Civil works for micro hydro power units

Compiled by

Christian Ardüser

Leif Karcheter

University of Applied Sciences
Northwestern Switzerland

Imprint

Authors:

Christian Ardüser, dipl. Bauing. FH

Leif Karcheter, dipl. Bauing. FH

Editing:

Prof. Dr.-Ing.Peter Gonsowski, Catherine Schultis

n|w University of Applied Sciences Northwestern Switzerland
School of Architecture, Civil Engineering and Geomatics

Institute of Civil Engineering, Gründenstrasse 40
CH-4132 Muttenz

T: +41 61 467 42 42
F: +41 61 467 46 10

bauing.habg@fhnw.ch

Supported by:

REPIC

Interdepartmental Platform for Renewable Energy and Energy Efficiency Promotion in International Cooperation www.repic.ch

REPIC is a joint initiative of

Schweizerische Eidgenossenschaft
Confédération suisse
Confederazione Svizzera
Confederaziun svizra

Swiss State Secretariat for Economic Affairs SECO

Swiss Agency for Development and Cooperation SDC

Swiss Federal Office for the Environment FOEN

Swiss Federal Office of Energy SFOE

Disclaimer:

Contents

Preface

This handbook was primarily composed due to the first international training on ***community based micro hydro power units*** in Sankoo, Ladakh September 2009. Experience had shown, that plenty of MHPU's had not achieved its lifetime expectation due to poor design mainly caused by lack of knowledge.

Since every potential site is different of any other, the engineer in charge will face many situations which could hardly be covered by a publication like this. However, the intention of this document is to promote the appropriate knowledge in the field of civil and hydraulic engineering. Therefore, the basic hydraulic principles are introduced to understand the mathematics behind the formulas. Basic flow and flood prediction methods are presented in order to find the calculation parameters for the hydraulic structures. Further, the main components of a micro hydro power plant such as intake, sand trap, forebay tank, penstock and supports are introduced. All designing and calculation approaches are accompanied by many drawings, examples and case studies for better education.

The present edition of this book has been adapted to the problems we had faced during our cooperation with the Ladakh ecological development group (LEDeG) and Bremen Overseas Research & Development Association (BORDA) in Ladakh, Nord India from 2007 to 2009.

Our special acknowledgements go to Prof. Dr.-Ing. Peter Gonsowski who was supporting us with his many years of experience on all technical issues, Prof. Dr. Dieter Mutz for his mentoring and enthusiasm during the whole project. Also we would like to announce our gratitude to Mrs. Catherine Schultis who edited the manuscript and dealt with the linguistically difficulties. Thanks are also due to Isabella Risorgi and Regula Ardüser for their valuable support.

Units and symbols

The following definition of units used in this handbook conforms to the *International System of Units* (abbreviated SI from the French Le Système International d'Unités) which is the modern form of the metric system. Generally, the system is devised around the convenience of the number ten.

SI base units:

Symbol	Meaning, Quantity	Unit	Notation
L	Length	m	metre
M	Mass	g	gram
T	Time	s	second
I	Electric current	A	ampere
T	Thermodynamic temperature	K	Kelvin

In some cases a prefix may be added to a unit to procedure a multiple of the original unit.

Symbol	Prefix	Factor	Scientific notation
p	pico	0.000’000’000’001	10^{-12}
n	nano	0.000’000’001	10^{-9}
μ	micro	0.000’001	10^{-6}
m	milli	0.001	10^{-3}
c	centi	0.01	10^{-2}
		1	10^{0}
h	hecto	100	10^{2}
k	kilo	1‘000	10^{3}
M	mega	1’000’000	10^{6}
G	giga	1'000'000’000	10^{9}
T	tera	1'000'000'000’000	10^{12}

Some of the dilated symbols and units which are often used in the context of civil and hydraulic engineering:

Symbol	Meaning, Quantity	Units	(Prefix) Notation
A	Area, cross section	m^2	square metre
a	Acceleration	$\mathrm{m/s}^2$	metre per square second
a	Wave velocity	$\mathrm{m/s}$	metre per second
c	Correction factor	-	dimensionless
d	Diameter	m	metre
DN	Diameter nominal	mm	(milli) metre
E	Elastic modulus	$\mathrm{kN/m}^2$	(kilo)Newton per square metre
E	Energy	$\mathrm{Ws, kWh}$	Watt second, (kilo) Watt hour
e	Present vapour pressure	$\mathrm{mbar, hPa}$	(milli) bar, (hecto)Pascal

e	Eccentricity	m	metre
e_w	Saturation vapour pressure	$\mathrm{mbar, hPa}$	(milli) bar, (hecto) Pascal ≈ 100 Pa
F	Force	N	Newton
f	Friction factor	-	dimensionless
F_r	Froude number	-	dimensionless
g	Acceleration due to gravity	$\mathrm{m/s^2}$	metre per second
h	Head, height	m	metre
h_E	Energy head	m	metre
h_{pr}	Height of precipitation	mm	(milli) metre
h_f	Head loss	m	metre
I	Moment of inertia	$\mathrm{m^4}$	metre to the power of 4
i_{pr}	Intensity of precipitation	$\mathrm{mm/h, mm/d}$	(milli) metre per hour, (milli) metre per day
k	Absolute roughness	mm	(milli) metre
k_{st}	Strickler coefficient	$\mathrm{m^{1/3}/s}$	metre to 1/3rd per meter
L	Length	$\mathrm{m, km}$	metre, (kilo) metre
n	Manning coefficient	$\mathrm{s/m^{1/3}}$	second per metre to 1/3rd
P	Power	kW	(kilo) Watt
p	Pressure	$\mathrm{N/m^2, Pa}$	metre per square metre, Pascal
Q	Volume flow rate	$\mathrm{m^3/s}$	cubic metre per second
q	specific flow rate	$\mathrm{m^3/s\ m^{-1}}$	cubic metre per second and meter
q	Load per unit length	$\mathrm{N/m}$	Newton per metre
R	Hydraulic radius	m	metre
Re	Reynolds number	-	dimensionless
RH	Relative humidity	$\%$	percent
S	Slope	$\mathrm{m/m}$	metre per metre
T	Temperature	$\mathrm{°C, °K}$	degree Celsius, degree Kelvin
T	Time interval	$\mathrm{s, min, h, d}$	second, minute, hour, day
T_{pr}	Time of precipitation	$\mathrm{min, h, d}$	minute, hour, day
U	Wetted perimeter	m	metre
V	Volume	$\mathrm{m^3}$	cubic metre
v	Velocity	$\mathrm{m/s}$	metre per second
w	Height of weir crest	m	metre
W_b	Body weight force	N	Newton

Creek symbols:

Symbol	Meaning, Quantity	Units	(Prefix) Notation
α_s	Flow reduction coefficient	-	dimensionless
ζ	Coefficient for local losses	-	dimensionless
η	Efficiency factor	-	dimensionless

μ	Dynamic viscosity	$\mathrm{kNs/m^2}$	(kilo)Newton second per square metre
μ	Weir shape coefficient	-	dimensionless
μ_k	Coefficient of kinetic friction	-	dimensionless
μ_s	Coefficient of static friction	-	dimensionless
ν	Kinematic viscosity	$\mathrm{m^2/s}$	square metre per second
ρ	Density	$\mathrm{kg/m^3}$	(kilo)gram per cubic metre
π	Ratio of a circle's circumference to its diameter	-	dimensionless
α	Coefficient of linear expansion	-	dimensionless

Greek alphabet:

Symbol (Capital, lower case)	**Name**
A, α	Alpha
B, β	Beta
Γ, γ	Gamma
Δ, δ	Delta
E, ε	Epsilon
Z, ζ	Zeta
H, η	Eta
Θ, θ	Theta
I, ι	Iota
K, κ	Kappa
Λ, λ	Lambda
M, μ	Mu
N, ν	Nu
Ξ, ξ	Xi
O, o	Omicron
Π, π	Pi
P, ρ	Rho
Σ, σ	Sigma
T, τ	Tau
Υ, υ	Upsilon
Φ, φ	Phi
X, χ	Chi
Ψ, ψ	Psi
Ω, ω	Omega

1 Energy from Water

Water is mostly underused as an energy source and is available in many places. Water recourses fluctuate over the year, therefore the associated production of energy varies. Flowing water contains potential energy which dissipates through the run-off in watercourses. This very capacity is used by the production of energy through hydropower. In technical terms the used difference of level is called head. Depending on head and flow rate, mechanical as well as electric energy can be generated.

1.1 What is Energy

Generally speaking, energy results from the weight of a body which is lifted to a certain height. In this case, it is water which is resides at a certain level (potential energy). The height is defined by the difference between the water level in the forebay tank and the axle of the freely suspended turbines (Pelton, Crossflow) respective to the after-bay gauge in the dammed turbines (such as Francis and Kaplan). Dammed means, in this case, that the water level in the after-bay lies higher than the axle of the turbine.

Kinetic energy is released when the lifted body is disengaged and drops to the ground. In our scheme, this is the water which runs through the penstock on to the turbine. The stream energy of water is then diverted by the bent turbine blades and is turned into a rotary motion and therefore into mechanical energy.

To create energy out of water two main factors are required:

- Q = Flow rate [m³/s]
- h = Gross head [m]

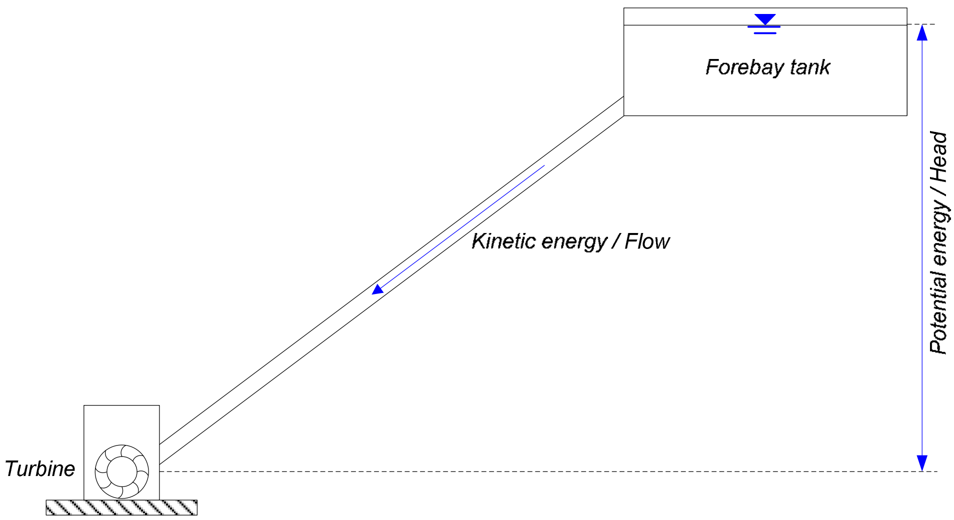


Figure 1.1 The Use of Hydropower

The weight of running water is a downwardly vectored power. The potency is due to the mass M and the acceleration due to the gravity g. Along with the vertical difference of level (head = h_{gross}) energy is calculated as follows:

$$E_{gross} = M \cdot g \cdot h_{gross} \text{ [Ws]}$$

The mass of water is due to the volume V and the water density ρ.

$$E_{gross} = V \cdot \rho \cdot g \cdot h_{gross} \text{ [Ws]}$$

The amount of water added to the turbine is measured in volume per time unit and is called flow rate Q. As released energy per time unit equals to the capacity in watts, the formula has to be rewritten.

$$P_{gross} = Q \cdot \rho \cdot g \cdot h_{gross} \text{ [W]}$$

Because of different losses, the energy generated by the turbine is less than the calculated gross output. Losses arise from friction in the penstock and turbine. When energy is provided to the consumer, additional losses occur at the generator and through the transmission wire. Thus a hydropower plant produces roughly half of the calculated gross output. Therefore one speaks of an overall efficiency η of a plant which varies in most instances between 0.5 and 0.7.

$$P_{net} = \eta \cdot Q \cdot \rho \cdot g \cdot h_{gross} \text{ [W]}$$

The water density is assumed to be approximately 1000kg/ m^3

$$P_{net} = \eta \cdot Q \cdot 1000 \cdot g \cdot h_{gross} \text{ [W]}$$

$$P_{net} = \eta \cdot Q \cdot g \cdot h_{gross} \text{ [kW]}$$

If the capacity of an installation needs to be roughly calculated, an efficiency of $\eta = 0.5$ and acceleration due to gravity of $g = 10\ m/s^2$ is set. In this way can one obtain fairly good data to estimate on-site whether an installation is feasible or not.

$$P_{estimate} = 0.5 \cdot Q \cdot 10 \cdot h_{gross} \text{ [kW]}$$

1.2 Water

Water (H_2O) is a chemical compound of the elements oxygen (O) and hydrogen (H). The term water is primarily used for the fluid condition of aggregation. In a solid state (frozen) one speaks of ice, in a gaseous state it is called steam. Water is the only chemical compound on earth which occurs in all of the three conditions of aggregation.

The characteristic or the ionic form of water has an influence on the design of the different structures of hydropower plants. Disregarding it can cause serious damage.

Ionic Form of Water

Fluid: Within the scope of using hydropower, the stored energy is primarily used in running water. A prime example is a flour mill which is powered by a water wheel. This method, used for hundreds of years, has been upgraded in the past 200 years and can now highly engineered produce electrical power.

Vaporous: When water is heated at an ambient pressure of 1.0 bar (sea level) to 100° Celsius, it starts to boil. Feeding water with heat energy through additional heating power will lead to vaporization. On top of the Mount Everest, 8848 m above sea level, the ambient pressure is much lower (0.4 bar) thus water starts boiling at 75° Celsius. Vapour pressure turbines can be powered by means of water vapour. This is an alternative method of producing mechanical energy and therefore of the production of electricity.

Example: in Ladakh, at an average height of 3500 m above sea level, water starts boiling at 90° Celsius.

Frozen: At a temperature of 0° Celsius, water starts to transform into a crystalline form. In colloquial language, this crystalline form is called ice. The density of ice (916.8 kg/m^3) is less than that of water (1000 kg/m^3) and causes the ice to swim on the water surface.

Icing leads to serious damage to constructions. Drifting ice which is washed up in water bodies can seal inlets and spill into the main construction and erode the foundations. Water transforming into ice expands and can consequently burst penstocks filled with stagnant water. Also, equalizing reservoirs which do not have moving water can break due to the outward vectored power. Mechanical parts such as sliders can becoming frozen and are therefore no longer functional. In case of an emergency, the system cannot be shut down and the generator can overheat or the turbine be destroyed.

1.2.1 Transport of solid matters

When using a turbine, water quality becomes very important. Fouling and all types of solids can lead to damages to constructions. Specific structural measures can minimize these damages.

A distinction should be drawn between bed load, floating matters and suspended load:

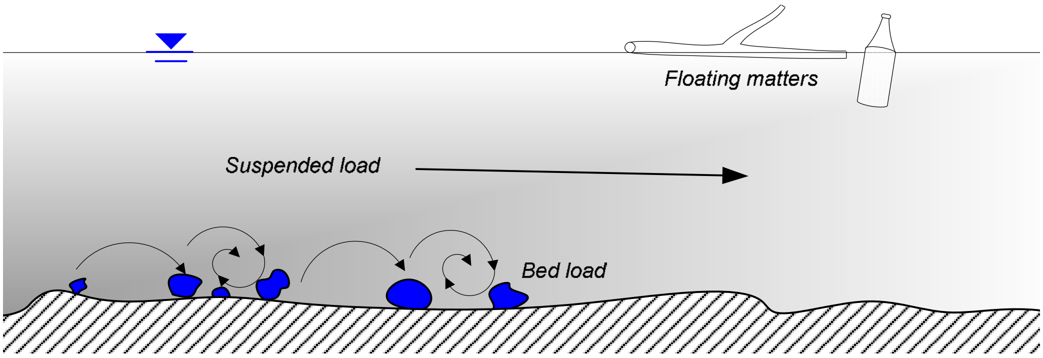


Figure 1.2 Transportation of solid matters

Bed load: The higher the flow rate, the stronger the power which affects the river bed, thus the larger the grain fraction which can be transported. The spectrum varies from grains of sand to pieces of rock during flooding. This transport mechanism results in the rolling motions of grains.

Larger pieces of rock can cause severe damage at intake through impact; furthermore, the inlet can become blocked by sediments. Smaller grains can reach the turbine shell through the channel and the penstock and so cause abrasion on mechanical parts. This situation must be prevented at all costs; otherwise, the longevity of the construction can be dramatically reduced.

Suspended load: This refers to diminutive mineral size fractions (sand and silt) that disperse from the ground due to turbulence and are brought into motion. Scooping clear water out of cloudy water (containing silt) and putting it into a bucket leads to a slow deposit of these solids on the ground within several days. After this, the clean water can be skimmed at the top.

Also, these small solids have to be removed from the water which is used for the turbine. Otherwise, damage can lead to the breakdown of these machines and enormous expenses arise from servicing the plant.

Floating matters: A distinction is drawn between natural floating solids and waste products from civilization. All these items, such as trees, branches, shelves, leaves, plastic, tins, bottles and so on, swim on the surface. Drifting cadavers are called floating solids as well.

When such solids reach penstocks, they produce a seal and thus a depression that leads to the demolition of the pipe. Also intakes and weirs can be blocked. Hence the use of a trash rack is essential.

1.3 Types of power plants

There are many different types of hydropower plants. They are categorized by the height of the head or by storage capability.

The height of the head h is the level difference between the water level above the turbine (forebay) and the water level after the turbine (afterbay). A distinction is drawn between three levels of pressure:

- High-pressure plant $50 < h > 2000$ [m]
- Medium-pressure plant $15 < h < 50$ [m]
- Low-pressure plant $h < 15$ [m]

In the case of storage power stations, the water is collected in a basin called ***reservoir***. This reservoir is either of natural origin or is created through impounding with a concrete dam or a retaining dam. The storage lake is fed by a natural inflow. With a storage power station, the water is stored over a specific period (several hours to months) so that electricity can be produced selectively when required. With a river power plant, the river is impounded and electricity is produced continuously by the running of the water.

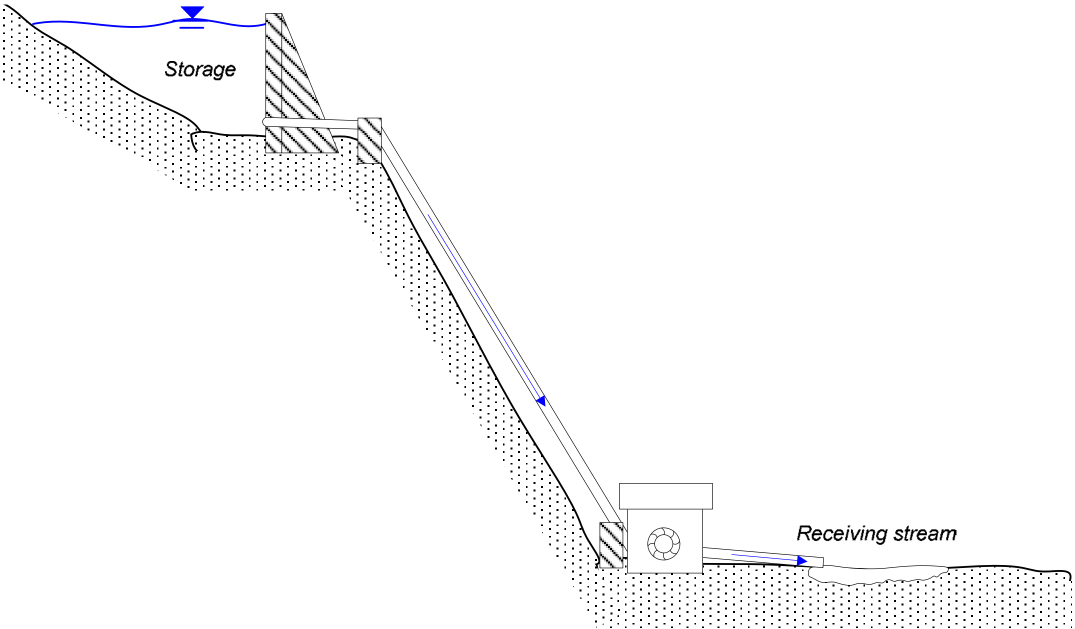


Figure 1.3 High-pressure hydro scheme with storage

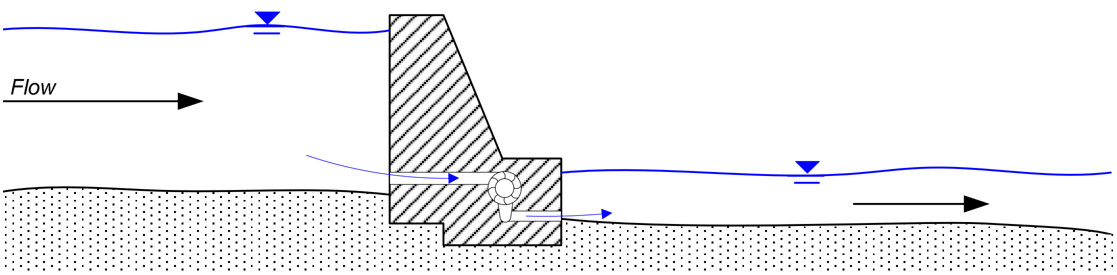


Figure 1.4 Low-pressure run-of-river hydro plant

High-pressure hydro schemes with storage are very common in combination with low-pressure run-of-river hydro plants as these types of construction are easily imbedded in the environment and are very cost effective.

Components of a hydropower plant

The constructional building of a hydropower plant is strongly reliant on the characteristics of the particular locations. Each construction has different basic conditions. The following illustration schematically illustrates which buildings are required to create a functional small hydropower plant. Generally small hydropower plants are established as either medium- or high-pressure plants. That is to say, a comparatively small amount of water produces electricity using a penstock and a high head.

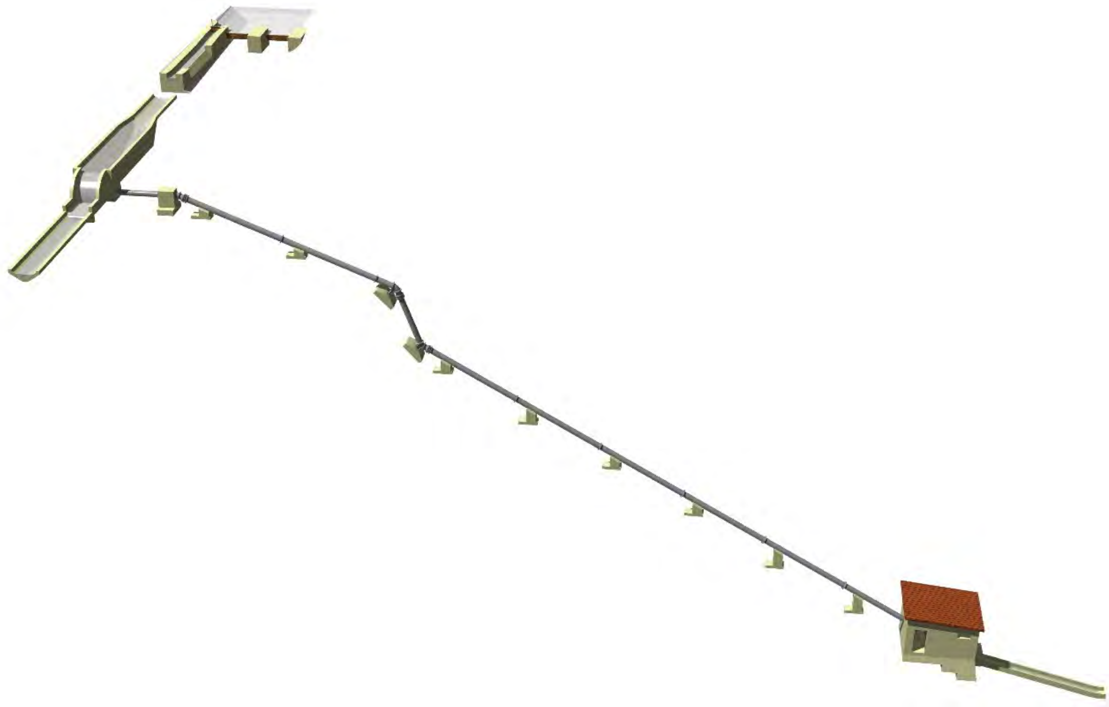
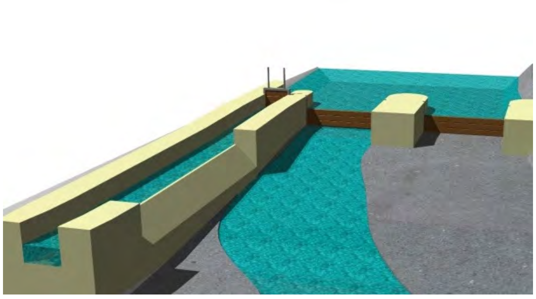


Figure 1.5 Major components of a micro-hydro scheme

To ensure that all the required water can be diverted out of the river into the open channel, a weir is needed. The water has to be guided through an opening in the river bank. This is called the intake opening or intake structure. Since silt and sand is bad for all mechanical devices, a settling basin is used. To preserve the elevation of the diverted water, the channel must follow the contour of the hillside with a gentle slope. At the end of the channel, the water enters the forebay tank and passes into the penstock. This pipe guides all the water to the lower positioned turbine. The shaft of the turbine wheel can rotate either mechanical devices or electrical generators. There are many different variations. For example, the channel can be eliminated and the penstock directly connected to the first settling basin. The design of the plant depends on the characteristics of the particular site.

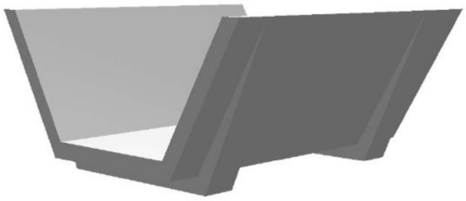
The functions of the construction required for a small hydro power plant are exemplified in the following chapter. The detailed mode of operation and the project planning is dealt with in later chapters.

Intake



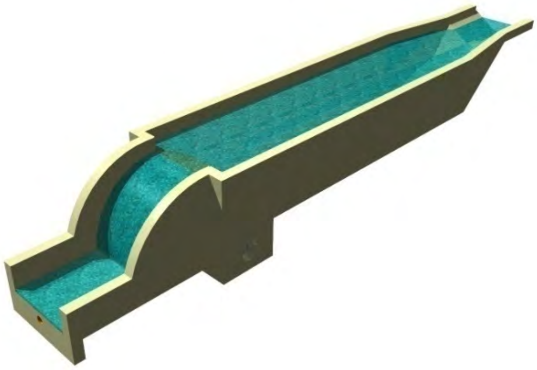
The intake is built directly in the river. A weir built crossways to the flow direction dams the water. This slack flow allows a regulation of the water conducted to the canal. In dry phases all the water can then be diverted directly into the canal. If there is too much water, however, the intake of the canal can be throttled. The extra water flows along the weir. In this way, one can avoid too much water entering the canal and possibly demolishing constructions. With strongly polluted water, a desilting basin must be arranged directly after the intake.

Channel



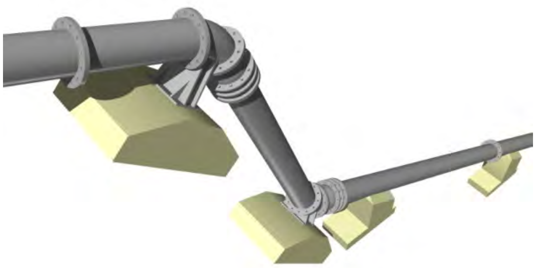
The canal leads the water from the intake to the forebay. It can be paved, concreted or constructed as an earth channel.

Desilting basin / forebay tank



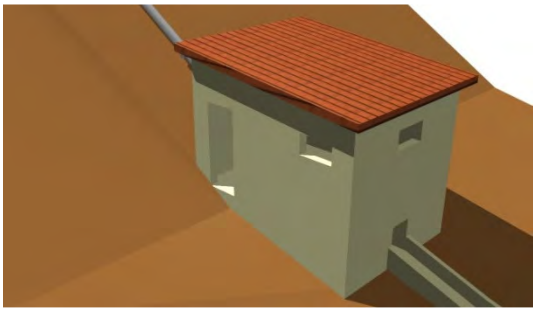
The assignment of the desilting basin (sand trap) is to settle out the particulate matters floating in the water to the bottom of the construction. The water which is used for the turbine can therefore be separated from these solids. Otherwise they will end up in the penstock and in the turbine which can lead to serious damage. The water volume of the forebay is made for balancing the variations of the water gauge while operating the tubine. In this example, the balancing basin is combined with the desilting basin. Therefore, the water volume of the desilting basin can be counted as that of the forebay tank, which leads to savings on construction material, because only one construction must be built.

Penstock



In the penstock, the water pressure is built up on the turbine. The water flows out of the forebay tank through the penstock directly on to the turbine. The water gauge in the forebay tank controlls the head.

Powerhouse and turbine



The turbine and the equipment required for the production of electricity are located in the powerhouse and therefore are protected from rain and other factors. The turbined water is subsequently restored in an open canal to the stream. The produced electricity is carried to the consumer using the transmission line.

Notice: No even two hydro plants are even precisely the same!

1.4 Losses

To calculate the effective capacity of a power plant, two factors are required, flow rate and head. What one must not disregard is that each element of the construction reduces capacity. Between the time when the water is extracted from the stream until the consumer receives the electricity, a substantial part of the power is converted to heat. This happens through friction between two different kinds of solids or prefabricated parts, for example:

- Running water in a steel penstock generates wall friction = heat
- When water hits the turbine, friction is excited = heat
- Rotary motion of the turbine shaft in the ball bearings produces friction = heat
- Power generation through rotary motion in the generator = heat
- The transmission of electricity through the transmission line simply said produces friction = heat
- Exception: The power reduction in the canal results from loss of water, which means the flow rate is fractionally reduced through evaporation, percolation (leak) or drainage.

Simply said, prefabricated parts that hold an added heat induce a great deal of friction. Therefore they are responsible for a high loss of power. Proper maintenance of a construction can avoid additional reduction of power (e.g., regular greasing of ball bearings).

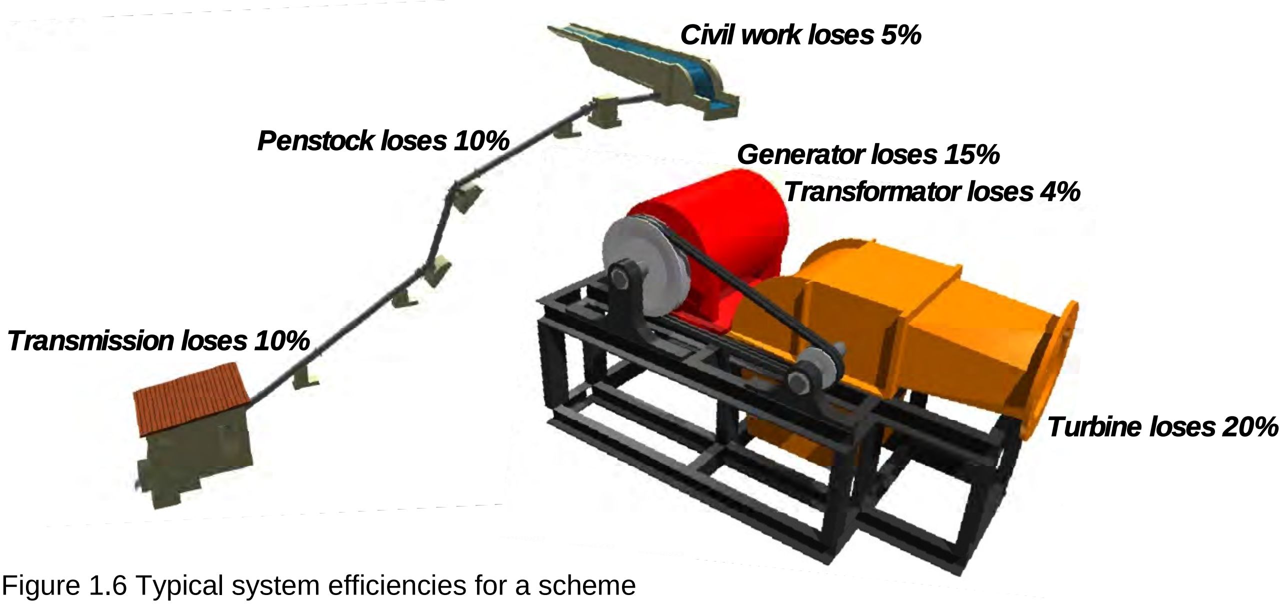


Figure 1.6 Typical system efficiencies for a scheme

The listed efficiencies, shown in the illustration above, are merely standard values. Some of them must be calculated on their own; others depend on the prefabricated parts (generator, turbine) and must be requested from the producer.

The different partial efficiencies are multiplied and added to equal the overall efficiency η.

According to the illustration the overall efficiency must be

$$\eta = \eta_{civil\,works} \cdot \eta_{penstock} \cdot \eta_{turbine} \cdot \eta_{generator} \cdot \eta_{transformator} \cdot \eta_{transmission\,line}\ [-]$$

$$\eta = 0.95 \cdot 0.9 \cdot 0.8 \cdot 0.85 \cdot 0.96 \cdot 0.9\ \ [-]$$

$$\eta = 0.5\ \ [-]$$

and therefore

$$P_{net} = \eta \cdot P_{gross}\ [\mathrm{kW}]$$

$$P_{net} = 0.5 \cdot Q \cdot g \cdot h_{gross}\ [\mathrm{kW}]$$

Example: power equation

Example 1: You are on a possible site and have to design a micro hydropower unit which must supply 40kW to a remote village. There is a potential slope which has a difference in height of 20 metres. Roughly how much water is needed?

$$P_{net} = \eta \cdot Q \cdot g \cdot h_{gross}$$

To find out the required flow the power equation must be changed:

$$Q = \frac{P_{net}}{\eta \cdot 10 \cdot h_{gross}} = \frac{40\ kW}{0.5 \cdot 10 \cdot 20\ m} = 0.4\ \left[\frac{m^3}{s}\right]$$

Flow is often measured in litres per second. In this case, it is important to change to the standard unit. In this handbook all the equations make use of System International (SI) units. Ensure that the correct units are always used when working with an equation from this handbook.

Example 2: You did flow measurements in a small stream near a feasible site. The head of 90 feet is measured roughly using a water-filled tube. The flow in the stream is more than 160 litres per second. How much power can be delivered to the village?

To ensure that the units are correct, use only SI units. Flow is given in m^3/s and head is listed in meters.

90 feet is about 27.5 metres (exact: 100 ft = 30.48 meters) and 160 l/s is 0.16 m^3/s.

$$P_{net} = 0.5 \cdot 0.15 \cdot 10 \cdot 27.5 = 22\ [\mathrm{kW}]$$

2 Baseline study

The idea behind a baseline study is to find out if a proposed site is socially or technically feasible or not. The survey should be used as an indicator which justifies the subsequent feasibility study, which will be time-consuming and costly. The survey can be done in one or two days, mostly in the field.

A baseline study is to clarify what the energy demand of a village community is. Additionally, it must project how the community will function in 5 to 10 years. Electricity brings progress and, therefore, potential for development (sawmills, radio and other technical devices). This will increase the demand for electricity. In the baseline study, this calculated demand will be compared with the capacity of a realizable hydropower plant on site. If it is not possible to provide sufficient electricity, the project will not be pursued.

In this chapter, we focus primarily on the technical feasibility of a baseline study. While the social aspect is not represented here, it is an essential component in building a successful, technically-practical hydropower plant. This should always be kept in mind.

2.1 Social survey: Study of potential user group

To discuss the capability of a community to operate and administer such an installation, it is essential that the social structure is examined. Such study will also allow us to gauge whether the community is able to partially finance the installation. Based on the layout of the village, the distribution of the electrical power can be predefined. If a village is sprawling, the grid is much more expensive than in a village where the buildings are grouped together. These and similar criteria must be evaluated based on the following points:

- Population and sex ratio
- Livelihood pattern
- Source of income
- Scope for electrification through grid
- Willingness to realize an MHPU
- Ability of contribution: Money, labour, material, etc.
- Present source of electricity
- Location, altitude and road connectivity of settlement
- Number of households
- Spread of households

Demand: Based on the number of houses and their occupants, how much electricity the village needs to cover their requirements must be extrapolated. Below are some standard values:

Light bulb (energy-saving):	20 W = 0.02 kW
Butter churn:	100 W = 0.1 kW
Carpentry machine:	3 kW
Storage water heater:	5 kW
Oil expeller:	3 kW
Flour mill:	3 kW

2.2 Technical survey: Study of potential energy supply

The technical part will be discussed in detail. The following points must be considered and precisely evaluated during a baseline study. The thereby determined data is the basis of information from which is decided whether the site is suitable for an MHPU.

- Find suitable site to set up an MHPU scheme
- Do water flow and land measurements
- Estimate potential hydraulic energy

Site survey / geological appearance

The plant sites cannot simply be chosen based on the optimal rate between flow and head. Knowledge of geology is required when exploring possible sites. A properly selected site increases the longevity of the hydropower plant. Structures cannot be built in hazardous areas nor on unstable subsoil. Sketches of the possible site must be made on site and these will function as references in the office. Do not forget to sketch and note all relevant geological information.

Slip zone / landslide:

A landslide is the downward movement of soil and rock mass where immense forces prevail. They can last from a few seconds (sudden/crop up) to years (predictable). These movements are caused by poor subsoil, varying soils and rocks which are not connected to each other, or by water flowing through the subsoil. Structures built in such locations could slide away and be destroyed. An indicator for slip zones are horizontal crevices in the hill. In slowly crawling areas, it seems as if the trees grow horizontally out of the ground and then straighten. A bent tree trunk is the consequence of the tree constantly adapting to the ground and then trying to continue grow normally, upwards toward the sky.

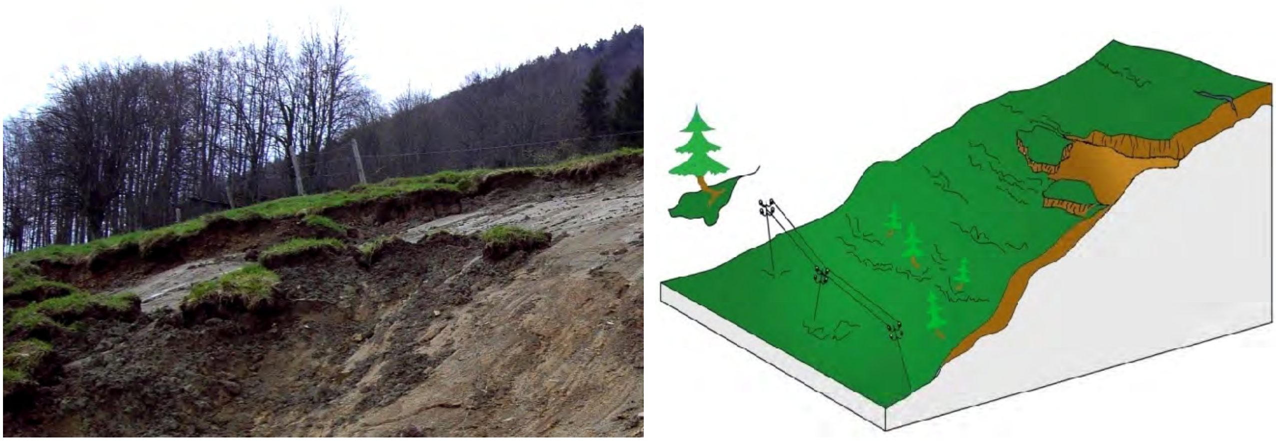


Figure 2.1 Landslide / schematic with curved trees, tilted poles and soil ripples in creep area

Storm gulley:

After or during extremely heavy rainfall, the ground cannot absorb all the water and the rainfall drains above ground. Mountain streams are suddenly formed in places where no water has flowed for years. These streams can carry large boulders and entire trees. A construction in this area will be destroyed and washed away within a short time. Such storm gullies, can be easily recognized. They usually look like a dry streambed in which no water has flowed for a long time; weak foliage flanks this streambed.

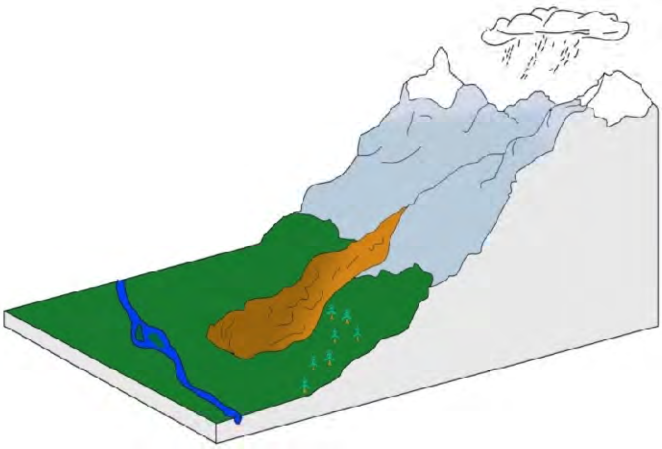



Figure 2.2 Actual storm gulley in the Himalaya / storm gulley schematic

Rockfall:

Brittle rock can break off unexpectedly and fall into the valley. Small stones, but also sizeable boulders can break off and can cause vast damage. These events usually occur in the colder seasons. Water flows into small crevices where it remains. When temperatures drop and the water freezes, its volume increases, since the volume of ice is greater than that of water. This process enlarges the crevice and can cause the boulder to break off, lose its hold and slide down the slope.

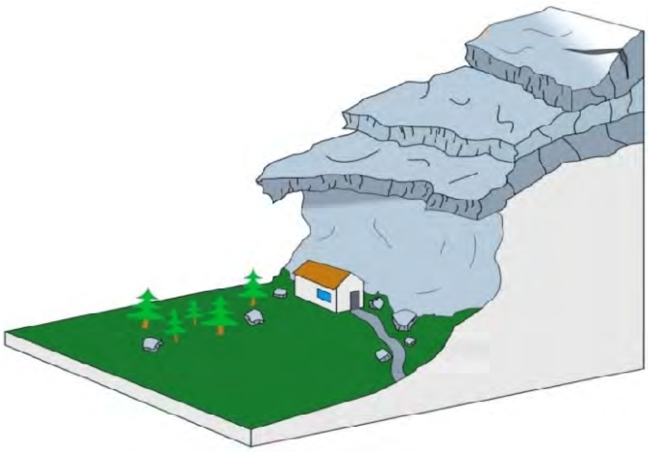



Figure 2.3 Rockfall schematic / large boulders on a road

Flood plain:

A seldom submerged zone close to the riverside is called the flood plain. Seldom means, in this case, every 20 years. But that does not mean it happens only every 20 years, this is an indicator for the amount of flood discharge.

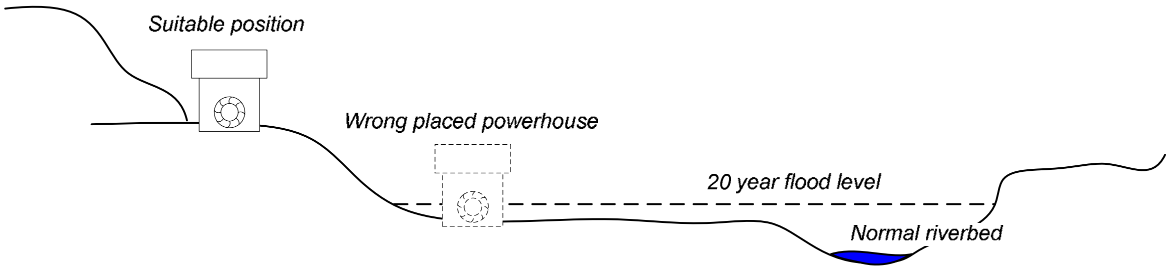


Figure 2.4 Wrongly placed powerhouse in a 20 year flood plane

If there is a map available, it is possible to do some pre-investigation before you go to the site. At a low-level flood plain, the area is shown without contours. People who have lived in the area a long time should be able to give detailed information on floods and flood levels. Typical indications of regular floods will be seen on site: debris which lines the river banks away from the actual river, ponds, secondary water courses with or without water, low areas near the river, mud and sand deposits.

When designing a powerhouse, an adequate location must be chosen. Do not forget that a flood plain is a risky area: The path of the river can change, sediments and debris can accumulate, the subsoil is mostly unstable for placing foundations and, during a flood, the whole powerhouse can be flushed away.

The civil works have to be built above 20-year flood plain levels. Intakes have to be built strong enough to withstand a flood. To protect the intakes, use wing walls made of gabions or masonry.

Wetland:

The soil of wetlands is saturated with moisture either seasonally or permanently. This subsoil is not stable and causes severe subsidence of foundations and, therefore, of the whole buildings. Wetlands consist of bogs, marshes and swamps. These areas are covered partially or completely by fordable pools of water.

Survey of subsoil:

It is important to examine the subsoil during the investigation of possible sites. Stable rock and firm soil, such as well-bedded gravel, are the most appropriate subsoil's for foundations. All parts of a foundation must be laid on an equally firm base, to avoid differential settling of the building. During such investigations, it is recommended to dig inspection trenches at different spots.

The purpose of the needed soil and rock is for assessment: is the location suitable for placing foundations (settling) or is the existing soil, sand, gravel and rock useful as construction material?

Characteristics of rock and soil:

	Code	Identification	Usefulness
Rock: Weathering and decomposition	Solid	Survives hammer blow. Bright clean. Cracks more than 0.3 cm apart.	Good base for foundations. Boulders can be used as ballast in cement weirs and anchor blocks.
	Weak	Breaks under hammer. Cracks 5 cm to 30 cm apart.	Reasonable base for foundations. Loose or crumbly material must be removed. Good for aggregate in cement work.
	Ghost	Keeps form of rock but crumbles easily. Equivalent to soil.	Unsuitable for foundations. Drive piles through or dig for deep concrete work.

	Code	Identification	Usefulness
Soil: Firmness and cohesion properties when wet and when dry	Gravel/ Gravelly soil	Small to medium stones. Crunches under foot.	If gravel content very high, forms good base for foundations. Gravel itself very useful as an aggregate in concrete. Gravelly soil drains well. Can be used to fill drainage trenches.
	Sand/ Sandy soil	Small grains, sugar-grain sized. Each grain separable and visible. Cannot be moulded by hand when wet or dry	Not suitable for foundations since sand can flow and compress when wet and dry. Important construction material - mix with cement.
	Loamy soils	Tiny grains scarcely visible. Grates between teeth. Moulds slightly in the hand when moist but cracks easily. Take care to distinguish between clay and sand.	Not suitable for foundations. Avoid partial presence of loamy or sandy soils as a foundation to prevent differential settling. Drive piles or dig for deep concrete work. Can be deceptively stable when dry but very mobile when wet. Weakened by frost action.
	Clay/ Clay soils	Invisible particles. Moulds well when moist can be formed into a ribbon between the fingers. Difficult to break by hand when dry. Impermeable. Expands when wet and shrinks and cracks when dry.	Very useful as an impermeable sealing material for channels. Medium load-bearing capacity. Suitable as foundation base for light construction work if well-designed concrete footings are inserted into clay bed.
	Peat/ Organic soils	Brown or black. Can have rotting smell. Found in deposits in marshy areas.	Not useful for construction. Must be removed. Highly compressible and unreliable. Can be used as a combustion fuel and for cultivation when oxygenated and limed.

Table 2.1 Characteristics of rock and soil (according to Adam Harvey's MHPU manual)

Potential energy

When possible sites have been selected, their power can be calculated using flow rate and head. Due to the simplified power equation, this can be done on site. This simple equation should be memorized: it is at the heart of all hydropower design work!

Supply:

$$P_{estimate} = 0.5 \cdot Q \cdot 10 \cdot h_{gross} \text{ [kW]}$$

2.3 Analysis and decision-making

The possible sites are compared and evaluated based on a number of criteria. In many cases, the most efficient plant may not the most appropriate. Many indicators decide the feasibility of the location and should be known at this point during the baseline study:

- Relationship between flow and head = electric power
- Water supply = is there enough water available to operate the turbine during each season
- Subsoil = can stable foundations be built
- Geological influences = landslide, storm gully, rockfall and so on
- Flood level = is the proposed powerhouse in the 20-year flood area
- Land tenure = is the owner willing to offer his land to the community
- Distance to the village = the farther away, the more expensive the electrical wire and the greater the loss of power

Decision according the collected data:

At the conclusion of a baseline study, the demand and supply are compared, to determine whether the favoured location/plant can provide adequate electricity.

Demand; social survey		**Supply; technical survey**
Rough amount of needed electrical energy. All anticipated energy consumption must be listed. A total of needed power must be calculated.	**<**	The estimated power output of the suggested site must deliver an equal or greater amount of electricity than the needs of the population.

Site is not suitable = no more research is necessary: project ends.

Site is suitable = move on and do a feasibility study. Furthermore, implement the collection of long-term flow data, if no data is available

2.4 Standardised form for technical baseline study

Heading of the organisation who conduct the survey

Name of the surveyor: ______________________________

Date of the survey: ______________

About the village

Name of settlement: ______________

Next bigger town: ______________ District: ______________

Road connection: Yes ☐ No ☐

If No, distance from the village to the road head: ______________

Demand

Number of households: ______________ Average number of

Number of villagers: ______________ rooms per household: ______________

Livelihood: ☐ self-sustaining Amount in percent: ______________

☐ cash income Amount in percent: ______________

During which hours do they use electric light: ______________

Size of farmland: ______________ [km^2] (1 km^2 = 1000 m · 1000 m = 1000'000 m^2)

To irrigate 1 km^2 you need 500 – 1500 m^3 water per day, depends on the region (dry ore wet area)

To run the turbine, we use water from an existing irrigation channel: Yes ☐ No ☐

In which months do they use water for irrigation: ______________________________

Electricity consumers	Power		Number		Total
☐ Light bulb (saving)	0.02	[kW]		=	[kW]
☐ Light bulb (normal)	0.2	[kW]		=	[kW]
☐ Carpentry machine	3	[kW]		=	[kW]
☐ Flour mill	3	[kW]		=	[kW]
☐ Storage water heater	5	[kW]		=	[kW]
☐ Butter churner	0.1	[kW]		=	[kW]
☐ Oil expeller	3	[kW]		=	[kW]
☐		[kW]		=	[kW]
☐		[kW]		=	[kW]
☐		[kW]		=	[kW]
				Grand total	[kW]

Technical site survey

Site number: ______________________

Site name/description: ______________________

Owner: ☐ village (site belongs to the community)

☐ private name of private owner: ______________________

View of site:

	A	B	C	D	E	F	G	H
1								
2								
3								
4								
5								
6								
7								
8								

☐ rough scale (1 : XX)	☐ roads and footpaths
☐ north direction	☐ numbers of houses in every cluster/hamlet
☐ distance from powerhouse to the village	☐ position of proposed intake
☐ length of irrigation channel (if existing)	☐ geological observations

Main data

Measured flow in river: ____________ [m^3/s] Date of measurement: ____________

Flow in channel: ____________ [m^3/s]

Head: ____________ [m]

Potential capacity

$$P_{estimate} = 0.5 \cdot Q \cdot 10 \cdot h_{gross} \ [\mathrm{kW}]$$

$$P_{estimate} = 0.5 \cdot \_\_\_\_\_\_ \left[\frac{m^3}{s}\right] \cdot 10 \cdot \_\_\_\_\_\_ \ [m] = \_\_\_\_\_\_ \ [\mathrm{kW}]$$

Geological survey	Specification:	Sector on view:
Slip zone near the proposed buildings:	☐	
Storm gulley	☐	
Rock fall area:	☐	
Wetlands:	☐	
Flood plain/area:	☐	

Subsoil at the proposed location of buildings

	rock solid or weak	**gravel** compact	**clay** sealing material	**poor subsoil** loam, org. soil
Intake	☐	☐	☐	☐
Channel	☐	☐	☐	☐
Forebay tank	☐	☐	☐	☐
Penstock foundations	☐	☐	☐	☐
Powerhouse	☐	☐	☐	☐
Building material*	☐	☐	☐	

*It is necessary to observe the surrounding area to find out where you get good building materials and aggregates. Such as solid stones, gravel or sand (used for concrete).

Demand/Supply analysis

To define if a site is suitable or not, you must compare the demands of the village and the possible supply of the new site. You must define when the villagers need electricity and when they need water for irrigation. Usually irrigation has first priority, since it is basic to agriculture and the economic security of the village. Milling and lighting have the next priority.

Time for irrigation:

$$\frac{farmland\ area\ \cdot needed\ irrigation\ water\ \cdot security\ factor\ [2]}{flow\ in\ irrigation\ channel\ \cdot hour}$$

$$\frac{\_\_\_\_\_\ [km^2] \cdot \_\_\_\_\_\ \left[\frac{m^3}{km^2}\right] \cdot\ 2}{\_\_\_\_\_\ \left[\frac{m^3}{s}\right] \cdot 3600\ \left[\frac{s}{h}\right]} = \_\_\_\_\_[h]$$

Daily graph

m^3/s					kW
	12 midnight	6 am	12 noon	6 pm	

This site is technically feasible Yes ☐ No ☐

Signature of observer: ____________________

2.5 Case study: Technical part of a baseline study

As part of a developmental assistance project, the task is to implement a baseline study in a remote village, which, at the earliest, will be connected to the public electricity grid in 15 to 20 years. This is why an isolated micro hydropower plant makes sense. Based on your work, a decision will be made as to whether the project will be continued. To streamline the survey, use standardised forms.

Information about the village:

- 90 households with an average of 8 inhabitants
- At present, a road which will end in the center of the village is being built
- The village is split into two unequal parts, one with 70 and one with 20 households
- The farmers practice grain agriculture
- There are a handful of carpenters and many trees
- There is an upper and a lower water channel with different flow rates
- The community is willing to build and maintain a micro hydropower plant, partial financing is possible

Power requirements of the village:

- Five light bulbs per household (20 W/bulb)
- Two flour mills (3 kW/Unit)
- Two carpentry machines (3 kW/Unit)
- Three 100-litre storage water heaters (5 kW/Unit)
- 30 Household application (500 W/Unit)

Information's about the proposed sites:

The lower earthen channel is massively developed. It can transport at the maximum 180 litres of water per second. Twenty litres must be available in summer for irrigation and drinking water for a neighbouring group of houses. The lower channel is therefore in the public domain. In order to increase the head, the channel must be extended across the storm gully. The storm gully has not carried any water in the last 15 years, according to the inhabitants of the village. The upper channel, on the other hand, belongs to three farmers and is used for the irrigation of their fields. It needs to be clarified whether the owners are willing to provide the channel for development of an MHPU. The flow is 110 l/s and all water would need to be used to operate the turbine. In August, 350 l/s of water flow in the river, but the amount of water decreases drastically and, in winter, less than 120 l/s must be expected.

Site 1: If the upper channel can be used, the net head can be up to 120 m. During irrigation season, the available amount of flow might be reduced down to 50 %. The channel is passing a storm gully on a stretch of 50 m. There are cracks in the soil indicating hillsides below the second part of the channel and the channel has been fixed multiple times. The powerhouse is located in a floodplain where two rivers meet. The transmission line to the village must be 1080 m. The penstock will probably be 474 m.

Site 2: The assumed net head is 70 m at its maximum. The channel must be extended by 250 m and will pass the storm gully on a stretch of 20 m. The powerhouse can be built at a fairly protected site. Only 160 l/s can be used for the turbine. The transmission line to the village must be 1240 m. The penstock will probably be 276 m.

Site 3: The assumed net head is 55 m at its maximum. Some trees must be felled to be able to build the forebay tank. The powerhouse can be built behind a large boulder, almost one meter above the water level. To increase the flow, the channel must be widened at a length of 400 m. The transmission line to the village must be 860 m. The penstock will probably be 210 m.

General plan of location:

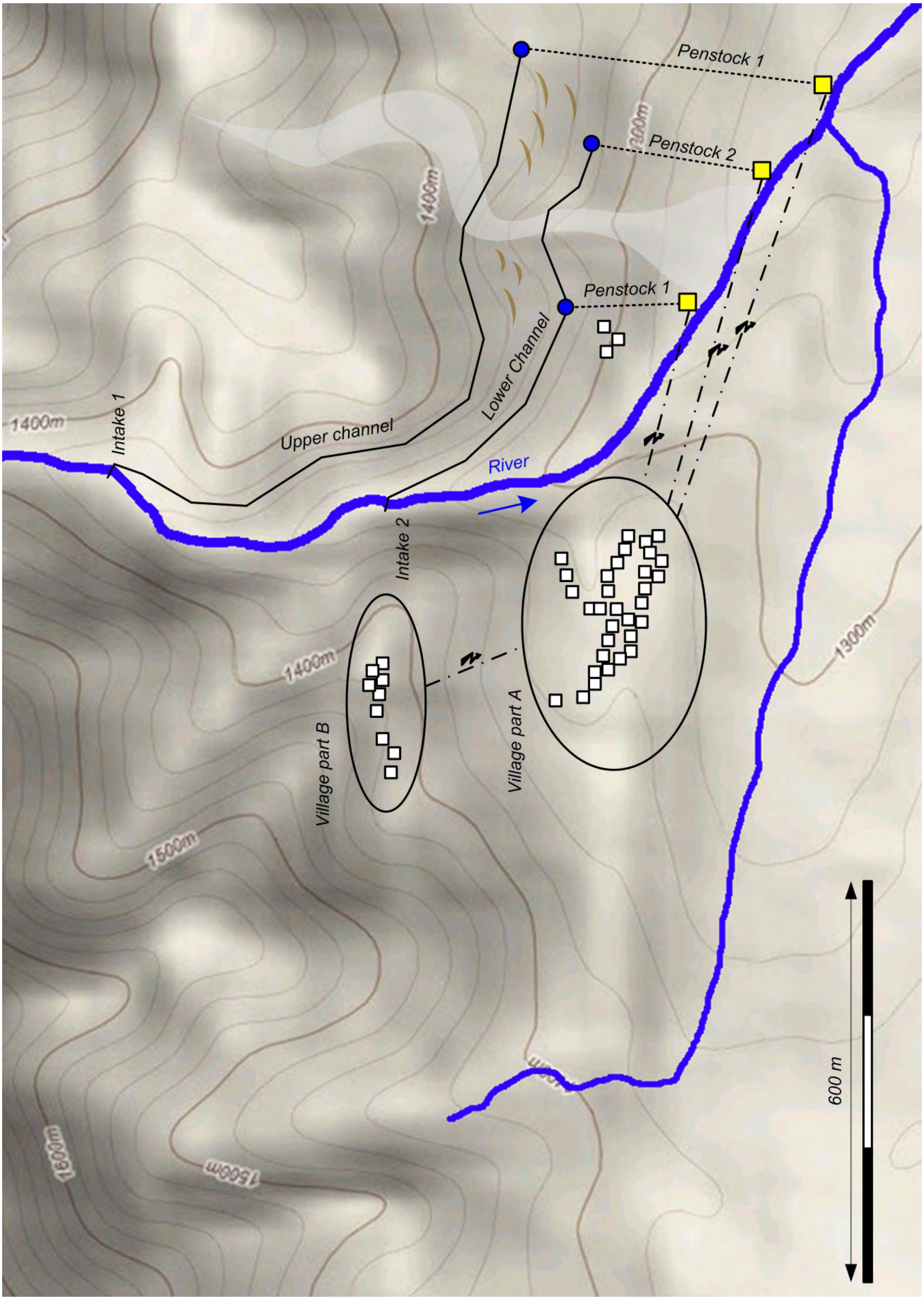


Figure 2.5 Plan of possible sites

Site analysis:

Estimate the output of all possible sites.

Site	Net head [m]	Volume flow rate [m^3/s]	Installed Capacity [kW]
1	120	**0.11**	**66**
2	70	**0.16**	**56**
3	48	**0.16**	**38**

Table 2.2 Potential output

Demand: social survey				**Supply: technical survey**
Day-time peak:				
Flour mill:	2 · 3 kW	6 kW		
Carpentry machine:	2 · 3 kW	6 kW		$Demand < P_{site\ 1} = 66\ [kW]$
Household applications:	30 · 0.5 kW	15 kW		$Demand < P_{site\ 2} = 56\ [kW]$
Storage water heater:	3 · 5 kW	15 kW		$Demand > P_{site\ 3} = 38\ [kW]$
Total I:		**42 kW**	**<**	
Night-time peak:				
Light bulb:	· 90 · 5 · 0.02 kW	9 kW		
Household applications:	30 · 0.5 kW	15 kW		
Storage water heater:	3· 5 kW	15 kW		
Total II:		**39 kW**		

Table 2.3 Demand/Supply analysis

To compare the different sites, list all important criteria down in a table. Based on this list, a decision is made as to figure out which criteria will influence the project either negative or positive.

Green: best choice; Yellow: 2nd choice; Red: last choice

	Capacity [kW]	Transmission [m]	Penstock [m]	Social aspects	Technical aspects
Site 1	66	1240	474	- Private channel	- Small landslide - Channel passes storm gully - Powerhouse in floodplain
Site 2	56	1080	276	- Buy land for channel extension	- Build channel extension - Channel passes storm gully
Site 3	38	860	210	- Fell some trees	- Widen channel to increase flow to reach demand (face water shortage)

Table 2.4 Analysis of different sites

Interpretation

The first site would have an excellent output, but is very costly due to the long penstock and transmission line. The private channel may give rise to maintenance issues which are difficult to solve. The irrigation season may be accompanied by a power shortage. Furthermore, the landslides may cause unforeseen repairs which force a shutdown of the unit each time. Also, one must protect the powerhouse from being flushed away if the water level rises, since two rivers meet next to the foundation.

The second site has average values, beside the fact that the channel must be extended by 250 m. To build the new channel, the village has to buy property from private landowners or do some other arrangements. The channel extend crosses the storm gully on a stretch of 20 m. An aqueduct could be a solution to bridge the storm gully. The powerhouse situation is safe compared to site1.

The third site does not reach the demand of 42 kW at the present situation, which would need a widening of the channel to increase the flow rate (up to 200 l/s). The widening would affect 400 m of the existing lower channel. By considering that the flow rate in the river might drop down below 120 l/s would have the consequences that during dry seasons the output will not reach the demand.

From this comparison, we can deduce that the maximum demand can be largely covered with the possible energy production at the second site. It can be assumed that less electricity will be needed in the beginning. But, the consumers will become used to the electricity and it can be expected that additional electrical devices will be purchased. Overproduction is sensible in the beginning. Also, when less water is used, less electricity is produced, but which, according to the following calculation, would mostly suffice:

$$Q_{required} = \frac{P_{daypeak}}{\eta \cdot 10 \cdot h_{gross}} = \frac{42\ kW}{0.5 \cdot 10 \cdot 70\ m} = 0.12\ \left[\frac{m^3}{s}\right]$$

With a reduction of 40 litres per second in the channel during winter time, it is still possible to produce enough power to satisfy the demands of the village. Since the flourmills will not be operated in winter, the demand will also slightly drop down:

$$Q_{Winter} = \frac{P_{nightpeak}}{\eta \cdot 10 \cdot h_{gross}} = \frac{39\ kW}{0.5 \cdot 10 \cdot 70\ m} = 0.11\ \left[\frac{m^3}{s}\right]$$

In case the flow rate goes below 0.11 m^3/s some arrangements need to be established. That could be for example a limited use of household applications.

3 Basic elements of hydromechanics

Hydromechanics is a sub-discipline in the wide field of mechanics. While mechanics in general deals with the interaction of solids under the influence of forces, hydromechanics deals with the interaction of fluids under the influence of forces. A global difference in solids and fluids can be characterised by the formability. Solids remain more or less in their original shape when they are exposed to forces and their structure will not deform until its specific strength has been reached, whereas a fluid starts to deform as soon as it is exposed to the slightest force.

Hydromechanics is a branch of applied science and engineering which focuses on the mechanical properties of liquids, principally water. Water has the advantageous attribute of not being compressible, unlike other fluids, which allows calculation relating its density to pressure. An incompressible fluid like water does not change its volume, while compressible fluids do.

Structures in contact with water can manage and guide the forces of either resting water or water in motion. The terms for these two behaviours are

- hydrostatics (in general the science of fluids at rest: fluid statics) and
- hydro dynamics (in general the science of fluids in motion: fluid dynamics)

Hydrostatics and hydrodynamics are branches of hydromechanics. The frequently used term ***hydraulics*** expresses the use of fluid to do work. In order to design a hydro-power system, such as an MHPU, it is essential to learn about the basic properties of hydraulics (hydromechanics). The principles of hydromechanics are necessary for judging and designing

- Dams
- Channels
- Spillways
- Pipe systems
- Storage and desilting basins
- Pumps
- Turbines
- River channel behaviour and erosion

The following pages give a short insight into the theoretical basics of hydromechanics.

3.1 Basics of hydrostatics

Hydrostatics is the science of fluids at rest and is a sub-field of hydro mechanics. The term usually refers to the mathematical treatment of the subject. It embraces the study of the conditions under which fluids are in stable equilibrium at rest. In hydrostatics the fluid that is examined is, in our case, water.

3.2 Hydrostatic pressure

In general, pressure is defined by force per unit area applied to an object in a direction perpendicular to the surface.

$$p = \frac{F}{A} \quad \left[\frac{\mathrm{N}}{\mathrm{m}^2}\right]$$

Where $F =$ normal Force, $A = Area\ (A = x \cdot y)$

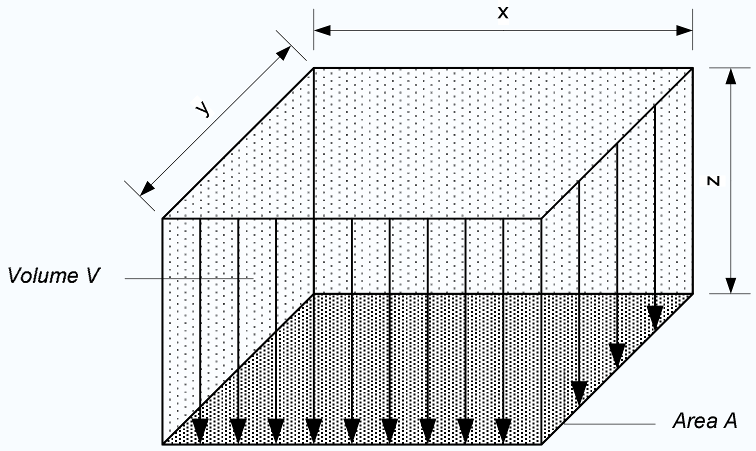


Figure 3.1 Area under hydrostatic pressure

Pressure is a scalar quantity, and has SI units of Pascal's; 1 Pa = 1 N/m^2 or 1 kPa = 1 kN/m^2. In hydrostatics, the pressure is caused by the weight of water resting against its boundaries such as against dams, weirs or pipe walls. The weight of water is, according to Newton's law, the product of mass and acceleration. The mass itself is formed by the volume and the density of, in this case, water. Acceleration is caused by gravity and therefore,

$$F = \rho_w \cdot V \cdot g \text{ [N]}$$

where $\rho_w =$ density of water, $g =$ gravitational acceleration, $V =$ volume of water ($A = x{\cdot}y{\cdot}z$).

The hydrostatic pressure is linear, increasing with the depth of the water. Therefore, the formula can be simplified and put into a function depending on depth z.

$$p(z) = \frac{\rho_w \cdot g \cdot x \cdot y \cdot z}{x \cdot y} = \rho \cdot g \cdot z \left[\frac{\mathrm{N}}{\mathrm{m}^2}\right]$$

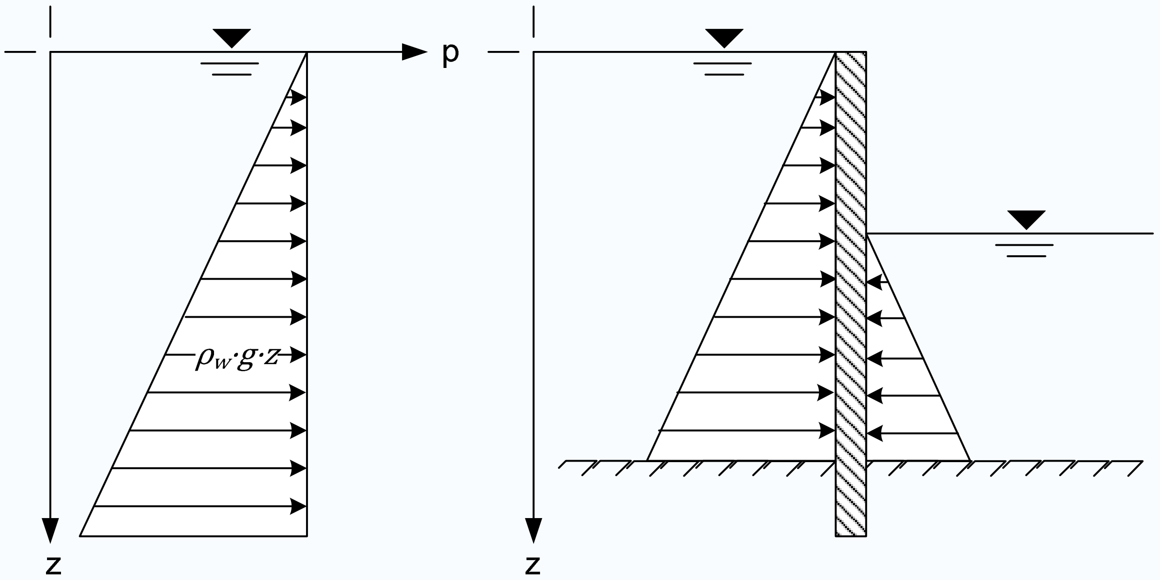


Figure 3.2 Hydrostatic pressure on vertical walls

If the water surface is considered to be the origin of a coordinate system ($z = 0$), hydrostatic pressure is zero. In some cases it is important to include the atmospheric pressure in the formula especially in pipe hydraulics when negative pressure is likely to occur ($p < 0$).

$$p(z) = \rho_w \cdot g \cdot z + p_a \left[\frac{\mathrm{N}}{\mathrm{m}^2}\right]$$

3.3 Buoyancy

Unlike solids, the pressure caused by fluids expands in all directions which results in buoyancy for bodies which stay in the fluid. Any arbitrarily shaped body which is immersed in a fluid, partly or fully, will experience the action of a vertical force originating from the depth-dependent liquid pressure.

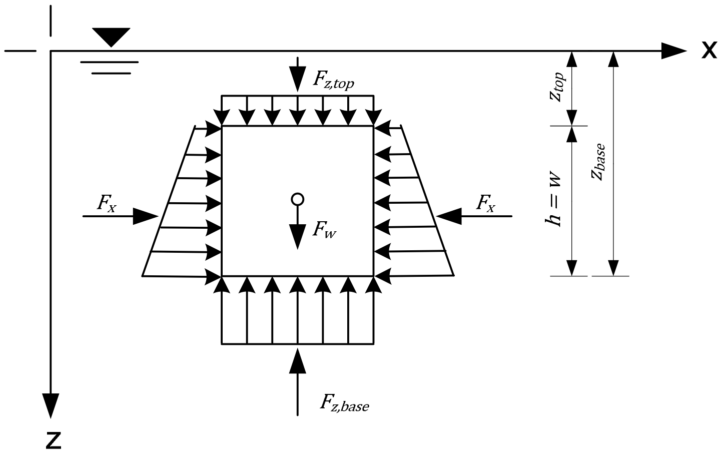


Figure 3.3 Cube exposed to hydrostatic pressure

A cube with a side length h, completely immersed in water, experiences hydrostatic pressure from all six directions. The corresponding compressive forces moving in horizontal directions (x and y-directions) neutralise each other because of their same magnitude. $F_x - F_x = 0$ as well as $F_y - F_y = 0$. The compressive force $F_{z,top}$ on top of the cube is smaller by the amount of $\rho \cdot g \cdot h$ than the compressive force $F_{z,base}$ from below against the cube. The difference of the vertical forces $\Delta F_z = F_{z,base} - F_{z,top}$ is equal to the weight of the volume of displaced fluid (V_{dis}), just pointing in opposite direction (buoyancy force).

$$F_B = F_{z,base} - F_{z,top} = \rho_w \cdot g \cdot V_{dis} \text{ [N]}$$

The density determines if a body floats or sinks:

$$sinking = 1 > \frac{\rho_{water}}{\rho_{body}} > 1 = floating$$

This explains why, for example, wood with a lower density ($\rho_{wood} \approx 600$ kg/m^3) than water ($\rho_{water} \approx 1000$ kg/m^3) floats and concrete with higher density ($\rho_{concrete} \approx 2500$ kg/m^3) than water sinks.

Let us look at two examples. First, take an obvious example: a ship. Its weight is balanced by a buoyant force from the displaced water, allowing it to float. When more cargo is loaded onto the ship, it sinks deeper into the water, displacing more water and thus needing a higher buoyant force to balance the increased weight. Buoyancy, however, may also occur unexpectedly in groundwater-saturated soil. Suppose a desilting tank is emptied for revision work and then the soil is completely saturated by groundwater; the structure of the tank might buoy upwards if the hydrostatic pressure forces become greater than the weight of the concrete structure.

Example: Empty channel under buoyancy force

Is the channel likely to buoy?

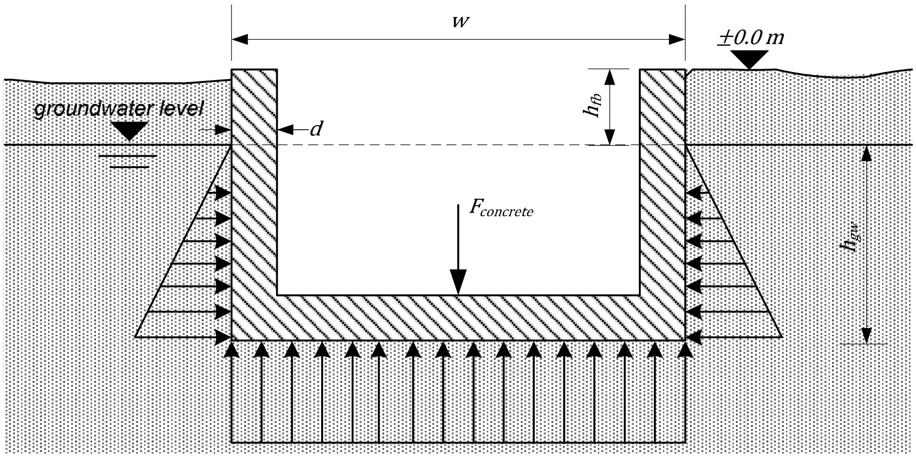


Figure 3.4 Channel under buoyancy

Given:

- Groundwater level -0.5 m
- Channel width $w = 3$ m
- Channel height $h = 1.8$ m
- Wall thickness $d = 0.2$ m
- Density of concrete $\rho_{concrete} = 2500$ kg/m^3

Buoyancy force:

$$F_B = \rho_w \cdot g \cdot V_{dis} = \rho_w \cdot g \cdot h_{gw} \cdot w = 1000 \cdot 9.81 \cdot 1.3 \cdot 3 = 38259\ [\mathrm{N}] = 38.259\ [\mathrm{kN}]$$

Weight force of concrete channel:

$$A_{concrete} = w \cdot h - (w - 2d) \cdot (h - d) = 3 \cdot 1.8 - 2.6 \cdot 1.6 = 1.24\ [\mathrm{m}^2]$$

$$F_{concrete} = A_{concrete} \cdot \rho_{concrete} \cdot g = 1.24 \cdot 2500 \cdot 9.81 = 30411\ [\mathrm{N}] = 30.411\ [\mathrm{kN}]$$

$$F_{concrete} = 30.411\ [\mathrm{kN}] < F_B = 38.259\ [\mathrm{kN}]$$

As can be seen, the channel is likely to buoy.

3.4 Hydrostatic compressive force on plane areas

Basically, the hydrostatic compressive force against a solid surface depends on hydrostatic pressure p and the extent of the involved area A. Since the hydrostatic pressure depends on the water depth z as well as the shape of the surface, each case must to be studied individually. In the following chapter, the focus is on symmetric plane areas which could appear in the form of dams, weirs or sheet pile walls.

For general derivation, it is assumed that hydro static pressure rests below the water surface of a basin, on an individual shaped area. The area A is located on an inclined x, y- plane under the angle of α. To quantify the compressive force on the area A, the following steps must be calculated

- The centre of gravity C_g of area A
- The hydrostatic pressure at the centre of gravity

Further, it will be of interest to know where the point of application C_a is located. The x-axis is assumed to be the line where the water surface and the x, y- plane cross. The water depths are displayed in the z-axis. For a clearer presentation, the x, y- plane is projected next to the lateral view.

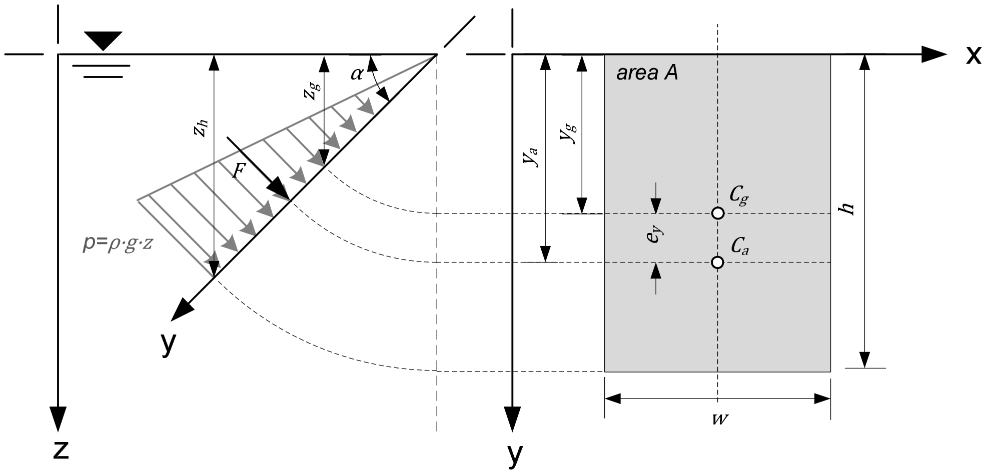


Figure 3.5 Plane area under hydrostatic pressure (lateral view left, front view right)

For a plane area $A = w{\cdot}h$, the resting hydrostatic pressure forms a triangular wedge. The compressive force for Figure 3.5 is calculated with

$$F = \int^{A} p(z)dz = \rho \cdot g \cdot \frac{z_h}{2} \cdot A \text{ [N]}$$

Since the hydrostatic pressure increases linearly, the average value is characteristic for right-angled shapes such as displayed in Figure 3.5. For other shapes, the hydrostatic pressure correspond to the centre of gravity C_g. Hence the formula changes to

$$F = \rho \cdot g \cdot z_g \cdot A$$

Due to increasing hydrostatic pressure at increasing depths, the centre of application C_a is generally located below the centre of gravity C_g. The distance from C_g to C_a is expressed by

$$e_y = y_a - y_g = \frac{I}{y_g \cdot A}$$

where y_a = distance between the x-axis and the centre of application C_a, y_g = distance between the x-axis and the centre of gravity C_g and I = the moment of inertia about its local x-axis**.** For certain symmetric shapes, the formulas for y_g, F and e_y are given in Figure 3.7.

Example: Trapezoid-shaped area under hydrostatic pressure

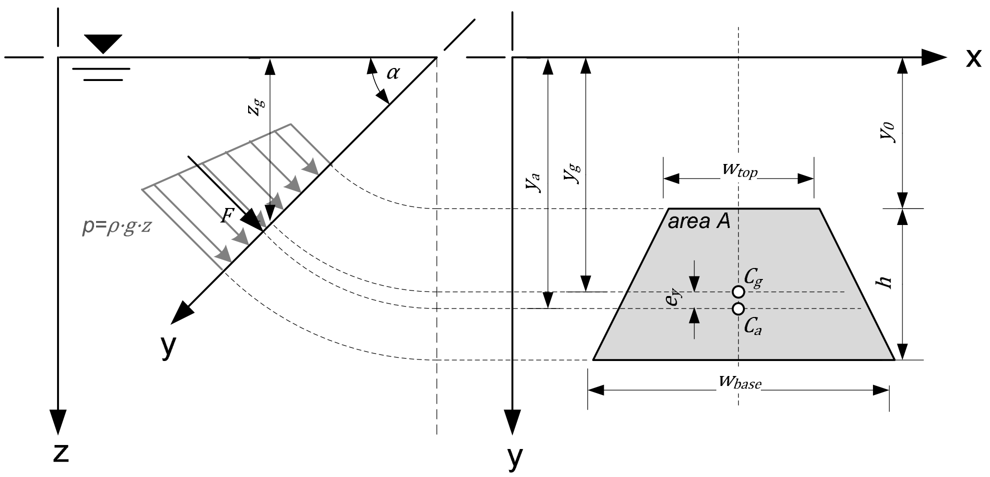


Figure 3.6 Trapezoid under hydrostatic pressure

Find the quantity of the compressive force F and centre of application C_a.

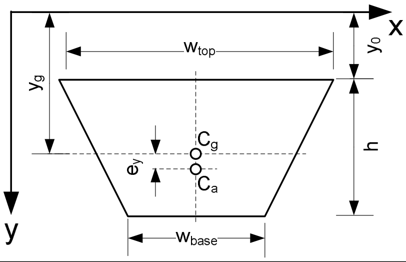
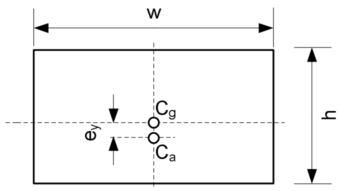
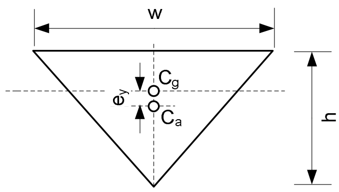
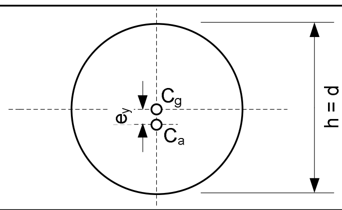
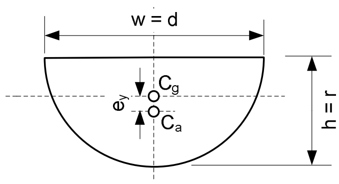
Shape	$y_g = \frac{z_g}{sin\alpha} =$	$\frac{F}{sin\alpha \cdot y_g} =$	$e_y =$
	$y_0 + \frac{h}{3} \cdot \frac{w_{top} + 2 \cdot w_{base}}{w_{top} + w_{base}}$	$\rho \cdot g \cdot h \cdot \frac{w_{top} \cdot w_{base}}{2}$	$\frac{h^2}{18} \cdot \frac{(w_{top} + w_{base})^2 + 2 \cdot w_{top} \cdot w_{base}}{(w_{top} + w_{base})^2 \cdot y_g}$
	$y_0 + \frac{h}{2}$	$\rho \cdot g \cdot w \cdot h$	$\frac{h^2}{12 \cdot y_g}$
	$y_0 + \frac{h}{3}$	$\rho \cdot g \cdot w \cdot \frac{h}{2}$	$\frac{h^2}{18 \cdot y_g}$
	$y_0 + \frac{d}{2}$	$\rho \cdot g \cdot \frac{\pi \cdot d^2}{4}$	$\frac{h^2}{16 \cdot y_g}$
	$y_0 + \frac{4 \cdot r}{3 \cdot \pi}$	$\rho \cdot g \cdot \frac{\pi \cdot r^2}{2}$	$\frac{h^2}{14.31 \cdot y_g}$

Figure 3.7 Compressive forces on symmetric-shaped areas

Given:

$\alpha = 45°\,; y_0 = 2\ [\mathrm{m}]; h = 2\ [\mathrm{m}];\ w_{top} = 0.2\ [\mathrm{m}];\ w_{base} = 0.4\ [\mathrm{m}]$

Calculate centre of gravity:

$$y_g = \frac{\sum_i^n y_i \cdot A_i}{\sum_i A_i} = \frac{3 \cdot (2 \cdot 2) + 3.33 \cdot (2 \cdot 1)}{(2 \cdot 2) + (2 \cdot 1)} = \frac{18.667}{6} = 3.111\ [\mathrm{m}]$$

Calculate hydrostatic pressure in centre of gravity:

$$p(z_g) = \rho \cdot g \cdot z_g = \rho \cdot g \cdot z_g \cdot \sin(45°) = 1000 \cdot 9.81 \cdot 3.111 \cdot \sin(45°) = 21580\ \left[\frac{\mathrm{N}}{\mathrm{m}^2}\right] = 21.580\ \left[\frac{\mathrm{kN}}{\mathrm{m}^2}\right]$$

Calculate compressive force:

$$F = p(z_g) \cdot A = 21.580 \cdot 6 = 129.5\ [\mathrm{kN}]$$

Calculate centre of application:

Triangle area A_1:

$$A_1 = 2 \cdot 1 = 2\ [\mathrm{m}^2];\ z_1 = 0.667\ [\mathrm{m}];\ I_1 = \frac{2 \cdot 2^3}{36} = 0.444\ [\mathrm{m}^4]$$

Square area A_2:

$$A_2 = 2 \cdot 2 = 4\ [\mathrm{m}^2];\ z_2 = 1\ [\mathrm{m}];\ I_2 = \frac{2 \cdot 2^3}{12} = 1.333\ [\mathrm{m}^4]$$

Moment of inertia according to Steiner:

$$I = \sum_{i=1}^{n} I_{x'i} + \sum_{i=1}^{n} A_{x'i} \cdot z_i^2 = I_1 + I_2 + A_1 \cdot z_1^2 + A_2 \cdot z_2^2 = 0.444 + 1.333 + 2 \cdot 0.667^{\,2} + 4 \cdot 1^2 = 6.67\ [\mathrm{m}^4]$$

$$e_y = \frac{I}{y_g \cdot A} = \frac{6.667}{3.111 \cdot 6} = 0.357\ [\mathrm{m}]$$

$$y_a = e_y + y_g = 0.357 + 3.111 = 3.467\ [\mathrm{m}]$$

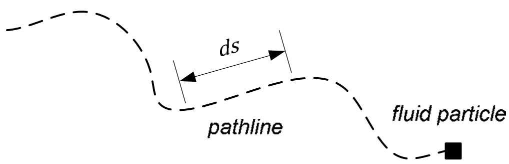
3.5 Basics of hydrodynamics

Hydrodynamics deals with water in motion under the influence of external forces and inert forces. The external forces are:

- gravitational forces (gravity)
- compressive forces
- frictional forces
- capillary forces

3.6 Definition of pathline, streamline and streamtube

The motion of water must be seen in relation to its artificial or natural boundaries. This could be flowing water in a hydraulic structure such as a pipe or a channel with an open surface. The boundaries are specified by pipe walls, channel walls or the open surface. There are different approaches describing how water moves within these boundaries. The pathline model assumes that water can be seen as a number of particles which are able to move from one position to another during a period of time t. The distance s covered by the particle is described as its pathline and may be different from any other particle's pathline within a specified group of particles. Pathlines are the trajectories that individual fluid particles follow.



$$ds = v \cdot dt$$

Another way to describe a moving water particle is the streamline. In a streamline, a water particle moves in a parallel direction to its boundaries and therefore both the direction vector and the velocity vector point in the same direction. The streamline defines the direction of the local velocity. By assuming that all water particles are moving in a parallel direction to each other, no fluid exchange between the different streamlines emerges. The total number of streamlines can be added together to create a streamtube which is limited by the boundaries of a structure or the open water surface.

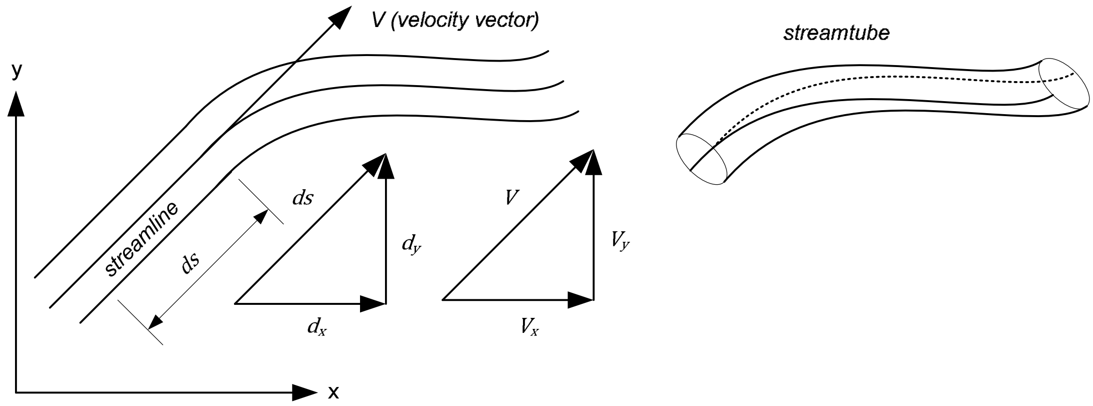


Figure 3.8 Single streamline (left), streamlines bundled into a streamtube (right)

The streamtube model helps to simplify the complex flow characteristics within the water by using, for example, the average velocity of all streamlines which are bundled in the streamtube.

3.7 Laminar flow, turbulent flow

Laminar flow, sometimes known as streamline flow, occurs when a fluid flows in parallel layers, with no disruption between the layers. It is the opposite of turbulent flow. In nonscientific terms, laminar flow is "smooth," while turbulent flow is "rough". For a practical demonstration of laminar and non-laminar flow, one can observe the smoke rising off an incense stick in a place where there is no breeze. The smoke from the incense stick will rise vertically and smoothly for some distance (laminar flow) and then will start undulating into a turbulent, non-laminar flow.

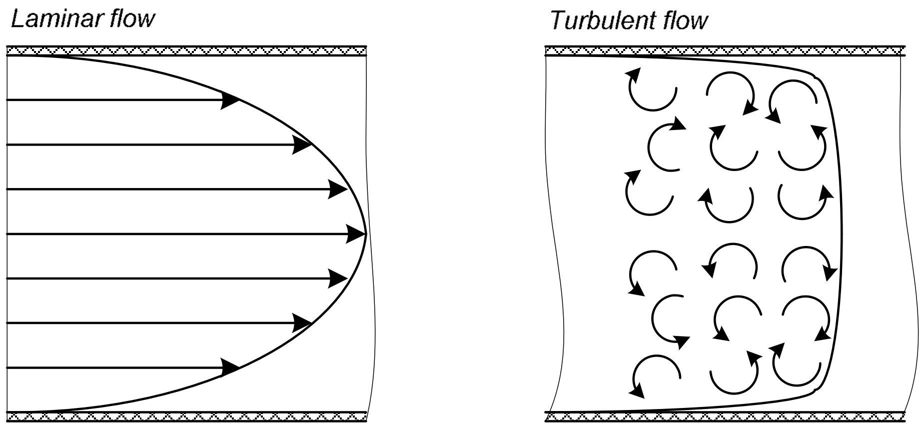


Figure 3.9 Laminar and turbulent flow pattern

To assess if water is likely to flow in a laminar or turbulent motion, the dimensionless Reynolds number comes into play. The formula displays the relation of inertial forces to viscous forces.

$$Re = \frac{v \cdot L}{\nu} \; [-]; \qquad where\ \nu = \frac{\mu}{\rho_w} \left[\frac{\mathrm{m}^2}{\mathrm{s}}\right]$$

Where

- V = velocity [m/s]
- L = characteristic length (e.g. diameter for pipes) [m]
- ν = kinematic viscosity (water at $10°C$ $\nu = 1.31 \cdot 10\text{-}6$) [m²/s]
- μ = dynamic viscosity [kNs/m²]
- ρ_w = density of water [kg/s]

Reynolds numbers of less than 2320 are generally considered to be of a laminar type. Reynolds numbers larger than 2320 indicate a turbulent flow. Viscosity is a measure of the internal resistance to flow and may be thought of as a measure of fluid friction. Internal resistance appears either as shear stress or extensional stress. In everyday terms (and for fluids only), viscosity is "thickness". Thus, water, having a lower viscosity is "thin", while honey, having a higher viscosity, is "thick".

Example: laminar and turbulent flow

What is the maximum velocity water can run through a pipe at before turning into a turbulent flow?

$$Re = \frac{v \cdot L}{\nu} < 2320 = \frac{v \cdot 0.2}{1.31 \cdot 10^{-6}}; v = \frac{2320}{0.2} \cdot 1.31 \cdot 10^{-6} < 0.016 \left[\frac{\text{m}}{\text{s}}\right]$$

3.8 Volume flow rate

Assuming that the direction of a moving fluid is pointing in the same direction as the velocity vector (streamline theory), area A must be perpendicular arranged to it. That would mean for parallel boundaries of a streamtube that area A is even, for other cases area A may be curved.

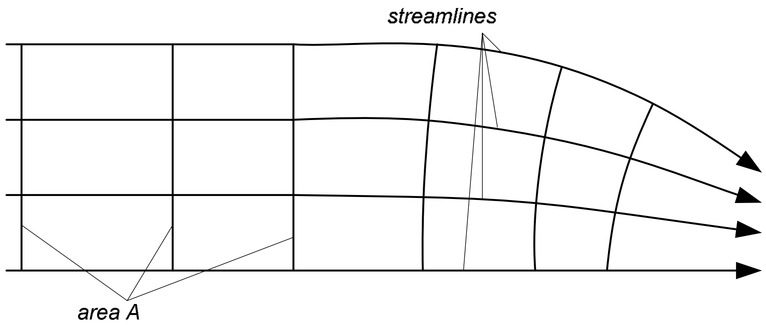


Figure 3.10 Plane and curved flow areas

For a period of time dt, a streamline passes perpendicularly through a given area A in the direction of the velocity vector v. Along the distance ds, the area A projects volume. Hence, the generated volume V is the scalar product of the given area A and the distance ds (unit: m³)

$$V = A \cdot ds = A \cdot v \cdot dt \; [\text{m}^3]$$

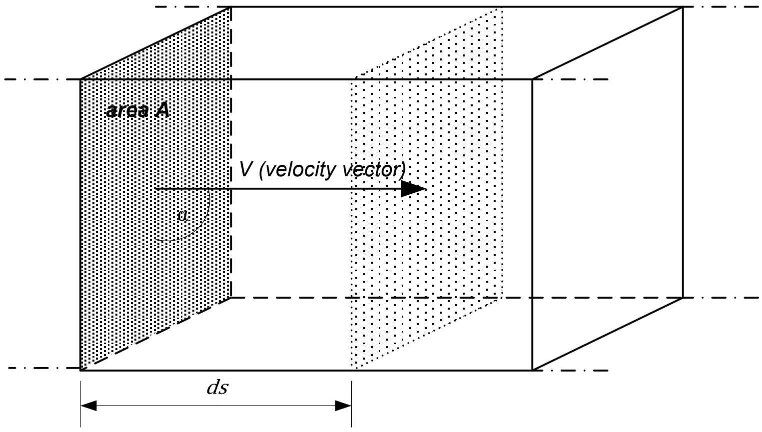


Figure 3.11 Volume flow in a cubic streamtube

Equally to the generated volume V, it is possible to generate the volume flow rate of fluid flow with

$$Q = A \cdot v = \frac{V}{dt} \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

The volume flow rate is usually represented by the symbol Q and the unit m^3/s.

For a more accurate prediction, the possibility of an uneven velocity distribution must be taken into account. In this case, the volume flow rate is equal to the content of a visualised field with the area A as a base area and a magnitude which conforms to its velocity at each point in area A (Figure 3.12). The total sum represents the total volume flow rate. Speaking in mathematical terms, the rate of fluid flow can be calculated by means of a surface integral.

$$Q = \int^{A} v_{(A)} \cdot dA \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

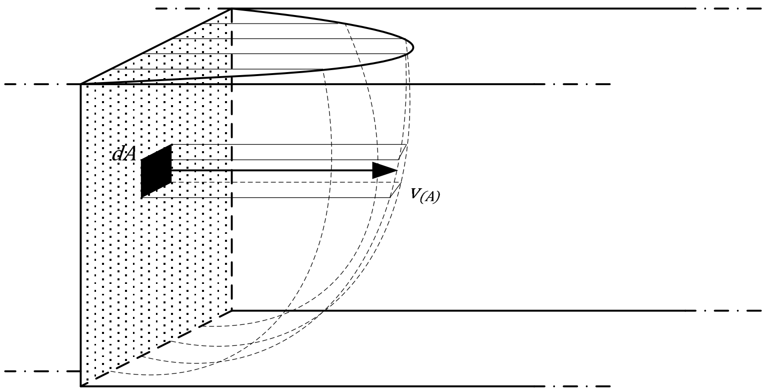


Figure 3.12 Individual volume flow rate for all area pieces dA

3.9 Continuity equation

The continuity equation describes a local form of conservation law. When a fluid is in motion, it must move in such a way that mass is conserved. To see how mass conservation places restrictions on the velocity field, consider the steady flow of fluid through a streamtube (that is, the inlet and outlet flows do not vary with time). The inflow and outflow are one-dimensional, so that the velocity V and density ρ_w

are constant over the area A (Figure 3.13).

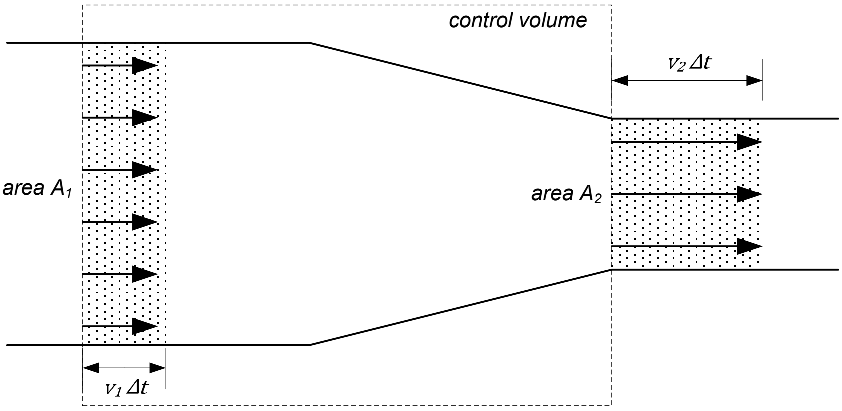


Figure 3.13 One-dimensional streamtube showing control volume

Applying the principle of mass conservation, it is assumed that there is no flow through the side walls of the streamtube. The mass comes in as A_1 goes out as A_2, (the flow is steady so that there is no mass accumulation). Over a short time interval Δt, the volume flow in is $A_1 = A_1 \cdot v_1 \cdot \Delta t$, and the volume flow out is $A_2 = A_2 \cdot v_2 \cdot \Delta t$. Therefore the mass going in is equal to the mass going out:

$$\rho_1 \cdot A_1 \cdot v_1 = \rho_2 \cdot A_2 \cdot v_2$$

Expressed by volume flow rate:

$$Q = A_1 \cdot v_1 = A_2 \cdot v_2 \;\; ; \quad Q_1 = Q_2$$

This is a statement of the principle of mass conservation for a steady, one-dimensional flow, with one inlet and one outlet. For a steady flow through a control volume with many inlets and outlets, the net mass flow must be zero, where inflows are negative and outflows are positive.

Example: Y-pipe

Pipe strand 1 of a Y-pipe junction has an internal diameter of 300 mm. The average velocity in pipe 1 is assumed $v_1 = 2$ m/s. The volume flow rate Q_2 of pipe branch 2 is assumed to be 0.25 m^3/s. What dimensions for the diameters d_2 and d_3 must be chosen to have the same average velocity in all pipe branches ($v_1 = v_2 = v_3$)?

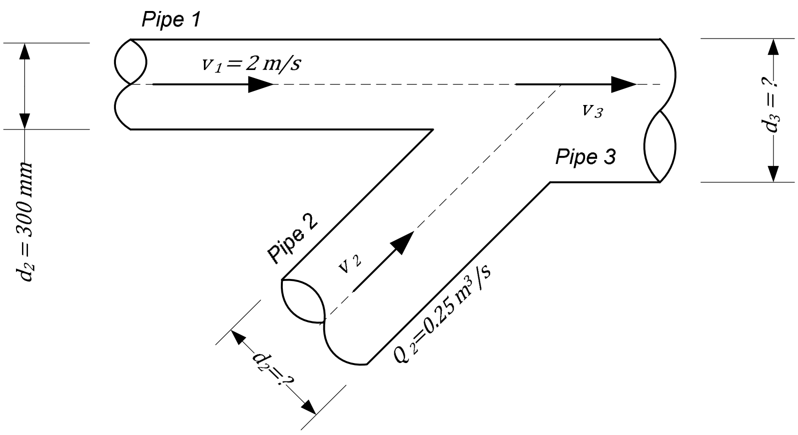


Figure 3.14 Y-pipe section

Volume flow rate pipe 1:

$$Q_1 = v_1 \cdot A_1 = v_1 \cdot \frac{\pi \cdot d_1^2}{4} = 2 \cdot \frac{\pi \cdot 0.3^2}{4} = 0.141 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

Volume flow rate pipe 3:

$$Q_3 = Q_1 + Q_2 = 0.25 + 0.141 = 0.391\ \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

Diameter of pipe 2:

$$d_2 = \sqrt{\frac{4 \cdot Q_2}{\pi \cdot v_2}} = \sqrt{\frac{4 \cdot Q_2}{\pi \cdot v_1}} = \sqrt{\frac{4 \cdot 0.25}{\pi \cdot 2}} = 0.399\ [\mathrm{m}]$$

Diameter of pipe 3:

$$d_3 = \sqrt{\frac{4 \cdot Q_3}{\pi \cdot v_3}} = \sqrt{\frac{4 \cdot Q_3}{\pi \cdot v_1}} = \sqrt{\frac{4 \cdot 0.391}{\pi \cdot 2}} = 0.499\ [\mathrm{m}]$$

3.10 Energy theorem & Bernoulli's principle

According to Newton's law of motion, each kind of matter (including fluid mass) changes its state of motion only when external forces are applied to it. Therefore, the applied forces F (gravitational-, compressive-, friction forces etc.) at every point in time and location on the fluid mass must be equal to the forces of inertia. Therefore inertia can be seen as the resistance of an object to a change in its state of motion.

$$F = m \cdot \frac{dv}{dt} = m \cdot a\ [\mathrm{N}]$$

The forces caused by friction or shear tension, which act not only on solid bodies, (for example, the pipe walls) but also within the fluid, are responsible for the irreversible change from hydraulic to thermal energy. This process is called dissipation, in common terms "energy loss". Although energy cannot be "lost", it may become useless (heat or noise, etc) for some applications. Energy loss occurs within all energy systems and cannot be prevented, only minimised. In the following chapter on energy equation, energy losses are ignored.

3.11 Energy equation for one-dimensional applications

Imagine a straight streamtube which is located above a reference x-axis with a cross section $A(x)$ which may vary along the flow direction x (Figure 3.15 Energy head of a streamtube). Further, an imaginary small inviscid (frictionless) fluid mass m with the density ρ_w and the weight $F_w = m \cdot g$ fills the volume $V=m/\rho$ at any spot x along the streamtube. The fluid mass is located along the axis of the streamtube which is located $z(x)$ above the reverence x-axis. In the streamtube, water moves with an average velocity $v(x)$ and a constant volume flow rate Q. The fluid mass experiences a certain amount of pressure $p(x)$. In total, the fluid mass carries three types of energy:

- Potential energy $= F_w \cdot z = m \cdot g \cdot z$
- Pressure energy $= V \cdot p = m \cdot p/\rho_w$
- Kinetic energy $= m \cdot v^2/2$

The total hydraulic energy of the fluid mass is

$$E = m \cdot g \cdot \left(z + \frac{p}{\rho \cdot g} + \frac{v^2}{2 \cdot g}\right) = F_w \cdot \left(z + \frac{p}{\rho \cdot g} + \frac{v^2}{2 \cdot g}\right)\ [\mathrm{kNm}]$$

By presuming that the fluid is inviscid, a loss-free flow can be assumed. The total amount of energy is the same at any place x along the streamtube axis, which means the energy level stage is constant.

Instead of calculating the energy according the above-mentioned formula, the specific energy (which, in this case, is length) can be expressed by the equation named after the Dutch–Swiss mathematician, Daniel Bernoulli.

$$\frac{E}{F_w} = h_E = z + \frac{p}{\rho \cdot g} + \frac{v^2}{2 \cdot g} \ [\mathrm{m}]$$

The energy is now expressed in energy head h_E where energy level describes the sum of potential, pressure and kinetic energy.

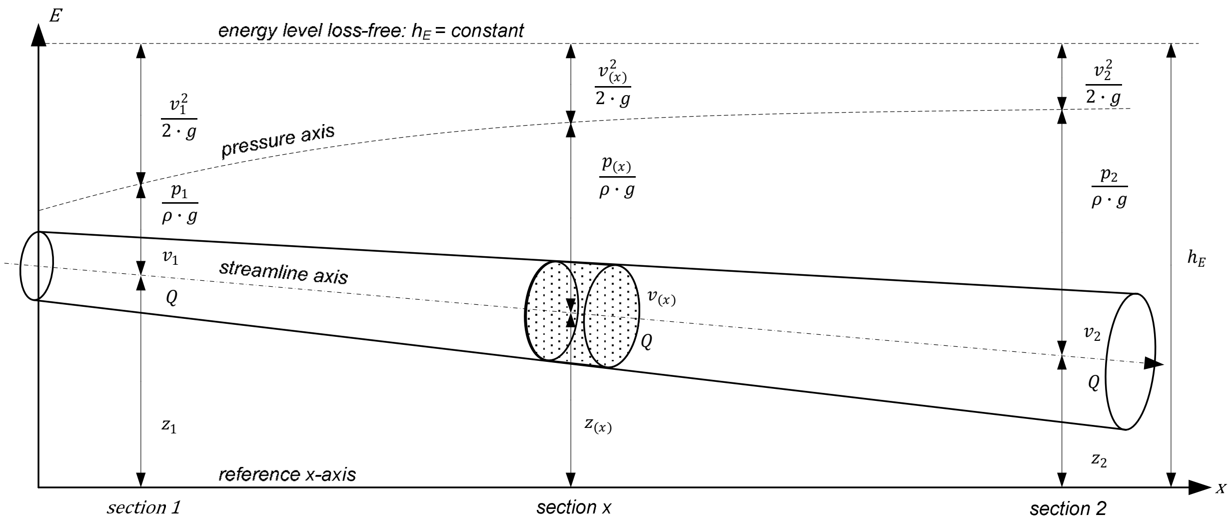


Figure 3.15 Energy head of a streamtube

In fluid dynamics, Bernoulli's principle states that for an inviscid flow, an increase in the speed of the fluid occurs simultaneously with a decrease in pressure or a decrease in the fluid's potential energy. The principle is equivalent to the principle of the conservation of energy. This states that in a steady flow, the sum of all forms of mechanical energy in a fluid along a streamline is the same at all points on that streamline. This requires that the sum of the kinetic-, pressure- and potential energy remains constant. If the fluid is flowing out of a reservoir, for example, the sum of all forms of energy is the same on all streamlines, because in a reservoir, the energy per unit mass (the sum of pressure and gravitational potential $\rho \cdot g \cdot h$) is the same everywhere.

Example: Venturi nozzle

A horizontally arranged Venturi nozzle is mounted into a pipe for measuring the volume flow rate. The pipe has a diameter $D_1 = 200$ mm and the measure nozzle diameter is $D_2 = 60$ mm.

a) Draw energy and pressure line referring the pipe axis (streamline axis)
b) What amount of water passes through the pipe (volume flow rate) when the pressure gauge displays a difference in pressure of $\Delta p = 5$ kN/m^2?

Assuming that no losses due to friction occur, the energy line is identical to the energy level and therefore, the following equations can be applied. Bernoulli equation:

$$z_1 + \frac{p_1}{\rho \cdot g} + \frac{v_1^2}{2 \cdot g} = z_2 + \frac{p_2}{\rho \cdot g} + \frac{v_2^2}{2 \cdot g}$$

$$\frac{p_1 - p_2}{\rho \cdot g} = \frac{v_2^2 - v_1^2}{2 \cdot g}$$

$$\frac{\Delta p \cdot 2 \cdot g}{\rho \cdot g} = v_2^2 - v_1^2$$

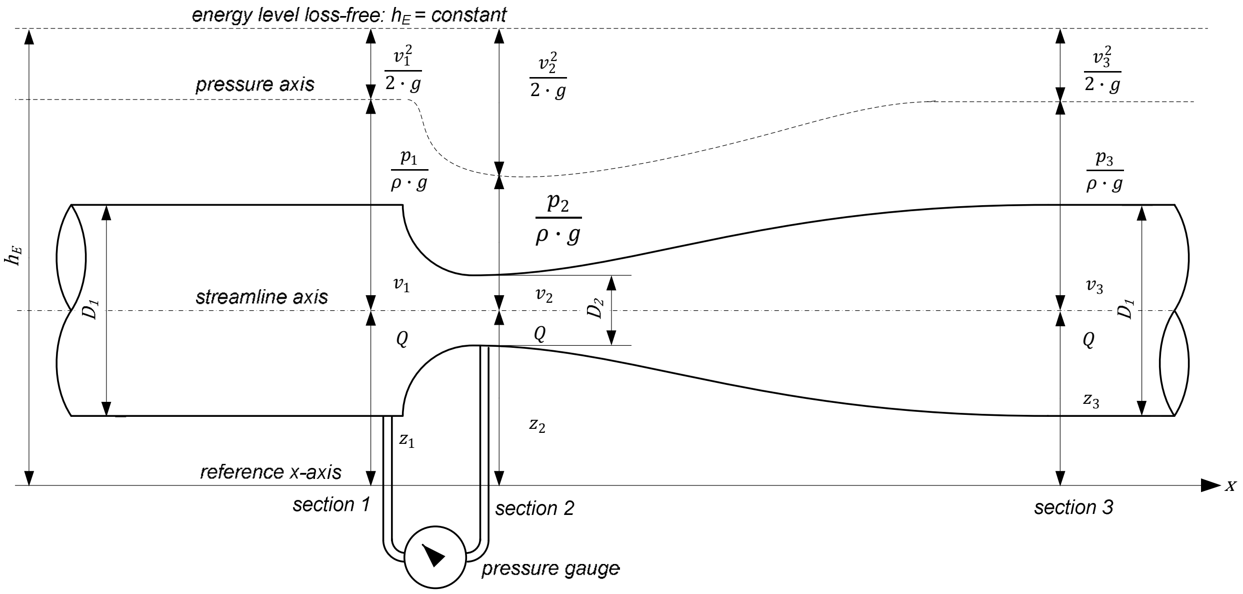


Figure 3.16 Venturi nozzle with pressure gauge

Continuity equation:

$$Q = v_1 \cdot A_1 = v_2 \cdot A_2; were\ A_1 = \pi \cdot \frac{D_1^2}{4}; and\ A_2 = \pi \cdot \frac{D_2^2}{4}$$

$$v_1 = v_2 \cdot \frac{A_2}{A_1} = v_2 \cdot \left(\frac{D_2}{D_1}\right)^2$$

Insert v_1 into Bernoulli equation:

$$v_2^2 - \left(v_2 \cdot \left(\frac{D_2}{D_1}\right)^2\right)^2 = v_2^2 \cdot \left(1 - \left(\frac{D_2}{D_1}\right)^4\right) = \frac{\Delta p \cdot 2}{\rho}$$

$$v_2 = \sqrt{\frac{\Delta p \cdot 2}{\rho \cdot \left(1 - \left(\frac{D_2}{D_1}\right)^4\right)}} = \sqrt{\frac{5 \cdot 2}{1 \cdot \left(1 - \left(\frac{0.06}{0.2}\right)^4\right)}} = 3.175 \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

Insert v_2 into continuity equation:

$$Q = v_1 \cdot A_1 = v_2 \cdot \pi \cdot \frac{D_2^2}{4} = 3.175 \cdot \pi \cdot \frac{0.06^2}{4} = 0.009 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

3.12 The principle of linear momentum in hydro dynamics

The principle of linear momentum in hydrodynamics appears where changes in direction of a flowing fluid can be observed. For a single bended streamtube starting with section A_1 and ending with section A_2, pressure forces may appear in both sections.

$$\vec{F} = \rho \cdot Q \cdot \vec{v}\ [\mathrm{N}]$$

Since the linear momentum has a direction as well as a magnitude, it is a vector quantity. The two forces F_1 and $-F_2$ acting on the corresponding sections of the streamtube create the force F. F is the geometrical sum of all forces having an impact on the fluid within the stream tube.

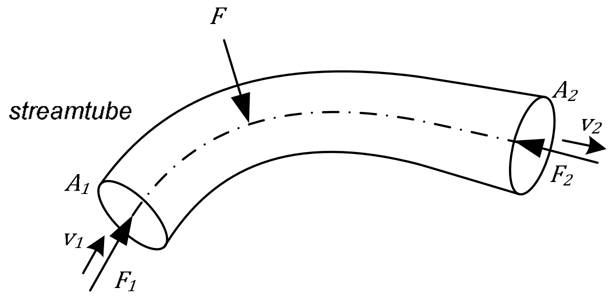


Figure 3.17 Bended stream tube

The forces F_1 and F_2 are fictive and always point inside the stream tube, despite the fact that the fluid is leaving the streamtube in section 2. The forces F_1 and F_2 are also called support forces and the formula is therefore called the support force theorem. According to the support force theorem, all forces which act from the shell of the streamtube against the flowing fluid must be equal to the support forces applying on the section A_1 and A_2.

$$\vec{F} = \vec{F}_{s1} + \vec{F}_{s2} \text{ [N]}$$

Supporting forces for a pressure pipe with a diameter d can be calculated by

$$F_s = p \cdot A + \rho \cdot Q \cdot v = \frac{\pi \cdot d^2}{4} \cdot (p + \rho \cdot v^2) \text{ [N]}$$

Supporting forces for rectangular-shaped channels with an open surface can be calculated by

$$F_s = \rho \cdot g \cdot w \cdot \frac{h^2}{2} + \rho \cdot Q \cdot v \text{ [N]}$$

where w is channel width and h is water depth.

Example: Pelton turbine

You want to calculate a nozzle for a Pelton turbine with a reduction in diameter from $D_1 = 200$ mm to $D_2 = 50$ mm. The water jet leaves the nozzle and enters the turbine casing and is therefore only exposed to atmospheric pressure which is $p_2 = p_{ap} = 0$. What is the force F acting against the nozzles flange (section 1-1) when the volume flow rate $Q = 0.120$ m^3/s?

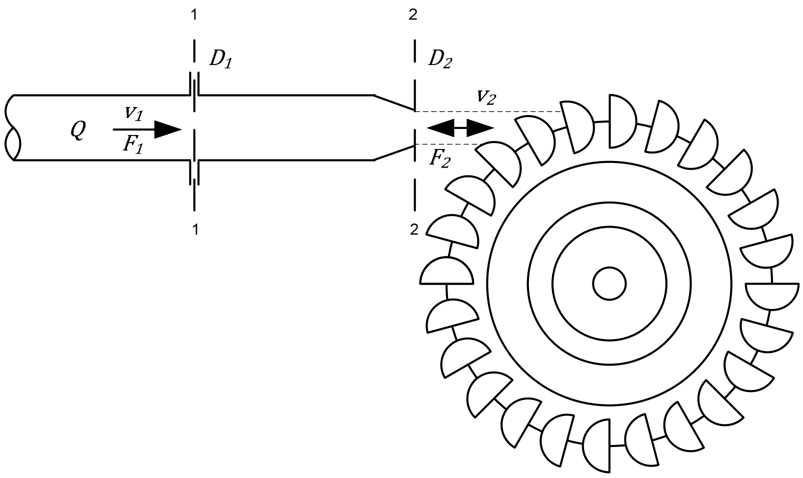


Figure 3.18 Pipe nozzle to increase kinetic energy

Velocity in section 1-1; 2-2 according continuity equation:

$$Q = A_1 \cdot v_1 = A_2 \cdot v_2$$

$$v_1 = \frac{Q}{A_1} = \frac{0.120}{\frac{\pi \cdot 0.2^2}{4}} = 3.820\frac{m}{s}\ ;\ v_2 = \frac{Q}{A_2} = \frac{0.120}{\frac{\pi \cdot 0.05^2}{4}} = 61.115\left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

Pressure in section 1-1 according Bernoulli equation:

$$z_1 + \frac{p_1}{\rho \cdot g} + \frac{v_1^2}{2 \cdot g} = z_2 + \frac{p_2}{\rho \cdot g} + \frac{v_2^2}{2 \cdot g}; were\ z_1 = z_2\ and\ p_2 = 0$$

$$p_1 = \left(\frac{v_2^2 - v_1^2}{2}\right) \cdot \rho = \left(\frac{61.115^2 - 3.820^2}{2}\right) \cdot 1000 = 1860257\left[\frac{\mathrm{N}}{\mathrm{m}^2}\right]$$

Resulting force acting against nozzle flange: (Force F_2 is pointing in the opposite direction to F_2 and is therefore negative)

$$F_{nozzle} = F_1 + (-F_2)$$

$$F_{nozzle} = p_1 \cdot A_1 + \rho \cdot Q \cdot v_1 - (p_2 \cdot A_2 + \rho \cdot Q \cdot v_2)$$

$$F_{nozzle} = 1860257 \cdot \frac{\pi \cdot 0.2^2}{4} + 1000 \cdot 0.120 \cdot 3.820 - (1000 \cdot 0.120 \cdot 61.115) = 51566\ [\mathrm{N}]$$

Example: Penstock bend

A penstock leading to a Pelton turbine is bent 60 degrees, and the diameter of the pipe is reduced from $D_1 = 400$ mm to $D_2 = 200$ m. The turbine shaft is arranged vertically, which means the penstock stands horizontally on the foundation block. The bent pipe section experiences, due to the supporting forces F_1 and F_2 , a resultant force F which drags on the foundation block. The internal pressure of the pipe in section 2-2 will be $p_2 = 49$ kPa. How large will the force F acting against the pipe bend be at a volume flow rate of Q = 0.120 m^3/s? Due to the short distance, the losses caused by friction can be ignored.

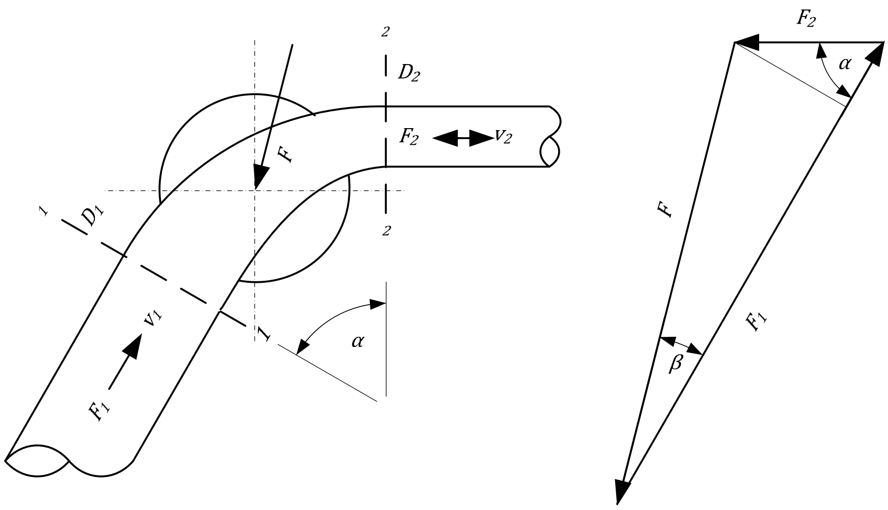


Figure 3.19 Supporting forces acting against an anchor block

Velocity v_1 and v_2 according to continuity equation:

$$Q = A_1 \cdot v_1 = A_2 \cdot v_2$$

$$v_1 = \frac{Q}{A_1} = \frac{0.120}{\frac{\pi \cdot 0.4^2}{4}} = 0.955\frac{m}{s}\ ;\ v_2 = \frac{Q}{A_2} = \frac{0.120}{\frac{\pi \cdot 0.2^2}{4}} = 3.820\left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

Pressure in section 1-1 according Bernoulli equation:

$$z_1 + \frac{p_1}{\rho \cdot g} + \frac{v_1^2}{2 \cdot g} = z_2 + \frac{p_2}{\rho \cdot g} + \frac{v_2^2}{2 \cdot g}; were\ z_1 = z_2$$

$$p_1 = \left(\frac{p_2}{\rho \cdot g} + \frac{v_2^2 - v_1^2}{2 \cdot g}\right) \cdot \rho \cdot g = \left(\frac{49000}{1000 \cdot 9.81} + \frac{3.820^2 - 0.955^2}{2 \cdot 9.81}\right) \cdot 1000 \cdot 9.81 = 55839 \left[\frac{\mathrm{N}}{\mathrm{m}^2}\right]$$

Forces:

$$F_1 = p_1 \cdot A_1 + \rho_w \cdot Q \cdot v_1 = 55839 \cdot \frac{0.4^2}{4} \cdot \pi + 1000 \cdot 0.120 \cdot 0.955 = 7132\ [\mathrm{N}]$$

$$F_2 = p_2 \cdot A_2 + \rho_w \cdot Q \cdot v_2 = 49000 \cdot \frac{0.2^2}{4} \cdot \pi + 1000 \cdot 0.120 \cdot 3.820 = 1998\ [\mathrm{N}]$$

Reflection force F according cosine rule:

$$F = \sqrt{F_1^2 + F_2^2 - 2 \cdot F_1 \cdot F_2 cos\alpha} = \sqrt{7132^2 + 1998^2 - 2 \cdot 1998 \cdot 7132 \cdot cos\alpha} = 6372\ [\mathrm{N}]$$

Angle between F_1 an F according law of sinus:

$$\frac{sin\beta}{F_2} = \frac{sin\alpha}{F}$$

$$\beta = \arcsin\left(\frac{F_2 \cdot sin\alpha}{F}\right) = \arcsin\left(\frac{1998 \cdot sin60}{6372}\right) = 15.76°$$

3.13 Open channel flow

In addition to flow of water in a close conduit such as a penstock or a pressure tunnel, we also face the situation of open channel flow which is characteristic for its free water surface. Unlike the flow in a close conduit, where the water is bounded by the pipe walls, the free water surface may vary in an open channel which makes calculations more problematic. Open channel flow is exposed to atmospheric pressure, which in pipe flow has only an indirect impact. However, in open channel flow, the hydraulic pressure is no portion in the total energy head h_E. Therefore, the Bernoulli formula can simplified to

$$h_E = z + \frac{v^2}{2 \cdot g}\ [\mathrm{m}]$$

where z is the vertical distance between a defined datum and the water surface in [m], v is the mean velocity in the regarded cross-section in [m/s] and g is the acceleration of gravity in [m/s^2]

3.14 State of flow in open channels

In open channel hydraulics, we can observe different flow behaviours: Sub-critical flow, critical flow and super critical-flow. Sub-critical flow goes along with great water depth, low flow velocity and with a gentle slope. This flow behaviour is characteristic for most streams and channels in nature. Super-critical flow occurs in shallow water with high flow velocity and steep slope. This flow behaviour is typical for mountain torrents and spillway chutes. The state of flow is represented by a ratio of inertial forces to gravity forces also known as the Froude number:

$$F_r = \frac{v}{\sqrt{g \cdot h}}\quad [-]$$

Where v is the mean velocity in [m/s], g is the acceleration of gravity in [m/s^2] and h is the hydraulic depth of the channel in [m]. The term below the fraction bar $(g{\cdot}h)^{0.5}$ is identified as the velocity of small gravity waves. In case F_r is equal to unity ($v=(g{\cdot}h)^{0.5}$), the flow is considered as critical. This means that the velocity in your channel is equal to the wave velocity. If *Fr* is less than unity ($v<(g\cdot h)^{0.5}$), the flow is subcritical and smaller than the wave velocity. If *Fr* is greater than unity ($v>(g\cdot h)^{0.5}$), the flow is supercritical and therefore greater than wave velocity.

The direct consequence of this behaviour can be observed when inducing waves by disturbing the flow. In case of critical flow, one front of such a wave would appear stationary to an observer standing on the channel banks. An upstream propagating wave (relative to the channel banks) indicates subcritical flow, whereas a downstream propagating wave indicates a supercritical flow. Therefore, the direction of the propagating waves can be used as a criterion for distinguishing between subcritical and supercritical flow.

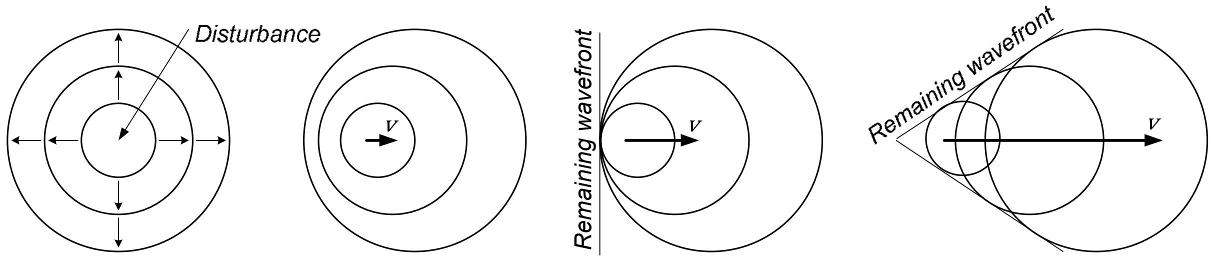


Figure 3.20 Propagation of small gravity waves with increasing velocity

Since the flow is dependent on boundary conditions such as water depth, flow velocity and slope, it is also likely to change its state of flow due to a change in the channel's shape or slope. In case of a drop in a shallow channel's slope, the velocity may increase with the result of surpassing the critical flow. This phenomenon is called a *hydraulic drop* and may occur upon a weir crest, a narrowing (Venturi channel) or below a control gate. The effect may also appear the other way round meaning that the velocity drops down below the critical flow, which causes a hydraulic jump.

3.15 Flow formula for uniform flow

Uniform flow has the typical features of constant depth, water area, velocity and discharge at every section of the channel. Therefore, the energy line must be parallel with the water surface as well as with the channel's bottom. With the empirical approach of the Manning equation,

$$Q = A \cdot \frac{1}{n} \cdot R^{2/3} \cdot \sqrt{S} \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

open channel flow can be quantified under the condition of uniform flow in [m^3/s].

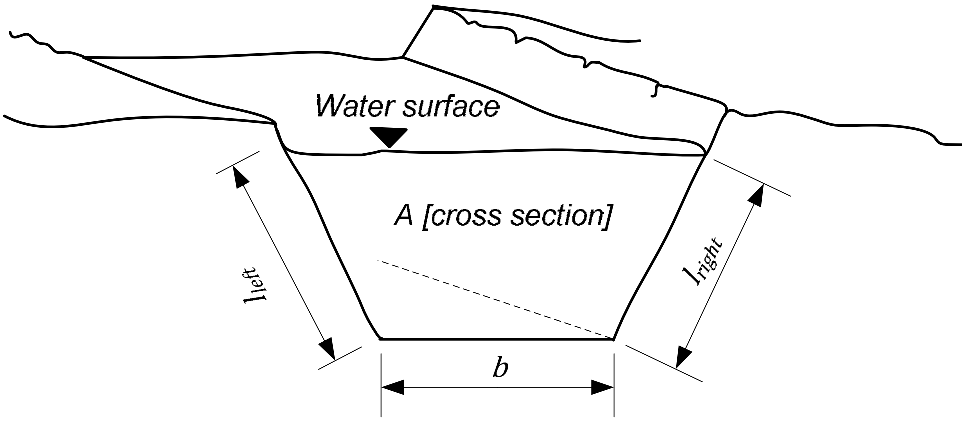
Where A is cross sectional water area [m^2], n is the roughness coefficient according to Manning [-] (select the Manning coefficient from Table 3.1), R is the hydraulic radius [m], S is the slope of either the energy level, water surface or channel bottom line [m/m].

$$S = \frac{\Delta h}{\Delta L} \left[\frac{\mathrm{m}}{\mathrm{m}}\right]$$

where Δh is the drop in head and ΔL is the regarded longitudinal section of the channel. The hydraulic radius is an auxiliary quantity containing the values of A (cross-sectional water area [m^2]) and P which is the wetted perimeter [m].

$$R = \frac{A}{P} \ [\mathrm{m}]$$

The hydraulic radius is not half the hydraulic diameter as the name may suggest. It is a function of the shape of the pipe, channel, or river in which the water is flowing. The hydraulic radius of a channel is defined as the ratio of its cross-sectional area to its wetted perimeter U (the part of the cross-section – bed and bank – that is in contact with the water).



Wetted perimeter of a trapezoid channel:

$U = b + l_{left} + l_{right}$ [m]

Figure 3.21 Wetted perimeter of trapezoid shaped channel

The greater the hydraulic radius, the more water can be transported by the channel. The highest values occur when the channel's shape is close to a semi-circle. In wide rectangular channels, the hydraulic radius is approximated by the flow depth. Roughness and hydraulic radius are therefore a measure of a river's or channel's efficiency (its ability to move water and sediment). It is used by civil engineers to assess channel and river cross-sections to transport water with a minimum of head loss.

For unconsolidated rock and riverbed material, you can derive the Manning coefficient by

$$n = \frac{d_{90}^{1/6}}{26}$$

For unconsolidated rock and riverbed material, you can derive Strickler coefficient by

$$k_{st} = \frac{26}{d_{90}^{1/6}}$$

where d_{90} is the predominant diameter (90 %) of stones, covering the riverbed.

The Strickler formula can be optional used for the Manning formula:

$$Q = A \cdot k_{st} \cdot R^{2/3} \cdot \sqrt{S}$$

	Roughness		
Surface texture of river/channel bed	**n [s/m$^{1/3}$]**	**k [mm]**	**k_{st} [m$^{1/3}$/s]**
Glass	0.010	0.003	100
Stainless steel	0.010	0.03	100
Plastic (PVC, PE)	0.010	0.05	100
Wood planed	0.011	0.3	90
Smooth concrete plastered	0.012	0.6	85
Smooth concrete	0.013	0.8	80
Prefabricated concrete	0.013	1.5	75
Concrete joint less	0.014	2	70
Stonework and brickwork, smooth	0.015	3	65
Steel pipes medium incrusted	0.015	3	65
Cobble stone pavement, smooth	0.015	3	65
Concrete, unrendered	0.015	3	65
Concrete, old	0.017	6	60
Stonework and brickwork, less accurate	0.017	6	60
Earth, smooth	0.017	6	60
Concrete, old, corroded	0.018	10	55
Stonework and brickwork, crude	0.020	20	50
Cobble stone pavement	0.022	20	45
Concrete, bad quality	0.022	20	45
Earth channels	0.025		40
Fine gravel		30	
Medium gravel		50	
Coarse gravel		75	
Channels with medium and coarse gravel	0.029	90	35
Earth channels infested with weeds and potholes	0.029	90	35
Natural riverbeds with boulders	0.033		30
River beds with heavy bed load	0.033		30
Earth channels with clumpy clay	0.033		30
Foreland covered with cover of vegetation	0.033		30
Mountain rivers with coarse boulders	0.040		25
Rock fill	0.050		20
Torrent	0.050		20

Table 3.1 Roughness coefficients according to Manning and Strickler

4 Elements of hydrology

Hydrology is the study of the movement, distribution and quality of water throughout the environment and thus addresses both the water cycle and water resources. The water cycle, also known as the hydrologic cycle, describes the continuous movement of water on, above, and below the surface of the Earth. Water can change states, e.q., liquid, vapour and ice, at various stages in the water cycle. Vaporising water from the oceans is transported by winds to the countryside and mountain ranges where it precipitates in the form of rain and snowfall. The runoff leading back to the oceans completes the cycle were the process starts again. Since the water cycle is truly a "cycle," there is neither beginning nor ending. Also, the amount of water active in this cycle is rather small. Only about 0.001 % of the worldwide water resources are vapour in the atmosphere. An even smaller amount of 0.00001 % moves in fluid state in rivers and streams. Another 0.62 % remains in freshwater lakes and groundwater aquifers, a further 2.15 % is stored in glaciers. The other 97.2 % of the worldwide water is in the oceans, which cover more than 70 % of our planet’s surface.

4.1 Water cycle

The sun, which drives the water cycle, causes evaporation from water surfaces and evapotranspiration from plants and trees. Rising air currents take the vapour up into the atmosphere where cooler temperatures cause it to condense into clouds. Air currents move clouds around the globe, where the particles collide, grow, and cause precipitation. Some fall as rain, some as snow and accumulate into ice caps and glaciers, which stores frozen water for thousands of years. Snowpacks may thaw and melt, and run off as snowmelt.

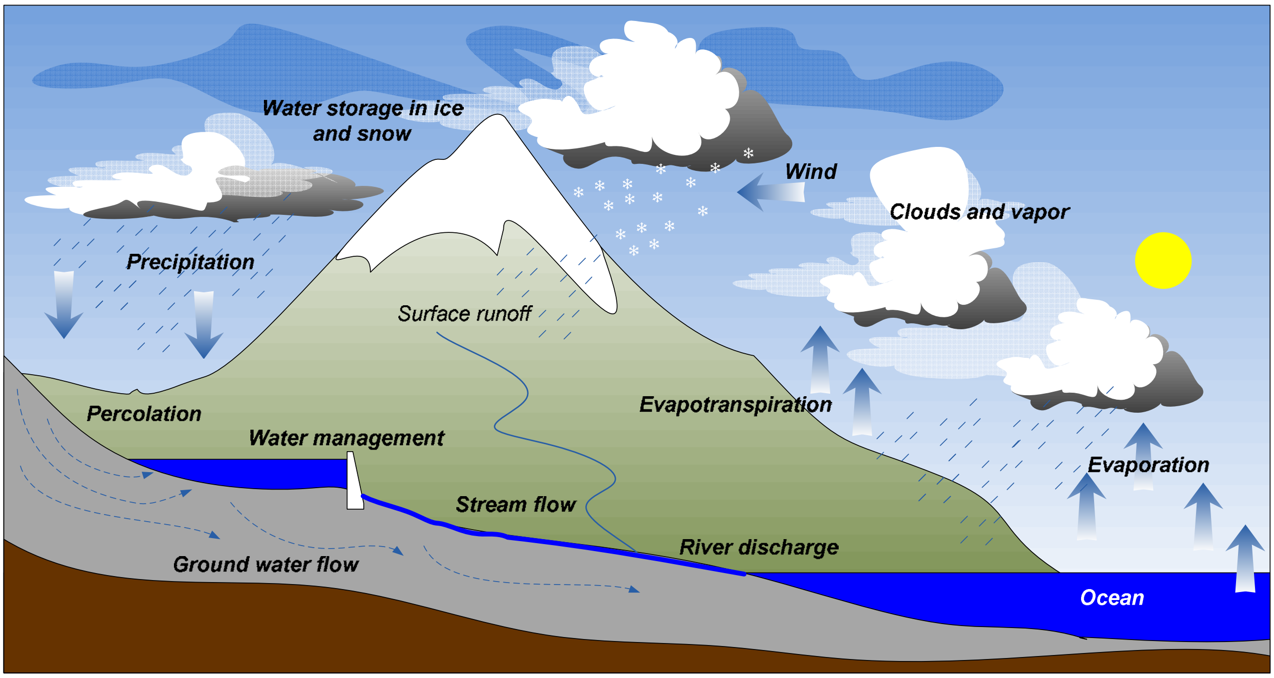


Figure 4.1 Water cycle

Most precipitation falls back into the oceans or onto land, where the precipitation flows over the ground as surface runoff. A portion of the runoff enters rivers through valleys in the landscape, with streamflow moving water towards the oceans. Runoff and groundwater are stored as freshwater in lakes. Not all runoff flows into rivers. Much of it soaks into the ground through infiltration. Some water infiltrates deep into the ground feeding aquifers, which store huge amounts of freshwater for long periods of time. Some infiltration stays close to the land surface and may seep back into surface-water bodies (and the ocean) as groundwater discharge. Some groundwater finds openings in the land surface and comes out as freshwater springs. Over time, the water returns to the ocean, where the water cycle started.

4.2 Precipitation

In meteorology, precipitation is any product of the condensation of atmospheric water vapour which is deposited on the Earth's surface. It occurs when the atmosphere becomes saturated with water vapour and the water condenses, falling out of solution (it precipitates). Moisture overridingly resulting from colliding weather fronts is a usual the reason for precipitation production. Precipitation is a major component of the water cycle, and is responsible for depositing most of the fresh water on the planet. The standard way of measuring rainfall or snowfall is with rain gauges. Precipitation intensity records are available from meteorological gauging stations and may show enormous regional differences.

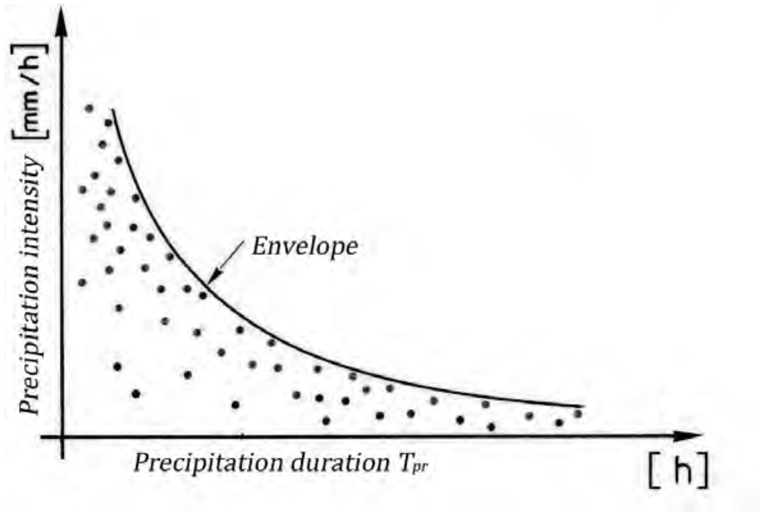


Figure 4.2 Precipitation intensity for a specific gauging station

Well-equipped gauging stations measure the duration and the total quantity of water in [mm] of every rain event. By dividing the quantity of precipitation h_{pr} by the precipitation duration T_{pr}, we obtain precipitation intensity i_{pr}.

$$i_{pr} = \frac{h_{pr}}{T_{pr}}$$

The precipitation intensity is measured in [mm/min], [mm/h] or [mm/d]. For each rain event a dot is plotted into a diagram like shown in Figure 4.2. After collecting data from numerous precipitation events, it is clear that the long precipitation events are less intense than the shorter ones.

4.3 Catchment area and watershed

A catchment area is a stretch of land where water from rain or snow-melt drains downhill into a body of water, such as a river, lake, groundwater aquifer, wetland or ocean. The catchment area acts like a funnel, collecting all the water within the area and channelling it into a waterway. Each catchment is separated topographically from adjacent catchments by a geographical barrier such as a ridge, hill or mountain, which is known as a watershed. A watershed, or drainage divide, is the line separating neighbouring catchments. In hilly countries, the divide lies along topographical peaks and ridges, but in flat countries (especially where the ground is marshy) the divide may be invisible – simply a notional line on the ground on either side of which falling raindrops will start a journey to different rivers, and even to different sides of a region or continent. The catchment area includes both the streams and rivers that convey the water as well as the land surfaces from which water drains into those channels, and is separated from adjacent areas by a watershed. Catchment areas can be subdivided into smaller areas in a hierarchical order where a number of small catchments form a large catchment. Simply said, a catchment is characterized by a valley and the river arising out of it. Therefore, the catchment is often called by its river's name.

4.4 Hydrology for hydro power

By analyzing the energy equation it can be seen that there are some factors which are constant (e= efficiency, ρ_w = water density and g= gravity) and others which depend on local circumstances such as the volume flow rate, described as Q. Q is the available amount of water per second (volume flow rate expressed in SI units: [m^3/s]). The larger the available amount of water Q, the larger the outcome from the energy equation

$$P_{net} = Q \cdot h_{gross} \cdot \eta \cdot \rho_w \cdot g \text{ [W]}$$

will be (see chapter 1.1).

To quantify the desired amount of power, it is first necessary to quantify the available runoff or discharge of the utilized water source. According to the already mentioned water cycle, the runoff depends on precipitation and may not be constant throughout the year. Rainfall is subject to seasonal changes and depends on the meteorological circumstances of the concerned area. This means that reliable flow data comes from long-term data. But unfortunately such databases do not exist for all rivers and streams in the world. The best way to collect these data is by measuring the runoff with an appropriate device over a period of a few years. Alternatively, it is possible to make flow predictions by analyzing the rainfall pattern of the involved catchment area. The hydrology study should be based on at least a year of daily records or should correlate to a recorded water source in a nearby area. To make exact predictions and to ensure that no dry or wet year falsifies the expected flow rate, records for many years are required.

These are common methods for collecting flow data:

- Flow measurement from river runoff
- Flow prediction using the area-rainfall method
- Flow prediction using the correlation method

In areas where precipitation appears as snowfall for several months, flow rate measurements of the concerned river are indispensable. The daily flow records should be collected in a chronological order and displayed in a hydrograph.

4.5 Hydrograph and flow duration curve

In surface water hydrology, a hydrograph is a time record of the discharge of a stream, river or watershed outlet. Rainfall is typically the main input into a watershed and the streamflow is often considered as the output of the watershed; a hydrograph is a representation of how a watershed responds to precipitation. In the case of high alpine areas, the melting water of glaciers and snowfall can be the major water source. Hydrographs are used for sizing the layout of the MHPU scheme, with the hydrograph displaying the river's discharge as a function of time. The hydrograph (black line) displayed in Figure 4.3 shows several peaks during the year which mark periods of heavy rainfall. To achieve representative view of the available flow it is useful to grade the discharge data by creating a flow duration curve (dotted line). The flow duration curve (FDC) shows answer to the amount of discharge in relation to the days of the year.

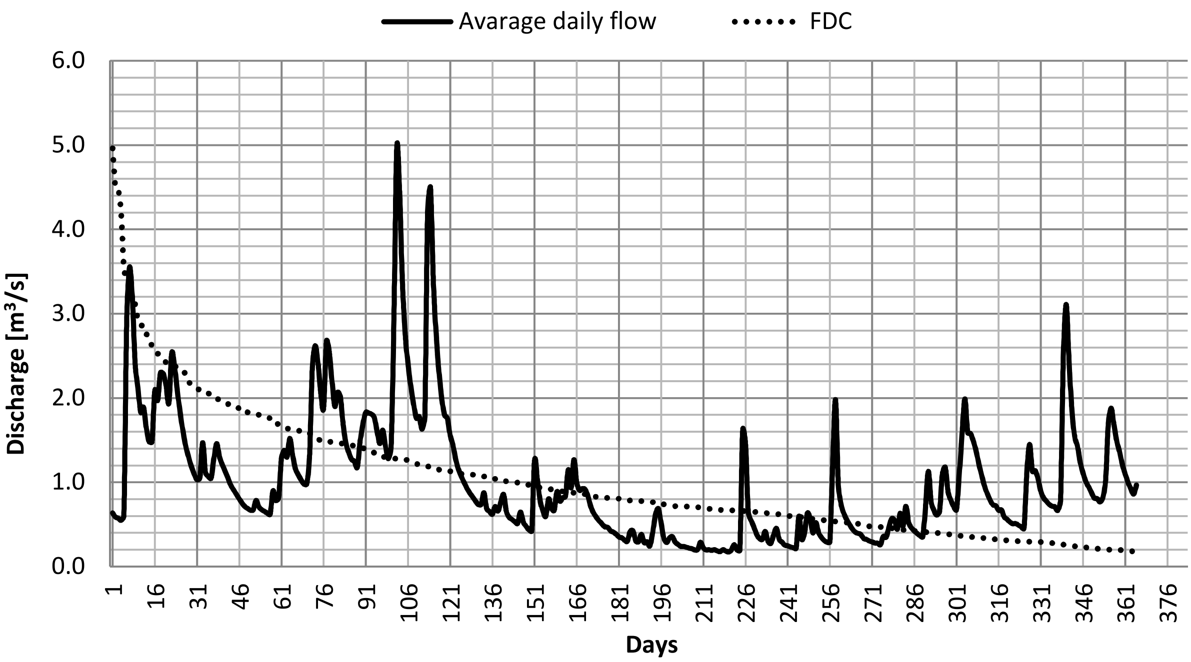


Figure 4.3 Annual hydrograph from a small river

In the example shown in Figure 4.3, a discharge of more than *0.2* m^3/s is available throughout the whole year, or a discharge of more than *1* m^3/s is available on less than 151 days of the year. Hydrographs recorded for many years give information about yearly changes, flood periods, dry and wet years. If a hydrograph of only a one-year period is used, you may have recorded a wet year which will lead to a too optimistic expectation of the output of your MHPU scheme. In order to be on the safe side, use the smallest discharge available from the whole year. In case more energy is used during a special time, you will have to match your energy demand with the energy supply, which depends directly on the discharge. In case the regarded waters are inhabited by fishes, it has to be insured that the riverbed never runs dry.

4.6 Flow prediction

To predict the discharge of a chosen catchment area, we define a system in which we quantify precipitation, storage and discharge. It is more or less the water cycle displayed in the water inventory equation:

$$inflow = outflow \pm storage - evaporation - seepage$$

It expresses the basic principle that, during a given time interval, the total inflow of an area must be equal to total outflow plus the net change in storage and evaporation and seepage loses. Inflow is characterised by the precipitation in the catchment area and outflow is the discharge out of the catchment area, which is some sort of runoff. Storage results when a portion of the precipitation remains in the catchment area for a certain time. Storage may occur in storage basins, swamps, snowfields or glaciers. In dry areas, evaporation puts a significant amount of the inflow back into the atmosphere and must be taken into account.

4.7 Area rainfall method

The first step for a flow prediction is the determination of a control point in your system, most likely the point where the discharge is diverted out of a river or lake. Secondly, the catchment area must be identified by finding the watershed of the surrounding area. Quantify the projected area of the catchment by blocking the map into little squares according to the map's scale.

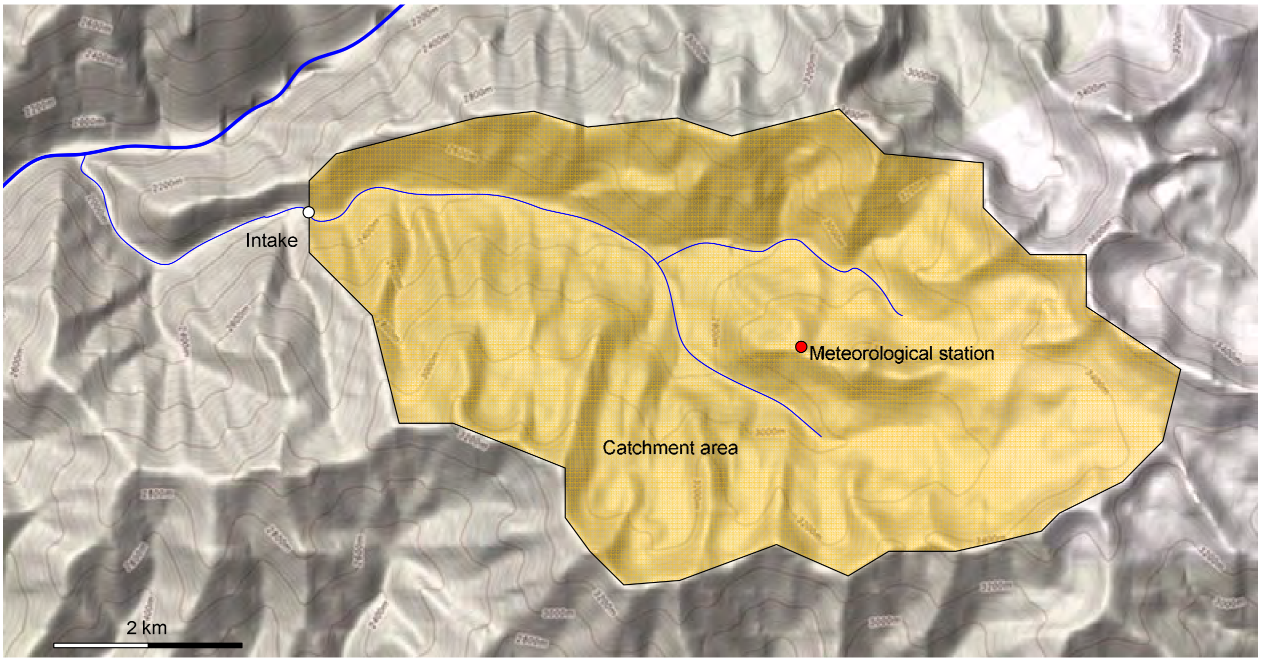


Figure 4.4 Catchment area of an MHPU scheme

An estimated average discharge at the considered intake point can be made from existing precipitation records at a nearby meteorological station or a preinstalled rain gauge. Precipitation is usually measured in mm which is equal to l/m^2. The total volume of rain passing through the control point each year is calculated by

$$V_{net} = A_{catchment} \cdot (h_{rain} - h_{sub} - h_{evaporation})$$

Consider that some of the rain does not pass through your control point, because it seeps into the underground and drains as sub surface flow. A much larger amount evaporates in dry and windy areas. The losses caused by seepage and evaporation are difficult to quantify and therefore it is useful to make assumptions to be on the safe side. Evaporation losses depend on water surfaces, irrigated surfaces and transpiring plants. The potential evaporation losses per day from water surfaces can be calculated by using the Penman formula:

$$h_{pot} = 0.26(0.5 + 0.15 \cdot v) \cdot (e_w - e) \left[\frac{\mathrm{mm}}{\mathrm{d}}\right]$$

Or

$$h_{pot} = 0.26(0.5 + 0.15 \cdot v) \cdot e_w \left(1 - \frac{RH}{100}\right) \left[\frac{\mathrm{mm}}{\mathrm{d}}\right]$$

where e_w = saturation vapour pressure [mbar] or [hPa]; e = present vapour pressure [mbar] or [hPa]; RH = relative humidity [%] and v = wind velocity above the water surface in [m/s]. Evaporation from normal terrain can be calculated with the same approach and by multiplying it with a factor c which represents the local conditions.

$$h_{evap} = c \cdot h_{pot}$$

where c = a factor depending on the season (for alpine areas use $c \approx 0.6$ for November to February, $c \approx 0.7$ for March, April, September and October, $c \approx 0.8$ for May to August). Calculating the losses caused by evaporation is very complicated and can be only done when a rainfall- runoff graph is available. An existing rainfall-runoff relation can also be adapted to predict losses in neighbouring areas. To create your own rainfall-runoff graph, match the yearly average runoff with the coinciding yearly average precipitation and draw it in a coordinate system. Form a regression line as described in chapter 4.9 following the created dot cluster (Figure 4.5 right).

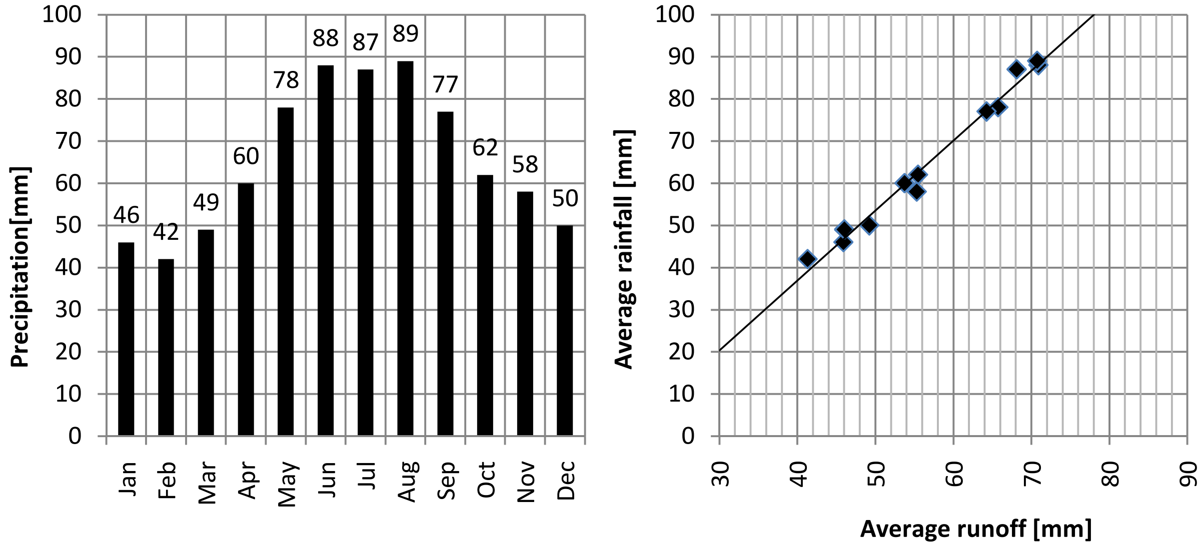


Figure 4.5 Average monthly precipitation recorded by meteorological station (left), rainfall-runoff relation with regression line (right)

For a rough estimate of a yearly or monthly discharge average, divide the net rainfall volume V_{net} by the measure period T concerned.

$$Q = \frac{V_{net}}{T}$$

4.8 Flow prediction by matching precipitation areas

Only in very few cases is long-term data for the proposed catchment areas available. If long-term flow data from neighbouring catchments is available, flow for your site can predicted by transferring the data, taking into account the precipitation area of the catchment. The precipitation area basically describes an area which causes the flow of the precipitating rain and snowfall. Therefore, it is obvious to consider flow in relation to its precipitation area. This is only applicable if the areas being compared have a similar topography, are close geographically and have a similar precipitation pattern. Under these conditions, you can utilise proportionality by calculating with the specific flow ($q = Q/A$), which means

$$q_x = q_1 \quad ; \quad Q_x = Q_1 \cdot \frac{A_x}{A_1}$$

where A_1 is the area of the gauged catchment and Q_1 is the corresponding flow. Should the catchment of interest be between two gauged sites, you may interpolate linearly:

$$Q_x = Q_1 + (Q_2 - Q_1) \cdot \frac{A_x - A_1}{A_2 - A_1}$$

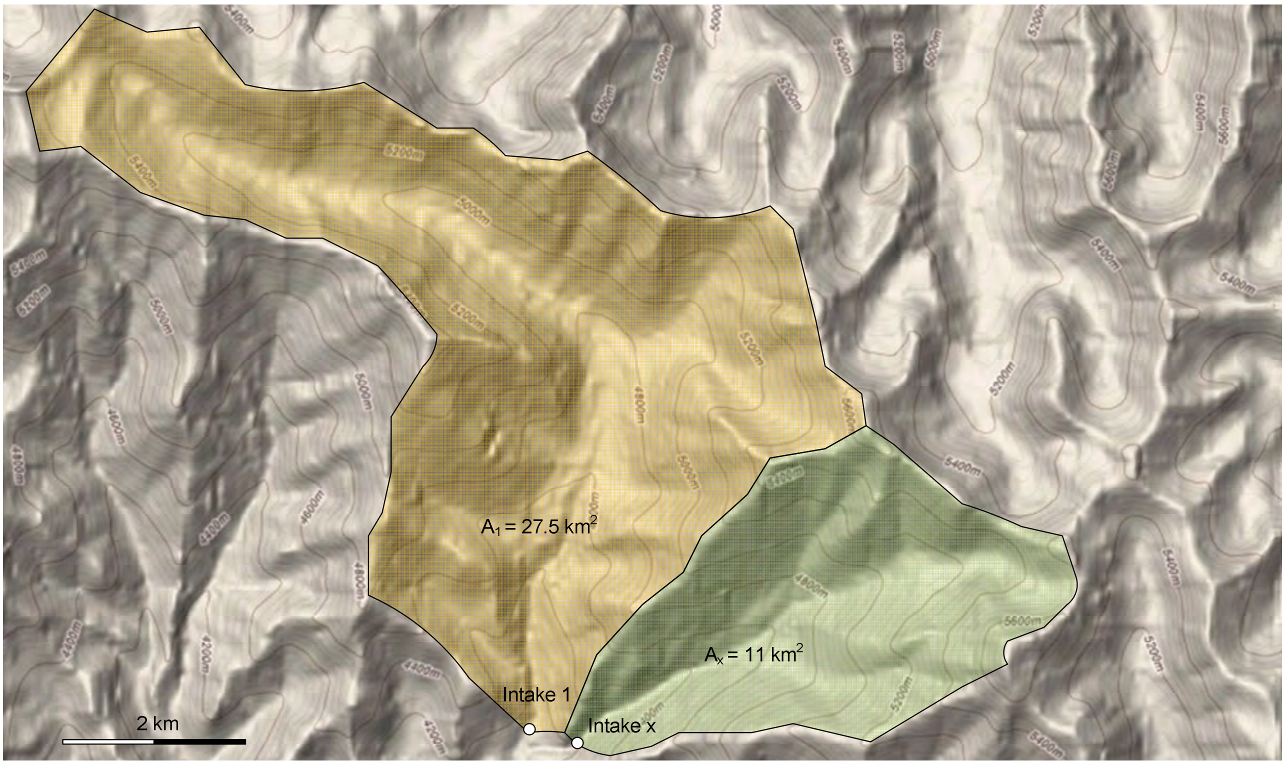


Figure 4.6 Two catchment areas which are close enough to apply data transfer from catchment A_1 to catchment A_x.

Example: Matching precipitation areas

Calculate the expected flow at the proposed intake x for the catchment area A_x by transferring the data from catchment A_1. Use the flow record taken at intake 1 :

Area A_1:	27.5	[km^2]
Area A_x:	11	[km^2]

Month	Discharge A1 [m^3/s]	Discharge Ax [m^3/s]
Jan	0.318	0.127
Feb	0.334	0.134
Mar	0.448	0.179
Apr	0.466	0.186
May	0.554	0.222
Jun	0.641	0.256
Jul	0.556	0.223
Aug	0.379	0.152
Sep	0.381	0.152
Oct	0.446	0.178
Nov	0.587	0.235
Dec	0.428	0.171

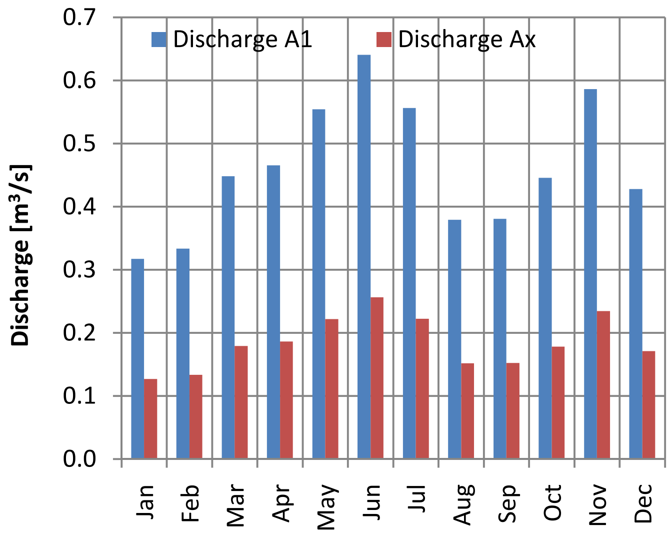


Figure 4.7 Flow data transferred by matching catchment areas A_x and A_1 from Figure 4.6

4.9 Flow prediction by correlation method

The flow can be predicted from a nearby gauged area if one has at least a few flow measurements from the proposed site A_x. For this, find the corresponding values of your measurements must be found in the long-term record of the known area A_1 and drawn into a Cartesian coordinate system. The correlating data will generate a scatter plot which should be roughly aligned as in Figure 4.8. Next, a regression line which will help you to find all corresponding flow values of your suggested site should be drawn. The mathematical approach for correlation by applying linear regression reads as follows:

$$Q_x = k_0 + k_1 \cdot Q_1 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

where

$$k_1 = \frac{\sum_{i=1}^{n}(Q_{xi} - \bar{Q}_x) \cdot (Q_{1i} - \bar{Q}_1)}{\sum_{i=1}^{n}(Q_{xi} - \bar{Q}_0)^2} \; [-]; \qquad k_0 = \bar{Q}_x - k_1 \cdot \bar{Q}_1 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

For a more accurate approach, use multiple correlations like

$$Q_x = k_0 + k_1 \cdot Q_1 + k_2 \cdot Q_2 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

or

$$Q_x = k_0 + k_1 \cdot Q_1 + \cdots k_n \cdot Q_{n2} \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

Once the correlation formula is defined, the flow duration curve and particularly the average monthly- and yearly flow can be derived for the place of interest. Therefore, it is advisable to initialise flow measurements on the proposed site to accumulate data for your correlation as early as possible. Attention should be paid to the fact that flood and low-water seasons must be correlated separately for more detailed predictions.

Example: Correlation method:

During the period of fundraising for the potential site, a number of flow measurements were undertaken in the allocated intake area. The collected flow data $Qx(i)$ and the corresponding data $Q1(i)$ from a long-term record of a nearby catchment is displayed in the table in Figure 4.8.

	Q_1	Q_x
Date	[m3/s]	[m3/s]
10.03.2008	0.980	0.412
24.03.2008	1.460	0.598
30.04.2008	1.560	0.612
06.05.2008	0.928	0.483
20.05.2008	0.652	0.231
25.05.2008	0.648	0.216
30.05.2008	1.260	0.611
01.06.2008	0.777	0.286
01.07.2008	0.314	0.095
12.08.2008	1.620	0.576
20.08.2008	0.418	0.157
03.10.2008	0.470	0.208

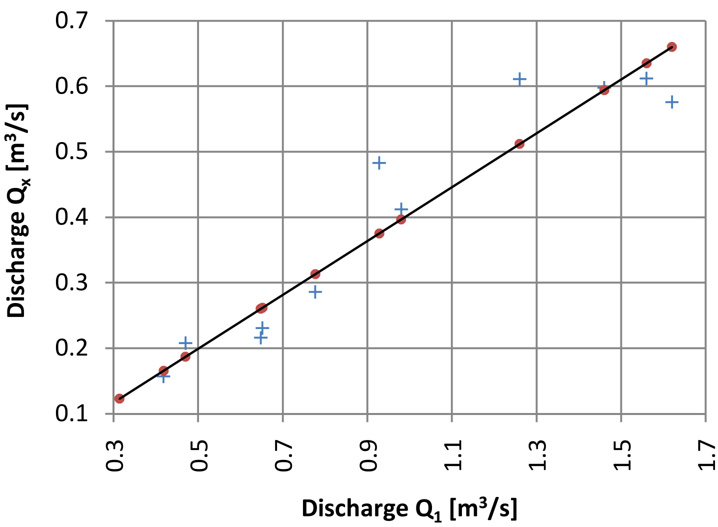


Figure 4.8 Table with corresponding discharge measurements and the resulting scatter plot (blue crosses) and regression line (black line)

First, the scatter plot is created with the pairs of values. Next, a table with 8 columns and as many rows as pairs of values i are available is prepared. Columns 2 and 3 display the discharge data. To create the regression line, the values required to form the regression coefficient k_1 and k_0 according to the above mentioned formula. The last row in the table displays the progress of the regression line.

i	Q_1(i)	Q_x(i)	(Q_1(i)-Q_1m)	(Q_x(i)-Q_xm)	(Q1(i)-Q1m)(Q_x(i)-Q_xm)	(Q_1(i)-Q_1m)^2	Qx(i)
1	0.980	0.412	0.056	0.038	0.002145	0.003145	0.397
2	1.460	0.598	0.536	0.224	0.120217	0.287385	0.594
3	1.560	0.612	0.636	0.238	0.151547	0.404602	0.635
4	0.928	0.483	0.004	0.109	0.000446	0.000017	0.375
5	0.652	0.231	-0.272	-0.143	0.038816	0.073939	0.262
6	0.648	0.216	-0.276	-0.158	0.043526	0.076130	0.260
7	1.260	0.611	0.336	0.237	0.079736	0.112952	0.512
8	0.777	0.286	-0.147	-0.088	0.012892	0.021585	0.313
9	0.314	0.095	-0.610	-0.279	0.170014	0.371998	0.123
10	1.620	0.576	0.696	0.202	0.140783	0.484532	0.660
11	0.418	0.157	-0.506	-0.217	0.109657	0.255952	0.166
12	0.470	0.208	-0.454	-0.166	0.075237	0.206040	0.187
				total:	0.945016	2.298277	

Table 4.1 Calculation of the regression line

The calculation, in detail, proceeds as follows:

Calculate the average discharges of Q_1 and Q_x:

$$\bar{Q}_1 = \frac{\sum_{1=1}^{n} Q_1}{n} = 0.924 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]; \quad \bar{Q}_x = \frac{\sum_{1=1}^{n} Q_x}{n} = 0.374 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

Calculate the regression coefficient k_1 by using the results from Table 4.1:

$$k_1 = \frac{\sum_{i=1}^{n}(Q_{xi} - \bar{Q}_x) \cdot (Q_{1i} - \bar{Q}_1)}{\sum_{i=1}^{n}(Q_{xi} - \bar{Q}_0)^2} = \frac{0.945016}{2.298277} = 0.411\ [-]$$

Now calculate the regression coefficient k_0 by using k_1 and the average discharges of Q_1 and Q_x:

$$k_0 = \bar{Q}_x - k_1 \cdot \bar{Q}_1 = 0.374 - 0.411 \cdot 0.924 = -0.006 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

The function of the regression line reads as follows:

$$Q_x = k_0 + k_1 \cdot Q_1 = -0.006 + 0.411 \cdot Q_1 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

If we enter the FDC values from the gauged area into the formula above, we obtain the FDC of the unknown area (Figure 4.9).

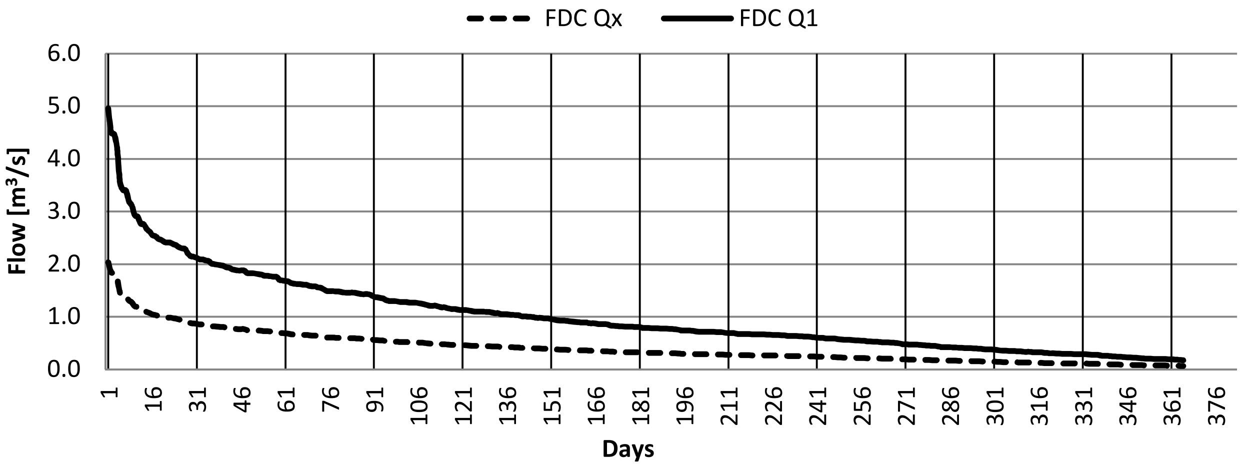


Figure 4.9 Black line: FDC of gauged catchment; dotted line: FDC of un-gauged catchment

4.10 Discharge measurement

Measuring the discharge of a river is the most reliable way to obtain flow data for your project. The advantage is that the real amount of water passing through the control section is recorded, without considering any unknown quantities like evaporation- and seepage losses. The disadvantage, however, is the fact that measure devices for larger rivers are more elaborate and costly to maintain. Also, it is difficult to organize a person to record the data on the spot.

4.11 Measuring river runoff with the velocity-area method

According to the continuity equation for a fluid of constant density flowing through a known cross-sectional area A, the product of A and the average velocity v will be equal to the volume flow rate Q:

$$Q = A \cdot v \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

When applying the velocity area method in the fields, it is important to choose a section as regular as possible with no pools, or changes in slope, inflows and outflows. Make a schematic drawing of the average cross-section A of the riverbed and calculate the cross-sectional area. Mark the beginning and end of the test track L and prepare a float body (e.g. a half-filled bottle). Now drop the float body a few meters before your test track into the centre of the river to allow acceleration of the float body. Take the time from where the body enters the test track until it reaches the measured end. Repeat this several times and calculate the average velocity by leaving out the slowest measurement. Due to uneven velocity distribution within the river's cross-section, the average velocity should be reduced by a correction factor c adapted to the cross-sections shape.

$$Q = A \cdot v = A_{mean} \cdot \frac{L}{t} \cdot c$$

Correction factors:

- Concrete channel, rectangular section, smooth $c = 0.85$
- Large, slow, clear stream $c = 0.75$
- Small, regular stream, smooth bed $c = 0.65$
- Shallow ($d \leqq 0.5$ m), turbulent stream $c = 0.45$
- Very shallow, rocky stream $c = 0.25$

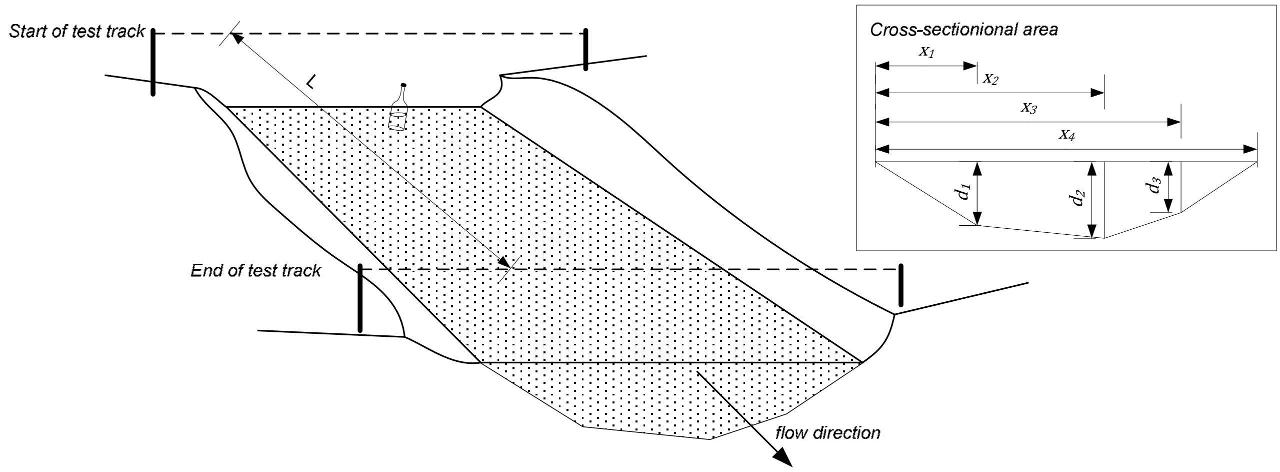


Figure 4.10 Runoff measurement using the velocity area method

The precision of the above-mentioned measuring method of volume flow rate increases if you choose a straight, long section with a smooth riverbed for the test track. Rocks and other large obstacles within the test track cause disturbances and turbulences, which can falsify the results.

4.12 Measuring river runoff with a measure weir

A measure weir has a clearly defined cross-section for which the overflow is deduced empirically or analytically. Unlike the velocity-area-method, there are few requirements when you apply a measure weir for your discharge research. The measure weir must be placed perpendicularly to the river. With the backwater piling up in the headrace, the oncoming flow is very even. The volume flow rate Q is quantified by measuring the difference in height h between the weir crest and the water level. There are different shapes to measure weirs, but rectangularly or triangularly shaped weirs are the easiest to produce and to apply in the field. For rectangular measure weirs, the following formula can be used to compute the volume flow rate.

$$Q(h) = \frac{2}{3} \cdot \mu \cdot b \cdot \sqrt{2 \cdot g} \cdot h^{1.5} \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

where b is the width of the weir opening, h the measured height and μ a factor which can be assumed with ≈ 0.6.

When preparing the measure weir, it is important to choose the correct weir geometry, because there are certain restrictions which apply to the weir formula.

$$w \geq 0.3\ [\mathrm{m}];\ \frac{h}{w} \leq 1; 0.025\ [\mathrm{m}]\ \leq h \leq 0.8\ [\mathrm{m}]$$

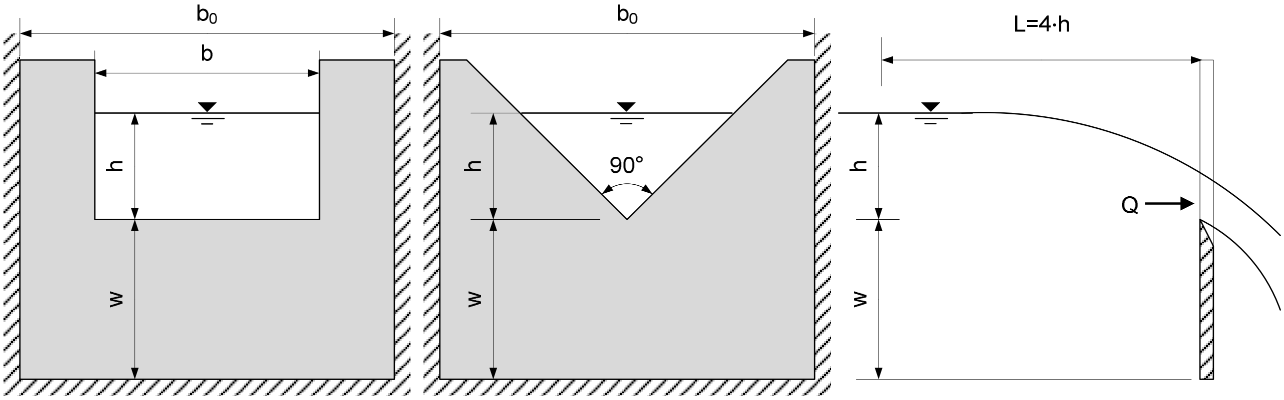


Figure 4.11 Rectangular and triangular measure weirs

Due to the increasing velocity towards the weir crest, the height h drops to a critical height. Therefore, the measurement of h should take place about $4 \cdot h$ away from the weir in the direction opposite to the flow (Figure 4.11.).

For small water amounts, the triangularly shaped measure weir, also called Thomson weir, is used. The advantage of the Thomson weir is its independence from turbulence in the headrace. Triangularly shaped measure weirs are standardized by ISO for a notch angle of 90 degrees.

$$Q(h) = 1.352 \cdot h^{2.483} \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

$$w \geq 0.45\ [\mathrm{m}];\ \frac{h}{w} \leq 0.4; 0.05\ [\mathrm{m}] \leq h \leq 0.38\ [\mathrm{m}];\ \frac{h}{b_0} \leq 0.2[\mathrm{m}]$$

By considering the above-mentioned restrictions, the error can be kept below 1 %.

To collect data for a longer period, it is advisable to install a permanent measuring device or to find a river section which is not likely to change due to deposits of debris or erosion. Choose a narrow section, where the water level rises visibly, thus increasing discharge and make a number of reference flow rate measurements to create a calibration curve. The values for the calibrations curve can either be collected by a measure weir or from measurements using the velocity-area method. A stationary, mounted measuring gauge provides the daily water level for a long term record.

4.13 Flood prediction

A flood is a relatively high flow or stage in a river, markedly higher than the usual. It is also the inundation of low land which may result from the flow. A body of water rises, swells and overflows land not usually thus covered. Floods may occur through heavy precipitation, and in mountainous regions are often accompanied by melting snow. The corresponding hydrograph is characteristic with its wave-shaped peak. Hence, a flood event is also called a flood wave because of the flow pattern in a stream, constituting a distinct progressive rise which culminates in a peak, followed by a recession.

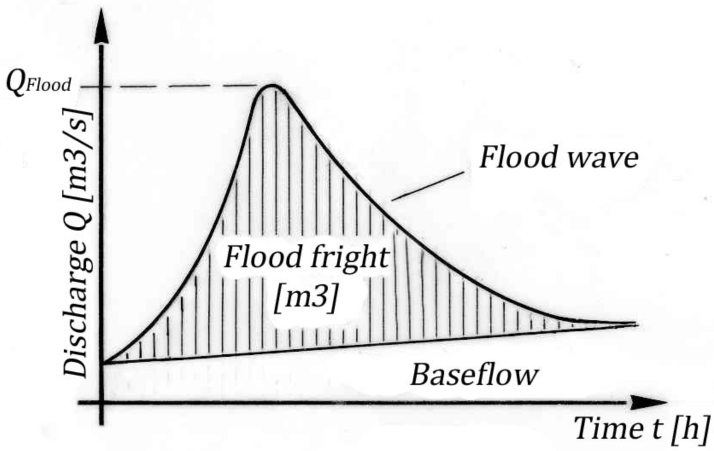


Figure 4.12 Flood hydrograph

Baseflow is the portion of streamflow which comes from groundwater and not directly from runoff. It is assumed that 50% of the water (in mountainous regions less) which percolates down to the shallow ground water contributes to baseflow.

In the following chapters, two approaches among many others will be presented:

- Flood prediction by ratio formula
- Flood prediction from riverbed research

The ratio formula takes a heavy rain event as the basis to simulate a flood wave. The riverbed research method quantifies a flood by analysing the shape, inclination and roughness of the riverbed.

4.14 Flood prediction using the ratio formula

Flood prediction using the ratio formula involves calculating the precipitation area and assumed intensity of a precipitation event. *Rainfall intensity* is the rate at which rainfall occurs expressed in depth units per unit time [mm/h]. It is the ratio of the total amount of rain to the length of the period in which the rain falls.

$$i_{pr} = \frac{Q_{pr}}{A} \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

Assume a known catchment area A [m^2] is being precipitated by heavy rainfall of an intensity i_{pr} for period of time T. If the rainfall drains without evaporation-, storage- and seepage loses into the next river, after a certain time the discharge of the river must be roughly the amount of

$$Q_{flood} = i_{pr} \cdot A \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

This would mean that the flood runoff Q_{flood} in the river must be equal to the amount of rain pouring on the precipitation area. In reality, the above-mentioned losses cause a decrease in the real flood runoff which can be calculated by defining a reduction coefficient α_s:

$$Q_{flood} = \alpha_s \cdot i_{pr} \cdot A \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

where

$$\alpha_s = \frac{Q_{flood}}{i_{pr} \cdot A} \; [-]$$

For rough flood predictions, use the following standard values of the runoff coefficient α_s for the following types of landscape:

- Fields and meadows $\alpha_s \approx 0.1$
- Plantations and forest $\alpha_s \approx 0.1$
- Rocky slopes $\alpha_s \approx 0.2$
- Urban settlements $\alpha_s \approx 0.4$

Since the intensity decreases over the time of precipitation, the critical rainfall must be found by calculation of the time which the water requires to enter the river plus the time which is needed for the water to reach the section of interest (e.g. your proposed intake).

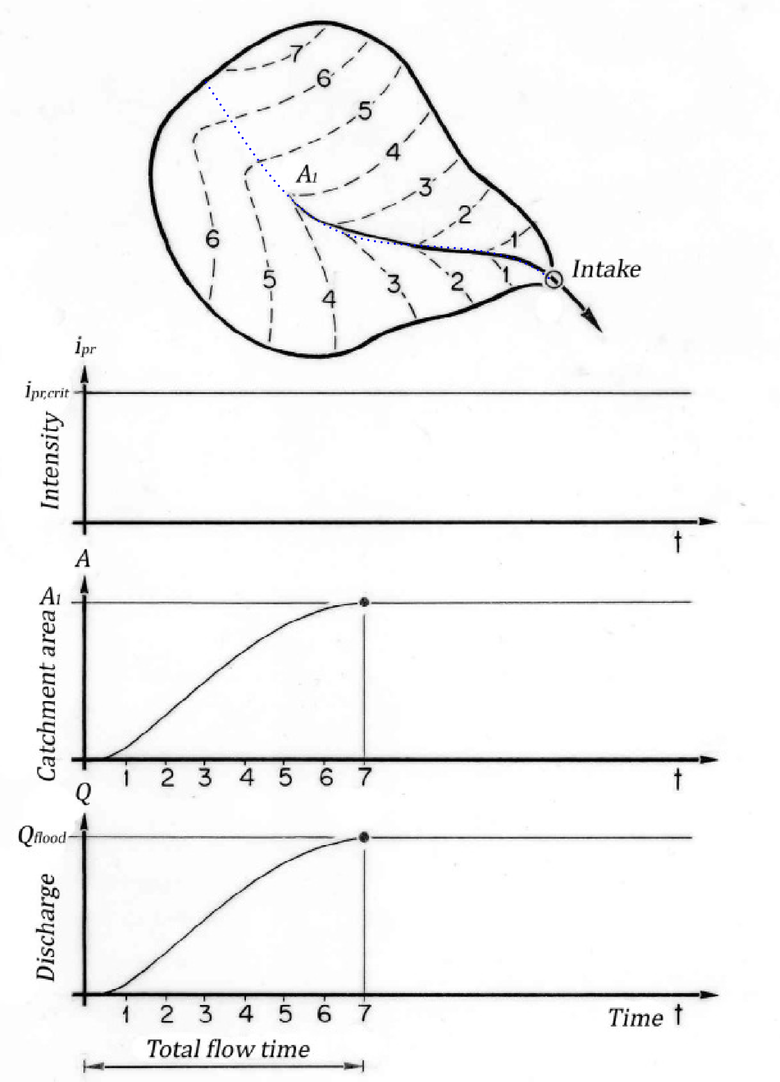


Figure 4.13 A sudden precipitation event strikes catchment area A_1.

Area A_1 distributes its water to the concerned section according to its isochrones, displayed in Figure 4.13. Firstly, the water must drain from the surface into the river over the period of time T_{drain}. Secondly, the water requires a time T_{flow} to reach the outlet of your catchment, in this case, your proposed intake. The time passing until the complete area A_1 is involved in the runoff (T_{drain} + T_{flow}) is called the *total flow* time $T_{tot.flow}$. This period is the determining factor for the choice of precipitation intensity $i_{pr.crit}$. As shorter precipitation events are of higher magnitude, try to identify the shortest precipitation duration T_{pr}, which is:

$$T_{pr} = T_{tot.flow} = T_{drain} + T_{flow}$$

For rough flood predictions use following standard values for T_{drain}:

- Shallow terrain T_{drain}=20 to 30 [min]
- Hilly terrain T_{drain}=15 to 20 [min]
- Steep terrain T_{drain}=10 to 15 [min]

To estimating the flow time in the river, you can use the Manning formula to find the flow velocity

$$v_{flow} = \frac{1}{n} \cdot \left(\frac{A}{U}\right)^{\frac{2}{3}} \cdot \sqrt{S}$$

where n represents the roughness of the riverbed according to Manning with the unit [s/m$^{1/3}$], U means the wetted perimeter measured in [m], and S is the slope measured in [m/m]. The river length L_{flow} divided by the flow velocity results in the flow time

$$T_{flow} = 86.4 \cdot \frac{L_{flow}}{v_{flow}} \quad [\mathrm{h}]$$

Both flow time plus the drain time result in the precipitation duration T_{pr}. If the surface texture of your catchment area is not properly researched, use the following estimation to calculate the precipitation time.

$$T_{pr} = T_{tot.flow} = \left(0.868 \cdot \frac{L_{F.tot}{}^3}{\Delta h}\right)^{0.385} \quad [\mathrm{h}]$$

where $L_{F.tot}$ [km] is the complete flow distance a rain drop has to move from the edge of the watershed to the outlet of the catchment area (Figure 4.13, blue dotted line). Δh [m] is the difference in head, measured from the watershed to the outlet.

Use the ratio formula method only for small catchment areas of ≈ 20 km^2.

Example: Flow prediction with the ratio formula

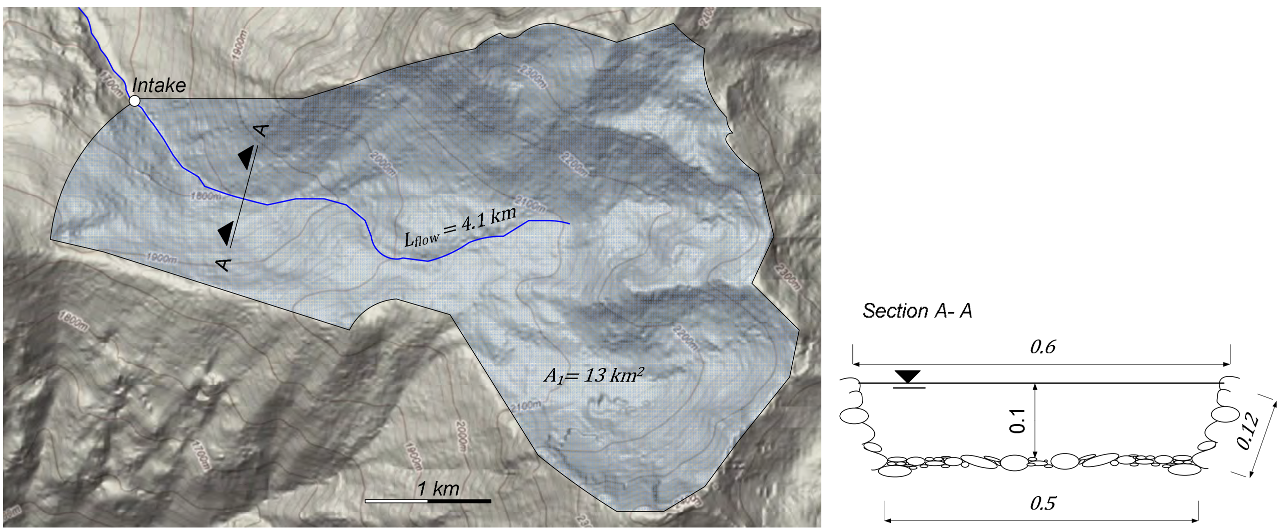


Figure 4.14 View of catchment and typical section of a river

While planning an intake, you are seeking a design flow which will strike your structure only in the worst case scenario. The building should be strong enough to withhold a flood event which statistically appears once in 50 years (flood with a recurrence interval of 50 or years).

Your catchment is in an alpine area of an average altitude at ≈ 2000 m above sea level, sizing about 13 km^2. Fifty percent of the area is covered by pine trees. The other 50 percent is rocky slopes. A small river cleaving through the valley measures 4.1 km backwards from the proposed intake and has an average slope of 9.5 %. The roughness coefficient according to Strickler is assumed to be $k_{st} \approx 20$. Use the ratio formula to define the flood discharge of your catchment area. Calculate the total flow time $T_{tot.flow}$ by taking the drain time T_{drain} and flow time T_{flow} into account. The rain intensities are displayed in Figure 4.14, where a different line is valid for each recurrence interval of 2.33, 10, 50 and 100 years. Assume for T_{drain}= 15 min (steep terrain). The wetted perimeter U of the channel section is calculated by adding together the width of bottom and bank slopes.

$$U = 0.5 + 2 \cdot 0.12 = 0.74 \text{ [m]}$$

Thus flow velocity is

$$v_{flow} = 20 \cdot \left(\frac{0.055}{0.74}\right)^{\frac{2}{3}} \cdot 0.095^{\frac{1}{2}} = 1.1 \left[\frac{\text{m}}{\text{s}}\right]$$

and

$$T_{flow} = \frac{4100}{1.1} = 3727\ [s] = 62\ [min]$$

total flow time

$$T_{tot.flow} = 15 + 62 = 77\ [min] \approx 1.3\ [h]$$

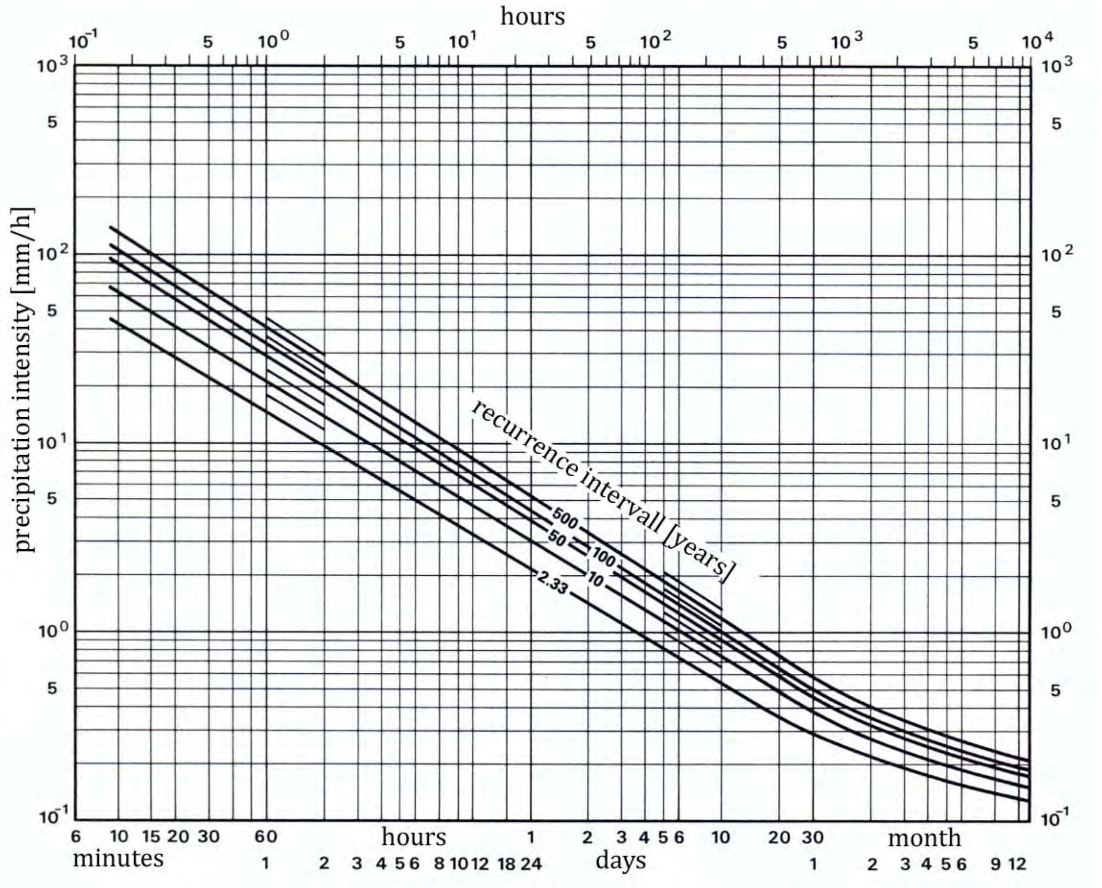


Figure 4.15 Precipitation intensity diagram from the local gauging station

Once you know the precipitation duration $T_{pr} = T_{tot.flow}$ choose the correct intensity from the diagram, in this case, $i_{pr} \approx 25$ mm/h. Now you may obtain the mean runoff coefficient (remember the area is partly covered with forest).

$$\alpha_s = \frac{\frac{A_1}{2} \cdot \alpha_{s.forest} + \frac{A_1}{2} \cdot \alpha_{s.slopes}}{A_1} = \frac{6.5 \cdot 0.1 + 6.5 \cdot 0.2}{13} = 0.15\ [-]$$

Now compute the peak flow

$$Q_{flood} = \alpha_s \cdot i_{pr} \cdot A_1 = 0.15 \cdot 25 \cdot \left[0.001 \frac{\mathrm{m}}{\mathrm{mm}} \cdot \frac{1}{3600} \frac{\mathrm{h}}{\mathrm{s}}\right] \cdot 13 \cdot \left[1000^2 \frac{\mathrm{m}^2}{\mathrm{km}^2}\right] = 13.54\ \left[\frac{m^3}{s}\right]$$

Be careful when you mix different units within the formula. It is advisable to write down all unit conversions.

4.15 Flood prediction from riverbed research

Flood prediction by observing flood traces in a riverbed supplies no accurate answer about how often a flood appears. It is rather a rough approach for obtaining an idea what approximate flow rate the river has seen in recent past. When we analyse the shape of a river during a field trip, we can see some indicators of previous flood events. For example:

- Shiny washed stones within the flood plain
- Missing vegetation within the flood plain
- Silt, mud and floating deposits along the bank slopes
- Floating deposits hanging in trees marking the flood stage.

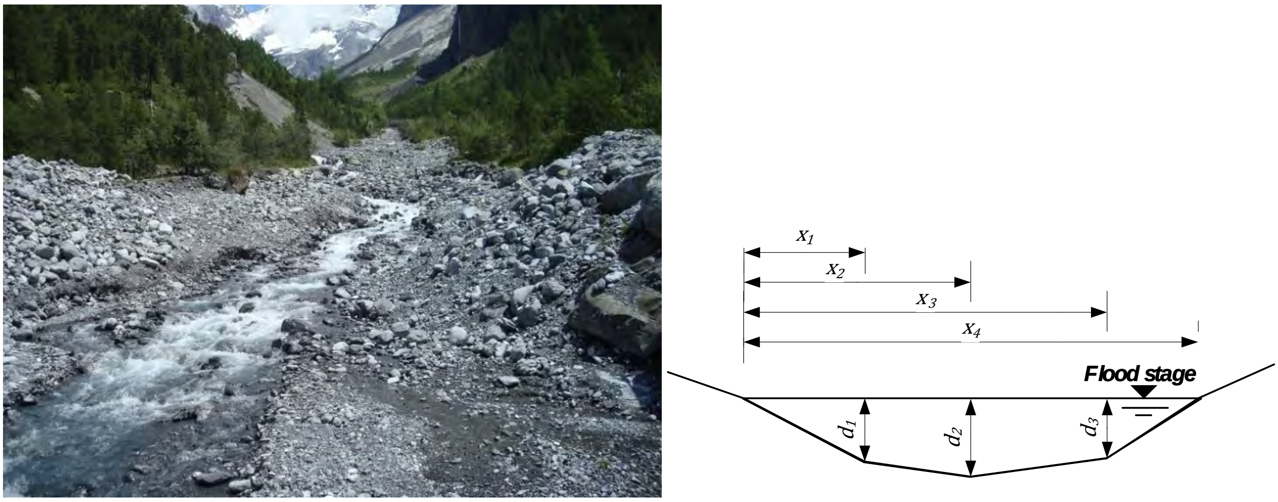


Figure 4.16 Riverbed with flood plain (left), assumed cross-sectional flood area (right) with wetted perimeter (blue line)

Flood prediction from riverbed research depends on the following parameters: roughness of riverbed, average slope of river and shape of river cross-section. You can determine the roughness coefficient n (Manning coefficient), by assuming an average diameter of the pebbles and rocks covering the riverbed. Find the slope S of the river by measuring the difference in head Δh and divide it over a certain difference of length ΔL.

$$S = \frac{\Delta h}{\Delta L}$$

Look for a section where no major drops or waterfalls falsify your slope. Also, areas where backwater is stored, have an impact on your results. Decide on a cross-section which considers your observed flood stage and measure the typical course of your riverbed to calculate the cross-sectional area A and wetted perimeter U (according Figure 4.16). Now obtain the flood discharge by using the Manning-formula described in chapter 3.15.

$$Q = A \cdot \frac{1}{n} \cdot R^{2/3} \cdot \sqrt{S} \left[\frac{m^3}{s}\right]$$

Remember that this method is very imprecise and should be verified by a more accurate procedure. However, in cases where no better data is available, it provides an impression of what flow to expect during a flood event. Table 3.1 shows a variety of roughness coefficients according Manning and Strickler.

For unconsolidated rock and riverbed material, the roughness-coefficient can derived with

$$n = \frac{d_{90}^{\frac{1}{6}}}{26} \; [-]$$

where d_{90} is the predominant diameter (90 %) of stones, covering the riverbed.

5 Intake building

The main task of the intake building is to divert water from a river or lake into an MHPU system via a channel, penstock, or in some cases, to have the water flow directly into the turbine chute. Since the available discharge of the water source may vary due to seasonal changes, the water level at the point of diversion must be hold constant at a certain stage. Hence, the intake must have some control device like a weir or similar barrage structure. A weir is a barrier across a river for the purpose of diverting a part of the runoff into a channel. In some cases, the weir is built to increase the difference between the upper and the lower water level. Further, the intake structure must be capable of managing the bed load which is carried by the river. In low water season, the bed load just contains silt and sand but during flood season, the river might carry heavy boulders. In general, the steeper the river slope is, the more bed load it carries due to increasing flow velocities. Both, low flow Q_{min} and high flow Q_{max} (or Q_{flood}) must be considered when planning the weir section of the intake. The diverting section must be large enough to manage the diverted flow for your MHPU or irrigation system (which will be called Q_{design}). In case of revision work, the inlet must be lockable by a regulation gate or a similar structure. In the context of small hydro power schemes, we look at two common types of intakes which can be applied in most cases:

- Side intake
- Ground intake

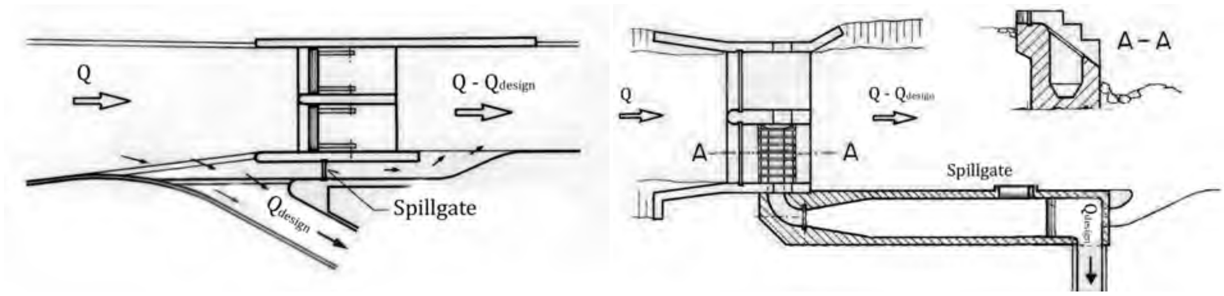


Figure 5.1 Side intake (left), ground intake (right)

The side intake is equipped with a diverting channel as well as a scouring channel or spill gate to return the bed load to the river. The ground intake is equipped with a trench that is arranged perpendicularly to the river. The trench is covered with a ground rake (trench rake) to avoid debris from falling into the trench. Both intake designs are completed by a weir to hold a minimum water level and allow floods to discharge.

Criteria for choosing either a side intake or a ground intake are listed in the following chart:

	Side intake	**Ground intake**
Q_{design}	up to 50% of Q_{river}	up to 100% of Q_{river}
River slope I > 10%	good	very good
River slope 10% > I > 1%	good	good
River slope 1% > I > 0.01%	good	bad
River slope 0.01% > I > 0.001%	good	bad
River slope I < 0.001%	bad	bad

Table 5.1 Assets and drawbacks of side intake and ground intake in relation to design flow and river slope

5.1 Elements of the intake structures

Intake is the general term for: headwork, diversion work or a diversion structure. It is a collective term for all works (weirs, diversion dams, head regulators, upstream and downstream river training works and their ancillary structures) required at intakes at main or principal channels to divert or control river flows and to regulate water supplies into the main channel or channels.

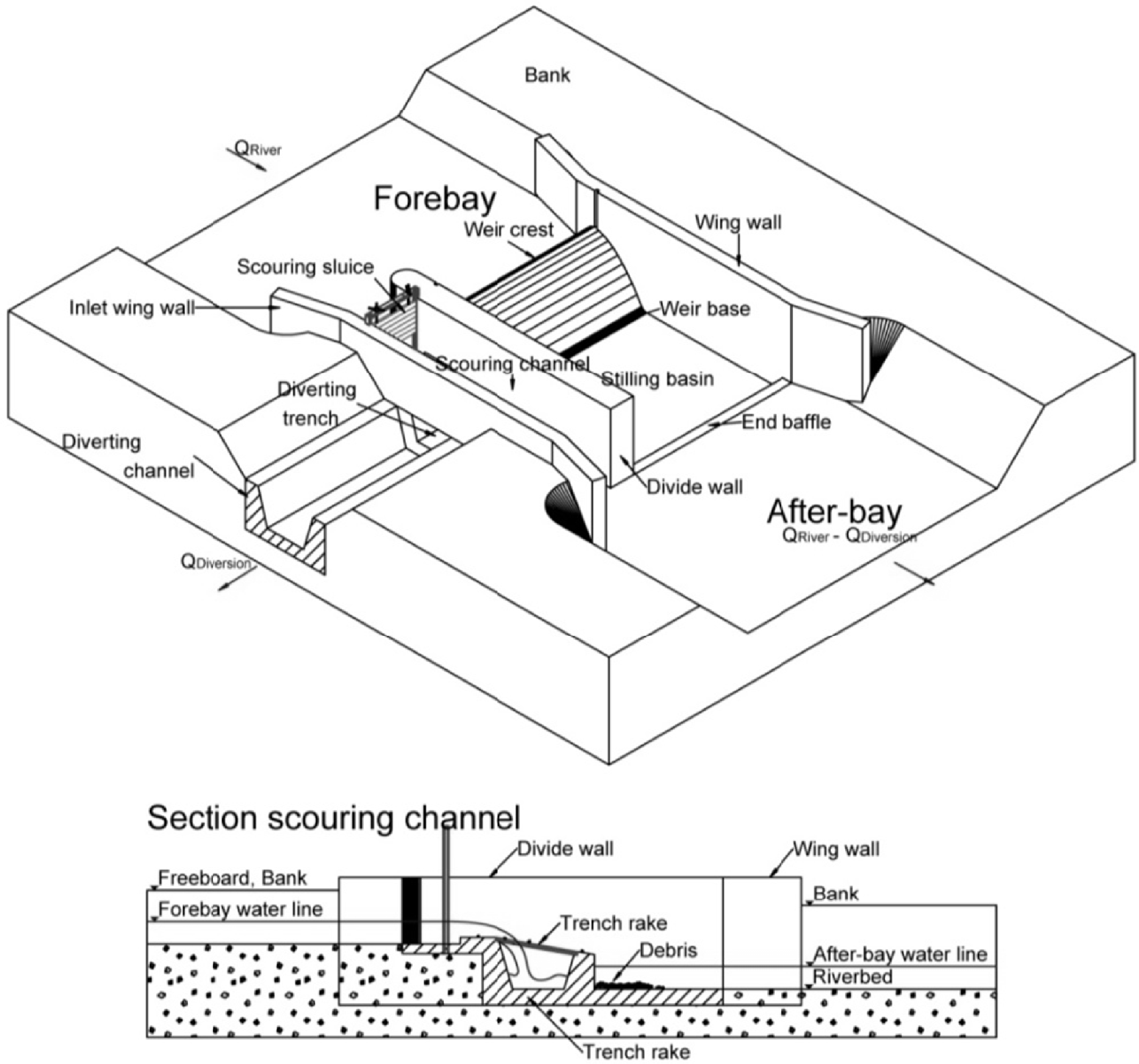


Figure 5.2 Elements of ground intake

Wing walls: The intakes side walls, improving flow conditions up- and downstream of the controlling section. By joining the abundant of the structure to an earth dike or the banks, the wing walls provide a longer path of percolation around the structure.

Scouring channel: The portion of a river channel leading water to the undersluices and away from it to join the river downstream of the weir. Often built to spill debris and silt deposits away from the diverting channel inlet.

Scouring sluices pocket: The portion of river channel upstream of the undersluices bounded by the divide wall, inlet of the head regulator structure and the undersluices.

Stilling basin: A structure below a spillway, chute or drop in which all or part of the energy dissipation occurs and into which kinetic energy is converted into turbulent energy.

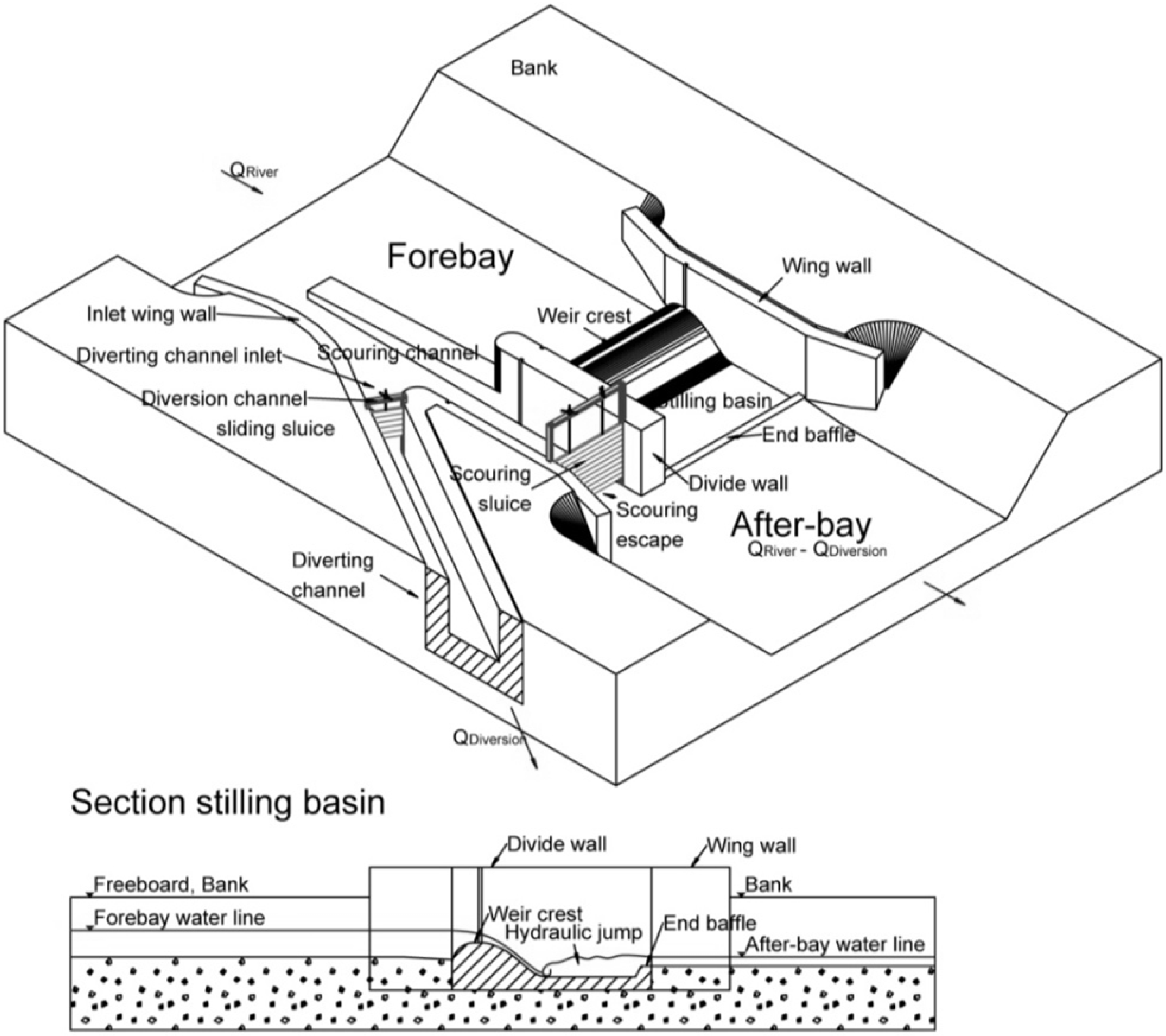


Figure 5.3 Elements of side intake

End baffle: A vertical, stepped slope or dentate wall constructed at the downstream end of a stilling basin.

Freeboard: The difference between the maximum flow line and the top of the bank or structure.

Forebay: The water immediately upstream of any structure. In some cases, this is a reservoir or pond at the head of a penstock.

After-bay or tailrace: The term may be applied to a short stretch of stream immediately after a structure.

5.2 Where to place the intake structure

If you have free choice where to place your intake, you are advised to find a spot in the outer part of a curve (Figure 5.4: section A-A left side, section C-C, right side). The river stretch displayed in the example below has cross-sections with typical scours (bent areas of rivers). A scour is the result of the erosive action of running water in rivers which excavates and carries away material from the bed and banks. Scours may occur in both earth and solid rock material. The erosion activity is concentrated in the outer part of a curve due to its higher flow speed, which is caused by centrifugal forces. Because of its advantageous oncoming flow, these places are ideal for placing a discharge inlet (diversion channel).

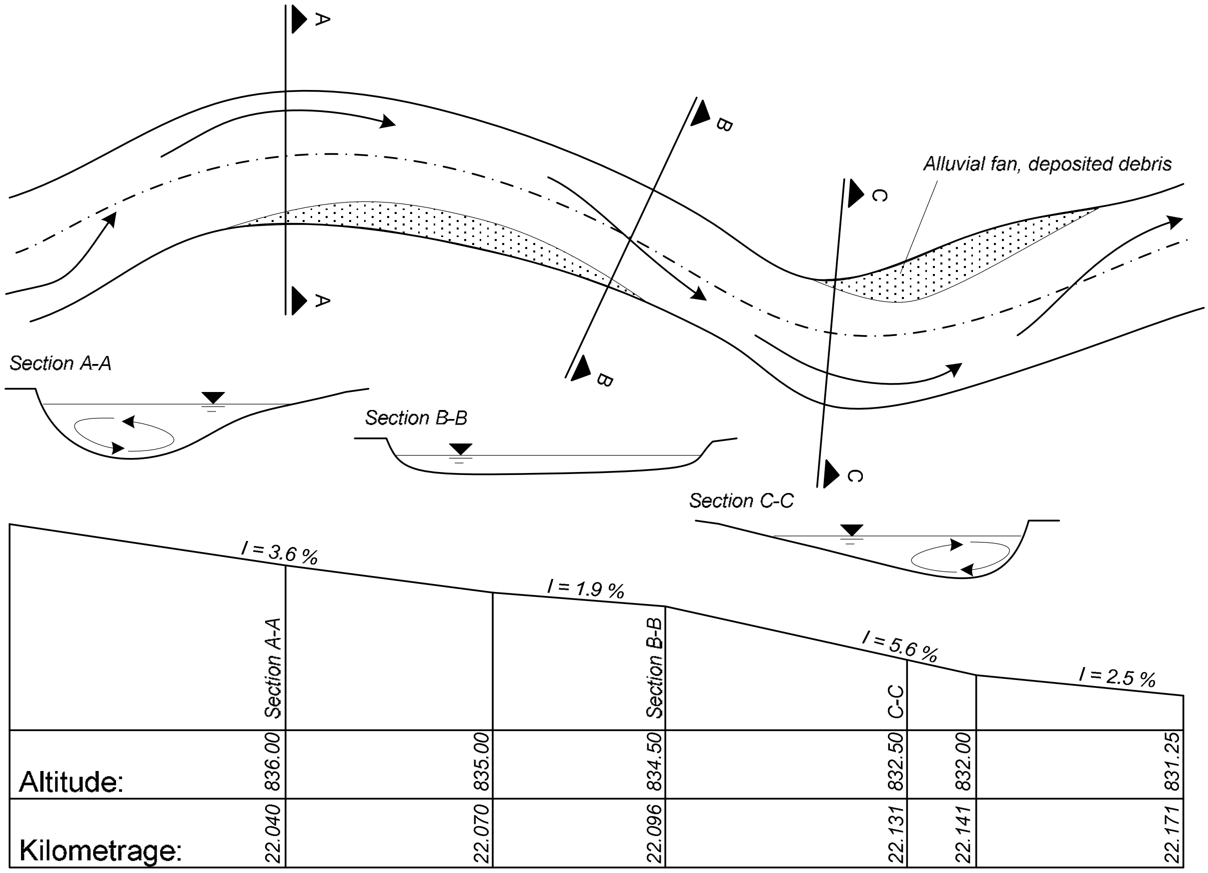


Figure 5.4 River scheme with cross-sections and longitudinal profile

The bed slope may also have some influence on your choice of a potential intake site, because the steeper the river, the shorter the area affected by the piling up backwater will be. Thus, you have to face a more lively stream with the disadvantages of erosion, higher flow speed and bed load.

5.3 Defining the backwater line

Before you start sizing the permanent weir, you need to find a minimum backwater line for your proposed intake structure. One of the determining criteria will be your targeted head, if you are planning a river runoff scheme, or the water level of your headrace, if you are planning a channel-fed MHPU scheme. If the design discharge Q_{design} for your system is known, find an appropriate cross-section area for the diverting channel by defining an approximate flow section. Use the Manning-Strickler formula to find the flow velocity and flow section for your diverting channel.

For example, your design flow $Q_{design} = 0.115$ m^3/s and the flow section A in your concrete channel is 0.75 m by 0.25 m. This means the minimum water stage of your forebay area must be 0.25 m above the river bed. To prevent silt and stones from coming into your channel, place a scouring channel in front of your diversion inlet. Now take the longitudinal profile of the chosen river section and see what impact on the backwater your intake structure may have.

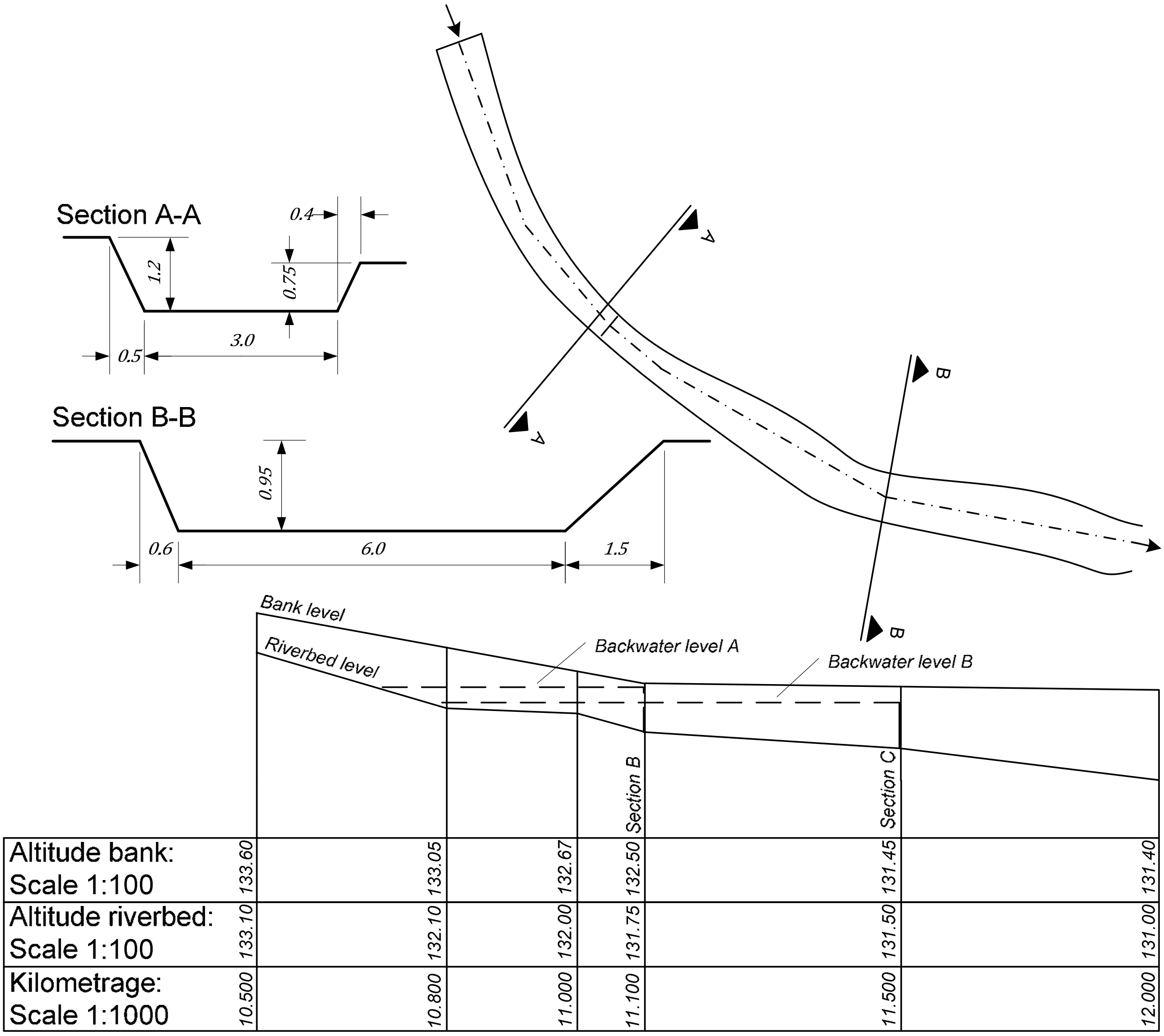


Figure 5.5 Two potential sites (section A-A and B-B) for intake structures

In some cases, the weir causes a very long banked-up water level behind it, which depending on the slope or inclination I of the riverbed. After having drawn the backwater level into the longitudinal profile of the concerned river section, you will notice how long the backwater pile up will be. In the example shown in Figure 5.5, an intake structure placed in section B-B with a weir height of 0.6 m, will have a minimum backwater curve of more than ~700 m. If the weir is placed in section A-A, you will reduce the backwater curve to ~400 m. The bank level in the profile gives information about whether the water stays in the bed or submerges the banks. In section A-A, a flood will most likely flow over the banks. Hence, it is absolutely necessary to consider a freeboard and, if needed, to enlarge the banks by building artificial dams in the concerned areas. Water running around or underneath the weir will excavate it in little time and may cause technical and economical damage. The freeboard must be placed above the flood water level. Also notice that the banked water may not be in a straight, horizontal line. Because of the flowage, the water level rises against flow direction. The exact calculation of water lines is complicated and therefore shall be omitted in the early steps of the project's procedure. For small projects, the above mentioned method is suggested.

5.4 Assessment of the intake structure

When designing an intake structure, the following three requirements must be considered.

1. The assumed design flood Q_{flood} must lead away without causing an unacceptable increase of the backwater or harming the building in any way.
2. The weir body should not be exposed to intolerable negative pressure.
3. The energy generated by the drop of head must be converted without causing erosion on either the continuing riverbed, the stilling basin or the weir itself.

Points one and two are addressed by the assessment of the weir body. Point three is taken care of by the careful design of a stilling basin.

5.5 Sizing of the weir

The hydraulic assessment of the weir body depends on the design flow Q_{flood} as well as on the space you have available at your river section. In the first step of the intake design, you have to find the appropriate width b (Figure 5.6) of the weir which serves as a control device for the backwater level in the forebay, where the diversion channel is tapped. To calculate the potential discharge over the weir, use the Poleni formula, when velocity in the backwater is very slow and therefore $v_0^2/(2 \cdot g) \approx 0$.

$$Q_{flood} = 2.95 \cdot \mu \cdot b \cdot H_B^{3/2} \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

If the width b is limited, then the flooding head H_B is calculated by:

$$H_B = \frac{Q_{flood}^{2/3}}{(2.95 \cdot \mu \cdot b)^{2/3}} \ [\mathrm{m}]$$

In case you want to obtain the needed width b of a certain flood Q_{flood} and a known flooding head H_B calculate the weir width according to the following formula:

$$b = \frac{Q_{flood}}{2.95 \cdot \mu \cdot H_B^{3/2}} \ [\mathrm{m}]$$

In case you want to obtain a weir shape coefficient μ by knowing the flooding head H_B, b and Q_{flood}, calculate according to the following formula:

$$\mu = \frac{Q_{flood}}{2.95 \cdot b \cdot H_B^{3/2}} \ [-]$$

where the width of the weir is described by b and the flooding head H_B, the weir shape coefficient is μ which relates to the weir shape as in Figure 5.6.

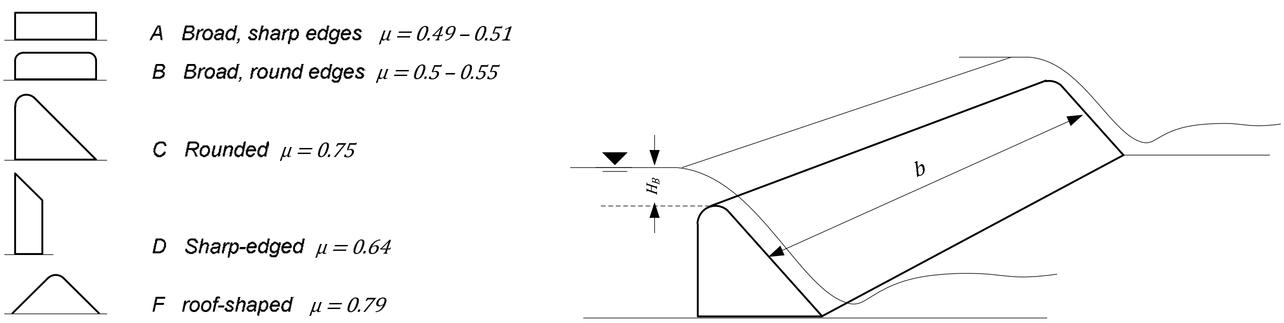


Figure 5.6 Weir crest shapes

5.6 Sizing a standard weir crest

The U.S. Army Corps of Engineers (USCE) developed a standard shape for a weir crest with a high discharge capacity μ and no prohibitive negative pressure on its lower slope. When regarding the peak point s, we differentiate between a left and a right branch. In mathematical terms, this means a curve in the up-water quadrant and a curve in the low-water quadrant.

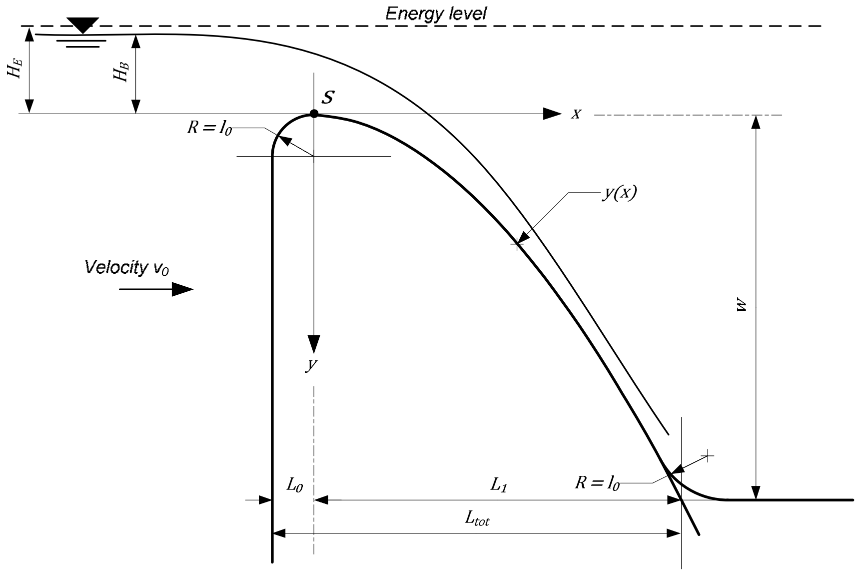


Figure 5.7 Standard weir shape according to USCE

In order to shape the weir crest, one must find the appropriate values for a specific situation. The low-water sloping branch of the weir is constructed by the function in relation of x according to:

$$y(x) = 0.5 \cdot \frac{x^{1.85}}{H_B^{0.85}} \text{ [m]}$$

The fillets are created by a radius which is assumed by

$$L_0 = 0.27 \cdot H_B \text{ [m]}$$

The base width l_1 is

$$L_1 = \left(2 \cdot w \cdot H_B^{0.85}\right)^{\frac{1}{1.85}} \text{ [m]}$$

and

$$L_{tot} = L_0 + L_1 \text{ [m]}$$

The discharge capacity factor in relation of flood head and energy level is calculated by $\mu = 0.75 \left(\frac{H_E}{H_B}\right)^{0.12}$:

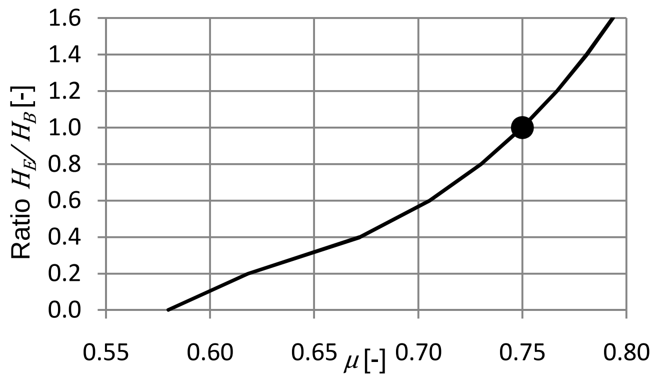


Figure 5.8 Capacity factor for a USCE standard weir crest

5.7 Sizing the trench weir

A trench weir can be seen as a special design of a ground intake as described in 5.5 and is classified as a ground intake. It is suggested for rivers with a strong inclination $I > 1\%$ and heavy bed load. Trench weirs or Tyrolean weirs, are common in mountainous regions, where both river slope and bed load are naturally high. The intake is equipped with a ground rake which allows the water to flow into the diverting channel and thus stopping the debris from entering the channel.

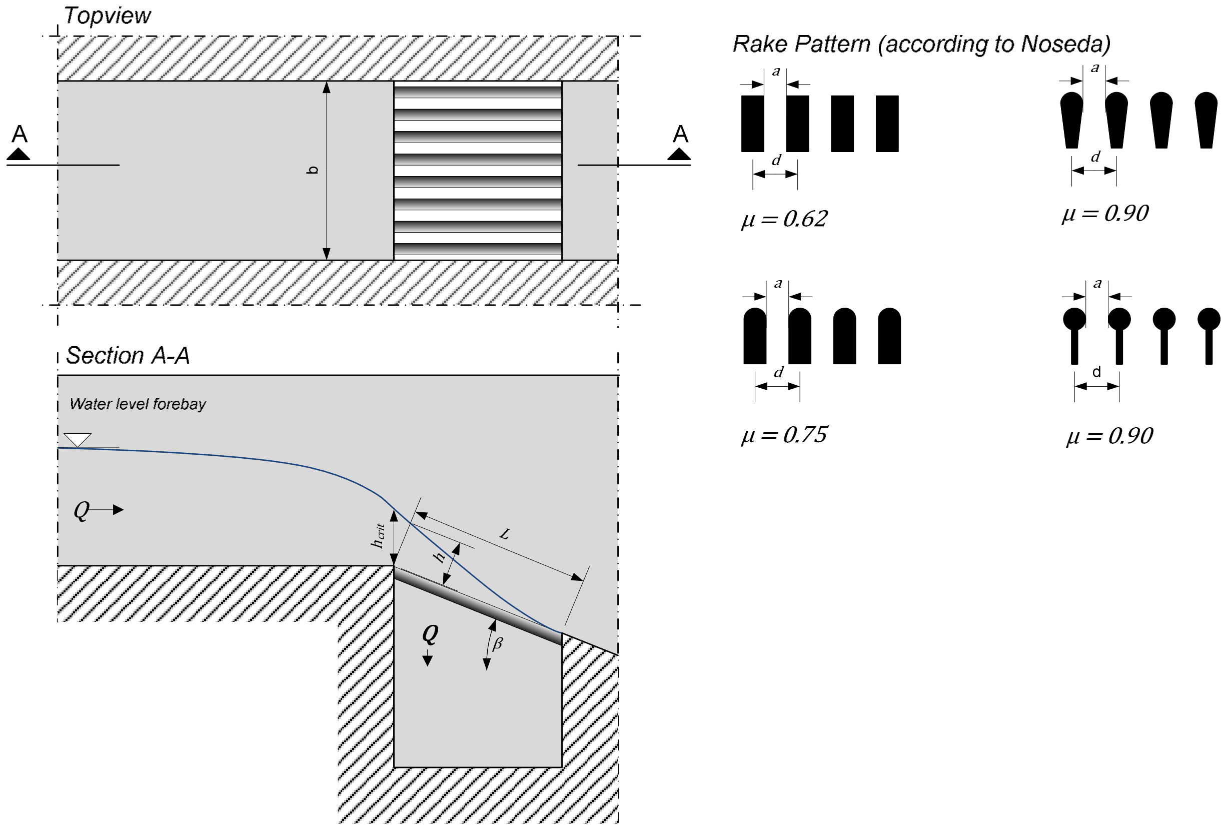


Figure 5.9 Trench rake arrangement

The hydraulic assessment of the rake design is as follows:

$$Q_{design} = \frac{2}{3} \cdot c \cdot \mu \cdot b \cdot L \cdot \sqrt{2 \cdot g \cdot h} \left[\frac{m^3}{s}\right]$$

where

- $c = 0.6 \cdot (a/d) \cdot (cos\beta)^{3/2}$
- a = clearance between rake bars
- d = distance between rake bars
- β = rake slope
- μ = flow coefficient as in Figure 5.9
- b = rake width
- L = rake length
- h_{crit} = critical water depth (~2/3 of water level in forebay)
- $h = h_{crit} \cdot \kappa$ (orthogonal water depth at the beginning of the rake as in Figure 5.10)

In low water season, when the river runoff is smaller than your diversion discharge, the river may run dry below your intake $Q_{river} < Q_{design}$. During high water season, more than the desired amount of flow will enter your diverting trench $Q_{river} > Q_{design}$. You are advised to arrange a spillway after the trench weir to guide surplus water back into the stream.

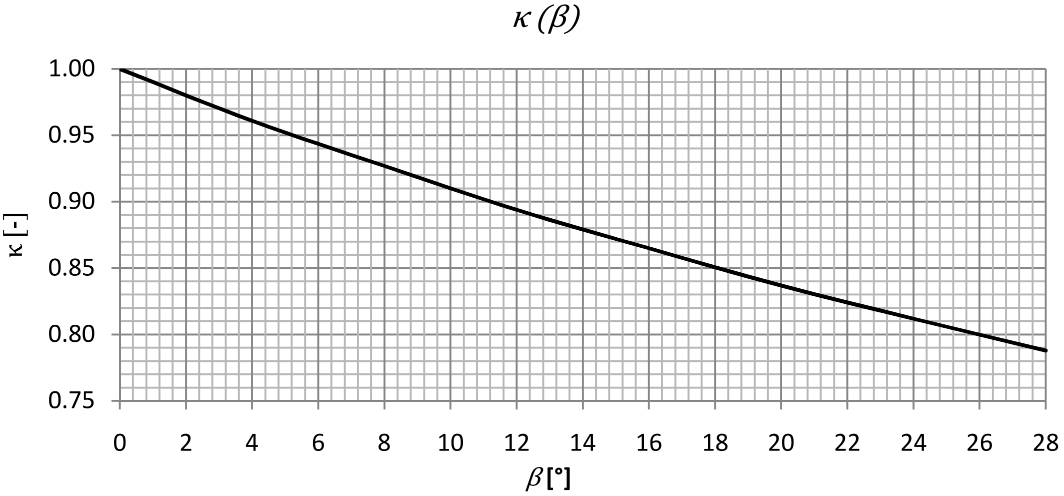


Figure 5.10 Correction factor κ to obtain the orthogonal water depth h at the beginning of the trench rake

Example: Trench weir design

For irrigation, 0.2 m^3/s is needed (Q_{design}). The average river runoff Q_{river} is around 1.0 m^3/s. Your task is to design an appropriate trench weir including a trench rake to obtain the needed discharge. The scouring channel in which the diverting trench is placed has a width $b = 1.0$ m. The rake slope is $10°$. The channel must be kept free of debris with a diameter bigger than 30 mm. The available rake bars are made out of square shaped steel tubes $50/80/5$ mm.

Since you want the design discharge of $Q_{design} = 0.2$ m^3/s, it is reasonable to arrange the intake structures in a way that a certain water level is guaranteed. This can be accomplished by placing the weir body at a stage which holds the desired water level in your forebay.

Assume that your water level in the forebay is kept at a minimum level of 0.25 m by a permanent weir.

The critical height h_{crit} in your scouring channel is:

$$h_{crit} = \frac{2}{3} \cdot h_{forebay} = \frac{2}{3} \cdot 0.25 = 0.16 \text{ [m]}$$

Now start sizing the trench rake by finding the factor c:

$$c = 0.6 \cdot \frac{a}{d} \cdot (cos\beta)^{3/2} = 0.6 \cdot \frac{30}{80} \cdot (\cos(10))^{3/2} = 0.22 \; [-]$$

Next, calculate the orthogonal water depth h at the beginning of the rake using Figure 5.10:

$$h = h_{crit} \cdot \kappa = 0.16 \cdot 0.91 = 0.15 \text{ [m]}$$

Now enter all factors into the formula and see if all the water can be diverted by your trench rake. For the rake length L, choose 1.0 m for the first trial.

$$Q = \frac{2}{3} \cdot c \cdot \mu \cdot b \cdot L \cdot \sqrt{2 \cdot g \cdot h} = \frac{2}{3} \cdot 0.22 \cdot 0.62 \cdot 1 \cdot 1 \cdot \sqrt{2 \cdot 9.81 \cdot 0.15} = 0.156 \left[\frac{\text{m}^3}{\text{s}}\right]$$

The diverting capacity in the first trial was not able to discharge your desired amount of water into the trench, therefore, you must change something in your design. In the second trial, your rake length L should be 1.3 m.

$$Q = \frac{2}{3} \cdot 0.22 \cdot 0.62 \cdot 1 \cdot 1.3 \cdot \sqrt{2 \cdot 9.81 \cdot 0.15} = 0.203 \left[\frac{\text{m}^3}{\text{s}}\right]$$

With a rake length $L = 1.3$ m the design discharge can be diverted out of the river.

5.8 Sizing stilling basin

After having found the length and shape of your weir crest, you may also want to design a stilling basin. This structure is necessary to eliminate of the redundant energy caused by the drop from the back water level (forebay) down to the tailrace water level (after-bay). For the hydraulic assessment of a rectangular- and horizontal stilling basin, use the following approach adapted from *Vischer* and *Huber*:

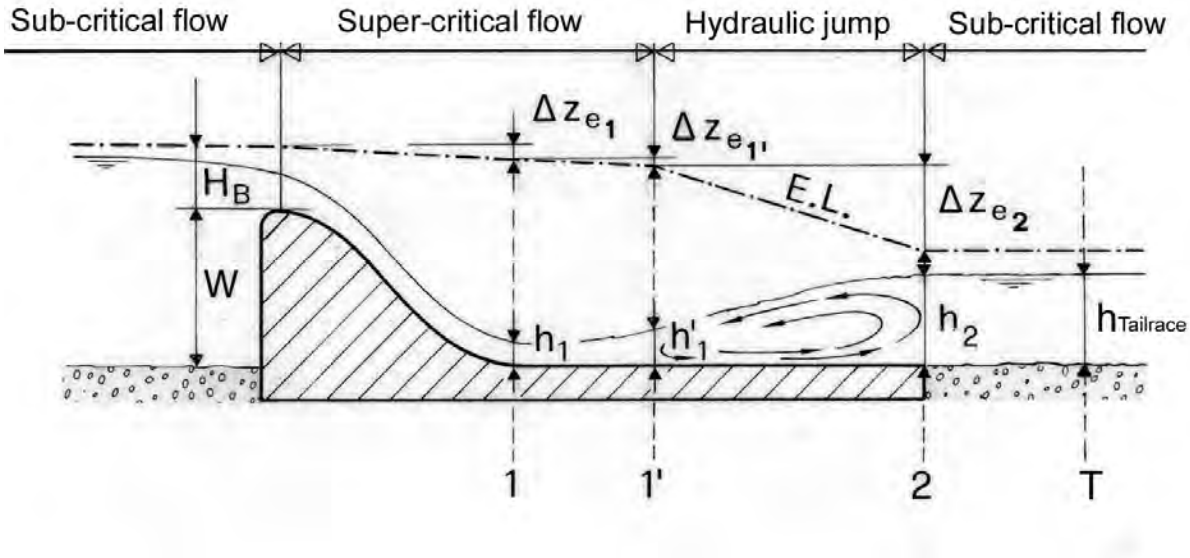


Figure 5.11 Permanent weir with stilling basin. Water level and energy level E.L.

Step 1: Since we know the energy level $EL = H_B + W$ from our previous calculation of the weir crest, by assuming a flow of Q_{flood}, we are able to predict the water level at the base of the weir (section 1-1) by applying the Bernoulli formula:

$$H_B + W = h_1 + \frac{v_1^2}{2 \cdot g} + \Delta z_{e1} \text{ [m]}$$

where Δz_{e1} is the head loss which can be assumed by $0.1 \frac{v_1^2}{2 \cdot g}$. The calculation can be further simplified by replacing v_1 with q/h_1 and rearranging the equation into this format:

$$H_B + W = h_1 + \frac{1.1 \left(\frac{q}{h_1}\right)^2}{2 \cdot g} \text{ [m]}$$

where the specific flow is $q = Q_{flood}/b$ [m^2/s]. The formula cannot be solved in one step, which makes an iterative approach necessary. Hence you must enter values for h_1 between zero and H_B to find the exact value which fulfils the equation. Once you have obtained the water height h_1 in section 1-1, calculate the restoring force S_1 in section 1-1 with the following equation:

$$S_1 = \rho \cdot g \cdot \frac{b \cdot h_1^2}{2} + \rho \cdot Q_{flood} \cdot v_1 \text{ [N]}$$

Step 2: Starting from the water level h_T in the tailrace, you can obtain the water height h_2 of the water level in section 2-2 in the lower part of the basin. In cases of a tailrace shape similar to the basin, use $h_2 = h_T$, otherwise calculate the height analogue to step 1 by using the Bernoulli principle. The opposite restoring force S_2 in section 2-2 can be calculated according to:

$$S_2 = \rho \cdot g \cdot \frac{b \cdot h_2^2}{2} + \rho \cdot Q_{flood} \cdot v_2 \text{ [N]}$$

Step 3: Now match the forces S_1 and S_2 and you may face one of three different cases:

Case A: $S_1 = S_2$. Section 1-1 coincides with section 1'-1' which has the result that the hydraulic jump is directly in front of the weir base. The length of the hydraulic jump l_2 can be estimated by obtaining the Froude number ($Fr_1 = v_1/\sqrt{g \cdot h_1}$) in section 1-1 and by finding the corresponding length from Figure 5.12. The length of your basin is l_2 described in Figure 5.12, which is the preferred case.

Case B: $S_1 > S_2$. The hydraulic jump will appear further downstream at section 1'-1' after reducing the flow velocity to $v_{1'}$ and after raising the water height up to $h_{1'}$.

$$h_{1'} = -\frac{h_2}{2} + \sqrt{\frac{h_2^2}{4} + \frac{2 \cdot Q_{flood}^2}{g \cdot b^2 \cdot h_2}} \ [\mathrm{m}]$$

The length l_1' can be computed by calculating the water level from section 1-1 downwards, (because of supercritical flow conditions) to section 1'-1' by using the Bernoulli principle. The length of the hydraulic jump l_2 can be estimated by obtaining the Froude number in section 1'-1' and by finding the corresponding length in Figure 5.12. The total length of your basin is $l_{1'} + l_2$.

Case C: $S_1 < S_2$. The hydraulic jump backs up to the weir base. The energy dissipation is rather poor and there is no common formula to size the length of the basin. Solve this problem by designing a flat weir slope as in Figure 5.13

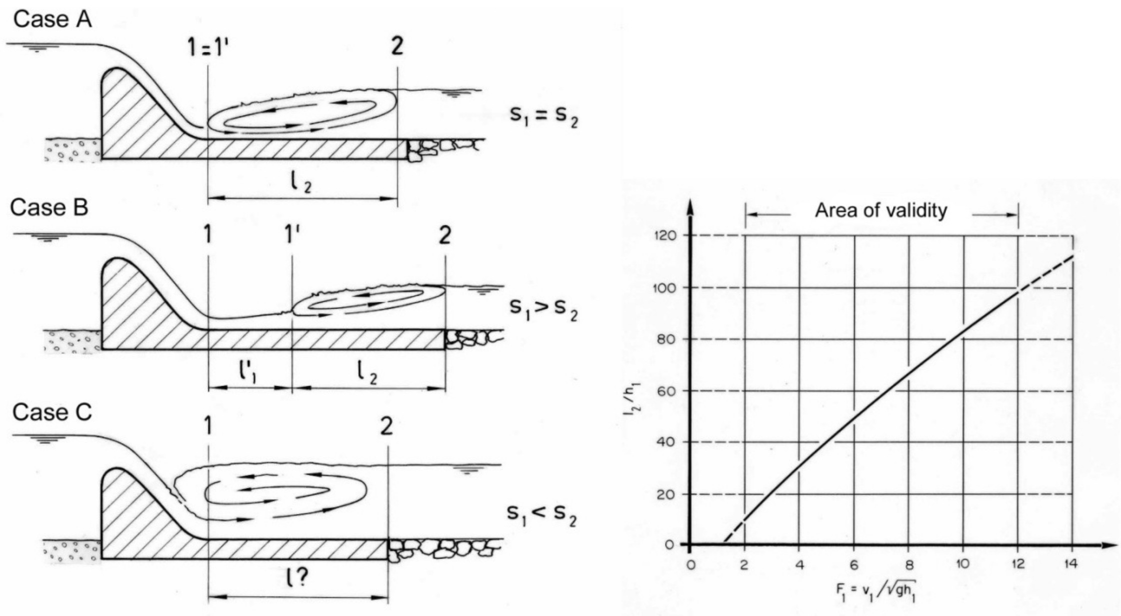


Figure 5.12 Potential appearance of hydraulic jump

If the Froude number is between 4 and 12, the basin length l_2 can be assumed by the following rule of thumb:

$$l_2 = 6 \cdot h_2 \ [\mathrm{m}]$$

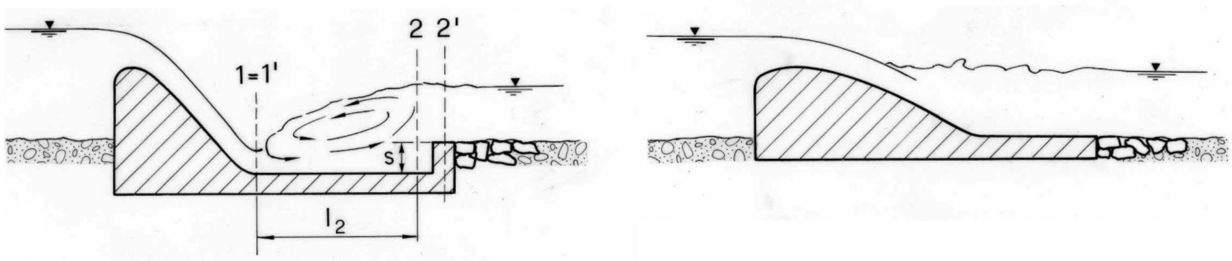


Figure 5.13 Alternative designs for case B (left) and case C (right)

It is obvious that whereas A is a rare case, B is the most common situation. To prevent a long basin construction, you are advised to design an end baffle with the height s as in Figure 5.13.

Sizing the stilling basin with an end baffle must be done in iterative steps, which means that you have to narrow down the value for s to fulfil the equilibrium of forces S_1=S_2 within the basin area. The approach is analogous to step 2, where you have to find the water height $h_{2'}$ at the end of your basin. Further, you start by guessing a value for the end baffle height s by setting $S_2 = S_{2'}$ and rearranging the equation to get h_2. Start with the calculation of $S_{2'}$:

$$S_{2'} = \rho \cdot g \cdot \frac{b \cdot (h_{2'} + s)^2}{2} + \rho \cdot Q_{flood} \cdot v_{2'} \text{ [N]}$$

Now repeat the calculation from Step 1 and obtain S_1 and match it with S_2. Repeat this procedure until you have found the appropriate value s to fulfil $S_1 = S_2$.

Example: Sizing intake with a standard weir and stilling basin

Design an intake structure to divert Q_{design} = 0.2 m^3/s from a river carrying water during a flood which statistically appears once in 50 years $Q_{flood} = 3.0$ m^3/s.

Boundary conditions:

- Required freeboard in forebay $h_{freeboard} = 0.7$ m
- Flow condition in forebay is sub-critical $\left(Fr = v/\sqrt{g \cdot h} < 1\right)$
- Water height in the after-bay can be assumed as $h_2 = 0.5$ m
- The gap between the two banks is 5.5 m (cross section in Figure 5.14.)

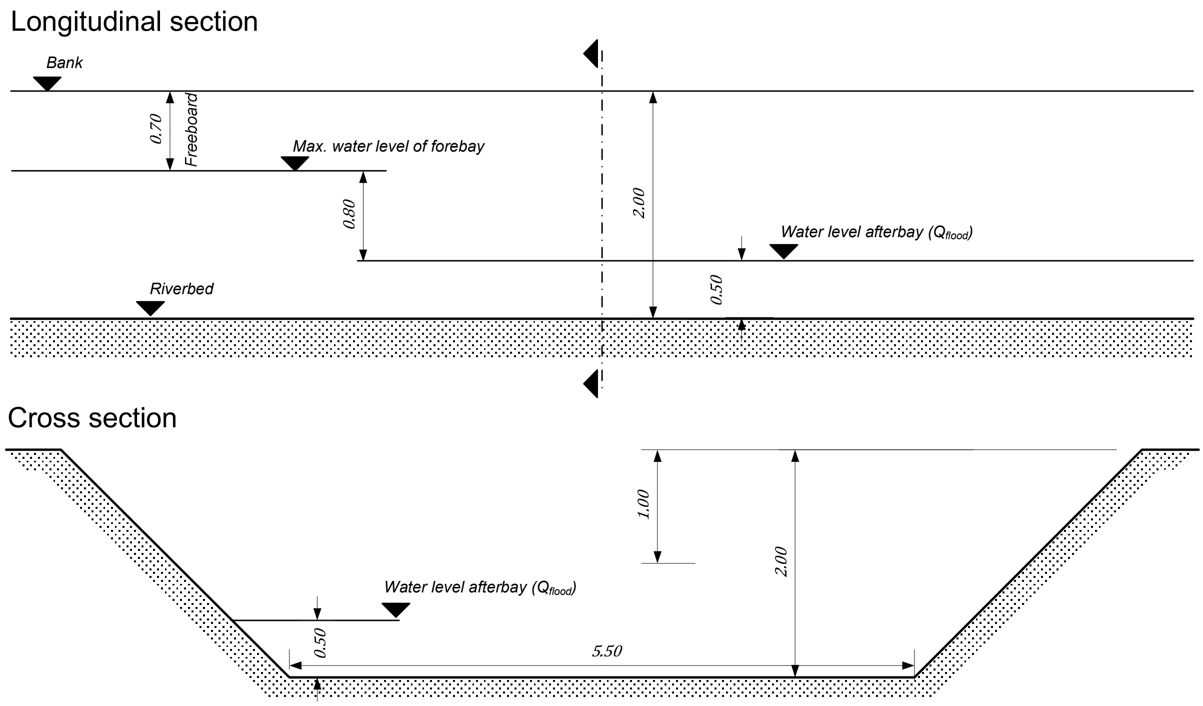


Figure 5.14 Longitudinal section and cross-section of proposed intake area

a.) Divide space into a diversion section and weir/stilling basin section:

Place wing walls to both sides with a width of 0.25 m, the scouring channel should be 1.0 m. The divider between the scouring channel and the stilling basin is exposed to more forces than the wing walls and shall therefore be twice as wide as the wing walls, thus 0.5 m. Subtracting all structure dimensions from the total river width w_{tot}, we obtain the weir length b:

$$b = w_{tot} - w_{wing} - w_{scouring} - w_{divider} = 5.5 - 2 \cdot 0.25 - 1.0 - 0.5 = 3.5m$$

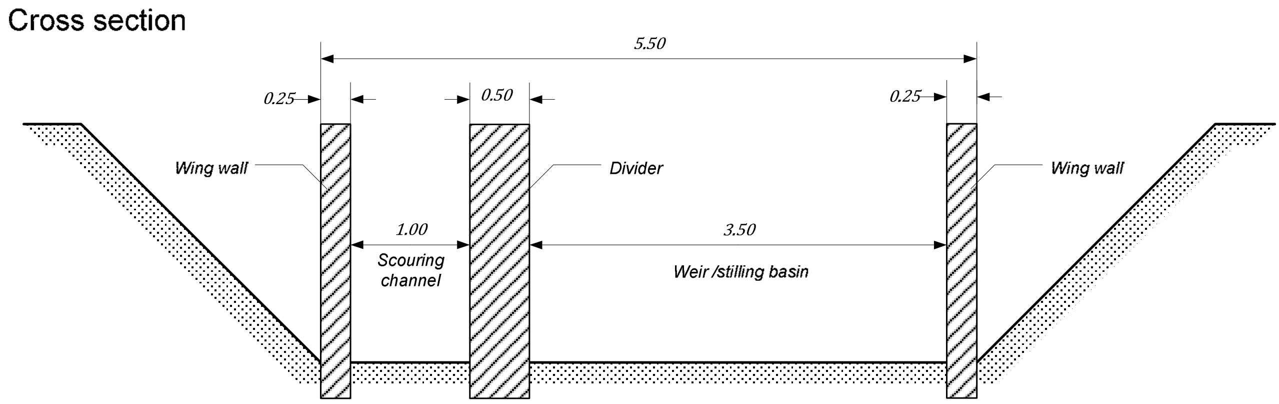


Figure 5.15 Partitioning of a river section

b.) Sizing the weir:

By assuming that the freeboard must be 0.7 m above the forebay water level, we first calculate the weir with an estimation of height of $w = 0.8$ m. Now calculate all the water passing over the weir without piling up the backwater more than 0.5 m. The weir shape coefficient shall be $\mu = 0.75$ (a flood scenario).

$$H_B = \frac{Q_{flood}^{2/3}}{(2.95 \cdot \mu \cdot b)^{2/3}} = \frac{3.0^{2/3}}{(2.95 \cdot 0.75 \cdot 3.5)^{2/3}} = 0.53 \text{ [m]}$$

The water level in the forebay is a little higher than it is supposed to be, but if we release a certain amount through the scouring channel, the water level will be below the freeboard level.

$$H_B = \frac{\left(Q_{flood} - Q_{design}\right)^{2/3}}{(2.95 \cdot \mu \cdot b)^{2/3}} = \frac{(3.0 - 0.2)^{2/3}}{(2.95 \cdot 0.75 \cdot 3.5)^{2/3}} = 0.50 \text{ [m]}$$

The weir height now allows a minimum water level of 0.8 m and a flood water level of 1.2 m without overshooting the freeboard.

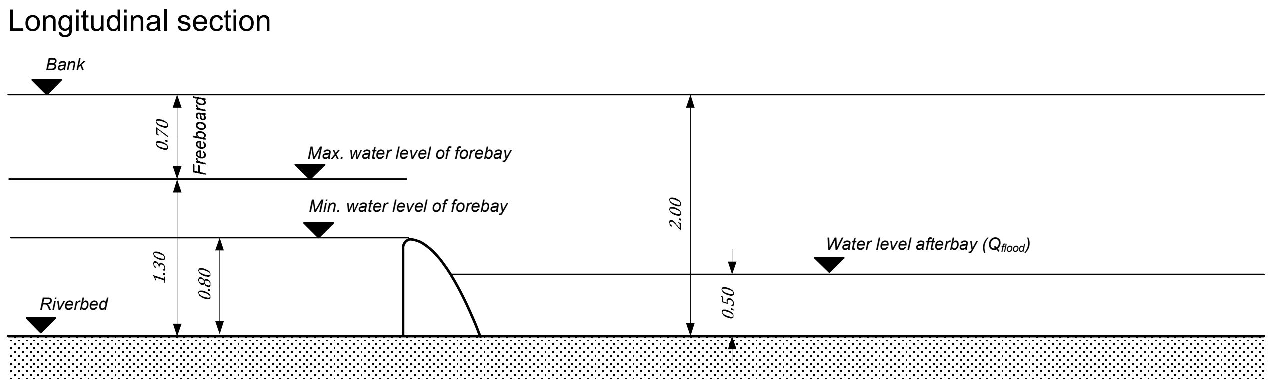


Figure 5.16 Longitudinal section with a maximum and minimum water level

c.) Sizing the stilling basin:

The stilling basin assessment is taken care of by the following three steps which are described in chapter 5.8:

Step 1: Find the restoring force S_1. where $w = 0.8$ m, $H_B = 0.5$ m, and $q = Q_{flood}/b = 0.8$ m^2/s. Remember that the water height h_1 at the base of the weir must be calculated by iteration.

$$H_B + W = 0.5 + 0.8 = 1.3\, m = h_1 + \frac{1.1\left(\frac{q}{h_1}\right)^2}{2 \cdot g} \text{ [m]}$$

Prepare a table with two columns, and solve for each value h_1 using the above-mentioned equation. h_1 will be somewhere between 0 and H_B. The number of values will form a curve like the one in Figure 5.17.

h_1	$H(h_1)$
0.10	3.36
0.15	1.60
0.20	1.02
0.25	0.77
0.30	0.66
0.35	0.62
0.40	0.60
0.45	0.61
0.50	0.63
0.55	0.66
0.60	0.69
0.65	0.73
0.70	0.77
0.75	0.81

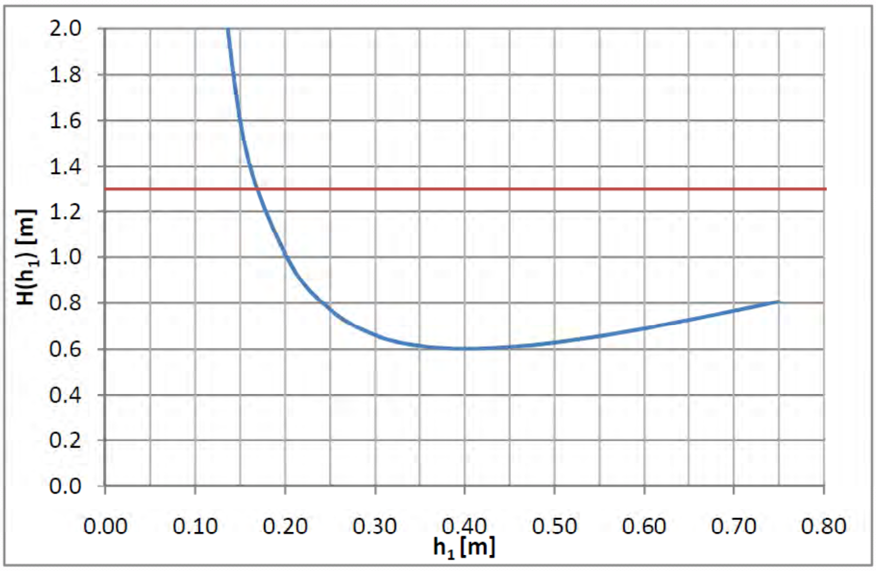


Figure 5.17 Blue line: energy head in relation to h_1; Red line: energy head $w + H_B$

The exact value for h_1, which fulfils the equation is where the two lines in your generated diagram cross each other (Figure 5.17). Hence, h_1 must be near 0.17 m. Now calculate S_1:

$$S_1 = \rho \cdot g \cdot \frac{b \cdot h_1^2}{2} + \rho \cdot Q_{flood} \cdot v_1 = 1000 \cdot 9.81 \cdot \frac{3.5 \cdot 0.17^2}{2} + 1000 \cdot 2.8 \cdot \frac{2.8}{(0.17 \cdot 3.5)} = 13673 \text{ [N]}$$

The restoring force leading downstream is approximately 13.7 kN.

Step 2: Finding the restoring force S_2. The water level in the afterbay can be estimated by $h_2 = 0.5$ [m].

$$S_2 = \rho \cdot g \cdot \frac{b \cdot h_2^2}{2} + \rho \cdot Q_{flood} \cdot v_2 = 1000 \cdot 9.81 \cdot \frac{3.5 \cdot 0.5^2}{2} + 1000 \cdot 2.8 \cdot \frac{2.8}{(0.5 \cdot 3.5)} = 8772 \text{ [N]}$$

The restoring force leading upstream is approximately 8.8 kN.

Step 3: Now match the forces S_1 and S_2

$$S_1 = 13.7 > 8.8 = S_2$$

According to Figure 5.12, the hydraulic jump is likely to move out of the stilling basin, excavating the riverbed. Therefore, you must equip your stilling basin with an end baffle. The top of the end baffle should be even with the riverbed. As a consequence, the stilling basin must be placed below the riverbed. However, lowering the stilling basin increases the height w of the weir, therefore you must recalculate. Estimate the end baffle height with $s = 0.35$ m.

$$S_{2'} = 1000 \cdot 9.81 \cdot \frac{3.5 \cdot (0.5 + 0.35)^2}{2} + 1000 \cdot 2.8 \cdot \frac{2.8}{((0.5 + 0.35) \cdot 3.5)} = 15040 \text{ [N]}$$

The corresponding force in section 1-1 is, according to the new energy head, $w+HB = 1.65$ m (obtained from Figure 5.17) and with $h_1 = 0.155$ m:

$$S_1 = 1000 \cdot 9.81 \cdot \frac{3.5 \cdot 0.155^2}{2} + 1000 \cdot 2.8 \cdot \frac{2.8}{(0.155 \cdot 3.5)} = 14864 \text{ [N]}$$

Now match the forces again:

$$S_1 = 14.9 < 15 = S_2$$

$$S_1 \approx S_2$$

As a result, with an end baffle of *0.35* m, the hydraulic jump stays within the stilling basin. Since the Froude number in section 1-1 is above *4*, the length l_2 between weir base and end baffle is obtained by the rule of thumb $l_2 = 6 \cdot h_2$.

$$Fr_1 = \frac{v_1}{\sqrt{g \cdot h_1}} = \frac{5.16}{\sqrt{9.81 \cdot 0.155}} = 4.18$$

$$l_2 = 6 \cdot h_2 = 6 \cdot 0.5 = 3.0\ [m]$$

Now design a standard weir crest according to chapter 5.6. It is recommended to prepare a table in which the coordinates for the weir slope are displayed. The y-values are calculated with the following formula:

$$y(x) = 0.5 \cdot \frac{x^{1.85}}{H_B^{0.85}}$$

Weir parameter [m]

Flooding head H_B:	0.50
Weir height w:	1.15
Fillet radius l_0:	0.14
Base length l_1:	1.14
Total base width l_{tot}:	1.28

Coordinates [m]

x -Value	Y (x)
0.00	0.00
0.12	0.02
0.23	0.06
0.35	0.13
0.46	0.21
0.58	0.32
0.69	0.45
0.81	0.60
0.92	0.77
1.04	0.96
1.15	1.17
1.14	1.15

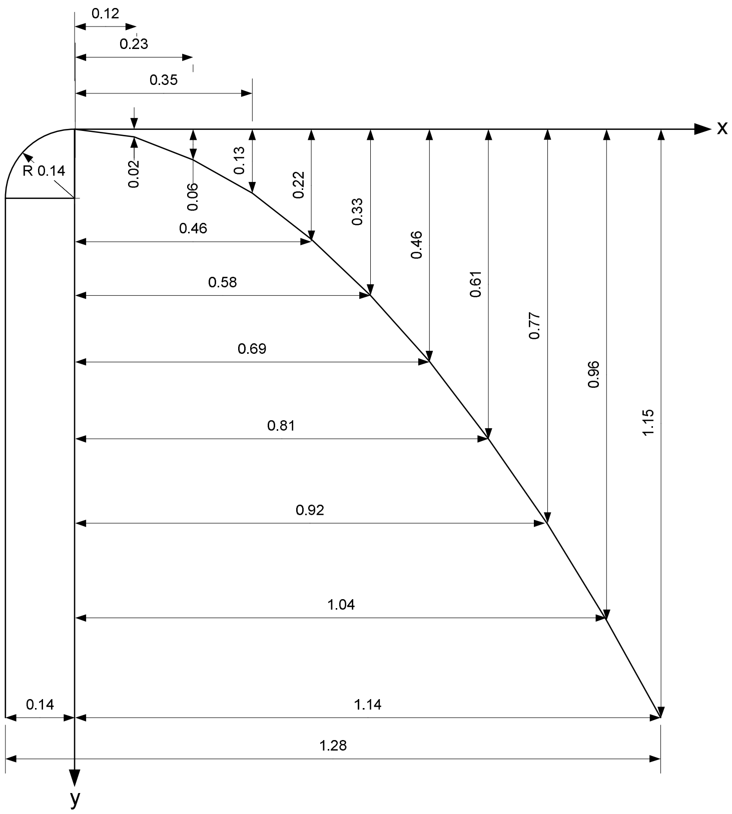


Figure 5.18 Weir crest design according to USCE

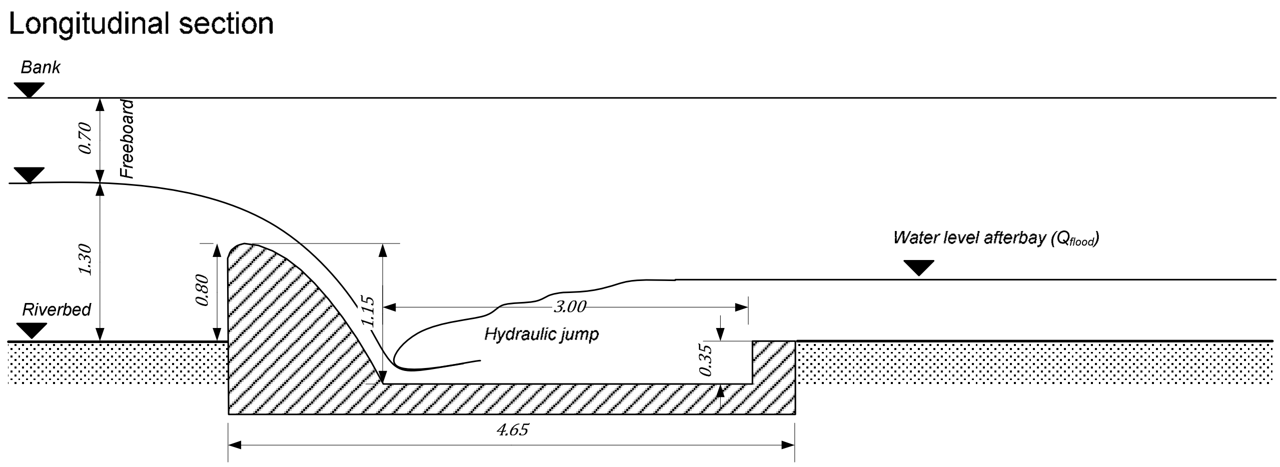


Figure 5.19 Weir layout with a stilling basin and end baffle

6 Settling basin and forebay tank

All the water which arrives from the river and passes into the turbine carries tiny particles of solid matter. These hard abrasive materials cause severe damage and can leads to the turbine runners are wearing after a short time. To eliminate this silt load, the water flow must be slowed in the structure in order for the particles to settle and collect on the basin floor. These deposits needs to be flushed periodically en sure that there is enough space for further deposits. It is essential to remove these sediments from the water. The building which houses the settling area should be placed at the channel entry or at the penstock entry. The structure at the penstock entry is called a forebay tank. If the river carries a high number of silt particles, the basin should be installed directly after the intake, and in this case, it is recommended that the channel be paved. Alternatively, it may be sufficient to combine the settling basin and forebay tank, which will also save building materials.

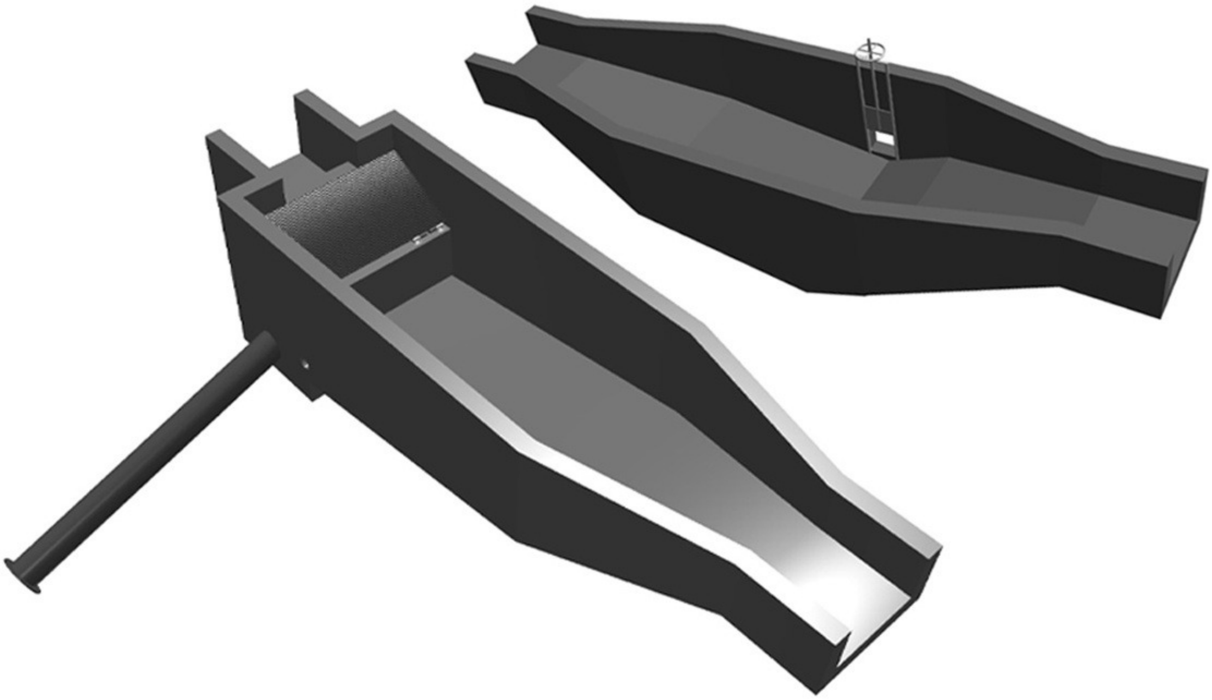


Figure 6.1 Left: Forebay tank with settling area; Right: Settling basin

Figure 6.1 shows a forebay tank with settling area and a settling basin which is placed at the channel entry. Different designs are possible. But all structures which contain a settling area must fulfill the following five important principles:

1. The dimensions of the basin (length and width) must be large enough ensure that the sediments can settle but not be so large that the structure is too massive and therefore too expensive.
2. A proper settling area must avoid flow turbulence and flow separation caused by sharp bends and area changes.
3. Enough space must be available to collect sediment
4. It must be possible to easily flush the deposits at sufficiently intervals.
5. Water which exits the flush gate must be led away from the basin and penstock foundations. Otherwise the soil supporting the installations will be flushed away. A paved spillway drain with walls should be built.

The designs which are shown in this chapter may not suit for your project exactly. Many different variations are possible, but all must achieve the above-mentioned five design principles.

6.1 Influence of silt load on mechanical devices

Picture 6.1 Damaged Pelton buckets caused by silt and sand

The source of the most serious damage is abrasion caused by silt and tiny sand particles which enter the turbine. Abrasion occurs when these particles enter the turbine at high speeds and act like a sandpaper on the runner blades of the turbine and/or the Pelton buckets. This is why it is essential that the silt and sand be filtered out of the water prior to entering the turbine. This can only be achieved with a settling basin of an appropriate size. Otherwise the turbine output will be reduced and, in the worst case, the turbine might be destroyed.

6.2 Design of a settling basin

The proper dimension of a settling basin significantly extends the lifespan of the building materials of a hydropower unit. In this particular case, the settling basins (or sand traps) are designed to allow sediment particles up to the size of 0.2 mm in diameter to settle. Specifically, all grains larger than 0.2 mm must be removed before the water enters the turbine. The maker of the turbine offers detailed specifications as to the maximum diameter of particles which may pass through the turbine safely without damaging the turbine blades. In some instances, larger particles may be allowed to pass through the turbine, which reduces the size of the settling basin. To reach the appropriate settling result, the flow velocity must to be reduced in order to minimize turbulence. Therefore, the cross-section of the basin should widen gently until the flow is slow enough to let the particles sink. The flowing water is quite sensitive to the recesses and edges of the structure. Consequently, the entrance walls of the basin can be widened to an angle of no more than 15°. In our case, the entrance will be too big. A rule of thumb states that this part should be as long as the width of the basin.

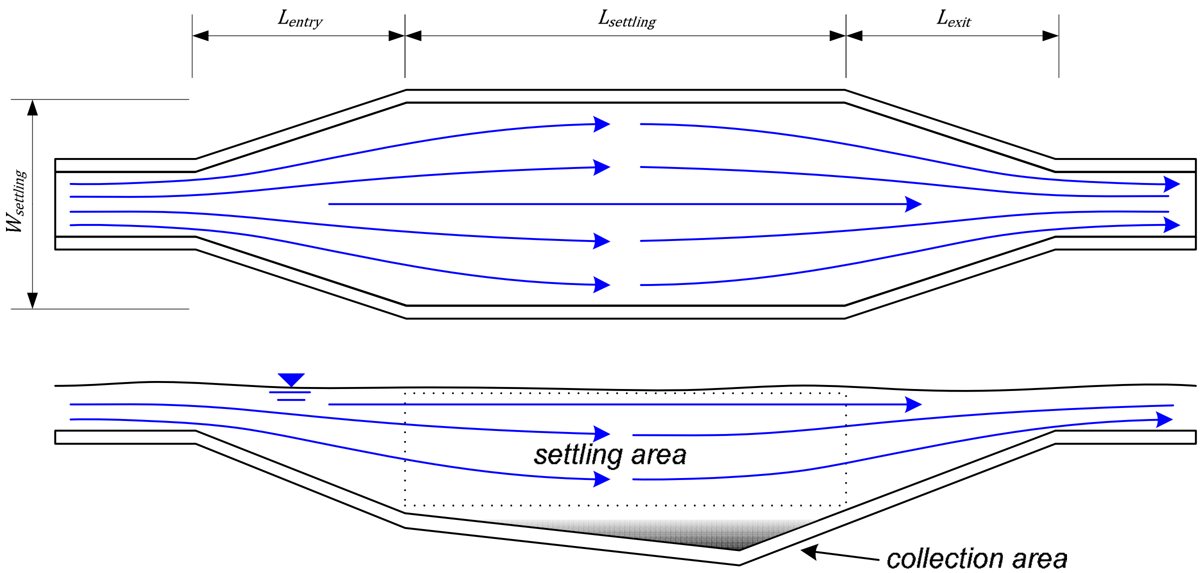


Figure 6.2 Low velocity throughout width, no turbulence in settling basin

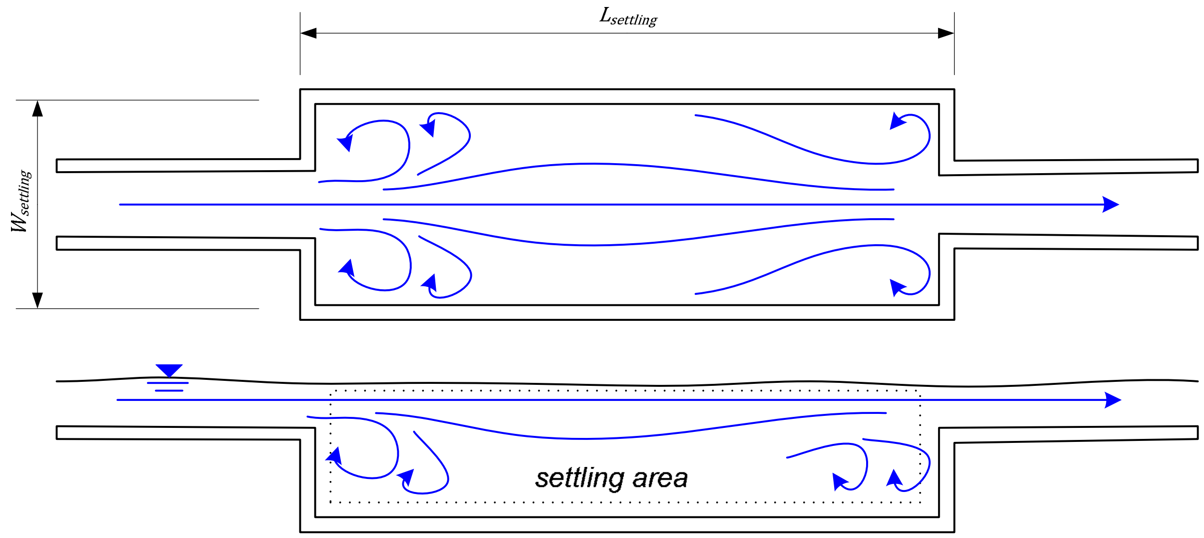


Figure 6.3 Incorrect settling basin design: high surface velocity and turbulence in corners

Dimension of a settling basin

Settling width: In order to reduce the flow velocity of the water in the settling area, the width ($W_{settling}$) of the basin must first be chosen. The width can be chosen based on the available space, but the width is usually between 2 to 15 times the width of the channel. In case of a trapezoid channel, the average width is used. If the channel runs through soil, it is recommended to pave the last five meters of the channel before the basin with concrete, which will improve the flow into the settling basin.

Settling height: The height of the settling area, too, must be customized to the location of the building. The flow of the water and the width of the structure should be known at this point. The depth of the basin can be determined with that information and the flow velocity. The flow velocity of the basin must be reduced until the particles that sink to the floor can no longer whirl up. Otherwise the sand catcher will not function properly. The maximum critical velocity can be estimated with the simplified Vischer & Huber (1982) formula; d is the diameter of the particles which should settle:

$$v_{crit} = 0.44 \cdot \sqrt{d} \; < 0.6 \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

$$d \; in \; [\mathrm{mm}] \; and \; v_{crit} \; in \; \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

With the help of the formula for volume flow rate we can now calculate the minimum basin depth:

$$Q = A \cdot v = W_{settling} \cdot H_{settling} \cdot v \; \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

H transformed ($H_{settling} = H_{min}$), and the flow velocity replaced by the critical velocity, we arrive at the minimum permitted water depth in the settling basin:

$$H_{min} = \frac{Q}{W_{settling} \cdot v_{crit}} \; [\mathrm{m}]$$

It is preferable to build a deeper basin if possible, because the length of the effective area of the settling area is then shortened. The maximum depth should not exceed the width of the basin by 1.25.

Settling length: The next step, calculating the length of the settling area, reveals the actual size of the structure.

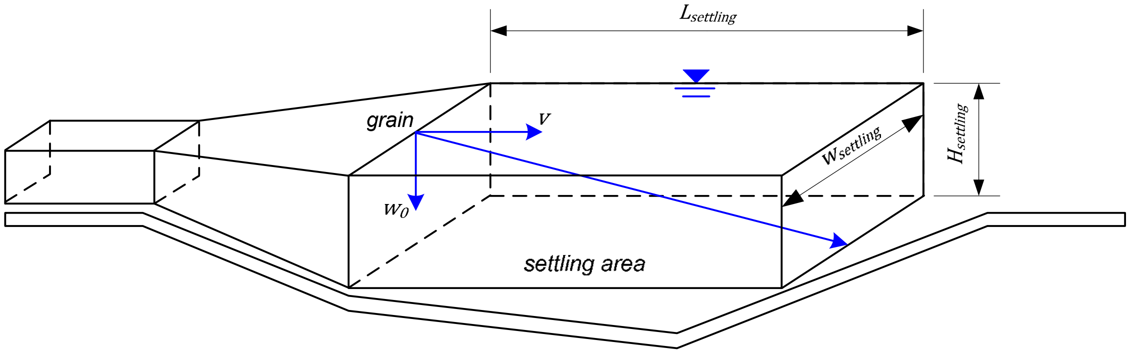


Figure 6.4 Settling behaviour of a grain in settling basin

The particle initially floats on the water and therefore, we now must calculate how long it will take for it to settle to the ground. For this, we need the flow velocity v and the vertical settling velocity w of the particle. The vertically acting speed can be calculated with the formula according to Zanke.

$$w_0 = \frac{100}{9 \cdot d} \cdot \left(\sqrt{1 + 1.57 \cdot 10^2 \cdot d^3} - 1\right) \quad \left[\frac{\mathrm{mm}}{\mathrm{s}}\right]$$

Where d = grain diameter [mm].

The formula is valid for particles in water at a temperature of 20° Celsius. If the water is colder, the particles sink at a slower speed. However, at the moment we will ignore this fact. In the following diagram, we can find the speed of the sinking particle according to its diameter. Be careful the diagram is charted logarithmically.

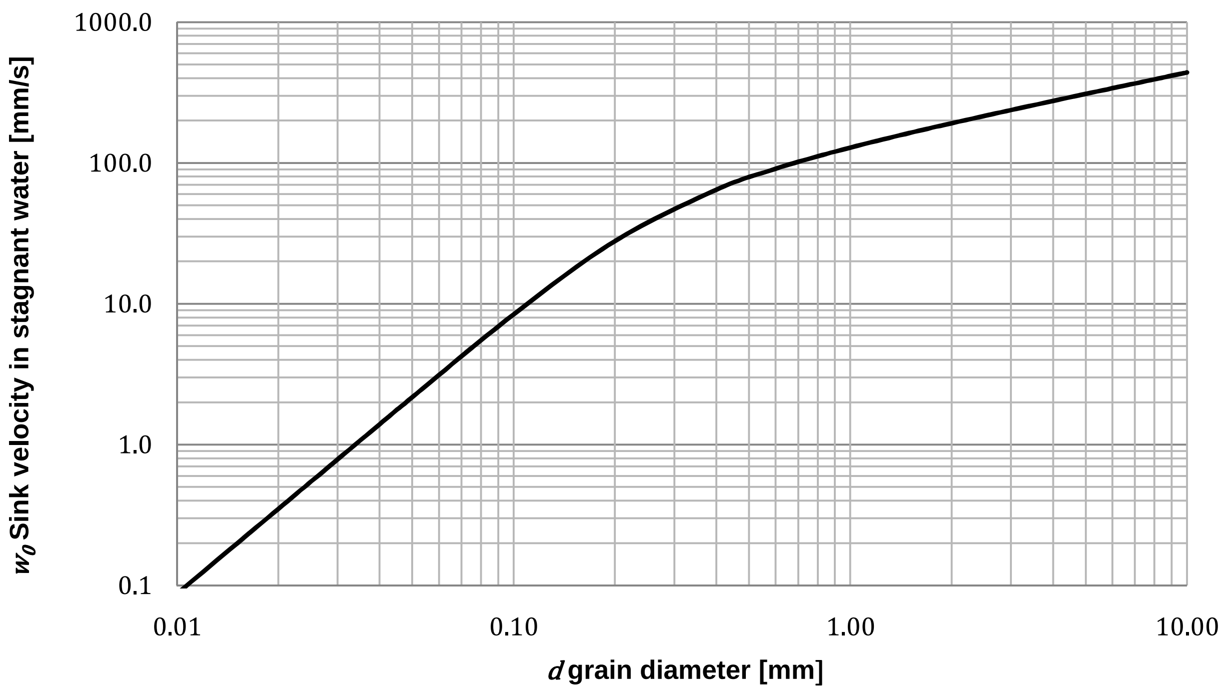


Figure 6.5 Sink velocity according to the grain diameter

Because the values displayed in the diagram are only valid for standing water, we must reduce them to continue the calculation. Flowing water has turbulences and therefore slows the settling velocity, which results in an increase of the settling distance.

$$w = \frac{w_0}{1000} - \left(v \cdot \frac{0.132}{\sqrt{H_{settling}}}\right) \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

$$w = must\ be\ larger\ than\ 0$$

The horizontal flow velocity of the water is known:

$$v = \frac{Q}{W_{settling} \cdot H_{settling}} \quad \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

With this data the settling length can be calculated:

$$L_{settling} = H_{settling} \cdot \frac{v}{w} \quad [\mathrm{m}]$$

A large basin width causes an unequal current in the settling tank which can prevent the settling of some particles. To solve this problem, several settling basins must be arranged side by side. To generate laminar flow, the width of the settling area should be no larger than 1/8$^{\text{th}}$ of its length $L_{settling}$. Otherwise, flow distribution walls which help avoid slipstreams must be installed and the silt cannot be transported into the channel or penstock. The outlet of a settling basin must be designed similarly to the shape of the entry.

Collection area and flushing of silt accumulation

The collection area is an area inside the settling basin exclusively utilized to collect the settled particles. The volume of the collection area cannot be added to the volume of the settling basin. Adequate space must be allocated in the settling basin for the collection area. As a rule of thumb, the following can be assumed:

$$V_{collection\ capacity} = \frac{V_{settling}}{4} = \frac{W_{settling} \cdot H_{settling} \cdot L_{settling}}{4} \quad [\mathrm{m}^3]$$

This formula gives us a benchmark and should be regarded as the minimum volume. The size can be enlarged if large amounts of sand and silt are expected. This may be necessary if the river has a steep decline or if the ground in the catchment area is rich in clay. Exact data about the flushing interval can be evaluated after a few months of service. The collecting area should never be overfilled. Otherwise the basin cannot be completely emptied and the valve of the flush gate cannot be opened.

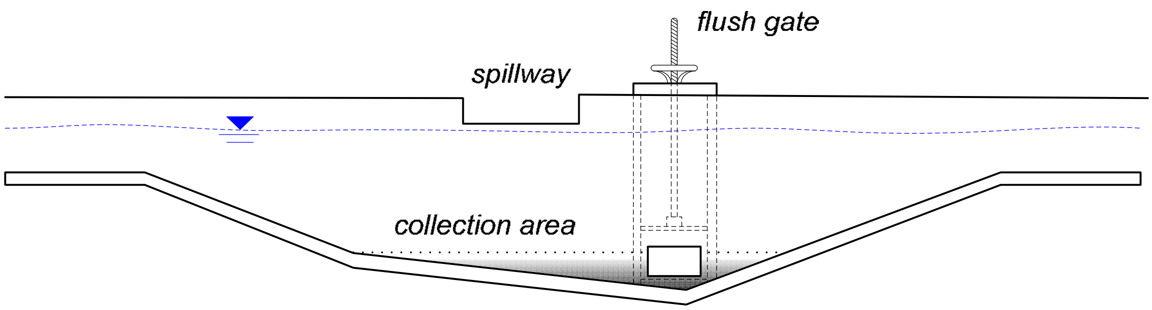
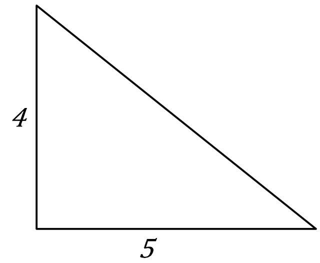


Figure 6.6 Settling basin with flush gate and spillway

An incline of 4:5 of the basin floor is most appropriate in order to guarantee the proper movement of the particles. This is suggested, but not always possible and compromises may be necessary. Where this is not possible, frequent manual maintenance with a broom may be necessary.



6.3 Design of a forebay tank

The main advantage of a forebay tank is to store sufficient water to prevent large fluctuations of the water level during turbine operation. In case the water level drops, balancing of the water volume prevents air from entering the penstock pipe, which would cause damage. Also, the forebay tank leads the water into the penstock pipe. It is prudent to combine the forebay tank with a settling basin. The water capacity from both structures can be combined, even though it is normally contained in the forebay tank. As a rule of thumb, the volume should be around 60 -100 times that of the designed turbine flow. It depends on the length of the penstock pipe the longer the pipe, the bigger the basin. Another positive outcome is the economic design of the combined structure which saves resources.

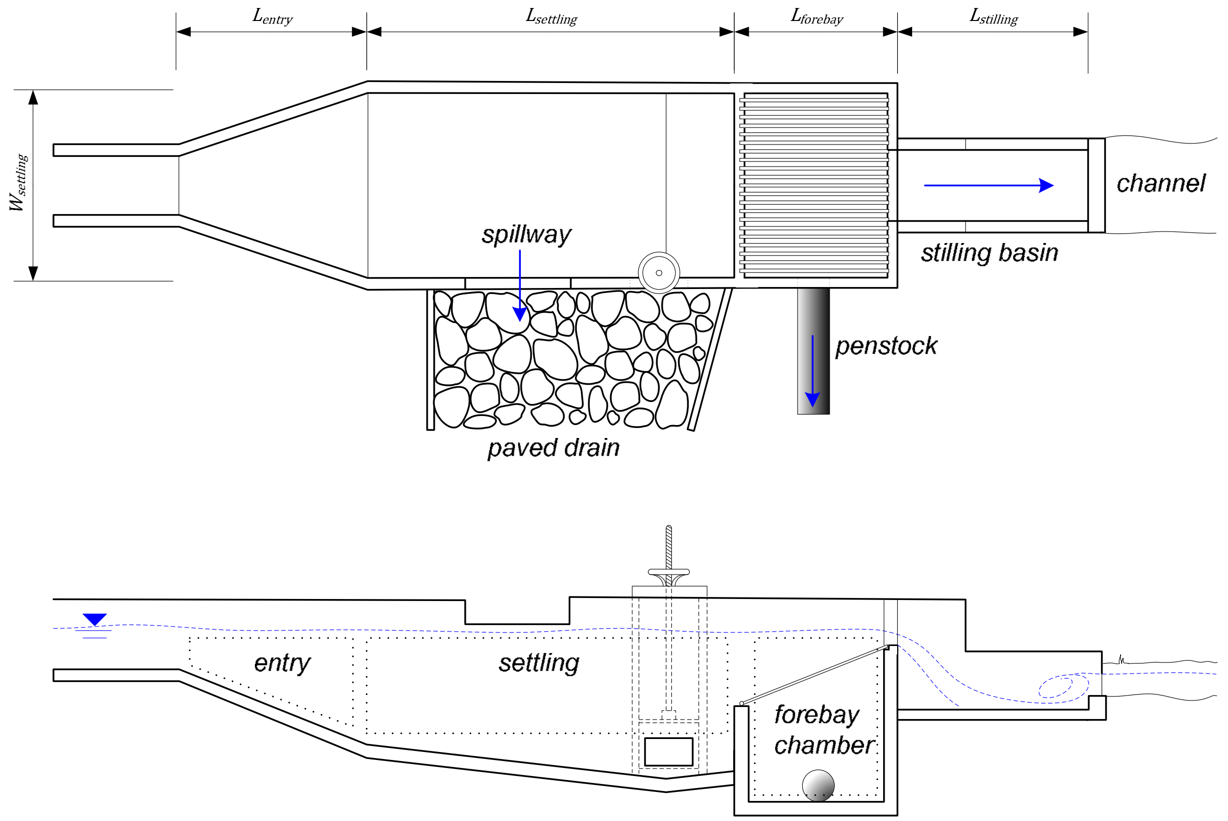


Figure 6.7 Possible design of a forebay tank including settling area

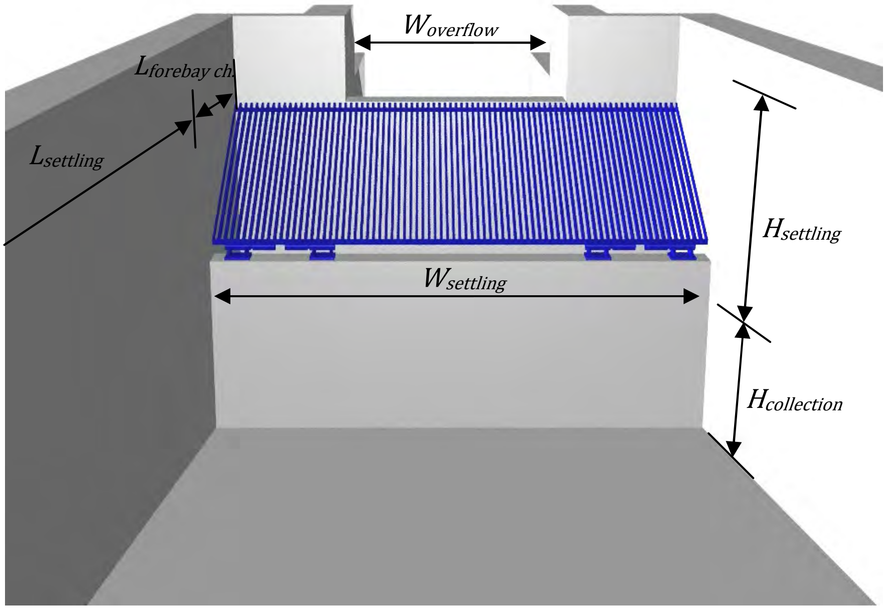
This structure is embedded into the channel so that the water first passes through the settling area and then into the forebay chamber. The dividing wall between these two sections must be about 20 cm higher than the collection depth of the settling basin. In the forebay section of the structure, the water is divided in the penstock, and the excess water is guided across an overflow back into the continuing channel. Channel, settling basin and forebay tank are best aligned axially to keep turbulences low.

The penstock is mounted to the concrete body of the forebay tank four times its diameter below the water level. This measure is necessary to prevent air from being sucked into the penstock or to create a tornado vortex.

The channel level after the forebay tank is lowered for structural reasons. Therefore, the overflow has a level difference. Due to the kinetic energy of the water, the ground below the overflow must be protected from erosion. With an adequate stilling basin (see intake chapter), the scour formation can be prevented.

6.3.1 Trash rack

A trash rack has to be mounted above the forebay tank to prevent floating debris from getting into the penstock. There are different types of trash racks, either thinly spaced vertical steel bars or rods or a plate with holes or slots. Only if there is a coarse trash rack further upstream should a strainer be used.



In this design, the top of the rake is located below the water surface to keep it clean and to guide debris over the overflow.

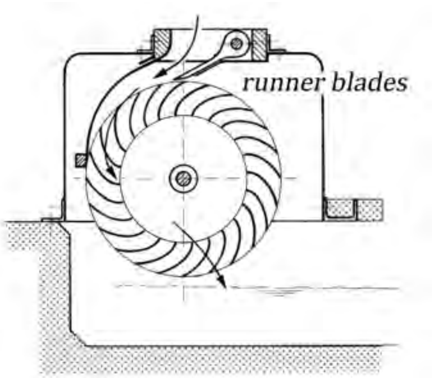
ank

The trash rack should be built strong enough to withstand the water pressure if the whole rack is blocked with floating debris. That means the water pressure above the rack, when there is a maximum level upstream and an insufficient level downstream, couldn't destabilize the rack. To allow simple maintenance during operation, the trash rack should be divided into sections. In this way, manual transportation is possible.

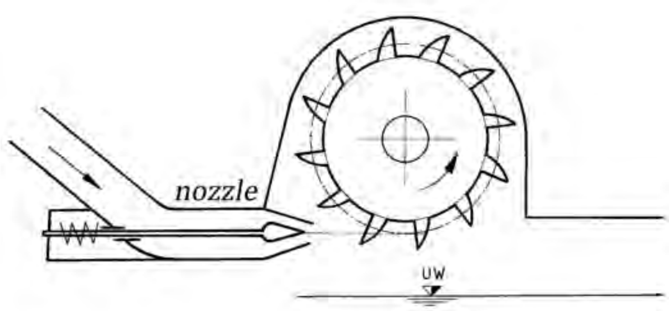
The clearance between bars, rods or the width of holes or slots should not be larger than:

- 0.5 times the nozzle diameter in case of Pelton turbine with fixed nozzle,
- 0.25 times the maximum clearance in a Pelton nozzle with needle valve or
- 0.5 times the distance between runner blades for other turbine types.

Otherwise, floatables could wash into the penstock and obstruct the turbine valve. If the space between the individual bars is too small, the gross head will drop down and the energy output will decrease. In flowing water, a trash rack is an obstacle.



Picture 6.2 Crossflow turbine



Picture 6.3 Pelton turbine and nozzle with needle valve

Head losses due to the bar space of a trash rack. The shapes and distances between the bars may vary. The design of the trash rack must be tailored to the available steel bars. Once a bar is chosen, the total number of bars can be determined based on the above-mentioned maximum distances.

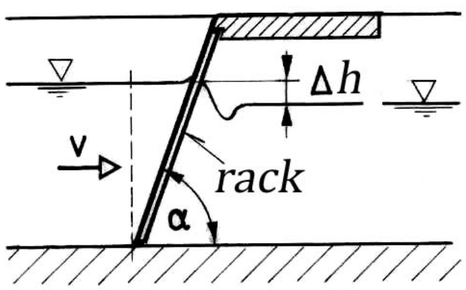
$$n = \frac{b}{(a+s)} - 1 \ [-]$$

When the trash rack is clean, head loss can be calculated according to Kirschmer and Mosonyi.

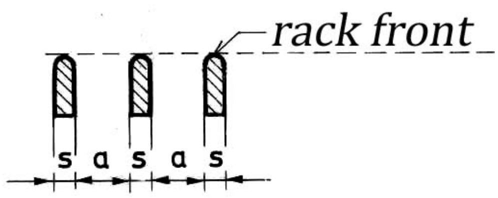
$$\Delta h = \xi \cdot \frac{v^2}{2 \cdot g} \ [-]$$

Loss coefficient with form coefficient if right angled approach flow.

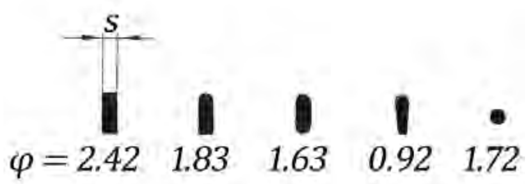
$$\xi = \varphi \cdot \left(\frac{s}{a}\right)^{4/3} \cdot \sin \alpha$$



Δh	Head loss [m]
n	Number of required steel bars
ξ	Loss coefficient
v	Average flow in forebay tank (max 0.5 to 1.0 m/s)
b	Width of rack [m]
a	Bar space [m]
s	Bar dimension [m]
α	Inclination of the trash rack [°]
φ	Form coefficient
g	Acceleration due to gravity [m/s^2]



Picture 6.4 longitudinal section of a rack



Picture 6.5 Form coefficient according to Kirschmer

Spillway

Every structure requires a well-designed emergency spillway should the regular drain be blocked by debris or floatables. The overflow must be constructed so that the exiting water does not erode the ground around other structures. That would prevent the structure from working properly. The spillways capacity is calculated equally to a weir (see chapter 5.5).

Picture 6.6 Flooded forebay tank

This forebay tank does not have a spillway. All surplus water simply flows over the structure unguided. Most of the water exits on the side (circle) and erodes the foundation of the forebay tank. Over time, the soil is removed and the foundation has no support, causing it to sag and break.

Picture 6.7 Temporary fixed forebay foundation

Portions of this foundation of this forebay tank have already been destroyed due to erosion. To prevent the tank from tipping, the villagers have arranged stones to support it. However, this is only a temporary solution since the foundation will eventually tip. The height of the wall was increased to prevent further overflow.

6.4 Case study: forebay tank with settling area

Boundary condition

In our case we have to design a forebay tank combined with a settling basin.

This will be a structure for a 25 kW micro hydro power plant. The usable head at this location is 30 meters. From the manufacturer of the turbine we know that the crossflow turbine permits particles with maximum diameter of 0.25 mm. Larger particles may not enter the penstock and the turbine.

The overall efficiency is 0.6.

Twenty litres per second must be guaranteed for use for irrigation and to pass through the forebay tank.

The channel has an average width of 1 m and currently transports 120 l/s. It can be extended since there is ample water available.

Picture 6.8 Proposed forebay tank location

Note regarding the calculation

After the expansion of the irrigation channel, the water in the channel will be 0.25 m high. To calculate the trash rake, we know that the distance between the runner blades in the crossflow turbine is 0.05 m, according to the turbine manufacturer. The dimensions of the available steel bars for the rake are 0.005 m x 0.04 m. The rake is more stable if the bars are mounted upright.

Introduction of procedure

Step I: Calculate the required flow in the forebay tank.

Step II: Define the width and the depth of the settling basin (use factor 2 in our example). Then, choose the proper sink velocity of the silt and sand particles and subsequently calculate the length of the settling basin. Additionally, the shape of the collecting area has to be designed.

Step III: Calculate the necessary balancing water volume in the combined building (forebay tank and settling basin). With this data, we can define the dimension of the forebay chamber.

Step IV: Now, the height of the overflow must be defined. Two situations must be considered: during the turbine operation, and during water flow over the overflow structure.

Step V: The head loss of the trash rack must be determined.

Solution of the forebay tank example

Step I: First, we must determine how much water must be guided through the structure to ensure that a sufficient amount is available for the operation of the turbine.

The power equation must be transformed:

$$P = \eta \cdot Q \cdot 10 \cdot h_{gross} \quad [\mathrm{kW}]$$

$$Q_{tubine} = \frac{P}{\eta \cdot 10 \cdot h_{gross}} = \frac{25}{0.6 \cdot 10 \cdot 30} = 0.138 \approx 0.14 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

In addition to the 140 liters, 20 liters must be added, which must be available during turbine operation. Consequently, the channel must be widened, and the forebay tank must be designed to accommodate 160 liters of water per second.

$$Q = Q_{turbine} + Q_{irrigation} = 140\frac{L}{s} + 20\frac{L}{s} = 160\frac{L}{s} = 0.16 \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

Step II: The maximum permitted flow velocity in the settling basin is determined with the approximation formula for critical velocity, and the known critical particle diameter. The velocity must be slower than 0.6m/s.

$$v_{crit} = 0.44 \cdot \sqrt{d} = 0.44 \cdot \sqrt{0.25} = 0.22 \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

$$v_{crit} = 0.22 < 0.6 \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

The settling speed of the critical particle in standing water can be deduced from the following diagram.

$$w_0 = \frac{100}{9 \cdot d} \cdot \left(\sqrt{1 + 1.57 \cdot 10^2 \cdot d^3} - 1\right)$$

$$w_0 = \frac{100}{9 \cdot 0.25} \cdot \left(\sqrt{1 + 1.57 \cdot 10^2 \cdot 0.25^3} - 1\right) = 38.145 \left[\frac{\mathrm{mm}}{\mathrm{s}}\right]$$

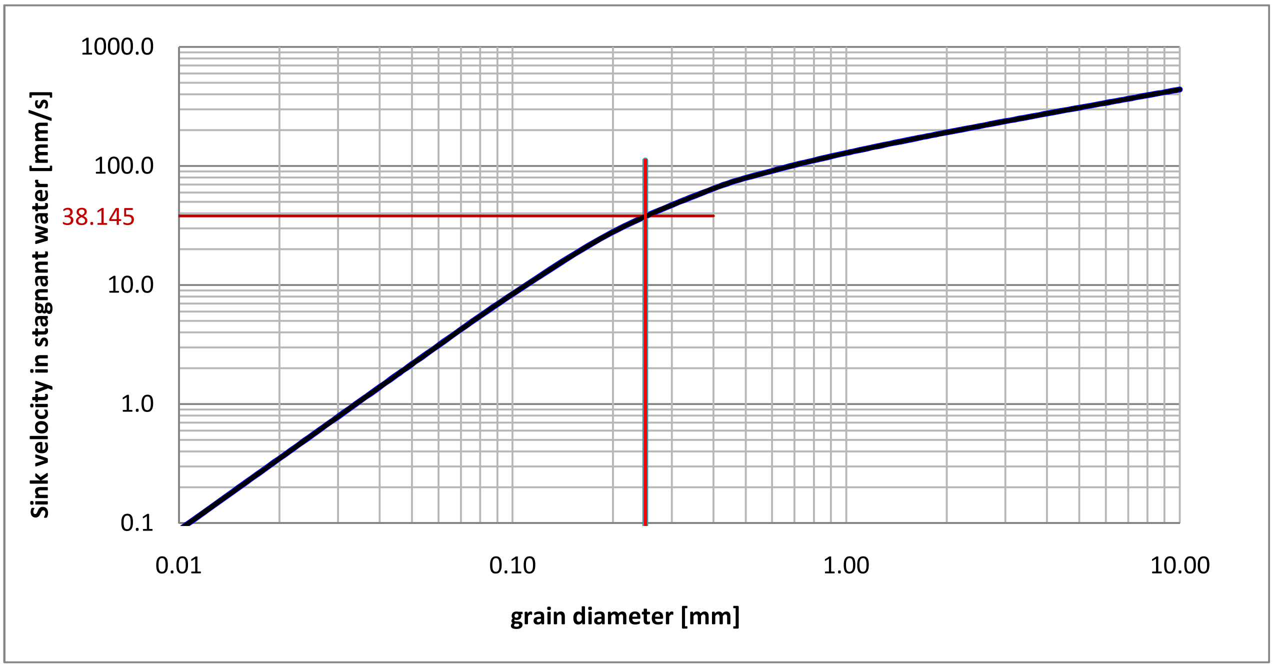


Figure 6.9 Sink velocity of a 0.25 mm grain in stagnant water

Now we can define the measurements of the settling area. On the basis of the available space on the hillside (see picture, proposed forebay tank location), the structure must be as narrow as possible.

$$W_{settling} = 2 \cdot W_{channel} = 2 \cdot 1 = 2 \ \ [\mathrm{m}]$$

With the computed values, we can now calculate the minimum settling basin depth. The settling area must minimally be this depth; otherwise the flow velocity in the basin will be too great.

$$H_{min} = \frac{Q}{W_{settling} \cdot v_{crit}} = \frac{0.16}{2 \cdot 0.22} = 0.36 \ \ [\mathrm{m}]$$

In most cases, the minimum depth is too low and a perfect settling of the particles is not possible. In these cases the settling height $H_{settling}$ must be increased, which also causes a shortening of the settling length. This, again, is advantageous when space is an issue. We value the depth between 0.3 – 1.0m and can then choose a suitable depth.

The settling speed must be reduced because particles sink slower in flowing water than in standing water. For this reason, we require the flow velocity of the water in the settling basin, which also changes with varying water depth.

$$v = \frac{Q}{W_{settling} \cdot H_{settling}} = \frac{0.16}{2 \cdot H_{settling}} \ \ \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

The settling speed w_0 of the 0.25 mm particle in standing water is 38.145 m/s. If the calculated velocity in moving water is smaller than 0, the associated settling depth is too small and we must choose a larger one.

$$w = w_0 - \left(v \cdot \frac{0.132}{\sqrt{H_{settling}}}\right) \ \ \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

$$w = must\ be\ larger\ than\ 0$$

With these values, we can now calculate the settling length.

$$L_{settling} = H_{settling} \cdot \frac{v}{w} \ \ [\mathrm{m}]$$

The following values were computed and presented in a table:

$H_{settling}$	v	w	$L_{settling}$
0.30	0.27	-0.026	impossible
0.40	0.20	-0.004	impossible
0.50	0.16	0.008	9.67
0.60	0.13	0.015	5.19
0.70	0.11	0.020	3.98
0.80	0.10	0.023	3.42
0.90	0.09	0.026	3.10
1.00	0.08	0.028	2.90

Table 6.1 Settling length according to the basin depth

We select the settling depth of 0.8m and thus obtain a relatively short settling basin. The calculated length is 3.42 m, which is rounded to 3.5 m.

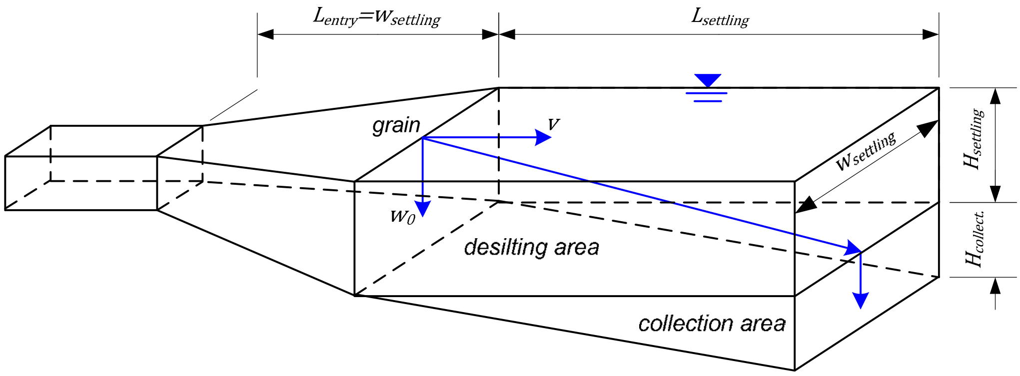


Figure 6.10 System of a settling basin

The dimensions of the settling basin are now determined. From this, we can derive the minimum volume of the collection basin.

$$V_{collection\ capacity} = \frac{V_{settling}}{4} = \frac{W_{settling} \cdot H_{settling} \cdot L_{settling}}{4} = \frac{2\ \cdot 0.8\ \cdot 3.5}{4} = 1.4\ [\mathrm{m}^3]$$

The volume of the collection basin is calculated as follows and is transformed for $H_{collection}$.

$$V_{collection\ capacity} = V_{collection}$$

$$V_{collection} = \frac{W_{settling} \cdot H_{collection} \cdot L_{settling}}{2}$$

$$H_{collection} = \frac{2 \cdot V_{collection}}{W_{settling} \cdot L_{settling}} = \frac{2 \cdot 1.4}{2\ \cdot 3.5} = 0.4\ [\mathrm{m}] \quad chosen \rightarrow 0.5\ [\mathrm{m}]$$

In this case it makes sense to round up the collection depth from 0.4 to 0.5 m. We obtain a larger collection volume and the value is simple to use in further calculations.

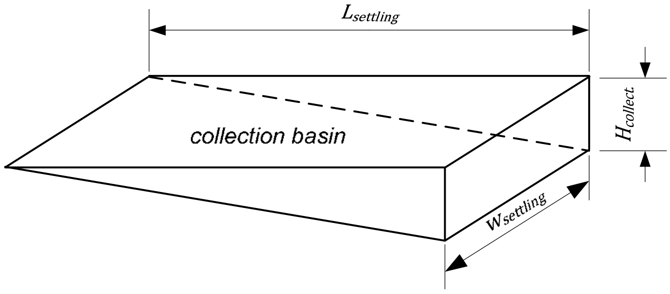


Figure 6.11 Volume of a collection basin

Step III: The inlet for the pressure pipe is connected directly behind the settling basin. The inlet is a structure at the diversion end of a conduit also called ***forebay chamber***. These combined structures are called the ***forebay tank***. The size is derived from the required balancing water volume, which must be available to offset fluctuations in the water height of the tank.

$$V_{forebay\ tank} = 75 \cdot Q_{turbine} = 75 \cdot 0.14 = 10.5 = 10.5\ \ [\mathrm{m}^3]$$

The combined water volume from the entry, the settling area and the forebay tank amount to the balancing water volume. It is important that the volume of the collecting area is not added here, since it is used to fill with bed load or suspended load and is therefore not available as water volume.

$$V_{forebay\ tank} = V_{balancing}$$

$$V_{balancing} = V_{entry} + V_{settling} + V_{forebay\ chamber}$$

We require the volume of the forebay chamber in order to continue our calculation. The remaining water volumes are known or can be determined from known measurements.

The water volume of the entry of the forebay tank can be calculated as a pyramidal frustum.

$$V_{entry} = \frac{L_{entry}}{3} \cdot \left(A_1 + A_2 + \sqrt{A_1 \cdot A_2}\right)$$

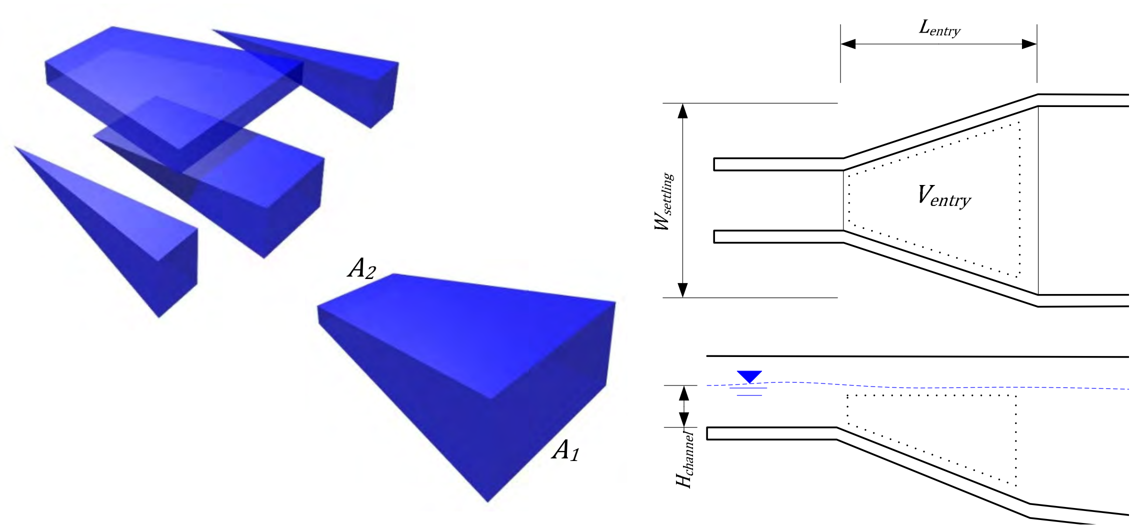


Figure 6.12 Entry water volume, tridimensional

Assumption: The water in the channel stands at 0.25m height after the expansion. In addition, the length of entry L_{entry} is the same as the settling width.

$$L_{entry} = W_{settling} = 2\ m$$

$$A_1 = W_{settling} \cdot H_{settling} = 2\ \cdot 0.8\ = 1.6\ [\mathrm{m}^2]$$

$$A_2 = W_{channel} \cdot H_{channel} = 1\ \cdot 0.25\ = 0.25\ [\mathrm{m}^2]$$

$$V_{entry} = \frac{L_{entry}}{3} \cdot \left(A_1 + A_2 + \sqrt{A_1 \cdot A_2}\right) = \frac{2}{3} \cdot \left(1.6\ + 0.25\ + \sqrt{1.6\ \cdot 0.25}\ \right) = 1.65\ [\mathrm{m}^3]$$

The water volume of the settling basin has the shape of a rectangular prism.

$$V_{settling} = L_{settling} \cdot W_{settling} \cdot H_{settling} = 3.5 \cdot 2 \cdot 0.8 = 5.6\ \ [\mathrm{m}^3]$$

Because we must find the dimensions of the forebay chamber, we transform the equation of the formula for the balancing water volume and arrive at the required water volume of the forebay chamber.

$$V_{balancing} = V_{entry} + V_{settling} + V_{forebay\ chamber}$$

$$V_{forebay\ chamber} = V_{balancing} - V_{entry} - V_{settling} = 10.5\ - 1.65 - 5.6 = 3.25\ \ [\mathrm{m}^3]$$

From here, we can calculate the measurements of the forebay chamber. Note that the penstock must be mounted below the water level at a distance four times its diameter. We know the diameter from the penstock calculations.

$$D_{penstock} = 14'' = 0.355\ [\mathrm{m}]$$

$$H_{forebay\ chamber} = 4 \cdot D_{penstock} + \frac{1}{2} \cdot D_{penstock} = 4 \cdot 0.355 + \frac{1}{2} \cdot 0.355 = 1.597\ [\mathrm{m}] \approx 1.6\ [m]$$

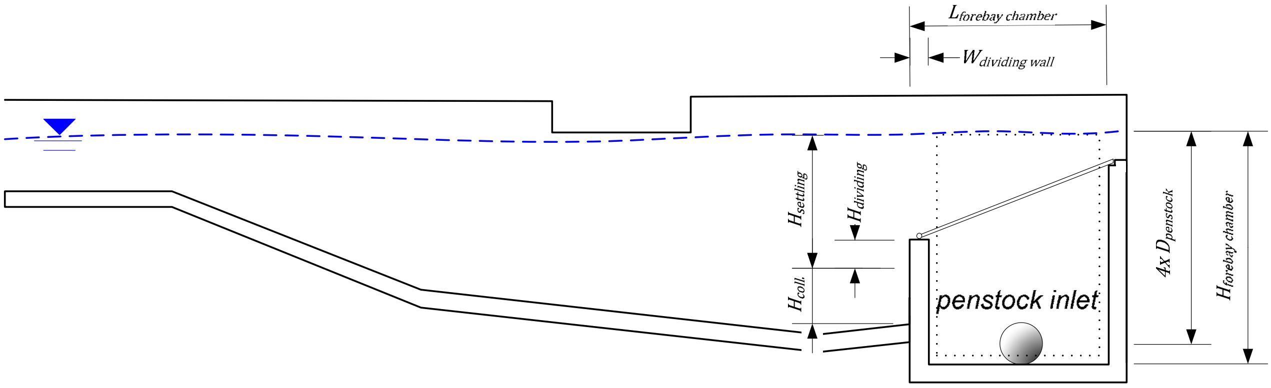


Figure 6.13 Forebay chamber with dimensioning

The volume of the forebay chamber also contains a wall, which divides the settling basin from the forebay chamber. Because this wall displaces water, its volume must be deducted from the total volume. The thickness of the wall is 0.15m. In addition, we know the collection height, which must be increased by 0.2m (see chapter Forebay Tank.)

$$V_{dividing\ wall} = \left[H_{forebay\ chamber} - \left(H_{settling} - H_{dividing}\right)\right] \cdot W_{settling} \cdot W_{dividing\ wall}$$
$$= [1.6 - (0.8 - 0.2)] \cdot 2 \cdot 0.15 = 0.3\ [\mathrm{m}^3]$$

$$V_{forebay\ chamber} = L_{forebay\ chamber} \cdot W_{settling} \cdot H_{forebay\ chamber} - V_{dividing\ wall}$$

The forebay tank should be constructed to allow for entry in case maintenance is required. It is recommended that the chamber be at least one meter long.

$$L_{forebay\ chamber} = \frac{V_{forebay\ chamber} + V_{dividing\ wall}}{W_{settling} \cdot H_{forebay\ chamber}} = \frac{3.25 + 0.3}{2 \cdot 1.6} = 1.10\ [\mathrm{m}]$$

Step IV: Once the dimensions of the structure are determined, the height of the overflow must be established. It is essential to build the overflow in a way so that the water level is stable while the turbine is in operation. With the Poleni formula from the Intake chapter, we can determine this height. It is a rectangular weir body with sharp edges.

The width b is known, and we calculate the flooding head H_B. The width of the weir is identical to the channel width.

$$W_{channel} = W_{weir} = b = 1\ [\mathrm{m}]$$

When the turbine is in operation, only 20 litres per second flow over the weir. The remaining 140 litres are funneled into the penstock. As a factor for a sharp edged weir $\mu = 0.5$ is selected:

$$H_{B1} = \frac{Q_{irrigation}^{\frac{2}{3}}}{(2.95 \cdot \mu \cdot b)^{\frac{2}{3}}} = \frac{(0.02)^{\frac{2}{3}}}{(2.95 \cdot 0.5 \cdot 1)^{\frac{2}{3}}} = 0.056\ [\mathrm{m}] \approx 0.06\ [\mathrm{m}]$$

When the turbine is not in operation, all water must be guided over the overflow. This causes the water level in the basin to rise. This water level in addition to the freeboard yields the total height of the structure.

$$H_{B2} = \frac{Q_{flood}^{\frac{2}{3}}}{(2.95 \cdot \mu \cdot b)^{\frac{2}{3}}} = \frac{(0.16)^{\frac{2}{3}}}{(2.95 \cdot 0.5 \cdot 1\,)^{\frac{2}{3}}} = 0.199\ [\mathrm{m}] \approx 0.2\ [\mathrm{m}]$$

Total building height

$$H_{building} = H_{forebay\ chamber} + (H_{B2} - H_{B1}) + freeboard$$
$$= 1.6\ m + (0.2 - 0.06) + 0.15 = 1.89\ [\mathrm{m}] \approx 1.90\ [\mathrm{m}]$$

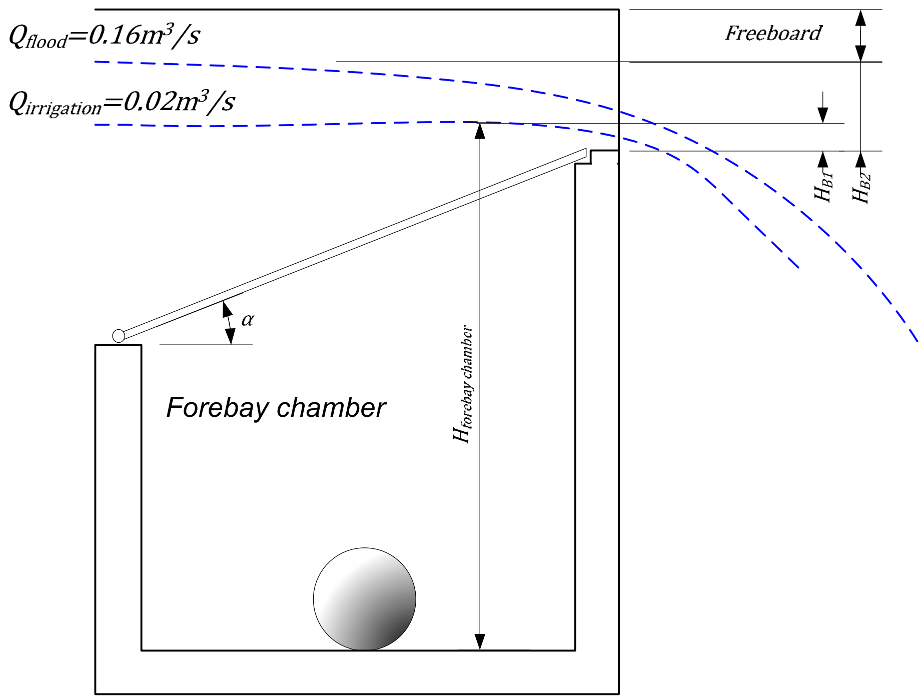


Figure 6.14 Overflow situation in the channel

The losses depend on the inclination of the trash rack.

$$\alpha = \tan^{-1}\left(\frac{H_{settling} - H_{dividing} - H_{B1}}{L_{forebay\ chamber} - W_{dividing\ wall}}\right) = \tan^{-1}\left(\frac{0.8\ - 0.2\ - 0.06}{1\ - 0.15}\right) = 32.4° \approx 30°$$

Step V: The rake causes head losses which can reduce the performance of the structure. The head decreases. Usually, the results are negligible. When the deficit is too large, the rake bars must be mounted at greater distances from each other.

We know from the turbine manufacturer that the distance between the runner blades is 0.05 m. With this information, we can pre-design the rake and determine whether it can be built that way. Half of the runner blade distance yields the distance between the rack bars.

$$a = \frac{0.05}{2} = 0.025\ [\mathrm{m}]$$

Loss coefficient for rectangular cross sections.

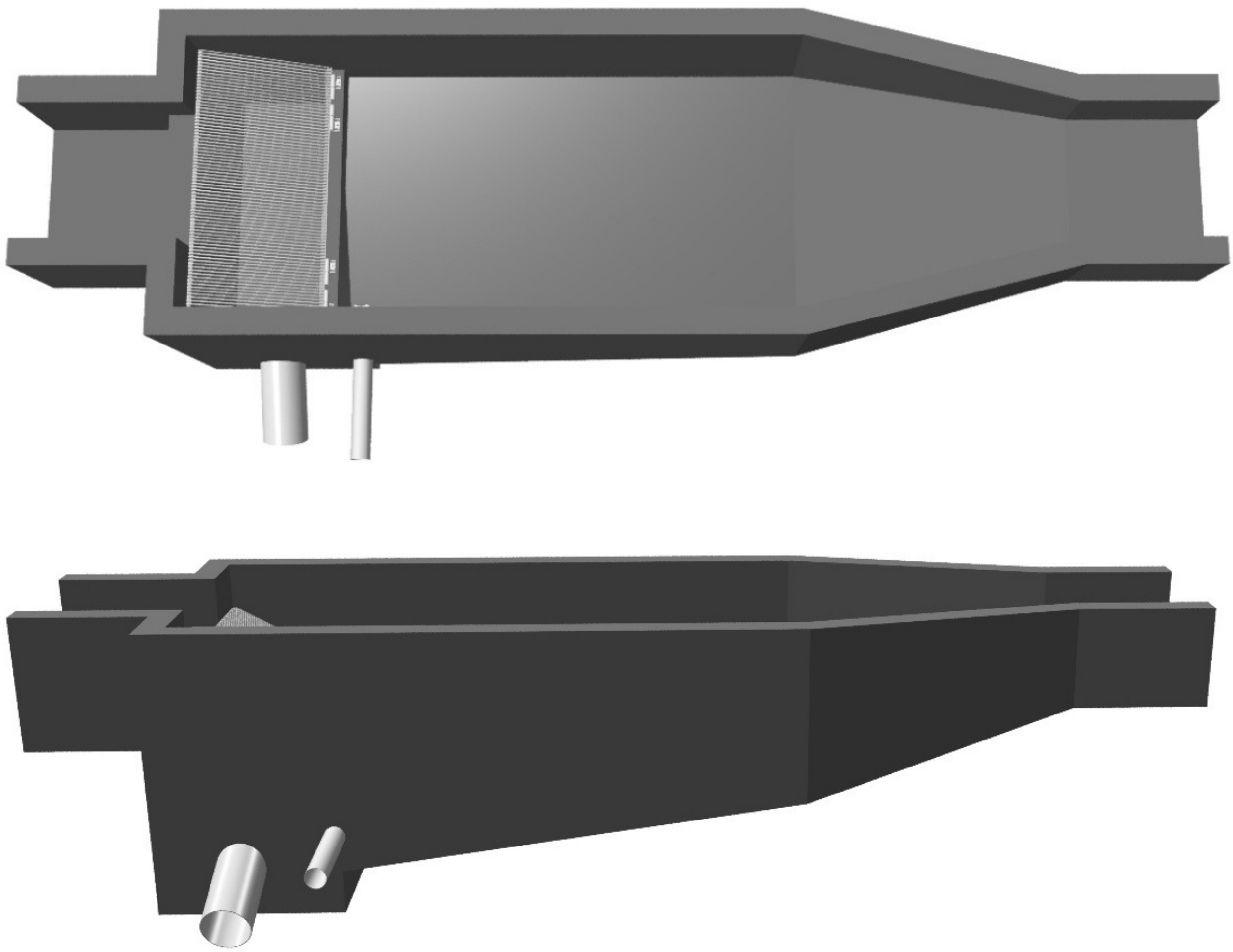
$$\xi = \varphi \cdot \left(\frac{s}{a}\right)^{4/3} \cdot \sin\alpha = 2.42 \cdot \left(\frac{0.005}{0.025}\right)^{4/3} \cdot \sin 30° = 0.141$$

Head losses according to Kirschmer and Mosonyi.

$$v = \frac{Q_{flood}}{W_{settling} \cdot H_{settling}} = \frac{0.16}{2 \cdot 0.8} = 0.1 \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

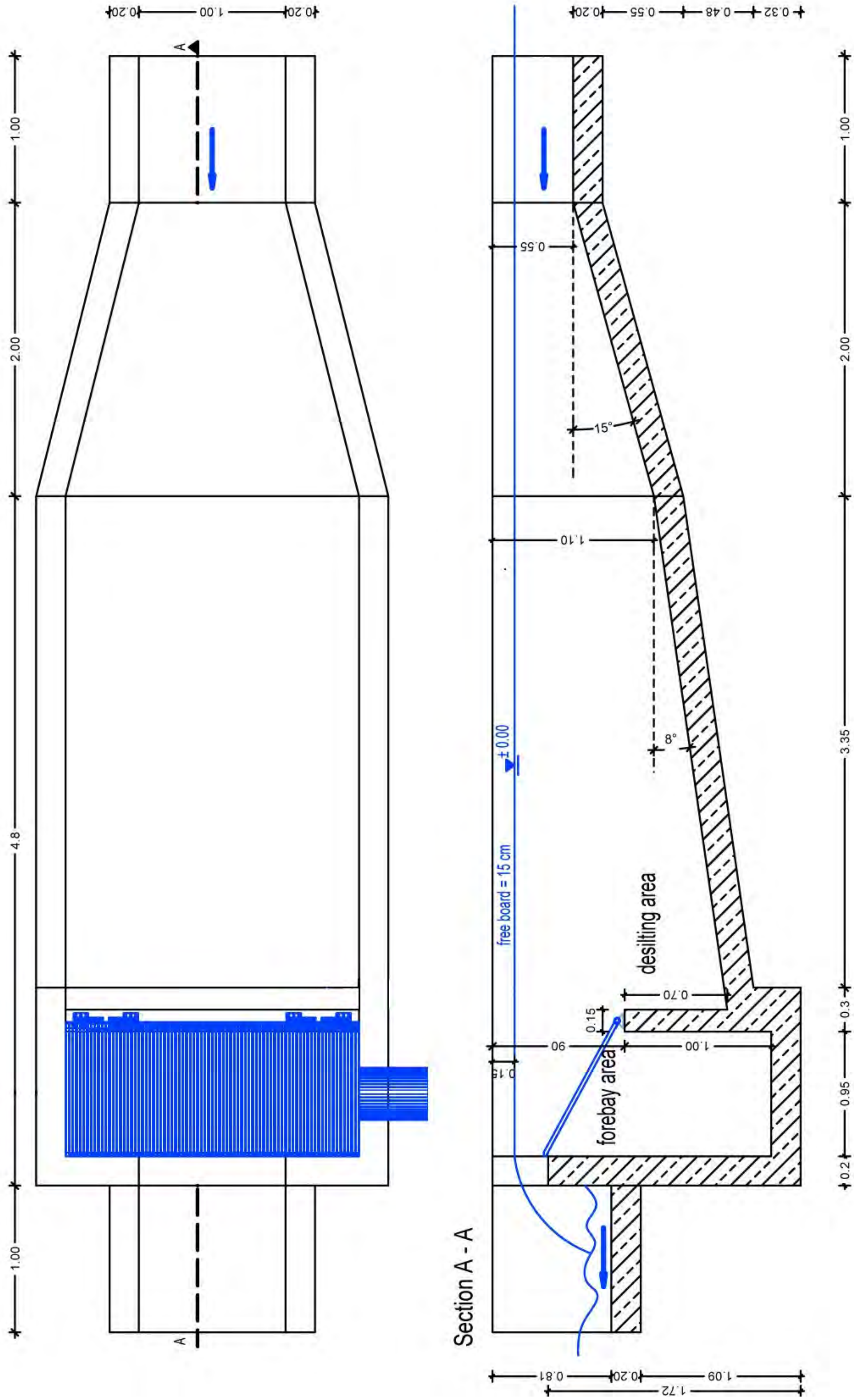
$$\Delta h = \xi \cdot \frac{v^2}{2 \cdot g} = 0.141 \cdot \frac{(0.1)^2}{2 \cdot 9.81} = 0.0005 \ [\mathrm{m}]$$

The head loss is very negligible, therefore we can omit it. The rake can be built with the available steel bars and the distance between the bars can also be implemented as suggested.



Picture 6.9 Designed forebay tank

Forebay tank scale 1 : 40



7 Penstock assessment

A penstock is a closed conduit for supplying water under pressure to a water wheel or turbine. The penstock is attached at its upper end to a forebay basin which feeds the penstock with the needed amount of water and prevents fluctuation of the head within the system due to load changes.

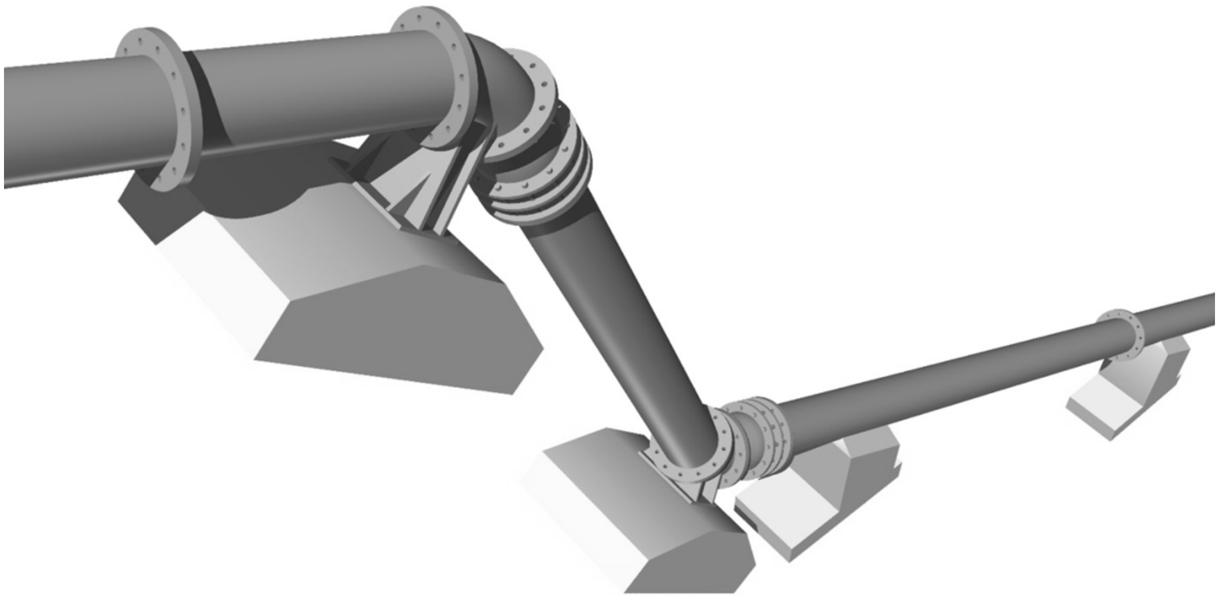


Figure 7.1 Above-ground steel penstock

The penstock costs are quite significant in the overall calculation of your scheme so you should have a closer look at the efficiency h_{net} / h_{gross}. The efficiency of the penstock is connected to the velocity of the water passing through the pipe. The higher the velocity, the more friction losses appear. Since the diameter of a pipe has a large influence on the velocity, the diameter of the pipe must be carefully chosen by considering its cost/performance ratio. The basic relation between flow and pipe diameter is obtained by the continuity equation:

$$Q = A \cdot v = \frac{d^2}{4} \cdot \pi \cdot v \quad \left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$$

where $Q = flow$ [m³/s]; $A = area$ [m²]; $v = velocity$ [m/s]; $d = diameter$ [m]; $\pi = 3.414$ [-]

Penstock losses have different effects on total efficiency:

- Friction losses (related to diameter, roughness and length)
- Bending losses (related to diameter, roughness, angle and bending radius)
- Inflow losses (related to diameter and design)
- Valve losses (related to diameter and design)

Since roughness has an influence on the whole length of the pipe, it forms the major part of the total losses.

7.1 Friction losses

When channelling water from your forebay tank to the turbine, losses occur due to friction and other obstacles within the penstock. *Friction loss* refers to the portion of pressure lost by fluids while moving through a pipe. Narrowness (which increases the flow velocity) and roughness are the variables in the following calculations. Thus, the diameter of your penstock is, besides the surface structure, the second parameter which you must optimize in your penstock assessment. Head loss can be calculated using the Darcy–Weisbach equation:

$$h_f = f \cdot \frac{L}{d} \cdot \frac{v^2}{2 \cdot g}\ [\mathrm{m}]$$

The corresponding pressure drop can then be evaluated as:

$$\Delta p = \rho \cdot g \cdot h_f$$

where h_f is the loss of head by friction (in [m] of water column); f is the friction factor from the Moody chart (displayed in Figure 7.2); L is the pipe length and d the internal diameter of your penstock, both measured in [m]; v the mean velocity of flow [m/s], and g the acceleration of gravity [m/s^2]; ρ is the density of water [kg/m^3].

The factor f is a dimensionless factor generated by the Reynolds number Re and the relative roughness expressed as k/d in which k represents the absolute roughness [mm]. In some publications the friction factor f is described as λ.

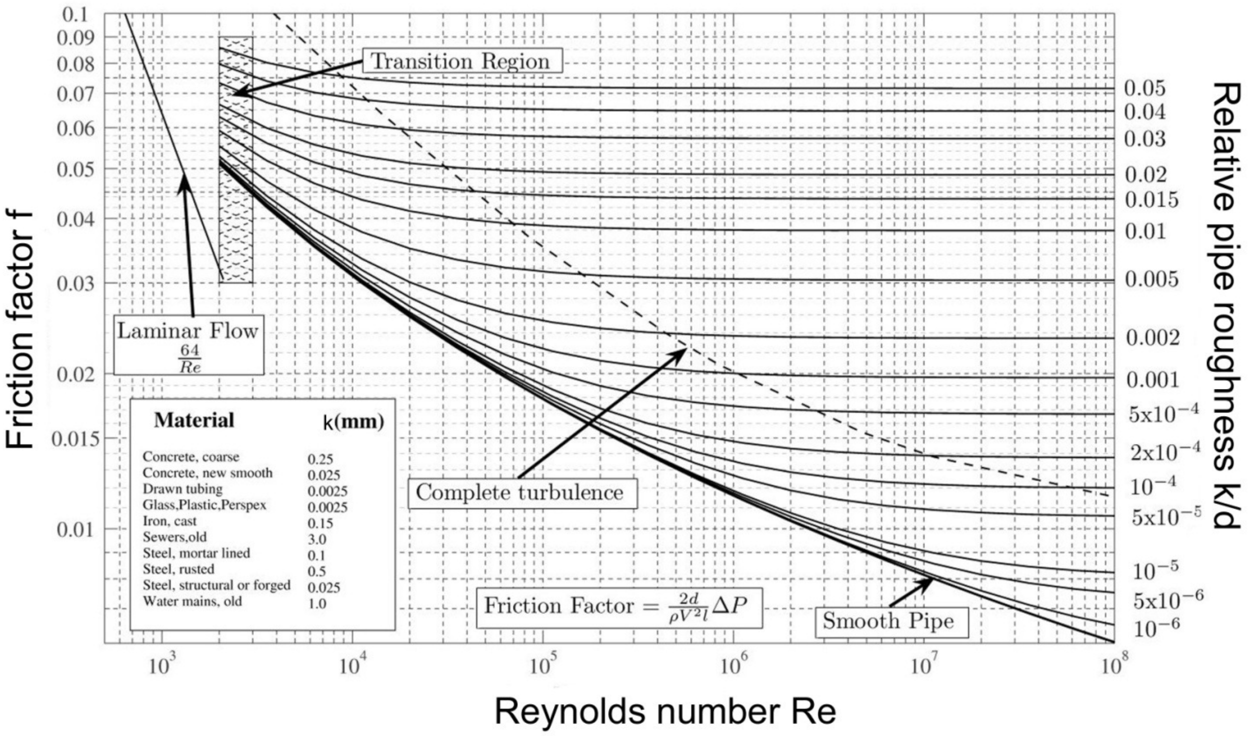


Figure 7.2 Moody chart showing friction factor plotted against Reynolds number for various stages of roughness.

The Reynolds number basically depends on the flow velocity where the concerned fluid is water and the temperature is constant.

$$Re = \frac{v \cdot d \cdot \rho}{\mu} \ [-]$$

where v is the mean velocity of flow [m/s], d the internal diameter of your penstock [m]; ρ is the density of water [kg/m^3]; and η stands for the dynamic viscosity of water [kg/ms] which depends on the temperature (Figure 7.3).

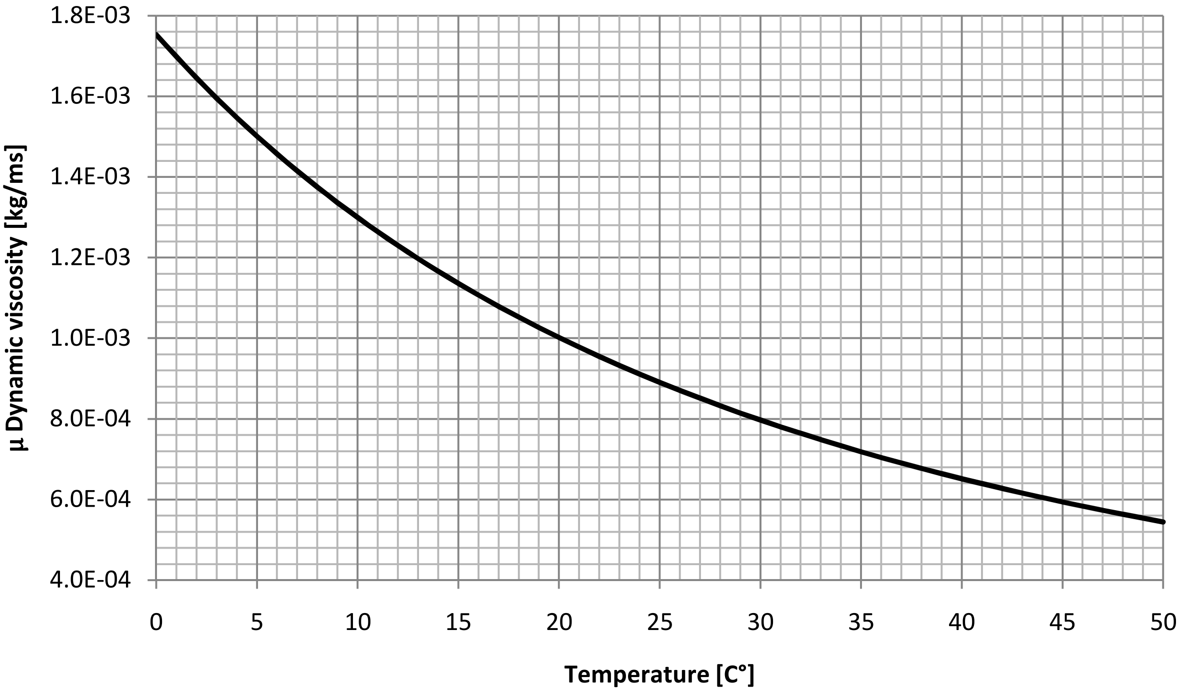


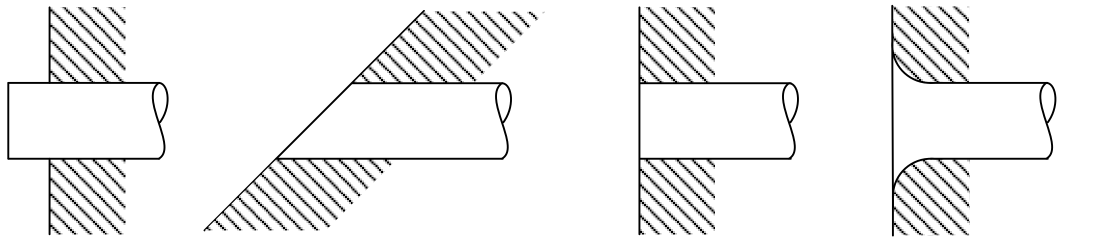
Figure 7.3 Dynamic viscosity of water in relation to temperature

7.2 Local losses

Local losses in the penstock may occur where a steady flow is disturbed. This appears at inflow sections, bends or sudden contractions, where turbulences lead to losses of the total head. Furthermore, head losses can be caused by fittings, valves and other obstacles within the penstock. Again, the loss is quantified by a factor in the velocity portion of the energy equation:

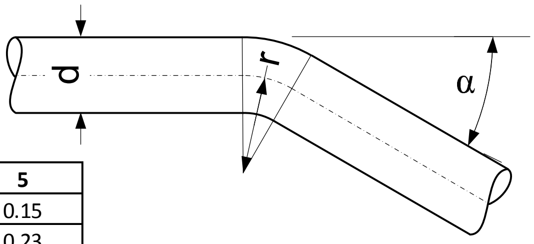
$$h_f = \zeta \cdot \frac{v^2}{2 \cdot g} \ [\text{m}]$$

Use the loss coefficients according to Adam Harvey's *Micro Hydro Design Manual*.



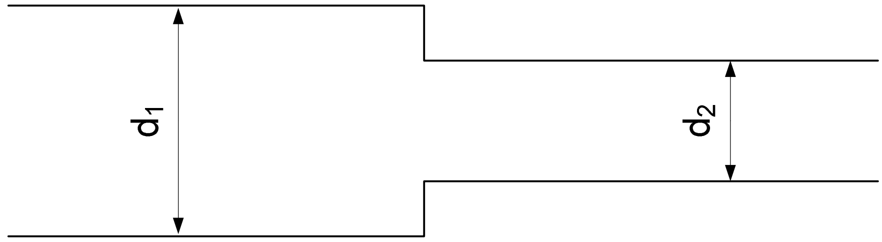
$\zeta = 0.5$ up to 1.0 $\quad$ $\zeta = 0.8$ $\quad$ $\zeta = 0.5$ $\quad$ $\zeta = 0.4$ down to 0.03

Figure 7.4 Head loss coefficients for penstock intakes from a forebay tank



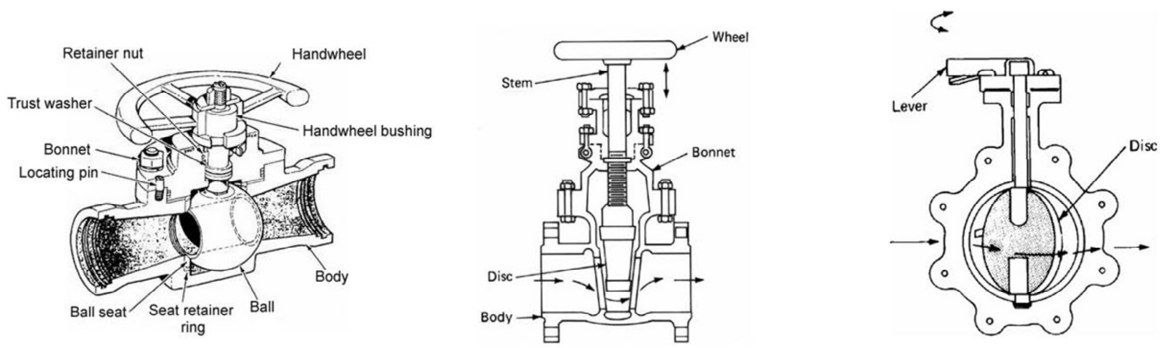
r/d	1	2	3	5
ζ (α=20°)	0.36	0.25	0.20	0.15
ζ (α=45°)	0.45	0.38	0.30	0.23
ζ (α=90°)	0.60	0.50	0.40	0.30

Figure 7.5 Head loss coefficients for bends



d_1/d_2	1.0	1.5	2.0	2.5	5.0
ζ	0.00	0.25	0.35	0.40	0.50

Figure 7.6 Head loss coefficient for sudden contractions



$\zeta = 0.0$ $\zeta = 0.1$ $\zeta = 0.3$

Figure 7.7 Head loss coefficients for valves

7.3 Penstock layout

In this chapter, the actions and reactions on the penstock are evaluated and interpreted. At a later stage, we will present a concept for the civil work of slide and anchor blocks.

Forces on the penstock

The penstock experiences a variety of forces such as water pressure, expansion, weight and deflection forces. Some of them are bound to the local conditions; others depend on the operation status. The problems caused by internal forces can be solved by designing the appropriate wall thickness of the penstock. Basically, the pipe sections have to resist the tensions of the different situations. Problems of expansion can be solved by expansion joints which compensate for the variation of pipe length caused by temperature differences.

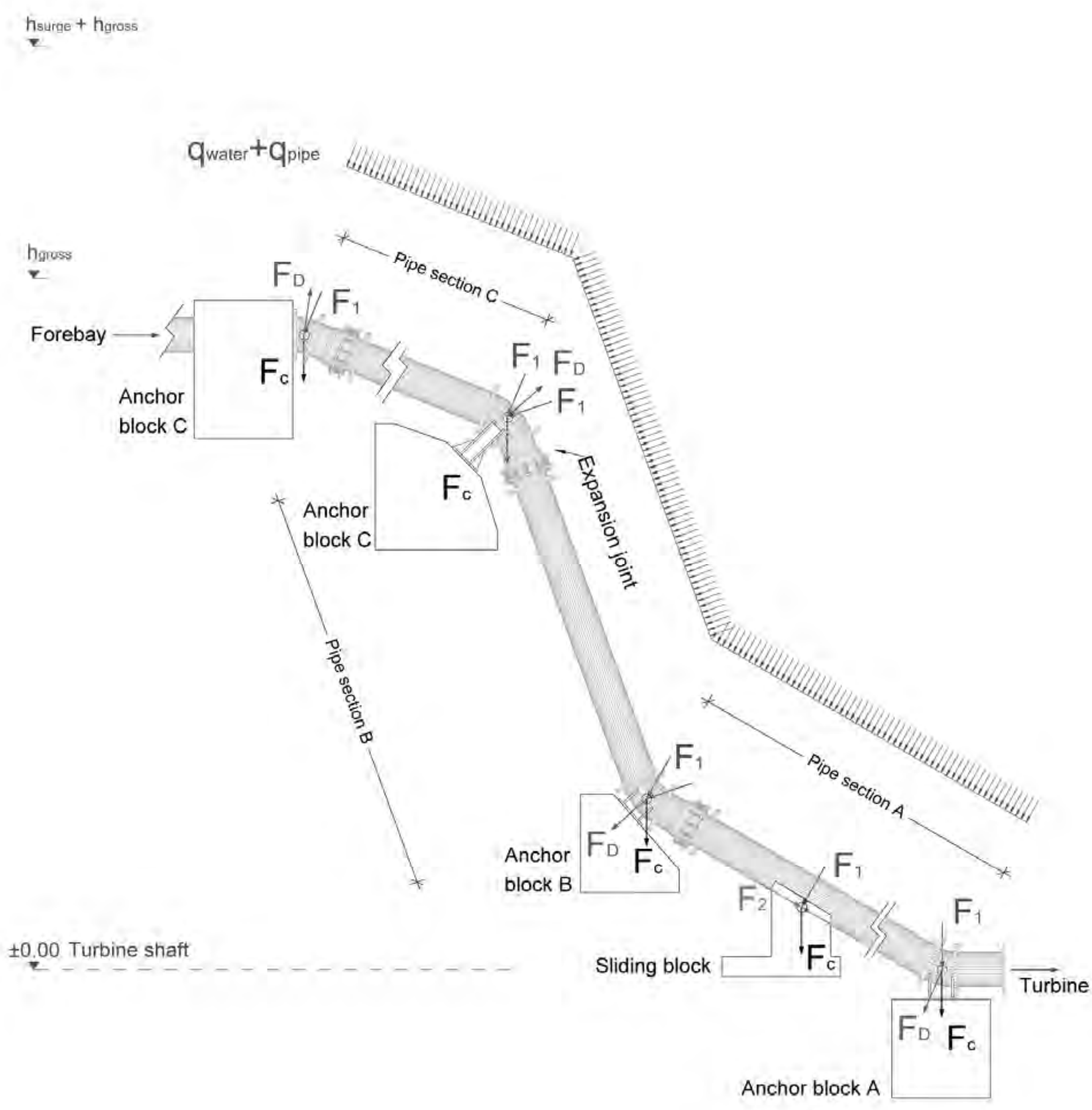


Figure 7.8 Forces on penstock

External forces are led into the ground by foundation blocks. For the design of these foundation blocks, first all possible combinations of forces must be defined. For example, two situations are most common for the assessment of a penstock and its bearings: The plant is either working or not. In these cases, you will have to assess all the possibilities which could affect the penstock.

Case 1

In load case 1, the turbine is not running, but water is already in the pipe. Possible forces acting on the penstock and foundations are:

- The weight force of steel pipe
- The weight force of standing water in the pipe
- Internal pressure caused by hydrostatic pressure

This situation takes place during periods when no power is needed, but still has to be available, for example, during day time when no power for light is necessary. The influence of an empty penstock is negligible, because the weight forces are only slightly different when compared to a filled penstock.

Case 2

In load case 2, the turbine is running and the water is flowing through the pipe. The following forces appear:

- The weight force of steel pipe
- The weight force of running water in the pipe
- Internal pressure caused by water hammer
- Bend forces caused by hydrostatic pressure
- Bend forces caused by a change of linear momentum
- Weight forces of anchor blocks

In this case, the marginal conditions are set to the highest limit to prevent any damage to the pipe system. This means that the total net head is assumed to be what it would be under surge pressure (water hammer). Of course, this event is unlikely to occur to its full extent, but it can be understood as a safety measure. One must consider that each time the valve is opened and closed, the pipe is exposed to surge pressure (water hammer). The amount of the surge pressure depends on how fast the valve gets opened or closed. The basic rule is: the slower the valve is operated, the smaller the pressure in the pipe rises or falls. It is a fact that the operating status changes during the day. This occurs when the water for the turbine is used for other purposes such as irrigation. Hence, we can say that the valve will be operated at least twice a day, which may potentially cause a pressure surge.

7.4 Water hammer

Water hammer is a pressure surge or pressure wave resulting when a fluid in motion is suddenly forced to stop or change its direction (momentum change). Water hammer commonly occurs when a valve is closed too fast at either at the beginning or the end of a pipeline system, and causing a pressure wave in the pipe. The water hammer causes a higher pressure than just the hydrostatic pressure caused by the water column in the pipe. It is thus the leading criteria for the assessment of the pipe wall's thickness. Surge pressure can be expressed in meter water column (h_{surge}), equal to hydrostatic pressure (h_{gross}). First, you must calculate the pressure wave velocity using the following equation:

$$a = \frac{\sqrt{\frac{E_w}{\rho}}}{\sqrt{\frac{d}{t} \cdot \frac{E_w}{E_{Penstock}} + 1}} \quad \left[\frac{\mathrm{m}}{\mathrm{s}}\right]$$

where E_w = module of elasticity of water ($E_w = 2 \cdot 10^9$ [N/m^2]) and $E_{Penstock}$ = module of elasticity of penstock (according to the manufacturer or as in Table 7.1), ρ = density of water, d = penstock diameter and t = wall thickness of penstock.

Material	Module of elasticity E [N/m2]	Coefficient of linear expansion α [m/m°C]	Ultimate tensile strength σ [N/m^2]	Density ρ [kg/m^3]
Steel	$200 \cdot 10^9$	$12 \cdot 10^{-6}$	$350 \cdot 10^6$	$7.8 \cdot 10^3$
PVC	$2.8 \cdot 10^9$	$54 \cdot 10^{-6}$	$28 \cdot 10^6$	$1.4 \cdot 10^3$
HDPE/MDPE	$0.2 - 0.8 \cdot 10^9$	$140 \cdot 10^{-6}$	$6\text{-}9 \cdot 10^6$	$0.9 \cdot 10^3$
Ductile iron	$170 \cdot 10^9$	$11 \cdot 10^{-6}$	$350 \cdot 10^6$	$0.7 \cdot 10^3$
Cast iron	$100 \cdot 10^9$	$10 \cdot 10^{-6}$	$140 \cdot 10^6$	$7.2 \cdot 10^3$
Concrete	$20 \cdot 10^9$	$10 \cdot 10^{-6}$	variable	$2.5 \cdot 10^3$
GRP	$20 \cdot 10^9$	$30 \cdot 10^{-6}$	$360 \cdot 10^6$	$2.2 \cdot 10^3$

Table 7.1 Physical characteristics of common materials

The surge head h_{surge} is calculated as followed:

$$h_{surge} = \frac{a \cdot v}{g} = \frac{a \cdot Q \cdot 4}{g \cdot d \cdot \pi} \ \text{[m]}$$

where Q = the design flow in the penstock, d = penstock diameter and g = acceleration of gravity. The total head for the pipe assessment is the sum of surge head and gross head:

$$h_{Total} = h_{surge} + h_{Gross} \ \text{[m]}$$

7.5 Expansion due to temperature differences

Your penstock is exposed to differences in temperature. There is a difference between day and night, summer and winter and even between the upper and the lower side of the penstock because only one side faces the sun. During turbine service, the water cools the pipe and keeps the temperature differences at a moderate level. During revision work or during winter months when the penstock is drained, the temperature differences are quite remarkable. Assume the greatest difference in temperature to calculate the expansion/ contraction length ΔL, to size the expansion joints.

$$\Delta L = \alpha \cdot (T_{Hot} - T_{Cold}) \cdot L \ \text{[m]}$$

where α = coefficient of linear expansion (according to the manufacturer or as in Table 7.1), $T_{Hot} - T_{cold}$ = maximum assumed temperature difference and L = length of the concerned straight penstock stretch. The expansion joint must be able to compensate for the additional length ΔL.

7.6 Sizing the penstock wall thickness

The penstock must withstand the biggest stress without any damage or symptoms of fatigue. In penstocks, the hydrostatic pressure, or the sum of hydrostatic and surge pressure, is the leading indicator for penstock wall assessment. The mathematical approach to calculate the minimum wall thickness t_{min} for heavy-walled penstock is as follows:

$$t_{min} = r_i \cdot \left(\sqrt{\frac{(\sigma_u - p_i)}{(\sigma_u + p_i)}} - 1 \right) \ \text{[m]}$$

where r_i = the internal radius of the penstock [m], σ_u = ultimate tensile strength (note that σ_u is a negative value) and p_i = internal maximum assumed pressure at the regarded penstock section.

It is advised to consider a safety factor to also ensure that disturbances in the material or hidden erosion will not lead to failures at maximum stress. A safety factor of $SF = 2.5$ *up to* 3.5 is adequate for most applications.

$$SF = \frac{t_{chosen}}{t_{min}} = 3.5 \ [-]$$

For thin-walled penstocks, use Barlow's formula which relates the internal pressure which a penstock can withstand to the dimensions and strength of its material:

$$t_{min} = \frac{p_i \cdot d}{\sigma_u \cdot 2} \ [\mathrm{m}]$$

For steel pipes, use additional correction factors to take into account that welding- seams and corrosion cause a reduction in the effective thickness.

$$t_{min} = \frac{p_i \cdot d}{\sigma_u \cdot V_s \cdot 2} + t_c \ [\mathrm{m}]$$

in which t_c = a corrosion loss of 1 to 2 mm and V_s = quality of the welding-seam:

- Single-sided weld: $V_s = 0.7$
- Double-sided weld: $V_s = 0.8$
- Double-sided weld, calcined: $V_s = 0.9$

Figure 7.9 Mild steel penstock on sliding blocks

7.7 Design of slide and anchor block

The penstock must be securely fixed to the ground at certain points. First, the penstock is attached to the forebay tank where the water enters the pipe. Secondly, anchor blocks are placed where the penstock has a bend, either in a horizontal or a vertical direction. Finally, the penstock is anchored somewhere in front of the turbine to withhold all forces from turbine casing and its base frame. All the above-mentioned bearings are anchor blocks which avoid movements of the penstock in any direction. Since the penstock is exposed to the atmosphere, the penstock will expand and contract as described in chapter 7.5. Hence, you must place an appropriate expansion/contraction-joint to compensate for the axial movement of the pipe in every straight section of your penstock alignment. Support blocks are needed to hold the penstock's position along the straight sections between the anchor blocks. These support blocks must allow the axial movement of the penstock and are therefore called *sliding blocks*.

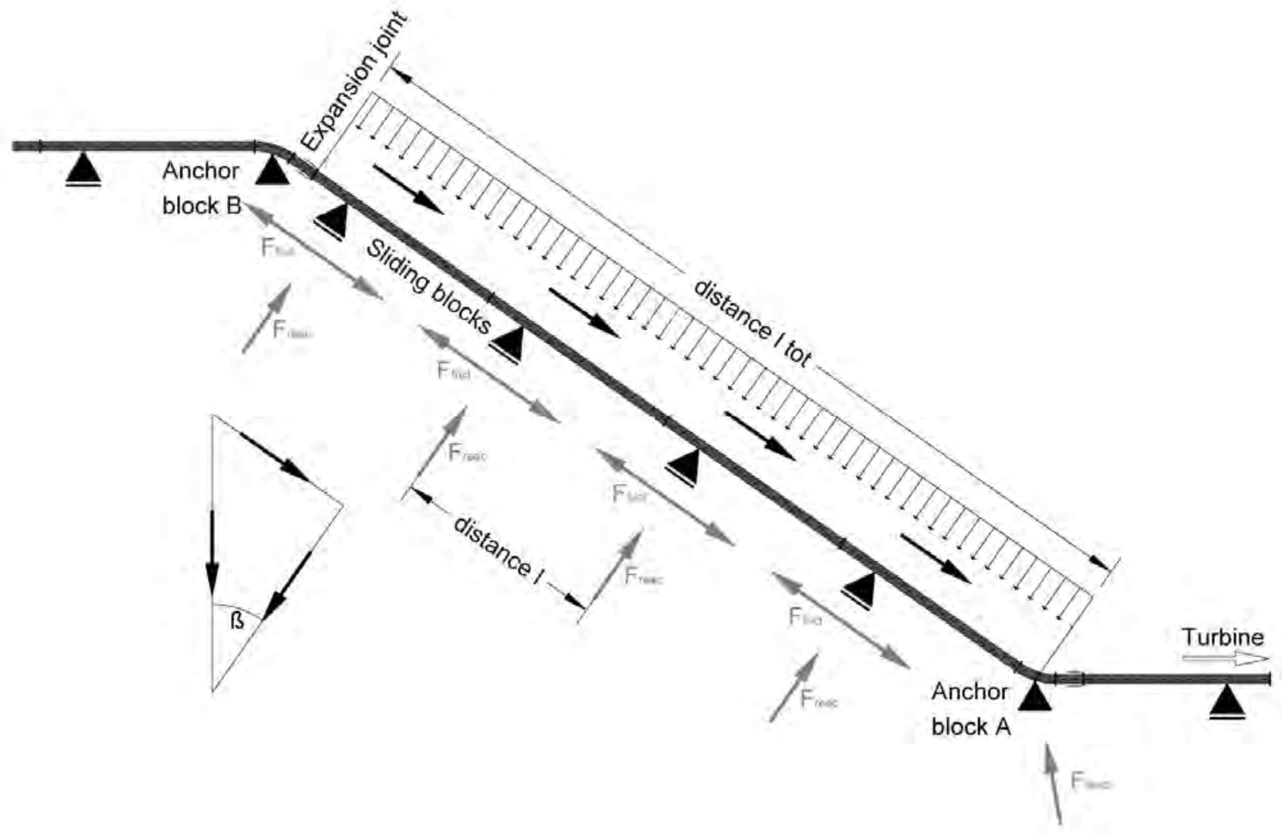


Figure 7.10 Self-weight on a penstock section between two anchor blocks

7.8 Forces acting on sliding blocks

Sliding blocks are exposed to the weight forces of the penstock and the bearing itself, as well as to the expansion and contraction of the penstock. They are designed to hold the penstock in position and to only allow movement in an axial direction. The design varies depending on the resistance of the bearing against sliding: the more friction the bearing creates against the expanding/contracting penstock, the larger the axial force and therefore, the larger the concrete mass needed. For example, a concrete bearing covered with tarred paper will reduce the friction forces to approximately 50 % of the weight forces $F_{pen\perp}$ caused by the penstock and the contained water. $F_{pen\perp}$ is a combination of the weight of water and the pipe itself. In special cases, even a superimposed load like snow must be taken into account.

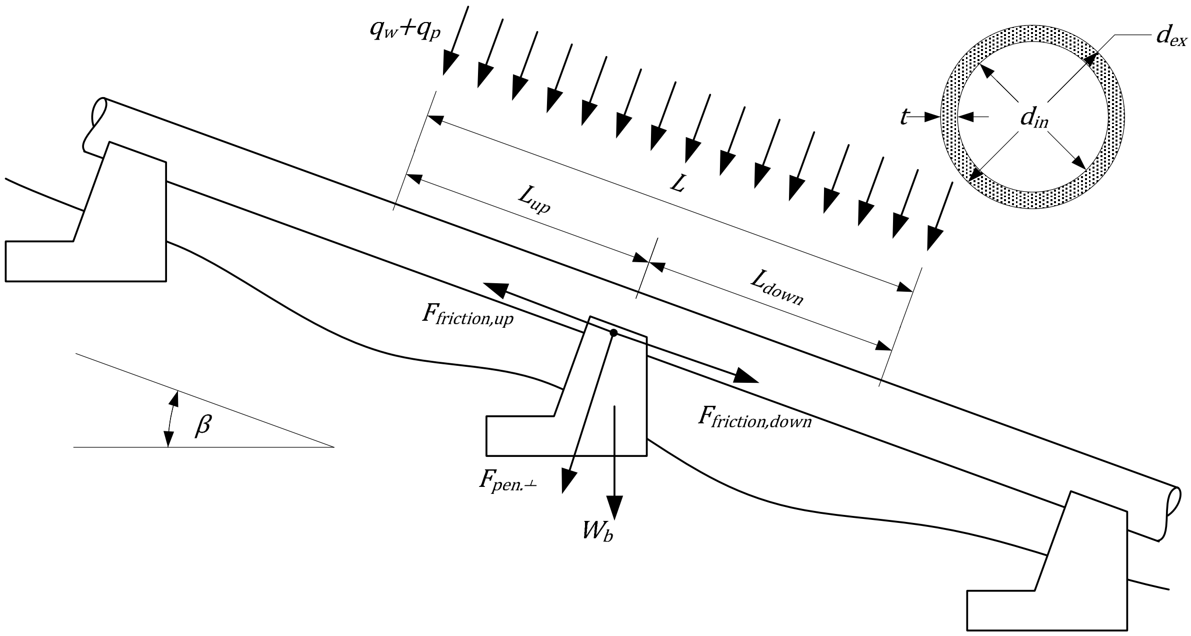


Figure 7.11 Forces acting on a sliding block

The forces acting on sliding blocks under the load of a penstock with an inclination angle β:

$$F_{pen\perp} = L \cdot (q_w + q_{pen} + q_x) \cdot \cos\beta \;\; [\mathrm{N}]$$

$$F_{friction} = \pm\mu_k \cdot F_{pen\perp} \;\; [\mathrm{N}]$$

$$W_b = V_b \cdot \rho_b \cdot g \;\; [\mathrm{N}]$$

where $F_{pen\perp}$ is the bearing force caused by penstock- and water weight; $F_{friction}$ is the friction portion of the moving penstock pointing either uphill or downhill (μ_k is the kinetic friction coefficient depending on the sliding block design which may vary from $\mu_k = 0.2 - 0.5$); and W_b is the weight force of the sliding block calculated with the volume V_b and density ρ_b of material used.

The penstock- and water weight per meter can be calculated with the following formulas:

$$q_{pen} = d_{ex} \cdot \pi \cdot t \cdot \rho_{Pen} \cdot g \;\; \left[\frac{\mathrm{N}}{\mathrm{m}}\right]$$

$$q_w = d_{in}{}^2 \cdot \frac{\pi}{4} \cdot \rho_w \cdot g \;\; \left[\frac{\mathrm{N}}{\mathrm{m}}\right]$$

in which d_{ex} is the external- and d_{in} the internal diameter of the penstock [m]; t is the wall thickness of the pipe [m], ρ_{pen} is the density of the penstock material and g is acceleration of gravity [m/s^2]. For further calculation, split the forces into horizontal and into vertical components.

Horizontal force components of the penstock forces are:

$$F_{pen\perp}x = F_{pen\perp} \cdot \sin(\beta) \;\; [\mathrm{N}]$$

$$F_{friction}x = F_{friction} \cdot \cos(\beta) \;\; [\mathrm{N}]$$

$$Total\ horizontal\ force\ component\text{: } F_x = F_{pen\perp}x + F_{friction}x \;\; [\mathrm{N}]$$

Vertical force components of penstock forces are:

$$F_{pen\perp}z = F_{pen\perp} \cdot \cos(\beta) \;\; [\mathrm{N}]$$

$$F_{friction}z = F_{friction} \cdot \sin(\beta) \;\; [\mathrm{N}]$$

$$Total\ vertical\ force\ component\text{: } F_z = F_{pen\perp}z + F_{friction}z \;\; [\mathrm{N}]$$

The resultant is computed as followed:

$$R = \sqrt{F_x^{\ 2} + F_z^{\ 2}} \ \text{[N]}$$

Since we consider the friction force to be either able to push uphill or downhill, we must analyse both scenarios. It is therefore useful to prepare a drawing which corresponds to the calculated vectors.

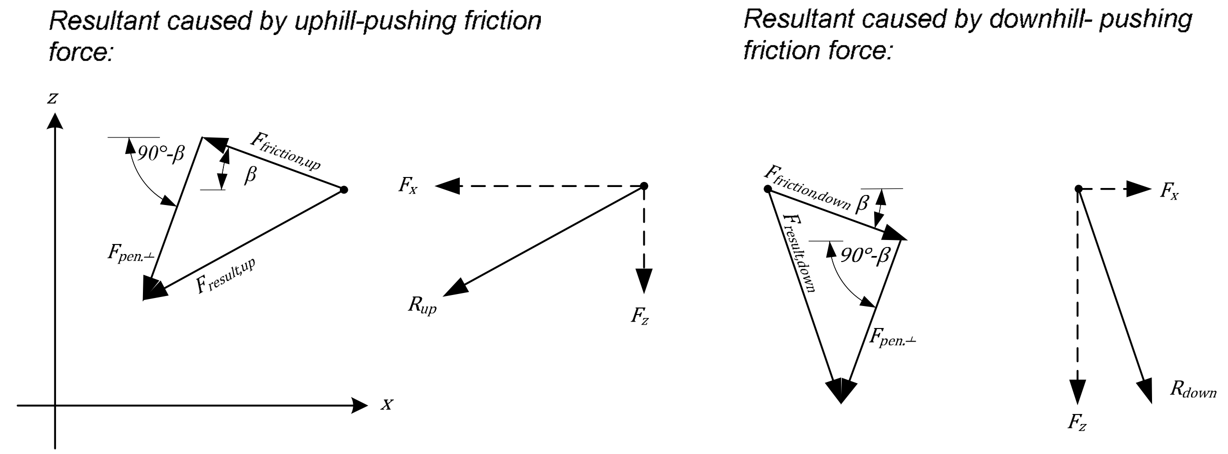


Figure 7.12 Force vectors of penstock load and corresponding resultants

Use a coordinate system which indicates positive x-values to the right and positive z-values pointing down.

Example: Calculate forces acting against a sliding block

Find the forces acting on the concrete sliding block displayed in Figure 7.13. The length of the concerned penstock stretch is $L = 6$ m, the steel penstock itself has an internal diameter of $d_{in} = 0.2$ m with a wall thickness of $t = 4.5$ mm. Consider a penstock slope of $\beta = 32°$. The friction coefficient is $\mu_k = 0.5$.

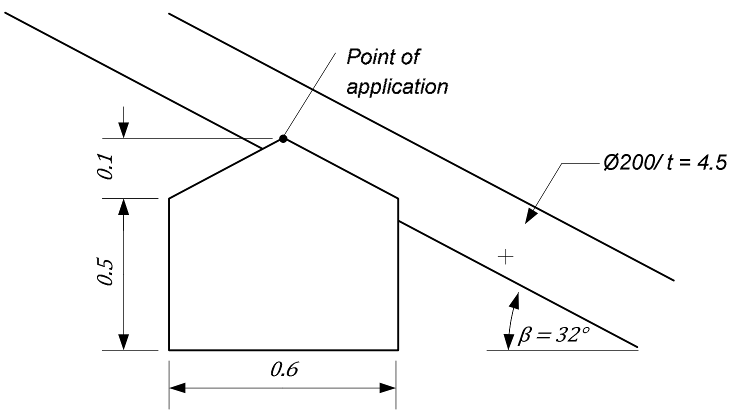


Figure 7.13 Example of sliding block (dimensions: 0.6/0.6/0.5)

First calculate the force resultant R_{up} caused by the uphill-pushing friction force $F_{friction}$:

$$q_{pen} = d_{ex} \cdot \pi \cdot t \cdot \rho_{Pen} \cdot g = 0.209 \cdot \pi \cdot 0.0045 \cdot 7850 \cdot (-9.81) = -0.228 \ \left[\frac{\text{kN}}{\text{m}}\right]$$

$$q_w = d_{in}^{\ 2} \cdot \frac{\pi}{4} \cdot \rho_w \cdot g = 0.200^2 \cdot \frac{\pi}{4} \cdot 1000 \cdot (-9.81) = -0.308 \ \left[\frac{\text{kN}}{\text{m}}\right]$$

$$F_{pen\perp} = l \cdot \left(q_w + q_{pen}\right) \cdot \cos\beta = \ 6.0 \cdot (0.308 + 0.228\,) \cdot \cos 32° = -2.727 \ \text{[kN]}$$

$$F_{friction} = \mu \cdot F_{pen\perp} = 0.5 \cdot 2.727 = \pm 1.364 \ \ \text{[kN]}$$

$$W_b = V_b \cdot \rho_b \cdot g = 0.6 \cdot \frac{(0.6 + 0.5)}{2} \cdot 0.5 \cdot 2400 \cdot 9.81 = 3.885 \text{ [kN]}$$

The horizontal force components are:

$$F_{pen\perp}x = F_{pen\perp} \cdot \sin(\beta) = 2.727 \cdot \sin(32) = -1.445 \text{ [kN]}$$

$$F_{friction}x = F_{friction} \cdot \cos(\beta) = 1.364 \cdot \cos(32) = -1.157 \text{ [kN]}$$

(Since the horizontal components point to the left, indicate the results negatively.)

$$Total\ horizontal\ force\ component: F_x = -1.445 - 1.157 = -2.602 \text{ [kN]}$$

The vertical force components are:

$$F_{pen\perp}z = F_{pen\perp} \cdot \cos(\beta) = 2.727 \cdot \cos(32) = -2.313 \text{ [kN]}$$

$$F_{friction}z = F_{friction} \cdot \sin(\beta) = 1.364 \cdot \sin(32) = 0.723 \text{ [kN]}$$

$$W_b z = W_b = -3.885$$

$$Total\ vertical\ force\ component: F_z = -F_{pen\perp}z + F_{friction}z - W_b z = -5.475 \text{ [kN]}$$

(The vertical components are also indicated negatively, because they are pointing down)

Now calculate the forces to achieve a resultant by adding the vectors. You can do this either graphically or arithmetically.

$$R = \sqrt{{F_x}^2 + {F_z}^2} = \sqrt{(-2.602)^2 + (-5.475)^2} = 6.062 \text{ [kN]}$$

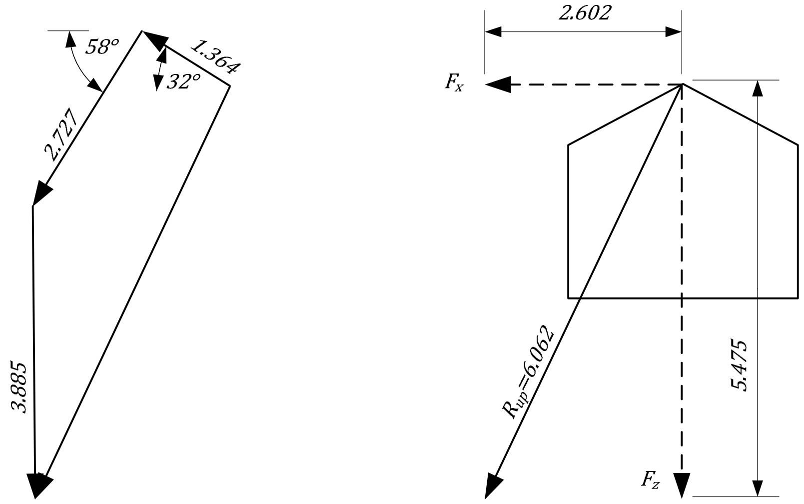


Figure 7.14 Resultant of an uphill-pointing friction force

Repeat the same procedure for the other case where the friction force pushes downhill.

To guarantee that the sliding block is strong enough to withstand the forces, ensure that the conditions of stability as in chapter 7.10 are fulfilled.

7.9 Forces acting on anchor blocks

Compared to a sliding block, an anchor block is designed to hold the penstock, not allowing any movement. As a result, all axial penstock forces between two expansion joints come to rest at this point. Since each action has a reaction, anchor blocks are sometimes quite massive to compensate for the attacking forces of the corresponding forces. Anchor blocks are exposed to following forces:

1. Forces caused by the weight of pipe and water
2. Forces caused by the pipe weight acting down the length of pipe (axial forces)
3. Bend forces caused by hydrostatic pressure
4. Bend forces caused by a change in of linear momentum
5. The weight forces of the anchor block

All forces must be arranged so that conditions of stability (chapter 7.10) are fulfilled.

1. Forces caused by the weight of the pipe and water are calculated as in chapter 7.8. In some cases, the anchor block is placed at a vertical or horizontal bend. In these situations, an uphill and a downhill portion must be expected.

$$Uphill\ portion: F_{pen\perp}up = L_{up} \cdot \left(q_w + q_{pen} + q_x\right) \cdot co\ s\ \beta_{up}\ \ [\mathrm{N}]$$

$$Downhill\ portion: F_{pen\perp}down = L_{down} \cdot \left(q_w + q_{pen} + q_x\right) cos\ \ \beta_{down}\ \ [\mathrm{N}]$$

In case the pipe is straight, use:

$$F_{pen\perp} = L \cdot \left(q_w + q_{pen} + q_x\right) \cdot co\ s\ \beta \quad [\mathrm{N}]$$

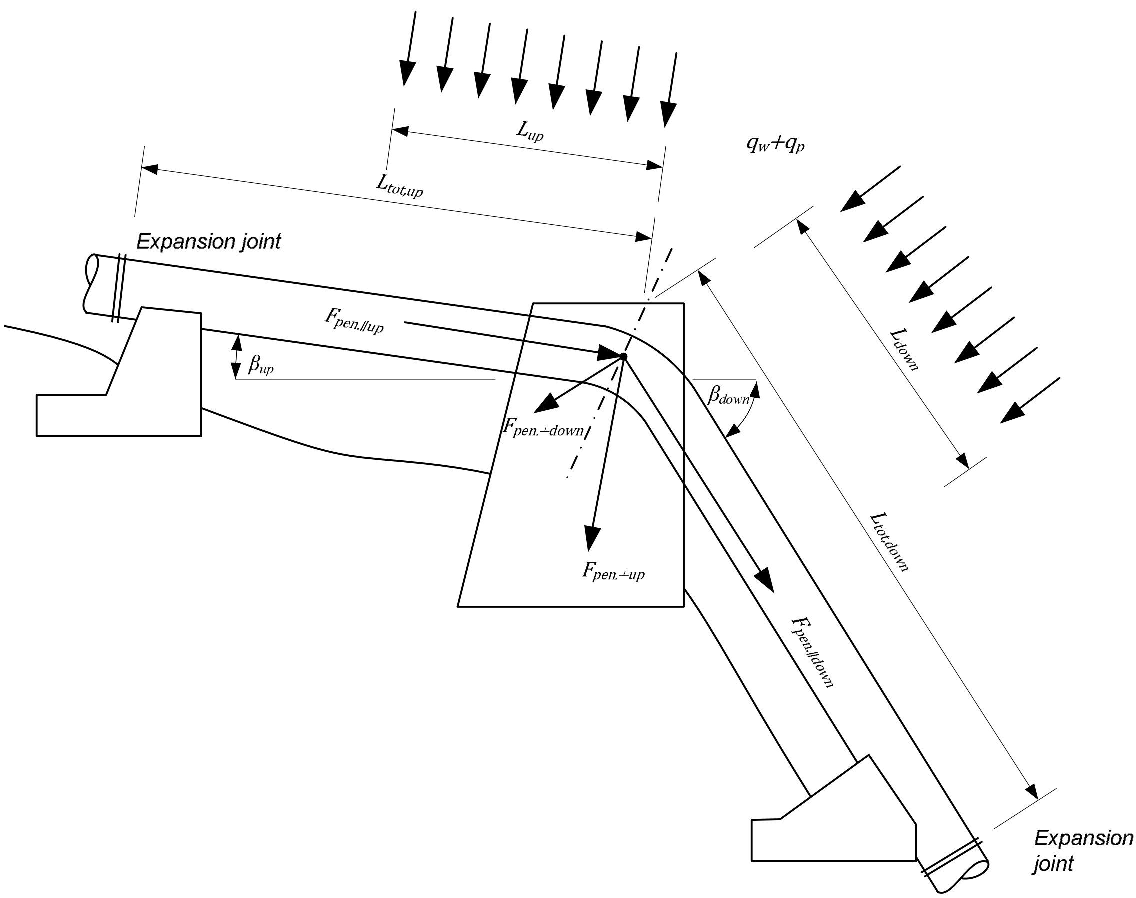


Figure 7.15 Anchor block exposed to perpendicular and parallel weight forces caused by the penstock

2. The component of pipe weight acting down the length (axial) of the pipe is calculated as following:

$$F_{pen\parallel}up = L_{up} \cdot (q_{pen} + q_x) \cdot sin\ \beta_{up}\ [\mathrm{N}]$$

$$F_{pen\parallel}down = L_{down} \cdot (q_{pen} + q_x) \cdot sin\ \beta_{down}\ [\mathrm{N}]$$

Consider that both uphill and a downhill component act on the block.

3. Bend forces caused by hydrostatic pressure:

$$F_{p,up} = p_{up} \cdot \pi \cdot \left(\frac{d_{up}}{2}\right)^2 \quad [\mathrm{N}]$$

$$F_{p,down} = p_{down} \cdot \pi \cdot \left(\frac{d_{down}}{2}\right)^2 \quad [\mathrm{N}]$$

$$F_p = \sqrt{{F_{p,up}}^2 + {F_{p,down}}^2 - F_{p,up} \cdot F_{p,down} \cdot \cos\ \alpha}\ [\mathrm{N}]$$

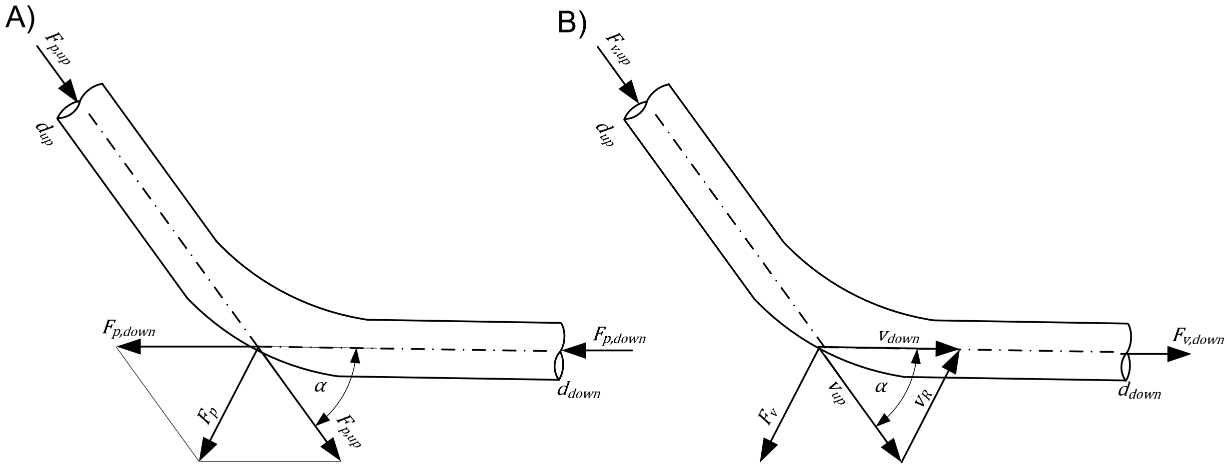


Figure 7.16 Bend forces: A) caused by hydrostatic pressure, B) caused by change of linear momentum

4. Bend forces caused by a change in linear momentum:

$$F_v = \rho_w \cdot Q \cdot \sqrt{{v_{up}}^2 + {v_{down}}^2 - 2 \cdot v_{up} \cdot v_{down} \cdot \cos\ \alpha}$$

in which v_{up} and v_{down} is the velocity of flowing water in the uphill and downhill portion of the penstock.

5. Weight forces of the anchor block:

$$W_b = V_b \cdot \rho_b \cdot g$$

Note that the weight does not necessarily act in the middle.

7.10 Conditions of stability

According to Adam Harveys *Micro Hydro Design Manual*, a sliding or anchor block is shaped and sized in such a way that it cannot move, regardless of the force imposed by the penstock. It must not sink in, slide along, or turn in the ground. This would allow the penstock to sag above it. Hence, the following conditions must be met:

Soil sinkage

The pressure the base of the block exerts on the soil is called the *base pressure* P_{base}. The base area A_{base} must be large enough, that it is less than the bearing capacity of the soil P_{soil}. The condition for stability is $P_{soil} > P_{base}$:

$$P_{soil} = \frac{F_z}{A_{base}}\left(1 + \frac{6 \cdot e}{L_{base}}\right) < P_{soil}$$

The values for P_{soil} are given in Table 7.2. Fz is the sum of all vertical forces acting on the bearing. The eccentricity e can be obtained by the following formula:

$$e = \frac{L_{base}}{2} - \frac{M}{Fv}$$

in which M is the momentum acting around the base's corner (Figure 7.17) and F_V is the sum of vertical forces.

$$M = F_x \cdot h_b + F_z \cdot \frac{L_b}{2} + W_b \cdot \frac{L_b}{2}$$

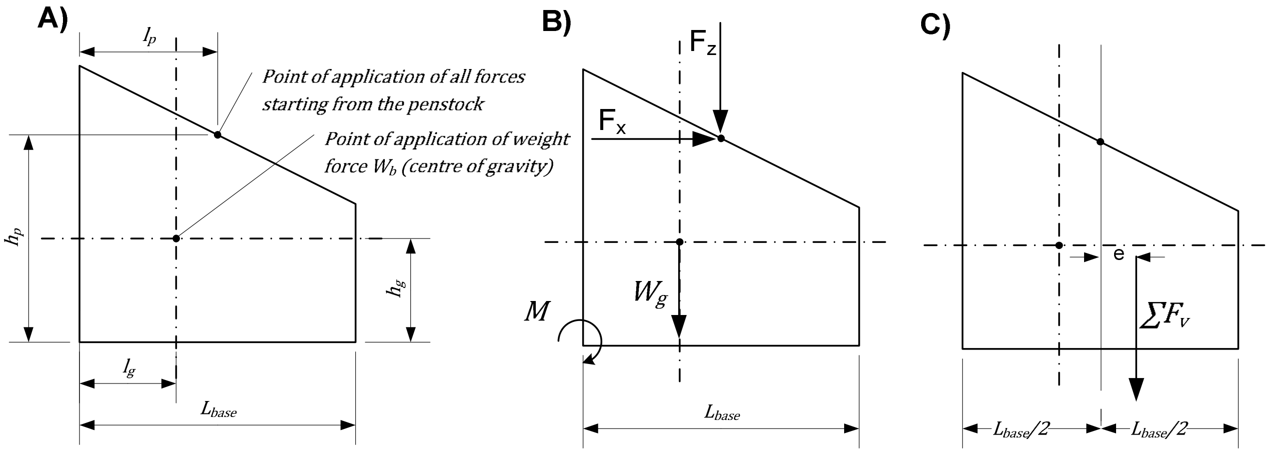


Figure 7.17 A) define the points of application. B) Obtain momentum around base corner. C) Calculate eccentricity.

Sliding

If the sum of the horizontal forces acting on the block is greater than the frictional resistance of the soil, the block will slide. Therefore, stability is given if:

$$\sum F_H = \mu \cdot \sum F_V$$

Where μ is the coefficient of friction between soil and the block's surface, normally taken $\mu = 0.5$. Design of the block base with serrations or with a step in it will also avoid sliding.

Toppling

All the vertical forces on the block can be resolved into one single force ΣF_V as shown in Figure 7.17. If this is too far toward the edge of the block it will tend to topple the block or turn it in the ground causing it to sink slightly (a slight sinkage can cause expensive damage to the penstock). To avoid this, you can design the block so that this force acts within the middle third of the length of the base L_{base}. The rule of thumb is:

$$e < \frac{L_{base}}{6}$$

Soil type	Maximum bearing pressure P_{soil} [N/m^2]		
Clay	180'000	to	220'000
Sand	200'000	to	320'000
Sand/gravel	300'000	to	400'000
Sand/gravel/clay	350'000	to	650'000
Rock	600'000	to	1'000'000

Table 7.2 Bearing capacities of soils

8 Bibliography

General references:

- Vischer Daniel, Huber Andreas (Springer, 2002, 6. Auflage)***Wasserbau***, ***Hydrologische Grundlagen, Elemente des Wasserbaus, Nutz und Schutzbauten an Binnengewässern.***
- Giesecke Jürgen, Emil Mosonyi (Springer, 2005, 4. Auflage) ***Waserkraftanlagen: Planung, Bau und Betrieb***
- Bretschneider H, Lecher K, Schmidt M (1982, 6.Auflage) ***Taschenbuch der Wasserwirtshaft***.
- Strobl Theodor, Zunic Franz (2006) ***Wasserbau, Aktuelle Grundlagen-Neue Entwicklungen.***
- Ausschuss für Internationale Zusammenarbeit im Kuratorium für Kulturbauwesen (1971) ***Fachwörterbuch für Bewässerung und Entwässerung***
- Harvey Adam (ITDG,1993) ***Micro-Hydro Design Manual, A guide to small-scale waterpower schemes***
- ***Henderson F. M. (Prentice Hall 1966) Open channel flow***
- Ven te chow (Mc Graw –Hill, 1959) ***Open channel hydraulics***
- Naudascher Eduard (Springer, 1987) ***Hydraulik der Gerinne und Gerinnebauwerke***
- Scheuerlein Helmut (Ernst & Sohn, 1984) ***Die Wasserentnahme aus geschiebeführenden Flüssen***
- Bollrich Gerhard (Huss, 2007, 6. Auflage) ***Technische Hydromechanik Band 1***
- Wikipedia.org
- FigureFigure 4.2,Figure 4.12,Figure 4.13,Figure 5.1,Figure 5.11,Figure 5.12,Figure 5.13 are adapted from: Vischer Daniel, Huber Andreas (Springer, 2002, 6. Auflage)***Wasserbau***, ***Hydrologische Grundlagen, Elemente des Wasserbaus, Nutz und Schutzbauten an Binnengewässern.***