COURSE HANDBOOK

RURAL HYDROPOWER CIVIL ENGINEERING

Decentralised schemes up to 500 kW

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Rural Hydropower Civil Engineering

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CONTENTS

1. COMPONENTS AND STRUCTURES

What this module is about

There are many components and structures that are required for the successful development of a small hydropower (SHP) scheme. For civil engineers to be able to plan and design structures, they need to understand the importance of these structures and how they relate with other components in the system. This module describes the components and structures that make up a small hydropower scheme.

Learning outcomes

At the end of this module, the participant is able to

- Describe the functions of SHP system components
- Identify and understand the importance of SHP system components
- Locate components in a new scheme

INTRODUCTION

Climate change and its drastic effects have caused members of the world community to turn their attention towards alternative energy sources. Fossil fuels (petroleum, coal) are the primary sources of energy amongst the developed nations. With developing nations, this energy is mostly composed of biomass. Over half of the world's electricity is produced by burning coal or petroleum fuels – a process, which adds more greenhouse gases, pollutes the atmosphere and contributes to adverse changes of climate conditions. On the other hand, these resources are being rapidly depleted. Reserves available for human use will run out in a few more decades at best. Renewable energy options such as small hydropower schemes can help address these issues.

Small-scale (pico, micro and mini) hydropower is one of the most environmentally harmless forms of energy generation available to us today. Small-scale hydro plants may be connected to conventional electrical grids as a source of low-cost renewable energy. Alternatively, they may be built in isolated areas where grid-based electricity is uneconomical or where there is no national grid or distribution network for electrical power. Since small hydro projects usually involve minimal reservoirs and civil construction work, they are seen as having a relatively low environmental impact compared to large hydro. This decreased environmental impact depends strongly on the balance between stream flow and power production. A well-designed small-scale hydropower system can blend with its surroundings with near-to-nil negative environmental impacts.

SUBCLASSIFICATION OF SMALL HYDRO

Small hydro can be further subdivided into mini hydro, usually defined as less than 1,000 kilowatts, and micro hydro, which is less than 200 kilowatts. Micro hydro is usually the application of hydroelectric power sized for smaller communities, single families or small enterprise.

Micro hydro plants may use custom-built turbines or mass-produced centrifugal pumps, connected in reverse to function as turbines. While these machines rarely have optimum hydraulic characteristics when operated as turbines, their low purchase cost makes them attractive for micro hydro class installations.

HOW DOES IT WORK?

Hydropower systems use the energy in flowing water to produce electricity or mechanical energy. The water flows through a channel **(penstock)** to a turbine where it strikes the bucket of the wheel, causing the shaft of the turbine to rotate. The rotating shaft is connected to an alternator or generator, which converts the motion of the shaft into electrical energy. This electrical energy may be used directly, stored in batteries, or inverted to produce utility-grade electricity. A small-scale hydroelectric facility requires a sizable water **flow** at a proper gradient, which is called **head**. These conditions are obtained without building elaborate and expensive facilities. Small hydroelectric plants can be developed at existing dams and have been constructed in connection with water-level control and irrigation schemes for rivers and lakes. By using existing structures, only minor new civil engineering works are required, which reduces the engineering costs of a scheme.

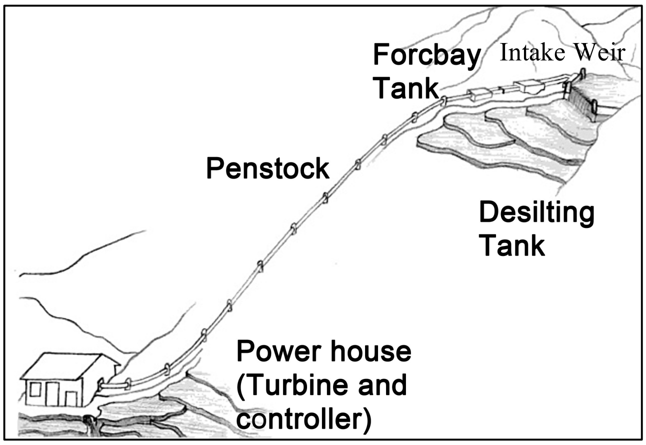


Figure 1.1 General layout of a hydro scheme

To generate energy from water, two main factors are required:

Gross head: The vertical distance between the tapping and the delivery of water in a power scheme is called gross head. This height difference is usually measured in metres. Gross head provides the potential energy which, when converted into kinetic energy, drives the turbine.

Flow rate: The quantity (volume) of water, which passes through the turbine per unit time is called the flow rate. It is measured in cubic meters per second.

Table 1.1: SWOT analysis for hydropower

Strengths	*Weaknesses*	*Opportunities*	*Threats*
• Cheapest renewable energy option • Water remains available for other purposes after use • Continuous availability of power on demand • Concentrated energy source given a reasonable head • Predictable quantity of available energy • Requires no fuel and only limited maintenance for low running costs (compared with diesel power) • Long-lasting and robust, systems can last 50+ years without major new investments • Simple enough to be transferred to and used in remote rural areas	• Mostly site-specific • Reliable site data must be collected to guarantee output • Lengthy planning and permit procedures • Seasonal variations affect performance • Dams and rivers collect water for electricity production, which alters the natural water flow system and thus deprives other needs. • Potential for serious disputes caused by changes to the river pathway and water shortages	• Large demand and supply gap • Major opportunities in hydroelectric consultancy in Nigeria and abroad • New sources of power generation • Opportunity to go global through tie-ups	• Rising cost of production • Competition from other new and environmentally friendly sources of power • Local opposition to environmental impacts and displacement of people

1.1. SYSTEM COMPONENTS

Various components help complete the transformation of water into watts. These components, in turn, depend on several techno-economic considerations that vary from case to case. It is primarily the type and layout of the components, which makes each project unique. Three basic elements are necessary in order to generate power from water: a means to create head, a conduit to convey water and a power plant. To provide these functional elements, the following civil components are employed in hydropower schemes.

GENERAL LAYOUT

A typical small hydropower plant will have the following components: reservoir, diversion structure, spillways, desilting arrangements, headrace channel, forebay (for micro hydro) or surge tanks, penstock, powerhouse consisting of electro-mechanical equipment and controllers, tailrace to channel water back to the riverbed, and transmission network to evacuate the power generated.

In smaller schemes, some of these components could be avoided by designing a single structure that incorporates more than one function. For example, desilting and spillway components could be integrated into one structure.

RESERVOIR

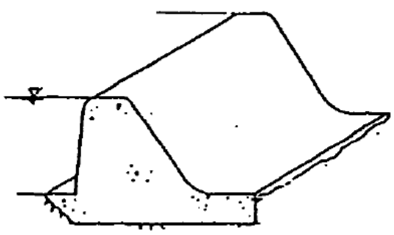
A reservoir is created by building a dam – a barrier that stops and stores water. The reservoir not only suppresses floods but also provides water for various needs, which include irrigation, human consumption, industrial use, aquaculture and navigation. Hydropower is often used in conjunction with dams to generate electricity.

In micro hydro schemes, the term **weir** is used instead of dam; because there is usually no storage. A weir is a low diversion structure built on the stream bed to divert the required flow while allowing the rest of the water to continue on its natural path. There are a number of basic types of dams/weirs:

- Concrete gravity dam
- Earth dam
- Rock-fill dam
- Gabion dam
- Concrete-reinforced gabion dam

Selecting a narrow location of the stream minimises construction costs. The dam must be easily accessible for maintenance work, even during the rainy seasons. The release of excess water, controlling of intake water and flushing out of the silt (desilting) behind the dam are important functions associated with the dam.

Concrete gravity dam: The entire body of the dam is built with concrete.

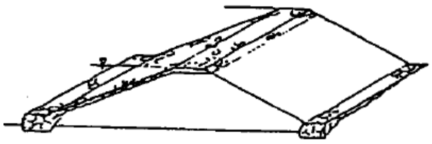


Foundation: Bedrock

River conditions: Not affected by the gradient, discharge or level of sediment load

Intake conditions: Good interception performance and intake efficiency

Earth dam: Earth is used as the main material for the body. Some form of rip-rap (rock or other material used to armour shorelines, streambeds, bridge abutments, pilings and other shoreline structures against scour and water erosion) covers this basic structure. In instances where seepage is anticipated, a core wall becomes necessary to mitigate this problem.

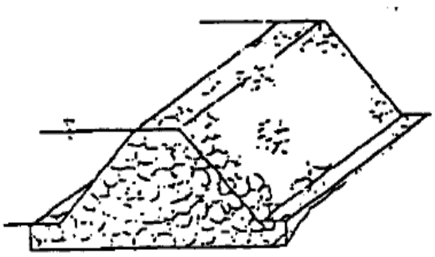


Foundation: Variable, from earth to bedrock

River conditions: Gentle flow, flooding can be easily handled

Intake conditions: Good intake efficiency, since excellent interception can be achieved with careful construction

Rock-fill dam: Gravel is used as the main material for the body. The introduction of a core wall may be necessary depending on the situation.

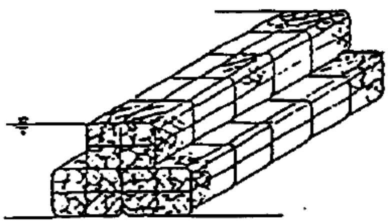


Foundations: Various, from earth to bedrock

River conditions: River where an earth dam could be washed away by normal discharge

Intake conditions: Limited to the partial use of river water due to low intake efficiency

Gabion dam: Gravel is wrapped by a metal net to improve the integrity.

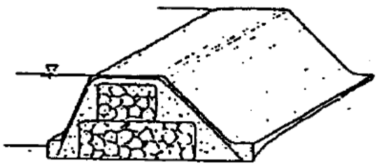


Foundations: Various, from earth to bedrock

River conditions: River where a rock-fill dam could be washed away by normal discharge

Intake conditions: Limited to the partial use of river water due to low intake efficiency

Concrete-reinforced gabion dam: The gabion dam surface is reinforced with concrete.



Foundations: Primarily gravel

River conditions: River where the metal mesh could be damaged due to strong flow

Intake conditions: Applicable when high intake efficiency is required

INTAKE STRUCTURES

An intake structure is an arrangement, which diverts water for the required use. The intake allows water to be taken from its source and then discharged into the conveyance system that routes it to the intended application (e.g. hydroelectric power generation). Arrangements for routing high flood discharge are also made. Extreme care must be taken in the selection of a location for an intake when developing small-scale hydropower schemes, as the cost of the intake facility is a major determinant for the cost-effectiveness of a project.

Intake designs depend on the following parameters:

- Placement on the stream (along a straight stretch or on a bend)
- Terrain properties (gradient of the channel or river basin, valley width, etc.)

- Flow quantity and diversion angle
- Maximum and minimum flows in relationship to planned quantity
- Materials carried (amount, size and frequency of rock loads transported by water, floating debris, silt and material in suspension)

Integral parts of the intake structure:

- Diversion weir
- Intake
- Antechamber (sedimentation zone)
- Trashrack
- Sluice gate
- Channel
- Spillways
- Sand trap
- Riverbed

There are two intake types: Side intakes and drop or bottom intakes.

» *Side intake*

Figure 1.2: Side intake

The side intake is the simplest and least expensive type of intake. It is also easy to build and maintain. This is the intake type, which has been used intuitively for many centuries by farmers all over the world. When designing an intake structure for a small-scale hydropower plant, the omission of an intake gate should be considered in order to reduce costs. For a small-scale hydropower plant, a side intake could be used as headrace. In this case, it is essential to avoid an inflow of excess water into the side intake. Water flows, which considerably exceed the design discharge, will ultimately destroy the headrace.

Note the following important points for the design:

- Always equip the intake with a closed tap instead of an open tap so that it becomes a pressure intake when the river level rises.
- Place the intake at a right angle to the direction of flow wherever possible to minimise the head of the approaching velocity at the time of flooding.
- As water inflow during floods exceeds the design discharge, ensure a fairly large spillway capacity at the settling basin or starting point of the headrace.
- Equip side intake models with a flushing gate.

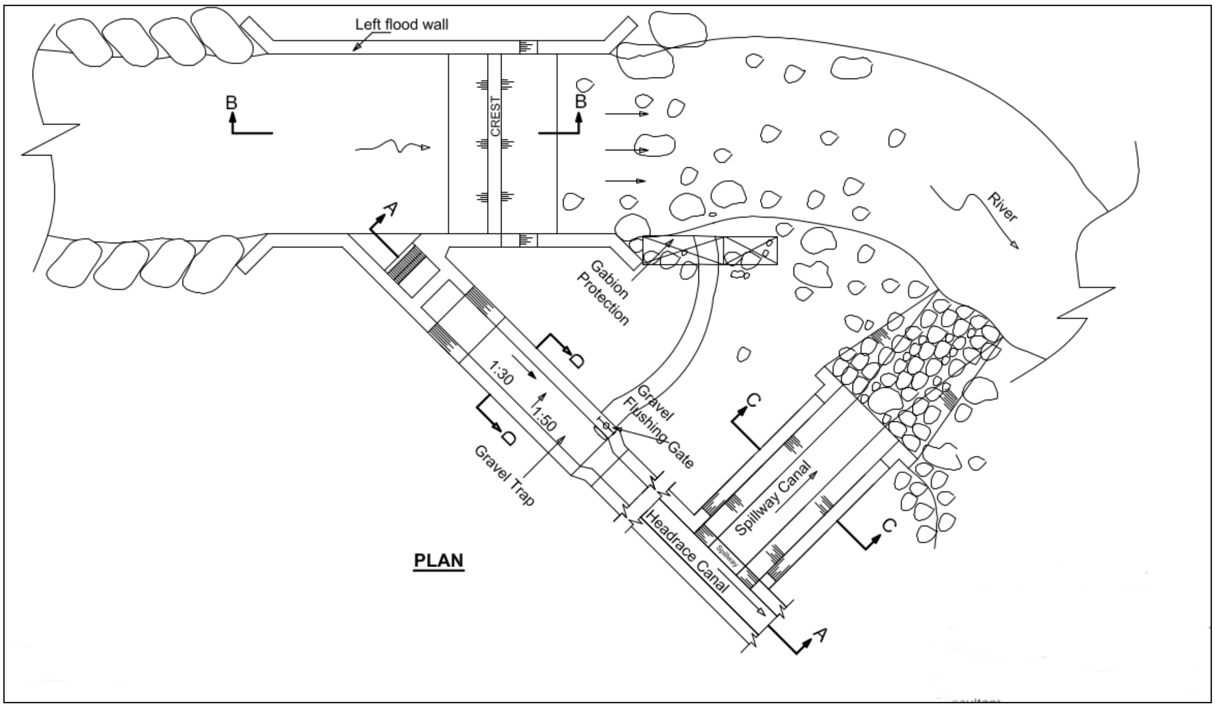


Figure 1.3: Layout of a typical side intake

» *Drop or bottom intake*

There are several types of simple intake designs, which aim at reducing the weir height and omitting the flushing gate (also known as the Tyrolean intake) for a hydropower plant. With a bottom intake, the weir is completely submerged. Excess water will pass the intake by flowing over the weir. The water to be diverted is taken in through a collection channel built into the river bottom and covered with a screen. The screen bars are laid in the direction of the current and inclined in the direction of the tailwater so that coarse bed load is kept out of the collection channel and transported further downstream. Particles, which are smaller than the gaps between the screen bars are introduced into the collection channel together with the water. Suitable flushing devices must later be used to separate these particles from the water for power generation. The bottom intake can be constructed at the same level as the riverbed or in the form of a sill.

Figure 1.4: Drop intake

Advantages	*Limitations*
• Very useful with fluctuating flows, even the smallest flows can be diverted • No maintenance required (if well designed)	• Expensive • Local materials cannot be used • Good design required to prevent sediment blockage

SPILLWAYS

» Spillway design

Figure 1.5: Srisailam Dam in India

One parameter of spillway design is the largest flood it is designed to handle. The structures must safely withstand the appropriate spillway design flood (SDF). A 100-year recurrence interval refers to the worst flood magnitude in 100 years. It may also be expressed as an exceedance frequency with a 1% chance of being exceeded in any given year. The volume of water expected during the design (or worst hypothetical) flood is obtained through hydrologic calculations of the upstream watershed. The return period is set by dam safety guidelines, based on the size of the structure and the potential loss of human life or property downstream.

Having established the design flood, the structure must be designed to safely handle and divert the flood. There are many spillway options, which need to be considered carefully. One simple and frequently used option an ogee weir (discussed in detail in module 3). The main advantage is simplicity, but to minimise the water level increase during a flood, the weir needs to be rather long, a requirement that results in an expensive construction. In most schemes, when a flood rises above an ogee weir, several metres of head are being wasted. To handle the same flood without such a large water level increase, a *piano key* weir or *hydroplus fusegate* should be seriously considered as options.

After the intake, a design feature must also be included to safely handle and divert excess water that may enter into the channels during a flood. Spillways are designed to permit a controlled overflow at certain points along the channel. Flood-level flows through the intake can be twice the normal channel flow, so the spillway must be large enough to divert this excess flow.

The spillway is a flow regulator for the channel. In addition, it can be combined with control gates to provide a means of emptying the channel. The spill flow must be fed back to the river in a controlled way so that it does not damage the foundations of the channel.

» Desilting / Settling basin

A settling basin is one of the most effective devices for removing sediment particles from flowing water. The reduced flow velocity in the settling basin is caused by an expansion of the channel cross-section over the length of the basin. This reduction in velocity also reduces turbulence. A reduction in velocity, and in turbulence, if adequate, stops the bed material from moving and also causes part of the suspended material to deposit. Once the minimum sediment size for removal has been determined, the next step is the design of the settling basin. For the settling basin, engineers must define the depth and length of the basin and decide on a removal method for the deposited material. To provide a safety margin, it is advised to construct the length of the settling basin (ls) as double the calculated number.

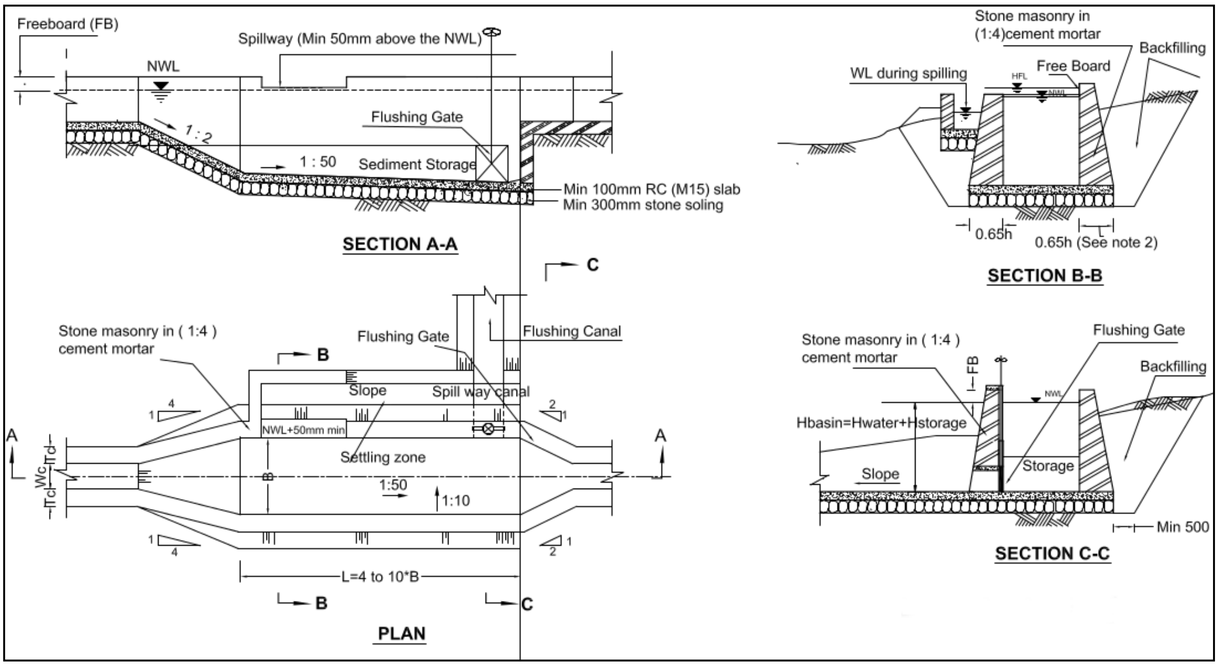


Figure 1.6: Desilting or settling basin

HEADRACE

The headrace for a micro hydropower plant usually adopts an exposed structure such as an open channel or a covered channel. This structure runs from the desilting tank to the forebay or surge tank and carries silt-free water. The channels may be of the following types:

- Open channel (reinforced concrete cement (RCC), or cut-and-cover and steel pipes are used along stretches where the slopes are geologically unstable)
- Polyvinyl chloride (PVC), high-density polyethylene (HDPE) or fibreglass
- Steel pipes
- Tunnels

For an open channel, the headrace cross-section could be trapezoidal with a thin concrete lining, or rectangular with masonry or concrete walls.

For access, provision needs to be made only for a footpath to access the channel, a provision which reduces costs as well as help avoid unnecessary hill slope cutting. Data on water loss of water due to seepage may be obtained from representative schemes so that its significance can be assessed.

Figure 1.7: Examples of masonry: Lined (left), earthen (middle) and wooden (right) channels

Figure 1.8: Earthen headrace

» *Determining cross-section and slope*

The dimensions of cross-section and slope should be determined in such a manner that the required turbine discharge can be economically guided to the head tank. Generally, the size of the cross-section is closely related to the slope. The headrace slope should be made gentler to reduce head loss (difference between water level at the intake and the forebay/surge tank); this causes a lower velocity, however, and thus a larger cross-section. Conversely, a steeper slope will create a higher velocity and smaller cross-section, but also a larger head loss. Generally, in the case of small hydro scheme, the slope of headrace will be determined as 1/500 – 1/1,500. Typical cross-sections used in small hydro are rectangular and trapezoidal.

For the same cross-sectional area A, and channel slope S, the channel with a larger hydraulic radius R delivers a larger discharge. That means for a given cross-sectional area, the section with the least wetted perimeter is the most efficient hydraulically. Semi-circular sections are consequently the most efficient. A semi-circular section, however, unless built with prefabricated materials, is expensive to build and difficult to maintain. The most efficient trapezoidal section is the half hexagon, whose vertical-to-horizontal side slope ratio is 1 to 0.577. Strictly speaking, this is only true if the water level reaches the level of the top of the bank. Actual dimensions have to include a certain freeboard (vertical distance between the designed water surface and the top of the channel bank) to prevent water level fluctuations from overspilling the banks. The minimum freeboard for lined canals is about 10 centimetres, and for unlined canals it should be about one third of the designed water depth or at least 15 centimetres. One way to prevent an overflow of the canal is to provide spillways at appropriate intervals; any excess water is then conveyed, via the spillway, to an existing streambed or gully.

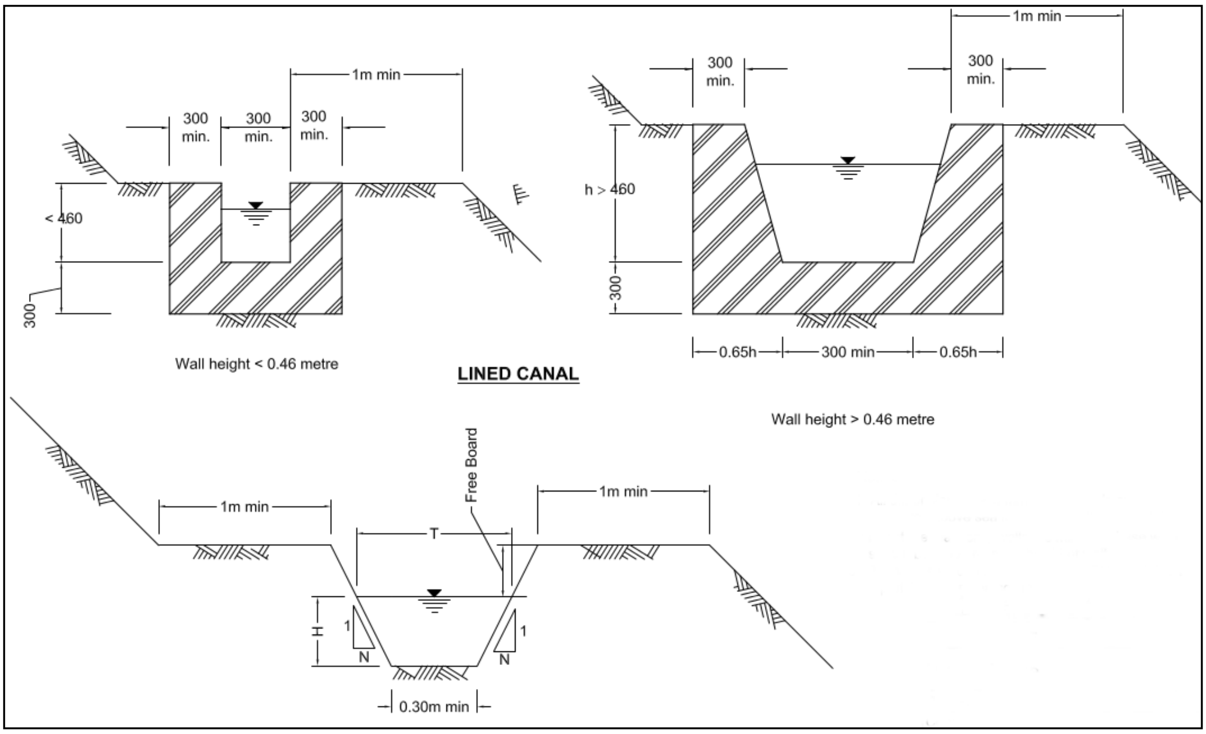


Figure 1.9: Types of channel cross-sections

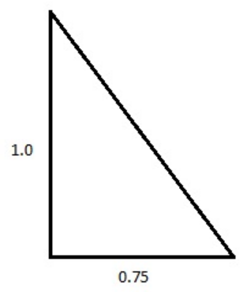


Figure 1.10: Slope for clayish soils

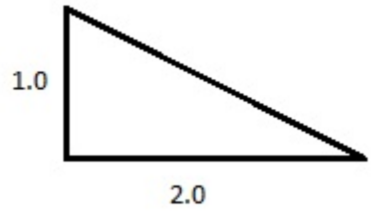


Figure 1.11: Slope for sandy soils

It should be noted that the best hydraulic section does not necessarily come with the lowest excavation cost. If the canal is unlined, the maximum side slope is defined by the slope at which the material will withstand the erosive effects of water (i.e. remain under water permanently without disintegrating). Clay slopes may stand at a slope consisting of 1 vertical unit to 3/4 horizontal units, whereas sandy soils must have flatter slopes (1 vertical, 2 horizontal).

» *Circumventing obstacles*

When siting channels, obstacles may be encountered such as ravines or streams, which need to be circumvented. The possibilities for circumventing these obstacles include going over, around or under them.

A flume is required to cross a stream. This prolongs the canal while retaining the same slope. Support for the flume can be provided by concrete or steel piles or it can span a given distance as a bridge. Steel pipes are often the best solution, since they may be used as the chord of a truss, which can be fabricated on site. Flumes do involve one potential problem: they make it hard to remove sediment deposited in the canal when filled with standing water.

» *Controlling leakage*

Water leakage along the canal should be minimised. Otherwise, large-scale damage can occur to the entire civil works due to the erosion of the canal banks and bottom. Note that water leakage *into* the canal also presents a potential challenge. Wherever sections come into contact with small streams or storm gullies, steps must be taken to divert that water away from the canal banks in a safe manner.

INTAKE SCREEN

To ensure that debris cannot get into the penstock and block the water passages through the turbine, the intake must be equipped with a screen. The spacer bars on the screen bars must be less than the minimum gap between the blades of a Francis turbine and less than the nozzle opening on a Pelton turbine when the needle is about 50% open.

Prior to designing the screen, consult the turbine manufacturer for information on the minimum gap in the runner or nozzle. A smaller gap might be necessary in certain cases to keep out (small) fish.

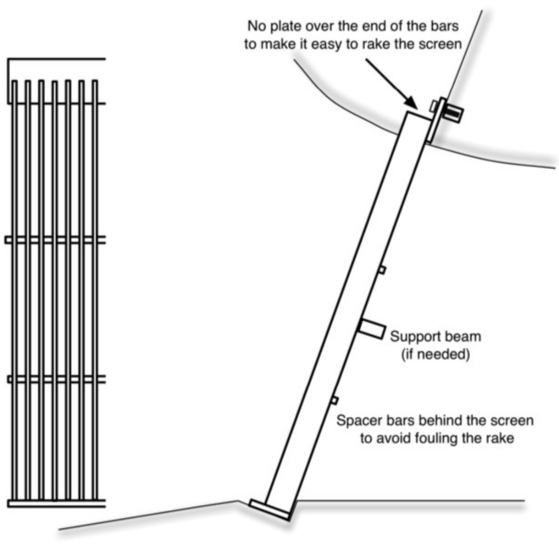


Figure 1.12: Intake screen

The screen itself must be carefully designed. It must have transverse bars to maintain the vertical bar spacing and to reduce the risk of the bars vibrating. If the screen is submerged by less than 3 metres, it should be designed for total blockage. If the head on the screen is submerged more than about 5 metres, designing for total blockage can be quite expensive, and one alternative is to monitor the head differential across the screen and close the intake gate if it gets too high.

FOREBAY OR POWER INTAKE

In small hydropower schemes, even in those with sizeable heads, the water intakes are horizontal, followed by a curve that leads to an inclined or vertical penstock. The design depends on whether the horizontal intake is part of a high-head or a low-head scheme. In low-head schemes, a good hydraulic design, in other words, a high-efficiency design which is often costlier makes sense because the head loss through the intake is relatively large compared to the gross head. In high-head schemes, the value of the energy lost in the intake will be small relative to the total head and the cost of increasing the intake size to provide a lower intake velocity and a better profile may not be justified.

In a power intake or forebay, several components need consideration:

- Approach walls to the trashrack should minimise flow separation and head losses
- Transition from rectangular to a circular cross-section to meet the entrance to the penstock
- Piers to support service gates and other mechanical equipment including trashracks
- Guide vanes for an even distribution of flow

A well-designed intake should not only minimise head losses but also preclude vorticity (the rotation of the fluid). Vorticity should be avoided because it interferes with the performance of turbines, especially for bulb and pit turbines. Vortices can ultimately:

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

The vapour phase forms when the hydrodynamic pressure in a flowing liquid drops below the liquid's vapour pressure. This phenomenon induces small bubbles, which are removed from the low-pressure region by the flow. The bubbles then collapse in regions of higher pressure. Their formation and subsequent collapse gives rise to what is called cavitation.

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

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

Lack of sufficient submergence and asymmetrical approach conditions seem to be the most common causes of vortex formation. An asymmetrical approach (Figure 1.13 left) is more prone to vortex formation than a symmetrical approach (Figure 1.13 right). Provided the inlet to the penstock is deep enough and the flow undisturbed, vortex formation is unlikely.

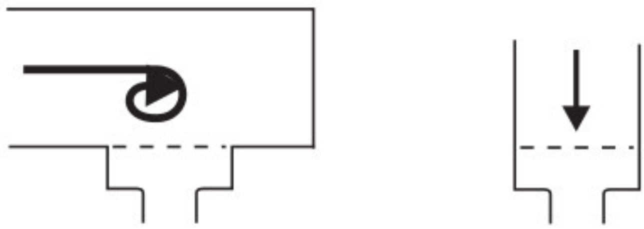


Figure 1.13: Penstock entries

» *Bypass gates*

In an emergency, it is always easier to open a valve quickly than to close a valve under high pressure. The opening of a bypass valve diverts the water away from the turbine without creating any water hammer effect on the penstock.

A bypass valve is also useful for the occasional flushing of penstock pipes to clear any unwanted objects or debris trapped in the pipeline.

PENSTOCK

The penstock is the pipe, which conveys water under pressure to the turbine. The major components of the penstock assembly are shown in Figure 1.14. The penstock is often the most expensive system component, hence engineers spend a lot of time to minimize its cost. The penstock must be strong enough to withstand the very high pressures, which result from a sudden obstacle to the flow of water, which create temporary pressure build up known as surge or water hammer pressures. In projecting the cost of the penstock, it is easy to underestimate the expense of peripheral items such as joints and coats of paint. The choice between one penstock pipe material and another can make a significant difference in overall cost if all these factors are included. For instance, plastic penstock piping may be cheap but the joints may be expensive or unreliable in some regions.

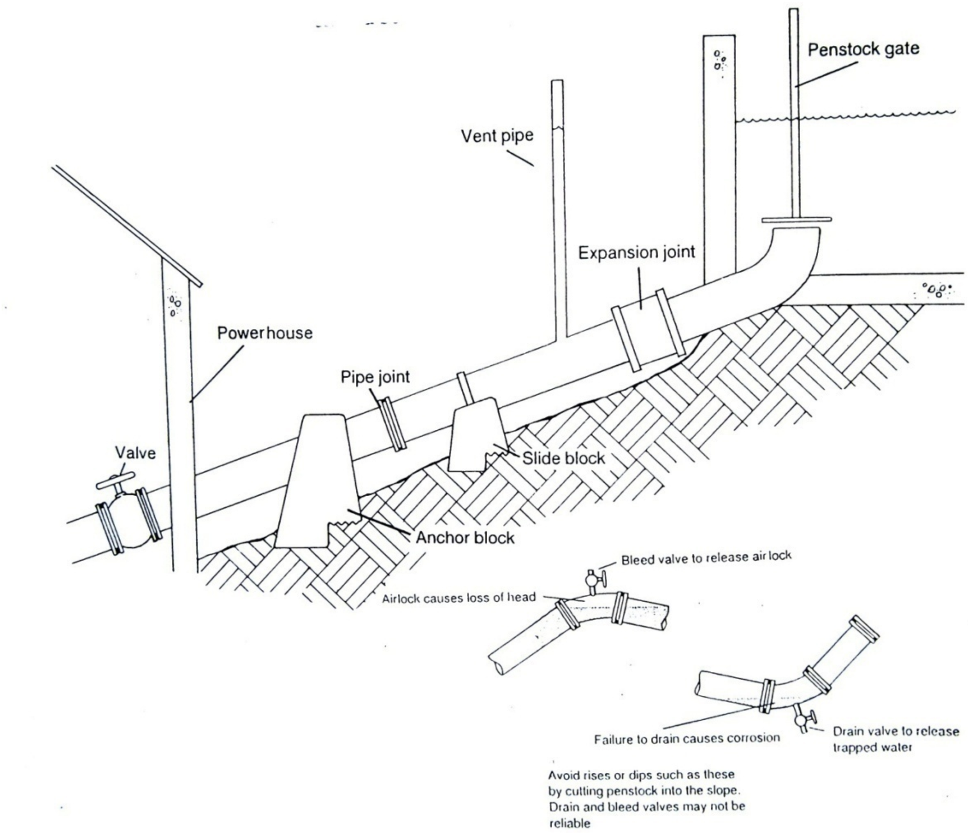


Figure 1.14: Components of a penstock assembly

» *Materials*

When deciding which material to use for a particular project, factors to be considered include design pressure, soil type, joining method, diameter and friction loss, weight and ease of installation, terrain, etc. Common materials used in penstock design for micro hydropower schemes include:

- Mild steel
- Unplasticised polyvinyl chloride (uPVC)
- High-density polyethylene (HDPE)
- Medium-density polyethylene (MDPE)
- Spun ductile iron
- Asbestos cement
- Pre-stressed concrete
- Wood stave
- Glass-reinforced plastic (GRP)

Of these materials, only three are commonly used: mild steel, PVC and HDPE. This handbook discusses only three of these materials in module 3.5.

» *Penstock support and anchorage system*

A penstock that is exposed or non-embedded requires an extensive support and anchorage system. This includes the following:

Saddles: Saddles are structures used to support the penstock. They take their name from their shape. Saddles provide continuous support and are designed to handle the full weight of the penstock with water.

Support blocks or ring girders: Support blocks or ring girders provide intermittent support to the penstock. Their spacing is decided by designing the penstock as a continuous beam supported at these points. Generally, the spacing between ring girders is kept between 10 to 50 metres.

Figure 1.15: Anchor block

Anchor blocks: Anchor blocks are required wherever there is a change in direction (vertical or horizontal) of the penstock to withstand the reaction forces caused by the change in flow direction. Anchor blocks are designed as fixed supports.

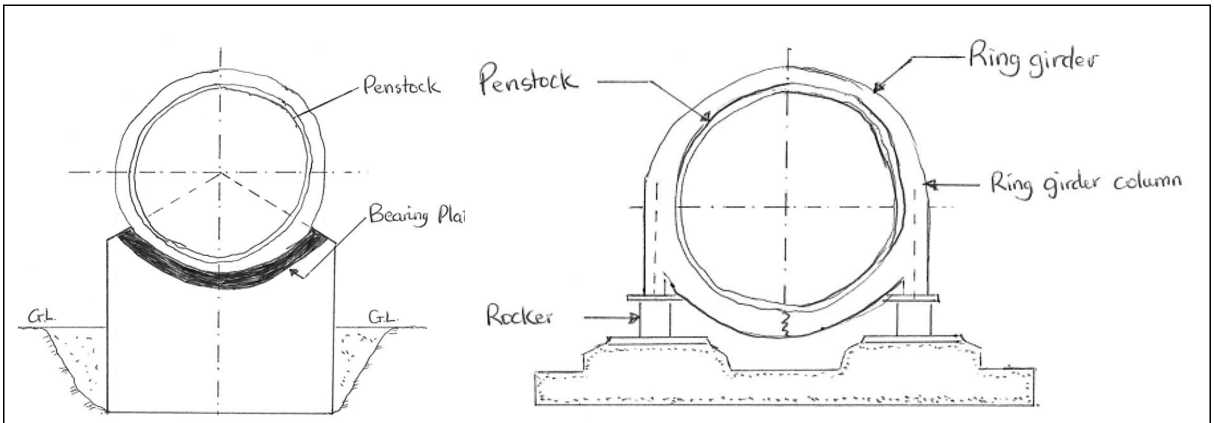


Figure 1.16: Penstock support types: Saddle (left) and ring grider support (right)

» *Penstock specials*

Penstock specials are associated components of the penstock. These are:

Bends (vertical, horizontal, or combined) are necessitated due to topographical constraints. However, bends should be smoother and the bend radius should be at least 5 times the penstock diameter.

Transitions (reducers or expanders) are necessary when the cross-sectional area of the penstock changes due to a merger or bifurcation/trifurcation. These changes must be designed to withstand internal pressure and properly streamlined to reduce hydraulic loss and damage from cavitation. The layout and analysis become very intricate due to intersecting cones and the intersection of cones and cylinders.

Manholes must be provided to allow for the periodic inspection of the penstock. One manhole should be provided as a minimum. Manholes need to be designed to have an effective seal to resist internal pressure.

One **expansion joint** should invariably be provided to accommodate expansion of the penstock and relative movements of the dam or powerhouse.

One **sleeve coupling** should be provided to accommodate vertical movement of the dam or powerhouse or foundation settlement.

POWERHOUSE

The powerhouse provides protection from weathering and other natural hardships for the electromechanical equipment that converts the potential energy of water into electricity. The number, type and power of the turbo-generators, their configuration, the scheme head and the geomorphology of the site are factors that control the shape and size of the building.

» *Location of the powerhouse*

Careful attention must be made when selecting the powerhouse location:

Figure 1.17: Powerhouse of Machkund, India, 120 MW installed capacity

Accessibility: It is desirable for the powerhouse to be located at a site with easy access for operation and maintenance purposes.

Conditions of the foundation: The foundations of the powerhouse must be strong enough to withstand the installation of heavy loads such as the electromechanical equipment. For a micro hydropower plant, a compacted gravel layer may be sufficient because of the relatively light equipment weight (approximately 2 to 3 tons per square metre).

Flood water level: The location of the powerhouse must avoid the level and section where the water flows to avoid scouring and to prevent the powerhouse from being flooded during high flows. In a situation where the flood stage is not recorded or established for a small-scale hydropower station, which is planned for a small river in a mountainous area, the flood water level could be assumed based on the information listed below that could be used to determine the ground elevation of the powerhouse with a sufficient margin:

- Information obtained from local residents
- Ground elevation of nearby structures (roads, embankments, bridges, etc.)
- Traces of flooding and vegetation boundaries

TAILRACE

After passing through the turbine, a short channel called the tailrace returns the water to the river. Relatively high exit velocities are common for impulse turbines, so the tailrace design should not compromise the powerhouse foundations. Rock rip-rap or concrete aprons should be installed between the powerhouse and the stream as a protective measure. Engineers should also ensure that during high-flow situations (e.g., floods in the river), the water in the tailrace does not exceed a certain level above which it would interfere with the turbine runner. Water levels in the tailrace influence turbine operation in reaction turbines and, more specifically, the onset of cavitation. This level also determines the available net head. In low-head systems, water levels in the tailrace can have a decisive influence on cost-effectiveness.

The design procedure for sizing the tailrace is the same as the channel; the only difference being that head-loss is not a concern to be worried about. This means that the velocity of the water can be very high and thus the cross-section of the tailrace can be reduced. However, material for construction needs to be selected carefully to avoid erosion of the canal due to high velocites. RCC is the usual choice for steep tailraces. Masonry (cement mortar), PVC, HDPE, steel pipes can also be used as tailrace. The selection of the pipe diameter is similar to the penstock design.

If the water from the powerhouse is carried through a long tailrace canal, the turbine regulation produces a surge which moves in the downstream direction. Such surges are the counter part of water hammer in pipe systems. In the tailrace canal, it is necessary to consider the worst surge height in determining the free board over the normal water level of the canal.

The location of the tailrace is determined using the same criteria as those applied for the powerhouse location because it is adjacent to the powerhouse. In other cases, the location of the tailrace is determined by taking the following four factors into consideration.

» ***Flood water level***

The tailrace channel should be preferably placed above the expected flood water level. When the base elevation of the tailrace (i.e. canal bed) is planned to be lower than the flood level, the location and base elevation of the tailrace must be decided in consideration of (i) suitable measures to deal with the inundation or seepage of water into the powerhouse due to flooding and (ii) a method to remove sediment which may be deposited in the tailrace channel.

» ***Riverbed fluctuation at tailrace***

When some amount of riverbed fluctuation is to be expected, the location of the water outlet must be chosen so as to avoid any problems with its operation due to sedimentation in front of the tailrace.

» ***Possibility of scouring***

If water is directly let out into a weak sandy streambed, it will erode the riverbed and create a pond. Over a period of time, this formation will continue to grow and erode the banks in the process. This is called scouring. Careful attention must be made to avoid the scouring of the riverbed and nearby ground. The selection of a location where protective measures can be easily applied is essential. As mentioned earlier, such protection measures can be a concrete apron or rock rip-rap.

» ***Flow direction of river water***

The tailrace must be directed (in principle, facing downstream) so as not to disrupt the smooth flow of the river water. Alternatively, a location should be selected which allows tailrace to face in the same direction as the river flow.

TRANSMISSION

In areas with a very high environmental sensitivity, the substation is enclosed in the powerhouse, and the transmission cables are laid along the penstock. Load flow studies need to be carried out to determine the optimum voltage and conductor size for any new transmission line or to ensure that a station connected to an existing line will not cause excessive voltage variations.

STATION EARTHING

Earthing or grounding systems are employed in situations and areas where major earth faults are common. The objective is to prevent the development of dangerous voltages that could result in electric shock or equipment damage. All metalwork must be connected to the same earthing system and this system needs to have the lowest possible resistance to the body of earth.

For optimal earthing, the reinforcing cage of the powerhouse building and other concrete structures are employed as the main earthing connection.

The reinforcing cage provides a far larger conductive area than buried cables or rods. In earthing terms, mass concrete serves as a good and consistent conductor of electricity. It should be noted that the lack of cables between the earthing system and equipment outside the powerhouse prevents the risk of cable damage or theft and makes the earthing system virtually indestructible. Main reinforcing bars at several points around the building should serve as connection points to the earthing system. Practical experience has shown that numerous contact points between the main reinforcing bars and traversing bars are more than sufficient when it comes to handling and distributing the earth fault current.

1.2. TYPES OF HYDROPOWER SCHEMES

The theoretical electrical power output from a hydropower scheme is calculated with the power equation below:

$$Power = Pressure \times Flow \times Efficiency$$

$$P = \rho \times g \times h \times Q \times e$$

Simplifying this equation with P in watts, Q in cubic metres per second (m^3/s) and h in metres, we get:

$$\begin{aligned} P &= 1000 \times 9.81 \times H \times (Q/1000) \times e \\ &= 9.81 \times Q \times H \times 0.51 \\ &= 5 \times Q \times H \end{aligned}$$

The efficiency is calculated by taking into account the efficiency of every component in the process of converting water to power:

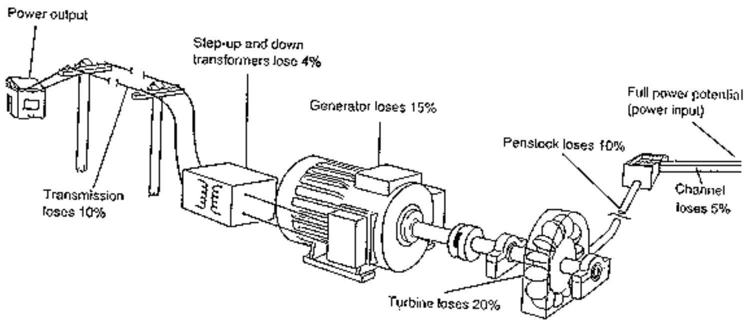


Figure 1.18: Efficiencies of different SHP components

$$\begin{aligned} \textit{Power output} &= e_{civil\ works} \\ &\times e_{penstock} \\ &\times e_{turbine} \\ &\times e_{transformer} \\ &\times e_{line} \\ &\times e_{power\ input} \end{aligned}$$

$$= 0.95 \times 0.9 \times 0.8 \times 0.85 \times 0.96 \times 0.9 \times \textit{power input} = 0.5 \times \textit{power input}$$

» Schemes types based on flow condition

The capacity, unit size and selection of equipment, along with equipment characteristics and specifications for the design of a hydropower station all depend upon the type of hydroelectric scheme and its classification with respect to head and size. There are three main types of hydropower schemes that can be categorised in terms of how the flow at a given site is controlled or modified. These are:

- Run-of-river (ROR) plants (no active storage scheme in Nigeria)
- Plants with significant storage (for example, Kainji Dam)
- Pumped storage

Run-of-river plants are different in design and appearance from conventional hydroelectric projects. Traditional hydro dams store enormous quantities of water in reservoirs, necessitating the flooding of large tracts of land. In contrast, most run-of-river projects do not require a large impoundment of water. A small dam is required to ensure that there is enough water to enter the penstock pipes that lead to the turbines.

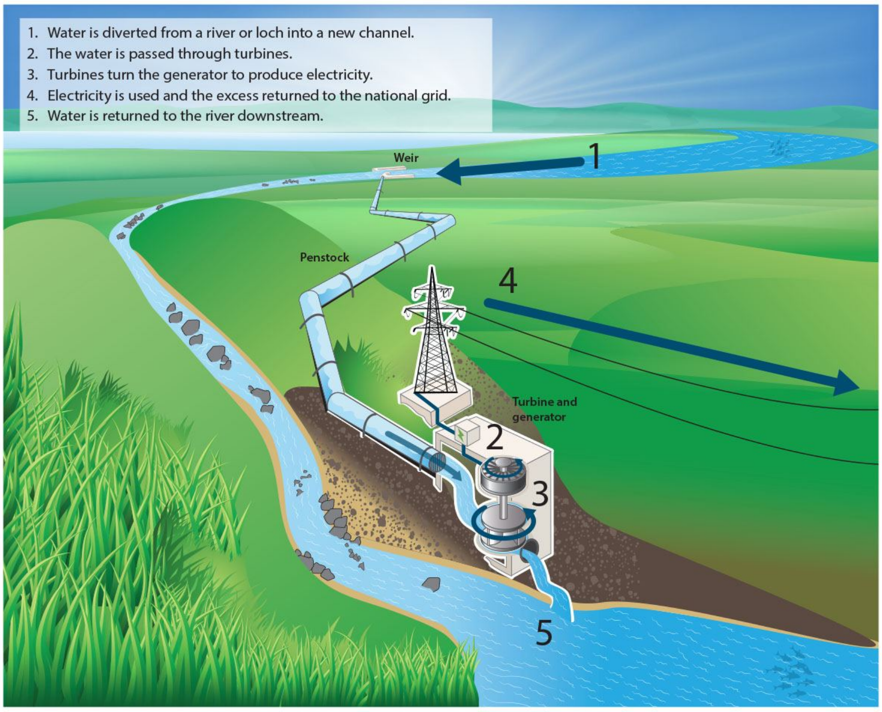


Figure 1.19: Run-of-the-river scheme – Courtesy Scottish Environment Protection Agency, www.sepa.org.uk

In the second type of scheme, **plants with significant storage**, the annual yield from the catchment (the area of land from which water flows towards a river) is stored completely or partially and then released according to some plan for the utilisation of storage. Water storage may serve a single purpose such as power development or it may target multiple purposes including irrigation, flood control, etc. Therefore, storage design will be governed by the intended use(s) of the stored water. If the scheme is only for power development, then the best use of the water will be to release controlled quantities based on power demand. Schemes with limited storage may be designed as peaking units. If the water project forms part of a large grid, then the storage is utilised for meeting the peak demands.

Figure 1.20: Example of large storage

In the third scheme discussed here, **pumped storage**, the basic principle is to convert the surplus electrical energy available in a system in off-peak periods to hydraulic potential energy.

The method stores energy in the form of gravitational potential energy of water, pumped from a lower elevation reservoir to a higher elevation. Low-cost off-peak electric power is used to run the pumps. During periods of high electrical demand, the stored water is released through turbines to produce electric power. Although, the losses occurring in the pumping process make the plant a net consumer of energy overall, the system increases revenue by selling more electricity during periods of peak demand, when electricity prices are highest.

Taking into account evaporation losses from the exposed water surface and conversion losses, energy can be recovered at a rate of 80% or more. This technique is currently the most cost-effective means of storing large amounts of electrical energy on an operating basis, but capital costs and the presence of appropriate geography are in addition, critical decision factors.

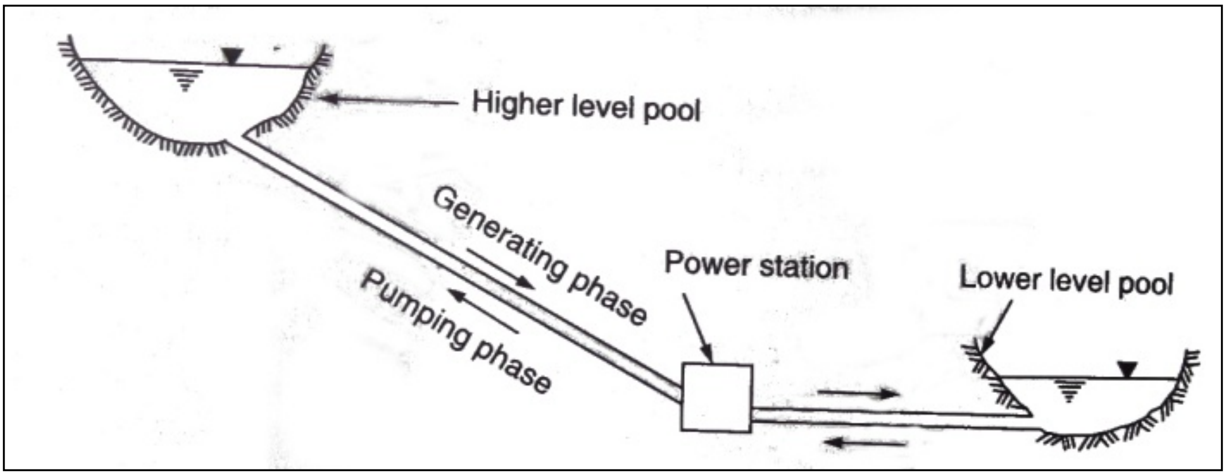


Figure 1.21: Pumped storage scheme

» *Based on head*

Hydropower plants are also categorized based on the amount of head. Practitioners generally differentiate three different types:

- Low head (up to 30 metres)
- Medium head (30 to 300 metres)
- High head (above 300 metres)

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

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

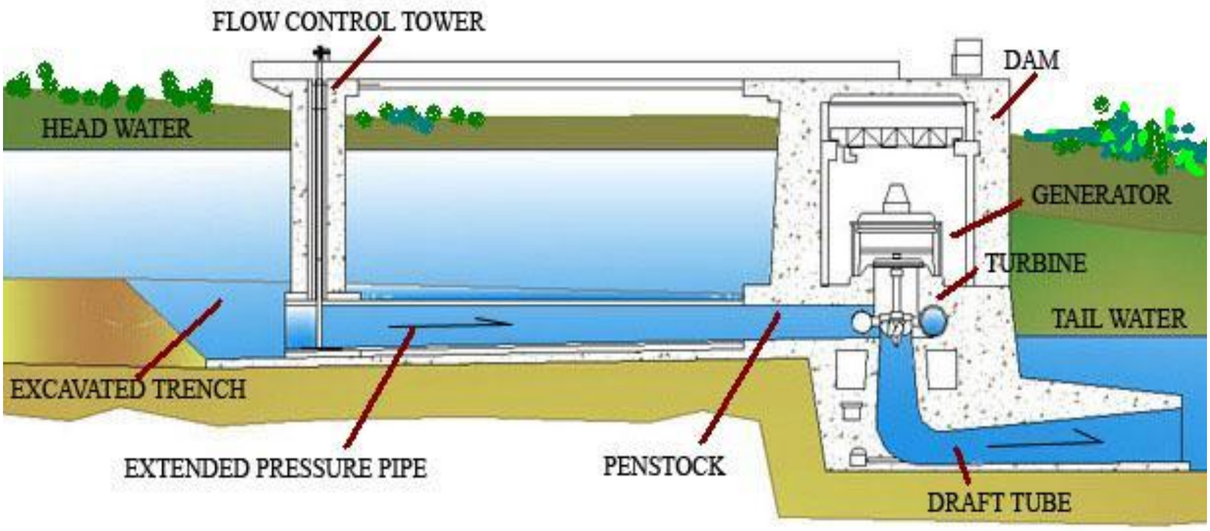


Figure 1.22: Low-head scheme

» *Schemes based on load and interconnection*

A **base load plant** operates continuously and generates constant power throughout the year. Examples of this type of plant include storage plants and ROR plants. On the other hand, a **peak load plant** supplies power during the peak hours only. Examples include a pumped storage plant or a ROR plant with pondage.

If a power station works independently, it is referred to as an **isolated plant**. These plants are generally installed in industrial settings (i.e. captive power plant). If a power station is connected with grid, it is called an **interconnected plant**.

1.3. TURBINE SELECTION

The hydraulic turbine is a mechanical device that converts the potential energy of water into rotary mechanical energy. Although, this handbook does not define guidelines for turbine design (a role reserved for the turbine manufacturers), it is appropriate to provide a few criteria to help choose the right turbine for a particular application and even to provide appropriate formulae to determine its main dimensions.

The conversion of potential energy happens in one of the two fundamental and basically different mechanisms:

- Water flows through the turbine expending its pressure along the runner surface. Turbines that operate in this way are called reaction turbines. The turbine casing is also exerted to the full operating pressure.
- Water pressure is converted into kinetic energy through high-speed jets, which hit the runner blades or buckets that are mounted on the periphery of the runner. These are called impulse turbines. Because the water falls into the tailwater with little remaining energy after striking the buckets, the casing can be lightweight and serves the purpose of preventing splashing.

Other limiting factors for turbines use:

- Efficiency at partial flow: If the turbine is required to run for substantial periods at partial flow, then its efficiency away from its optimum point is important.
- Strength and hydraulic stability: The turbine may not actually be able to function over the full power and speed range deduced from its specific speed (N) and head limitations. Propeller turbines, for example, cannot operate over a wide range of flows and heads. Even small changes affect performance drastically.
- Site-specific features: It may not be possible to excavate deep enough to fit a particular type of turbine, or the generator design may limit the permissible turbine speed.

IMPULSE TURBINES

An impulse turbine is a basic turbine category defined as a turbine equipped with one or more free jets discharging into an aerated space and impinging on the buckets of a runner. Efficiencies are often 90% and above. There are several types of impulse turbines, which are discussed below.

» Pelton turbines

Single-nozzle impulse turbines have a very flat efficiency curve and may be operated down to loads of 20% of rated capacity with good efficiency. For multi-nozzle units, the range is even broader because the number of operating jets can be varied.

Control of the turbine is maintained by hydraulically operated needle nozzles in each jet. In addition, a jet deflector is provided for emergency shutdown. The deflector diverts the water jet from the buckets to the wall of the pit liner. This feature provides surge protection for the penstock without the need for a pressure valve because the load can be rapidly removed from the generator without changing the flow rate.

Control of the turbine may also be accomplished by the deflector alone. On these units, the needle nozzle is manually operated and the deflector can divert a portion of the water jet for lower loads. This method is less efficient and normally used to regulate turbine speed under constant load.

Runners of the modern impulse turbine consist of a one-piece casting. Runners with individually attached buckets have proven to be less dependable and, on occasion, have broken away from the wheel, causing severe damage to the powerhouse. Integral cast runners are difficult to cast, costly, and require long delivery times. However, impulse turbine cost less to maintain than reaction turbines as they are free of cavitation problems. Excessive silt or sand in the water, however, will cause more wear on the runner of an impulse turbine than on the runner of most reaction turbines.

Figure 1.23: Pelton turbine

Water exits the turbine through the tailrace. The draft tube connects the exit point of the turbine with the tailrace. The function of the draft tube is to reduce the velocity of discharged water to minimise the loss of kinetic energy at the outlet, which could otherwise lead to low pressures that will damage the turbine.

Draft tubes are not required for impulse turbines. The runner must be located above maximum tailwater to permit operation at atmospheric pressure. This requirement necessitates an additional head loss (reduction in head) for an impulse turbine not required by a reaction turbine.

Impulse turbines may be mounted horizontally or vertically. The additional floor space required for the horizontal setting can be compensated by lower generator costs on single-nozzle units in the lower capacity sizes. Vertical units require less floor space and are often used for large-capacity multi-nozzle units. Vertical shaft multi-jet turbines are generally selected for large flow installations, whereas horizontal shaft turbines are suitable for those applications that have less water available.

» *Turgo turbines*

Another type of impulse turbine is the Turgo impulse. This turbine is higher in specific speed than the typical impulse turbine. Eric Crewdson originally patented this turbine in 1920. The difference between a Pelton unit and a Turgo is that, on a Turgo unit, the jet enters one side of the runner and exits the other side. The Turgo unit operates at a higher specific speed, which means for the same runner diameter as a Pelton runner, the rotational speed can be higher. The application head range for a Turgo unit is 15 metres to 300 metres. Turgo units have been used for applications up to 7,500 kilowatts.

» *Cross-flow turbines*

A cross-flow turbine is an impulse type turbine with partial air admission. This impulse turbine, also known as *Banki-Michell* in remembrance of its inventors, or *Ossberger* after a company, which has been making it for more than 50 years, is used for a wide range of heads overlapping those of Kaplan, Francis and Pelton. It can operate with discharges between 20 litres per second and 10 cubic metres per second, with heads ranging between 1 and 200 metres.

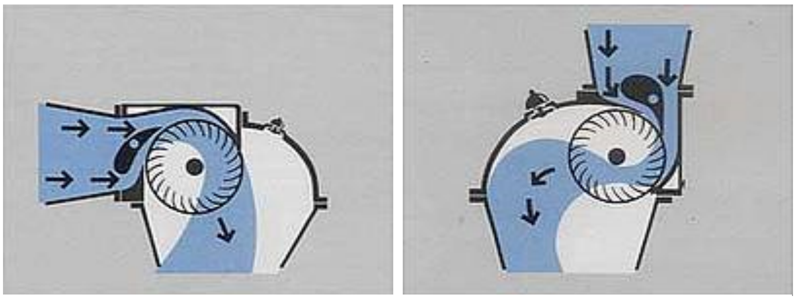


Figure 1.24: Cross-flow turbines horizontal and vertical entry types

Performance characteristics of this turbine are similar to those of an impulse turbine, and consist of a flat efficiency curve over a wide range of flow and head conditions. The wide range is accomplished by use of a guide vane at the entrance, which directs the flow to a limited portion of the runner depending on the flow. This operation is similar to the operation of multi-jet impulse turbines.

Peak efficiency of the cross-flow turbine is less than that of other turbine types previously discussed. Efficiency of the cross-flow turbine is about 60–65%, with maximum efficiency seldom exceeding 80%. The cross-flow turbine **is manufactured in Nigeria** by Engineering Materials Development Institute (EMDI) at Akure.

On cross-flow turbines, the largest runner size is about 1.2 metres in diameter. This limits the capacity of individual units, but multi-unit installations can be used. Cross-flow turbines are sometimes equipped with a conical draft tube creating a sub-atmospheric pressure in the turbine chamber. Therefore, the difference between the turbine centre-line elevation and the tail water is not lost on cross-flow turbines as is the case for an impulse turbine. Air is admitted into the chamber through an adjustable air inlet valve used to control the pressure.

Cross-flow turbines are free from cavitation, but are susceptible to wear when the water contains excessive silt or sand particles. Runners are self-cleaning and, in general, maintenance is less complex than for the other turbine types.

REACTION TURBINES

In reaction turbines, the second major turbine category discussed here, water enters at full pressure and the pressure is dissipated over the entire surface of the runner. This places the entire turbine casing under pressure as well. Reaction turbines are classified as Francis (mixed flow) or propeller (axial flow) turbines. Propeller turbines are available with both fixed blades and variable pitch blades (Kaplan).

» Francis turbines

Figure 1.25: Francis turbine

Francis turbines are radial flow reaction turbines, with fixed runner blades and adjustable guide vanes, used for medium heads. In the high-speed Francis turbine, the admission of water is always radial but the outlet is axial.

Francis turbines can be set in an open flume or attached to a penstock. For small heads and SHP configurations, open flumes are commonly employed. Steel spiral casings are used for higher heads, designing the casing so that the tangential velocity of the water is constant along the consecutive sections around the circumference. This implies a cross-sectional casing area that differs along the length of the spiral. Small runners are usually made in aluminium bronze castings. Large runners are fabricated from curved stainless steel plates, welded to a cast steel hub.

» *Kaplan and propeller turbines*

Figure 1.26: Kaplan turbine

Kaplan and propeller turbines are axial-flow reaction turbines, generally used for low heads. The Kaplan turbine has adjustable runner blades and may or may not have adjustable guide vanes. If both blades and guide vanes are adjustable, it is described as *double-regulated*. If the guide vanes are fixed, the turbine is *single-regulated*. Unregulated propeller turbines are used when both the flow and head remain practically constant.

The Kaplan turbine was an evolution of the Francis turbine. The head ranges from 10 to 70 metres and the output from 5 to 200 megawatts. Runners are between 2 and 11 metres in diameter. The flow enters radially and makes a right angle turn before entering the runner in an axial direction. The control system is designed so that the variation in blade angle is coupled with the guide vane setting in order to obtain the best efficiency over a wide range of flows. The blades can rotate with the turbine during operation, through links connected to a vertical rod sliding inside the hollow turbine axis.

Since the blades are rotated in high-pressure hydraulic oil bearings, a critical element of Kaplan design is to maintain a positive seal to prevent emission of this oil into waterways. The turbine does not need to be at the lowest point of water flow as long as the draft tube remains filled with water.

» *Bulb turbines*

Bulb turbines were developed about 50 years ago and are now a very well-established technology. The main application range is for heads from 3 to 20 metres and outputs from 1 to 50 megawatts. The draft tube outlet is usually square and these turbines do not have a large spiral casing, thereby taking up less lateral space than vertical Kaplan turbines. This makes them particularly attractive if large flows must be handled and lateral space is limited. Bulb turbinestend to be more efficient than vertical Kaplan turbines because the water flows straight into the turbine and the draft tube is straight. Generators for bulb turbines have a very low inertia and it is virtually impossible to incorporate a flywheel, so most bulb units are not capable of controlling the frequency on an isolated system.

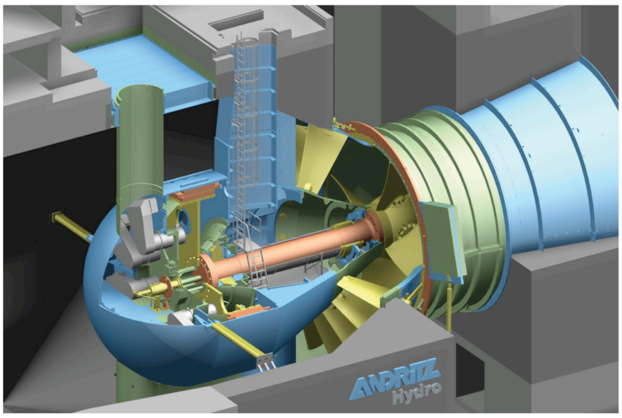


Figure 1.27: Bulb turbine

» *Pumps as turbines (PATs)*

Standard centrifugal pumps may be operated as turbines by directing flow through them from the pump outlet to the inlet. Since they have no flow regulation, they can operate only under relatively constant head and discharge levels. Pump storage schemes (operating both as pumps and turbines) brought this possibility. The advantages of using pumps as turbines (PATs) are:

- Pumps of all shapes and sizes are manufactured worldwide and the market for small pumps is enormous.
- Pumps are much cheaper as they are mass produced.
- Pump technology, its operation and maintenance is easily understood.
- No real design or drawing costs involved for pumps.

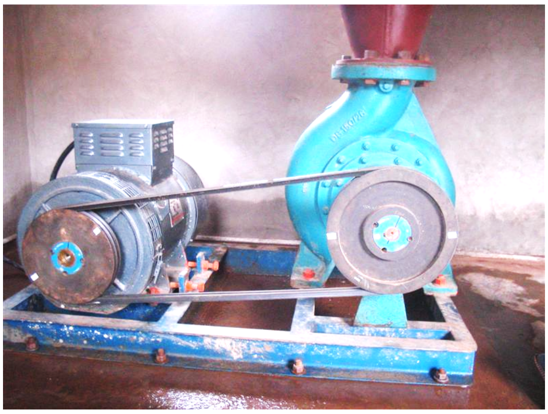


Figure 1.28: Pump as turbine

The operational range is wide and depends upon the selected pump's parameters. Pumps are usually used as turbines in schemes with heads ranging from 5 to 100 metres.

» *Draft tubes*

In reaction turbines, to reduce the remaining kinetic energy in the water as it exits the runner, a draft tube or diffuser stands between the turbine and the tailrace. A well-designed draft tube allows, within certain limits, the turbine to be installed above the tailwater elevation without losing any head. As the kinetic energy is proportional to the square of the velocity, one objective of the draft tube is to reduce the outlet velocity. An efficient draft tube would have a conical section but the angle cannot be too large, otherwise flow separation will occur. The optimum angle is 7 degrees, but to reduce the draft tube length, and therefore its cost, the angle could be increased to 15 degrees. Draft tubes are particularly important in high-speed turbines, where water leaves the runner at very high speeds.

To maximise efficiency, there must be a straight conical section of draft tube for at least one runner diameter downstream of the runner. Most draft tubes incorporate a right angle bend and it is very important to ensure that flow around the bend is uniform without flow separation or reversal. For optimal draft tube efficiency, the cross-sectional area should contract slightly around the bend. A bend with a constant area is less efficient, and an expanding bend must be avoided.

With large vertical turbines, optimising the hydraulic design of the draft tube is a very important process. To minimise excavation costs, the manufacturer will be compelled to keep the draft tube as shallow as possible. This means that the bend is quite sharp and, in some cases, the straight section of draft tube below the runner will be quite short. If a draft tube has been model tested with good results and the model design is reproduced faithfully, good performance can be guaranteed.

A draft tube consisting of a straight conical section followed by a slightly contracting

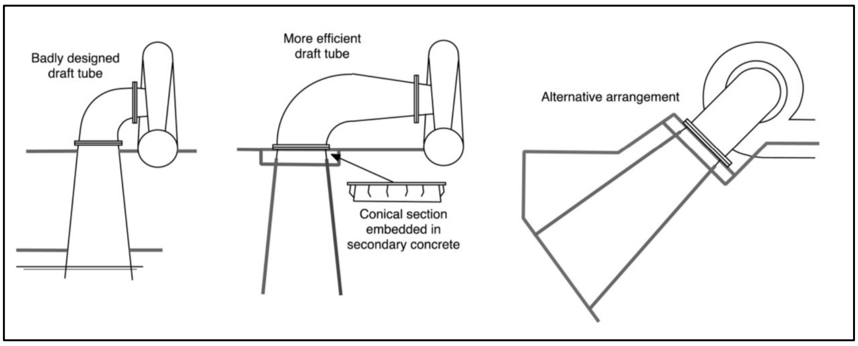


Figure 1.29: Draft tube options

bend and then by a second conical section will always be more efficient. If a dismantling joint is installed at the end of the first conical section or at the end of the bend, inspection and maintenance are simplified.

Should the draft tube diameter measure more than a metre or so, it is always a good idea to provide a manhole (or an inspection port for smaller runners) so that the runner can be inspected easily, and, with runner outlet diameters in excess of about 1.3 metres, so that a person can gain access to the draft tube for minor repair operations or a close inspection.

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2. SITE ASSESSMENT

What this module is about

There are numerous considerations to take into account when assessing the suitability of a site for a hydropower scheme. Civil engineers need to have a comprehensive grasp of these considerations while planning construction works for a hydropower scheme. This module describes the hydrological and geological features that are relevant to the successful deployment of a hydropower scheme.

Learning outcomes

At the end of this module, the participant is able to

- Measure the head and flow of a stream
- Assess the potential of a site
- Assess the geological features of the site
- Define the site layout

INTRODUCTION

In most micro hydropower schemes, the selection of an appropriate site takes place as an iterative process. Usually, some community non-governmental organisations (NGOs) with previous exposure to micro hydropower technology approach funding agencies, consultants, or manufacturers, depending upon their financial resources and the size of the scheme. The technicians concerned undertake a site visit to assess whether the site is feasible for a micro hydro installation. Based on the feasibility report submitted by these technicians, the community members and other process stakeholders decide whether to proceed further with the development of the scheme.

Once the decision has been made to proceed with the scheme and if it is situated in the upper range of micro hydro (say, above 20 kilowatts), then a detailed survey of the project area is undertaken and a detailed design report is prepared. Approvals for loans, subsidies, and grants by funding agencies and banks are based on this report. There are frequent meetings between the concerned parties during this stage. For the lower end of micro hydro (say, less than 20 kilowatts), the manufacturers usually undertake both the design and installation.

Apart from socio-economic factors such as the need for affordable electricity and a balance of supply and demand, technically speaking the selection of an appropriate site depends on the following two factors:

- Stream flow
- Topography

It should be noted that designers have little control over the flow in the stream. However, they do have some control over the topography. They can choose different alignments for the intake, headrace and penstock. They can also modify the local topography through excavation, building of structures and by undertaking measures to enhance soil stability.

WHY SITE INVESTIGATION?

Site investigation is the preliminary work carried out to establish the suitability for construction of the various options (or the most feasible option if it is apparent) through the investigation of soils, slope stability, flood levels, surface water movement and subsidence (movement of the earth's surface).

Note: In most civil engineering work, the unexpected happens. Site investigation aims to predict what this might be so that the engineer can prepare a design that will deal with it.

It should be noted that head and flow measurements serve to establish the options available for micro hydro development for the site. The site investigation then assesses the site's suitability for each alternative. In this process, engineers can choose the optimum layout where more than one option appears to be feasible. Site conditions are also recorded during the site investigation stage so that there is adequate information for the detailed design phase. The principles of site investigation are:

- Take your time and be thorough. A return visit to collect information missed the first time is costly and inadequate civil design even more so.
- Visit all possible sites. Gain a full appreciation of the available options.
- Talk to local people, especially those who have carried out construction work in the area. Since most of the rivers in Nigeria have not been gauged, stream flow data is not usually available. Therefore, it is important to talk to local people to get a feel for potential flood levels for rare flood events (for the past 20 to 50 years).
- Stay focussed on the bigger picture; aim to raise understanding and awareness of changes in the site over time.

2.1. HYDROLOGY

INTRODUCTION

Hydrology is concerned with the distribution of water on the earth's surface as well as its movement over and beneath the surface, and through the atmosphere. This wide-ranging definition suggests that all water comes under the remit of a hydrologist, while in reality it is the study of fresh water that is of primary concern. Knowledge of hydrology is fundamental to hydropower professionals.

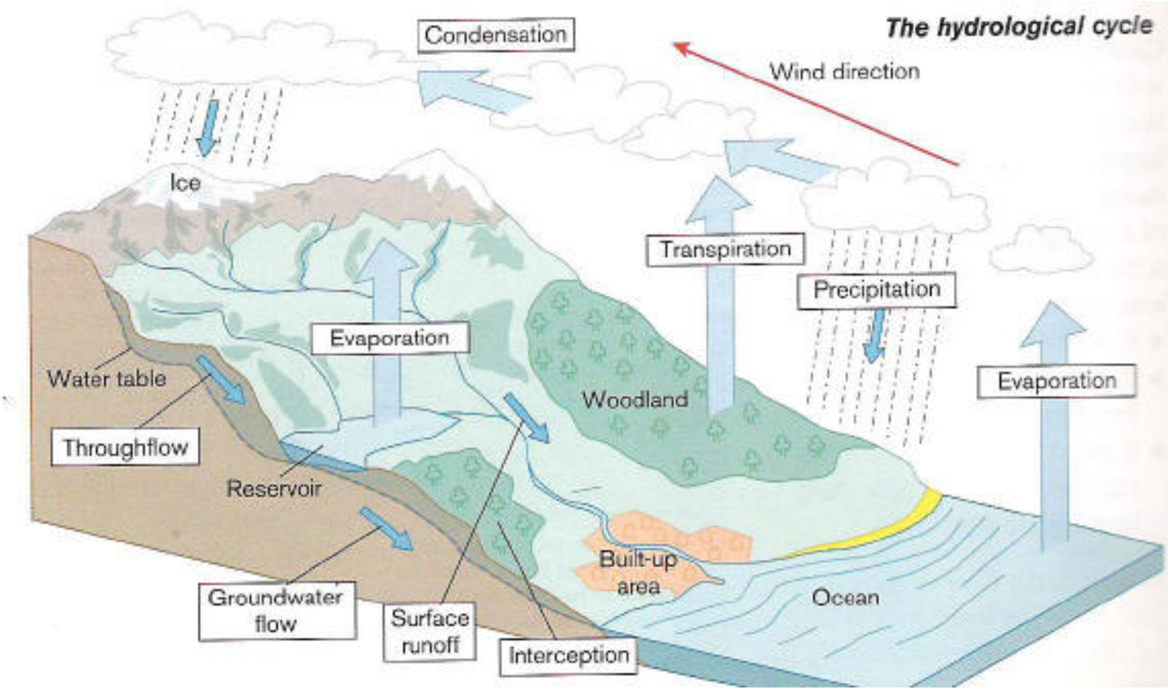


Figure 2.1: The hydrological cycle

As a starting point for the study of hydrology, it is useful to consider the hydrological cycle. This is a conceptual model of how water moves around between the earth and atmosphere in different states as a gas, liquid or solid. As with any conceptual model, it contains many gross simplifications; these are discussed in this section. Hydrology can be subdivided into:

- Surface water hydrology
- Sub-surface or ground water hydrology

For hydropower projects, only the water flowing directly at the project site over the surface is relevant; as such, surface hydrology is of greater significance. Sub-surface hydrology or groundwater hydrology does not have a direct bearing on hydropower development and hence, is not discussed further in this context.

PRECIPITATION

Precipitation is the release of water from the atmosphere to reach the earth's surface. The term covers all forms of water released by the atmosphere, including snow, hail, sleet and rain. It is the main water input entering a river catchment area and therefore needs careful assessment in any hydrological study. Although, rainfall is relatively straightforward in terms of measurement (other forms of precipitation are more difficult), it is notoriously difficult to measure accurately and, to compound the problem, is also extremely variable within a catchment area. Studying the rainfall and the resultant run-off is of greater importance to hydropower engineers.

» *Measuring rainfall*

For hydrological analysis, it is important to know how much precipitation has fallen and when this occurred. The usual expression of precipitation is as a vertical depth of liquid water. Rainfall is measured by millimetres or inches depth, rather than by volume such as litres or cubic metres. The measurement is the depth of water that would accumulate on the surface if all the rain remained where it had fallen. Rainfall is measured using:

- Rain gauges
- Radar observation
- Satellite observations

RAIN GAUGES

Three common types of rain gauges are:

- Non-recording rain gauges: These gauges are observed manually and show the total amount of rainfall at the rain gauge station during the measuring interval. The rainfall collected in the bottle is measured daily once or twice at a definite time using a special measuring cylinder, usually one with a 1,000 millimeters (mm) capacity.
- Recording rain gauges: These are self-recording automatic rain gauges that record cumulative rainfall continuously over time and show this development in the form of a chart.
- Automatic radio reporting rain gauges: In remote and inaccessible mountainous areas where the manual collection of rainfall data is not normally possible, the weather station uses an automatic radio reporting rain gauge. Such gauges rely on telemetry and are deployed in what is known as a real-time system. Hydrometeorological data (such as run-off, rainfall, etc.) is collected through remote sensors, transmitted and used almost simultaneously in a computer system for flood forecasting or reservoir operation.

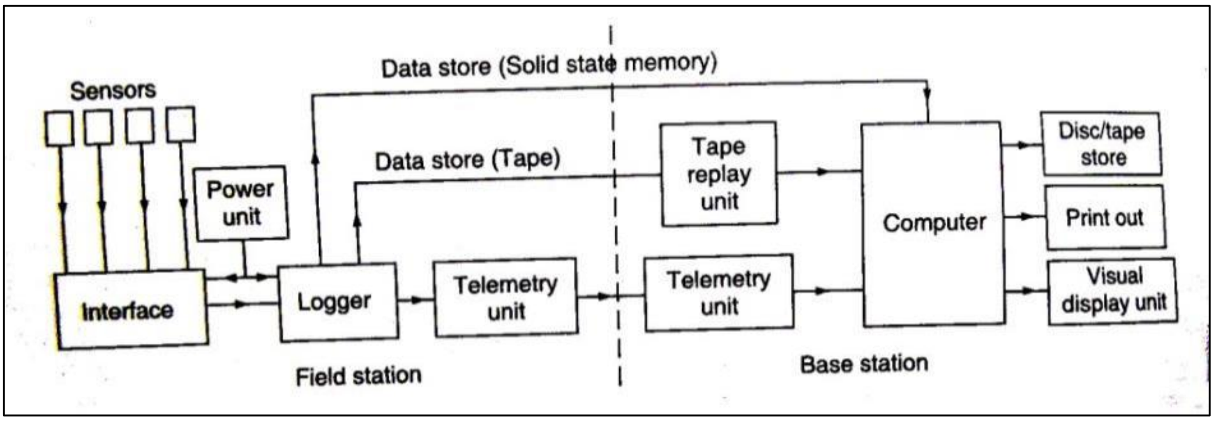


Figure 2.2: Automatic weather station diagram

RADAR AND SATELLITE OBSERVATIONS

Radar is an important means to analyse storm mechanics, determine the real extent, and assess the orientation and movement of rainstorms. Radar signals reflected by the rain are helpful in determining the magnitude of storm precipitation and its real distribution. Radar has a range of 50 to 60 kilometres.

Weather satellites used to assess precipitation have become a common feature of hydrometeorology. Large convective storms can now be easily identified using remote sensory technology and satellite imagery. These storms are correlated with ground-based precipitation measurements.

» *Interpretation of rain gauge data*

The methods described above all measure rainfall at a precise location. However, in reality, we need to know how much rain has fallen over a far larger area such as a catchment. To move from point measurements to a spatially distributed estimation, it is necessary to employ some form of spatial averaging. The usual practice, therefore, is to install a number of rain gauges distributed over the catchment area. The number may be approximately one rain gauge for an area of about 150 to 250 square kilometres. The optimum number of gauges would depend upon the spatial variability of the rainfall as well the desired accuracy level.

There are different statistical techniques that address these spatial distribution issues, and with the increasing use of Geographic Information Systems (GIS), it is often a relatively trivial matter to do the calculation. As with any computational task, it is important to have a good knowledge of how the technique works so that any shortcomings are fully understood. Three techniques are described here:

ARITHMETIC MEAN METHOD

This method computes the rainfall by applying a formula to work out a simple arithmetic mean (average) of the precipitation occurring at each station in the area. Thus, if P_1, P_2 P_3....P_n are the precipitation averages provided by stations 1, 2, 3…n respectively, then we have

$$P = \frac{P_1 + P_2 + P_3 \ldots P_n}{n}$$

This method to calculate a simple arithmetic mean is only expedient when uniformly spaced rain gauges are used to sample the catchment. Topographical diversity should also not be present. If these conditions ever actually existed, only a single rain gauge would be required to sample an area. Use of a simple averaging technique is therefore very rare.

THIESSEN'S POLYGONS

A US engineer who was active around the turn of the twentieth century, Thiessen devised a simple method to overcome the problem of uneven rain gauge distribution within a catchment.

This method is based on the assumption that the precipitation value at a rain gauge station is representative of the rainfall for the surrounding area. An arbitrarily constructed polygon is used to depict the catchment area for which the rain gauge station gives this representative value. Thiessen proposed that these polygons be constructed as shown in Figure 2.3.

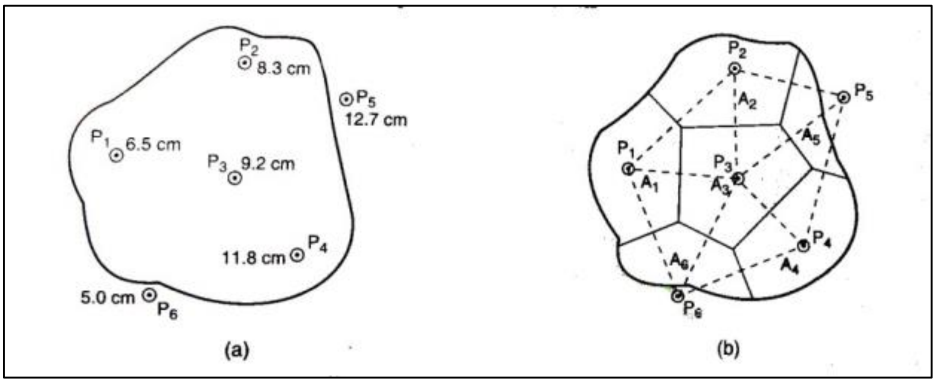


Figure 2.3: Thiessen's polygons

Adjacent stations are joined by straight (dotted) lines, which divide the entire basin area into a series of triangles. Perpendicular lines that bisect each dotted line are then drawn to create a series of polygons. Each polygon encloses a representative rain gauge station for a particular area.

If P_1, P_2, P_3 … P_n stand for the station rainfall records enclosed by polygon, the areas of which are respectively A_1, A_2, A_3, ….A_n , then the mean precipitation of P for the entire basin area A is expressed by

$$P = \frac{A_1P_1 + A_2P_2 + A_3P_3 + \ldots + A_nP_n}{A_1 + A_2 + A_3 + \ldots + A_n}$$

$$= \frac{A_1P_1}{\sum A} + \frac{A_2P_2}{\sum A} + \frac{A_3P_3}{\sum A} + \ldots + \frac{A_nP_n}{\sum A} = \frac{\sum_{i=1}^{i=n} A_iP_i}{\sum_{i=1}^{i=n} A_i}$$

Here, $\frac{A_1}{\sum A} + \frac{A_2}{\sum A} + \frac{A_3}{\sum A}, \ldots,$ etc., are termed as weightage factors for the corresponding rain gauge. These weightages are constant for different storms within the scope of this method.

With each rain gauge station weighted according to its position in the considered basin area, the Thiessen polygon method can be viewed as superior to the arithmetic mean method. Stations within or very near the basin area are therefore automatically weighted higher than those near but outside the basin. This method is used for comparatively small and moderate size basins of up to about 500 square kilometres. This method would be more suitable than the isohyetal method (refer Figure 2.4) if a particular basin or catchment has fewer rain gauges than the recommended minimum.

ISOHYETAL METHOD

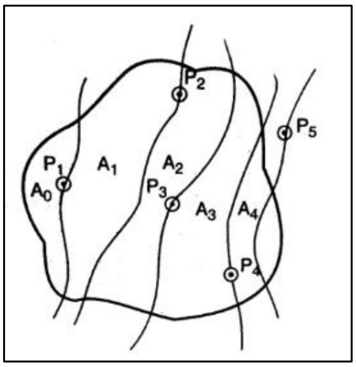


Figure 2.4: Isohyetal map

Isohyetal maps are no more than contour maps of precipitation. Each isohyet (or each contour line) represents equal rainfall along its length.

For estimating the mean precipitation, the points of equal rainfall are joined to construct the isohyets of various magnitudes. The area between the two successive isohyets is measured with the help of a planimeter (also known as a platometer) and the mean of the rainfall on the two adjacent isohyets is assumed to be occurring in the enclosed area. Thus, if P_{12} is the mean of precipitations represented by the two successive isohyets of magnitudes P_1 and P_2, and the area between two isohyets is A_1, and P_{23} is the mean of the precipitations P_2 and P_3, the area between the isohyets of P_2 and P_3 being A_2, and so on, then the mean precipitation over the entire area can be expressed by

$$P = \frac{A_1 \times P_{12} + A_2 + P_{23} + \ldots + A_n \times P_{(n)(n+1)}}{A_1 + A_2 + \ldots + A_n}$$

For boundary areas, suitably extrapolated values may be assumed. Thus for the area A_0, an average value $P_0 = P_1 + \Delta P$ where ΔP is based on the general rainfall trend. It may be noted that the factor $(A_n/\sum A)$ is a constant for each rainstorm. This may be expressed as a weightage factor for the n^{th} rain gauge.

This method is suitable for isolated studies in hilly or rugged and large areas more than 5,000 square kilometres and is more accurate compared to other methods.

REGIONAL PRECIPITATION ANALYSIS

Precipitation data is often subjected to regional analysis in order to find large-scale regional patterns. A variety of such maps can be prepared for obtaining insights into rainfall patterns. The precipitation pattern in Nigeria is shown in Figure 2.5 and Figure 2.6.

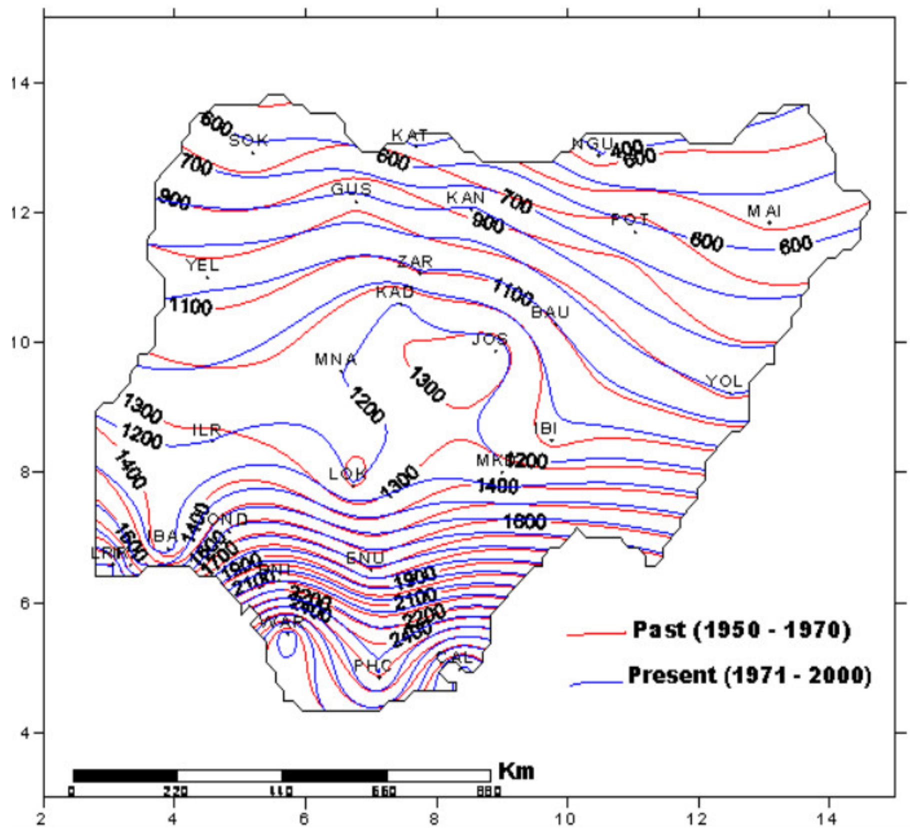


Figure 2.5: Hyetograph of Nigeria

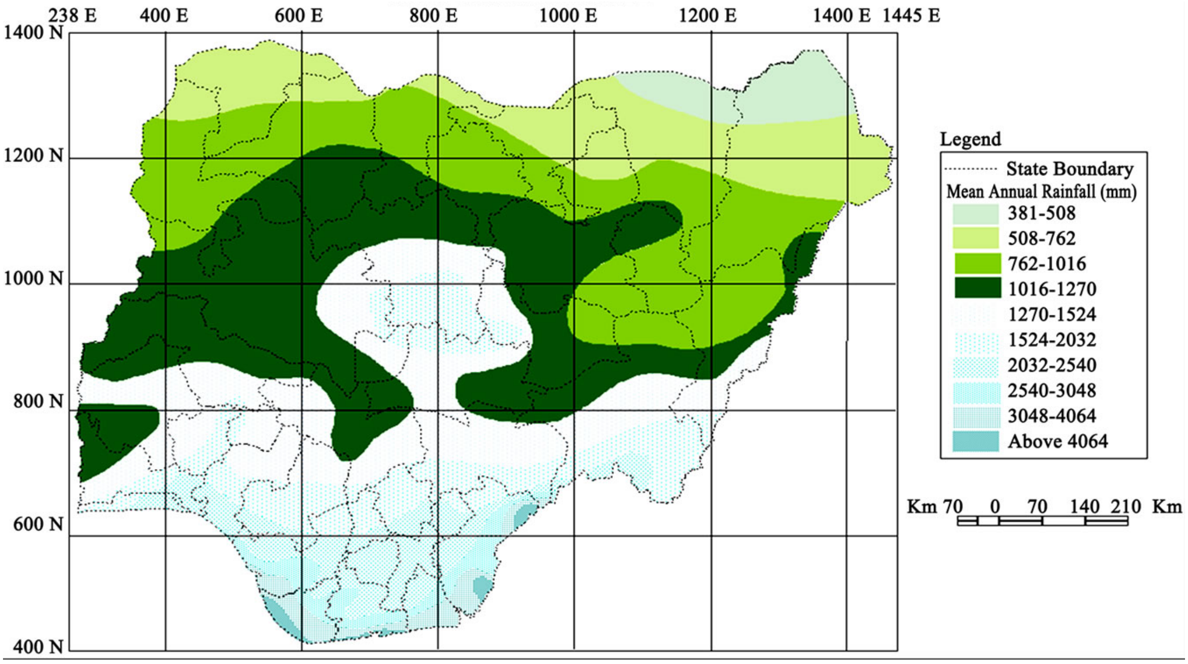


Figure 2.6: Mean annual rainfall in Nigeria

RUN-OFF AND STREAM FLOW

» Catchment area

A catchment or river basin can be defined as the area of land from which water flows towards a river and then in that river to the sea. The terminology suggests that the area is analogous to a basin where all water moves towards a central point (i.e. a drain, or in this case, the river mouth). The common denominator of any point in a catchment is that wherever rain falls, it will end up in the same place: where the river meets the sea (unless lost through evaporation). A catchment may range in size from a matter of hectares to millions of square kilometres.

In terms of its topography, a river basin can be defined based on the assumption that waterfall on a surface always flows downhill. This makes it possible to draw a catchment boundary, which defines the actual catchment area for a river basin. However, it should be noted that the assumption that all water makes its way *down* to a river is not always correct. In certain cases, the underlying catchment geology is complicated and water can flow as groundwater into another catchment area, creating a problem for the definition of *catchment area*.

These problems aside, the catchment does provide an important spatial unit for hydrologists to consider how water is moving about and being distributed at a certain time.

» Run-off

A rather general term, run-off refers to water moving to a channelised stream after having fallen to the ground as precipitation. Water can move at varying velocities on or below the surface. When it comes into contact with a stream, the water is channelled towards the oceans in a process called *stream flow* or *riverflow*. Stream flow is expressed in terms of discharge: the volume of water over a defined time period.

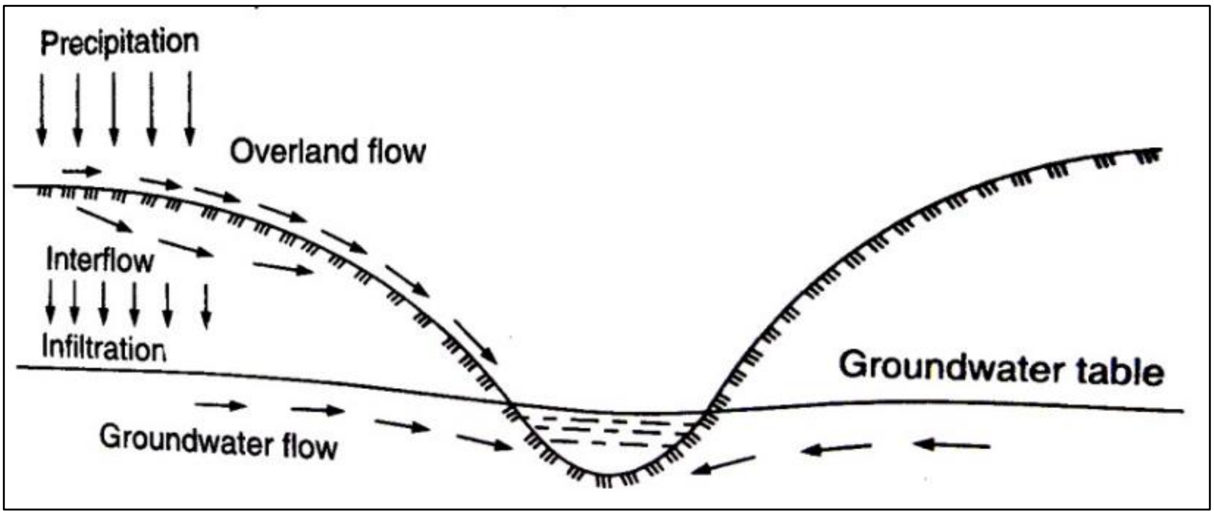


Figure 2.7: Run-off showing flow types

Overland run-off or flow starts soon after water falls to the ground in the form of precipitation. Interflow is a slow process, and groundwater flow is the slowest. After a storm, water defined as overland flow and interflow would make contact with the stream after a number of hours. The groundwater flow response, in contrast, may take days. Overland flow and interflow can therefore be grouped as an analytical category. As this portion of run-off is called *direct run-off*, it stands to reason that groundwater flow is then referred to as *indirect run-off*.

Calculating run-off is beyond the scope of this course. For now, it is sufficient to know the fundamental operation of the catchment area.

AREA-RAINFALL METHOD TO PREDICT FLOW

Careful examination of a map shows that a number of possibilities exist for the design of a hydro scheme. In the best-case scenario, the hydrologic authorities would have installed a gauging station in the stretch of stream under consideration and stream flow time series data would have been gathered regularly over several years. The records in general may include:

- Measured stream flow data for a number of gauged sites
- Stream flow characteristics for these sites such as mean flow and flow duration curves
- Run-off maps

These stream flow indicators can all be collected by the concerned department. For reasonable assumptions, the data collection period must date back at least 20 years. The object of stream gauging is to estimate the discharge in the stream.

If an appropriate stream flow time series cannot be found, and sufficient time is available, the discharge may be directly measured for at least a year – a one-off measurement of instantaneous flow in a watercourse is of little use. Several methods are available to measure the discharge.

VELOCITY-AREA METHOD

This is a conventional method for medium to large rivers, involving the measurement of the cross-sectional area of the river and the mean velocity of the water flow; it is a useful approach for determining the stream flow with minimal effort. An appropriate point must be selected on a relatively straight, smoothly flowing section of the river to be gauged. The river at this point should have a uniform width, and the area must be well-defined and clean.

The top water level (or stage of the river) will increase and decrease with variances in discharge. Observe the stage on a daily basis at the same time. A *measuring board* – or large-scale ruler with metre and centimetre markings can be used for this purpose.

Measuring the area: To compute the cross-sectional area of a natural watercourse it should be divided into a series of trapezoids. Measuring the trapezoid sides, by marked rules, such as the Figure 2.8 below, illustrates, the cross-section would be given by:

$S = b \times \frac{h_1 + h_2 + \cdots + h_n}{n}$

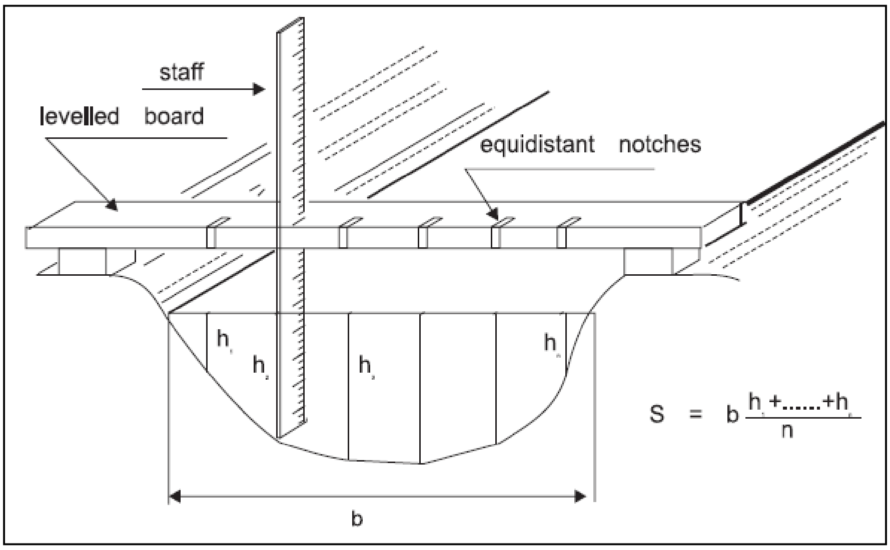


Figure 2.8: Measuring the cross-section of the river

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

- **Float method:** A series of nearly submerged floats, like a wooden plug or a partially filled bottle, are placed in the centre of the stream flow and then timed as they move over a measured length of stream. A flow velocity is obtained by averaging the results over a large number of trials. This velocity must then be reduced by a correction factor, which estimates the mean velocity as opposed to the surface velocity. By multiplying averaged and corrected flow velocity, the volume flow rate can be estimated.
- **Propeller current meters:** This method is more accurate than the float method. A current meter consists of a shaft with a propeller or revolving cups connected to one end. The propeller is free to rotate and the speed of rotation is related to the stream velocity. A simple mechanical counter records the number of revolutions of a propeller placed at a desired depth. By averaging the readings recorded evenly throughout the cross-section, an average speed of the stream can be obtained.

  In the case of medium to large rivers, observations are made by lowering the meter from a bridge, though if the bridge is not a single-span construction there will be divergence and convergence of the streamlines caused by the piers, which can result in considerable errors. In many instances, however, the gauging site will have no bridge but it should be placed in a section of the river that is as straight and uniform as possible. If it is deep and the river is in a flooded state, a cable must be provided to stabilise the boat, together with a lighter measuring cable to determine horizontal position in the cross-section.

- **Electromagnetic current meters:** An electro-magnetic (e/m) current meter is an instrument that measures electrical induction. It is free of moving parts and mounted in a fully enclosed streamlined probe. This probe can be mounted on rods and held at various depths. It may also be configured in a cable suspension.

  In terms of its advantages, this meter is smaller and offers a wider measurement range than the propeller meters. It can also withstand very low velocities when propeller meters become erratic, making it an expedient choice for a broad spectrum of stream flow conditions. It is highly sensitive and less vulnerable to debris and weeds. These two properties enable its use in heavily polluted or densely vegetated streams.

- **Dilution method:** This method is relatively easy to perform, reasonably accurate (error probability is less than 7%), and reliable for a wide range of stream types. It gives better results if the stream is more turbulent. Using this approach, a spot check of the stream flow can be taken in less than 10 minutes with very little equipment.

The method involves the following steps: First, a bucket of heavily salted water is poured into the stream. The cloud of salty water in the stream starts to spread out while travelling downstream. After moving some distance downstream, it will have filled the width of the stream. The cloud will have a leading segment with low saline content, a middle segment with high saline content, and a lagging segment, which is again low in saline. The saltiness (salinity) of the water can be measured with a conductivity meter. If the stream is small, it will not dilute the salt very much, so the electrical conductivity (measured in microsiemens, µS) of the cloud (which is greater, the saltier the water) will be high. Therefore, low flows are indicated by high conductivity and vice versa. The flow rate is therefore inversely proportional to the degree of conductivity of the cloud.

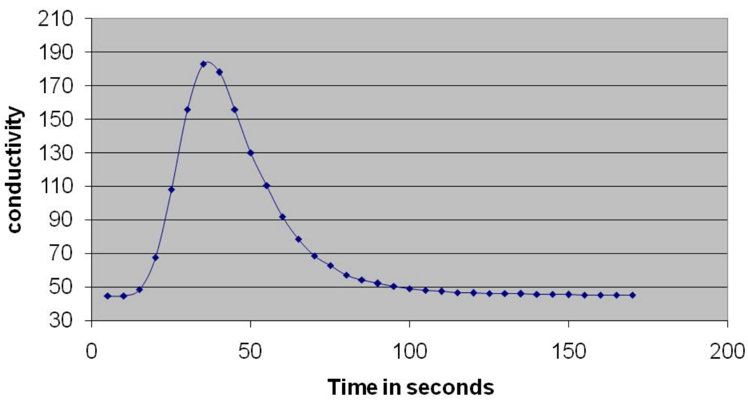


Figure 2.9: Dilution graph (µS/s)

The above phenomenon assumes that the cloud passes the probe in the same amount of time in each case while the slower the flow, the longer the cloud takes to pass the probe. Flow is therefore inversely proportional to the cloud passing time. For this flow measurement, also called the *salt gulp* method, the necessary equipment is a bucket, table salt, a thermometer, and a conductivity meter (with a 0 to 1,000 millisecond range).

Select two points in a stream, 20 to 70 metres apart. A 20-metre interval should be chosen for turbulent streams, in which salt will be distributed quickly. Otherwise, a higher value should be chosen. Throw the bucket of heavily salted water into the river at the upstream point. The salty cloud will start to spread and will have filled the stream width at a certain point downstream. The cloud will have a leading segment, a middle segment, and a lagging segment (with weak-strong-weak salt concentrations, respectively). Immerse a probe rod connected to a conductivity meter close to the streambed, or ideally at mid-depth, at the second point downstream. Avoid fast-flowing river sections for this test. Salt is electrically conductive, so the saltiness of water can be determined using an electrical conductivity meter. If the stream is moving slowly, it will take a long time for the cloud to pass. The relative lack of water also means that the salt concentration will stay high and the conductivity is high, as well.

Based on these considerations, it becomes clear that high conductivity and a long cloud passing time are characteristic of low flows. High flows, on the other hand, are characterised by weak dilutions, low conductivity, and a short passing time. Flow is thus inversely proportional to both cloud passing time and the degree of conductivity. Once the meter reading begins to increase, take a conductivity reading every five seconds. The reading will rise, peak, and return to the base level. If the reading is uneven, the salt has failed to properly mix. In this case, repeat the test with a longer distance. If the peak reading is less than twice the base reading, repeat the test with a larger quantity of salt.

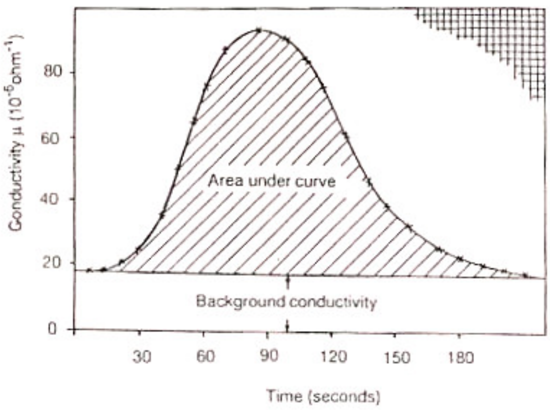
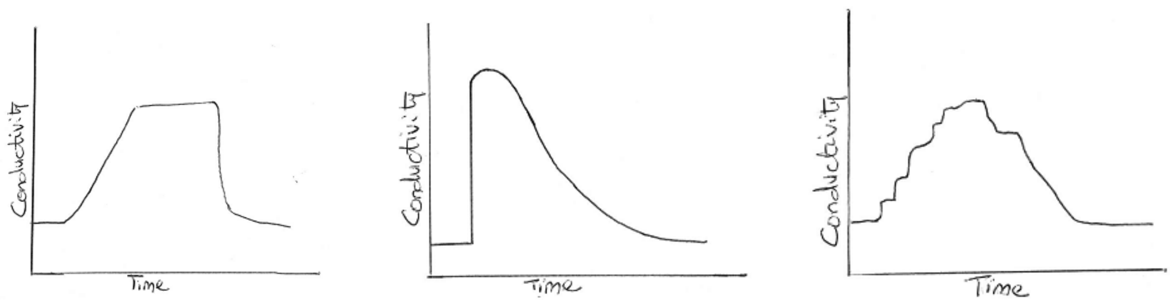


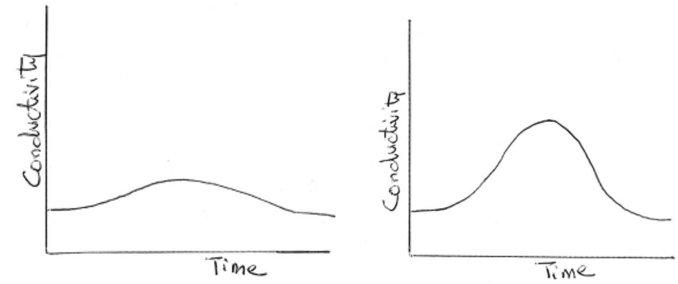
Figure 2.10: Dilution graph



a. Meter saturated, change scale or use less salt

b. Badly skewed, use longer distance

c. Uneven, salt not mixed. Use longer distance



d) Insufficient response. Use more salte. Perfect response

Figure 2.11: Dilution graphs and interpretation

In a short time interval *dt*, the average conductivity of water while the cloud passes is measured by the conductivity meter in μ (ohm x10^{-6}), which can be converted to an average salt concentration by multiplying with the conversion factor k. In the same time period *dt*, the volume of water passing is Q *dt* (m^3/s x s). The average mass of salt passing in *dt* is therefore μkQ*dt*. All these masses added together must be equal to the mass of salt (Msalt) originally placed in the stream.

$$\therefore M_{salt} = \sum \mu\, k\, Q\, dt$$

Because M is the only factor varying with time, the above equation can be expressed as

$$M_{salt} = k\, Q \sum \mu dt$$

$$\text{or } Q = M_{salt} / k \int \mu dt$$

Where the integral is evaluated over the whole period during which the cloud passes. The conversion factor varies with temperature, so it must be based on a temperate measurement during the site visit.

- The conductivity meter reading over time (abscissa) is plotted. The area under the curve represents the value of ∫µdt and is denoted by A.
- A is given in units of (conductivity x time), i.e. [ohm $^{-1}$ x 10 $^{-6}$ x s].
- The reciprocal of conversion factor k^{-1} is obtained from the graph of the conductivity per concentration of salt v/s temperature.
- The unit of $k^{-1}$is [ohm^{-1} x 10 $^{-6}$]/ (milligrams/litre)
- Then Q = M k^{-1} / A litres per second.

WEIR METHOD (STAGE CONTROL METHOD)

The flow measurement weir is a weir containing a notch through which all the water in the stream is made to flow or pass. The flow rate can be determined from the difference in height between the upstream water level and the bottom of the notch. For reliable results, the crest of the weir must be sharp and sediment must be prevented from accumulating behind the weir.

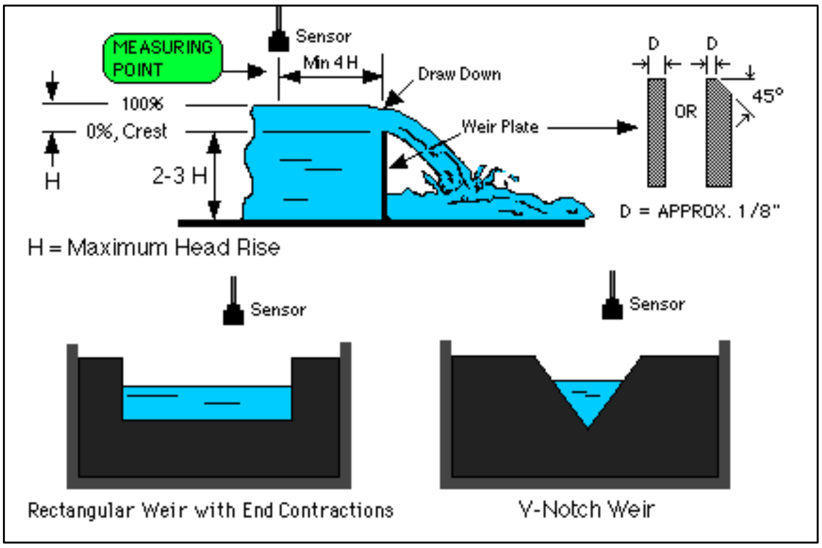


Figure 2.12: Flow measurement weir for two different notches

Weirs can be made of concrete, metal or even timber and must always be oriented at right angles to the flow. The weir should be located at a point where the stream is straight and free from eddies. Upstream, the distance between the point of measurement and the crest of the weir should be at least twice the maximum head to be measured. There should be no obstructions to the flow near the notch and the weir must be perfectly sealed against leakage.

For larger flows, a rectangular notch is suitable while a triangular notch (90°) is suitable for small flows with wide variations. The notch of weirs must be wide enough to accommodate the largest expected discharge. For reliable results, the notch must have sharp crests. The measurement method to determine the head *h* is very simple. A stake is driven into stream bed at a distance of at least four times the head, on the upstream so that the top of the stake is exactly at the crest level of the notch. The water level depth above this stake yields the head. The head is measured by keeping a scale or a levelling stave on top of the stake and the water levels are measured for different periods.

Then the discharge is calculated by:

For a rectangular notch the flow formula $Q = 1.8 \times [L - 0.2h] \times h^{3/2}$

and for a 90° triangular notch $Q = 1.4 \times h^{5/2}$

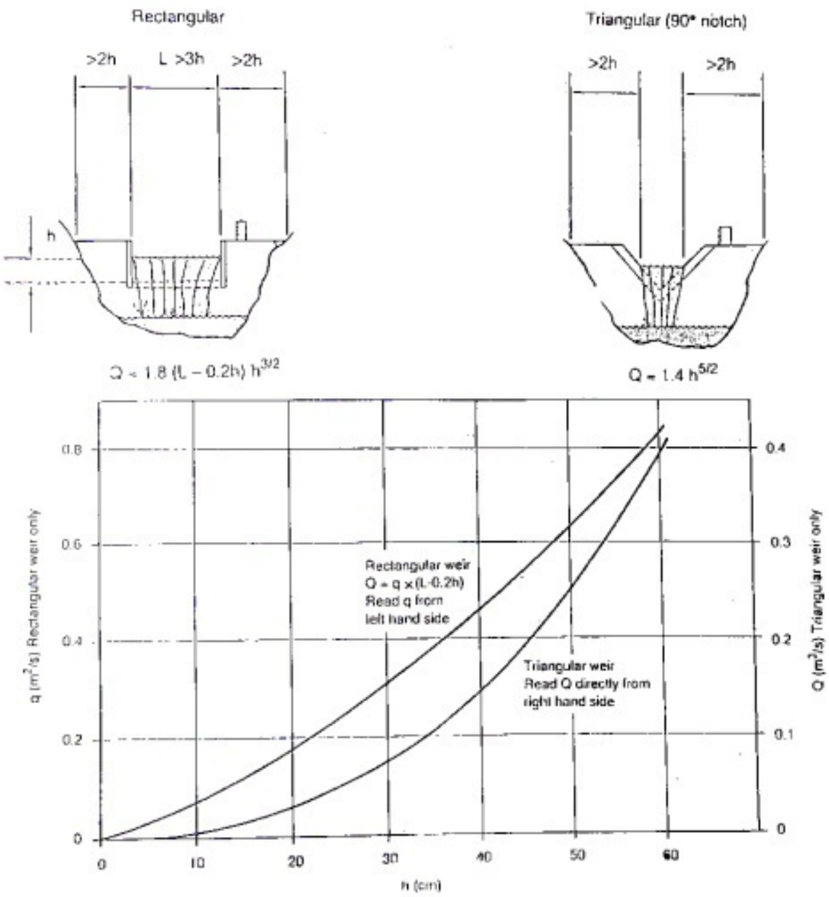


Figure 2.13: Typical measurement weirs

» ***Flow characteristics***

Stream flow data can additionally be represented using various diagrams, further analysis and studies. These possibilities include:

HYDROGRAPH

This graph creates a discharge-time curve of the flow. The discharge is plotted on the Y-axis. The time of the respective measurement is plotted on the X-axis in days, months, years or hours. Figure 2.14 shows the hydrographs for two streams. From the flow characteristics, it may be observed that the flow in summer is very low and it increases significantly during monsoon season.

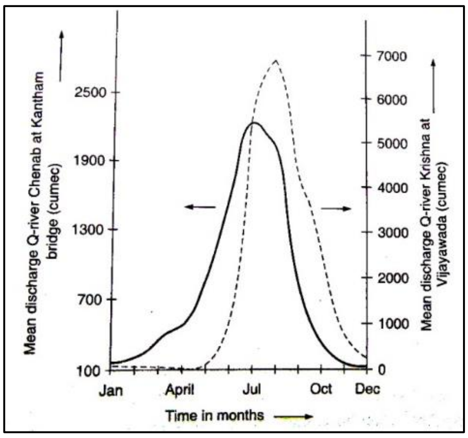


Figure 2.14: Hydrograph

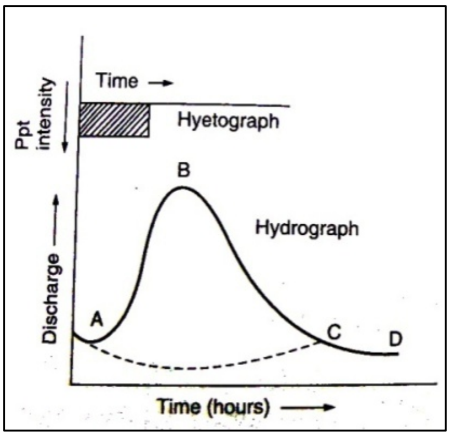


Figure 2.15 Hydrograph after a storm

The hyetograph shows a storm starting at zero hours and lasting for a few hours. Its peak value is reached at B and the subsequent flood gradually recedes. The dotted line AC indicates the division between direct run-off and base flow. The area below the curve ABCD gives the total flood volume.

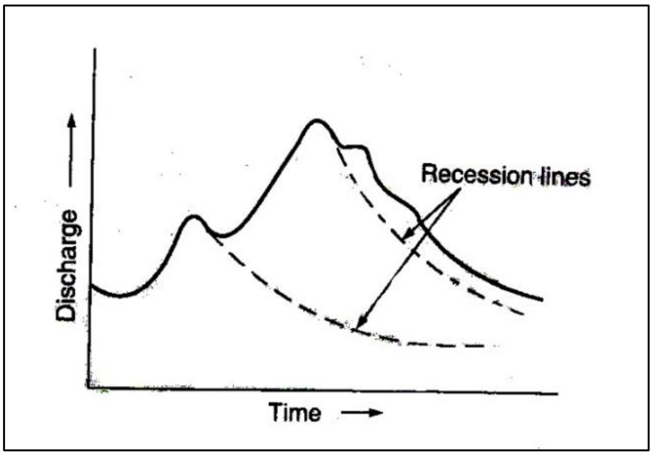


Figure 2.16 Hydrograph with kinks

Actual recorded floods for rivers may not necessary be indicated by single-peak curves, but instead show multiple peaks and kinks. These shapes indicate complexities of the storm or catchment peculiarities. A single-peak hydrograph is generally preferred for flood analysis. Complex hydrographs can be resolved into simpler depictions by drawing hypothetical recession lines.

FLOW DURATION CURVE

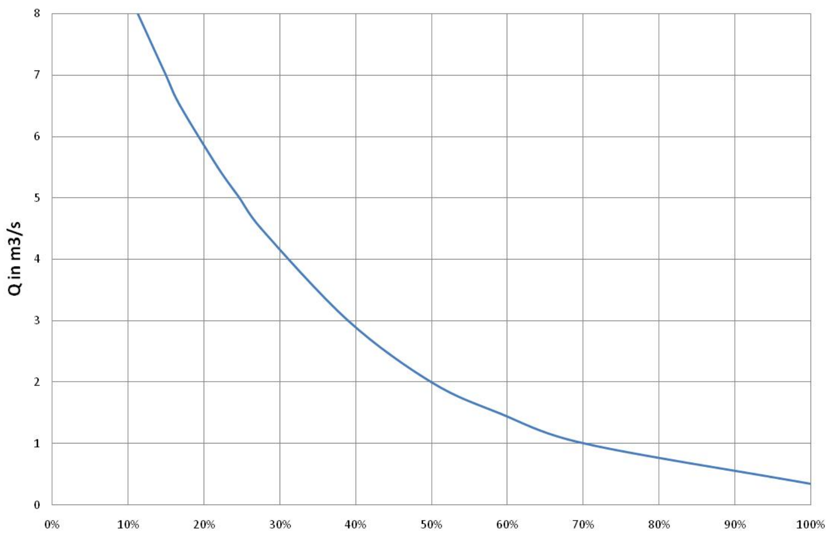
This is the most useful way of organizing discharge data by plotting a flow duration curve (FDC) that shows for a particular point on a river the proportion of time during which the discharge there equals or exceeds certain values. It can be obtained from the hydrograph by organising the data by magnitude instead of chronologically. If the individual daily flows for one year are organised in categories, a graph such as presented below will be obtained, which represents the ordinates arranged in order of magnitude instead of chronologically.

Nowadays, when most gauging stations are computerised, the easiest way to derive a FDC is to transpose the digital data onto a spreadsheet, sort it in descending order, and classify the data as in the below table, either manually or by using a simple macro. Once it is complete, the same spreadsheet can be used to draw the FDC using its graph-building functions.

Flow duration curves are used in assessing the dependability of the discharge. In other words, from the below plot, a discharge of 1.0 cubic metre is available for about 70% of the time. Then it is said that a 1.0 cubic metre per second flow has a 70% dependability.

Table 2.1: Flow duration curve summary table and plot: Percent of time discharge is equalled or exceeded

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



When designing a hydropower scheme, normally a 90% dependable flow rate is considered. For the above example, this would equal 0.5 cubic metres per second.

Curve accuracy can be improved when the flow duration curve is plotted using average daily discharges. Figure 2.17 indicates the flow duration curve with the daily and monthly average discharges for the same observation set. This diagram allows us to observe that an error rate of 10 to 15% can occur when estimating minimum and maximum flow rates. This should be kept in mind when working with flow duration curves.

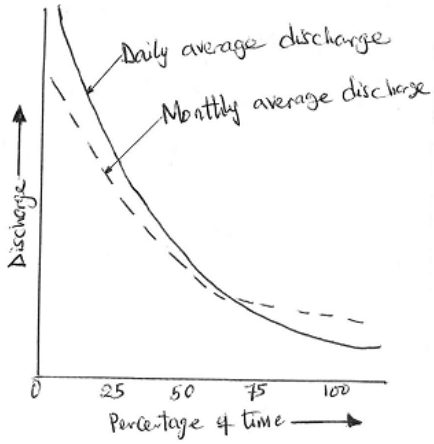


Figure 2.17: Daily versus monthly FDC

Flow duration curves are plotted using average monthly flow values. A better approach is to use the entire recorded data and to summarise it in a single curve. Two commonly used methods:

- Total period method: This method uses all available data to draw the flow duration curve. If a ten-year record is available, it would yield 120 average monthly flow values. These values are first tabulated in ascending order, starting from the driest and ending with the wettest month in the entire period. The resulting curve would be drawn using these 120 values.

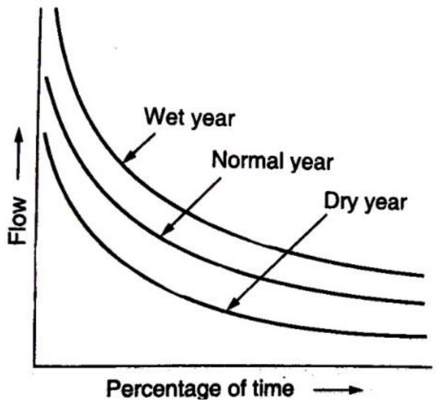


Figure 2.18: Long-term FDC

- Calendar year method: In the calendar year method, the average monthly values for a year are arranged in ascending order. Then the average flow values, corresponding with the driest month, second driest month, and so on up to the wettest month, are derived using the arithmetic mean of all values of the same rank. These averages are then applied to plot the final flow duration curve consisting of 12 points.

It should be noted that greater accuracy is obtained with the total period method.

» *Rainfall run-off relation*

The observed stream flow data can be analysed by hydrographs, flow duration curves, etc. A correlation between the rainfall and the corresponding run-off must also be established. There are various factors that influence the run-off rate:

PRECIPITATION CHARACTERISTICS

These include the type or nature of storm, its intensity, extent and duration. Variations in storm intensity with respect to area as well as time are also important. Greater intensity produces a greater run-off for the same total precipitation volume. Storms producing mild rains over longer periods produce a relatively small run-off. If two storms follow in close succession, the second storm produces a relatively greater run-off because water loss due to infiltration is markedly reduced. This saturated soil state is known as antecedent precipitation condition. Snowstorms only produce run-off with melting snow. This run-off has different characteristics.

METEOROLOGICAL CHARACTERISTICS

Meteorological factors such as temperature, humidity, wind and pressure variation produce significant effects on run-off. Higher temperatures and wind velocities increase evaporation while humidity decreases evaporation. Transpiration also increases with these same factors. The pressure distribution in the atmosphere helps the movement of storms. If the storm follows the stream flow, the run-off rate will be greater. Otherwise, less run-off will result.

CATCHMENT CHARACTERISTICS

Catchment characteristics include the catchment size, shape, surface, orientation, altitude, topography and geology. Larger catchments result in greater run-off. Catchment size does not correlate with the run-off in a linear manner. Storms are seldom effective over the entire area of larger catchments. Only a part of catchment may therefore be affecting the flow at a given time.

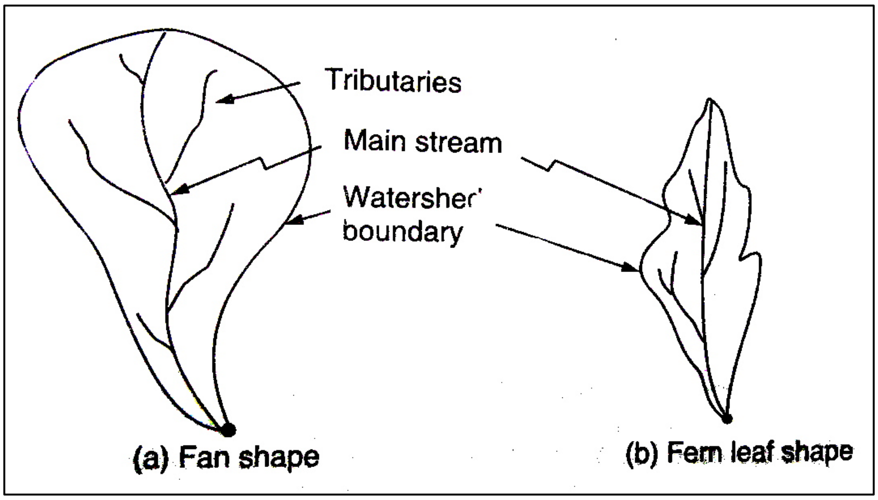


Figure 2.19: Catchment area types

Catchment shape also has an effect on the run-off. A fan-shaped catchment (Figure 2.19(a)) produces more intense flooding than a fern-shaped catchment (Figure 2.19(b)). In the fan-shaped catchment, the tributaries are all roughly the same length and the run-off reaches its various end points almost simultaneously. With the fern-shaped catchment, the reverse is the case.

In addition to the basin shape and storage characteristics, the shape of the hydrograph for any basin depends on different variables, for instance how long it takes water to move through the basin. The travel time depends on the average flow velocity but is presumed to be roughly proportional to the distance from the outflow point. The time required for travel from the farthest point of the basin up to the outflow point is referred to as the time of concentration. This is a significant variable. Storms that last longer than the time of concentration will be able to produce run-off from the entire catchment area. Major flooding would be the result.

The catchment surface has substantial effect on run-off. A bare surface produces more run-off whereas a cultivated surface with substantial vegetation will produce less run-off. Paved surfaces, as in cities, produce greater run-off.

Catchment surfaces located on the orographic or windward side of mountains receive greater precipitation and hence greater run-off. Surfaces located on the lee (sheltered) side, on the other hand, receive less precipitation and less run-off. High-altitude catchments receive greater precipitation and greater run-off.

Topographically steep catchments produce greater run-off because of quicker drainage, resulting in less opportunity for evaporation and absorption. The permeability of the geological strata controls the abstraction rate and the run-off. An unfissured rock surface converts almost all precipitation into run-off.

STORAGE CHARACTERISTICS

Of the total precipitation over a catchment area, a portion is stored within the area and does not readily appear as run-off. It is released later in the form of delayed run-off. There may be a number of depressions, lakes, and pools, etc., which detain the precipitation either temporarily or permanently. This is known as the depression stage. The water depths in the channel network in the catchment increase with increasing flow, as such some water is automatically stored in the additional channel volume. This is known as channel storage. Taken as a combined unit, these two characteristics are referred to as valley storage. The magnitude of flood and run-off depends on the characteristics of valley storage.

» *Flood run-off*

A standard project flood (SPF) is derived from standard project storm using the unit hydrograph. This diagram, in turn, is used to obtain the probable maximum flood (PMF) from the probable maximum precipitation (PMP). If the flood hydrograph can be developed, all information such as flood volume, its return period, rise and fall of its levels, etc., is readily accessible.

EVALUATION OF GROSS HEAD

There are several methods to measure the available head. Some are more suitable for low-head sites, but too time-consuming, and even prone to error, when used at high-head sites. In either case, several separate head measurements should be taken at each site.

Another key factor is that the gross head varies with the riverflow and does not remain constant. The tailwater often rises faster than the headwater as riverflow increases, a phenomenon which reduces the total available head. This variation in head is much less than the variation in flow; it can, however, have a sizeable impact on available power. This is particularly applicable for low-head schemes where even 50 centimetres is critical. For an accurate assessment of the available gross head, measurements of headwater and tailwater levels must be taken to reflect the full range of river flows. Some of the more common methods/techniques used for the measurement of head:

WATER-FILLED TUBE

The method is useful for low head sites, since it is cheap and quite accurate. With this method, two or three separate readings must be taken to make sure that final results are correct and reliable. In addition, the results should be cross-checked against measurements made by another method, for instance the water-filled tube and pressure gauge method.

WATER-FILLED TUBE AND PRESSURE GAUGE

This is probably the best of the simple methods available but has some limitations. The two sources of error are out-of-calibration gauges and bubbles in the hose. To avoid the first error, the gauge should be caliberated before and after each major site survey. To avoid the second, a clear plastic tube allowing to see bubbles should be used.

DIGITAL INSTRUMENTS

Nowadays, with digital theodolites, electronic digital and laser levels, and especially with the electronic total stations, the job of measurement has been simplified.

Modern electronic digital levels provide an automatic display of height and distance within about 4 seconds with a height measurement accuracy of 0.4 mm, and an internal memory that can store approximately 2,400 data points.

A total station or TST (total station theodolite) is an electronic theodolite integrated with an electronic distance meter (EDM) to read gradient distances from the instrument to a particular point. Angles and distances are measured around the total station to points under survey, and the coordinates (X, Y, and Z or easting, northing and elevation) of surveyed points relative to the total station position are calculated using trigonometry and triangulation.

DUMPY LEVELS AND THEODOLITES

The dumpy level (or builder's level) is the conventional instrument for measuring head and should be used whenever possible. It should be noted that these units must be operated by seasoned experts and they require precise calibration. Dumpy levels are combined with staffs for head measurements taken in a series of stages. The device allows the operator to take readings visually on a graduated staff held by a colleague, while ensuring that the line of sight is exactly horizontal. Stages are usually limited by the length of the staff to a height change (usually of no more than 3 metres). This method requires an unobstructed view; ample vegetation at a location makes it difficult to apply this method. Dumpy levels only allow a horizontal sight reading. Theodolites, on the other hand, can also measure vertical and horizontal angles, providing enhanced versatility and allowing faster work.

Figure 2.20: An engineer taking readings from a dumpy level

SIGHTING METERS

Hand-held sighting meters (also called inclinometers or Abney levels) allow the user to measure the angle of inclination of a slope. They are compact and small, and may include a range finder to eliminate the problem of measuring linear distance. The error in estimation is typically between 2 and 10% depending on how skilfully the meter is applied.

MAPS

Large-scale maps are useful for approximate head values, but are not always available or completely reliable. For high-head sites (>100 metres), 1:50,000 maps are useful for pre-feasibility studies and are generally available.

ALTIMETERS

Altimeters are quite useful for high-head pre-feasibility studies. Surveying altimeters generally give errors in a range of as low as 3% in 100 metres. Atmospheric pressure variations need to be allowed for, and however, this method cannot be generally recommended except for approximate readings (pre-feasibility studies).

HAND-HELD GPS DEVICE

This is a very useful tool for site visits. It can help to locate and mark the major components of the scheme. The data can be then easily laid over a map and compiled. Elevation readings associated with this device are not accurate. They are usually in the range of ±7 metres.

2.2. GEOLOGY

When visiting a proposed hydropower site, a geological survey should be conducted. After this visit, the surveyor should aim to return home with some information on these aspects:

- Future surface movements. A site may have loose rock slopes, for example, that can be disturbed by construction work or heavy rain. Other indicators include dry mud, which points to mud flows, storm gullies that may take torrents and rock flows during heavy rainfall, clues of flood behaviour at valley base level.
- Future sub-surface movements. This includes factors such as landslip and subsidence.
- Rock and soil types. To design the foundations of civil works, information is required to select materials for use in channel construction, and to assess which building materials are available on site.

Just as in the hydrological study, the on-site survey must be preceded by careful use of a map, if one is available. Try also to obtain aerial photographs of the proposed micro hydro area. Use the map and photographs to sketch out the basic geological characteristics of the area. Sketches should be brought along during site visits and added to observations. Figure 2.21 contains an example sketch. Make sure to survey slopes high above the proposed hydro installation, since activities such as landslides far above the structure are likely to have knock-on effects causing geological activity near the installations. Also, extend your survey to include the land below, as ground movements there have repercussions on overlying civil works.

Construction work required for the hydro project may very well trigger disturbances, which activate movements on and below the surface. Over time, the installation could cause a disturbance; water may leak from channels and tanks into the hillside below, for example. Appropriate measures should be taken to prevent these effects.

The geological survey mainly aims to assess the best locations for proposed civil works and to estimate the costs of their construction costs and future maintenance. Useful items for a geological survey include:

- Notebook, pencil, squared paper, calculator, reference tables, sketches from map
- Measuring tape, compass, map
- Hammer (to inspect rock for weathering properties)
- Pickaxe/spade for inspection trenches
- Measuring cylinder to settle out clay, loam, sand constituents of soil
- Measuring kit for soil permeability
- Equipment to measure head and flow

Local knowledge is an essential information source. After touring the area, speak to long-term residents. Then follow up what they have related by returning to the site and checking their estimations of flood levels, size of landslides, and so on. See how these indicators match up against your initial measurements and assessments.

Figure 2.21 shows a river where water is drawn at an intake site. This figure could serve as a sketch for a proposed hydro installation. It contains contours along with the proposed channel to convey water to a tank (i.e. the forebay) at the top of a delivery pipe (i.e. the penstock).

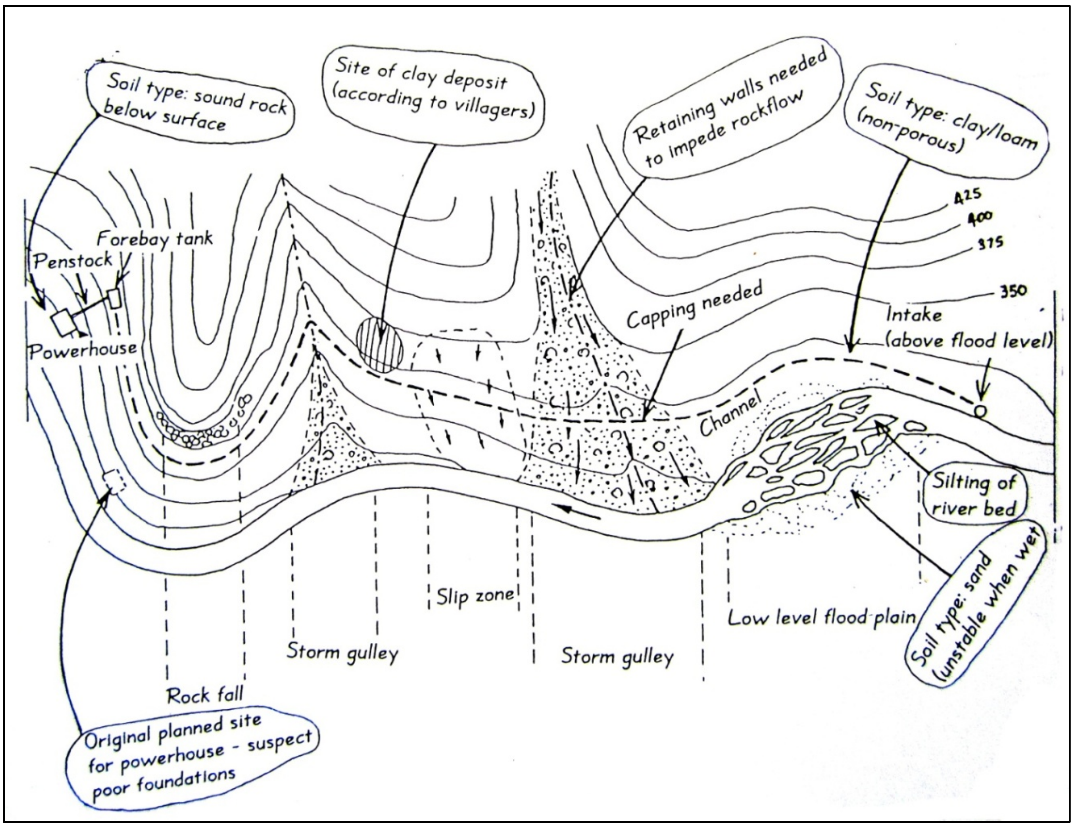


Figure 2.21: Example of a site visit sketch

LOOSE SLOPES

Loose slopes are usually plain to see, often indicated by debris collecting at their base. These slopes must be stabilised above and below installations to protect civil works. Introducing vegetation (plants or seedlings) such as local grasses and bushes has proven to be an ideal method for stabilization.

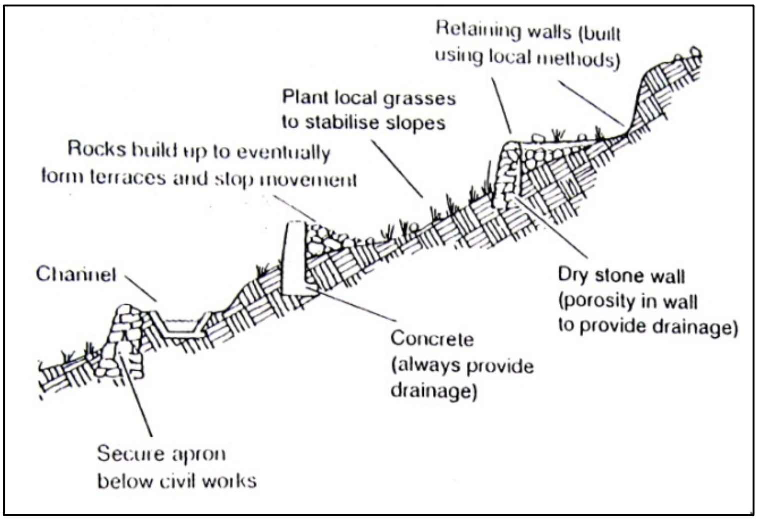


Figure 2.22: Retaining walls and terracing

Dense areas of nearby vegetation may be thinned out and transplanted to a loose slope. This is always required on slopes where construction work has stripped away all vegetation and on soils and rubbles, which have been moved. This is known as *made-up ground*, which should never be used as the foundation for civil works.

Terraces or retaining walls present additional stabilisation methods, as can be seen in Figure 2.22. Water drainage must be included for all terrace constructions to avoid their collapse. Gentle traverse slopes can be integrated in terraces as a protective measure for the penstock and powerhouse in order to prevent erosion and surface sliding.

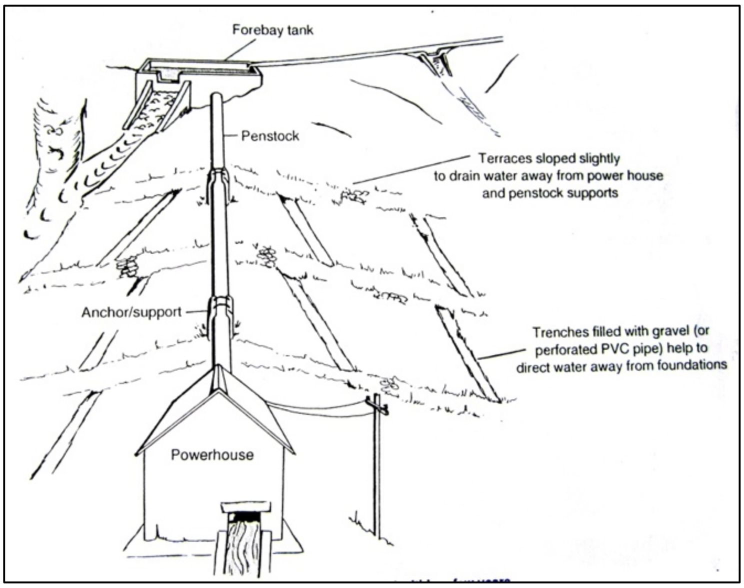


Figure 2.23: Traverse sloping

The slopes below the civil works are as important as the slopes above. The same techniques of grass-planting and terracing must be applied to stabilise the lower slopes. This is because movement below can cause collapse above.

A secure retaining wall or *apron* can be built immediately below any civil construction if there is reason to suspect lower slope instability and planting/terracing methods are not helpful.

STORM GULLIES

Water torrents may result on hillsides after unusually heavy rain or snow. Upland lakes may also overtop, resulting in a sudden release. The water will frequently erode gashes or gullies in the hillside. These formations are easily detectable. The torrent usually sweeps away loose rocks and boulders (boulders become lighter in water and are easily moved). The torrent can therefore destroy virtually anything in its path. Many poorly situated micro hydro installations have fallen victim to torrents. This danger can be spotted early on by looking for rock debris that has accumulated at the foot of dry streambeds and eroded sections of the hillside.

When surveying, make sure to identify small gullies that might appear insignificant at first glance, but nevertheless, become forceful torrents in a bad ten-year storm. The presence of vegetation on these gullies does not mean they are harmless. While vegetation can spring up within two years, the gulley may only release a torrent every ten years. Loose rock deposits will often be located along the gulley bed in this case.

Civil works should not be located in or below a gulley. Whenever a traverse structure to cross a channel cannot be unavoided, engineers must take precautions as shown in Figure 2.24. Always avoid attempting to conduct water in a small gulley underneath a channel in a culvert. Instead, direct the water channel across a bridge *or channel crossing* (see Figure 2.25).

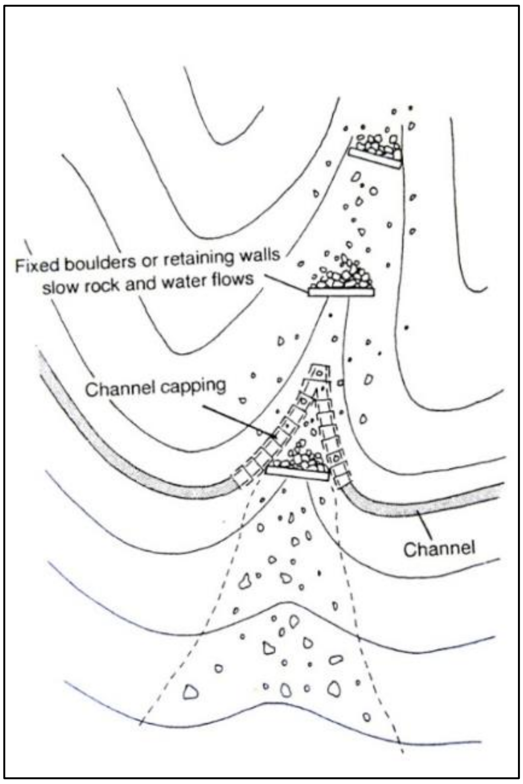
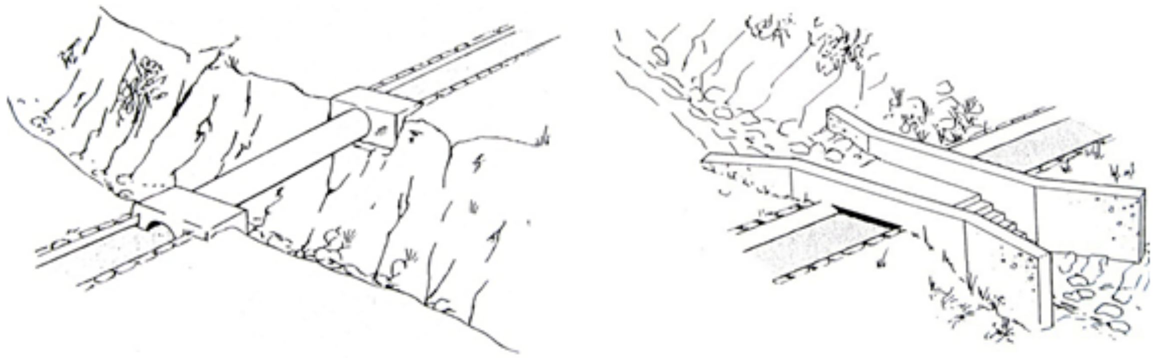


Figure 2.24: Storm gulley



a. Pipe bridge or aqueduct crossing a ten-year storm gulley. The bridge must be located above potential rock flow.

b. Storm gulley directed over the channel. This is often safer than running a culvert below the channel, since small culverts are easily blocked in a storm

Figure 2.25: Bridge and storm gulley

FLOOD PLAINS

Low-level flood plains are frequently shown by a contour-free area on maps. Surveyors will be able to spot on-site indications of periodic flooding, and long-term residents will be able to provide information on floods. Typical flooding signs include ponds, low ground near the river, debris on the river banks or trapped in bushes that is suspected to be waterborne, secondary watercourses (either wet or dry), deposits consisting of sand gravel and dried mud (the latter often with shrinkage cracks). Flood plains pose various risks, which include sediment build-up, potential changes to river paths, and sandy ground, which can be unsuitable for placing foundations.

Always construct powerhouses and intakes above flood plain levels. Wing walls can sometimes be employed to protect intakes from flooding.

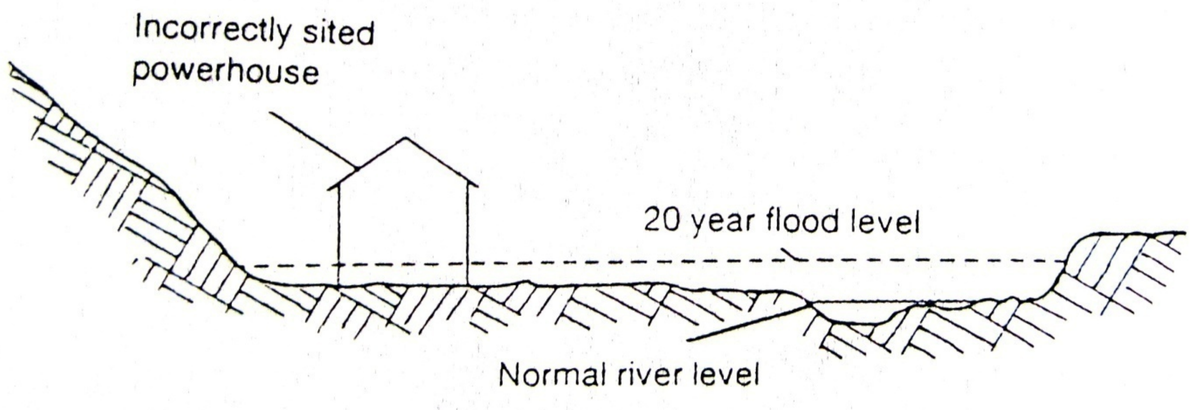


Figure 2.26: Always build the powerhouse above the 20-year flood level, which should be determined by speaking to local residents and searching for signs of flood debris (gravel, dry mud, flotsam, sand)

FUTURE SUB-SURFACE MOVEMENTS

» ***Landslip***

The geological sketch can be used to represent a landslip zone. Viewed from the side, this same effect can be seen in Figure 2.27. Surveyors can detect slip zones by carefully scanning an area for sudden steps, or cracks, in the hillside. Steps are located found where a fault line comes into contact with the surface. Even steps, which have acquired vegetation and become rounded may indicate fault lines which have the potential to be reactivated. At times, a fault can also be indicated by a sudden dip or rise in a path, or a broken section of wall. A further indication is an oddly shaped tree trunk. Water seepage from cracks on the slope can indicate the lower exposed end of a fault line.

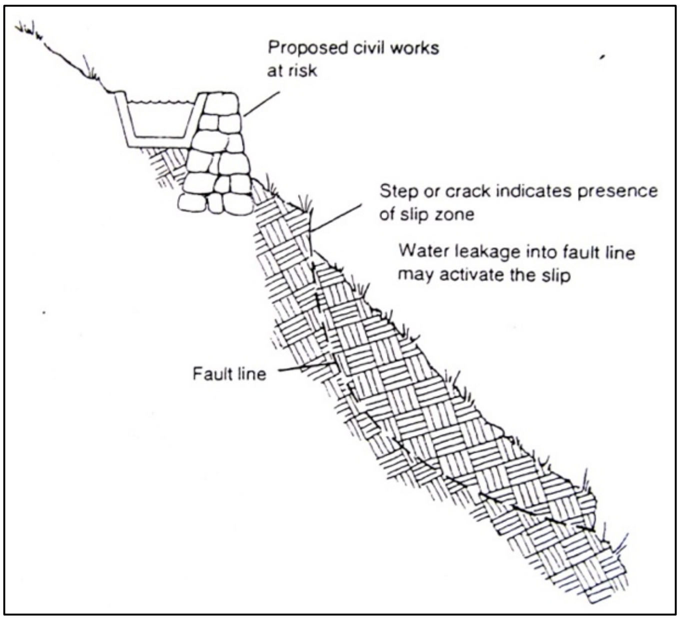


Figure 2.27: Landslip or *rotation*

Disturbances caused by construction may trigger slip, which will eventually lead to a collapse when occurring below a civil construction. Leaks from a channel, forebay, or spillway may also enter and activate the fault line. Careful direction of channel and spillway drains is therefore very important.

In addition to the considerations outlined above, it is important to study the larger area surrounding the site. Landslip occurrences in some distance away on similar ground indicate that danger is nearer at hand. These cases of slip may have occurred because of a disturbance, for instance the stripping of trees and grasses for cultivation purposes. Similar activity may occur near the hydro installation in the future, in which case precautions should be taken.

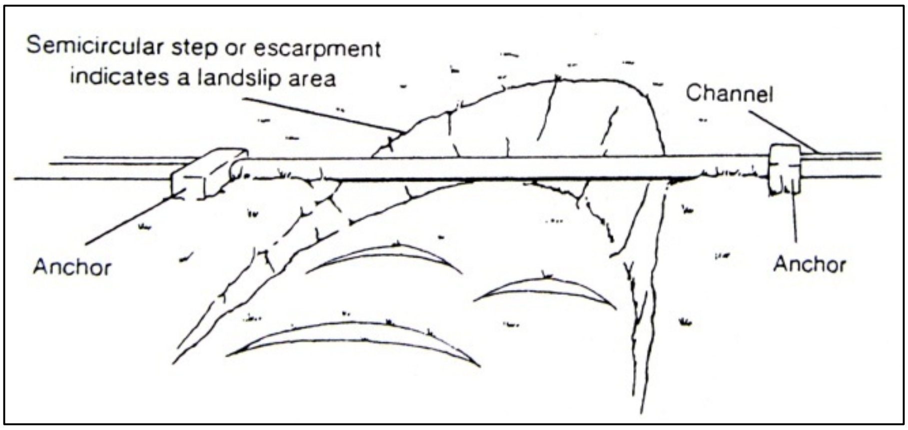


Figure 2.28: Penstock traversing a slip zone

The general rule is to avoid the location of works near landslip faults. In the rare case where a channel is to be traversed in a suspected slip zone, a pipe or an open aqueduct can be used, but only if the traverse is short (see Figure 2.28). The traverse length is limited by the length of pipe or aqueduct that can support its own weight and the weight of water inside it. For longer lengths, a pipe bridge must be built, or catenary wires (secured to anchors) used to support the pipe. In the latter case, anchors must be placed outside the slip zone area to stabilise the construction. If a conventional channel is built across a suspected zone, the essential precaution is to provide ample culverts and bridges in order to ensure that water descending the slopes does not build up behind the channel walls.

» *Layer faulting*

Severe instability of slopes can also result from a second type of sub-surface fault pattern. This is referred to as *layer faulting* and consists of parallel fault layers that slope downward with the slope. This phenomenon can be identified where debris builds up at the foot of the slope. Layers of sub-surface material may also be exposed on the hillside. Previously described measures to correct movement of ground surface (retaining walls, vegetation, terracing) may be sufficient to stabilise the slope. More expensive methods must otherwise be used. Gabions are one frequently used method. Contractors for irrigation or road building can advise on the use of gabions to stabilise slopes above and below proposed works. Gabions can normally be constructed on site using a wire mesh to enclose rock rubble to form a large stable mass. Galvanised steel wire measuring 2.5 to 3.0 millimetres in diameter and formed in hexagonal sections are used to form the mesh (for anti-corrosion properties ranging from 15 to 20 years). The use of gabions is a highly cost-effective method for building retaining walls and weirs at the river intake site, and should be considered seriously for the stabilisation of slopes and to protect the powerhouse from riverbank erosion. For extreme layer faulting, the slippage layers can be held together using pins. These pins must be long (e.g. 5 metres), made from stainless steel, and inserted by a qualified specialist.

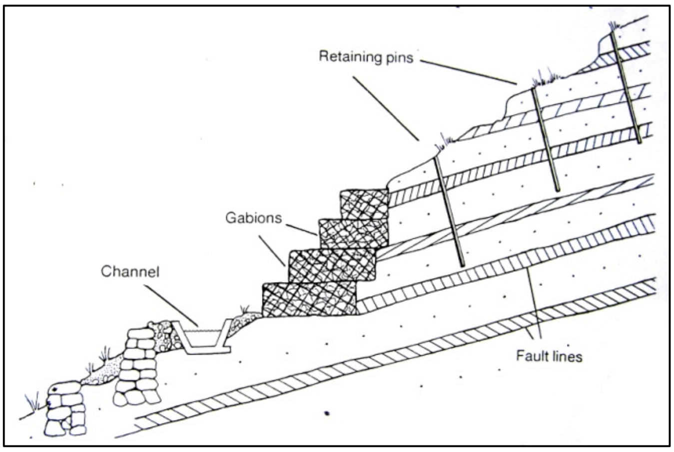


Figure 2.29: Cross-section of slope with measures to mitigate against faults

Instability is sometimes caused by fault lines sloping in the direction of the hillside. As the rock weathers, it loses cohesiveness and crumbles away. Slippage along the fault lines contributes to surface movement. Attempts to stabilise such a slope are expensive and difficult. Gabions (wire cages enclosing a mass of stones) can be used to anchor the slope, as shown. Retaining pins are more expensive to purchase and install. Specialist advice should be sought in cases like these.

» *Subsidence*

Subsidence is a third type of sub-surface movement, which is caused by soluble rocks such as limestone in conjunction with chemical and water interactions below ground. This risk can often be spotted by the presence of limestone outcrops (yellowish-white rock frequently containing small holes). Dissolving rock and underground water can be indicated by the sudden appearance and disappearance of streams on the visible surface. Another clear sign is the presence of *sinkholes*. These holes are small, circular surface formations (with a diameter of 2 to 10 metres) that appear as cone-shaped depressions. At their deepest point, they can have a small point into which water trickles, or they may be dry. Sinkholes form when the underlying limestone dissolves until the soil above collapses.

Never place civil works such as penstock anchors in these types of areas; otherwise, there will be a risk of the collapse of foundations. This is a particular danger in cases where newly introduced watercourses, such as the spill from a forebay tank, are not carefully diverted as they may accelerate sub-surface subsidence activity. The presence of unusually acidic water in rivers or streams, the effluent perhaps of a mining operation upstream, presents an additional possible risk.

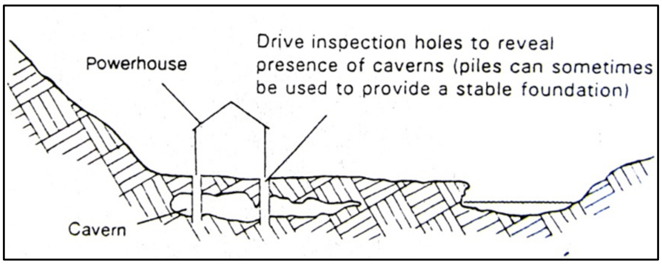


Figure 2.30: Subsidence

Sandy soils, or soils containing subsurface sand layers, pose dangers of movement underground. Additional mass above the sand, and seepage of water into these layers, can promote movement and then subsidence. Always inspect soil types below the surface and avoid constructing on sandy soils, or drive piles deep below the weak layers.

» *Soil and rock types*

Your survey and sketch of the proposed installation area should include careful notes as to soil and rock types. It is worthwhile to dig inspection trenches at various points around the surveyed area. The purpose of the analysis is to assess:

- Suitability of soil/rock for placement of foundations and likelihood of movement of foundations.
- Suitability of soils and rock for use as construction materials.

It is well known that foundations must be laid on firm soil or rock. Where these are not found at a reasonable depth from the surface, or found only partially, piles may be driven deeper underground until firm material is found. All parts of a foundation must be laid on an equally firm base, to avoid differential settling of the building. Table 2.2 lists techniques for identification of various soil types and their usefulness in construction and foundations.

Table 2.2: Rock and soil characteristics

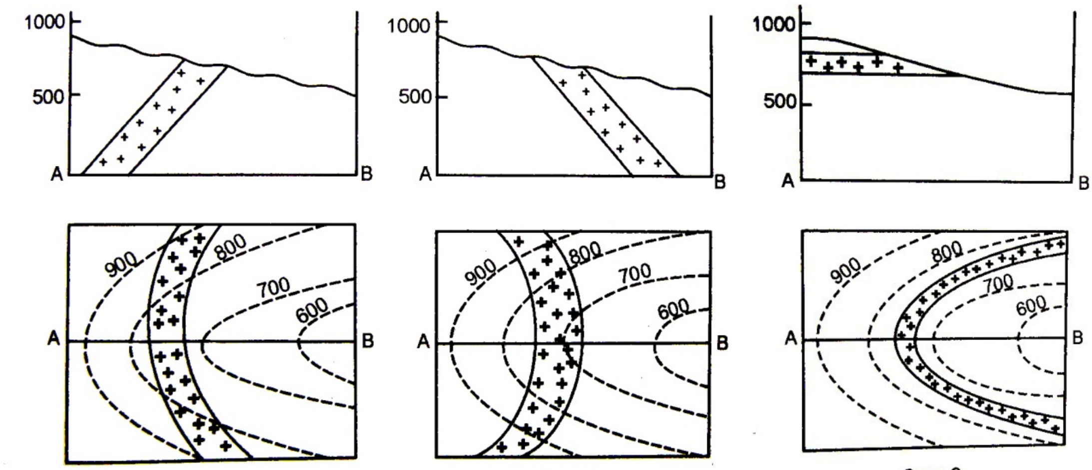
Code	*Identification*	*Usefulness*
Rock – Weathering and decomposition		
Solid	Withstands hammer blow. Bright clean. Cracks more than 30 cm apart	Good base for foundations. Boulders can be used as ballast in cement weirs and anchor blocks
Weak	Breaks under hammer. Discoloured. Cracks 5 cm to 30 cm apart	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
Soil – Firmness and cohesion properties when wet and when dry		
Gravel/ gravelly soil	Small to medium stones. Crunches under foot	Forms a good base for foundations when gravel content is very high. Gravel itself very useful as 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 soil	Tiny grains scarcely visible. Grates between teeth. Moulds slightly in the hand when moist but cracks easily. Take care to distinguish from clay or 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

Code	Identification	Usefulness
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 cultivating when oxygenated and limed

GEOLOGICAL STRUCTURES

» Outcrops

Exposed bedrock is called an outcrop. An outcrop may appear as a rounded knob out in a field, as a ledge forming a cliff or ridge, on the face of a stream cut, or along human-made roadcuts and excavations.



***Case 1*:** Dip occurs in the direction opposite to the valley slope: V points up the valley.

***Case 2*:** Dip in bed occurring in the same direction as the valley slope and at a greater angle: V points down the valley.

***Case 3*:** Horizontal beds or bed dipping in the same direction as the valley slope and at a smaller angle: V points up the valley and is longer than in *case 1*.

» Strike and dip

The angle between an imaginary horizontal line (the strike line) on the plane and the direction to true north is called the strike. A special compass is used to measure the strike.

The dip is the angle of the plane's slope. In more precise terms, it is the angle between a horizontal plane and the dip line (an imaginary line parallel to the steepest slope on the structure), as measured in a vertical plane perpendicular to the strike. The dip angle is measured using a clinometer, a type of protractor. A horizontal plane has a dip of 0°, and a vertical plane has a dip of 90°.

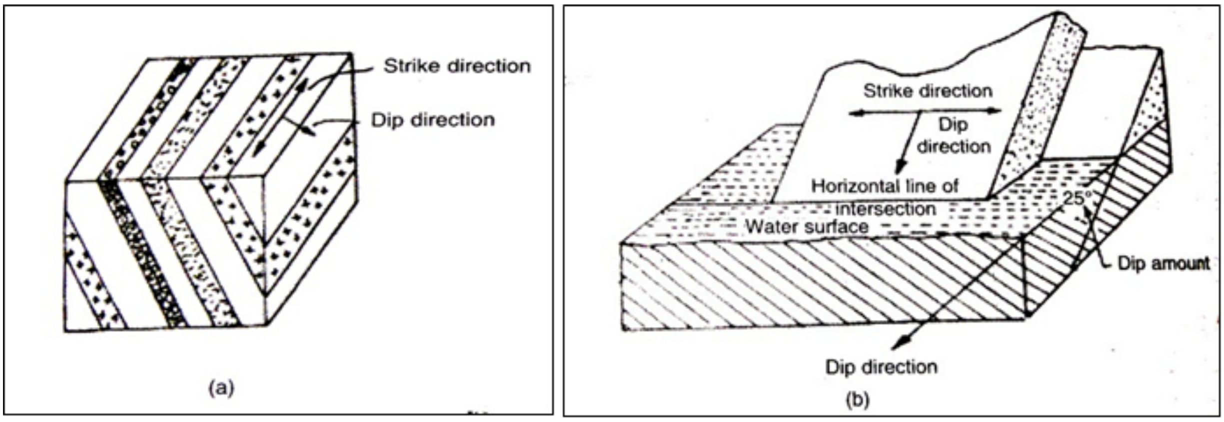


Figure 2.31: Strike and dip

» *Folds*

Picture a flap carpet on the floor. If you push on one end of the carpet, it will crease into a series of wave-like curves. In a similar way, stresses occurring during mountain building can warp or bend the bedding and foliation (or other planar features) in rock. The resulting curved shape in the rock layer is called a fold.

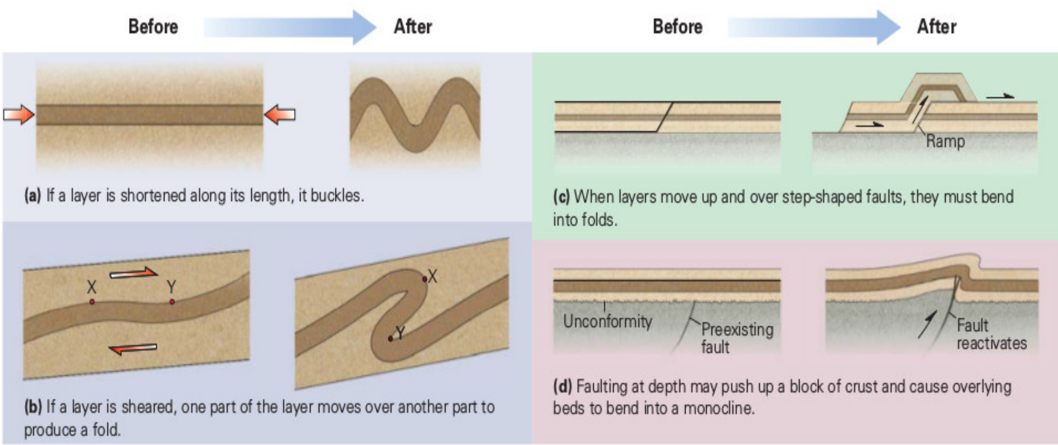


Figure 2.32: Causes of folding

Figure 2.33: Examples of folds

» *Faults*

Fractures on which sliding and slip events occur are called faults. Faulting can generate earthquakes. Like joints, faults are planar structures; their orientation can therefore be represented by strike and dip.

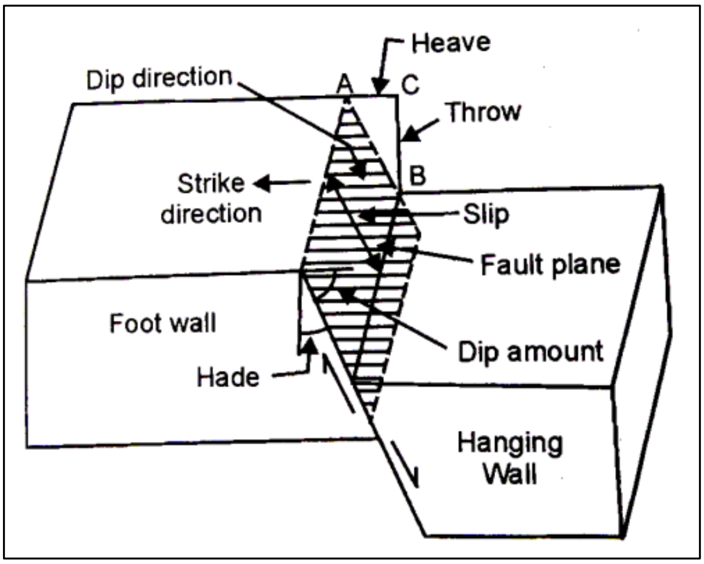


Figure 2.34: Fault

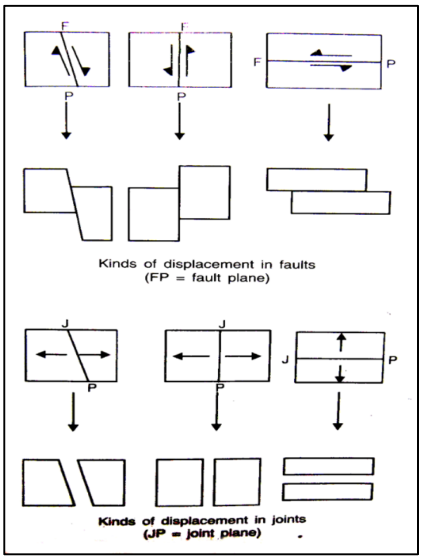


Figure 2.35: Movement in faults and joints

Earth's history includes numerous examples of fault formation and activity. Some faults are still active, since there is evidence of sliding in recent geologic time; most, however, are inactive, as sliding has ceased long ago. Some faults intersect the ground surface and their movement displaces the ground. Others remain invisible at the surface level unless later exposed by erosion. They accommodate the sliding of rocks in the crust, beneath the earth's surface.

» *Lithological evidence*

Lithological indicators of faulting include fault drags, brecciation, slickensides, mineralised areas, repetition, and strata omission and offset of beds.

SLICKENSIDES

Rocks along fault planes often bear marks of relative displacement in the form of parallel striations or narrow grooves on plain surfaces. They commonly exhibit a very polished surface. Frictional resistance caused by slow relative displacement results in this smooth appearance. Rocks exhibiting these characteristics are called *slickensides*. Slickensides appear scattered near and around the fault plane when erosion occurs. When spotted in a field, their presence therefore suggests a nearby fault.

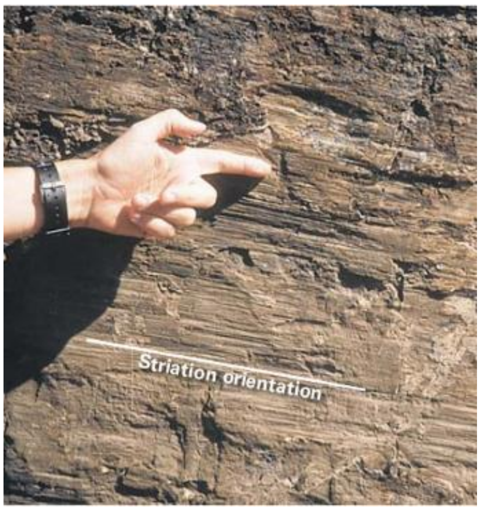


Figure 2.36: Slickensides

FAULT DRAG

Along the fault plane, depending upon the absolute movement involved, the adjacent beds may exhibit a type of bending which is referred to as fault drag. This formation is evidence of faulting.

BRECCIATION

When faulting results in numerous fractures or crushed rock that forms angular fragments, this phenomenon is called brecciation. When cementation introduces coherency to these fragments at a later date, they are called fault breccias. *Gouge* is the term used to refer to very fine crushed fragments. Fault breccias or gouge provides evidence of faulting.

Figure 2.37: Breccia

MINERAL-RICH ZONES:

Highly fractured rocks on either side of the fault plane are called shear zones. These zones are present when faulting extends over a larger area. Shear zones are the frequently the site of different types of economic or industrial minerals. Localised mineral-rich shear zones are an indication of faulting.

» ***Topographical evidence***

Topographical indicators of faulting include various surface features: fault scraps, parallel deflection of valleys, offset ridges, suddenly deviating streams, aligned occurrence of springs, drainage reversal, straight river courses (for considerable stretches), and straight steep coastal lands. All these topographical features, on closer observation, are the result of relative displacement that occurs during faulting and due to lithological factors.

Figure 2.38: Prominent vertical joints in red sandstone beds, Arches National Park, Utah

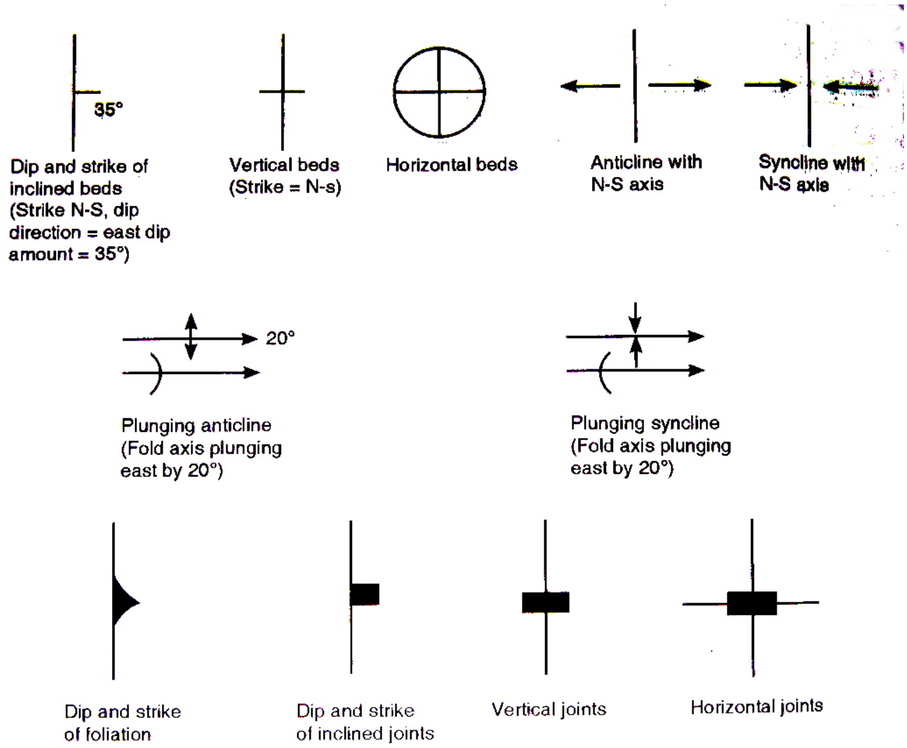
JOINTS

Joints are natural cracks, formed in response to tension in rock under brittle conditions. Veins develop when minerals precipitate as water passes through cracks.

The deposit of strata at a location does not necessarily occur as a continuous process. An interval in which no deposition (nondeposition) and/or erosion occurs is referred to as *unconformity*. Geologists recognise three types of unconformity: angular unconformity, nonconformity, and disconformity.

Figure 2.39: Example of unconformity. The layers above were deposited long after the beds below had tilted

» *Common symbols*



SLOPE STABILITY AND LANDSLIDES

» *Landslides*

Natural landslides are a normal phenomenon associated with the process of erosion and denudation. Slips often happen due to indiscriminate cutting during engineering operations. To prevent the occurrence of such artificial slips it is necessary to know the conditions under which slopes become unstable.

Landslides tend to take place where topographical and geological conditions collude to cause an unstable rock mass. Undercutting by streams, sideward erosion of banks, cuts for the excavation of roads, railways, canals, etc., make the unsupported mass unstable. The downward movement of the unstable mass due to gravity is aided by the presence of water. Earth tremors, blasting or heavy traffic may dislodge unstable masses and trigger sliding. Heavy rain is another cause. Many landslides have taken place due to the increased water content that results. Unconsolidated and semi-consolidated materials have a smaller angle of repose when wet than when dry. In such cases, the mass will be stable when the material is dry and once it is wet, it becomes unstable due the increase in water content. An important artificial cause responsible for bringing about waterlogging is the elevated water table around the reservoir created by the dam.

» *Slope stability*

In consolidated rocks, the stability of slopes will be determined by the frequency of divisional planes like joints, bedding planes, planes of schistosity, etc., and their disposition with respect to slope surface.

In general, the fewer the divisional planes, the more stable the slope. With a dip of divisional planes and slopes in opposite directions, the slope will be stable irrespective of the degree of the two. Where dip and slope are in the same direction, divisional planes with a dip steeper than the slope will not cause instability. Nevertheless, with dip and slope in the same direction, and the dip less than the slope, sliding will be possible along the divisional planes and the slope will become unstable (see Figure 2.40 right). However, even when in the same direction, dips and slopes measuring 15° or less may be considered safe.

To avoid slips during construction, all cuts and excavation in hillsides must be properly designed, taking into account the nature of the material concerned and its structural features. In consolidated rocks, the relationship between divisional planes and slope should be determined and slopes that will be unstable because of unfavourable relationships between the two should not be produced.

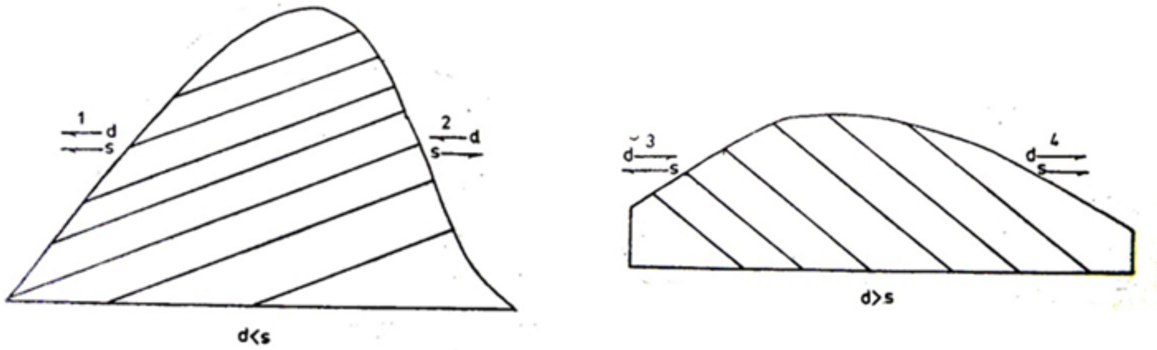


Figure 2.40: Stable (left) and unstable slopes (right)

GEO-STRUCTURAL FEATURES AT A DAM SITE

» Dip and strike

The bedded rocks are stronger in compression, and can bear greater stresses when applied normal to the bedding planes. Thus, the desired conditions are that the resultant thrust should be perpendicular to the bedding planes. The beds dipping gently upstream offer optimal resistance to the resultant thrust and also, obstruct the leakage of water better than those dipping downstream.

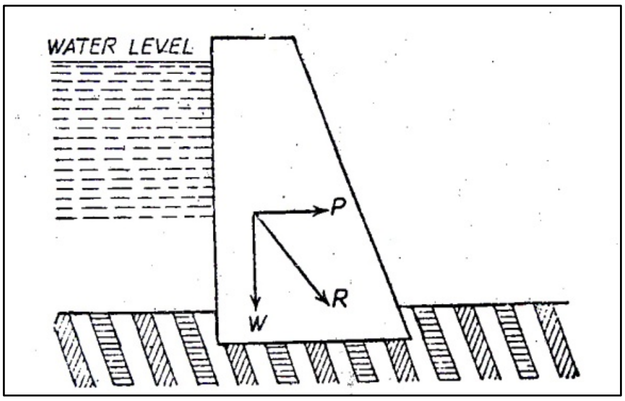


Figure 2.41: Rock bed dipping gently upstream

» Folds

The folded rocks are always under a considerable strain and the same is released whenever any kind of excavation is done through them or they are disturbed by external forces. It is therefore desirable that a highly folded rock should be always avoided.

If the engineer is compelled to adopt such a site, he/she should see that the foundations of the dam rest on the upstream limb of the fold; if the fold is anticlinal in nature. On the other hand, if the fold is syncline in nature, the foundations of the dam should rest on the downstream limbs of the folds to avoid leakage. It is observed that the danger of leakage is always greater beneath the strata in the case of a synclinal fold. **In no case should the foundation rest on the axis of the fold.**

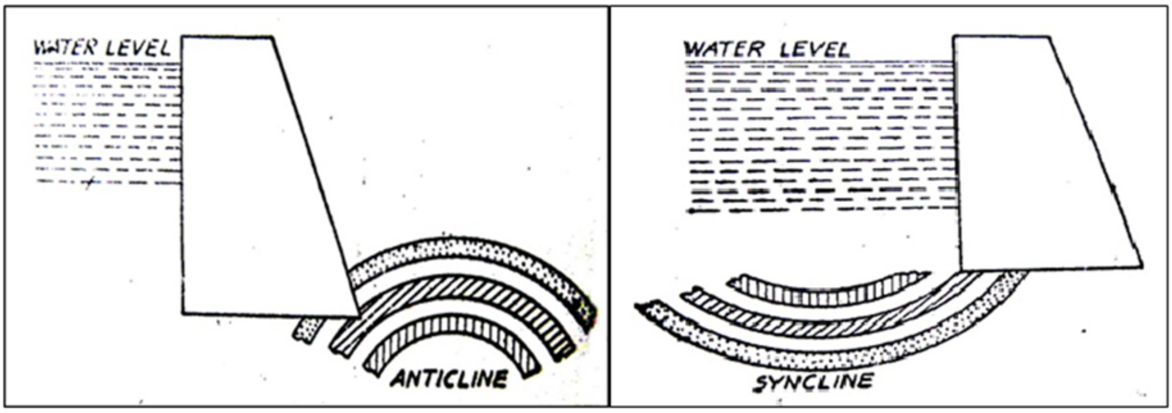


Figure 2.42: Locating a dam on a fold

» ***Fault***

It is always desirable to avoid a site on a fault, as the movement along the existing fault plane is much easier than along other planes. Even a slight disturbance may damage the structure constructed on a fault. In unavoidable circumstances, it is advisable to place the foundation of a dam upstream of the fault and not downstream of the fault.

The nature, extent and age of the fault should be thoroughly investigated. The different zones of a small fault can be improved effectively by grouting; whereas in the case of wider zones, all the weak material should be removed and refilled with rich cement concrete. This should be done if there is no possibility of movement along the fault plane.

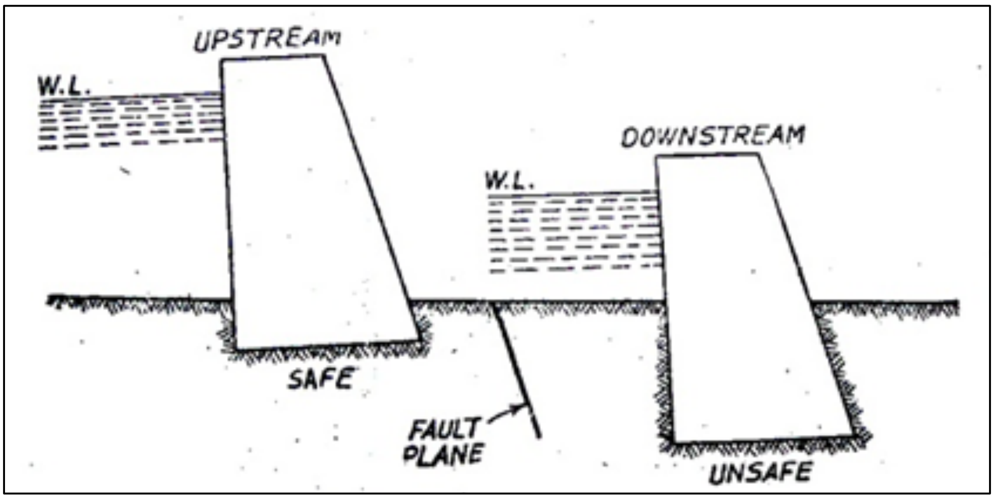


Figure 2.43: Locating a dam in an area of fault

» ***Joints***

A joint is the weakest point in a structure. Joints in underlying rocks at the dam site will result in the leakage of water. With the passage of time, this leakage may even endanger the structure. Such a condition will change from bad to worse if a part of the underlying rocks contains limestone, as the joints are enlarged by the dissolution of the rock.

If the joints are local in character or the area affected is small, grouting will improve the site. However, if the underlying rocks are heavily jointed, **the site should be rejected straightaway**.

RESERVOIRS

When dams are constructed across a river, artificial lakes called reservoirs form along its course due to the impounding of the natural flow of river water. There can be one or more reservoirs on a river and each dam shall have its associated reservoir. The chief purpose of constructing a dam in most cases is to store the water in reservoirs for various purposes. Thus, a reservoir is expected to be beneficial in different ways for a long time to come. If this objective is not achieved for some reason or the other, it indicates the failure of that reservoir as well as the failure of the connected dam.

Hence, geological investigations are conducted in advance to study the suitability of the site as a reservoir. These investigations must be carried out very thoroughly in advance because once the reservoir is put into active service, the underlying area remains submerged permanently under water and becomes inaccessible or difficult for any future studies. These studies include many geological and non-geological site-based aspects such as water-tightness of the reservoir; life of the reservoir (i.e. silting rate); reservoir capacity and area; effect of evaporation; possible submergence of valuable economic mineral deposits, fertile land and forests; submergence of places of interest such as temples and historical monuments; ecological effects and so on. After proper assessment of all these factors, a reservoir location is selected.

» *Capacity of the reservoir*

The reservoir's capacity is very important and it depends on the existing topography and the proposed top water level (TWL) of the reservoir.

» *Effect of evaporation*

The natural evaporation process reduces the quantity of water in the reservoir. Loss of water due to evaporation will be still more in regions having hot and dry climates. Evaporation losses of water are known to have been reduced in parts of Africa by the growth of water cabbage and the use of chemicals. Cetyl alcohol can be used, for example, to form a thin insulating layer on the water surface and reduce evaporation.

» *Water-tightness and influencing factors*

As a consequence of weathering, which is a natural process, the surface is covered by loose soil and below it lies the fractured rock (i.e., subsoil). The massive bedrock occurs further below. When the dam is constructed, the impounded water accumulates in large quantities in a reservoir, which covers a very large area.

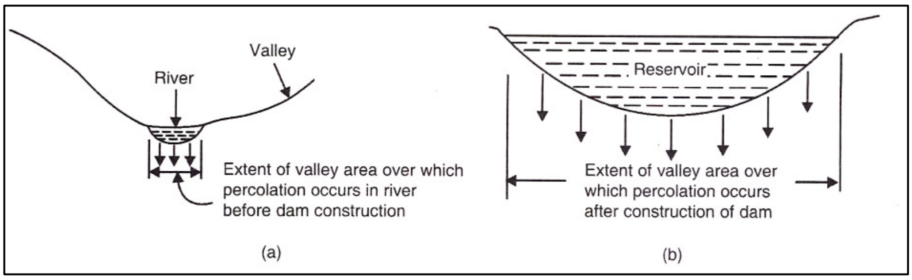


Figure 2.44: Water percolation in dams

This means percolation will occur over all such areas. Furthermore, due to the considerable height of the water in the reservoir, significant hydrostatic pressure develops, which will make the leakage more effective on the sides and the floor of the reservoir, particularly at its deepest end, next to the dam. Thus, leakage may reach an alarming level.

» *Buried river channels*

These are more frequent in glaciated regions and are also a source of leakage when they occur at the reservoir site. The Drac river in the south of France experienced severe leakage of reservoir water. The course of the pre-glacial Drac extends across the present river gorge at two points upstream of the Sautet dam, its floor being at a deeper level than that of the present river. The old channel was filled with permeable sands and gravels, through which heavy loss of reservoir water occurred.

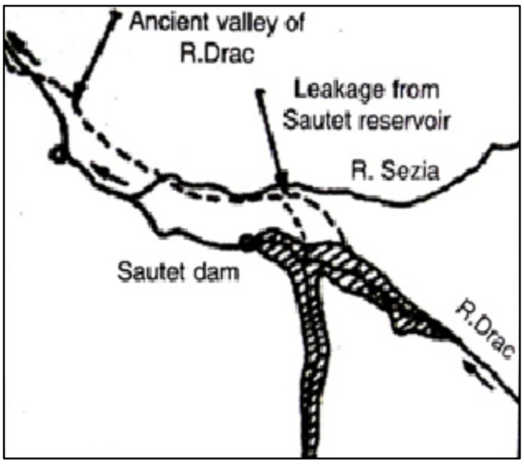


Figure 2.45: Buried river channels

INFLUENCE OF ROCK TYPES

The rock types occurring at a reservoir site are also a strong influence factor for the water-tightness of a reservoir basin. Permeable and porous rocks (i.e. aquifers) will cause water leakage and are thus undesirable at the reservoir site. Common rock types and their influence are discussed below.

IGNEOUS ROCKS

Due to their composition, texture and formation, granites and other intrusive igneous rocks are neither porous nor permeable. When present at the reservoir site, they will not cause water leakage unless they contain other defects such as faults, shear zones or joints. Extrusive igneous rocks such as basalt and other volcanic formations are not desirable as they do frequently become porous and permeable.

SEDIMENTARY ROCKS

Sedimentary rocks are more important in this context than igneous rocks by virtue of their frequency and wide aerial extent. Shale is the most abundant sedimentary rock followed by sandstones and limestones.

- Shale is an extremely fine-grained sedimentary rock, which is very porous but not permeable (i.e. aquicludes). Shale occurring at the reservoir site will thus not cause any leakage. Its occurrence at the dam site is considered undesirable due to its incompetence and slipperiness.
- Sandstone, the next common sedimentary rock, is an aquifer and hence, has a tendency to cause leakage. Careful examination is needed, however, to determine whether this leakage will be severe if present at the reservoir site. Sandstone has varying porosity and permeability depending on its cementation and the composition of the cementing material. Less porous and less permeable sandstones are well-cemented and compact, and therefore less prone to leakage.
- Limestone, the third most common sedimentary rock at the reservoir site, is generally undesirable. There may be some exceptions to this rule. Limestone is compact and dolomitic or siliceous. This means that its porosity is not only negligible, but it also possesses reasonable hardness and durability. These characteristics make limestone a desirable rock at the reservoir site. Other limestone types, though compact, are undesirable because the circulation of carbon dioxide-bearing waters along the cracks and fractures of these rocks makes them susceptible to the formation of caverns. Although

massive limestones seem to be waterproof on the surface, they may contain internal caverns and cause leakage.

- Of the less common sedimentary rocks such as conglomerates, breccias and laterites, the conglomerates and breccias may cause leakage at the reservoir site. The extent of leakage depends on their porosity. Breccias, which generally occur in fault zones, represent tweak, porous and permeable zones. Laterites, which are highly porous and naturally cause excessive leakage.

METAMORPHIC ROCKS

A very common metamorphic rock, gneiss behaves like granite, i.e. it is neither porous nor permeable. Schist, on the other hand, is generally a source of weakness and leakage due to its strong foliation and soft minerals, which are prone to cleavage. Quartzite, a compact rock, is neither permeable nor porous thanks to its quartz content (which is durable, hard and free of cleavage) and granulose structures. The occurrence of quartzite at reservoir sites contributes to water-tightness. Though compact, marble contains calcium carbonate composition and calcite (which is soft and with excellent cleavage sets), making it unreliable in terms of water-tightness. Slate tends to cause leakage because of its typical slaty cleavage, but it's very fine-grained nature helps hold leakage in check.

» *Influence of geological structures*

Tectonic forces often cause different geological structures in natural rocks such as folds, faults and joints. These structures may occur together or alone and may be simple or complicated. The simple presence of these structures has a significant influence on decreasing or increasing the leakage through the rocks at a reservoir site. This means that the inherent leakage character of rocks is modified by geological structures. Granite, for example, is notably nonporous; however, intense joints or faults on the same rock may cause profuse leakage. Easy detection of all kinds of structures including simple tilting is only possible with sedimentary rocks and foliated metamorphic rocks. Massive igneous rocks and granulose metamorphic rocks do not reveal tilting, folding or faulting. Faults may ofcourse appear as fractures. Structures that are detectable in all kinds of rocks include joints, shear zones and fault zones. In terms of possible water leakage, folding or tilting is not relevant in igneous rocks or granulose metamorphic rocks. Unconformities, joints, faults, and rock cleavage, however, do influence leakage.

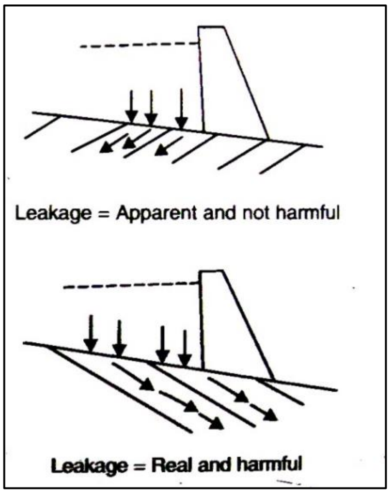


Figure 2.46: Leakage types

Looking at the different structures, the bedding planes (of simply tilted beds or of strata limbs of folds), joint planes and fault planes have the common characteristic that all these formations represent weak planes and provide scope for leakage. This depends on their attitude or, in other words, their strike and dip. If upstream dip is present, a serious problem is not present, since because any leakage that occurs will be guided back in the upstream direction only. There is then no effective loss of water due to leakage. A downstream dip is undesirable, because under the powerful thrust caused by the height of the water column in the reservoir, water is forced along the weak planes to reach the downstream side. Undesirable loss of reservoir water and development of uplift pressure on the dam are the results.

When determining the role of structures, lithology (i.e. rock types), topography and the position of different beds at the reservoir site will be viewed as the same in this context. Topographically, another parallel valley occurs next to the valley containing the reservoir at a lower level. In lithological terms, when we examine rock types, a permeable bed (e.g. sandstone or cavernous limestone) can be found between the impermeable beds (e.g. shales). As for their geological characteristics, all beds are conformable and striking parallel to the length of the valley. With respect to their position, the permeable bed occurs at the reservoir rim and all beds are dipping towards the same side.

Case 1 shows the possibility of reservoir water leakage into the adjacent lower-lying valley. Leakage occurs because the tilted permeable bed is also exposed in the adjacent valley.

Case 2 shows how folding can prevent such leakage.

Case 3 shows how a faulting can prevent leakage as well. In this case, along the fault plane, the permeable bed through which reservoir water percolates is intersected by an impermeable bed, which prevents leakage.

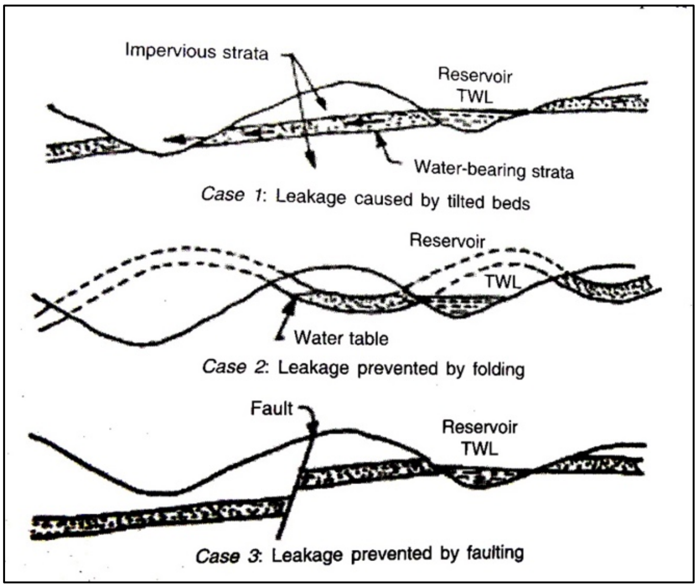


Figure 2.47: Leakage in different fault scenarios

The proceeding cases are hypothetical and meant only to highlight the influence of the geological structure in terms of reservoir water leakage. Nevertheless, they reveal that thorough geological studies must be employed to assess the water-tightness of a reservoir basin.

PREVENTING LEAKAGE

The previously mentioned details stress the possibility of reservoir water leakage due to various reasons and the various undesirable consequences. Leakage can, however, be partly controlled naturally and by man-made efforts as follows.

Natural silting: When silt and clay (transported by the river) settle on the reservoir floor, they fill openings of all kinds and form a layer on the country (natively occurring) rocks. Clay, being impermeable, will prohibit leakage through the openings (e.g. pore spaces, fracture planes, shear zones). Leakage is thus reduced without any special treatment by the natural silting process.

Covering weak zones with concrete slabs: Under unfavourable conditions, to prevent leakage, suitable covering such as the placement of concrete slabs may have to be considered. Of course, it will be impracticable to treat large areas in this manner.

Cut-off walls and sheet piles: To reduce percolation below the dam, cut-off walls and sheet piles are placed beneath the dam. In certain cases, however, these preventive measures may not help create an effective reservoir. The Jerome Reservoir (Idaho), Hondo Reservoir (New Mexico) and the Cedar Creek Reservoir (Texas) are a few examples from the United States of failed reservoirs due to excessive leakage.

» ***Influence of water table (effluent and influent conditions)***

Depending on its position relative to the river level, the water table either allows the groundwater to seep into the river or vice versa. If the water table is situated at a very high level and intersects the sides of the river valley, groundwater will seep and be added to the river. Rivers fed by groundwater are referred to as efficient rivers.

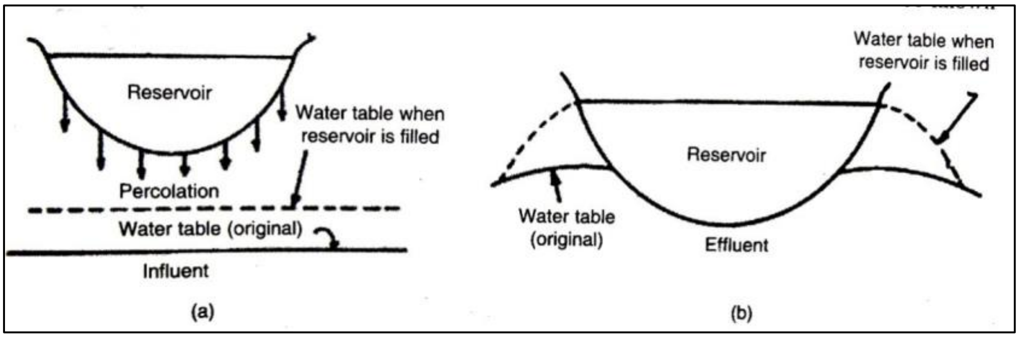


Figure 2.48: Effect of water table on a reservoir

Conversely, if the water table occurs at a considerable depth below the river, river water will percolate through the country rocks (utilising primary and/or secondary porosity) and reach the groundwater. The river therefore loses water. This process continues until the water table rises to meet the river level. In other words, tremendous leakage occurs where the water table runs deep. These types of rivers are called influent rivers.

If the river happens to be *efficient* at the reservoir site, no leakage shall occur. All ground openings are already saturated by the groundwater. Regardless of the porosity and permeability of rocks at the reservoir site as well as their inherent geological structures, no loss of reservoir water shall occur at this site. When the reservoir is full, of course, it will contain a considerably high water level (i.e. higher than the regional water table). This condition may lead to minor leakage into the sides of the valley and raise the water table level to meet the reservoir water level.

If the river is *influent* at the reservoir site, on the other hand, leakage shall occur. Precautions to prevent leakage will be ineffective and downward percolation becomes inevitable due to gravity. As this leakage continues, the groundwater becomes enriched and the water table is forced to rise. Based on the larger objective, i.e. water-tightness of the reservoir site, this rise in the water table is obviously of no advantage (unless the original water table level happened to be very shallow). Water table maps can be prepared to determine whether a river is effluent or influent and, if influent, at what depth the water table is present. These maps are contour maps in which contours join the points of equal height or depth of the water table occurrence in a region. Adequate data about the water level position in a number of wells or bore wells in the nearby vicinity must be available to prepare these maps. Information on seasonal fluctuations of such contours should, of course, also be available for the proper assessment of prevailing groundwater conditions. Water table maps interpreted together with regional toposheets of the region can enable an easy interpretation as to the effluent or influent nature of the concerned river.

Perched water tables should be noted with caution in this context. Rivers or streams flowing over perched water tables may appear to be effluent. However, the suitability of these sites for reservoirs depends on the shape and the extent of impermeable formations that have caused the development of such unusual perched water conditions. These sites are unsuitable as reservoir locations.

Ponds, springs or seepage frequently occurring along the river indicate a high regional water table and an effluent river. If these features are absent, the water level in the nearby wells (when left untapped for a few days) indicates the underground water table position. This helps to estimate the influent nature of the river.

» ***Reservoir silting (orreservoir life)***

Reservoir silting is as harmful as reservoir leakage and both can cause the reservoir to fail if they occur at serious levels. One example of failure due to silting is the New Lane Austin Dam on the Colorado river. Rapid silting over 13 years left the reservoirs almost completely filled and rendered the dam and reservoir ineffective. At the Cat Creek Canyon Dam in Nevada, a thunderstorm washed down the entire drainage area, almost filling its reservoir, within just 30 minutes.

The basic nature of a river is to cause erosion when it has energy and deposit eroded sediments whenever it loses its energy. The main source of a river's energy is its velocity. By virtue of steep gradients that prevail at the origins of rivers (i.e. hills or mountains), rivers will be moving very fast with a high velocity and therefore be highly energetic. This energy in turn, enables the river to cause rapid erosion, which means large amounts of sediments are formed. These sediments are carried by the river further along a reservoir, which acts as a barrier and checks the natural onward flow of water. This causes a loss of energy of the river and, as a consequence, transported sediments (i.e. silting) will be deposited. Thus, a reservoir, which stores the impounded water is also the place of active deposition of sediments (i.e. silting).

Silting which commences at the bottom of the reservoir continues, and the thickness of deposited sediments gradually increases with time. Silting simultaneously reduces the reservoir's water storage capacity. Since erosion is an effective process in the upstream course of a river, sediments (the products of erosion) are continuously added to the body of the river. The river transports these sediments and dumps into the reservoir in a never-ending process. The reservoir's capacity to hold water gradually vanishes and, finally, only sediments remain in the reservoir, but no water. To ensure that the storage capacity of the reservoir never falls below requirements during the design life cycle, silting must be taken into account. The total silt volume likely to be deposited dam's life cycle is therefore estimated. That approximate volume, known as dead storage, is left unused to allow for silting. Dead storage generally makes up more than one quarter of the total capacity. The remainder is known as effective storage or live storage.

» ***Measure to control the silting process***

The forgoing information makes it very clear that, from an economic point of view, the success of a reservoir depends on the rate of silting or the life of the reservoir.

Careful geological examination of the catchment area and of other upstream regions of a reservoir site reveals the zones, which contribute the bulk of the sediments to the river. Usually, areas with loose soil, highly weathered rocks (i.e. rocks, which are highly disintegrated and decomposed), weak argillaceous rocks like mud or claystones and rocks gently dipping on the downstream side are the sources, which readily contribute sediments to the river when it flows over them.

Similarly, the highly fractured zones like fault zones, shear zones and places, which are intensely jointed also contribute a large portion of sediments to the river. Furthermore, the valley side with a uniform slope allows sediments to roll down easily and join the river. Thus, river sediments come from different sources, which can all ultimately be attributed to erosion. The following measures help reduce erosion with reference to the forgoing contexts:

- Vegetation
- Covering with slabs on weak zones
- Terracing of the slope and construction of retaining walls
- Check dams and settling basins
- Diversion of sediment-loaded waters
- Stilt outlets

LANDSLIDE EVENTS

When a reservoir is put into active service, landslides are likely to occur. Hence, the possible after-effects of the creation of a reservoir should be considered carefully. Due to a reservoir, large masses of rock, which were never under water will be submerged and further large masses are likely to be waterlogged by the raising of the water table around the reservoir. This makes it necessary to ascertain, which rocks will be affected and how they will behave under the new conditions. Since water is the single most important cause of landslides, the scope of their occurrence should be examined carefully.

» ***Case study on dealing with slope stability***

The Beas Dam is a 116-metre high earth and rock-fill dam that crosses the river Beas in the Punjab State. The most important geological feature responsible for the instability of the slopes in the vicinity of the tunnel intake in the left abutment is the disposition of an inter-bedded sandstone-clay-shale sequence which is exposed on slopes steeper than the dip. This potential instability is further aggravated by the under-cutting of the toe of the slopes by the river and the occurrence of persistent joints (with a North West-South East trend).

Important engineering structures have been located within or close to a major slide area in the left abutment. Due to the adverse geo-topographical condition of the slopes, they have been subjected to a major slide in the past and a minor slide recently, and unless the slopes are suitably corrected, similar slides can take place in future.

One of the proposals for stabilizing the slopes of the tunnel intake area is to flatten them from 1.5:1 to 2.5:1 (and locally 1:1) and with gravel packs to cover the contacts of clay/shale and sandstone so as to increase the resistance of shearing in planner slides. The slope cuts should also be restricted to predetermined heights with suitable berms, depending upon the results of stability analysis. Any deep-seated shear failure along the clay gouge seams in the thick clay/shale beds exposed to the free surfaces at the river side should also be taken into account by providing concrete buttress walls, suitable drainage, and a dowelled concrete lining on the slopes above the intake bench.

Some measures are also necessary to stabilise the main slide area upstream of the tunnel intakes. These corrective measures would include the provision of suitable slope cuts with berms and drainage tunnels and partial removal of the hill on the top, in order to physically eliminate the volume of material during any major slide into the reservoir.

From a distance of about 200 metres upstream of the intake portals of the tunnels, the river has more or less carved a strike valley measuring 2.4 kilometres in length. Along this stretch, there are two old landslides on dip slopes and exposed clay/shale and sandstone. It was apprehended that as the Ramganga reservoir would submerge this landslide up to a considerable height, the reservoir would potentially cause further landslides in steps due to the changed conditions caused by the reservoir operations, until a very thick sandstone band inhibited the failure of the slopes. Moreover, these slides, if unchecked, would possibly affect the safety of the dam and the operation of the intake structure located immediately downstream.

Geological studies were carried out to estimate the quantity of material, which might be brought down by probable future slides. These studies indicated the maximum amount of resulting debris, which may be of the order of 25 million cubic metres. However, an engineering analysis of the problem indicated that large-scale landslides are unlikely because i) the natural slopes are flatter than the dip slopes and ii) the factor of safety obtained in the stability analysis of slopes is quite high for normal earthquake conditions. Further, the impact of major slides was analysed and it was decided that no remedial measures were required to control the anticipated slides and, even if they do occur, they will most likely not pose any serious threat to the proper functioning of the dam and the tunnels.

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3. SYSTEM DESIGN

What this module is about

For a hydropower scheme to be successful, the design and implementation of the different system components needs to be carefully carried out. Poor system design will ultimately lead to system failure. In this module, participants shall be introduced to international best practices governing the design and implementation of rural hydropower schemes.

Learning outcomes

At the end of this module, the participant is able to

- Select an appropriate layout for micro hydropower (MHP) civil works
- Design and plan the civil works and select appropriate materials
- Understand best practices and common pitfalls
- Supervise construction of rural hydropower civil works

INTRODUCTION

Hydropower development includes a number of structures, the design of which will be dependent upon the type of scheme, local conditions, access to construction material and also, local building traditions in the country or region. Some important characteristics one should consider are:

- No two sites are the same. The design depends on the site conditions.
- Design engineers may have a say about the materials used in construction, but they have limited control over the ground on which the structure stands. Geological conditions must be taken into account and alternative sites considered to avoid problems with instability.
- Failure of civil works can be very dangerous and expensive. Similarly, poor performance or over-design is also uneconomical.

Although no two micro hydro sites are identical, all of them require certain common components of different dimensions to convey water from a stream or river to the power generation units and back to its source.

3.1. DIVERSION WORKS

The diversion works for a micro hydropower scheme control the flow of water from the source river into the headrace. They comprise a diversion weir (usually), an intake, and sometimes river training works (river training works consist of protection walls on the riverbanks upstream of the weir). The diversion works are part of the headworks, and serve the following functions:

- Maintaining the design flow with nominal head losses during rainy and dry seasons
- Preventing the boulders, timber, leaves, etc. from entering the channel
- Safely containing flood flows in the river and away from the micro hydro system to minimise structural damage

Of these, the principal function is to minimise the sand particles (silt) entering into the channels because they can cause numerous problems for the civil works and the turbine.

SELECTION OF WEIR/INTAKE LOCATION

Consider the following principles when selecting the location for the weir.

» Minimal disturbance to the natural stream flow

Avoid high and permanent weirs (larger than 1 to 2 metres) across the total width of the river. This type of damming will result in the rapid deposition of sediments and modify the present river course. The intake will be rendered dry and useless. The design and construction of weirs requires careful consideration to avoid blocking the flow of floods in the rainy season. Therefore, locate intakes such that the natural water level in the stream at low flow is suitable for the intake level of the channel. This will allow the channel intake structure to be built at stream level. The only measures required within the stream or riverbed itself will be measures to stabilise the present state of the stream.

» Location in naturally sheltered areas

To divert water from a stream that may rise considerably during rainy periods, the intake should be located behind or beneath large, immobile boulders or rocks, as they will limit the water that can enter the intake, and divert flood flows and river-borne debris. Also, take advantage of stable banks and rock outcrops.

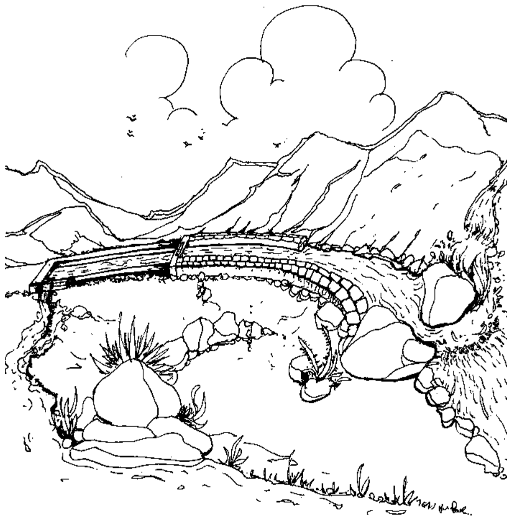


Figure 3.1: Intake location with natural shelter

» Location on the outside of a bend

Straightaway channel sections are ideal for the construction of the intake structure in order to ensure a smooth, steady flow of water to the intake and to prevent scouring of the riverbanks downstream of the intake. If this is not practicable and the river follows a bending course, it will naturally tend to deposit sediment on the inside of the river bends. At these curved junctions, the flow closest to the riverbed changes in direction compared with the surface flow. The result is a spiral flow, which carries the bed load to the inner side of the river bend. Gravel and sand banks can be seen at the inside bend of all streams and rivers, i.e. the bed load is diverted from the deflecting bank. When the flow decreases, consequently, the river width decreases from the inside of the bend. Intakes should thus not be sited on the inside of a bend.

» Stability of hillside slope

A landslide or unsteady slope located near an intake weir site is cause for concern, as it could create an obstruction at the water intake due to landslide sediments or erosion. Always give ample consideration to the stability of nearby hillsides when selecting the intake location.

» Use of existing civil structures

In small-scale hydropower development, the use of existing civil structures such as check dams, intake facilities for agriculture and irrigation channels can contribute to the reduction of the development cost. Carefully consider how the intake can be situated to make use of civil structures already in place.

CLASSIFICATION OF WEIRS

» Based on function

Storage weirs: Sometimes used to store water, given significant storage upstream of the weir.

Diversion weirs: Used to raise upstream water level and divert water into offtake channels.

Pick-up weirs: Constructed at a point downstream of a storage dam. Water released from the dam is diverted to the offtake channels at these weirs.

» Based on design aspects

Gravity weirs: Create a counterweight to fully resist the uplift pressure caused by the head of the water seeping below the weir.

Non-gravity weirs: Feature a weir floor designed continuous with the divided piers as a reinforced structure such that the weight of concrete slab together with the piers safeguard the structure from the uplift pressure. Mostly used in big rivers and small hydro projects. **Not applicable to micro hydro sites.**

» Based on construction material

VERTICAL DROP WEIR

This type of weir consists of an impervious horizontal floor or apron and a masonry wall with vertical faces or slightly inclined faces. Details are shown in Figure 3.2. On the upstream and downstream ends of the floor, cut-off walls or piles are provided up to scour depths. The upstream end of floor is equipped with a block protection; the downstream end with a graded inverted filter. Launching aprons or pervious aprons are provided at the ends on either side. This type of weir is suitable for any type of foundation.

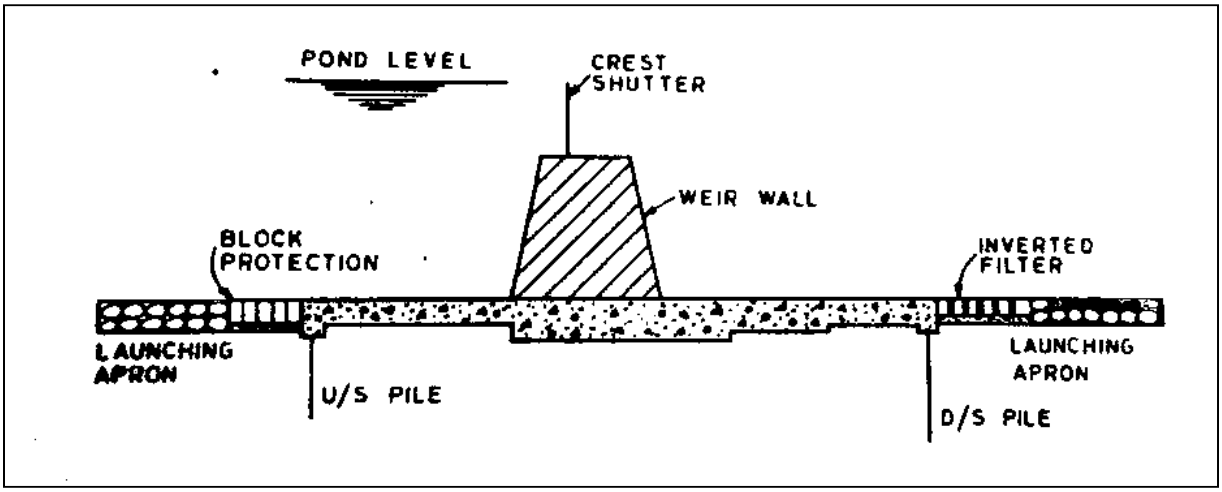


Figure 3.2: Vertical drop weir

ROCK-FILL WEIRS WITH SLOPING APRONS

These weirs consist of a masonry wall and dry-packed boulders laid in the form of a glacis, or sloping aprons on the upstream and downstream side of the weir, with a few intervening core walls as shown in the Figure 3.3.

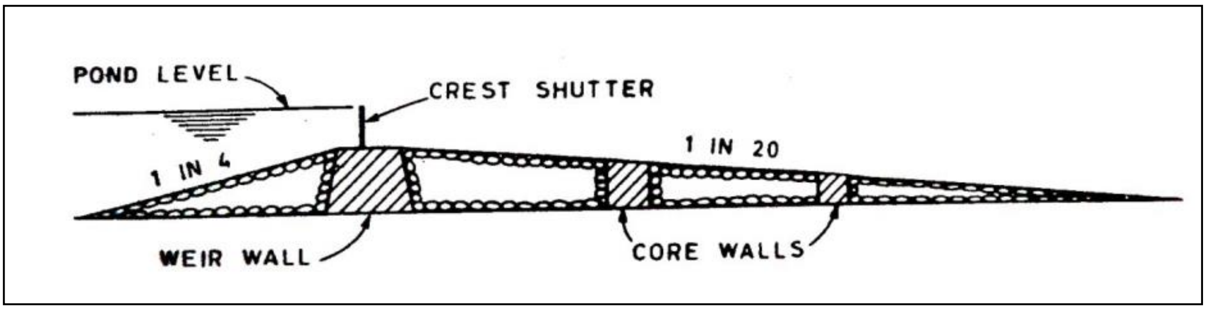


Figure 3.3: Rock-fill weirs

The downstream glacis or slope of apron is generally made very flat. It is the simplest type of construction and its stability is not amenable to exact theoretical treatment. Since rock-fill weirs require rock fragments, they are only economical when rock is abundantly available near the site.

CONCRETE WEIR WITH DOWNSTREAM GLACIS

Weirs of this type are of recent origin. Cut-off walls or sheet piles of sufficient depth are provided at the end of floors. Sometimes an intermediate pile is also introduced as shown in Figure 3.4. Considerable energy of flowing water is dissipated on the downstream end as a hydraulic jump is formed. At the end of floors on either side, protective measures are provided as in the case of the vertical drop weir. This type is commonly adopted nowadays and is usually considered for pervious foundations.

GABION WEIR

If there is no significant boulder movement along the river stretch at the intake area, a gabion weir may be an appropriate solution. If properly designed and constructed, the advantage of a gabion structure is that, unlike concrete and masonry structures, it can tolerate some ground movement without significant damage. The weir design should include checking:

- Safety against scour (with a foundation on rock or large boulders or a downstream *counterweir* to form a stilling pool)
- Seepage control (through the use of an impermeable membrane)
- Stability against overturning and sliding
- Safe bearing capacity of the foundation.

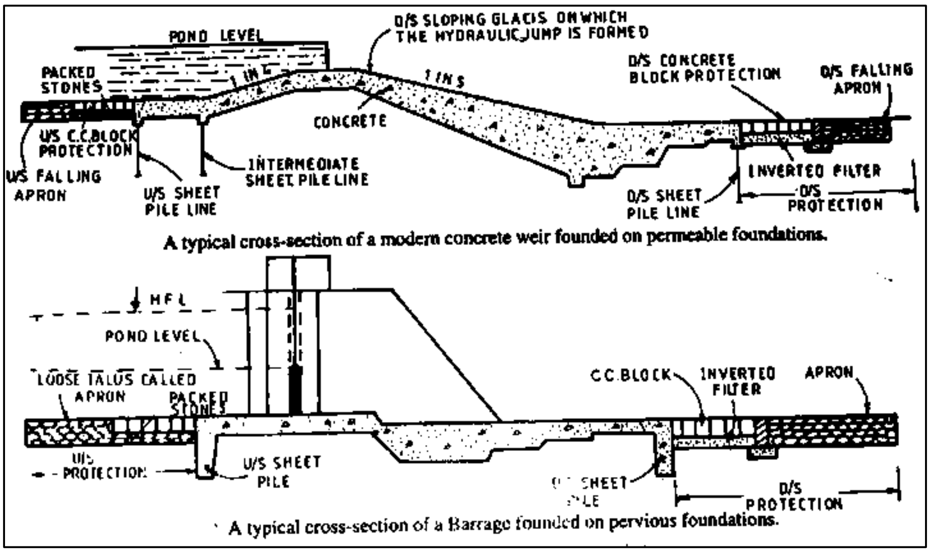


Figure 3.4: Foundation details for weirs

OTHER TYPES

Another method, capable of remote control, is the inflatable weir, which employs a reinforced rubber bladder instead of concrete, steel or wood flashboards. This offers an alternative to more conventional methods of weir construction, with the inherent advantages of low initial cost, simple operation and minimal maintenance if available locally, which is not the case in Nigeria.

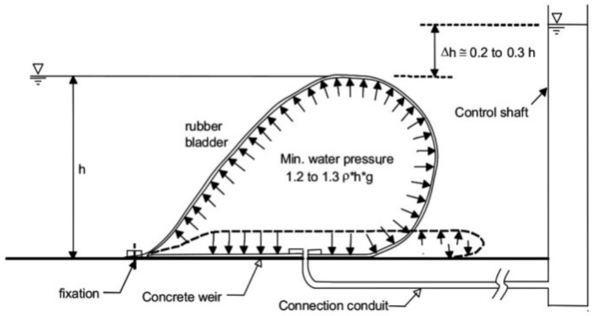


Figure 3.5: Cross section of an inflatable weir

Figure 3.6: Rubber dam in China

Inflatable weirs are effectively flexible gates designed as a reinforced, rubber bladder. Air or water is used to inflate the rubber bladder, which is anchored by anchor bolts embedded into the concrete foundation. Just like any other gate, the inflatable weir needs an opening and closing mechanism. When filled with water or compressed air, the weir is elevated; when deflated, the weir lies flat on its foundation in a fully opened position. This requires an air compressor or water pump connection to the rubber bladder via a pipe. An economic system is created by a large width-to-height ratio for the weir. Inflatable weirs offer significant advantages over conventional systems in cases where system management and operational safety are rather critical. The upstream water level and the inner bladder pressure are monitored by an electronic sensor. A constant level is maintained in the intake entrance by a microprocessor making small changes in the inner bladder pressure. Either a back-up diesel/gasoline generator or a stand-alone photovoltaic system can be used to power the microprocessor system. A similar device can regulate the inflatable weir to maintain a pre-set upstream water level to avoid flooding land.

In rivers where sudden flooding or surges are common, inflatable weir systems deflate the bladder fully and automatically. Given the dimensions of a typical weir, i.e. two metres high and thirty metres wide, this can be accomplished in less than thirty minutes.

DESIGN CONSIDERATIONS OF THE WEIR

The design of weirs and barrages is carried out in two phases:

» ***Hydraulic design***

Hydraulic design deals with the evaluation of hydraulic forces acting on the structure and the determination of the most economical structure configuration with the best functional efficiency.

When situated on a permeable foundation, the hydraulic design of weirs and barrages is considered in relation to sub-surface and surface flow conditions. Various design aspects relating to sub-surface flow require engineers to determine uplift pressures, the length of impervious floor, sheet pile or cut-off depths at the upstream and downstream ends of the impervious floor, and protective works.

Design aspects relating to surface flow, on the other hand, involve determining the pond level, afflux (upstream water level increase caused by the weir), upstream floor levels and the crest of weir or barrages, weir crest shape, waterway, and the effect of regression and energy dissipation measures.

» *Structural design*

The structural design consists of dimensioning various parts of the structure to enable it to safely resist all the forces acting on it. Important design considerations:

- Since the dam must be safe from overturning under all possible load conditions, the contact stress between the foundation and the dam must be greater than zero at all points. To assure this condition, the resultant of all horizontal and vertical forces – including the weight of the dam – must pass through the centre third of the base.
- Any sliding of the dam must also be excluded. The static friction coefficient, i.e. all horizontal forces divided by all vertical forces, must remain between 0.6 and 0.75.

» *Afflux*

Afflux occurs when the water level on the upstream side exceeds the normal level due to a weir construction. The value of afflux relative to the design flood is an important indicator for the length and crest of the structure, as well as for the downstream cistern, cut-off depth, bed protection, and river training works.

The maximum permissible afflux value depends on various factors like upstream river conditions, the measured back water effect, and the submerged area coming and its importance.

The Indian Standard IS 6966 recommends an afflux of 1 metre for weirs or barrages located on alluvial rivers in the upper and middle reaches of the river, and 0.3 metre in the lower reaches of the river with flat gradients. For weirs or barrages located on rivers with shingle or boulder beds and banks, an afflux exceeding 1 metre is permissible. Higher afflux levels have both advantages and disadvantages:

Advantages	*Disadvantages*
▪ Low-cost narrow weir ▪ Makes the river flow modular at all stages thanks to the formation of a standing wave or hydraulic jump, even at the maximum flood level ▪ Helps the operation of silt excluders and silt extractors. Generally provides better control of the weir	▪ Higher cost of gates and river training works ▪ Greater intensity of discharge ▪ Greater scour depth, involving additional length and thickness of launching aprons on upstream and downstream ends ▪ Greater depth of cut-offs ▪ Greater water level difference on the upstream and downstream ends ▪ Greater risk of failure due to outflanking.

DISCHARGE OVER WEIR

As stated earlier, placing a weir across the river raises the water level. Any excess flow that is not withdrawn into the intake flows over the weir. The discharge over the weir is given by the following equation:

$$Q = C_w \text{ x } l_{weir} \text{ x } (h_{overtop})^{1.5}$$

Where

Q = Discharge over the weir in m^3

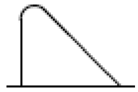
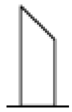
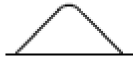
L_{weir} = Length of weir in metres

$h_{overtop}$ = Head over the weir crest level in metres

C_w = Weir coefficient which varies according to the weir profile

C_w for different weir profiles is shown in Table 3.1. In micro hydro schemes, the weir is usually broad with round edges; C_w is therefore 1.6.

Table 3.1: Weir coefficient

Shape		***Coefficient***
Broad; sharp edges		1.5
Broad; round edges		1.6
Rounded		2.1
Sharp		1.9
Roof-shaped		2.3

INTAKE: INTRODUCTION AND GENERAL LAYOUTS

Intake structure types are chiefly distinguished by the method used to divert water from the river. Micro hydropower schemes commonly implement the following intake types:

» ***Side intake***

Figure 3.7: Intake top wall

Side intakes are simple and less expensive than other types of intakes. They are easy to build, operate and maintain. Gates should be provided for a side intake if the river is relatively large and thus flows into the intake need to be regulated, and at times stopped, such as during emergency or maintenance work. However, if the river flow is low, intake gates may not be essential and it may suffice to provide a spillway downstream of the intake.

One should note that the provision of orifice-like structures (i.e. solutions as simple as a wall on top of an intake channel, as shown in Figure 3.7, will help to control water entry during floods. Even with this type of orifice, however, higher flows (i.e. higher than the flows required for full power gen eration) will enter the intake during flood events. A spillway should be installed at an appropriate location directly downstream of the intake to ensure that only the desired flow is conveyed further downstream. This will both improve the performance of the downstream structures (e.g. the gravel trap and settling basin) and reduce risks due to uncontrolled overflows. When overflows do occur, the channel between the intake and the spillway should also be equipped to handle excess water that enters during flood. Below is an example of a side intake. The side intake model must be equipped with a flushing gate.

In the case of a small-scale hydropower plant, the headrace is usually an open channel. When this type of headrace is employed, it is essential to avoid the inflow of excess water, which considerably exceeds the design discharge, as it will directly lead to the destruction of the headrace. Important points for design:

Figure 3.8: Side intake

- It is necessary for the intake to have a closed tap instead of an open tap so that it becomes a pressure intake when the water level of the river rises.
- The intake should be placed at a right angle to the river flow direction wherever possible to minimise the head of the approaching velocity at the time of flooding.
- As the water inflow during flooding exceeds the design discharge, the spillway capacity at the settling basin or starting point of the headrace should be fairly large.

TRASHRACKS FOR SIDE INTAKES

Trashracks on side intakes prevent the entry of boulders, cobbles, floating logs and branches into the headrace. Trashracks can be made from flat steel, angles, tees or round bars welded together at defined intervals. At the intake the trashrack is also known as the *coarse trash rack* since its bars are spaced wider compared to the trashrack at the forebay.

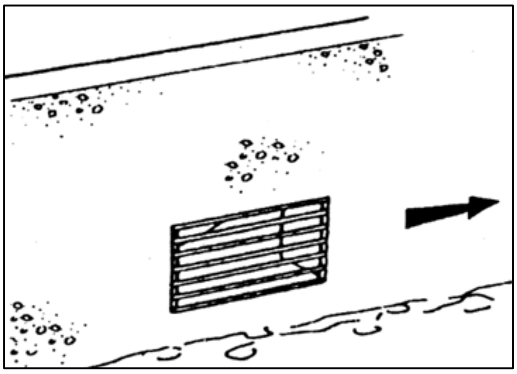


Figure 3.9: Thrash rack for side intake

Boulders can frequently impact the coarse trashrack, so it needs to be robust and employ thick steel sections. Factors such as the length and width of the opening, the nature of the sediment load, and the required flow, will determine an appropriate clear spacing, which may vary between 50 and 200 millimetres.

ORIFICE DESIGN

An orifice is normally included downstream of the trashrack on a side intake. Water is initially drawn into the headrace. At times, the side intake is merely a continuation of the headrace channel up to the riverbank. Whenever feasible, however, an orifice should be incorporated to limit excessive flows during floods. An orifice is an opening in the intake that conveys river water towards the headrace. The design flow can pass through the orifice under normal conditions (i.e. low flow), but it restricts higher flows during floods. The discharge through an orifice for submerged condition is defined by the equation below:

$$Q = A \times C \times \sqrt{2g(h_r - h_h)}$$

$$V = C \times \sqrt{2g(h_r - h_h)}$$

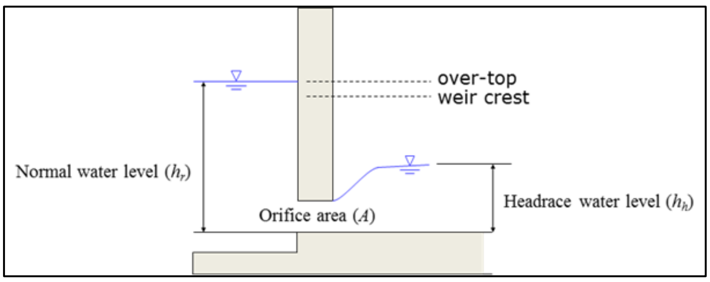


Figure 3.10: Cross-section showing orifice

Where

- Q = Discharge through the orifice in m^3/s
- V = Velocity through the orifice
- A = Cross-sectional area of orifice in m^2
- h_r = Water level in the river next to the orifice relative to a datum
- h_h = Water level in the headrace canal measured from the same datum as h_r.
- g = Acceleration due to gravity = 9.8 m/s^2
- C = Coefficient of discharge of the orifice and is dependent on the shape of orifice

The value of C decreases with the amount of turbulence induced by the intake. For a sharp edged and roughly finished concrete or masonry orifice structure, this value is as low as 0.6 and for carefully finished aperture it can be up to 0.8. For micro hydro, the recommended velocity (V) through the orifice during normal flow is 1.0-1.5 m/s.

Using these equations and factors, the size of the orifice is calculated as follows:

- Assuming a maximum velocity of 1.5 m/s through the orifice, calculate the required area of the orifice opening using Q = V x A.
- For a rectangular opening, A = W x h

Where W is the width and h is the height of the orifice. Set h according to the river and ground conditions and calculate W.

Example: Sizing of an orifice

Choose a suitable size of an orifice for a design flow of 250 l/s. The normal water level in the river is 0.8 metres above the bed level. The design flood level is about 0.7 metres above the normal water level. What is the discharge through the orifice during such a flood?

⇒ Q = 0.250 m^3/s

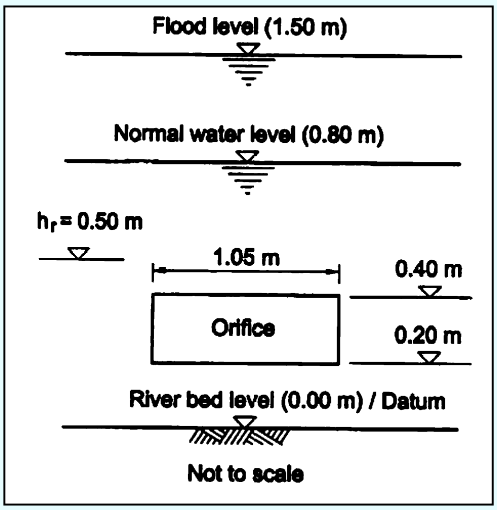
⇒ Set V = 1.2 m/s

⇒ Orifice area (A) = Q/V = 0.25/1.2= 0.21 m^2

⇒ Set orifice height (h) = 0.20 metres and width of orifice (W)=A/h =0.21/0.2 =1.05 metres

⇒ Set bottom of orifice 0.2 metres above the river bed level. This will minimise the bed load.

⇒ Also, set the datum at the river bed level.



Set water level at headrace canal, h_h = 0.5 metres with respect to the datum (i.e. 100 mm above the upper edge of orifice to ensure submerged condition). Note that later the headrace canal will have to be designed accordingly.

$$Q = A \times C \times \sqrt{2g(h_r - h_h)}$$

Assume C = 0.6 for roughly finished masonry orifice.

$$Q = 0.21 \times 0.6 \times \sqrt{2 \times 9.8 \times (0.8 - 0.5)}$$

⇒ = 0.31 m^3/s or 310 l/s

⇒ Q required = 250 l/s: Therefore, orifice size is OK.

⇒ Discharge through the orifice during flood flow:

⇒ h_r - h_h = 0.8 + 0.7 - 0.5 = 1.0 metres

⇒ Q_{flood} = 0.21 x 0.6 x SQRT(2 x 9.8 x 1.0) = 0.56 m^3/s

⇒ Q_{flood} = 560 l/s

Note that excess flood flow can be discharged via a spillway at the gravel trap or a suitable location.

» *Drop or bottom intake*

Several simple intake designs aim to reduce the weir height and omit the flushing gate for a hydropower plant. At a bottom intake (also known as the Tyrolean intake), the weir is completely submerged. Excess water will pass the intake by flowing over the weir. Bottom intakes are preferable in locations with minimal sediment movement along the riverbed, because they tend to withdraw more bottom water than surface water. This intake model was first used for SHP and irrigation systems in the early 1900s in alpine regions of Europe.

Figure 3.11: Drop intake

The water to be diverted is taken in through a collection channel built into the river bottom and covered with a screen. The bars of the screen are laid in the direction of the current and inclined in the direction of the tailwater so that coarse bed load is kept out of the collection channel and transported further downstream. Particles which are smaller than the spaces between the screen bars are introduced into the collection channel together with the water. Later on, suitable flushing devices must separate these particles from the water for power generation. The bottom intake can be constructed at the same level as the riverbed or in the form of a sill.

Advantages	*Limitations*
▪ Very useful with fluctuating flows; even the lowest flow can be diverted ▪ No maintenance required (if well designed)	▪ Expensive ▪ Local materials not useable ▪ Good design required to prevent sediment blockage

DESIGN OF BOTTOM INTAKE

The following equation is used for the design of a bottom intake:

$$Q_A = \tfrac{2}{3} c \times \mu \times b \times l \times \sqrt{2gh}$$

Where:

Q_A=	Design discharge into the intake in m^3/s
b =	Width of the bottom intake in metres
l =	Length of the trashrack in metres. It is recommended that the trashrack length (l) be increased by 20%, i.e., l= 1.2 x $l_{calculated}$. This will ensure that there will be adequate flow when the trashrack is partially blocked by wedged stones and branches.
h =	2/3 x h_E
h_o =	Initial water depth in metres in the river upstream of the intake
h_E =	$h_o + v_o^2 / 2g$.
Q_A=	Design discharge into the intake in m^3/s
g =	Acceleration due to gravity (m/s^2)
C =	Correlation factor for bottom overfall

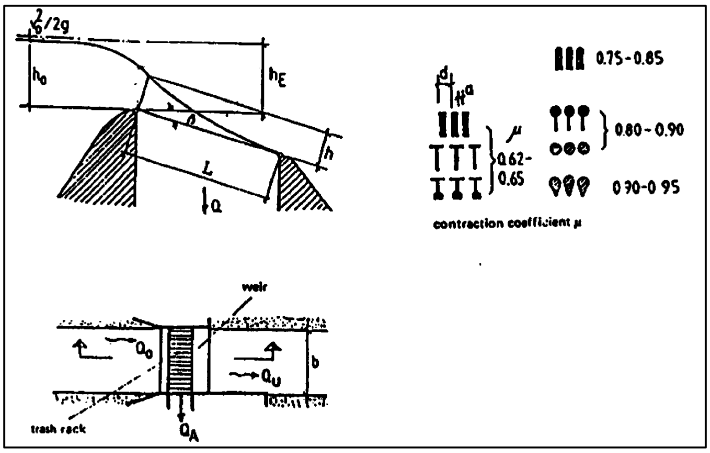


Figure 3.12: Symbols used in the bottom intake equation

Note that as can be seen in Figure 3.12 h_E is actually the initial water depth in the river plus the velocity head of the river ($v_o^2/2g$). For steep rivers, the flow velocity should be measured since the velocity head can be high.

X = A function of the inclination of the trashrack (P)

C = Correction factor for submerged overfall = 0.6 x 4-a/d (cos ß)$^{3/2}$

a = Clear spacing of the trashrack bars in metres

d = Centre-to-centre distance between the trashrack bars in metres

ß = Angle of inclination of the trashrack with respect to the horizontal in degrees

µ = Contraction coefficient for the trashrack, which depends on the shape of the bars

Table 3.2: Values for *ß and x*

ß	*x*	*ß*	*x*
0°	1.000	14°	0.879
2°	0.980	16°	0.865
4°	0.961	18°	0.851
6°	0.944	20°	0.837
8°	0.927	22°	0.825
10°	0.910	24°	0.812
12°	0.894	26°	0.800

In Figure 3.12, Q_o is the river flow upstream of the intake and Q_u is the excess flow in the river downstream of the intake. To solve the bottom intake equation, either the length or the width of the intake opening must be set before the other dimension can be calculated.

The dimension selected depends on the site conditions. If the trashrack length is too small, for example, excavation of the headrace channel in the riverbed will need to be deeper, a requirement which may prove difficult. The bottom intake length should generally equal the headrace channel width, and the width should match the river channel.

» *River training works*

A flood protection wall along the riverbank may be required if there is a high probability of flood damage to the initial headrace and other structures such as the gravel trap and settling basin. Such walls are also called river training structures since they confine the river channel. The wall height should be greater than or at least equal to the design flood level.

» *Gates*

Usually fixed wheel gates or radial gates are provided. Follow the standard regional code of practice for the design.

Example: Sizing of a bottom intake

A suitable site has been located for a bottom intake. The river width at this area is 5 metres and the depth is 0.5 metres (i.e. h_o = 0.5 metres). A velocity of 3 m/s was measured at the intake site. The design flow (QA) required for power generation is 0.40 m^3/s. Select an appropriate size for the bottom intake.

⇒ Choose 20 mm diameter round bars for the trashrack.

⇒ μ = 0.85 for round bars (from Figure 3.12) Set the clear spacing between the bars, a = 12 mm

⇒ Centre-to-centre distance between bars, d = 32 mm

Set the inclination of the trashrack ß = 8° (trashrack inclination should be equal to or slightly greater than the river gradient)

⇒ For ß = 8°, X = 0.927

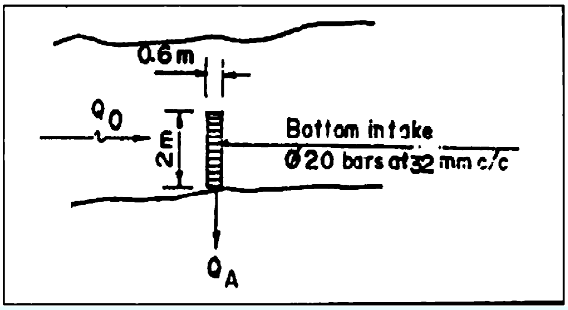
⇒ $h = 2/3\ X\ h_E$

⇒ $h_E = 0.5 + 3^2/2g = 0.96$ metres

⇒ Or h = 2/3 x 0.927 x 0.96 = 0.59 metres

⇒ $c = 0.6\ a/b \cos^{3/2}ß$

⇒ $c = 0.6 \times (0.012 / 0.032) \times \cos^{3/2} (8°) = 0.22$



Dimensions for the bottom intake of example

Now use the bottom intake equation:

$$Q_A = \tfrac{2}{3}\, c \times \mu \times b \times l \times \sqrt{2gh}$$

$$Q_A = \tfrac{2}{3} \times 0.22 \times 0.85 \times b \times l \times \sqrt{2 \times 9.8 \times 0.59}$$

$$Q_A = 0.42 \times b \times l$$

⇒ With Q_A = 0.40 m^3/s: B x l = 0.40/0.42 = 0.95 m^2 or l = 0.95 / b

Select the width of the trashrack, b = 2 metres, l = 0.95/2 = 0.48 metres. Increase the length by 20%: l= 0.48 x 1.2 = 0.57 metres. The proposed dimensions of the bottom intake are as follows:

⇒ Width of the opening, b = 2.0 metres (at right angles to the flow)

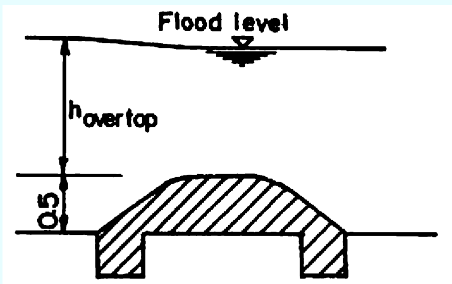
⇒ Length of the opening, l = 0.6 metres (parallel to the river flow)

⇒ Trashrack bar size = 20 mm diameter round bars

⇒ Bar spacing = 32 mm centre to centre

Example: Height calculation for the flood protection wall

A broad crested weir has been placed across a river for a micro hydro intake as shown in the figure. The weir height is 0.5 metres and the length 5 metres. How high should the flood protection wall be for a 20-year return flood of 11 m^3/s?



⇒ Equation for flow over a weir

$$Q = C_w \text{ x } l_{weir} \text{ x } (h_{overtop})^{1.5}$$

⇒ So, $h_{overtop} = [\frac{Q}{(C_w \text{ x } l_{weir})}]^{0.667}$

⇒ Note that C_w is 1.5 for broad crested weir with rounded edges (Table 3.1).

⇒ $h_{overtop} = [\frac{11}{(1.6 \text{ x } 5)}]^{0.667}$ = 1,24 metres

Height of flood protection walls from river bed level = 0.5 metres + $h_{overtop}$ + 0.3 metres (allow 300 mm of freeboard) = 2.04 metres.

3.2. STORAGE STRUCTURES

Dams are mainly storage structures. A dam is a barrier constructed across a river for impounding water during floods and releasing the water later, as and when required, for various purposes such as irrigation, water supply, hydropower generation, etc. Dams can have a moderate head with a large storage capacity or a high head with a small storage capacity.

CLASSIFICATION OF DAMS

For small hydro schemes, dams can be classified based on the following criteria:

- Function
- Purpose
- Shape
- Construction material
- Hydraulic or structural design

» Classification based on functions

Storage dams are mainly for storage and subsequent use, as and when required, for hydropower generation, irrigation, water supply etc. **Diversion dams**, on the other hand, are constructed to raise the water level and divert water through an appropriate system. Such schemes have limited or no storage capacity. In cases of perennial rivers or streams, the flow is assured and as such, a diversion scheme supplies the required water quantity to planned projects. A third dam type, **detention dams** have the function of detaining flood waters to control flooding with flood water released in a controlled manner. Generally, these dams are kept empty prior to anticipated floods.

» Classification based on application

Dams can be constructed for various different applications. Irrigation, water supply, hydropower generation, fish farming, flood control, etc. are just some different purposes. Schemes that serve multiple purposes are referred to as multipurpose projects.

» Classification based on shape

The shape of a dam is designed to suit structural requirements. Shapes can include a trapezoidal dam section, arch dams, etc.

» Classification according to construction material:

Dam construction types are mainly chosen based on readily available materials. In general, stone masonry, concrete and reinforced concrete cement (RCC), earth, rock and rock fragments are commonly used construction materials. These dams are thus distinguished as masonry dams, concrete dams (plain and reinforced), earth dams, rock-fill dams, etc. These are the popular categories under this classification.

Furthermore, steel dams (made of steel plates), timber dams (made of timber cribs) and inflatable dams (made of rubber and neoprene) are also occasionally adopted and can be used with very low heights.

» Classification based on size

Based on the hydraulic head and gross storage capacity, dams are classified as shown below.

Table 3.3: Classification of storage and head

Classification	*Gross storage*	*Hydraulic head*
Small	0.5 to 10 million m^3	7. 5 metres to 12 metres
Medium	10 to 60 million m^3	12 metres to 30 metres
Large	> 60 million m^3	Greater than 30 metres

FACTORS GOVERNING THE SELECTION OF DAM TYPE

The choice of a dam type depends on numerous factors. The paramount considerations are safety and economy. Dams have to satisfy stability tests for

- Unusually high floods
- Shock loads which may be due to earthquakes and sudden changes in reservoir levels

Another important governing factor is the overall economy of the project. The overall economic analysis is done taking into account the total capitalised cost of construction as well as operation and maintenance throughout its lifetime. Comparing various alternate proposals will yield the best solution with the lowest costs.

Apart from these two basic considerations, several technical and equally important factors, which govern the selection of dam type are discussed below.

» Topography

The topography dictates the initial choice of the dam type. The shape of the valley or gorge may be broadly classified in three categories.

- Narrow-necked valleys or V-shaped valleys with sound rock abutments on either side would be an ideal choice for an arch dam. However, for a cost-effective arch dam, the top width of the valley should preferably be less than four times its height.
- A moderately wide U-shaped valley with a sound rock foundation is best suited to a gravity dam or a buttress dam.
- For a wide valley with a soil foundation extending to a considerable depth, an embankment dam would be the ideal choice. Sometimes a masonry dam can be implemented and flanked by an earth dam on either side. The location of an overflow spillway is an important factor for earth dams.

» Geology and foundation conditions

The foundation ultimately comes into contact with all the forces acting on a dam, including its own weight. For the site and foundation, detailed geological studies must be fully investigated before finalising the choice of a dam type. The geological character and the thickness of strata, their inclination, permeability and relation to underlying strata, existing faults, fissures, etc., are just some factors that affect the foundation conditions. A high bearing capacity and resistance to erosion and percolation can be had with good rocky foundations.

Coarse sand and gravel foundations are another common foundation type. These foundations do not offer high bearing capacity; high gravity dams therefore should not be considered in such cases. Suitable choices may include earth dams, rock-fill dams, and low concrete gravity dams (to heights of 15 metres). Greater percolation can be expected through the foundation.

With silt and fine sand foundations, the major problems are excessive percolation, as well as piping and erosion at the downstream toe of the dam. Earth dams and low-level gravity dams (up to about 8 metres in height) are only the dam types that will be viable on these foundations. Nevertheless, the problems of percolation, erosion, etc., must be addressed carefully.

Clay foundations are unsuitable for all dam types as they would result in substantial settlement with unconsolidated clay and high moisture content. An earth dam may be planned in certain situations, but these dams also require special treatment to consolidate the foundation.

» ***Availability of construction materials***

The choice of a dam type also depends on the construction material available locally or near the construction site. Readily available stone, aggregate, sand, gravel, and crushed stone is favourable to a concrete or masonry dam. An abundance of soils suitable for construction, on the other hand, will point toward an earth dam. Locally available materials reduce transportation costs, which has a positive effect on the overall economy.

» ***Spillway and its location***

The safe disposal of flood water necessitates a spillway. The spillway size, type and location also have some impact on the choice of the dam site. An overflow spillway generally speaks for a masonry or concrete dam.

» ***Environmental considerations***

Undesirable developments that may arise from a high dam with a large reservoir may include the involuntary resettlement of inhabitants, submergence of large tracts of agricultural land, forest, mineral bearing lands, deforestation, and the loss of wildlife, plant life, aquatic and animal life.

» ***Earthquake zone***

For a proposed dam in an earthquake zone, the dam should be able to resist earthquake shocks without damage. Though any type of dam designed to consider earthquake forces can be adopted in these areas, earth dams and concrete gravity dams are the best suited types in this respect. Though Nigeria is not prone to earthquakes, it is vital to be aware of the effects of earthquake zones as the Earth’s movement cannot be predicted accurately.

» ***General considerations***

The choice of a dam type also depends on various other factors like river diversion during construction, availability of labour, availability of construction equipment and general expertise, site accessibility.

If it is not possible to divert a river during construction, an earth-fill dam cannot be constructed and the choice will fall between a masonry dam or concrete dam, since these types allow floods to pass over the constructed portion of dam.

» Dam selection

Table 3.4: Characteristics of different dam types

Type	Notes and characteristics
Embankment earthfill	▪ Suited to either rock or compressible soil foundation or wide valleys. ▪ Can accept limited differential settlement given relatively broad and plastic core. ▪ Cut-off to sound, i.e. less permeable. ▪ Horizons required. ▪ Low contact stresses. ▪ Requires range of materials, e.g. for core, shoulder zones, internal filters, etc.
Rock-fill	▪ Rock foundation preferable; can accept variable quality and limited weathering. ▪ Cut-off to sound horizons required. ▪ Rock-fill suitable for all-weather placing. ▪ Requires material for core, filters, etc.
Concrete gravity	▪ Suited to wide valleys, provided that excavation to rock is less than 0.5 metres. ▪ Limited weathering of rock acceptable. ▪ Check discontinuities in rocks with regard to sliding. ▪ Moderate contact stress. ▪ Requires imported cement.
Buttress	▪ As gravity dam but higher contact stresses require sound rock. ▪ Concrete saved relative to gravity dam 30–60%.
Arch and cupola	▪ Suited to narrow gorges, subject to uniform sound rock of high strength and limited. deformability in foundation and most particularly in abutments. ▪ High abutment loading. ▪ Concrete saving relative to gravity dam is 50–85%.

SITE SELECTION FOR DAMS

Some important considerations for the selection of a dam site include:

- The hydrological and geological or geotechnical characteristics of a catchment and site are the principal determinants establishing the technical suitability of a reservoir site. An assessment of anticipated environmental consequences of the construction and operation of the dam must also be undertaken.
- Certain functional and technical requirements must be fulfilled to deem a reservoir site satisfactory.
- The functional suitability of a site is governed by the balance between its natural physical characteristics and the specific purpose of the reservoir. Factors such as the catchment hydrology, available head and storage volume must be matched to operational parameters set by the project.
- Technical acceptability requires the presence of a satisfactory site for a dam, the availability of suitable construction materials and the integrity of the reservoir basin with respect to leakage.

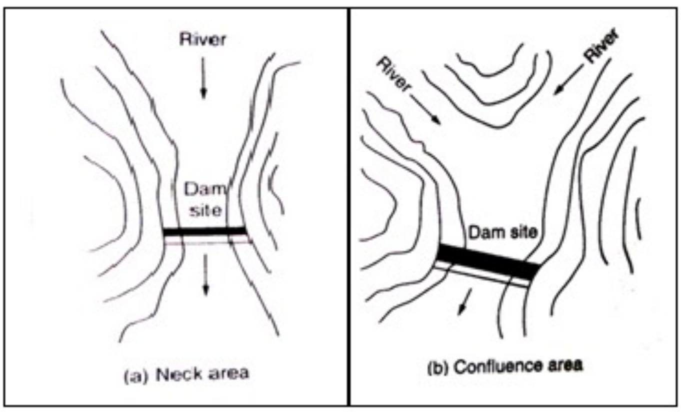


Figure 3.13: Siting the dam

Additional key considerations should be taken into account:

- A shorter dam length will definitely translate to lower costs for construction work. In this context, the dam should be located where the river valley forms a neck. Where storage is the prime factor, a valley, which provides a large storage capacity would probably be the best location. Based on the site topography, it may be possible to locate the dam downstream of a confluence point of two rivers. This is advantageous due to larger storage capacity available in both the river valleys and the greater inflow of water as well.
- For a diversion dam, as no storage is necessary, a site that decreases the length of the water conveyance system may be chosen.
- As explained in the previous section, suitable foundations should be available as otherwise appropriate foundation treatments may be necessary to improve the foundation properties. An ideal location like a low-saddle portion for a non-integrated spillway, economic procurement of bulk of construction materials, the value of property and other lands in the submergence, socio-environmental consequences, etc., also play a major role in the selection of dam site.
- Locate the dam on high ground as compared to the river basin. This will reduce costs and facilitate drainage of the dam section.
- Low-level gaps in the surrounding hills have to be plugged off with suitable subsidiary dams. The water-tightness of the reservoir basin is an important requirement.
- The site should be easily accessible to facilitate transportation of workers and personnel, material, machinery and various other essential items to and from a nearby rail or road network.
- The dam site should be such that the reservoir will not silt up too quickly. In such cases, a suitable location should be thoroughly investigated while considering the sediment-carrying properties of the river.

» *Reconnaissance, preliminary investigations and final investigations*

RECONNAISSANCE

Reconnaissance work involves visiting all available sites and collecting information such as geological data, without any sub-surface exploration. It also involves the approximate estimation of stream flow data, storage capacity and available head. Approximate spillway requirements and scouting out possible sites are included as well. Information about available environmental resources and potential changes after dam and reservoir construction must be collected for study.

PRELIMINARY INVESTIGATION

The second step is a preliminary investigation in which sufficiently precise data from various sites are collected. Usually, the following items are required:

- Survey with the resulting topographic site map
- Preliminary geological investigation and corresponding report
- Investigations about the availability of construction materials such as stone, soil, gravel, sand, concrete aggregate, etc.
- Determination of public utilities such as roads, railway lines, telephone lines, pipe lines, power plants which might be affected by the dam construction

- Hydrologic studies
- Determination of the quantity of silt carried by the stream
- Checking high flood level marks for their use in determining the spillway capacity

After comprehensive studies of various sites and associated estimates, one of the several sites is selected for final investigation.

FINAL INVESTIGATION

In the final investigation phase, the following principal items are covered:

- Sufficiently precise site survey and preparation of topographical maps to serve all design purposes and dam construction
- Determination of type of dam to be constructed
- Planning of the foundation treatment based on sub-surface investigations
- Demarcation of lands for the sites of structures and for other purposes
- Determining the extent of the submergence area, lands and villages coming under submergence, arrangements of rehabilitation, planning of environmental balance, etc.
- Obtaining all essential information for an accurate cost estimate
- Determination of final location of the dam, construction equipment, diversion of the river, coffer dams, construction of highways, roads and rail lines, probable sources of construction material and all other requisite information for the design of the dam
- Hydraulic design of dam and spillway

GRAVITY DAM

A gravity dam is a solid structure of masonry or concrete, which resists all forces exerted on it by its own weight. Since all these acting forces are transmitted to the foundation, a solid rock foundation is essentially required for the construction of the dam. Most gravity dams are equipped with an overflow spillway section. Thus, a gravity dam may consist of two sections, one being the non-over flow section and the other being the overflow spillway section.

Concrete dams can be built faster with more effective quality control. In many current construction projects, concreting is done with mechanised equipment. Masonry dams require hundreds of masons and other workers. These dams can be preferred in regions where there is large-scale unemployment and readily available manpower.

Gravity dams cannot be considered feasible in few situations where good rocky foundations are not present and where the height of dam is expected to exceed about 25 metres.

Before beginning construction, river diversion should be planned by constructing cofferdams that comply with the site conditions. Cofferdams are temporary structures made of steel piles or temporary dykes to retain water and safely divert it downstream through bypass.

Dam construction is planned in stages. It is divided in various blocks and concrete is placed in a particular block to raise it by 1.5 to 2 metres at a time. Concrete placement is followed by vibration and subsequent curing. Joints between the blocks are treated as expansion joints.

Similar stages are also employed in the planning of masonry dams. Rubble placed up to a height of about 1.5 metres is used to cover the block area. Vertical perforated pipes are placed in a staggered arrangement over the entire area of block. Then cement mortar is pumped through the pipes at a specific pressure to seal off the gaps up to the top of the block.

After curing, core samples are taken from the constructed blocks for lab testing and quality assurance purposes. In some situations, drilling and grouting with cement can be done to seal off minor cracks or cavities in the construction.

Suitable drainage galleries should be provided at different heights with downstream access. These galleries are proved in the body, along the length of the dam to stop percolation and safely drain off the water to the downstream side.

Figure 3.14: The Grande Dixence Dam (Switzerland) is the world's highest concrete gravity dam (285 metres)

Suitable arrangements to dissipate energy should be installed downstream of the overflow spillway. For gated spillways, either radial gates or lift-type sliding gates are provided to control flooding. Hoisting arrangements are installed at the top of dam.

EMBANKMENT DAMS

The foundation requirements for a gravity dam are very specific and inflexible. Embankment dams, on the other hand, need not have a rocky foundation, but can be built on soil. Other characteristics of embankment dams:

- Require only simple earth handling machinery
- Largely reduce the need for expensive materials such cement, etc.
- Can be designed to utilise locally available materials
- Overall costs of embankment dams quite economical compared to gravity dams
- Strength of earth dams increases with age
- Preferable to gravity dams in seismic zones due to their inherent adaptability

Embankment dams may be earth dams, rock-fill dams or a combination of the two. Advancements in heavy-duty machinery and the knowledge of soil mechanics have had a sizeable impact on the construction of embankment dams. More than 71% of all dams worldwide are embankment dams. Tehri Dam in Uttarakhand and Mula Dam in Maharashtra (both in India), Mica Dam (Canada), Oroville Dam (USA), and Nurek Dam (Tajikistan) are just a few outstanding examples of embankment dams.

» *Earth dams*

These dams are also known as earthen or earth-fill dams. The embankment is raised in small layers and properly compacted into the required section, which is usually trapezoidal. These dams alternatively referred to as rolled-fill dams, since compacted layers require substantial rolling. Raising the embankment by jetting soil slurry is an alternate method used in some situations. These dams are then called hydraulic-fill dams.

SECTIONAL CHARACTERISTICS OF AN EARTH DAM

Reasonable degrees of imperviousness and stability under all working conditions are the two basic requirements for earth dams. The former requirement is not possible in the presence of sandy soils and the fine-textured clayey soils do not satisfy the stability requirement. To achieve optimum design requirements, it is necessary to carefully select and blend the earth to be used for construction. Normally, earth dams tend to apply the following design features:

- Homogeneous section
- Diaphragm type
- Zoned sections

Homogeneous section: The entire section of dam is made up of a single soil type, which is usually sandy clayey soil. Use of this type involves limitations and is currently restricted to smaller dams.

Diaphragm type: In this dam type, a solid concrete or masonry wall is built in the core of the earth dam.

This imparts imperviousness, but the homogeneity of the earth mass is lost. Post-construction settling of embankments and the foundation may cause cracking of the diaphragm. This section type has become obsolete.

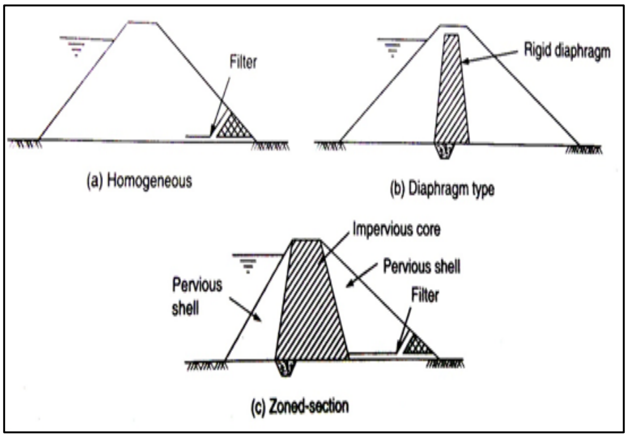


Figure 3.15: Types of earth dams

Zoned section: Currently a popular choice, this type is used in the construction of earthen embankment dams. The dam cross-section is divided into a number of zones. The outer zone should be more pervious so that they have a free draining property (facilitate drainage). The inner zone is made of an impervious material (clayey soil) in order to check the seepage.

Bigger dams will have more than two zones with different soil properties for each zone. Seepage patterns of homogeneous and zoned sections shown in Figure 3.16 explain the importance of an impervious core to prevent seepage.

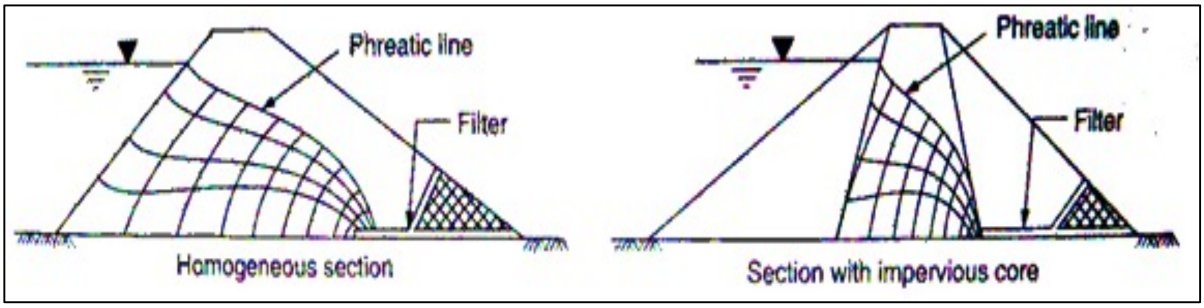


Figure 3.16: Seepage pattern of earth dams

CAUSES OF EARTH DAM FAILURE

Failure of an earthen dam is mainly caused due to piping (constant removal of soil grains due to seepage), over-topping, sloughing loss in shear strength due to saturation, instability of slopes, conduit leakage, slope damage caused by rain and wave actions, settling of the foundation, etc. Possibilities of such failures should be considered while designing the dam.

SEEPAGE CONTROL

Seepage through the foundation causes dam failure due to piping and sloughing. To prevent these effects, seepage must be checked. Underlying seepage also exerts uplift pressure. Effective measures for seepage control include providing deep cut-offs, taken up to the impervious foundation layer (use only partial cut-offs if the impervious layer is at a greater depth), and backfilling the same with compacted clayey soil (black soil). The cut-off width is usually 0.2 to 0.5 times the maximum head. A depth up to 15 to 45% of the total dam height is desirable.

Figure 3.17: Fujinuma dam in Japan failed due to an earthquake

Cut-offs may be diaphragm walls of concrete sheet piles. Other effective measures to check seepage are upstream clay blankets (e.g. Tarbela dam, Pakistan) and grout injections to form a grout curtain (e.g. Ramganga dam, India).

The success of grouting depends on the underlying stratum. In one case where grouting was adopted, it was observed that the grout was oozing out at several spots further downstream. Later, instead of grouting, a deep cut-off trench measuring about 2 metres wide and 18 to 24 metres extending down to impervious strata was created and backfilled with flexible concrete specifically designed for this purpose. The deep cut-off trench revealed deeply seated sand, gravel patches, and gullies at several points along the length of the dam.

» ***Providing drainage***

Drainage in earth dams is usually provided by a horizontal sand filter with an inverted filter at the downstream end as shown in Figure 3.18. At the downstream end, this filter is connected to a rock toe provided at downstream toe of dam. A more effective method of drainage is to install a slant inverted filter or vertical filter at the downstream edge of the impervious core, extending nearly to the top of dam and connected to horizontal filter at the bottom.

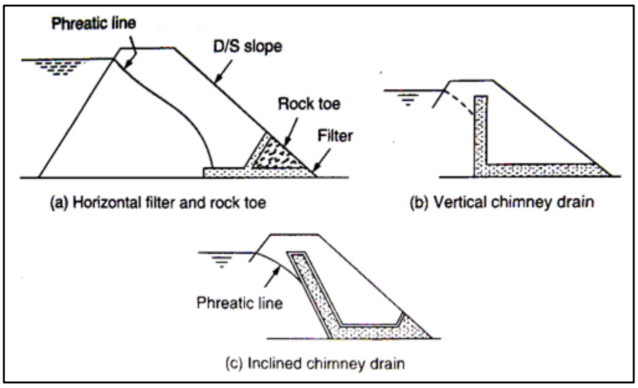


Figure 3.18: Types of drains

UPSTREAM SLOPE PROTECTION

Slope protection is provided on the dam using any one of these methods:

- Dumped stone rip-rap
- Hand-placed rip-rap
- Grouted rip-rap
- Concrete slabs or concrete blocks
- Bituminous paving
- Planting or turfing of slopes.

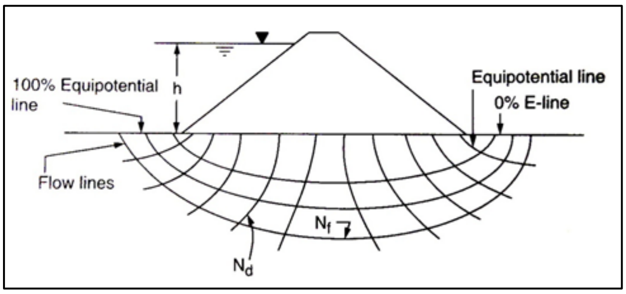


Figure 3.19: Flow net

The first two types are the most commonly adopted. The thickness of rip-rap depends on wave height and impact forces. Considering this relationship, the thickness may vary between 45 and 100 centimetres. Empirically, this measurement should be 1.5 times the average rock size (approximately 20 to 35 centimetres). In some regions (in Russia and neighbouring countries), reinforced concrete cement (RCC) slabs are provided for the protection of slopes. It is safer to provide RCC slabs with a thickness of 20 to 50 centimetres, measuring 10 by 10 metres, for protection in view of high waves of the order 1.75 to 2 metres.

EQUIPMENT FOR EMBANKMENT CONSTRUCTION

Various machinery and equipment is used to move and compact the soil used on an earth dam:

- Excavating equipment, e.g. scrapers, shovels, JCBs/bucket excavators, draglines, etc.
- Hauling equipment, e.g. trucks, dumpers, belt conveyors, tractors, etc.
- Spreading machinery, e.g. bulldozers, graders
- Compacting machinery and equipment, e.g. sheep foot rollers, plain rollers, pneumatic tyred rollers, smooth wheel rollers, vibratory rollers, pneumatic compactors, rammers, etc.

Compaction is done layer by layer at an optimum moisture level. Field samples are taken after laying, spreading, watering and mixing the soil, and the moisture content is checked for each layer. Coarse gravel and sand compaction is done using vibratory rollers.

» *Rock-fill dams*

Rock pieces form the main structural element of this dam type. Rock-fill is done by hand-picking and laying rubble, or by the loose dumping or vibratory compaction of rock-fill in layers. Though usually founded on rocky soil, the rock need not be as strong as that required for gravity dams. These dams exhibit a higher degree of resistance to earthquake forces. Nurek, the highest dam in Tajikistan, is a rock-fill dam.

Figure 3.20: Rock-fill dam

The impervious layer or core shown in the cross-section is classified into two main types.

- **Impervious membrane type:** The impervious layer consists of a membrane of concrete, asphaltic concrete or often, steel. It is placed on the upstream slope of the rock-fill. The membrane rests on a rubble cushion layer placed over dumped rock-fill.
- **Earth core type:** an impervious core (say, of clayey soil) is provided in the body of the dam, as in the case of earth dams. The core is separated from the rock-fill by designed graded filters on either side of the core. The maximum rock size is governed by the transportation method. Current practice is to use smaller stones, but in any case, finer materials are not permitted. Membrane-type cores are expensive. For heads greater than 100 metres, membranes are seldom considered, since the membrane behaviour is not clear under high heads. The thickness of the concrete membrane depends on the maximum head of water. Around 1% of the head is generally adopted. Nominal reinforcement, both ways, is also provided to the extent of 0.5%.

Dumped rock-fill has a natural slope of 1.4:1. The downstream slope is usually kept at this slope or slightly flatter. For dams with a membrane core, the upstream slope is kept at 1.4:1 to 1.75:1. For higher earth core dams, the upstream slope is flatter, up to 2.5:1. The dam section can be zoned when using rock fragments of varying properties. Weaker rock fragments are placed in areas subject to minimal stress.

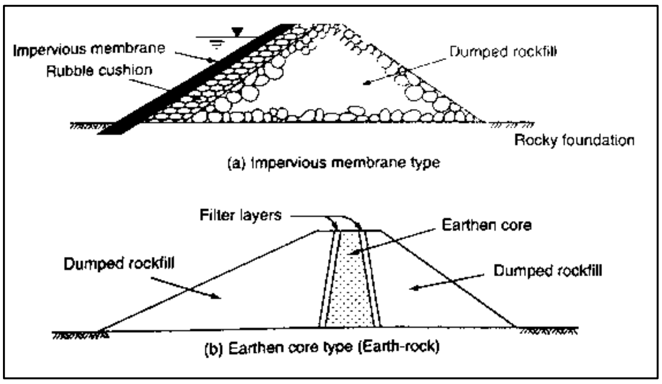


Figure 3.21: Cross-section of typical rock-fill dam

Care is to be taken in the construction of rubble cushioning below the membrane. The stones are laid on stable surfaces and voids filled with quarry spalls. The thickness of the cushion at the top should range from 1.5 to 3 metres. At the bottom of the cushion, the thickness should generally be equal to 5% of the total height. Readers can find additional details for these types of dams in relevant literature.

CONCRETE-FACED ROCK-FILL DAMS (CFRD)

These dams are zoned rock-fill embankment dams sealed on the upstream side by a thin concrete slab, beneath which the face support and transition zones should be provided. CFRDs are better suited to locations where a rock foundation is available close to the plinth area and where suitable core material is not available but rock-fill is abundant. Advantages of CFRDs:

- Shorter ancillary works compared to earth dams
- Settlement of rock-fill is relatively small; rock-fill remains stable even without a face slab
- Multistage construction possible
- Grouting can proceed parallel to the placement of rock-fill
- Face slab construction can be done faster and economically by slip forming
- Drainage galleries in abutments not required
- Parapet can be erected on the crest and connected to face slab
- Only the plinth requires manual labour during foundation clean-up.

Figure 3.22: Dhauliganga dam, India

Another advantage is that the high-friction angles achieved with compacted rocks and gravel fills enables a dam design with steeper slopes and thus a reduced volume.

The thickness of face slab is normally determined as:

$$t = 0.3 + 0.002\ H$$

where H is the head of water.

The slab thickness varies. It has been observed that the hydraulic gradient becomes critical when the hydrostatic pressures exceed a certain height. The hydraulic gradient curve increases with the height of dam. Thus, depending upon the situation, the above formula can be modified. For dams with a height greater than 120 metres, for example, the thickness can be calculated as:

$$t = 0.3 + 0.003\ H \text{ or } t = 0.3 + 0.004\ H \text{ or } t = 0.3 + 0.0045\ H$$

ROCK-FILLED CONCRETE (RFC) DAMS

Rock-filled concrete structures are made by using self-compacting concrete (SCC). Rock blocks measuring about 30 centimetres in width are poured into ready-mix SCC to produce RFC. SCC fills the void between blocks due to its good fluidity and produces RFC on setting. SCC is able to flow under its own weight and completely fill the form work while maintaining homogeneity, even in the presence of congested reinforcement, and then consolidate without the need for vibration. It is very advantageous for use in situations of congested reinforcement, narrow or tight spaces, etc. and the bonding behaviour of the SCC with the reinforcement is better than that of normal concrete. SCC can be produced by using powder content, a viscosity agent, or a mixture of both.

» *Inflatable dams*

The inflatable dam was invented by Jacques Mesnager, a professor, in France. Engineers in Japan further developed the dam to its current status. Modern inflatable dams are composed of air-filled tubes made of laminated rubber and nylon measuring 10 to 25 millimetres in thickness and clamped to a concrete sill. They can retain a maximum water height of about 6 metres, while the smaller dams may retain a height of about 0.4 metres. Span lengths may vary up to 200 metres. These dams are flexible, environmental friendly, and similar in function to small weirs.

» *Dam failure or collapse*

The failure of a dam can be catastrophic and result in considerable damage to property and the loss of life. Analyses have shown that the following design/ construction/operational/natural/human factors can result in the full or partial failure of a dam:

- Inadequate capacity to pass the flood or the malfunctioning of spillway/sluice gates
- Significant seepage, resulting in an eroded foundation or greater uplift forces
- Large amounts of settling in the embankment height or foundation levels
- Seepage along the outlet surface resulting in internal erosion
- Non-homogeneous foundations giving rise to settling
- Liquefaction of soils in embankments
- Cracks due to extreme stresses in the body of the dam
- Unserviceable outlet works
- Failure of the rocky strata in the valley due to water pressure
- Earthquake effects
- Faulty reservoir operation, improper maintenance and inadequate monitoring of the dam
- Harmful vegetation and animal burrows
- Damage to the upstream slope protection surface in earth dams
- Human factors: faulty design, wars, terrorism, sabotage, etc.

» *Dam safety*

The issue of dam safety is complex. The following guidelines, however, are helpful in improving the safety:

- Careful and comprehensive investigation of the foundation geology and soil/rock mass properties using modern exploration methods. Liquefaction potential in soil strata needs be assessed.
- Thorough investigation of flood hydrology and flood probability, and of flood risks during a) construction and b) the lifetime of the dam
- Provision of adequate spillway capacity and bottom sluices and an emergency spillway which can function as a fuse plug
- Strict operating rules for spillways and sluices and rigid enforcement of these rules; frequent inspection of gates and a preventive maintenance programme
- Proper filters for earth dams; adequate foundation treatment for all dams
- Periodic checks for undue or uneven settlement
- Competent design using modern methods of analysis and well-planned regional states of construction
- Independent quality control during construction period and well-defined norms of acceptance criteria to ensure compliance
- Proper and automatic recording of instrumentation and continuous monitoring of the structural behaviour of the dam. Among the dam monitoring instruments are vibrating wire piezometers, settlement monitors, jointmeters, crackmeters, concrete strain gauges, soil deformation gauges, foundation extensometers, total pressure cells, etc.
- Laboratory and in-situ testing of rock masses and interpretation for all dams other than embankment dams
- Careful design of filters in embankment dams and investigation regarding the depression of clayey material used in embankment cores
- Establishing rules for emergency situations and catastrophic natural phenomena such as flash floods, landslides and earthquakes
- Inspection by an expert committee to ensure safety of all existing dams as well as those under construction
- Dam safety is as important as the design and construction of a dam. The International Commission on Large Dams (ICOLD) has been highly instrumental in creating awareness in this respect among the dam builders.

» *Rehabilitation of dams*

Adverse climatic and environmental factors may affect the imperviousness of the upstream face of dam and may even result in cracks (due to shrinkage or the cooling of concrete). Materials used to impart impermeability will age significantly with time and their quality thus becomes impaired. Infiltration can begin to occur through the upstream face, reducing the drainage system efficiency and potentially increasing uplift. Dam stability may become compromised. Rehabilitation of the dam in such a situation is necessary. Cement grouting and/or the construction of a concrete wall, along with various other structural measures, can be adopted for this purpose. One rather modern method involves a fully watertight synthetic geo-membrane on the upstream face of the dam, together with a good drainage system behind it. Accidental seepage occurring through the membrane, accumulating between the liner and the dam body, is then drained by the system. Geo-membranes have also been linked to positive outcomes for underwater repairs and in multiple layers, together with vertical stainless steel strips as anchors.

3.3. SPILLWAYS

A spillway is a waterway provided to dispose of surplus floodwaters from the reservoir of a dam. It is absolutely necessary to provide a safe passage through which the surplus floodwaters are disposed of downstream to avoid danger of overtopping. These structures are invariably provided for all dams and can be likened to safety valves for the dams.

To maximise reservoir storage capacity of reservoir, the spillway should not start discharging until the water reaches a predefined level, called full reservoir level (FRL). Another marker, the maximum water level (MWL) denotes the highest permissible water level in the reservoir. The height differential between FRL and MWL is known as flood lift. At MWL, the spillway should provide a discharging capacity to ensure the overflow is at least equal to the inflow discharge. Then the reservoir level will never exceed MWL.

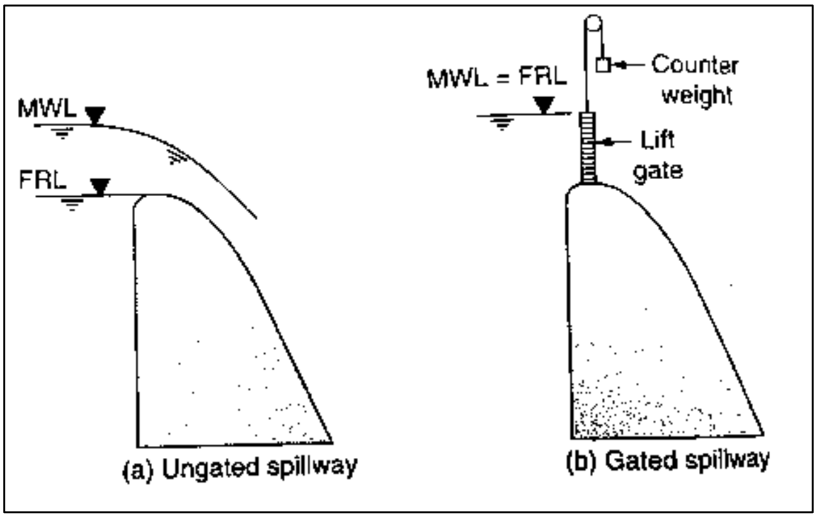


Figure 3.23: Overfall spillways

In a dam structure, a spillway may be installed as an integral component containing an overflow section. Depending on the topography and pragmatic considerations, this section may be located at the centre or at any end of the dam. The spillway may be an independent structure on the upstream basin in a suitable position like a saddle or in a location that is well aligned with the topography so that the downstream of the spillway tailrace channel joins with the river stream on the downstream section of the dam. Overflow spillways can optionally be equipped with a gate on the top of the crest. The crest level determines the FRL on an ungated spillway. However, on a spillway equipped with gates, additional storage capacity is available up to the top of gates during off flood seasons. Here the FRL coincides with the MWL.

DESIGN FLOOD

Spillways of major reservoirs are designed for the probable maximum flood (PMF). This flood level is the result of probable maximum precipitation (PMP). In practice, the spillway is designed for a rare flood event whose return period is as high as 1 in 10,000 years. If it is not possible to work out the PMF value, the highest probable flood that will recur in 1,000 years can be taken into consideration.

It is essential to provide a sufficient capacity so that surplus floodwater is discharged safely, keeping the water level in the reservoir at a predetermined full reservoir level (FRL). An insufficient spillway capacity leads to overtopping of the dam, resulting serious damage or dam failure. An overestimation of capacity will result in an uneconomical design.

TYPES OF SPILLWAYS

Depending on the locations, various possible types of spillways are listed below.

- Free overfall or straight drop spillway
- Overflow spillway
- Chute spillway
- Side channel spillway
- Shaft spillway
- Stepped spillway
- Labyrinth spillway

» Free overfall spillway

A free overfall or straight drop spillway is a spillway in which the control structure is a low-height narrow-crested weir with a nearly vertical downstream. The water flowing over the crest falls as a free jet away from the downstream face. Sometimes the crest is extended in the form of an overhanging lip to direct small discharges away from the downstream face of the spillway. The inner side of the nappe is ventilated to prevent pulsating or fluctuating jets. If no artificial protection is provided on the downstream overfall section, the falling jet usually causes the scouring of the streambed and a deep plunge pool is formed. Floor blocks and an end sill may be provided to help establish a hydraulic jump and thus reduce downstream scour. The free overfall spillway is commonly used for low earth dams. This type is also suitable for thin arch and other dams having their downstream face nearly vertical and permitting the free fall of water. This type is not suitable for high drops on yielding foundations, as the apron will be subjected to large impact forces. For this reason, the freefall spillways are ordinarily used where the hydraulic drops are less than 6 metres. The design details of vertical drop weirs apply.

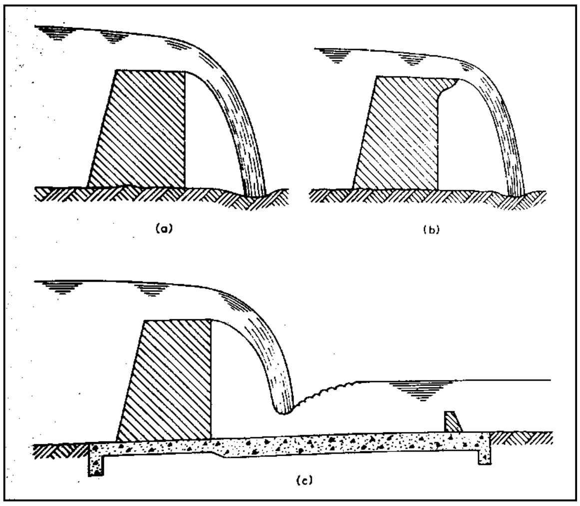


Figure 3.24: Cross-section of free overfall spillway

» Overflow/overfall spillway

These spillways are generally constructed as an integral part of the main dam by using a section of the dam as an overflow section with its crest at FRL. Such spillways should be provided in conjunction with solid gravity dams and in valleys of sufficient width to accommodate the required crest length. The downstream face of the dam may be modified so as to act as a glacis; the toe portion of the dam is suitably designed to prevent erosion due to high velocity flow.

The overfall spillway may be either controlled or uncontrolled. In the former, the control can be achieved in two ways. First, by a permanent gate, which can be lifted when necessary either automatically or manually. Second, by the provision of temporary planks, generally wooden, which have to be introduced towards the end of every flood season. Smaller and less crucial spillways may have such temporary devices, while spillways on major dams are usually equipped with permanent crest gates. Although crest gates increase the total cost of dam, they are increasingly used because they allow water to be stored effectively up to the top level of the gate, instead of the spillway crest level, without increasing the maximum water level of the reservoir in any way.

Since the floodwater passes over the crest of the spillways, the crest must be carefully designed to withstand the maximum flood. An ideal crest profile would correspond to that of the underside of the nappe in case of a sharp-crested weir, as otherwise negative pressures will develop on the glacis. An ogee weir is highly similar to this profile or shape.

» *Chute spillway*

In situations where reasons such as an erodible stream bed, a narrow valley, the use of embankment dams or other reasons, which make it impossible or undesirable to provide the overflow spillway, some sort of independent spillway is required. The chute or open channel or through spillway is one such type that can be employed.

The chute spillway is installed along the abutment of the dam or in a saddle along the reservoir rim. It has a steep-sloped open channel called a chute or trough. This element carries the surplus floodwater from the reservoir. This spillway then takes its name from this carrier channel. Depending on the natural levels of the saddle, this spillway may or may not require a control structure. If the saddle has a higher level than the full reservoir level, it is excavated to the latter level and left to serve as a flat-crested weir. On the other hand, if the saddle has a lower level than FRL, a weir must be built up to that level, which is usually constructed in the shape of an ogee to obtain a high discharge coefficient.

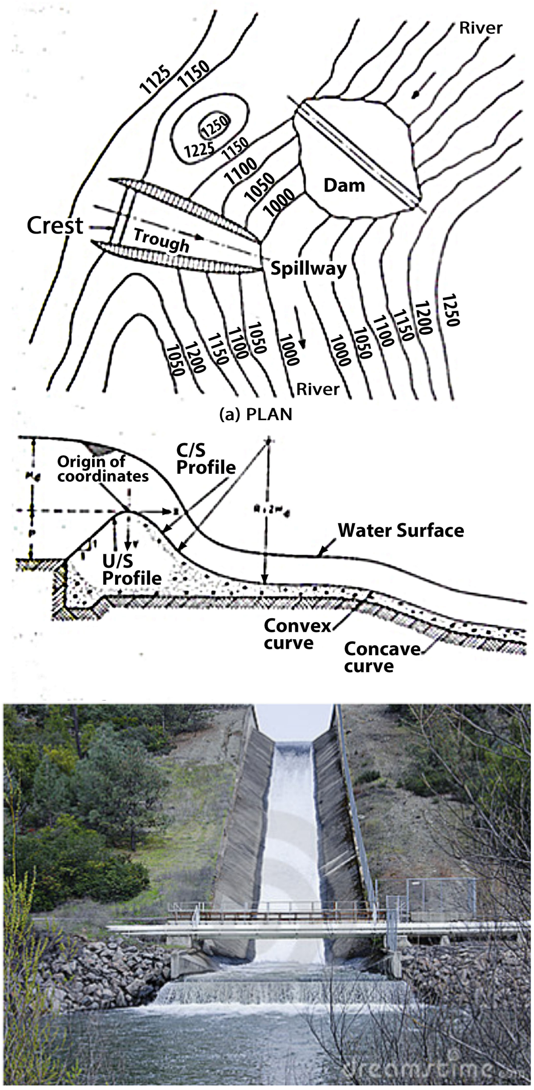


Figure 3.25: Chute spillway on a map (top) and at Napa dam, California (bottom)

» *Side channel spillway*

This spillway type is one in which after flowing over the crest, the water is carried in a channel parallel to the crest. In a chute spillway, water flows at right angles to the crest after passing over it. In contrast, in the side channel spillway, water flows parallel to the crest after passing over it. A side channel spillway is suitable for dams in narrow canyons and for other situations where overflow spillway cannot be provided and where space required for a chute spillway of an adequate crest length is not available. The control structure for this spillway is also a weir, which is usually ogee-shaped. Another situation where side channel spillways can be given serious consideration is in embankment dams. Design considerations require a side channel of sufficient capacity to carry the maximum flood

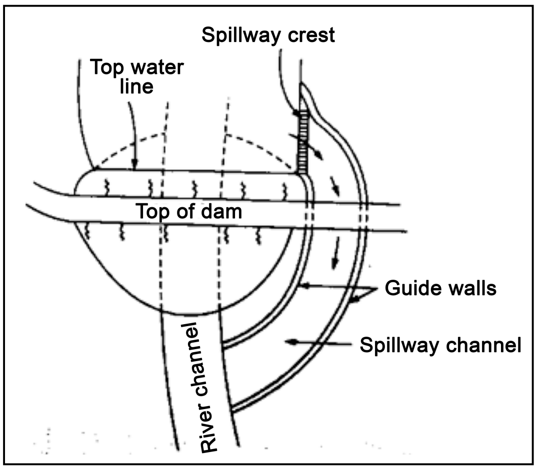


Figure 3.26: Side channel spillway

discharge without submerging weir crest to an extent that restricts the weir discharge capacity. Side channel sections are mostly trapezoidal. The design of the side channel spillway requires the computation of the water surface profile in the channel after the water passes over the crest.

» *Stepped spillway*

In this spillway type, the downstream overflow face of the spillway is constructed as an open channel having a series of steps (falls) that may be horizontal, inclined or pooled (which act as intermediary energy dissipation basins). Energy dissipation is achieved by jet breakup, through the formation of hydraulic jump or by jet mixing on the steps. Such spillways have been found to be especially compatible and economical with RCC dams. The types of flow regimes that could occur in such spillways are nappe flow and skimming flow. When the spillway slope is flat or the discharge is slow, the water flows down in a series of plunges from one step to the next as a nappe flow. In the case of the nappe flow, as the jet falls over the steps, its energy dissipates due to the jets breaking up in air, the formation of partially or fully developed hydraulic jump, or by jet mixing on the steps. In a case of skimming flow, the water flows down the stepped space as coherent stream skimming over the steps and cushioned by the recirculating fluid trapped between them.

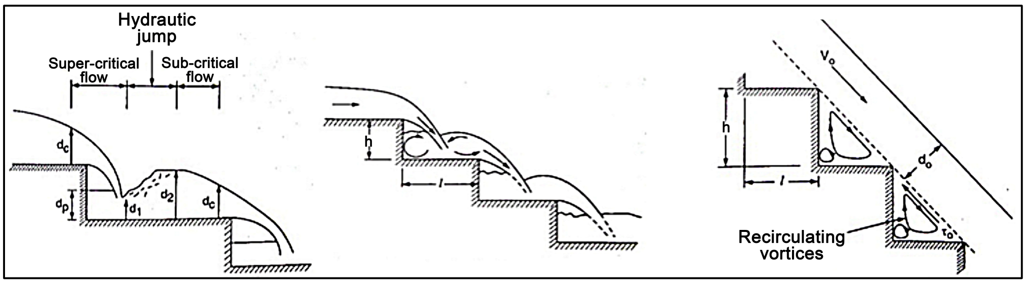


Figure 3.27: Water flow in stepped spillways

» *Labyrinth weir spillway*

In order to pass a greater flood, the length of a spillway crest must be increased. However, for a limited channel width of a channel and the lateral space available as required by a conventional spillway, the crest length of the weir can be increased by providing a labyrinth weir spillway. The crest length is increased without having to increase the top width of the chute, by providing a series of interconnected V-shaped weirs as shown in Figure 3.28. This way, the specific discharge is increased. The weir can be designed with a RCC wall upon a flat bay and in a trapezoidal layout. The vertical wall is generally up to 4 metres high, since it may not be hydraulically suitable for to negotiate large discharges to be negotiated and may require stronger larger reinforcement and base anchors. Other types of spillway, but not discussed are; piano key weir, siphon spillway and breaching section (or emergency spillway).

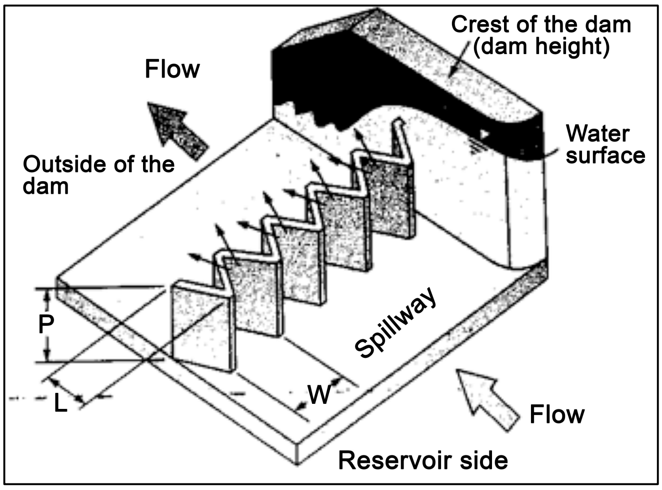


Figure 3.28: Labyrinth spillway

3.4. WATER CONVEYANCE SYSTEM

Depending on the topography of the terrain, various types of conveyance systems have to be provided. From the downstream of the intake structure, the water conveyance system should be provided to convey water up to the forebay. From the forebay, penstocks are provided to carry water up to the turbine inlet.

After the water has passed the intake, it enters the conduit and is conducted from there to the turbines at the powerhouse. Alternately, the conduit is provided up to a certain length, after which the penstocks carry the discharge up to the turbine. Conduits used in hydroelectric practice are either open conduits, which include channels and flumes or closed conduits which include concrete/RCC, steel, and wood stave pipes

HEADRACE CHANNEL

The headrace is an important component of any hydropower scheme. Proper planning, design and maintenance is of great importance if the hydropower scheme is to operate efficiently.

» Alignment

If the powerhouse is separated from the headworks, the water must be conveyed to the delivery point, e.g. a forebay. In this context, in-depth geological studies are important to ensure that the system components are located in safe areas. To properly align the power channel, contour maps as well as topographical maps with the locations of the headworks, powerhouse, penstock, etc., are required. On the contour map, consider the area 150 metres on either side of the centre line of the power channel. Use a map with a scale of 1:20,000 to 1:50,000. The power channel alignment has to be marked with all alternatives, which are to be verified on site for their suitability in all respects.

When the general topography of the terrain is moderate with gentle slopes, open channels are appropriate. The channel alignment is generally taken along the falling contour so as to minimise the head loss due to bed gradient. Loss of head means loss of power. Other considerations include cuts and fills, slopes for cutting sections and embankment sections, the foundation requirement, lining to be provided, etc.

While passing along the falling contour, it is obvious that the cross-section of the channel would show high sloping ground on one side and falling ground on the other. It is important to stabilise the uphill cut slope with some kind of safeguard to prevent the fallout of loose stone blocks into the channel. Certain channel stretches may need to be supplemented with an artificially created embankment on the lower bank. The alignment should not pass through geologically unsafe areas. If unavoidable, remedial measures have to be planned.

To arrive at the most feasible alignment (economically and technically), it may be necessary to consider a number of alternative alignments.

» Channels

CONCRETE CHANNELS

The majority of micro hydropower schemes do not use concrete headrace channels due to their costs. However, these channels rule out virtually all seepage. For short crossings, reinforced concrete channels are sometimes used. High-density polyethylene (HDPE) headrace pipes are generally more cost-effective than concrete channels.

EARTH CHANNELS

These channels are constructed by simply excavating the ground to the required shape. Such channels are used on stable and gently sloping ground. Seepage can be high in the channels depending on the soil type. If there are signs of instability in a headrace section, or if seepage from the channel is likely to contribute to forms of slope instability such as landslides, this channel type should not be selected. However, for headrace alignments on stable ground where seepage is not likely to cause instability, earth channels are the most economical option.

Figure 3.29: Earth channel

CEMENT MORTAR CHANNELS WITH BRICK MASONRY

If an earth channel does not appear to be feasible, the second option to be considered is a cement mortar channel with brick masonry. Compared to an earth channel, there will be less seepage from this channel type. For similar flows, the cross-section of this channel can be smaller than that for earth channels because a higher velocity is acceptable (without causing erosion), as will be discussed later.

Figure 3.30: Cement channel

OTHER CHANNEL TYPES

Timber channels require the use of hardwood and skilled labour. These can be possible for short crossings and aqueducts, or where timber is inexpensive and available is abundant quantities, as in most rural areas in Nigeria.

Other channel types include ferrocement sections, concrete pipes, soil cement channels, etc.

» ***Headrace channel lining***

Headrace channels should preferably be lined, since:

- Lining is more efficient hydraulically, thus ensuring a smaller cross-section and lower frictional forces, which enable higher velocities for the same discharge capacity resulting in improved economy.
- Lining minimises water loss due to seepage.
- Lining reduces the need for repairs and maintenance, and thus increases the availability of the hydropower scheme.
- Lining ensures a lower cost of operation and maintenance.
- Lining minimises weed growth.

Lined channels are capable of withstanding higher flow velocities without scouring. They greatly minimise loss due to seepage and improve the hydraulic flow by reducing Manning's co-efficient, n.

Various types of lining are adopted channels:

- Simple stone paving
- Masonry and brick lining

- Concrete lining (cast in-situ) and pre-cast tile lining
- Shotcrete (cement-gun concrete) lining
- Asphalt or bitumen lining
- Bentonite lining
- Plastic lining

Any material that gives a fairly good water-tightness at a reasonable cost can be considered for lining. The material should not permit any weed growth and lend itself to easy application to produce a smooth surface.

Simple stone paving prevents the erosion of the bed and sides but does not help in reducing seepage. Stones also produce considerable friction. Moreover, masonry and brick linings are relatively rough, though they reduce seepage.

Concrete lining is an extensively used option as it offers low frictional resistance, as well as controlling seepage and erosion. In cases of concreting, construction joints will have to be provided at about 10 metres centre to centre (c/c). Normally, a thickness of 100 to 150 millimetres is used for the concrete lining. For junctions at cross-drainage works like an aqueduct, a siphon, etc., a thicker lining may be required. These are often RCC.

Asphalt and bitumen are water-tight lining materials that are more elastic than concrete, yet comparatively less durable and reliable. Bentonite may also be considered. This solution consists of a mixture of bentonite and wet earth in a compacted layer that is spread over a thin layer of dry bentonite.

CHANNEL DESIGN

When it comes to channel design, there are number of relevant criteria for micro hydropower schemes:

» *Capacity*

The design flow should be accommodated by the headrace channel with adequate freeboard. The latter refers to the difference in elevation between the top of the channel bank and the design water level. The river water level is high during wet seasons and therefore the design flow may be exceeded at the intake, a situation, which calls for spillways and escapes to discharge the excess flows. If falling debris or other obstructions block the channel, the entire flow needs to be safely discharged into a nearby gully or stream before it results in further instability problems.

» *Velocity*

The velocity should be kept sufficiently low to prevent erosion of the channel bed and walls. Table 3.5 shows the recommended maximum velocity for different channel types. When the velocity is too low, aquatic plants and moss will start to grow on the channel and reduce the cross-sectional area. To prevent the growth of aquatic plants, 0.4 metres per second should be maintained as a minimum velocity. In the headrace channel leading up to the settling basin, the velocity should be high enough to prevent the deposition of sediments.

» *Head loss and seepage*

As mentioned earlier, head loss and seepage need to be minimised. Head loss is governed by the channel slope. Seepage can be controlled by choosing the construction materials (earth, mud or cement mortar channels, etc.) that are appropriate for the ground conditions.

» *Side slopes*

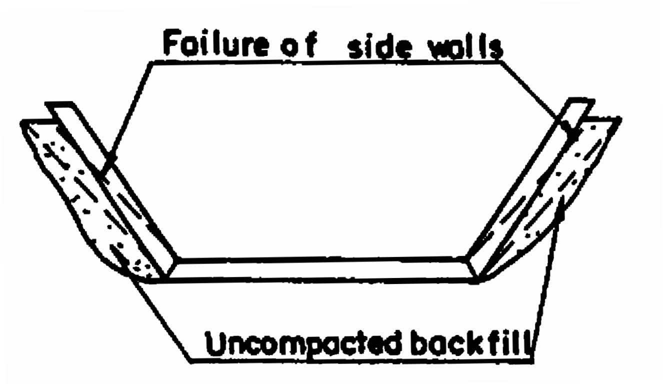


Figure 3.31: Failure of side walls

Channel should ideally be shaped as semi-circles, since this form can convey the maximum flow for a given cross-sectional area. In practice, however, semi-circular channels are difficult to construct and a trapezoidal shape (which is similar to a semi-circle) is often used. For masonry channels in cement mortar or continuous plain concrete channels, rectangular shapes (i.e. vertical walls) are recommended unless the backfill can be well compacted or it is possible to excavate the required trapezoidal section. This is because the side walls of trapezoidal cement masonry and plain concrete channels will depend on backfill for support. Cracks may arise in these walls or at the channel bed (causing seepage), since proper compacting of the backfill behind the walls, as shown in Figure 3.31, may prove difficult. For different channel types, recommended side slopes are shown in Table 3.5.

» *Stability*

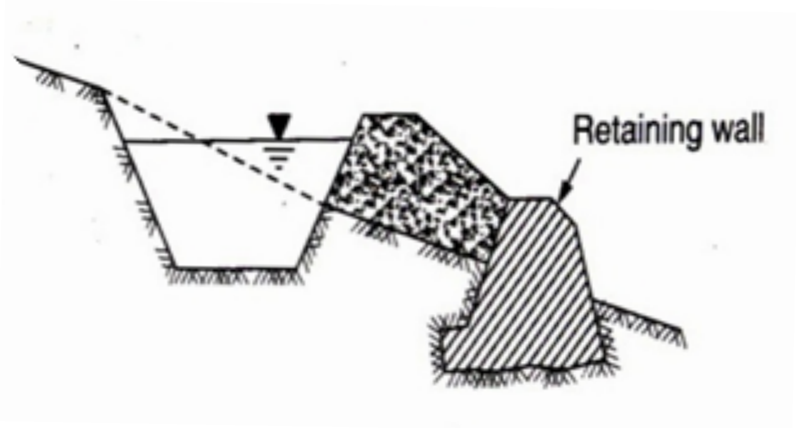


Figure 3.32: Bank supporting wall

Not only should the channel be on stable ground but the areas above and below the alignment also need to be stable. The channel design should address stability issues such as protection against falling rocks, landslides and storm run-off. Covering channels by placing concrete slabs (or flat stones) and some soil cover (to absorb the impact of falling rocks) can be an appropriate solution if a small length of the channel is vulnerable to falling rocks.

» *Economics*

Like any other engineering structure, the channel design should minimise costs. Particularly for long headrace channels, an optimised design will contribute substantially to a cost-effective project on the whole. Optimising the design and costs requires keeping the channel alignment as short as possible (unless longer lengths would help avoid unstable areas and crossings) as well as minimising excavation and the use of construction materials, especially stone and cement. In a micro hydro scheme, for example, a cement masonry channel could be used only for sections containing porous soil and/or seepage, with landslides being the probably result. Earth and stone masonry in mud mortar channels could be used in the same scheme along stretches where seepage and associated risks are not expected. Where the headrace channel makes up a large part of the overall project cost, it would be worthwhile to optimise the channel dimensions.

» *Useful tables for channel design*

Table 3.5: Flow velocities

Material	Maximum velocity to avoid erosion Depth < 0.3 metres	Depth < 1.0 metres
Sandy loam	0.4 m/s	0.5 m/s
Loam	0.5 m/s	0.6 m/s
Clay loam	0.6 m/s	0.7 m/s
Clay	0.8 m/s	1.8 m/s
Concrete	1.5 m/s	2.0 m/s
Masonry	1.5 m/s	2.0 m/s

Minimum velocity: In order to avoid silt clogging the channel, the flow should not be too slow. If the water is always clear, this is not a problem. Silty water should not move at less than 0.3 m/s.

Table 3.6: Side slopes

	Material	Side slope 'N'
a	**Trapezoidal sections**	
	Sandy loam	2
	Loam	1.5
	Clay loam	1
	Clay	0.58
	Concrete	0.58
b	**Rectangular sections**	0

Figures are for 'lining' (strengthening) with the material, but not for sealing. For sealed channels, use the N value of the surrounding soil and not the material.

Table 3.7: Characteristics of cross-sections

Types of cross-section	Wetted perimeter (P)	Top width (T)
Rectangle	$B + 2 \times h$	B
Trapezoid	$B + 2 \times h \times \sqrt{(1 + N^2)}$	$B + 2 \times h \times N$
Triangle	$2 \times h \times \sqrt{(1 + N^2)}$	$2 \times h \times N$

Table 3.8: Roughness coefficient (water depth less than 1 metre)

Channel type	Description	n
Earth channel	Clay, with stones and sand, after ageing	0.020
	Gravelly or sandy loams, maintained with minimum vegetation	0.030
	Lined with coarse stones, maintained with minimum vegetation	0.040
	Vegetated (useful to stabilise soil); water depth 0.7 metres	0.050
	water depth 0.3 metres	0.070
Rock cut	Heavily overgrown, water depth 0.3 metres	0.150
	Smooth and uniform	0.035
	Jagged and irregular	0.045
	Very jagged and irregular	0.060
Masonry and concrete	Stone masonry in mud mortar, dry stone masonry	0.035
	concrete Stone masonry in cement mortar using rounded stones	0.030
	Stone masonry in cement mortar using split stones (dressed)	0.020
	Concrete (according to finish)	0.013 – 0.017
Cement plaster		0.013
Wooden canals	Planed, well-jointed boards	0.011
	Unplaned boards	0.012
	Older wooden canals	0.015
Metal canals	All types	0.020
Mountain streams	Dominant bed material:	
	Gravel (up to 60 mm)	0.030
	Cobbles (up to 200 mm)	0.040
	Boulders (up to 600 mm)	0.050
	Large boulders (> 600 mm)	0.070

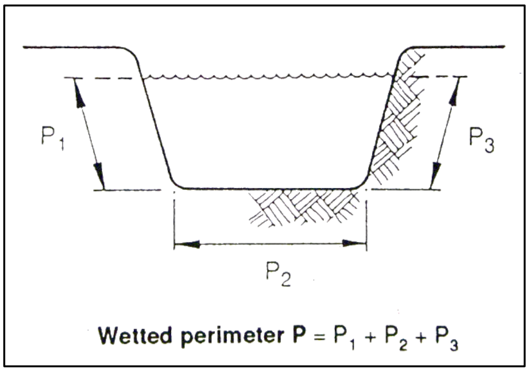


Figure 3.33: Wetted perimeter

Freeboard allowance is the amount the channel is oversized to allow for higher flows than the design flow. The channel is not permitted to spill when carrying excess water, because otherwise it will quickly damage the channel walls and the hillside on which it is built. A common freeboard allowance is 20% – a channel with this allowance can accommodate 1.2 times the design flow. The wetted perimeter (P) is the distance over which the cross-section wets the canal bed and sides during normal flow.

The hydraulic radius (R = A/P) is a nominal quantity describing channel efficiency. If the channel has a large cross-sectional area and relatively small wetted perimeter, this implies it is efficient and can develop the required velocity with little head loss.

Before starting to calculate the dimensions of any particular channel section, you will need to first decide on its length (l), and the material with which it is made, lined or sealed. (*Lined* means that strength is added, whereas sealing only reduces friction).

PROCEDURE

1	Choose a suitable velocity (v). Do not exceed the values given in Table 3.5.
2	From Table 3.5, find the side slope N. For rectangular channels N=0. From Table 3.8, estimate roughness of the wet surfaces (n). Choose a suitable freeboard allowance (F): F = 1.3 is suitable for most purposes. Calculate cross-sectional area A using the equation $A = Q \times \frac{F}{v}$
3	Calculate the channel height (h), bed width (B), and the top width (T). X is an index, which expresses the chosen side slope N in terms of a semi-circle. This minimises friction loss. $X = 2 \times \sqrt{(1+N^2)} - 2 \times N$ $\quad B = h \times X$ $\quad h = \sqrt{\frac{A}{(X+N)}}$ $\quad T = B + (2 \times h \times N)$ Note that for rectangular channels N = 0 and X = 2, so, $h = \sqrt{\frac{A}{2}}$ $\quad T = B = 2 \times h$ for a rectangular channel the width is twice the height. For stable uniform flow in a long channel, it is best to keep the velocity below the critical limit, $v_c = \sqrt{(A \times g / T)}$. Calculate v_c and reset v in step 1 following the rule $v < 0.9\ v_c$
4	From Table 3.7 calculate the wetted perimeter (P). Calculate the hydraulic mean radius from the equation R = A/P. The slope S can be calculated using Manning's equation $S = \left(n \times \frac{v}{R^{0.667}}\right)^2$ You can now construct the channel section with the required slope and cross-sectional dimensions. Note of the head loss (l = length of channel section): Head loss = l x S.
5	The channel is sized for a flow of Q x F. Check that freeboard height is not less than 0.15 metres for unlined channels. Avoid a narrow width of less than 0.25 metres as they can be easily blocked. If the velocity seems low or high, adjust accordingly.
6	Add all the head losses for each section to find total head loss. If this too big or too small, repeat all steps with different velocity.

» *Sediment control*

SEDIMENT PROBLEM

Most rivers contain substantial sediment in the form of gravel, sand or finer material. The type of sediment depends on the river characteristics, catchment geology and the discharge. Intakes are designed and placed to limit the sediment that enters the micro hydro system but cannot eliminate it entirely. Intakes merely prevent the entry of large rocks and cobbles and minimise the influx of gravel and finer sediment. In a suspended state, sediment can cause severe wear on the turbine runner, seals and bearings, since the flow has a high velocity at the runner. This wear reduces the turbine efficiency and eventually causes it to fail. At micro hydro sites without settling basins, numerous turbine runners have been completely destroyed within years after installation.

DESILTING/SETTLING BASINS

Settling basins settle the suspended particles in the diverted river flow. All rivers contain sediment, so it follows that all micro hydro schemes should have a settling basin. For small schemes, this may simply be a widened channel section. A very simple flushing mechanism may be employed, which is acceptable provided that damaging sediment does not reach the turbine.

Figure 3.34: Desilting tank with flush out valve

In many micro hydro schemes, it is possible to combine the settling basin and forebay tank. The following design principles should be kept in mind:

- Length and width should be large enough to allow settling of particles but not too long making it very expensive.
- Deposits should be flushed out periodically. Provision for this should be included.
- Water removed during flushing should be safely returned to the stream or away from other civil works like the penstock, anchor blocks, powerhouse, etc. This prevents soil erosion.
- Sharp area changes and bends, which will create flow separation and turbulence should be avoided.
- Sufficient capacity must be allowed for collection of sediment.

» *Flow separation and turbulence*

Two effects must be avoided in the design of silt basins. These are turbulence and flow separation. Figure 3.35 shows an incorrect design, which encourages both effects. Turbulence should be avoided since it stirs up the silt bed load and does not permit settling. Flow separation is the tendency of a body of water to move quickly through the basin from entry to exit, carrying sediment load with it.

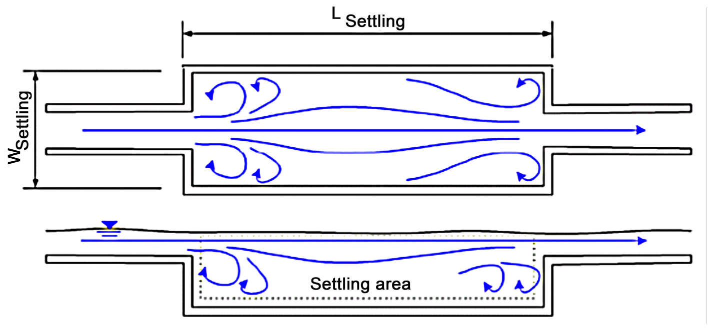


Figure 3.35: Incorrect design – high velocity in centre stream and tubulence in corners

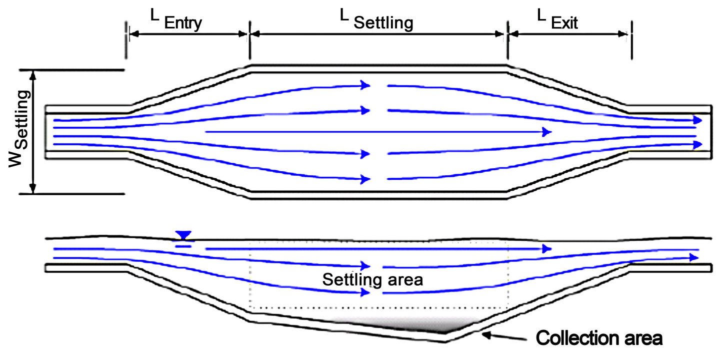


Figure 3.36: Correct design for a settling basin

» *Width and length of settling areas*

Figure 3.36 shows the dimensions that are critical. The full length of the basin is divided into three portions. l_{entry}, $l_{settling}$, and l_{exit}. The central area is for settling. The depth of the basin is divided into two portions, $D_{settling}$ and $D_{collection}$.

As the water enters the basin, it will slow down as the effective cross-sectional area of the basin expands. When the basin is full, the sediment particles will travel forwards faster. The speed at which the particles drop vertically depends on their size, shape, density and the extent of turbulence in water. Assuming non-turbulent water, the settling velocity ($v_{vertical}$) of small particles is given in Table 3.9 below.

In most micro hydro schemes, it is sufficient to remove particles bigger than 0.3 mm in diameter, which have a settling velocity of more than 0.03 metres per second.

Table 3.9: Vertical velocities of particles

Particle size mm	$V_{vertical}$ *m/s*
0.1	0.02
0.3	0.03
0.5	0.1
1.0	0.4

To design a settling basin, choose any width ($W_{settling}$) which is between 5 and 15 times the width of the channel. $D_{settling}$ can be equal to the channel water depth. The length of the basin is then found from this equation:

$$l_{settling} = \frac{Q}{W_{settling} \times v_{vertical}}$$

If the derived length is inconvenient to build, adjust the width until both the width and length present more convenient dimensions. The following three figures explain the working of the desiling tank.

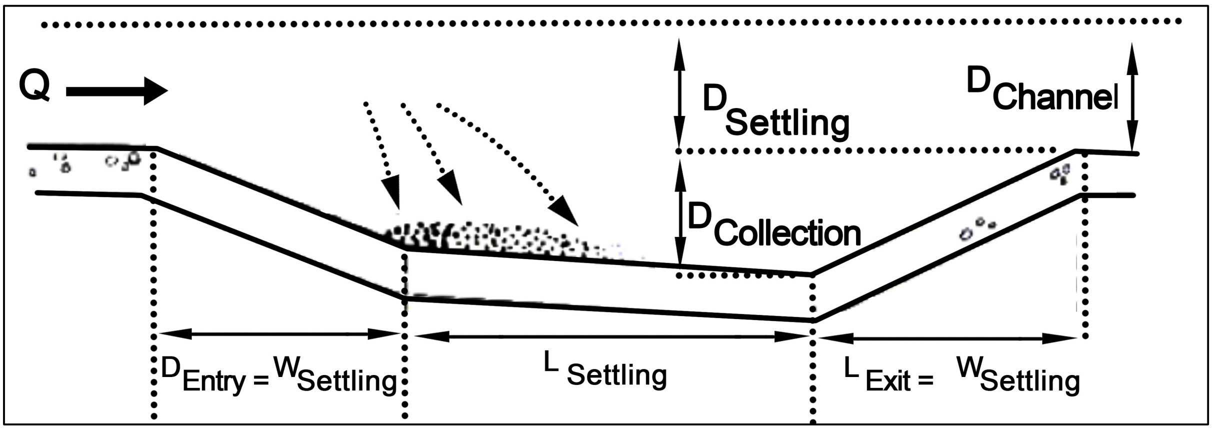


Figure 3.37: Design considerations for settling basin

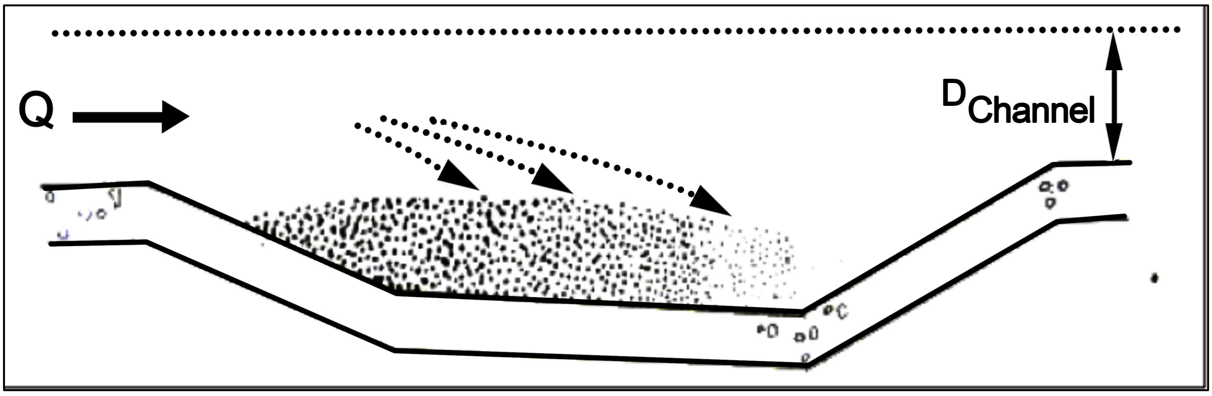


Figure 3.38: Design considerations for settling basin

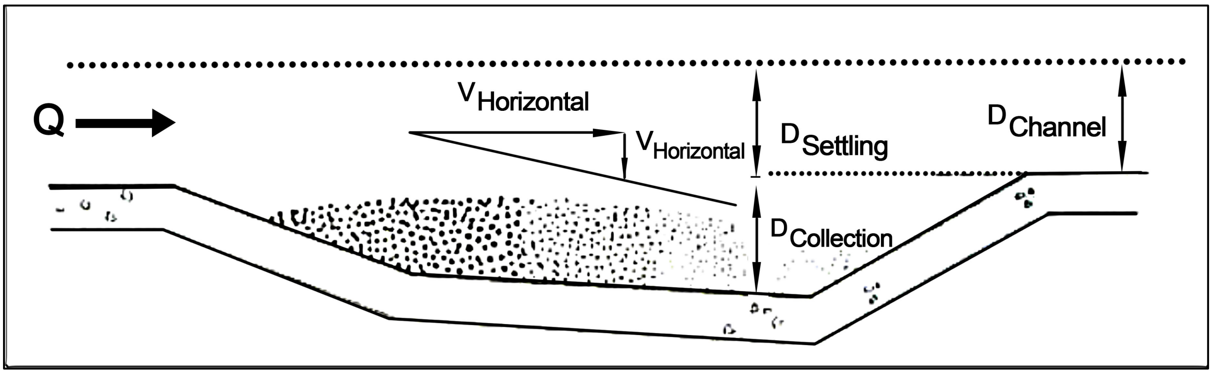


Figure 3.39: Design considerations for settling basin

MEASURING SEDIMENT

Silt load can be easily measured by filling a bucket with water from the stream a number of times. Samples should be taken in a different place or at a different depth. Let the water stand until silt is settled out and then separate and weigh the solid matter. This process is highly uncertain unless there is some confidence that the worst part of the year (following a heavy rain) has been chosen.

Example: Sizing of silt basin

Design flow of a scheme is 0.132 m³/s. The channel depth is 0.5 metres and the turbine manufacturer has told you that particles larger than 0.3 mm should be avoided. Measurements indicate a silt load of 0.5 kg/m³. Design a suitable silt basin with the collection tank depth on the basis of a reasonable emptying frequency.

- ⇒ $Q = 0.132$ m³/s
- ⇒ Choose $W_{settling}$ to be 2 metres. Refer to Table 3.9 and select the $V_{vertical}$ as 0.03
- ⇒ $l_{settling} = \dfrac{Q}{W_{settling} \times v_{vertical}} = \dfrac{0.132}{2 \times 0.03} = 2.2$ metres

Note that in order to avoid turbulence tapered entrance and exit lengths are required.
The design rule for these is to make them each equivalent in length to one basin width.

A possible emptying frequency would be twice a day (every twelve hours).
In twelve hours the intake will suck in:

- ⇒ Silt load = Q x T x S = 0.132 x 12 x 3600 x0.5 = 2850 kg
- ⇒ Density of sand = 2600 kg/m³. Assume a packing density of 50%.
- ⇒ Volume capacity of silt = $\dfrac{1}{0.5} \times \dfrac{1}{2600} = 0.77 \times 10^{-3}$ m³/kg
- ⇒ Required collection tank capacity = $0.77 \times 10^{-3} \times 2850 = 2.2$ m³

Given basin dimensions of 2.0 metres ($W_{settling}$) and 2.2 metres ($l_{settling}$) this implies a collection tank depth of:

- ⇒ $D_{collection} = \dfrac{TankCapacity}{W_{settling} \times l_{settling}} = \dfrac{2.2}{2.0 \times 2.2} = 0.5$ metres

» Forebay

The forebay is a tank at the entrance to the penstock pipe. It allows for the flow to transition from an open channel to a pressure flow, while also maintaining the submergence depth for the penstock pipe to avoid vortex formation and providing storage when the flow fluctuates in the turbine. The forebay tank can also serve as a final settling basin. Sometimes, the forebay structures are even combined with the settling basin. The forebay component should always contain an overflow spillway to allow spilling of the entire flow in case of emergency plant closure and excess flow in case of excessive load changes.

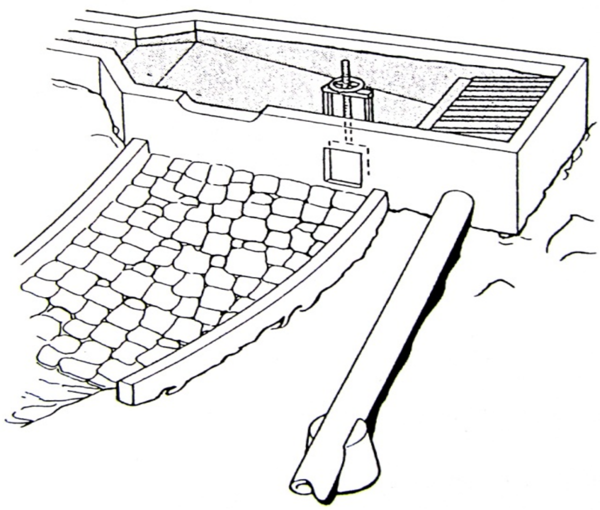


Figure 3.40: Forebay tank

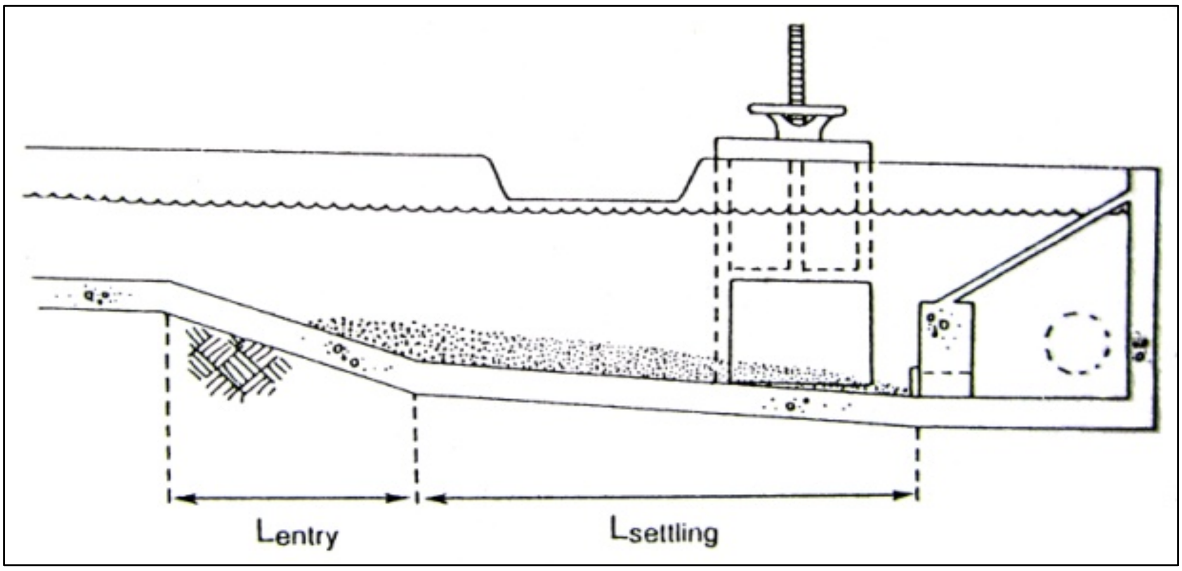


Figure 3.41: Side view of forebay tank

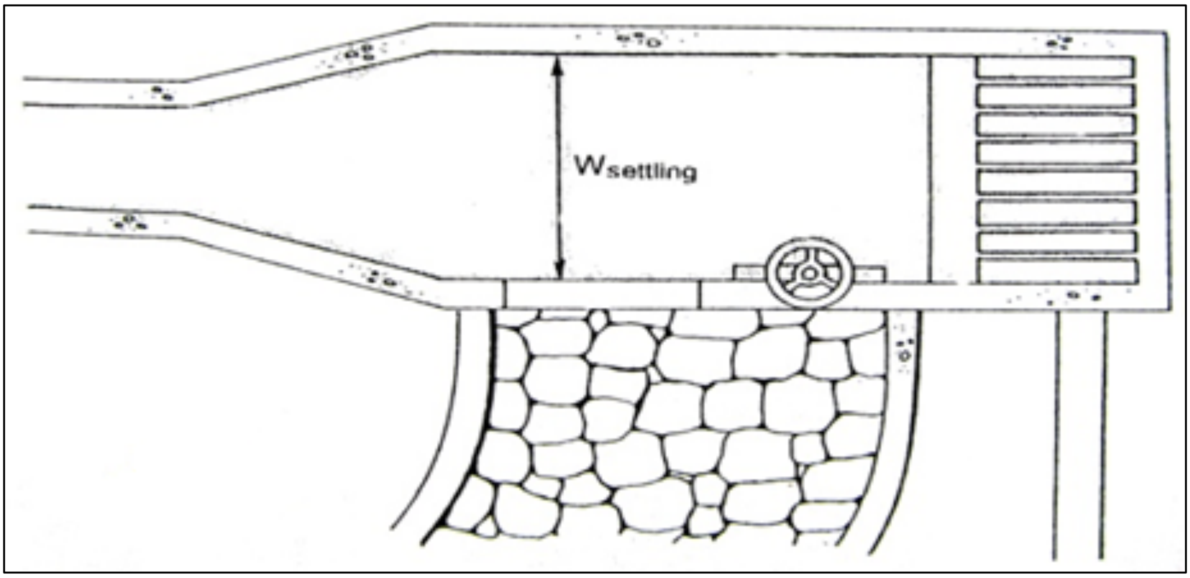


Figure 3.42: Plan view of forebay tank

FOREBAY SIZE

The minimum size of the forebay should be such that a person can get in and clean it. The minimum clear width required for this is 1 metre. Even if the sediment load is not expected in the forebay, it may sometimes reach this structure such as when the settling basin is filled in quickly during the wet season or there is a small landslide. If a person can get into the forebay and clean it occasionally or during the annual maintenance period, limited sediment accumulation will not be a problem.

Whenever possible, the forebay tank should be dimensioned to hold 15 seconds of the design flow, which is safely stored in the tank above the minimum pipe submergence level. This is more critical if the scheme uses a headrace pipe instead of a channel. Minor transient surges may occur in the headrace pipe, which result in an uneven flow. The 15-second storage capacity helps to balance such uneven flows.

» *Trashrack*

The trashrack at the forebay should be placed at a 1:3 gradient for both efficient hydraulic performance and ease of cleaning (e.g. by raking). To minimise head loss and blockage, the recommended velocity through the trash rack is 0.6 metres per second, but a maximum of 1 metre per second could be used.

Note that the trashrack bars should be vertical, since horizontal bars are difficult to clean. The bar spacing should be about half the nozzle diameter for Pelton turbines and half the runner width for cross-flow turbines. This spacing prevents debris from obstructing the turbines and minimises the chances of surge.

» *Cross-drainage works*

In its length, the power channel may have to cross several valleys, small tributaries, local depressions, roads or rail tracks, etc. It is therefore necessary to investigate in detail and plan as to which of the structures need to be provided to safely negotiate across the power channel.

The various cross-drainage works and other structures that may need to be provided are shown in the list:

- Aqueduct
- Siphon aqueduct
- Slab/pipe drains
- Super passages
- Road/railway bridge

The required structures need to be designed to minimize any inevitable head loss in the power channel.

» *Catchwater drains*

The catchwater drain captures and diverts surface water (from storms or land drainage systems, etc.) coming from higher lying areas before it enters the power channel. Water trapped by catchwater drains, in turn, must be diverted to a natural watercourse containing cross-drainage structure. Otherwise the silt/soil, debris, boulders, etc. carried by storm water flowing on the surface enters the power channel and may cause severe damage.

Catchwater drains usually have a sufficient trapezoidal cross-section and excavated material should always be deposited on the downhill side of the drain. These soils should be properly stabilised to prevent them from being washed away during rains.

However, there are certain dangers with catchwater drains that must be carefully considered:

- Surface water usually carries a high amount of silt, debris, etc., and if not properly built, the drain can quickly silt up or become clogged.
- They probably receive less maintenance.
- They may be ploughed up or blocked off by people using the land.

These risks can be reduced if the drains are carefully designed and properly built and maintained.

» ***Landslides or slope protection***

This is an additional factor that must be carefully addressed. Geological studies should be conducted to investigate any existing slip zones. Slopes should be retained through proper measures.

Instable slopes may occur due to fault lines sloping in the direction of the hillside. As rock weathers, it loses cohesiveness and gradually crumbles. Slippage occurring along fault lines will produce surface movement. Attempts to stabilise such a slip zone can prove to be expensive and difficult. Gabions can be installed to anchor the slope, as shown.

WATER HAMMER

A gate, or valve, at the end of the penstock pipes controls the discharge to the turbine. As soon as this governor-regulated opening is altered, the pipe flow has to be adjusted to the new magnitude of flow. This results in rapid pressure oscillations in the pipe, often accompanied by a sound similar to a hammer. Hence, this phenomenon is called *water hammer*.

When designing a water conveyance system for hydropower projects, special consideration has to be given to transient flow conditions, particularly if the conveyance system is long. Such transient flows are due to the governor operation of the turbine, which changes the discharge values. For the penstock pipes, a flow change triggers a high-pressure wave, which sweeps the penstocks and may damage the system. In open channels, this results in surging water waves. The wave velocity travelling upstream can potentially attain velocities faster than sound.

Extreme cases can develop due to

- Switching on the turbine directly to full load from a state of rest
- Sudden removal off the entire normal load by the turbine.

These two cases correspond to full gate opening and full gate closing of the turbine gates respectively. Various other special combinations of the valve operation that may result in water hammer pressure and resonance may also have to be investigated.

RESONANCE IN PENSTOCKS

Besides water hammer pressure, another important aspect of investigation in the penstock design is that of resonance. When subject to a periodic excitation with natural periods of the systems, severe oscillations of the pressure head develops on the penstock piping system. The system is then *in resonance* and faces the risk of damage due to the resulting auto-oscillations. Periodic excitation of the penstock pipe usually occurs due to the rhythmic closing and opening of a penstock valve at the turbine inlet. Besides this, hunting of the governor, vibration of walls, and leakage through faulty seals may be some other factors that can introduce resonance.

CHANNEL SURGES

In hydropower conveyance systems, sometimes long power channels are used to feed the powerhouse. In such cases, the sudden regulation of discharge at the turbine produces a water wave or surge, which moves upstream. Similarly, if the water from the powerhouse is carried through a long tailrace channel, the turbine regulation produces a surge, which moves in the downstream direction. Such surges are the counterpart of water hammer in pipe systems. In the power channel or tailrace channel, it is necessary to consider the worst surge height in determining the freeboard over the normal water level of the channel.

3.5. PENSTOCKS

The penstock constitutes a major expense in the total micro hydro budget. It is therefore worthwhile to optimise the penstock design to minimise both lifetime running costs and the initial purchase cost. To ensure low maintenance costs, care should be taken to place the penstock anchors and supports on stable slopes and to find firm foundations.

The penstock is the pipe, which conveys water under pressure to the turbine. The major components of the penstock assembly are shown in Figure 3.43.

In projecting the cost of the penstock, it is easy to underestimate the expense of peripheral items such as joints and coats of paint. The choice between one penstock pipe material and another can make significant differences in overall cost if all these factors are included. For instance, plastic penstock piping may be cheap, but the joints may be expensive or unreliable in some regions. Guidelines for selecting a penstock are provided in the details of following sections.

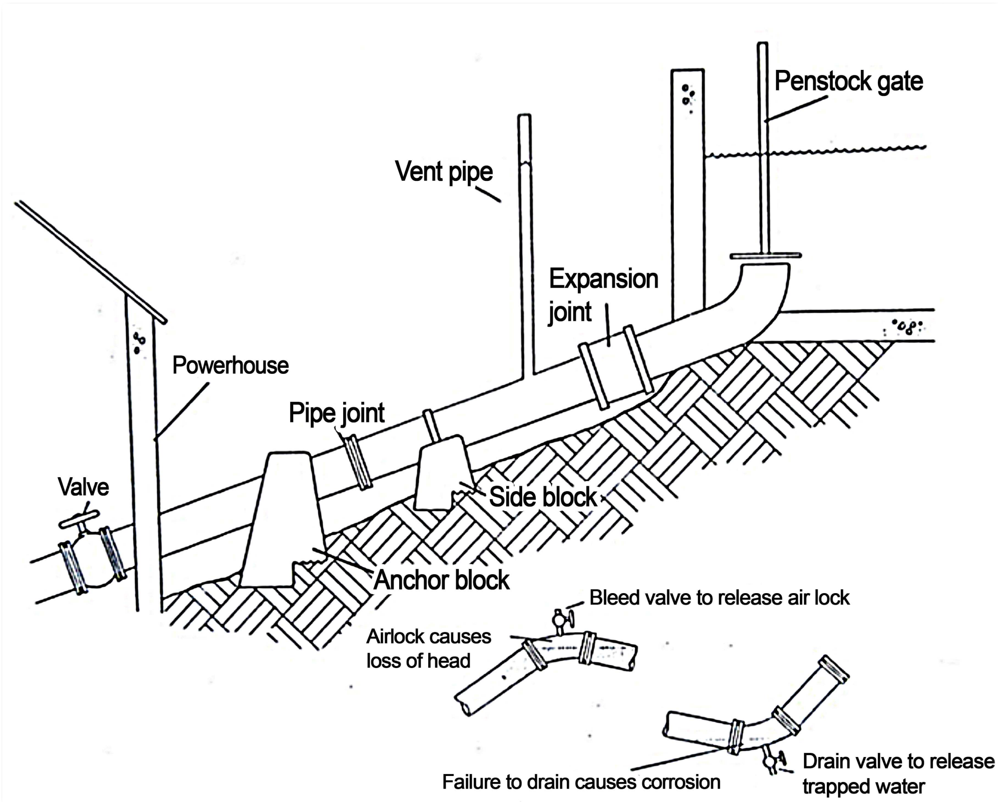


Figure 3.43: Components of penstock assembly

» *Materials*

The following factors have to be considered when deciding which material to use for a particular project:

- Design pressure
- Method of jointing
- Diameter and friction loss
- Weight and ease of installation
- Site accessibility
- Terrain
- Soil type
- Design life and maintenance
- Weather conditions
- Availability
- Relative cost
- Transport to site

The following materials are often used for penstocks in micro hydro schemes:

- Mild steel
- Unplasticised polyvinyl chloride (uPVC)
- High-density polyethylene (HDPE)
- Medium-density polyethylene (MDPE)
- Spun ductile iron
- Pre-stressed concrete
- Woodstave
- Glass-reinforced plastic (GRP)

Of these, three are most commonly used: mild steel, polyvinyl chloride (PVC) and HDPE. This handbook discusses only these three materials (see chapter 1 for details).

MILD STEEL

The most common penstock material in micro hydro schemes is steel. It is relatively cheap, local production does not pose a problem, and it requires no elaborate production machinery. Steel plate is rolled into a cylinder and then the seam is welded. Various diameters and thicknesses are possible. Mild steel has medium friction loss and, when coated with a protective layer, it can have a useful life of up to 20 years. Though it resists most mechanical damage, buried steel penstocks are subject to a greater corrosion risk. The pipes are quite heavy, but can be manufactured in lengths convenient for transport and installation as required. Mild steel pipes may be joined by flanges, on-site welding, or mechanical joints.

UNPLASTICISED POLYVINYL CHLORIDE (UPVC)

An extremely popular choice for micro hydro schemes worldwide, this pipe is relatively cheap, widely available in diverse sizes from 25 to over 500 millimetres, and appropriate for high-pressure applications. The wall thickness can be varied to achieve different pressure ratings; however, the outside diameter remains constant for various ratings in a given diameter. Lightweight and easy to transport and lay, it also has very good friction loss characteristics and is resistant to corrosion. Nevertheless, uPVC is relatively fragile, and prone to mechanical damage (e.g. from falling rocks), particularly at low temperatures.

HIGH-DENSITY AND MEDIUM-DENSITY POLYETHYLENE (HDPE AND MDPE)

MDPE and HDPE pipes offer a good, yet somewhat more costly, alternative to uPVC. Available diameters range from 25 millimetres to over 1 metre. Diameters on the smaller end of this range (up to 100 millimetres) usually come in rolls of 50 or 100 metres. As they are easy to install, they are particularly useful for SHP schemes. MDPE and HDPE pipes have excellent friction loss and anti-corrosion properties, and withstand sunlight without deterioration. The pipes can be joined by heating the ends and fusing them under pressure. Special equipment is required, which presents a disadvantage; for smaller diameter pipes, however, mechanical compression fittings offer an economical option.

» *Sizing and costing*

The penstock is often the most expensive system component. Therefore, some effort should be spent minimizing its cost. The penstock must be strong enough to withstand the very high pressures, which result from a sudden obstacle to the flow of water. These temporary pressures are known as surge or water hammer pressures. They travel throughout the pipe in waves of positive and negative pressure. Figure 3.44 shows how a sudden blockage causes a total head in the pipe that can climb to more than 150% of the normal operating pressure. The negative pressure wave can sometimes destroy the penstock by causing it to collapse, rather than the positive wave (which bursts it). However, the penstock should be designed for the worst positive surge, such that it has a wall thickness suitable to accommodate both the negative and positive surge.

The hydro scheme can also be designed to minimise surge pressure in order to eliminate the need, and the expense, of an increased pipe wall thickness. Using a multi-jet Pelton as the turbine (with separate valves on each jet), for example, can reduce the penstock cost. This is because any sudden blockage will most likely stop the flow through only one of the jets; the resulting surge pressure will be much less than if the full flow had been obstructed. A lower surge pressure can enable the use of a thinner-walled and more inexpensive penstock. To reduce surge, another good design option is to integrate a slow-closing mechanism for the valve used at the turbine end of the penstock (e.g. not a mechanism that uses a quarter-turn lever). This minimises the risk of a high surge pressure and a penstock with thinner walls can be used with greater confidence.

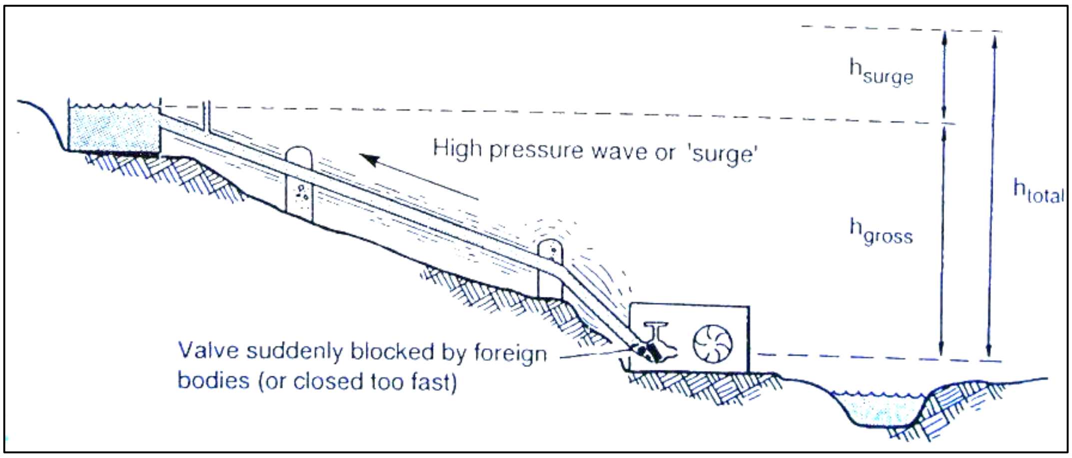


Figure 3.44: Sudden obstruction conditions

To choose the best diameter and type of penstock, a comparison must be made, which depends on the following factors:

- Purchase cost of the penstock and ancillary costs (joining, supports, delivery, installation, design, etc.)
- Penstock maintenance costs. Costs for repainting, drain clearing, etc.
- Power delivered by the penstock after taking into account its friction loss
- Knock-on costs or savings. Refers to the financial implications of the power delivered by the penstock, for instance, considering where a penstock with significant loss may allow the use of smaller drive systems and an alternator or other driven device. These are the electromechanical cost implications.
- Part flow. The relative importance of demand on the hydro installation at part flow or full flow. If stream flow is particularly low during the dry season, but more than sufficient in the wet season, the best penstock may be the one, which is most cost-effective during part flow, and its efficiency at full flow may be allowed to fall off.
- Matching. Identification of the limits to penstock efficiency imposed by the need to match penstock net head and turbine speed at both part flow and full flow, as explained in the text above.
- Peak power. Peak power is the final 10% or 20% of power produced. In some installations, it is particularly important that this power is available because no other power source is present to run all appliances at certain crucial stages of a manufacturing

process. For instance, an agricultural process may require 7 kilowatts (kW) at certain times and not be able to function if only 6 kilowatts are provided. In this case, an expensive low-friction penstock may be a necessary choice. In contrast, this same process may be powered partly by a diesel engine or grid electricity and only use hydropower as an auxiliary energy source. In this case, the extra 1 kW can be provided by the other energy source and a cheaper hydro scheme can be installed without any resulting disadvantages. This factor can be thought of alternatively as the minimum power limit.

- Additional benefit. Positive trade-off received for paying out extra costs for a penstock with less head loss. The additional benefit is usually a mixture of qualitative and quantitative factors:
- 8a. Qualitative additional benefit. The idea of additional benefit covers the previously mentioned benefits, for instance whether or not the improved penstock provides good matching (factor 6) and satisfies the peak power requirement (factor 7). It is possible to have zero benefit or even disadvantages.
- 8b. Quantitative additional benefit. In certain types of installations, the additional benefit can be calculated in actual monetary sums. For instance, your installation serves as a power source to a factory, which is already powered by grid electricity. Every kilowatt it produces earns revenue in the form of savings made by not using grid electricity. If you install a bigger penstock, which has a 3% head loss instead of one with a 7% head loss, you might produce an additional 5 kilowatts. Over ten years this will earn perhaps USD 7,000 in extra income.
- Additional cost. This is the extra amount you pay for a bigger penstock, which has less head loss. It is also the extra money you save or pay in knock-on costs (factor 3 above). Additional cost is always measured in monetary terms, unlike additional benefit. Continuing our example above, we might find that the 3% penstock (the bigger one) costs an extra USD 5,000. It earns USD 7,000, so it is a better choice. However, the smaller penstock that produces 5 kilowatts less may also enable the use of a smaller turbine/alternator set for a cost savings of USD 4,000. The smaller penstock will then be more advantageous, as long as factors 6 and 7 – the qualitative benefits – are also satisfied.
- Total costs of the complete hydro scheme, as projected in approximate terms. This can also be expressed as the target figure for cost per kilowatt delivered by the scheme. If previous schemes in the same region have, for instance, cost USD 1,400 per kilowatt and have been successful in operation and economic return, you might adopt this figure as a target.

» *Useful data for penstock design*

The following tables and charts provide a quick guide for determining the appropriate design of penstocks for micro hydro schemes.

Table 3.10: Physical properties of various commonly used materials

Material	*Young's modulus (E) N/m²*	*Linear expansion co-efficient (a) m/m °C*	*Ultimate tensile strength (S) N/m²*	*Density (ρ) kg/m³*
Steel	200×10^9	12×10^{-6}	350×10^6	7.8×10^3
uPVC	2.8×10^9	54×10^{-6}	28×10^6	1.4×10^3
HDPE/MDPE	$0.2 – 0.8 \times 10^9$	140×10^{-6}	$6 - 9 \times 10^6$	0.9×10^3

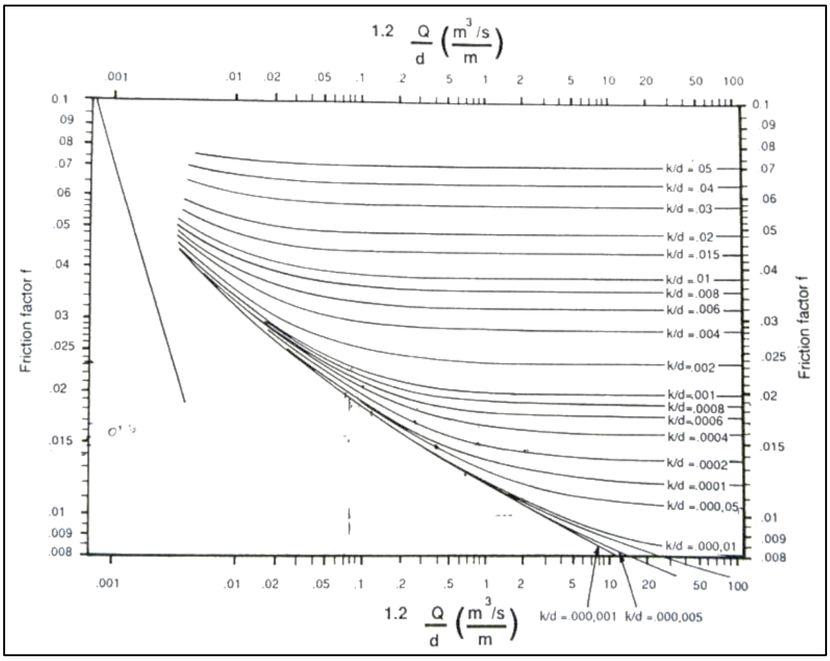


Figure 3.45: Moody chart

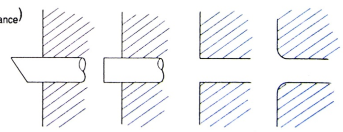
Table 3.11: Pipe roughness values k in mm

Material		*Age/condition* *Good (<5 years)*	*Normal (5 to 15 years)*	*Poor (>15 years)*
Smooth pipes PVC, HDPE, MDPE, Glass fibre		0.003	0.01	0.05
Concrete		0.06	0.15	1.5
Mild steel	- Uncoated	0.01	0.1	0.5
	- Galvanised	0.06	0.15	0.3
Cast iron				
New		0.15	0.3	0.6
Old	Slight corrosion	0.6	1.50	3.0
	Moderate corrosion	1.5	3.00	6.0
	Severe corrosion	6.0	10.00	20.0

Table 3.12 Turbulence losses in penstocks

Head loss coefficients for intakes ($K_{entrance}$)

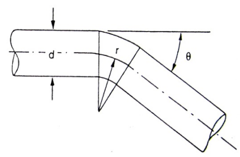
Entrance profile



$K_{entrance}$	1.0	0.8	0.5	0.2

Head loss coefficients for bends (K_{bend})

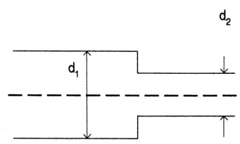
Bend profile



r/d		1	2	3	5
K_{bend}	(Θ = 20°)	0.36	0.25	0.20	0.15
K_{bend}	(Θ = 45°)	0.45	0.38	0.30	0.23
K_{bend}	(Θ = 90°)	0.60	0.50	0.40	0.30

Head loss coefficents for sudden contractions ($K_{contraction}$)

Contraction profile



d_1/d_2	1.0	1.5	2.0	2.5	5.0
$K_{contraction}$	0	0.25	0.35	0.40	0.50

The total turbulence head loss ($h_{turb\ loss}$) is given by

$$h_{turb.loss} = \frac{v^2}{2g} \times \left(K_{entrnace} + K_{bend} + K_{contraction} + K_{valve}\right)$$

Note that there may be more than one bend or contraction. K in all cases represents a head loss coefficient.

» Design procedure –

FRICTION LOSS AND DIAMETER

1	Establish the gross head and turbine design flow value, Q_{net} $Q_{net} = Q_{gross} - Q_{irrigation} - Q_{seepage}$
2	Measure or estimate the length of penstock needed. If you are doing a detailed calculation, use a value measured at the actual proposed penstock site. If you are doing a rapid costing/feasibility study, and an accurate map is available, the penstock length can be estimated very roughly from the map. The penstock length (l) is given by simple trigonometry: $l_{pipe} = \sqrt{l_{horizontal}{}^2 - h_{gross}{}^2}$ Where $L_{horizontal}$ is the horizontal distance from forebay to powerhouse measured on the map. The contours give h_{gross}.
3	Choose a material and establish which internal diameters are available for the penstock, picking one for a first try. This is *d*.
4	Estimate the extent of pipe inside wall roughness (k) or use the normal values given on in Table 3.11. Calculate *k*/*d*, using the same units for the top and bottom. Also calculate 1.2 Q/d, using SI units. Read the friction factor value *f* from the Moody chart (Figure 3.45). See footnote*[1].
5	Calculate the head lost due to pipe wall friction ($h_{wall\ loss}$). $h_{wall.loss} = \dfrac{f \times l_{pipe} \times 0.08 \times Q^2}{d^5}$
6	Calculate the velocity of water in the penstock (v): $v = \dfrac{4 \times Q}{\pi d^2}$ Usually, turbulence losses are minor compared to the effect of wall friction (this is not always true for multi-jet Pelton turbines, since manifold bend losses can be high and should be calculated). If you are doing a quick initial sizing calculation, and you think turbulence effects are minor, skip step 7. Referring to Table 3.11, calculate turbulence losses ($h_{turb\ loss}$) in entrance sections, bends, valves, and other obstructions such as manifolds: $h_{turb.loss} = \dfrac{v^2}{2g} \times \left(k_{entrnace} + k_{bend1} + \ldots + k_{valve1}\right)$ Where *k* is a factor associated with the bend or obstruction, and g is 9.8 m/s^2, add together the friction losses to obtain the total friction loss ($h_{friction\ loss}$): $h_{friction\ loss} = h_{wall\ loss} + h_{turb\ loss}$
7	Calculate the percentage loss of head due to friction and the net head (h_{net}): % loss = $\dfrac{h_{friction}}{h_{gross}} \times 100$ $h_{net} = h_{gross} - h_{friction}$
8	Enter the details in a table. A template is suggested at Table 3.13.
9	Calculate wall thickness as described in the next section.
10	Choose another material and/or diameter and repeat steps 3 to 10.

[1] The Moody chart appears in some textbooks in a slightly different form, with a friction factor 4 times smaller than the values given here. If such a chart is used, the friction factor must be multiplied by 4 before use in the equation for a wall loss given above. In all textbooks, the Moody chart is given in terms of Reynolds number on the horizontal axis. In fact 1.2 Q/d is the Reynolds number for water at a nominal temperature.

WALL THICKNESS

1	Consult a pipe manufacturer's table, and choose a pipe with an internal diameter *d* and associated wall thickness *t*.
2	Calculate the pressure wave velocity *a* from the following equation, using Table 3.10 to find *E*. Remember all units are SI, so the wall thickness t is expressed in metres. $$a = \frac{1400}{\sqrt{1+\left(\frac{2.1\times10^9\times d}{E\times t}\right)}}$$
3	Calculate velocity, surge head (h_{surge}) and total head (h_{total}): $$v = \frac{4\times Q}{\pi d^2}$$ $$h_{surge} = \frac{av}{g}$$ (g is always 9.8 m/s²) $h_{total} = h_{gross} + h_{surge}$
4	If the pipe is steel, it is subject to corrosion and to welding or rolling defects. Its effective thickness is therefore less than the nominal thickness quoted by the manufacturer. To find effective thickness ($t_{effective}$) for steel pipes: Welding: Divide by 1.1 Flat rolled: Divide by 1.2 Corrosion: Subtract for 10 years life: 1 mm Subtract for 20 years life: 2 mm If the pipe is PVC, refer to the pipe manufacturer for low temperature thickness correction factors. For instance, $t_{effective}$ may be 0.5 x t, if temperatures are sub-zero. Avoid bending and point stresses. In tropical climates, if the PVC pipes are carefully laid, $t_{effective}$ = t.
5	Calculate the safety factor (SF) from the equation below, using Table 3.10 to find *S* and remembering that all units are SI, so t is in metres: $$SF = \frac{t_{effective}\times S}{5\times h_{total}\times10^3\times d}$$
6	If the safety factor is below 3.5, reject this penstock option and repeat the above calculation for stronger walled options. In certain circumstances it is legitimate to accept a safety factor of 2.5, but only if the surge head is calculated as shown above and if all of the following conditions are met: ▪ Staff members are experienced with similar pressures and materials ▪ Slow- closing valves and design to avoid *emergency slam* stoppage of flow ▪ Low damage costs and safety risks (gentle slopes, low heads) ▪ Careful pressure testing to total head before commissioning
7	Enter the calculation results in a table such as Table 3.13 together with head loss and cost benefit calculations.

Table 3.13: Template for comparison of penstock options

Parameter	Penstock material uPVC	HDPE	PVC	Steel
Nominal diameter *(mm)*				
Internal diameter *(metre)*				
Nominal wall thickness *(mm)* t				
Wave velocity *(m/s)* a				
Estimated total head *(metre)* h_{total}				
Effective wall thickness *(mm)* $t_{effective}$				
Calculated safety factor (SF)				
SF acceptable?				
Assumed roughness *k (mm)*				
Head loss *(%)*				
Penstock cost *(NGN)*				
Comment				
Assuming one of the pipes as a reference				
Additional cost *(NGN)*				
Additional benefit *(kW)*				
Total cost *(NGN)*				
Total benefit *(kW)*				
Comparison *(NGN/kW)*				

ANCHORS AND SLIDE BLOCKS

An anchor block encases a penstock and limits pipe movement in all directions. Anchor blocks should be included at all sharp horizontal and vertical bends, because forces exist at these points, which will tend to move the pipe out of alignment. Anchor blocks are also needed to counteract axial forces along extended straight sections of the penstock.

Slide blocks are short columns that are placed between anchor blocks along straight sections of exposed penstock pipe. These structures prevent the pipe from sagging and becoming overstressed. However, the slide blocks need to allow pipe movement parallel to the penstock alignment, which occurs due to thermal expansion and contraction.

» Anchor blocks

An anchor block normally consists of reinforced concrete, which is keyed to the penstock. It prevents the penstock from moving relative to the block. Vertical or horizontal bends in the penstock should always contain an anchor block. These components should also be located at the point where the penstock enters the powerhouse, to protect the turbine. Straight pipe sections should contain anchors, each one next to an expansion joint.

For small installations with low to medium head (below 20 kilowatts and 60 metres head) and straight penstocks (buried or exposed), anchors can be designed as follows: place one anchor every 30 metres, by keying the pipe to one cubic metre of concrete for every 300 millimetres of pipe diameter– in other words, a 200 millimetre pipe requires only 0.67 cubic metres of concrete.

The *1-cubic-metre-per-300-mm*-diameter rule can be extended to cover pipe bends by applying this instruction: double the mass for bends of less than 45°; triple the amount of concrete for sharper bends.

For downward vertical bends, the entire concrete mass must bear down on the pipe. Ensure this either by placing the pipe near the bottom of a larger block or by inserting reinforcing bars. This method should not be used for heads that exceed 60 metres, for pipe diameters above 300 millimetres, or for schemes with ratings above 20 kilowatts. A full analysis must be conducted in these cases.

The pipe is best keyed to the anchor by attaching a collar or flange to the pipe at the point where it is bedded in concrete. Glue can be used for PVC pipes; for steel pipes, any object can be welded in place. Steel inside the anchor can be protected from corrosion by tarring and wrapping the pipe in tar sheet (roofing sheet). This will help prevent a small air gap between the steel and the concrete (which will shrink slightly) in which otherwise water would collect.

» *Expansion joints*

Ambient temperature variations affect the penstock pipes, especially if these are metal (e.g., mild steel) and are laid overground. Depending on the ambient temperature, the penstock either expands or contracts. These variations in temperature are especially important if the turbine does not function continuously, or when the penstock is dewatered for repair, resulting in thermal expansion or contraction.

The ideal method for installation of penstocks is to bury them. In which case, the use of expansion joints and concrete anchor blocks can then be reduced or eliminated. However, it is not always feasible to bury the entire length of the penstock pipe due to site conditions.

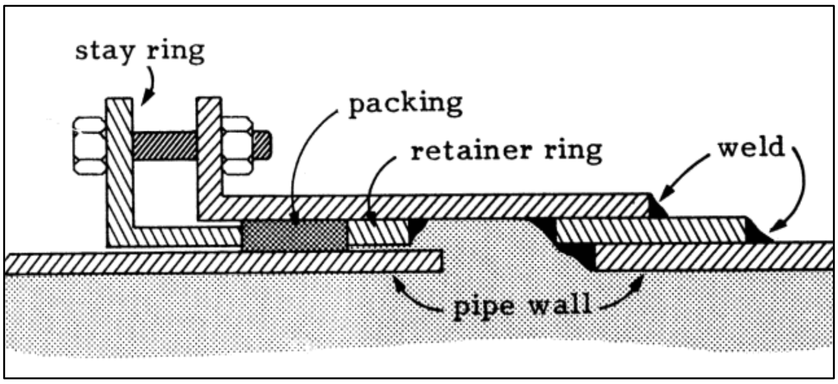


Figure 3.46: Cross-section of expansion joint

This thermal expansion or contraction does not cause additional stresses on the penstock pipe if one end of the pipe is free to move. Since this is not possible between two anchor blocks where both ends of the pipe are rigidly fixed, an expansion joint should be installed in between to allow for this thermal stress. While the stress may be well within the elastic limit of the pipe material and would have little influence on the pipe itself, the thrust caused by the expansion may transfer high stress on the anchor blocks. By providing expansion joints, material and cost can be saved in the the anchor blocks, especially where the site topography and lack of appropriate lifting equipment create difficulties in construction.

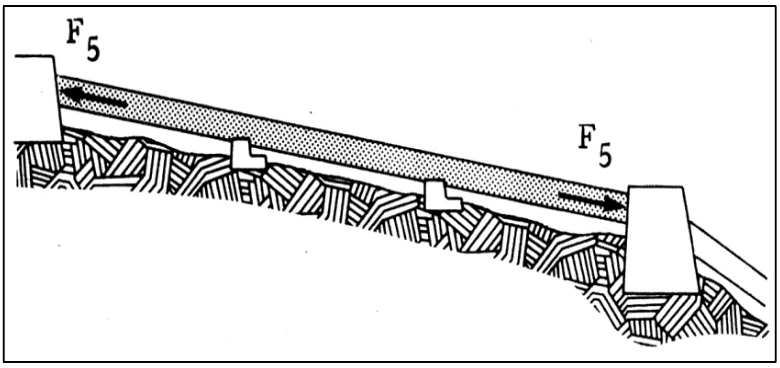


Figure 3.47: Force due to expansion

$$F_e = C_e \Delta T \, E \pi D t$$

Where

F_e = Force due to extension
C_e = Coefficient of extension
ΔT = Change in temperature
E = Young's modulus
D = Penstock diameter
t = Wall thickness

In absence of expansion joints, mechanical joints may be used as they can also be used as expansion joints. Mechanical joints can also be used to connect two different pipe materials such as steel to a PVC. Furthermore, mechanical joints can also handle slight misalignments in the penstock. This facilitates quick installation and allows lateral movement of anchor blocks and foundations, such as may occur due to landslips.

Figure 3.48: Mechanical joint

» *Slide blocks*

A slide block holds the weight of pipe and water and restricts the pipe from moving upward or sideways, but does allow it to move longitudinally. It is used to remove weight-related bending stresses without incurring the cost of a full anchor. The side block also stops the pipe from detaching internally due to upwards or sideways movement caused by internal pressurization.

Slide blocks are unnecessary for a buried penstock; the pipe should, however, be laid into a trench on a bed of sand or gravel of consistent quality, with no big stones which could puncture or cause stress to concentrate at the pipe wall. Accumulated stress will eventually cause the PVC pipe to fail due to vibrations caused by the water movement. Concentrated stress may also cause a buried pipe to burst when a vehicle passes over it.

» *Block spacing*

Anchors and slide blocks both support the penstock. The maximum block spacing may be calculated so that the pipeline does not collapse between supports when carrying a full load. This depends on an accurate prediction of crumpling of the top of the pipe, which is difficult. It is preferable to use Table 3.13 for spacing of supports. If in doubt, one sliding support should generally be used for each pipe length.

For steel pipes, the key criterion is the jointing system. One support per length is needed for any flexible coupling method. For flanges constructed to British Standards (minimum thickness 16 millimetres), the pipes can be considered as one piece. For more cost-effective flange thicknesses (1 or 2 times the pipe thickness), one support per length is needed.

When using PVC, the first rule is to bury these pipes wherever possible. If you must use PVC pipe above ground, it should never be allowed to bend, as it is susceptible to fatigue weakening. Follow the maker's recommendations, and include one support per length whenever possible. Cast iron, ductile iron and concrete pipes commonly use spigot and socket joints. These can withstand only minimal bending, so use one support per length.

Table 3.14: Support spacing in metres

	***Diameter** (mm)*				
	100	**200**	**300**	**400**	**500**
Thickness 2 mm	2	2	2.5	3	3
Thickness 4 mm	3	3	3	4	4
Thickness 6 mm	4	4.5	5	6	6

» Force calculations

For larger hydropower schemes (i.e. those exceeding 20 kilowatts), the quick method outlined is not reliable. Instead, a full calculation of forces must be conducted. To do this calculation, start by distinguishing between the pipe segment upstream of the anchor (with the length l_u), and the pipe segment downstream (with the length l_d). Calculate the forces for each segment separately and then add the results.

Figure 3.49 shows an anchor with a flexible coupling or expansion joint above it that permits longitudinal pipe movement without affecting the pipe further upstream. This coupling marks the end of the upstream pipe at the distance l_u (u for upstream) from the anchor block. To find the weight of water and pipe material in the upstream pipe segment, apply the formula:

$$(W_P + W_W) \times l_u$$

W_w represents the water weight per unit length of pipe in N/m.

The pipe segment weight exerts a vertical downward force. It can be divided into a component acting down the axis of the pipe (longitudinally) which we may call F_4 (see Figure 3.49) and a component acting perpendicular to the pipe which is one part of F_1 (see Figure 3.50). Note that our anchor block (or slide block) must have a base area, which is large enough to stop it from sinking into the ground as a result of force F_1. This also helps it to be *squat* enough to prevent it from toppling as a result of F_4 (or in the case of a slide block, the friction caused by F_2 shown in Figure 3.50, which we can approximately calculate as 0.25 to 0.5 x F_1, depending on the effectiveness of the friction-reduction arrangement).

Another very important force on an anchor block is the hydrostatic force on a bend, which is termed F_3 (see Figure 3.51). If the anchor is fixed to a straight section of pipe, then F_3 is zero. Large hydrostatic forces are commonly experienced in the powerhouse or just outside, where the pipe bends to join the turbine inlets. Care must be taken to provide anchors capable of withstanding F_3.

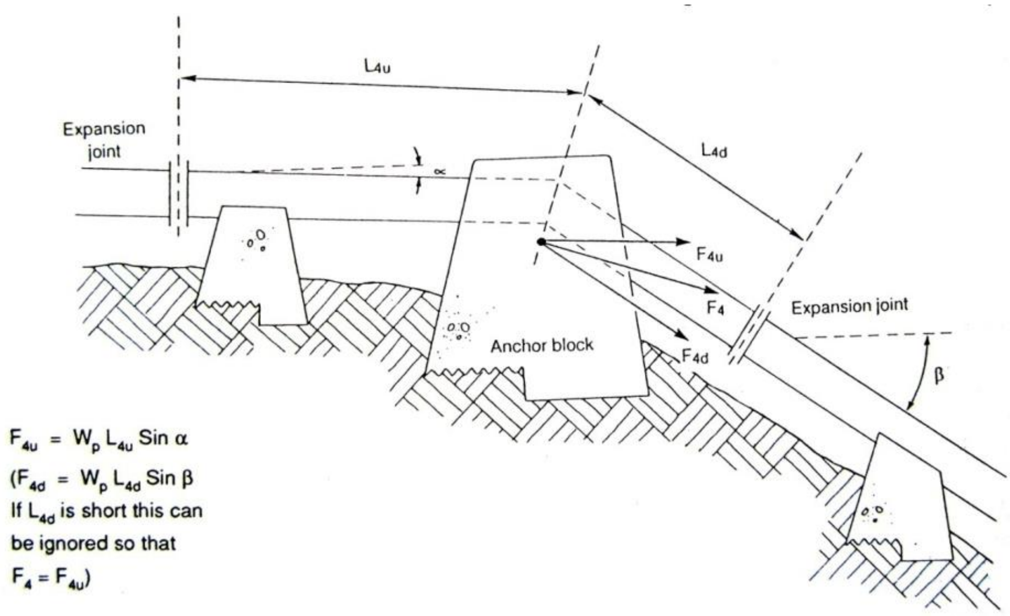


Figure 3.49: Force F4 due to weight of the pipe

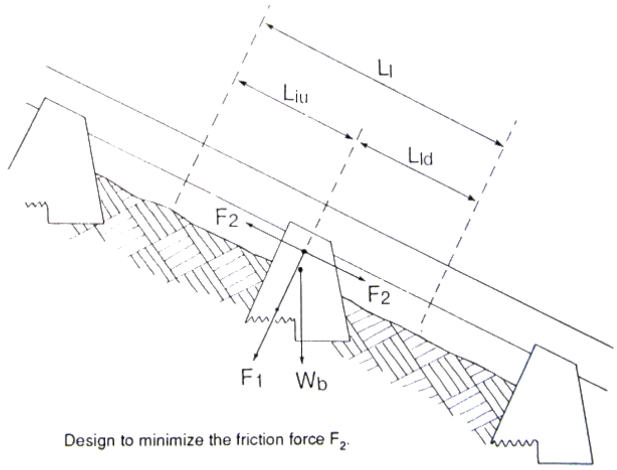


Figure 3.50: Forces acting on a side block

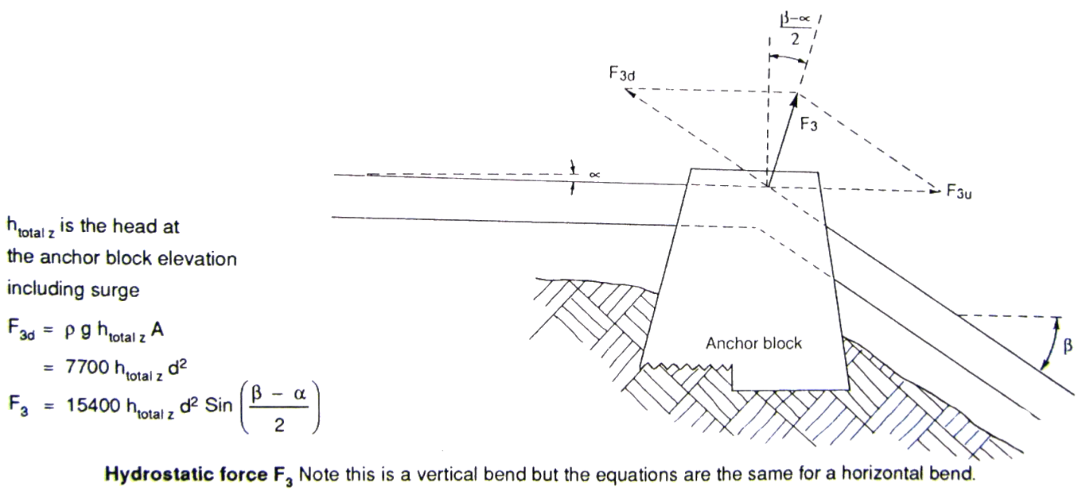


Figure 3.51: Hydrostatic force on anchor blocks

Example

A PVC penstock with an internal diameter of 100 millimetres is chosen for a 2 kilowatt (output power) site. The pipe will be buried at a depth of 1 metre in sandy/gravelly soil. Most of its length will cover a hillside, as shown in the diagram, until the last 5 metres prior to entry at into the powerhouse. It is then reduced to a 50 mm flexible pipe, which takes a sharp 90° bend to form a joint with a pump turbine installation. The head is 50 metres and the pipes are delivered in 6 metres segments. The wall thickness of the pipe is 5 millimetres. The pipe has one horizontal and three vertical bends. Your task is to position and size all blocks.

1 **Slide block:** Not required for support. Instead, a carefully prepared bed of gravel or sand must be laid in the trench as a safe foundation for the pipe. No sharp stones or large items such as bricks should be found in the bed. Include restraints (to prevent movement upward and side to side) while testing the penstock for leaks (before burying).

2 **Anchor block spacing:** Because this is a SHP installation (less than 20 kilowatts), it is suitable to apply the basic rule of thumb for securing the penstock. Though the penstock is to be buried, the soil is loose and it may allow some movement of the pipe, even underground. Therefore, anchor blocks spaced at 30 metres intervals would be a sensible precaution. These blocks will be useful when pressure-testing the pipe before filling the trench. An anchor must be placed at the entry to the powerhouse, for example at the final 45° vertical bend located 5 metres before the powerhouse. Three more anchors in total are needed at the bends, i.e. two more buried vertical bends and one more buried horizontal bend (referred to as the thrust block).

3 **Anchor block sizing:** For straight sections, following the *1-m³-per-300-mm-diameter rule* discussed above, each block would be 0.3 cubic metres, with the dimensions 0.7 x 0.7 x 0.7 metres. For vertical bends, you may double the mass of concrete calculated for straight segments for bends less than 45°, and triple for 45° or sharper bends. In all cases of downward vertical bends, the concrete mass should remain above the pipe to restrain upward movement. This can be accomplished either by inserting steel bars or by increasing the height of the block and placing the pipe near the bottom of the block. Soil is not a reliable medium to restrain movement, so even for the final vertical bend, the anchor block must be sized as for other vertical bends or a second thrust block used.

4 **Thrust block size:** The bend angle is 45°; the bearing area will thus be 1.3 x 0.2 = 0.26 m². The trench is 0.5 metres deep. The top of the thrust block should therefore be at least 0.25 metres below the surface and its height could be 0.25 metres. Its width is therefore 1 metre.

5 **Exposed vertical bend:** This bend occurs after the valve just as the penstock enters the turbine. Note that it has a reduced diameter. Calculate the forces F_{3u} and F_{3d} separately in accordance with Figure 3.51. Horizontal and vertical restraints can then be supplied, for instance, in the form of angle irons bolted to the powerhouse floor, or by connecting the pipe at either end using suitable joints that can withstand the tension force.

6 **Reinforcing bars:** And concrete blocks are strong in compression but weak in tension. To improve their performance in terms of tension, either oversize the block (using gravity to reduce tension forces) or use steel reinforcing bars. In all cases, prepare the concrete correctly to ensure some strength in tension.

3.6. MAINTENANCE OF CIVIL COMPONENTS

A breakdown can cause a lot of tension and it is important to remain calm and communicative under these conditions. Mechanics must communicate with other people who may have more operating experience with the plant or with people who were there at the time of the breakdown. This information will help in coming to a logical conclusion of what is the cause of the breakdown and it becomes easy to rectify the same.

Preventive maintenance is an important aspect of the micro hydro installation and this is often ignored. For example, if there is a leak in the channel, every effort must be made to divert the water along a safe path into the stream until the leak is repaired. If this is not timely done the water will wash away the side of the bank and ultimately there will be an earth slip which will be every expensive to repair. In the case of machinery, reference should be made to the manufacturer literature, which generally highlights the equipment that needs regular maintenance.

Maintenance records should be kept and will serve to monitor the plant. The record should consist of maintenance work carried out and parts repaired/replaced. This should contain the date and a brief description of what happened. This record should also contain the stock of spare parts and any part that is not available should be ordered. Experience has shown that it is best to stock spare parts worth 10% of the total value of the equipment.

Safety in the place of work is also a major consideration. The mechanic or operator should know what is dangerous in terms of materials, conditions and equipment. At all times they should know how and where to use protective equipment in an emergency. Electrical shock can cause injuries and even death. Moving parts such as belts and flywheels can cause serious injury. When working on the plant it is always better to stop the plant completely. The correct tools and also correct or recommended spare parts should be used for both the safety of the plant and also the people. Never override, bypass or overrate any safety equipment installed in the plant for any reason.

Maintenance needs should be identified during the design stage itself and, on commissioning of the projects documented and communicated with the owner and operator.

WEIRS

Weirs are masonry or concrete structures placed across the river from where the water for the turbine is taken. In most cases, this will need very little maintenance other than **when there is heavy rain**, which will bring in some large stones or some vegetation, which may block the water from entering the intake channel. It may also be necessary to remove silt from behind the weir as it may collect and block the entrance of the intake. However, it is not necessary to remove all the silt from behind the weir as in most cases the silt helps to keep a good seal and prevents leaks on the weir.

If there are leaks on the weir, they will show up during the dry periods of the year. It is during this time of the year that it is necessary to get as much water into the channel as is available in the stream. If there are leaks on the weir, this will not be possible. A temporary measure will be to block the leak with sand bags. The **dry periods of the year is also the best time to inspect** the weir for cracks and carry out repairs where required.

Repairs may take the form of patch work on broken or cracked places on the weir. The wing walls on the side of the banks may need repair and it may be a good time to do this too as floor water can erode into the banks if the wing walls are damaged. The erosion of the banks could have serious implications on the working of the plant.

Some weirs have in them a flushing gate or a blank plate, which when open, will help flush the sand out of the weir. This opening will also divert the water when work is done on the weir.

More care should be taken when repairing high weirs as they should remain strong after repair. It is also advisable not to raise the height of the weir as the foundations, etc., as the result may not be strong enough to withstand higher water levels.

INTAKES

The intake is the point from where the water leaves the stream and enters the channel. It is necessary to inspect this area **at least daily** especially during the wet periods of the year. This is because during the rainy season the stream will carry more debris and silt which could block the intake. Some intakes have built in steel bars or trashracks, which will need clearing from time to time.

The intake may also have a wall above the channel, which will limit the flow of water during flood and in some cases it may be a sluice gate that does the same function. It is best to keep the opening to a limit, as it will prevent damage to the channel during flood.

During flood or high flow conditions in the stream, the channel should take only the required amount of water. If excess water is allowed into the channel, it will overflow along the channel causing the sides of the channel to wash away.

Some intakes may be fitted with a short length of pipe, often it may get blocked. Pipes are best cleared by sending water through them. However, if it is blocked with vegetation then it will need some manual cleaning.

OVERFLOWS AND SPILLWAYS

Excess water entering the channel should be put back into the stream and in well designed micro hydro systems this is always done. The excess water should leave the channel at the overflow points and some arrangement should be made to lead the water back to the stream without eroding the soil around the overflow. In most cases, there are some form of steps or some kind of rubble masonry work, which leads the water back into the stream.

In some schemes, there is provision to close these overflows and spillways with stop logs during the dry period. This is to get as much water down the channel during this time of the year. However, one of the dangers of wooden stop logs is that they could be stolen. It would be good precaution to bolt them down in some way. These spillways should not be permanently built up with masonry as it will be a problem during wet periods of the year.

Overflows and spillways should be **inspected periodically** like the rest of the civil works for various damages that may occur over time, such as cracks. These should be promptly repaired or the damage will increase and will become more expensive to repair later.

CHANNELS

Most schemes will have some form of channel to bring the water from the stream into the hydropower plant. There are many types of channels like the earth channel, the rubble masonry lined channel, and the concrete channel. Some schemes have a combination of all three.

One of the important things about a channel is the speed at which the water is moving inside. An earth channel is regarded as slow and the concrete channel is regarded as fast with the lined channel in between.

If the water in any channel is moving faster than designed, it will erode the channel unduly and if the water is moving too slow the channel tends to get blocked with silt. Considering this, it is best to try and keep the flow within these limits. When channels are maintained or repaired, this should be kept in mind.

The channel should be **inspected daily** and any stones or vegetation removed from it. If there is silt in the channel, it should be moved into the silt tank and removed from there.

General maintenance of a channel should prevent leaks and repair leaks as soon as they appear. If leaks are not attended to, they get worse and sometimes wash away the ground that holds the channel. In earth channels, it is often noticed that the sides collapse and block the flow of water.

Drains to carry rain water away from the channel should always be looked at and cleaned or repaired as necessary. In general, drains over the channels are better than drains under the channels because the under drains get blocked during heavy rains and damage the channel. It is always better to drain the rain water away from the channel and into the stream than allowing it to get into the channel.

Aqueducts are steel or concrete section of channel, which are used to span a ravine. The inspection on these sections should include the support and the structure in general. **Steel aqueducts should be painted annually** with anti-corrosive paints and if they leak due to corrosion, they should be repaired or replaced. Some steel aqueducts have rods across the top to strengthen the section and these too should be inspected often.

Sometimes pipes are used in the channel to bring the water to the forebay tank. These are low pressure pipes. As far as maintenance is concerned, these pipes may get blocked occasionally and need to be cleaned. The inspection should take into account the supports and joints along the pipe. The ground both above and below the pipes should be carefully looked at as it has been seen that stones falling on to the pipe from above can damage these pipes.

A more recent problem with channels is that people use the water in the channel to cultivate the land, and very often, the channel is broken or blocked for this purpose. The inspection of the channel should take this into account and if it is a problem, a solution can be to install one or more small PVC pipes (50 mm diameter) in the side of the channel to supply water for cultivation or any other use. The foot path along the channel should be in good condition so that the operator can at all times walk along the channel to inspect same.

SILT TANKS, FOREBAY TANKS AND RESERVOIRS

Silt tanks form an important part of a micro hydro scheme as it is the silt tank that determines the wear on the turbine. As a result, it is wise to keep the silt tank in good shape at all times. The silt is collected in the silt tank because the speed of the water at the silt tank is reduced allowing the silt to fall to the bottom. The collected **silt should be removed daily** through the flushing gate, if not it will collect up to its limit and excess will be passed into the turbine. During rainy periods, the need for silt removal will increase as more silt is carried by the water.

Other than daily removal of the silt, the tank needs very little maintenance. Probably the occasional masonry repair, which could be carried out during the dry period of the year. Flushing valves may need attention as there are moving parts, which will need to be lubricated once a week or so. Care should be taken about the water leaving the flushing valve; this should be taken back to the stream without causing any erosion to the soil under the silt tank.

Sometimes, a silt tank designed a few decades ago is now insufficient to serve the purpose. This is because the land upstream has now been cleared of forest, which causes more silt to flow in the stream. Another reason could be that due to the wear on the turbine over the years, the machine now needs more water than the original design. Keeping this in mind, the silt tank should be cleaned as often as possible and if an opportunity arises, it should be enlarged.

Forebay tanks are where the penstock is connected to the channel. In some schemes, the forebay tank and the silt collecting tank are adjoined while in others, the silt tank is a few metres away from the forebay tank. In the latter case, the channel conveying the water from the silt tank to the forebay tank should be lined. A forebay tank will consist of a trash rack, an overflow or spillway and a sluice gate to shut off the water to the turbine.

The **trash rack needs daily maintenance** or cleaning, as it is here that all the water borne vegetation etc. is stopped from entering the turbine. The trash rack should be cleaned as often as possible and during rainy periods it **may be necessary to clean it twice daily**.

It is very important that only water gets pass the trash rack as any other material could reduce the output of the turbine or could also damage the turbine. Keeping this in mind, it is always better to inspect and repair the trash rack as often as necessary. When repairs are carried out, they should be done in such a way that cleaning becomes easy and for this purpose, **suitable rakes** should be made.

On certain schemes, the channels bring in the water into a large forebay tank where some sort of storage is possible. The penstock is connected to this reservoir and the turbine can be operated for a few hours at a higher power output. The reservoir also acts as a large silt tank and in some cases needs to be cleaned after a period of operation. Generally, trash racks are also placed in the reservoir but needs less cleaning than those in the channels.

PENSTOCKS

Cast iron pipes need very little maintenance other than **attending to leaks** at the joints. This is common with the spigot and socket joints if the pipes have been moved for some reason but can be repaired easily with lead wool. Flange type joints, which are bolted together need no maintenance other than inspection and replacement of damaged bolt-nut assemblies, if any.

Cast iron pipes will remain trouble free as long as they remain on firm ground and their supports are in good condition. If for some reason the ground becomes unstable or a support falls off, the pipe is in danger. If the pipe is damaged or broken, it is difficult and expensive to repair.

The above is also true for the other types of penstocks but mild steel penstocks tend to corrode very fast. The only external protection that can be given is a coat of paint and thus the pipe surface where paint has worn off should be **repainted during the annual maintenance**. PVC or other plastic penstocks on the other hand should be kept away from direct sunlight, this is done again by painting the exposed parts of the penstock or by burying the pipe with a soil cover.

Penstock **supports should be inspected often** and any damage should be repaired immediately, neglecting this could cause extensive damage to the penstock itself. Once supports are broken or damaged, the pipe has to support itself, which it cannot do and therefore it breaks under its own weight plus the weight of water. If pipe of same specification is not available, it is then necessary to replace the pipe with similar pipe, which should be rated to take up the same pressure.

The drainage around penstock supports should be such that the water is moved away from the supports, as running water will erode their foundation. It has been seen on some hydro schemes that poor drainage has caused a few pipe lengths to move from its original position due to earth slips. Penstocks could easily be damaged by falling rocks and wherever possible, loose rocks should be removed from above the penstock.

VALVES

Valves are installed along the penstock and most of them are in the power house, though sometimes they are placed outside. Valves generally **tend to leak with time,** which is not a major problem if the leakage is not high. Water leaks into the power house could be stopped by repairing the sealing arrangement on the valve, but if the valve does not shut the water off completely, it requires a fair amount of skills to undertake repair.

Valves need **occasional lubrication**. Sometimes large valves are provided with a small bypass valve installed to make the opening of the large valve easy. The bypass valve removes the pressure on the large valve and also reduces the wear and tear on it while it is being opened or closed.

Valves installed on the penstock should be fully opened while operation and **should not be used as flow control valves,** which work differently. Flow control valves are generally installed on the turbine.

FURTHER READING

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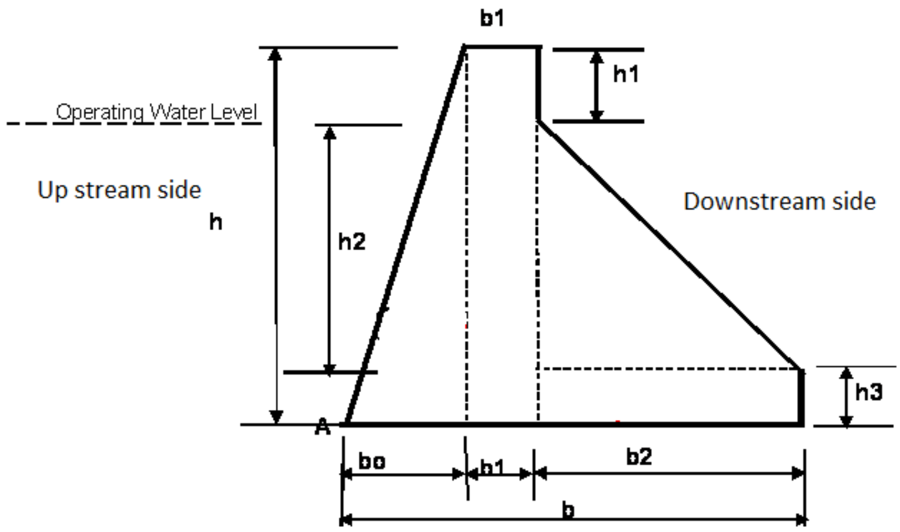
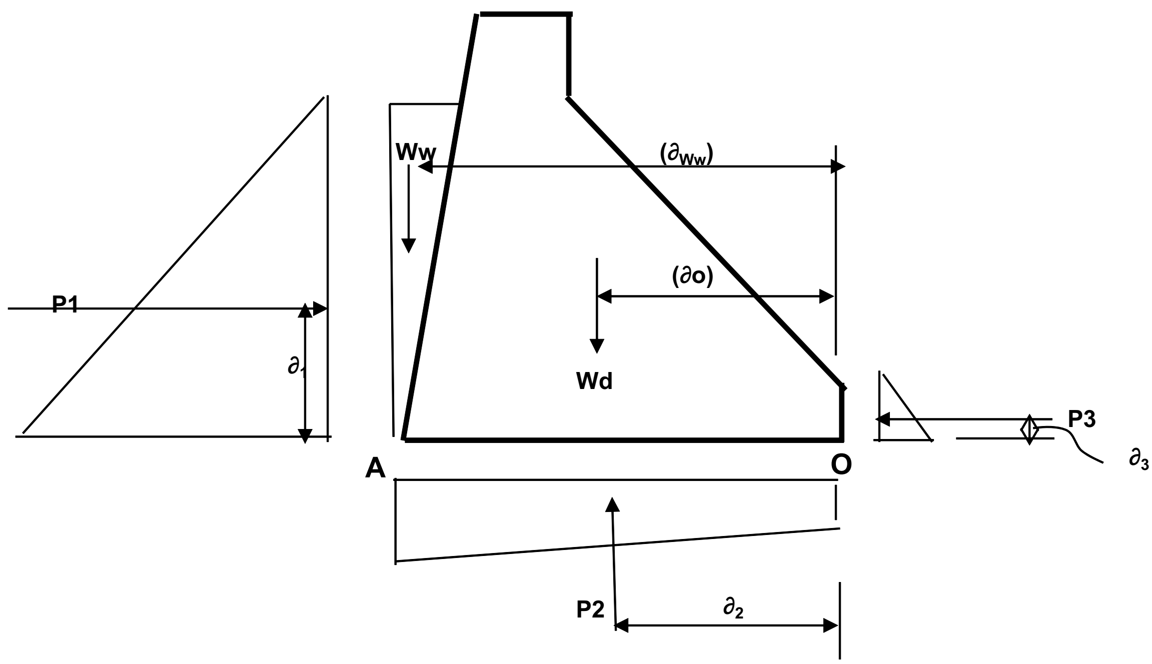
ANNEX

STRUCTURAL CALCULATION OF WEIR DESIGNS

A simple weir will have an upstream slope, middle stem and downstream slope.



FORCES ACTING ON THE WEIR



P1 = Hydrostatic force due to water upstream. This force acts to the right in the figure

P2 = Uplift force due to water under the weir. This force acts upwards

P3 = Hydro static force due to to tail water. This force acts to the left in the figure

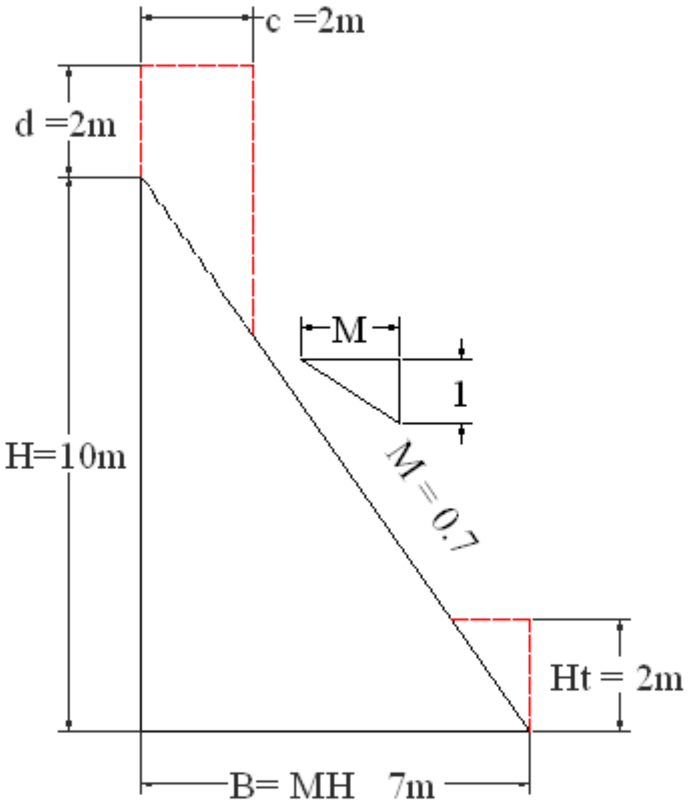
Ww = Weight of the water on the upstream face of the weir

Wd = Weight of the weir

∂ = Corresponding distance from where these forces act

WORKED EXAMPLE

Design a concrete gravity dam of 10 metres height.



Note: Analysis based on 1 metre dam length across the stream.

Given parameters:

$\gamma_w = 10$ kN/m^3

$\gamma_s = 16$ kN/m^3

$\gamma_c = 24$ kN/m^3

$k_{a,s} = 0.3$

$\alpha = 1$

$\eta = 0.6$

Hydro static pressure:

$$P_w = \frac{1}{2} \times \gamma \times h^2$$

= 0.5 x 10 x 10² = 500 kN/metres

Sediment pressure:

$$P_{s,a} = \frac{1}{2} \times K_{a,s} \times \gamma_s \times h^2$$

= 0.5 x 0.3 x 18\6 x 10² = 240 kN/metres

For simplicity, neglect the top weight, then:

Weight of the weir (a triangle) =

$$W_1 = \frac{1}{2} \times h \times B \times \gamma_c$$

= 0.5 x 10x 7 x 24 = 840 kN/metres

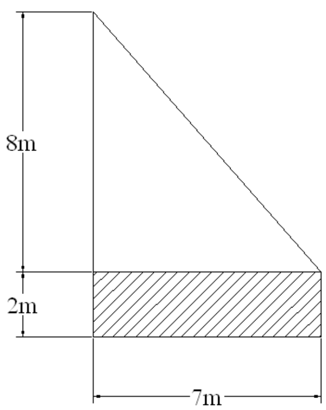
Hydrostatic pressure due to tail-water

= $P_{w.t} = \frac{1}{2} \times \gamma \times {h_t}^2$

= 0.5 x 10 x 2² = 20 kN/metres

Now calculate the uplift force:

$$P_u = P_{u,t} + P_{u,h}$$



for the rectangular section, $P_{u,t} = h \times B \times \gamma$ = 2 x 7 x 10 = 140 kN/metres

For the triangular section, $P_{u,H} = \frac{1}{2} \times h \times B \times \gamma$ = 0.5 x 8 x 7 x 10 = 280 kN/metres

Sum of vertical forces,

$$\sum F_V = W_1 - P_{u,t} - P_{u,h}$$

= 840 - 140-280 = 420 kN/metres

Sum of horizontal forces,

$$\sum F_h = P_w + P_s - P_{w,t}$$

= 500 + 240 - 20 = 720 KN/metres

Now check factor of safety against sliding:

$$FoS = \mu \times \frac{\sum F_V}{\sum F_H} = 0.6 \times 420/720 = 0.35 < 1.0$$

Go for bigger size (metres > 0.7)

Try to increase weir width B

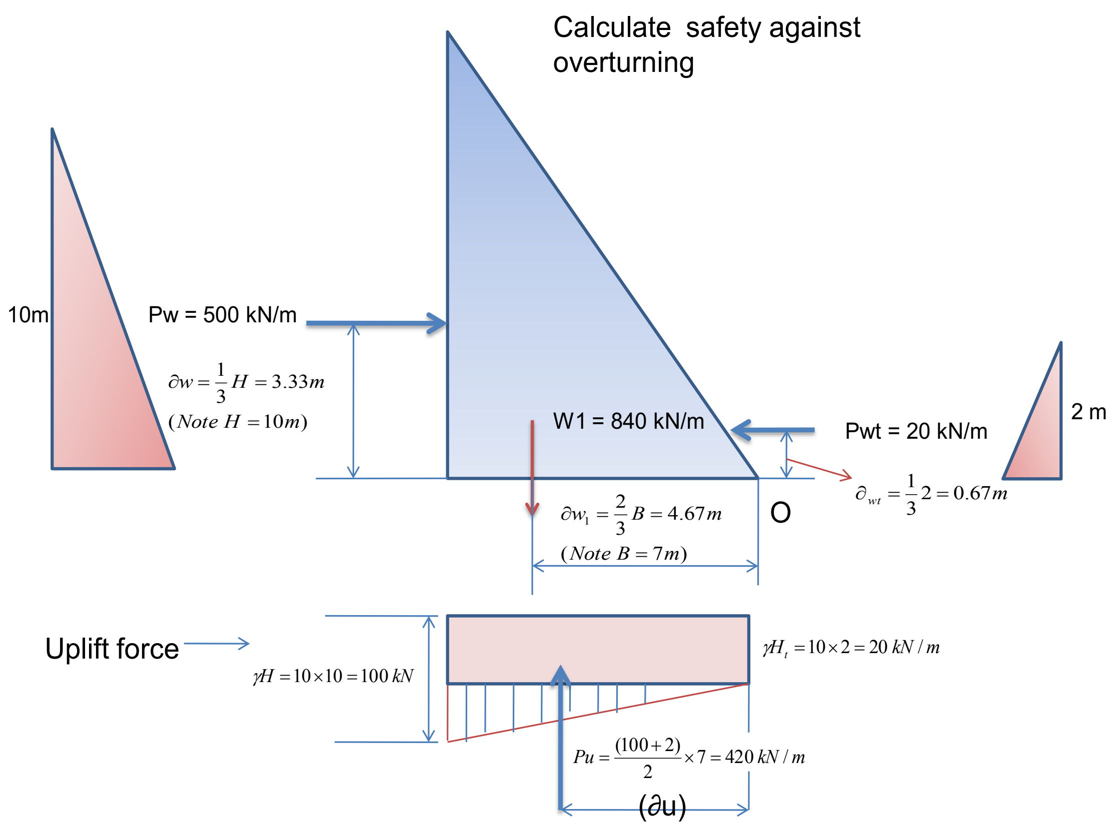
Such that Safety factor is just larger than 1.0, say 1.2

and or

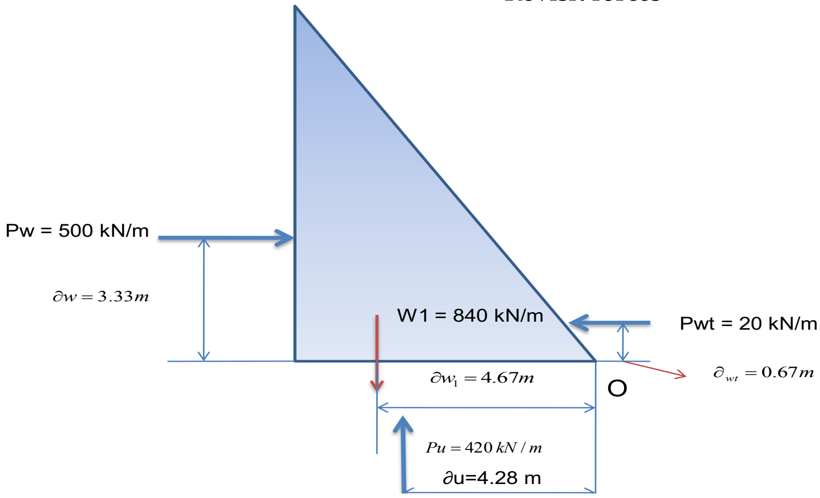
+ provide / grout the upstream face bottom or sheet piling

+ provide drainage gallery

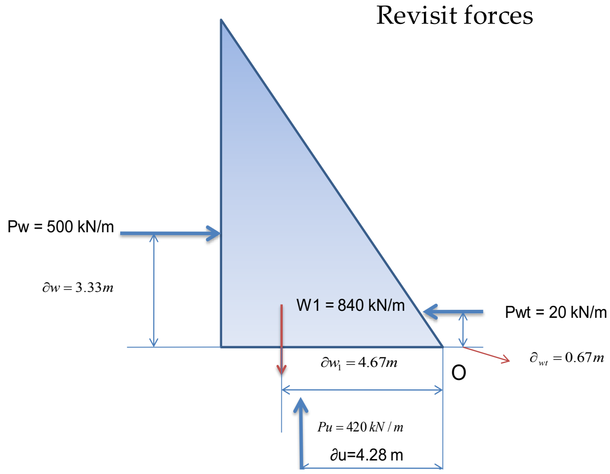
+ provide inclination towards upstream



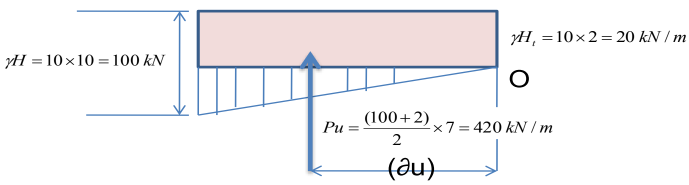
Revisit forces



Revisit forces



Calculate the distance from 'O' where the uplift force acts



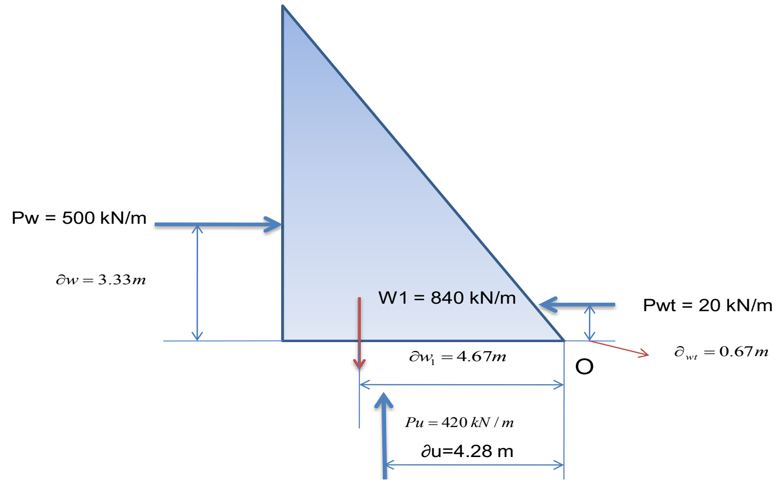
Area of rectangle

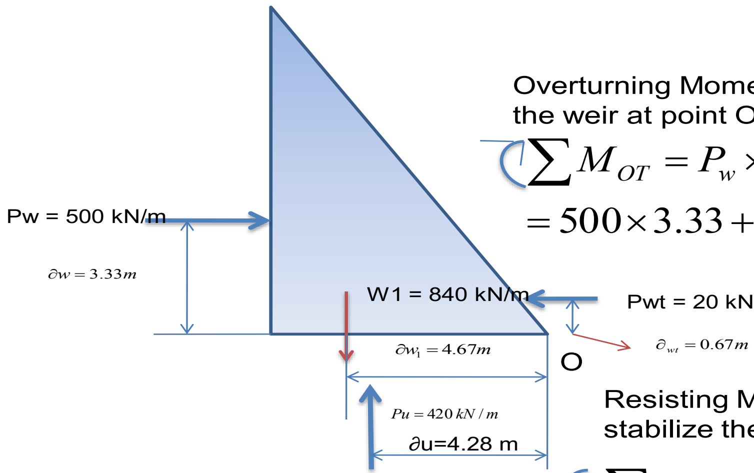
Area of Triangle

$$\partial_U = \frac{(20\times 7)\frac{7}{2} + (0.5\times 80\times 7)\times \frac{2}{3}7}{(20\times 7)+(0.5\times 80\times 7)} = 4.28m$$

Now we have all the forces and the locations where they act

Revisit forces





Overturning Moment: Moment that will turn the weir at point O:

$$\sum M_{OT} = P_w \times \partial_w + P_u \times \partial_U$$

$$= 500 \times 3.33 + 420 \times 4.28 = 3196\ kNm$$

Resisting Moment: Moment that will stabilize the weir at point O:

$$\sum M_{stb} = W_1 \times \partial_{w1} + P_{wt} \times \partial_{Ut}$$

$$= 840 \times 4.67 + 20 \times 0.67 = 3936\ kNm$$

Factor of safety against overturning

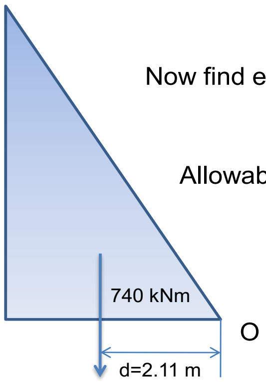
$$FoS_{OT} = \frac{\sum M_{Stb}}{\sum M_{OT}} = \frac{3936}{3196} = 1.23 > 1.0 \quad OK!$$

Therefore, structure is safe against overturning

Now check for sinking or Bearing Pressure

Unbalanced Moment $\sum M = \sum M_{STB} - \sum M_{OT} = 3936 - 3196 = 740\ kNm$

Location where unbalanced Moment acts $d = \frac{\sum M}{\sum V} = \frac{740}{350} = 2.11\ m$



Now find eccentricity $e = \left|\frac{B}{2} - d\right| = \frac{7}{2} - 2.11 = 1.39\ m$

Allowable eccentricity $e_{allowable} = \frac{B}{6} = \frac{7}{6} = 1.17m$

Since actual eccentricity is more than allowable eccentricity, the width is too small!!

Now calculate the bearing pressure using the following equation:

$$P_{base} = \frac{\sum V}{A_{base}}\left(1 + \frac{6e}{B}\right) = \frac{350}{7 \times 1} + \left(1 + \frac{6 \times 1.39}{7}\right) = 110\ kN/m^2$$

Generally allowable bearing pressure in soil is ~150 kN/m^2 and in bedrock it can be ~400 kN/m^2.

Therefore, the structure is safe against sinking.
However, it is not safe against sliding and the eccentricity is not in the middle third of the base → both indicating a larger base width (B) is required!!