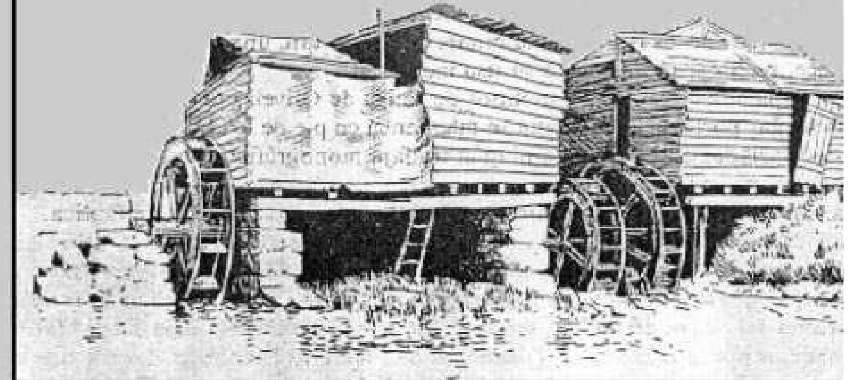
LAYMAN'S GUIDEBOOK

on how to develop a small hydro site



European
Small Hydropower
Association

DIRECTORATE GENERAL FOR ENERGY (DG XVII)

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DIRECTORATE GENERAL FOR ENERGY (DG XVII)

LAYMAN'S HANDBOOK ON HOW TO DEVELOP A SMALL HYDRO SITE

(Second Edition)

June 1998

EUROPEAN COMMISION

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This handbook, an updated version of the original “Layman’s Handbook on how to develop a Small Hydro Site”, published by the Commission in 1993, has been written, in the frame of the ALTENER programme, under contract with the Commission of the European Communities (Directorate General for Energy, DG XVII). It has not been designed to replace professional expertise but it is hoped it is comprehensive enough to advise laymen on all necessary procedures that should be followed to develop a site. However its content includes enough technical information, so a non-specialist engineer would be able to produce a primary feasibility report.

Hydraulic engineering is based on the principles of fluid mechanics. However until now there does not exist, and probably never will, a general methodology for the mathematical analysis of the movement of the fluids. Based on the large amount of accumulated experience there exists many empirical relationships to achieve practical engineering solutions with the movement of the water, the fluid that concerns hydroelectricity. Chapter 2, based on part of the original chapter 5 – written by Eric Wilson - is devoted to this subject.

All hydroelectric generation depends on falling water. The first step to develop a site must address the availability of an adequate water supply. Chapter 3 is entirely devoted to this subject, and particularly to comment on the **European Atlas of Small Scale Hydropower Potential,** developed by the Institute of Hydrology in the UK, on behalf of ESHA and with the financial aid of the DG XVII.

Experience shows that many small hydro plants have failed because they were poorly designed, built or operated. Most of these failures – seepage under the weir, open channel slides – occurred through a lack of proper geological studies of the site. Chapter 4 incorporates guidelines on such studies.

Hydraulic structures and ancillaries represent almost fifty per cent of the investment cost. If poorly designed they will require such high maintenance costs that the investment will become unprofitable. Chapter 5 is devoted to these structures.

Turbines transform the potential energy of water to mechanical rotational energy, which in turn is transformed into electrical energy in the generators. Chapter 6 is devoted to the study of turbines and generators and to the devices employed to control them.

Although since the publication of the first edition of the Layman’s Handbook many sites have been developed in the E.U, the installed capacity would be greater if the administrative procedures to authorise the use of water had been simpler. Many hundreds of authorisation requests are pending approval, mainly because of supposed conflict with the environment. Chapter 7, “Environmental impact and its mitigation”, intends to provide a few guidelines to help the designer to propose mitigating measures that can be easily agreed with the licensing authorities. The various papers presented to HIDROENERGIA and more specifically to the European Workshop on THERMIE “Strategies to overcome the environmental burden of small hydro and wind energies” that was held at Vitoria in October 1996, constitute the basis of this chapter.

An investor decides to develop a small hydro site in order to obtain a reasonable profit. To do that his decision should be based on sound economic principles. Chapter 8 shows how the financial mathematics can help to calculate the cost of the kWh produced annually, and to compare different possible alternatives for the scheme.

Chapter 9 reviews the administrative procedures and buy-back tariffs nowadays in force. Unfortunately the trend toward deregulation of the electricity market makes the situation very volatile, preventing accurate reporting of the market from an institutional viewpoint.

Acknowledgements

Although based on the original version, the handbook has been entirely rewritten. The original chapter 5 has been split in two: chapter 2, a fundamental treatment of engineering hydraulics, and chapter 3 devoted exclusively to the water resource and to the possibilities offered by the **European Atlas of Small Scale Hydropower Potential.** The Institute of Hydrology (IH) in the UK, on behalf of ESHA, has developed this computer program, with the financial aid of the DG XVII, as a tool to enable potential investors to define the hydrological potential, for any ungauged site within the European Union. We acknowledge the co-operation of IH, and more specifically of Gwyn Rees and Karen Kroker, by allowing us to reproduce entire paragraphs of the "Technical Reference and User Guide" of the Atlas.

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Two well known experts, Bryan Leyland from Australia and Freddy Isambert from France, presented to HIDROENERGIA 95 two papers, dealing with the topic "lessons from failures", describing several schemes that, due to a lack of adequate geological studies, failed outrageously during its operation. On the base of these experiences a new chapter, Chapter 4, devoted to the technologies employed to study the site in depth, was introduced. This chapter has been almost entirely written by Alberto Foyo, Professor of Ground Engineering at the E. T. S. I. C. C. P, Polytechnic Cantabria University.

Other sources of inspiration in the composition of the handbook were "Micro Hydropower Source" by R. Inversin (NRCA 1986), the volume 4 of the "Engineering Guidelines for Planning and Designing Hydroelectric Developments" (ASCE 1990) and "Hydraulic Engineering Systems" (N.C.Hwang and C.E. Hita 1987). The authorisation by Inversin to reproduce the Appendix X of his book, dealing with the physical description of the waterhammer phenomena, is much appreciated. We appreciate the spirit of collaboration of the authors of hydraulic papers; all of them gave their authorisation to reproduce their papers-

We should thank Eric Wilson for his efforts to correct the English text, both for style and content. If any errors are still present it will be unquestionably the fault of the author.

And finally our acknowledgement to President Henri Baguenier, who solicited the support of the DG XVII to commission the writing of the handbook and to facilitate the relationship with the ALTENER Committee.

Celso Penche
June 1988.

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1. Introduction

1.0 A free fuel resource potentially everlasting.

Following the United Nations Conference in Rio on the Environment and Development, the European Union committed itself to stabilising its carbon dioxide (CO_2) emissions, primarily responsible for the greenhouse effect, at 1990 levels by the year 2000.Clearly Europe will not be able to achieve this ambitious target without considerable promotion of energy efficiency and a major increase in the development of renewable energy sources. The European Commission is well aware of this fact and one of the ALTENER objectives is to double, from now to the year 2010, the electricity generated by renewable resources.

From the beginning of electricity production 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 CO_2 emissions by more than 67 million tons a year. But whereas the conventional hydro requires the flooding of large areas of land, with consequent serious environmental and social costs, the properly designed small hydro schemes (less than 10 MW installed capacity) are easily integrated into local ecosystems.

Small hydro is the largest contributor of electricity from renewable energy sources, both at European and world level. At world level, it is estimated there is an installed capacity of 47.000 MW, with a potential –technical and economical – close to 180.000 MW. At European level, the installed capacity is about 9.500 MW, and the EC objective for the year 2010 is to reach 14.000 MW..

The large majority of small hydro plants are «run-of-river» schemes, meaning simply that the turbine generates when the water is available and provided by the river. When the river dries up and the flow falls below some predetermined amount, the generation ceases. This means, of course, that small independent schemes may not always be able to supply energy, unless they are so sized that there is always enough water.

This problem can be overcome in two ways. The first is by using any existing lakes or reservoir storage upstream. The second is by interconnecting the plant with the electricity supplier's network. This has the advantage of allowing automatic control and governing of the frequency of the electricity but the disadvantage of having to sell the energy to the utility company at its price -the 'buy-back' rate-, which can be too low. In recent years, in most of the member states, the rate has been fixed by national governments, who, conscious of the environmental benefits of renewables, have been making provision for increasing the "buy-back" rates. Portugal, Spain and Germany have proved that reasonable "buy-back" rates are essential to increase the generation of electricity with renewables.

With the announced deregulation of the European electricity market, the small producers will be in a weak position to negotiate the purchase of their electricity by the utilities. But national governments cannot dispense with renewables in their effort to curb CO_2 emissions, and must found ways, perhaps similar to the British NFFO to support generation by renewables.

1.1 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); in France the limit was established at 8 MW and UK favour 5 MW. Hereunder will be considered as small any scheme with an installed capacity of 10 MW or less. This figure is adopted by five member states, ESHA, the European Commission and UNIPEDE (International Union of Producers and Distributors of Electricity).

1.2 Site configurations

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

According to the head, schemes can be classified in three categories:
- High head: 100-m and above
- Medium head: 30 - 100 m
- Low head: 2 - 30 m

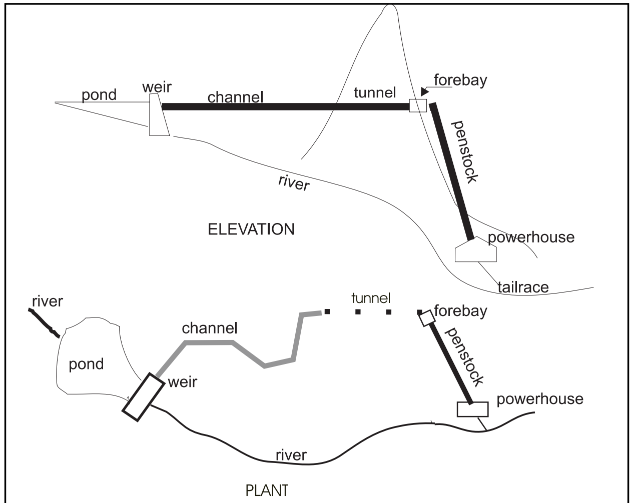


figure 1.1 High head scheme

These ranges are are not rigid but are merely means of categorising 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 an canal or in a water supply pipe

1.2.1 Run-of-river schemes

In the «run-of-river» schemes 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 – the minimum technical flow of the turbine equipping the plant -, generation ceases.

Medium and high head schemes use weirs to divert water to the intake, from where it is 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, with larger latitude in slopes, can be an economical option. At the outlet of the turbines, the water is discharged to the river, via the tailrace.

Occasionally a small reservoir, storing enough water to operate only on peak hours, when "buy-back" rates are higher, can be created by the weir, or a similarly sized pond can be built in the forebay, using the possibilities provided by geotextiles.

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

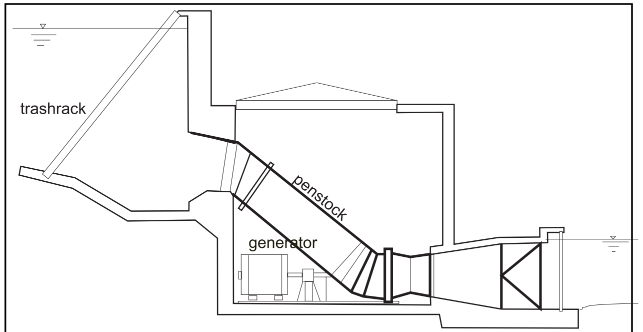


figure 1.2

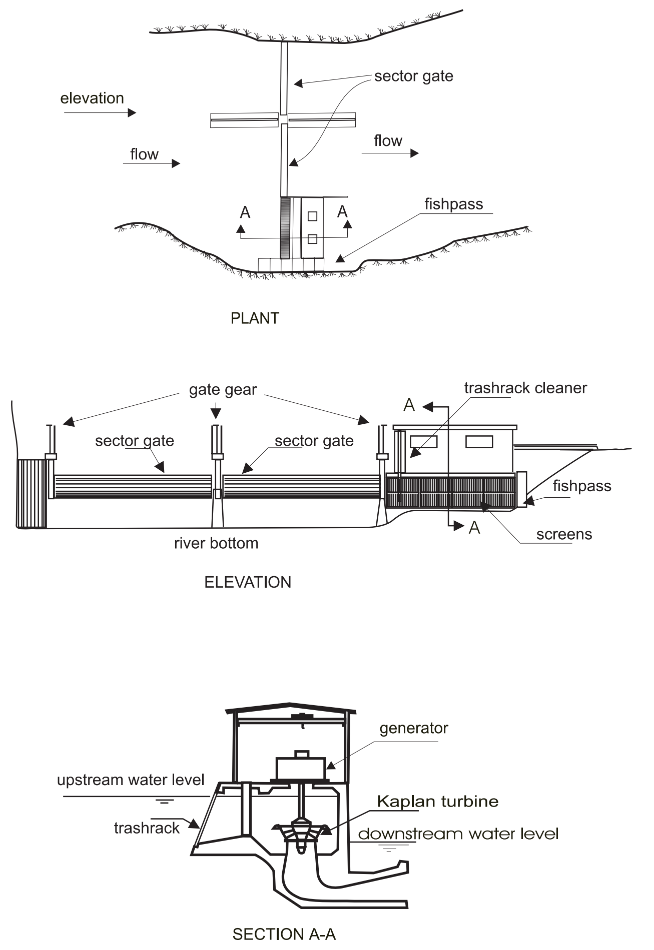


figure 1.3

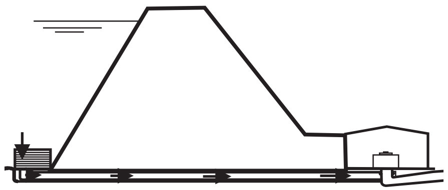


figure 1.4

fish ladder.

1.2.2 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 – flood control, irrigation network, water abstraction for a big city, recreation area, etc, - it may be possible to generate electricity using the discharge compatible with its fundamental usage or the ecological flow of the reservoir.

The main question is how to link headwater and tailwater by a waterway and how to fit the turbine in this waterway. If the dam already has a bottom outlet, as in figure 1.4, the solution is clear. Otherwise, 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 with heads up to 10 meters and for units of no more than 1.000 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

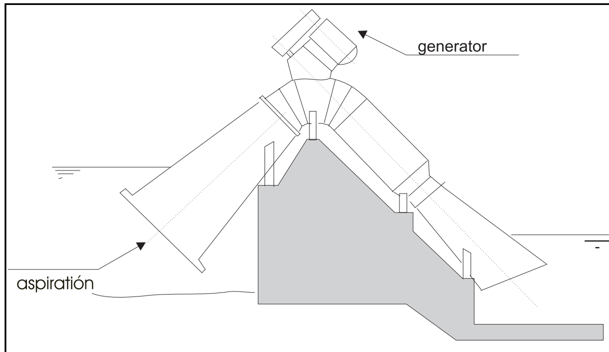


figure 1.5
siphon intake

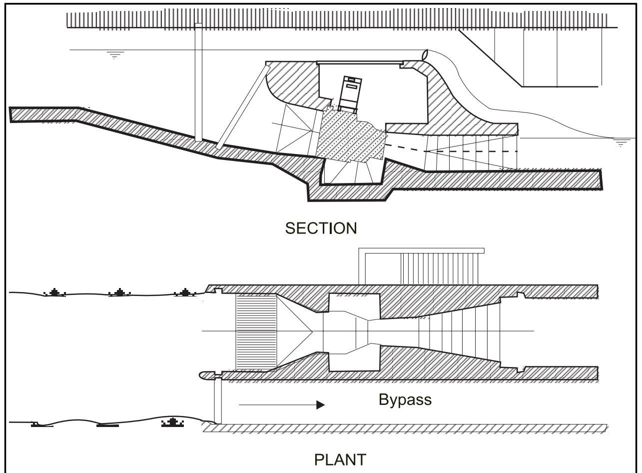


figure 1.6

top of the dam or on the downstream side. The unit can be delivered pre-packaged to the works, and installed without major modifications of the dam.

1.2.3 Schemes integrated with an irrigation canal

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

- The canal is enlarged to the required extent, to accommodate the intake, the power station, the tailrace 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 ensure 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, because the widening of the canal in full operation is an expensive option.
- 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

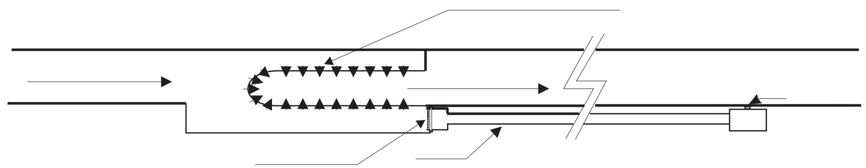


figure 1.7

spillway. To reduce the with 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, once through the turbine, is returned to the river via a short tailrace. As generally, fish are not present in canals, fishpasses are unnecessary.

1.2.4 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 waterhammer, that would endanger the pipe, is avoided.

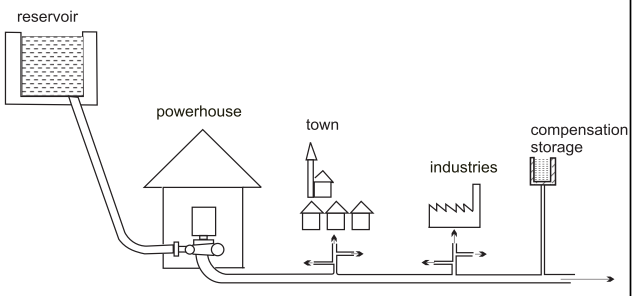


figure 1.8

Waterhammer 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 pound. The control system maintains automatically, and unattended, the level of the pound. In case mechanical shutdown or load rejection closes the turbine, the valve of the main bypass can also maintain the level of the pound automatically. Occasionally if the main bypass valve is out-of-operation and overpressure occurs, an ancillary bypass valve is rapidly opened by a counterweight and is subsequently closed. All the closing and opening operations of these valves however 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 subjecto the counter-pressure of the network, as is shown in figure 1.8.

1.3 Planning a small hydropower scheme

The definitive project of a scheme comes as the result of a complex and iterative process, where, always having in view the environmental impact, the different technological options are compared from an economic viewpoint.

Although 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 authorisations

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 the 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. The gross head can usually be considered as constant, but the flow varies over the year. To select the most appropriate hydraulic equipment, estimate its potential and calculate 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 straightforward 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 topographic work much simpler and faster. To produce a flow-duration curve on a gauged site has no mystery; to produce such a curve at an ungauged site requires a deeper knowledge of the hydrology. In Chapter 3 various methods for measuring the quantity of water flowing in a stream are analysed and hydrologic models to calculate the flow regime at ungauged sites are discussed. . Fortunately, new computer package programs will ease that task and in Chapter 3 one of these programs (HydrA) is presented.

Chapter 4 presents the techniques – orto-photography, RES, GIS, geomorphology, geotectonics, etc – used nowadays for site evaluation, preventing potential future failures. Some of these failures are analysed and conclusions about how they might have been avoided are explained.

In Chapter 5 the basic layouts are developed 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 nowadays unattended, the control systems, based on personal computers, are reviewed.

Environmental Impact Assessment is required to attain authorisation to use the water. Although several recent studies have shown that small hydropower having no emissions nor producing 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, which can be applied in the economical evaluation of a scheme. Various methodologies of economic analyses are described and illustrated with tables showing the cash flows generated by the schemes.

Institutional frameworks and administrative procedures in various UE member-states are reviewed. Unfortunately the recent electricity industry's deregulation make itimpossible to detail a situation that was fairly clear few years ago, when ESHA produced in December 1994 and under contract with the E.C., Directorate General for Energy, DGXVII, the report " Small Hydropower. General Framework for Legislation and Authorisation Procedures in the European Union"

2. Fundamentals of Hydraulic Engineering

2.0 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 large amount of accumulated experience, certainly there are particular solutions to specific problems. Experience that goes back as far as 2500 years ago, when a massive irrigation system, that is still operative, was built in Siechuan, China, and to the Roman Empire's builders of the aqueducts.

2.1 Water flow in pipes

The energy 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, h_1 is the elevation head, 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 at point 1 is then the algebraic sum of the potential energy h_1, the pressure energy P_1/γ , and the kinetic energy $V_1^2/2g$.

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 appeared as a straight line all along the pipe, indicating laminar flow. The water flows in laminae, like concentric 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 paraboloid of revolution and the average velocity (figure 2.1) is 50% of the maximum centre line velocity.

If the flow rate is gradually increased, a moment is reached when the thread of colour 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. Osborne Reynolds, near the end of last century, performing this carefully prepared experiment found that the transition from laminar flow to turbulent flow depends, not only on the velocity, but also on the pipe diameter and the viscosity of the fluid, and can be described by the ratio of the inertia force to the viscous force, This ratio, known nowadays as the Reynolds number, can be expressed, in the case of a circular pipe, by the equation:

$$N_R = \frac{D\,V}{\nu} \qquad (2.2)$$

where D (m) is the pipe diameter, V is the average water velocity (m/s), and ν is the kinematic viscosity of the fluid (m^2/s). .

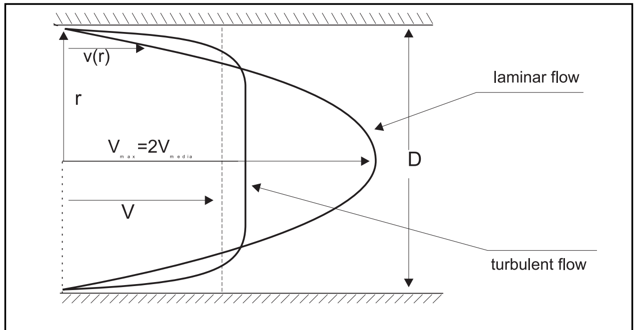


figure 2.1

Experimentally 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 N_R=2000 but varies with the experimental 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 20ºC. Calculate the largest flow-rate for which laminar flow can be expected
The kinematic viscosity of water at 20ºC is υ = 1 x 10^{-6} m^2/s.
Accepting a conservative value for N_R = 2000
V=2000 / (10^6x0,06) = 0.033 m/s
Q = AV = /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 nearby 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 turbulence there is a more intensive particle mixing action, and hence a higher viscous dissipation. Consequently the energy losses in pipe flow 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 energy h_f is lost

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

due mainly to the friction of the water against the pipe wall, and secondarily to the

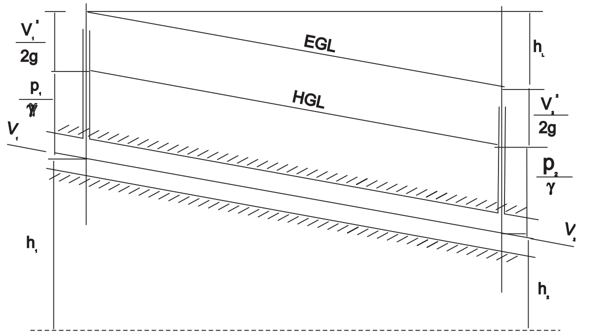


figure 2.2

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. The question is, how h_f can be evaluated?

2.1.1 Los of head due to friction

Darcy and Weisbach, applying the principle of conservation of mass to a control volume – 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, travelling through pipes:

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

where f, friction factor, is a dimensionless number, L the length of 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

$$f = \frac{64\mu}{\rho VD} = \frac{64}{N_R} \tag{2.5}$$

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 N_R 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:

$$h_f = \frac{64\mu}{\rho VD}\times\frac{L}{D}\times\frac{V^2}{2g} = \frac{32\mu LV}{\rho g D^2} \tag{2.6}$$

showing that the specific head loss, in laminar flow, is proportional to V and inversely proportional to D².

When the flow is practically turbulent (NR>>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
Fiberglas 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 sublayer. When N_R increases, the sublayer's thickness diminishes. Whenever the roughness height "e" is resolutely lower than the sublayer 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*

$$\frac{1}{\sqrt{f}} = 2\log\left(\frac{N_R\sqrt{f}}{2.51}\right) \tag{2.7}$$

At high Reynolds numbers, the sublayer thickness becomes very small and the friction factor *f* becomes independent of N_R 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*

$$\frac{1}{\sqrt{f}} = 2\log\left(3.7\frac{D}{e}\right) \tag{2.8}$$

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

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{e/D}{3.7} + \frac{2.51}{N_R\sqrt{f}}\right) \tag{2.9}$$

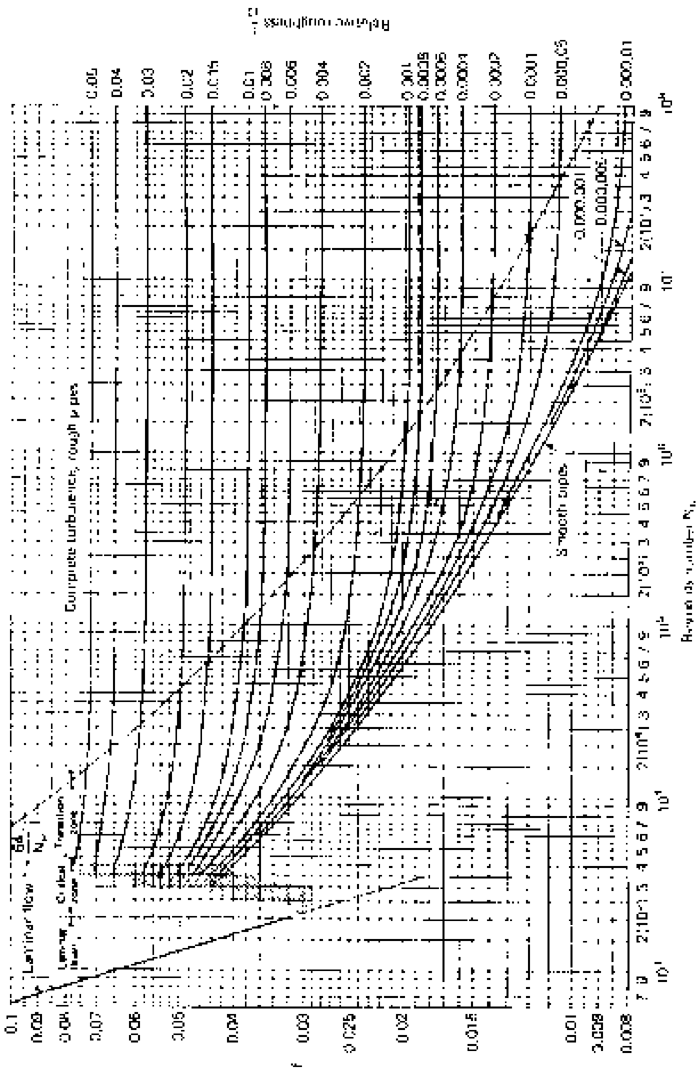


Figure 2.3 *Friction factors for flow, in pipes , the Moody diagram 'Trans. ASME, vol 66, 1944'*

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 2.3)

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 N_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 N_R and e/D (equation 2.9)
4. a complete 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 m³/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

$N_R = DV/\upsilon = (0.9 \times 1.886)/1.31 = 1.3 \times 10^6$ ($\upsilon = 1.31\ 10^{-6}$)

In the Moody chart for e/D = 0.00062 and $N_R = 1.3 \times 10^6$ we found f=0.019

From equation (2.4)

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

Those not fond of nomographs can use an electronic spreadsheet to derive $\alpha = \sqrt{1/f}$ from equation 2.9

$$\alpha = -2\log\left(\frac{e/D}{3.7} + \frac{2.51}{N_R}\alpha\right)$$

As the variable is on both sides of the equation an iterative calculation is needed. We use an Excel97 spreadsheet (figure 2.4) to do it. In figure 2.5 there is a list of the formulae that should be introduced on each cell. Once introduced the formulae

Example 2.4 - Steel pipe

Q	1.2	m³/s	f	alpha	alpha
D	900	mm	0.025	6.32455532	7.43162852
V	1.8863	m/s		7.43162852	7.42203156
L	500	m		7.42203156	7.42211430
Nr	1,300,000			7.42211430	7.42211359
e	0.6			7.42211359	7.42211359
e/D	6.6667E-04			7.42211359	7.42211359
f	0.0182				
un	1.31E-06				
hf	1.8289	m			

figure 2.4

and the data, the sheet should look as in figure 2.4. In this case we guessed a value of 0,025 for f, equivalent to α=6.3245. In the spreadsheet it can be seen how the value of alpha is converging to the final value of α=7.4221136, that automatically gives the final value for f = 0.0182 and a head loss h_f=1.829 m.

In Internet there are two home pages, one corresponding to the PENNSTATE University, Department of Mechanical Engineering, and the other AeMES Department, University of Florida, each having an online computer program to calculate the friction factor f, by introducing the Reynolds number and the Roughness parameter. It is much faster than the two above-mentioned methods

B3	q	G6	-2*LOG($C$9/3.7+2.51/$C$7*F6)
C3	1.2	B7	Nr
D3	m^3/s	C7	1.3E+06
E3	f	F7	+G6
F3	alpha	G7	-2*LOG($C$9/3.7+2.51/$C$7*F7)
G3	alpha	B8	e
B4	D	C8	0.6
C4	900	F8	+G7
D4	mm	G8	-2*LOG($C$9/3.7+2.51/$C$7*F8)
E4	0.025	B9	e/D
F4	+1/SQRT(E4)	C9	+C8/C4
G4	-2*LOG($C$9/3.7+2.51/$C$7*F4)	F9	+G8
B5	V	G9	-2*LOG($C$9/3.7+2.51/$C$7*F9)
C5	+4*C3/(C4/1000)^2/PI	B10	f
D5	m/s	C10	+1/G9^2
F5	+G4	B11	nu
G5	-2*LOG($C$9/3.7+2.51/$C$7*F5)	C11	+1.31*10^-6
B6	L	B12	hf
C6	500	C12	+C10*C6/C4*1000*C5^2/(2*9.81)
D6	m	D12	m
F6	+G5		

figure 2.5

and more precise than the Moody Chart. The Internet addresses are, respectively
http://viminal.me.psu.edu/~cimbala/Courses/ME033/me033.htm
http://grumpy.aero.ufl.edu/gasdynamics/colebrook.htm

Applying both online computer programs to the data of example 2.2 the answer was respectively f=0.01787 and f=0.01823, both complete up to 10 decimals. Observe that the second value is practically identical to the one attained with the spreadsheet.

The formula (2.9) can be used to solve almost any kind of problem with flows in close pipes. For example, if you want to know what is the maximum water velocity flowing in a pipe of diameter D and length L, without surpassing a friction headloss h_f you only need to use an independent variable μ

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

Substituting N_R by its value in (2.2) and f by its value in (2.4) becomes

$$\mu = \frac{gD^3 h_f}{Lv^2} \qquad (2.11)$$

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

$$N_R = -2\sqrt{2\mu}\,\log\left(\frac{e/D}{3.7} + \frac{2.51}{\sqrt{2\mu}}\right) \qquad (2.12)$$

An equation that makes it possible to plot the N_R evolution with μ for different values of e/D, as shown in figure 2.6, a variation of the Moody Chart where N_R can be estimated directly.

Example 2.3

Estimate the flowrate of water at 10ºC that will cause a friction headloss 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 = $4x10^4$, after computing μ .

$$\mu = \frac{9.81 \times 1.5^3 \times 2}{1000 \times 1.31^2 \times 10^{-12}} = 3.86 \times 10^{10}$$

$$N_R = -2\sqrt{2 \times 3.86 \times 10^{10}}\,\log\left(\frac{4 \times 10^{-4}}{3.7} + \frac{2.51}{\sqrt{2 \times 3.86 \times 10^{10}}}\right) = 2.19 \times 10^6$$

$$V = \frac{N_R v}{D} = \frac{2.19 \times 1.31}{1.5} = 1.913\,\mathrm{m/s};\ Q = 3.38\,\mathrm{m^3/s}$$

Also based on the Colebrook-White equation there exists some other nomographs, to compute the friction headloss on a pipe, given a certain flow and a certain pipe

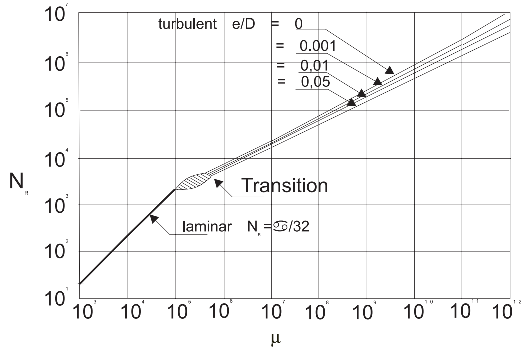


figure 2.6

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..

Empirical formulae

Over the years many empirical formulae, based on accumulated experience, have been developed. They are, in general, not based on sound physical 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
5. 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

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

Where n is the Manning roughness coefficient, P is the wetted perimeter (m), A is cross-sectional area of the pipe (m²) and S is the hydraulic gradient or headloss by linear meter.

Applying the above formulae to a full closed circular cross section pipe:

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

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

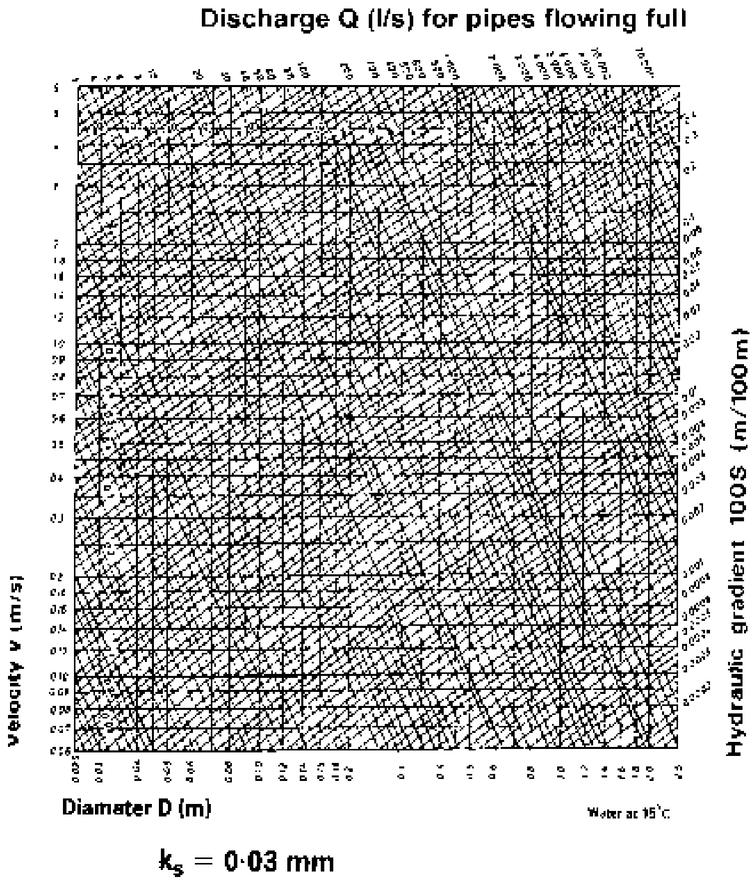


Table 2.2 Manning coefficient n for several commercial pipes

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 attained 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 headloss applying the Manning formulae

Accepting n=0.012 for welded steel pipe

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

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

Example 2.5

Compute, using the Colebrook equation and the Manning formulae, the friction headloss on a welded pipe 500 m long, of respectively 500 mm, 800 mm, 1200 mm, and 1500 mm diameter, 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 doesn't 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 North America for pipes larger than 5 cm diameter and flow velocities under 3 m/s the Hazen-Williams formulae is used:

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

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

2.1.2 Loss of head due to turbulence

Water flowing through a pipe system, with entrances, bends, sudden contraction and enlargements of pipes, racks, valves and other accessories experiences, in addition to the friction loss, a loss due to the inner viscosity. This loss also depends of the velocity and is expressed by an experimental coefficient K multiplying the kinetic energy $v^2/2g$.

2.1.2.1 Trash rack (or screen) losses

A screen or grill is always required at the entrance of a pressure pipe. The flow of water through the rack also gives rise to a head loss. Though usually small, it can be calculated by a formula due to Kirchmer (see figure 2.7)

$$h_t = K_t\left(\frac{t}{b}\right)^{4/3}\left(\frac{V_0^2}{2g}\right)\sin\Phi \qquad (2.16)$$

where the parameters are identified in figure 2.7.

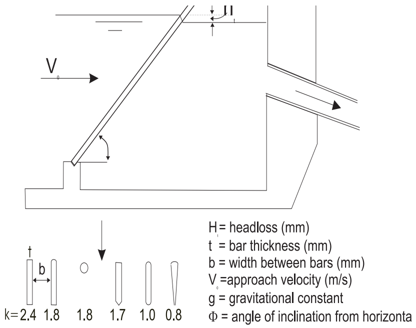


figure 2.7

If the grill is not perpendicular but makes an angle β with the water flow (β will have a maximum value of 90º for a grill located in the sidewall of a canal), there will be an extra head loss, as by the equation

$$h_\beta = \frac{V_0^2}{2g}\sin\beta$$

2.1.2.2 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 turbulence.

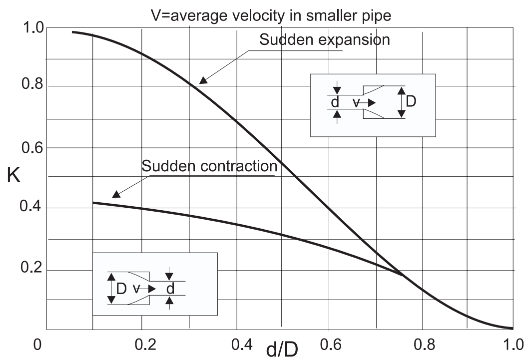


figure 2.8

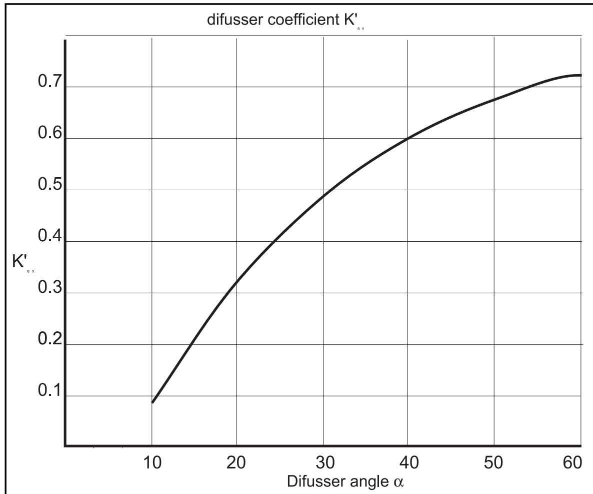


figure 2.9

The flow path is so complex that, at least for the time being, it is impossible to provide a mathematical analysis of the phenomenon. The head loss is estimated multiplying the kinetic energy in the smaller pipe, by a coefficient K_c that varies with the indes of contraction d/D

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

For an index up to d/D = 0.76, K_c approximately follows the formula

$$K_c = 0.42(1 - d^2/D^2) \qquad (2.18)$$

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

In sudden expansion the loss of head can be derived from the momentum consideration, and is given by

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

where V_1 is the water velocity in the smaller pipe. Figure 2.8 is a graphic representation of the K_c and K_{ex} values as a function of d/D.

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

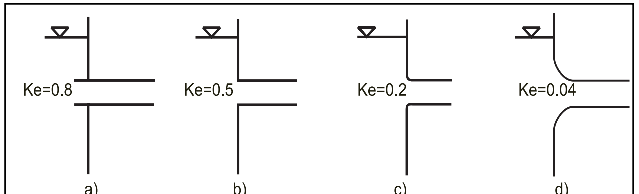


figure 2.10

In the confuser the head loss varies with the confuser angle as it is shown in Table 2.3 where K'_c values are experimental:

Table 2.3 K'_c for different confuser angles

Angle	K'_c
30º	0.02
45º	0.04
60º	0.07

In the diffuser the analysis of the phenomenon is more complex. Figure 2.9 shows the experimentally found values of K_{ex} for different diffuser angles. The head loss is given by:

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

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

An entrance to the pipe is, otherwise, an extreme case of sudden contraction. Figure 2.10 shows the value of the K_e coefficient that multiplies the kinetic energy $V^2/2g$ in the pipe.

2.1.2.3 Loss of head in bends

Pipe flow in a bend, 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 2.11. 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 2.11, 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 90º, is almost proportional to the bend angle.

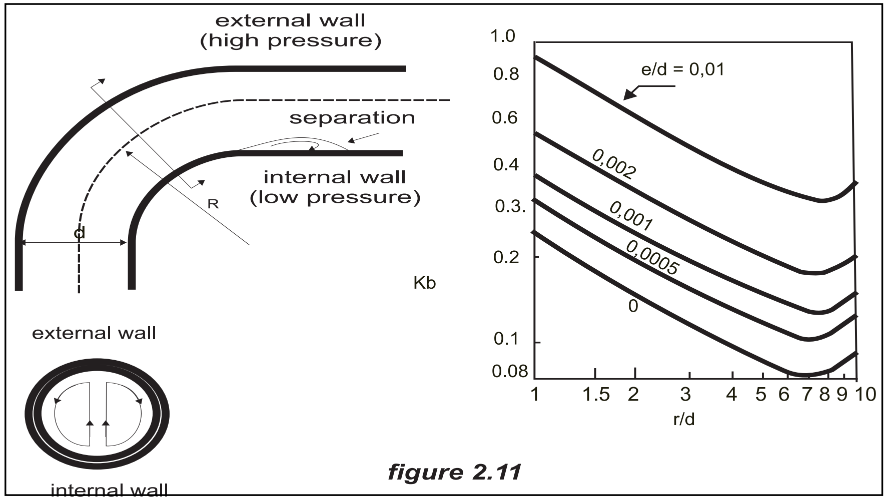


figure 2.11

The problem is extremely complex when successive bends are placed one after another, close enough to prevent the flow from becoming stabilised at the end of the bend. Fortunately this is hardly ever the case on a small hydro scheme.

2.1.2.4 Loss of head through valves

Valves or gates are used in small hydro scheme 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 the water flowing through an open valve depends on the type and manufacture of the valve. Figure 2.12 shows the value of K_v for different kind of valves.

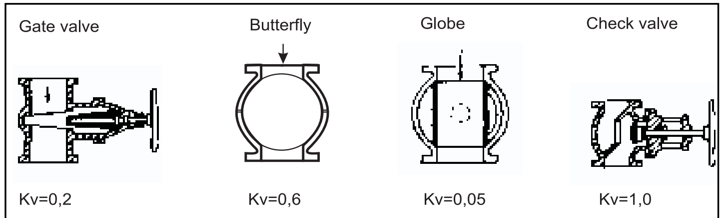


figure 2,12

2.1.3 Transient flow

In steady flows, where 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 occur, 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 *waterhammer* and its effects can be dramatic: the penstock can burst from overpressure or collapse if the pressures are reduced below ambient. Although being transitory the surge pressure induced by the waterhammer 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

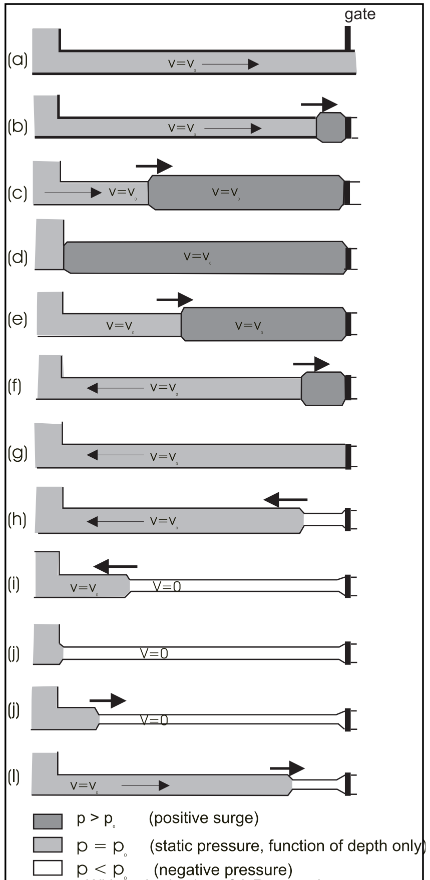
$$F = m\frac{dV}{dt} \qquad (2.21)$$

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; 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, Irvine, from Appendix F of his "Micro-Hydropower Sourcebook", is one of the best physical explanations of the phenomenon. Figure 2,13 illustrates how a velocity change caused by an instantaneous closure of a gate at the end of a pipe creates pressure waves travelling within the pipe.

Initially, water flows at some velocity «V_0» as shown in (a). When the gate is closed, the water flowing within the pipe has a tendency to continue flowing because of its momentum. Because it is physically prevented from so doing, it «piles up» behind the gate; 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 «v_0» is destroyed, the water is compressed, and the pipe is expanded its entire length (d). At this point, the water's kinetic energy has all been converted to strain energy of the water (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_0» (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



With authorisation of A.R. Inversin author of "Micro-Hydropower Sourcebook."

figure 2.13

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, T_C, 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:

$$c = \sqrt{\frac{K \times 10^{-3}}{1 + \frac{KD}{Et}}} \quad (2.23)$$

where K = bulk modulus of water 2.2x10^9 N/m^2
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, (time lower than the critical one T), all the kinetic energy of the water will be converted on an overpressure, and its value in meters of water column, will be

$$P = \frac{c\Delta_V}{g} \quad (2.24)$$

where Δ_V is the change of water velocity.

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 Allievi formula:

$$\Delta_P = P_0\left(\frac{N}{2} + \sqrt{\frac{N^2}{4} + N}\right) \quad (2.25)$$

where P_0 is the gross head and

$$N = \left(\frac{\rho L V_0}{P_0 t}\right)^2 \quad (2.26)$$

where ρ = water density (kg/m^3)
V_0 = water velocity (m/s)
L = total pipe length (m)
P_0 = static pressure (m column of water)
t = closure time (s)

The total pressure experienced by the penstock will be $P = P_0 + \Delta_P$.

In chapter 6, several examples related to penstock design will clarify the above physical concepts.

For a more rigorous approach it would be necessary to take into consideration not only the fluid and pipe material elasticity, as 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 m)ethods, together with some worked examples.

2.2 Water flow in open channels

Contrary to what happen in closed pipes, where 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, because *a priori* the shape of the surface is unknown. 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 2.14 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.

2.2.1 Clasification of open channel flows

Under the **time criterion** a channel flow is considered *steady* when the discharge and the water depth at any section of the stretch does not change with time. and *unsteady* when one or both of them changes with time.

Based on the **space criterion**, an open channel flow is said to be *uniform* if the discharge and the water depth at any section of the stretch do not change with time, and is said to be *varied* when the discharge and the water depth change along its length. The flow could be *varied steady* if the unidimensional approach can be applied and *varied unsteady* if not. Figure 2.15 represents different kind of flows: steady, varied steady (GV), and varied unsteady (RV)

As in the fully closed pipe flows, channel flows also follow the Bernoulli 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.

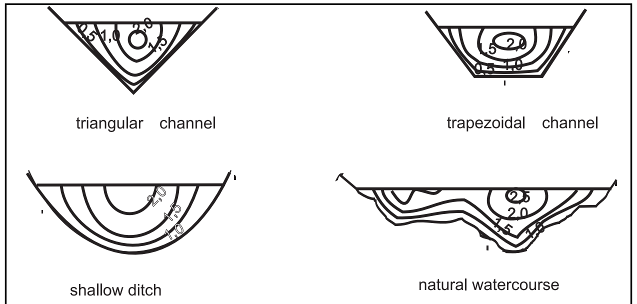


figure 2.14

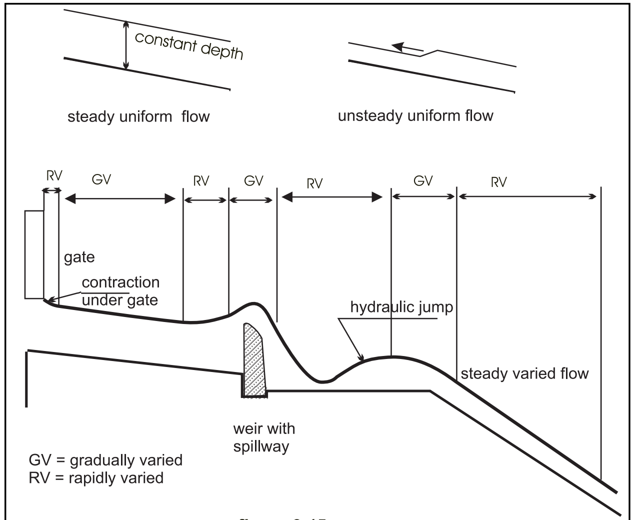


figure 2.15

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

$$V = C\sqrt{R_h S_e} \qquad (2.27)$$

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:

$$C = \frac{1}{n} R_h^{1/6} \qquad (2.28)$$

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

$$V = \frac{1}{n} R_h^{2/3} S_e^{1/2} \qquad (2.29)$$

or alternatively

$$Q = \frac{1}{n} A R_h^{2/3} S_e^{1/2} \qquad (2.30)$$

The parameter $AR_h^{2/3}$ has been defined as the section factor and is given, for various channel sections, in table 2.4. 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 as first approximations.

From (2.30) it may be deduced that for a channel with a certain cross-section area A and a given slope S_e 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 6 the selection of the channel section is considered from the construction viewpoint, balancing efficiency, land excavation volumes, construction methods, etc

2.2.3 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 varied steady flows are parallel. On a channel with a sensibly constant reasonable slope (figure 2.16 a), the pressure head at any submerged point is equal to the vertical distance measured from the free surface to the 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 2.16 b): 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 2.16 c). Consequently the resulting pressure head, for water flows along a straight line, a convex path and a concave path is respectively

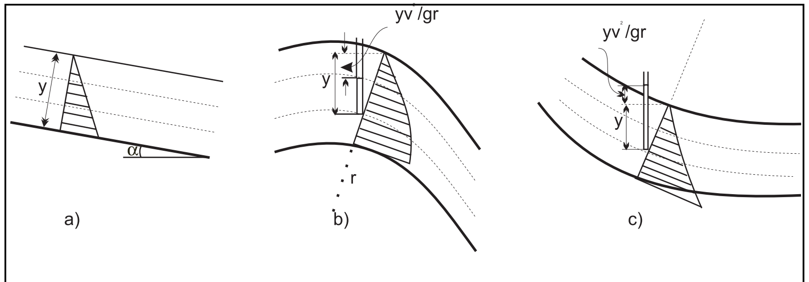


figure 2.16

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

where γ is the specific weight of water, y the depth measured from the free water surface to the point, V the water velocity at that point and 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 take 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 it can be used $\alpha = 1$, a reasonable value when the slope is under 0.018 (α <101). Equation 2.32 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 2.17 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 2.36) 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, dA/dy = T, where T is the top width of the channel section (see figure 2.17).

By definition $$Y = \frac{A}{T} \qquad (2.36)$$

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

Substituting in equation (2.37) dA/dy by T and A/T by Y:

$$\frac{Q^3}{gA^3}\frac{dA}{dy} = \frac{Q^2}{gA^2}\frac{T}{A} = \frac{V^2}{g}\frac{1}{Y} = 1;\ \frac{V}{\sqrt{gY}} = 1 \qquad (2.37)$$

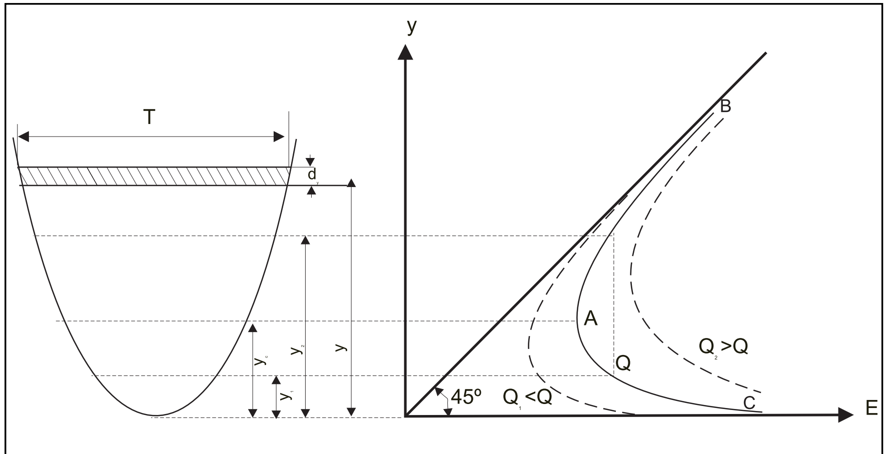


figure 2.17

The quantity $\frac{V}{\sqrt{gY}}$ is dimensionless and known as the *Froude number*.

When N_F= 1 as in equation (2.37), the flow is in the critical state; the flow is in the supercritical state when N_F<1 and in the subcritical state when N_F>1. Figure 2.17 can be analysed in this way. The AB line represents the supercritical flows, and the AC the subcritical ones.

As shown in figure 2.17, 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.

The second term of equation (2.37) can be written:

$$\frac{Q^2}{g} = YA^2 \tag{2.38}$$

In a rectangular channel Y = y and A=by; equation (2.38) may be rewritten

$$\frac{Q^2}{g} = y^3 b^2$$

In the critical state y = y_c being y_c the critical depth and

$$y_c = \sqrt[3]{\frac{Q^2}{gb^2}} = \sqrt[3]{\frac{q^2}{g}} \tag{2.39}$$

where q=Q/b is the discharge per unit width of the channel.

Table 2.4 shows the geometric characteristics of different channel profiles and Table 2.5, taken from Straub (1982) 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 2.5 $\Psi = \alpha Q^2/g$ = 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

$$y_c = 0.81\left(\frac{\Psi}{z^{0.75}b^{1.25}}\right)^{0.27} - \frac{b}{30z} = 0.86m$$

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 a sudden increase in the slope of a channel to be connected to another; for to spillway design profiles;/g the free surface behind a gate etc. Nevertheless in most cases the designer should make use of empirical formulae based on past experience.

Table 2.4 Geometrical properties of typical open channels

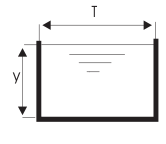
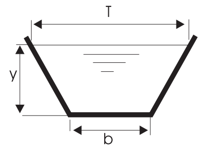
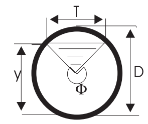
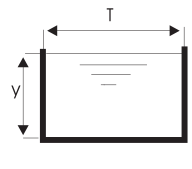
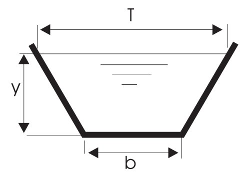
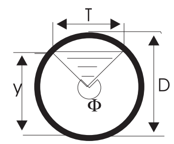
			
Area A	by	(b+zy)y	$\frac{1}{8}(\Phi - sen\Phi)D^2$
Wetted perimeter P	b+2y	$b+2y\sqrt{1+z^2}$	$1/2\phi D$
Top width of section T	b	b+2zy	$2\sqrt{y(D-y)}$
Hydraulic radius R	$\frac{by}{b+2y}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$\frac{1}{4}\left(1-\frac{sen\Phi}{\Phi}\right)D$
Hydraulic depth D	y	$\frac{(b+zy)y}{b+2zy}$	$\frac{1}{8}\left(\frac{\Phi - sen\Phi}{sen\frac{\Phi}{2}}\right)D$
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 2.5 (Straub 1982) $\Psi = \alpha\, Q^2/g$

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

2.3 Computer programs

There are quite a few computer programs that help to solve all kind of problems with open channels. We will simply refer to the Flow Pro 2.0, from Professional Software for Engineering Applications (PSA), a shareware that can be found in INTERNET, at the address http:\\www.prosoftapps.com for an evaluation copy.

The first step in computing a water surface profile is to select the Channel Type. You can do this by clicking the Channel Type menu and selecting Trapezoidal, Circular, Ushaped or Elongated Circular. The program title will reflect your selection, and the input fields will change accordingly.

Once a channel type has been selected and all of the required inputs have been entered, you can compute the water surface profile by selecting Compute from the Tools|Water Surface Profile menu. Flow Pro will compute the profile along with the normal and critical depths, the profile and flow types. The water surface profile grid will contain the tabulated data, which can be saved and imported into any spreadsheet for further analysis.

Flow Pro will classify the type of flow in a water surface profile. The flow type will be classified as either subcritical or supercritical. The profile computations start at the downstream end of the channel for subcritical flow, and the upstream end for supercritical flow. This is due to the location of the control depth for each type of flow.

For subcritical flow, the control depth is typically critical depth at the downstream end of a free discharging outfall or the height over a downstream weir. Supercritical flow has an upstream control depth such as the depth of flow under a gate. The

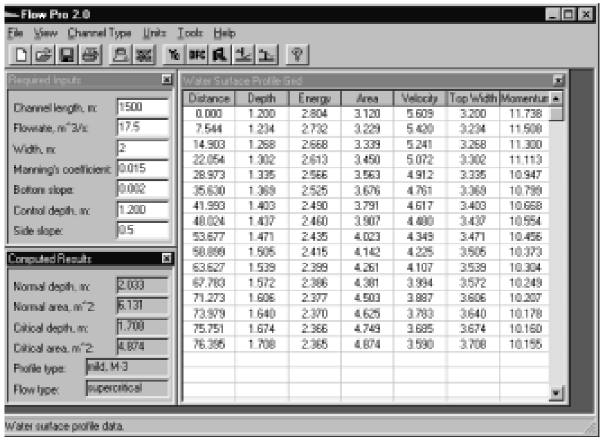


figure 2,18

water surface profile grid data will start the computations at zero, and continue until normal or critical depth is reached or until the channel ends. It is important to note the type of flow, so the direction the calculations proceed in the channel is understood.

Flow Pro will continue to calculate the profile along the length of the channel until the depth reaches normal or critical depth or the channel ends (whichever occurs first).

Figure 2.18 shows the dialog box with the depth, flow rate, slope and roughness of a certain canal, with the required inputs and the computed results.

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3 The water resource and its potential

3.0 Introduction

All hydroelectric generation depends on falling water. Streamflow is the fuel of a hydropower plant and without it, generation ceases. Accordingly, the study of any potential hydroelectric scheme must first of all address the availability of an adequate water supply. For an ungauged watercourse, where observations of discharge over a long period are not available involves the science of hydrology; the study of rainfall and streamflow, the measurement of drainage basins, catchment areas, evapotranspiration and surface geology.

Figure 3.1 illustrates how the water by flowing from point A to point B, regardless of the path B along the watercourse, an open canal or a penstock B it loses energy according to the equation:

$$P = QH\gamma$$

Where P is the power in kW lost by the water, Q the flow in m^3/s, H_g the gross head in m and γ the specific weight of water, being the product of its mass and the gravitational acceleration (9.81 kN/m^3).

The water can follow the riverbed, losing the power through friction and turbulence. Or it can flow from A to B through a pipe with a turbine at its lower end. The water would lose the same amount of power, in pipe friction, turbulence in the inlet, bends, valves, etc and in pushing its way through the turbine. In the later case it is the power lost in pushing through the turbine that will be converted by it to mechanical energy and then, by rotating the generator, to electricity. It can be seen that the objective of a good design is to minimise the amount of power lost between A and B, so the maximum amount of power may be available to rotate the generator.

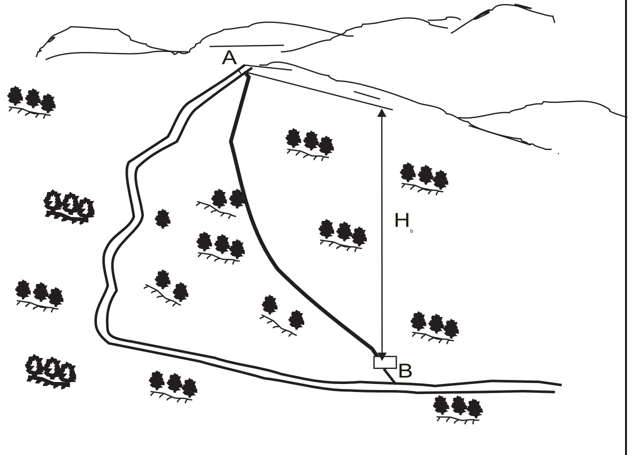


figure 3.1.

Therefore to estimate the water potential one needs to know the variation of the discharge throughout the year and how large is the gross available head. In the best circumstances the hydrologic authorities would have installed a gauging station, in the stretch of stream under consideration, and streamflow time series data would have been gathered regularly over several years.

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

Whatsoever, the first step to take is to look out for streamflow time series, in the stretch of river in question, if possible, or if not, in other stretches of the same river or in another similar nearby river, that permit to reconstitute the time series of the referred stretch of river.

3.1 Streamflow records

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.

INFOHYDRO includes a Manual and a computerised data

The INFOHYDRO Manual contains information concerning the entire INFOHYDRO and its operation. It also contains all hydrological information available at present in INFOHYDRO. Thus, the Manual comprises in a single volume comprehensive information on the Hydrological Services of the countries of the world and their data-collection activities. Chapter IV of the INFOHYDRO manual contains tables giving the numbers of observing stations maintained by the countries of the world as follows:

- Precipitation
- Evaporation
- Discharge
- Stage (water level)
- Sediment and water quality
- Groundwater

The INFOHYDRO Manual may also be purchased from WMO at a price of chf 132. Request WMO No. 683, INFOHYDRO Manual, (Operational Hydrology Report No. 28).

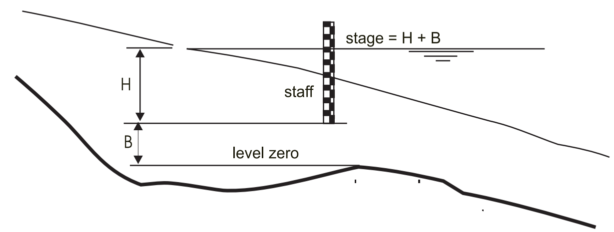


figure 3.2
stage measurement in a gauging station

The INFOHYDRO is a computerised database, and data can also be supplied on diskette. Requests should be addressed to:

The Secretary-General
World Meteorological Organization
41, Avenue Giuseppe Motta
P.O. Box 2300
CH-1211 GENEVA 2
Switzerland
Telephone: (+41 22) 730 81 11
Facsimile: (+41 22) 734 23 26
Cable: METEOMOND GENEVE
Telex: 23 260 OMM CH

3.2 Evaluating streamflows by discharge measurements

If appropriate streamflow time series cannot be found, and there is time, the discharge may 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:

3.2.1 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 streamflow 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, and 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 the style of a levelling staff- with the discharges.

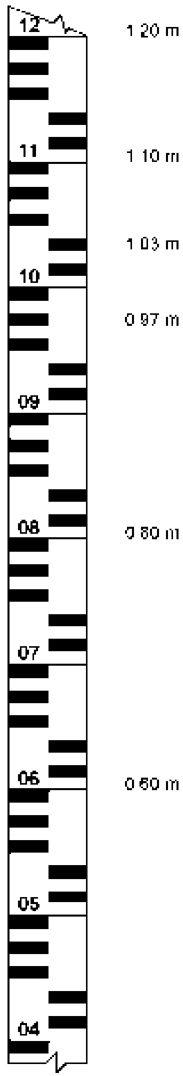


figure 3.3

Figure 3.3 shows a suitable marking system. In modern gauging stations, instead of a board, that requires regular observations, any one of several water-level measurement sensors available, which automatically register the stage, may be used. Periodic discharge measurements from the lowest to the highest are made over a time period of several months, to calibrate the stage observations or recordings.

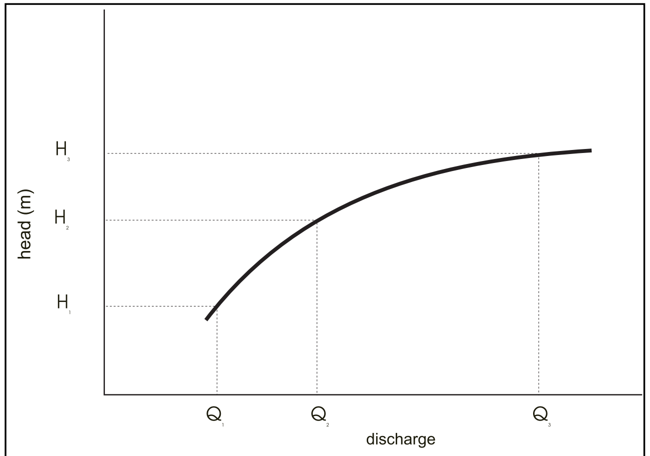


figure 3.4
rating curve

The correlation stage-discharge is called a rating curve (figure 3.4) 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 to begin measuring the low flows, and use the data to start to draw a curve that correlates the flows and the 'n' Manning coefficient. Later on the method of the river slope (section 3.3.4) can be used to estimate the high flows, often impossible to measure with the other methods.

The rating curve (figure 3.4) is represented by

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

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

$$Q_1 = a(H_1+B)^n$$
$$Q_2 = a(H_2+B)^n$$

By measuring a third point, corresponding to a discharge Q_3 and a stage H_3

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

consequently:

$$(H_3+B)^2 = (H_1+B)(H_2+B)$$ and therefore

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

There are ISO recommendations[2,3] for the correct use of this technique.

3.2.1.1 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 3.5). Measuring the trapezoid sides, by marked rules, such as figure 3.5 illustrates, the cross-section would be given by

$$S = b \times \frac{h_1 + h_2 + \ldots + h_n}{n} \qquad (3.3)$$

3.2.1.2 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.

By a floater
A floating object, which is largely, submerged B for instance a wood plug or a partially filled bottle B is located in the centre of the streamflow. The time t (seconds) elapsed to traverse a certain length L (m) is recorded. The surface speed (m/s)

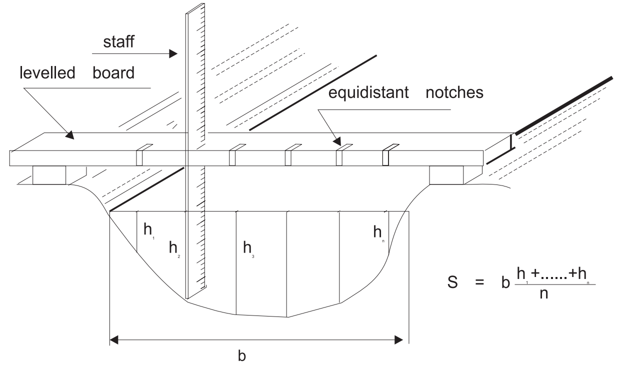
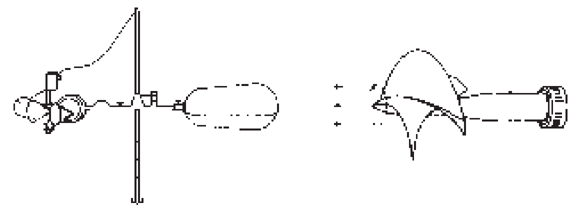


figure 3.5 measurement of cross-section

would be the quotient of the length L and the time t. To estimate the average flow speed, 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 (0.75 is a well accepted value)

By a propeller current-meter
A current-meter is a fluid-velocity-measuring instrument. A small propeller rotates about a horizontal shaft, which is kept parallel to the streamlines by tail fins. The instrument is ballasted to keep it as nearly as possible directly below the observer. Another version of the instrument has a circlet of small conical cups disposed horizontally about the suspension axis. (figure 3.6)

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, over a short period (say 1 or 2 minutes). These observations are converted into



a) caps current-meter b) propeller current-meter

figure 3.6
current-meters

water velocities from a calibration curve for the instrument. By moving the meter vertically and horizontally to a series of positions whose co-ordinates 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, though if the bridge is not a single-span one there will be divergence and convergence of the streamlines caused by the piers, which can cause considerable errors. In many instances, however the gauging site, which should be in as straight and uniform a reach of a river as is possible, will have no bridge and if it is deep and in flood, a cable to hold some 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, either from a manned cable car or directly from the cable car, the instrument in this latter case being positioned by auxiliary cables from the riverbanks.

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 ballasted 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 diameters exceed 500m, 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 circuitry.

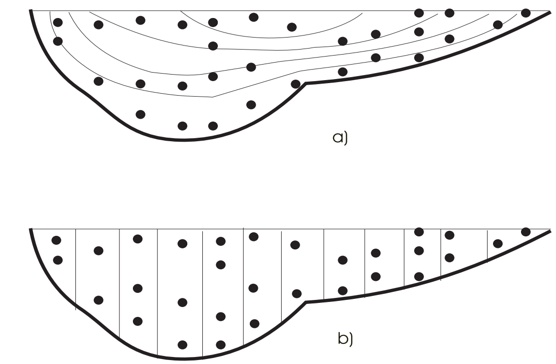


figure 3.7

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

The discharge at the cross-section is best obtained by plotting each velocity observation on a cross-section of the gauging site with an exaggerated vertical scale. **Isovels** or contours of equal velocity are then drawn and the included areas measured by a planimeter. A typical cross-section, so treated, is shown in figure 3.7 a). Alternatively, the river may be subdivided vertically into sections and the mean velocity of each section applied to its area, as in figure 3.7 b) In this method the cross-sectional area of any one section 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 Manning's n. In this way a knowledge of the n values of the river at various stages will be built up, which may prove most valuable in extending the discharge rating curve subsequently.

To ensure uniformity in the techniques of current-meter gauging ISO has published various recommendations[12;3]

3.2.2 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

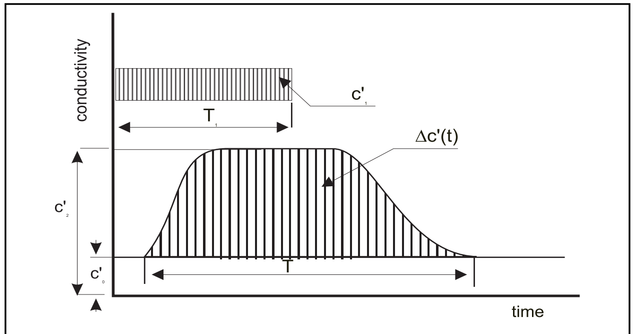


figure 3.8

concentration level, or administered in a single dose as quickly as possible, known as **gulp injection**. In this 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. Analysis of the samples is by an automated colorimetric procedure that estimates the concentration of very small amounts of the chromium compound by comparison with a sample of the injection solution. The equipment is expensive and specialised [4].

Nowadays the above methods have been substituted by the method developed by Littlewood[7,] requiring 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 of Aastad and Sognen[8].[9]

The discharge is measured by gradually discharging 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. In that way it is possible to plot a conductivity-time curve, along a time T as in figure 3.8. The average of the ordinates of this curve represents the average of the difference in conductivity, between the salt solutions and the streamwater, upstream the injection point. If a small volume, v, of the particular strong solution is added to a large volume V* of the streamwater, and the differences in conductivity Δc* are measured, the discharge will be then given by the equation:

$$Q = \frac{V}{T} \times \frac{V^*}{v} \times \frac{\Delta c^*}{\overline{\Delta c'}} \qquad (3.5)$$

where V = volume of injection solution
T = duration of solute wave (s)
v = volume of the strong solution added to a larger
V* = volume of streamwater
Δc* = change in conductivity (ohm^{-1}) consequence of the dilution of v in V*
$\overline{\Delta c'}$ = ordinate's average curve conductivity-time

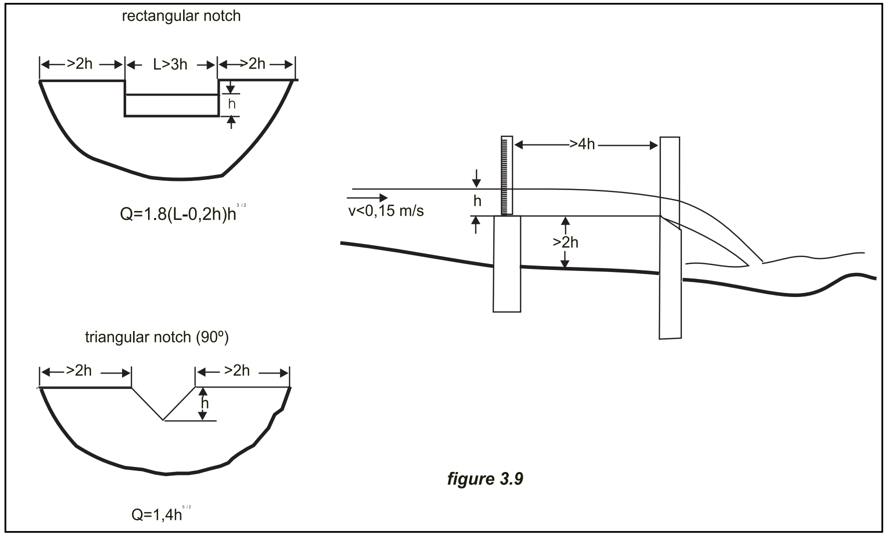


figure 3.9

3.2.3 Weir method

If the watercourse being developed is reasonably small (say < 4 m^3/s) then it may be possible to build a temporary weir. This is a low wall or dam across the stream to be gauged with a notch through which all the water may be channelled. Many investigations 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.

Several types of notch can be used - rectangular, vee or trapezoidal. The V-notch is most accurate at very low discharges but the rectangular or trapezoidal 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 3.9.

Flumes can be used similarly, 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.

In most cases of small-hydro development, such structures are too expensive and adequate flow data can be derived by simpler methods. Appropriate guidance and formulae may be found in references [10, 11, 12, 13, 14,15.]

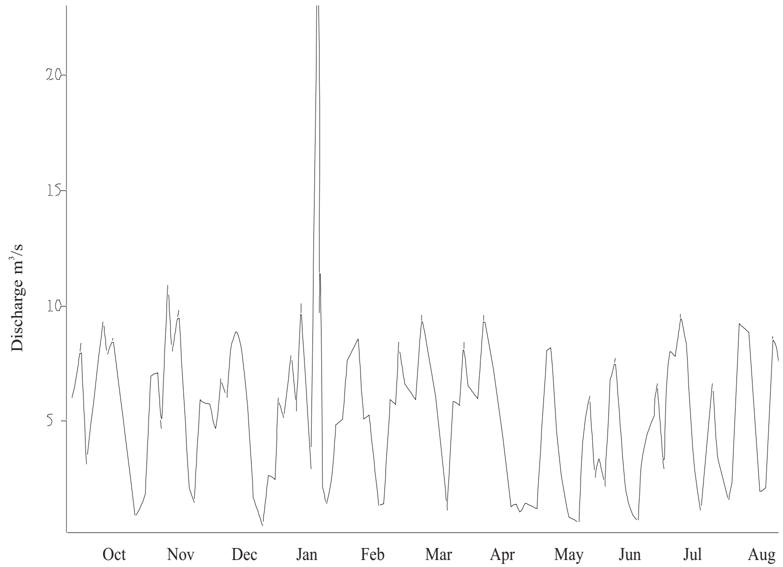


figure 3.10

3.2.4 Slope-area method

This method depends on hydraulic principles 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 water-surface level at the time of the flow measurement, upstream and downstream of the discharge-measuring site. These marks can subsequently be used to establish the water slope (S). Cross-sectional measurements will yield the area (A) and hydraulic radius of the section (R). Once known these parameters the discharge is computed by the Manning formula

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

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 objection may be partially met 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 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

3.3 Streamflow characteristics

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

3.3.1 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 3.10)

3.3.2 Flow Duration Curves (FDC)

Another way of organising discharge data is by plotting a flow duration curve (FDC), that 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: - e.g

	No of days	% of the year
Flows of 8.0 m3/s and greater	41	11.23
Flows of 7.0 m^3/s and greater	54	14.90
Flows of 6.5 m^3/s and greater	61	16.80
Flows of 5.5 m^3/s and greater	80	21.80
Flows of 5.0 m^3/s and greater	90	24.66
Flows of 4.5 m^3/s and greater	100	27.50
Flows of 3.0 m^3/s and greater	142	39.00
Flows of 2.0 m^3/s and greater	183	50.00
Flows of 1.5 m^3/s and greater	215	58.90
Flows of 1.0 m^3/s and greater	256	70.00
Flows of 0.35 m^3/s and greater	365	100.00

then a graph like figure 3.11 will be obtained, which represents the ordinates of figure 3.10 arranged in order of magnitude instead of chronologically.

Nowadays, when most gauging stations are computerised, the easiest way to derive a FDC is to transpose the digital data to a spreadsheet, sorting them in descending order, and 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 (such as has been draw figure 3.11).

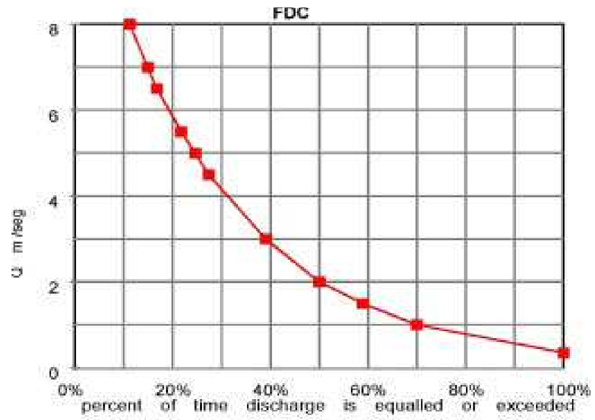


figure 3.11

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 ordinate (Q) to a logarithmic scale, and a normal probability scale used for the frequency axis. On such a graph, if the logarithms of the discharges are normally distributed, then the FDC plots as a straight line. Figure 3.12 represents figure 3.11 with the vertical axis in logarithmic scale.

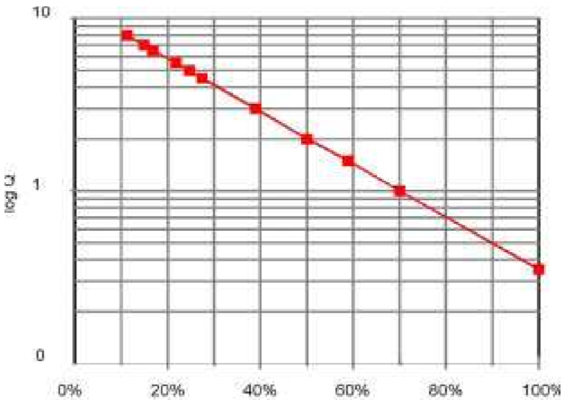


figure 3.12

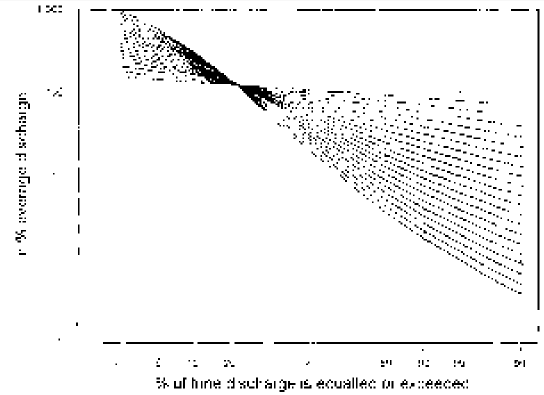


figure 3.13

3.3.3 Standardised FDC curves

FDCs for different rivers can be compared when presented in this more compact way, by standardising them. The discharges are divided firstly by the contributing catchment area and secondly by weighted average annual rainfall over the catchment. 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 3.13 shows twenty FDCs corresponding to catchment areas of different geological composition, drawn to a double logarithmic scale.

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.

3.3.4 Evaluating streamflows at ungauged sites

When there are no flow records at a particular location it is necessary to proceed from first principles. Rainfall data are normally available from national agencies on an annual-average basis, but often only on a fairly small scale. 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, immediately studies are considered. Even one year's records will help in the production of a synthesised FDC.

The first step then is to estimate the mean annual flow Q_m (also referred to as ADF or average daily flow). In UK the mean flow is estimated using a catchment water balance methodology: the long term average annual catchment runoff 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 /year, it is assumed that actual evaporation is equal to potential. This relationship between SAAR is given by

$r = 0.00061 \times SAAR + 0.475$ for $SAAR < 850$ mm
$r = 1.0$ for $SAAR \geq 850$ mm

Actual evaporation is calculated using $AE = r \times PE$

The average runoff depth (AARD in millimetres) over the catchment area (AREA in km^2) is converted to mean flow in m^3s^{-1} by:

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

In other countries it may need modification, using similar methods. For instance, in Spain, the water balance methodology does not yield feasible results, whereat the equation to represent mean flow is given by a modified empirical equation:

The meanflow over catchment is then:

$$Q_m = Runoff \times AREA \times 3.17 \times 10^{B5}$$

where Q_m is given in m^3s^{-1}, the runoff in mm and the AREA in km^2.

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 streams. 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, the soils have been categorised into 29 discrete groups to represent different physical properties and the hydrological response of soils. 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 area, 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 3.13. 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.

3.3.5 European Atlas of Small Scale Hydropower Resources

Although using the above methodology is a rather lengthy process, the flow regime of the site, represented by the FDC, can be easily estimated. To aid local authorities, water resource planners and potential investors, to asses the feasibility of developing small hydro schemes anywhere in the European Union, the Institute of Hydrology in the UK, has developed the **European Atlas of Small Scale Hydropower Potential**. The Atlas has been developed on behalf of the European Small Hydropower Association (ESHA) with the financial aid of the E.C, DGXVII in the frame of the ALTENER Programme.

The Atlas, which is presented as a menu driven Microsoft Windows ™ compatible software package, incorporates methods for deriving flow duration curves at ungauged sites and standard engineering methods for using these curves to estimate the hydropower generation potential for the commercial turbine types. To estimate the hydro potential of a site the Atlas proceeds as follows:

1 estimation of the catchment characteristics for the site, including catchment area, average rainfall, average potential evaporation and appropriate low flow statistic;
2 estimation of the flow regime within the catchment, represented by the flow duration curve, using above catchment characteristics:

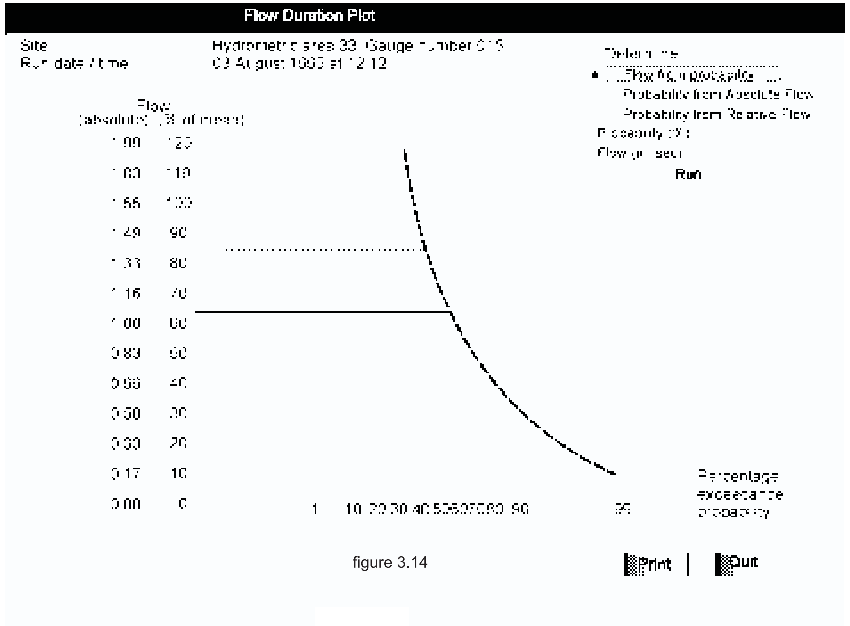


figure 3.14

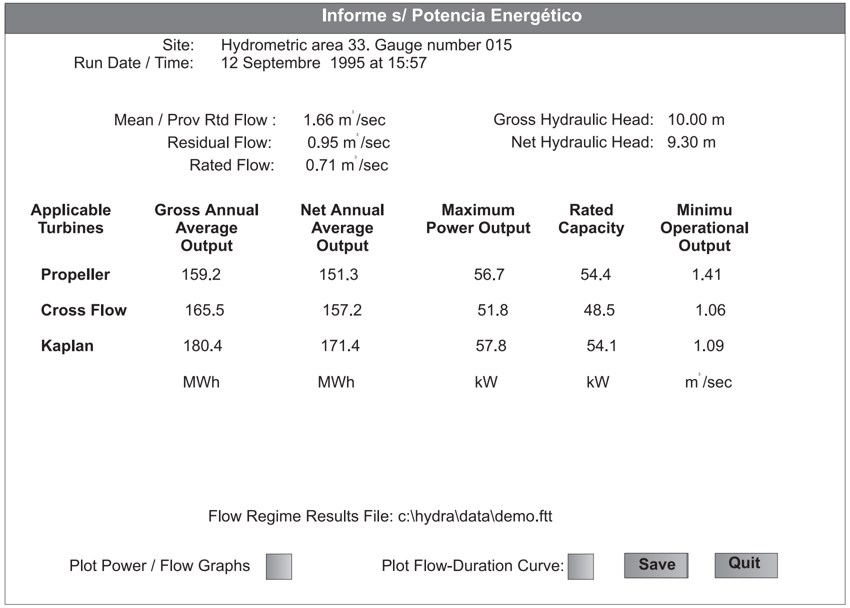
3 estimation of power potential for a range of suitable turbines based on the estimated flow duration curve

To accomplish the task the user is required to define the catchment boundary.

The estimation of the catchment characteristics then proceeds by the program:

1. Calculating the catchment area;
2. Transposing the catchment boundary onto thematic catchment characteristic maps to estimate catchment average values of annual average rainfall, potential evaporation and the fractional extent of individual soil units;
3. Estimating the mean flow using a water balance model incorporating the parameters thus determined.
4. Calculation of a standardised low flow statistic using the appropriate relationship between flow and soil characteristics (assigned to hydrological response units as appropriate).

Graphical and tabular output may be obtained at each stage in the estimation procedure within the software. Figure 3.14 shows the flow duration curve of a site in UK. The box at the upper right is used to obtain the probability of exceedence for an absolute or relative flow, just as the flow corresponding to a certain exceedence.



Informe s/ Potencia Energético

Site: Hydrometric area 33. Gauge number 015
Run Date / Time: 12 Septembre 1995 at 15:57

Mean / Prov Rtd Flow : 1.66 m^3/sec
Residual Flow: 0.95 m^3/sec
Rated Flow: 0.71 m^3/sec

Gross Hydraulic Head: 10.00 m
Net Hydraulic Head: 9.30 m

Applicable Turbines	Gross Annual Average Output	Net Annual Average Output	Maximum Power Output	Rated Capacity	Minimu Operational Output
Propeller	159.2	151.3	56.7	54.4	1.41
Cross Flow	165.5	157.2	51.8	48.5	1.06
Kaplan	180.4	171.4	57.8	54.1	1.09
	MWh	MWh	kW	kW	m^3/sec

Flow Regime Results File: c:\hydra\data\demo.ftt

Plot Power / Flow Graphs Plot Flow-Duration Curve: **Save** **Quit**

figure 3.15

The flow duration curve, in conjunction with user defined head and design flow parameters, is used to calculate energy and power output, which can potentially be anticipated at the site. Figure 3.15 shows a power potential report where gross and net average annual output and rated capacity for various possible turbines are clearly indicated.

The computer program is easy to operate and yields very interesting results. The package in its different modules permits the modification of the input data, coming from the previous module.

3.3.6 FDC's 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 or periods of record. 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.

3.3.7 Water pressure or 'head'

3.3.7.1 Measurement of gross head

The gross head is the vertical distance that the water falls through in generating power, 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 impose the methods to be employed.

In the past the best way to measure it was by levelling with a surveyor's level and staff, but the process was slow. Accurate measurements were made by a tachometer or less accurately by a clinometer or Abney level. Nowadays with digital theodolites, the electronic digital levels and especially with the electronic total stations the job has been simplified. The modern electronic digital levels provides an automatic display of height and distance within about 4 seconds with a height measurement accuracy of 0,4 mm, and the internal memory makes it possible to store approximately 2,400 data points. Surveying by Global Positioning Systems (GPS) is already practised and a handheld GPS receiver is ideal for field positioning, and rough mapping.

3.3.7.2 Estimation of net head

Having established the gross head available it is necessary to allow for the losses arising from trash racks, pipe friction, bends and valves. In addition to these losses,

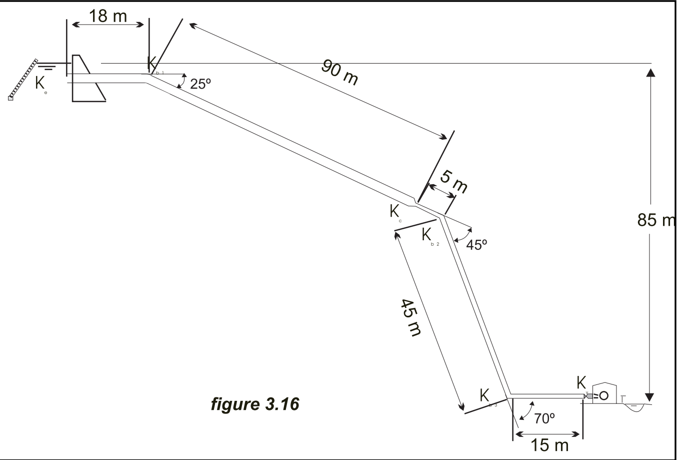


figure 3.16

certain types of turbines must be set to discharge to the atmosphere above the flood level of the tail water (the lower surface level). The gross head minus the sum of all the losses equals the net head, which is what is available to drive the turbine. Example 3.1 will help to clarify the situation

Example 3.1

Figure 3.16 shows the pipe layout in a small hydropower scheme. The nominal discharge is 3 m³/s and the gross head 85 m. The penstock has 1.1 m diameter in the first length and 0.90 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 trashrack inclined 60º 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 trashrack area is estimated by the formula:

$$S = \frac{1}{K_1}\left(\frac{t}{t+b}\right)\frac{Q}{v_0}\frac{1}{\operatorname{sen}\alpha}$$

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 trashrack has an automatic cleaner, is equal to 0.80. Assuming v_0 = 1 m/s, S=5.07 m². For practical reasons a 6 m² trashrack may be specified, corresponding to a v_0 = 0.85 m/s, which is acceptable.
The headloss traversing the trashrack, as computed from the Kirschner equation

$$h_r = 2.4\left(\frac{12}{70}\right)^{3/4}\frac{0{,}8^2}{2\times 9{,}81} = 0{,}007\ m$$

The friction losses in the first penstock length are a function of the water velocity, 3.16 m/s.
The entrance to the pipe has a bad design and coefficient K_e = 0.8 (see figure 2.11)
The head loss in the first length according to Manning's equation is:

$$\frac{h_f}{108} = \frac{10{,}29\times 0{,}012^2\times 3^2}{1{,}1^{5{,}333}} = 0{,}008$$

The headloss coefficient in the first bend is K_b = 0.085 (one half of the corresponding loss of a 90º bend); in the second K_b = 0.12 and in the third K_b =0.14
The taper pipe, with an angle of 30º, 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 = 4.72 m/s)
The friction headloss in the second length is computed in the same way as the first one, and accordingly h_f/65 = 0.0234 (water velocity in second span is 4.72 m/s)
The coefficient of headloss in the gate valve is K_v= 0.15.
Therefore the friction headloss are estimated as

0.008 x 108 + 0.0234 x 65 = 2.385 m

The turbulence headloss will be as follows:

In the trashrack	0.007 m
In the pipe entrance 0.8 x 0.508	0.406 m
In the first bend 0.085x0.508	0.043 m
In the second bend 0.12x1.135	0.136 m
In the third bend 0.14x1.135	0.159 m
In the confusor 0.02x1.135	0.023 m
In the gate valve 0.15x1.135	0.170 m

The total head loss is equal to 2.385 m friction loss plus 1.375 m turbulence loss, giving a net head of 81.24 m. This represents a loss of power of 4.42% which is reasonable. Improving the pipe entrance the loss coefficient will diminish by almost 39 cm.

3.4 Residual, reserved or compensation flow

An uncontrolled abstraction of water from a watercourse, to pass 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 results for aquatic life.

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.

It is in the interest of the hydro-power developer to keep the residual flow as small as is acceptable to the licensing authority, since in seasons of low flow, its release may mean generation being stopped if there is insufficient discharge to provide both it and minimum turbine discharge. On the other hand the lack of flowing water can endanger the life of the aquatic biota. In Chapter 7 the subject will be treated in depth from an environmental viewpoint.

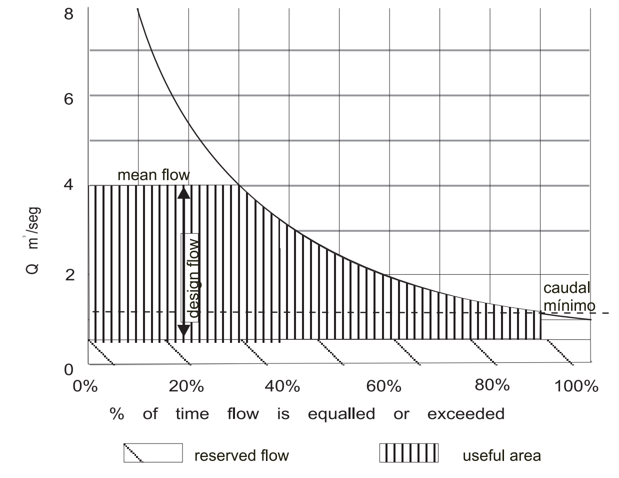


figure 3.17

3.5 Estimation of plant capacity and energy output

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

Figure 3.17 illustrates the FDC of the site it is intended to evaluate. Usually the design flow is assumed to be, in a first approach, the difference between the mean annual flow and the reserved flow. In actual practice is strongly recommended to evaluate the plant for other design flows in order to choose, the one that yields the best results. Once the design flow is defined (Q_m-Q_{res}), and the net head estimated, suitable turbine types must be identified. The suitable turbines are those for which the design flow and head plot within the operational envelopes (figure 3.18). Figure 3.17 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 gross average annual energy (E in kWh) is a function

$$E = fn\,(Q_{median}, H_n, \eta_{turbine}, \eta_{generator}, \eta_{gearbox}, \eta_{transformer}, \gamma, h)$$

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 Q_{median}

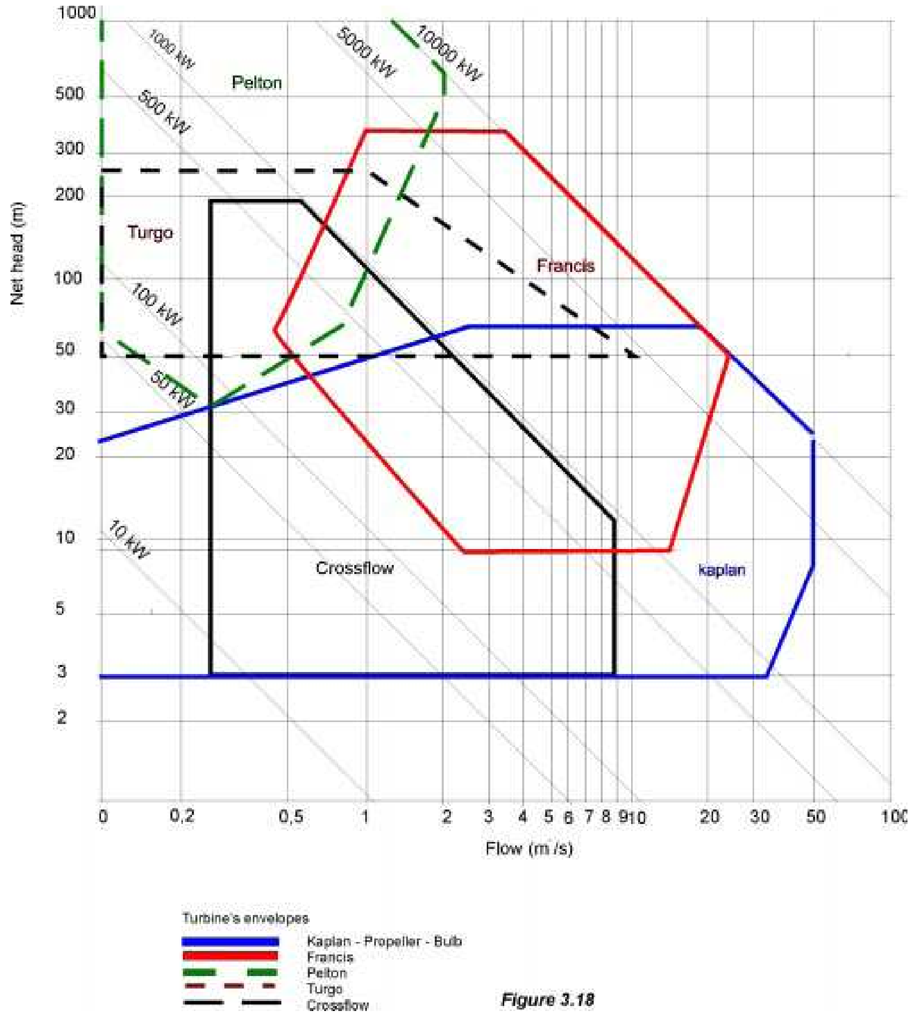


Figure 3.18

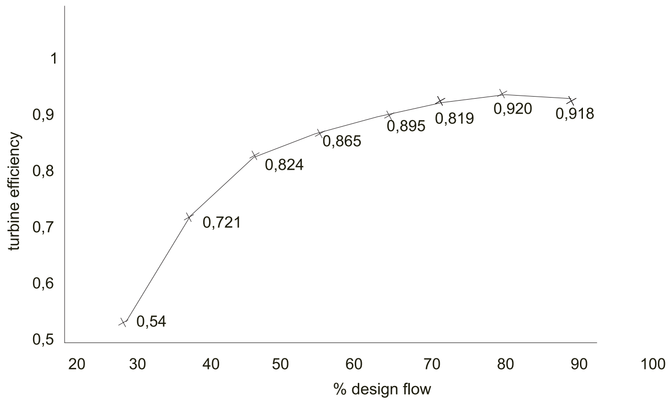


figure 3.19

$\eta_{generator}$ = generator efficiency

$\eta_{gearbox}$ = gearbox efficiency

$\eta_{transformer}$ = transformer efficiency

h = number of hours for which the specified flow occurs.

The software package uses a procedure to calculate the energy. It divides 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 $\eta_{turbine}$ value is defined for the corresponding efficiency curve and the energy contribution of the strip is calculated using the equation:

$$\Delta E = W.Q_{median}.H.\ \eta_{turbine}.\ \eta_{generator}.\eta_{gearbox}.\ \eta_{transformer}.\gamma.\ h$$

where
- W = strip width = 0.05 for all strips except the last one that should be calculated
- h = number of hours in a year
- γ = specific weight of the water (9.81 KNm^{-3})

The gross average energy is then the sum of the energy contribution for each strip.

The capacity of each turbine (kW) it 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 can be seen the curves of turbine efficiency against flow for the commercial turbines. Table 3.1gives the minimum technical flow for different types of turbines, as a percentage of the design flow.

Table 3.1 Minimum technical flow of turbines

Turbine type	Q_{min}
Francis spiral	30
Francis open flume	30
Semi Kaplan	15
Kaplan	15
Cross flow	15
Pelton	10
Turgo	10
Propeller	65

The **European Atlas of Small Scale Hydropower Potential** includes a module to compute both the installed capacity and annual energy output of every appropriate turbine, and prepare a complete report on the results. Anyone can estimate both power and energy output by hand, simply by calculating areas, but it is tedious work that can be shortened with the aid of the software package

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

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

If a turbine operate with a bigger flow than the design flow Q_d, under a head H_1 smaller than the rated head H_d, the flow admitted by the turbine will be:

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

Headwater elevation versus spillway discharge is easy to compute. According to the spillway theory,

$$Q = CLH^{3/2} \qquad (3.8)$$

where Q = Discharge over spillway
C = Spillway discharge coefficient
L = Length of the spillway crest
H = height of the water surface level above the spillway crest

The value of C depends on spillway shape, and may be found in hydraulic reference books.

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, equation (3.8) is applied to the excess flow, which passes over the spillway. 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 (it suffices to add the discharge going through the turbine).

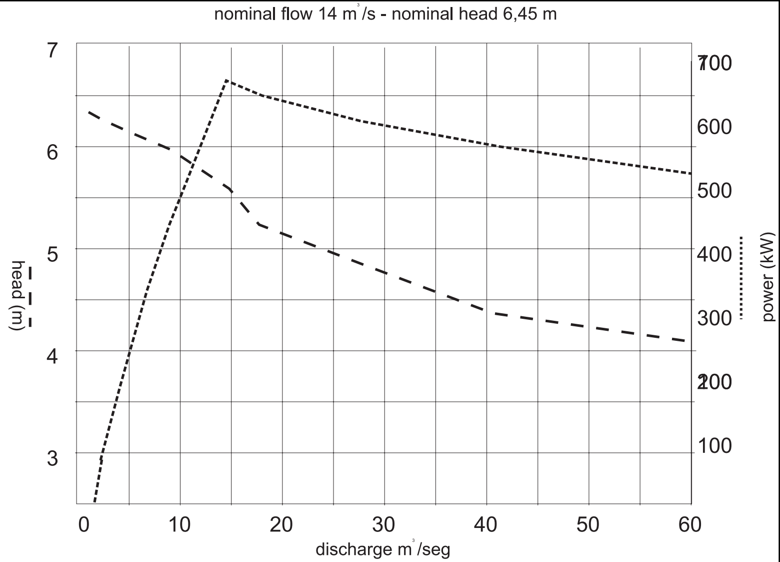


figure 3.20

The tailrace level is more difficult to estimate. The Hydrologic Engineering Center (HEC) of the US Army Corp of Engineers, has developed a computer program, HEC-HMS, that can be usefull for that purpose. It can be downloaded, free of charge from INTERNET, http://www.hec.usace.army.mil/software/software_distrib/hec-hms/hechmsprogram.html

Figure 3.20 shows how the head varies with the flow in a real case and its influence on the power delivered at different river discharges.

3.5.2 Another methodology to compute power and annual energy output

If the European Atlas software package is not available, the use of an electronic spreadsheet, with a model such as the one shown in Table 3.2 is suggested, especially in low head schemes where the flow passing through the turbine is a function of the nominal head and the actual head, corresponding at this flow.

The discharge passing through the turbine would be the river discharge, less the reserved flow, except if it exceeds maximum turbine discharge or other constraints on turbine discharge are encountered. If the head is smaller than the rated head, the discharge admitted by the turbine will be given by the equation:

$$Q_i = Q_d \sqrt{H_i / H_d} \tag{3.9}$$

where the suffix 'i' indicate the parameters corresponding to the point i in the FDC and the suffix 'd' the design parameters. The power P in kW will be given by the product of Q, H, η (global efficiency in %) and 0.0981. The energy output by the power multiplied by ΔT and the total number of hours in the year, less 5% downtime. 'Downtime' is the time when the plant is unavailable through malfunction, maintenance or shortage of water.

In table 3.2 the «River discharge» shows the river discharge less the reserved flow. After some iterative calculations, it was decided to have as design flow, the corresponding to the 50% exceedence - 46 m^3 /s - with a rated head 6.45 m. The curve head-river discharge is reflected in line 4 (Head), and the flow going trhough the turbine is shown in line 5, function of river discharge and net head. The turbine, a double regulated Kaplan, will have an installed power of 2.450 kW. The calculation procedure is clearly explicited in the table.

Table 3.2

	10%	20%	30%	40%	50%	60%	70%	80%	85%	90%	95%	100%
River discharge (m3/s)	70,00	60,67	53,78	49,33	46,00	43,52	40,78	37,97	36,33	34,70	32,70	26,30
Nominal head (m)	**6,45**											
Rated flow (m3/s)	**46,00**											
Head (m)	4,50	4,95	5,40	6,10	6,45	6,55	6,60	6,62	6,63	6,64	6,65	6,66
Flow through the turbine (m3/s)	38,42	40,30	42,09	44,73	46,00	43,52	40,78	37,97	36,33	34,70	32,70	26,30
Global plant efficiencyl (%)	0,83	0,83	0,83	0,84	0,84	0,84	0,84	0,84	0,84	0,84	0,83	0,82
Power (kW)	1.408	1.624	1.851	2.249	2.445	2.349	2.218	2.071	1.985	1.887	1.771	1.409
Delta T (%)		10%	10%	10%	10%	10%	10%	10%	5%	5%	5%	5%
E (GWh)		1.262	1.446	1.706	1.953	1.995	1.900	1.785	844	806	761	662
Annual energy output (GWh)	15.118											

3.5.3 Peaking operation

Electricity prices at peak hours are 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, preferably in peak hours. To compute this volume considering that:

Q_R = river flow (m^3/s)
Q_D = rated flow (m^3/s)
Q_P = flow in peak hours (m^3/s)
Q_{OP} = flow 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

The needed storage volume V will be given by:

$$V_R = 3.600 \ t_P\left(Q_P - \left(Q_R - Q_{res}\right)\right)$$

If the pound should be refilled in off-peak hours

$$t_P\left(Q_P-\left(Q_R-Q_{res}\right)\right)<t_{OP}\left(Q_R-Q_{res}\right) \text{ thence}$$

$$Q_P \le \left(Q_R-Q_{res}\right)\frac{t_{op}-t_p}{t_p}$$

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

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

3.6 Firm energy

A run-of-river scheme cannot, in general, guarantee a firm energy. On the contrary a group of small hydro run-of-river schemes, located in different basins of a country possibly can, because the low flow seasons may not occur at the same time.

If a small hydro scheme has been developed to supply energy to an isolated area, the rated flow should be the one corresponding in the FDC to the exceedence probability of, at least, 90%. But even in these conditions the electricity supply cannot be guaranteed 90% of the time, because the FDC is related to the long term and does not necessarily apply in dry years.

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4. Site evaluation methodologies

4.0 Introduction.

Adequate head and flow are necessary requirements for hydro generation. Consequently site selection is conditioned by the existence of both requirements.

For the flow, chapter 3 lists the addresses of the international and national organisations where stream data are recorded, underlining the availability of specialised databases. With the European Atlas of Small Scale Hydropower Resources, by introducing the catchment geographic definition, the mean flow and the Flow Duration Curve for any specific site may be estimated. If the scheme is located in a country where databases for the Atlas do not exist, one of the methodologies detailed in the chapter 3 may help to get the required results.

The gross head may be rapidly estimated, either by field surveying or by using the GPSs (Global Positioning System) or by ortophotographic techniques. With the aid of the engineering hydraulic principles brought out in chapter 2 the net head may be computed. Flow and head estimation should no longer be a problem.

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 sensibility of the site, are most important. That is why a thorough knowledge of the scheme 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.

4.1 Cartography

In the industrialised countries maps to the required scale 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: the topographical relief –land, no matter how flat, is never horizontal – and the tilt of the optical axis of the camera.

Modern cameras 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 detecting slope instability and the engineer gather data necessary for dam, open channels and penstock construction.

Depending on the required accuracy, the digitised photographs can be geocoded (tied to a coordinate system and map projection) and orthorectified. Distortion from the camera lens is removed by using ground control points from maps, survey data or client's 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 ortophotos. Both hard copy and digital ortophotos in diskettes can be produced.

With those maps is possible to locate the intake, trace the open channel and the penstock and locate the powerhouse, with precision enough for the feasibility studies and even for the phase of bidding. With stereoscopic photographs geologic problems can often be spotted, specially those concerning slope stability, that can cause dangerous situations.

4.2 Geotechnical studies

Frequently, the need to proceed with detailed geological studies of the site is 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 estimates, in a first approach, of the security of the dam foundations, the stability of the slopes and the permeability of the terrain. However sometimes this general information should be complemented with fieldwork of drilling and sampling.

Hydraulic structures should be founded on level foundations, with adequate side slopes and top 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 for the first phase of the project. The problem is especially acute in high mountain schemes, where the construction may be in the 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 a 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, product of the surface mechanical weathering of the rock masses, and solifluction processes, very active in high mountain environments where the subsoil is seasonally or perennially wet, are some of the features that can compromise channel stability. Drainage treatments, bench constructions and gunite 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 because its 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).

4.2.1 Methodologies to be used

Within geological science, there is a wide spectrum of geomorphologic techniques that can be used including the following 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, based on a topographic one, 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 tri-axial consolidation facilitate the surface formation classification, to be included in the above mentioned map.

Geophysical studies
A geophysical investigation either electric or seismic by refraction will contribute to a better knowledge of the superficial formation's thickness, the location of the landslide sections, the internal water circulation, and the volumetric importance of potential unstable formations.

Structural geological analysis
Although not properly a geomorphologic technology it 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:

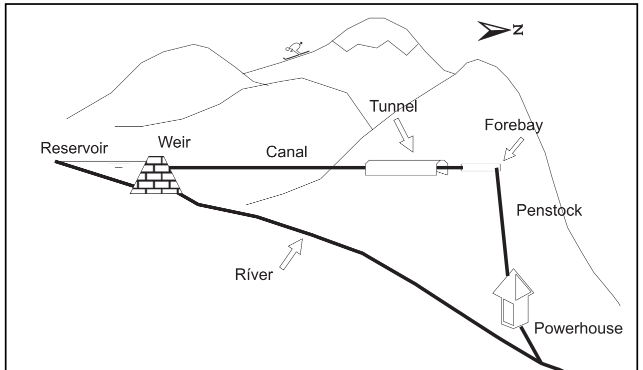


figure 4.1

- Permeability tests in boreholes, such as Lugeon or Low Pressure Test, to define the water circulation in the foundation.
- Laboratory tests to determine the compression strength of the samples to define their consolidations characteristics.

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

4.2.2 Methodologies. The study of a practical case.

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 m^3
- 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

4.2.2.1 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 or the rock mass must be anchored.

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

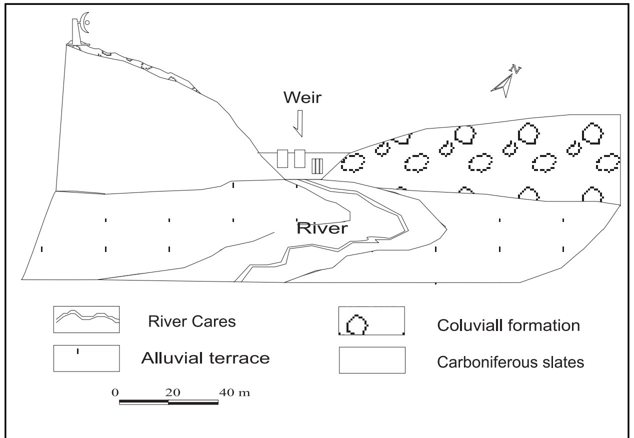


figure 4. 2

formation; the right one less steep is associated to a colluvial formation.

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.

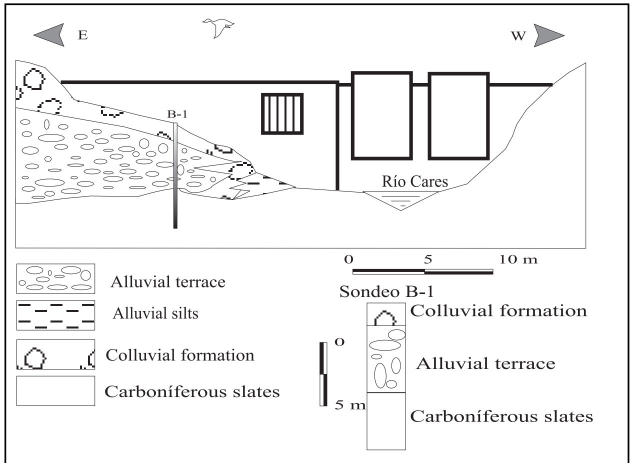


figure 4.3.

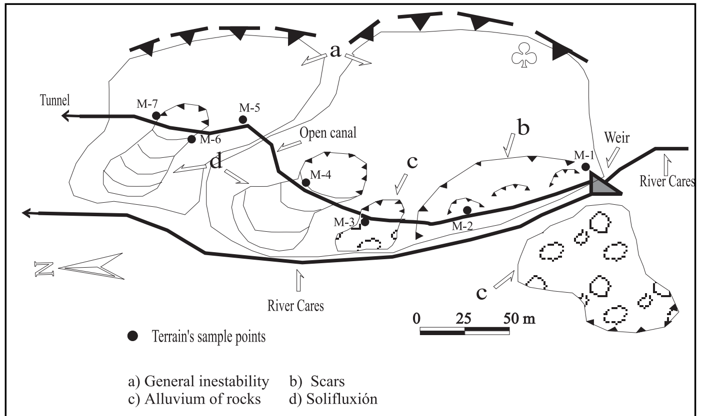


figure 4.4.

4.2.2.2 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

Photo 4.2

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

Photo 4.4

4.2.2.3 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 existing 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 from the point marked in figure 4.4 with the word "tunnel". Figure 4.5 shows

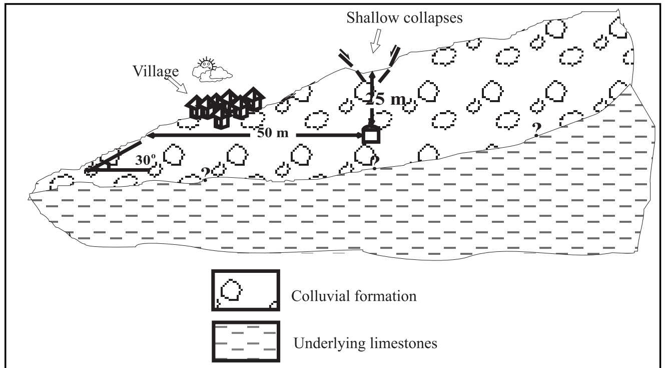
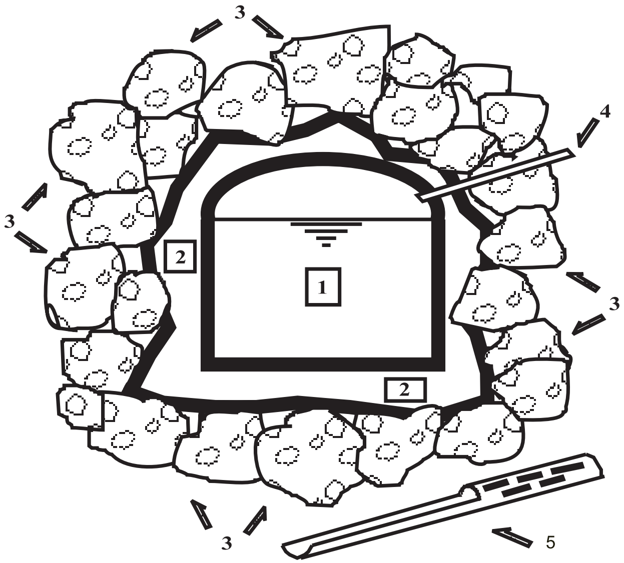


figure 4.5



1. Canal in tunnel
2. Injected concrete
3. Coluviall formation
4. Drain
5. Detail of the draining pipe

figure 4.6

a schematic cut of the tunnel under the colluvium and figure 4.6 illustrates the concrete lining conforming 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 out of place here. Excavation by tunnelling machines unfeasible. The excavation had to proceed meter by meter 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.

Photo 4.5

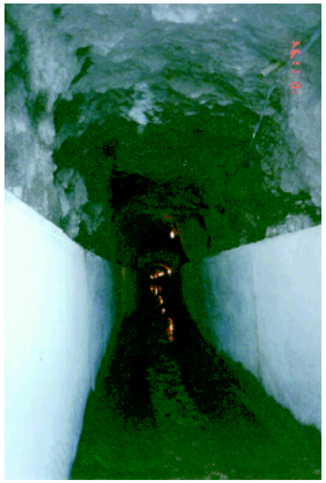
The construction of tunnels through rocky massifs should take into account two important geologic characteristics:

- The lithologic variation along its trace, that 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 – 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 bendings, 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 sound were completely altered. Its response to the excavation was of course very different from the response of the rest of the massif. Only by knowing in time the presence of this fault could the tunnel be excavated without unexpected incidents. Figure 4.8 shows in greater detail how the tunnel was excavated through the fault zone. 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 4.7



4.2.2.4 The powerhouse

Due to the presence of large and heavy equipment units the powerhouse stability must be completely secured. Settlements cannot be accepted in the powerhouse. If the geologic condition of the ground cannot guarantee the stability of the foundation it must be strengthened.

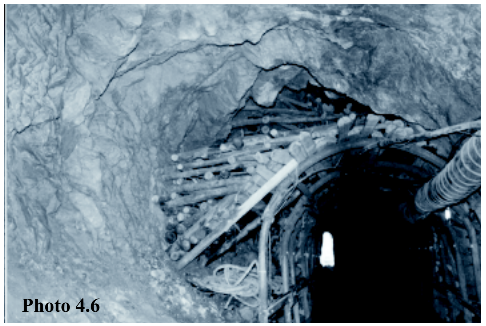


Photo 4.6

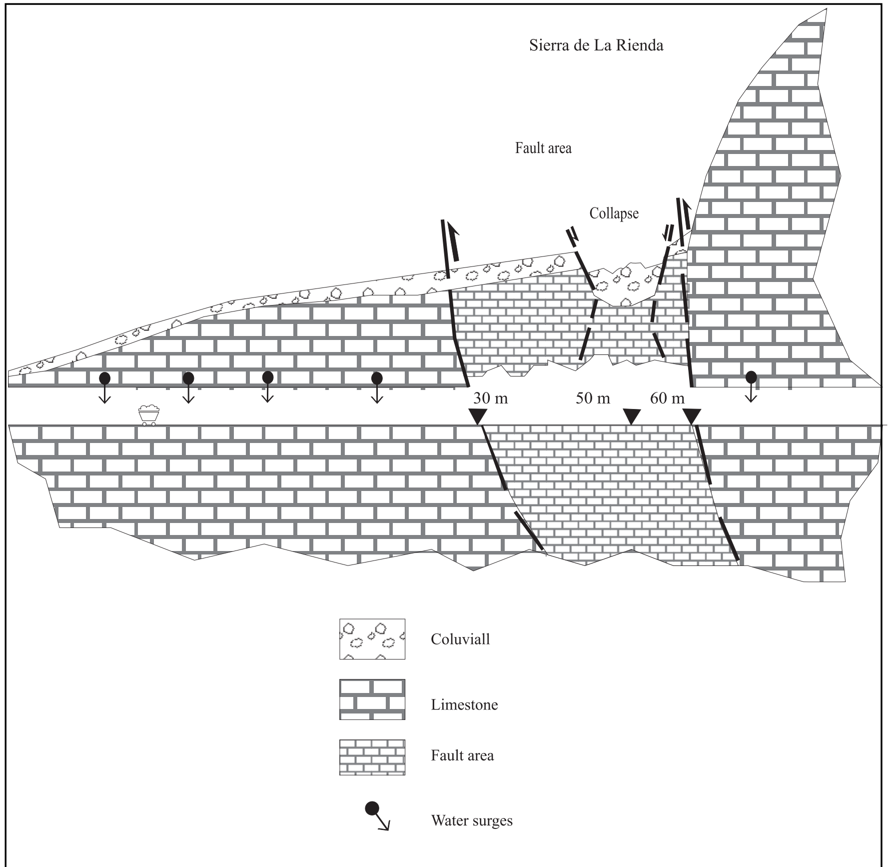


figure 4.7

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

The traditional cement grouting presents some difficulties and in any case its results never will be satisfactory when the terrain is as heterogenous and permeable as exists in fluvial terraces. A new injection technique, jet grouting, can guarantee the

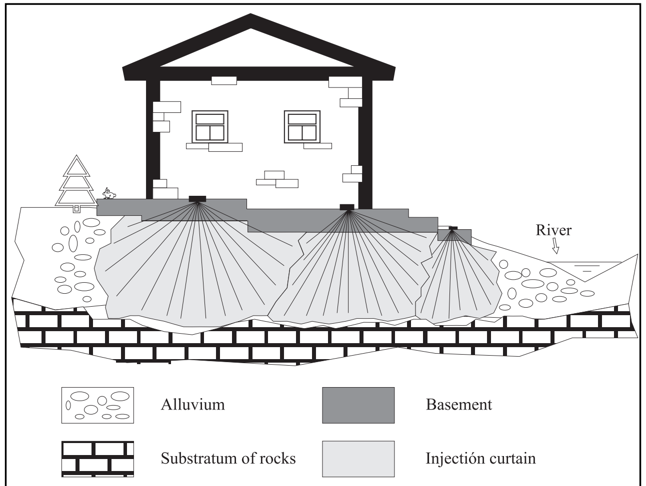


figure 4.9

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 4.9 illustrates the results of the jet-grouting operation which was performed to reinforce the terrain supporting the powerhouse.

4.3 Learning from failures

Two well-known experts, Bryan Leyland of Australia and Freddy Isambert from France, presented to HIDROENERGIA95 Conference, that was held at Milan, two independent papers dealing with the topic "lessons from failures". Mr Leyland quoting Mr Winston Churchill –"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. And according to Mr Issambert "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

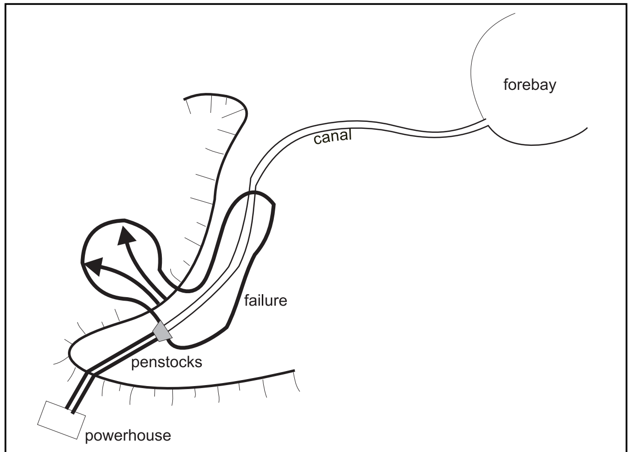


figure 4.9

their repetition". And 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, those 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.

Ruahihi canal failure (New Zealand)
As shown in figure 4.10 the scheme had a 2000 m 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.

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 1100 m of pipes. This increased the length of the penstocks from 750 m to 1850 m and required that water hammer pressures have to be reduced because the original concrete pipes could only withstand a very limited overpressure.

Photo 4.8

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 canal failure (Spain)
The La Marea scheme has a spiral Francis turbine of 1 100 kW installed capacity for a discharge of 1.3 m^3/s and a 100-m head. As shown in figure 4.11 the scheme includes a small weir for the water intake, provided with a ladder fish pass. From the intake a rectangular canal built in reinforced concrete (3 x 2 m section) is followed by another 600 m 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 100-m above the powerhouse.

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

The lessons learned were:
- Weathered sandstone gives bad results against landslide, specially on slopes with an angle over 35º to the horizontal.
- Hydraulic canals should be built to guarantee their waterproofness; alternatively a draining system should be devised so the water leakage can not affect the terrain.
- The replacement of an open canal by a low pressure pipe on a steep slope may

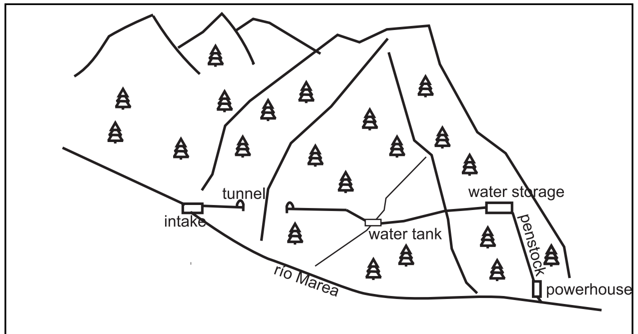


figure 4.10

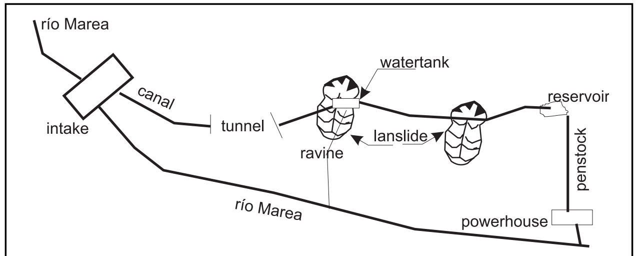


figure 4.11

be the best option, because it will be watertight and because its anchorage on the slope will require only a few strong points.

Seepage under a weir (France)
This case concerns a small weir, which is the structure furthest upstream of a 600 kW project comprising 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 cutoff 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 4.11

The hydraulic canal in a low-head 2 MW scheme
The hydraulic canal - 5 m wide and 500 m long – goes along the river and close to it. 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

There are other cases that could be described to show the effects of misjudgment 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.

5. Hydraulic structures

5.1 Structures for storage and water intake

5.1.1 Dams

The dam is a fundamental element in conventional hydraulic schemes, where it is used to create a reservoir to store water and to develop head. In relatively flat terrain, a dam, by increasing the level of the water surface, can develop the head necessary to generate the required energy. The dam can also be used to store, during high flow seasons the water required to generate energy in dry seasons. Notwithstanding this, due to the high cost of dams and their appurtenances, they are seldom used in small hydro schemes.

If a scheme is connected to an isolated net, and if the topography is favourable, a dam can be built to store excess water when the flow is high or the demand low to make it available at times of low flow or increased demand.

Where a reservoir is built for other purposes –irrigation, water supply to a city, flood regulation, etc- it can be used by constructing a plant at the base of the dam to generate energy as an additional benefit.

5.1.2 Weirs

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 overflow it. When the scheme is large enough this diversion structure becomes a small dam, 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 should be constructed on rock and in their simplest version consist of a few boulders placed across the stream (figure 5.1). When the rock is deep, excavation is needed, and a sill constructed of gabions - steel mesh baskets filled with stones- can be used (figure 5.2).

In larger structures the weir may be a small earth dam, with an impervious core

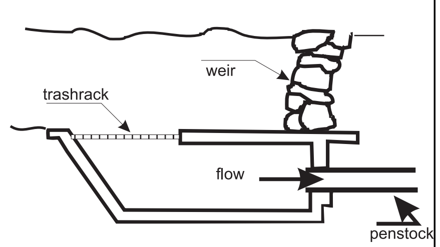


figure 5.1

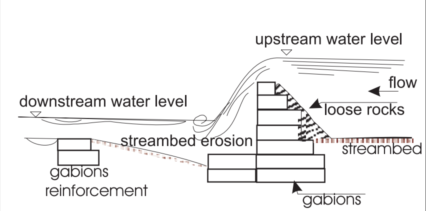


figure 5.2

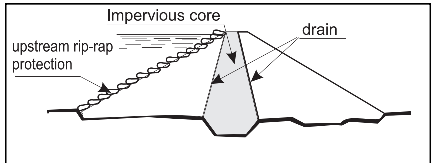


figura 5.3

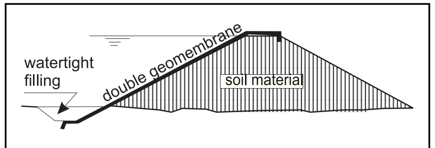


figure 5.4

which extends well into the impervious foundation, located in the central portion of the dam. (Figure 5.3). This core is generally constructed of compacted clayey material. If this material is not available in the site a properly welded geotextiles sheet must cover the upstream embankment to provide the required waterproofing (figure 5.4)

If clayey material doesn't exist in the site but sand and gravel are easily found, the construction of a concrete dam can be considered. If the stream is subjected to sudden floods that require the construction of large spillways, very expensive to build in an earth dam, concrete dams, where the spillways is easily integrated (photo 5.1) may be advisable. However if the scheme is located on a seismic area, rigid structures such as concrete dams should be avoided, and earth dams are more suitable. In very cold climates the required precautions to be taken with the freshly poured concrete can be so costly that the construction of a concrete dam is not feasible.

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 exceed15 m, the crest length is less than 500 m and the stored water is less than1 million cubic meters. These parameters are important, because of the complicated administrative procedures associated with the construction of large dams. The great majority of small dams in small hydro schemes are of the gravity type, commonly founded on solid rock and where their stability is due to their own weight. If a dam is less than 10 m high it can be built on earth foundations, but allowable stresses must not be exceeded and the possibility of piping due to

Photo 5.1

seepage under the dam minimised, through the use of aprons or cut-offs. For the foundation it will be necessary to know the shear strength, compressive strength and Poisson's ratio.

The dam must be stable for all possible loading conditions (figure 5.5): hydrostatic forces on the upstream and downstream faces; hydrostatic uplift acting under the base of the dam; forces due to silt deposited in the reservoir and in contact with the dam; earthquakes forces that are assumed to act both horizontally and vertically through the centre of gravity of the dam (if the dam is located in a seismically active zone); earthquake forces induced by the relative movements of the dam and reservoir etc.

Since the dam must be safe from overturning under all possible load conditions therefore the contact stress between the foundation and the dam must be greater than zero at all points. To assure this condition the resultant of all horizontal and vertical forces –included the weight of the dam- must pass through the middle one-third of the base. The upstream face is usually vertical whereas the downstream face has a constant inclination. It is also necessary to guarantee that the dam doesn't slide, so the static friction coefficient –all the horizontal forces divided by all the vertical ones- must remain between 0.6 and 0.75.

5.1.2.1 Devices to raise the water level.

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 (photo 5.2). 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 (figure 5.6 a). 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 illustrated in figure 5.6.b is somewhat easier to remove.

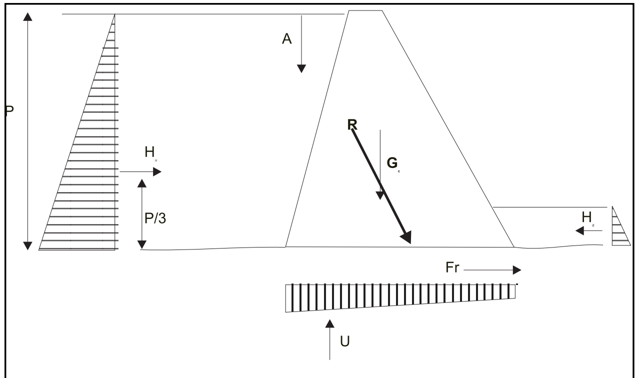
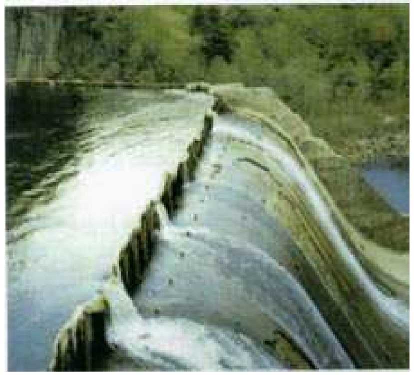


figure 5.5



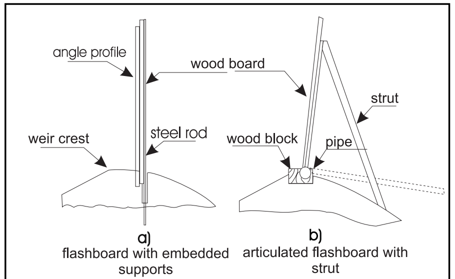


figure 5.6

Photo 5.2

In low head schemes with integral intake and powerhouse –see figure 1.3- the best way to increase the head without risking upstream flooding, is the sector gate. A hydraulic system or an electric motor opens the gate, so that the water passes underneath.

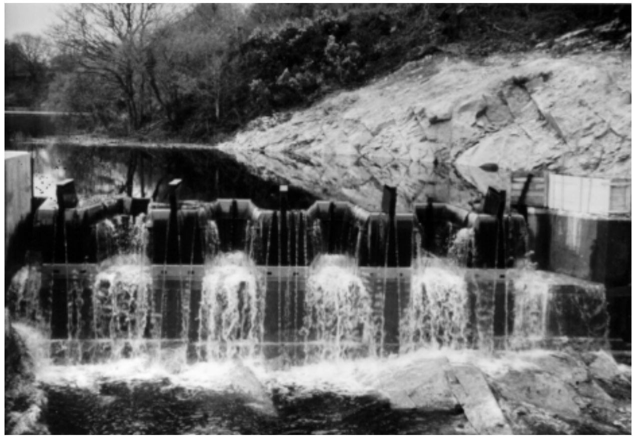
In large installations, but also sometimes in small ones , it is advisable to place fusegates, such as those supplied by Hydroplus[1]. In the event of a major flood, when the water reaches a pre-set level, one or more of the fusegates –basically hinged structures- will tilt to increase the section of the spillway (photo 5.3).

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 initial 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 5.7) 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 (photo 5.4); 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.

Photo 5.3



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 5.5 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 manageable 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. The free space between panels or between panel and the buttress are closed by a synthetic rubber flap anchored to one of the panels[2].

Photo 5.4

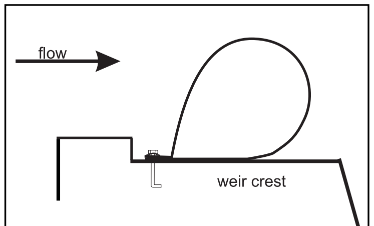


figure 5.7

Photo 5.5

5.1.3 Spillways

In the south of Europe, with a clear difference between dry and wet season flows, flood flows can have catastrophic effects on whatever structure is built in the stream. To avoid damage the excess water must be safely discharged over the dam or weir. For this reason carefully designed overflow passages are incorporated in dams 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.

Photo 5.6

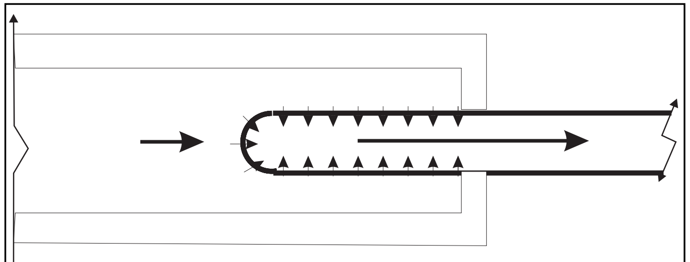


figure 5.8

The commonest type of spillway is the overflow gravity type (photo 5.6). Basically it is an open channel with a steep slope and with a rounded crest at its entry. To minimise the pressure on the surface of the spillway the profile of the crest should follow the same curve as the underside of the free-falling water nappe overflowing a sharp crest weir. This trajectory varies with the head, so the crest profile is the right one only for the design head H_s. If $H>H_s$ negative pressure zones tend to develop along the profile and cavitation may occur. Recent work suggests that fortunately, separation will not occur until $H>3\ H_s$. The U.S. Waterways Experimental Station[3] has provided a set of profiles that have been found to agree with actual prototype measurements.

Photo 5.7

The discharge may be calculated by the equation

$$Q=CLH^{3/2} \tag{5.1}$$

where C is the coefficient of discharge, L is the length of the spillway crest and H is the static head. The coefficient of discharge C is determined by scale model tests; its value normally ranges between 1.66 for broad crested weirs to 2.2 for a weir designed with the optimum profile, when the head equals the design head.

In some small hydropower schemes –e.g. small scheme in an irrigation canal- there is not enough space to locate a conventional spillway. In these cases, U shaped (figure 5.8 and photo 5.7) or labyrinth weirs (figure 5.9) should help to obtain a higher discharge in the available length.

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 as illustrated in Fig 5.10 [4]. When the water level rises above the elbow of the siphon the water begins to flow, down the conduit just as in an overflow spillway, 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.

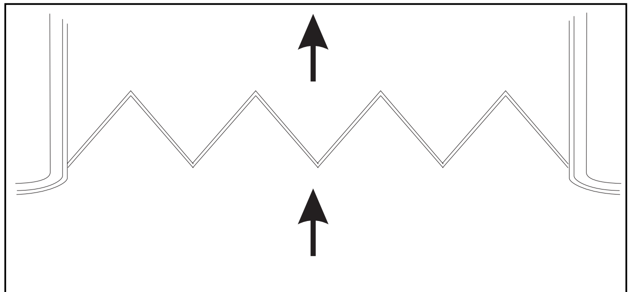


figure 5.9

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 deprimed 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,

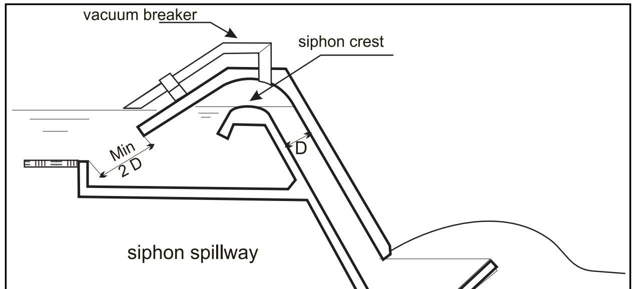


figure 5.10

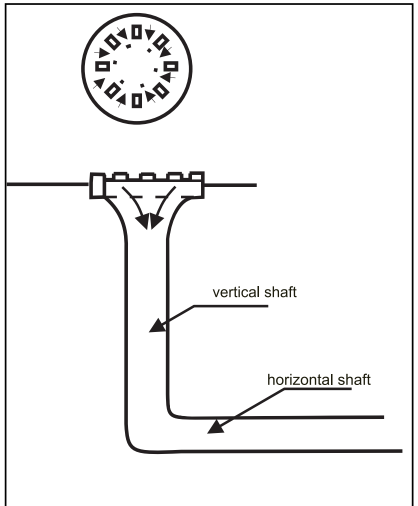


figure 5.11

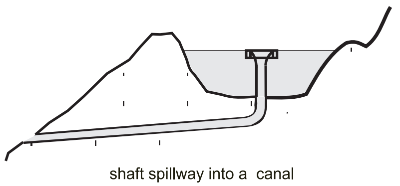


figure 5.12

should normally not exceed 5 m. Further details on this kind of spillway can be found in the literature[6].

Shaft or "glory hole" spillways are rarely used in small scale-hydro. As illustrated in Fig 5.11 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. Figure 5.12, reproduced from Inversin[5] illustrates a shaft installed to evacuate the excess water in a channel, where a side-spillway could generate a landslide by saturating the terrain. The US Bureau of Reclamation reports (USBR)[6,7] describe the design principles for these spillways.

5.1.4 Energy dissipators

The discharge from a spillway outlet is usually supercritical and so may produce severe erosion at the toe of the dam, especially if the streambed is of silt or clay. To avoid such damage, a transition structure known as a stilling basin must be constructed to induce the formation of a hydraulic jump, where the water flow changes from supercritical to subcritical. The USBR has published a set of curves to be used in the design of stilling basins[8].

5.1.5 Low level outlets

Low level outlets in small hydropower schemes are used to perform, together or independently, the downstream release and the evacuation of the reservoir, either in an emergency or to permit dam maintenance. In general a low level-conduit with a cone valve at the exit or a sliding gate at the inlet is enough to perform both functions. At the exit, if the flow is supercritical, the provision of energy dissipators should be considered.

5.1.6 River diversion during construction

In small hydropower schemes the construction may be completed, in some cases, within the dry season, but in many others, diversion arrangements will be necessary. Suitable diversion structures include the following:

Gabions with geotextiles on the upstream faces
Earth dikes with riprap protection
Inflatable weirs
Sheetpile diversion dams

The techniques of their construction and their practical use require the advice of specialised engineers.

5.2 Waterways

5.2.1 Intake structures

The Glossary of Hydropower Terms -1989 defines the intake as "a structure to divert water into a conduit leading to the power plant". Following the ASCE Committee on Hydropower Intakes[11], the water intake in this handbook is defined as a structure to divert water to a waterway –not specifying what type of waterway: a power channel or a pressure conduit- and reserving the word *forebay* or power intake, to those intakes directly supplying water to the turbine, via a penstock.

A water intake must be able to divert the required amount of water into the power canal or into the penstock without producing a negative impact on the local environment and with the minimum possible headloss. 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 – fish diversion systems, fishpasses- characteristics of each project.

Even if every year new ideas for intake design are proposed –advances in modelling, new construction materials, etc- the fundamental hydraulic and structural design concepts have not changed much in many years, and are not likely to change in the future. Over the years, many intakes have been designed; vast quantities of trash have been removed; and large amounts of sediments have been sluiced. From all that accumulated experience we now know what works and what does not work, and this experience together with fundamental hydraulic principles, the designer can develop better and effective intakes, precluding future incidents.

5.2.1.1 Water intake types

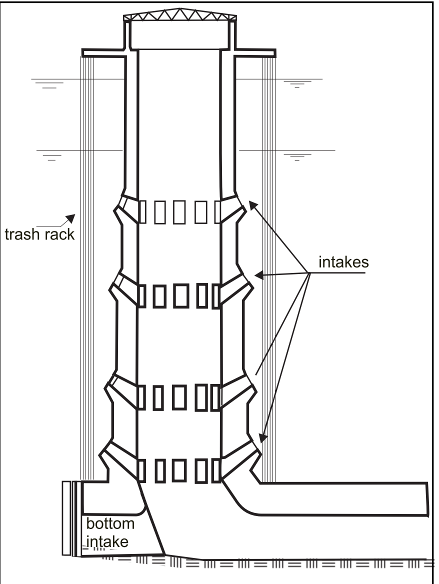


figure 5.13

The first thing for the designer to do is to decide what kind of intake the scheme needs. Notwithstanding the large variety of existing intakes, these can be classified according to the following criteria:

- The intake supplies water directly to the turbine via a penstock (figure 5.1). This is what is known as power intake or forebay.
- The intake supplies water to other waterways –power canal, flume, tunnel, etc- that usually end in a power intake (figure 1.1 Chapter 1). This is known as a conveyance intake
- The scheme doesn't have any conventional intake, but make use of other devices, like siphon intakes or "french intakes" that will be described later.

In multipurpose reservoirs –built for irrigation, drinking water abstraction, flood regulation, etc- the water can be withdrawn through towers with multiple level ports, permitting selective withdrawal from the reservoir's vertical strata (figure 5.13) or through bottom outlets (figure 5.14)

The siphon intake (figure 5.15) renders intake gates unnecessary, and the inlet valves (provided each unit has its own conduit) may also be eliminated, reducing the total cost by 25-40 per cent, and reducing the silt intake. The water flow to the turbine can be shut off more quickly than in a gated intake, which is beneficial in a runaway condition. Photo 5.8 shows a siphon intake built on an existing dam, with very small civil works. The siphon can be made of steel, or alternatively in countries where the procurement of fabricated steel is difficult, in reinforced concrete, with the critical sections lined in steel.

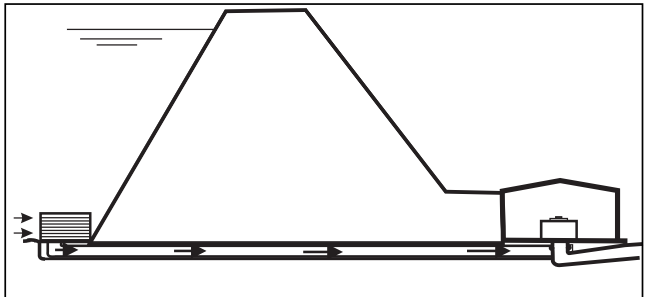


figure 5.14

Photo 5,8

The "french" or drop intake (figure 5.16) 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 5.9 shows a drop intake installed in a mountain stream in Asturias (Spain). In France EDF has improved this type of intake, placing the bars as cantileversto avoid the accumulation of small stones commonly entrained by the water (figure 5.17)

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

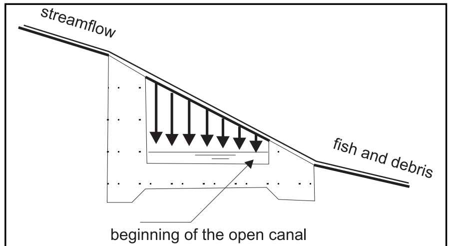


figure 5.16

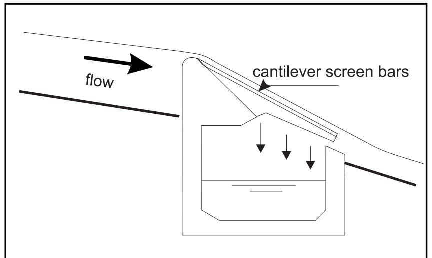


figure 5.17

Photo 5.9

silt basin and sediment ejection system can be omitted. The intake (photo 5.10) is patented by AQUA SHEAR and distributed by DULAS[11] in Europe.

5.2.1.2 Intake location

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.

Photo 5.10

Photo 5.11

The orientation of the intake entrance to the flow is a crucial factor in minimising debris accumulation on the trashrack, a source of future maintenance problems and plant stoppages. The best disposition[9] of the intake is with the screen at right angles to the spillway (figure 5.1) so, that in flood seasons the flow entrains 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 entrain and accumulate trash at the entrance. If for any reason the intake entrance should be parallel to the spillway, it is preferable to locate it close to the spillway so the operator can push the trash away to be carried away by the spillway flow. (See photo 5.11 in a dry season where all the water went through the turbine)

The water 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. Spillways have been already considered in depth in 5.1.3, as other components will be later.

5.2.2 Power intake

The power intake is a variant of the conventional intake, usually located at the end of a power canal, although sometimes it can replace it. Because it has to supply water to a pressure conduit –the penstock- its hydraulic requirements are more stringent that those of a conveyance intake.

In small hydropower schemes, even in high head ones, water intakes are horizontal, followed by a curve to an inclined or vertical penstock. The design depends on whether the horizontal intake is a component of a high head or a low head scheme. In low head schemes a good hydraulic design –often more costly than a less efficient one- makes sense, because the head loss through the intake is comparatively large related to the gross head. In high head schemes, the value of the energy lost in the intake will be small relatively to the total head and the cost of increasing the intake size to provide a lower intake velocity and a better profile may not be justified.

In a power intake several components need consideration:
- Approach walls to the trashrack designed to minimise flow separation and head losses
- Transition from rectangular cross section to a circular one to meet the entrance to the penstock
- Piers to support mechanical equipment including trashracks, and service gates
- Guide vanes to distribute flow uniformly
- Vortex suppression devices

The velocity profile decisively influences the trashrack efficiency. The velocity along the intake may vary, from 0.8 – 1.0 m/sec through the trashrack to 3 – 5 m/sec 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; a reasonable velocity of the flow approaching the trashrack provides the dimensions of the rectangular section.

The research department of “Energy, Mines and Resources” of Canada[10] commissioned a study of entrance loss coefficients for small, low-head intake structures to establishing guide lines for selecting optimum intakes geometries. The results showed that economic benefits increase with progressively smoother intake geometrics having multiplane roof transition planes prepared from flat formwork. In addition, it was found that *cost savings from shorter and more compact intakes were significantly higher than the corresponding disbenefits from increased head losses.*

The analyses of cost/benefits recommend the design of a compact intake –it appeared that the length of the intake was unlikely to be the major factor contributing to the overall loss coefficient- with a sloping roof and converging walls (figure 5.18, alternative 2 in the study). The K coefficient of this transition profile was 0.19. The head loss (m) in the intake is given by

$$\Delta h = 0.19\, v^2/2g \qquad (5.2)$$

where v is the velocity in the penstock (m/sec).

A well-designed intake should not only minimise head losses but also perclude vorticity. Vorticity 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

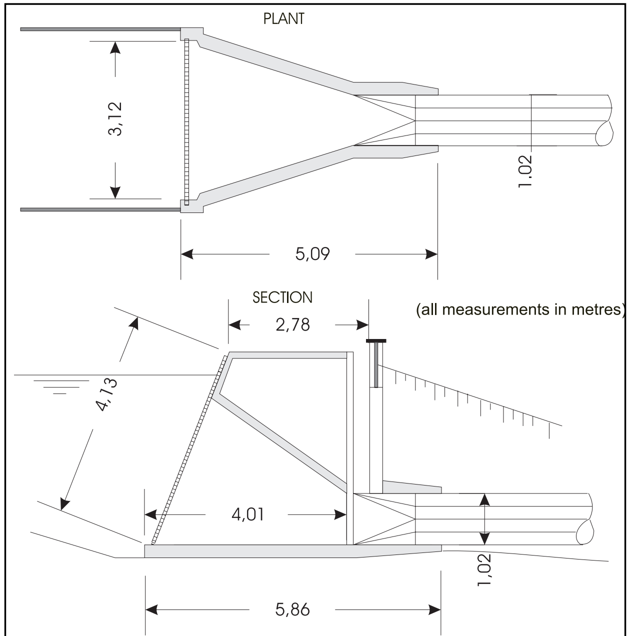


figure 5.18

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 commonest causes of vortex formation. An asymmetric approach (figure 5.19 a) is more prone to vortex formation than a symmetrical one (figure 5.19b). Providing the inlet to the penstock is deep enough, and the flow undisturbed vortex formation is unlikely.

According to Gulliver, Rindels and Liblom (1986) of St. Anthony Falls hydraulic laboratories, vortices need not be expected provided (figure 5.19)

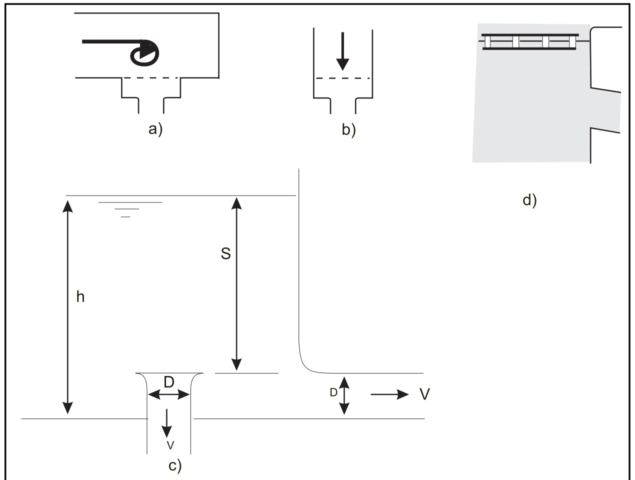


figure 5.19

$$S > 0.7D \text{ and } N_F = \frac{V}{\sqrt{gD}} < 0.5 \qquad (5.3)$$

After applying the above recommendations, if there is still vortex formation at the plant and it is impossible to increase the submergence of the penstock entrance or increase its diameter- the situation can be improved by a floating raft which disrupts the angular movement of the water near the surface (figure 5.19 d)

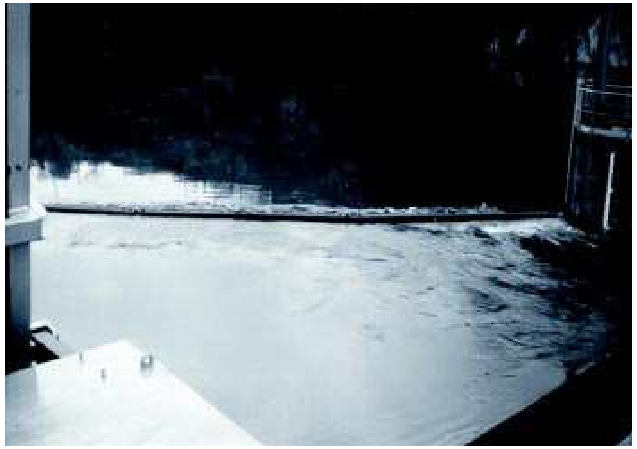
5.2.3 Mechanical equipment

5.2.3.1 Debris management in intakes

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

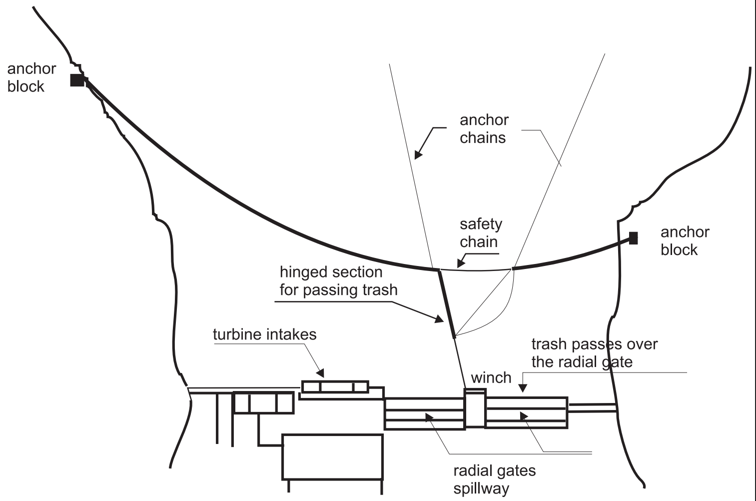
Trashracks are fabricated with stainless steel or plastic bars. Since the plastic bars can be made in airfoil sections, less turbulence and lower head losses result. The bar spacing varies from a clear width of 12 mm for small high head Pelton turbines to a maximum of 150 mm for large propeller ones. The trashrack should

Photo 5.12



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 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 5.13



(reproduced from ASCE publication)

figure 5.20

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 (photos 5.12 and 5.13) 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 total surface of the screen will be given by the equation:

$$S = \frac{1}{K_1}\left(\frac{b+a}{a}\right)\frac{Q}{V_0}\frac{1}{\sin\alpha} \tag{5.4}$$

Where: S = Total area of the submerged part of the screen
Q = Rated flow
V_0 = Approach velocity
b = Bar width
a = Space between bars

K_1 = Coefficient related to the partial clogging of the screen:
no automatic raker 0.20-0.30;
automatic raker with hourly programmer 0.40-0.60;
automatic raker plus differential pressure sensor 0.80-0.85
α = Angle of the screen with the horizontal

For computing head losses in clean trashracks, the Kirscmer formula, detailed in Chapter 2, section 2.2.2.1, is commonly used. This formula is only valid when the

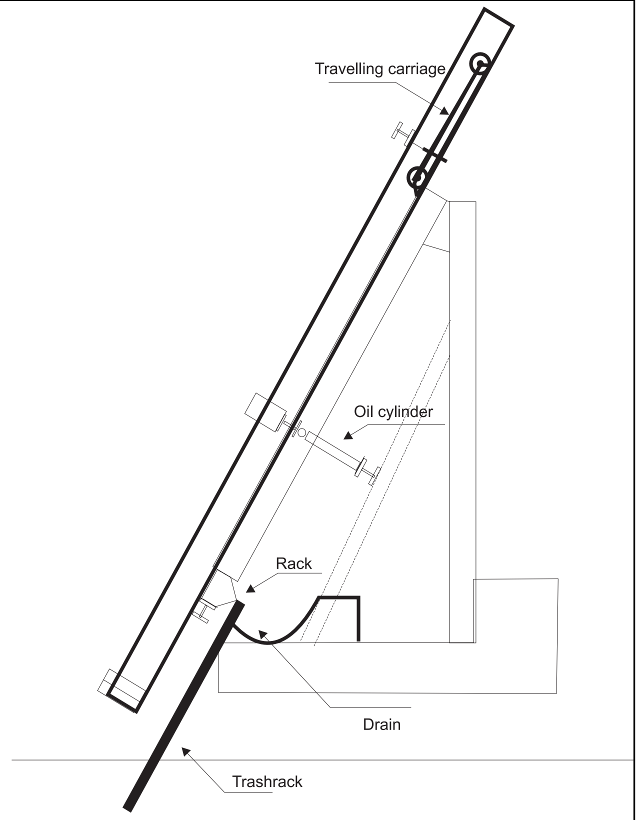


figure 5.21

flow approachs the screen at right angles. Otherwise the head losses increase with the angle, and can be up to 18 times the value computed by the Kirchsmer formula. The additional head loss can be computed by the formula

$$h_\alpha = K_2 \left(\frac{b}{a}\right)^{4/3} \frac{V_0}{2g} \sin\alpha \qquad (5.5)$$

where h_α is the head loss in m, α angle between the flow and the perpendicular to the screen (α max = 90º for screens located in the sidewall of a canal) and V_0 and g there are the same values as in the Kirschmer formula. If the flow is not perpendicular to the screen it is preferable to use round bars instead of profiled wire. Anyhow it is more important to keep the screen free of clogging because the head loss computed by the above formulae is insignificant when compared with the headloss arising from a partial clogging of the screen.

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 30º 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 5.21 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 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.

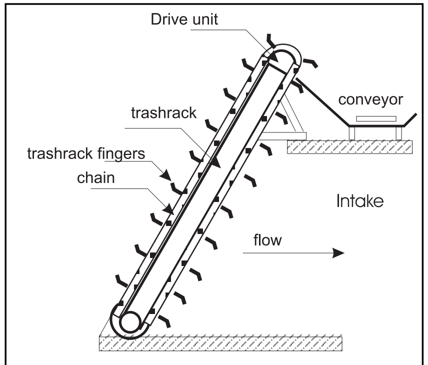


figure 5.22

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 5.14).

A less common type is represented in figure 5.22. A hydraulic driven chain system pulls some steel fingers through the trashrack. The fingers, at the upper travel position dump the collected trash to a conveyor belt for

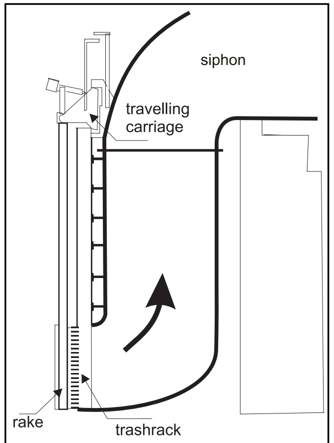


figure 5.23

Photo 5.14

automatic removal

The figure 5.23 illustrates a very particular raker located at the entrance of a siphon intake in "Le Pouzin" reservoir [12]. Initially no automatic raker was foreseen because the screen was located very close to the spillway and the plant was attended. The bars were placed horizontally and it was assumed that the flow would deal with the trash easily. However it was observed that the trashrack was clogged too often and a special horizontal raker was designed. The raker begins its cleaning movement upstream and moves downstream so the spillway flow contributes to cleaning it. An electrically-propelled carriage moves the raker and the approach action is provided by an endless screw.

5.2.3.2 Sediment management in intakes

Location of intakes, as detailed in 5.2.1.2 is particularly important in this respect. Open channels have a tendency to deposit sediments on the inner sides of bends, but when the intake is located at the outer side of the bend floodwaters may damage it. To overcome this problem, the best solution is to locate the intake structure in a relatively straight section of the river. Design of an intake for sediment exclusion can be adverse for other purposes such as fish protection. For example limiting the velocity at the screen approach to permit small fish to escape can result in deposition of sediments, up to actually blocking the entrance. Locating the intake entrance on a non-eroding bedrock streambed would prevent entrance of the sediment but the

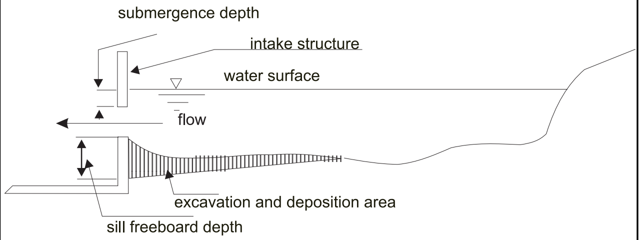


figure 5.24

construction costs will be increased. Figure 5.24 shows the invert of the intake sill raised above the river bottom to reduce the inclusion of bed load and heavy suspended materials near the bottom. The intake sill is kept off the river bottom to avoid the sliding of the sediment along the bed. Using the spillway to entrain the sediments that otherwise would cumulate in front of the intake is a good management technique.

When significant quantities of suspended sediments are expected to enter the intake large-size particles must be removed, using a sediment-excluding structure. The sediment-trap can be located immediately downstream of the intake, where the flow velocity is reduced. Well designed it should be able to remove all particles over 0.2 mm and a considerable portion of those between 0.1 and 0.2 mm. Such a structure is essential for heads over 100 m. A good example of a sediment-trap with an appropriate purging system and sufficient deceleration is shown in Fig 5.25.

Recently new sediment sluicing system which minimises the sluicing time and the wasted water has appeared in the market. One of these, the SSSS (Serpent Sediment Sluicing System) has been described in detail in the issue 9 –spring/summer 1993- of ESHA Info.

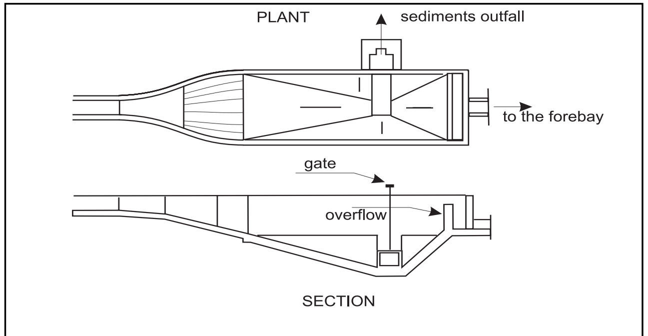


figure 5.25

5.2.3.3 Gates and valves

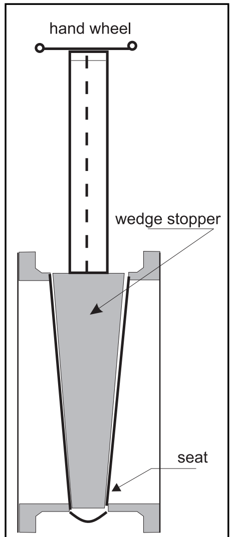


figure 5.26

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 - should be temporary 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 open 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 5.15

Photo 5.16

When used in a high-pressure conduit the water pressure that force the stopper against its seat makes the valve difficult to operate. This difficulty is overcome with a wedge-shaped stopper (figure 5.26), 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[13]. They can be made of natural rubber, styrene-butadiene or chloroprene compounds. The seal path is located adjacent to the roller path.

Small sliding gates controlling the flow can be raised by using either a wheel-and-axle mechanism (Photo 5.15), a hydraulic cylinder (Photo 5.16) or an electric actuator on a screw thread.

In butterfly valves a lens shaped disk mounted on a shaft turns to close the gap (figure 5.27). 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. Butterfly valves are simple, rugged and uncomplicated and can be operated manually or hydraulically. Photo 5.17 shows a large butterfly valve being assembled in a powerhouse and photo 5.18 shows a butterfly valve, hydraulically operated, with an ancillary opening system and a counterweight, at the entrance to a small Francis turbine.

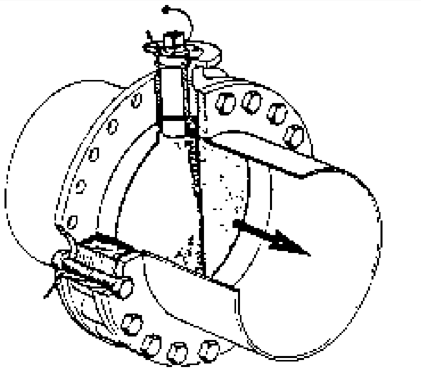


figure 5.27

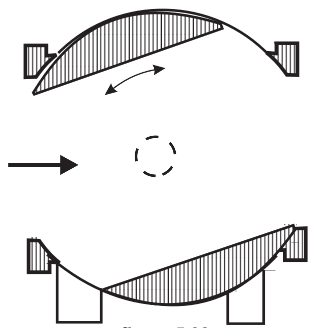


figure 5.28