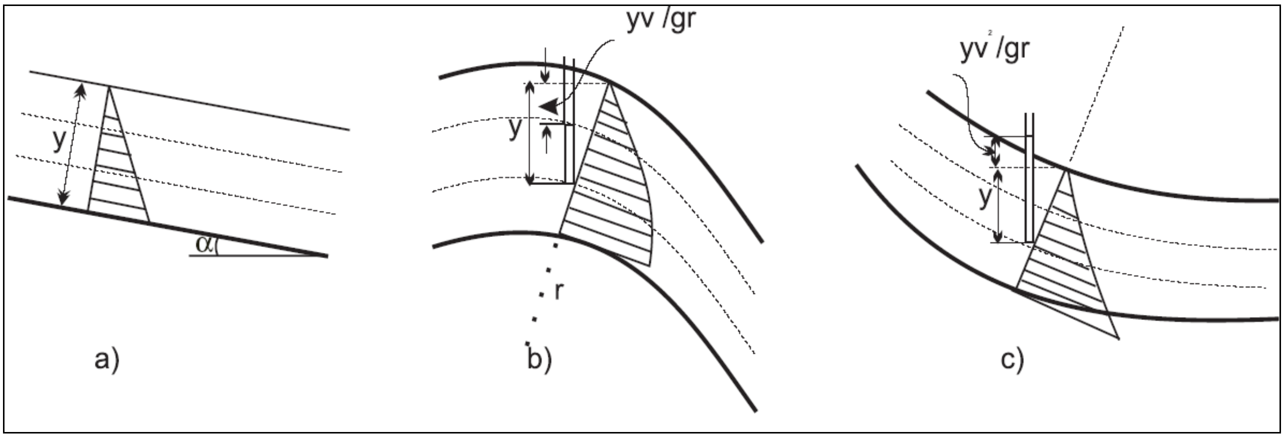


Figure 2-12 Pressure distribution for channels with vertically curved bed

$$\frac{P}{\gamma} = y\ (a);\ \frac{P}{\gamma} = y - y \cdot \frac{V^2}{rg}\ (b);\ \frac{P}{\gamma} = y + y \cdot \frac{V^2}{rg}\ (c) \qquad (2.31)$$

where:

γ = the specific weight of water y = the depth measured from the free water surface to the point, y = hCosα, h being the flow depth normal to the channel bottom

V = the water velocity at that point,
r = the radius of curvature of the curved flow path.

The specific energy in a channel section or energy head measured with respect to the bottom of the channel at the section is:

$$E = y + \alpha \frac{V^2}{2g} \qquad (2.32)$$

where α is a coefficient that takes into account the actual velocity distribution in the particular channel section, whose average velocity is V. The coefficient can vary from a minimum of 1.05 for a very uniform distribution, to 1.20 for a highly uneven distribution. Nevertheless in a preliminary approach a value of α = 1 can be used, a reasonable value when the slope is under 0.018 (α <1.03°). Equation 2.32 then becomes:-

$$E = y + \frac{V^2}{2g} \qquad (2.33)$$

A channel section with a water area A and a discharge Q, will have a specific energy:

$$E = y + \frac{Q^2}{2gA^2} \qquad (2.34)$$

Equation (2.34) shows that given a discharge Q, the specific energy at a given section, is a function of the depth of the flow only. When the depth of flow y is plotted, for a certain discharge Q, against the specific energy E, a specific energy curve, with two limiting boundaries, like the one represented in figure 2.13 is obtained. The lower limit AC is asymptotic to the horizontal axis, and the upper AB to the line E=y. The vertex point A on the specific energy curve represents the depth y at which the discharge Q can be delivered through the section at a minimum energy. For every point over the axis E, greater than A, there are two possible water depths. At the smaller depth the discharge is delivered at a higher velocity and hence at a higher specific energy - a flow known as supercritical flow. At the larger depth the discharge is delivered at a smaller velocity, but also with a higher specific energy - a flow known as subcritical flow. In the critical state the specific energy is a minimum, and its value can therefore be computed by equating the first derivative of the specific energy (equation 2.34) with respect to "y" to zero.

$$\frac{dE}{dy} = -\frac{Q^2}{gA^3}\frac{dA}{dy} + 1 = 0 \qquad (2.35)$$

The differential water area near the free surface, $\delta A/\delta y = T$, where T is the top width of the channel section (see figure 2.13). By definition:-

$$Y = A/t \qquad (2.36)$$

The parameter Y is known as the "hydraulic depth" of the section, and it plays a key role in studying the flow of water in a channel.

Substituting in equation (2.35) $\delta A/\delta y$ by T and A/T by Y one obtains:

$$\frac{V}{\sqrt{gY}} = 1 \qquad (2.37\ a)$$

Where:

$$F_r = \frac{V}{\sqrt{gY}} \qquad (2.37\ b)$$

The quantity F_r is dimensionless and known as the Froude number. When $F_r = 1$, as in equation (2.37 a), the flow is in the critical state. The flow is in the supercritical state when $F_r > 1$ and in the subcritical state when $F_r < 1$. In Figure 2.13, the AB line represents the supercritical flows and the AC the subcritical ones. As shown in figure 2.13, a family of similar curves can be drawn for the same section and different discharges Q. For higher discharges the curve moves to the right and for lower discharges to the left.

In the critical state, $y = y_c$ (y_c being the critical depth). It can be obtained from equation (2.37 a). For a rectangular channel, the critical depth is given by:

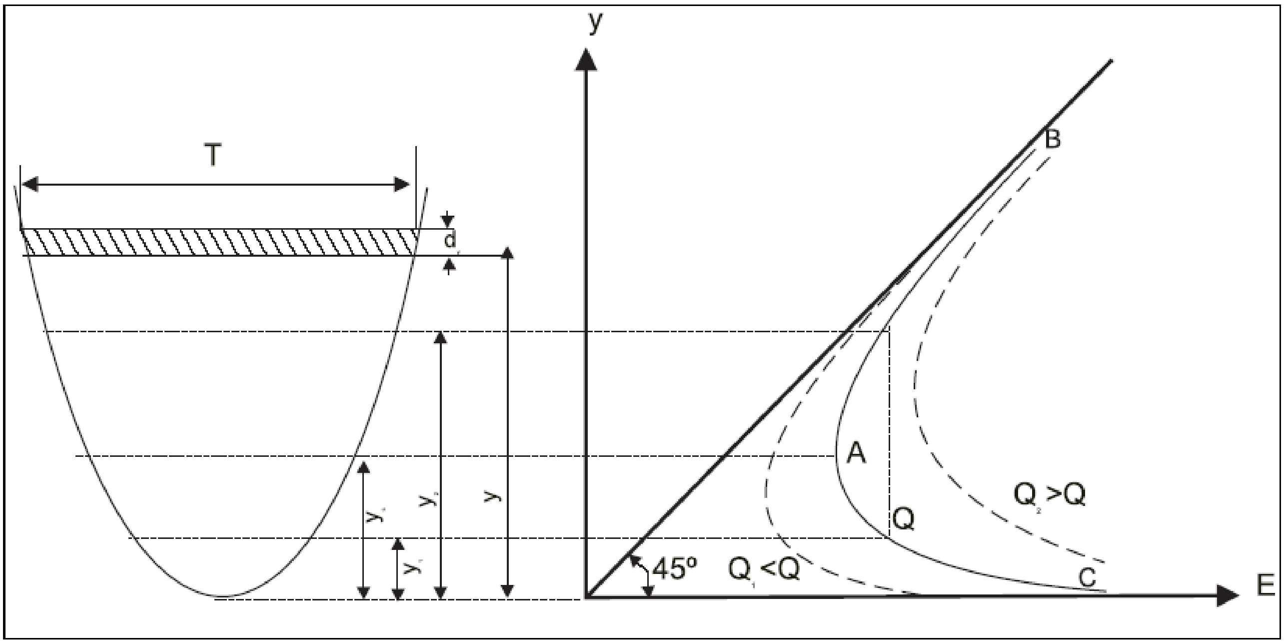


Figure 2-13 Specific energy as a function of flow depth

$$y_c = \sqrt[3]{\frac{Q^2}{gb^2}} = \sqrt[3]{\frac{q^2}{g}} \qquad (2.38)$$

where q=Q/b is the discharge per unit width of the channel. Table 2.5 shows the geometric characteristics of different channel profiles and Table 2.6, taken from Straub (1982) presents the empirical formulae used to estimate y_c, in non-rectangular channel.

Example 2.6

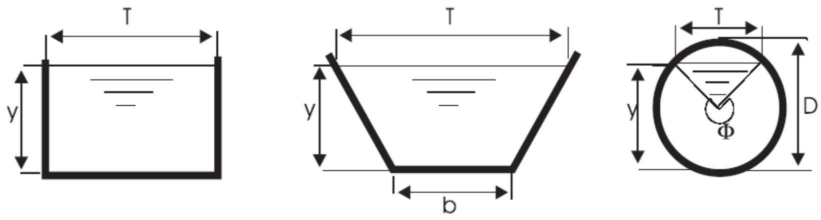
In a trapezoidal section channel where b=6 m and z = 2, compute the critical depth flow for a discharge of 17 m3/s.

From table 2.6 Ψ = α Q2/g = 29.46 for α=1
The solution is valid provided 0.1 < Q/b^2 < 0.4; as q/b^2 = 0.19 it is valid

$$y_c = 0.81\left(\frac{\Psi}{z^{0.75}b^{1.25}}\right)^{0.27} - \frac{b}{z}\Psi = 0.86m$$

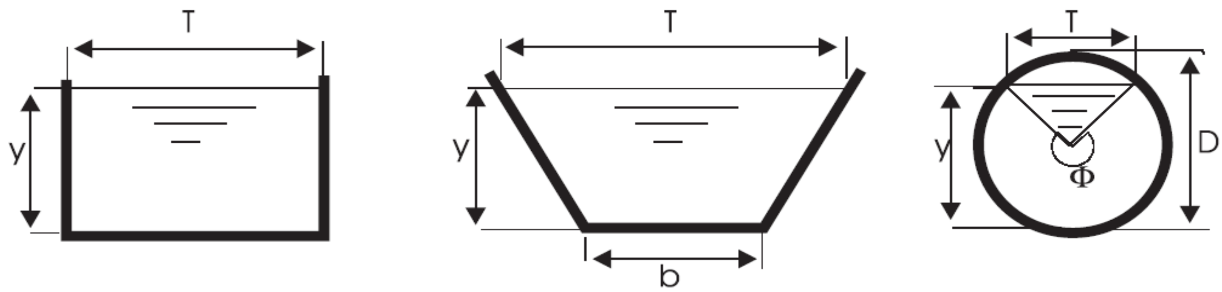
The estimation of the critical depth, and the supercritical and subcritical ones, permits the profile of the free surface to be determined, in cases such as, the sudden increase in the slope of a channel, the free surface upstream from a gate and spillways, etc..

Table 2-5 **Geometrical characteristics of different channel profiles**



Area A	by	(b+zy)y	$\frac{1}{8}(\Phi - sen\Phi)D^2$
Wetted perimeter P	b+2y	$b+2y\sqrt{1+z^2}$	$1/2\phi D$
Top width of section T	b	b+2zy	$2\sqrt{y(D-y)}$
Hydraulic radius R	$\frac{by}{b+2y}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$\frac{1}{4}\left(1-\frac{sen\Phi}{\Phi}\right)D$
Hydraulic depth D	y	$\frac{(b+zy)y}{b+2zy}$	$\frac{1}{8}\left(\frac{\Phi - sen\Phi}{sen\frac{\Phi}{2}}\right)D$
Section factor	$by^{1,5}$	$\frac{[(b+zy)y]^{1,5}}{\sqrt{b+2zy}}$	$\frac{\sqrt{2}(\theta - sen\theta)^{1,5}}{32\sqrt{sen\,½\theta}}D^{2,5}$

Table 2-6 Empirical formulae used to estimate y_c, in typical channel.



$\left(\frac{\Psi}{b^2}\right)^{1/3}$	$0,81\left(\frac{\Psi}{z^{0,75}b^{1,25}}\right)^{0,27} - \frac{b}{30z}$	$\left(\frac{1,01}{d^{0,25}}\right)\Psi^{0,25}$

Moody Diagram

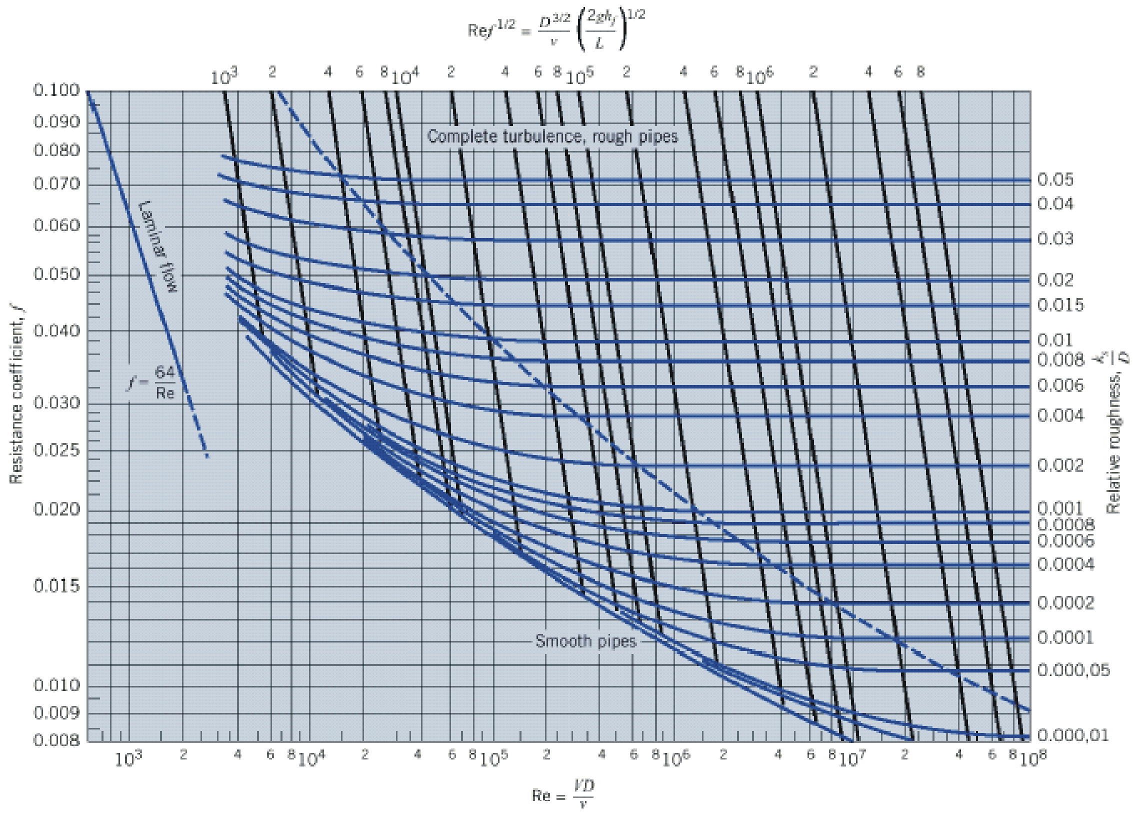


Figure 2-14 Moody's Chart: Friction factors for pipe flow

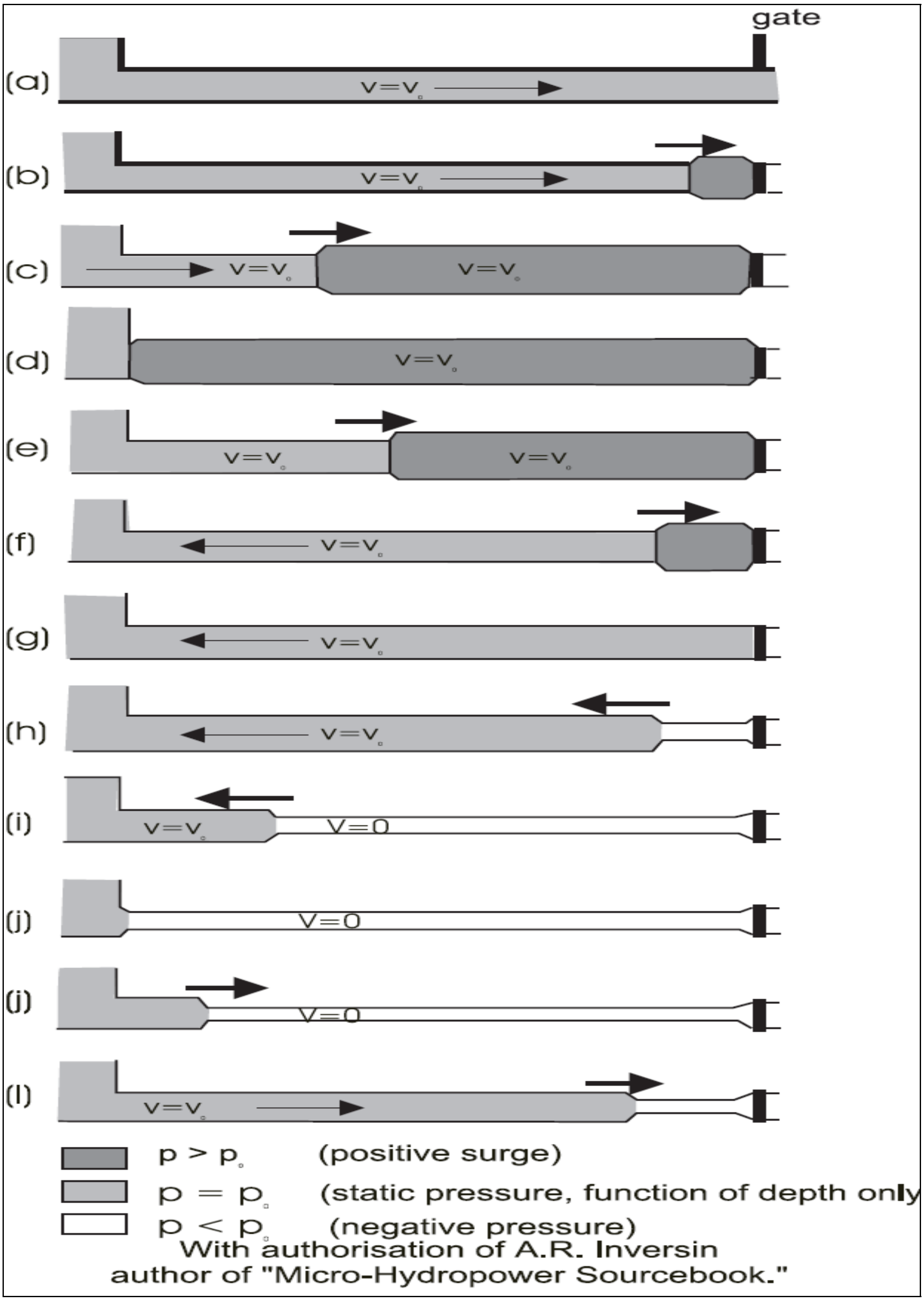


Figure 2-15 Illustration of pressure wave in pipes

Bibliography

1. N.H.C.Hwang and Carlos Hita, "Fundamentals of Hydraulic Engineering Systems", Prentice Hall Inc. Englewood Cliffs, New Jersey 1987

2. F.H. White, "Fluid Mechanics", MacGraw-Hill Inc. USA

3. A. Piqueras, "Evacuación de Broza" (in Castillan), ESHA Info nº 9 summer 1993

4. L. Allievi, The theory of waterhammer, Transactions ASME 1929

5. H. Chaudry. Applied Hydraulic Transients, Van Nostrand Reinhold Co. 1979

6. V.L. Streeter and E.B. Wylie, Hydraulic Transients, McGraw-Hill Book Co., New York 1967

7. J. Parmakian. Waterhammer analysis. Dower Publications, New York 1963

8. R.H. French, "Hidráulica de canales abiertos" (in Castillan), McGraw-Hill/Interamericana de Mexico, 1988

9. V.T. Chow, Open Channel Hydraulics, McGraw-Hill Book Co., New York 1959

10. V.L. Streeter and E.B. Wylie, Fluid Mechanics, McGraw-Hill Book Co., New York 1975

11. A.C Quintela, « Hidráulica » (in Portuguese), Ed. Calouste Gulbenkian Foundation, 1981

12. J. Dubois, “Comportement hydraulique et modélisation des écoulements de surface" (in French),
Communication LCH n° 8, EPFL, Lausanne 1998.

13. E. Mosonyi, “Water power development”, Tome I and II, Akadémiai Kiadó Budapest, 1987/1991

Other references on the topics of this subject :
W.King and E.F. Brater, Handbook of Hydraulics, McGraw-Hill Book Co., New York 1963
R. Silvester, Specific Energy and Force Equations in Open-Channel Flow, Water Power March 1961

[i] By Jonas. Rundqvist (SERO), Pedro Manso (EPFL) and Celso Penche (ESHA)

CHAPTER 3: EVALUATING STREAMFLOW

CONTENTS

3. EVALUATING STREAMFLOW 44
3.1. Introduction 44
3.2. Stream flow records 45
3.3. Evaluating stream flows by discharge measurements 46
3.3.1. Velocity-area method 46
3.3.2. Weir method 53
3.3.3. Slope-area method 54
3.4. Stream Flow Characteristics 55
3.4.1. Hydrograph 55
3.4.2. Flow Duration Curves (FDC) 55
3.4.3. Standardised FDC curves 56
3.4.4. FDCs for particular months or other periods 58
3.4.5. Water Pressure or "head" 58
3.5. Residual, reserved or compensation flow 61
3.6. Estimation of plant capacity and energy output 61
3.6.1. How the head varies with the flow and its influence on the turbine capacity 63
3.6.2. Peaking operation 64
3.7. Firm energy 65
3.8. Floods 65
3.8.1. Flood Control Design 65
3.8.2. Statistical analysis of flood data 66
3.8.3. Hydrological modelling of the catchment area 68
Bibliography 69

LIST OF FIGURES

Figure 3-1 Schematic layout of a hydro development 44
Figure 3-2 Measuring the river stage, definitions 46
Figure 3-3 Rating curve 48
Figure 3-4 Measuring the cross-sectional area 49
Figure 3-5 Conductivity time curve 52
Figure 3-6 Discharge measurements using weirs and notches 53
Figure 3-7 Example of hydrograph 54
Figure 3-8 Example of a flow duration curve (FDC) 55
Figure 3-9 Example of FDC with logarithmic scale 56
Figure 3-10 Example of standardised FDCs 57
Figure 3-11 Conveyance system (example 3.1) 59
Figure 3-12 Residual flow 61
Figure 3-13 Example of turbine efficiency as a function of flow 63
Figure 3-14 Variation of net head vs. river flow 64
Figure 3-15 Components of hydrological model 68

LIST OF TABLES

Table 3-1 Typical values of Manning's n for watercourses 54
Table 3-2 Minimum technical flow of turbines 62
Table 3-3 Typical design flood criteria 66
Table 3-4 Probability of occurrence 66

LIST OF PHOTOS

Photo 3-1 Gauging station in a river 47
Photo 3-2 Current meters 50

3. EVALUATING STREAMFLOW[i]

3.1. Introduction

All hydroelectric generation depends on falling water. This makes hydropower extremely site dependent. First of all, a sufficient and dependable stream flow is required. Secondly, the topographic conditions of the site must allow for the gradual descent of the river in a river stretch be concentrated to one point giving sufficient head for power generation. This head can be created by dams or by leading the water in parallel to the river in a waterway with low head losses compared to the natural stream, or very often, by a combination of both.

Planning for the exploitation of a river stretch or a specific site is one of the more challenging tasks that face a hydropower engineer, since there are an unlimited number of practical ways in which a river or site can be exploited.

The hydropower engineer has to find the optimum solution for plant configuration, including dam type, water conveyance system, installed generating capacity, location of various structures etc. The success of the hydropower engineer depends on experience and an almost "artistic" talent, since a strictly mathematical optimisation approach is impossible, due to the number of possibilities and site-specific conditions.

When a site has been identified as topographically suitable for hydropower, the first task is to investigate the availability of an adequate water supply. For an ungauged watercourse, where observations of discharge over a long period are not available, it involves the science of hydrology, the study of rainfall and stream flow, the measurement of drainage basins, catchment areas, evapotranspiration and surface geology.

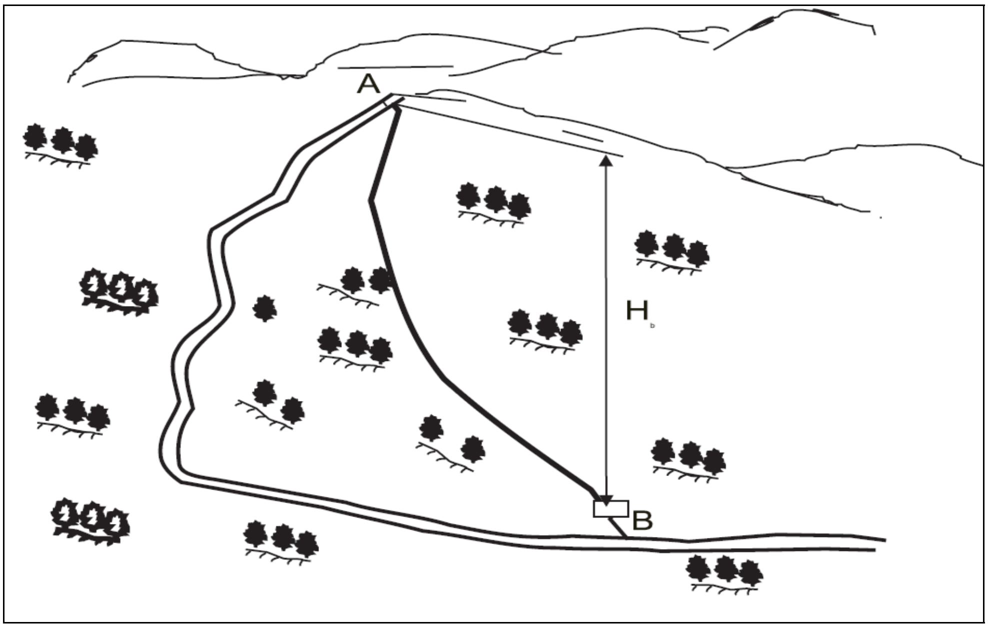


Figure 3-1 Schematic layout of a hydro development

Figure 3.1 illustrates how the water flowing from point A to point B, with elevations Z_A and Z_B, loses potential energy corresponding to the drop in elevation. This loss of potential energy occurs regardless of the path along the watercourse or via an open canal, penstock and turbine. The potential energy lost can be converted to power lost according to the equation:

$P = Q \cdot H_g \cdot \gamma$

Where:

P is the power in kW lost by the water
Q is the flow in m3/s
H_g is the gross head in m, = $Z_A - Z_B$, and
γ is the specific weight of water, (9.81 kN/m^3).

The water can follow the riverbed, losing power through friction and turbulence resulting in a marginal rise in the temperature of the water. Or it can flow from A to B through an artificial conveyance system with a turbine at its lower end. In this case the power will be used mainly for running a turbine, and a smaller part of the power is lost in friction in the conveyance system. In the latter case it is the power lost in pushing through the turbine that will be converted to mechanical energy and then, by rotating the generator, to produce electricity.

The objective is to reduce construction costs while conserving the maximum amount of power available to rotate the generator. To estimate the water potential one needs to know the variation of the discharge throughout the year and how large the gross available head is. In the best circumstances the hydrologic authorities would have installed a gauging station in the stretch of stream under consideration, and stream flow time series data would have been gathered regularly over several years.

Unfortunately, it is rather unusual for regular gauging to have been carried out in the stretch of river where the development of a small hydro scheme is proposed. If, however, it does happen, then it will suffice to make use of one of several approaches that can be used to estimate the long-term average annual flow and the flow duration curve for the stretch in question (these approaches will be explained later).

Whether or not regular gauging has taken place, the first step is to do some research, to ascertain if there are stream flow records for the stretch of river in question. If not, then in other stretches of the same river or a similar nearby river that permits the reconstitution of the time series for the referred stretch of river.

3.2. Stream flow records

In Europe, stream flow records can be obtained from national hydrological institutes. These stream flow records can be of several different types, each useful for the evaluation of the generating potential of the considered site. These include:-

- Measured stream flow data for a number of gauged sites
- Stream flow characteristics for these sites such as mean flow and flow duration curves (both expressed as actual flow and generalised as runoff per unit area of the catchment)
- Runoff maps, etc

There is a United Nations organisation, the “World Meteorological Organisation”, with a hydrologic information service (INFOHYDRO) whose objective is to provide information regarding:

- National and international (governmental and non-governmental) organisations,
- Institutions and agencies dealing with hydrology;
- Hydrological and related activities of these bodies;

- Principal international river and lake basins of the world;
- Networks of hydrological observing stations of countries - numbers of stations and duration of records;
- National hydrological data banks - status of collection, processing and archiving of data;
- International data banks related to hydrology and water resources.

Further information can be obtained at www.wmo.ch (At the date of printing, the INFOHYDRO database was going through a major revision and was not available)

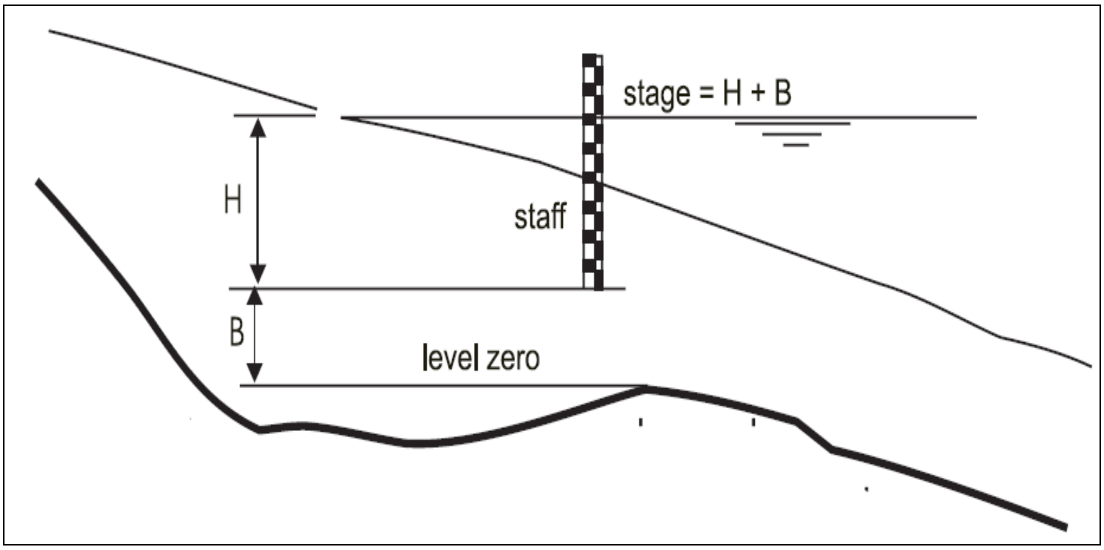


Figure 3-2 Measuring the river stage, definitions

3.3. Evaluating stream flows by discharge measurements

If appropriate stream flow time series cannot be found, the discharge should preferably be directly measured for at least a year. A single measurement of instantaneous flow in a watercourse is of little use. To measure the discharge several methods are available:

3.3.1. Velocity-area method

This is a conventional method for medium to large rivers, involving the measurement of the cross sectional area of the river and the mean velocity of the water through it. It is a useful approach for determining the stream flow with a minimum effort. An appropriate point must be selected on a relatively straight, smoothly flowing portion of the river to be gauged (figure 3.2).

The river at this point should have a uniform width, with the area well defined and clean.
As discharge varies, the top water level (termed the stage of the river) rises and falls. The stage is observed daily at the same time each day on a board - marked with metres and centimetres. In modern gauging stations, instead of a board, that requires regular observations, any one of several water-level measurement sensors is available which automatically register the stage. To calibrate the stage observations or recordings, periodic discharge measurements from the lowest to the highest are made over a time period of several months. Photo 3.1 shows a gauging station in a river.

Photo 3-1 Gauging station in a river

The correlation stage-discharge is called a rating curve (figure 3.3) and permits the estimation of the river discharge by reading the river stage. To draw this curve, both the stage and the discharge must be simultaneously read. It is strongly recommended that to begin measuring the low flows, one should use the data to draw a curve that correlates the flows and the 'n' Manning coefficient. Later on the river slope-area method (section 3.3.3) can be used to estimate the high flows, which are often impossible to measure with the other methods.

When a rating curve has been graphically established, based on a number of readings, its mathematical formulation can be readily derived, which facilitates interpretation of the stage readings. The rating curve (figure 3.3) is represented by the function:-

$$Q = a\,(H+B)^n \qquad (3.1)$$

Where a and n = constants

H = river stage as measured or recorded

B = correction factor to get the actual level

To compute B (see figure 3.2) the data corresponding to two discharges should be noted, such as

$$Q_1 = a\,(H_1+B)^n$$

$$Q_2 = a\,(H_2+B)^n$$

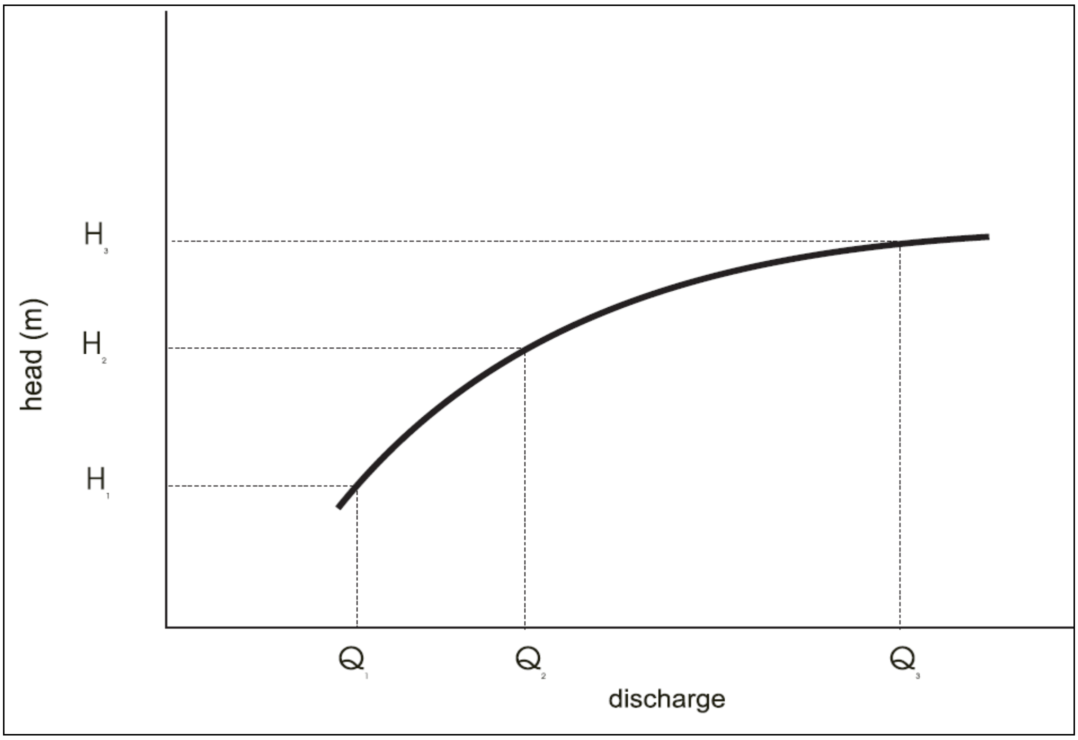


Figure 3-3 Rating curve

By introducing a third "factual" reading H_3 vs. Q_3, where Q_3 (indexes in figure 3.3 are not representative) is defined as the square root of the product of Q_1 and Q_2, and the corresponding H_3 is taken from the graphical representation of the rating curve Q_3, this can be expressed as:-

$$Q_3 = \sqrt{Q_1 \cdot Q_2} = a(H_3 + B)^n = \sqrt{a(H_1 + B)^n \cdot a(H_2 + B)^n}$$

consequently:

$$(H_3 + B)^2 = (H_1 + B) \cdot (H_2 + B)$$

and therefore:

$$B = \frac{H_3^2 - H_1 H_2}{H_1 + H_2 - 2H_3} \qquad (3.2)$$

There are ISO recommendations for the correct use of this technique.

Measuring the cross-sectional area

To compute the cross-sectional area of a natural watercourse it should be divided into a series of trapezoids (figure 3.4). Measuring the trapezoid sides, by marked rules, illustrated in figure 3.4, the cross-section would be given by:-

$$S = b\,\frac{h_a + h_2 + \ldots.. + h_n}{n} \qquad (3.3)$$

Measuring the velocity

Since the velocity both across the flow and vertically through it is not constant, it is necessary to measure the water velocity at a number of points to obtain a mean value. There are several ways of doing this, two of which are discussed below.

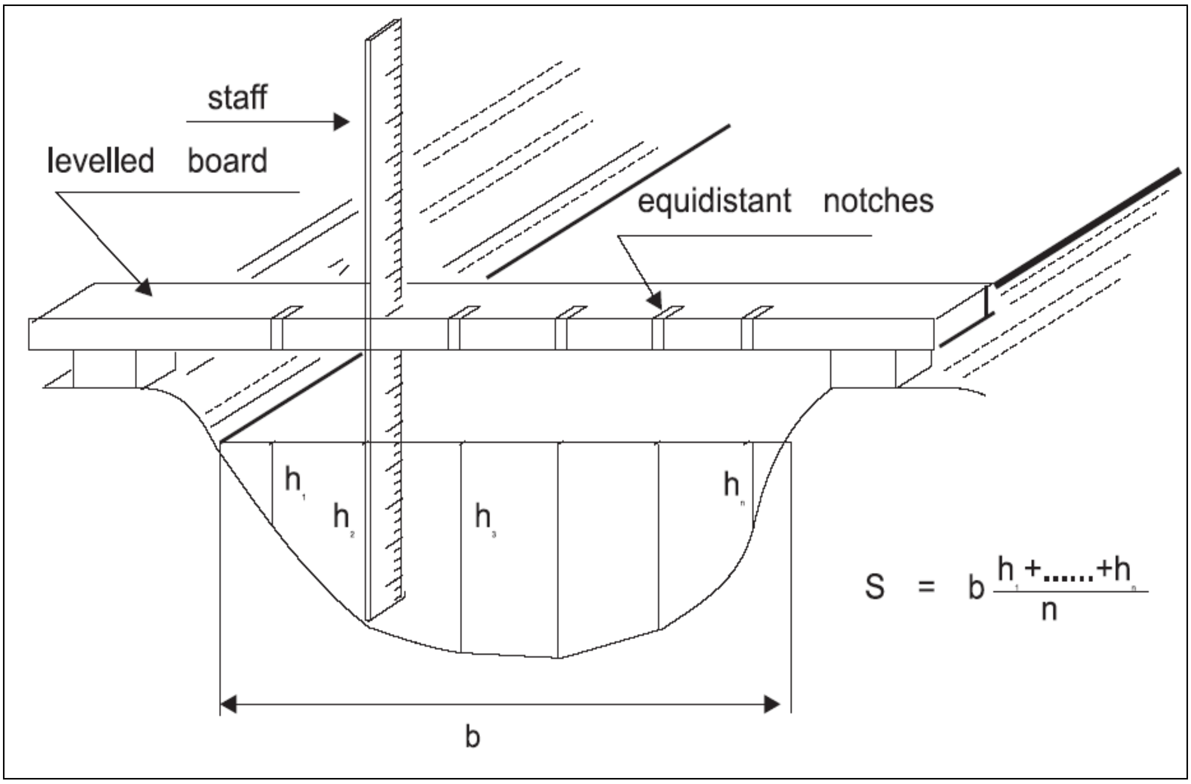


Figure 3-4 Measuring the cross-sectional area

By float

A floating object, which is largely submerged (for instance a wood plug or a partially filled bottle) is located in the centre of the stream flow. The time t (seconds) elapsed to traverse a certain length L (m) is recorded. The surface speed (m/s) would be the quotient of the length L and the time t. To estimate the mean velocity, the above value must be multiplied by a correction factor that may vary between 0.60 and 0.85 depending on the watercourse depth and their bottom and riverbank roughness (0.65 is a well accepted value). The accuracy of this method is dependant on the range of correction factor.

By mechanical current-meter

A current-meter is a fluid-velocity-measuring instrument. Current meters are classified in two types:-
Vertical axis rotor with cups: This type of instrument has a circle of small conical cups, disposed horizontally which rotate about the suspension axis. (Photo 3.2 right photo) These current meters operate in lower velocities than the horizontal axis rotor types, and have the advantage of bearings being well protected from silty waters. The rotor can be repaired in the field.
Horizontal axis rotor with vanes (propeller): A small propeller rotates about a horizontal shaft, which is kept parallel to the stream by tail fins. (Photo 3.2 left photo) The instrument is weighted to keep it as directly as below the observer as possible. This rotor has the advantage of being less likely to disturb the flow around the measuring point and also for being less likely to become entangled by debris.

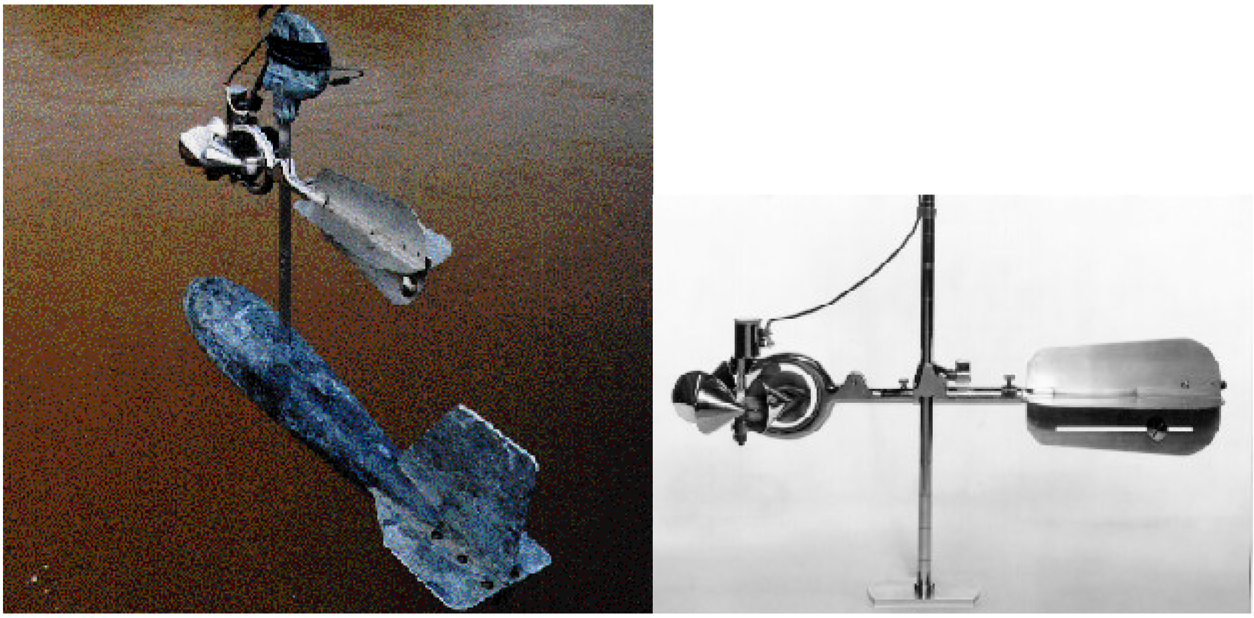


Photo 3-2 Current meters

Each revolution of the propeller is recorded electrically through a cable to the observer and the number of revolutions is counted by the observer, or automatically by the instrument itself, over a short period (say 1 or 2minutes). These observations are converted into water velocities from a calibration curve for the instrument (modern instruments, with microprocessor technology will compute this and display it almost immediately). By moving the meter vertically and horizontally to a series of positions (whose coordinates in the cross-section are determined), a complete velocity map of the cross-section can be drawn and the discharge through it calculated.

In the case of medium to large rivers, observations are made by lowering the meter from a bridge, however, if the bridge is not single-span there will be divergence and convergence of the streamlines caused by the piers, and this can cause considerable errors. In many instances, however, the gauging site, which should be in as straight and uniform a reach of the river as possible, will have no bridge. In such cases, particularly if it is deep and in flood, a cable to hold a stable boat must be provided, together with a lighter measuring cable to determine horizontal position in the cross-section.

Since the drag on a boat, with at least two occupants and suspended current-meter, is considerable, a securely fastened cable should be used. The presence of suitable large trees at a particular site often necessitates its choice for this reason. Alternatively, for very large rivers, cableways are sometime used to suspend the meter, directly from a cable car, the instrument in this latter case being positioned by auxiliary cables from the riverbanks or from the cable car itself.

Depths should always be measured at the time of velocity observation since a profile can change appreciably during flood discharges. Observers should also remember such elementary rules as to observe the stage before and after the discharge measurement, and to observe the water slope by accurate levelling to pegs at the water level as far upstream and downstream of the gauging site as is practicable, up to (say) 500m in each direction.

As water velocities increase in high floods the weighted current meter will be increasingly swept downstream on an inclined cable. The position of a meter in these circumstances can be found reasonably accurately if the cable angle is measured. Ballast can be increased but only within limits.

Rods can be used to suspend the meters but a rigid structure in the boat will then be required to handle the rods, calling for a stable platform on a catamaran-type of craft. Rod vibration and bending are common in deep rivers unless large diameter rods are employed, in which case the whole apparatus is getting very heavy and unmanageable.

By electro-magnetic current-meter

An electro-magnetic (e/m) current-meter is an electrical induction-measurement instrument, with no moving parts, mounted in a totally enclosed streamlined probe. The probe can be mounted on rods and held at various depths or suspended on a cable.
The e/m meter has the advantages of being smaller and having a wider measurement range than the propeller meters. It is particularly useful at very low velocities when propeller meters become erratic. Its sensitivity and lower vulnerability to fouling from weeds and debris make it attractive for use in heavily polluted or weedy streams.
Each unit is provided with a surface control box with a digital display and dry-cell batteries. A set of stainless steel wading rods is also standard equipment. Latest models have built-in battery-charger circuits.

It will be appreciated that since each river is unique, a careful assessment of its width, depth, likely flood velocities, cable-support facilities, availability of bridges, boats, etc. needs to be made before a discharge measurement programme can begin.

The discharge at the chosen measuring point is best obtained by plotting each velocity observation on a cross section of the gauging site with an exaggerated vertical scale. Isovels (velocity profiles – contours of equal velocity) are then drawn and the included areas measured by a planimeter. Alternatively, the river may be subdivided vertically into sections and the mean velocity of each section applied to its area. In this method the cross-sectional area of any one section, where measurements, are taken should not exceed 10 per cent of the total cross-sectional area.

A check should always be made using the slope-area method of section 3.3.4 and a value obtained for the Manning's number “n”. In this way knowledge of the n values of the river at various stages will be built up and this may prove most valuable for subsequently extending the discharge-rating curve.
To ensure uniformity in the techniques of current-meter gauging, ISO has published a number of recommendations.

By dilution methods.

Dilution gauging is particularly suited to small turbulent streams where depths and flows are inappropriate for current metering, and flow structures would be unnecessarily expensive. The method involves the injection of a chemical into the stream and the sampling of the water some distance downstream after complete mixing of the chemical in the water has occurred. The chemical can either be added by constant-rate injection until the sampling downstream reveals a constant concentration level, or administered in a single dose as quickly as possible, known as “gulp injection”. In the latter case, samples over a period of time disclose the concentration-time correlation. In both cases the concentration of chemical in the samples is used to compute the dilution, and hence, the discharge of the stream can be obtained.

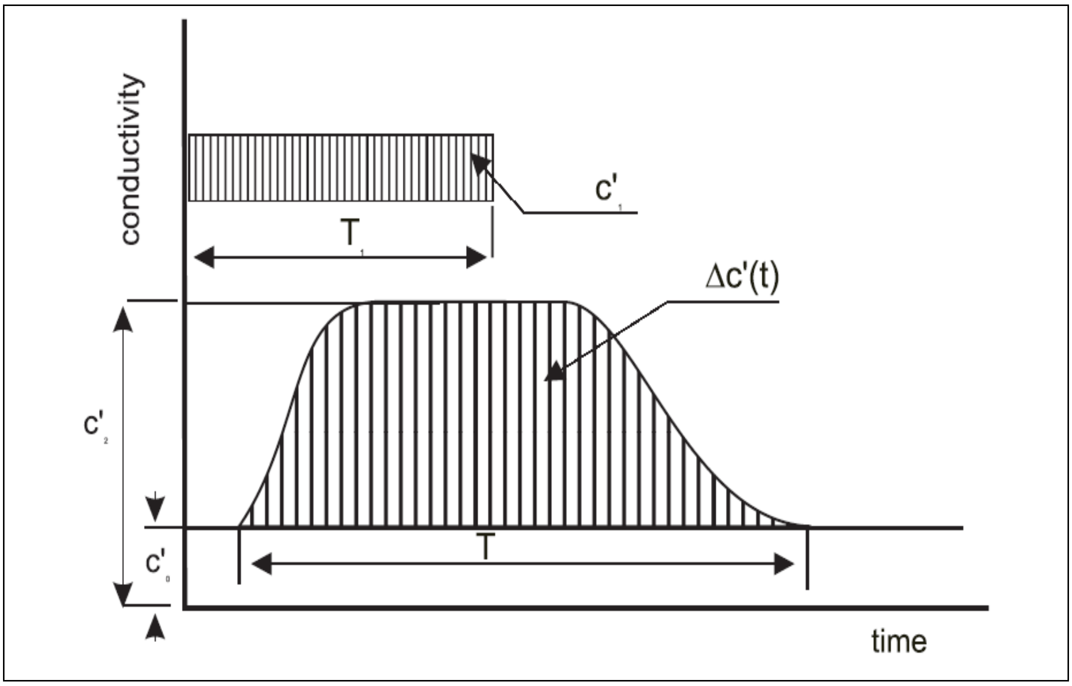


Figure 3-5 Conductivity time curve

Analysis of the samples is by an automated colorimetric procedure that estimates the concentration of very small amounts of chromium compound by comparing with a sample of the injection solution. The equipment is expensive and somewhat specialised.

Recent developments have substituted the above procedures by a method developed by Littlewood[7], requiring only simple and relatively cheap equipment. The method depends on the electrical conductivity of solutions of common salt (NaCl) in the stream water and is a version of the relative-dilution gauging method developed by Aastad and Sognen.

The discharge is measured by gradually rleasing a known volume (V) of a strong salt solution (c^1) into the stream at a known rate (q), and measuring, at short intervals, the change in conductivity of the water at the downstream end of the mixing length. It is then is possible to plot a conductivity-time curve, along a time T as in figure 3.5. The average of the ordinates, of this curve, represent the average of the difference in conductivity, between the salt solutions and the stream water, upstream from the injection point. If a small volume, v, of the particular strong solution is added to a large volume V^* of the stream water, and the differences in conductivity Δc^* are measured, the discharge will be then given by the equation:

$$Q = \frac{V}{T_2} x \frac{V^*}{v} x \frac{\Delta c^*}{\overline{\Delta c'}} \qquad (3.5)$$

where:

- v = volume of injection solution
- T_2 = duration of solute wave (s)
- V = volume of the strong solution added to a larger
- V^* = volume of stream water
- Δc^* = change in conductivity (ohm-1) consequence of the dilution of v in V^*
- Δc^* = ordinate's average curve conductivity-time

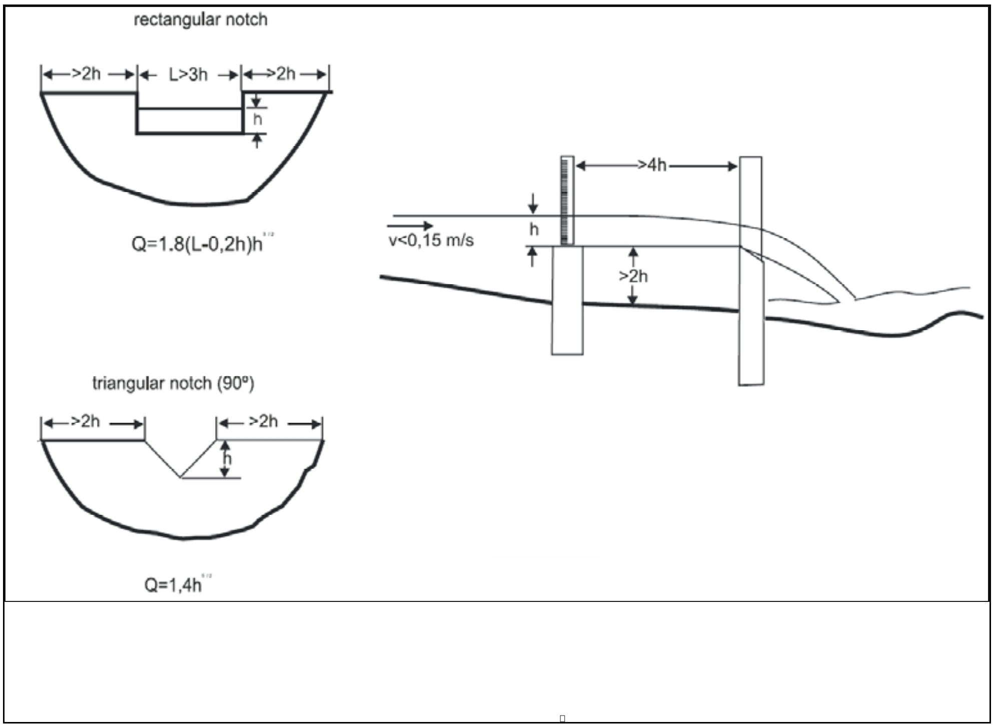


Figure 3-6 Discharge measurements using weirs and notches

3.3.2. Weir method

If the watercourse being developed is reasonably small (say < 4 m3/s) then it may be possible to build a temporary weir. This is a low wall or dam across the stream/river to be gauged with a notch through which all the water may be channelled. Many studies have established accurate formulae for the discharge through such notches. A simple linear measurement of the difference in level between the upstream water surface and the bottom of the notch is sufficient to quantify the discharge. However, it is important to measure the water surface level some distance back from the weir (at least four times the depth over the base of the notch) and to keep the notch free of sediment and the edge sharp and chamfered on the downstream side of the top of the weir/notch.

Several types of notches can be used:-

- Rectangular,
- V-notch
- Trapezoidal.

The V-notch is the most accurate at very low discharges but the rectangular or trapezoidal types are capable of a much wider range of flows. The actual notches may be metal plates or planed hardwood with sharp edges, built to the dimensions of figure 3.6.

Another fairly accurate method is to construct a “Flume”. A “Flume” is where a stream is channelled through a particular geometrically shaped regular channel section for some distance before entering a length of different cross-section, usually made so by side contraction or steps in the bed, generally in the shape of a “Venturi”. These structures have the advantage over weirs in that they do not obstruct the flow or “ponding” of the water upstream, they can also be very accurate and provide a permanent gauging station.

To ensure uniformity in the techniques of current-meter gauging ISO has published various recommendations. The catalogue with ISO recommendations can be obtained at:

http://www.iso.ch/iso/en/CatalogueListPage.CatalogueList?ICS1=17&ICS2=120&ICS3=20

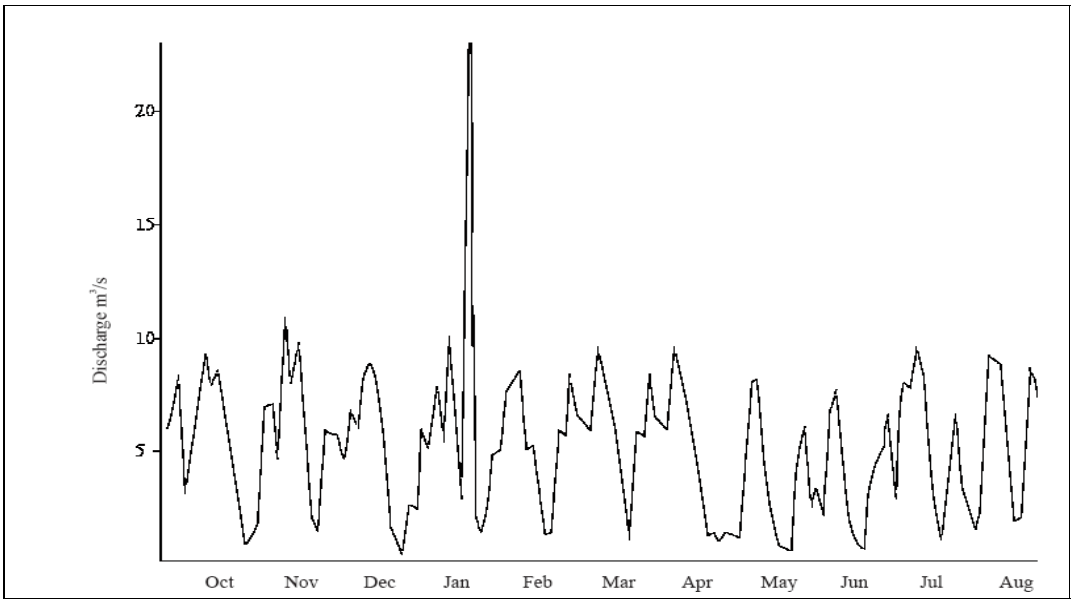


Figure 3-7 Example of hydrograph

3.3.3. Slope-area method

This method depends on some of the hydraulic principles described in Chapter 2 and is useful for high flows where other methods are impractical. It presupposes that it is practical to drive in pegs or make other temporary elevation marks at the water-surface level (upstream and downstream of the discharging site) at the time the flow measurements are taken. These marks can subsequently be used to establish the water slope (S). Cross-sectional measurements are taken to establish the area (A) and hydraulic radius of the section (R). Once these parameters are known the discharge is computed by the Manning formula

$$Q = \frac{A \cdot R^{2/3} \cdot S^{1/2}}{n} \qquad (3.6)$$

This method is sometimes criticised because of its dependence on the value of n. Since n for natural streams is about 0.035, an error in n of 0.001 gives an error in discharge of 3 per cent. This error may be partially reduced by plotting n against stage for all measured discharges, so that the choice of n for high stages is not arbitrary but is taken from such a plot. If a high flood slope can be measured, then this method may well be the best one for such flows. Typical values of Manning's n for watercourses and common pipe materials are given Table 3.1

Table 3-1 Typical values of Manning's n for watercourses

Watercourses	n
Natural stream channels flowing smoothly in clean conditions	0.030
Standard natural stream or river in stable conditions	0.035
River with shallows and meanders and noticeable aquatic growth	0.045
River or stream with rods and stones, shallows and weedy	0.060

3.4. Stream Flow Characteristics

A programme of stream gauging, at a particular site over a period of years, will provide a table of discharges that, to be of any use, has to be organised into a usable form.

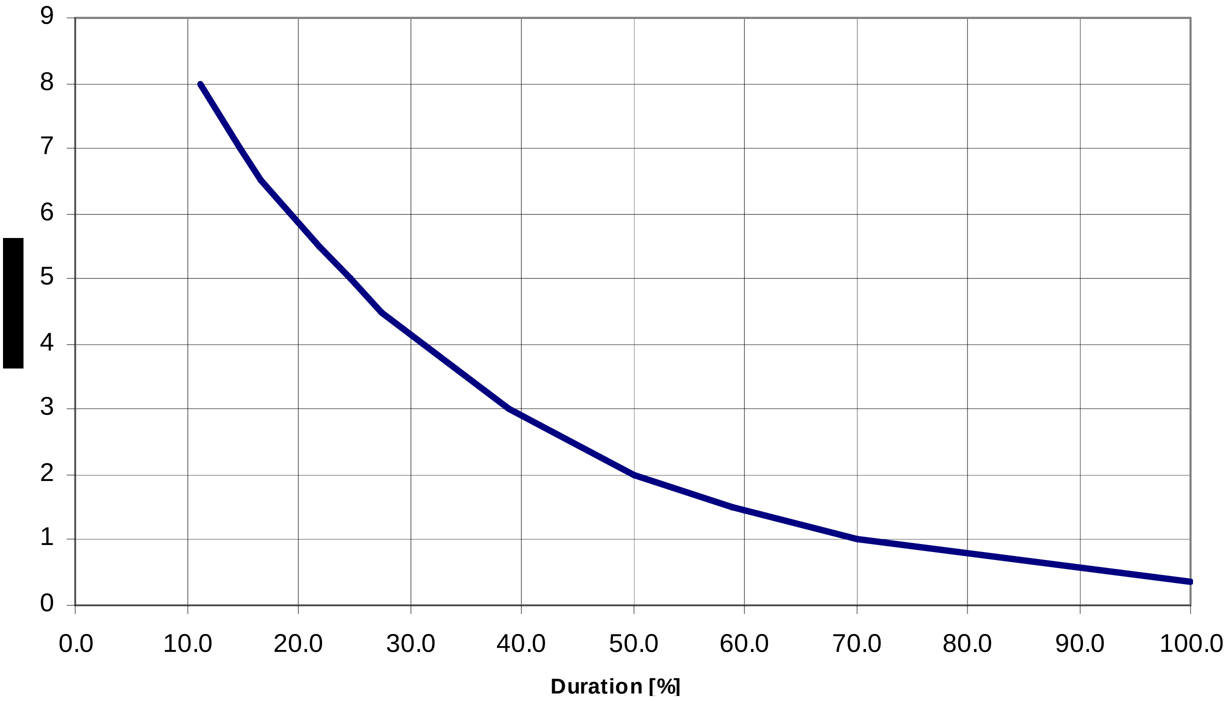


Figure 3-8 Example of a flow duration curve (FDC)

3.4.1. Hydrograph

One way of doing this is to plot them sequentially in the form of a hydrograph, which shows discharge against time, in chronological order (see figure 3.7).

3.4.2. Flow Duration Curves (FDC)

Another way of organising discharge data is by plotting a flow duration curve (FDC) An FDC shows for a particular point on a river the proportion of time during which the discharge there equals or exceeds certain values. It can be obtained from the hydrograph by organising the data by magnitude instead of chronologically. If the individual daily flows for one year are organised in categories as shown below:-

	No of days	% of the year
Flows of 8.0 m3/s and greater	41	11.23
Flows of 7.0 m3/s and greater	54	14.9
Flows of 6.5 m3/s and greater	61	16.8
Flows of 5.5 m3/s and greater	80	21.8
Flows of 5.0 m3/s and greater	90	24.66
Flows of 4.5 m3/s and greater	100	27.5
Flows of 3.0 m3/s and greater	142	39
Flows of 2.0 m3/s and greater	183	50
Flows of 1.5 m3/s and greater	215	58.9

Flows of 1.0 m3/s and greater	256	70
Flows of 0.35 m3/s and greater	365	100

If the above figures are plotted then a graph like figure 3.8 will be obtained, which represents the ordinates of figure 3.7 arranged in order of magnitude instead of chronologically.

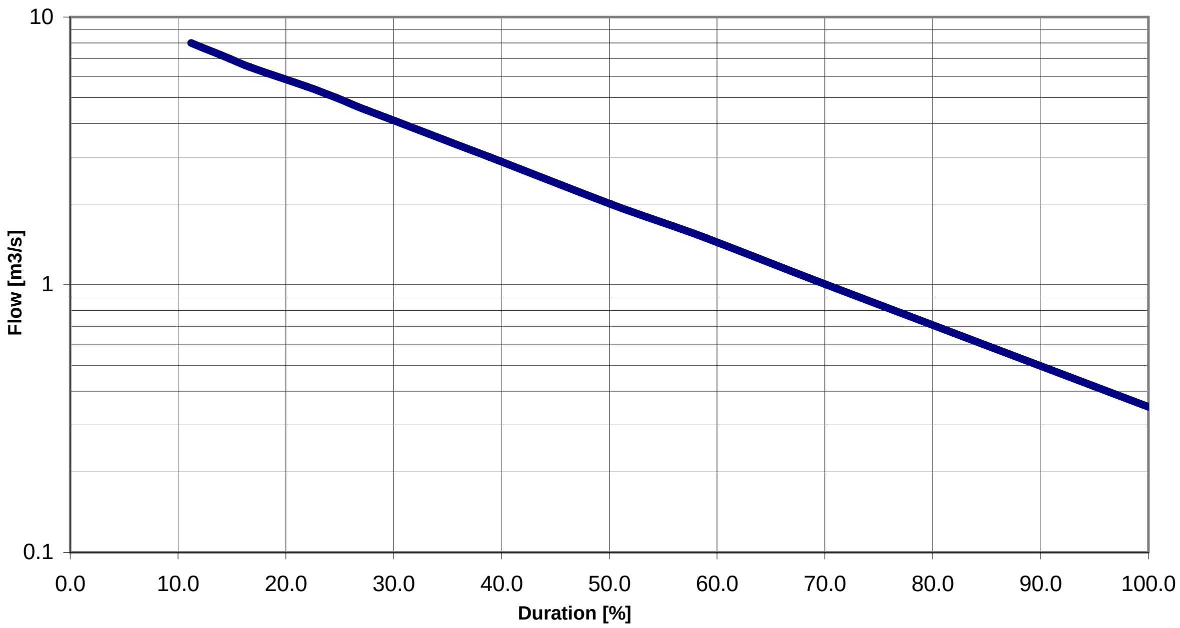


Figure 3-9 Example of FDC with logarithmic scale

Most gauging stations (in the EU) are computerised, the easiest way to derive a FDC is to transpose the digital data to a spreadsheet, sorting them in descending order, and then by hand or by using a simple macro, classify the data as in the above table. Once done, the same spreadsheet, using its graphic building capability will draw the curve FDC (like the one drawn in figure 3.8).

For many rivers the ratio of peak to minimum discharges may be two or more orders of magnitude and FDCs for points on them are often more conveniently drawn with the (Q) ordinate to a logarithmic scale, and a normal probability scale used for the frequency axis. On these, logarithmic graphs, the discharges are distributed, such that the FDC plots as a straight line. Figure 3.9 represents the graph in figure 3.8 with the vertical axis in logarithmic scale.

3.4.3. Standardised FDC curves

FDCs for different rivers can be compared by presenting them in a standardised way. The discharges are divided firstly by the contributing catchment areas and secondly by weighted average annual rainfall over the catchment areas. The resulting discharges, in m^3/s or litres /s, per unit area, per unit annual rainfall (typically $m^3/s/km^2/m$) can then be compared directly. Figure 3.10 shows twenty FDCs corresponding to catchment areas of different geological composition, drawn to a double logarithmic scale. A collection of regional flow-duration curves shows the effect of a basin's superficial geology on the shape of the curves. If the flow duration curves of different catchments are standardised by the catchments mean flow, certain low flow statistics, such as Q95 can be used to describe the whole flow duration curve.

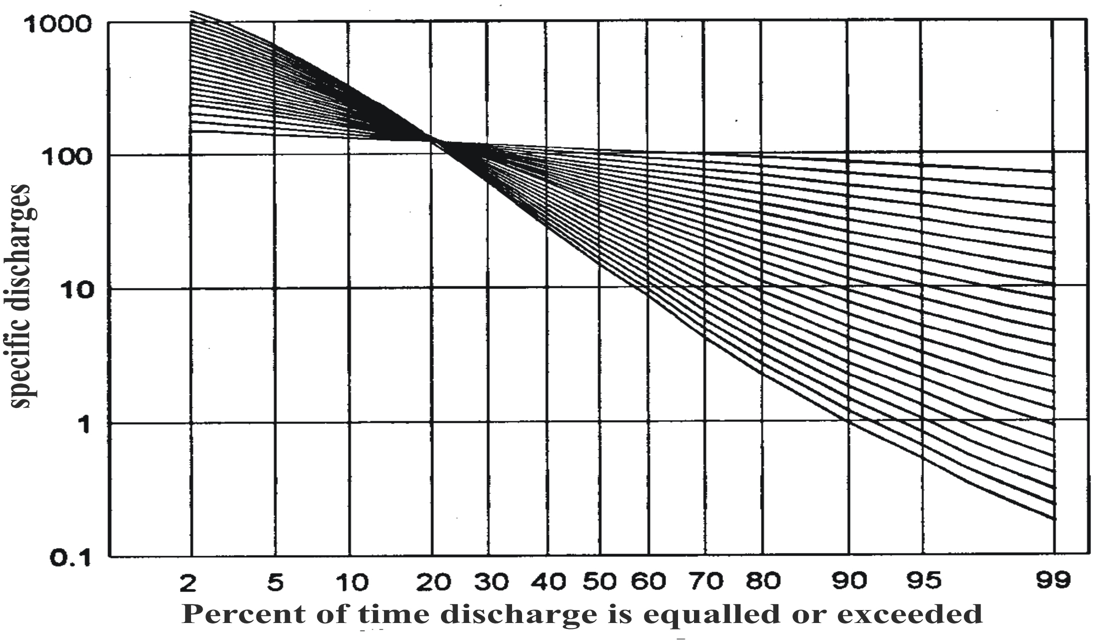


Figure 3-10 Example of standardised FDCs

Another method for standardising FDCs is to express Q in terms of Q/Qm, where Qm is the mean flow. The use of such a non-dimensional ordinate allows all rivers, large and small, to be compared on the same graph. If sufficient records are available from neighbouring rivers of similar topographical character in a similar climate, these methods can be very useful in evaluating stream flows at ungauged sites. If we know the FDC for another stretch of the same river, it will be possible to extrapolate it using the ratio of areas of the respective catchment basins.

When there are no flow records for a particular location it is necessary to proceed from first principles. Rainfall data is normally available from national agencies on an annual-average basis, but often only on a fairly small scale. Therefore, attempts should always be made to find local records, which will indicate seasonal variation. Failing that, a standard rain gauge should be installed in the catchment area, as soon as the studies are being considered. Even one year's records will help in the production of a synthesised FDC.

The first step is to estimate the mean annual flow Q_m [also referred to average daily flow (ADF)].

In UK the mean flow is estimated using a rainfall catchment water balance methodology, the long term average annual runoff of rainfall can be assumed to be equal to the difference between standard average annual rainfall (SAAR) and actual evaporation (AE). Catchment values of SAAR and potential evaporation are estimated from the rainfall and potential evaporation (PE) maps.

Actual evaporation is estimated from potential evaporation using a scaling factor r, where r increases with SAAR and hence increasing water availability. For catchments with annual average rainfall in excess of 850mm per year, it is assumed that actual evaporation is equal to potential. This relationship between SAAR is given by:

r = 0.00061 x SAAR + 0.475 for SAAR < 850 mm
r = 1.0 for SAAR > 850 mm
Actual evaporation is calculated using: AE = r x PE

The average runoff depth (AARD in millimetres) over the catchment area (AREA in km^2) is converted to mean flow in m^3/s by:

$$Q_m = (AARD \times AREA) / 31536$$

Although the mean annual flow gives an idea of a stream's power potential, a firmer knowledge of the stream's flow regime, as obtained from a flow duration curve, is needed. The flow duration curve depends on the type of soil on which the rain falls. If it is very permeable (sand) the infiltration capacity will be high and the groundwater will be a large proportion of flow in the stream. If it is impermeable (rock) the opposite will be the case. The catchments of high permeability and large groundwater contributions will therefore tend to have more regular discharges with less fluctuating flow than rocky catchments, where the variations will be great and will reflect the incidence of rainfall to a much greater extent.

In UK, for instance, soils have been categorised into 29 discrete groups that represent different physical properties and hydrological response. The classification system is referred to as the Hydrology of Soil Types (HOST) classification. By measuring the areas of each of these categories, within the catchment, as a proportion of the whole, the BFI (Base Flow Index) can be computed. Knowing the BFI of the catchment, a standardised FDC can be selected from figure 3.11. Multiplying the ordinates of the selected FDC by the catchment Qm, the particular flow duration curve of the site is obtained.

In Spain, the distribution of the soils has been identified from the Soil Map of the European Communities (CEC, 1985), which is based on the FAO/UNESCO Soil Classification of the World. Nineteen soils are represented within the gauged catchments considered in the study.

There are actually many watershed models that permit calculation of the runoff for a certain catchment basin taking into account the average daily rainfall, the potential evapotranspiration, the soil composition, the basin slope and area, the stream length, and other parameters. All those programs allow an analysis of the snowmelt and its contribution to the discharge, and also the creation of flood inundation maps, flood depths maps and flood impact maps.

3.4.4. FDCs for particular months or other periods

It is always important to know when, during the year, water will be available for generation. This is required when considering the economics of schemes in those networks where tariffs, paid by utilities to independent producers, vary with the season of the year and time of day.

FDCs can be produced for particular periods of time as well as for particular years. Indeed, it is standard practice to prepare FDCs for six "winter" months and six "summer" months. This can be taken further, to obtain FDCs for individual months, if so desired. It is simply a matter of extracting the flow records for a particular month from each year of record and treating these data as the whole population. If sufficient flow records for this process do not exist, then the rainfall record can be used.

3.4.5. Water Pressure or "head"

3.4.5.1.Evaluation of gross head

The gross head is the vertical distance that the water falls through in giving up its potential energy (i.e. between the upper and lower water surface levels).

Field measurements of gross head are usually carried out using surveying techniques. The precision required in the measurement will limit the methods that can be employed.
In the past, the best way to measure gross head was by levelling with a surveyor's level and staff, however this was a slow process. Accurate measurements were made by a tachometer or less accurately by a clinometer or Abney level. Nowadays with digital theodolites, electronic digital and laser levels and especially with the electronic total stations the job has been simplified.

The modern electronic digital levels provide an automatic display of height and distance within about 4 seconds with a height measurement accuracy of 0.4 mm, and the internal memory that can store approximately 2,400 data points. Surveying by Global Positioning Systems (GSM) is now used widely and a handheld GPS receiver is ideal for field positioning, and rough mapping.

3.4.5.2. Estimation of net head

Having established the gross head available, it is necessary to make allowances for the losses, from trash racks, pipe friction, bends and valves. In addition to these losses, certain types of turbines need to discharge their water to atmosphere, above the level of the tail water (the lower surface level).

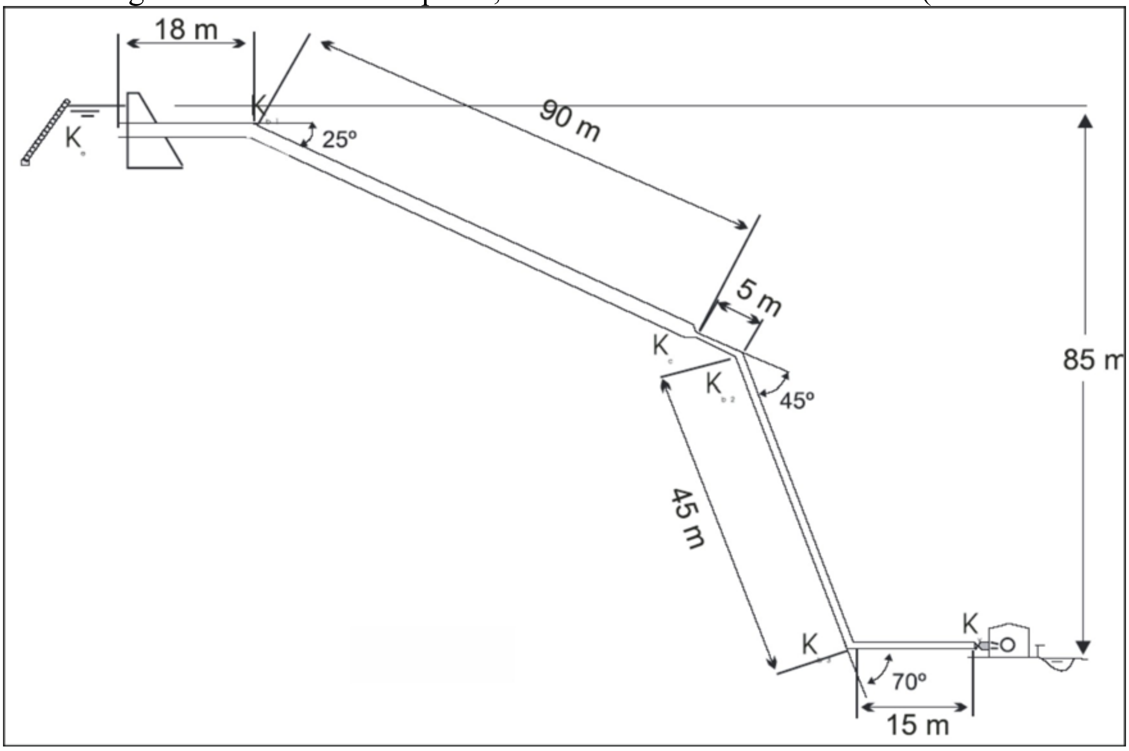


Figure 3-11 Conveyance system (example 3.1)

The gross head minus the sum of all the losses equals the net head, which is available to drive the turbine. Example 3.1 will help to clarify the situation:

Example 3.1

Figure 3.13 shows the pipe layout in a small hydropower scheme. The nominal discharge is 3 m3/s and the gross head 85 m. The penstock is 1.5 m diameter in the first length and 1.2 m in the second one. The radius of curvature of the bend is four times the diameter of the pipe. At the entrance of the intake there is a trash rack inclined 60^0 with the horizontal. The rack is made of stainless steel flat bars, 12 mm thick and the width between bars is 70 mm. Estimate the total head loss?

According to experience the velocity at the entrance of the rack should be between 0.25 m/s and 1.0 m/s. The required trash rack area is estimated by the formula:-

$$S = \frac{1}{K_1}\left(\frac{t}{t+b}\right)\frac{Q}{V_0}\frac{1}{\sin\alpha}$$

where S is the area in m2, t the bar thickness (mm), b the distance between bars (mm), Q the discharge (m3/s), V_0 the water velocity at the entrance and K_1 a coefficient which, if the trash rack has an automatic cleaner, is equal to 0.80. Assuming $V_0 = 1$ m/s, S=5.07 m^2. For practical reasons a 6 m^2 trash rack may be specified, corresponding to a $V_0 = 0.85$ m/s, which is acceptable. The headloss traversing the trash rack, as computed from the Kirschner equation

$$h_r = 2{,}4\left(\frac{12}{70}\right)^{3/4}\frac{0{,}8^2}{2\cdot 9{,}81} = 0{,}007\ m$$

The friction losses in the first penstock length are a function of the water velocity, 1.7 m/s. The entrance to the pipe has a good design and coefficient $K_e = 0.04$ (see figure 2.11). The head loss in the first length according to Manning's equation is:

$$\frac{h_F}{L} = 0{,}00177;\ h_f = 0{,}19m$$

The headloss coefficient in the first bend is $K_b = 0.085$ (one half of the corresponding loss of a 901 bend); in the second $K_b = 0.12$ and in the third $K_b = 0.14$ The taper pipe, with an angle of 30^0, gives a loss in the contraction $h_c = 0.02$ m (for a ratio of diameters 0.8 and a water velocity in the smaller pipe =2,65 m/s)

The friction headloss in the second length is computed in the same way as the first one, and accordingly

$$\frac{h_F}{L} = 0{,}0169;\ h_f = 1{,}10m$$

The coefficient of headloss in the gate valve is $K_v = 0.15$. Therefore the headloss due to friction is estimated to be

0.19 + 1,10 = 1.29 m

The additional headlosses will be as follows:-

• In the trash rack		0.007 m
• In the pipe entrance	0.04 x 0,147	0.059 m
• In the first bend	0.085 x 0.147	0.013 m

• In the second bend	0.12 x 0,359		0.043 m
• In the third bend	0.14 x 0,359		0.050 m
• In the confusor	0.02 x 0,359		0.007 m
• In the gate valve	0.15 x 0,359		0.054 m
		Headlosses	0,233 m

The total head loss is equal to 1,29 m friction loss plus 0,23 m in local losses, giving a net head of 83.48 m. This represents a loss of power of 1,8% which is reasonable.

3.5. <u>Residual, reserved or compensation flow</u>

Uncontrolled abstraction of water from a watercourse (e.g. passing it through a turbine) even if it is returned to the stream close to the intake, could lead to sections of the watercourse being left almost dry with serious impacts on aquatic life.

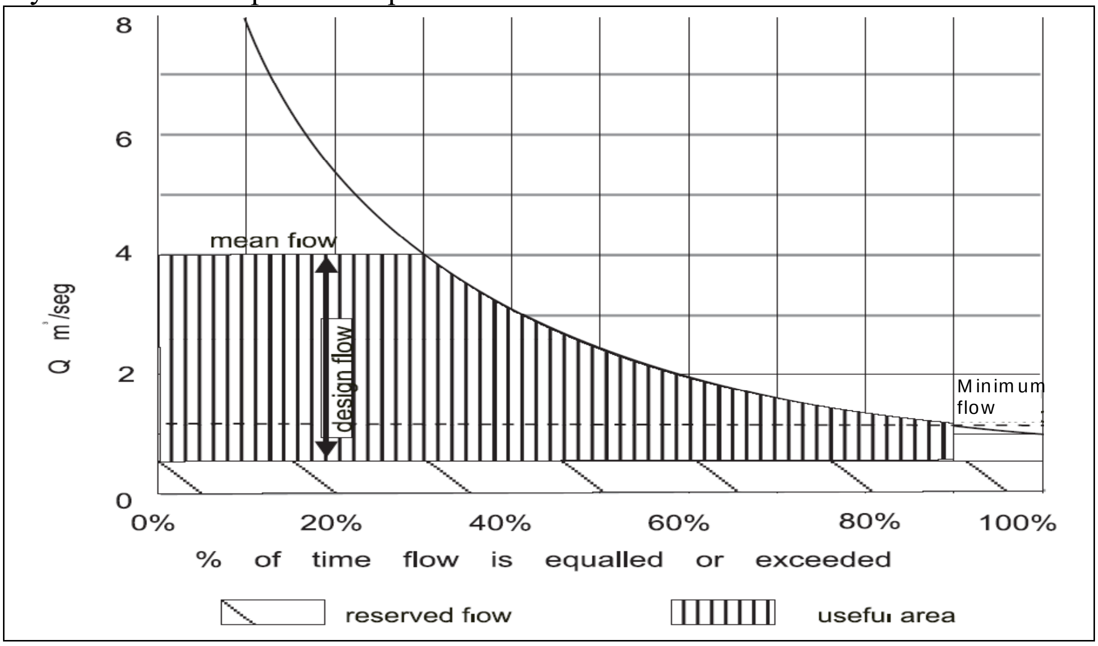


Figure 3-12 Residual flow

To avoid this happening, permission to divert water through a hydro turbine or a licence to abstract from a river or stream will almost always specify that a certain residual flow should remain. The residual flow is sometimes called other names, depending on the country, or authority responsible, e.g. "reserved flow", "prescribed flow" and "compensation flow" are terms commonly used.

This residual flow should be carefully evaluated since a flow that is too small would cause damage to aquatic life in the stream. On the other hand an unnecessarily large flow effects the power production and especially so in periods of low flow, thus reducing the benefits of the installation.

3.6. <u>Estimation of plant capacity and energy output</u>

The FDC provides a means of selecting the right design discharge, and by taking into account the reserved flow and the minimum technical turbine flow, an estimate of the plant capacity and the average annual energy output.

Figure 3.12 illustrates the FDC of the site it is intended to evaluate. The design flow has to be identified through an optimisation process, studying a range of different flows, which normally gives an optimum design flow significantly larger than the difference between the mean annual flow and the reserved flow. Once the design flow is defined and the net head estimated, a suitable type of turbine must be identified (refer chapter 6).

Figure 3.12 shows the useable region of the flow duration curve. Every selected turbine has a minimum technical flow (with a lower discharge the turbine either cannot operate or has a very low efficiency) and its efficiency is a function of the operating discharge.

The average annual energy production (E in kWh) is a function of:

$E = fn(Q_{median}, H_n, \eta_{turbiner}\ \eta_{gearbox}, \eta_{transformer}, y, h)$

Where:

Q_{median} = flow in m3/s for incremental steps on the flow duration curve
H_n = specified net head
$\eta_{turbine}$ = turbine efficiency, a function of Qmedian
$\eta_{generator}$ = generator efficiency
$\eta_{gearbox}$ = gearbox efficiency
$\eta_{transformer}$ = transformer efficiency
y = specific weight of the water (9.81 KN/m3)
h = number of hours for which the specified flow occurs.

The energy production can be calculated by dividing the useable area into vertical 5% incremental strips starting from the origin. The final strip will intersect the FDC at Q_{min} or $Q_{reserved}$ which ever is larger. For each strip Q_{median} is calculated, the corresponding $h_{turbine}$ value is defined for the corresponding efficiency curve, and the energy contribution of the strip is calculated using the equation:

$E = W \times Q_{median} \times H \times ç_{turbine} \times ç_{generator} \times ç_{gearbox} \times ç_{transformer} \times Õ \times h$

Where: W = strip width = 0.05 for all strips except the last one that should be calculated
h = number of hours in a year
y = specific weight of the water (9.81 KN/m3)

The average annual energy production is then the sum of the energy contribution for each strip. The capacity of each turbine (kW) will be given by the product of their design flow (m^3/s), net head (m), turbine efficiency (%), and specific weight of the water (kNm^{-3}).

In Chapter 6, curves of turbine efficiency against flow for the commercial turbines are shown. Table 3.2 gives the minimum technical flow for different types of turbines as a percentage of the design flow.

Table 3-2 Minimum technical flow of turbines

Turbine type	Q_{min} (% of Q_{design})
Francis	50
Semi Kaplan	30
Kaplan	15

Pelton	10
Turgo	20
Propeller	75

3.6.1. How the head varies with the flow and its influence on the turbine capacity

Depending on the river flow and the flow admitted to the turbines, the head can vary significantly.

The upstream water level may vary with flow. If the intake pond is controlled by an overflow weir without any gates, the water level will rise with the flow. However, if the intake pond is controlled by gates in order to operate at a specified reservoir level, the water level may remain constant even during high flow periods. During low flow periods, the upstream water level may also be lower due to draw down of the reservoir.

The head losses in the adduction system varies with the square of the admitted flow, and thus for low flow seasons with low turbine flow the head loss in the adduction system can be substantially reduced.
The downstream water level may vary with the flow. This depends on the water body into which the water is discharged. If discharging directly into a headwater pond controlled by gates in a downstream development, the water levels may remain almost constant even for higher flows. If the water is discharged into a natural stream, the water levels again may vary considerably.

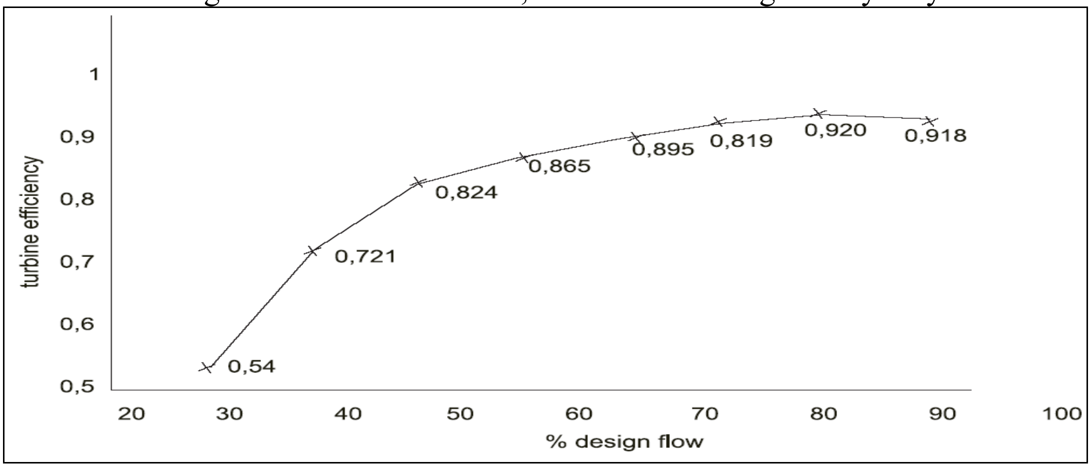


Figure 3-13 Example of turbine efficiency as a function of flow

In medium and high head schemes the head can be considered constant, because variations in the upper or lower surface levels are small compared with the head. In low head schemes, when the flow increases over the value of the rated flow of the water surface level, both in the intake and in the tailrace, may increase but at different rates, so that the head can potentially increase or decrease.

If a turbine operates with a head $H_1 = Z_{Upstream} - Z_{Downstream}$, other than the rated head H_d, the flow admitted by the turbine will be:- note finished at 13/0305

$$Q_1 = Q_d \cdot \sqrt{\frac{H_1}{H_d}} \qquad (3.7)$$

Headwater level is normally kept at spillway crest level when all the river discharge passes through the turbines. When the river discharge exceeds maximum turbine discharge, excess flow will pass over the spillway. The reservoir level corresponding to different spillway flows can easily be calculated. In this case measuring the head on the spillway crest we have at the same time the level of the intake water surface and the river discharge (including the discharge from the turbines).

The tailrace level is more difficult to estimate. The Hydrologic Engineering Centre (HEC) of the US Army Corp of Engineers in Davis, California, has developed a computer program, HEC RAS, that can be downloaded free of charge from INTERNET (http://www.usace.army.mil). Although freely available and straightforward to use, the results as always depend on the quality of the input. Figure 3.14 shows an example of how the head varies with the flow in a real case and its influence on the power delivered at different river discharges.

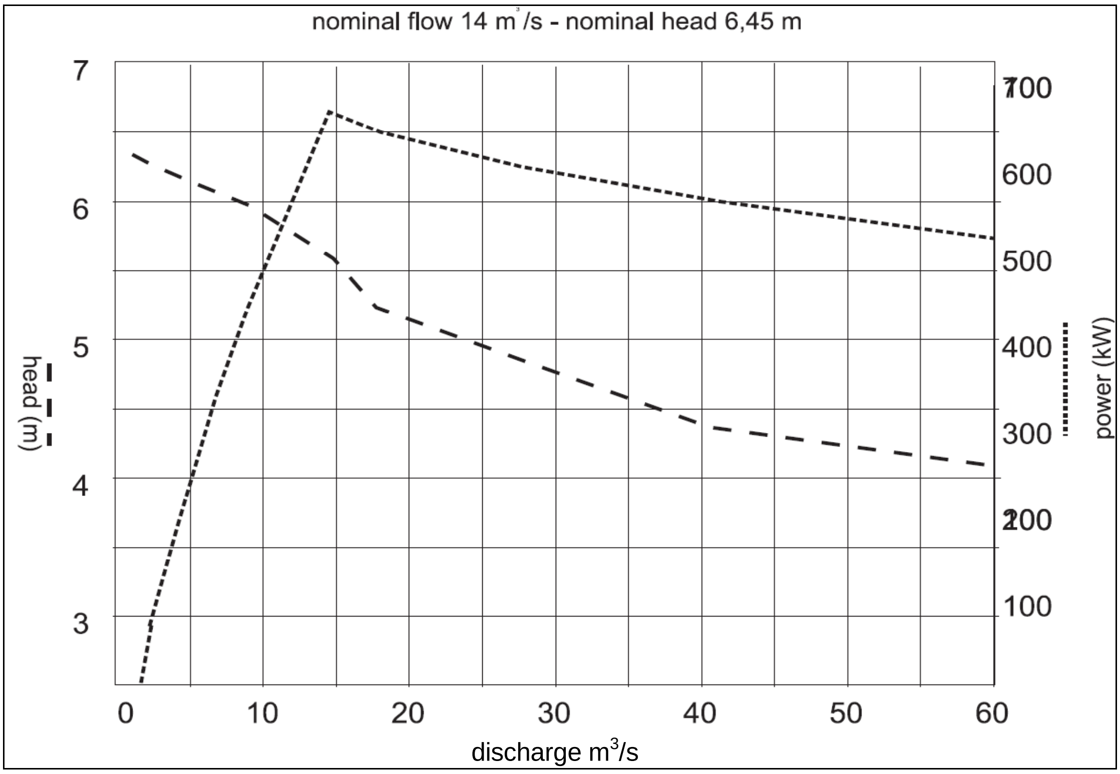


Figure 3-14 Variation of net head vs. river flow

3.6.2. Peaking operation

Electricity prices at peak hours can be substantially higher than in off-peak hours, hence the interest in providing an extended forebay or pound, big enough to store the water necessary to operate, at maximum during peak hours. To evaluate this volume:-

Q_R = river flow (m^3/s)
Q_D = rated flow (m^3/s)
Q_P = flow needed to operate in peak hours (m^3/s)
Q_{OP} = flow needed to operate in off-peak hours (m^3/s)
t_P = daily peak hours
t_{OP} = daily off-peak hours (24 - t_P)
Q_{res} = reserved flow (m^3/s)
Q_{tmin} = minimum technical flow of turbines (m^3/s)
H = head (m)

The volume V will be given by:-

$V_R = 3.600 \cdot t_P \cdot (Q_P-(Q_R-Q_{res}))$
If the pound should be refilled in off-peak hours
$t_P (Q_P-(Q_R-Q_{res}))\delta t_{OP} (Q_R-Q_{res})$
hence :-

$Q_p \leq t_{op}-t_p/t_p \cdot (Q_R-Q_{res})$

the flow available to operate in off-peak hours will be:-

$$Q_{OP} = \frac{24(Q_R - Q_{res}) - t_P Q_P}{t_{OP}} > Q_{\min}$$

3.7. Firm energy

Firm energy is defined as the power that can be delivered by a specific plant during a certain period of the day with at least 90 –95% certainty. A run-of-river scheme has a low firm energy capacity. A hydropower plant with storage does, however, have considerable capacity for firm energy.

If the hydropower scheme is to be connected to an electrical network that includes several types of power and where the hydropower installations are geographically distributed, as is the case in Europe, the firm power capacity of singular plants may, not be important.

If a small hydro scheme has been developed as the single supply to an isolated area, the firm energy is extremely important. As failure to meet demand, could result in power shortages and blackouts.

3.8. Floods

The stream flow is the fuel of the plant, but stream flow in the form of floods is also a potential threat to all structures built in rivers. Therefore the hydrological investigation must address not only water availability for production, but also frequency and severity of floods so as to design flood protection and control into the scheme. The design flood is not only characterized by its peak value of flow, but a hydrograph flood flows should show the distribution of the flow over time.

3.8.1. Flood Control Design

It is important to distinguish between inflow floodwater and required spillway capacity, since considerable routing effects take place in reservoirs.
For reservoirs with dams that are at risk from high floodwaters it is usual to consider two differing criteria:

1. **Maximum Inflow Design Flood** - that the facilities should be able to accommodate the floodwater, without unacceptable risk of dam failure or other serious damages to the structures. This flood is normally defined as the PMF, (Probable Maximum Flood) or similar.

2. **Normal Operation Design Flood** - that the facilities should be able to accommodate floodwater without exceeding normal conditions of operation. This flood is usually defined as a flood with a specific return period.

Whereas for medium and low hazard dams, the requirements often discard the routing effects of the reservoir, and specify that the spillway capacity shall exceed the peak flow of a flood with a specific return period, typically between 100 and 1000 years.

The requirements regarding the design flood are usually specified in national legislation or industry guidelines, and distinguish between high, medium and low hazard structures. In table 3.3 below, typical design flood requirements are given:

Table 3-3 Typical design flood criteria

Structure	Design Flood
High Hazard	Maximum Inflow Design Flood: PMF, Probable Maximum Flood or similar. Alternatively 10.000-year flood Normal Operation Design Flood: 1000-year flood.
Medium Hazard	100- to 1000-year flood
Low Hazard	Typically 100-year flood although in some countries no formal requirements exist.

With a 100-year flood, an annual probability of occurrence is understood to be 1/100. In other words, the Return Period is the inverse of the frequency. In the table below, the probability of occurrence during different life spans for different event frequencies is shown.

Table 3-4 Probability of occurrence

Life Span Frequency (Return Period)	10 years	50 years	100 years	200 years
0,01 (100)	9,6 %	39 %	63 %	87 %
0,001 (1 000)	1 %	5 %	9,5 %	18 %
0,0001 (10 000)	0,1 %	0,5 %	1%	2%

The economically optimal design flood return period for a specific dam, considering the marginal cost of additional spillway capacity and the cost of failure, is usually higher than the 100-year flood even for low hazard structures.

3.8.2. Statistical analysis of flood data

There are basically two ways of arriving at a design flood:

- Statistical analysis of stream flow records
- Hydrological modelling of the catchment area

Typically, statistical analysis is used for less important structures that would not cause dramatic consequences to life and society in case of failure, whereas hydrological modelling is required for important and potentially dangerous dams in case of failure.

The object of the hydrological modelling is to arrive at a Probable Maximum Flood, or similar, to be used for dam and spillway design. Frequency analysis is a statistical method to calculate the probability of an event based on a series of previous events.

The technique for estimating the return period of flows is straightforward and based on records of annual maximum flows. For the evaluation, a probability distribution that fits to the phenomenon must be chosen. Generally logPearson III is recommended for flood estimation since it allows for non-symmetrical probability distributions around the mean value, which is often the case in hydrology, however the lognormal distribution is still widely used.

The non-symmetrical distribution is expressed in a skew coefficient. LogPearson III and the calculation of the skew coefficient are very sensitive for short data series. Therefore, it is recommended to use a modified skew factor based not only on the actual data series, but also includes general experience for the specific geographical region.

In the graphical method, the annual maximum floods are arranged in order of size and then plotted on probability paper applicable for the desired distribution. Generally the ordinate represents the value and the abscissa represents the probability. The data are expected to fit, as close as possible, to a straight line. The graph can then be used for interpolation, extrapolation or comparison purposes.

In case of extrapolation, effects of errors are magnified and caution is recommended.

In the analytical method, the mean value, standard deviation as well as the skew coefficient (in case of logPearson III) of the logarithmic value of the flow record is calculated. Based on the desired frequency, a frequency factor is read from a diagram. The logarithms of floods corresponding to certain frequencies are then calculated as the mean value plus the standard deviation multiplied with the corresponding frequency factor. The logarithms are then converted to actual flow values.

Both methods are explained in more detail in standard hydrology textbooks.
As an illustrative example the 100-year flood is calculated using the analytical method for the lognormal and logPearson III probability distribution based on the following time series of annual maximum flows:

Flow (m^3/s)	**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
1970-	65	32	45	87	34	29	26	35	42	41
1980-	36	29	55	46	31	26	34	31	39	61

The steps are as follows:
1: Calculate the logarithmic value of the flow records
2: Calculate the mean of the logarithms
3: Calculate the standard deviation of the logarithms
(3b: Calculate the skew factor for LogPearson III)
4: Read the frequency factor for the desired probability (f = 0,01)
5: Calculate the logarithm of the 100-year flow
6: Convert the logarithm to a flow value:

Using the LogNormal distribution the 100-year annual maximum flow is estimated at 83 m3/s, and for the Log Pearson III distribution almost 25 % higher, or 103 m3/s. Which value is the more correct? This example illustrates that even though the methods are straightforward, a good professional judgment is required as to applicability and choice of method.

3.8.3. Hydrological modelling of the catchment area

In order to arrive at a design flood using hydrological modelling, a design rainfall is introduced to a hydrological model comprising various components. The design rainfall is combined with other critical factors such as soil moisture content, snow melting, ground water magazine contents etc.

This task is best left to the experts.

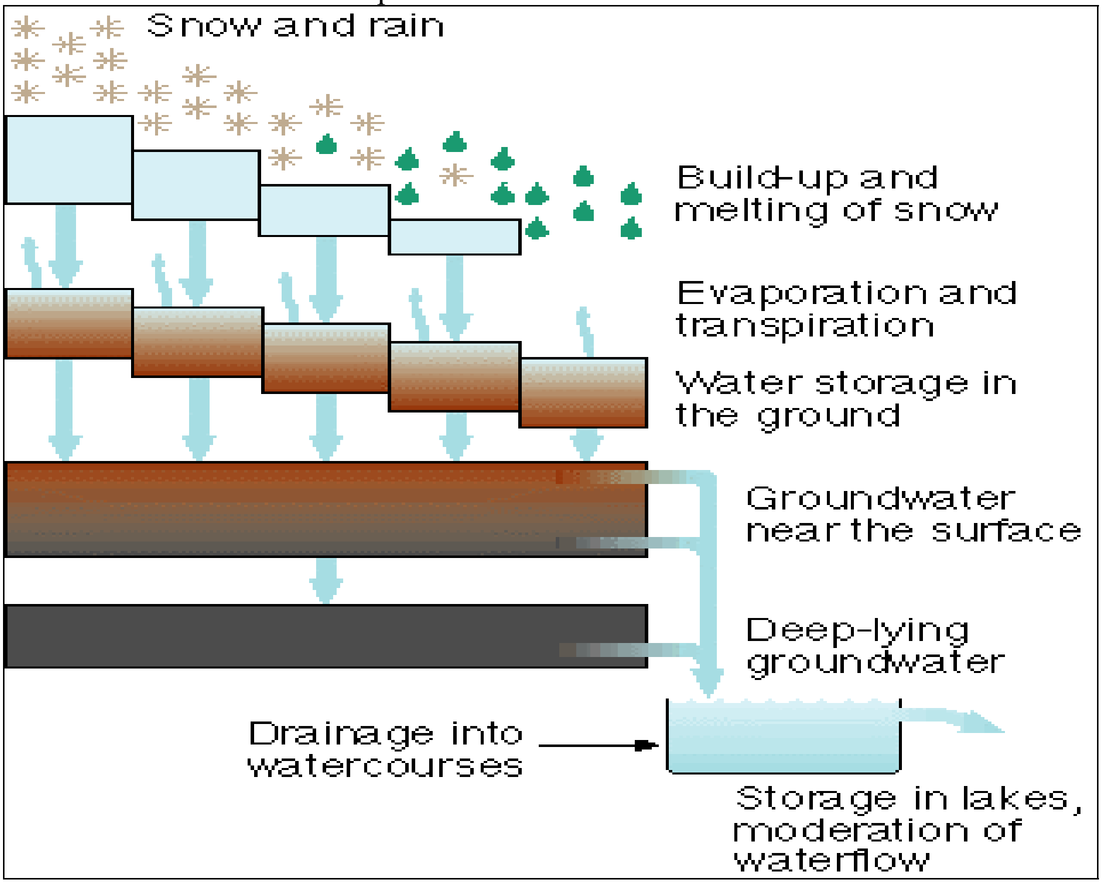


Figure 3-15 Components of hydrological model

Bibliography

1. José Llamas, "Hidrología General. Principios y Aplicaciones". Servicio Editorial de la Universidad del País Vasco, 1933.

2. ISO 1100-1: 1996 "Measurement of liquid flow in open channels. Part 1: Establishment and operation of a gauging station".

3. ISO/DIS 110-2 "Measurement of liquid flow in open channels – Part 2: Determination of the stage-discharge relation" (revision of ISO 1100-2: 1982).

4. ISO 2537: 1988 "Liquid flow measurement in open channels – Rotating element currentmeters".

5. ISO 955-1: 1994 "Measurement of liquid flow in open channels – Tracer dilution methods for the measurement of steady flow – Part 1: General".

6. ISO 3846: 1989 "Liquid flow measurement in open channels by weirs and flumes – Rectangular broad-crested weirs".

7. ISO 3847: 1977: "Liquid flow measurement in open channels by weirs and flumes – End-depth method for estimation of flow in rectangular channels with a free overfall".

8. ISO 4359-1983 "Liquid flow measurement in open channels: Rectangular, trapezoidal and Ushaped flumes".

9. ISO 4360: 1984 "Liquid flow measurement in open channels by weirs and flumes – Triangular profile weirs".

10. ISO 4362: 1992 "Measurement of liquid flow in open channels – Trapezoidal profile"

[i] By Jonas Rundqvist (SERO), Bernhard Pelikan (ÖVFK), Vincent Denis (MHyLab) and Celso Penche (ESHA)

CHAPTER 4: SITE EVALUATION METHODOLOGIES

CONTENTS

4. SITE EVALUATION METHODOLOGIES 72
4.1. Introduction 72
4.2. Cartography 72
4.3. Geochemical Studies 73
4.3.1. Methodologies to be used 73
4.3.2. Methodologies. The study of a practical case. 75
4.3.2.1. The weir 75
4.3.2.2. The open channel 76
4.3.2.3. The channel in tunnel 78
4.3.2.4. The powerhouse 82
4.4. Learning from failures 82

LIST OF FIGURES

Figure 4-1 Schematic representation of the site 74
Figure 4-2 Weir location and structures of both slopes 75
Figure 4-3 Geological section of the colluvial formation 76
Figure 4-4 Geomorphologic scheme of the channel trace 76
Figure 4-5 A schematic cut of the tunnel under the colluvium 79
Figure 4-6 Concrete lining forming the final section of the canal 79
Figure 4-7 A thrust fault, present in the La Rienda tunnel 81
Figure 4-8 results of the jet-grouting operation 82
Figure 4-9 Scheme of Ruahihi canal 83
Figure 4-10 Longitudinal scheme of La Marea plant 86
Figure 4-11 Plan view f La Marea plant 86

LIST OF PHOTOS

Photo 4-1 General view of the right-side slope 77
Photo 4-2 Local instabilities generated during the excavation works 77
Photo 4-3 One of the existing sliding scarps before the beginning of the works 78
Photo 4-4 A view of the Cordiñanes colluvium, under which the tunnel runs 78
Photo 4-5 View of tunnelling works 80
Photo 4-6 View of the tunnel lining 80
Photo 4-7 View of the tunnel lining 81
Photo 4-8 The effects of failure 84
Photo 4-9 La Marea basin 85
Photo 4-10 Weir undermined by seepage 87
Photo 4-11 Weir undermined by seepage 87
Photo 4-12 Channel destroyed by uplift pressure 88

4. SITE EVALUATION METHODOLOGIES[i]

4.1. Introduction

Adequate head and flow are necessary requirements for hydro generation. Consequently these parameters are important factors in site selection.

Chapter 3 outlines the available methodologies for evaluating the flow that is available for power production.

In this chapter, the methodologies needed, in order to evaluate the suitability of a site for hydropower development, are presented.

The gross head may be rapidly estimated, either by field surveying or by using a GPS (Global Positioning System) or by orthophotographic techniques. With the aid of the engineering hydraulic principles outlined in Chapter 2 the net head can be determined.

Nevertheless, the selection of the most appropriate technical solution for the site will be the result of a lengthy, iterative process, where the topography and the environmental issues for a particular site, are most important. That is why a thorough knowledge of the principles is needed to avoid dangerous failures in the operation of the plant.

Surveying technologies are undergoing a revolutionary change, and the use of the technologies mentioned above may greatly assist in scheme design and reduce its cost.

4.2. Cartography

In industrialised countries, scaled maps are usually available. The E.U. territory has been or is being digitised, and cartography at scale as large as 1:5 000 is already available. On the other hand, in the developing countries, the developer will be fortunate if he can find maps at 1:25 000.

Aerial photographs of topography can be substituted for maps if they cannot be found at the required scale. However aerial photographs are unlike maps in one important respect. A map has a uniform or controlled variable scale, the latter being dependent on the choice of map projection. The aerial photograph, on the other hand, does not have a constant or uniformly changing scale. Aside from lens imperfections, which for all practical purposes can be considered negligible, two major factors are responsible for variations in the scale of a photograph:-

1. The topographical relief - land, no matter how flat, is never horizontal – and…
2. The tilt of the optical axis of the camera.

Modern cameras are able to remove distortion, resulting from their axial tilt. Furthermore aerial photographs can be viewed stereoscopically or in three dimensions. The stereoscopic effect enables the geologist to identify rock types, determine geologic structures, and detect slope instability and the engineer is able to gather data necessary for a dam, open channels and penstock construction.

Depending on the required accuracy, the digitised photographs can be geocoded (tied to a co-ordinate system and map projection) and orthorectified. Distortion from the camera lens is removed by using ground control points from maps, survey data or clients GPS vectors. This is a very cost-effective way to orthorectify aerial photographs. Resolutions of 30 cm to one metre can be expected with digital orthophotos. Both hard copy and digital orthophotos in diskettes, or CDROM can be produced.

With these maps it is possible to locate the intake, trace the open channel and penstock and locate the powerhouse, with precision enough for the feasibility studies and even for the contractors to engage in the bidding phase for construction.

With stereoscopic photographs geologic problems can often be spotted, especially those concerning slope stability that can cause dangerous situations.

4.3. Geochemical Studies

Very often, the need to proceed with detailed geological studies of a site, are underestimated. In many cases with regrettable consequences - seepage under the weir, open channel slides etc.

Fortunately in the E.U. member states and in many other countries all over the world, good geological maps permit initial estimates, for the security of the dam foundations, the stability of the slopes and the permeability of the terrain. However sometimes this information, should be complemented, with fieldwork particularly, drilling and sampling.

Hydraulic structures should be founded on level foundations, with adequate side slopes and widths, not subject to stability problems. There are a good number of slope stability computer programs, ranging from a simple two-dimensional approach to the sophisticated three-dimensional full colour graphic analysis. The catalogue of failures, especially in channel design is so large that a minimum geomorphologic study of the terrain should be recommended in the first phase of the project. The problem is especially acute in high mountain schemes, where the construction may be in a weathered surface zone, affected by different geomorphologic features such as soil creep, solifluction, rotational and planar soil slides and rock falls.

The weir and its corresponding reservoir can be affected by the instability of the superficial formations that can be present within its zone of influence, but at the same time the pond itself can affect these same formations. If the weir has to be founded on unconsolidated ground the variation of water level can generate instability on the reservoir's wetted slopes.

Along the open channel many geomorphologic features can adversely affect its selected line, which, together with a steep slope of the terrain, may lead to potential instability. Colluvial formations, a product of the surface mechanical weathering of the rock masses, and solifluction processes, are very active in high mountain environments where the subsoil is seasonally or perennially wet – these are some of the features that can compromise channel stability.

Drainage treatments, bench constructions and gunnite treatments, among many others, may be recommended. At the end of the canal, the forebay acts as a mini-reservoir for the penstock. Frequently, authorities require that all the water retaining embankment sections undergo stability analysis regardless of their configuration. The layout of the penstock, usually placed on a steep slope, poses problems both for its anchoring blocks and visual impact.

Deep in the valley, frequently built on an old river terrace, the powerhouse foundation poses problems that many times only can be solved by using techniques as up today as the jet grouting (see 4.2.2.4).

4.3.1. Methodologies to be used

In geological science, there is a wide spectrum of geomorphologic techniques that can be used including the most common ones:-

- **Photogeology**. As mentioned above photogrammetry - at scales from 1:10 000 to 1:5 000 – allows the geologist to identify rock types, determine geologic structures, and detect slope instability.
- **Geomorphologic maps**. The result of photogrammetric analysis complemented with the results of the field survey must be combined on a Geomorphologic Map. This map is based on a topographic one and is drawn at a scale between 1:10 000 and 1:5 000, duly classified using simple symbols, should show all the surface formations affecting the proposed hydraulic structures.
- **Laboratory analysis.** Traditional laboratory tests such as soil grading and classification, and triaxial consolidation facilitate the surface formation classification. The results should be included in the geomorphic map.
- **Geophysical studies**. A geophysical investigation either electrical or seismic (by refraction) will contribute to a better knowledge of the superficial formation thickness, the location of the landslide sections, the internal water circulation, and the volumetric importance of potentially unstable formations.
- **Structural geological analysis**. Although not a proper geomorphologic technology can help to solve problems in the catchment area and in those cases where hydraulic conduits must be tunnels in rock massifs. The stability of the rock and seepage in the foundation of hydraulic structures are problems that can be solved by this methodology, avoiding dramatic incidents during the operation.
- **Direct investigations - Borehole drilling.** This is uncommon for small hydro scheme development. However when the dam or weir has to be founded in unconsolidated strata, a drilling programme, followed by laboratory tests on the samples extracted is essential. Some of these recommended tests are:-
    1. Permeability tests in boreholes, such as Lugeon or Low Pressure Test, to define the water circulation in the foundation.
    2. Laboratory tests to determine the compressive strength of the samples to define their characteristics.

Complementing the above tests a geophysical refraction seismic essay to define the modulus of dynamic deformation of the rock massif in depth can be recommended in the case of high dams.

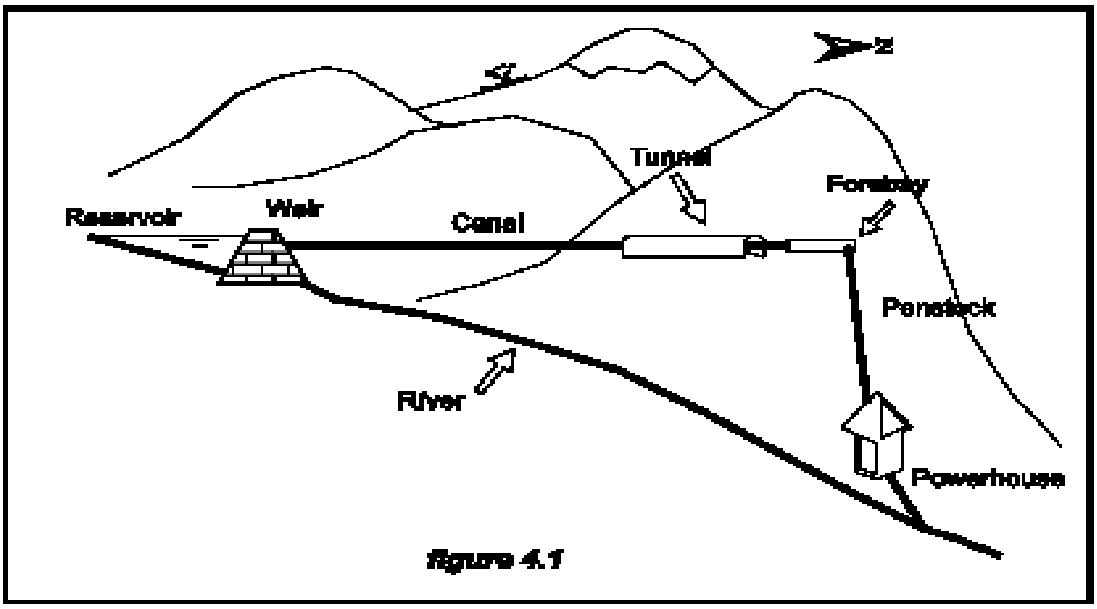


Figure 4-1 Schematic representation of the site

4.3.2. <u>Methodologies. The study of a practical case.</u>

A short report on the geomorphologic techniques used in the Cordiñanes scheme, a high mountain scheme located in the Central Massif of Picos de Europa (Leon, Spain) will help to demonstrate the scope of the above-mentioned studies. Figure 4.1 is a schematic representation of the site, which includes:-

- A gravity weir 11.5 meters high over foundations
- A reservoir with a storage capacity of 60 000 m3
- An open channel 2475 m long (776 m are in tunnel)
- A forebay at the end of the tunnel
- A 1.4 m diameter penstock, 650 m long with a 190 m drop
- A powerhouse

4.3.2.1.The weir

International regulations require that if there is a potential for direct shear failure or whenever sliding is possible along joints or faults, rock foundations must be analysed for stability. When necessary, additional rock excavation may be required.

Figure 4.2 shows the weir location and illustrates the entirely different structures of both slopes, the left one, steeper, follows the nearly vertically bedded slate formation; the right one less steep is associated to a colluvial formation.

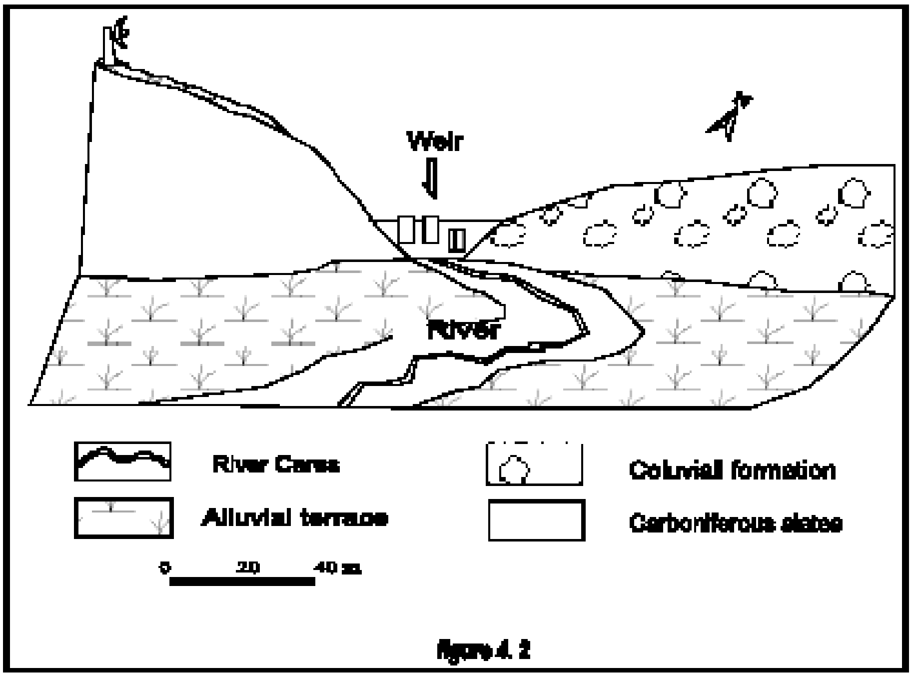


Figure 4-2 Weir location and structures of both slopes

Figure 4.3 shows the geological complexity of the colluvial formation. The borehole drilling B-1 illustrates the existence of an alluvial terrace under the colluvial formation. Each formation behaves in a different way to the requirements of the weir foundation.

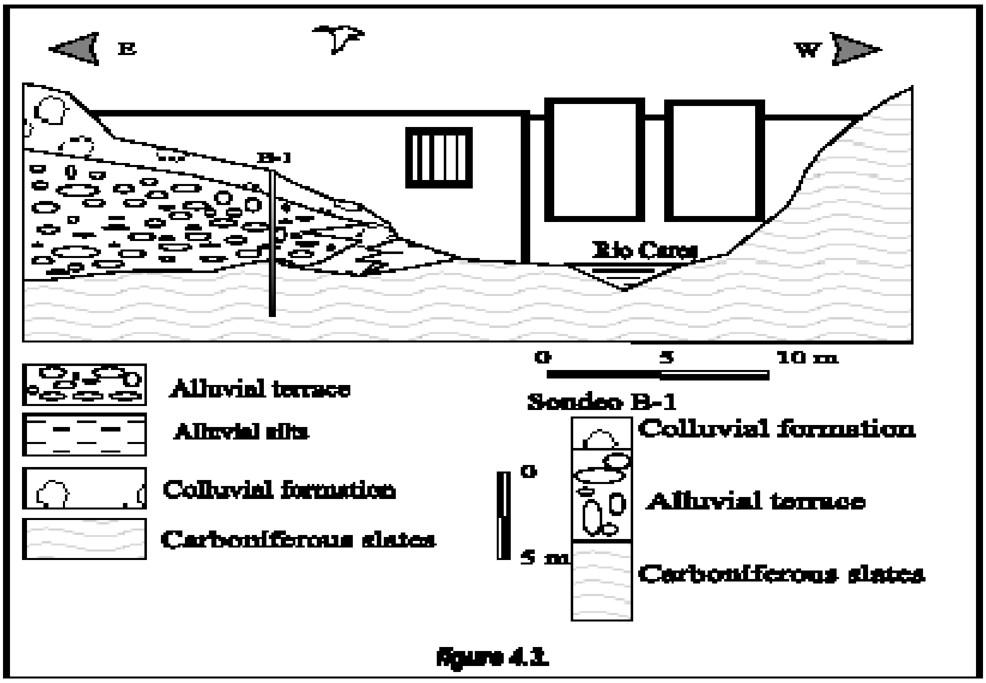


Figure 4-3 Geological section of the colluvial formation

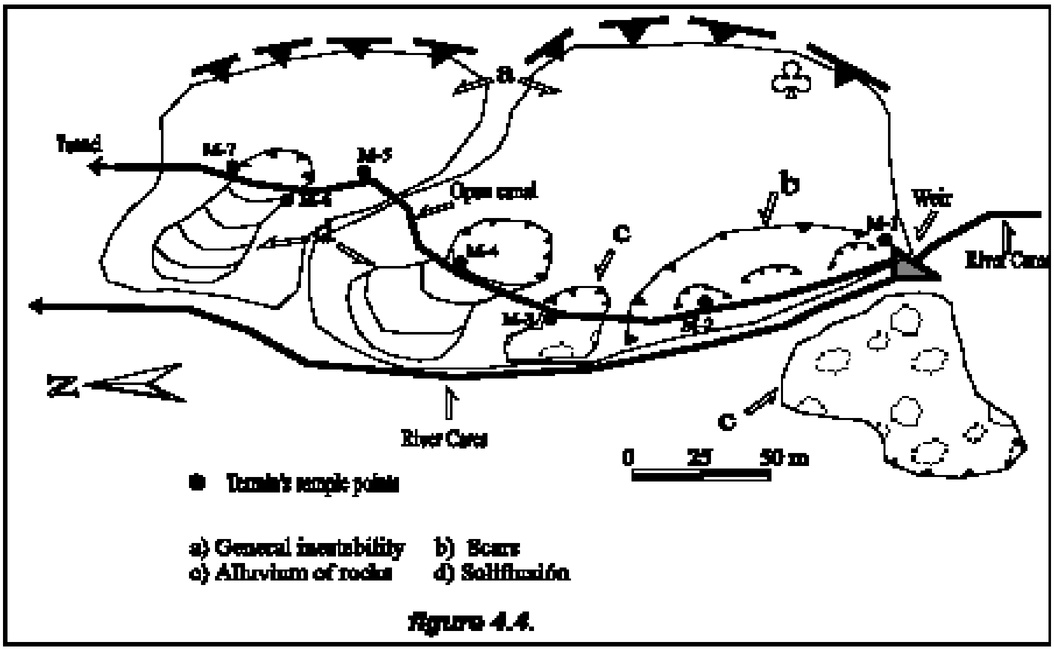


Figure 4-4 Geomorphologic scheme of the channel trace

4.3.2.2.The open channel

Figure 4.4 shows a geomorphologic scheme of the channel trace. Two large independent unstable zones (b and c) can be seen in the right side of the river. Photographs 4.1 and 4.2 show a general view of the right-side slope and the local instabilities generated during the excavation works, just as a detail of one of these instabilities. Photograph 4.3 shows one of the existing sliding scarps before the beginning of the works.

Photo 4-1 General view of the right-side slope

Photo 4-2 Local instabilities generated during the excavation works

The foundation of the channel should meet two requirements:-

- Must be stable. Channels are rigid structures and do not permit deformations.
- Should be permeable. Channels do not support thrusts or uplift pressures.

The geologic studies should aim to avoid settlements in the channel and to provide adequate drainage to hinder the thrust and uplift stresses. The study should conclude with a recommendation to guarantee the stability and suppress the uplift pressures

Photo 4-3 One of the existing sliding scarps before the beginning of the works

Photo 4-4 A view of the Cordiñanes colluvium, under which the tunnel runs

4.3.2.3.The channel in tunnel

The tunnel construction must comply with the following requirements:-

- The excavation will be conditioned by the geologic formations that must traverse, either a rock massif or a superficial formation.
- The tunnel, being a hydraulic channel should be stable and watertight. Consequently the geologic formations that exist in the massif to be traversed must be known in detail.

Photograph 4.4 shows a view of the Cordiñanes colluvium, under which the tunnel runs. Figure 4.5 shows a schematic cut of the tunnel under the colluvium and figure 4.6 illustrates the concrete lining forming the final section of the canal.

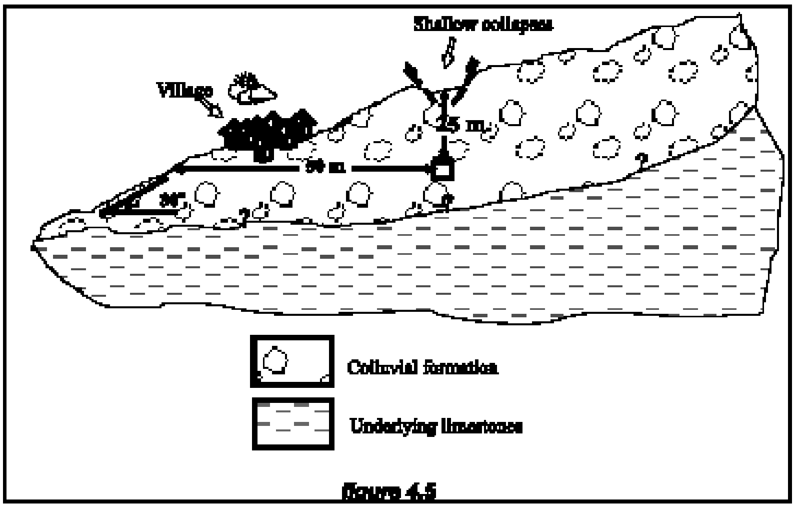


Figure 4-5 A schematic cut of the tunnel under the colluvium

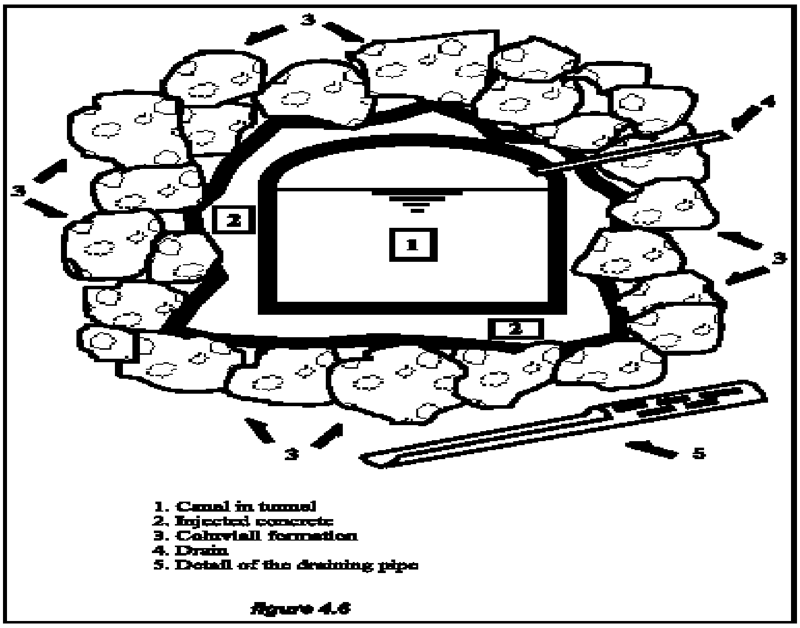


Figure 4-6 Concrete lining forming the final section of the canal

The excavation works were extremely difficult due to the large variety and heterogeneity of the blocks, which varied in size from simple stones to blocks of several cubic meters. The use of large explosive charges was not permitted here. The use of tunnelling machines was not feasible. The excavation had to proceed metre by metre, using small explosive charges to reduce the size of the blocks, which could not be handled (Photograph 4.5).

The concrete lining was also difficult. Zone 2 in figure 4.6 was filled by injecting grout. In fact this injection not only filled the empty space but also enclosed the supporting structure and reinforced the loose terrain around the tunnel. This terrain is very permeable so to avoid lateral pressures or uplift pressures a draining system was put in place.

The construction of tunnels through rocky massifs should take into account two important geologic characteristics:-

- The lithologic variation, along its trace can decisively influence the construction method to be used.
- The structural stability, of the massif along the trace. Even if the massif is lithologically coherent the distribution of the potential discontinuities in stratification planes, joints, fissures - will be far from homogeneous. Once again the knowledge of all those discontinuities must be based on a detailed structural geological study.

As well as the relatively small discontinuities referred above, the designer should also deal with the large tectonic discontinuities -large bending, faults, invert faults- that not only affect the work itself but also the future operation of the canal.

Figure 4.7 shows a thrust fault, present in the La Rienda tunnel, second part of the tunnel of Cordiñanes close to the forebay built right at the end of the tunnel. Due to the strains and deformations supported in the past by this mass of rocks, the rocks originally found were completely altered. The response to this excavation was of course very different from the response of the rest of the massif. Only by knowing the presence of this fault beforehand could the tunnel be excavated without unexpected incidents. As photographs 4.6 and 4.7 illustrate, the supporting structure during the tunnel construction was very different in this area to the one used in the rest of the work.

Photo 4-5 View of tunnelling works

Photo 4-6 View of the tunnel lining

Photo 4-7 View of the tunnel lining

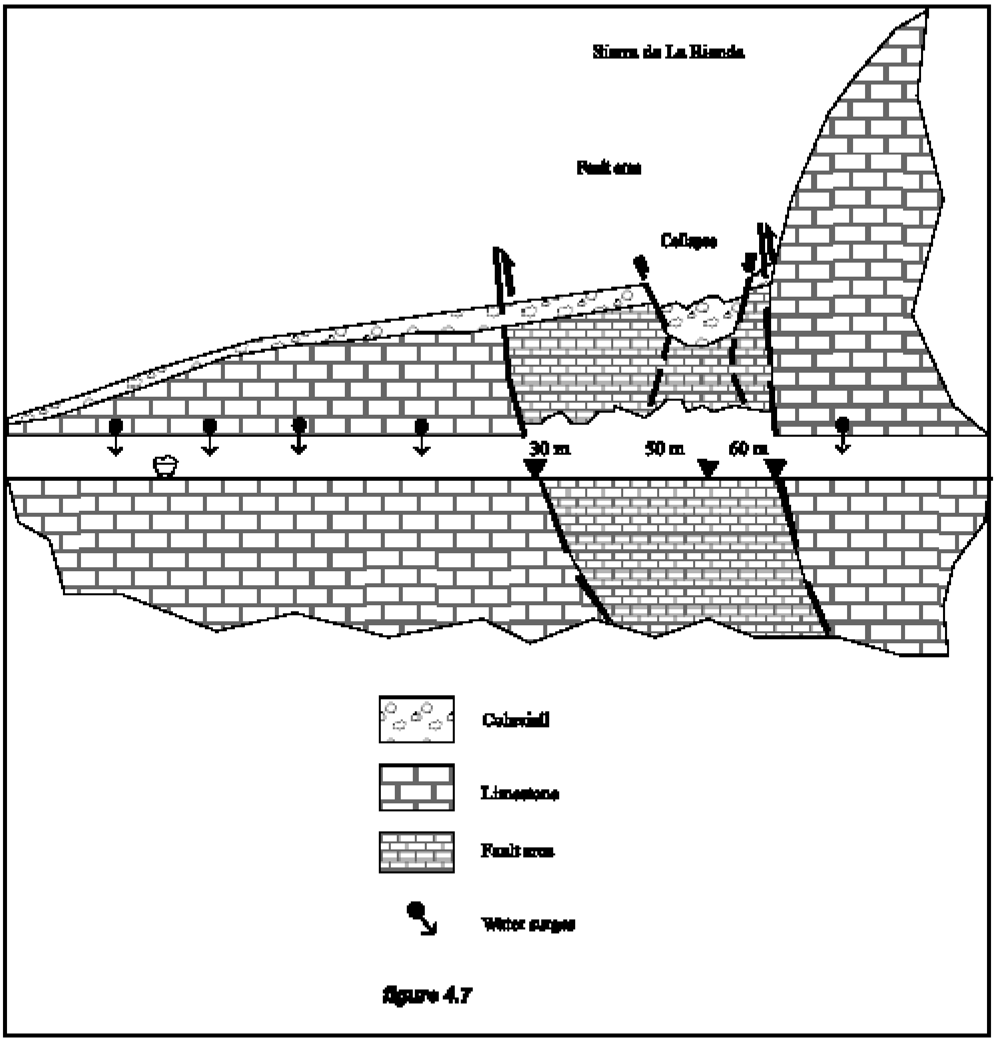


Figure 4-7 A thrust fault, present in the La Rienda tunnel

4.3.2.4.The powerhouse

If the powerhouse is founded on rock, the excavation work will remove the superficial weathered layer, leaving a sound rock foundation. If the powerhouse is to be located on fluvial terraces near the riverbanks that do not offer a good foundation then the ground must be reinforced.

The traditional cement grouting presents some difficulties and in many cases its results are not satisfactory when the terrain is as heterogeneous and permeable as exists in fluvial terraces. A new injection technique, jet grouting, can guarantee the terrain consolidation, replacing alluvial sediments by an injected curtain. The technique, widely used by the DOE (Department of Energy of the U.S) to cut the seepage in the underground storage reservoir for toxic wastes, is however very expensive at present. Figure 4.8 illustrates the results of the jet-grouting operation that was performed to reinforce the terrain supporting the powerhouse.

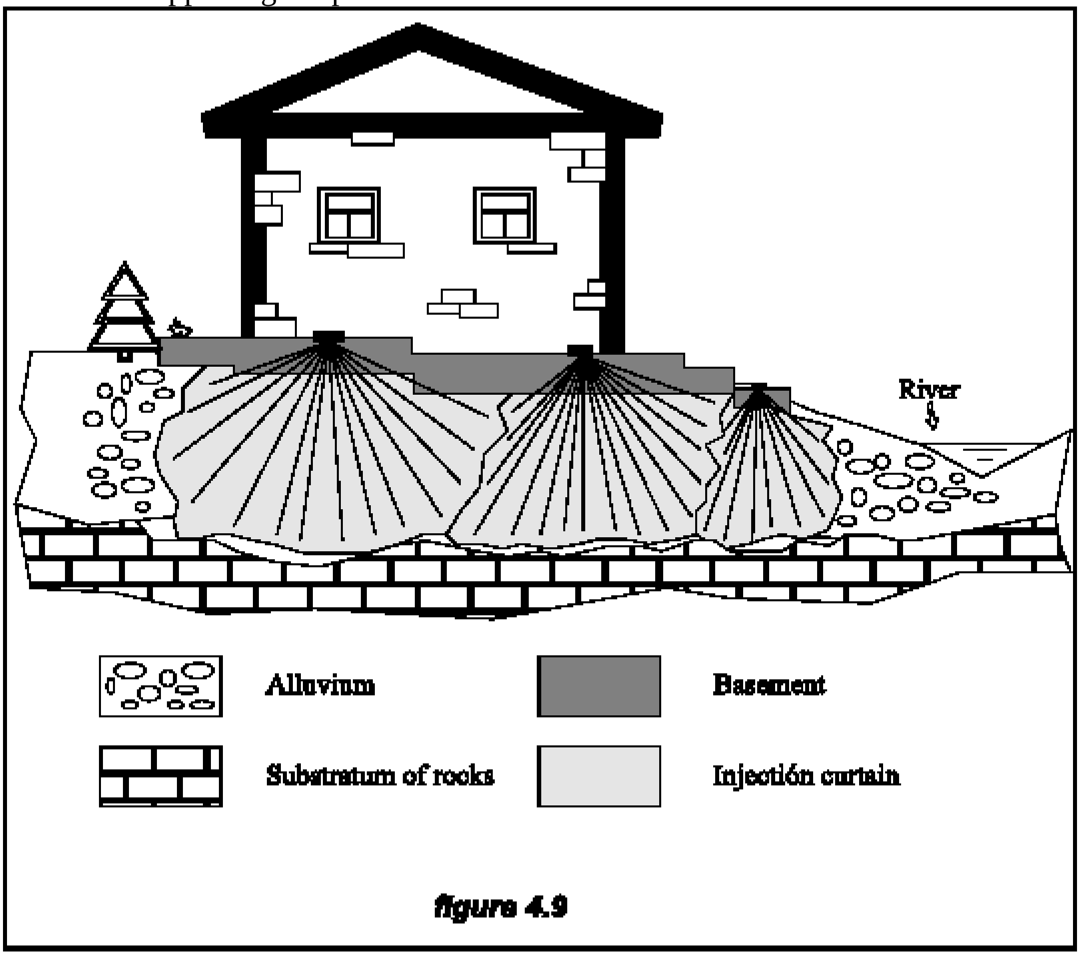


Figure 4-8 results of the jet-grouting operation

4.4. <u>Learning from failures</u>

Two well-known experts, Bryan Leyland of Australia and Freddy Isambert from France, presented at the HIDROENERGIA95 Conference two independent papers dealing with the topic **"Lessons from failures"**.

Mr Leyland quoting Sir Winston Churchill (the famous UK Prime Minister) – **"he who ignores history is doomed to repeat it"** - claims that if one does not want to repeat the mistakes of others, the reasons for their failures must studied and understood. According to Mr Isambert "case studies have shown that a number of small hydro plants have failed because they were poorly designed, built or operated".

The authors presented, with the aid of graphics and photographs, several examples of schemes that failed in the commissioning of the plant or later in the operation, and produces considerable loss of money and dramatic delays.

Professor Mosony wrote in ESHA Info no. 15, "a fair and open discussion about failures is indispensable in order to learn from failures and consequently to avoid their repetition". Quoting Marcus Tullius Ciceron (106-43 BC) "Every human being can make a mistake, but only the idiot persists in repeating his mistake". From the accounts of failures reported at HIDROENERGIA, together with more than 50 others described in the ASCE publication "Lessons Learned from the Design, Construction and Operation of Hydroelectric Facilities", of which 28 of them concern schemes of less than 10 MW capacity, examples have been selected for discussion below. They demonstrate the importance of studying in depth, the stability of canals and the effects of uplift pressure on hydraulic structures.

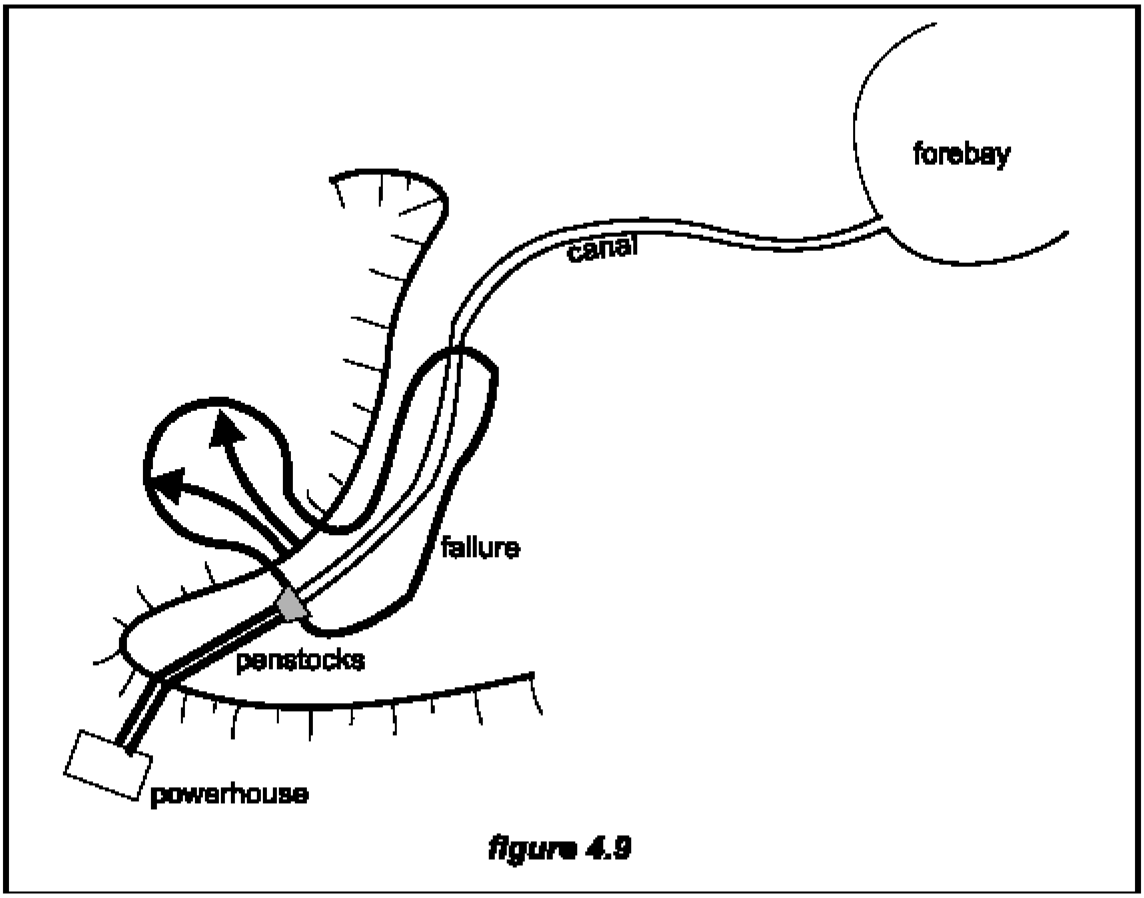


Figure 4-9 Scheme of Ruahihi canal

Ruahihi canal failure (New Zealand)

As shown in figure 4.9 the scheme had a 2000m canal laid along a side slope, leading to 750 m of concrete and steel penstocks. The canal was excavated in soft ignimbrite (debris from a volcanic explosion) and lined with a type of volcanic clay.

The brown ash dried and cracked during construction but due to its unusual characteristics, the cracks did not seal when the canal was filled. So water leaked into the ignimbrite below. When these leaks appeared perforated pipes were driven in to drain the bottom of the slope. This hid the problem and also made it worse because the leaking water caused caverns to form in the fill.

Photo 4-8 The effects of failure

On the day after the scheme was officially opened, a large section of the canal suddenly collapsed. Photograph 4.8 illustrates the magnitude of the catastrophe. Many options were examined and finally it was decided that the only viable option was to replace the failed section of canal with 1100m of pipes.

This increased the length of the penstocks from 750 m to 1850 m and required that water hammer pressures had to be reduced, because the original concrete pipes could only withstand a very limited overpressure. It was necessary to modify the relief valves and the inlet valves so that there would only be a 3% pressure rise under the worst conditions. A surge chamber was not an option because the ground could not take the extra weight. Fortunately the turbine manufacturer was very cooperative and had faith in the ability of his relief valves to limit the pressure rise to 3%, which they did. The refurbishment was completed ahead of time and under budget.

The lessons learned were:

- The characteristics of volcanic materials are highly variable and often undesirable;
- When a canal leaks, be sure the problem is fully understood before repairs commence;
- When the alternative is to abandon a failed scheme, consider the seemingly impossible there may not be a lot to lose!

Photo 4-9 La Marea basin

La Marea canal failure (Spain)

The La Marea scheme has a spiral Francis turbine of 1 100kW installed capacity a discharge of $1.3m^3/s$ and a 100m head. As shown in figure 4.11 the scheme includes a small weir for the water intake, provided with a fish ladder. From the intake a rectangular canal built in reinforced concrete (3 x 2m section) is followed by another 600m long canal in tunnel. At the outlet of the tunnel a reservoir was built to store water for peak operation. The reservoir was built by compressing a mix of sand and clay, and unfortunately proved to be insufficiently watertight. From the reservoir another canal, built with prefabricated sections of concrete with thin steel plates between, brings the water to the forebay, located 100m above the powerhouse.

The canal lies on a steep slope on strongly weathered sandstone. Heavy rain was pouring over the canal both during its construction and during its commissioning. Immediately after opening the intake gate, the reservoir was filled and the water began to seep into the terrain. The wetted sandstone could not resist the shear stresses and a landslide broke the reservoir embankment (photograph 4.9), and large masses of material reached the river, and through the river, to the coast.

The reservoir was replaced by a construction in reinforced concrete, which up to the present day has served no useful purpose. Later on, the second section of the canal and the prefabricated reach, started to leak. The terrain became saturated and, unable to resist the shear stresses, failed in a rotational slide. About 200m of canal were replaced by a low-pressure welded steel pipe that up to now has been performing adequately. The pipe runs under a storage pond, waterproofed by a thermo-welded plastic sheet, and ends in the forebay.

The lessons learned were:-

- Weathered sandstone gives poor performance when resisting landslide, especially on slopes with an angle over 35º to the horizontal.

- Hydraulic canals should be built to guarantee their watertightness; alternatively a drainage system should be devised so the leakage does affect the terrain.
- The replacement of an open canal by a low pressure pipe on a steep slope may be a better option, because it will be watertight and it will require only a few anchorage points

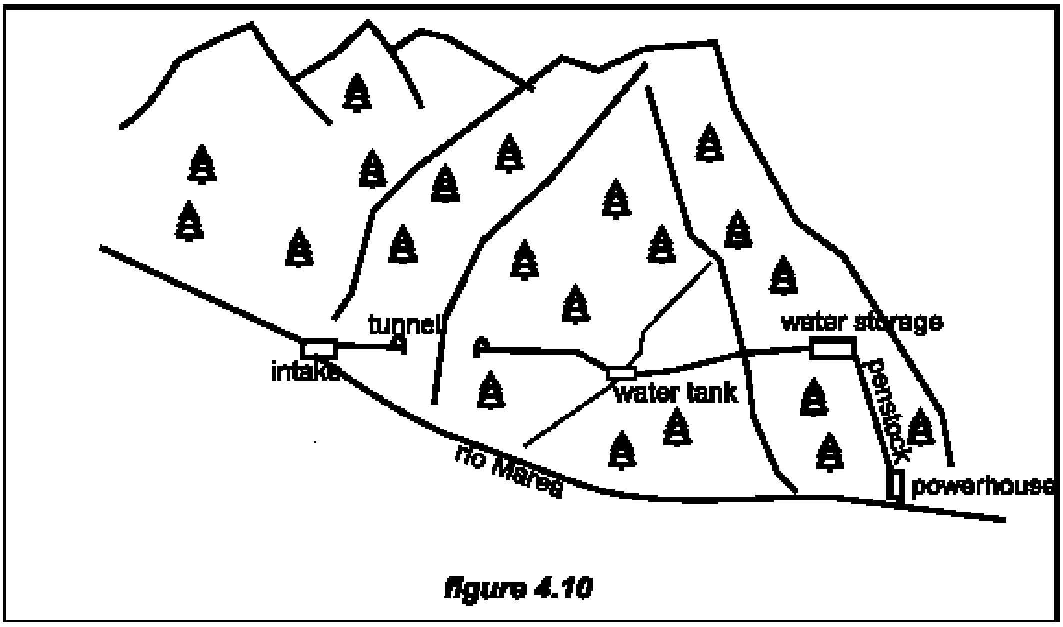


Figure 4-10 Longitudinal scheme of La Marea plant

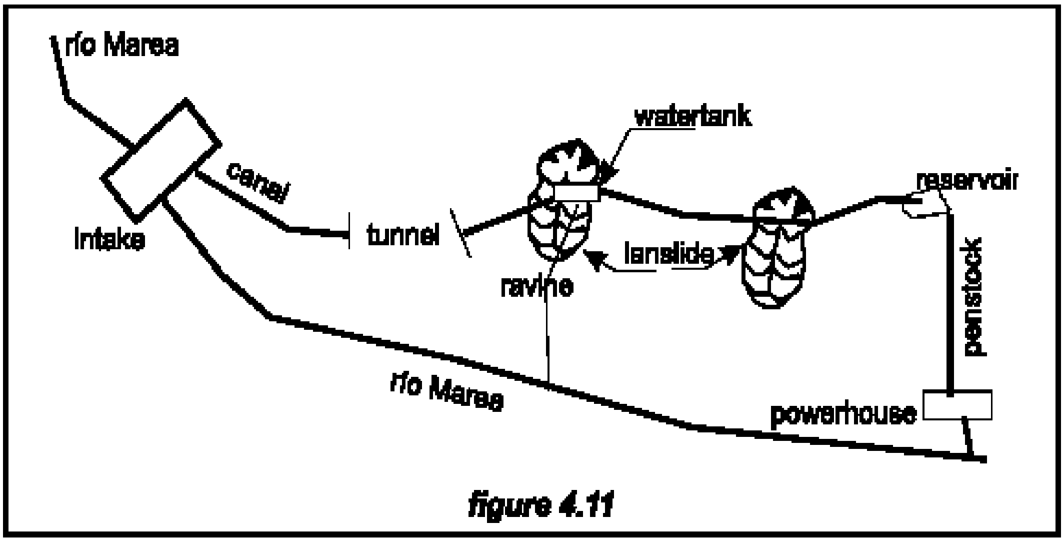


Figure 4-11 Plan view f La Marea plant

Seepage under a weir (France)

This case concerns a small weir, which is the furthest structure upstream of a 600kW project comprising of a buried culvert, a penstock and a powerhouse. The operating personnel had noticed minor leakage at the downstream toe of the dam. The small reservoir was emptied, and a trench was excavated so that the contact between the structure and the foundation could be examined. It was then revealed that a conduit had formed between the upstream and the downstream faces of the weir (photo 4.11), which was actually founded on permeable deposits without a cut-off trench.

The weir in this condition would have eventually failed by undermining the foundation.
The key issues to learn from this case were the lack of a geomorphologic survey and inadequate supervision of the design and construction of the weir.

Photo 4-10 Weir undermined by seepage

Photo 4-11 Weir undermined by seepage

The hydraulic canal in a low-head 2MW scheme
The hydraulic canal, 5m wide and 500m long ran along side a river. The river was known to experience frequent flash floods. On one particular day, a flood occurred which was later calculated to be a 100 year event. When the flood occurred, the turbines were stopped and all the gates closed. The headrace channel had been almost emptied by leakage, and the channel was destroyed by uplift pressure (photo 4.12). In this case the key technical issues were hydraulics, structural stability and design.

Photo 4-12 Channel destroyed by uplift pressure

There are other cases that could be described to show the effects of poor judgement during either the design or the construction phase. Such case studies show the number and diversity of parameters that can cause failures. It is also unfortunately evident that design, construction and site supervision are often carried out by companies, which may offer lower costs, but have little experience of hydraulic works.

---

[i] By Luigi Papetti (Studio Frosio), Jonas Rundqvist (SERO) and Celso Penche (ESHA)

CHAPTER 5: HYDRAULIC STRUCTURES

CONTENTS

5 HYDRAULIC STRUCTURES 93
5.1 Introduction 93
5.2 Dams 93
5.2.1 Embankment Dams 95
5.2.2 Concrete Dams 95
5.2.3 Other Dam types 97
5.2.4 Loads and stability for concrete dams 98
5.2.5 Dam Safety 99
5.3 Weirs and spillways 100
5.3.1 Weirs 101
5.3.2 Gated Spillways 103
5.3.3 Other spillways 104
5.4 Energy dissipating structures 109
5.5 Intake structures 109
5.5.1 General 109
5.5.2 Intake types 110
5.5.3 Head losses 114
5.5.4 Trashracks 115
5.5.5 Vorticity 119
5.6 Sediment traps 120
5.6.1 General 120
5.6.2 Efficiency of a sediment trap 121
5.6.3 Design 121
5.7 Gates and valves 122
5.8 Open channels 126
5.8.1 Design and dimensioning 126
5.8.2 Excavation and stability 130
5.9 Penstocks 134
5.10 Tailraces 149
Bibliography 150

LIST OF FIGURES

Figure 5.1: A zoned embankment dam with moraine core 95
Figure 5.2: Typical geometry for arch and cupola dams (single curvature arch dam to the left) 97
Figure 5.3: Masonry dam with vertical concrete upstream wall 98
Figure 5.4: Typical timber dams 98
Figure 5.5: Loads on concrete dams 99
Figure 5.6: Fixed and mobile spillway structures 101
Figure 5.7: Discharge characteristics for weirs 102
Figure 5.8: Weir configurations 103
Figure 5.9: Discharge characteristics for gated spillways 104
Figure 5.10: Flashboards, articulated & embedded 105
Figure 5.11: Inflatable weir 106

Figure 5.12: Schematic layout of siphon spillway 108
Figure 5.13: Schematic view of morning glory (shaft) spillway 108
Figure 5.14: Labyrinth weir 109
Figure 5.15: Secondary currents in river bends 111
Figure 5.16: Typical layout of lateral intake 112
Figure 5.17: Secondary current along the outer bend of a curved river 113
Figure 5.18: “Tyrolean” intake 114
Figure 5.19: French drop intake: a canal built in the streambed and covered by a trashrack 115
Figure 5.20: Trash boom layout 116
Figure 5.21: Formula for computing head losses 117
Figure 5.22: Oleo-Hydraulic cylinders 118
Figure 5.23: Minimum degree of submergence 119
Figure 5.24: Sediment traps 121
Figure 5.25: Wedge-shaped stopper 123
Figure 5.26: Butterfly valves 124
Figure 5.27: Globe and rotary valves 124
Figure 5.28: Channel design 131
Figure 5.29: Rectangular reinforced canal 131
Figure 5.30: Materials used for protection 132
Figure 5.31: Penstock 134
Figure 5.32: Penstock with concrete anchor blocks and expansion joints 136
Figure 5.33: Manufactured steel pipe 137
Figure 5.34: Energy loss 139
Figure 5.35: Friction and turbulence head losses 141
Figure 5.36: Surge tower 147
Figure 5.37: Surge height versus time 147

LIST OF TABLES

Table 5.1: Intake characteristics 111
Table 5.2: Hydraulic parameters for common canal cross-sections 127
Table 5.3: Optimum profile for different channel sections 128
Table 5.4: Different material’s characteristics 137

LIST OF PHOTOS

Photo 5.1: Examples of gravity (RCC) and buttress dams 96
Photo 5.2: Example of an arch dam 97
Photo 5.3: Failure of a small dam, the breach and the flooding downstream 100
Photo 5. 4. Ogee weir 103
Photo 5.5: Articulated flashboard 105
Photo 5.6: Flashboard controlled by inflatable rubber bladder 106
Photo 5.7: Hydroplus fusegates 107
Photo 5.8: Drop intake 113
Photo 5.9: Prefabricated booms 116
Photo 5.10: Telescopic hydraulic cylinders 118
Photo 5.11: wheel-and-axle mechanism 123
Photo 5.12: Hydraulic Cylinder 124
Photo 5.13: Large butterfly valve 125

Photo 5.14: Butterfly valve hydraulically operated 125
Photo 5.15: Tainter gate (left) and housing of its sector on a concrete pier 126
Photo 5.15: Canal in the Cordinañes 132
Photo 5.16: Lateral spillway 133
Photo 5.17:Uplift 133
Photo 5.18: Flume 133
Photo 5.19: Water jet 148

5 HYDRAULIC STRUCTURES[1]

5.1 Introduction

A hydropower development includes a number of structures, the design of which will be dependant upon the type of scheme, local conditions, access to construction material and also local building traditions in the country or region. The following structures are common in a hydro scheme:

- Diversion structure
    - Dam
    - Spillway
    - Energy dissipation arrangement
    - Fish pass
    - Residual flow arrangements
- Water conveyance system
    - Intake
    - Canals
    - Tunnels
    - Penstocks
    - Power house

Design aspects and common solutions for these structures are presented below.

5.2 Dams

Dams and weirs are primarily intended to divert the river flow into the water conveyance system leading to the powerhouse. Dams also produce additional head and provide storage capacity. The choice of dam type depends largely on local topographical and geotechnical conditions. For instance if sound rock is not available within reasonable excavation depth, rigid structures such, as concrete dams are difficult. Conversely, for narrow valleys, it can be difficult to find space for separate spillways, and concrete dams can be the natural choice with their inherent possibilities to integrate spillways etc in the dam body.

In the Nordic countries the ice age has left us with wide and open valleys and moraine material in abundance. Not surprisingly the vast majority of dams are embankment dams with a central core of moraine. South of the Alps natural clays suitable for dam core are not in abundance and the topography in many locations favour concrete dams.

According to the ICOLD (International Committee of Large Dams), a dam is considered "small" when its height, measured from its foundation level to the crest, does not exceed 15 m, the crest length is less than 500 m and the stored water is less than 1 million cubic meters. These parameters can be important, because of the complicated administrative procedures often associated with the construction of large dams.

World wide, embankment dams are the more common partly due to the following characteristics, which they possess:

- Can be adapted to a wide range of foundation conditions.
- Construction uses natural materials, which can often be found locally, limiting needs for long transportation.
- The construction process can be continuous and highly mechanized.
- The design is extremely flexible in accommodating different fill materials.

Disadvantages with embankment dams are that they are sensitive to overtopping and leakage, and erosion in the dam body and its foundation. There is a higher mortality rate among embankment dams as compared to concrete dams.

Concrete dams on the other hand have drawbacks that correspond to the pros of the embankment dams:

- Require certain conditions with respect to the foundations.
- Require processing of natural materials for aggregate at the site, hauling of large quantities of cement and has a labour intensive and discontinuous construction process, leading to large unit costs.

On the other hand concrete dams have several advantages:

- They are suitable for most ranges of topography that is for wide and narrow valleys, provided that foundation conditions are right.
- They are not very sensitive to overtopping.
- A spillway can be placed at the crest, and if required over the entire length of the dam.
- Chambers or galleries for drainage, tubing and ancillary works can readily be housed within the dam body.
- Powerhouses can be placed right at the toe of the dam.

The development of the Concrete Faced Rockfill Dam (CFRD) neutralizes many of the drawbacks with core-type embankments. In particular, sensitivity to leakage and erosion is reduced, and dependence of good core material is removed.

The development of the Roller Compacted Concrete Dams (RCC-dams) introduces a continuous, highly mechanised construction process and low unit costs.

New large dams are almost always CFRD and RCC designs.

5.2.1 Embankment Dams

Homogeneous dams: These dams are used for low embankments (<4m) and often as secondary dams. For dam safety reasons, some type of drainage is almost always provided.

Zoned embankment dams: These are used for dam heights from 4m and up. Constructions are extremely sensitive to the engineering design and construction, and it is therefore vital to engage highly skilled consultants and contractors require experienced site-supervision engineers. Critical components of these dams are the core, the transition zones (filters) surrounding the core and drainage capacity of the dam toe (see figure 5.1).

Embankments dams with membrane: The membranes can be of different types and be located either at the upstream front of the embankment or vertically in the centre of the embankment. Membranes can be made from concrete (as in the CFRD), asphalt (Norwegian type) or in the form of a geomembrane on the upstream slope.

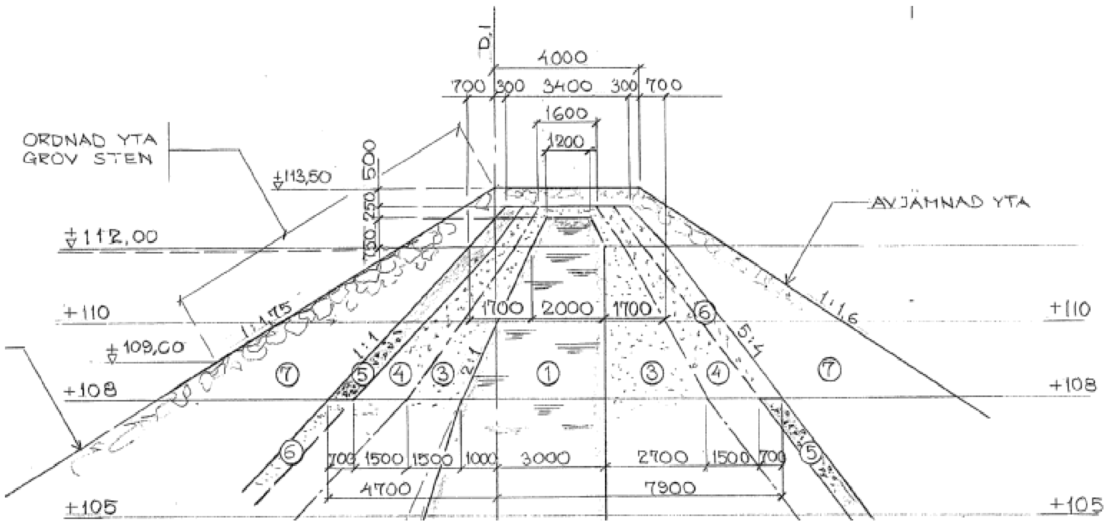


Figure 5.1: A zoned embankment dam with moraine core

Embankment dams are often categorised according to the main fill material, for example, rock-fill dams, or earth-fill dams.

5.2.2 Concrete Dams

Generally, concrete dams are categorized according to how they function statically, and fall into one of the following groups.

Gravity dams: These are dependent on their own mass for stability. Their cross-section is basically triangular in order to provide adequate stability and stress distribution across the foundation plane. The upper part is normally rectangular in order to provide adequate crest width for installation and transportation.

Design issues include stability analysis (sliding and overturning), stress control, temperature control during construction to avoid cracking, control of uplift pressures under the dam, etc. In photo 5.1 a gravity dam constructed of RCC (left photo) is shown. Note the characteristic stepped downstream slope.

Buttress dams: These dams consist of a continuous upstream face that is supported by buttresses at regular intervals. The upstream face is normally divided into vertical sections by dilatation joints, each section being supported by a buttress. Cross-sections are similar to those of gravitation dams. In colder climates, the upstream face can be susceptible to freezing of the water contained in the concrete, damaging the concrete. For this reason buttress dams in such locations are often covered along the downstream contour of the buttresses in order to provide climate control. The right-hand photo in photo 5.1 shows an example of a buttress dam. Note that the spillway is also a buttress type structure.

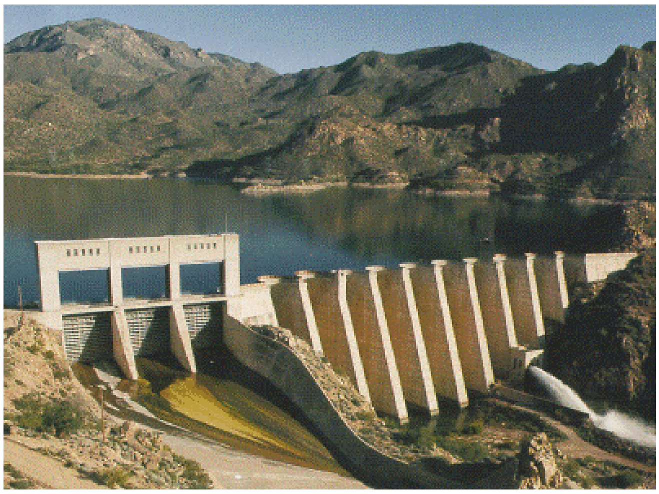


Photo 5.1: Examples of gravity (RCC) and buttress dams

Arch and Cupola dams: These dams function structurally as horizontally laid out arches that transfer the water pressure on the upstream face into the abutments rather than into the foundation. Arch dams can be designed with a constant radius over the dam height, or with varying radii (Cupola dams). Arch dams with a constant radius have a vertical and "straight" cross-section. These dams will be subject to considerable vertical strain forces since the deformation of the dam will tend to be greatest in the vertical centre of the dam. This requires that the dam be heavily reinforced to avoid cracking with accompanying leakage.

The Cupola dam is designed to have only compression forces for all directions and at all sections. This requires the radius of the curvature to vary over the dam height, which produces a curved vertical cross-section.

The arch and cupola dams are structurally efficient and greatly reduce the required volume of concrete. They require, however, a narrow valley topography and strong foundation rock in the abutments. In photo 5.2 an example of an arch dam is shown, and in figure 5.2 the typical geometry for single curvature arch dams versus double curvature cupola dams is displayed.

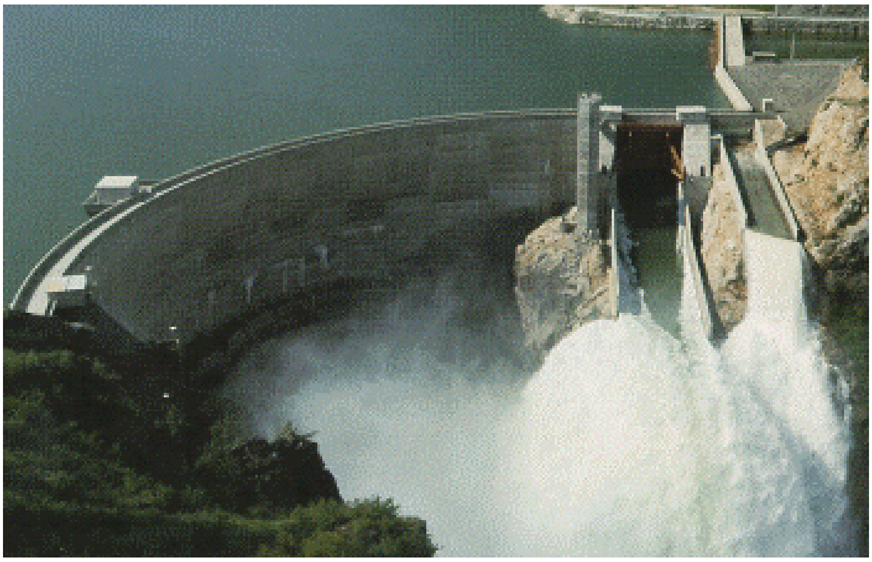


Photo 5.2: Example of an arch dam

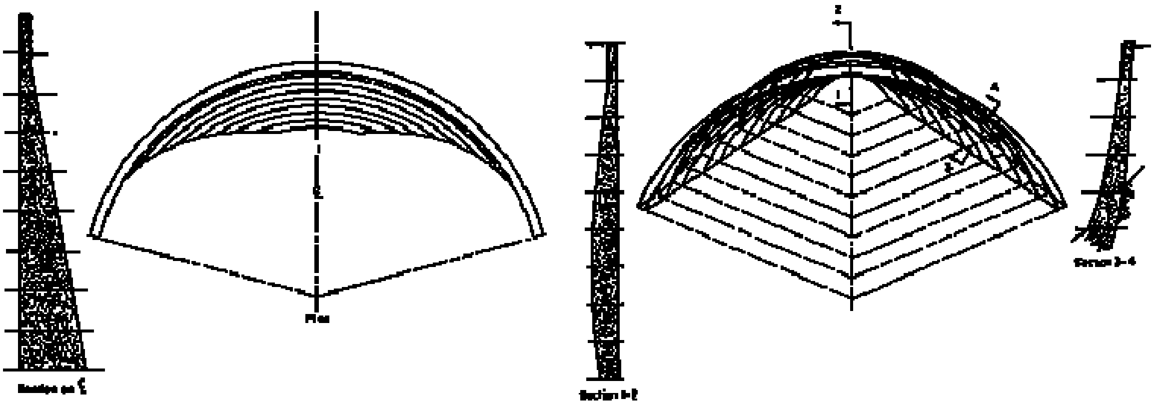


Figure 5.2: Typical geometry for arch and cupola dams (single curvature arch dam to the left)

5.2.3 Other Dam types

Another type of concrete dam is the **spillway dam**, which can be gated or ungated. A gated dam with large spillway openings compared to the dam height is often designed to function as a buttress dam, whereas higher spillway dams with relatively small spillway openings normally are designed to function as a gravity dam.

An ungated spillway dam is often referred to as a weir for lower dam heights. Weirs and spillways are described in more detail below.

An old dam type still prevailing is the **masonry dam**. This dam was prevalent during the early days of industrialization, utilizing the building techniques present at that time. The masonry structure functioned as the load bearing structure and water tightness was provided by either vertical timber sheeting on the upstream face or by filling impervious soils upstream of the masonry structure. Figure 5.3 shows an example of a masonry dam, with an upstream wall. In many ways these dams resemble the CFRD, a development in embankment dams, and they share a number of advantageous characteristics.

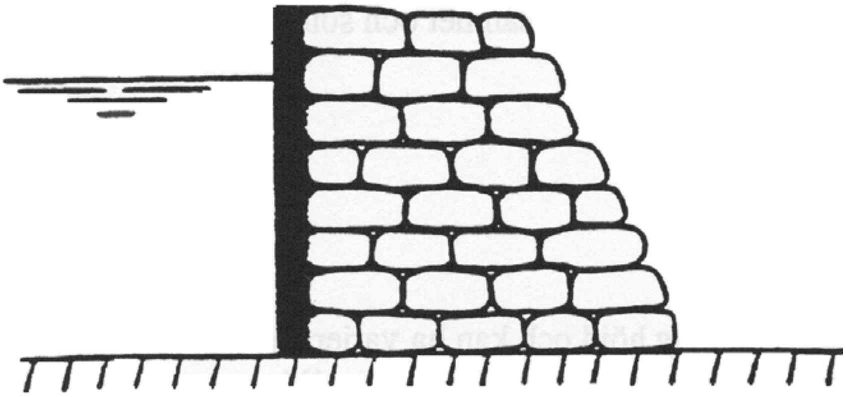


Figure 5.3: Masonry dam with vertical concrete upstream wall

Timber dams: These dams can still be found although due to their limited durability they are becoming increasingly scarce. These dams were constructed in two ways, as is shown in figure 5.4.

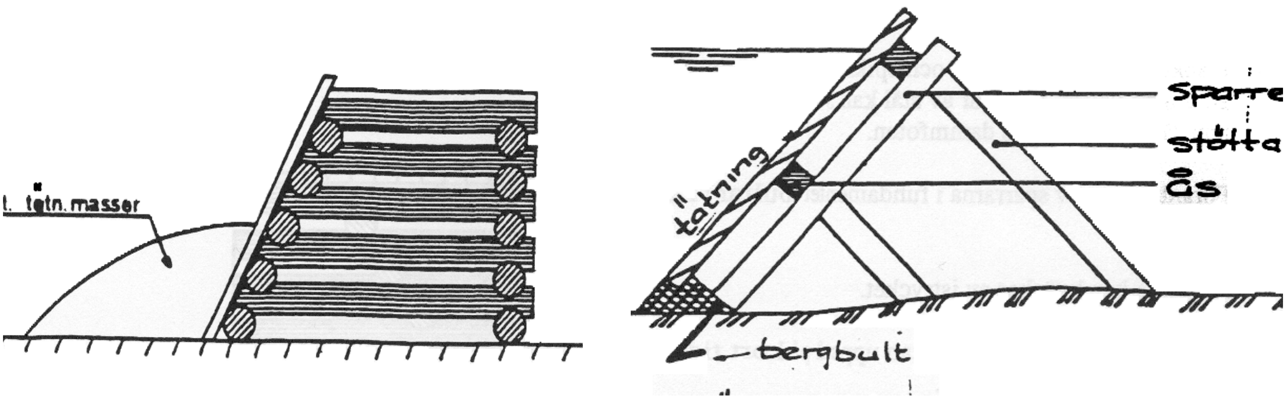


Figure 5.4: Typical timber dams

5.2.4 <u>Loads and stability for concrete dams</u>

In figure 5.5, the typical loads acting on concrete dams are shown. H denotes horizontal loads and V vertical loads. The horizontal loads are: 1; Lateral water pressure, 2; pressure from soil or deposited sediments, 3; Ice pressure, 4; Loads from floating objects and debris, 5; downstream water pressure, 6; dynamic acceleration from earthquakes, 7; incremental water pressure during earthquakes. The vertical loads are: 1; self-weight of the dam, 2; weight of water on inclined upstream surface, 3;uplift pressure from pore water, 4; dynamic load from earthquakes. There is also a small vertical load corresponding to the weight of water on the inclined downstream slope.

The understanding of uplift pressures and their importance for gravity dams has gradually increased. The very existence of uplift pressures was not known until the beginning of the 20th century. For the first gravity dams, made as masonry dams, uplift pressures were basically eliminated due to the effective drainage provided by the porous structure of the masonry. As masonry was replaced by concrete in new dams these dams were designed applying the same well-proven dimensions used for masonry dams, which in many cases led to failure of the dams.

Modern concrete dams provide drainage in the form of drainage galleries, by drilling drainage holes into the foundation rock. Using grouting curtains reduces foundation leakage. These measures can be effective, but require maintenance. Concrete dams built as late as in the 1980's regularly show weaknesses due to the very optimistic assumptions regarding uplift pressures and the ineffectiveness of individual counter measures.

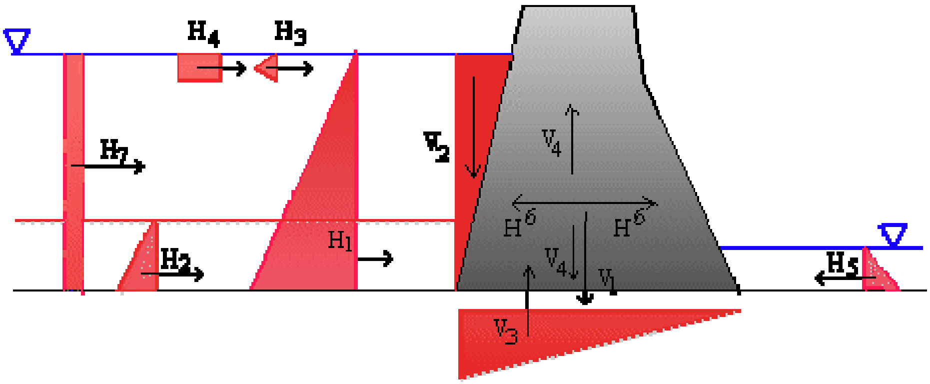


Figure 5.5: Loads on concrete dams

Concrete dams are designed for:

- Stability against rotation and overturning
- Stability against translation and sliding
- Over-stress and material failure

5.2.5 <u>Dam Safety</u>

Dams have been identified as "the single man-made structures capable of causing most deaths". Hazards with dam failure have largely been associated with large dams and reservoirs, but depending on localization and circumstances even smaller and medium sized dams and reservoirs can be potentially dangerous, and considering their large number they do pose a significant threat to health and environment. In Sweden, for example, the only fatality as a result of dam failure was caused by the failure of a dam less than 4m high. Photo 5.3 shows two photos of the failure of a "small" dam. The left photo shows the breach and the right photo show the damage downstream.

In order to identify potentially hazardous dams most countries now employ a classification system for dams, requiring dam owners to classify their dams. The hazard level is described and identified subjectively using terms such as low, significant and high (USACE 1975).

Dam safety can be improved by installation of monitoring systems, performing reviews and undertaking dam inspections on a regular basis.

Photo 5.3: Failure of a small dam, the breach and the flooding downstream

5.3 Weirs and spillways

A dam failure can have severe effects downstream of the dam. During the lifetime of a dam different flow conditions will be experienced and a dam must be able to safely accommodate high floods that can exceed normal flow conditions in the river by orders of magnitude. For this reason carefully designed overflow passages are incorporated in dams or weirs as part of the structure. These passages are known as *spillways*. Due to the high velocities of the spilling water, some form of energy dissipation is usually provided at the base of the spillway.

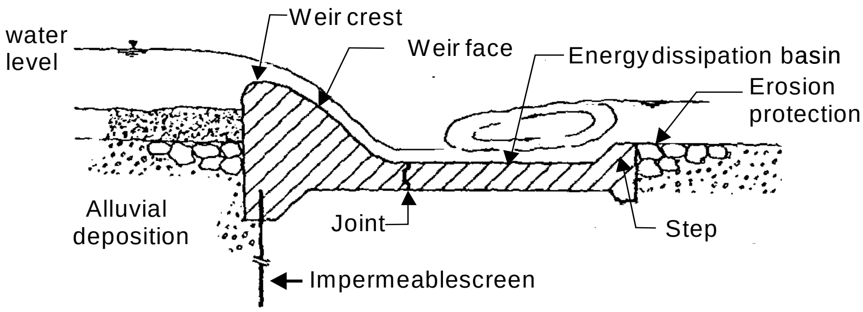
The large majority of small hydro schemes are of the run-of-river type, where electricity is generated from discharges larger than the minimum required to operate the turbine. In these schemes a low diversion structure is built on the streambed to divert the required flow whilst the rest of the water continues to flow over it. Such a structure is commonly known as a *weir*, whose role is not to store the water but to increase the level of the water surface so the flow can enter into the intake.

Weirs and spillways can be subdivided into *fixed* and *mobile* structures (Figure 5.6). Smaller fixed structures are generally referred to as weirs, whereas larger structures are often referred to as spillways. Spillways are often divided into ungated and gated spillways, corresponding to fixed and mobile structures, the ungated spillway in fact being a large-scale weir.

Fixed storage structures, such as weirs and ungated spillways have the advantage of security, simplicity, easy maintenance, and are cost effective. However, they cannot regulate the water level and thus both the water level and energy production fluctuates as a function of discharge.

Mobile storage structures such as gated spillways can regulate the water level such that it stays more or less constant for most incoming flow conditions. Depending on gate configuration and discharge capacity they may also be able to flush accumulated sediment downstream. These structures are generally more expensive than fixed structures, for both construction and maintenance, and their functioning is more complicated.

Fixed structure



Mobile structure

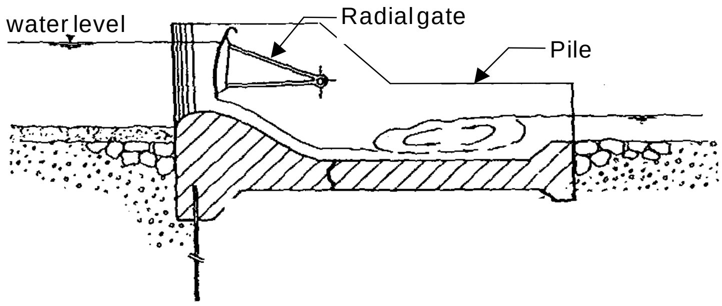


Figure 5.6: Fixed and mobile spillway structures

5.3.1 Weirs

Weirs can be constructed perpendicular, angular or lateral compared to the river axis. Most often the weir crest is rectilinear and perpendicular to the river axis. For relatively low downstream water levels, the weir controls the flow and defines the relationship between the upstream water level and the discharge. As a function of the type of weir, different discharge relationships are obtained as indicated in Figure 5.7.

The sharp-crested weir is easy to construct and relatively cost-effective. Its discharge is defined by means of a coefficient C_d. Special attention has to be paid to the shape of the downstream face of the upper part of the weir in order to obtain sufficient aeration between the lower nappe (sheet of water that flows over the weir) of the jet and the structure. If the lower nappe of the jet sticks to the structure, vibrations may be transferred from the flow to the structure.

The broad-crested weir is often applied for temporary structures or for structures of secondary importance, such as in case of temporary flow diversion. Its design is simple and cost-effective. The hydraulic conditions are far from optimal, expressed by a low discharge coefficient and the presence of under-pressures along the weir crest and downstream face. The discharge depends on the form of the structure.

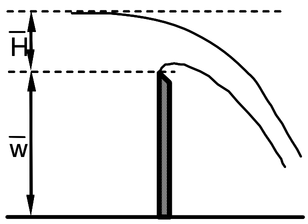
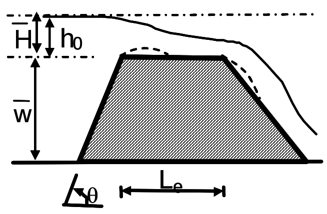
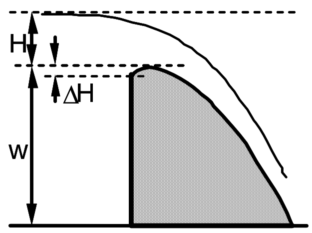
Type	Design	Discharge relationship	Characteristics
Sharp-crested weir		$Q = b\cdot\bar{C}_d\cdot\bar{H}^{3/2}\cdot\sqrt{2g}$ $\bar{C}_d = 0.42$	Simple design Cost effective
Broad-crested weir		$Q = b\cdot c_e\cdot\bar{C}_d\cdot\bar{H}^{3/2}\cdot\sqrt{2g}$ $\bar{C}_{d,mean} = 0.42$ $c_e = 1-\frac{2\sin\theta}{9(1+\xi_e^4)}$ $\xi_e = \frac{\bar{H}-\bar{w}}{L_e}$	Simple design, underpressures on crest Cost effective
Ogee weir		$Q = b\cdot C_{dD}\cdot H^{3/2}\cdot\sqrt{2g}$ $C_{dD} = 0.494$ (for $H = H_D$)	Highest discharge Costly design

Figure 5.7: Discharge characteristics for weirs

The ogee weir is hydraulically the most ideal solution giving the highest discharge coefficient. Its curved shape is defined by the jet trajectory that would appear for the design discharge H_D. For lower or higher discharges, over-or under-pressures will appear along the downstream face. For discharges much higher than the design discharge, these under-pressures may lead to cavitation and damage to the downstream concrete face. Recent work suggests fortunately that separation will not occur until $H > 3H_D$. The U.S. Waterways Experimental Station has provided a set of profiles that have been found to agree with actual prototype measurements. The exact relationship between the discharge coefficient and the ratio H/H_D can be found in Sinniger & Hager (1989).

For downstream water levels that are equal to or higher than the spillway crest level, the spillway becomes progressively submerged and its corresponding discharge decreases. Furthermore, in presence of piles, the governing discharge will depend on the shape and dimensions of the piles. All these aspects influence the functioning of a spillway and for a detailed and correct design; the reader is referred to classical works in this field, such as Sinniger & Hager (1989).

Photo 5. 4. Ogee weir

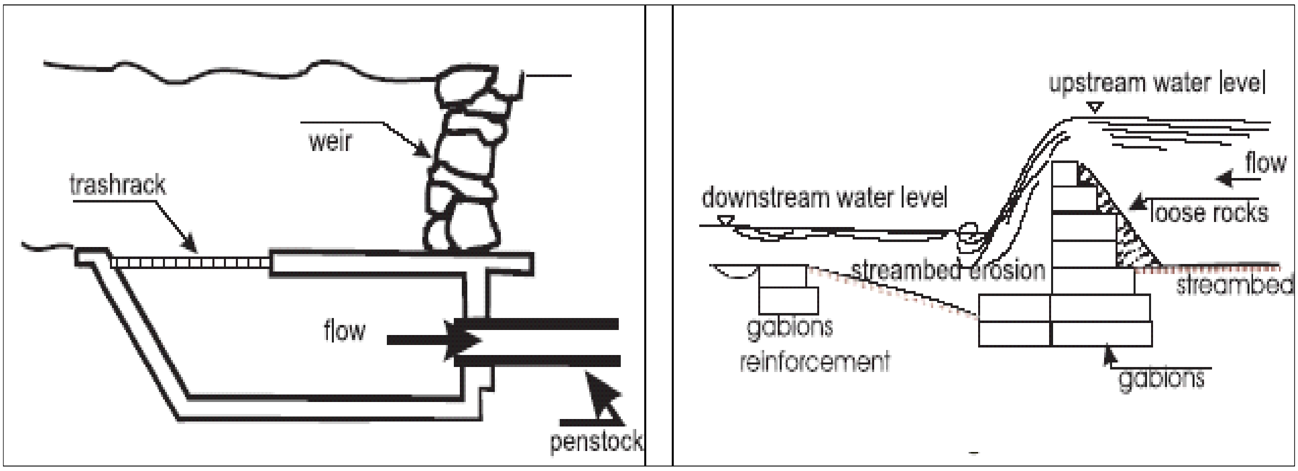


Figure 5.8: Weir configurations

5.3.2 Gated Spillways

The installation of mobile elements on dams or weirs allows control of the flow conditions without changing the water level. This is performed by means of gates, which are designed such that, when the gate is fully open (and the structure functions as if it where fixed) the discharge has to pass the structure without noticeable water level increase upstream. Gate operation needs permanent maintenance and an external energy source. As a result, there is a risk that the gate remains blocked during floods.

The most used types of gates are presented in Figure 5.9. Depending on the type of gate, the possible gate movements are rotating, sliding or turning. The discharge through the gates depends not only on the type of gate and the relative gate opening and gate lip angle, but also on the shape of the supporting weir.

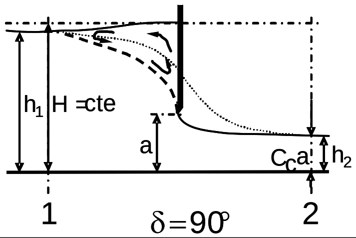
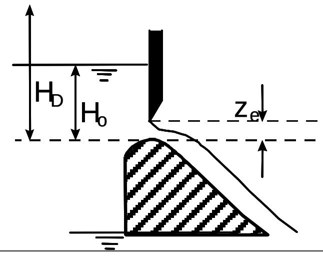
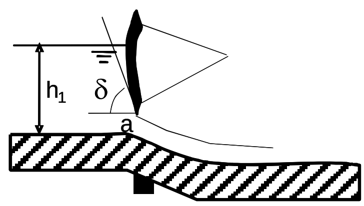
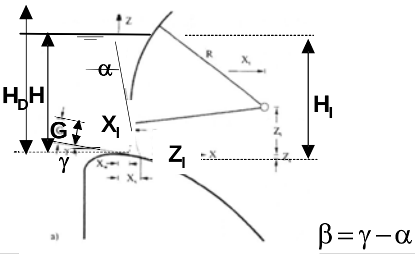
Type	Position	Design	Discharge relationshi
Flat gate	rectangula canal		$Q = ab\cdot C_d\cdot\sqrt{2gh_1}$ $C_d = C_{d0} - \exp\left[\frac{a}{2\cdot h_1}\left(1-\frac{\delta^2}{b}\right)\right]$ $C_{d0} = 0.98\cdot\left[\frac{4+5\cdot e^{-0.76\cdot\delta}}{9}\right]$
	ogeeweir		$\frac{Q_g}{Q_D}=\left[\frac{H_0^{3/2}}{H_D}-\left(\frac{H_0}{H_D}-\frac{z_e}{H_D}\right)^{3/2}\right]\cdot\left(\frac{1}{6}+\frac{z_e}{H_D}\right)^{1/9}$ $Q_D = bC_{dD}\cdot H^{3/2}\cdot\sqrt{2g}$ $C_{dD} = 0.494$
Sectoror radialgate	rectangula canal		$Q = ab\cdot C_d\cdot\sqrt{2gh_1}$ $C_d = C_{d0} - \exp\left[\frac{a}{2\cdot h_1}\left(1-\frac{\delta^2}{b}\right)\right]$ $C_{d0} = 0.96\cdot\left[\frac{4+5\cdot e^{-0.76\cdot\delta}}{9}\right]$
	ogeeweir	 $\beta=\gamma-\alpha$	$Q = H_D\cdot b\cdot G\cdot C_{dg}\cdot\sqrt{2gH_e}$ $C_{dg} = 0.90\cdot\left(1-\frac{\beta}{27^\circ}\right)\cdot\left(\frac{H}{H_D}\right)^{0.12}$ $G=\left[1-\frac{2}{g}\cdot\left(\frac{x_I}{H_D}\right)^{3/2}\right]\cdot\left[\frac{z_I}{H_D}+\frac{1}{2}\cdot\left(\frac{x_I}{H_D}\right)^{1.85}\right]$

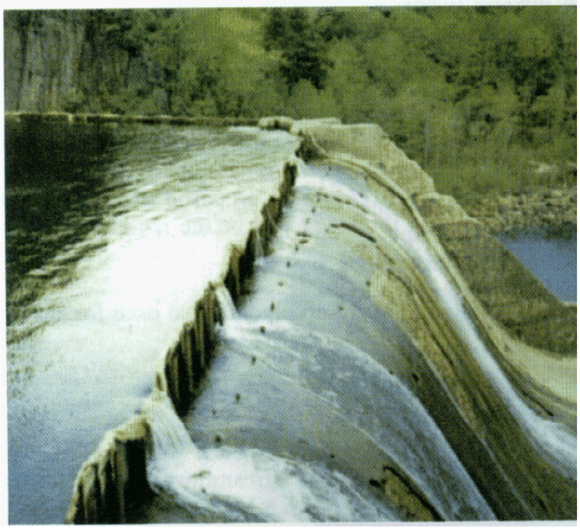
Figure 5.9: Discharge characteristics for gated spillways

More detailed design accounts also for the shape of the gate lip. Furthermore, the above-presented discharges are only valid for un-submerged flow conditions. Similar to the fixed structures, when the downstream water level becomes equal to or higher than the crest level, the mobile structure becomes progressively submerged and the corresponding discharge decreases. For more information, the reader is encouraged to consult classical textbooks on this subject.

5.3.3 Other spillways

Flashboards

To raise the water level slightly behind the weir to ensure adequate depth of water at the intake, without endangering the flooding of the upstream terrain, *flashboards* may be installed on the crest of the weir (Figure 5.10). The flashboards are commonly made of wood and supported by steel pins embedded in steel sockets (pipes cut down to size) in the spillway crest. The flashboards have to be removed by hand during flood flows so that high water levels do not flood the upstream terrain, an operation that in such circumstances is very difficult. The articulated flashboard is somewhat easier to remove.



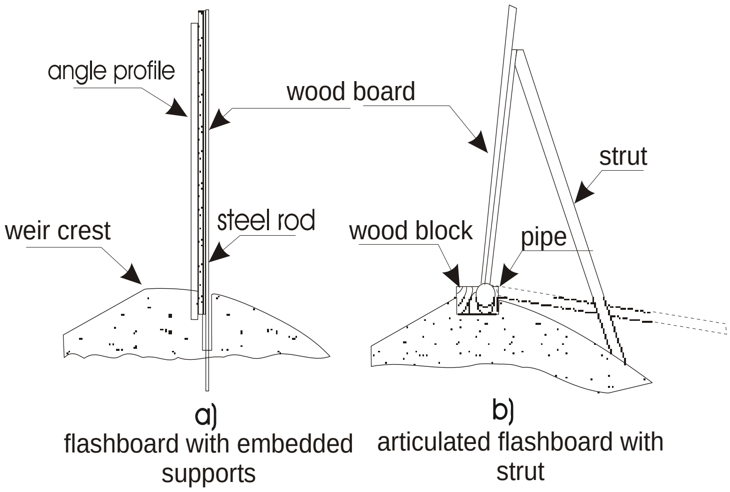


Figure 5.10: Flashboards, articulated & embedded

Photo 5.5: Articulated flashboard

<u>Inflatable weirs</u>

Another method, capable of remote control, is the *inflatable weir*, which employs a reinforced rubber bladder instead of concrete, steel or wood flashboards. This offers an alternative to more conventional methods of weir construction, with the inherent advantages of low inital cost, simple operation and minimal maintenance.

In effect, inflatable weirs are flexible gates in the form of a reinforced, sheet-rubber bladder inflated by air or water, anchored to a concrete foundation (Figure 5.11) by anchor bolts embedded into the foundation. Like any other gate, the inflatable weir needs a mechanism by which it is opened and closed. The weir is raised when filled with water or air under pressure. An air compressor or a water pump is connected, via a pipe, to the rubber bladder. When the bladder is filled the gate is raised; when it is deflated the weir lies flat on its foundation, in a fully opened position. The system becomes economic when the width of the weir is large in relation to the height.

When the management and operational safety of the system is rather critical, the use of inflatable weirs can give substantial advantages over conventional systems. An electronic sensor monitors the upstream water level and the inner pressure of the bladder. A microprocessor maintains a constant level in the intake entrance by making small changes in the inner pressure of the bladder. To avoid flooding land, a similar device can regulate the inflatable weir regulated to correspond to a pre-set upstream water level.

Inflatable gate control systems can be designed to fully deflate the bladder automatically in rivers prone to sudden water flow surges. On a typical weir, two meters high and thirty meters wide, this can be done in less than thirty minutes. Photo 5.6 illustrates a new type of inflatable weir - patented by Obermeyer Hydro - where the sheet rubber incorporates a steel panel that behaves as a flashboard, which is quickly and easily moved in the event of sudden floods. By controlling the pressure in the rubber blade the steel panels may be more or less inclined, varying the level of the water surface. The system incorporates an additional advantage: the rubber blade is always protected against boulders carried during flood flows (buoyancy causes heavy boulders to loose a portion of their weight in water, making it easier for the flood flow to carry them downstream). A synthetic rubber flap anchored to one of the panels closes the free space between panels or between panel and the buttress.

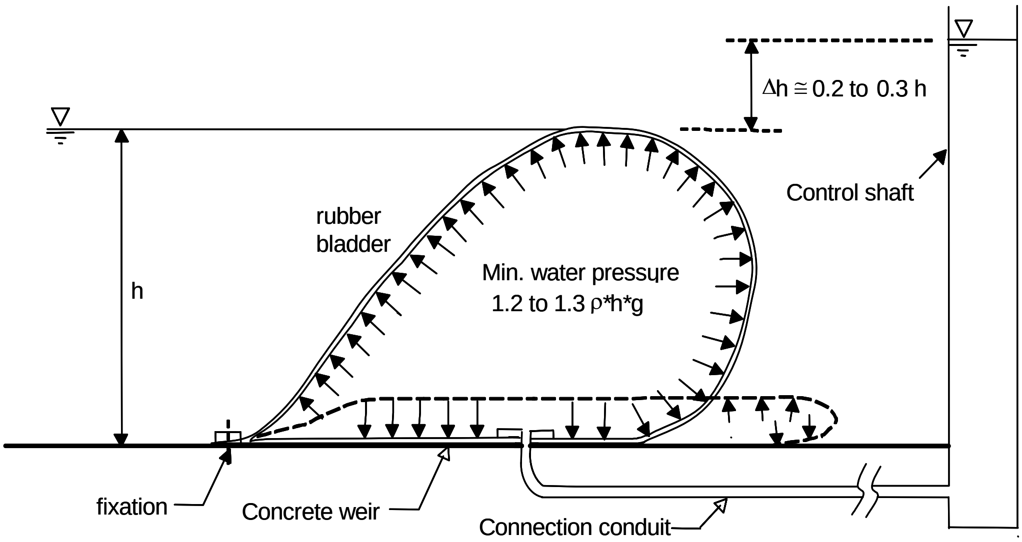


Figure 5.11: Inflatable weir

Photo 5.6: Flashboard controlled by inflatable rubber bladder

Fusegates

In large installations, but also sometimes in small ones, it is advisable to place fusegates, such as those supplied by Hydroplus[2]. In the event of a major flood, when the water reaches a pre-set level, one or more of the fusegates (basically hinged structures) will tilt to increase the section of the spillway (Photo 5.7).

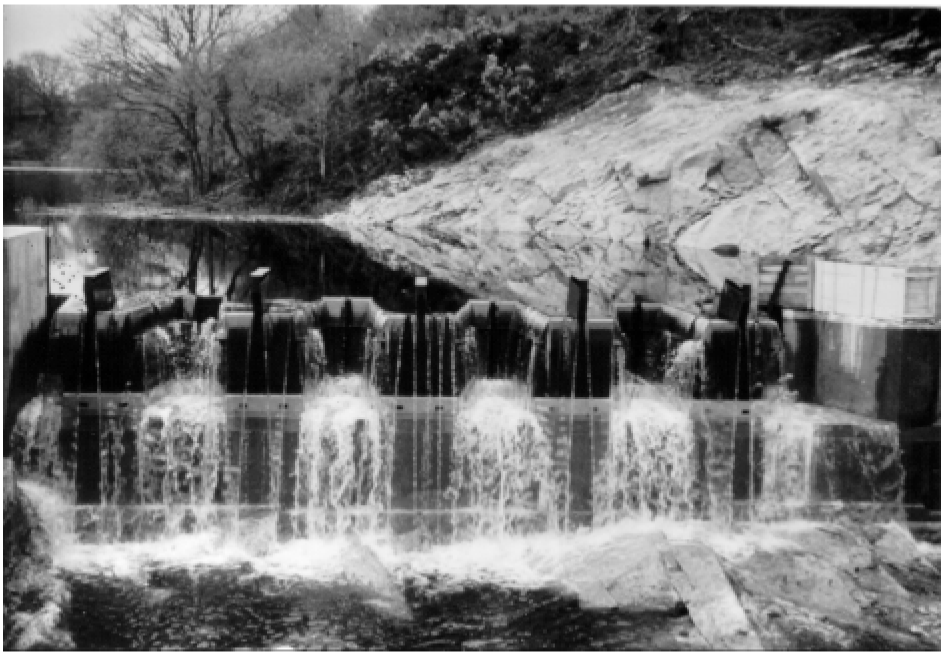


Photo 5.7: Hydroplus fusegates

Siphon spillways

Alternatively where space available for the spillway is limited, a siphon spillway or a shaft spillway may be used. Both solutions help to keep the upstream water level within narrow limits. A siphon spillway is basically a curved enclosed duct (Figure 5.12). When the water level rises above the elbow of the siphon, water begins to flow down the conduit just as in an overflow, but it is when it rises further that the siphon is primed and increases the discharge considerably. Usually siphons are primed when the water level reaches or passes the level of the crown, but there are designs where priming occurs when the upstream level has risen only to about one third of the throat height.

If badly designed, the siphon process can become unstable. At the beginning the siphon discharges in a gravity mode, but when the siphon is primed the discharge suddenly increases. Consequently the reservoir level drops, the siphon is de-primed and the discharge is reduced. The level of the reservoir increases anew until the siphon primes again, and the cycle of events is repeated indefinitely, causing severe surges and stoppages. Multiple siphons with differential crest heights or aerated siphons can be the solution to this problem. When the siphon is primed the flow through a siphon spillway is governed, as in penstocks, by Bernoulli's equation. Assuming that the velocity of water in the conduit is the same at the inlet and outlet, the head loss may be calculated from the formulae in Chapter 2, paragraph 2.2.1.

If the pressure at the crown of the siphon drops below the vapour pressure, the water vaporises forming a large number of small vapour cavities, which entrained in the flow, condense again into liquid in a zone of higher pressure. This phenomenon is known as cavitation and it can be extremely damaging. To avoid it, the distance between the crown of the siphon and the maximum level at the reservoir, depending on height above sea level and prevailing barometric pressure, should normally not exceed 5 m. Further details on this kind of spillway can be found in the literature[3].

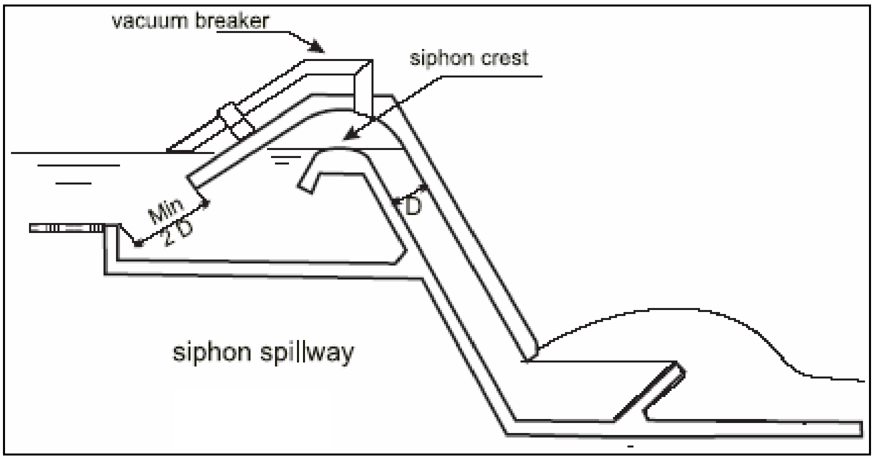


Figure 5.12: Schematic layout of siphon spillway

Shaft (or Morning glory) spillways

Shaft spillways are rarely used in small scale-hydro. As illustrated in Figure 5.13, a shaft spillway incorporates a funnel-shaped inlet to increase the length of the crest, a flared transition which conforms to the shape of the nappe as in the overflow spillway though it is sometimes stepped to ensure aeration, a vertical shaft and an outlet tunnel that sometimes has a slight positive slope to ensure that at the end it never flows full. The US Bureau of Reclamation reports (USBR) 6 and 7 describe the design principles for these spillways.

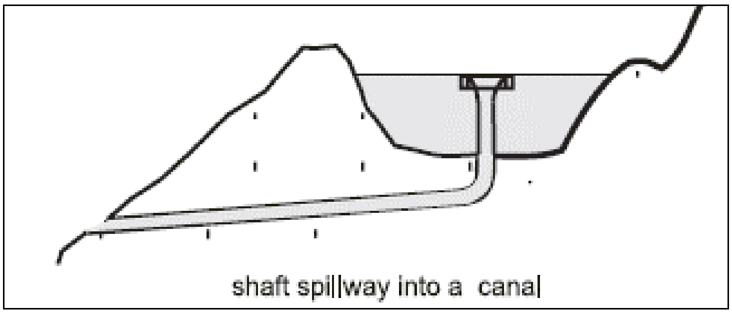


Figure 5.13: Schematic view of morning glory (shaft) spillway

Labyrinth weir

In some small hydropower schemes (e.g. small schemes in an irrigation canal) there is not enough space to locate a conventional spillway. In these cases, U shaped or labyrinth weirs (Figure 5.14) should help to obtain a higher discharge in the available length.

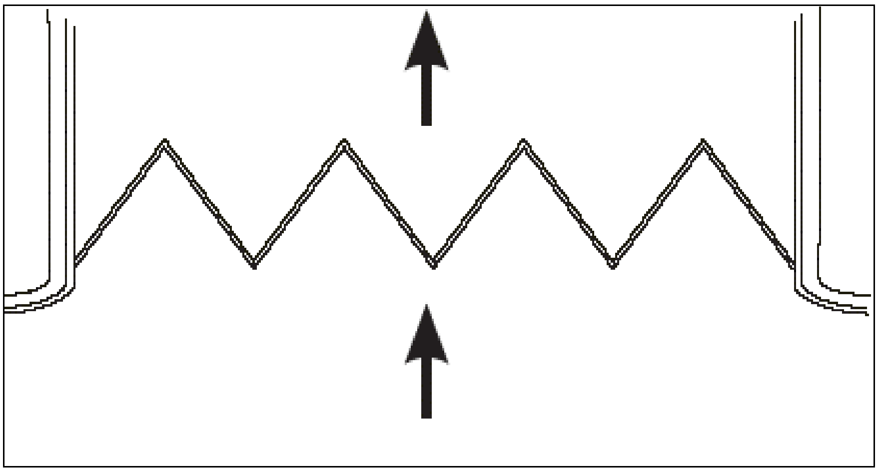


Figure 5.14: Labyrinth weir

5.4 Energy dissipating structures

The discharge from the aforementioned fixed or mobile structures is usually supercritical at the outlet. The corresponding high flow velocities and turbulence may produce severe erosion at the toe of the structure, especially if the riverbed is not erosion resistant, such as for example in the case of silt, clay, loose sand, gravel or even fractured rock.

To avoid such damage, several structural solutions may be applied, some of them being very costly. The most often used solutions are:

- Stilling basin
- Baffled apron drop
- Plunge pool
- Chute cascades

Most of these structures dissipate the flow energy by the formation of a hydraulic jump, which dissipates a lot of energy over a relatively short distance. The design and construction of energy dissipating structures is quite complex and vast and the reader is encouraged to contact specialized engineers. More detailed information can be found for example in Vischer & Hager (1995).

In RCC-dams the stepped chute downstream of the spillway has proven effective in reducing flow velocities and reducing the dimensions of the subsequent stilling basin.

5.5 Intake structures

5.5.1 General

A water intake must be able to divert the required amount of water into a power canal or into a penstock without producing a negative impact on the local environment and with the minimum possible head losses. Also, a major challenge consists of handling debris and sediment transport. The intake serves as a transition between a stream that can vary from a trickle to a raging torrent, and a controlled flow of water both in quality and quantity. Its design, based on geological, hydraulic, structural and economic considerations, requires special care to avoid unnecessary maintenance and operational problems that cannot be easily remedied and would have to be tolerated for the life of the project.

A water intake designer should take three criteria into consideration:

- Hydraulic and structural criteria common to all kind of intakes
- Operational criteria (e.g. percentage of diverted flow, trash handling, sediment exclusion, etc.) that vary from intake to intake
- Environmental criteria characteristics of each project (eg requiring fish diversion systems, fish passes, etc).

The location of the intake depends on a number of factors, such as submergence, geotechnical conditions, environmental considerations (especially those related to fish life) sediment exclusion and ice formation, where necessary. The orientation of the intake entrance to the flow is a crucial factor in minimising debris accumulation on the trashrack, a source of possible future maintenance problems. The best disposition of the intake is with the screen at right angles to the spillway so, that during flood seasons, the flow pushes the debris over its crest. The intake should not be located in an area of still water, far from the spillway, because the eddy currents common in such waters will accumulate trash at the entrance.

The intake should be equipped with a trashrack to minimise the amount of debris and sediment carried by the incoming water; a settling basin where the flow velocity is reduced, to remove all particles over 0.2 mm; a sluicing system to flush the deposited silt, sand, gravel and pebbles with a minimum of water loss; and a spillway to divert the excess water.

5.5.2 Intake types

The first thing for the designer to do is to decide what kind of intake the scheme needs. These can be classified according to the following criteria:

- *Power intake*: The intake supplies water directly to the turbine via a penstock. These intakes are often encountered in lakes and reservoirs and transfer the water as pressurized flow.
- *Conveyance intake*: The intake supplies water to other waterways (power canal, flume, tunnel, etc.) that usually end in a power intake (Figure 1-1 Chapter 1). These are most frequently encountered along rivers and waterways and generally transfer the water as free surface flow.

Conveyance intakes along rivers can be classified into lateral, frontal and drop intakes. The main characteristics of these three types are summarized in Table 5.1.

Table 5.1: Intake characteristics

		River slope	River width B	Plan view of river	Sediment transport
Lateral intake	**in outer river bend**	0.001% < J < 10%	All widths	Curved path is optimal	Strong bedload, small suspended transport (Qeq < Qcr)
	with gravel deposition canal	0.01% < J < 10%	B < 50 m	Possible rectilinear path if countermeasures	Strong bedload with continuous flushing, strong suspended load
Frontal intake	**with gravel deposition tunnel**	0.01% < J < 10%	B < 50 m, (B < 500 m for economical dams/weirs)	Rectilinear is optimal, curved path is possible if countermeasures	Strong bedload with continuous flushing, very strong suspended load
Drop intake		J > 10% favorably, possible already at 2.5%	B < 50 m, (B < 500 m is possible for dams/weirs over part of river width)	Rectilinear	Strong bedload (only large grain sizes)

The lateral intake functions by using a river bend or by using a gravel deposition channel. The former is presented in Figure 5.15. This intake favourably applies the presence of a strong secondary current along the outer bend of the curved river. This secondary current prevents bedload from entering the intake. The installed discharge Q_{ep} has to be smaller than 50 % of the critical river discharge Q_{cr}, where the latter is defined as the discharge for which the bedload transport starts.

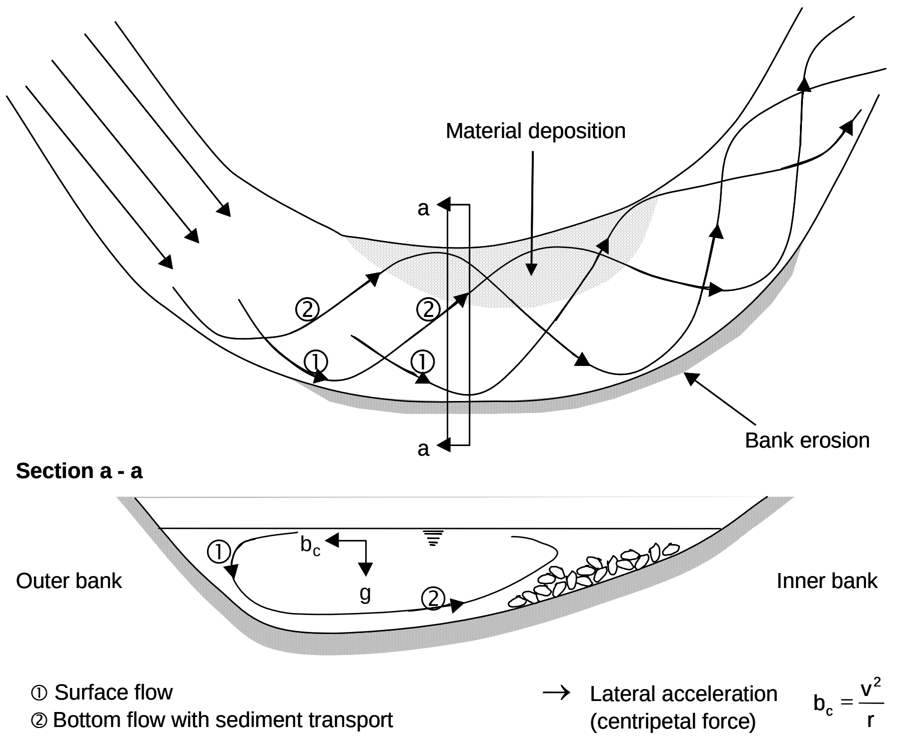


Figure 5.15: Secondary currents in river bends

The latter type of lateral intake uses a gravel deposition canal in front of the intake in order to prevent both bed and suspended load from entering the intake. Hence, there is no discharge restriction. The channel makes use of a gravel weir of minimum 1-1.5 m, as indicated in Figure 5.13. Furthermore, its slope should be at least 2%, preferably 5%. The channel bottom has to be protected against abrasion (using high quality concrete, stones, etc.). A partially submerged wall (0.8-1.0 m submersion) is installed in order to prevent debris from entering the intake.

The main elements of the lateral intake structure are presented in Figure 5.16: a mobile weir/dam, gravel deposition channel and intake with trashrack.

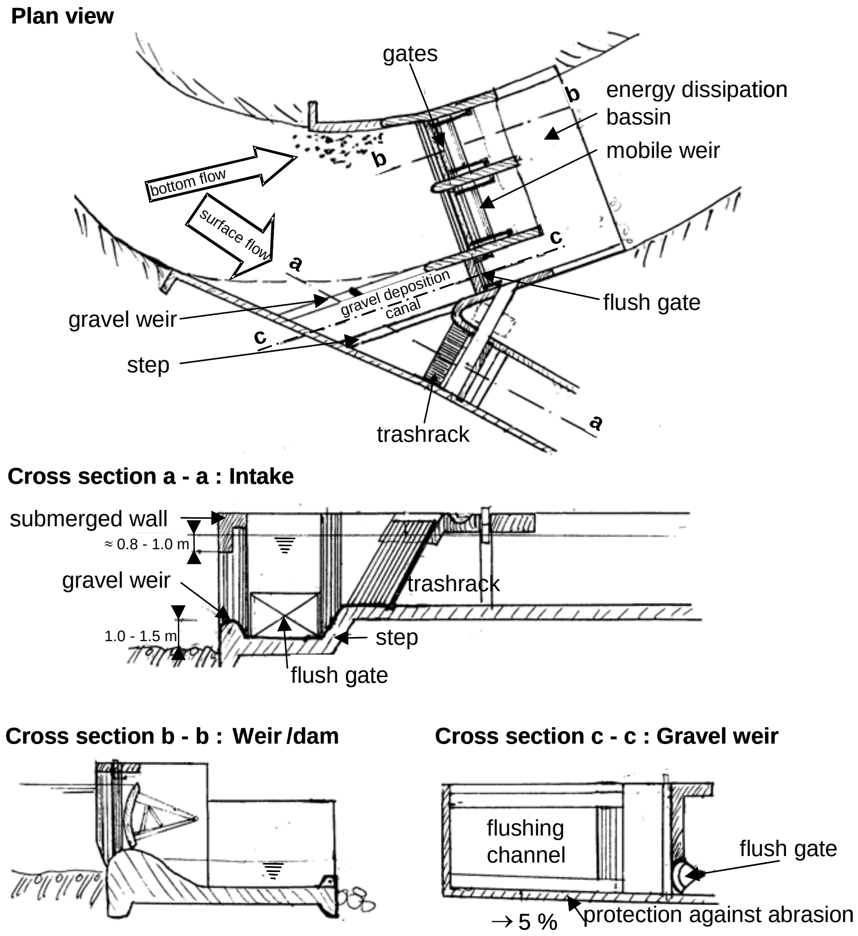
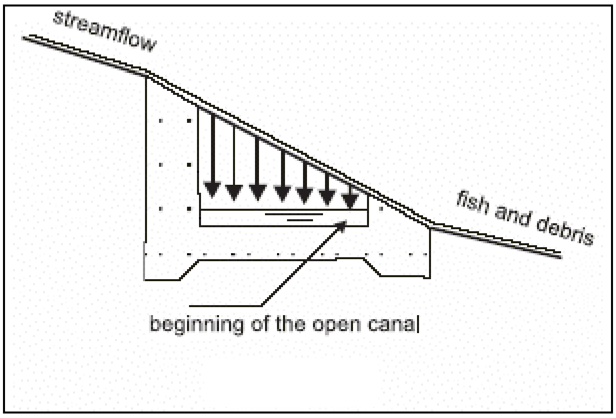


Figure 5.16: Typical layout of lateral intake

The frontal intake is always equipped with a gravel deposition tunnel and is well adapted for rectilinear river reaches. The deposition tunnel has to be flushed in a continuous manner and the maximum river width is 50 m. A major advantage of this type of intake is its ability to handle large quantities of both bed and suspended load. However, this needs continuous flushing and thus large losses of water. The frontal intake is largely applied in regions with very large bed and suspended loads, such as for example in India and Pakistan. In Europe, its application is largely restricted.

The drop intake is generally used in steep sloped rivers, such as torrents, and for rectilinear reaches. The "French" drop intake (Figure 5.17) is essentially a canal built in the streambed, stretching across it and covered by a trashrack with a slope greater than the streambed slope. The trashrack bars are oriented in the direction of the streamflow. Photo 5.8 shows a drop intake installed in a mountain stream in Asturias (Spain).



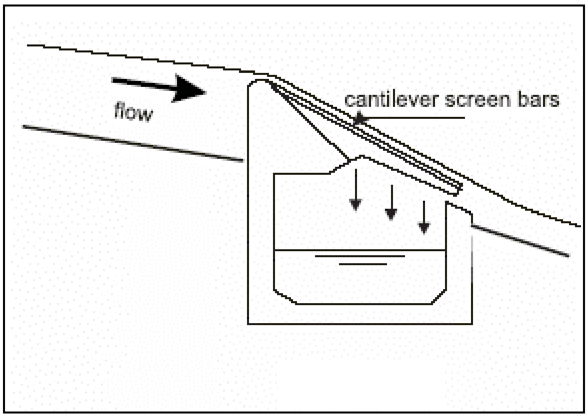


Figure 5.17: Secondary current along the outer bend of a curved river.

Photo 5.8: Drop intake

The Coanda type screen is an advanced concept of the drop intake, incorporating the "Coanda effect", well known in the ore separation industry, to separate fish and debris from clean water. Essentially it consists of a weir with a downward sloping profiled surface of stainless steel wire screen mesh on the downstream side and a flow collection channel below the mesh - as in the drop intake. The mesh wires are held horizontal -unlike the drop intake- and are of triangular section to provide an expanding water passage. Water drops through the mesh with debris and fish carried off the base of the screen. The screen is capable of removing 90% of the solids as small as 0.5 mm, so a silt basin and sediment ejection system can be omitted. The intake is patented by AQUA SHEAR and distributed by DULAS 11 in Europe.

In the Alps, a drop intake has been developed that is particularly adapted to very steep torrents in high mountainous regions with difficult access, called the “Tyrolean” intake (Figure 5.18).

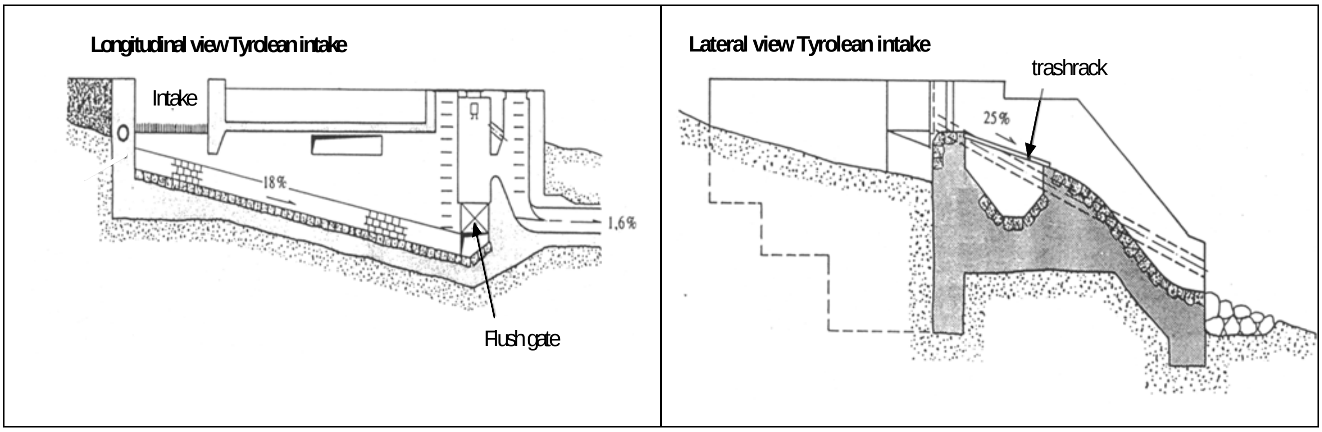


Figure 5.18: "Tyrolean" intake

Power intakes are mostly used on lakes and reservoirs. The water is transferred under pressure and the problems associated with these kinds of intakes are different than for conveyance intakes. For example, sediments are much less able to enter the intake, although they may pose a problem by deposition in the lake itself. On the other hand, pressurized intakes with low pressure heads contain the risk of vortex formation at their entrance and thus the formation of air pockets inside the downstream conduit. This is discussed later on.

5.5.3 Head losses

For small hydro plants, *head losses* can be of huge importance to the feasibility of the project and should thus be minimized as much as possible. Accounting for the following issues can do this:

- Approach walls to the trashrack designed to minimise flow separation and head losses
- Piers to support mechanical equipment including trashracks, and service gates
- Guide vanes to distribute flow uniformly
- Vortex suppression devices
- Appropriate trashrack design

The velocity profile decisively influences the trashrack efficiency. The velocity along the intake may vary from 0.8 - 1 m/s through the trashrack to 3 - 5 m/s in the penstock. A good profile will achieve a uniform acceleration of the flow, minimising head losses. A sudden acceleration or deceleration of the flow generates additional turbulence with flow separation and increases the head losses. Unfortunately a constant acceleration with low head losses requires a complex and lengthy intake, which is expensive. A trade-off between cost and efficiency should be achieved. The maximum acceptable velocity dictates the penstock diameter; the need for a reasonable velocity of the flow approaching the trashrack dictates the dimensions of the rectangular section.

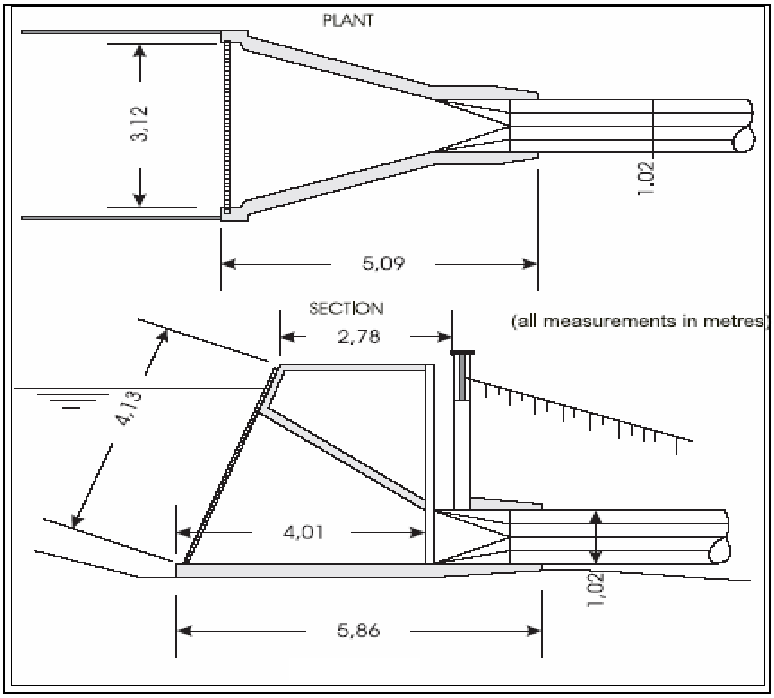


Figure 5.19: French drop intake: a canal built in the streambed and covered by a trashrack

The research department of "Energy, Mines and Resources" of Canada commissioned a study of entrance loss coefficients for small, low-head intake structures to establish guidelines for selecting optimum intake geometry. The results showed that economic benefits increase with progressively smoother intake geometry having multiplane roof transition planes prepared from flat form work. In addition, it was found that cost savings from shorter and more compact intakes were significantly higher than the corresponding disadvantages from increased head losses.

Analyses of cost/benefits suggests that the best design is that of a compact intake with a sloping roof and converging walls (Figure 5.19, alternative 2 in the study), whilst the length of the intake is unlikely to be the major factor contributing to the overall loss coefficient. The K coefficient of this transition profile was 0.19. The head loss (m) in the intake is given by:

$$\Delta H = 0.19\ V^2/2g \qquad (5.1)$$

where V is the velocity in the penstock (m/s). Head losses due to the trashrack depend on spacing and shape of the bars, orientation of the trashrack compared to the flow and eventual obstruction due to debris and are discussed in more detail below.

5.5.4 Trashracks

One of the major functions of the intake is to minimise the amount of debris and sediment carried by the incoming water, so trashracks are placed at the entrance to the intake to prevent the ingress of floating debris and large stones. A trashrack is made up of one or more panels, fabricated from a series of evenly spaced parallel metal bars. If the watercourse, in the flood season, entrains large debris, it is convenient to install, in front of the ordinary grill, a special one, with removable and widely spaced bars (from 100 mm to 300 mm between bars) to reduce the work of the automatic trashrack cleaning equipment.

Trashracks are fabricated with stainless steel or plastic bars. Since the plastic bars can be made in airfoil sections, less turbulence and lower head losses result. The bar spacing varies from a clear width of 12 mm for small high head Pelton turbines to a maximum of 150 mm for large propeller turbines. The trashrack should have a net area (the total area less the bars frontal area) so that the water velocity does not exceed 0.75 m/s on small intakes, or 1.5 m/s on larger intakes, to avoid attracting floating debris to the trashrack. Trashracks can be either be bolted to the support frame with stainless steel bolts or slid into vertical slots, to be removed and replaced by stoplogs when closure for maintenance or repair is needed. In large trashracks it must be assumed that the grill may be clogged and the supporting structure must be designed to resist the total water pressure exerted over the whole area without excessive deformation.

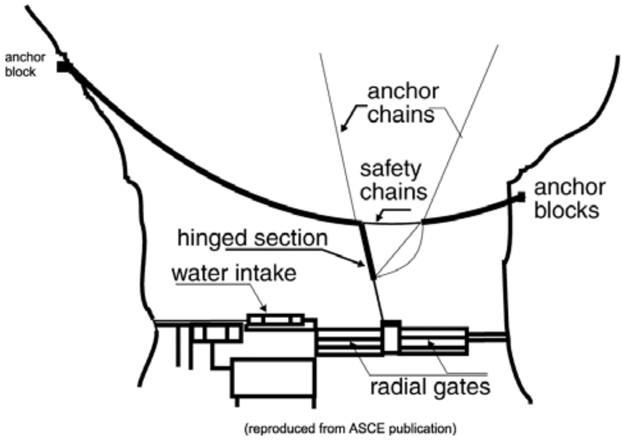
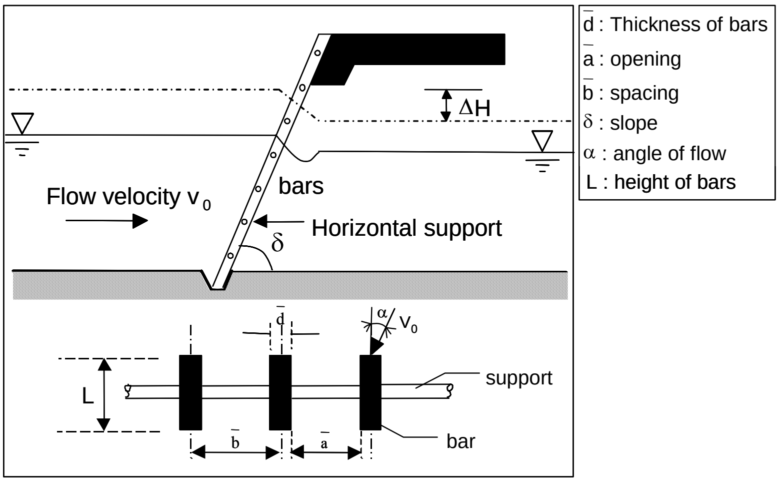


Photo 5.9: Prefabricated booms

Figure 5.20: Trash boom layout

When the river entrains heavy debris, floating booms may be located ahead of the trashracks. The simplest boom consists of a series of floating pieces of timber connected end to end with cables or chains. However modern booms are built with prefabricated sections of steel and plastic (Photo 5.9) supported by steel cables. Their location is critical, because their inward bowed configuration does not lend itself to a self-cleaning action during flood flows. Figure 5.20 (reproduced from reference 11) shows a rather complex trash boom layout designed for a dual-purpose: preventing boats passing over the spillway and protecting the adjacent intake. A section of the boom is hinged at one end of the fixed section so that winches can handle the other end to let the trash pass downstream over the spillway, when large quantities are passing.

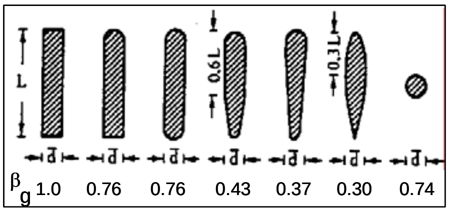
The trashrack is designed so the approach velocity (V_0) remains between 0.60 m/s and 1.50 m/s. The maximum possible spacing between the bars is generally specified by the turbine manufacturers. Typical values are 20-30 mm for Pelton turbines, 40-50 mm for Francis turbines and 80-100 mm for Kaplan turbines.



$$\Delta H = \zeta_g \cdot \frac{v_0^2}{2 \cdot g}$$

$$\zeta_g = \beta_g \cdot \xi \cdot c \cdot (\sin \delta) \cdot \kappa$$

β_g : Shape factor of bars



ξ : Head loss factor

$$\xi = f\left(\frac{L}{d}, \frac{A_g}{A_0}\right)$$

for $\frac{L}{d} \approx 5$ and $\frac{\bar{a}}{b} > 0.5$:

$$\xi = \frac{7}{3}\left(\frac{\bar{a}}{b} - 1\right)^{4/3}$$

c : Trashrack coefficient

$c = 1$: Non-obstructed rack

$1.1 < c < 1.3$: Rack with mechanical cleaning

$1.5 < c < 2$: Rack with manual cleaning

α \ $\bar{d}/\bar{a}$	1,0	0,9	0,8	0,7	0,6	0,5	0,4	0,3	0,2
0°	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
10°	1,06	1,07	1,08	1,09	1,10	1,11	1,12	1,14	1,50
20°	1,14	1,16	1,18	1,21	1,24	1,26	1,31	1,43	2,25
30°	1,25	1,28	1,31	1,35	1,44	1,50	1,64	1,90	3,60
40°	1,43	1,48	1,55	1,64	1,75	1,88	2,10	2,56	5,70
50°	1,75	1,85	1,96	2,10	2,30	2,60	3,00	3,80	---
60°	2,25	2,41	2,62	2,90	3,26	3,74	4,40	6,05	---

Figure 5.21: Formula for computing head losses

As can be seen, the head loss coefficient depends on several factors, such as for example the way of cleaning of the rack. The presented equations (Figure 5.21) are strictly only valid for rectangular bars, but experience has proven that they can also be used for other bar shapes. Cleaning of trashracks is very important to reduce possible head losses through the system. Manual cleaning is very difficult, especially during floods. Therefore, mechanical cleaning is recommended.

Another formula for computing head losses in clean trashracks is the Kirscmer formula, detailed in Chapter 2, section 2.2.2.1. This formula is only valid when the flow approaches the screen at right angles.

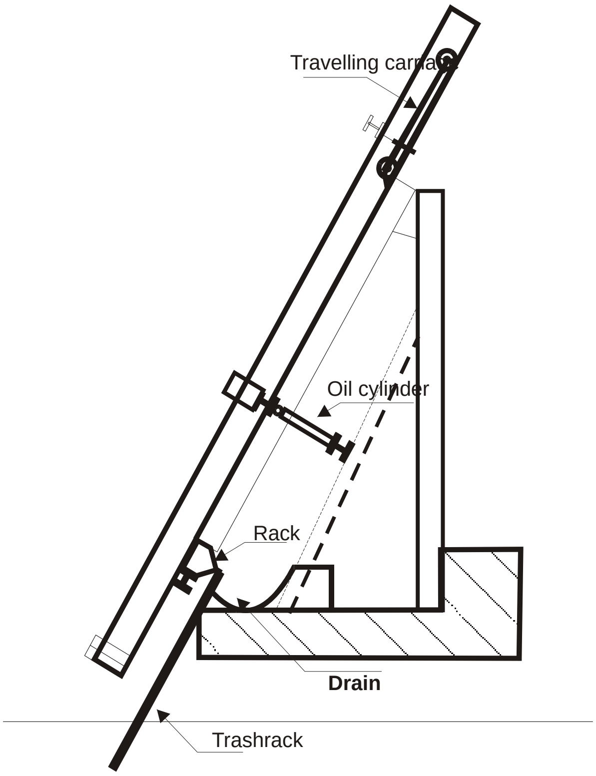


Figure 5.22: Oleo-Hydraulic cylinders

Photo 5.10: Telescopic hydraulic cylinders

The trashrack should be removable for repair and maintenance and provided with facilities to clean it. To facilitate the hand cleaning of the trashrack it should be inclined at an angle 300^{o} from the horizontal although steeper angles are often used. Trashracks can be cleaned by hand up to 4 meters depth. A horizontal platform above high-water level should be provided to facilitate the operation. On unattended plants operated by remote control, mechanical rakers are used. The mechanical raker can be designed to be operated either on a timed basis or on a head differential basis. The latter uses a sensor to detect the drop in head across the trashrack. An accumulation of trash on the trashrack creates an increased differential head across the trashrack. The raker begins when a predetermined differential head is reached.

The raker in Figure 5.22 is operated through oleo-hydraulic cylinders. The secondary cylinder pushes out or retracts the raker, which rides on a hinged arm. The raker pushes out in its way down to the bottom of the screen and then retracts to travel up along the screen. The raker itself is a series of prongs protruding from a polyamide block that moves along the spaces between the bars. The trash is conveyed to the top to be dumped on a conduit or on to a conveyor. If dumped into a conduit a small water pump delivers enough water to wash the trash along the canal. The problem of trash disposal must be solved case by case, bearing in mind that a trash raker can remove large amount of debris.

When the trashrack is very long the trash raker described above is assembled on a carriage that can move on rails along the intake. Automatic control can be programmed to pass along the supporting structures without human aid. Using telescopic hydraulic cylinders the raker can reach down to 10 m deep, which combined with the almost limitless horizontal movement, makes it possible to clean large surface screens (Photo 5.10).

5.5.5 Vorticity

A well-designed intake should not only minimise head losses but also preclude *vorticity*. Vorticity can appear for low-head pressurized intakes (power intakes) and should be avoided because it interferes with the good performance of turbines - especially bulb and pit turbines. Vortices may effectively:

- Produce non-uniform flow conditions
- Introduce air into the flow, with unfavourable results on the turbines: vibration, cavitation, unbalanced loads, etc.
- Increase head losses and decrease efficiency
- Draw trash into the intake

The criteria to avoid vorticity are not well defined, and there is not a single formula that adequately takes into consideration the possible factors affecting it. According to the ASCE Committee on Hydropower Intakes, disturbances, which introduce non-uniform velocity, can initiate vortices. These include:

- Asymmetrical approach conditions
- Inadequate submergence
- Flow separation and eddy formation
- Approach velocities greater than 0.65 m/sec
- Abrupt changes in flow direction

Lack of sufficient submergence and asymmetrical approach seem to be the most common causes of vortex formation. An asymmetric approach is more prone to vortex formation than a symmetrical one. When the inlet to the penstock is deep enough and the flow is undisturbed, vortex formation is unlikely.

Empirical formulas exist that express the minimum degree of submergence of the intake in order to avoid severe vortex formation. Nevertheless, no theory actually exists that fully accounts for all relevant parameters. The minimum degree of submergence is defined as shown in Figure 5.23.

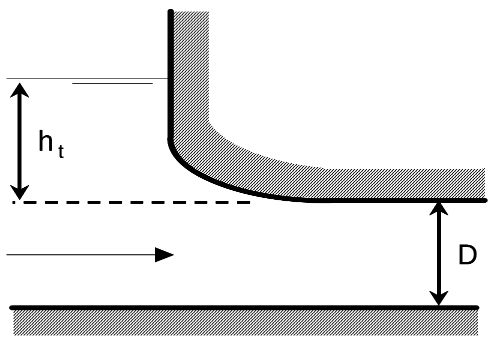


Figure 5.23: Minimum degree of submergence

The submersion is defined as h_t. The following formulas express the minimum values for h_t:

KNAUSS $$h_t \geq D \cdot \left(1 + 2.3 \cdot \frac{V}{\sqrt{g \cdot D}}\right) \tag{5.2}$$

NAGARKAR $$h_t \geq 4.4 \cdot \left(V \cdot D^{0.50}\right)^{0.54} \tag{5.3}$$

ROHAN $$h_t \geq 1.474 \cdot V^{0.48} \cdot D^{0.76} \tag{5.4}$$

GORDON $$h_t \geq c \cdot V \cdot \sqrt{D} \tag{5.5}$$

with $c = 0.7245$ for asymmetric approach conditions

$c = 0.5434$ for symmetric approach conditions

It is important to highlight that V is the velocity inside the downstream conduit in m/s and D is the hydraulic diameter of the downstream conduit in m.

Beside a minimum submersion, constructive measures might help to prevent vortex formation. For example, asymmetric flow conditions may be prevented by means of vertical walls, piles, screens, floating rafts or by appropriate design of the entrance shape.

5.6 Sediment traps

5.6.1 General

Conveyance intakes are designed on rivers in order to eliminate possible floating debris and bedload transport. However, they cannot prevent the entrance of suspended sediment transport. For this, a sediment trap is projected downstream of an intake. The main objective of such a trap is to avoid sedimentation of downstream structures (canals, shafts, etc.) as well as to limit the possible damage of sediments on the hydro mechanical equipment.

A sediment trap is based on the principle of diminishing the flow velocities and turbulence. This results in a decantation of suspended sediments in the trap. This diminishing is obtained by an enlargement of the canal, controlled by a downstream weir as shown in Figure 5.24.

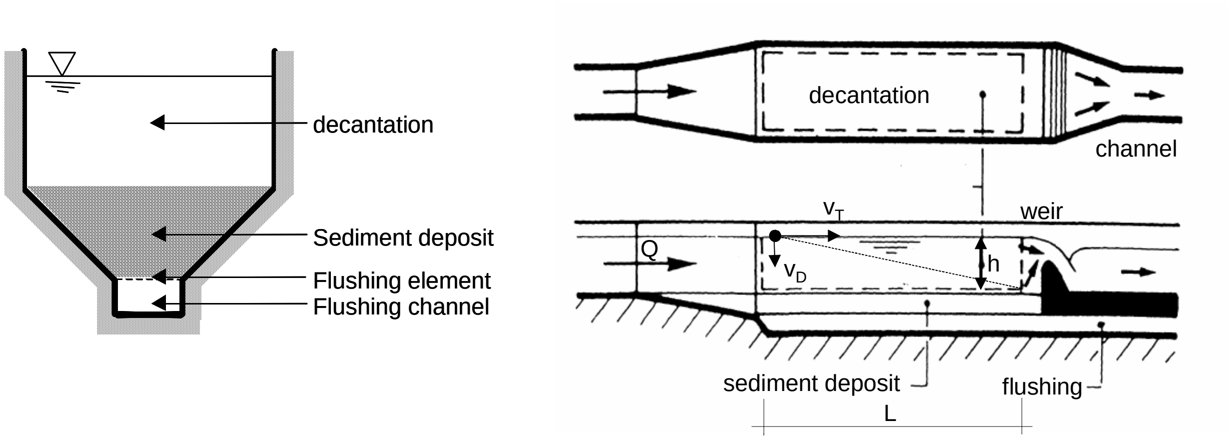


Figure 5.24: Sediment traps

A sediment sluicing system that minimises the sluicing time and the wasted water can be used[4].

5.6.2 Efficiency of a sediment trap

The efficiency of the sediment trap is defined by the grain diameter that deposits in the trap. The choice of efficiency depends on the type of hydro mechanical equipment and on the gross head difference of the power plant. For a Francis turbine, the abrasive power of sediment grains is expressed as a function of the velocity of the grains and the gross head of the plant as follows:

$$P_e = \mu \cdot \forall \cdot \frac{\rho_s - \rho_E}{R} \cdot V^3 \qquad (5.6)$$

in which μ is a friction coefficient between the turbine blades and the grains, $\forall$ is the volume of the grains, ρ_s and ρ_E are the densities of grains and water, R is the radius of the blades and V is the grain velocity. The volume of the grains is directly related to the efficiency of the trap.

Reparation intervals of Francis turbines are around 6-7 years for a sediment trap efficiency of 0.2 mm, 3-4 years for an efficiency of 0.3 mm and 1-2 years for an efficiency of only 0.5 mm. It is obvious that the cost of a sediment trap increases with its efficiency. Hence, an optimum efficiency may be found as a function of the construction costs, the energy losses, the reparation costs of the turbines and the exploitation costs. Experience has shown that the most economical solution is around 0.2 mm efficiency for severe conditions (significant gross head, quartz particles) and around 0.3 mm for normal conditions.

5.6.3 Design

The necessary length of a sediment trap is defined by the equipped discharge of the intake and by the chosen efficiency of the trap (grain diameter that still deposits inside the trap). The length has to be such that all grains have the time to deposit before leaving the trap. This happens when the deposition time t_D equals the transfer time t_t. The former is defined as h/v_D and the latter as L/v_T (see Figure 5.24). Hence, the minimum length required to deposit a grain of diameter d_D is given:

$$L \geq \frac{Q}{v_D \cdot B} \qquad (5.7)$$

The width B has to stay smaller than 1/8 times the length L and also smaller than twice the flow depth h. The deposition velocity v_D is defined by the Newton or Prandtl formula for spherical particles and under ideal conditions, i.e. pure water, no turbulence and no wall effects. It depends on the form drag of the particle, which on its turn depends on the Reynolds number. For real situations, no formula exists and experiments should be carried out. For practice, the empirical formula of Zanke is often used as a first-hand approach in still water flow conditions:

$$v_D = \frac{100}{9 \cdot d}\left(\sqrt{1 + 1.57 \cdot 10^2 \cdot d^3} - 1\right) \tag{5.8}$$

in which v_D is expressed in mm/s and the grain diameter d in mm. This expression is strictly valid for T = 20° and a grain-to-water density ratio of 2.65.

For turbulent flow conditions, the deposition velocity decreases and the following expression becomes more appropriate:

$$v_D = v_{D0} - \alpha \cdot v_T \geq 0 \tag{5.9}$$

in which v_{D0} is the deposition velocity in still water and α a reduction factor (in $[1/m^{1/2}]$) expressed as a function of the trap water depth h (m):

$$\alpha = \frac{0.132}{\sqrt{h}} \tag{5.10}$$

Finally, for appropriate design, the critical transfer velocity of the trap has to be defined. This critical velocity defines the limit between the suspension regime and the deposition regime. If the velocity is too high, deposited sediments risk to be entrained again by the flow. For a Manning-Strickler roughness value of $K = 60\ m^{1/3}/s$ ($K = 1/n$, average value for concrete) and for a grain-to-water density ratio of 2.65 the following formula is valid:

$$v_{cr} = 13 \cdot R_h^{1/6} \cdot \sqrt{d} \tag{5.11}$$

Typical values for v_{cr} are 0.2-0.3 m/s.

Further information regarding design and construction details can be found for example in Bouvard (1984).

5.7 Gates and valves

In every small hydropower scheme some components, for one reason or another (maintenance or repair to avoid the runaway speed on a shutdown turbine, etc) need to be able to be temporarily isolated. Some of the gates and valves suited to the intakes for small hydro systems include the following:

- Stoplogs made up of horizontally placed timbers
- Sliding gates of cast iron, steel, plastic or timber
- Flap gates with or without counterweights
- Globe, rotary, sleeve-type, butterfly and sphere valves

Almost without exception the power intake will incorporate some type of control gate or valve as a guard system located upstream of the turbine and which can be closed to allow the dewatering of the water conduit. This gate must be designed so it can be closed against the maximum turbine flow in case of power failure, and it should be able to be opened partially, under maximum head, to allow the conduit to be filled.

For low pressure the simplest type of gate is a stoplog; timbers placed horizontally and supported at each end in grooves. Stoplogs cannot control the flow and are used only to stop it. If flow must be stopped completely, such as when a repair is needed downstream, the use of two parallel sets of stoplogs is recommended. They should be separated by about 15 cm, so that clay can be packed in between. Gates and valves control the flow through power conduits. Gates of the sliding type are generally used to control the flow through open canals or other low-pressure applications. This is the type of flow control used on conveyance intake structures where, if necessary, the flow can be stopped completely to allow dewatering of the conduit. Cast iron sliding-type gates are those mostly used for openings of less than two square meters. For bigger openings fabricated steel sliding gates are cheaper and more flexible. Gates of the sliding type are seldom used in penstocks because they take too long to close. The stopper slides between two guides inside the gate.

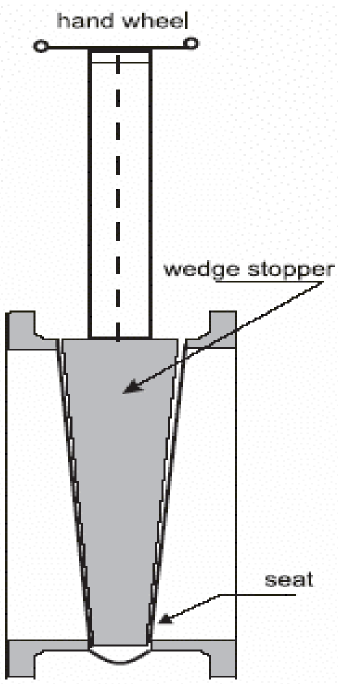


Photo 5.11: wheel-and-axle mechanism

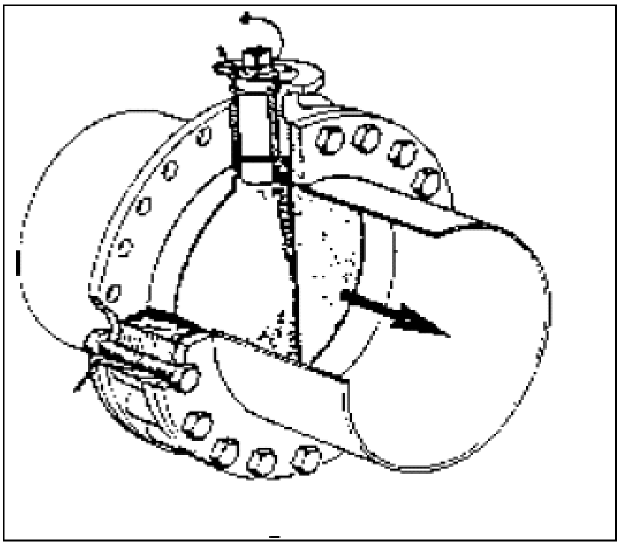
Figure 5.25: Wedge-shaped stopper

When used in a high-pressure conduit the water pressure that forces the stopper against its seat makes the valve difficult to operate. This difficulty is overcome with a wedge-shaped stopper (Figure 5.25), so that the seal is broken over the whole face as soon as it rises even a small distance. To provide a good seal around a sliding gate different kinds of rubber seals are used. They can be made of natural rubber, styrene-butadiene or chloroprene compounds. The seal path is located adjacent to the roller path.

Using a wheel-and-axle mechanism (Photo 5.11), a hydraulic cylinder (Photo 5.12) or an electric actuator on a screw thread can raise small sliding gates controlling the flow.

In butterfly valves a lens shaped disk mounted on a shaft turns to close the gap (Figure 5.26). Under pressure each side of the disk is submitted to the same pressure, so the valve is easy to manoeuvre and closes rapidly. Butterfly valves are used as the guard valves for turbines and as regulating valves. Is easy to understand that when used for regulation their efficiency is rather low because the shaped disk remains in the flow and causes turbulence.

Photo 5.12: Hydraulic Cylinder



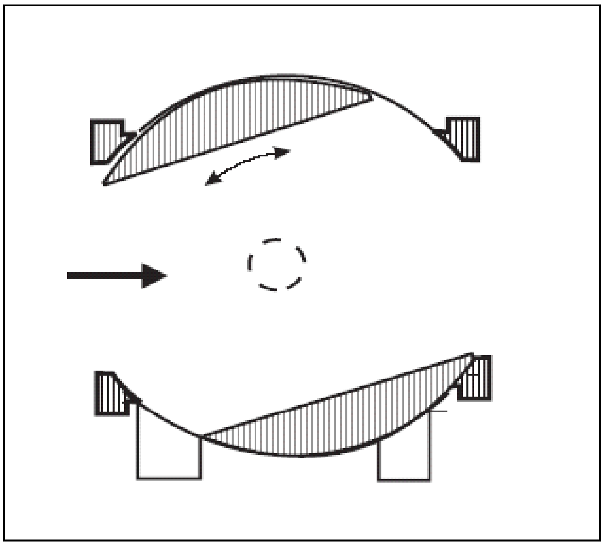


Figure 5.26: Butterfly valves

Figure 5.27: Globe and rotary valves

Butterfly valves are simple, rugged and uncomplicated and can be operated manually or hydraulically. Photo 5.13 shows a large butterfly valve being assembled in a powerhouse and Photo 5.14 shows a butterfly valve, hydraulically operated, with an ancillary opening system and a counterweight, at the entrance to a small Francis turbine.

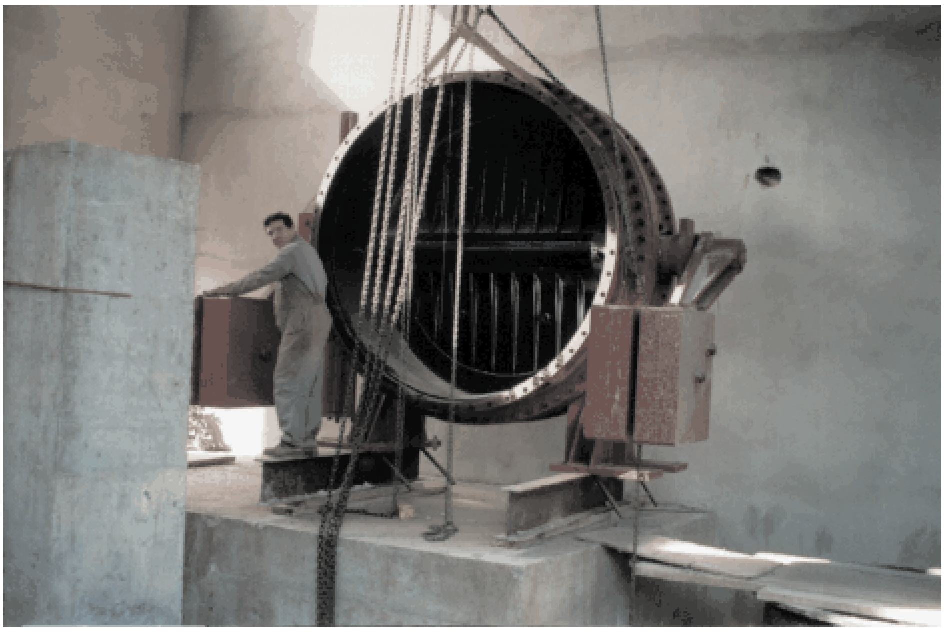


Photo 5.13: Large butterfly valve

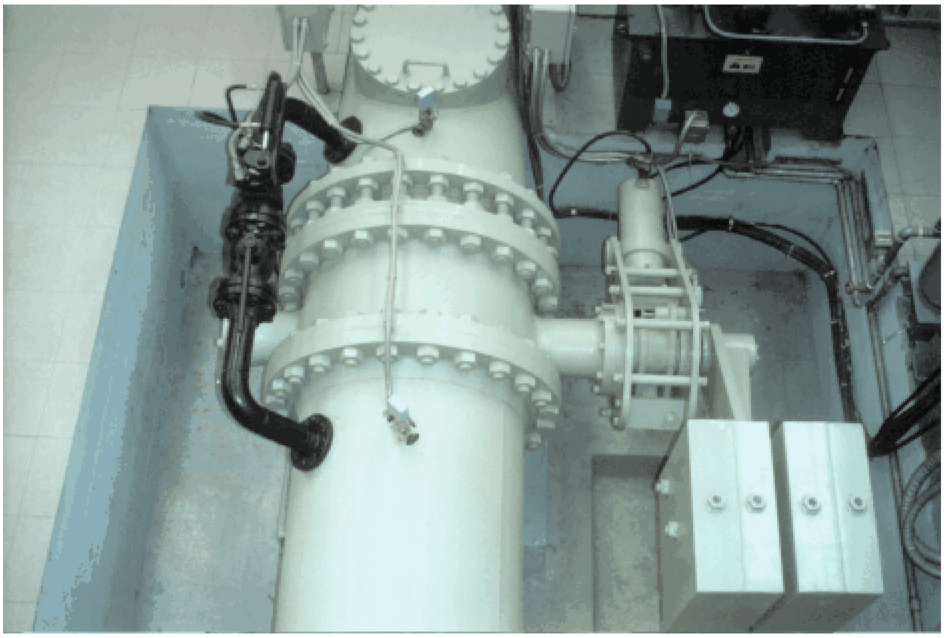


Photo 5.14: Butterfly valve hydraulically operated

Globe and rotary valves (Figure 5.27) have lower head losses than the slide and butterfly gate valves and are also commonly used in spite of their higher price.

The radial gates, conceptually different, are a method of forming a moveable overflow crest and allow a close control of headwater and tailwater. Photo 5.15 shows a Tainter gate at the left, ready to be installed, and the housing of the sector on a concrete pier at the right. The radial gate is operated by raising or lowering to allow water to pass beneath the gate plate. The curved plate that forms the upstream face is concentric with the trunnions of the gate. The trunnions are anchored in the piers and carry the full hydrostatic load. Because the hydrostatic load passes through the trunnions, the lifting force required by the hoisting mechanism is minimised. The head losses in gates and valves are relatively high, especially when they are operated as regulating devices. For further details refer to Chapter 2, Section 2.2.4 and the enclosed bibliography.

Photo 5.15: Tainter gate (left) and housing of its sector on a concrete pier

5.8 Open channels

5.8.1 Design and dimensioning

The flow conveyed by a canal is a function of its cross-sectional profile, its slope, and its roughness. Natural channels are normally very irregular in shape, and their surface roughness changes with distance and time. The application of hydraulic theory to natural channels is more complex than for artificial channels where the cross-section is regular in shape and the surface roughness of the construction materials - earth, concrete, steel or wood - is well documented, so that the application of hydraulic theories yields reasonably accurate results.

Table 2.4, Chapter 2, illustrates the fundamental geometric properties of different channel sections.

In small hydropower schemes the flow in the channels is in general in the rough turbulent zone and the Manning equation can be applied:

$$Q = \frac{A \cdot R^{2/3} \cdot S^{1/2}}{n} = \frac{A^{5/3} \cdot S^{1/2}}{n \cdot P^{2/3}} \qquad (5.12)$$

where n is Manning's coefficient, which in the case of artificial lined channels may be estimated with reasonable accuracy, and S is the hydraulic gradient, which normally is the bed slope. Alternatively:

$$S = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{Q \cdot n}{AR^{2/3}}\right)^2 \qquad (5.13)$$

The above equation applies when metric or SI units are used. To use Imperial or English units the equation must be modified to:

$$Q = \frac{1.49 A^{5/3} \cdot S^{1/2}}{n \cdot P^{2/3}} \qquad (5.14)$$

where Q is in ft^3/s; A in ft^2 and P in ft. n has the same value as before.

The above equation shows that for the same cross-sectional area A and channel slope S, the channel with a larger hydraulic radius R, delivers a larger discharge. That means that for a given cross-sectional area, the section with the least wetted perimeter is the most efficient hydraulically. Semicircular sections are consequently the most efficient. A semicircular section however, unless built with prefabricated materials, is expensive to build and difficult to maintain. The most efficient trapezoidal section is the half hexagon, whose side slope is 1 vertical to 0.577 horizontal. Strictly, this is only true if the water level reaches the level of the top of the bank. Actual dimensions have to include a certain freeboard (vertical distance between the designed water surface and the top of the channel bank) to prevent water level fluctuations overspilling the banks. Minimum freeboard for lined canals is about 10 cm, and for unlined canals this should be about one third of the designed water depth with a minimum of fifteen centimetres. One way to prevent overflow of the canal is to provide spillways at appropriate intervals; any excess water is conveyed, via the spillway, to an existing streambed or to a gully.

Table 5.2: Hydraulic parameters for common canal cross-sections

Type of Channel	Manning's n
Excavated earth channels	
Clean	0.022
Gravelly	0.025
Weedy	0.030
Stony, cobbles (or natural streams)	0.035
Artificially lined channels	
Brass	0.011
Steel, smooth	0.012
Steel, painted	0.014
Steel, riveted	0.015
Cast iron	0.013
Concrete, well-finished	0.012
Concrete, unfinished	0.014
Planed wood	0.012
Clay tile	0.014
Brickwork	0.015
Asphalt	0.016
Corrugated metal	0.022
Rubble masonry	0.025

It should be noted that the best hydraulic section does not necessarily have the lowest excavation cost. If the canal is unlined, the maximum side slope is set by the slope at which the material will permanently stand under water. Clay slopes may stand at 1 vertical to 3/4 horizontal, whereas sandy soils must have flatter slopes (1 to 2).

Table 5.3 defines for the most common canal sections the optimum profile as a function of the water depth y, together with the parameters identifying the profile.

Table 5.3: Optimum profile for different channel sections

Channel section	Area A	Wetted perimeter P	Hydraulic radius R	Top width T	Water depth d
Trapezoid: half hexagon	$1.73\ y^2$	3.46 y	0.500 y	2.31 y	0.750y
Rectangle : half square	$2\ y^2$	4 y	0.500 y	2 y	y
Triangle: half square	y^2	2.83 y	0.354 y	2 y	0.500y
Semicircle	0.5πy2	π y	0.500 y	2 y	0.250πy

Example 5.1

Assuming a flow depth of 1 m, a channel base width of 1.5 m and side slopes of 2 vertical to 1 horizontal, a bed slope of 0.001 and a Manning's coefficient of 0.015, determine the discharge (Q), the mean velocity (V).

According to Table 2.4 for b=1.5, x=1/2 and y=1

A=(1.5+0.5x1)x1=2m2; $P = 1.5 + 2x\sqrt{1+0.5^2} = 3.736m$

Applying 5.6) for A=2 and P=3.736

$$Q = \frac{1}{0.015}x\frac{2^{5/3}}{3.736^{2/3}}x\sqrt{0.001} = 2.78m^3/s$$

V=Q/A=2.78/2=1.39 m/s

Example 5.2

Determine the slope knowing the discharge and the canal dimensions. Assuming a canal paved with smooth cement surface (n=0.011), a channel base of 2 m, side slopes with inclination 1v:2h and a uniform water depth of 1.2 m, determine the bed slope for a discharge of 17.5 m^3/s.

Applying the formulae of table 2.4:

$$S = \left(\frac{17.5 \cdot 0.011}{5.28 \cdot 0.717^{2/3}} \right)^2 = 0.002$$

When the canal section, the slope and discharge are known and the depth "d" is required, equation 5.6 - nor any other - does not provide a direct answer, so iterative calculations must be used.

Example 5.3

A trapezoidal open channel has a bottom width of 3 m and side slopes with inclination 1.5:1. The channel is lined with unfinished concrete. The channel is laid on a slope of 0.0016 and the discharge is 21 m^3/s. Calculate the depth.

According to 5.6 the section factor:

A=(b+zy)y = (3 + 1.5y)y   P=b+2y(1+z2)0.5 = 3+3.6y

Compute the factor section for different values of y, up to find one approaching closely 6.825:

For y = 1.5 m A=7.875, R=0.937, AR2/3=7.539

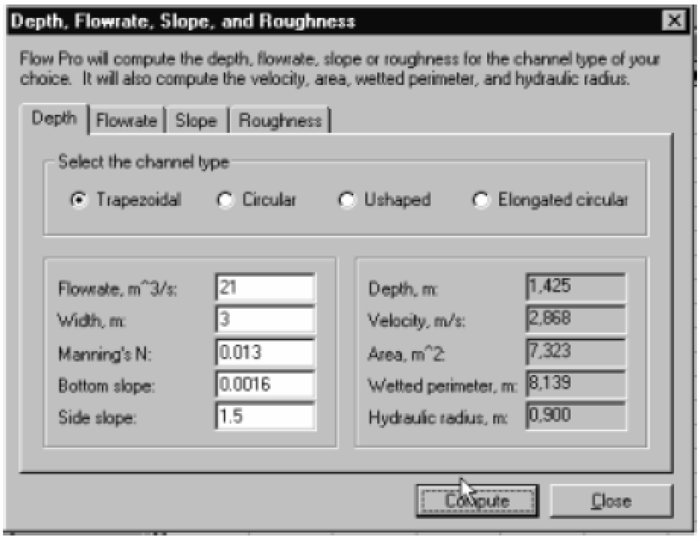
For y = 1.4 m A=7.140, R=0.887. AR2/3=6.593

For y = 1.43 m A=7.357, R=0.902, AR2/3=6.869

According to the above results the normal depth is slightly under 1.43. Using the software program FlowPro, mentioned in Chapter 2 it would be instantaneously calculated, as shown in the enclosed captured screen: a depth of 1.425, with A=2.868, P=8.139, R=0.900 and a section factor 6.826

Summarising, the design of fabricated channels is a simple process requiring the following steps:

- Estimate the coefficient n from table 5.2
- Compute the form factor AR2/3=nQ/S1/2 with the known parameters in second term
- If optimum section is required apply values in table 5.3. Otherwise use values in table 2.4
- Check if the velocity is high enough to form deposit or aquatic flora
- Check the Froude number NF to determine if it is a subcritical or a supercritical flow
- Define the required freeboard



Example 5.4

Design a trapezoidal channel for an 11 m^3/s discharge. The channel will be lined with well-finished concrete and the slope 0.001.

Step 1. Manning n = 0.012

Step 2. Compute form factor

$$AR^{2/3} = \frac{nQ}{\sqrt{S}} = \frac{0.012x11}{\sqrt{0.001}} = 4.174$$

Step 3. Not intended to find the optimum section.

Step 4. Assuming a bottom width of 6 m and side slopes with inclination 2:1 compute the depth d by iteration as in example 5.3.

d = 0.87 m A = 6.734 m^2

Step 5. Compute the velocity

V = 11/6.734 = 1.63 m/s OK

Step 6. Total channel height. The tables of the US Bureau of Reclamation (USA) recommend a freeboard of 0.37 m. The FlowPro software would provide all these results.

5.8.2 Excavation and stability

In conventional hydropower schemes and in some of the small ones, especially those located in wide valleys where the channels must transport large discharges, the channels are designed in the manner shown in Figure 5.28. According to this profile, the excavated ground is used to build the embankments, not only up to the designed height but to provide the freeboard, the extra height necessary to account for the height increase produced by a sudden gate closing, waves or the excess arising in the canal itself under heavy storms. These embankment channels although easy to construct are difficult to maintain, due to wall erosion and aquatic plant growth.

The stability of the walls is defined by the eventual sliding of the material. This sliding can be enhanced by rapid water level changes in the canal. The velocity of water in unlined canals should be kept above a minimum value to prevent sedimentation and aquatic plant growth, but below a maximum value to prevent erosion.

If the canal is unlined, the maximum velocity to prevent erosion is dependent on the mean grain diameter of the bank material d_m:

$$V \leq 5.7 \cdot d_m^{1/3} \cdot R_h^{1/6} \tag{5.15}$$

Where R_h stands for the hydraulic radius of the canal. For grain diameters of 1 mm and hydraulic radius of 1 to 3 m, critical velocities of 0.6-0.7 m/s are obtained. For grain diameters of 10 mm, the critical velocities are between 1.2 and 1.5 m/s for the same hydraulic radius. The above equation can be used for grain diameters larger than 0.1 mm.

For cohesive soils, the critical velocities are between 0.4 and 1.5 m/s. Concrete-lined canals may have clear water velocities up to 10 m/s without danger. Even if the water contains sand, gravel or stones, velocities up to 4 m/s are acceptable.

On the other hand, to keep silt in suspension after the intake, the flow velocity should be at least 0.3-0.5 m/s. To prevent aquatic plant growth, the minimum velocities are 0.5-0.75 m/s and the minimum water depths are 1.5 to 2.0 m.

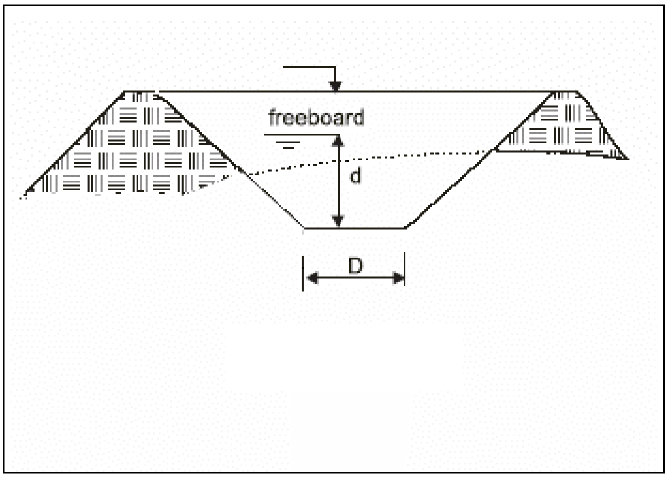


Figure 5.28: Channel design

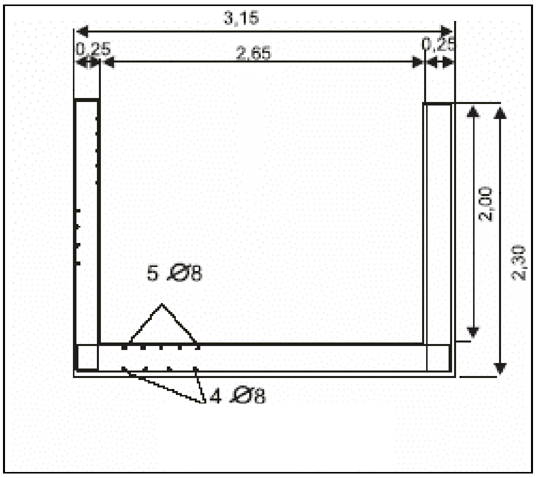


Figure 5.29: Rectangular reinforced canal

An appropriate lining provides bank protection. Possible materials to be used for protection are vegetation, rock blocks with or without mortar, bituminous material, or concrete. Some examples are presented in Figure 5.30.

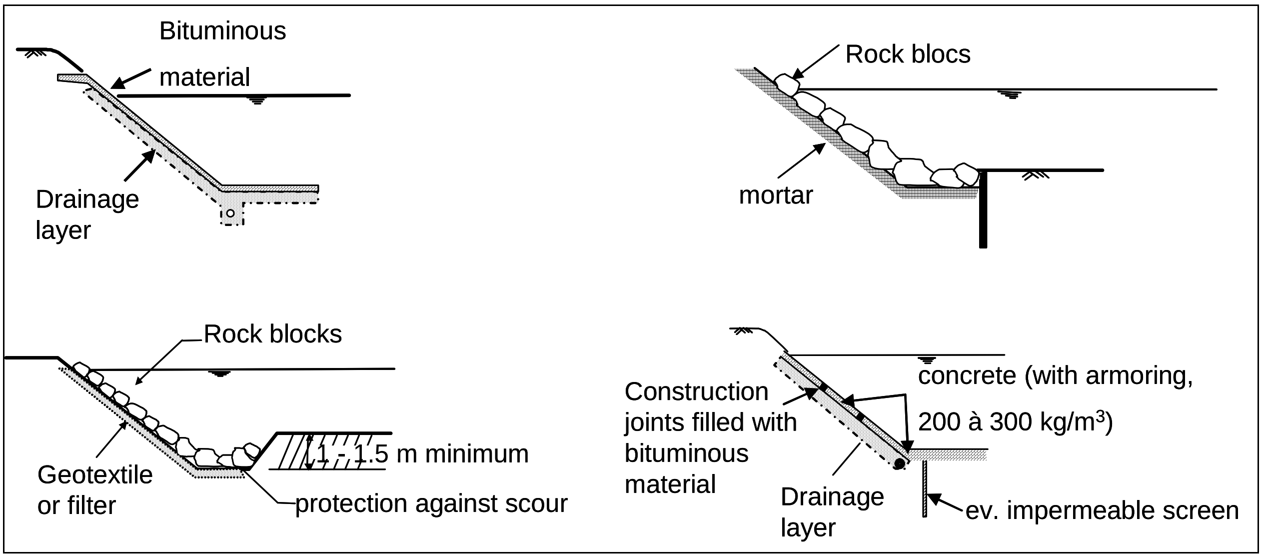


Figure 5.30: Materials used for protection

In high mountain schemes the canal is usually built from reinforced concrete, so that environmental legislation may require it to be covered and revegetated. Figure 5.29 shows the schematic section of a rectangular reinforced concrete canal in the Cordinañes scheme, referred to in Chapter 4 and Photo 5.15 shows the same canal not yet covered with the concrete slab that would serve as a basis for new ground and new vegetation. Sometimes, to ensure that no seepage will occur, the canal is lined with geotextile sheets, to prevent landslides consequent to the wetting of clayey material.

As is shown in the following examples, once the canal profile has been selected it is easy to compute its maximum discharge.

Photo 5.15: Canal in the Cordinañes

Photo 5.16: Lateral spillway

To ensure that the channel never overflows, endangering the slope stability, and in addition to provide a generous freeboard, a lateral spillway (as in Photo 5.16) should be provided.

Before definitely deciding on the channel route, a geologist should carefully study the geomorphology of the terrain. Photo 5.17 shows clearly how uplift can easily ruin a power channel (6 m wide and 500 m long, in a 2 MW scheme). On one particular day, a flood occurred which was later calculated to be a 100 year event. At the time the flood occurred, the headrace channel had been empty, and uplift pressures destroyed the channel. Consideration should also be taken of the type of accidents detailed in Chapter 4, section 4.4.

Photo 5.17:Uplift

Photo 5.18: Flume

Circumventing obstacles

Along the alignment of a canal obstacles may be encountered, and to bypass them it will be necessary to go over, around or under them.

The crossing of a stream or a ravine requires the provision of a flume, a kind of prolongation of the canal, with the same slope, supported on concrete or steel piles or spanning as a bridge. Steel pipes are often the best solution, because a pipe may be used as the chord of a truss, fabricated in the field. The only potential problem is the difficulty of removing sediment deposited when the canal is full of still water. Photo 5.18 shows a flume of this type in China.

Inverted siphons can also solve the problem. An inverted siphon consists of an inlet and an outlet structure connected by a pipe. The diameter calculation follows the same rules as for penstocks, which are analysed later.

5.9 Penstocks

Arrangement and material selection for penstocks

Conveying water from the intake to the powerhouse (this is the purpose of a penstock) may not appear a difficult task. However deciding the most economical arrangement for a penstock is not so simple. Penstocks can be installed over or under the ground, depending on factors such as the nature of the ground itself, the penstock material, the ambient temperatures and the environmental requirements.

A flexible and small diameter PVC penstock for instance, can be laid on the ground, following its outline with sand and gravel surrounding the pipe to provide good insulation. Small pipes installed in this way do not need anchor blocks and expansion joints.

Larger penstocks are usually buried, as long as there is only a minimum of rock excavation required. Buried penstocks must be carefully painted and wrapped to protect the exterior from corrosion, but provided the protective coating is not damaged when installed, further maintenance should be minimal. From the environmental point of view the solution is optimal because the ground can be returned to its original condition, and the penstock does not constitute a barrier to the movement of wildlife.

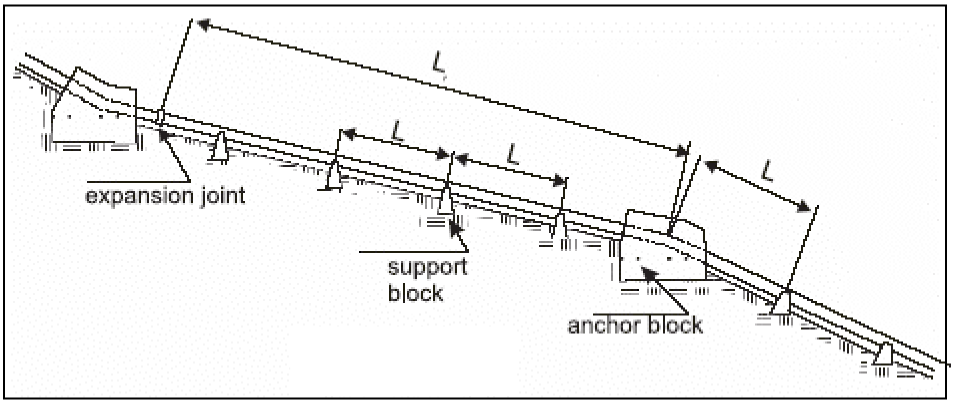


Figure 5.31: Penstock

A penstock installed above ground can be designed with or without expansion joints. Variations in temperature are especially important if the turbine does not function continuously, or when the penstock is dewatered for repair, resulting in thermal expansion or contraction. Usually the penstock is built in straight or nearly straight lines, with concrete anchor blocks at each bend and with an expansion joint between each set of anchors (Figure 5.31). The anchor blocks must resist the thrust of the penstock plus the frictional forces caused by its expansion and contraction, so when possible they should be founded on rock. If, due to the nature of the ground, the anchor blocks require large volumes of concrete, thus becoming rather expensive, an alternative solution is to eliminate every second anchor block and all the expansion joints, leaving the bends free to move slightly. In this case it is desirable to lay the straight sections of the penstock in steel saddles, made to fit the contour of the pipe and generally covering 120 degrees of the invert (Figure 5.32). The saddles can be made from steel plates and shapes, with graphite asbestos sheet packing placed between saddle and pipe to reduce friction forces. The movement can be accommodated with expansion joints, or by designing the pipe layout with bends free to move.

If a pipeline system using spigot and socket joints with O-ring gaskets is chosen, then expansion and contraction is accommodated in the joints.

Today there is a wide choice of materials for penstocks. For the larger heads and diameters, fabricated welded steel is probably the best option. Nevertheless spiral machine-welded steel pipes should be considered, due to their lower price, if they are available in the required sizes. For high heads, steel or ductile iron pipes are preferred, but at medium and low heads steel becomes less competitive, because the internal and external corrosion protection layers do not decrease with the wall thickness and because there is a minimum wall thickness for the pipe.

For smaller diameters, there is a choice between: manufactured steel pipe, supplied with spigot and socket joints and rubber "O" gaskets, which eliminates field welding, or with welded-on flanges, bolted on site (Figure 5.33); plain spun or pre-stressed concrete; ductile iron spigot and socket pipes with gaskets; cement-asbestos; glass-reinforced plastic (GRP); and PVC or polyethylene (PE) plastic pipes. Plastic pipe PE14 is a very attractive solution for medium heads (a PVC pipe of 0.4 m diameter can be used up to a maximum head of 200 meters) because it is often cheaper, lighter and more easily handled than steel and does not need protection against corrosion. PVC15 pipes are easy to install because of the spigot and socket joints provided with "O" ring gaskets. PVC pipes are usually installed underground with a minimum cover of one metre. Due to their low resistance to UV radiation they cannot be used on the surface unless painted, coated or wrapped. The minimum radius of curvature of a PVC pipe is relatively large (100 times the pipe diameter)– and its coefficient of thermal expansion is five times higher than that for steel. They are also rather brittle and unsuited to rocky ground.

Pipes of PE16 – (high molecular weight polyethylene) can be laid on top of the ground and can accommodate bends of 20-40 times the pipe diameter (for sharper bends, special factory fittings are required). PE pipe floats on water and can be dragged by cable in long sections but must be joined in the field by fusion welding, requiring a special machine. PE pipes can withstand pipeline freeze-up without damage, may be not available in sizes over 300 mm diameter.

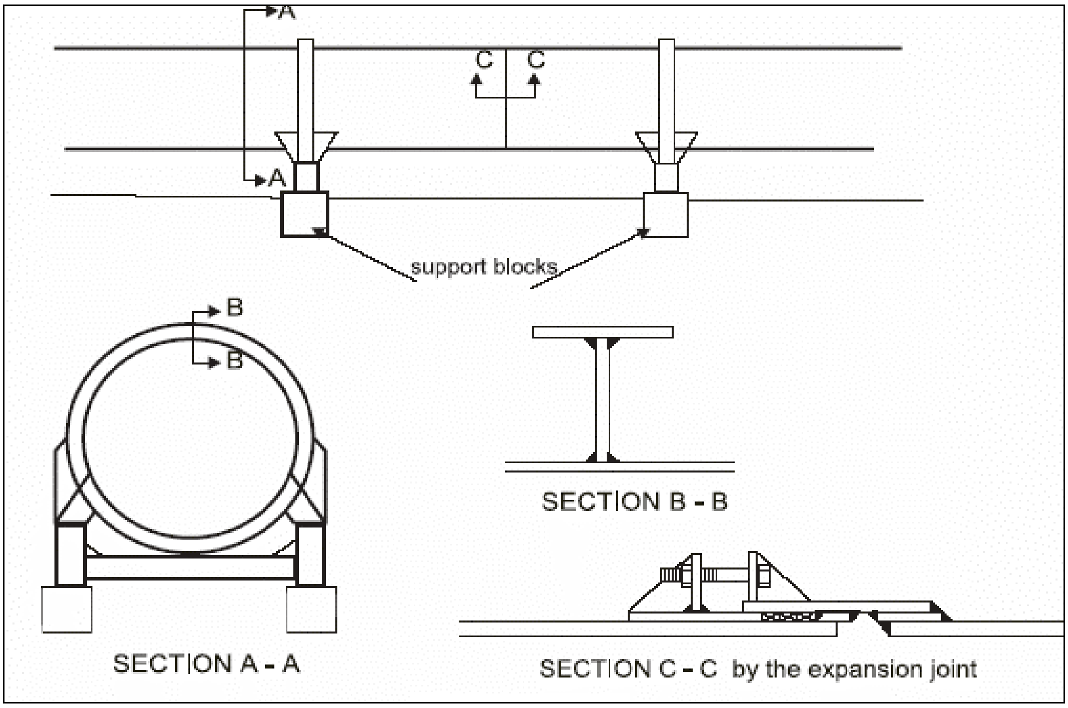


Figure 5.32: Penstock with concrete anchor blocks and expansion joints

Concrete penstocks, both pre-stressed with high tensile wires or steel reinforced, featuring an interior steel jacket to prevent leaks, and furnished with rubber gasket spigot and socket joints constitute another solution. Unfortunately their heavy weight makes transportation and handling costly, but they are not affected by corrosion.

In developing countries, pressure creosoted wood-stave, steel-banded pipe is an alternative that can be used in diameters up to 5.5 metres and heads of up to 50 metres (which may be increased up to 120 meters for a diameter of 1.5 metres). The advantages include flexibility to conform to ground settlement, ease of laying on the ground with almost no grade preparation, no requirement for expansion joints and no necessity for concrete supports or corrosion protection. Wood-stave pipe is assembled from individual staves and steel bands or hoops that allow it to be easily transported even over difficult terrain. Disadvantages include leakage, particularly in the filling operations, the need to keep the pipe full of water when repairing the turbine, and considerable maintenance such as spray coating with tar every five years. Table 5.3 shows the main properties of the above material. Some of these properties are not always typical, particularly the values of the Hazen Williams coefficient which depends on the surface condition of the pipe.

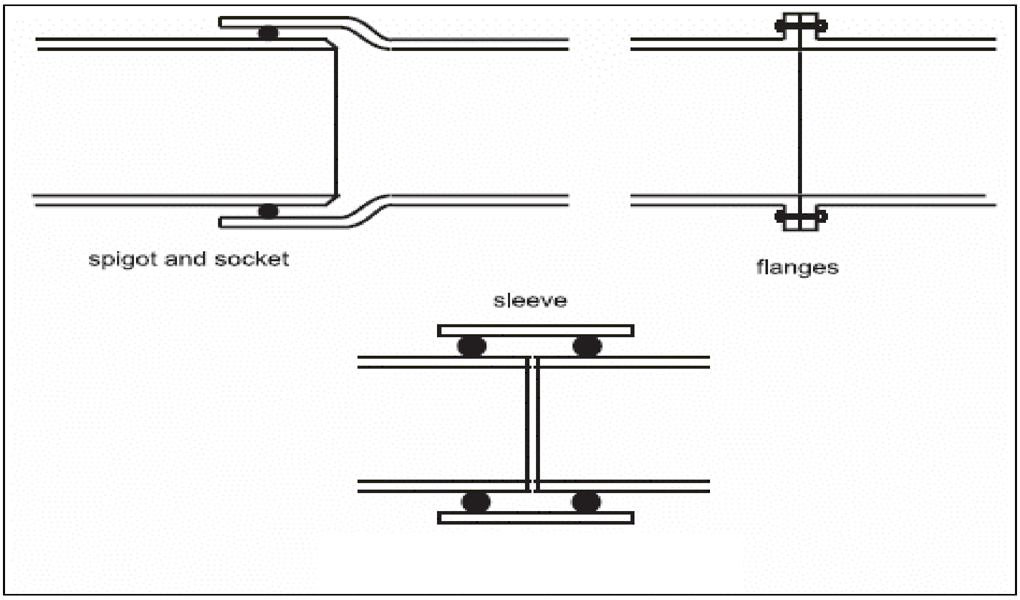


Figure 5.33: Manufactured steel pipe

Table 5.4: Different material's characteristics

Material	Young's modulus of elasticity E(N/m^2)E9	Coefficient of linear expansion a (m/m ^{0}c)E6	Ultimate tensile strength (N/m^2)E6	n
Welded Steel	206	12	400	0.012
Polyethylene	0.55	140	5	0.009
Polyvinyl Chloride (PVC)	2.75	54	13	0.009
Asbestos Cement	n/a	8.1	n/a	0.011
Cast iron	78.5	10	140	0.014
Ductile iron	16.7	11	340	0.013

Hydraulic design and structural requirements

A penstock is characterised by materials, diameter, wall thickness and type of joint:

- the material is selected according to the ground conditions, accessibility, weight, jointing system and cost,
- the diameter is selected to reduce frictional losses within the penstock to an acceptable level,

- the wall thickness is selected to resist the maximum internal hydraulic pressure, including transient surge pressure that will occur.

Penstock diameter

The diameter is selected as the result of a trade-off between penstock cost and power losses. The power available from the flow Q and head H is given by the equation:

$P=QH\gamma\eta$

where Q is the discharge in m^3/s, H the net head in m, γ the specific weight of water in kN/m^3 and η the overall efficiency.

The net head equals the gross head minus the sum of all losses, including the friction and turbulence losses in the penstock, that are approximately proportional to the square of the velocity of the water in the pipe. To convey a certain flow, a small diameter penstock will need a higher water velocity than a larger diameter penstock, and therefore the losses will be greater. Selecting a diameter as small as possible will minimise the penstock cost but the energy losses will be larger and vice versa. Chapter 2 details the friction loss calculations, putting special emphasis on the graphic representation of the Colebrook equations (the Moody diagram and the Wallingford charts) and on the Manning's formula. In this chapter the above principles are used and some examples will facilitate their application in real cases.

A simple criterion for diameter selection is to limit the head loss to a certain percentage. Loss in power of 4% is usually acceptable. A more rigorous approach is to select several possible diameters, computing power and annual energy. The present value of this energy loss over the life of the plant is calculated and plotted for each diameter (Figure 5.34). On the other side the cost of the pipe for each diameter is also calculated and plotted. Both curves are added graphically and the optimum diameter would be that closest to the theoretical optimum.

Actually the main head loss in a pressure pipe are friction losses. The head losses due to turbulence passing through the trashrack, in the entrance to the pipe, in bends, expansions, contractions and valves are minor losses. Consequently a first approach will suffice to compute the friction losses, using for example the Manning equation:

$$\frac{h_f}{L} = 10.3\frac{n^2Q^2}{D^{5.333}} \tag{5.16}$$

Examining the above equation, it can be seen that dividing the diameter by two would lead to the losses being multiplied by 40.

From this it follows that:

$$D = \left(\frac{10.3 \cdot n^2Q^2 \cdot L}{h_f}\right)^{0.1875} \tag{5.17}$$

If we limit h_f at 4H/100, D can be computed knowing Q, n and L, by the equation:

$$D = 2.69\left(\frac{n^2 Q^2 L}{H}\right)^{0.1875} \tag{5.18}$$

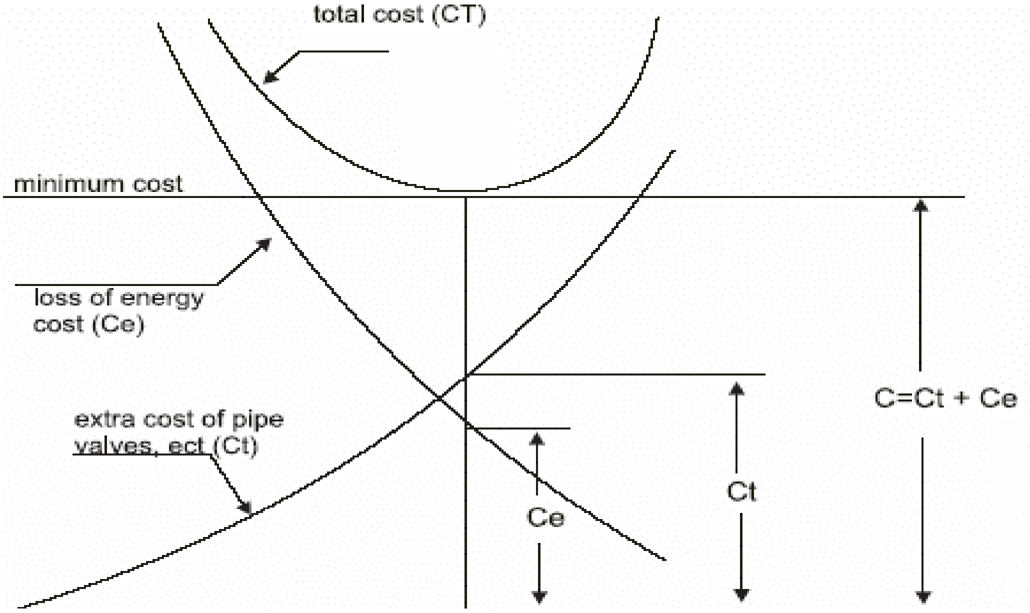


Figure 5.34: Energy loss

Example 5.5

A scheme has a gross head of 85 m, a discharge of 3 m^3/s, and a 173 m long penstock in welded steel. Calculate the diameter so the power losses due to friction do not surpass 4%.

According to equation (5.18):

$$D = 2.69\left(\frac{3^2 x 0.012^2 x 173}{85}\right)^{0,1875} = 0.88\, m$$

We select a 1m steel welded pipe and compute all the losses in the next example

Example 5.6

Compute the friction and turbulence head losses in a scheme as that illustrated in Figure 5.35. The rated discharge is 3 m^3/s and the gross head 85 m. The steel welded penstock diameter 1.0 m. The radius of curvature of the bends is four times the diameter. At the entrance of the power intake there is a trashrack with a total surface of 6 m^2, inclined 60^0 to the horizontal. The bars are 12-mm thick stainless steel bars, and the distance between bars is 70 mm.

The flow velocity approaching the screen is: (with K1=1)

$$V_0 = 3x\frac{70+12}{70}x\frac{1}{6}x\frac{1}{0.866} = 0.7\, m/s$$

The head loss through the trashrack is given by the Kilchner formula:

$$h_f = 2.4x\left(\frac{12}{70}\right)^{4/3} x\frac{0,7^2}{2x9.81}x0.866 = 0.0049\,m$$

The head loss at the inlet of the penstock is given in Figure 2.11, Chapter 2: K=0.08. The velocity in the penstock is 3.82 m/s, so the head loss at the inlet:

he= 0.08 x 3.822/(2 x 9.81) = 0.06 m

The gross head at the beginning of the penstock is therefore

85-0.005-0.06=84.935 m

The friction loss in the penstock, according Manning equation (2.15) is:

$$h_f = \frac{10.3x0.012^2 x3^2}{1.0^{5.333}}x173 = 2.30\,m$$

The Kb coefficient for the first bend is 0.05. The coefficient for the second bend Kb=0.085 and for the third bend Kb=0.12. The head losses in the three bends amount to:

(0.05 + 0.085 + 0.12) x 3.822/(2 x 9.81) = 0.19 m.

The head loss in the gate valve 0.15 x 3.822/(2 x 9.81) = 0.11 m

Summarising: head loss in trashrack plus pipe inlet: 0.065

head loss in three bends and valve : 0.30 m

head loss by friction in the penstock: 2.30 m

Total head loss: 2.665 m equivalent to 3.14% of the gross power.

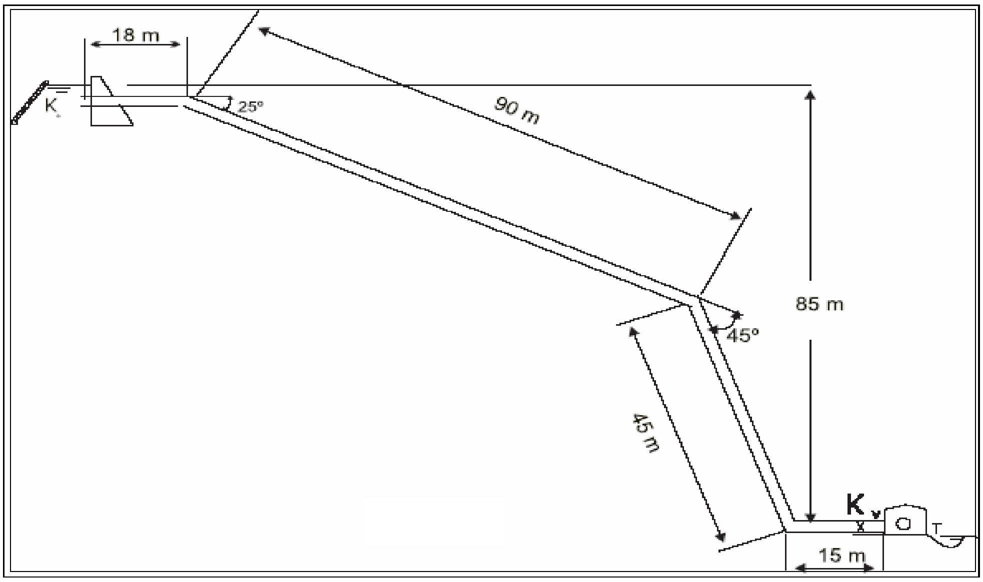


Figure 5.35: Friction and turbulence head losses

Wall thickness

The wall thickness required depends on the pipe material, its ultimate tensile strength (and yield), the pipe diameter and the operating pressure. In steady flows (discharge is assumed to remain constant with time) the operating pressure at any point along a penstock is equivalent to the head of water above that point. The wall thickness in this case is computed by the equation:

$$e = \frac{P_1 \cdot D}{2\sigma_f} \tag{5.19}$$

where

e = Wall thickness in mm

P_1= Hydrostatic pressure in kN/mm^2

D = Internal pipe diameter in mm

σ_f = Allowable tensile strength in kN/mm^2

In steady pipes the above equation is modified by:

$$e = \frac{P_1 \cdot D}{2\sigma_f \cdot k_f} + e_s$$

where

e_s= extra thickness to allow for corrosion

k_f= weld efficiency

k_f = 1 for seamless pipes

k_f = 0.9 for x-ray inspected welds

k_f = 1.0 for x-ray inspected welds and stress relieved

σ_f = allowable tensile stress (1400 kN/mm^2)

The pipe should be rigid enough to be handled without danger of deformation in the field. ASME recommends a minimum thickness in mm equivalent to 2.5 times the diameter in metres plus 1.2 mm. Other organisations recommend as minimum thickness t_{min}=(D+508)/400, where all dimensions are in mm.

In high head schemes it can be convenient to use penstock of uniform diameter, but with different thickness as a function of the hydrostatic pressures.

A certain area of the penstock can remain under the Energy Gradient Line and collapse by sub-atmospheric pressure. The collapsing depression will be given by:

$$Pc = 882500x\left(\frac{e}{D}\right)^3 \tag{5.20}$$

where e and D are respectively the wall thickness and diameter of the pipe in mm.

This negative pressure can be avoided by installing an aeration pipe with a diameter in cm given by:

$$d = 7.47\sqrt{\frac{Q}{\sqrt{P_c}}} \tag{5.21}$$

provided $P_c \leq 0.49$ kgN/mm^2 ; otherwise d=8.94$\sqrt{Q}$.

Sudden changes of flow can occur when the plant operator or the governing system opens or closes the gates rapidly. Occasionally the flow may even be stopped suddenly due to full load rejection, or simply because an obstruction becomes lodged in the nozzle of a Pelton turbine jet. A sudden change of flow rate in a penstock may involve a great mass of water moving inside the penstock. The pressure wave which occurs with a sudden change in the water's velocity is known as water hammer; and although transitory, can cause dangerously high and low pressures whose effects can be dramatic: the penstock can burst from overpressure or collapse if the pressures are reduced below ambient. The surge pressures induced by the water hammer phenomenon can be of a magnitude several times greater than the static pressure due to the head, and must be considered in calculating the wall thickness of the penstock.

Detailed information on the water hammer phenomenon can be found in texts on hydraulics, and information is given in Chapter 2, section 2.2.3. Some examples will show the application of the recommended formulae.

As explained in Chapter 2, the pressure wave speed c (m/s) depends on the elasticity of the water and pipe material according to the formula:

$$c = \sqrt{\frac{10^{-3} k}{\left(1 + \frac{kD}{Et}\right)\rho}} \qquad (5.22)$$

where

k = bulk modulus of water 2.1x109 N/m^2

E = modulus of elasticity of pipe material (N/m^2)

t = wall thickness (mm)

The time taken for the pressure wave to reach the valve on its return, after sudden closure is known as the critical time:

T= 2L/c (5.23)

For instantaneous closure (the pressure wave reaches the valve after its closure) the increase in pressure, in metres of water column, due to the pressure wave is:

$$P = c\frac{\Delta_v}{g} \qquad (5.24)$$

where Δ_v is the velocity change.

Examples 6.4 and 6.5 shows that surge pressures in steel pipes are more than three times greater than in PVC, due to the greater stiffness of the steel.

Example 5.7

Calculate the pressure wave velocity, for instant closure, in a steel penstock 400mm diameter and 4mm-wall thickness.

Applying the above equations gives:

$$c = \sqrt{\frac{2.1\times10^6}{1+\frac{2.1\times10^9\times400}{2.1\times10^{11}\times4}}} = 1024\,m/s$$

b) The same for a PVC pipe 400 mm diameter and 14 mm wall thickness.

$$c = \sqrt{\frac{2.1\times10^6}{1+\frac{2.1\times10^9\times400}{2.75\times10^9\times14}}} = 305\,m/s$$

Example 5.8

What is the surge pressure, in the case of instant valve closure, in the two penstocks of example 5.7, if the initial flow velocity is 1.6 m/s?

a) steel penstock:

$$P_s = \frac{1024x4}{9.8} = 417\,m$$

b) PVC penstock:

$$P_s = \frac{30^5 x4}{9.8} = 123\,m$$

As the example 5.8 shows, the surge pressure in the steel pipe is three times higher than in the PVC pipe, due to the greater rigidity of the steel.

If the change in velocity occurs in more than ten times the critical time T, little or no overpressure will be generated and the phenomenon may be ignored. In between, if T>2L/c, P_s will not develop fully, because the reflected negative wave arriving at the valve will compensate for the pressure rise. In these cases the Allievi formula may compute the maximum overpressure:

$$\Delta P = P_o\left(\frac{N}{2} \pm \sqrt{\frac{N^2}{4} + N}\right) \qquad (5.25)$$

where P_0 is the hydrostatic pressure due to the head and:

$$N = \left(\frac{LV_0}{gP_0t}\right)^2 \qquad (5.26)$$

where:

V_0 = water velocity in m/s

L = total penstock length (m)

P_0 = gross hydrostatic pressure (m)

t = closing time (s)

The total pressure experienced by the penstock is $P = P_0 + \Delta P$

The next example illustrates the application of the Allievi formula, when the closure time is at least twice but less than 10 times the critical time.

Example 5.9

Calculate the wall thickness in the penstock analysed in example 5.6 if the valve closure time is 3 seconds.

Summarising the data,

Gross head: 84.935 m

Rated discharge: 3 m^3/s

Internal pipe diameter 1.0 m

Total pipe length: 173 m

Estimating in a first approach at 5 mm wall thickness to compute the wave speed c:

$$c = \sqrt{\frac{2.1\times10^6}{1+\frac{2.1\times10^9 x1000}{2.1\times10^{11} x5}}} = 836.7\, m/s$$

The closure time is bigger than the critical one (0.41 s) but smaller than 10 times its value, so the Allievi formula can be applied.

The water velocity in the pipe is:

$$V = \frac{4x3}{\pi x1.0^2} = 3.82\, m/s$$

N would be computed for a gross head in the pipe of 84.935 m

$$N = \left(\frac{3.82x173}{9.81x84.935x3}\right)^2 = 0.070$$

and therefore

$$\Delta_P = 84.935\left(\frac{0.07}{2} \pm \sqrt{0.07+\frac{0.07^2}{4}}\right) = +25.65\ m; -19.58\ m$$

The total pressure would be 84.935+25.65 = 110.585 tf/m2 = 11.06 kN/mm^2.

It requires a wall thickness:

$$e = \frac{11.06x1000}{2x1400} + 1 = 4.95\, mm$$

That agrees with the initial estimation and covers the specification for handling the pipes in the field (tmin=2.5x1+1.2=3.7 mm)

To compute the air vent pipe diameter:

$$P_c = 882500\left(\frac{5}{1000}\right)^3 = 0.11\,kN/mm^2$$

And the diameter:

$$d = 7.47\sqrt{\frac{3}{\sqrt{0.11}}} = 22.46\ cm$$

The waterhammer problem becomes acute in long pipes, when the open channel is substituted by a pressure pipe all along the race. For a rigorous approach it is necessary to take into consideration not only the elasticity of fluid and pipe material, as above, but also the hydraulic losses and the closure time of the valve. The mathematical approach is cumbersome and requires the use of a computer program. For interested readers, Chaudry 19, Rich 20, and Streeter and Wylie 21 give some calculation methods together with a certain number of worked examples.

To determine the minimum pipe thickness required at any point along the penstock two water hammer hypotheses should be taken into consideration: normal water hammer and emergency water hammer. Normal water hammer occurs when the turbine shuts down under governor control. Under these conditions, the overpressure in the penstock can reach 25% of the gross head, in the case of Pelton turbines, and from 25% to 50% in the case of reaction turbines (depending on the governor time constants). The turbine manufacturer's advice should be taken into consideration. Emergency water hammer, caused for example by an obstruction in the needle valve of a Pelton turbine, or a malfunction of the turbine control system, must be calculated according to the aforementioned equation.

In steel penstocks, the compounded stresses (static plus transitory) are a function both of the ultimate tensile and yield strength. In the case of normal water hammer, the combined stress should be under 60% of the yield strength and 38% of the ultimate tensile strength. In the case of emergency water hammer, the combined stresses should be under 96% of the yield strength and 61% of the ultimate tensile strength.

Commercial pipes are often rated according to the maximum working pressure under which they are designed to operate. The pressure rating of a pipe already includes a safety factor, and sometimes may include an allowance for surge pressures. Safety factors and surge pressure allowances depend on the standards being used.

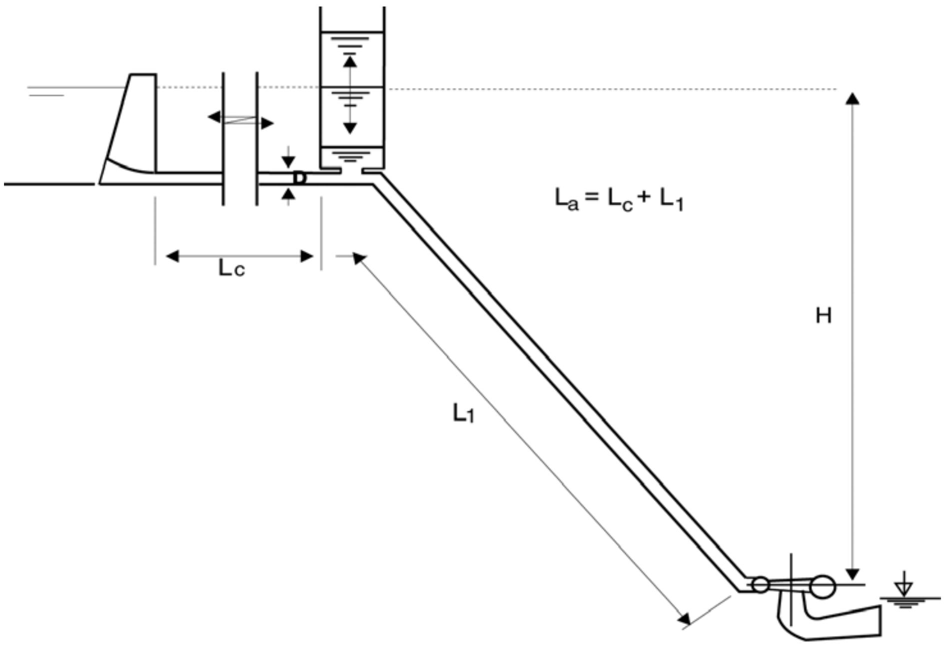


Figure 5.36: Surge tower

If the scheme is liable to surge pressure waves a device to reduce its effects must be considered. The simplest device is the surge tower, a sort of large tube, connected at its base to the penstock and open to the atmosphere. The fundamental action of a surge tower is to reduce the length of the column of water by placing a free water surface closer to the turbine (Figure 5.36). Some authors consider that the surge tower is unnecessary if the pipe length is inferior to 5 times the gross head. It is also convenient to take into account the water acceleration constant t_h in the pipe:

$$t_h = \frac{V \cdot L}{gH} \qquad (5.27)$$

where

L = length of penstock (m),

V = flow velocity (m/s) and

H = net head (m).

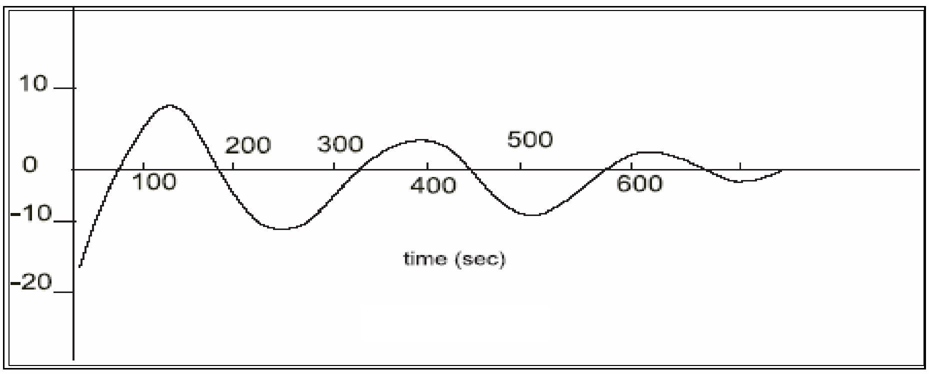


Figure 5.37: **Surge height versus time**

Photo 5.19: Water jet

If t_h is inferior to 3 seconds the surge tower is unnecessary but if it surpasses 6 seconds, either a surge tower or another correcting device must be installed to avoid strong oscillations in the turbine controller.

With the valve open and a steady flow in the penstock, the level of the water in the tower will correspond to the pressure in the penstock - equivalent to the net head. When by a sudden closure of the valve the pressure in the penstock rises abruptly, the water in the penstock tends to flow into the tower, raising the level of the water above the level in the intake. The level in the tower then begins to fall as the water flows from the tower into the penstock, until a minimum level is reached. The flow then reverses and the level in the tower rise again and so on. Figure 5.37 shows a graph plotting the surge height versus time. The maximum height corresponds to the overpressure in the penstock due to the waterhammer. The throttling introduced by a restricted orifice will reduce the surge amplitude by 20 to 30 per cent. The time t_h plays an important role in the design of the turbine regulation system. In a badly designed system, the governor and the tower surge can interact, generating speed regulation problems too severe for the governor to cope with.

In instances, when the closure time of the turbine valves must be rapid, a relief valve placed in parallel with the turbine, such that it opens as the turbine wicket gates close, can be convenient. This has the effect of slowing down the flow changes in the penstock[5]. Photo 5.19 shows the water jet ejecting from the open valve.

Saddles, supporting blocks and expansion joints

The saddles are designed to support the weight of the penstock full of water, but not to resist significant longitudinal forces. The vertical component of the weight to be supported, in kN, has a value of:

$$F1=(Wp+Ww)\cdot L\cdot \cos\Phi \qquad (5.28)$$

where

Wp = weight of pipe per metre (kN/m)

Ww = weight of water per metre of pipe (kN/m)

L = length of pipe between mid points of each span (m)

Φ = angle of pipe with horizontal

The design of support rings is based on the elastic theory of thin cylindrical shells. The pipe shell is subject to beam and hoop stresses, and the loads are transmitted to the support ring by shear. If penstocks are continuously supported at a number of points, the bending moment at any point of penstock may be calculated assuming that it is a continuous beam, and using the corresponding equation. The rings are welded to the pipe shell with two full length fillet welds and are tied together with diaphragm plates

The span between supports L is determined by the value of the maximum permissible deflection L/65000. Therefore the maximum length between supports is given by the equation:

$$L = 182.61 \cdot \sqrt[3]{\frac{(D + 0.0147)^4 - D^4}{P}} \qquad (5.29)$$

where D = internal diameter (m) and P = unit weight of the pipe full of water (kg/m).

5.10 Tailraces

After passing through the turbine the water returns to the river trough a short canal called a tailrace. Impulse turbines can have relatively high exit velocities, so the tailrace should be designed to ensure that the powerhouse would not be undermined. Protection with rock riprap or concrete aprons should be provided between the powerhouse and the stream. The design should also ensure that during relatively high flows the water in the tailrace does not rise so far that it interferes with the turbine runner. With a reaction turbine the level of the water in the tailrace influences the operation of the turbine and more specifically the onset of cavitation. This level also determines the available net head and in low head systems may have a decisive influence on the economic results.

BIBLIOGRAPHY

1. http://www.obermeyhydro.com

2. H.C. Huang and C.E. Hita, “Hydraulic Engineering Systems”, Prentice Hall Inc., Englewood Cliffs, New Jersey 1987.

3. British Hydrodynamic Research Association, “Proceedings of the Symposium on the Design and Operation of Siphon Spillways”, London 1975.

4. Allen R. Inversin, “Micro-Hydropower Sourcebook”, NRECA International Foundation, Washington, D.C.

5. USBR, “Design of Small Canal Structure”, Denver Colorado, 1978a.

6. USBR, “Hydraulic Design of Spillways and Energy Dissipaters”, Washington DC, 1964.

7. T. Moore, “TLC for small hydro: good design means fewer headaches”, HydroReview, April 1988.

8. T.P. Tung y otros, “Evaluation of Alternative Intake Configuration for Small Hydro”, Actas de HIDROENERGIA 93. Munich.

9. ASCE, Committee on Intakes, “Guidelines for the Design of Intakes for Hydroelectric Plants”, 1995.

10. G. Munet y J.M. Compas, “PCH de recuperation d’energie au barrage de “Le Pouzin””, Actas de HIDROENERGIA 93, Munich.

11. G. Schmausser & G. Hartl, “Rubber seals for steel hydraulic gates”, Water Power & Dam Construction September 1998.

12. ISO 161-1-1996 “Thermoplastic pipes for conveyance of fluids – Nominal outside diameters and nominal pressures – Part 1: Metric series.”

13. ISO 3606-1976 “Unplasticized polyvinyl chloride (PVC) pipes. Tolerances on outside diameters and wall thickness.”

14. ISO 3607-1977 “Polyethylene (PE) pipes. Tolerance on outside diameters and wall thickness.”

15. ISO 3609-1977 “Polypropylene (PP) pipes. Tolerances on outside diameters and wall thickness.”

16. ISO 4065-1996 “Thermoplastic pipes – Universal wall thickness table.”

17. H. Chaudry, “Applied Hydraulic Transients”, Van Nostrand Reinhold Company, 1979.

18. J. Parmakian, “Waterhammer Analyses”, Dover Publications, Inc, New York, 1963.

19. Electrobras (Centrais Eléctricas Brasileiras S.A.) “Manual de Minicentrais Hidrelétricas.”

20. M. Bouvard, “Mobile barrages and intakes on sediment transporting rivers” IAHR Monograph, AA Balkema, 1984.

21. Sinniger & Hager, “Constructions Hydrauliques”, PPUR, Lausanne, 1989.

[1] By Erik Bollaert (LCH-EPFL), Jonas Rundqvist (SERO) and Celso Penche (ESHA)

[2] J.L. Brennac. “Les Hauses Hydroplus”, ESHA Info n° 9 Estate 1993

[3] USBR “Design of Small Dams” - 3rd ed., Denver, Colorado, 1987.

[4] One of these, the SSSS (Serpent Sediment Sluicing System) has been described in detail in the issue 9 -spring/summer 1993- of ESHA Info

[5] In the ESHA NEWS issue of spring 1991 there is a description of such a valve.