1. Components of small scale hydropower

General

Depending on the specific site typical components of small hydropower plant consist of the following:

- Civil works components (headwork, intake, gravel trap with spillway, headrace canal, forebay, penstock pipe, powerhouse and tailrace)
- Powerhouse components (turbines, generators, drive systems and controllers)
- Transmission/distribution network

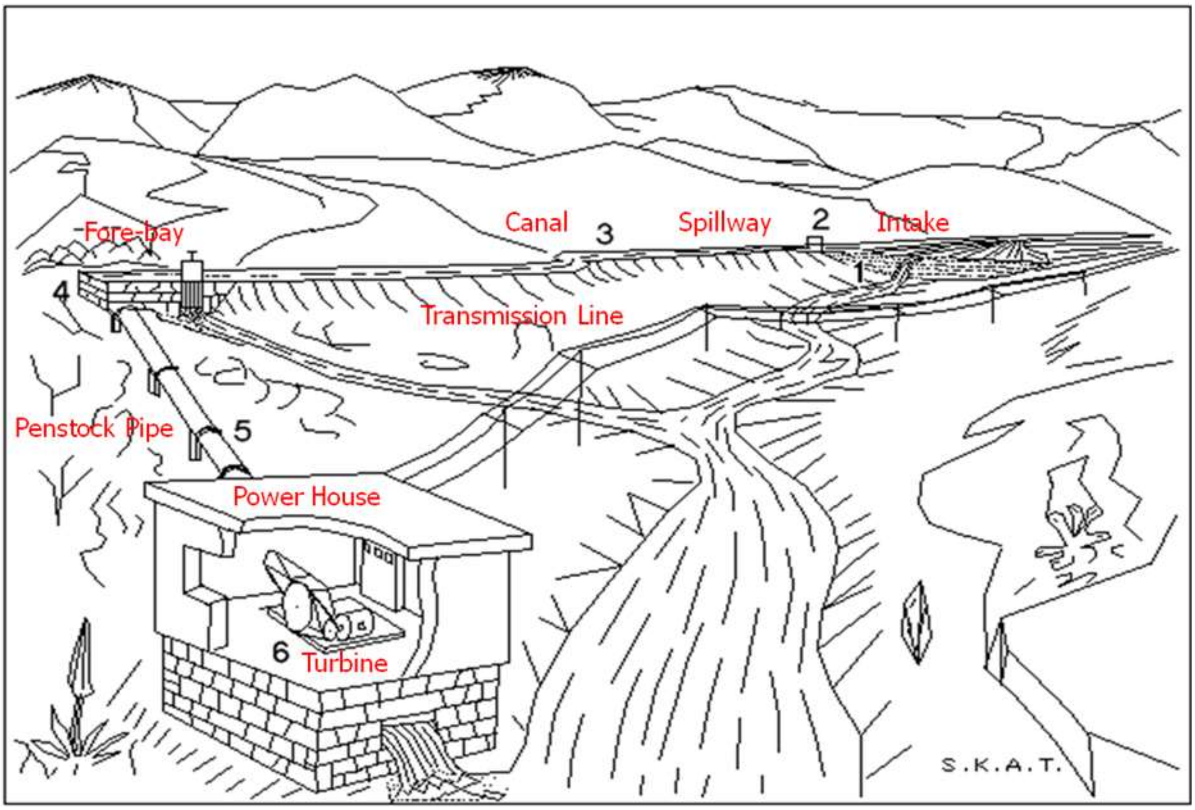


Figure 1. 1 components of micro hydropower

1.1 Civil Works Components

Civil works structures control the water that runs through a Small scale -hydropower system, and conveyances are a large part of the project work. It is important that civil structures are located in suitable sites and designed for optimum performance and stability. Other factors should be considered in order to reduce cost and ensure a reliable system, including the use of appropriate technology, the best use of local materials and local labor, selection of cost-effective and environmentally friendly structures, landslide-area treatment and drainage-area treatment.

Figure 1. 2 civil work components

1.1. Head works

Head works consist of the weir the water intake and protection works at the intake to safely divert water to the headrace canal.

1.1.1 Intake

The intake conveys the required flow of water from the source stream and diverts it in to the headrace of the Small scale -hydropower system. It is designed and located precisely to ensure that the full design-flow rate goes to the turbine.

Because many Small scale -hydropower systems are run-of-river systems, a low-head dam or weir could be used to hold back the water in order to provide a steadier flow of water, depending on the site.

Figure 1. 3 Intake

1.1.2 Headrace Canal

The headrace canal carries the design flow from the intake to the forebay. Generally, the canal runs parallel to the river at an ever-increasing difference in elevation, which gives the Small scale -hydropower system its head. The canal cross section and alignment should be designed for optimum performance and economy in order to reduce losses due to leakage. It could be an open channel of efficient section to transport the water into the forebay.

Figure 1. 4 Head race canal

1.1.3 Forebay

The forebay tank connects the channel and the penstock. The tank allows fine silt particles to settle before the water enters the penstock. A fine trash rack is used to cover the intake of the penstock to prevent debris and ice from entering and damaging the turbine and valves.

Figure 1. 5 Forebay

1.1.4 Penstock Pipe

The penstock pipe transports water under pressure from the forebay tank to the turbine, where the potential energy of the water is converted into kinetic energy in order to rotate the turbine.

The penstock is often the most expensive item in the project. It is therefore worthwhile to optimize its design in order to minimize its cost. The choice of size and type of penstock depends on several factors that are explained briefly in this section. Basically, the trade-off is between head loss and capital cost.

Head loss due to friction in the penstock pipe depends principally on the velocity of the water, the roughness of the pipe wall and the length and diameter of the pipe. The losses decrease substantially with increased pipe diameter. Conversely, pipe costs increase steeply with diameter. Therefore, a compromise between cost and performance is required. The design philosophy is to first identify available pipe options, select a target head loss of 5 to 10 percent or less of the gross head, and keep the length as short as possible. Several options for sizes and types of materials may need to be calculated and evaluated in order to find a suitable penstock pipe.

Several factors should be considered when deciding which material to use for a particular penstock: design pressure, the roughness of the pipe’s interior surface, method of joining, weight and ease of installation, accessibility to the site, design life and maintenance, weather conditions, availability, relative cost and likelihood of structural damage.

The pressure rating of the penstock is critical because the pipe wall must be thick enough to withstand the maximum water pressure; otherwise there will be a risk of bursting. The pressure of the water in the penstock depends on the head; the higher the head, the higher the pressure.
The most commonly used materials for a penstock is mild steel because of their suitability, availability and affordability. Layout of the penstock pipelines depends on their material, the nature of the terrain and environmental considerations;
They are generally surface-mounted or buried underground. Special attention is necessary where a penstock is installed in a very cold environment.

Figure 1. 6 Penstock and it supporting structures

1.1.5 Powerhouse and Tailrace

The powerhouse is a building that houses the turbine, generator and controller units.

Although the powerhouse can be a simple structure, its foundation must be solid.

The tailrace is a channel that allows the water to flow back to the stream after it has passed through the turbine.

Figure 1. 7 power house and tail race

1.1.6 Powerhouse Components

1.1.6.1 Turbines

A turbine unit consists of a runner connected to a shaft that converts the potential energy in falling water into mechanical or shaft power. The turbine is connected either directly to the generator or is connected by means of gears or belts and pulleys, depending on the speed required for the generator.

The choice of turbine depends mainly on the head and the design flow for the proposed micro hydropower installation. The selection also depends on the desired running speed of the generator.

Figure 1. 8 Turbine

1.1.6.2 Turbine Efficiency

Typical efficiency ranges of turbines are given in table 4.1 Turbines are chosen or are sometimes tailor-made according to site conditions. Selecting the right turbine is one of the most important parts of designing a Small scale -hydropower system, and the skills of an engineer are needed in order to choose the most effective turbine for a site, taking into consideration cost, variations in head, and variations in flow, the amount of sediment in the water and overall reliability of the turbine. For Small scale -hydropower cross flow turbine (T-15 model) with efficiency of 65-85 is practiced and being effective on so many Indonesian micro hydropower schemes.

Table 4. 1 Typical efficiency of turbines

Turbine type		Efficiency Range
Impulse Turbines	Pelton	80-90
	Turgo	80-95
	Cross - flow	**65-85**
Reaction Turbines	Francis	80-90
	Pump-as-turbine	60-90
	Propeller	80-95
	Kaplan	80-90

1.1.6.3 Generators

Generators convert the mechanical (rotational) energy produced by the turbine to electrical energy; this is the heart of any hydro electrical power system. The principle of generator operation is quite simple: when a coil of wire is moved past a magnetic field,
a voltage is induced in the wire. Alternating current (AC) generators are also referred to as alternators. They generate varying voltages, which alternate above and below the zero voltage point. It is this process that produces AC electricity.

There are two types of generators: synchronous and asynchronous. Synchronous generators are standard in electrical power generation and are used in most power plants. Asynchronous generators are more commonly known as induction generators.
Both of these generators are available in three-phase or single-phase systems. System capacity, type of load and length of the transmission/distribution network dictate whether a single- or three-phase generator should be used.

Induction generators are generally appropriate for smaller systems. They have the advantage of being rugged and cheaper than synchronous generators. The induction generator is a standard three-phase induction motor, wired to operate as a generator. Capacitors are used for excitation and are popular for smaller systems that generate less than 10 to 15 kW.All generators must be driven at a constant speed to generate steady power at the frequency of 50 Hz.

1.1.6.4 Electronic Load Controllers

Water turbines, like petrol or diesel engines, will vary in speed as load is applied or disconnected. Although not a great problem with machinery that uses direct shaft power, this speed variation will seriously affect the frequency and voltage output from a generator. It could damage the generator by overloading it because of high power demand or over-speeding under light or no-load conditions.

Traditionally, complex and costly hydraulic or mechanical speed governors similar to larger hydro systems have been used to regulate the water flow into the turbine as the load demand varied. Over the last two decades, electronic load controllers (ELCs) have been developed that have increased the simplicity and reliability of modern micro hydropower systems.

Figure 1. 9 Electronic Load controller

Electro-Mechanical Components of Small scale -hydropower

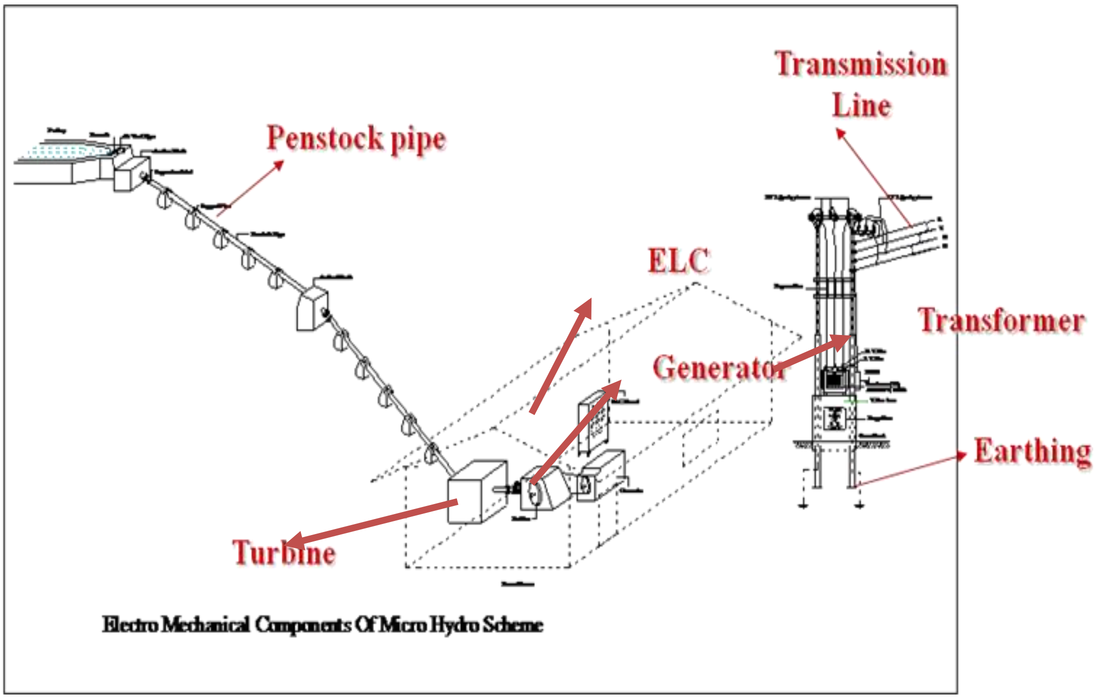


Figure 1. 10 **Electro-Mechanical Components of Small scale -hydropower**

Transmission and distribution Network

Electric power transmission or "high-voltage electric transmission" is the bulk transfer of electrical energy, from generating power plants to substations located near population centers. This is distinct from the local wiring between high-voltage substations and customers, which is typically referred to as electric power distribution. Transmission lines, when interconnected with each other, become high-voltage transmission networks.

Figure 1. 11 Transmission and distribution Network

2 SITE SELECTION AND LAY OUT PLAN

2.1. GENERAL

The selection of appropriate site and layout of micro-hydro power scheme is an important and an iterative process for sustainability of the project, which also meet the demand of targeted community.

A brief note is necessary on finding out demand side of the project so that suitable site selection and layout can be arranged for designing of the project which can supply the need.

Domestic lighting

The domestic lighting is the major electricity market in the rural communities of Nepal. Keeping affordability of majority of people in rural area, in average 100 watt per house hold can be assumed to find out electricity demand. However, real demand should be gathered by visiting each and every household.

Business Market

Apart from the major demand on domestic lighting, other important and normal markets are agro-processing (hulling, grinding, oil expelling, fruit drying), saw mills, rural bakery and water lifting etc. However, normally these industries are run during day time and therefore, major governing factor for sizing of MH project should be based on demand of domestic lighting.

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

- Stream Flow
- Topography

The power available from a micro-hydropower scheme is a function of both the flow and the head. The head depends on the topography. Micro hydro becomes technically viable only if the combination of head and flow are such that the demand of the targeted community can be met. Under normal circumstances, the low season flow of the river should be used while calculating the power output. It should be noted that designers have little control over the flow available in the stream. However, they have some control over the topography. They can choose different alignments for the intake, headrace and penstock alignments. They can also modify the local topography through excavation, building of structures and by undertaking soil stability enhancement measures.

Proper planning and coordination in the initial stage of the project will keep costs to a minimum and reduce delays.

2.2. PRINCIPLES OF SITE INVESTIGATION

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

In most of civil works, commonly the unexpected events happen during the construction. Site investigation aims to predict what this might be so that the engineer can prepare a design which will deal with it.

It should be noted that the measurement of the head and flow serves to establish the options available for development of the site for micro-hydropower. The site investigation then assesses the suitability of the site for each alternative. The site investigation process helps to 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.

There is usually a limit to the time and funds available for site investigations. It is always difficult to know when adequate work has been completed. The key is to work efficiently and to think carefully about where more thorough investigation is required. The principles of site investigation are:

- Take time and be thorough to avoid return visit to collect information missed the first time which is costly and inadequate for civil design even more so.
- Walk all over the site. Gain a full appreciation of the options available.
- Talk to local people, especially those who have carried out construction work in the area. Since most of the rivers in the mountains and the middle hills of Nepal have not been gauged, stream flow data are not usually available. Therefore, it is important to talk to local people to get a feel of the flood levels for rare floods events (say 20 years to 50 years return period)

2.3. SELECTION OF ALTERNATIVE LAYOUTS

2.3.1. AN OVERVIEW OF THE SITE

This will involve viewing the site from a physical vantage point and taking time to consider the practical design and construction of the alternative layouts. Each possible layout will require construction work on different parts of the potential site and the surveyor should therefore note on which part each component of the scheme will be located. The overview should also note features from the topography such as slope stability as well as land use and ownership which may affect the design of the scheme. A sketch map of the each site plan should be made so that later a complete schematic diagram can be drawn.

2.3.2. LOCATION OF COMPONENTS

For each of the alternative layouts selected during the site visit, locations of each component should be identified and then included in the site plan. Powerhouse, forebay and intake are selected by an iterative process. Usually the first step is to identify a suitable location for the powerhouse as this will contain most expensive parts and machines, then to choose a position for the forebay giving the requisite gross head, and then to survey the path of the proposed headrace to reach the intake.

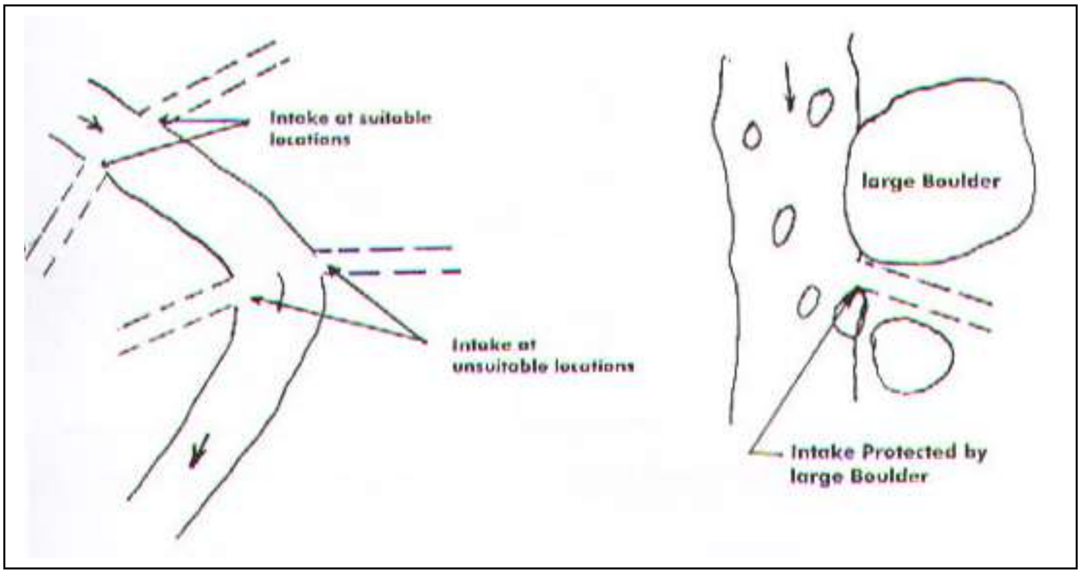
Once all alternatives have been identified, the most suitable layout should be chosen. Apart from the project cost, the following are the main criteria that should be used to select the most suitable layout:

- The combination of head and flow of each alternative should be such that the required minimum power output is feasible.
- Preference should be given to the simplest layout in terms of design and construction.
- The shorter the alignment of the scheme is, the less the requirement for construction materials. Hence, such a scheme can be constructed faster and at a lower cost.
- Finally, technical parameters such as overall slope stability, floods risks, and other site specific issues should be considered as stated under.

2.3.2.1. INTAKE

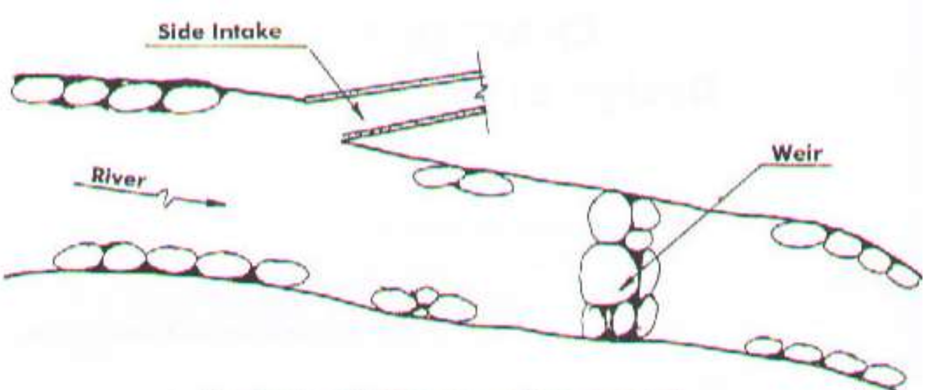
Make sure that water can be diverted away from the river and towards the headrace canal. Does the river course appear stable or does it look like it will meander? Think about floods and flood levels. Does the river carry large boulders? If so, think about temporary diversion works rather than a permanent weir. Apart from the above factors following are the specific aspects to be considered in selecting intake.

- the stream should be straight upstream and downstream.
- should not be located at bend. If it has to be located at bend, it should be on the outer side of bend.
- should not be located at the place having large gradient at upstream.
- should be located at a place where the stream is relatively permanent.
- should be located at a place where a weir can be constructed at downstream of the intake mouth.
- can be protected by locating downstream of large boulder.



2.3.2.2. WEIR

- may not be needed if the inherent features of the stream automatically divert adequate water to the intake during dry season.
- should be located about 5 – 20 meters downstream of the intake.
- should be located at a narrow width of the stream or at a place with large boulder which can be used as a part of the weir.



Side Intake with Temporary Diversion Weir

2.3.2.3. SPILLWAY

- should be located as close as possible to the intake.
- should have a good path available for easy drain out of the spill flow back to the stream.

2.3.2.4. HEADRACE CANAL

In general the head race alignment should be on level to slightly sloping ground. If the alignment is steep, consider using a headrace pipe instead of a canal. Try to get the alignment away from the river as early as possible to minimize flood damage. Provide escapes upstream of areas where the canal might be blocked by landslides. If seepages from the headrace canal can trigger landslides, think about lining the canal or consider using pipes.

- should pass through stable terrain not threatened by landslides, storm gully, falling rock or similar.
- should follow the shortest reasonable route with minimum crossings.
- should not contain obstructions that make construction difficult and expensive.

2.3.2.5. SETTLING BASIN & GRAVEL TRAP

Generally the first settling basin should be located as close to the intake as possible so that any sediment is removed early and the maintenance of the canal is minimized. If river carries significant amount of gravel during the monsoon then a gravel trap should be constructed immediate after the intake location to flush out gravels into the stream or nearby gully. If the canal is very short settling basin should be combined with forebay tank to minimize the cost.

- should be located on stable ground as close as possible to intake and downstream of the spillway.
- should have a good and short path available for easy drain out of overflow and washout without damaging the structure.

2.3.2.6. FOREBAY TANK

Allowance should be made for final settling of sediments. Generally, this structure should be located just uphill of the transition area where the ground profile changes from level to steep. Is there a possibility to discharge the entire flow safely from a spillway in case of system malfunction?

- should be located on stable ground from where the desirable head can be achieved.
- should have a safe and short path available for easy drain out of overflow and washout without damaging the structures.
- should have a safe path to discharge the entire flow safely from spill way in case of system malfunction.

2.3.2.7. PENSTOCK

The penstock alignment should start where the ground profile gets steeper. An ideal ground slope would be between 1:1 and 1:2 (V:H). The flatter the ground slope, the less economic is the penstock. It is difficult to manually lay penstock, construct support piers and anchor blocks if the slope is greater than 1:1. Also try to minimize bends since these will require additional anchor blocks.

- should be on the stable ground, not liable to slip as a result of heavy rain, landslide etc.
- should be short and straight with less horizontal and vertical bends.

2.3.2.8. ANCHOR BLOCK AND SUPPORT PIER

- should be on stable ground, not on loose debris.

2.3.2.9. POWERHOUSE

- should be easily accessible and near to the load centers.
- should be located on stable ground safe from flash flood, landslide, river undercut etc. and from where the desirable head can be achieved.
- should be above the 50 years flood level of the stream.

- should be located at a place where the tailrace can be constructed to carry water leaving turbine to the stream safely without damaging the powerhouse.

2.3.2.10. TAILRACE CANAL

- The tailrace should be oriented downstream to prevent floodwater, debris and bed load from being funneled into it towards the powerhouse
- Tailrace should be protected from the stream into which emerging from the turbine is discharged.

2.3.2.11. TRANSMISSION AND DISTRIBUTION LINE

- The route should be the shortest to the load center to make the project inexpensive
- Should be erected on stable ground which is safe from landslide and flood
- Maximum spacing between two hardwood poles should be 35 meters and in regards of steel pole it should be not greater than 50 meters or as per manufacturer's specification.

2.4. GEOTECHNICAL CONSIDERATIONS

2.4.1. GEOLOGY

Geology of the site is critical to the design, costs and future performance of the civil works of the schemes. Geological characteristic of a site can be grouped in the following way:

- Slope stability- the degree of stability of the hillsides of the site.
- Soil and rock types- Foundation conditions, and liability to seepage undermining and subsidence around structures planned for the site.

2.4.2. SLOPE STABILITY

In geological terms, the hills and mountains of Nepal are young and unstable. They could be likened to a pile of sand in that the excavation along a slope, easily results in the sliding of the land above, especially when a further triggering mechanism occurs (particularly during the monsoon). Common triggering mechanisms are the following:

- Surface water
- Ground water
- Undercutting of slope by excavation

The stability of slopes will affect the design of all components of a micro-hydro scheme and should therefore be analyzed thoroughly, particularly in the following key areas:

- Above and below proposed canal routes
- Below the proposed location of a settling basin or forebay tank
- Along the proposed penstock alignment
- Above and below the propose location of the powerhouse

Threats in these areas will either take the form of weakening of the support around the foundations through land slipping away or collapsing, or damage to structures through falling debris.

Indicators of slope instability

Section of slope	Indicator of instability
Upper, middle or lower hill-slopes	Tension cracks along slope, ground falls sharply, trees leaning down slope or bending upwards from the base, water springs or seepage at base, displacement of paths, fence or posts.
Exposed faces of landslide	Fresh rock faces exposed, presence of soft, weathered rock, open joints in rock, tension cracks, overhangs and loose rock, water springs or seepage at base of rock face
Debris slopes	Fresh debris at base of slope, tree roots exposed, loose debris which moves underfoot, profiles steeped towards base of slope, debris littered with dead or overturned woody plants and grass clumps.

The slope stability is indicated by the following features of the slope of rock face:

- Complete vegetation cover, including trees standing vertically.
- Straight, even slope profile
- Rock surfaces covered with moss, lichen or a weathered skin
- Hard, impermeable rock
- Rock with no or few joints
- Closed rock joints
- Well packed debris, especially with fine material packed into voids between coarse material
- Well established trees and shrubs
- No active gully (although a stable gully system may be present)

The increased knowledge of the site gained from a thorough investigation of the slopes will influence the design of the whole scheme, particularly the location of the principal structures.

The recommendations from the investigation of slope stability should follow two basic rules:

- Never construct on fill, that is, land which has been built-up or filled using excavated material.
- Avoid the location of structures close to landslide zones

2.4.3. SOIL AND ROCK TYPES

The type of soil or rock also affects the foundation of structures and the canal type. For example, if the soil type is sandy loam, a larger foundation depth is required. On the other hand, structures may be built directly on hard rock without any excavation. Similarly, lining may not be required for headrace if the soil type is clay. However, lining will be required if the alignment is through sandy soil.

2.5. OTHER CONSIDERATIONS

2.5.1. FLOOD RISK

In site investigation, the concern is for the selection of the best option for the design of the scheme. Therefore, knowledge of flood levels is important at the two extremes of the micro-hydro scheme, the intake and powerhouse, or at other parts of the scheme which may be vulnerable to flood damage from the river.

Flood levels may be predicted by hydrological calculation from available data to give the 20 years or 50 years flood level, or by consulting local people. Ideally both method should be used to give reliable estimate. Always allow a margin of error so that a rare flood event is allowed for and think carefully about how the floods will affect each of the proposed lay outs for the project. The location of the powerhouse higher on a slope will reduce the available head and therefore have an important impact on the capacity and the economics of the project

Subsidence is caused by the location of acid substances in the local groundwater acting on soluble rocks such as lime-stones, by the presence of rocks which are liable to splitting and foliation of by underground caverns which are prone to collapse. The presence of thick layers of very sandy soil may also lead to subsidence.

These characteristics are identified by careful observation of the site. Limestone outcrops, sinkholes (holes of 2-10 meters in diameter which form when the limestone beneath dissolves, causing the soil above to collapse), the appearance of streams or other seepage from depressions or cracks in the ground surface are examples of characteristics to look for.

Undermining refers to the action of surface water on the foundations of structures. The intake of the scheme and the penstock are particularly prone to undermining where surface water threatens the structures, but the headrace canal is also vulnerable.

2.5.2. CROSS DRAINAGE

Sometimes, because of the nature of the topography, the headrace canal/or the penstock alignments will need to cross gullies and small streams. Note that dry slopes are more stable than saturated slopes. Surface water can be diverted by constructing various types of cross drainage works. For example, catch drains can be constructed uphill from the MH alignment to divert the surface runoff. Catch drains are small channels that divert surface runoff (thus catch it) and divert it into rear-by gullies or natural drainage.

Another example of cross drainage works is the use of a super-passage. This is a covered headrace canal arrangement such that the surface runoff flows over it whereas the design flow is safely conveyed in the canal.

2.5.3. WATER RIGHTS

Sometimes there can be water use conflicts between the proposed MH scheme and other prior uses of the source stream. For example, if there is an irrigation scheme downstream of the proposed MH intake which may received less water (once the MH Plant is commissioned), there will be conflicts. Such water rights issues should be resolved before implementing the micro hydro scheme.

It should be noted that irrigation and micro hydro can be co-coordinated if an agreement with all concerned parties is reached in the initial stage. This is because irrigation water is not required throughout the year and therefore water can be used for power production at other times. This may result in less or even no power available during peak irrigation period. However, if the electricity users are also owners of the irrigated land, they can prioritize their needs such as by irrigating in the afternoons and nights and producing power during mornings and evenings.

2.5.4. LAND OWNERSHIP AND LAND USE

The surveyor should note down the issues concerning land use and ownership. If the alignment transverses through a farmer’s paddy field, the land may have to be bought by the project. Another example is that an open channel headrace may be technically feasible but the designer may have to choose a buried pipe if the headrace alignment is along cultivated land. Similarly, sediment flushing and spillway flows need to be safely diverted away from cultivated land

3. Flow calculation, Estimation & Design flow determination

A. RIVER FLOW CALCULATION

Calculation of the discharge from the velocity and depth measurements can be made in several ways. Two of these methods are mid section and mean section methods we be presented here.

i. Mid-section method

In this method the vertical in each elemental strip of the cross-section where the velocity measurements are made (one-point method or two-point method) is taken at the middle of the strip. The measured water depth (by sounding) in the vertical is taken as the mean depth of flow in the strip. If *b* is the equal width of each elemental strip then the discharge in the strip is given by

$\Delta Q=(bd)V_{0.6d}$ …………………………………………………………… 1 for shallow strip

$\Delta Q = bd\frac{V0.2d+V0.8d}{2}$…………………………………………..…………………2 for deep water strips

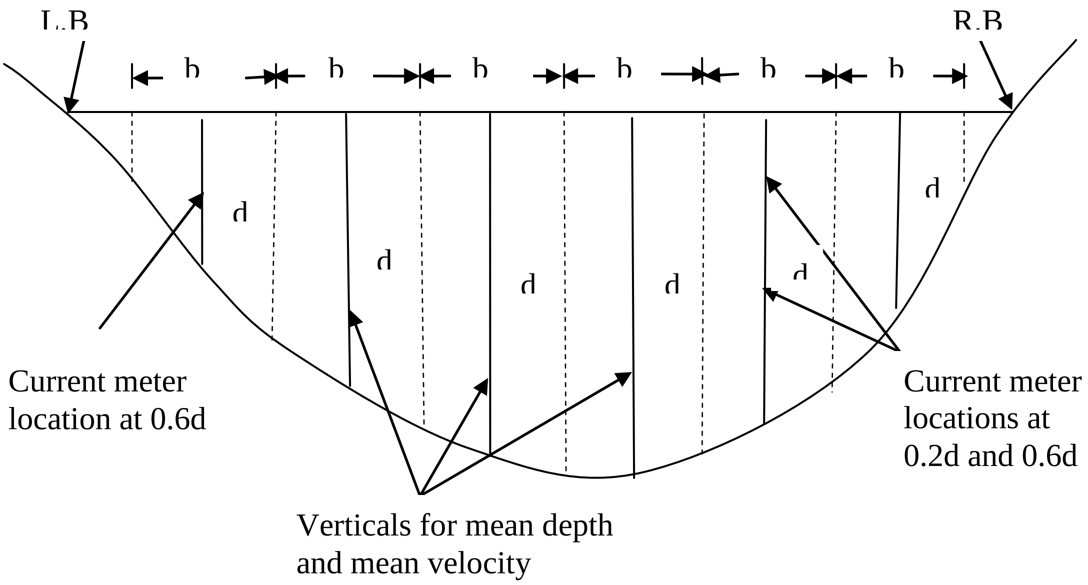
Where *b* = width of each elemental strip

d = average depth of flow in the particular strip (measured as the depth of flow in the vertical taken at the middle of the strip)

bd = area of cross-section of flow in the strip considered

$V_{0.6d}$ = velocity of flow at 0.6*d* below the free surface of water in the vertical taken at the middle of the strip

$V_{0.2d}$ = velocity of flow at 0.2*d* below the free surface of water in the vertical taken at the middle of the strip



Discharge through each elemental strip of width b

$$\Delta Q=(bd)\frac{V_{0.2d}+V_{0.8d}}{2} \text{ (or) } (bd)V_{0.6d}$$

Figure 1

$V_{0.8d}$ = velocity of flow at 0.8*d* below the free surface of water in the vertical taken at the middle of the strip

The discharge in the stream is given by

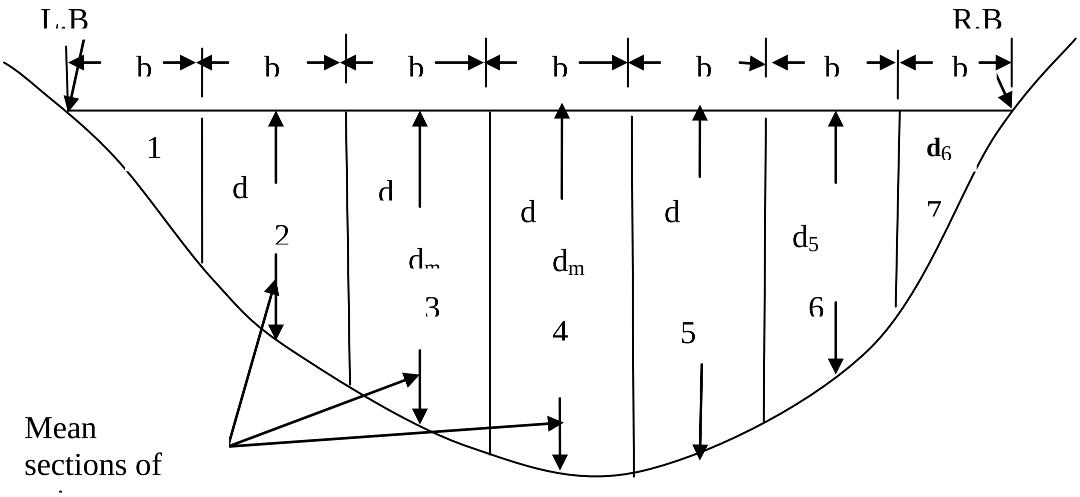
$$Q=\sum \Delta Q \quad \text{………………………………………………………………. 3}$$

In this method, the discharge in the two elemental triangular strips near the ends is not included in the computation of discharge.

ii. Mean-section method

In this method, depths and velocities are measured in the verticals taken at the ends of each elemental strip of width b. For example, in strip 1, the depths of flow measured in the verticals considered at the ends of strip are d_1 and d_2. The mean depth of flow in strip 1 is taken as the average of the depths of flow measured at the verticals considered at the ends of the strip, that is, $d_m = (d_1 + d_2) / 2$. The velocities of flow are measured in the vertical at the ends of the strip at 0.6d (for shallow strips) and at 0.2d and 0.8d for deep water strips. For example in strip 1, the velocity is measured at 0.6d in the vertical at the left end of the strip, that is, $V_1 = V_{0.6d} = V_{0.6d_1}$ and the velocities are measured at 0.2d and at 0.8d I the vertical at

the right end of the strip, that is $V_2 = \frac{V_{0.2d}+V_{0.8d}}{2} = \frac{V_{0.2d_1}+V_{0.8d_1}}{2}$. Then, the discharge in elemental strip 1 is given by,



$\Delta Q = d_m V_m$

Strips 1 to 7

d_m = depth of flow at mean section of strip

For strip 2, $d_m = \frac{d_1+d_2}{2}$ and $V_m = \frac{V_1+V_2}{2}$

where V_1 and V_2 are determined as $V_{0.6d}$ for shallow strip

V_1 and V_2 are determined as $\frac{V_{0.2d}+V_{0.8d}}{2}$ for deep strip

Figure 2

$\Delta Q = b\frac{d_1+d_2}{2}\,\frac{V_1+V_2}{2}$……………………………………………………………..4

Where $V_1 = V_{0.6d} = V_{0.6d_1}$ (shallow vertical)

and $V_2 = \frac{V_{0.2d}+V_{0.8d}}{2} = \frac{V_{0.2d_1}+V_{0.8d_1}}{2}$ (deep vertical)

Thus, stream discharge is givenby

$Q = \sum \Delta Q$……………………………………………………5

The mean-section method is considered to be more accurate, but the mid-section method is faster and hence is generally used.

Example 1: The following data were collected for a stream at a gauging station. Compute the discharge.

Distance from one end of water surface (m)	Depth, d (m)	Immersion of current meter below water surface					
		at 0.6d		at 0.2d		at 0.8d	
		Rev.	sec.	Rev.	sec.	Rev.	sec.
3	1.4	12	50				
6	3.3			38	52	23	55
9	5.0			40	58	30	54
12	9.0			48	60	34	58
15	5.4			34	52	30	50
18	3.8			35	52	30	54
21	1.8	18	50				

Rating equation of current meter: V = 0.3N + 0.05; N = revolutions per second; V = velocity in meters per second; Rev. = Number of revolutions; sec = time taken in seconds for revolutions.

Solution. Discharge in each elemental strip is determined by the mid - section method.

Discharge through each elemental strip of width b is given by

$$\Delta Q=(bd)\frac{V_{0.2d}+V_{0.8d}}{2}$$ (for intermediate strips) and

$$\Delta Q = (bd)V_{0.6d}$$

Where d = mean depth of flow in the strip under consideration

$V_{0.2d}$, $V_{0.6d}$ and $V_{0.8d}$ are velocities of flow measured at 0.2d, 0.6d and 0.8d in the strip under consideration.

The discharge in the stream is given by

$$Q = \sum \Delta Q$$

In this method, the discharges in the two elemental triangular strips near the ends (Left bank and right bank) are not included in the computation of total discharge Q.

Sample calculations:

For end strip 1, $\Delta Q = (bd)V_{0.6d}$ = (3 x 1.4) 0.122 = 0.51 cumec

For intermediate strip 2, $\Delta Q = (bd)\dfrac{V_{0.2d} + V_{0.8d}}{2}$

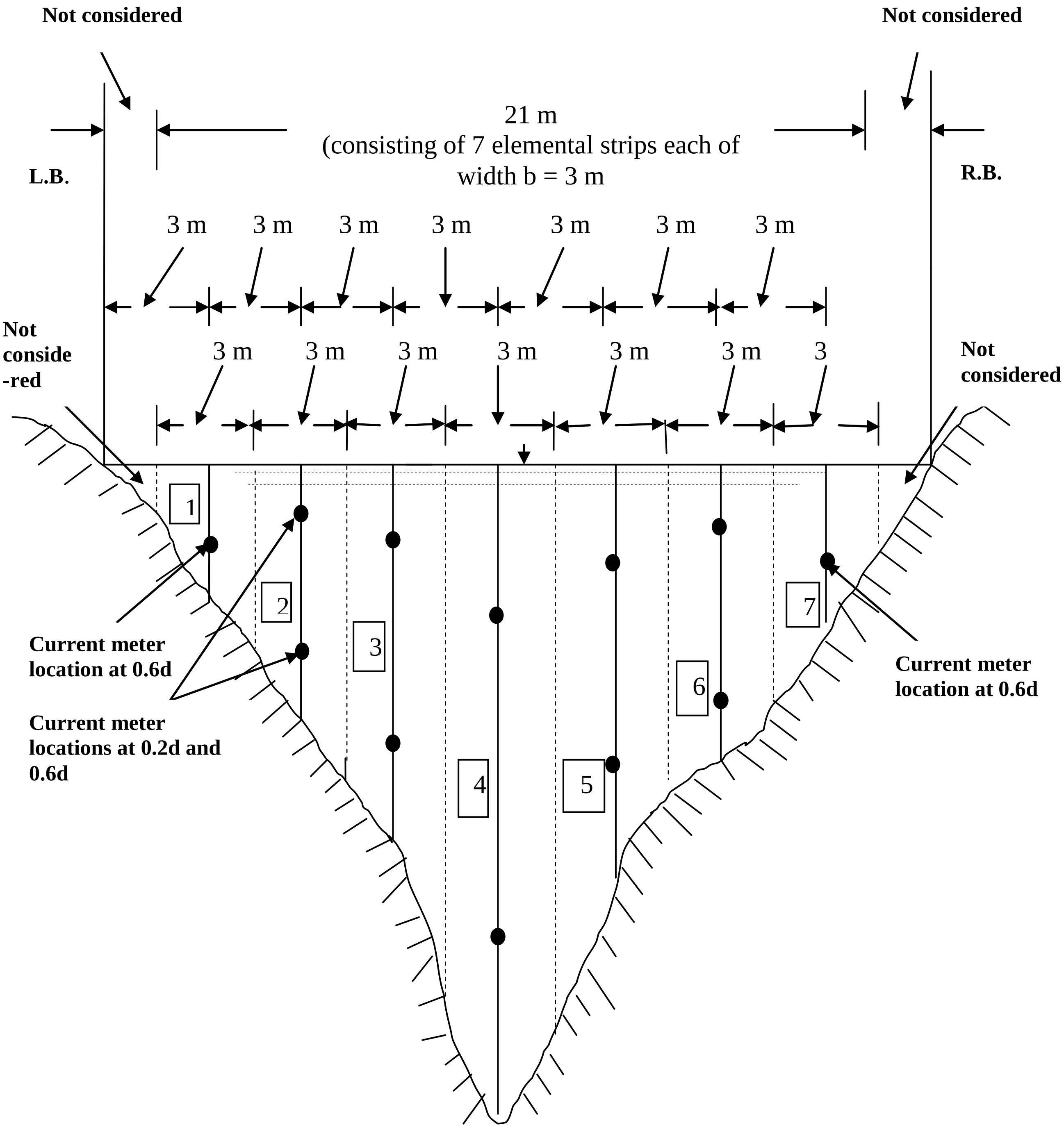


Figure 3

Distance from one end of water surface (m)	Depth, d (m)	Immersion of current meter below water surface						Average velocity of flow in strip, V_{ave} m/s	Discharge in strip $\Delta Q = (bd)V_{ave}$ (b = 3 m)
		Depth of velocity measurement		Rev. R	sec. t	rps N=R/t	V = 0.3N + 0.05 m/s		
3	1.4	0.6d	0.84	12	50	0.24	0.122	0.122	0.51
6	3.3	0.2d	0.66	38	52	0.73	0.269		
		0.8d	2.64	23	55	0.42	0.176	= (0.269 + 0.176) / 2 = 0.223	2.20
9	5.0	0.2d	1.00	40	58	0.69	0.257		
		0.8d	4.00	30	54	0.56	0.218	= (0.257 + 0.218) / 2 = 0.236	3.54
12	9.0	0.2d	1.80	48	60	0.80	0.290		
		0.8d	7.20	34	58	0.59	0.227	= (0.290 + 0.227) / 2 = 0.259	7.00
15	5.4	0.2d	1.08	34	52	0.65	0.245		
		0.8d	4.32	30	50	0.60	0.230	= (0.245 + 0.230) / 2 = 0.238	3.86
18	3.8	0.2d	0.76	35	52	0.67	0.251		
		0.8d	3.04	30	54	0.56	0.218	= (0.251 + 0.218) / 2 = 0.234	2.67
21	1.8	0.6d	1.08	18	50	0.36	0.158	0.158	0.85
									Q = 20.53 cumec

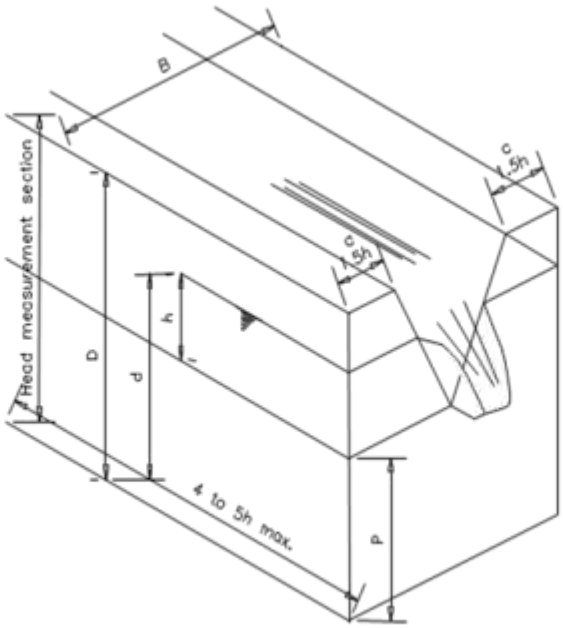
B. Weir Methods

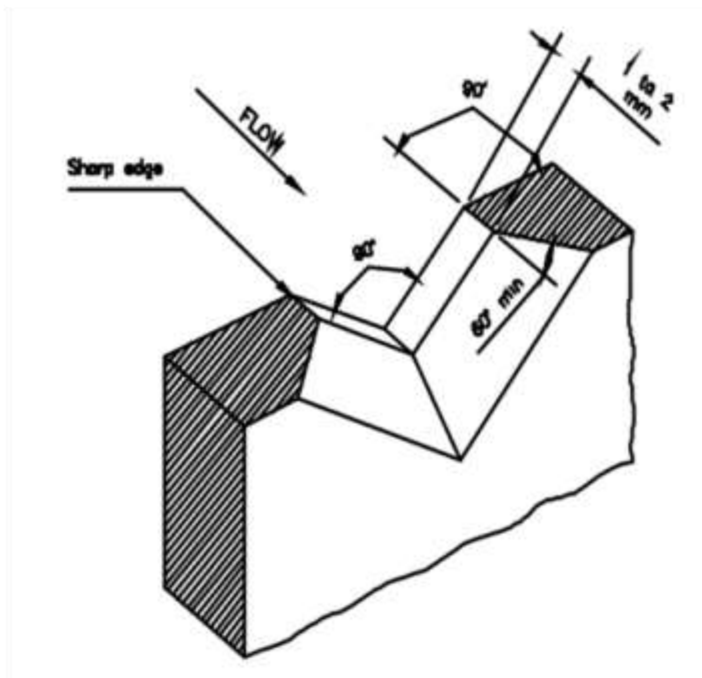
When discharge is small and the use of current meter or float measuring method is impossible, the weir is built and discharge is measured by measuring the overflow depth at the river.

A Weir is perhaps the most accurate way to measure small and medium sized streams. All the water is directed through an area that is exactly rectangular, making it very easy to measure the height and width of the water to compute FLOW.

Triangular (Smaller flow) and rectangular (Larger flow) are the most common once

Triangular (V notch)





The basic equation of discharge for a V notch weir is:

$Q = C \times (8/15) \times (2g)^{0.5} \times \tan(\theta/2) \times h^{5/2}$ …………………………………………6

Where

Q = discharge in m^3/s

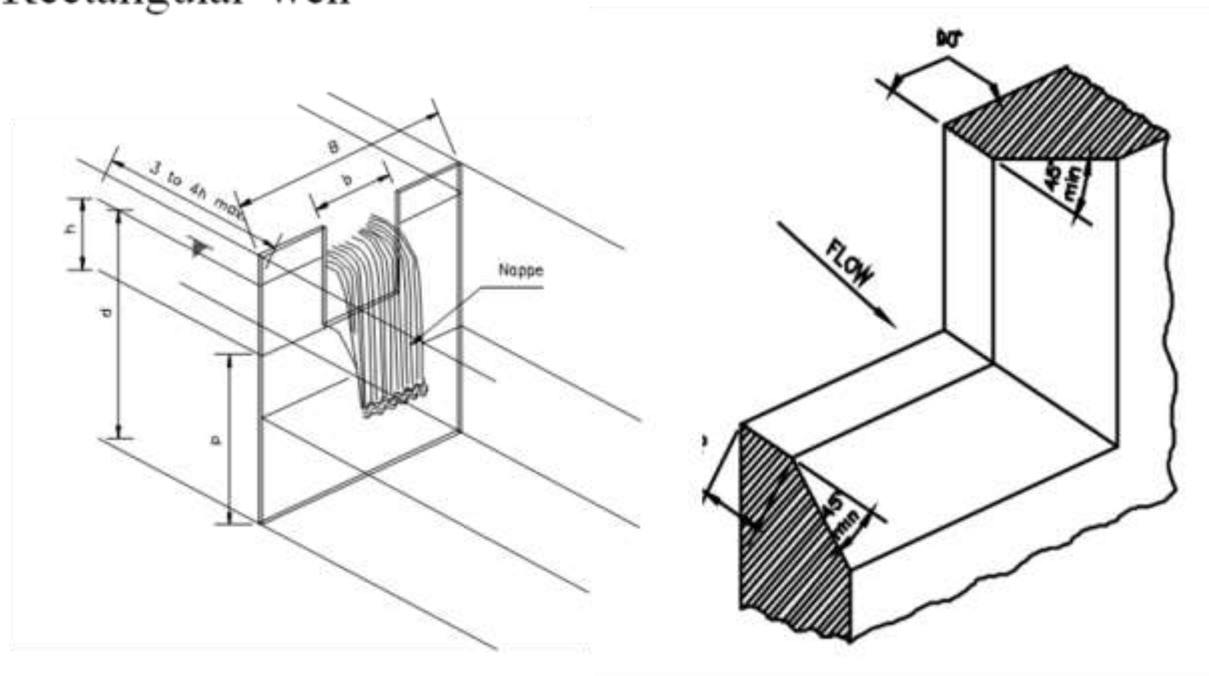
h = head over the weir measured from the vertex of the notch

g = acceleration due to gravity = 9.81 m/s^2

θ = The angle of the notch

C = coefficient of discharge

Rectangular weir



The basic equation of discharge for a rectangular weir is:

$Q = C \times b \times h^{1.5}$ ………………………………………………………………..7

Where,

Q : Discharge (m3/s)

h = Head over weir in m

b = Width of the weir in m

C = Weir coefficient of discharge which is close to 0.6.

C. **Stage- discharge rating curve**

It is a curve showing the relationship between stream discharge Q and the stage (water level) measured at a stream gauging site.

What is the usefulness of stage – discharge rating curve?

The stage – discharge rating curve is useful in serving as a ready reckon for reading the stream discharge knowing the measured stage (water level) at the gauging site. A developed stage – discharge rating curve for a gauging site is applicable as long as the cross – section of the stream at or near the gauging site has not very much altered. Periodical gauging are conducted (say, once in 3 years) to verify the developed rating curve. If there are changes in the stream discharges for different measured stages, then the rating curve needs to be revised.

If h_o is the vertical gauge reading corresponding to zero – discharge and h is the vertical gauge reading corresponding to discharge Q, then the gauge height is $(h – h_o)$ for this discharge Q. The relation between stream discharge and gauge height can be expressed as

$Q = a(h - h_o)^b$ ……………………………………………………………………….8

Where *a* and *b* are constants for any stream gauging station.

Equation (6) is non-linear.

It can be linearised by taking logarithms on both sides of it.

Taking log on both sides of equation (6)

$\ln(Q) = \ln(a) + b\ln(h - h_o)$ ……………………………………………………………9

Equation (7) is in the form of a straight line equation given by $Y=c+mX$

Where, $Y=\ln(Q)$; $c=\ln(a)$; $m=b$; and $X = \ln(h - h_o)$

Hence, the plot of $(h-h_o)$ and Q on a log – log paper gives a straight line whose slope is b and Y – intercept is a; a = Q when $(h – h_o) = 1$

2. FLOW ESTIMATING METHODS

Estimates of stream flow statistics often are needed for sites on streams where no data are available.

The two methods used most commonly to estimate statistics for un gauged sites are the drainage-area ratio method and regression equations. The drainage-area ratio method is most appropriate for use when the ungauged site is near a stream gauging station on the same stream (nested).

Regression equations can be used to obtain estimates for most un gauged sites. Additional details on application of these methods are provided below.

i. Drainage-Area Ratio Method

The drainage-area ratio method assumes that the streamflow at an ungaged site is the same per unit area as that at a nearby, hydrologically similar streamgaging station used as an index. Drainage areas for the ungaged site and the index station are determined from topographic maps. Streamflow statistics are computed for the index station, and then the statistics (numerical values) are divided by the drainage area to determine streamflows per unit area at the index station. These values are multiplied by the drainage area at the ungaged site to obtain estimated statistics for the site.

$$Q_u = (A_u/A_g)^b Q_g \ldots\ldots 10$$

where Q_u is the estimated flow statistic for the un gauged site, Au is the drainage area for the un gauged site, Ag is the drainage area for the stream gauging station, Q_g is the flow statistic for the stream gauging station, and b, depending on the state, may be the exponent of drainage area from the appropriate regression equation, a value determined by the author of the state report, or 1 where not defined in the state report.

ii. REGRESSION EQUATIONS

Multiple linear-regression analysis (regression analysis) has been used to develop equations for estimating stream flow statistics for un gauged sites. In regression analysis, a stream-flow statistic (the dependent variable) for a group of data collection stations is statistically related to one or more physical or climatic characteristics of the drainage areas for the stations (the independent variables). This results in an equation that can be used to estimate the statistic for sites where no stream flow data are available. Equations can be developed by use of several different regression analysis algorithms. The various algorithms use different methods for minimizing differences between the values of the dependent variable for the stations used in the analysis (the observed values) and the corresponding values provided by the resulting regression equation (the estimated or fitted values). Choice of one algorithm over another depends on the characteristics of the data used in the analysis and on the underlying assumptions for use of the algorithm. Equations obtained by use of regression analysis take the general form

$$Yi=b_o+b_1X_1+b_2X_2+b_3X_3+\ldots\ldots b_n X_n+\dot{\varepsilon} \ldots\ldots 11$$

where Y_i is the estimate of the dependent variable for site i, X_1 to Xn are the n independent variables, b0 to bn are the n + 1 regression model coefficients, and εi is the residual error (difference between the observed and estimated value of the dependent variable) for site i. Assumptions for use of regression analysis are

(1) Equation 2 adequately describes the relation between the dependent and the independent variables,

(2) The mean of the ε_i is zero,

(3) The variance of the ε_i is constant and independent of the values of Xn,

(4) The εi are normally distributed, and

(5) The ε_i are independent of each other (Iman and Conover, 1983, p. 367). Regression analysis results must be evaluated to assure that these assumptions are met. Stream flow and basin characteristics used in hydrologic regression usually are log normally distributed; therefore, transformation of the variables to logarithms is usually necessary to satisfy regression assumption 2.

This method is most commonly applied when the index gauging station is on the same stream as the ungaged site because the accuracy of the method depends on the proximity of the two, on similarities in drainage area and on other physical and climatic characteristics of their drainage basins.

HYDROLOGICAL ANALYSIS AND DESIGN FLOW DETERMINATION

For the development of any small hydropower scheme an essential first step is to determine whether there is sufficient and reliable amount of water available to make the scheme economically viable. As a standard practice a gauging station should be set up and the discharge should be observed for at least two lean seasons.

The main requirements of hydrological analysis suggested for Micro hydro power design are:

- annual runoff and its monthly distribution;
- daily flow duration curve;
- design flood;
- Stage–discharge relation curve.

Design Annual Runoff

The analysis of design annual run off consists of the analysis of annual runoff (or annual average discharge), and the distribution of runoff (by month) in the design year. The findings of the annual runoff analysis are indispensable for flow regulation, power generation calculation, and installed capacity determination.

In case of where more than 15 years' recorded data are available, direct frequency analysis is possible. Given an annual runoff sequence with terms from high to low range, the empirical frequency of the m^{th} term is calculated by the expectancy formula:

$$f = \frac{m}{n+1} * 100\% \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 12$$

Where

m is rank

n is number of data

The natural flow duration curve

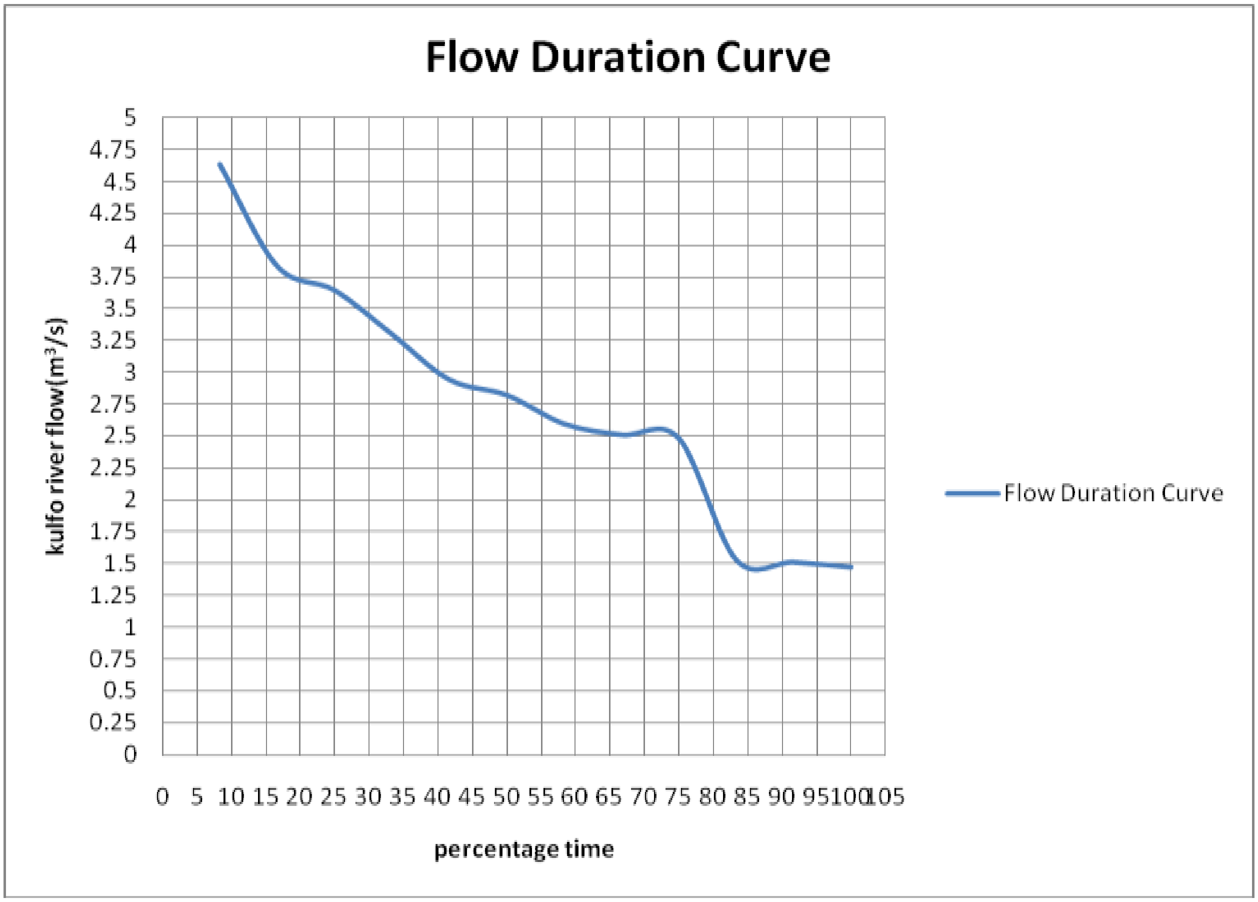
It is useful tool for MHP design in runoff the river schemes and daily regulation schemes.

A flow duration curve plotted by monthly average flows has a tendency towards a higher design discharge that sometimes produces quite large errors. This is because the monthly average used will mask within the month variation. Consequently, the designer should obtain an average daily flow series as far as possible in order to build a daily flow duration curve.

If we have sufficient daily flow data, the daily flow duration curve can be constructed on the basis of findings from three typical years.

Computing a flow duration curve at a gauged point can be carried out by two methods: the ranked flow method and the class interval method. Using the ranked flow technique, a time series of flows is rank –ordered according to the magnitude of flow. The rank-ordered values are then assigned order numbers beginning with the largest. The order number are then divided by the total number in the recorded and multiplied by 100 which represents the percentage of time intervals that the particular average flow has equaled or exceeded during the period of record.

The flow is then plotted versus the respective percentage exceedance. Using class interval technique, the flow range is categorized into class intervals from the highest to the lowest order. A tally made for a number of flows in each class and the number of value greater than each class can be accumulated .The number of values greater than each class is divided by the total number of flows to obtain percentage exceedance. This percentage exceedance is plotted versus the upper (or mean) class interval to obtain the flow duration curve.



The flow corresponding to the 95% of exceedance that consider as design flow after subtracting 5-10% for losses and 10-15% for downstream ecology. From the above natural flow duration curve the flow for the 95% of exceedence is 1.5m^3/s ,the the design flow for SHP design is taken by considering losses and foe ecology case appropriately after studying that particular site condition.

4. Socio Economic aspects of SHP site and Demand and supply analysis

4.1. Introduction

Socio-economics is a multidisciplinary approach to studying economic phenomena and the social implications and effects that result from those phenomena. Socio-economics is the study of the relationship between society and economic phenomena (for this particular training the phenomena is due to MHP). Socio economic aspect deals with the positive and negative aspect of the project on the economy of the society in terms of environmental consequences and impacts due to implementation of MHP.

The socio economic aspect of MHP can easily be digested considering the following question-answer part that generalizes the relation of the project and socio economic aspect.

- What social and economic effects can be attributed to a particular MHP project and which would not have occurred in a comparable situation without project? Explain the relationship between the project and the benefiting community/ies.

- ✓ The project will allow the rural community to exploit its significant economic potential. To increasing liveliness to the community through improved commercial activities, health condition, and education, thus bringing about rural transformation and poverty alleviation.

- ✓ The project will improve the socio-economic conditions through creation of directly and indirectly employments.

- ✓ Improving education services due to improved and longer illumination period, which will encourage reading habits of village students.

4.2. SHP and Society

From the planning through the operation stages of a decentralized Small scale hydropower plant, the condition and perceptions of the rural people must be considered. Introducing small hydropower to rural people involves more than installing the equipment; it also involves an understanding of numerous social factors.

Project Implementation

There are two major methods of introducing Small scale hydro to rural areas each method has advantages and disadvantages. In one method, either governmental or nongovernmental organizations implement the project, and the local people purchase power from them. With this method:

- ➢ The local people usually will have a passive attitude toward the project, because they take no part in it and purchase the power from others.

- ➢ The project will neither motivate the local people nor help to develop their self confidence, because they neither originate the idea nor implement the project. Motivation and self-confidence are very important for rural development in general.

- ➢ The project will emphasize technology and management only.

In the other method, the local people implement the project cooperatively. With this method:

- Because the idea comes from the local people themselves, they will gain a sense of achievement, self-confidence and self-reliance, rather than depending on help from others. Experience shows that local involvement influences the use of the electricity generated by the project.

- Technicians must devote considerable time and patience to the project to achieve the type of success described above.

- Engineers and technicians should have a working knowledge of or at least be aware of the social condition and perception of the local community. In developing countries, such persons are still rare.

No matter which approach is used, the local people must be prepared to accept hydroelectricity.

Operational Cost

Rural people do not fully understand the concepts of operational costs and depreciation of Small scale hydro. To rural people, water is something that comes from nature. For hydropower projects, rural people consider only the cost of initial implementation because they regard water and thus waterpower equipment as something that will last forever. They do not readily comprehend why operation and maintenance procedures entail costs. This attitude may pose an obstacle to management of the project. Moreover, in many developing countries, rural projects are implemented, at no cost to the rural community. This policy may reinforce the lack of understanding of the nature of post-implementation costs of micro hydro projects.

Changing misperceptions is a long process. Studies should attempt to determine the readiness of the society for micro hydro by examining the country's social groups, economy and traditional ways of managing credit, finance, sharing, etc. Experience shows that standard management techniques will face many obstacles and, therefore, will be difficult to use. If a study of these aspects of a society indicates that change is possible, the community's leaders can be offered Achievement Motivation Training (AMT). Experience shows that AMT greatly influences attitudes and can be used to influence rural people’s management methods. Then the use of electricity for productive activities will become feasible.

4.3. Socio-Economic Aspects

4.3.1. Social Effects

Both the positive and the negative social aspects of hydropower give rise to issues that need to be addressed. On the positive side, a hydropower facility generates a lot of revenue from a natural resource, a river. Inevitably questions arise about the sharing of these revenues among local communities, government and investors, which are often government corporations. In some countries, individuals and companies can own water rights, in other countries they are leased, and in yet other countries the government owns all water rights.

Independently of the legal provisions, the good will and cooperation of local communities in the project area are very important for its success, and the developer will usually work out arrangements to share some of the benefits with local communities. This may take many forms including business partnerships, royalties, development funds, equity sharing, job creation, improvements of roads and other infrastructures, recreational facilities, sharing of revenues, payment of local taxes, or granting preferential electricity rates and fees for other water-related services to local companies and project-affected populations. Aside from the revenues, the water resources themselves are often shared among irrigation, fisheries, flood and drought management, navigation, recreation and tourism, etc. The necessity to share creates a fundamental need for sound water resources policies on which processes can be constructed to arbitrate among competing claims and to manage sharing arrangements on a day-to-day basis.

On the negative side, those hydro projects that include a dam and a reservoir sometimes require the involuntary displacement of people from the area to be inundated. Throughout the world, expropriation has become increasingly difficult for all types of infrastructure projects, including hydropower projects. Unfortunately, the historic record is not unblemished: there are too many instances where displaced populations were not treated fairly. Today, disrespect of basic human rights is no longer tolerated. Modern communications have empowered civil society and interest groups to make their voices heard. The solution is easy to say, but difficult to do: make sure that affected people are better off after the project, both in the short and the long-term. Experience has been gained with resettlement problems and there are now a number of ‘good practice’ projects where this issue was handled successfully.

The question of fisheries is both an environmental and a social issue. The environmental aspect involves avoiding or compensating for damages to species and habitats. The social aspects concern fisheries as a livelihood, a source of protein in the diet or a recreational opportunity. In warmer climate zones, and especially in tropical areas, the creation of still standing water body such as reservoirs can lead to increases in water borne diseases. Hydropower planners and operators must be aware of these risks, and take appropriate measures to prevent them.

4.3.2. Environmental Effects

The facts that hydropower is renewable, produces very low greenhouse gas emissions and creates no atmospheric pollutants are its main environmental advantages. At the same time, hydropower projects modify aquatic and riparian ecosystems, which can have significant adverse effects according to the project's specific site conditions. Nevertheless, these disadvantages have to be weighed against the advantages.

There are some widely held misconceptions on this subject. One is that 'small hydro' is always more benign than 'large hydro' and for this reason small hydro should be included widely in renewable energy programs and large hydro not. In fact, stressing the principle of eco-efficiency, one large hydro project can well be more benign than 100 small ones generating the same amount of electricity.

A second misconception is that large hydropower is renewable, but not 'sustainable' because of its negative impacts. A factor possibly contributing to this misconception is the enormous diversity of existing hydro projects, and consequently the variety of environmental impacts that may be encountered under different circumstances. For new projects, it is often possible to avoid or mitigate most of the impacts through good planning, design and operation, so that in the end the benefits prevail over detriments.

The detailed Socio-Economic aspects of SHP site with regard to social and environmental impacts are discussed below;

Impact of the energy systems on the local environment and communities

- **Positive impacts:**
    - Source of income
    - Technology demonstration and transfer
    - Essential energy services to the community
    - Reduced emissions into the atmosphere
    - Transportation of the power
    - Flexibility into electrical power system
    - Recreation and tourism
    - Agriculture and industries
    - Portable water supply

- **Negative impacts:**
  - Competition for resources
  - Air emissions
  - Soil and water pollution
  - Damage to the ecosystem
  - Visual intrusion
  - Biological diversity
  - Fisheries
  - Noise
- **Dam Drawbacks to Human Communities:**
  - Human Displacement
  - Flooding of Cultural Sites

  (Archeological and Modern)
  - Social disruption
  - Cost overruns
  - Socio-economic centralization
- **Dam Drawbacks to the Environment:**
  - Ecosystem Destruction
  - Fish Blockage and Wildlife Losses
  - Large-Scale Flooding Due to Dam Failures
  - Sedimentation and Salinity
  - Herbicide and Other Toxic Contamination
  - Evaporative Losses
  - Nutrient Flow Retardation
  - Release of greenhouse gasses

Environmental Consideration and Protection work

No or very little environment effect will be occurred due to the installation of a small scale run off river type SHP plant. However, some negative environmental impact is possible especially if the plant is not properly designed / installed. Thus, Necessary precautions need to be made during the survey & project implementation period in order to reduce the environmental impacts.

Necessary precautions to be made;

- Seepage from any component/structure should be minimized and a drainage system should be provided whenever possible.

- Extensive exercise should be made to fell a minimum number of trees and bushes, which may be possible by choosing an appropriate route for the canal and transmission line.

- Canal and penstock route should be surveyed and designed in such a way that the extent of excavation is minimum since the natural stability of the ground is disturbed due to too much excavation.

- Some minor environmental problems may also be encountered when the flow in the original stream becomes very low or it dries. Thus, minimum prescribed flow should be left in the stream, even though this means reducing the power output.

Protection work

At the time of survey the following things should be clearly identified:

- Flood prone area/part in intake, powerhouse and tail race part
- Land slide and land slide prone area
- Loose soil or rock falling
- The length of the protection-needed area should be accurately measured and the surveyor should provide suggestion for the appropriate protection work.

Significant impacts on riverine & downstream ecosystem

- Loss of riverine & terrestrial biodiversity
- Adverse impacts on livelihoods in floodplains
- Poor record of ecosystem mitigation
- Some reservoirs have created habitats for biodiversity
- Rreservoirs emit green-house gases Move beyond the simple "balance-sheet" approach that
- Trades off losses and gains between groups
- Impoverishes some people
- Excludes people and limits awareness
- Overlooks sustainability aspects
- Induces conflict and higher costs

Define whose rights & what risks

- Rights
- No party's rights should extinguish another's
- Where rights compete – negotiated agreements are needed
- Turning Conflict into Consensus

- Gain public acceptance
- Assess options
- Address existing dams
- Sustain rivers and livelihoods
- Recognize entitlements and share benefits
- Ensure compliance
- Share rivers across boundaries

Gaining public acceptance

- Dams affect existing rights & create wide range of risks
- Opportunities exist for achieving a higher level of equity
- Recognise rights of indigenous & tribal peoples
- Achieve outcomes through binding formal agreements

- **Comprehensive options assessment**
    - Failure to adequately define needs & assess options led to dispute
    - An early and open examination of options can avoid poor projects
    - Raise the significance of social & environmental aspects
    - Increase the effectiveness of existing systems as a priority

- **Address existing dams**
    - Most dams that will operate in the 21^{st} century already exist
    - Considerable scope exists for improving benefits
    - Remedy outstanding social issues; enhance mitigation, restoration & enhancement of ecosystems
    - Use licenses to formalize operating agreements
- **Sustain rivers and livelihoods**
    - Rivers support millions of livelihoods
    - Dams cause significant and often irreversible effects on ecosystems
    - Value rivers, ecosystems & endangered species
    - Emphasise avoidance of impacts
    - Maintain ecosystem integrity through environmental flows

Recognize entitlements & share benefits

- Many people displaced - many more unrecognised
- Including those who depend on a river's resources
- Recognise rights and assess risks as a basis for negotiations
- Agree legally enforceable entitlements
- Adversely affected people become first among beneficiaries

Ensure compliance

- Many policies and guidelines exist but often a failure to fulfill obligations
- Need a compliance plan covering all commitments to people and the environment
- Introduce initiatives to reduce corruption
- Develop incentive framework for compliance

- **Share rivers across boundaries**
    - Conflicts over transboundary rivers due to power imbalance
    - Experience suggests disputes can be resolved
    - Endorse the UN Convention principles
    - Go beyond sharing water - to sharing the benefits
    - Encourage consistent policies for financing agencies

Improved Outcomes for Nations, Affected People & the Environment

Development opportunities for all by;

- Respecting human rights
- Meeting development needs for water, food & energy
- Sustainable resource use

Fundamental Considerations for Hydropower Development

- Socio-Environmental Impact and its mitigation
    - Impacts of Reservoir Construction
    - Impacts during operation & maintenance
    - Impacts due to installed infrastructure; transmission lines,
- Socio- Economic evaluation of the project and financing potential
    - Investment fundamentals

- Economic Evaluation of investment schemes
- Financial Viability Analyses
- Tariffs and incentive schemes

Social aspects

- Consideration of Land ownership.
- Legal consideration
- Water rights
- Regional state regulatory permits
- Federal licensing

2.4. Socio-Economic Analysis

Before a new power scheme can be built, its technical and economic feasibility has to be established in order to convince both the developer and his backers - governments and financing agencies - that the proposed project is sound and credit-worthy. Soundness implies satisfactory technical performance and reliability of the project throughout its asset life. Credit-worthiness demands that:

- ✓ The project offers the least-cost solution for a given supply of electricity consistent with technical soundness;
- ✓ Redemption charges on the capital invested in the project and the operating expenses are adequately covered by the revenue the project can earn.

Credit-worthiness is conventionally judged on the basis of economic merit, i.e. on an evaluation of quantifiable costs and benefits from which suitable judgement factors, such as benefit/cost ratios or internal rates of return, are derived. If the anticipated return is at or above the opportunity cost, the economic merit of the project is considered to be adequate and the investment in it is considered to be justified. The lower the return in relation to the opportunity cost the smaller is the incentive to invest in this project and the lower its merit or credit rating. On the basis of the evaluation of quantifiable parameters, the project may have to be rejected if its credit rating is too low since funding for it is unlikely to be forthcoming. This approach may well be the right one for project selection. A project that fails the test of economic viability may have to be redesigned, or replaced by a more promising one.

In addition to the directly assessable benefits, an electric supply brings also intangible benefits which are generally not quantifiable. If the economic merit of a project is at or below the borderline for credit-worthiness, intangible benefits may well improve its attractiveness and may help in coming to a decision in its favour. Such benefits can arise on two levels:

- On the national level by way of encouraging social and economic advancement of a hitherto disadvantaged area or region and consequently of an improved social and economic homogeneity for the country concerned;
- On a local level by way of the provision of a socially important service, by improved employment opportunities and by strengthening of the local economy.

Socio-economic analysis deals essentially with the study of intangible factors and with their impact on the merit of a proposed scheme. The impact can be very great, particularly in a developing country environment where the social and economic benefits of an improved electricity supply may far outweigh any directly attributable advantages. On the other hand, the merit may be seriously impaired by whatever disturbance this project might create.

It is important therefore to appraise socio-economic impacts in the planning stages of every project and to determine in what way such impacts could affect a decision in its favour. Such assessment is made difficult by the fact that it can be no more than subjective because of the difficulty of quantifying socio-economic impacts in an objective and universally acceptable way but an effort towards impact appraisal should nevertheless be attempted.

Economic Stimulation

Stimulation of the local economy is potentially the most valuable benefit which an ample supply of electricity - in substitution or support of non-commercial energy - can bring. Benefit can result from a number of activities:

- Improved agricultural outputs brought about by better irrigation,
- The local processing of agricultural produce,
- More intensive local, industrial or commercial activity which can lead, in turn, to a move away from a subsistence economy to a monetary economy.

Quantification of economic impacts is difficult but not impossible. Scenarios can be set up to permit evaluating the returns that might be achieved from creating employment, however limited, and from stimulating local economic activity. Such scenarios could be in the form of test cases for which the economic inputs and outputs are valued and the economic benefit then numerically computed. The result can provide factors for estimating the extent of the economic impact and this can be related in turn to the amount of investment needed to achieve this impact. An economic overview rating can thus be developed which, although by no means precise, can be used to support an investment decision on the particular energy supply scheme under consideration.

2.5. Environmental Analysis

Environmental and socio-economic impacts can be difficult to separate. They are inter-related at both the local and the national level, depending on the scale of the impact that is being experienced, or anticipated to be experienced. Environmental impacts deal with the effect of a scheme on ambient conditions - ecology, flora, fauna, amenity and living conditions; these enter partly or even wholly into the socio-economic situation.

Methods for appraisal are judgemental in both cases and rely largely on empirical rather than directly quantifiable factors. Both call for a considerable investigative effort even in the pre-investment stages; it is often difficult to secure adequate funds for a sufficiently far-reaching investigation. There is a basic difference in the ultimate objective:

- Environmental analysis aims to find a solution that creates a tolerable impact at whatever the cost entailed; if the project cannot bear this cost it will have to be discarded.

- Socio-economic analysis looks for ways of supporting a project through quantified and hitherto intangible factors in its favour without adding to its cost; such quantification can also cause a project to be abandoned if its negative features should cause its merit to be reduced below an acceptable limit.

An attempt at objective valuation will in both cases avoid decision-taking on an emotive basis. Depending on the nature, scale and conception of a project, environmental disturbance can be far-reaching, particularly with nuclear and thermal generation. The impact is more constrained in the hydro case and does not usually range beyond the local area and the river course on which the development is to take place. The distinction between environmental and socio-economic issues becomes greater as their assessment moves away from the 1ocal level. Environmental matters can touch on the political and policy-making plane - even internationally, whereas socio-economic issues remain well within the national sphere and often do not reach beyond the local level.

Socio-economic features affecting primarily the local level:

- Effect on local amenity (visual, noise, recreation, disturbance during construction, amenity needs for new residents).

- Changes in land use (residential and occupational displacement due to changes in water levels and site requirements, opportunities created through soil improvement and water storage).

- Housing situation (displacement, needs for additional housing, related services and infrastructure - but excluding housing directly associated with the project and corresponding services which should be included in the project cost estimates).

- Employment (direct and indirect, temporary and permanent, displacement).

- Economic activity (opportunities created by the physical features of the development - land drainage, river training, water storage, project construction and operation; indirect benefits arising from electrification and the scope offered for advancement in the monetary economy but also the effects of disturbance and displacement brought about by the project).

- Social activity (improvement of living standards, enhancement of the quality of life through the provision of an electricity supply).

- Infrastructure development (access, communications, transportation, utilities and services but only those features which do not form part of the direct costs of the project and to be included in the project cost estimates).

- Financial effects (fiscal, income levels, consequential investment requirements - but excluding financing charges for the project itself and directly project-related expenditure which form part of the project investment).

Socio-economic features impacting at the national level

- Economic features (reduced economic disparities - sectoral and regional and stimulation of employment-creating activities, enhanced contributions to national income and national product, benefits of electrification, but also a possible subsidisation requirement for local electricity usage).

- Social features (reduced social disparities in living standards, reduced need for governmental support, enhanced quality of life).

- Infrastructural features, (greater development potential for the local infrastructure, formation of growth centres in outlying areas, encouragement of secondary activities).

Socio-economic and environmental features overlap to some extent and this makes it questionable whether strict separation and independent analysis of the two issues is warranted. The ultimate objective is to determine the feasibility of a new power scheme which caters for a growing need for energy but, on the other hand, has to face increasing sensitivity to all disturbance of ambient conditions.

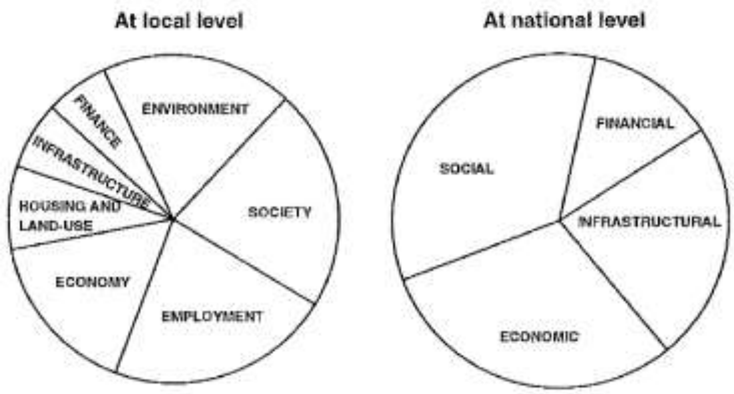


Figure-: Sectoral impacts

Quantification

If socio-economic factors are to be used in support of the decision-making process for or against a hydro project proposal, it is important that they should be quantified as far as this is possible. Some factors can be quantified fairly readily on the basis of local investigations, for example:

- Changes in land use which have direct commercial or financial consequences;
- Housing or re-housing needs;
- Creation of employment or unemployment;
- Changes in commercial, agricultural, industrial or agro-industrial activity;
- Changes in fiscal contributions resulting from increased, or reduced, local activity.

Economic Analysis

Socio-economic effects can be taken into account as a benefit or as an extra cost in the conventional economic project analysis. Only parameters which can be expressed in monetary terms can be included in the numerical analysis. Intangible and unquantifiable features may help in appraising the results of this analysis and forming a judgement on the risks involved in adopting a given solution. If the risks appear to be large, a higher return may be asked for to offer some compensation, say an internal return several percentage points above the opportunity cost. The opposite approach may be appropriate if unquantifiable benefits are very substantial.

For numerical analysis, the Net Effective Cost (NEC) of a project can be computed as a capital sum by the equation

$$NEC = C + 0 - S$$

Where: C = the capital investment, and re-investment, in the project and also the capital and capitalized socio-economic costs (disbenefits),

O = capitalised annually recurring operating, maintenance and other project-related expenses, including recurring costs for socioeconomic impact mitigation (e.g. regular compensation payments),

S = capitalised benefits achieved by the project including quantifiable socio-economic benefits (once-off or recurring figures).

The net effective cost can also be expressed as an annual sum on a 'per kW' basis. In this case, all the quantities entering into the equation have to be levellised to form equal annual installments so that

C = capital charges (interest and depreciation) on the total cost of the project, per kW and per annum, for a given annuity rate,

0= annually recurring operating expenses per kW installed,

S = annually recurring savings and benefits, per kW, levellised at the given annuity rate.

None of the items making up the NEC will be the same in every year. It is essential therefore to capitalise all expenditures and benefits over the asset life and then compute the NEC either in terms of the present-worths (as capital sums) or levellised annually recurring costs (making use of the capital recovery factor).

A convenient procedure is to establish the annual cash flow over the asset life of the project. Costs (C and 0) are included for the year in which they arise as a positive entry and benefits (S) as a negative entry. Disbenefits are treated as additional costs. In years in which the value of S exceeds the sum of C + 0, the cash entry will be negative. The cash flow is then present-valued for a range of discount rates. The discount rate at which the NEC is zero will be the internal rate of return for the project.

Since there can be some uncertainty over the valuation of socio-economic benefits or costs, the estimates included in S can be varied for different assumptions and the corresponding internal rates of return calculated. The sensitivity of the results to changes in assumptions can thus be tested. An element of judgement cannot be avoided when deciding on the acceptability of a given result.

Discussion of project appraisal processes is generally focused on the comparative analysis of alternative solutions. The method described here deals with only one solution but the basic concept of the two approaches is in fact the same. Alternative solutions for satisfying given electricity requirement are set up in order to determine the comparative benefit which the chosen project is likely to achieve. This benefit is expressed by the term S in the equation above. In order to identify the benefit, it may still be necessary to prepare cost estimates for an alternative project so that the savings obtained with the chosen project can be fully evaluated.

A number of socio-economic features will probably defy quantification and one may question whether it is worth taking account of only that part of the features that can be quantified - often only with difficulty. The approach suggested here is that every impact that can be quantified should be included in the analysis. More effort should be devoted to quantification so that, in time, experience values will be established which make it possible to place greater weight on socio-economic appraisal and reduce emotive judgement.

Case Study

Socio-Economic Environment

The socio-economic assessment has been carried out with the objective to provide a comprehensive analysis of the existing socio-economic conditions of the population in the future reservoir area.

Population and Settlement: In 2006, an estimated 253,412 people were living in the 67 Peasant Associations located around the Gibe III reservoir area. The average population density within the Peasant Associations located around the reservoir area is 127.8 persons per km^2. More than 13 different ethnic groups live around the future reservoir area and the major ethnic groups are - Hadiya (25.3 percent), Wolayita (23 percent), Oromo (20.4 percent), Dawro (6.9 percent), Kembata (6.7 percent), Tembaro (5 percent), and less than one percent of Amhara, Keffa and Sodo Gurage, Silte and Sebatbet gurage. The rest of the ethnic groups constitute less than 3.3 in total.

Agriculture and livestock populations: The main areas of farming are confined to the middle or upper slopes of the hills where the settlements are situated. The farmers in the project area (mainly on the high land) produce small quantities of a wide range of crops (15-20 different crops), including cereals, roots, tubers, pulses, spices, coffee and fruits.

The main livestock populations in the project area are cattle, sheep, goats, poultry and equines. Few farmers own oxen, although this varies throughout the project area.

Public Health: The major health problems of the project area are reported to be infectious diseases and malnutrition. Waterborne and vector borne diseases are also prevalent in the area. The project areas are highly endemic for malaria with continuous transmission and it is by far the most important of the diseases. The presence of several rivers (tributaries to Omo River) provides ideal breeding habitats for mosquitoes.

Cultural, Religious, Historical and Archaeological Sites: The UNESCO designated heritage site is not in close proximity to the proposed Gibe III dam and reservoir areas. The lower valley of the Omo River which is designated as a UNESCO heritage site is located far downstream and will not be affected by this scheme. No visible archaeological remains, which have scientific, cultural, public, economic, ethnic and historic significances, have been observed in the area and dam sites. The sites have no archaeological importance. However, a Chance Find Protocol has been prepared to cover any unexpected finding.

4.2. Demand and Supply Analysis

4.2.1. Demand Analysis

The need for and the purpose of demand forecasts are fully recognized. Not only the size but the “shape” of the demand is important factor in planning the power supply. By shape is meant the daily, seasonal and annual variation of the demand curve.

A lot of planning information can be derived from the demand curves of supply systems. They will indicate need for regulation of watercourse contemplated for development as they give information on the water needed for generation on a daily, seasonal and annual basis. Such demand curves also provide data needed to determine the size of generation, installations, and unit size and transmission facilities.

The minimum installation in the development should at least satisfy the energy and power demand required by the load curve often termed *as firm power or energy* and the maximum size can also be fixed by referring the peak demand.

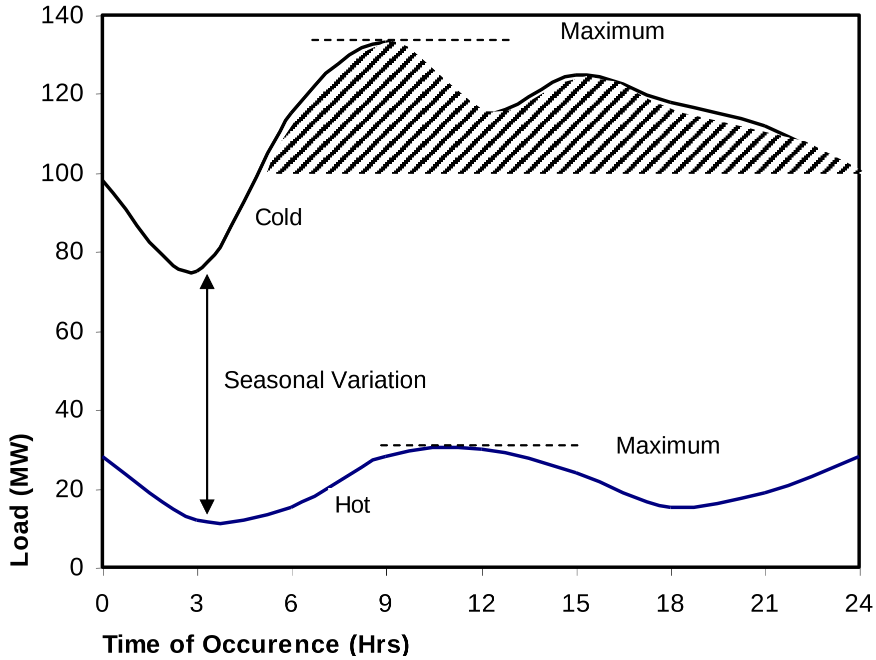


Figure-2: Typical 24 Hours Load Curve

The term *"firm"* is given to supply which can be guaranteed at all times or at large percentage of the time (e.g. 90% of the time). This type of supply is distinguished from the supply governed by the availability of water, which is often termed *"secondary"*. Supply available as a result of seasonal excess of water or abnormal runoff is termed *"surplus"* as the alternative to generation is letting the water runoff (spilling). Some of the river discharge is by nature firm, usually the minimum flow, but its share of the total discharge can be increased by introducing regulation of the river, i.e. provision of storage reservoirs from which water can be drawn during dry periods.

The value of having guaranteed supply of water and the additional costs involved in regulation is reflected in the price of electricity and firm supply commands a higher price than secondary and surplus power and energy. The highest priced energy, however, is often the supply termed *"peaking"*. By peaking is meant the load which can be supplied to meet the variation in demand in a supply system. It is measured as excess of the average demand over a period of time, day, season or year.

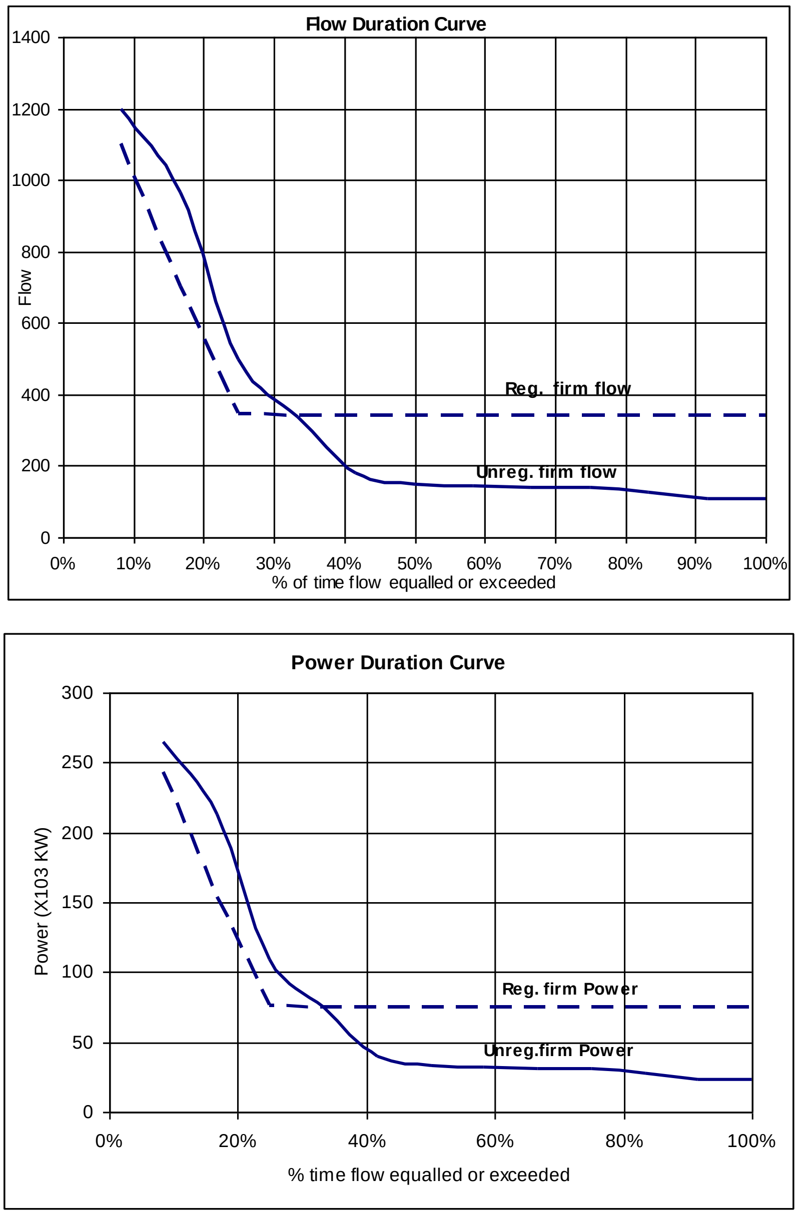


Figure-3: Unregulated and Regulated firm power

4.2.2. Supply Analysis

The network of consumers which can be reached by a generation scheme is known as the supply system. Supply systems have to be studied in connection with the planning of new generation facilities in order to ensure that the new scheme will satisfactorily fit the system and the purpose for which it was originally conceived. The additional power should also be fully compatible with the requirements of the system it will supply. System studies will have to explore:

- The influence of the new scheme on the operation of the existing system and on the structure of its production costs
- The effects of the new scheme on the expansion of the system
- The optimum dimensioning of the new scheme in relation to system requirements and the phasing of its development
- The transmission capacity and any strengthening of the network needed for absorbing the output from the new generation scheme

In order to achieve a balanced and orderly development of the power supply to an area, the planning has to be based on reliable knowledge of the market, the present and the future demand. Power market surveys are means of evaluating the present and potential markets for electric energy in a defined area. They are based on registration of the prevailing demand and supply situation, adjusting for possible suppressed demand due to under supply, high tariffs, etc. as well as overuse due to excess capacity in the system, price subsidies and similar inducements.

The market survey will consider the effects on the use of electric energy within the survey are of such factors as:

- Geographical location
- Natural resources
- Industrial development
- New power uses
- The economic status and prospective growth of the population
- Substitution loads

5. Design aspects of civil components of SHP

Every component of the system that envisages coming up with tangible power output should be dimensioned and analyzed under the framework of safety, economy and sustainability.

The following considerations should be kept in mind while conducting the design and analysis works of any Small scale hydro power plant.

1. Operation and maintenance of the system should not be too complicated to be handled by the community.
2. There should be maximum utilization of local materials and resources to create sense of self-reliance among the beneficiaries.
3. The system should be as simplified as possible to enable the community to grasp the ideas and working principles easily.
4. The work force and construction methodology should be organized in such a way that community should be participated according to the requirements.

5.1. Civil Work components

All hydraulic structures found at a diversion site and along a water conveyance route are categorized under this group. These structures control and regulate both the quality and quantity of water flowing in to the turbine.

- Diversion weir and Coffer dam
- Intake
- Weir
- Settling Basin
- Headrace
- Fore bay
- Support Structures
- Power House/Tailrace

5.1.1. Coffer dam

Cofferdam is a temporary dam or barrier used to divert the stream flow and enclose the area dry during construction. The design of an adequate cofferdam involves the problem of construction economics. The height of which a cofferdam should be constructed may involve an economic study of cofferdam height versus diversion work capacity, including diversion of design flood.

But the site select for the weir site is to narrow, and that cause a difficulty for diversion and placing the cofferdam. What is planned is to apply a method of two stage construction. Instead of diverting the river into a tunnel or channel, it is some times more economical to have two stage construction of weir. In such a case, the flow is, first of all, diverted and confined to one side of the channel by constructing semi circular coffer dam. Then construction progresses in free water zone. On the next stage the diverted water is allowed to overtop or to pass through outlet on the side of already constructed portion of the weir/intake.

This method is suitable only during construction of concrete weir. Generally, for this site the cofferdam is constructed of materials available at the site i.e. earth fills cofferdam.

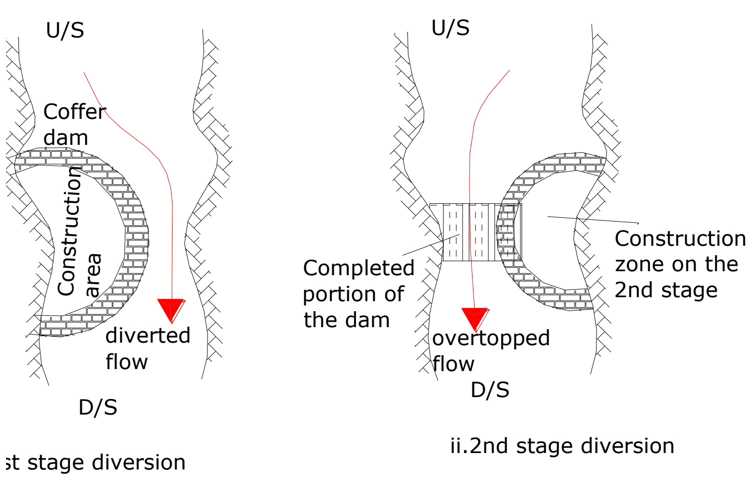


Figure 1. 12

Figure 1. 13Typical photo of Beshuro gute site's coffer dam

5.1.2. Diversion Weir

The main function of the diversion weir is to create sufficient head for off-taking channel and hence to ensure that the channel flow is maintained when the river flow is low. Among various types of weirs, an ogee weir is efficient in disposing off floods anticipated during rainy seasons.

All components of the weir would be constructed from masonry, except the pillars for Flush and intake gate. Both hydraulic and structural designs of each component are listed below.

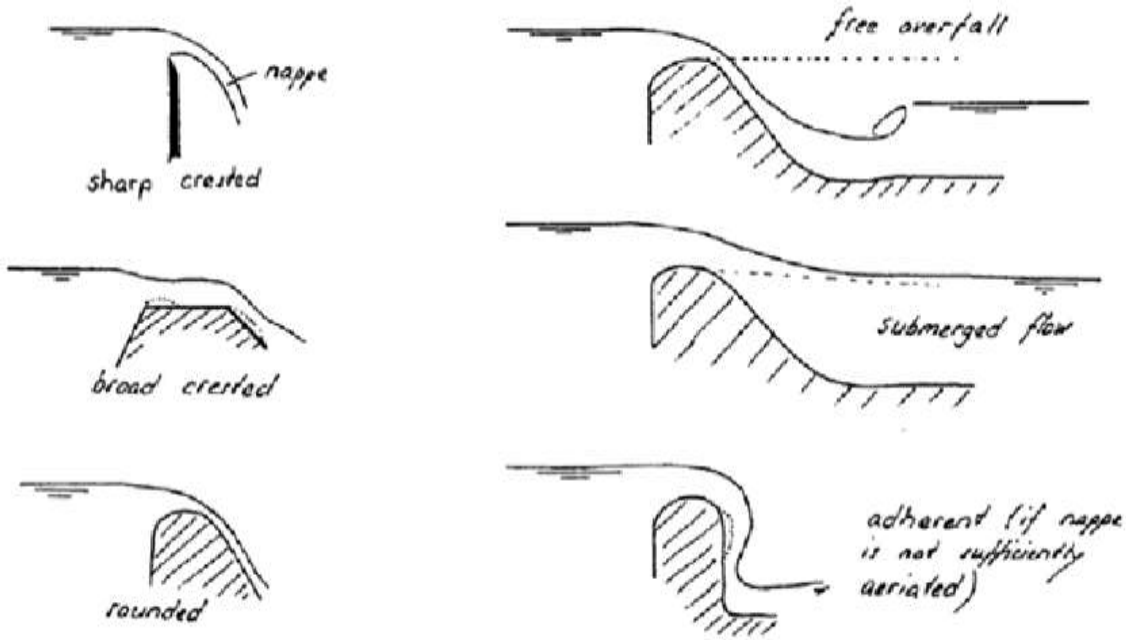


Figure 1. 14Types of weir

Diversion weirs can be broadly classified in to three

i. Broad crested weir
ii. Sharp crested weir
iii. Ogee weir

Crest Shape of Overflow Spillways

- Bazin's made comprehensive laboratory investigation for nappe shapes.
- The used of Bazin's data in design will produce a crest shape that coincides with lower surface of as aerated nappe over a sharp-crested weir.
- Such a profile is known as Bazin profile Advantage

– Should couse no negative pressure on the crest (the presence of negative pressure will lead to danger of cavitation damage).

– In selecting a suitable profile avoidance of negative pressure should be considered an objective, along with such other factors as maximum hydraulic efficiency, practicability, stability & economy

- On the basis of the Bureau data, The U.S Army. Corps of Engineers has developed several standard shapes at its Waterways Experimental Station. Such shapes designed as the WES standard spillway shapes, can be expressed by the following equation:-

$$X^n = K\, H_d^{\,n-1}\, Y$$

Where

- X and Y are Coordinates of the crest profile with the origin at the highest point of the crest.
- H_d is the design head excluding the velocity head of the approach flow
- K & n are parameters depending on the slope of the upstream face. values of k & N are given as flows

Table 1. 1 values of K and n

Slope of upstream face	k	n
Vertical	2.000	1.850
3 pm 1 (V = H)	1.936	1.836
3 on 2	1.939	1.810
3 on 3	1.873	1.776

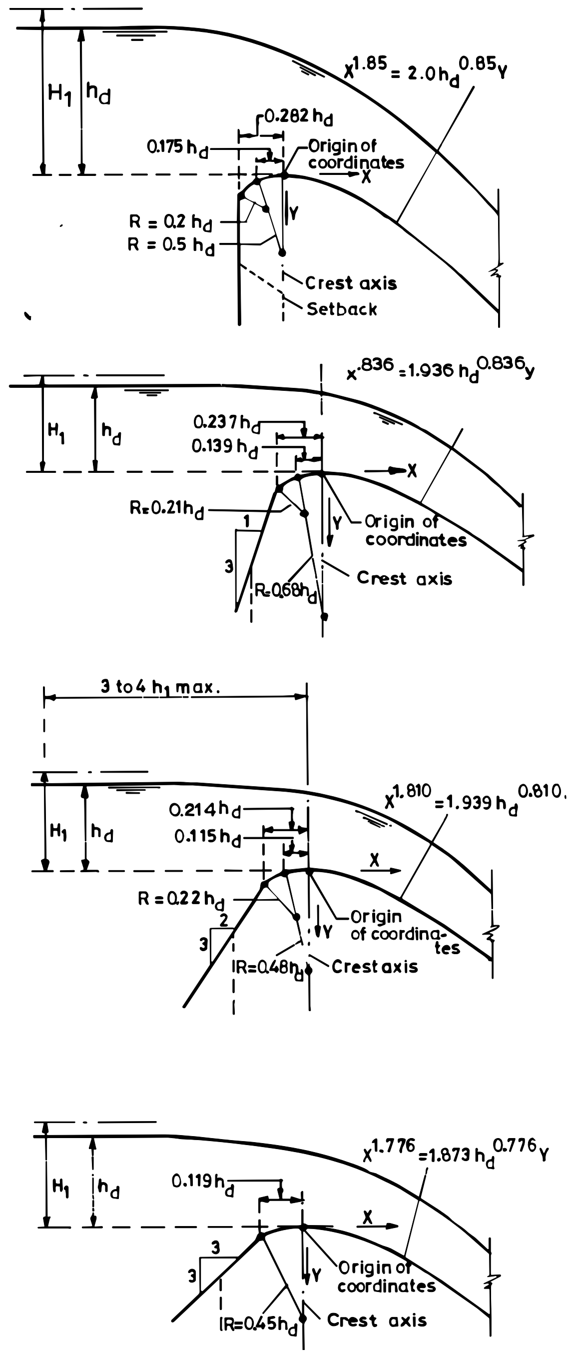
Discharge of WES Spillway

- The discharge over a spillway can be computed by an equation in the form of SCW/BCW

$Q = CLH_e^{1.5}$

H_e the total energy head on the crest, including the velocity head in the approach canal.

- The effect of the approach velocity is negligible when height h of the spillway is greater than $1.33H_d$ ($h > 1.33\ H_d$), where the design head exclude the approach velocity head.
- Under this condition, i.e. $h/H_d > 1.33$, $H_e = H_d$ can be taken (the approach velocity head is negligible) and the coefficient of discharge C has been found to be C = 2.21



X1.85 = 2.0 hd0.85Y
H1
hd
0.282 hd
0.175 hd
Origin of coordinates
X
Y
R = 0.2 hd
R = 0.5 hd
Crest axis
Setback
x.836 =1.936 hd0.836y
0.237hd
0.139 hd
R=0.21hd
1
3
R=0.68hd
Origin of coordinates
Crest axis
3 to 4 h1 max.
X1.810 =1.939 hd0.810.
0.214hd
0.115hd
R = 0.22 hd
2
3
R=0.48hd
Origin of coordina-tes
Crest axis
X1.776 =1.873 hd0.776Y
0.119hd
R=0.45hd
Origin of coordinates
Crest axis

Figure 1. 15

Intake Structures

Intake structure is a structure to divert water into a conduit leading to the power plant or water ways and provided as an integral part or in isolation from the diversion weir.

A water intake must be able to divert the required amount of water into the power canal or into the penstock without producing a negative impact on the local environment and with the minimum possible head loss.

- A water intake designer should take three criteria into consideration:

– ***Hydraulic and structural criteria common to all kind of intakes***

– ***Operational criteria, e.g. percentage of diverted flow, trash handling, sediment exclusion, etc- that vary from intake to intake***

– ***Environmental criteria, fish diversion systems, fish passes- characteristics of each project.***

- ***Functions of Intakes***

– To control flow of water in to the conveyance system.

– To provide smooth, easy and vortex or turbulence free entry of water in the conveyance system this is to minimize head loss.

– To prevent entry of coarse river born trash matter such as boulders, logs, tree branches etc. Provision of trash racks at the entrance achieves this function.

- The water intake should be equipped with
- a ***trash rack*** to minimize the amount of debris and sediment carried by the incoming water;
- ***a settling basin*** where the flow velocity is reduced, to remove all particles over 0.2 mm;

- a ***sluicing system*** to flush the deposited silt, sand, gravel and pebbles with a minimum of water loss; and
- a ***spillway*** to divert the excess water.

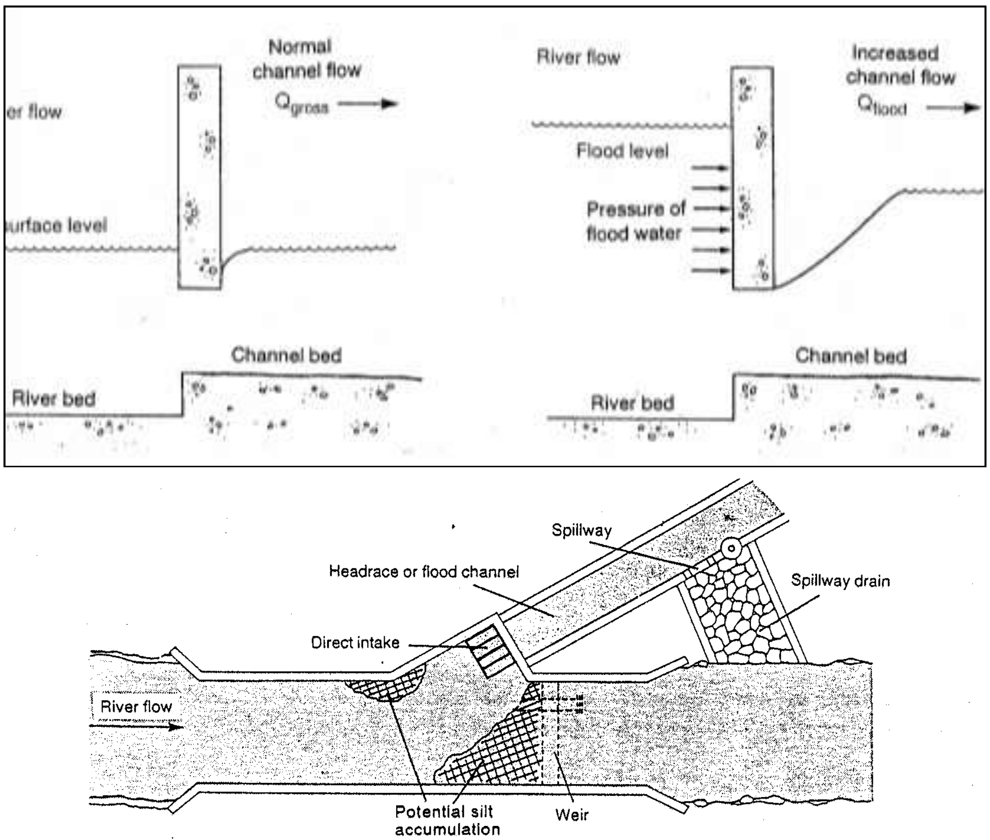


Figure 1. 16 Cross section of intake

- One of the major functions of the intake is to minimize the amount of debris and sediment carried by the incoming water, so trash racks are placed at the entrance to the intake to prevent the ingress of floating debris and large stones.

- A trash rack is made up of one or more panels, fabricated from a series of evenly spaced parallel metal bars.
- Vertical strips:
- Size: 5x40 to 5x75
- Spacing: =< 75mm
- Velocity: 0.2m/s.
- .

For the condition of no vortices at intakes, the following empirical relations may be used:

$$Y_s > 0.30V\sqrt{D} \quad \text{for symmetrical approach}$$
$$Y_s > 0.40V\sqrt{D} \quad \text{for asymmetrical approach}$$

Where, Y_s is necessary submergence depth,

V is velocity at inlet to the canal,

D is diameter of the conduit.

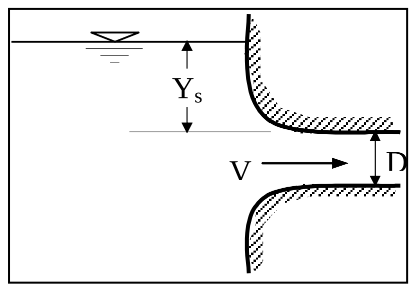


Figure 1. 17

Inlet Aeration

Intakes normally have a bulk head gate at the front and a control gate inside on the downstream side. An air vent is always provided just downstream of a control gate. The functions are:

i) To nullify vacuum effect, which could be created when the penstock is drained after control gate closure

ii) Intake gates operate under conditions of balanced pressure on both sides of the gate. Thus the conduit is required to be filled with water through a by-pass pipe. The entrapped air is therefore driven out through the air vent.

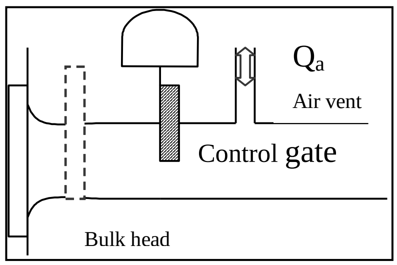


Figure 1. 18

Size of the air vent

i. 4th Congress on Large Dams (ICOLD)

Area of air vent =10% of control gate area

ii. USBR design guide: Capacity of air vent = 25% of conduit discharge

Water conveyance system

After flowing through the intake structure, the water must pass through the water conveyance system may be either of closed conduit type, as shown in Figure1 (tunnel off-taking from upstream of the river diversion) or could be open-channels as shown in Figure 2. High pressure intakes, for example as in the entry to penstocks (Figure 9) would be either reinforced concrete lined or steel lined. In this section we discuss the various types of water conducting passages.

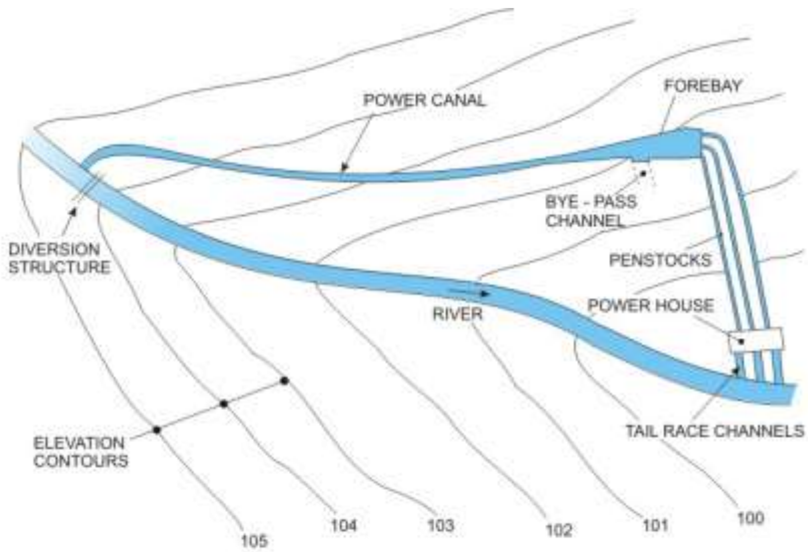


Figure 1. 19 Alignment of a power canal along a hill slope

Head Race

Head race may be a power canal, a pressure tunnel, or a pipe, which in most cases conveying water from intake structure to surge tank, forebay or pressure shaft, depending on the arrangement of the scheme.

Canals

Canals are appropriate choice when the general topography of the terrain is moderate with gentle slopes. However, when the ground is very steep and rugged, it becomes uneconomical to construct canals as it follows longer distances and/or needs provision of cross-drainage works and deep cuts and fills at a number of appropriate locations. In such cases, it is advisable to go for tunnels or pipes. The choice, in fact, has to be made based on economic analysis. Where the topography of the region presents special formations, the alternating use of open-canal and open-surface tunnel sections may ensure the most economical development.

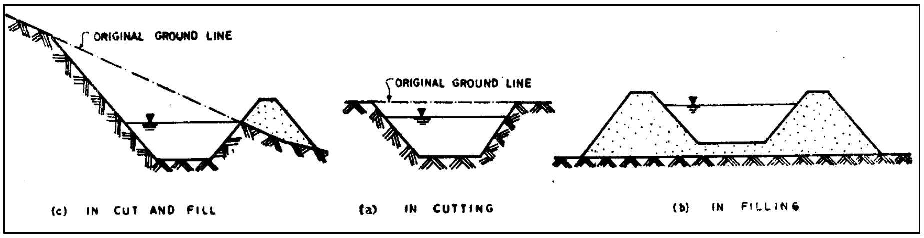


Figure 1. 20

Canal Lining

Power canals may be lined or unlined. The lined canals are usually lined with impervious material such as concrete, masonry, or clay.

Canal lining might be carried out to:

- Reduce seepage losses
- Reduce canal surface roughness
- Prevent the growth of weeds
- Reduce damage caused by erosion, rodents, and livestock
- Reduce the required volume of excavation
- Permits the use of rectangular x-section

Canal Design

Canal design involves determination of the following:

- Carrying capacity, velocity of water in the canal & roughness coefficient of the canal surface, slopes, Cross-sectional profile of the canal

Carrying Capacity and Velocity

For the hydraulic design, the discharge is computed from continuity equation as **Q=V.A.** The mean velocity, **V,** is determined from any of the continuity equations. The roughness coefficient is specified from the bed material type.

A *Chezy's equation*

$$V = C\sqrt{RS}$$

Several equations are available to determine the value of C.

i) Bazin's formula $C = \dfrac{87}{1+\dfrac{m}{\sqrt{R}}}$ Where m is roughness factor

ii) Maning's Formula $C = \dfrac{1}{n}R^{1/6}$ Where n is Maning's roughness coefficient

Table 1. 2 Permissible flow velocities in a canal

Maximum velocity		Minimum velocity
Bed Material	V_{max} (m/s)	V_{min} (m/s)
Sand	0.4	To keep any sediment from settling out, the minimum velocity in a canal should not be less than 0. 3 m/s.
Sandy loam	0.6	
Loam	0.6	
Clayey loam	0.8	
Clay	2.0	
Gravel	3.0	
Masonry	3.5	
Asphalt	4.0	
Concrete	5.0	

Power Canal Slopes

In plain areas use slope between 5 to 20 cm/km (0.005 to 0.02 %). In mountainous areas slopes are as steep as 1 to 2 m/km. The canal bed slope can also be estimated using the Manning's equation: $S = \frac{n^2 V^2}{R^{4/3}}$

The slope found from the above equation should nearly coincide with the available natural topography. Otherwise, a different slope should be computed by choosing other values for the velocity within the permissible limit until a satisfactory result is obtained.

Table 1. 3 Values of roughness coefficient 'n' for different canal materials

Canal material		Roughness coefficient
Earth canals	Clay, with stones and sand	0.020
	Gravelly or sandy loam	0.030
	Lined with coarse stones	0.040
Rock canals	Medium coarse rock muck	0.037
	Rock muck from careful blasting	0.045
	Very coarse rock muck	0.060
Masonry canals	Brickwork, well pointed	0.015
	Normal masonry	0.017
	Coarse rubble masonry	0.020
Concrete canals	Smooth cement finish	0.010
	Concrete, unplastered	0.015
	Coarse concrete	0.018
	Irregular concrete surfaces	0.020

Cross-sectional Profile

The material in which the canal is constructed generally dictates its cross-sectional profile. The common cross-sections used for canals are described in the following paragraphs.

A ***semi-circular*** cross-section is the most efficient profile because, for a given canal slope and cross-sectional area, it conveys the maximum flow. However, this form is impractical to excavate. It is therefore used primarily with materials which lend themselves to this shape. Examples are prefabricated concrete, sheet metal, and wood-stave sections.

A ***trapezoidal*** cross-section is the most widely used profile for both lined and unlined canals excavated in earth. If the canal is unlined, the maximum side slope is set by that slope at which the material will permanently stand under water. The magnitude of the side slope of a lined trapezoidal canal depends on the nature of the material on which the lining will rest, but usually steeper than unlined canals. In general, it should be nearly equal to the angle of repose of the natural soil so that no earth pressure is exerted on the back of the lining. The bank of a lined canal resting on almost any free-draining material requires slopes not steeper than 1:1.

For a trapezoidal canal with a given side slope, the most efficient cross-section is one in which a semi-circle can be inscribed in the wetted area. For this section, it can be shown that the length of either sloping side of the wetted area is half its top width.

Table 1. 4 Suggested side slopes for trapezoidal canals

Canal material	Side slope (V:H)
Firm rock	1:1/4
Fissured and disintegrated rock	1:1/2
Clay	1:3/4
Clayey loam	1:1
Loam	1:3/2
Sandy loam	1:2
Sand	1:3
Lining	1:1

A ***rectangular*** cross-section is often most appropriate when excavation is undertaken in firm rock. It is also commonly used when the canal incorporates properly constructed masonry walls. Use of a rectangular canal reduces the excavation required. For the most efficient rectangular cross-section, the width of the canal is twice the depth of the wetted area and, like a trapezoidal section, is a section in which a semi-circle can be inscribed.

Freeboard Allowance:

Freeboard is provided above the design water level for safety purposes. For earth canal the lower limit is 35 cm and the upper limit is 140 cm. Generally the free board = [0.35+1/4h] m. Where h is depth of flow. Allowances should be made for bank settlements. For lined canals, the top of the lining is not usually extended for the full height of the free board. Usually it is extended to 15cm to 70cm above the design water level

Settling Basins

The water drawn from a river for power generation may carry suspended sediment particles. This silt load may be composed of hard abrasive materials such as ***quartz*** and will cause damage or wear to the hydro-mechanical elements like turbine runners, valves, gates and penstocks. To remove this material a structure called ***settling basin*** should be constructed, where the velocity of the flow will be reduced resulting in settling out of the material, which has to be periodically or continuously flushed out.

In order to satisfy the requirement for a good hydraulic performance the basin is divided into three main zones: *inlet zone*, *settling zone*, and *outlet zone*.

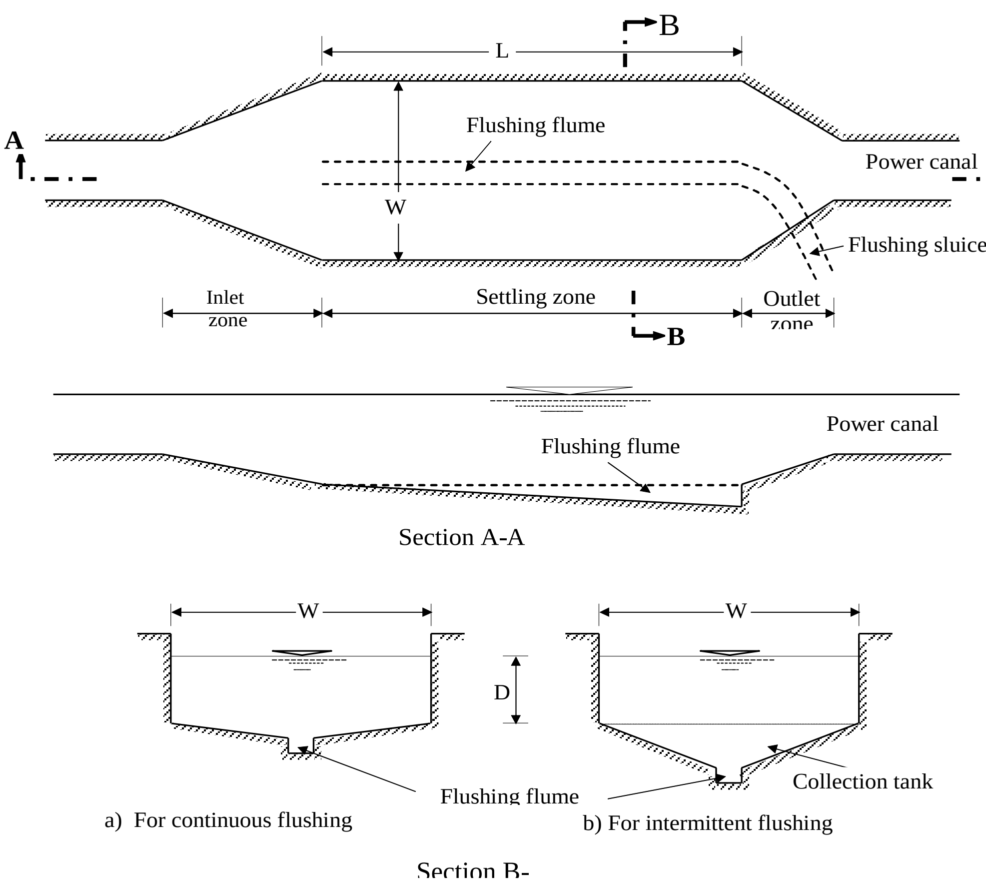


Figure 1. 21: Settling basin

Inlet Zone:

The main function of the inlet is to gradually decrease the turbulence and avoid all secondary currents in the basin. This is achieved by decreasing the flow velocity through gradually increasing the flow cross-section, i.e., by providing gradual expansion of the width and depth.

To achieve optimum hydraulic efficiency and effective use of the settling zone, the inlet needs to distribute the flow uniformly over the cross-section of the basin. To achieve uniform flow distribution, the following techniques, in addition to the provision of gradual expansion, may be adopted at the inlet zone:

- Use of submerged weir
- Use of baffles
- Use of slotted walls

Settling Zone:

This is the main part of the basin where settling of the suspended sediment is supposed to take place. The dimensions of this zone can be determined through calculations.

Outlet Zone:

This is a kind of transition provided following the settling zone to facilitate getting back the flow into the conveyance system with the design velocity by gradually narrowing the width and depth. The outlet transition may be more abrupt than the inlet transition.

Design of Settling Basins

The design principle of settling basin must consider the following points:

1. The settling basin must have length and width dimensions which are large enough to cause settling of the sediments but not so large that the basin is over expensive and bulky.

2. It must allow for easy flushing out of deposits, undertaken at sufficiently frequent intervals.

3. Water removed from the flushing exit must be led carefully away from the installation. This avoids erosion of the soil surrounding and supporting the basin foundations.

4. It must avoid flow turbulence caused by introduction of sharp area changes or bends, and they must avoid flow separation.

5. Sufficient capacity must be allowed for collection of sediment.

- The silt particles begin to collect, fall and the lightest ones will fall at the end of the basin:

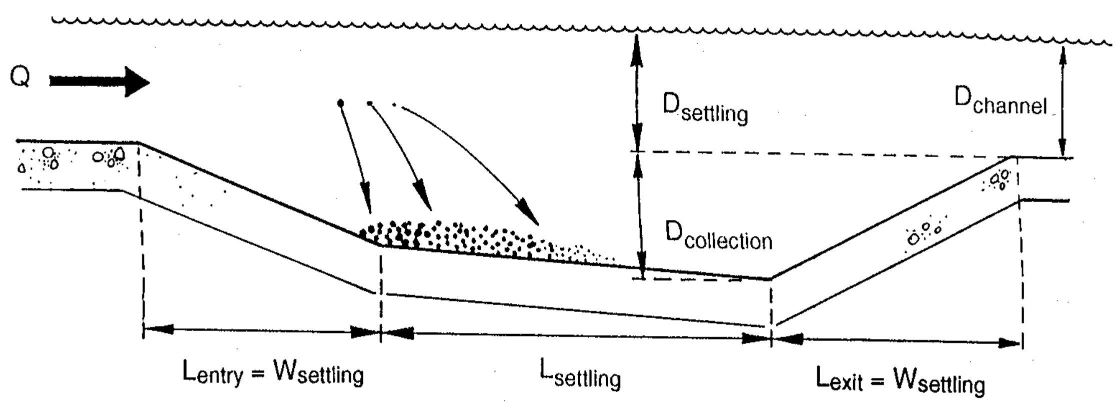


Figure 1. 22

- After a period of time the collection tank will be full. The lightest particles still fall at the end of the basin:

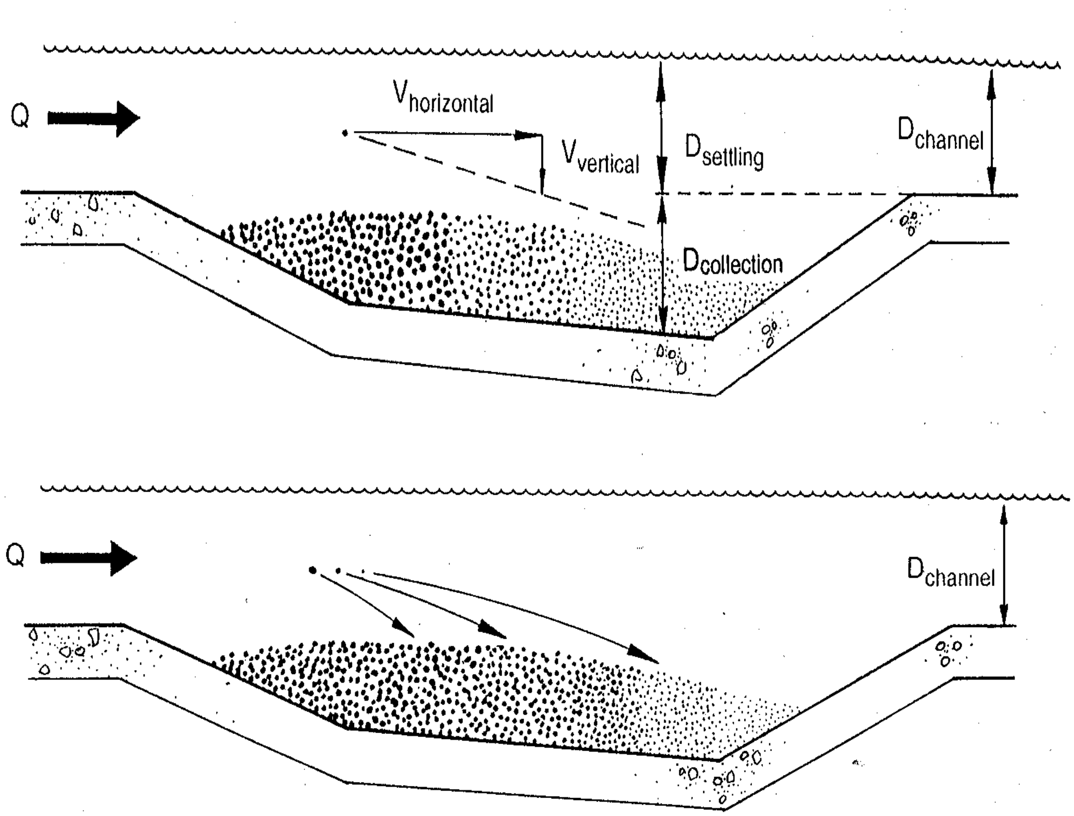


Figure 1. 23

Design neglecting the effect of turbulence (simple settling theory):

Here, the effect of turbulent flow upon settling velocity is neglected. Three basic relations may be written for the determination of the required basin dimension.

Denoting the depth of the basin by *D* and its width by *W*, the discharge passing through the basin is:

$$Q = W\,D\,V$$

Where, *V* is the flow-through velocity.

The second equation expressing the relation between the settling velocity,**ω** the depth of the basin *D*, and the settling time *t* is:

$$t = \frac{D}{\omega}$$

Finally, the length of the basin will be governed by the consideration that water particles entering the basin and sediment particles conveyed by them with equal horizontal velocity should only reach the end of the basin after a period longer than the settling time. Thus, even the smallest settling particle may reach the bottom of the basin within the settling zone. In other words, the retention period should not be shorter than the settling time. The required length of the basin is thus:

$$L = V\,t$$

Eliminating ***t*** from the last two equations two relations can be established between the six parameters governing the hydraulic design:

$$\left.\begin{array}{l} Q = W\,D\,V \\ L = \dfrac{D\,V}{\omega} \end{array}\right\}$$

Obviously a solution of the problem is not possible unless four of the six quantities are known. The discharge *Q* is usually known. The settling velocity ω is defined by the initially specified degree of removal and, as mentioned previously, can be established by calculation or experimentally. The highest permissible flow-through velocity *V* should also be specified, considering that particles once settled should not picked up again. According to *Camp*, the critical flow-through velocity is estimated from:

$$V = a\sqrt{d_l} \qquad [m/s]$$

Where, *d* is the equivalent diameter of the smallest sediment particle to be settled in mm and ***a*** is a constant given as:

$a = 0.36, \quad for\ d_l > 1\ mm$

$a = 0.44, \quad for\ 0.1\ mm < d_l < 1\ mm$

$a = 0.51, \quad for\ d_l < 0.1\ mm$

[Modern tendency is to use V =0.4 to 0.6 m/s]

Depth of basin should be specified considering that long and/or wide basins are economical than deep ones. The depth of settling basins in waterpower projects is generally between 1.5 and 4 m with flow-through velocities not higher than 0.5 m/s. Hence W and L can be computed.

Check

From Q=DWV *gives* V=Q/WD

and from L=V*t *gives* V=L/t

Therefore, Q/WD = L/t *gives* Q*t = WDL (Water conveyed to tank = Volume of tank)

Penstock

The penstock is high pressure pipeline between forebay (surge tank or reservoirs) and the turbine. The design principles of penstocks are the same as that of pressure vessels & tanks but water hammer effect has to be considered. For short length, a separate penstock for each turbine is preferable. For a moderate heads & long distances a single penstock is used to find two or more turbines through a special branching pipe called Manifold.

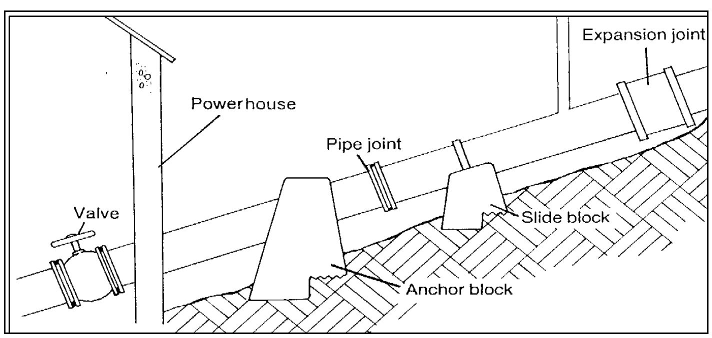


Figure 1. 24

Classification of penstock.

Classification may based on :

1. The material of construction
2. Method of support
3. Rigidity of connection and support.
3. Number of penstocks

1) Material of construction

Factors for the choice of material are: head, topography & discharge. Various materials used are steel, R.C., asbestos cement, PVC, wood stove pipes, banded steel, etc. The following factors have to be considered when deciding which material to use for a particular project:

Required operating pressure, Diameter and friction loss, Weight and ease of installation Accessibility of site, Cost of the penstock. Design life, Availability and Weather conditions

2) Method of support

A penstock may be either buried or embodied underground (or inside dams) or exposed above ground surface & supported on piers.

Buried penstocks: are supported on the soil in a trench at a depth of 1 to 1.5m and back filled. The general topography of the land should be gentle sloping and of loose material.

Table 1. 5

Advantages	*Disadvantages*
1. Continuity of support given by the soil provides better structural storability.	1- Difficulty in inspection
2. Pipe is protected from high temperature fluctuations	2- Possibility of sliding on step slopes

3. Conservation of natural land escape	3- Difficulty in maintenance
4. Protection from slides , storms & sabotage.	4- Expensive for loge diameter in rocky soils.

Exposed penstocks: supported on piers or saddles.

Table 1. 6

Advantages	*Disadvantages*
1- Ease in inspection of defects & maintenance	1- Direct exposure to weather effect
2- Economy in rocky terrain & large diameters.	2- Development of longitudinal stress due to support and anchorage, thus requiring expansion joints
3- Stability is insured with proper anchorage	

When the situation warrants, partly buried system, may be adopted this combines the advantages of both systems.

Hydraulics

Permissible velocities

3 to 5 m/s (no abrasion property settled water) for properly settled water in exceptional cases up to 5m/s may be tolerated.

Therefore: $A = \frac{Q_0}{v_0} = \frac{\pi D^2}{4} \Rightarrow D = 1.128\sqrt{\frac{Q_0}{v_0}}$

Head losses

i) Frictional head loss => $h_f = \lambda \frac{L}{D}\frac{v^2}{2g}$

ii) Other local losses $h_{f_c} = k\frac{v^2}{2g}$ where *k* = local loss coefficient

Table 1. 7

	k
Penstock with gradual transition entrance	0.10 - 0.20
Open butterfly valve (depending on disk thickness)	0.05 - 0.25
Needle valves	0.20 - 0.25
Bends (depending on deflection angle)	0.05 - 0.15

Net head: $H = H_g - \sum$ losses where H = net head, H_g = gross head.

For H_g : elevation of water level at the fore bay or reservoir at the upstream end, and at d/s end – free tail water level in reaction type turbines (Francis) or elevation of jet nozzle in case of impulse/action turbine (Pelton)

Economical Diameter of Penstock

The diameter of the penstock is determined from economic consideration and then checked to see that acceptable velocities are not exceeded

Two approaches - Graphical (economic analysis)
- Empirical equations

i) **Graphical approach:** D – f (capital cost, running cost)

If D is small, large h_f, reduction in output, loss in revenue. If D_1 is large, small h_f, greater output , larger initial cost .

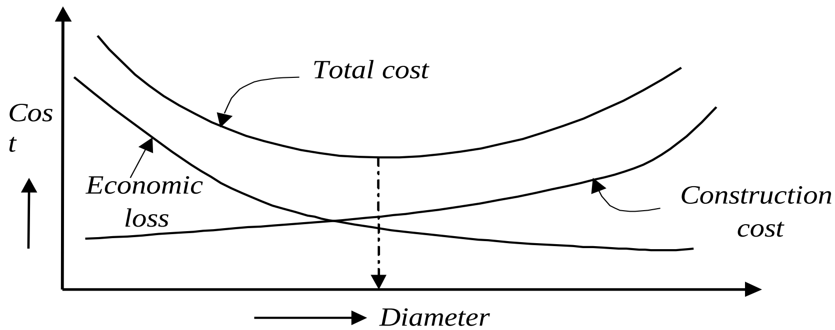


Figure 1. 25: Economical Diameter of Penstock

For preliminary design various **empirical formula** are available

1) *SARKARIA'S Formula*

$$D = \frac{0.62\,P^{\,0.43}}{H^{0.65}}$$

D – penstock diameter (m)

P - hp transmitted by the pipe

H – net head e the end of the penstock is m.

2) *USBR*

$$v = 0.125\sqrt{2gH}$$

v – Optimum velocity

3) *Donald's formula*

$$D = 0.176\left(\frac{P}{H}\right)^{0.466}$$

4) *Fahlbusch (2982)*

$$D = 0.52\,H^{0.17}\left(P/H\right)^{0.43}$$

Structure Analysis of the Penstock

- It is necessary to construct pressure grade line
- In addition to pressure heads, to water hammer pressure have to be determined

From previous

$$C = \sqrt{\frac{E_m}{\rho}} \quad \text{Where} \quad \frac{1}{E_m} = \frac{1}{K} + \frac{D}{TE}$$

For instant closure i.e. $t_c \leq \frac{2L}{C}$

$$h = \frac{Cv_o}{g}$$

For all other closure $t_c > \frac{2L}{C}$, the max pressure rise at the valve

$$h_{\max} = \frac{h}{C} \; \frac{2L}{t_c} = \frac{C\,v_o}{g\,C} \cdot \frac{2L}{t_c} = \frac{2v_o\,L}{g\,t_c}$$

The value of water hammer pressure rise as computed above may be taken at the turbine gate, reducing to zero at intake or surge tank level. Values at intermediate location may be calculated assuming a straight line variation

Thus, design head H = static head plus water hammer head.

For thin walled vessels, where $\frac{D}{t} \geq 20$

$$\sigma = \frac{pD}{2t}$$

The design pressure, p=(H

$$t = \frac{PD}{2\sigma}$$

In the above $\sigma \leq \phi\, \sigma_a$; σ_a = allowable stress

N= is coefficient depending on joint type. For welded joint, (ϕ = 0.85 – 0.95)

For steel used in penstocks usually a factor of safety of 3 to 3.5 is used. Thus for material with ultimate tensile strength of 3700 kg/cm^2; $\sigma_a \leq$ 1200 kg/cm^2

Thus for design purposes, $$t \geq \frac{pD}{2\phi\sigma_a}$$

For protection against coating deterioration add 1 to 3mm to the above value.

For thick welded piper where $D/t <$ 20 , the following formula giver sufficient accuracy

$$t = \frac{D}{2}\left(\sqrt{\frac{\phi\sigma_a + 0.07H}{\phi\sigma_a - 0.13H}} - 1)\right) + (1\ to\ 3)mm$$

The ASME gives the formula for thickness as $$t = \frac{pr}{\sigma_a\phi - 0.6P} + 0.15$$

Where t in cm

p pressure in kg/cm^2

r internal radius in cm

σ_a design stress in kg/cm^2

ϕ joint efficiency factor

0.15cm is allowance for corrosion

In case where the negative water column gradient falls below the penstock center line, there is danger of collapse of the penstock due to external atmospheric pressure. The external pressure p_e likely to result in collapse may be computed from the formula by Mayer

$$p_e = \frac{3EI}{r^3} = \frac{24EI}{D^3}\ kg/cm^2$$

I =moment of inertia of x-section of the pipe ring $\frac{t^3}{12}$ m^3

E =modulus of elasticity of steel

Introducing a S.F. η, $Pe = \frac{1}{\eta}\ 2\left(\frac{t}{D}\right)^3$ n=2 for burried pipes; n=4 for exposed pipes

$t = D\sqrt[3]{\frac{\eta\ Pe}{2E}}$

For example for complete vacuum, $t = D\sqrt[3]{\frac{4\times 1}{2\times 2\times 10^6}} = 0.01D$

Penstock Joints

Penstock pipes are generally supplied in standard lengths, and have to be joined together on site. There are many ways of doing this, and the following factors should be considered when choosing the best jointing system for a particular scheme.

- Relative costs
- Ease of installation
- Suitability for chosen pipe material
- Degree of joint flexibility

Methods of pipe jointing fall roughly into four categories:

- Flanged joints
- Spigot and socket joints
- Mechanical joints
- Welded joints

Flanged Joints:

Flanges are fitted to each end of individual pipes during manufacture, and each flange is then bolted to the next during installation as shown in Figure 1.15.

A gasket or other packing material is necessary between each flange of a pair. Flange jointed pipes are easy to install, but flanges can add to the cost of the pipe. Flange joints do not allow any flexibility. They are generally used to join steel pipes, and occasionally ductile iron pipes.

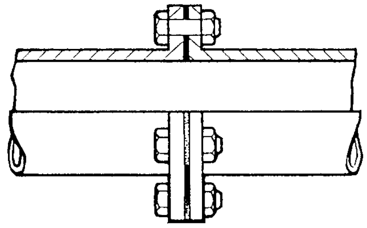


Figure 1. 26 Flanged joints

Spigot and Socket Joints:

Spigot and socket joints are made by either fitting a collar to, or increasing the diameter during manufacture of, one end of each pipe such that the internal diameter of the collar or increased internal diameter of the pipe is the same as the external diameter of the pipe.

A good seal is required between each pipe section, and this is achieved by either providing a rubber seal or special glue called solvent cement, depending up on the material of which the pipes are made.

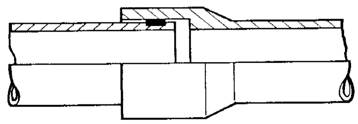


Figure 1. 27 Spigot & socket joints

Spigot and socket joints are generally used to join ductile iron, PVC, concrete and asbestos cement pipes.

Mechanical Joints:

Mechanical joints are rarely used on penstocks because of their cost. One important application of it is for joining pipes of different material or where a slight deflection in the penstock is required that does not warrant installing a bend.

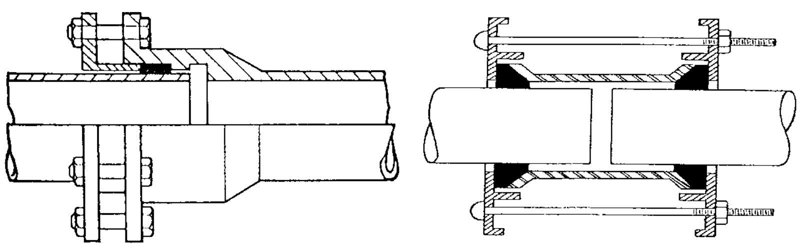


Figure 1. 28 Mechanical joints

Welded Joints:

Welded joints are used on penstocks made of steel. Steel pipes are brought to the site in standard lengths, and then welded together on site. One advantage of welding on site is that changes in the direction of the pipe can be accommodated without preparation of a special bend section. It is relatively cheap method, but has the drawback of needing skilled site personnel.

Expansion Joints:

A penstock, specially exposed ones, will change in length depending on temperature fluctuations. If it is fixed the thermal expansion forces are substantial. It is possible to relieve these forces by incorporating special joints called *expansion joints*, which allow the pipe to expand and contract freely.

For short penstocks, provision of a single expansion joint may be sufficient, but for long penstocks with a multiple anchor blocks expansion joints should be placed below each anchor block. Another alternative to take care of thermal expansion is to take in to account the forces that result from it in designing anchors.

Penstock Supports and Anchors

Slide blocks, anchors, and thrust blocks all serve the same basic function – to constrain movement of the penstock. Different terms are used with these structures simply to indicate which specific function they serve, and this is discussed in the following paragraphs, see Figure 5.32 for the sketches.

Slide Blocks:

A slide block, also called supporting pier, carries the weight of pipe and water, and restrains the pipe from upward and sideway movements, but allows it to move longitudinally. In most cases the spacing between slide blocks are assumed equal to the length of each pipe.

If the penstock is buried, slide blocks are unnecessary; rather instead the pipe is laid in a trench on a bed of sand or gravel of consistent quality, with no big stones which could cut into the pipe or cause stress concentrations on the pipe wall.

Forces that act on slide blocks include:

Weight of the pipe and enclosed water, as slide blocks do not resist longitudinal forces, only the component of the weight perpendicular to the pipe will be considered.

Friction forces on the blocks. This is due to the longitudinal movement of the pipe over the blocks caused by thermal expansion and contraction.

Weight of the block itself

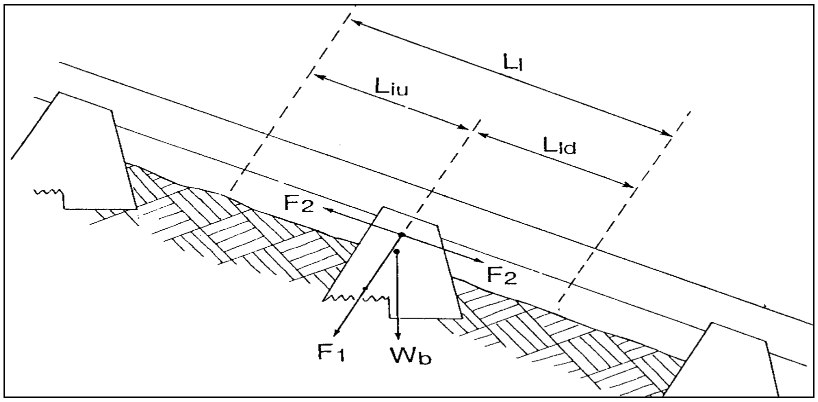


Figure 1. 29 Forces on a slide block

Anchor Blocks:

An anchor block consists of a mass of reinforced concrete keyed to the penstock so that the penstock cannot move in any way relative to the block. It is designed to withstand any load the penstock may exert on it.

Anchors are often used at bends (horizontal and vertical) and before entrance to the powerhouse They can also be used along long straight sections of penstock, each one next to expansion joint.

Because an anchor is keyed to the penstock pipe and is also frequently located at a bend in the pipe, more forces act on an anchor than on a slide block. The major forces which act on anchor blocks are the following:

- Weight of the pipe and enclosed water
- Hydrostatic force on a bend
- Friction forces on slide blocks located between the anchor and expansion joint
- Thermally induced stresses, when expansion joints are not incorporated
- The weight of the anchor block itself

Thrust Blocks:

These are a special form of anchor whose sole purpose is to transmit forces primarily caused by hydrostatic pressures at horizontal bends along a buried penstock to undisturbed soil which provides the reaction force. However, if the bend is vertical, an anchor block is still used if the back filled soil is not able to resist this force.

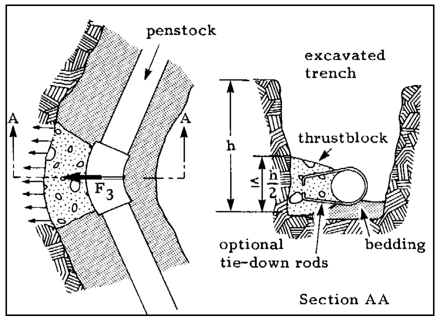


Figure 1. 30 Thrust block

Conditions of Stability for Supports and Anchors:

For any penstock support or anchor to be stable and fulfill its intended purpose, the following conditions must be met:

The structure should be safe against *sliding*. For sliding not to occur:

$$\sum H < \mu \sum V$$

Where, $\sum H$ and $\sum V$, respectively, are the sum of all horizontal and vertical forces, and μ is the coefficient of friction between the structure and the ground often assumed as 0.5.

The structure should be safe against *overturning*. For this condition to be fulfilled, the resultant force should act within the middle third of the base. In other words,

$$e < \frac{L_{base}}{6}$$

Where, e is eccentricity of loading and L_{base} is length of the structure base.

The pressure transmitted to the foundation must be within the safe bearing capacity of the foundation material. This can be expressed as:

$$\left\{\begin{matrix} Maximum\ pressure \\ by\ the\ structure \end{matrix}\right\} = \frac{\sum V}{A_{base}}\left(1+\frac{6e}{L_{base}}\right) < \left\{\begin{matrix} Bearing\ capacity\ of \\ the\ foundation\ soil \end{matrix}\right\}$$

Penstock Valves

Valves are usually installed at two places in a penstock. One valve is provided at the upstream end of the penstock, i.e., at the forebay or immediately after the surge tank, and is called *penstock inlet valve*, while the second is provided at the downstream end of the conduit, immediately ahead of the turbine, and is named as *turbine inlet valve*. The upper valve is sometimes replaced by a gate.

The main purpose of penstock inlet valve is for dewatering of the penstock in case maintenance of the penstock is required. But, it can be omitted for short penstocks where the closure of the power canal or power tunnel is possible from the intake.

The main purpose of turbine inlet valve is to close the penstock while the turbine is inoperative. It can also act as an emergency shut-off device. This valve cannot be omitted except under especial case where the penstock supplies a single unit having installed the penstock inlet valve. The number of turbine inlet valves required at a power station is governed by the number of turbine units installed, but not by the number of penstocks, as a single penstock can serve a number of units through a manifold at the end. There are varies types of valves for use in hydropower installations. The most frequently applied include:

- Gate valves
- Butterfly valves
- Spherical valves
- Needle valves

The type to be applied should be determined individually for each case after considering the various factors involved. For details on valves, reference can be made to hydropower and hydraulic structures textbooks.