HYDROPOWER ENGINEERING II

LECTURE NOTE

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1. Hydraulic Turbines

1.1 General

Hydraulic turbines may be considered as hydraulic motors or prime movers of a water power development, which convert water energy (hydropower) in to mechanical energy (shaft power). The shaft power developed is used in running electricity generators directly coupled to the shaft of the turbine, thus producing electrical power.

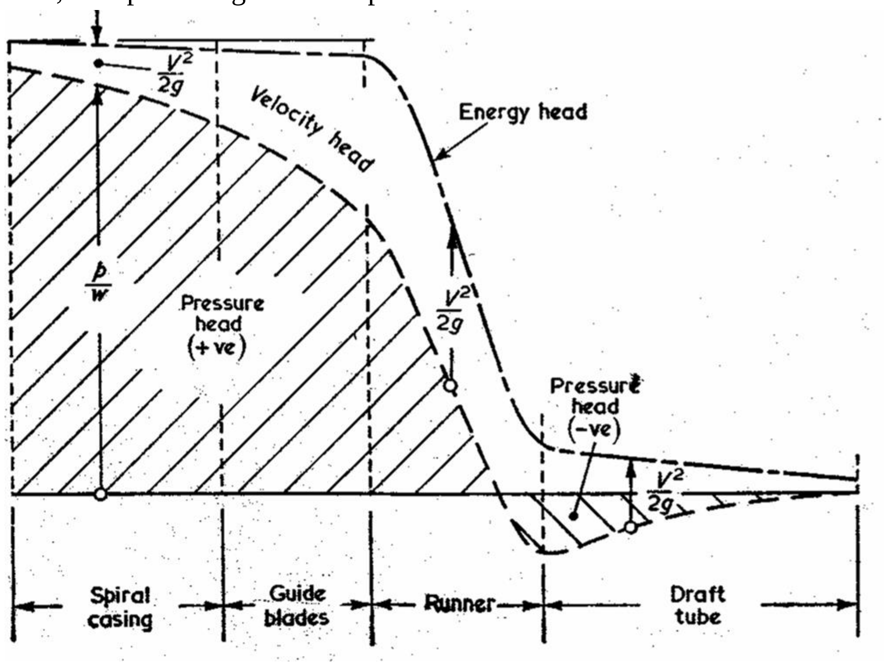


Figure 1.1: Energy head variation in turbine passages

1.2 Classification

All types of turbines basically fall in to two categories impulse and reaction turbines.

Impulse turbine: All the available potential energy is converted in to kinetic energy with the help of contracting nozzle/s. The water after impinging on the curved vanes or bucket is discharged freely to the downstream channel (eg. Pelton wheel)

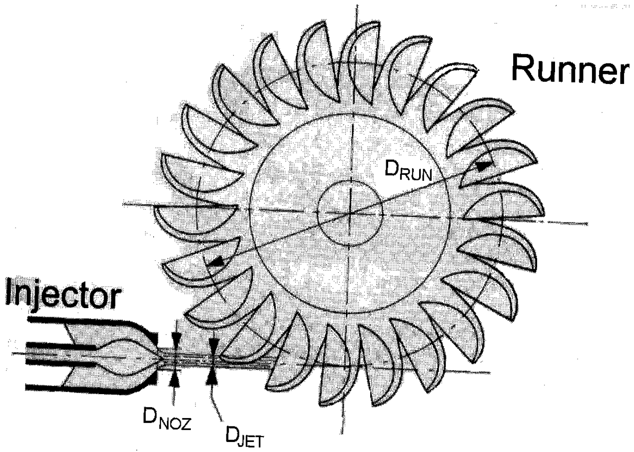


Figure 1.2: Example of Pelton turbine arrangement (single nozzle)

Reaction turbines: In this type the water enters the turbine in a circumferential direction in to the scroll case and moves into the runner through a series of guide vanes, called wicket gates. The available energy partly converted to kinetic energy & substantial magnitude remains in the form of pressure energy (e.g. Francis, Kaplan, Propeller, Bulb, etc)

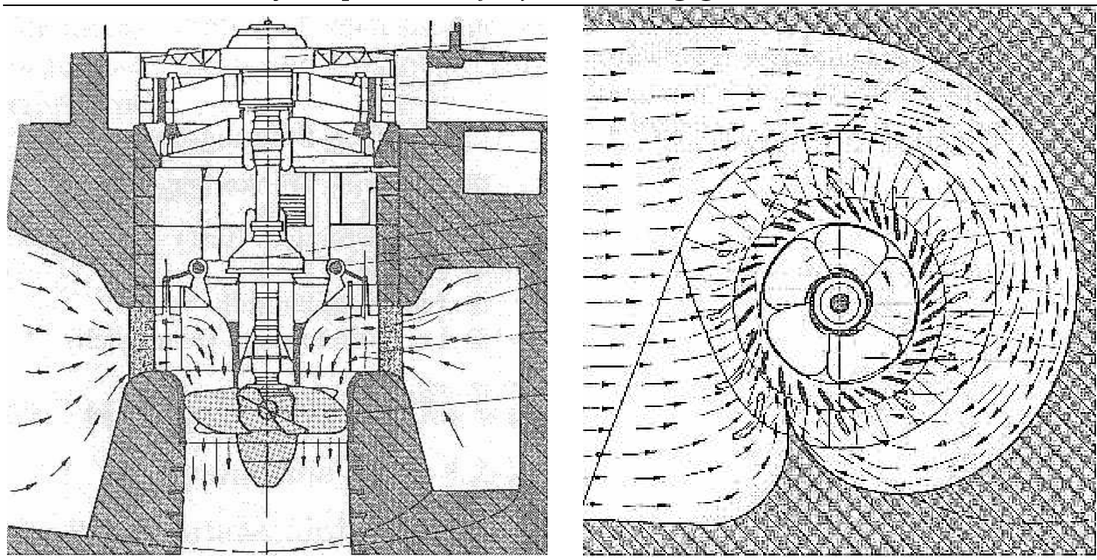


Figure 1.3: Representation of flow pattern in Kaplan turbine

Turbines may also be classified according to the main direction of flow of water in the runner as

- Tangential flow turbine (Pelton wheel)
- Radial flow turbine (Francis)
- Mixed flow turbine (modern Francis)
- Axial flow turbine of fixed blade (Propeller) or movable blade (Kaplan or bulb) type.

Furthermore, turbines may be classified based on head, discharge, speed, specific speed.

1.3 Characteristics of Turbines

Specific speed: is useful parameter for the selection of turbine for a given condition: It is defined as the speed at which a geometrically similar runner would rotate if it were so proportioned that it would develop 1 Kw when operating under a head of 1m , and expressed as (from dimensional analysis)

$$N_s = N\frac{\sqrt{P}}{H^{5/4}}$$

Where N_s = Specific speed
N = rotational speed (rpm)
P = Power developed (kw)
H = effective head (m)

Turbine or synchronous speed: Since turbine & generator are fixed, the rated speed of the turbine is the same as synchronous speed of the generator. The speed N, for synchronous running is given by:

$$N = 120\, f/p$$

Where *f* = frequency cycle/sec (50-60 cycles/sec.)
p = number of poles (divisible by 4 for head up to 200 m)
(Divisible by 2 for head above 200 m)

The speed of a turbine is an important parameter of design. The higher the speed, the smaller the diameter of the turbine runner & the cheaper the generator coupled to the turbine. High speed, however, makes a turbine more susceptible to cavitation.

Speed factor or peripheral coefficient, ϕ: he ratio of the peripheral speed, ***u***, of the bucket or vanes at the nominal diameter, D, to the theoretical velocity of water under the effective head, ***H***, acting on the turbine is called the speed factor or peripheral coefficient, ϕ

$$\phi = \frac{u}{\sqrt{2gH}} = \frac{\omega\, r}{\sqrt{2gH}}$$

But $\boldsymbol{\omega}$ in rad/sec; $\omega = \frac{2\pi N}{60}$ *and* $r = D/2$

Therefore, $\phi = \frac{\pi DN}{60\sqrt{2gH}} = \frac{DN}{84.6\sqrt{H}}$ Where, D and H in m; N in rpm

The following table suggests appropriate values of ϕ, which give the highest efficiencies for any turbine, the head & specific speed ranges & the efficiencies of the three main types of turbine.

Type of runner	ϕ	Ns	H (m)	Efficiency (%)
Impulse	0.43 – 0.48	8-17 17 17-30	>250	85-90 90 90-82
Francis	0.6 – 0.9	40-130 130-350 350-452	25-450	90-94 94 94-93
Propeller	1.4-2.0	380-600 600-902	<60	94 94-85

Thus in general:

- Pelton turbines are used for high heads & low discharges
- Francis types are used for medium & high head plants (has adjustable guide vanes but the runner is a disc with fixed passage)
- Propeller & Kaplan (Kaplan has adjustable blades) types are used for low head plants with large discharges.

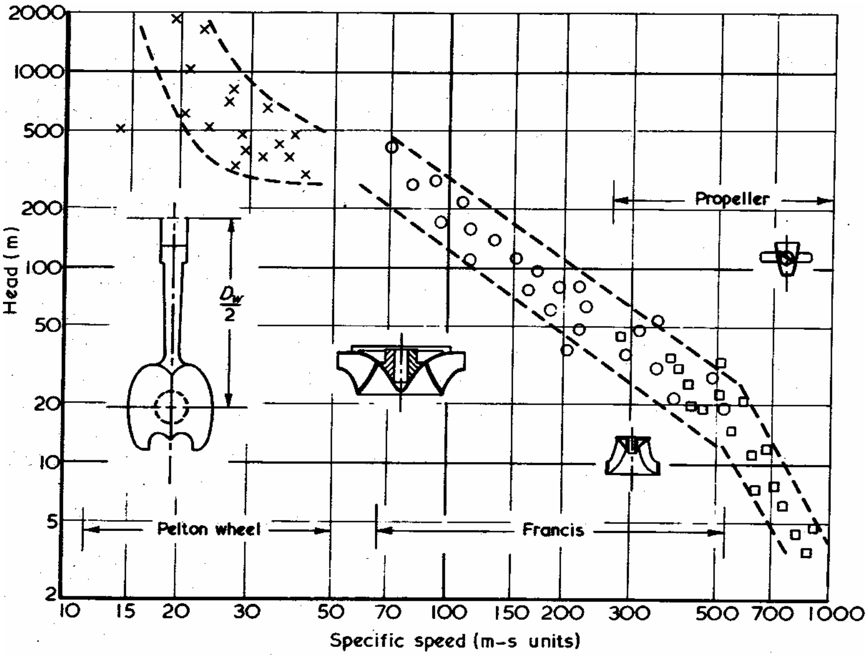


Figure 1.4: Application of turbine based on head and specific speed

Runaway Speed: If the external load on the machine suddenly drops to zero (sudden rejection) and the governing mechanism fails at the same time, the turbine will tend to race up to the maximum possible speed, known as runaway speed. This limiting speed under no-load, maximum-flow must be considered for safe design.

Type of runner	Runaway speed (% of normal speed)	Acceptable head variation (% of design head)	
		Minimum	Maximum
Impulse (Pelton)	170 - 190	65	125
Francis	200 - 220	50	150
Propeller	250 - 300	50	150

1.4 Procedure in Preliminary Selection of Turbines

1. From design Q and H, calculate approximate P that can be generated , $P = \eta \gamma Q H$
2. From $N = 120\dfrac{f}{p}$ calculate N (or assume) & compute N_s. From this, the type of turbine can be suggested

3. Calculate D from: $\phi = \dfrac{DN}{84.6\sqrt{H}}$

If D is found to be too large, either N can be increased or more units may be adopted. For approximate calculations of runner diameter; the following empirical formula may be used (Mosony)

$D = a\left(\dfrac{Q}{N}\right)^{1/3}$ Where, D in m, Q in m^3/s, N in rpm

a = 4.4 for Francis & **Kaplan;** a = 4.57 for Kaplan.

Or $D = \dfrac{7.1\sqrt{Q}}{(N_s + 100)^{1/3} H^{1/4}}$ for Propeller, H in m,

Nominal diameter, D, of Pelton wheel and d_j is diameter of the jet:

$$D = 38\sqrt{\frac{H}{N}}$$

$$d_j = 0.542\sqrt{\frac{Q}{H}}$$

Jet ratio given by $m = \dfrac{D}{d_j}$, is important parameter in design of Pelton wheels.

Number of buckets, $n_b = 0.5\,m + 15$ (good for $6 < m < 35$)

It is not uncommon to use a number of multiple jet wheels mounted on the same shaft so as to develop the required power.

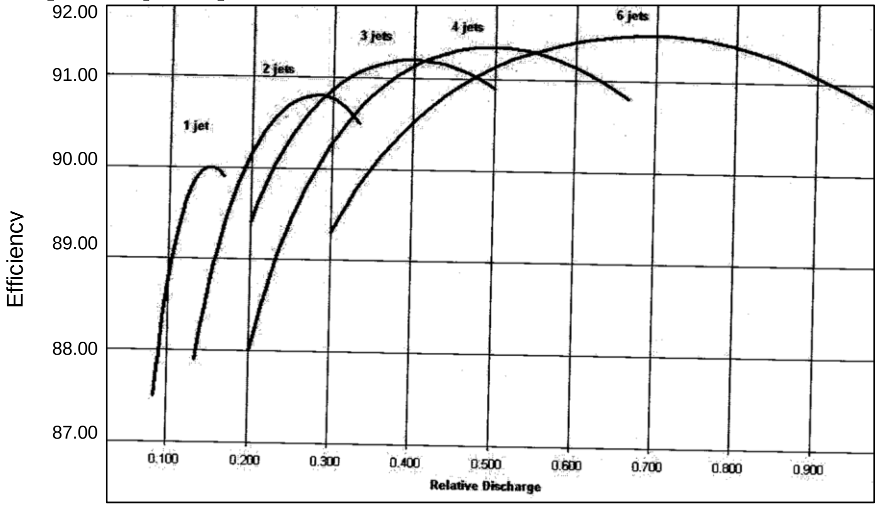


Figure 1.5: Efficiency of a 6-jet Pelton turbine with automatic selection of number of active jets

Hydraulic turbines (runner) are designed for optimum speed & maximum efficiency at design head. But in reality, head and load conditions change during operation & it is extremely important to know the performance of the unit at other heads. This is furnished by manufacturer's curve.

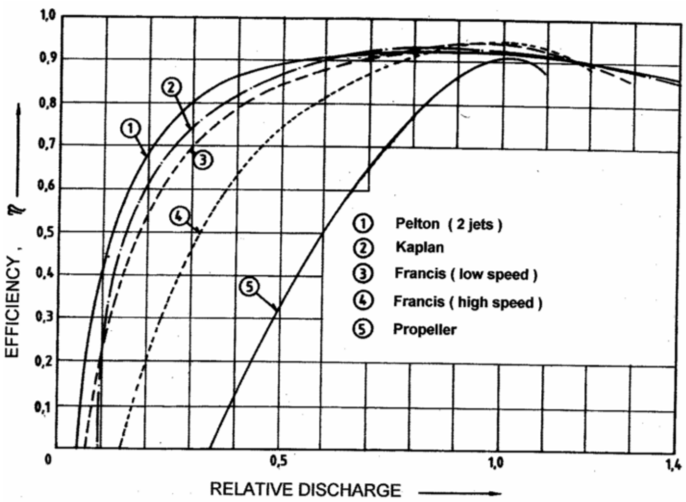
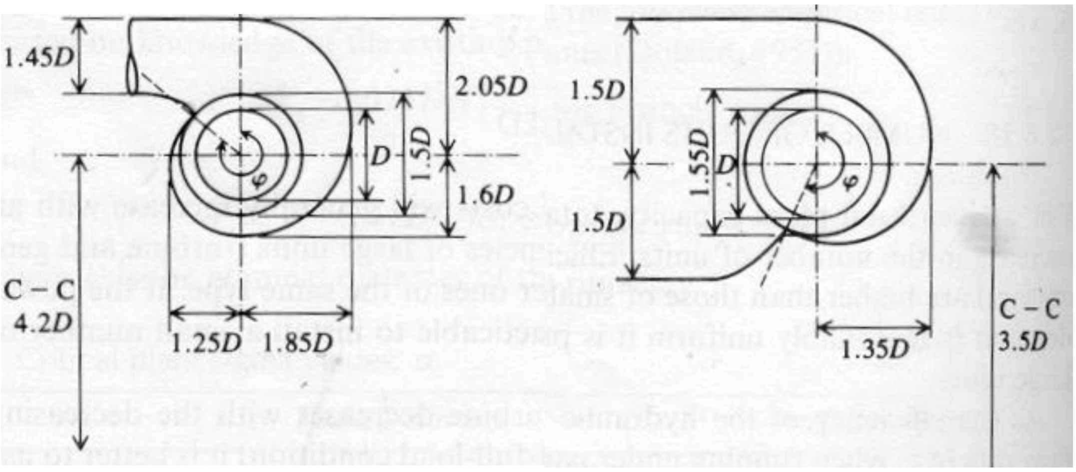


Figure 1.6: Turbine efficiency versus relative discharge for different turbine type

1.5 Turbine Scroll Case

A scroll case is the conduit directing the water from the intake or penstock to the runner in reaction type turbine installation (in case of impulse wheels a casing is usually provided only to prevent splashing of water & lead water to the tail race). A spiral shaped scroll case of the correct geometry ensures even distribution of water around the periphery of the runner with the minimum possible eddy formations.



a) Francis turbine with steel spiral case b) Propeller turbine with partial spiral

Figure 1.7: Recommended dimensions of scroll casings

These kinds of spiral case will generally used in medium and high head installations where discharge requirement is low.

The design of the shape of the spiral case is governed by the flow requirements. Initial investigation should be based on the following assumptions:

a) spiral case of constant height
b) an evenly distributed flow in to the turbine
c) no friction losses

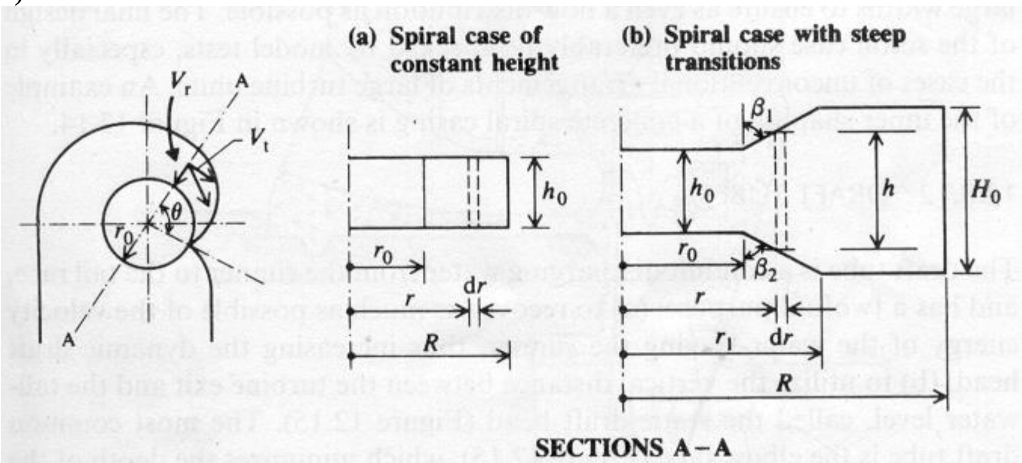


Figure 1.8: Typical cross-sections of spiral case

Referring to Fig 1.8 (a) the discharge in section of spiral case defined by angle θ is

$q = \frac{Q\theta}{2\pi}$ Where Q is the total discharge to the runner

$v_t = \frac{k}{r}$ Where $k = 30\frac{\eta g H}{N\pi}$ (from the basic Euler equation for the power absorbed by the machine)

And the discharge through the strip ***dq*** is given by

$$dq = v_t\ h_0\ dr = \frac{k}{r} h_o\ dr \quad \therefore \quad q = \int_{r_0}^{R} k\ h_o \frac{dr}{r} = \frac{Q\theta}{2\pi} \quad or\ \ln\frac{R}{ro} = \frac{Q\theta}{2\pi\ k\ h_o}$$

This shows for given vortex strength, k, a definite relationship exist between Q & R.

The most economical design of a power station substructure and the narrowest spiral case can be obtained by choosing a rectangular section adjoining the guide vanes (entrance ring) by step transition (symmetrical or asymmetrical)

$h = h_0 + \alpha(r - r_0)$ h where $\alpha = \cot\beta_1 + \cot\beta_2$

$$\frac{Q\theta}{2\pi k} = \int_0^{r_1} h\frac{dr}{r} + \int_{r_1}^{R} H_0\frac{dr}{r}$$

Replacing and integrating

$$\frac{Q\theta}{2\pi k} = h_0 - \alpha\, r_0 \ln\left(\frac{r_1}{r_0}\right) + H_0 - h_0 + H_0 \ln\left(\frac{R}{r_1}\right)$$

Knowing r_1 from $r_1 = \left(\frac{H_0 - h_0}{\alpha}\right) + r_0$, the value of R defining the shape of the spiral case can be determined. The height H_0 at any angle θ may be assumed to be linearly increasing from h_0 at the nose towards the entrance. Shape at various θ is determined by assuming existence of uniform velocity equal to entrance velocity, $v_0 \cong 0.2\sqrt{2gH}$ *and* $q_i = \frac{Q\theta_i}{2\pi}$

$A_i = \frac{q_i}{v_0} = 0.18\frac{Q\theta_i}{\sqrt{H}}$ area of cross-section at angle θ_i

1.6 Draft Tubes

A draft tube is a conduit discharging water from the turbine runner to the tailrace. It is employed in conjunction with reaction type turbines, and has twofold purposes:

- To recover as much as possible of the velocity energy of the water leaving the runner, which otherwise would have gone to waste as an exit loss, thus increasing the dynamic draft head.
- To utilize the vertical distance between the turbine exit and the tail-water level, called the static draft head. In other words, to allow the turbine to be set at higher elevation without losing the advantage of elevation difference.

The most common is elbow type which minimizes the depth of substructure compared to vertical cone; it also has a desirable effect in directing the flow in the direction of the tail water.

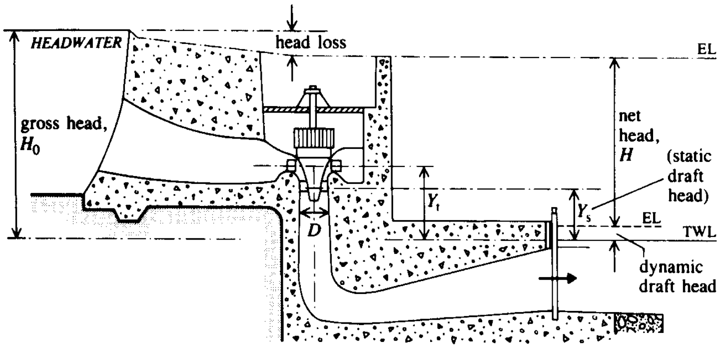


Figure 1.9: Elbow-type draft tube

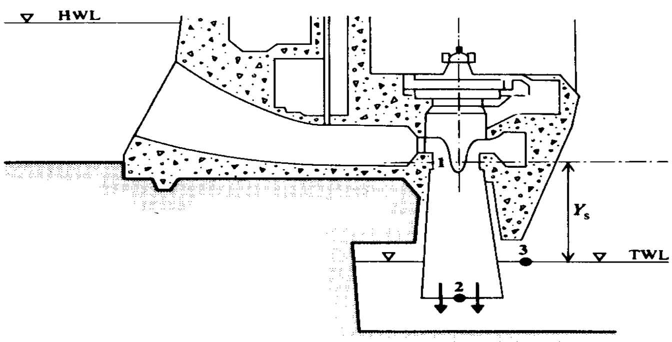


Figure 1.10: Straight conical draft tube

The straight conical draft tubes are the simplest in design and the most efficient type, but they are rarely used in actual practice. This is because, for effective recovery of velocity head, the outlet section has to be many times the inlet section of the draft tube. For smooth eddy-free flow (flow with no separation), the angle of flare of the tube has to be limited to 4 to 8 degrees. Hence, a considerable long tube is necessary to achieve the desired result. This increases the depth of excavation of the substructure, making it uneconomical and unsuitable from cavitation view point.

The elbow-type draft tube is often adopted, because of the following advantages it offers over the conical type:

- Minimizes the required depth of excavation
- Directs the flow in the direction of the tail-water flow
- Allows the provision of gate at the outlet of the tube which can facilitate the de-watering of the turbine for repairs, if necessary.

However from constructional point of view, the elbow draft tube presents more problems. Further more, the change of shape in the elbow naturally increases the turbulent losses in the draft tube.

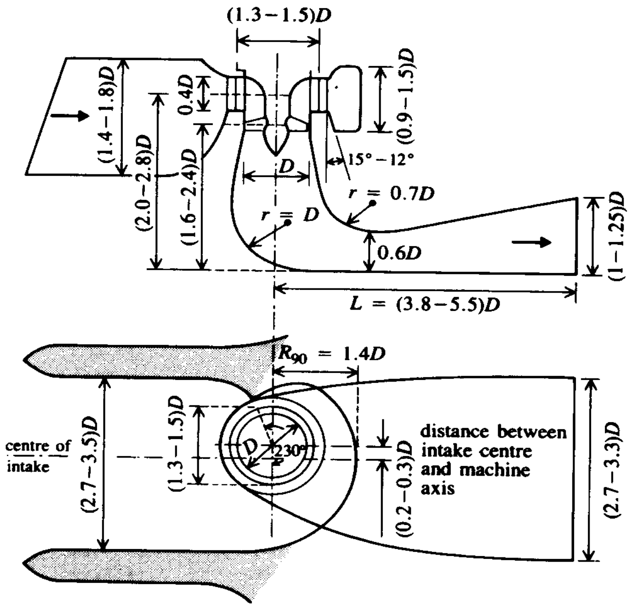


Figure 1.11: Recommended dimensions of an elbow-type draft tube *(after Mosonyi)*

Elbow type divided in to 3 – parts: vertical, bend, and almost horizontal sections. Between 1 & 3 in Figure 1.10, $Y_s + \frac{P_1}{\gamma} + \frac{v_1^2}{2g} = \frac{P_a}{\gamma} + \frac{v_2^2}{2g} + H_L$

Therefore, $\frac{P_1}{\gamma} = \frac{P_a}{\gamma} - Y_s - \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g} - H_l \right)$

$$H_d = \eta_d \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g} \right)$$

Where η_d = efficiency of the draft tube

In order to avoid cavitation at the exit from the runner the condition $\frac{P_1}{\gamma} > \frac{P_v}{\gamma}$

1.7 Cavitation in Turbine & Turbine Setting

Cavitation results pitting, vibration & reduction in efficiency & is certainly undesirable. Cavitation may be avoided by suitably designing, installing, and operating the turbine in such a way that the pressures with in the units are above the vapor pressure of water.

Referring Figures 1.10, Y_s is the most critical factor in the installation of reaction turbines.

$$\sigma = \left(\frac{H_a - H_v \ - \ Y_s}{H} \right)$$

σ = Cavitation coefficient or Plant Sigma

$H_a - H_v = H_b$ = barometric pressure (10.1 at see level)

H = effective head.

$\mathbf{Y_{s,max} = H_b - \sigma_c H}$ (Thoma's formula, bottom of turbine setting)

If Y_s is negative runners must be below TWL. Where σ_c is the minimum (critical) value of Φ at which cavitation occur.

	Francis runners					Propeller runners	
N_s	75	150	225	300	375	600	750
σ_c	0.025	0.10	0.23	0.40	0.64	0.8	1.5

The above may be approximated by

$$\sigma_c = 0.0432\left(\frac{N_s}{100}\right)^2$$ For Francis

$$\sigma_c = 0.28 + 0.0024\left(\frac{N_s}{100}\right)^3$$ For propeller

The preliminary calculation for the elevation of the distributor above the TWL, Y_t is

$Y_t = Y_s + 0.025DN_s^{0.34}$ For Francis

$Y_t = Y_s + 0.025D$ For propeller

Where D is the nominal diameter of the runner

1.8 Generators and Turbine Controls

Generators transform mechanical energy into electrical energy. Although most early hydroelectric systems were of the direct current variety to match early commercial electrical systems, nowadays only three-phase alternating current generators are used in normal practice. Depending on the characteristics of the network supplied, the producer can choose between.

Synchronous generators equipped with a DC excitation system (rotating or static) associated with a voltage regulator, to provide voltage, frequency and phase angle control before the generator is connected to the grid and supply the reactive energy required by the power system when the generator is tied into the grid. Synchronous generators can run isolated from the grid and produce power since excitation is not grid-dependent

Asynchronous generators are simple squirrel-cage induction motors with no possibility of voltage regulation and running at a speed directly related to system frequency. They draw their excitation current from the grid, absorbing reactive energy by their own magnetism. Adding a bank of capacitors can compensate for the absorbed reactive energy. They cannot generate when disconnected from the grid because are incapable of providing their own excitation current.

Synchronous generators are more expensive than asynchronous generators and are used in power systems where the output of the generator represents a substantial proportion of the power system load. Asynchronous generators are cheaper and are used in large grids where their output is an insignificant proportion of the power system load. Their efficiency is 2 to 4 per cent lower than the efficiency of synchronous generators over the entire operating range. In general, when the power exceeds 5000 kVA a synchronous generator is installed.

Recently, variable-speed constant-frequency systems (VSG), in which turbine speed is permitted to fluctuate widely, while the voltage and frequency are kept constant and undistorted, have entered the market. This system can even ''synchronize'' the unit to the grid before it starts rotating. The key to the system is the use of a series resonant converter in conjunction with a double feed machine. Unfortunately its cost price is still rather high and the maximum available power too low.

The working voltage of the generator varies with its power. The standard generation voltages are 380 V or 430 V up to 1400 kVA and at 6000/6600 for bigger installed power. Generation at 380 V or 430

V allows the use of standard distributor transformers as outlet transformers and the use of the generated current to feed into the plant power system. Generating at medium voltage requires an independent transformer MT/LT to supply the plant services.

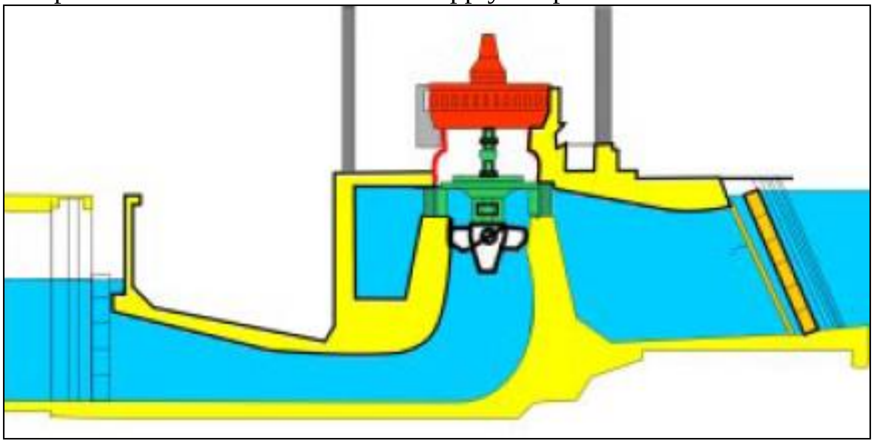


Figure 1.11: Generator Set up

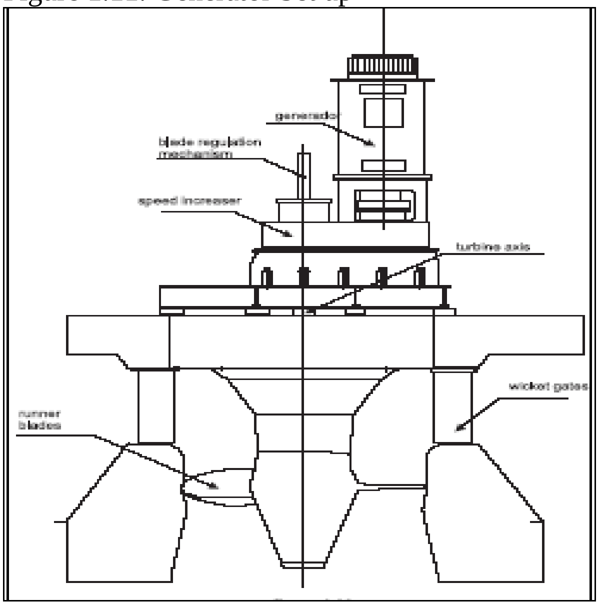


Figure 1.12: Generator Installation

Turbines are designed for a certain net head and discharge. Any deviation from these parameters must be compensated for, by opening or closing control devices such as the wicket-vanes or gate valves to keep constant, either the outlet power, the level of the water surface in the intake or the turbine discharge. In schemes connected to an isolated net, the parameter to be controlled is the runner speed, which controls the frequency. The generator becomes overloaded and the turbine slows-down. In this case there are basically two approaches to control the runner speed: either by controlling the water flow to the turbine or by keeping the water flow constant and adjusting the electric load by an electric ballast load connected to the generator terminals.

In the first approach, speed (frequency) regulation is normally accomplished through flow control; once a gate opening is calculated, the actuator gives the necessary instruction to the servomotor, which results in an extension or retraction of the servo's rod. To ensure that the rod actually reaches the calculated position, feedback is provided to the electronic actuator. These devices are called speed governors.

In the second approach it is assumed that, at full load, constant head and flow, the turbine will operate at design speed, so maintaining full load from the generator; this will run at a constant speed. If the load decreases the turbine will tend to increase its speed. An electronic sensor, measuring the frequency, detects the deviation and a reliable and inexpensive electronic load governor, switches on preset resistances and so maintains the system frequency accurately.

The controllers that follow the first approach do not have any power limit. The Electronic Load Governors, working according to the second approach rarely exceeds 100 kW capacities.

1.9 Turbine Control

Governors

A governor is a combination of devices and mechanisms, which detect speed deviation and convert it into a change in servomotor position. A speed-sensing and amplified to excite an actuator, hydraulic or electric, that controls the water flow to the turbine. In a Francis turbine, where to reduce the water flow you need to rotate the wicket-gates a powerful governor is required to overcome the hydraulic and frictional forces and to maintain the wicket-gates in a partially closed position or to close them completely.

Several types of governors are available varying from purely mechanical to mechanical-hydraulic to electro-hydraulic. The purely mechanical governor is used with fairly small turbines, because its control valve is easy to operate and does not require a big effort. These governors use a fly ball mass mechanism driven by the turbine shaft. The output from this device .the fly ball axis descends or ascends according to the turbine speed- directly drive the valve located at the entrance to the turbine.

The most commonly-used type is the oil-pressure governor that also uses a fly ball mechanism lighter and more precise than that used in a purely mechanical governor. When the turbine is overloaded, the fly balls slowdown, the balls drop, and the sleeve of the pilot valve rise to open access to the upper chamber of the servomotor. The oil under pressure enters the upper chamber of the servomotor to rotate the wicket-gates mechanism and increase the flow, and consequently the rotational speed and the frequency.

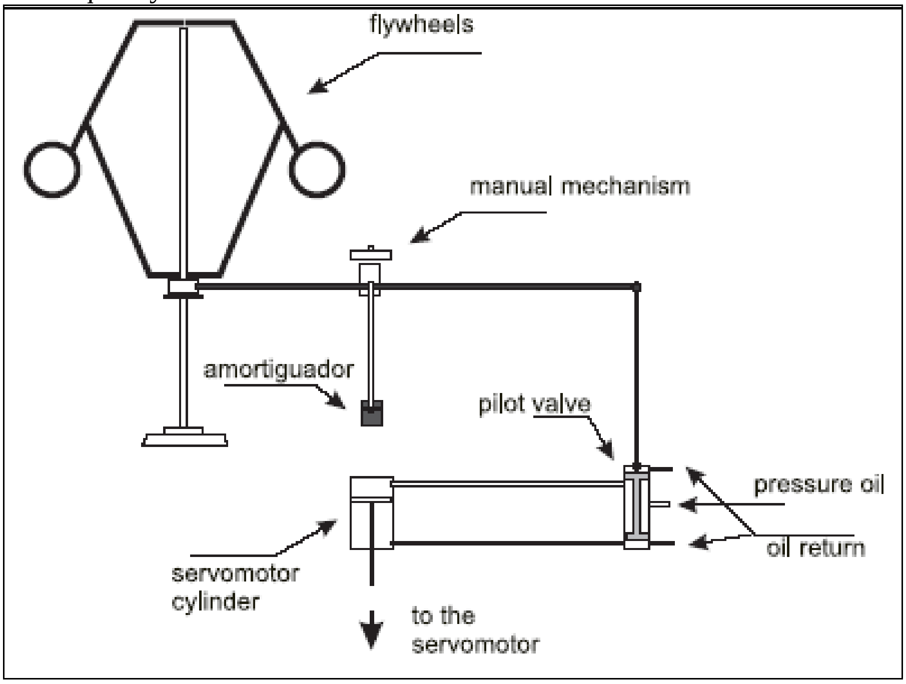


Figure 1.13: Oil-pressure governor

In an electro-hydraulic governor a sensor located on the generator shaft continuously senses the turbine speed. The input is fed into a summing junction, where it is compared to a speed reference. If the speed sensor signal differs from the reference signal, it emits an error signal (positive or negative) that, once amplified, is sent to the servomotor so this can act in the required sense. In general the actuator is powered by a hydraulic power unit consisting of a sump for oil storage, an electric motor operated pump to supply high pressure oil to the system, an accumulator where the oil under pressure is stored, oil control valves and a hydraulic cylinder. All these regulation systems, as have been described, operate by continuously adjusting back and forth the wicket-gates position. To provide quick and stable adjustment of the wicket-gates, and/or of the runner blades, with the least amount of over or under speed deviations during system changes a further device is needed. In oil pressure governors this is achieved by interposing a ''dash pot'' that delays the opening of the pilot valve. In electro-hydraulic governors the degree of sophistication is much greater, so that the adjustment can be proportional, integral and derivative (PID) giving a minimum variation in the controlling process.

An asynchronous generator connected to a large net, from which it takes its reactive power to generate its own magnetism, does not need any controller, because its frequency is controlled by the mains. Notwithstanding this, when the generator is disconnected from the mains the turbine accelerates up to runaway speed with inherent danger for the generator and the speed increaser, if one is used. In such a case it is necessary to interrupt the water flow, rapidly enough to prevent the turbine accelerating, but at the same time minimizing any water hammer effect in the penstock.

To ensure the control of the turbine speed by regulating the water flow, certain inertia of the rotating components is required. Additional inertia can be provided by a flywheel on the turbine or generator shaft. When the main switch disconnects the generator the power excess accelerates the flywheel; later, when the switch reconnects the load, the deceleration of this inertia flywheel supplies additional power that helps to minimize speed variation.

2. HYDROPOWER PROJECT INVESTIGATION AND PLANNING

2.1 Investigation of Resources

- Basic Investigations

Hydropower investigations are often carried out as part of an inventory of water resources, to identify, register and catalogue the hydropower resources existing in river basins, areas and districts. They may also be carried out countrywide to prepare inventories of the complete hydropower potential in a nation.

The main purpose of such investigation is to register the available resources and to determine size and other qualities. Investigated projects are often ranked according to size, costs, priority, etc.

Having no established development purpose such investigations are often termed as *"basic investigations"*

- Purpose oriented Investigations

Other hydropower investigations are carried out for specific purposes, i.e. in order to meet identified needs for electric power through finding suitable supply. Such investigations are purpose oriented in as much as their objective is, among *available hydropower resources, to identify and select the best projects for the stated purposes.*

Purpose oriented investigations have *specific terms of reference to meet.* They are far more comprehensive than basic investigations and are organized accordingly.

It should be noted that purpose oriented investigations will benefit greatly when basic investigations have been carried out in the area of interest as valuable data and information will be available from the start.

2.2 Investigation of Hydropower Projects

A proposed hydropower project which supposed to meet for an established demand for electric power and energy *must be adapted to the physical conditions at hand.* Precise and reliable knowledge about the m*arket situation, socio-economic trends and development plans* are needed in order to make predictions about the future need for electricity and to establish a demand (or load) forecast. In this connection not only the size of the demand needs to be known but also the type of load, peaking needs, etc.

Other requirements:

- value of future electricity supply to determine the benefits side of proposed hydropower project
- long term data on hydrology and meteorology to determine river discharges, floods
- topographic data
- detail knowledge of the geomorphology and geology of the project area and sites needed in connection with planning of project layout and design of the structures and other facilities
- necessary expertise and skills and wide and varied experience from hydropower investigations

2.2.1 Planning Parameters and Data

Several planning parameters and comprehensive data and information are needed for investigation of hydropower resources and planning of hydropower projects. The main data are derived from:

Forecast of demand for electricity, and from studies of:

- hydrology
- topography
- geology, soils and materials

Important issues, indirectly part of the planning process, are:

- environmental constraints
- socio-economic considerations
- electricity tariffs, and tariff policy

These issues influence project planning and project formulation and also contribute to project costs

2.2.2 Power Market

Demand:

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, 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.

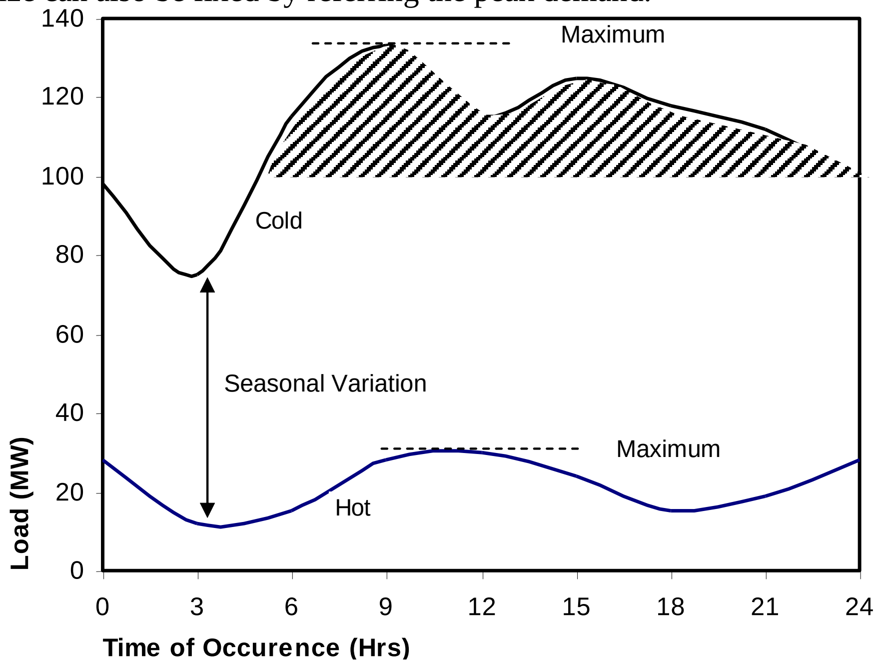


Figure 2.1 Typical 24 Hours Load Curve

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 run off (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.

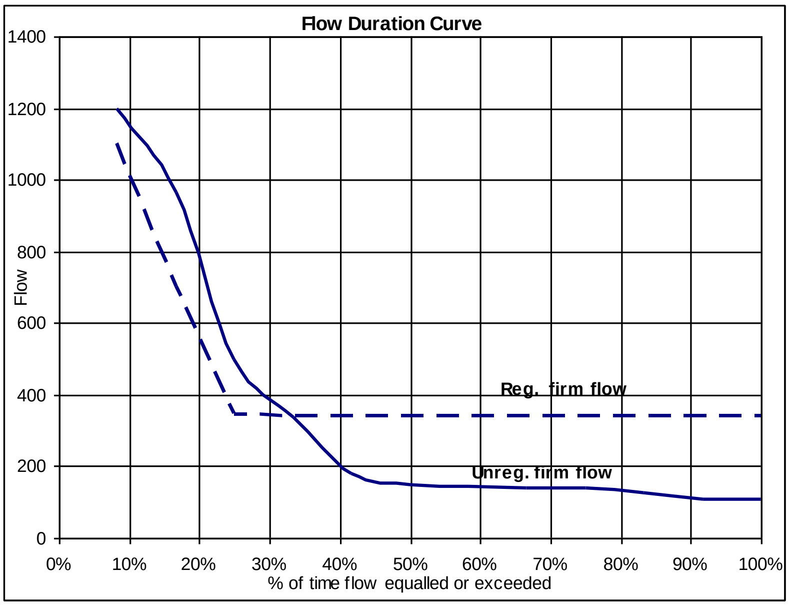


Figure 2.2: Unregulated and Regulated firm flow

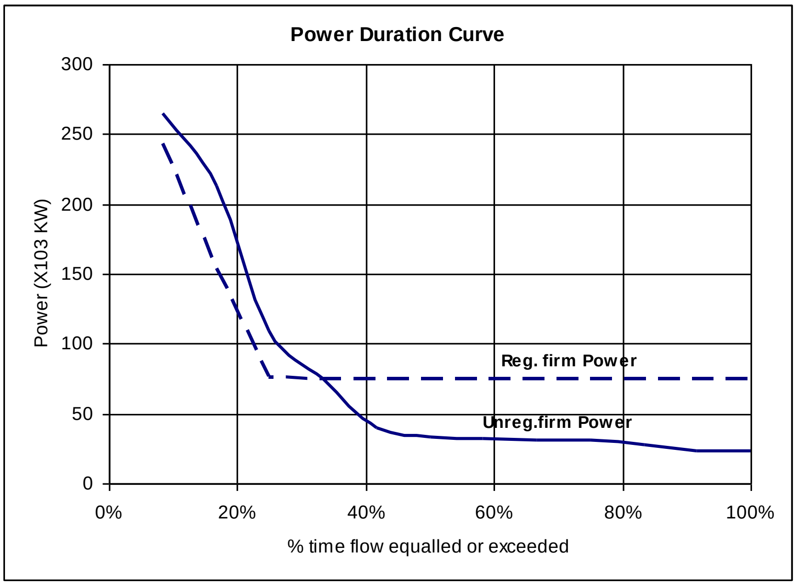


Figure 2.3: Unregulated and Regulated firm power

2.2.3 Supply system:

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

2.2.4 Power Market surveys:

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

2.2.5 Demand forecast:

As hydropower development has long lead-time, it is necessary to be guided by a long term demand prognosis. Normally demand forecast cover at least ten years or more. They are organized in such a way that periodic updating is easy to perform.

- base case
- low case
- high case

2.3 Hydrology

Hydrological studies will provide data on the flow of water, one of the main parameters used in hydropower planning.

Precipitation and hence water supply, varies widely between geographical locations, from season to season and from year to year. Each of these variations has a profound effect on the planning for the control and use of water resources.

All planning in hydrology terms is predicted on the assumption that the past history of water occurrence will be repeated in future. In other words, plans for control and use of water are based on the assumption that the precipitation and stream flow conditions which have been observed in the past can be expected to occur, within reasonable limits of similarity, in the future, except if stream flows are modified by acts of Man.

Obviously the ideal foundation for water resources planning would be comprehensive records, covering an infinite period of years, of precipitation and other climatic conditions, stream flows and groundwater conditions. Unfortunately, such records seldom exist, and the records that are available in most instance fall far short of the ideal.

The generation of hydropower does not imply consumption of water except as a result of incidental evaporation, especially from reservoirs. The extent to which power production will affect the use of water for other purposes will depend on a number of factors such as:

- the location and capacity of power plants
- the nature of power to be produced, that is, RoR power, firm power or peaking power

- the amount of forebay and afterbay regulation provided
- and the relative preference assigned to the uses of water for various purposes

The determination of the water requirement for power production is probably best accomplished by "trial and error" methods including incremental analyses and will require close coordination and integration of power, economic and social studies.

Hydrological data:

- `historical series of daily or monthly flows

Rainfall data

- historical series of daily, monthly or annual total of rainfall

Basic hydrological studies are required mainly to determine *water discharge and hydraulic head.*

2.3.1 Flow duration studies:

A useful way of treating the time variability of water discharge data in hydropower studies is by utilizing flow duration curves. A flow duration curve is a plot of flow versus the percent of time a particular flow can be expected to be exceeded. A flow duration curve merely *reorders the flows in order of magnitude instead of the time ordering of flows versus time plot.*

Methods of computing:

- rank-ordered technique
- class-interval technique

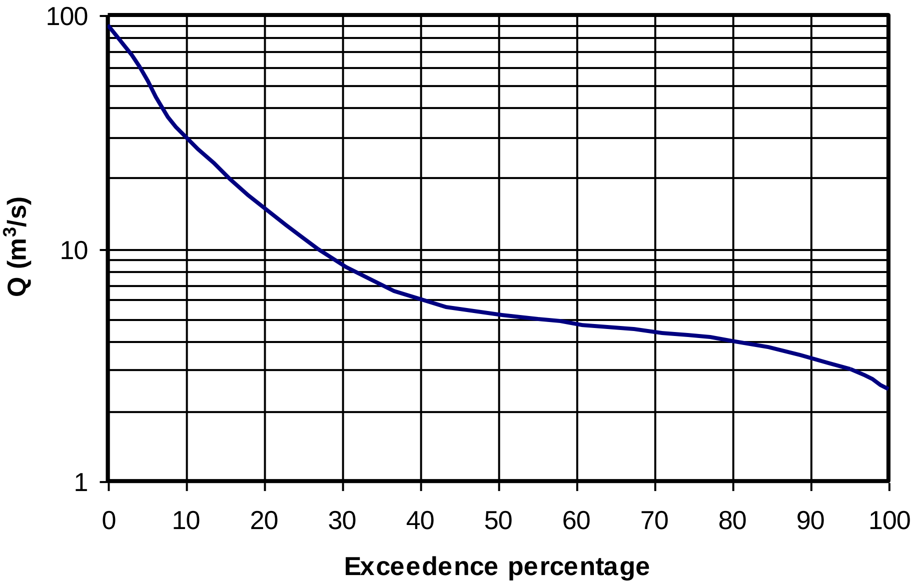


Figure 2.4: Flow Duration Curve (FDC)

The longer the record, the more statistically valuable the information that results from the flow duration curve.

2.3.2 Estimation of flow to ungauged sites:

All too often the stream flow data that are available from measured gauging stations are not from location for which a hydropower site analysis is to be made. Methods are required to develop extrapolation of measured flow duration data which will be representative of a given site on a stream. There are several methods to estimate flows from ungauged catchments. Regional frequency analysis, sequential flow analysis and use of Parametric Flow Duration Curve are some of them.

A regional frequency analysis involves regression analysis of gauged catchments within the general region. Through this technique, sufficiently reliable equations can often be derived for peak flow of varying frequency given quantifiable physical basin characteristics and rainfall intensity for a specific duration. Once these equations are developed, they can be then be applied to ungauged basins within the same region and data of similar magnitude used in developing the equations.

A regional analysis usually consists of the following steps:

- Selecting components of interest, such as mean and peak discharge
- Selecting definable basin characteristics of gauged watershed: drainage are, slope, etc.
- Deriving prediction equations with single or multiple linear regression analysis
- Mapping and explaining the residuals (differences between computed and observed values) that constitute "unexplained variances" in the statistical analysis on a regional basis.

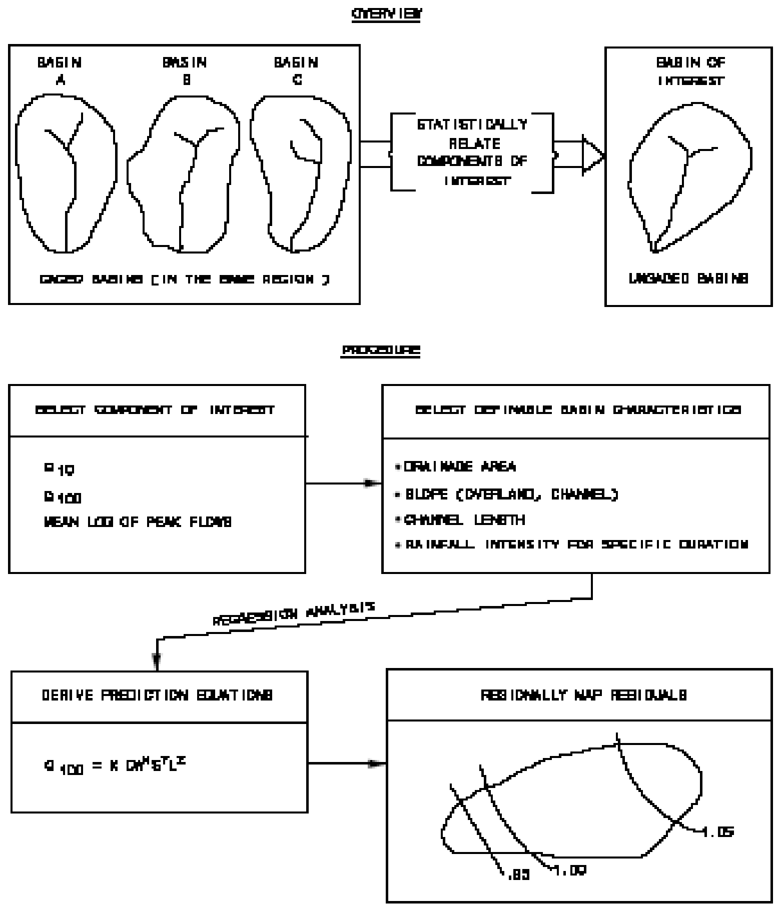


Figure 2.5: Regional Frequency Analysis Procedures

Some of the equations may have the form:

$$Q_2 = 0.24A^{0.88}P^{1.58}H^{0.80}$$

$$Q_5 = 1.20A^{0.82}P^{1.37}H^{0.64}$$

$$Q_{10} = 2.63A^{0.80}P^{1.25}H^{0.58}$$

$$Q_{25} = 6.55A^{0.79}P^{1.12}H^{0.52}$$

$$Q_{50} = 10.4A^{0.78}P^{1.06}H^{0.48}$$

$$Q_{100} = 15.7A^{0.77}P^{1.02}H^{0.43}$$

Where:

Q = peak discharge

A = drainage area

P = mean annual precipitation

H = altitude index

The basic approach in regulated and sequential flow analysis can be explained by referring to the physiographic layout of figure 1.6. in this case a measured record for a considerable length of time is assumed to be available at reservoir outlet A. the location for which flow data are needed is at point B. the flow at B is the inflow from an area of considerable extent where there is no stream gauge record, plus inflow from the operations of a reservoir at station A. A normal annual precipitation map of the entire drainage area is required. Also records from nearby stream gauge (station C) on an unregulated stream that can be considered to represent the sequential variation of runoff from drainage area M (Crosshatched area) are required. These long term records must cover the same period for which regulated flow data are available at station A.
First an estimate must be made of the average annual runoff from area. This is done by planimetering the isohyetal map of normal annual precipitation and getting the normal annual water input into area M, the volume of water per year. Then a coefficient of runoff for the area on an annual basis must be estimated. This can be done by referring to records of nearby gauges on streams that have essentially the same hydrologic characteristics. Multiplying the normal annual precipitation input value by the runoff coefficient gives the average annual runoff from the area M.

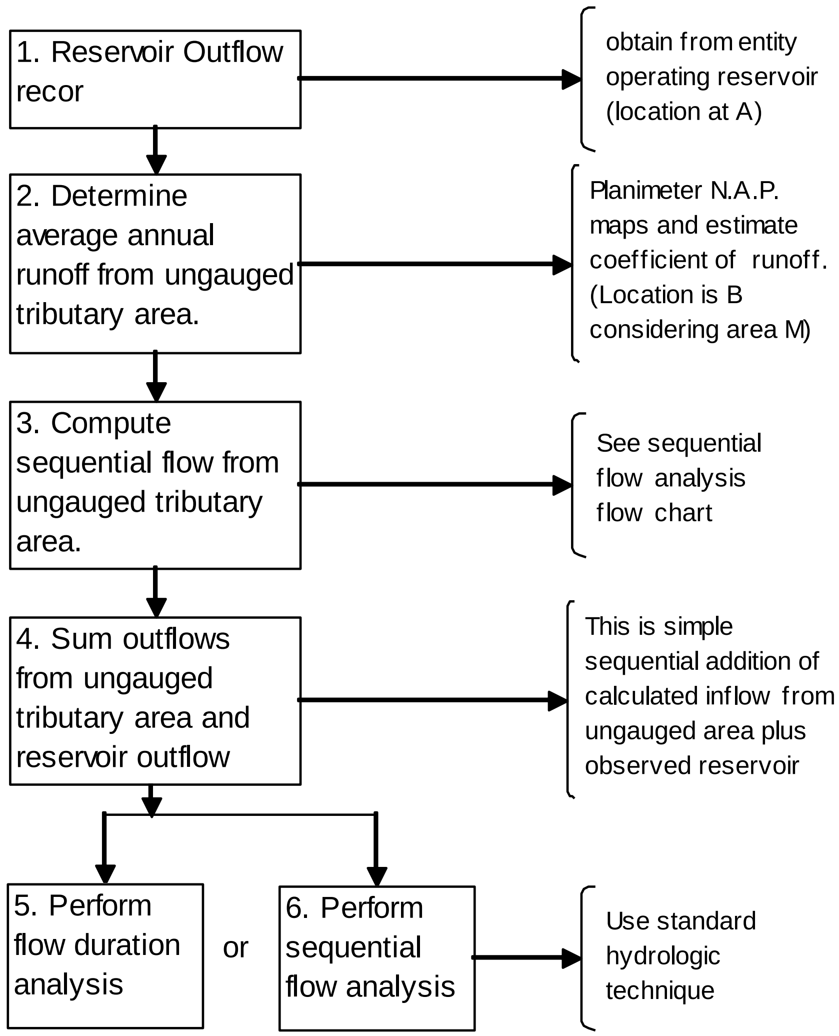


Figure 2.6: Method for determining flow duration of regulated flow combined with ungauged inflow

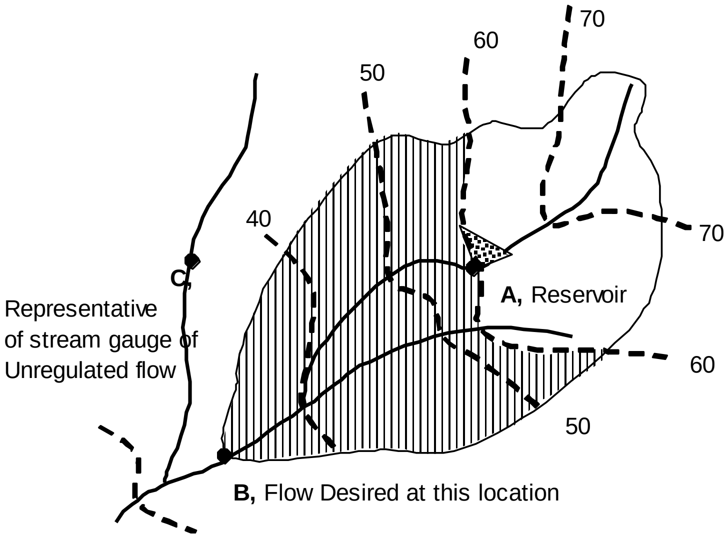


Figure 2.7: Physiographic layout

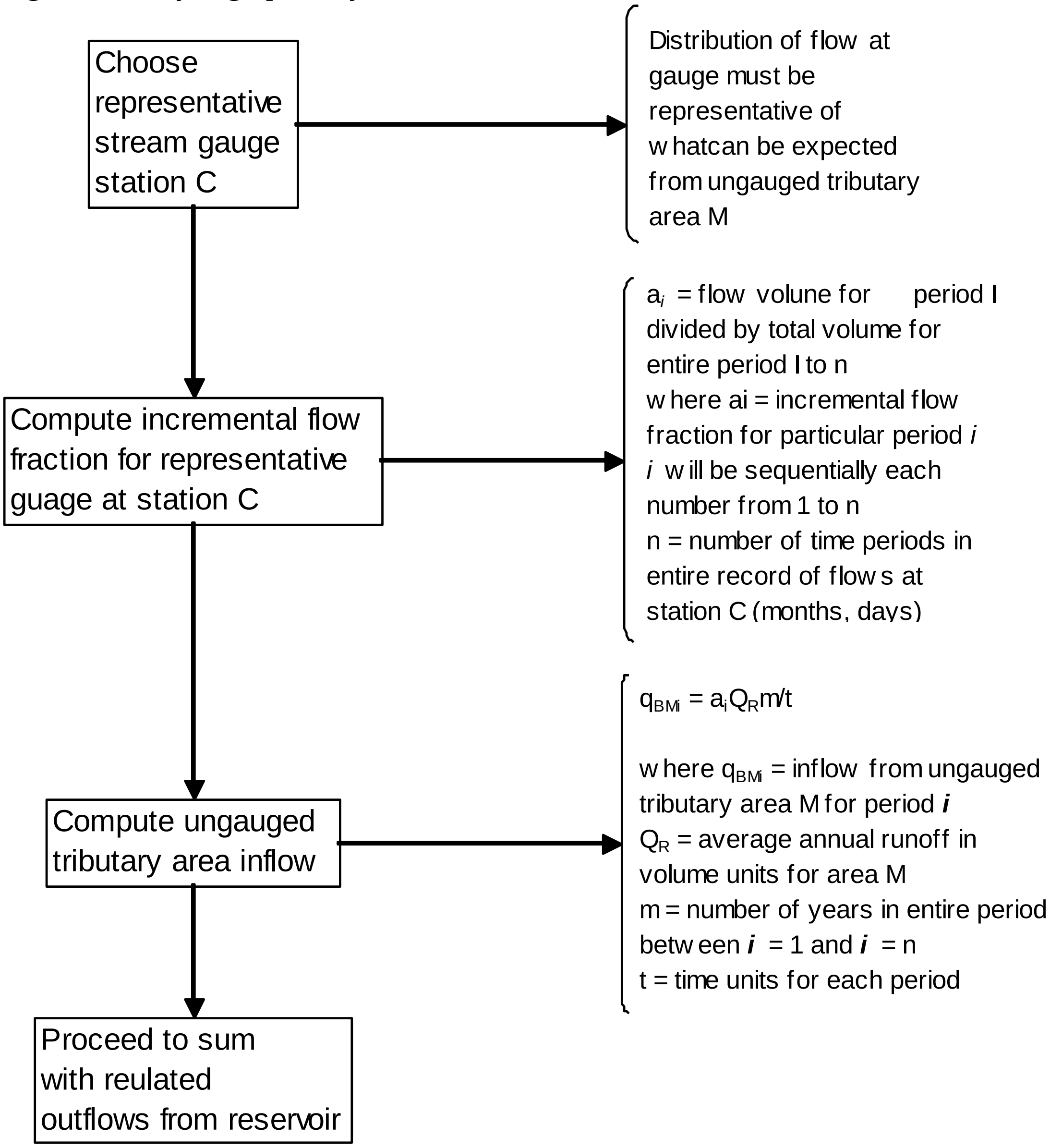


Figure 2.8: Flow diagram for computing sequential flow magnitudes from ungauged tributary area.

A sequential flows coming off area M must be computed. The time increments or periods must correspond to the records of discharge available from reservoir operation. First a flow record at station C must be obtained and studied. The record at C is assumed to have the same time distribution of flow as the runoff coming off area M. an incremental fraction of flow, a_i, for an increment of time in the total desired time period must be obtained for the representative gauge C. Figures 2.8 and 2.9 give flow diagrams for a step by step procedure to calculate the sequential inflow from the ungauged area labeled M in figure 2.7. Once the sequential flows have been calculated it is a simple procedure to add, sequentially the flow from the ungauged tributary to the regulated flows.

In regions where stream flow does not vary with respect to the contributing drainage area flow duration curves can be plotted for the gauged sites. From these flow duration curves are developed a family of parametric duration curves in which flow is plotted against the average

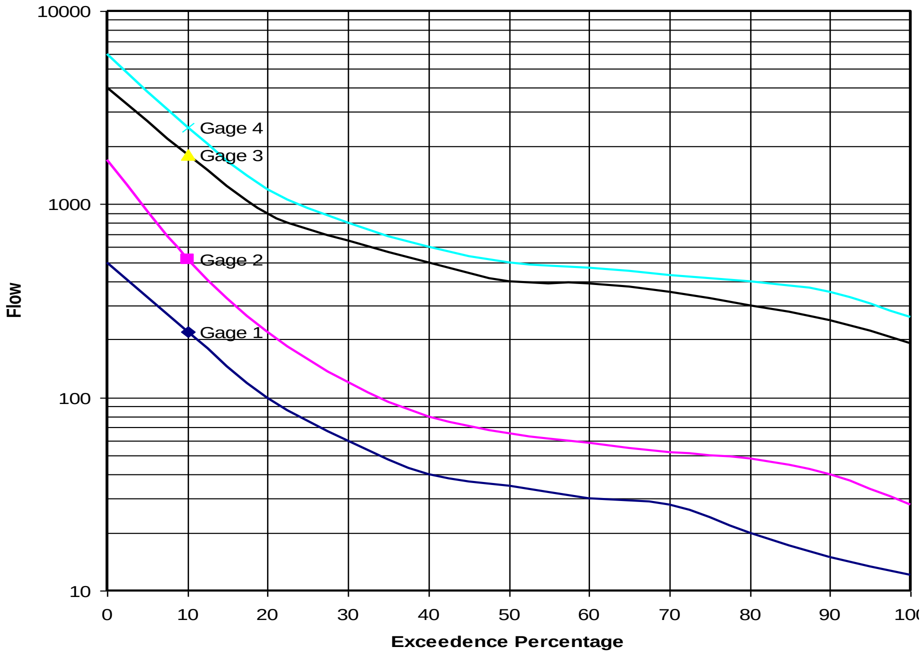


Figure 2.9: FDC for gauging stations in a homogeneous drainage basin

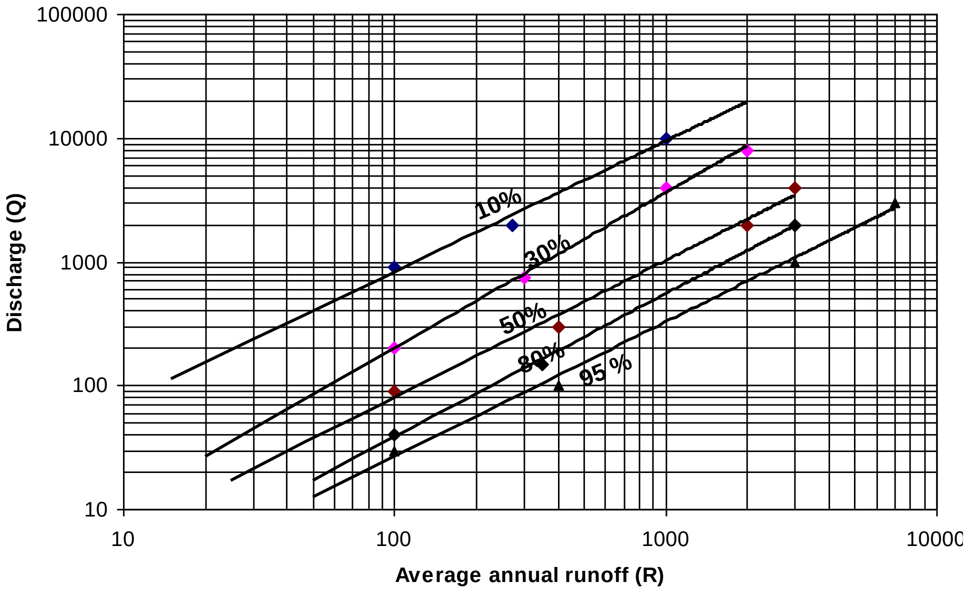


Figure 2.10: Parametric flow duration curve

annual runoff ($\overline{R}$) or annual discharge, $\overline{Q}$ at the respective gauges for several exceedence interval percentages. A separate curve is developed for each exceedence interval used. A correlation analysis is then performed to obtain the best-fitting curve for the data taken from the measured records of stream flow.

To use the parametric flow duration curves effectively, it is necessary to determine the average annual discharge, $\overline{Q}$, at the point or location on the stream for which a hydropower analysis to be made. Isohytal maps developed for normal annual precipitation in a river basin are helpful for determining the annual discharge. The records of precipitation and stream flow data should represent the same period of record.

Utilizing the records of average annual precipitation input to the basins at measured streams nearby or having similar hydrologic characteristics, a runoff coefficient is estimated for the drainage basin being studied. The product of this coefficient and the computed normal annual precipitation input to the basin and the basin area can be used to calculate the average annual discharge as:

$$\overline{Q} = \frac{k\,\overline{P}\,A}{T} \qquad (1.1)$$

With the average runoff annual discharge estimate it is possible to enter the parametric flow duration curve and determine values of flow for different exceedence percentages for which the parametric flow duration curve has been developed.

2.3.3 Energy and Power Analysis using Flow Duration approach

In processing regulated and unregulated flow data, it is important to recognize that in the power equation, flow is the primary limiting factor. When a Run-Off-River type of power study is done and a flow duration analysis is used, the capacity or size of the hydropower units determines the maximum amount of water that will go through the unit or units. This is dictated by the nominal runner diameter and the accompanying outlet area and draft tube.

In the figure below Q_c is the discharge capacity of the plant under the design head. This Q_c is the discharge at full gate opening of the runner under design head. Even though to the left of Q_c on the flow duration curve the stream discharge is greater, it is not possible to pass the higher discharge through the plant. If the reservoir or pondage is full, water must be bypassed by a spillway.

To the right of the runner discharge capacity point, Q_c, it should be noted that all the water that can go through the turbine is the amount flowing in the river at the particular percent of time point. This shows that full-rated power production will not be produced. With pondage it is possible to alter this for short periods of time, but the total amount of energy output can not be increased.

If hydraulic head and the expected losses in the penstock are known, it is possible to generate a power duration curve from the flow duration curve.

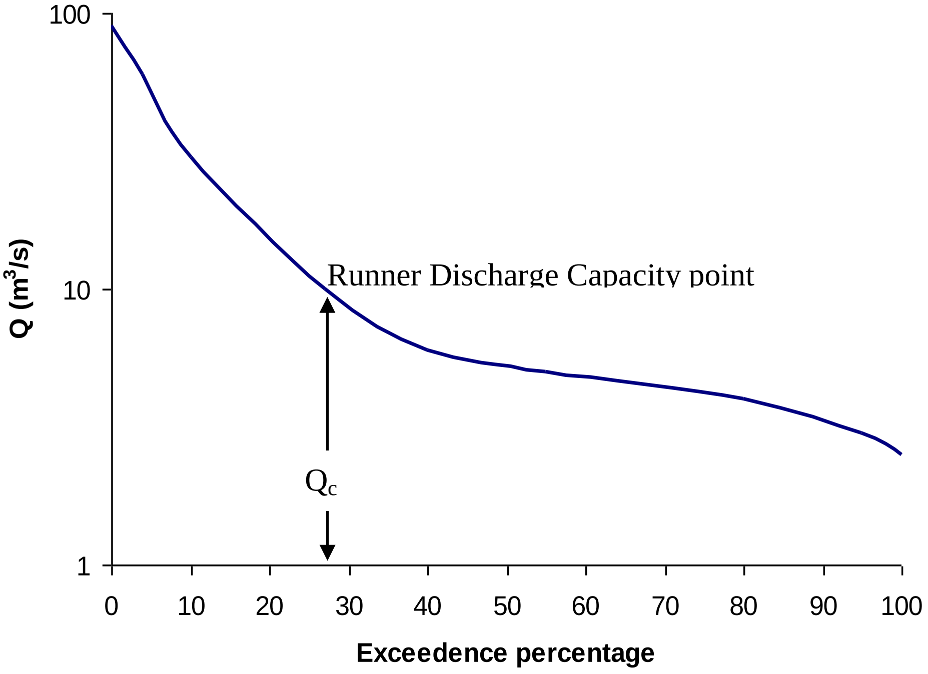


Figure 2.11: Flow duration curve showing discharge capacity value

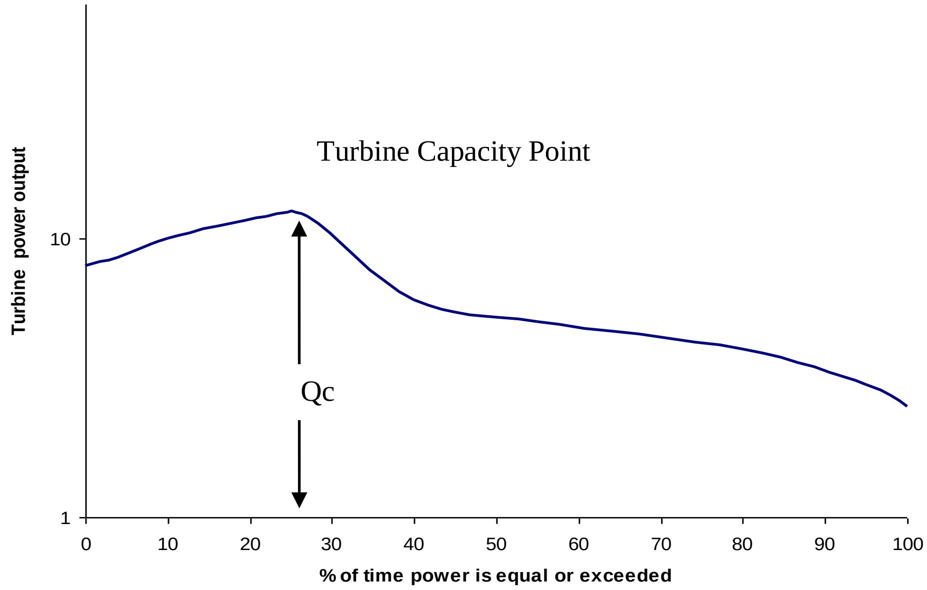


Figure 2.12: Power duration curve

2.3.4 *Water pressure or 'Head'*

Measurement of gross head:

The gross head is the vertical distance that the water falls through in generating power, i.e. between the upper and lower water surface levels. Field measurements of gross head are usually carried out using surveying techniques. The precision required in the measurement will impose the methods to be employed.

In the past the best way to measure was by leveling with a surveyor's level and staff, but the process was slow. Nowadays with digital theodolites, the electronic digital levels and especially with the electronic total stations the job has been simplified. The modern electronic digital levels provides an automatic display of height and distance within about 4 seconds with a height measurement accuracy of 0.4 mm, and the internal memory makes it possible to store approximately 2,400 data points. Surveying by Global Positioning Systems (GPS) is already practiced and a handheld GPS receiver is ideal for field positioning, and rough mapping.

Estimation of net head:

Having established the gross head available it is necessary to allow for the losses arising from trash racks, pipe friction, bends and valves. In addition to these losses, certain types of turbines must be set to discharge to the atmosphere above the flood level of the tail water (the lower surface level). The gross head minus the sum of all the losses equals the net head, which is what is available to drive the turbine.

2.3.5 Residual, reserved or compensation flow

An uncontrolled abstraction of water from a watercourse, to pass it through a turbine, even if it is returned to the stream close to the intake, could lead to sections of the watercourse being left almost dry with serious results for aquatic life.

To avoid this incident, permission to divert water through a hydro turbine or a license to abstract from a river or stream will almost always specify that a certain residual flow should remain.

It is in the interest of the hydro-power developer to keep the residual flow as small as is acceptable to the licensing authority, since in seasons of low flow, its release may mean generation being stopped if there is insufficient discharge to provide for the turbine. On the other hand the lack of flowing water can endanger the life of the aquatic biota

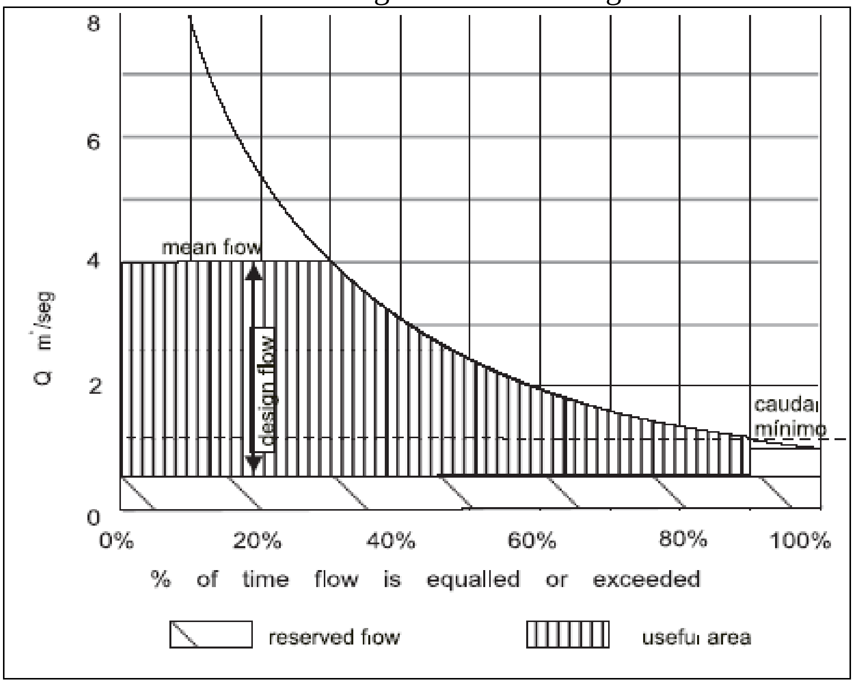


Figure 2.13: Residual, reserved or compensation flow:

2.3.6 Tail water Relationships

As release of water over spillways and other releases in to the stream immediately below a hydropower plant are made, the tail water elevation below the outlet to the turbine will fluctuate. Therefore, it is important to develop a tail water elevation versus discharge curve over the complete range of flow that is to be expected. Preparing such a curve requires an adequate contour map of the channel area and an estimation of velocity in the channel at various stages of flow. Information on normal tail water, maximum tail water, and minimum tail water elevations is necessary to determine design head and to determine the appropriate turbine setting. Estimating stream channel velocity can be made using slope-area calculations that involve conventional Manning's open-channel-flow equation.

2.4 Estimation of plant capacity and energy output

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

Figure 2.13 illustrates the FDC of the site it is intended to evaluate. Usually the design flow is assumed to be, in a first approach, ***the difference between the mean annual flow and the reserved flow***. In actual practice is strongly recommended to evaluate the plant for other design flows in order to choose, the one that yields the best results. Once the design flow is defined ***(Q_m-Q_{res})***, and the net head is estimated, suitable turbine types must be identified. The suitable turbines are those for which the design flow and head plot within the operational envelopes (figure 2.14). Every selected turbine has a minimum technical flow (with a lower discharge the turbine either cannot operate or has a very low efficiency) and its efficiency is a function of the operating discharge.

The gross average annual energy (E in kWh) is a function E = fn (Q_{median}, H_n, $\eta_{turbine}$, $\eta_{generator}$, $\eta_{gearbox}$, $\eta_{transformer}$, γ,h)

Where:

Q_{median} = flow in m^3/s for incremental steps on the flow duration curve

H_n = specified net head

$\eta_{turbine}$ = turbine efficiency, a function of Q_{median}

$\eta_{generator}$ = generator efficiency

$\eta_{gearbox}$ = gearbox efficiency

$\eta_{transformer}$ = transformer efficiency

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

The strip is calculated using the equation:

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

Where:

W = strip width

h = number of hours in a year

γ = specific weight of the water (9.81 KN/m^3)

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

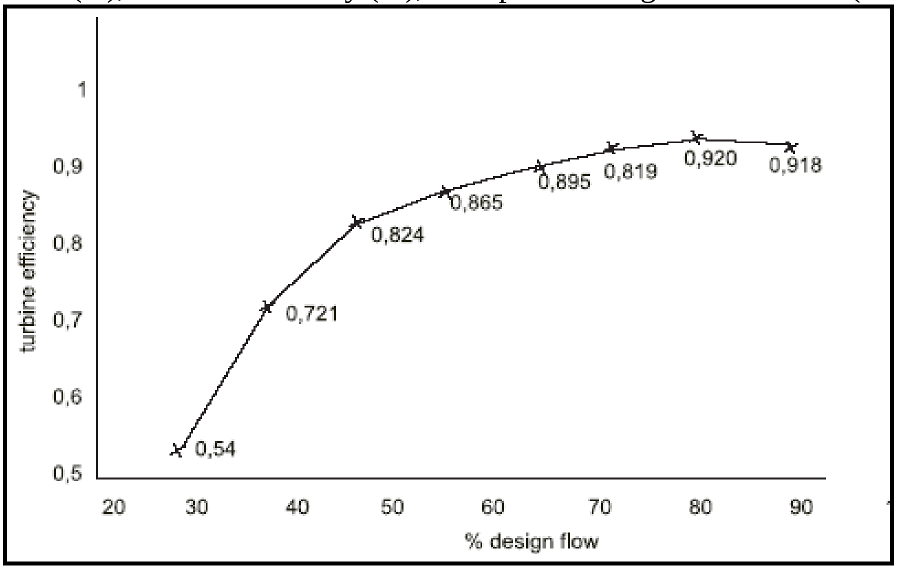


Figure 2.15: Typical Turbine efficiency

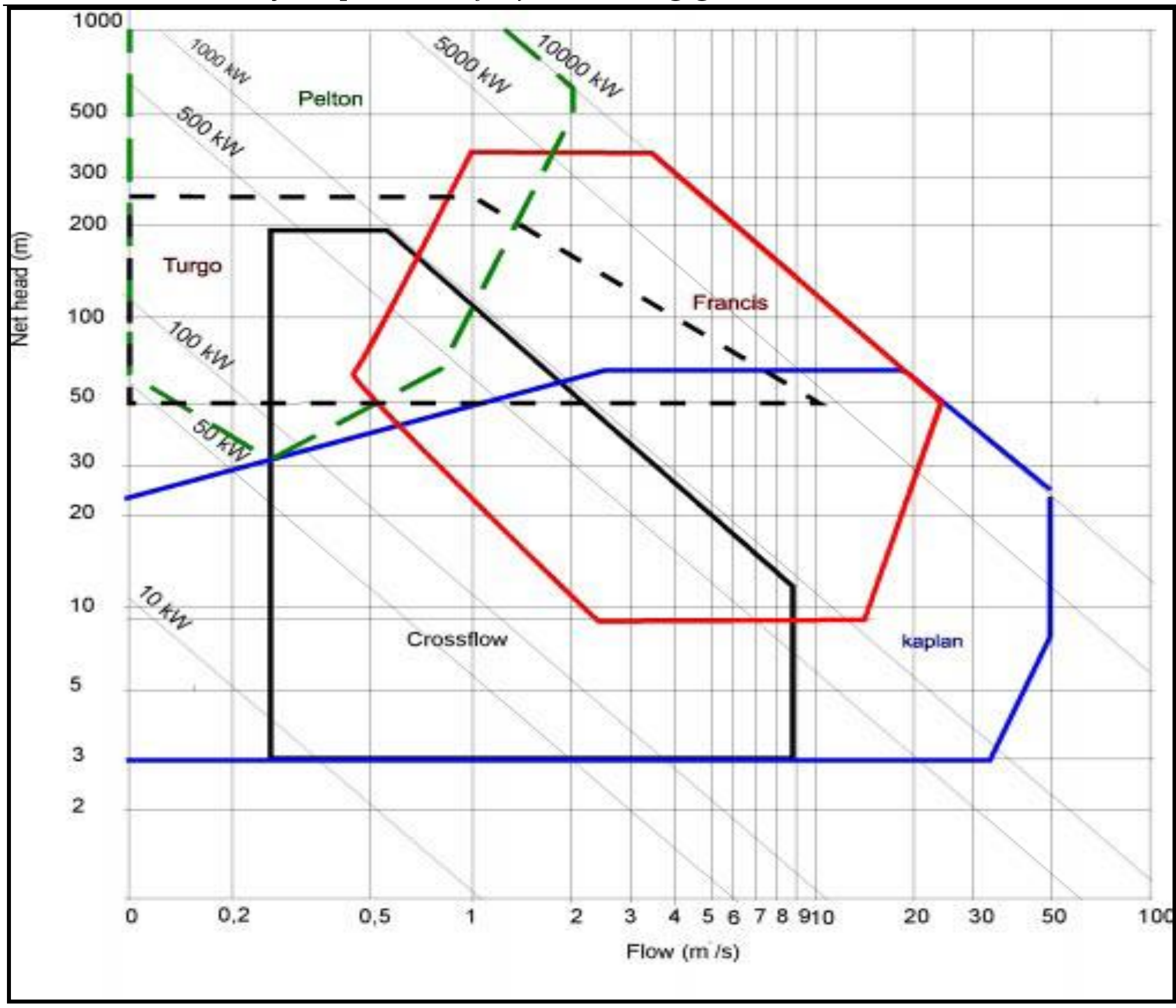


Figure 2.16: Turbine Envelopes

Table 2.1: Minimum technical flow of turbines

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

2.4.1 Plant capacity Determination

Turbine selection and plant capacity determination require that rather detail information has been determined on head and possible plant discharge. In theoretical sense, the energy output, E, can be expressed mathematically as plant output or annual energy in a functional relation as:

$E = f(h, q, TW, d, n, H_s, P_{max})$ (1.2)

Where h = net effective head

q = plant discharge

TW = tail water elevation

d = diameter of runner

n = generator speed

H_s = turbine setting elevation above tail water

P_{max} = maximum output expected or desired at plant

It is seen that there are numerous parameters that can be varied to achieve the best selection. The usual practice is to base selection on the annual energy out put of the plant and the least cost of that energy for the particular scale of hydropower installation. Thus one must recognize that determination of plant capacity requires analyses that vary the different parameters in equation (1.2) while applying economic analysis.

Limits of use of turbine types

For practical purposes there are some definite limits of use that need to be understood in the selection of turbines for specific situations. Impulse turbines normally have most economical application at head above 300 m.

For Francis turbines the units can be operated over a range of flows from approximately 50 to 115% best efficiency discharge. Below 40%, low efficiency, and rough operation may make extended operation unwise. The upper range of flow may be limited by instability or the generator rating and temperature rise. The approximate limits of head range from 60 to 125% of design head.

Propeller turbines have been developed for heads from 2 to 70m but are normally used for heads less than 30 m. For fixed blade propeller turbines the limits of flow operation should be between 75 and 100% of best-efficiency flow. Kaplan units may be operated between 25 and 125% of the best efficiency discharge. The head range for satisfactory operation is from 20 to 140% of design head.

Table 2.2: Minimum technical flow of turbines

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

Determination of number of units

Normally, it is cost effective to have minimum number of units at a given installation. However, multiple units may be necessary to make the most efficient use of water where flow variation is high. Factors such as space limitations by geological characteristics or existing structure may dictate larger or smaller units. The difficulty of transporting large runners sometimes makes it necessary to limit their size. Larger units require construction in segments and field fabrication with special care. Field fabrication is costly and practical only for multiple units where the cost of facilities can be spread over many units. Runners may be split in two pieces, completely machined in the factory and bolted together in the field. This is likewise costly, and most users avoid this method because the integrity of the runner cannot be assured.

Figure 2.17 shows how multiple units can be used effectively to take advantage of low flow variation. At the design stage of analysis and with availability of standardized units, it may be desirable to consider as alternatives a single full-capacity unit, two or more equal size units, and two or more unequal size units to determine the optimum equipment selection

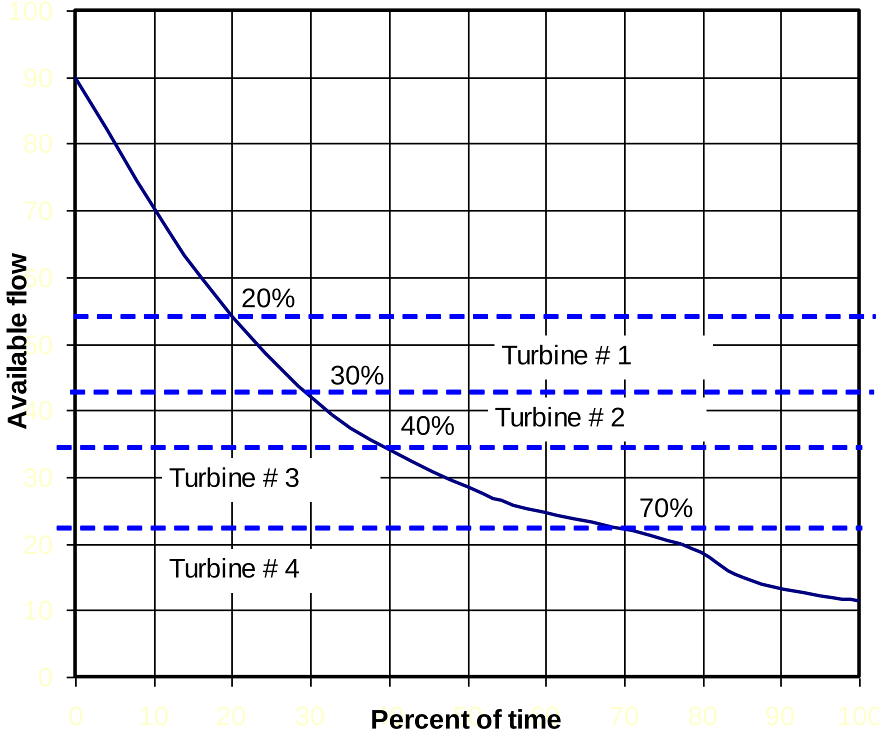


Figure 2.17: Effective use of multiple units

Selection of Most Economical Units

An economic analysis must be done in order to justify the optimum installation. When the curve of total cost of installation crosses the total benefit gained the optimum installation can be decided depending on the local situation. The flow chart shown in figure 2.10 can be followed for turbine selection.

Example is presented how to optimize the most economical installation. The computation is done following the steps given in the flow chart. In table 2.3 the energy for each increment of 10% of the time is determined by considering the average output for the increments. The total energy is then the sum of the 10 increments.

In table 123, the second row gives the value for various flow capacities for alternative sizes of power plants. In the example the plant capacity was varied from 11.68 MW to 6.28 MW. Using flow capacities for 0, 8, 10, 20, 30 and 40 exceedence percentages, and the table was completed to determine net annual benefits and thus most economical size of unit. This required a determination of the project life and the discount rate for money necessary for capital investment. The capital recovery cost was computed using a 7% discount rate and a plant life of 40 years. The investment and annual operating costs are estimated.

1. Obtain river flow data for each percent of time, 0% through 100%

2. Determine headwater elevation at each flow characteristics by flow duration curve. On Run-of river plants this is often constant

3. Determine tail water elevation at each flow characteristics by the flow duration curve

4. Estimate head loss through hydro systems. This will vary with penstock and draft tube

5. Compute a net head for each of the flows characterized. Note: as river flows increase, tail water rises and reduces net head

6. Estimate plant efficiency. This can be made constant for pre-feasibility level design

7. Choose a wheel or plant capacity flow. This full-gate flow will be limited by runner diameter and selected penstock size

8. Compute plant discharge at all flow values for each exceedence percentage. Note: at river flows greater than plant capacity the plant discharge may be less than depending on the net head

9. Compute power output at each percent time under investigation

10. Compute annual energy output for given plant capacity. Repeat this for four to five plant capacities.

11. Estimate the annual plant costs for each of the plant capacities investigated, using estimating curves.

12. With annual energy output calculate plant benefits based on average expected value of power, Birr/KWh

13. Plot a curve or develop table to show where maximum net benefit is obtained

Figure 2.18: Flow Chart of turbine selection procedure

The annual benefit is computed multiplying the energy produced by the unit sale value. In this case 0.26 Birr/Kwh is used for the unit sale. Plotting annual costs and benefits against the installed capacity will then permit a determination of the optimum plant capacity by showing where the maximum benefit is or where marginal benefit equals marginal cost. This is shown in figure 2.19, where the optimum installation is shown as 10 MW.

Table 2.3: Computation table for turbine capacity selection

	Duration (30%)					
	0	10	20	30	40	50
River Discharge(m3/s)	283.10	179.77	133.06	110.41	96.25	87.76
Head (m)	4.72	5.73	6.40	7.01	7.47	7.96
Plant Discharge(m3/s)	110.41	110.41	110.41	110.41	96.25	87.76
Efficiency	0.89	0.89	0.89	0.89	0.89	0.89
Power Output(MW)	4.55	5.52	6.17	6.76	6.28	6.10
Percent Time(%)		10.00	10.00	10.00	10.00	10.00
Energy(MWh)		4414.15	5121.95	5662.46	5708.66	5418.61
	Duration (30%)					
	60	70	80	90	100	
River Discharge(m3/s)	79.27	75.02	72.19	63.70	28.31	
Head (m)	8.38	8.69	8.99	9.30	9.51	
Plant Discharge(m3/s)	79.27	75.02	72.19	63.70	28.31	
Efficiency	0.89	0.89	0.89	0.89	0.89	
Power Output(MW)	5.80	5.69	5.67	5.17	2.35	
Percent Time(%)	10.00	10.00	10.00	10.00	10.00	
Energy(MWh)	5210.72	5033.02	4974.45	4746.77	3294.03	
River Discharge(m3/s)	**49584.81**					

Table 2.4: Computational table for Economic capacity selection

	0	8	10	20	30	40
Plant discharge (m3/s)	283.10	196.00	179.80	133.06	110.41	96.30
Plant Capacity, P(MW)	11.68	9.81	8.99	7.44	6.76	6.28
Capital cost (mBirr)	133.98	121.37	117.89	104.40	95.27	88.39
Capital recovery cost (mBirr)	10.04	9.09	8.83	7.82	7.14	6.62
Annual operating cost (mBirr)	6.24	5.39	5.24	5.18	4.99	4.92
Total Annual Cost (mBirr)	16.27	14.48	14.07	13.00	12.12	11.54
Annual energy out put (10^3XKWh)	56853.29	55991.04	54986.43	52095.30	49584.81	47256.80
Annual benefits (mBirr)	14.84	14.61	14.35	13.60	12.94	12.33
Net benefits (mBirr)	-1.43	0.13	0.28	0.60	**0.82**	0.80

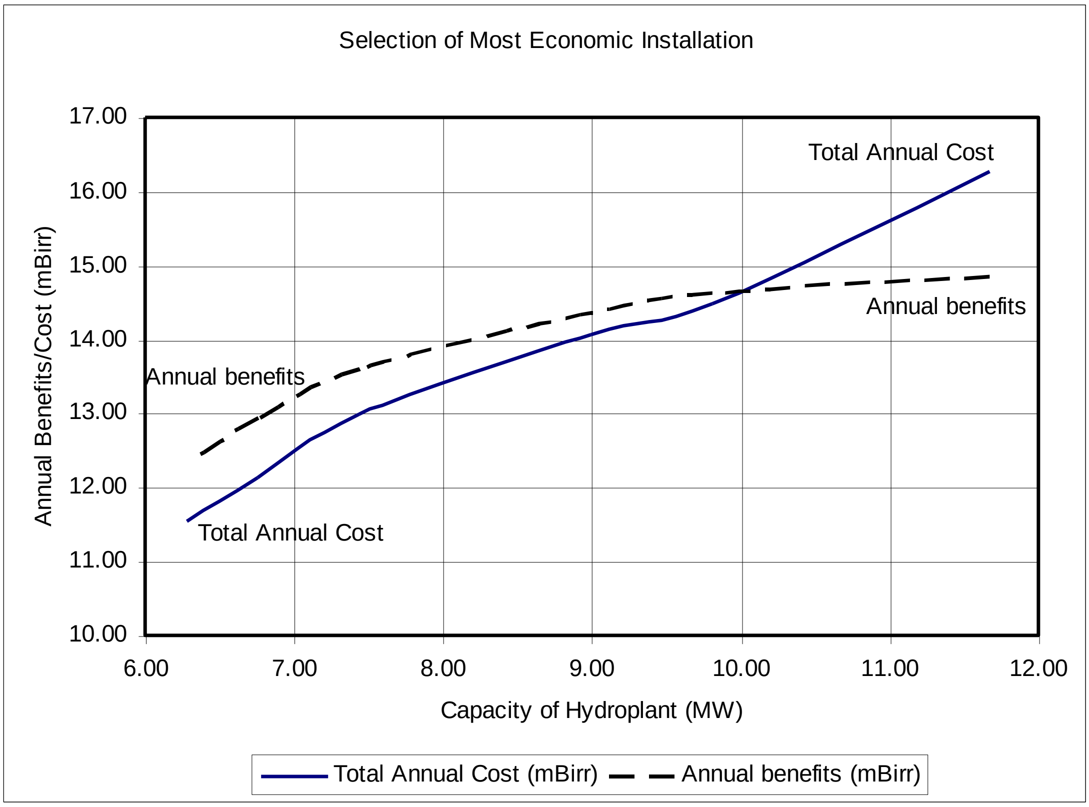
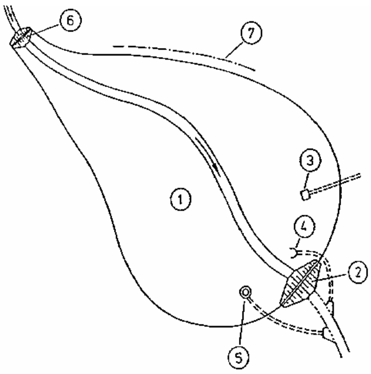


Figure 2.19: Benefits and costs versus plant capacity

2.5 Reservoir (storage) capacity

Reservoir capacity is determined by means of mass curve procedure of computing the necessary capacity corresponding to a given inflow and demand pattern. Reservoir capacity has to be adjusted to account for the dead storage, evaporation losses and carry over storage.



Storage **(1)**
Dam **(2)**
Appurtenant Structure: Intake and Spillage Structures:
Spillage Structure: Spillway **(5)**
Intake: Service Intake **(3)** and Bottom Outlet **(4)**
Surveillance Structure (in Dam, in Foundation as well as Valley Sides)
Diversion **(6)**
Service Road **(7)**

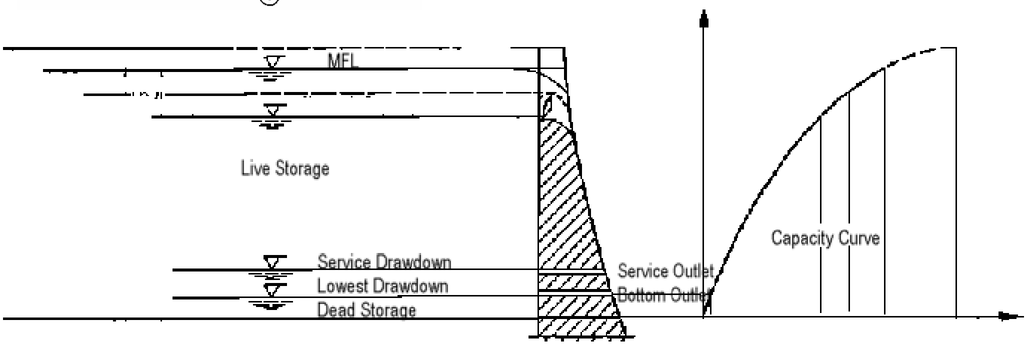


Figure 2.20: Storage Components

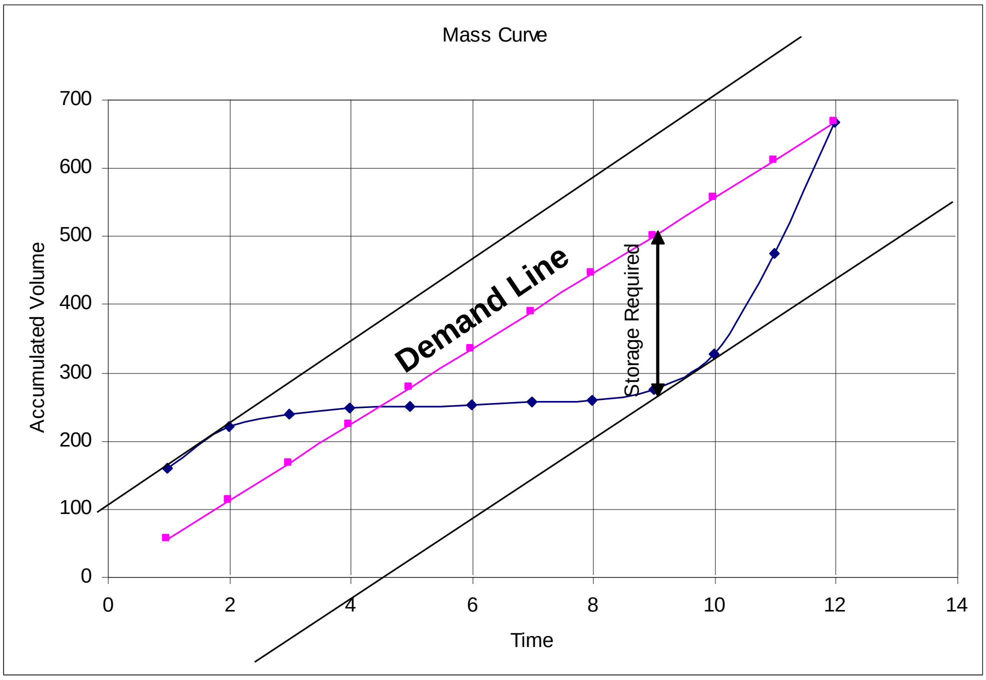


Figure 2.21: Reservoir capacity determination: Mass Curve Procedure

2.5.1 Area Capacity Curves

Most hydropower developments involve an impoundment behind a dam. The water in storage in the impoundment is released; the head water elevation changes and this will influence the design of the plant and the pattern of operation. Therefore, it is necessary to have a storage or pondage volume versus impounding surface elevation curve. At the same time there is a need to know surface area versus reservoir elevation. This information can be obtained by plannimetering a contour map of the reservoir area and making necessary water volume calculations and water surface area. The two curves are typically combined into what is termed an area-capacity curve.

2.5.2 Reservoir Rule Curves

When releases from reservoir are made, the schedule of releases is often dictated by considerations other than just meeting the flow demands for power production. The needs for municipal water supply, for flood control, and for down stream irrigation use dictates certain restraints. The restraints are conventionally taken care of by developing reservoir operation rule curves that can guide operating personnel in making necessary changes in reservoir water releases.

To be effective, rule curves often require the use of rather careful and extensive reservoir operation studies using historical flow data and estimates of demand for water that are likely to occur in the future.

2.5.3 Evaporation Loss Evaluation from reservoirs

Where there is a reservoir involved in a hydropower development there is a need to assess the effect of evaporation loss from the reservoir surface. This loss in warmer climate is considerable.

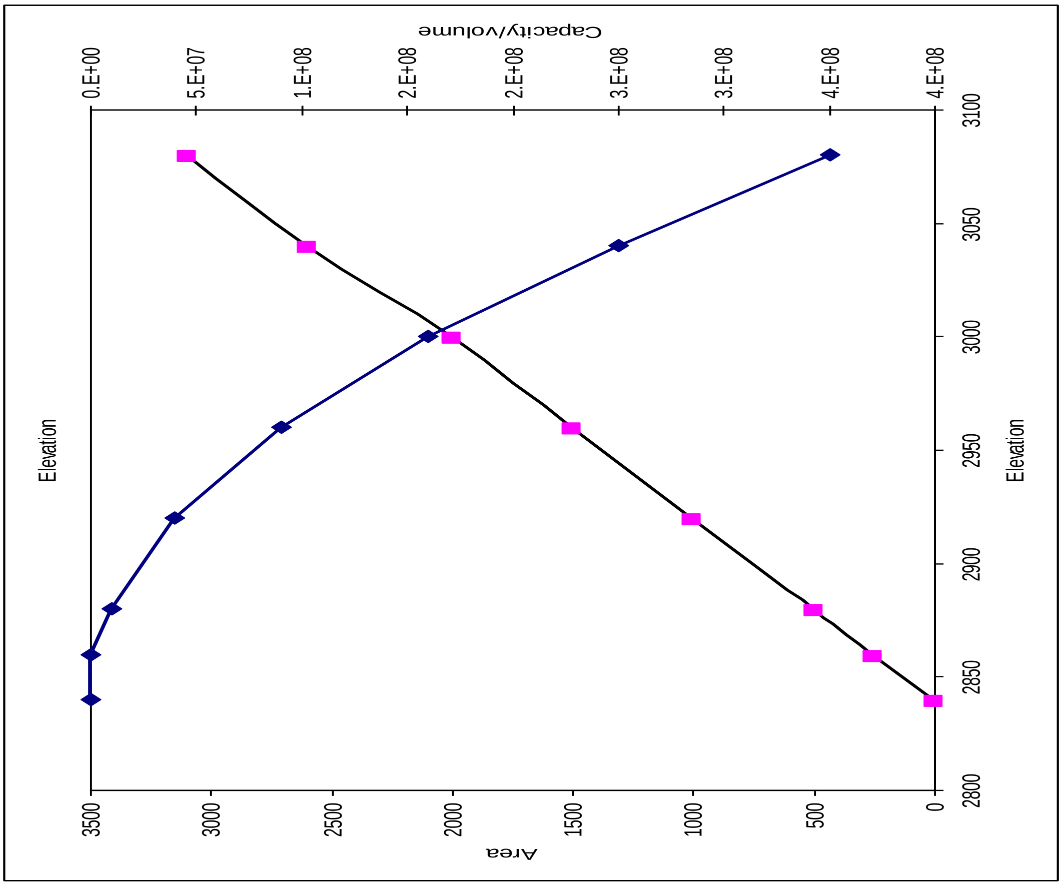


Figure 2.22: Typical area-capacity curve

2.5.4 Spillway Design Flood Analysis

Many hydropower developments require a dam or a diversion that blocks the normal river flow. This then requires that provision be made for passing flood flows. Spillway design flood analysis treats a unique type of hydrology that concerns the occurrence of rare events of extreme flooding. Flood frequency analysis is a well defined procedure for spillway design flood determination. it is customary on large dams and dams where failure might cause a major disaster to design the spillway to pass the probable maximum flood. For small dams, spillways are designed to pass a standard project flood.

2.6 Geotechnical studies

Hydraulic structures should be founded on level foundations, with adequate side slopes and top widths, not subject to stability problems. The catalogue of failures, especially in channel design is so large that a minimum geomorphologic study of the terrain should be recommended for the first phase of the project. The problem is especially acute in high mountain schemes, where the construction may be in the weathered surface zone, affected by different geomorphologic features such as soil creep, solifluction, rotational and planar soil slides and rock falls.

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

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

perennially wet, are some of the features that can compromise channel stability. Drainage treatments and bench constructions are among many others may be recommended.
At the end of the canal the forebay acts as a mini-reservoir for the penstock. It is required that all the water retaining embankment sections should undergo stability analysis regardless of their configuration. The layout of the penstock, usually placed on a steep slope, poses problems both for its anchoring blocks and because it's visual impact. Deep in the valley, frequently built on an old river terrace, the powerhouse foundation poses problems that many times only can be solved by using techniques as up today as the jet grouting.

2.6.1 Methodologies to be used

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

Photogeology:
Photogrammetry at scales from 1:10 000 to 1:5 000 -allows the geologist to identify rock types, determine geologic structures, and detect slope instability.

Geomorphologic maps:
The result of photogrammetric analysis complemented with the results of the field survey must be combined on a Geomorphologic Map. This map, based on a topographic one, drawn at a scale between 1:10 000 and 1:5 000, duly classified using simple symbols, should show all the surface formations affecting the proposed hydraulic structures.

Laboratory analysis:
Traditional laboratory tests such as soil grading and classification, and tri-axial consolidation facilitate the surface formation classification, to be included in the above mentioned map.

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

Structural geological analysis:
Although not properly a geomorphologic technology it can help to solve problems in the catchment area and in those cases where hydraulic conduits must be tunnels in rock masses. The stability of the rock and seepage in the foundation of hydraulic structures are problems that can be solved by this methodology, avoiding dramatic incidents during the operation.
Direct investigations: *Borehole drilling:*
When the dam or weir has to be founded in unconsolidated strata, a drilling program, followed by laboratory tests on the samples extracted is essential. Some of these recommended tests are:

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

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

2.7 Environmental Issues

In the case of hydropower developments there seems to be a growing interest in conservation as opposed to utilization of resources. Hydropower developers contend that development of hydro-resources, providing pollution free energy, represents sound management of natural resources. However, there is no escaping the fact that hydropower projects encroach on the environment. This the developers must accept. They must also ensure that their projects are planned to cause minimum environmental disturbance. Projects should be judged on

environmental as well as technical and economical feasibility. Only projects having acceptable levels of environmental disturbances should be implemented.
The extent of environmental disturbance depends on plans and layout. Negative effects can be kept to a minimum if the environment is considered as planning parameter. Attention must be given to the following issues:
In developing hydropower natural runoff and streams are diverted in to man made waterways, canals, pipes, tunnels, etc. thus reducing the flow over a length of the river during part of the year.
The effect of diversion on a stretch of a river can be mitigated through spilling water at the intake and securing a minimum flow in the river at all times. The effect of the minimum flow may be increased by constructing low weirs of stones, thus creating ponds to maintain water depth. This technique improves the conditions for aquatic life and gives affected rivers a better visual appearance.
Transfer of water from one river basin to another can cause problems affecting downstream water quality and quantity and it can influence conditions for aquatic life.
In the case of river regulation the downstream water flow and quality of water may be greatly altered. Storage may inundate areas of agricultural and ecological importance, displace resident population, cause changes in water conditions and micro climates and increase pests, weeds, evaporation and siltation. Storage projects are therefore increasingly objected to although they may also have considerable positive effects, such as flood mitigation, fish production, recreation facilities, water sports, etc.

2.8 Project Appraisal and Socio-Economic Considerations

Hydropower projects are normally appraised by their direct benefits and the monetary value they can earn on invested capital.
The social benefits resulting from adequate supply of electricity are not considered in the appraisal of hydropower projects as they are difficult to quantify or include in the economic calculations.

The social benefits result from the provision of light, heat and motive power. In addition there is the greatly enhanced quality of life which electrification can bring about.
Economic benefits from electrification arise in two ways:

- directly, through employment opportunities during construction, both the actual construction and the provision of material and supply of components
- indirectly, by simulating the local economy and creating commercial and industrial activities, providing employment opportunities and training for the local population

The main direct benefit may only last during actual construction of the project. Operation and maintenance of hydropower plants are not labour intensive. The employment opportunities they create are therefore few.
The training effect of the construction period may, however, result in skills for local people which can be put to good use after completion of the project.

2.9 Planning

The hydropower development cycle

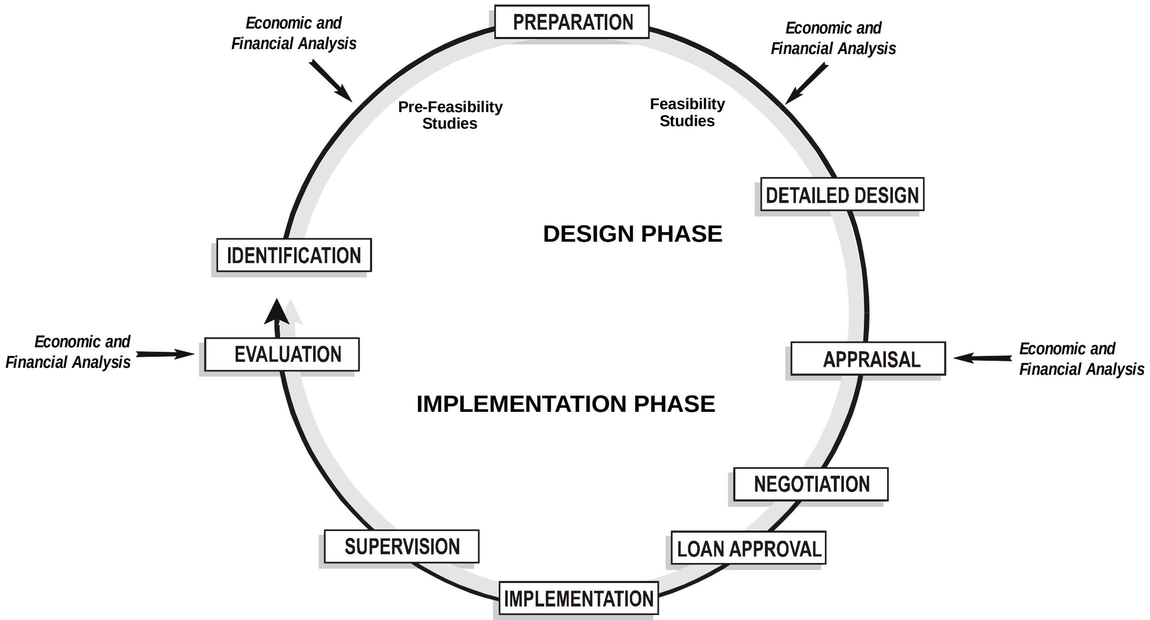


Figure 2.23: Hydropower Project Cycle

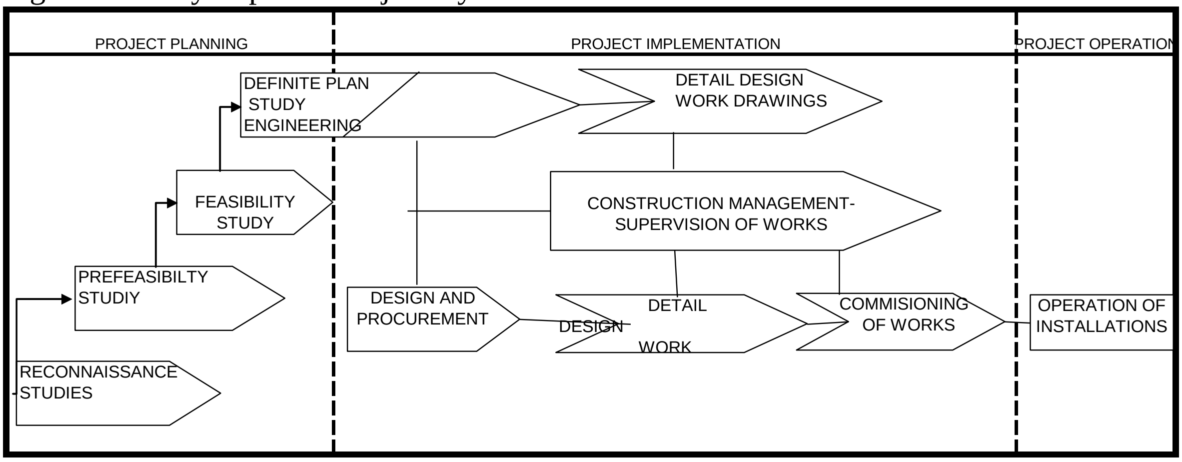


Figure 2.24: Hydropower Project Cycle

The hydropower development cycle consists of three main parts, each covering on of the three periods in the life of hydropower projects:

- Preconstruction
- Implementation
- Operation

Development of hydropower follows well defined stages. Each stage takes the project a step forward in the development cycle, based on the findings from the actual and previous stages.

The major part of investigations, planning and design takes place in the first phase. Normally, the investigation and planning of hydropower projects pass several milestones before projects are accepted for implementation.

There may be many project possibilities and a large number of alternatives to be investigated. Each project has different physical properties and conditions which have to be considered in order to obtain a good basis for planning and design.

Project investigation, planning and design are normally organized in several consecutive studies which are listed here in increasing order of detail, importance and reliability:

- Reconnaissance studies
- Prefeasibilty studies
- Feasibility studies

The first stage of hydropower investigations is the identification of projects. If this has not been done as part of the resources inventories, it must be carried out as part of the reconnaissance study.

Reconnaissance studies are normally the first step of project oriented planning. Such studies are of a preliminary nature as their purpose is not to investigate projects in detail, but like basic hydropower investigations, to identify and investigate the available hydropower resources. The specific purpose leading to reconnaissance investigations is usually defined for electric power and energy, i.e. existing power markets or demand forecasts.

Reconnaissance studies are organized along the same lines as the planning studies to follow, prefeasibility, feasibility, etc. but with much lesser detail and accuracy requirements. Having all planning studies similarly organized will facilitate investigations as well as reporting.

Being the first step of project planning, reconnaissance studies are concerned with identification as well as investigations of projects which are suitable for the stated purpose. The purpose being the supply of an identified power market with electric power and energy.

And hence, the main objectives of reconnaissance studies may be listed as:

- to identify suitable power projects for the stated purpose
- to investigate apparent alternative solution for inclusion in the plans, or rejection
- to investigate and study the various projects and project alternatives to the confidence level required
- to compare the candidate and formulate the project best suited for the stated purpose
- to record lower ranked projects and project alternatives for future reference
- to provide preliminary cost figures and implementation schedules for the selected project

The second organized step in hydropower investigation and planning is called prefeasibility study. In this phase one or more identified projects are brought one step further in the planning process.

The main purpose of pre feasibility investigations is to:

- establish the need and justification for the project
- formulate a plan for developing the project
- determine the technical, economical and environmental practicability of the project
- define the limits of the project
- ascertain local interest in and the desire for the project
- make recommendations for further action

The next stage, or feasibility investigation, is a comprehensive analysis and detailed study of the contemplated project, directed towards its ultimate authorization, financing, design and construction. The feasibility study is carried out in order to determine the engineering (technical), economical and environmental feasibility of the projects. The feasibility study report will provide the necessary information from which the owners can decide whether or not to go for implementation of the project, i.e. to proceed with the definite plan studies, final design and construction of the project. It also serves as application documentation for the development license.

Feasibility investigations include analysis of resources:

- estimates of net economic values to be produced
- estimates of cost of development and construction
- estimation of cost of operation, maintenance and replacement
- assessment of the impact of implementation of the project will have on the environment and the cost of mitigating the effects

3. POWER PLANT STATIONS: Conventional type of Power Stations

3.1 Components of Hydropower projects

Generally three basic elements are necessary in order to generate power from water: a means of creating head, a conduit to convey water, and a power plant. To provide these functions, the following components are used: dam, reservoir, intake conduit or penstock, surge tank power house, draft tube and tail race.

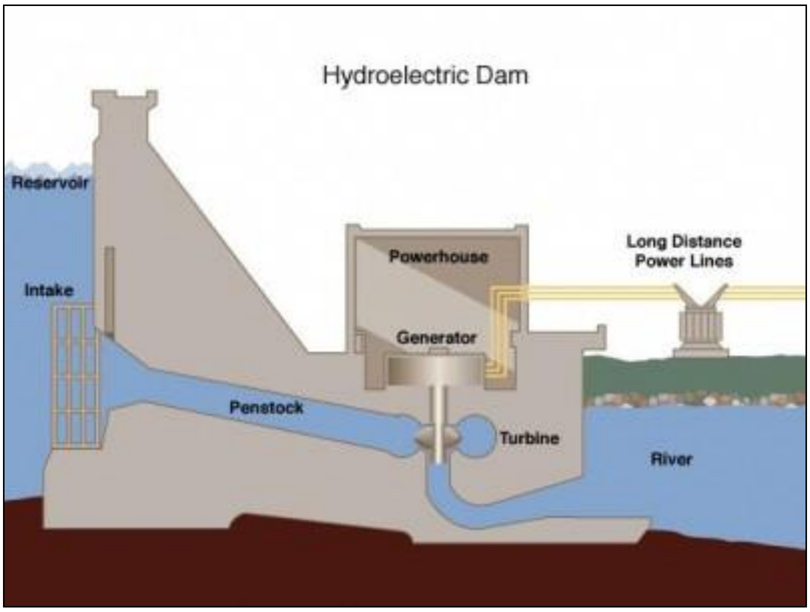
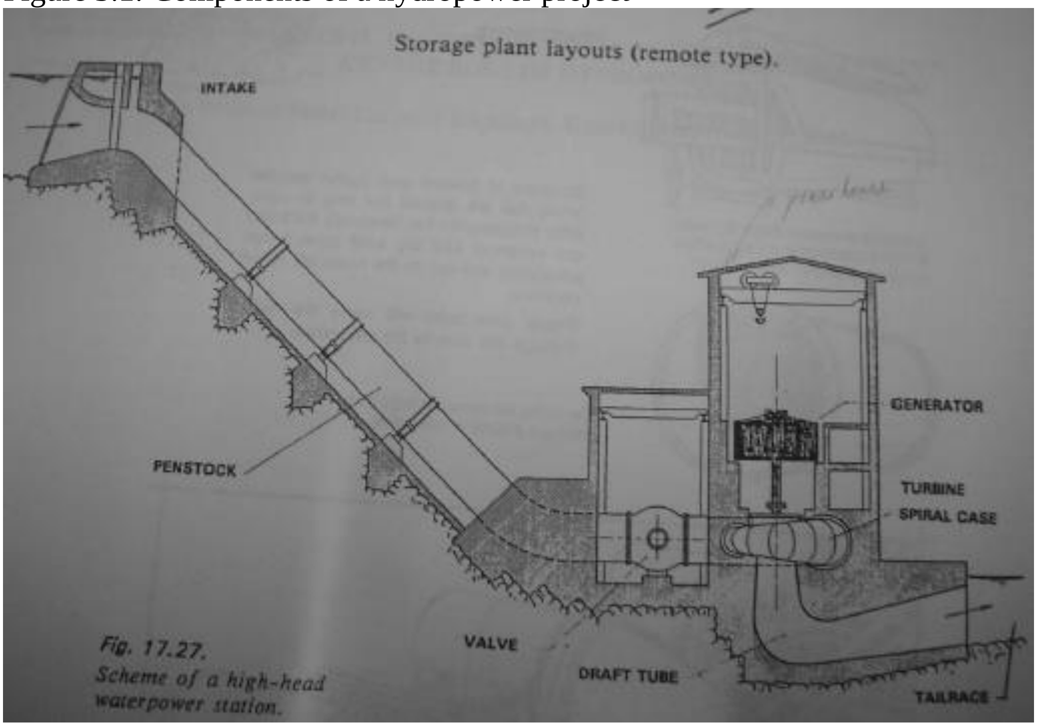


Figure 3.1: Components of a hydropower project



Main components:

Dam: to create the head necessary to move turbines and impound. Storage is used to maintain the daily or seasonal flow variations.

***Reservoir*:** Consists of the Water impoundment behind a dam.

Intake: directs water from reservoir in to the penstock Gates or valves are used to shut off the flow of water to permit emergency unit shut down or turbine and penstock maintenance. Racks or screens prevent trash and debris from entering the turbine units. Projects that are required to use water at a selected temperature must have multi-level intakes in order to control inlet water temperature by mixing waters obtained from different levels.

Penstock: conveys water from the intake structure to the power house and can take many configurations, depending up on the projects layout. For multi-unit installations it is often desirable to serve several Units with a single penstock, and manifolds or bifurcation structures are provided to direct flow to individual units.

Surge tanks: Flow through a penstock can change rapidly during the operation of power plants. As long as flow is steady and constant, pressure changes on the conveyance conduits are minimal. However, pressure changes within the conduit become greater as the rate of change of flow increases. This phenomenon is known as water hammer and is caused by a change of momentum within the water column. When there is a rapid changes in flow water hammer effects can become serious. Surge tanks are constructed on the conduit to reduce momentum changes due to water hammer effects.

Surge tanks are often necessary in medium and high head hydropower projects, particularly where there is a considerable distance between the water source and power plant. Surge tanks or chambers can also be provided on the draft tube where discharge conduits are very long.

3.2 Power house

The power house shelters the turbines, generating Units, control and auxiliary equipments, and sometimes erection and service areas. The power house location and size is determined by site conditions and project layout. It could be located within the dam structure adjacent to it or some distance away from the dam. The power house would be located to economically maximize available head while observing site physical and environmental constraints.

3.2.1 Power house types

There are four types of power house configurations (structure), three of which are classified according to how the main generating unit, are housed: Indoor, Semi-out door, Outdoor and Underground.

Indoor: this type of structure encloses all of the power house components under one roof.

***Semi-out door*:** this powerhouse has a fully enclosed generator room. The main hoisting and transfer equipment is located on the roof of the plant and equipment is handled through hatches located in the roof.

Outdoor: a generator room is not provided with this type of power house structure. Generators are enclosed in a weather proof individual cubicles or enclosures and are recessed in to the floor.

Underground: this type of powerhouse is often used in mountainous areas where there is limited space available to locate a power plant. It is also used to minimize penstock length in these areas since the penstock can be located directly below the reservoir. Pumped storage powerhouses are often located underground in order to shorten the penstock and obtain deep settings on the turbines.

The selection of powerhouse configuration and structure should be based upon both Fixed Operation and Maintenance (O&M) costs. The lower capital cost associated with out door and semi-out door power plants is often offset by increased equipment and Operation and Maintenance costs. The final selection of powerhouse for any given site would be made after a detailed cost study, usually performed in the feasibility design stage

3.2.2 Power House planning

The basic requirement of a power house is the functional utility and the aesthetic requirements. Planning the power house should be harmonious with the surrounding.
A power house of a hydropower may be

i. Surface Over ground power house
ii. Under ground power house

A surface power house has no space limitation where as an Underground power house has space limitation. The surface power houses need an architectural planning so that they fit in with the general landscape. If a particular area is affected by landslides and if the underlying geology is suitable, an underground powerhouse is the obvious choice. For low head power plant and small scale developments surface power house is the economical choice

Design of the powerhouse is primarily a structural and architectural problem and the size of the building is governed by the requirements to accommodate the generator, the spiral casing and the outlet area of the draft tube.

For feasibility studies powerhouse layout dimensioning can be done using theoretical and empirical relations of the power house components. For final design it so customary for the turbine and generator manufactures to furnish dimensions for the interiors of the spiral casing, draft tube, and generator assembly.

The following items of equipment are considered for planning and dimensioning of the power house:

i. Hydraulic equipment:
   i. Turbines
   ii. Gate and gate valves
   iii. Relief valves of penstocks
   iv. Governors
   v. Flow measuring equipment
ii. Electrical equipment:
   i. Generator
   ii. Excitors
   iii. Transformers, pumps, cooling systems, connections, funs and plate forms
   iv. Switching equipment:
      a. Low tension buses
      b. Switch board panels
      c. Switch board equipment and instruments
      d. Oil switching and
      e. Reactors
   a. High tension system:

a. *Buses*
b. *Oil circuit breakers*
c. *Lightening arrestors*
d. *Out going connections*

b. Auxiliaries:
a. *Storage batteries*
b. *Station lighting*

iii. Miscellaneous equipment:
i. Crane
ii. Work shops
iii. Office rooms
iv. Other facilities,(clinic, Store , etc)

The machine in the power house can be either vertical mounting or horizontal mounting. A horizontal mounting machine requires more floor space but less height. A vertical mounting machine requires less floor space but more height. For larger capacity installations, it is ideal choice to have vertical mounting. In general power houses are oriented differently to accommodate excavation and site preparation problems.

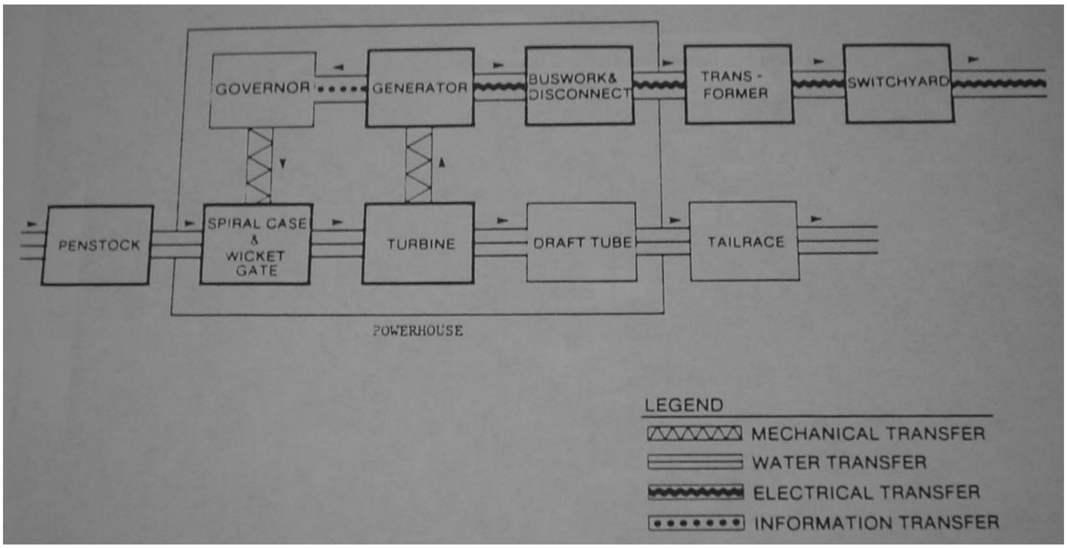
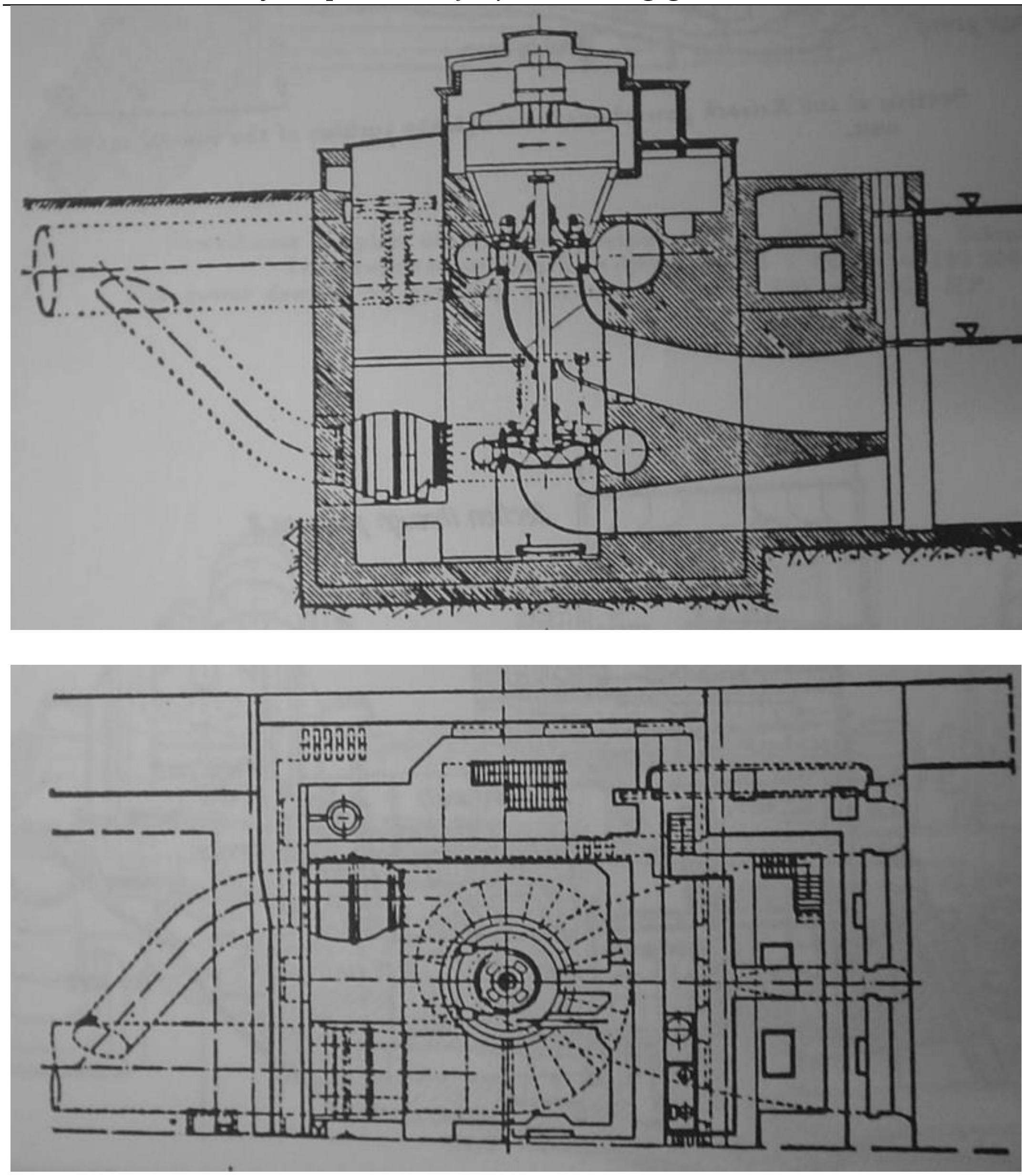
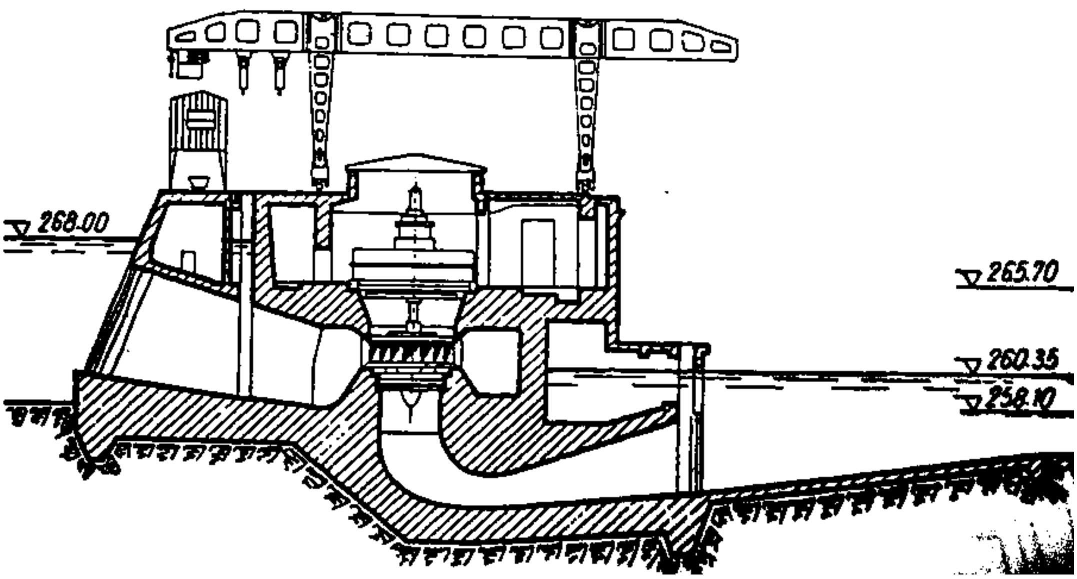
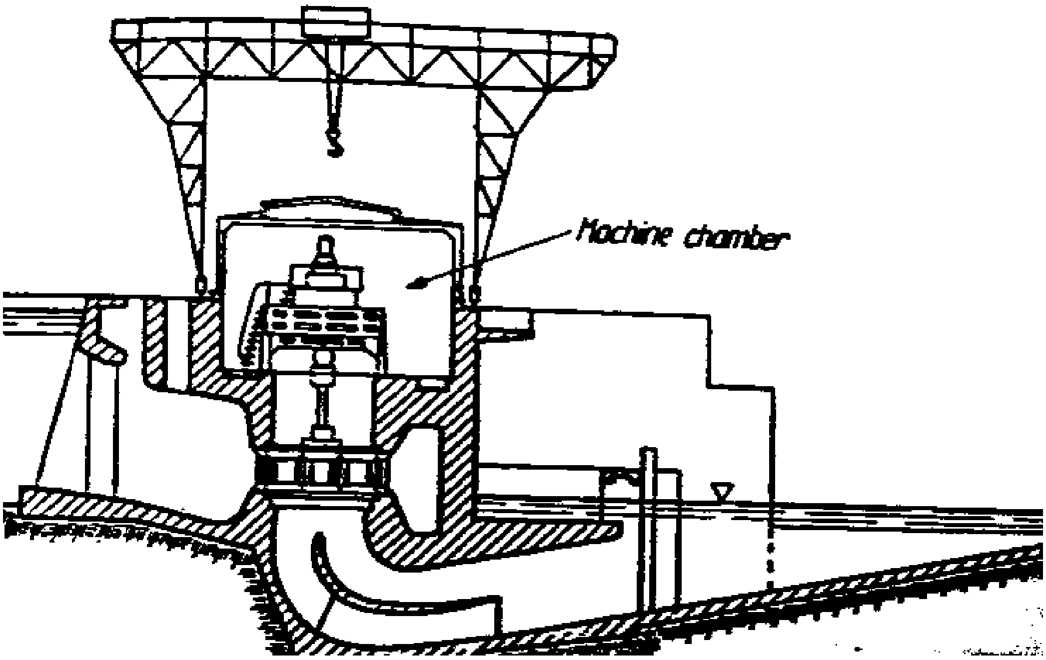


Figure 3.2: Power House System Network



a) High Head





b) Low Head

Figure 3.3: Typical Cross section of Power House

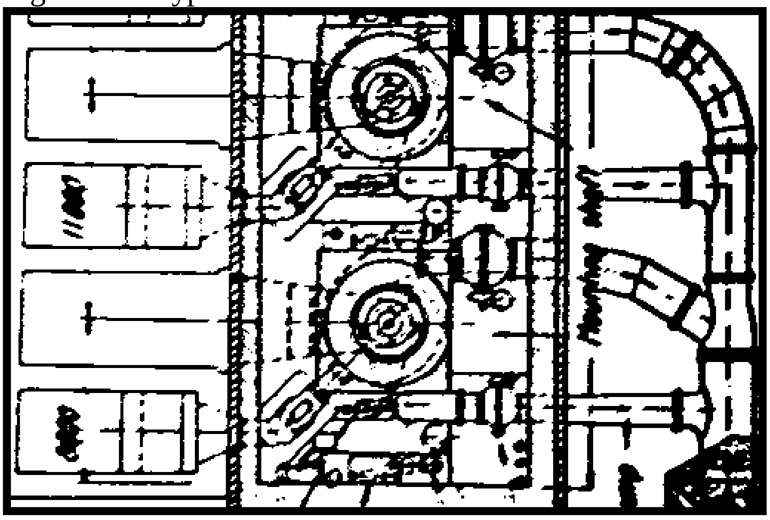


Figure 3.4: Typical plan of the Generator floor

3.2.3 Components of a power house

A power house can consist three main components in general:

i. Sub- Structure
ii. Intermediate structure and
iii. Super structure

The substructure of a power house is the portion below the turbine level. The super structure is the foundation of the power house which consists of steel and concrete structures necessary to form the draft tube, support the turbine stay ring and generator. It also gives accommodation for drainage facilities, tail water and access galleries to the substructure. The substructures transmit the load to the foundation.

Horizontal setting has advantage compared to vertical setting in the following aspects:

- Reduction in civil works because of less excavation
- Combination of sub and intermediate structures (only sub structure)
- A smaller height of power house, and
- Use of conical draft tube

The arrangement is also advantageous for easy inspection and accessibility during maintenance.

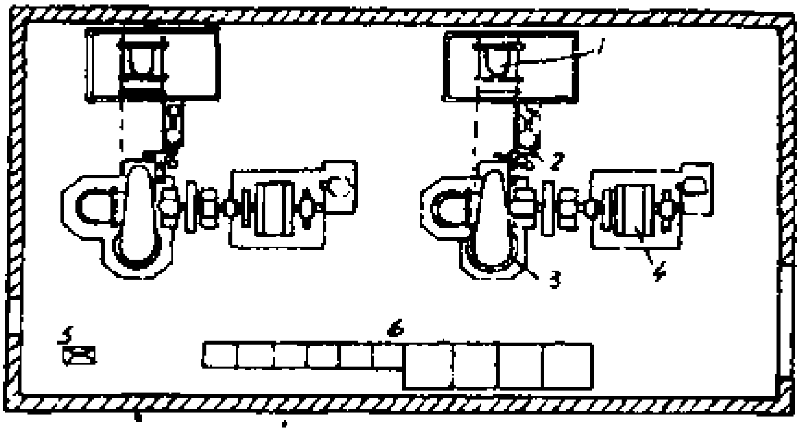
3.3 Layout and dimensions of power house

3.3.1 Layout of Generating Units for small hydropower:

Suitable turbines: Horizontal Francis turbine and impulse (diversion type plant)

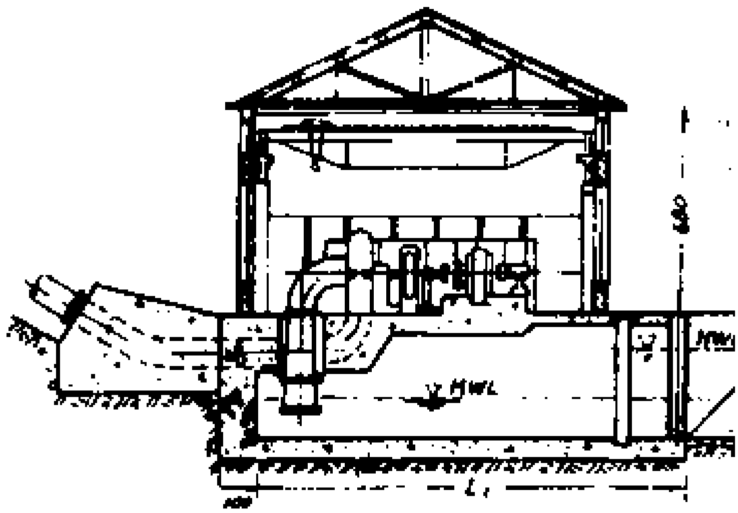
Two types of layout s are generally used namely with horizontal Francis turbines:

i. A Unit axis parallel to the power house axis
ii. A Unit axis perpendicular to the power house axis

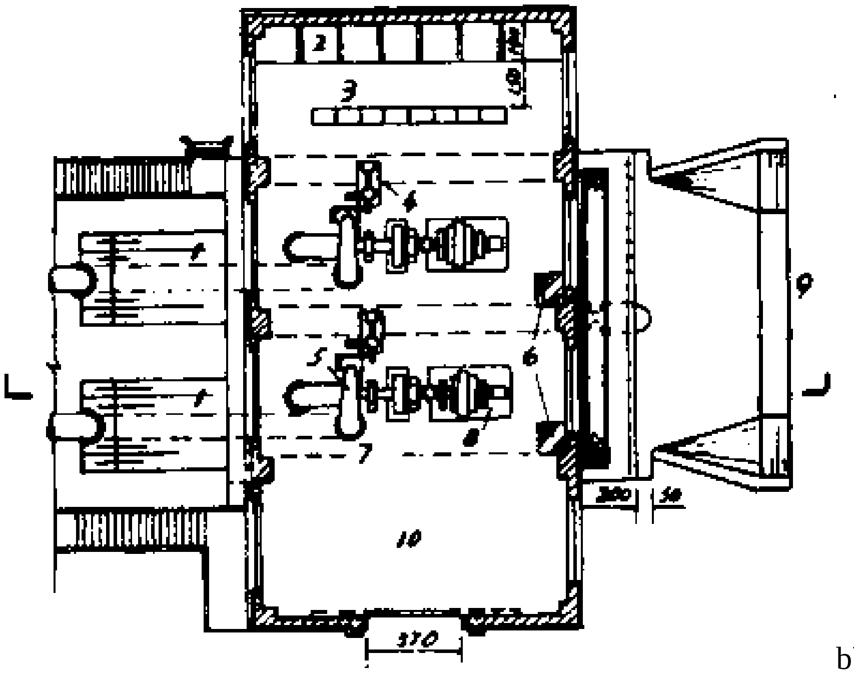


Legend
1. Valve
2. Governor
3. Turbine
4. Generator
5. Transformer
6. Control Panel and switch board

Figure 3.5: Unit axis parallel to the power house axis



a) Cross Section



Legend
1. Anchor Block
2. Switch board
3. Control Panel
4. Governor
5. Turbine
6. Man hole
7. Fly wheel
8. Generator

b) Plan

Figure 3.6: Unit axis perpendicular to the power house axis

The advantages of unit axis parallel to the power axis are:-

- A better approach to the turbine
- A smaller with to the power house

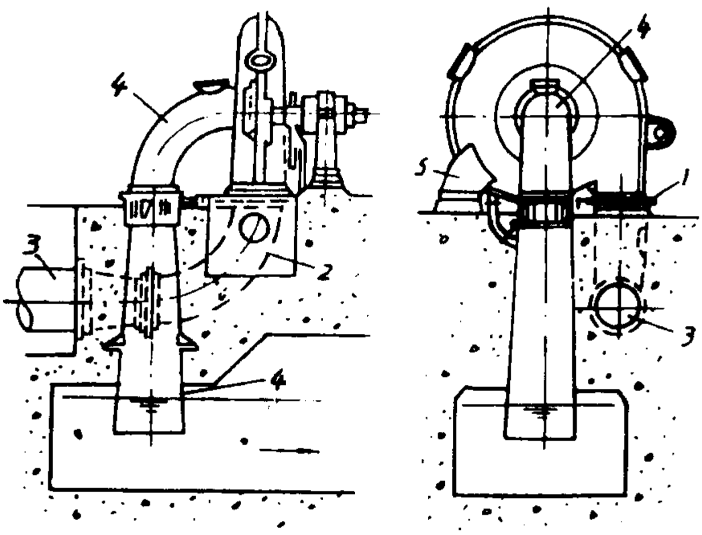
And its disadvantage:

- The larger space between units, which is unsuitable for the arrangement of branching pipes in front of the power house

Main advantage of unit axis perpendicular to the power house axis is the smaller space between units.

In either of the layouts, the control panel and/or switch board can be properly arranged in different areas. The working bay can be eliminated or provided at one end of the power house and the valve can be located in the power house or in front of the power house on the basis of the actual conditions mentioned earlier. The cable ducts and other ducts can be arranged under the power house floor with out any difficulty.

It should be noted that when the setting elevation is high or the suction head is large, the inlet of the spiral case is arranged vertically down ward, and when the setting elevation is low or the suction head is small the inlet of the spiral case is arranged horizontally.



Legend
1. Inlet of spiral case
2. Pipe bend
3. Penstock
4. Bend to draft tube

Figure 3.7: Vertically downward inlet in the spiral case

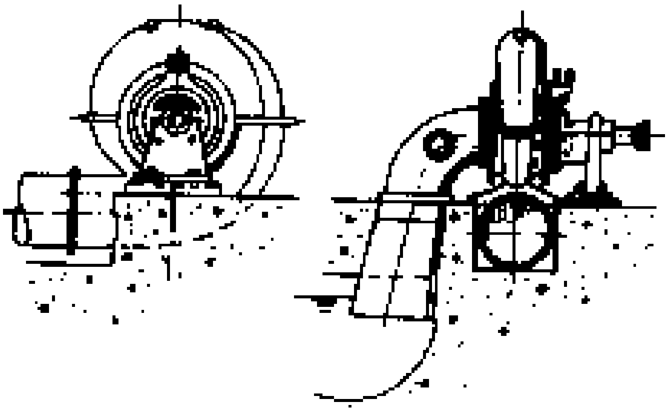


Figure 3.8: Layout for horizontal Francis turbine

Generally speaking, horizontal Francis turbines are unsuitable for those cases in which the tailrace level varies greatly during flooding. Generally, a high tailrace level can be prevented by a water proof wall of a power house, or with a special wall or dyke behind the power house. In this case a sump wall should be properly provided.

3.3.1.1 The dimensions of power houses

The dimensions in the plant are determined by the dimensions of the generating units or by the dimensions of the spiral case particularly when the head is low.

The width of the working bay is generally equal to about one unit bay, if the unit is planned to be major overhauled in the power house. If the unit is planned to be major overhauled outside the power house there can be no working bay in the power house. The width of the unit bay is so determined that the clearance between the two units or between the unit and the wall, should be sufficient for the erection and disassembly of the unit, generally, about 2m. The passageway, for the operators should be 1-1.5 m, and the clearance between the switch board / control panel and other apparatus should be at least 2m, and that the switchboard and the wall should be about 0.8m. For the side unit, its unit bay should have an additional width (about 1m per Units).

The determination of the setting elevation of the turbine is very important for the Power House design, taking in to consideration the minimum tailrace level and the suction head of the turbine.

The height of the Power House is mainly determined by over head craning of the heaviest part of the unit.

3.3.2 Preliminary dimensions of power House for Medium and large Hydro (Reaction Turbine installation

1. *Unit spacing in terms of discharge (for steel scroll case)*

Discharge in m^3/s	Unit spacing in m
25	10
50	13
75	15
100	17
150	20
200	22
250	24

2. *Unit spacing in terms of discharge diameter*

Discharge diameter of runner in m	Unit spacing in terms of discharge diameter (m)
1	5.5
2	5.1
3	4.7
4	4.4
5	4.2
6	4.0

Width and height of the power house in also calculated based on the capacity of the unit and crane span required.

3. *Width of power house*

Capacity of unit in 1000 KVA	Crane span in m for operating head in m				
	25	***50***	100	150	200
10	16.2	***12.5***	10.7	-	-
20	-	16.0	13.0	11.6	11.0
30	-	18.3	15.3	13.7	12.5
40	-	-	17.1	15.3	14.0
50	-	-	18.3	16.5	15.3
60	-	-	-	17.7	16.5
70	-	-	-	-	18.3

4. *Height of the power house*

Capacity of unit in 1000 KVA	Height to crane rail from generator floor in meter of operating head in m				
	25	***50***	100	150	200
10	16.8	***12.6***	11.1		
20		14.8	13.1	12.3	11.5
30			14.4	13.4	12.6
40			15.8	14.4	13.8
50			16.8	15.6	14.4
60			17.8	16.4	15.1
70			18.4	17.4	15.8
80				18.0	16.4

Figure 3.9: Discharge diameter of a runner

Figure 3.10: Section of a power house, Example

5. *Other formulae*

i. Unit spacing :

   a. width if draft table + wall thickness

   b. E + B + Wall thickness

ii. Width of power house:

   F+ C+ 2 + 1.85 D3

   D_3 = discharge diameter

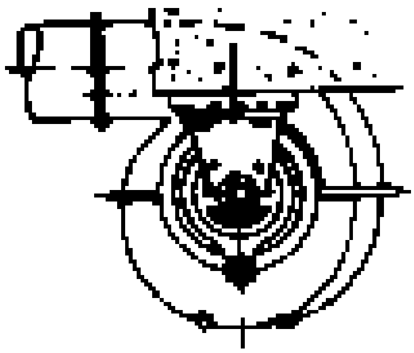


Figure 3.11: Scroll Case

iii. Mosonyi's formula :

   Unit spacing

$$\left(s.s - \frac{N_s}{200}\right)D_3$$

   N_s = specific speed

iv. J.J Donald's formula :

   Unit spacing $= 3.5$ *to* $6\ D_3$

6. *Determination of discharge diameter, D_3 (Mosonyi's formula)*

$$D_1 = \phi * \frac{\sqrt{H}}{N} \quad \text{(Entrance Diameter)}$$

$$D_3 = \left(0.5 + \frac{Ns}{400}\right) \quad \text{(Exit Diameter)}$$

$\phi =$ Peripheral coefficient

N = speed of turbine in rpm

7. *Guthrie Brown's formula*

$$D_3 \left\{\frac{90Q}{N}\right\}^{1/3}$$

Where, Q discharge at full load in m^3/s

3.3.2.1 Bay's Dimension

The three essential bays in a power house complex are:-

i. Unit bay or machine hall
ii. Erection bay
iii. Control room

Machine hall:

Length: the length of the machine hall depends on the number of units and the size of machine. For vertical alignment machine the centre to centre distance is controlled by the size of the scroll casing layout. Standard layout indicates a distance of ***4.50 to 5.0D,*** where ***D*** is the turbine out let diameter. Added to this dimension is the minimum clearance of ***2 to 3m.*** So, the preliminary dimension between centre to centre of two units is ***5.0D + 2.5m.*** For higher specific speed it can be ***4.0D + 2.5*** or smaller. Knowing the number of machines, the total length of the machine hall can be worked out. The additional bay for the erection loading can be one unit length.

Width: the width of the machine hall is determined by the size and clearance spacing between the walls – needed as gangway. Since the gangway requirement is of the order of ***2.5m,*** as a first approximation the width of the power house can be presumed to be at least equal to the centre to center distance of two machines. Unnecessary increase in width will increase the length of the Electrically Operated Trail (EOT***)*** or Mechanically Operated Trail (MOT) and the roof structure. In the Machine hall, the generator placing is not exactly at the centre of the machine hall but towards one side so as to provide enough operation space for the crane operator.

Height: the height of the Machine hall is fixed by the head room requirement (about ***2 to 2.5m***) of the crane operation. The hall must have a height which will enable the crane to lift the rotor of the generator or the runner of the turbine clear of the floor without any obstruction. To this clearance, space is to be added the depth of crane girder and the head room for the operating cabin.

Loading and erection bay:

This space is required for unloading or loading heavy machines and for its erection. Small assembly is also done on the space. The stator of the generator which come in smaller segments are also assembled on the loading bay. The size of the erection should be sufficient to keep the larger parts like the rotor of the generating unit.

Control bay:
The control bay houses the control equipment. It can be adjacent to the unit bay as in most power houses. Signal is sent from the control bay to the operating bay from where the operation control is achieved. Most of the controls are operated by remote control from the control bay.

Service crane:
The crane should be designed for such a capacity that it can lift the heaviest component in the power house. Normally, the heaviest part is the rotor and the stator.

Cable and bus bar:
These are placed in the cable ducts made in the floor of the generator in the bus bar galleries (cable galleries). High voltage cables should be carried separately.

Switch Yard:
This is the yard with step up transformers. This should be located near the power house. In most cases switch yards are kept out side the power house.

4. UNDER GROUND POWER HOUSE

If there is a gorge and a valley, an underground power house may be economical. Other factors for choosing under ground power stations are frequent seismic activities landslides and snow avalanches. An important characteristic of the under ground power plant station is its flexibility of layout. The shortest possible layout through various feasible alignments can be draw up with minimum size of pressure conduits and omissions of anchors and valves. The basic requirement for the feasibility of the underground power house is the availability of good sound rock at the desired location and depth. Underground power house are also safer during war attacks.

Most of the power projects that came in Europe after World War –II are underground power houses.
Some of the underground power stations in the world:

- Portage Mountain (Canada) - 2300MW
- Komano (Canada)- 832 MW
- Vianden (Luxembourg) 920MW
- Tddiki (India)- 840MW
- Tekeze Hydropower (Ethiopia)-300MW

4.1 Location of underground power stations

Depending upon the rock quality, tunneling ease and overall economics, the power houses may be located in various ways.

1. The whole power house may be totally underground:

2. The generator may be in a pit but the super structure may be on the surface

3. Semi-Underground, here the generator may be located on the surface while other units, such as turbines may be under ground

4. The power house may be placed in a cut where the stable rock exists, the units may be placed in a cut in the rock

4.2 Arrangements of underground power stations

The type of layout of underground power plants depends largely on the positions of head and tail water levels, control valves, turbines, generators, transformers, control room, access shafts and ventilation shaft. The Other factors responsible for the location of such a plant are the topography, geology and the head to be developed.

According to Mosonyi, the various characteristic types and layouts of the power station could be described with reference to head and tail water levels as follows.

Characteristics types of under ground power development

i. Upstream Station or head development
ii. Downstream station or tail development
iii. Intermediate station development
iv. Diagonal Tunnel alignment with air cushion surge tank

The upstream station or Head development (Swedish type of development): in this type of development, the power station is located close to the intake and thus water is directly fed from the reservoir/forebay to the generating units.

This arrangement is suitable for low head (25-50 m) and high discharge condition in the continuously sloping or mildly rolling terrains. A surge tank could be provided at the entrance to the tunnel to protect it from the water hammer during sudden opening and closure of turbines.

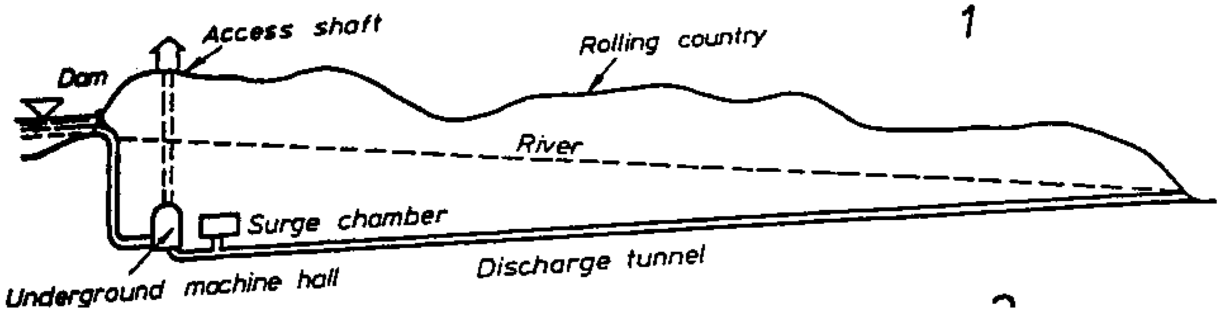


Figure 4.1: The upstream power station or Head development (Swedish type of development)

Downstream Development or Tail race development (Swiss type of development): this type of Development has its characteristics in a long and nearly horizontal pressure tunnel together with pressure shafts and a short tail race tunnel. Such a development is most suited for a rugged terrain and where high heads of the order of several hundred meters can be utilized.

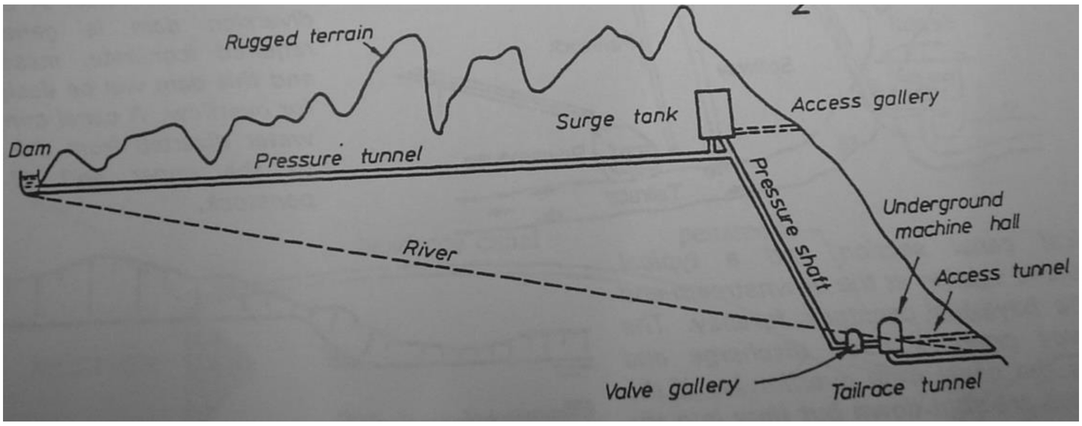


Figure 4.2: Downstream or Tail-race development power station arrangement (Swedish type of development)

Intermediate station Development (Italian arrangement): the characteristics of this type of arrangement are a long head-race tunnel and a long tail race tunnel. The consequent pressure variations due to long tunnels are taken care of by surge tank both upstream and downstream of the power house.

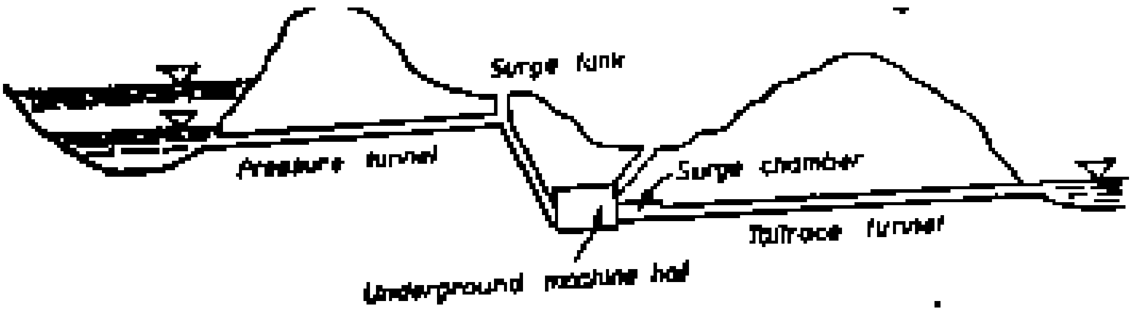


Figure 4.3: Intermediate power station arrangement (Italian type of development)
Diagonal Tunnel alignment with air cushion surge tank (Norwegian solution):

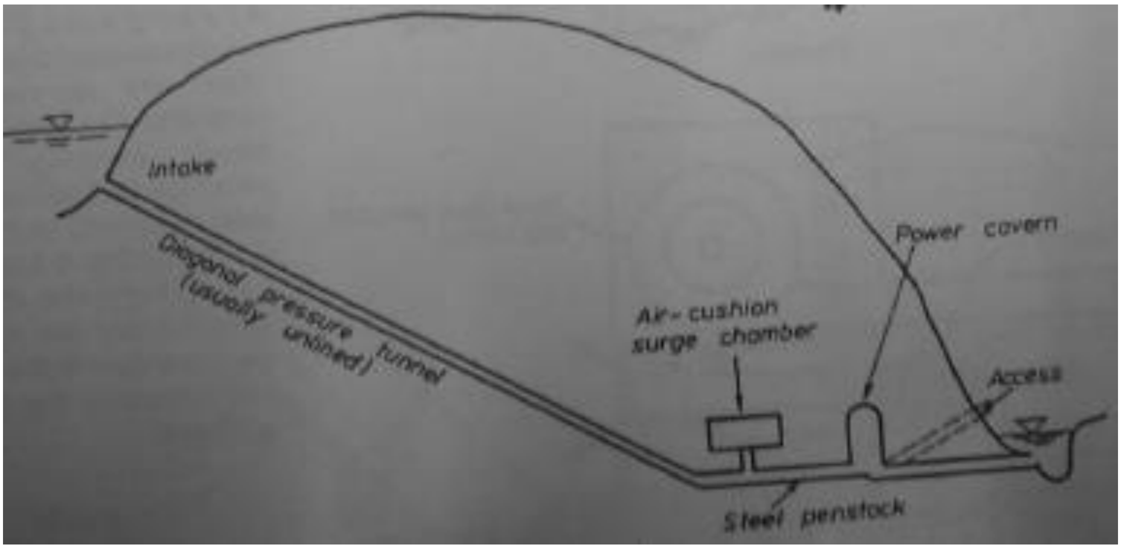


Figure 4.4: Diagonal alignment with air cushion surge tank (Norwegian Solution)

4.3 Comparing above–ground and underground power house stations (Basics for comparison of power station alternatives)

- *An underground power house is more costly than a powerhouse on the surface, implying that underground powerhouse should be considered only when this solution is the only option available due to site topography*
- *In many cases a number of parameters concerning investment costs, risks, operation and maintenance will have to be considered before a conclusion can be drawn as to whether the optimum layout of a hydropower plant shall have powerhouse on the surface or underground*
- *In some case the assessment of rock conditions will be decisive in determining whether an underground or a surface powerhouse is the most favorable solution. However, in general the type, head, and capacity of the power plant and the topography of the project site provide more important parameters for the decision*

Some of the main parameters to be considered in the assessment of the overall plant layout are:
The dam /Reservoir Type: the dam itself creates the head to be utilized in the power plant and so the waterways will be short. Maximum head for a pure design of this type is restricted today by feasible dam heights to about 250-300m.

The powerhouse is integrated as part of the dam structure, located on the surface at the foot or adjacent to the foot of the dam or may be located underground within one of the mountainsides/abutments. At narrow sites with favorable rock conditions and heads our 40 – 50 m, a concrete arch dam and an underground powerhouse often offer the most cost effective solution. With less favorable rock conditions a narrow site may not be the best dam location. In such cases wider sites allowing a surface design may offer an alternative, which give less total costs even with significantly possible higher dam costs. In suitable topography, which may allow a penstock and power house on the surface without excavation of excessive open cuts, surface powerhouse may give the cheapest solution even with excellent rock condition.

The low dam /log water way type: this type of powerhouse layout is characterized by a low dam where most of the head is created by a long water way releasing water down stream of natural rapids in the same river basin (Trans-basin diversion). Hydropower plants with heads of more than 250 m will have some of these characteristics.
A powerhouse located near the intake creates layout with a long tailrace tunnel in mountainous locations. This design requires long access tunnels and pumping of leakage water during excavation. On the other hand there is no much risk of losing water from a pressurized headrace tunnel. Location of the powerhouse in rock near the intake may be dictated by topography.

Figure 4.4: The development of the general layout of high power plant

4.4 The functions of the powerhouse complex

The overall function of the powerhouse complex of a hydropower plants is to transform the potential energy of water and head in to electric energy with the highest possible rate of efficiency under reliable and safe operational conditions.

i. Hydraulic system:
  - *Intake with trash rack*
  - *Rock trap and stone rack*
  - *Bypass arrangements and energy dissipaters*

ii. Generating system:
  - *Turbines*
  - *Switch gear*

iii. Auxiliary systems:
  - *Power house cranes*
  - *Cooling water system*
  - *Drainage system*
  - *Ventilation*

iv. Operational Aspects and Emergencies:
  - *Emergency chambers for fire risks, floods*

4.5 The power House complex

4.5.1 Main characteristics of underground Power plants

Flexibility in powerhouse Location and plant layout:

- *Selection of an Underground powerhouse implies* ***great flexibility*** *in the overall plant layout and location of the powerhouse itself. In principle the powerhouse may be placed anywhere along the water way, and the surface facilities at the tailrace out let will require only minor space. Consequently, the alignment of the waterway may be selected among several options, optimized to topography and geology of the project area and adjusted to suitable locations of powerhouse, tailrace outlet and adits. Plants of the long waterway type have the* ***largest degree of flexibility*** *as regards to overall layout and powerhouse location.*

Cost saving potential:

- *Comparing the general development layout plan in figure 4.4, the penstock and steel lining represent notable parts of the construction cost of headrace is stage 1 and 2. The pressure shaft of stage 2 is shorter than required for the penstock in stage1. In addition the thickness needed for a steel lining embedded in rock is less. The result is cost saving for stage 2 due to lower steel weight.*

Total plant Efficiency:

- *The steel parts are the most costly sections of the headrace. Optimum design gives higher specific losses in steel parts than in the waterway in general. A surface powerhouse implies longer steel parts than the underground alternative, thus, by applying equal optimization criteria for the two options, the underground plant will achieve the lesser total losses. Consequently, an underground development means higher total plant efficiency and therefore more effective utilization if the natural resources.*

Operational stability:

- *Due to long distance from the turbine to the surge chamber, surface plants may be unsuited for satisfying all technical criteria for stable operation. Plants with underground powerhouse are more stable than the surface plants.*

Deference, operational Reliability and personnel safety:

- *As strategic infrastructure, an underground powerhouse is less vulnerable under war like events than surface option and easier to prefect against sabotage .*

Structural Design:

- *Being completely fixed in its cavern, a powerhouse in the underground can be designed very efficiently from a structural point of view, as any need for overall support will easily be provided by the rock confinement. On the contrary, a powerhouse on the surface may, in order to achieve appropriate safety against sliding or uplift failure, require larger concrete volumes than what is needed for structural reasons alone.*
- *Steel lining embedded in rock will have similar advantage. All reactive forces from the pipe are transferred directly through the concrete surround to the rock. The rock will prevent any longitudinal movement of the lining and there is no need for expansion joints.*

Operation and Maintenance:

- *Embedded steel linings need less maintenance than exposal penstocks on the surface, which are subjected to deteriorating effects from changing temperature, sunshine, storm and rain and from frost and snow in clod climates. There will be a need to maintain external corrosion coating, expansion joints, erosion protection etc. While embedded linings will need maintenance of the inside coating only. Further, lining in rock will be shorter than surface options.*

Conditions for Construction and Erection:

- *Excavation for surface powerhouse will normally take less time than the access and caverns for underground option. Therefore, construction of an underground powerhouse will normally take longer time than surface alternatives. If the powerhouse including erection works is on the **critical path** for project implementation, the **construction schedule** may be decisive for the choice of alternatives. In cases with either very larger dams or long waterway, powerhouse works including erection on commissioning will not usually be on the critical path for implementation.*

Environmental impacts:

- *Whether located at surface or underground, the powerhouse itself will hardly cause serious environmental concerns. It may be assumed though, that an underground plant, occupying less surface area, will generally get higher environmental merits than a surface development. It causes less loss of forest or other valuable surface assets than a surface plant and gives no negative visual impact of a penstock on the hillside.*

4.6 Overall Plant layout

An underground hydropower plant will consist of:

i. headrace system with intake tunnel
ii. tailrace system with tailrace tunnel and outlet structure
iii. power house in one or more caverns with a system of tunnels serving various functions
iv. certain facilities on the surface

The location and alignment of the power plant will depend on the conditions of rock cover, rock type, access roads, construction adits, (Topography and geological conditions)

- Minimum need for heavy rock support
- Adjusting the vertical alignment to follow favorable strata of sedimentary rocks or locate the headrace as pressurized tunnel in igneous rock below weaker sedimentary rocks

The general design criterion, which has to be satisfied at any point in an unlined pressurized tunnel or shaft, is that the minimum principal stress in the adjacent rock mass is higher than the maximum future water pressure.

The embedded steel lining is the most expensive part of the headrace. Therefore, the penstock has to be designed as short as possible for the actual head in the geological formation. The next important task is to determine a suitable location and orientation of the powerhouse cavern in as short a distance as possible from the end of the unlined part of the headrace. The most important objectives are to ensure the stability of the powerhouse and adjacent tunnel system and avoiding leakage directly in to the power house. A **"Design as you go"** procedure with the possibility to adjust the relative positions of the two components after excavation has reached the powerhouse area is a recommendable approach, especially for low head or medium head development allowing rather short penstocks. At this point a detailed study should be made of on the system of joints around the powerhouse cavern with special focus on any risk of intersecting faults. Therefore, the location and final elevation of the end of the unlined tunnel can be adjusted to minimize the risk of short cut leakage in to the cavern. To further reduce risk of leakage along the penstock a fan shaped grouting curtain can be done to cover the cover the concrete surround at the penstock inlet and the adjacent rock mass.

The necessary length of the steel lining will depend on the head, the rock quality and the existence of crack systems and possible faults.

4.7 Powerhouse Tunnel system

The powerhouse needs a tunnel system to serve various needs, which can be divided in to two sets of requirements, ***one set, related to the period of construction*** and the other ***for the future operation of the power plant.***

During construction the tunnel system will have to serve as access for excavation of all parts of the Power House, for transport of excavated material and ventilation, supply of electricity, water, compressed air and other support from the outside for performance of the civil works and erection. Further, the main access tunnel has to be designed for the largest electromechanical components to be transported in to the power house.

After commissioning, the tunnel system around the powerhouse will need to serve a different set of functions, the main ones being:

- Main access in to the powerhouse
- Branch-off tunnels to other installations like transformer cells or separate transformer cavern , tailrace gate chamber, concrete plug with steel bulkhead , etc,
- Alternative emergency exit from the power house to the surface
- Routing of high voltage cables from the generators to the transformers and from the transformers to the switchyard on the surface
- Routing of signal and control cables
- Supply and evacuation of air for ventilation
- Tailrace surge chamber, etc

Multipurpose aspects of tunnels: to minimize the total cost, multipurpose functions of the tunnel layout should be a ***main design principle***. Every tunnel and shaft may serve several and different functions during the period of construction and after commissioning.

Example:

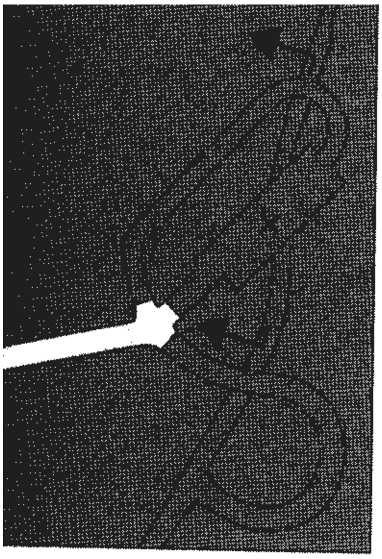
- *45^0 inclined shaft serving for the high–voltage cable connection , for supply of ventilation air and as alternative escape route*
- *Combined cavern for the main transformers and machinery for tailrace gates and with the same cavern as the starting point for a tunnel loop for excavation of the top heading of the powerhouse cavern*
- *transport access tunnel to the tailrace latter serve as surge chamber*

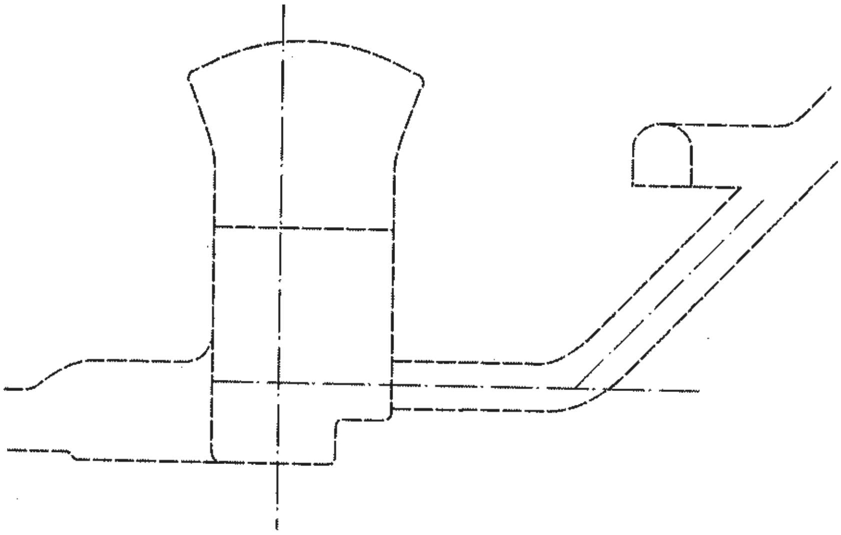
4.8 Excavation Equipment and construction procedures

It is important to recognize that excavating a tunnel sloping downwards means little inconvenience. The main one is the continuous need for pumping of water from the working face. Therefore, instead of a constant downward slope to overcome a moderate difference in elevation, it is a recommended design to start on a minimum upward slope from the entrance. In this way self-drainage is achieved for the tunnel stretch near the surface. The difference in elevation is overcome by a concentrated steeper downward slope toward the end of the tunnel

Steeper tunnel slope reduce total tunnel lengths. At least 2% should be selected for effective self drainage.

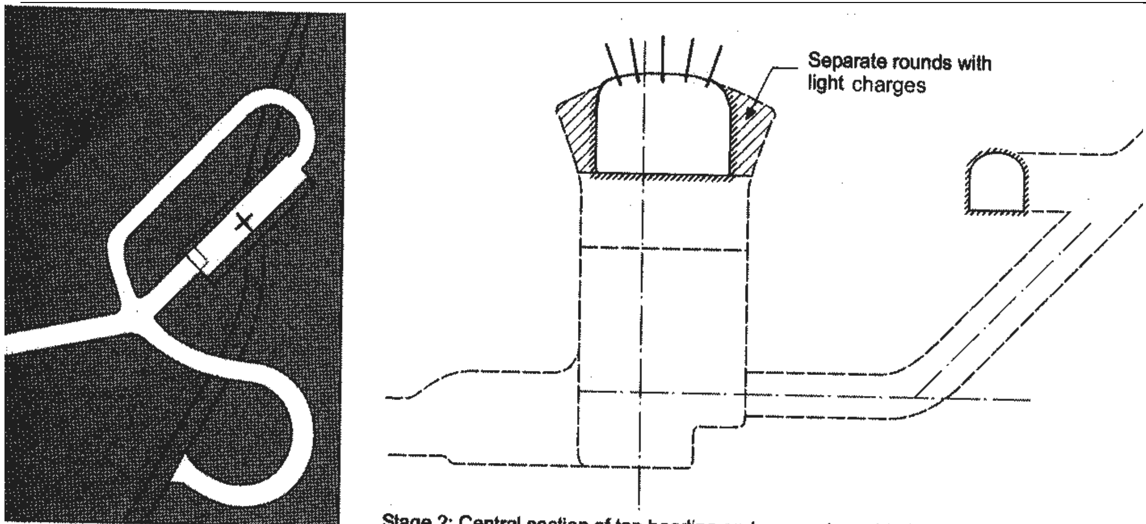
Figure 4.5: Plan and cross section of an underground Hydropower plants with unlined waterways (Multipurpose tunnel layout)



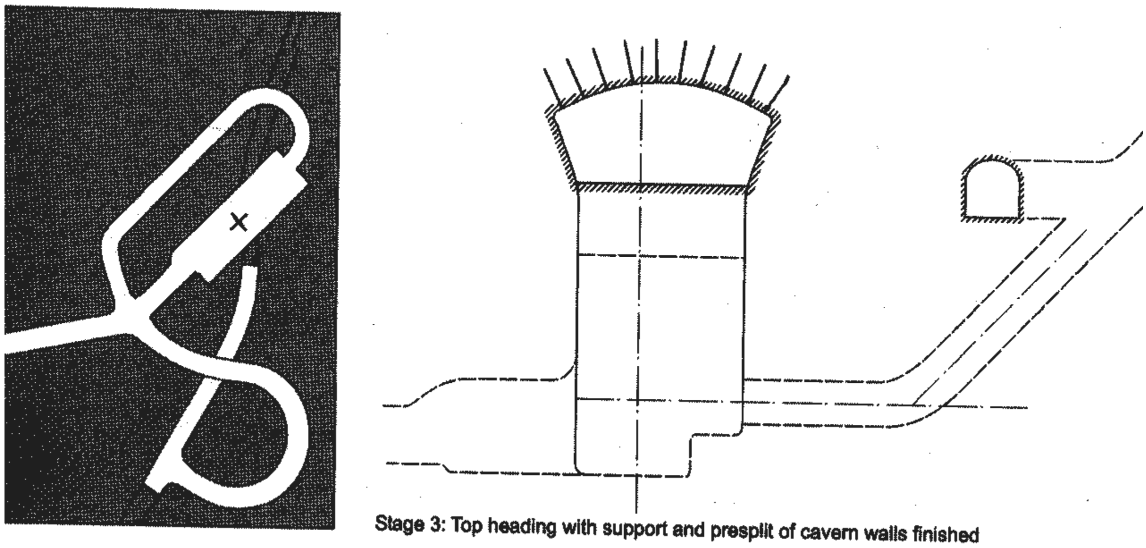


Stage 1: Main access tunnel excavated to powerhouse area

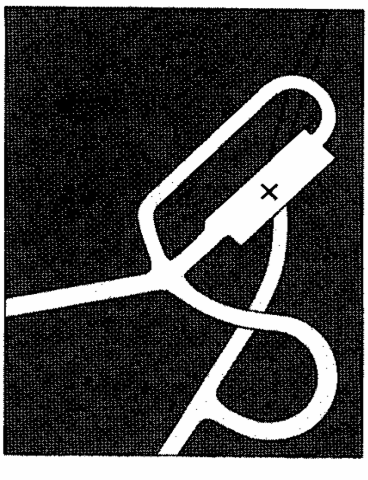
A typical sequence of excavation, concrete works and erection for an underground powerhouse with one Pelton unit is presented in the figure below.

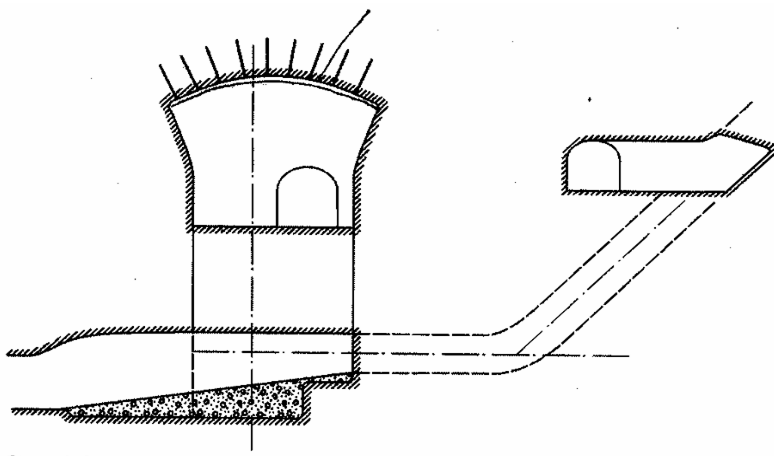


Stage 2: Central section of top heading and access tunnel to tailrace excavated

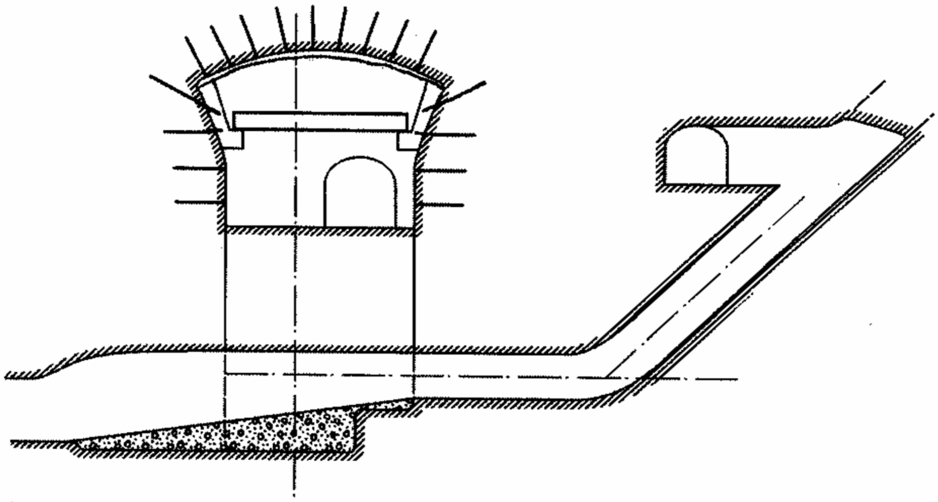


Stage 3: Top heading with support and presplit of cavern walls finished

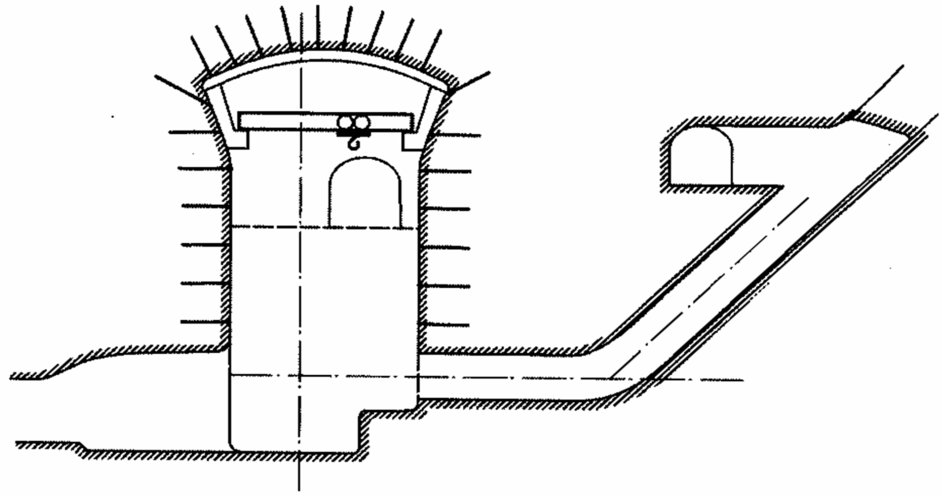




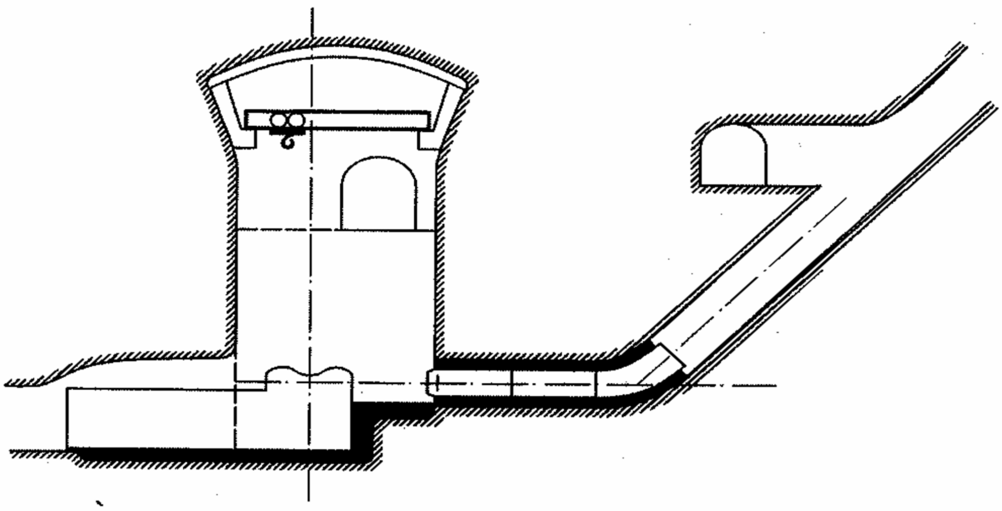
Stage 4: Ceiling support and excavation to machine hall level finished.
Turbine pits excavated



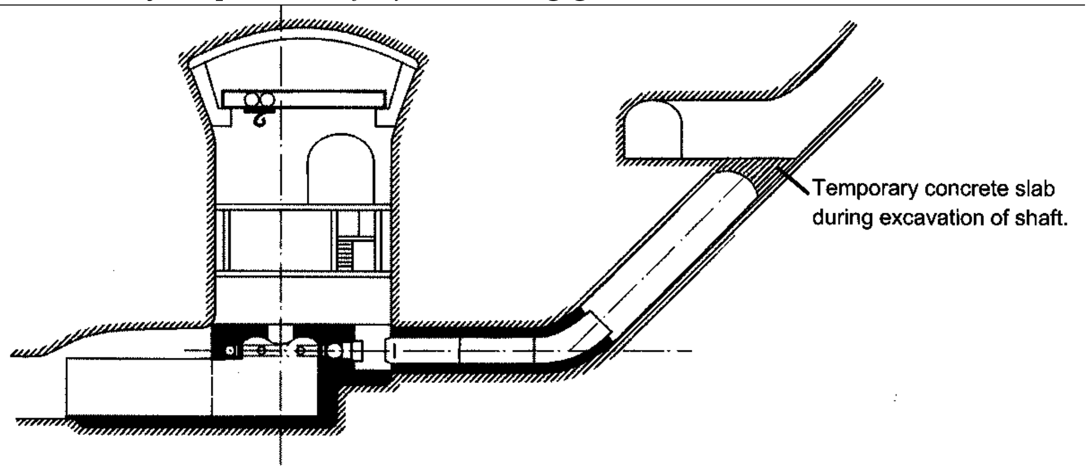
Stage 5: Lower part of pressure shaft excavated. Preliminary erection of crane bridge



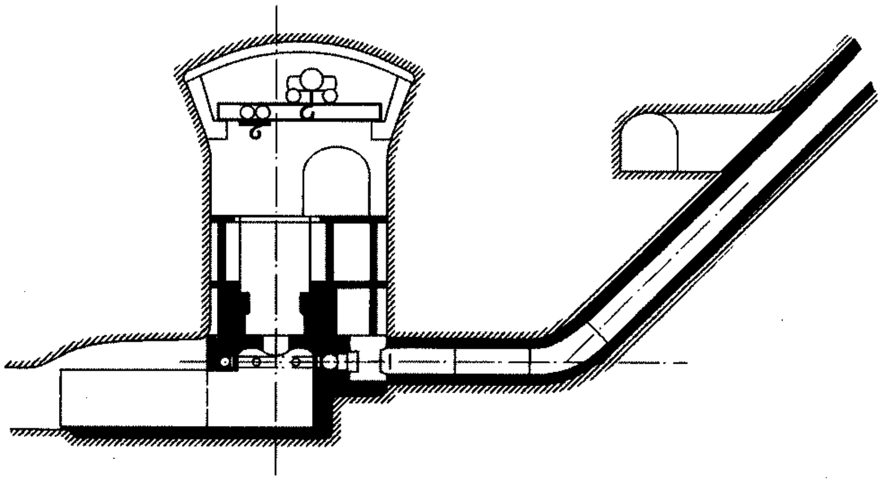
Stage 6: Excavation and support finished.
Erection of auxiliary hoist



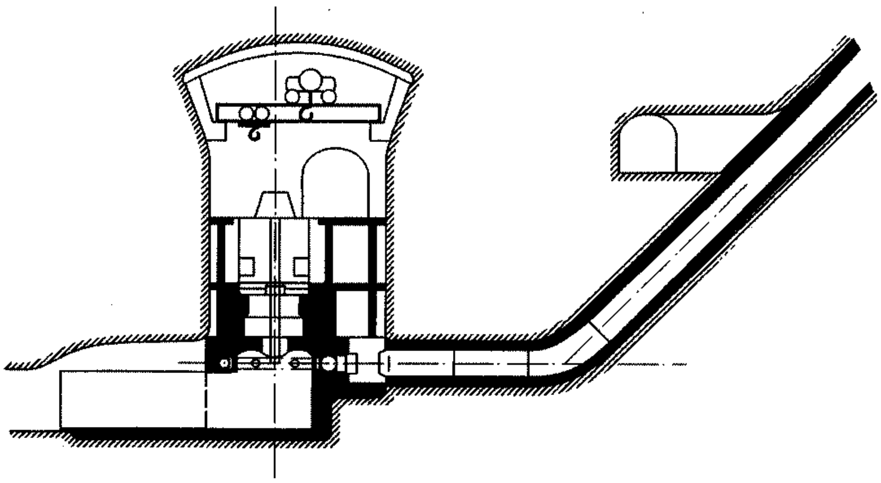
Stage 7: Erection and concreting of steel casing in turbine pit
and concluding part of penstock



Stage 8: **Erection and concreting distributor.**
Concrete adjacent part of substructure



Stage 9: Concrete structure finished.
Main hoist erected. Erection of steel lining ongoing



Stage 10: Erection of generating unit and steel lining completed

Figure 4.6: Stages of excavation in underground power stations.

4.9 Transformer Arrangements and Locations

The transformer location will greatly influence the arrangement of an underground powerhouse. Due to the cost of high amperage connection between generators and transformers, transformers location at surface may only be economically viable by the shallowest seated power house. Aspects of maintenance, operation and deference all favour transformers underground, hence the transformers are located within the machine hall or in an adjacent separate cavern.

- In a machine hall
- Between generators
- In extension of power house cavern
- In separate cavern

4.10 Hydropower Tunnels

Tunnels are underground conveyance structures constructed by special tunneling methods without disturbing the natural surface of the ground. In many modern high head plants, tunnels form an important engineering feature.

Tunnels have the advantages of:

i. Providing direct and short route for the water passage thus resulting in considerable saving in cost
ii. Quicker completion due to simultaneous tunneling work at many points
iii. Protection of natural land scape

Tunnels of hydropower projects fall into two categories: *water carrying tunnels* and *service tunnels*.

a) ***Water carrying tunnels***: these include head race or power tunnels, tail race tunnels or diversion tunnels. Flows in water tunnels are usually under pressure (pipe flow), but sometimes free-flow (open channel flow) can be experienced, especially, in tailrace tunnels. The design of free-flow tunnels follows the same principles as used in the design of open canals.

- *Head race tunnels*: are tunnels that convey water to the surge tank. These are pressure tunnels
- *Tail race tunnels*: could be free flowing or pressure tunnels depending on the relative position of turbine setting and tail water level.
- *Diversion tunnels*: are constructed for the purpose of diverting the stream flow during construction period. Normally they are not of high pressure but should have sufficient flood carrying capacity. Such tunnels either plugged with concrete or converted in to some use such as spillway tunnel at the completion of the project.

b) ***Service tunnels****:* these may be:

- *Cable tunnels*: to carry cables from underground power house to the switch yard
- *Ventilation tunnels*: fitted with fans at the open end to supply fresh air to the underground
- *Access or approach tunnels*: these are passage tunnels from surface to underground power house.

Shape: Tunnels are either circular or non-circular in shape.

Circular tunnels: are most suitable structurally. They are more stable when the internal pressure is very high.

Non-circular tunnels: have a flat floor, nearly vertical or gently flaring walls and arching roofs. The horse-shoe shape is the most popular and convenient from construction point of view.

Commonly adopted shapes:

a) Circular shape b) Horseshoe shape

Figure 4.6: Tunnel shapes

Alignment: Tunnels have generally small bottom slopes, i.e. tunnels or aligned nearly horizontal. Shaft is a tunnel with vertical alignment or inclined shaft when it is steeply inclined to the horizontal. It is very crucial to investigate in detail the geology of the strata through which a tunnel would be passing. Sound, homogenous, isotropic, and solid rock formations are the most ideal ones for tunneling work. However, such ideal conditions are rarely present, and rather the rock mass exhibits various peculiarities. There may be folds, faults, joint planes dipping in a particular direction, weak strata alternating with good strata, etc. Thus, the alignment of a tunnel should be fixed keeping in view these phenomena. The alignment, for instance, should as far as possible avoid major fracture planes.

***Lining**:* Lining is a protective layer of concrete, R.C. or steel on the inner surface of the tunnel to improve the efficiency and structural stability of the tunnel. Tunnels in good, sound rock may be left unlined.

Lining of tunnels is required:

i. For structural reasons to resist external forces particularly when the tunnel is empty and when the strata is of very low strength.
ii. When the internal pressure is high, i.e. above 100m
iii. When reduction in frictional resistance and therefore the head loss is required for increasing capacity
iv. For prevention or reduction of seepage losses
v. For protection of rock against aggressive water

In the case of low-pressure tunnels the tunnel surface may frequently be left unlined except for visible fissures. A watertight lining is usually required for tunnels operating under medium and high heads. Seepage is more likely to occur as the head increases, water may leak through the smallest fissures and cracks. Moreover, under high-pressure it may penetrate the otherwise watertight rock and render it permeable.

Tunnel Design Features: **t**he design of hydropower tunnels include *alignment; choosing the appropriate geometric shape, longitudinal slope and flow velocity; computation of head loss, rock cover (overburden), lining requirements and economic x-section* come in to play.

Alignment: in aligning water tunnels, the following points should be taken in to account:

- Length of the tunnel: as much as possible short route should be followed
- Location of surge tanks & adits: the alignment should provide convenient points for surge tanks & adits.
- Rock cover (overburden): sufficient rock cover should be available along the alignment
- Discontinuities: the alignment should, if possible, avoid crossing of weakness zones, joint planes, etc. If crossing of these features is unavoidable, suitable direction of crossing should be considered.
- Rock quality: good quality of rock mass should be sought in aligning the tunnel

Geometrical Shape:

- The choice of the cross-sectional profile of a tunnel depends on:

- Hydraulic considerations: Circular is preferable
- Stability considerations: Circular is preferable
- Convenience for construction: D-shaped is preferable
- Available tunneling equipment :

Longitudinal Slope: the minimum slope for a pressure tunnel is limited on the basis of dewatering requirements. And also the longitudinal profile of the tunnel should be such that the roof remains below the hydraulic pressure line by 1 to 2 m. Likewise, the tunneling method and the equipment employed for transportation of the excavated material (rail or wheel transport) can limit the maximum slope possible to provide. The usual practice is to keep the slope of power tunnel gentle till the surge tank and then steeper (even vertical) for the pressure shaft.

Flow Velocity: the allowable velocities in tunnels depend upon whether it is lined or unlined. In unlined tunnels, a velocity of 2 to 2.5 m/s is the upper limit, while in concrete lined tunnels 4 to 5 m/s is often employed. The velocities for the pressure shafts, which are generally steel lined, are usually higher than that in the power tunnel. The normal range of velocities is between 5 to 8 m/s.

Rock Cover (overburden):for pressure tunnels, it is obvious that the overburden on the roof of the tunnel serves to balance the effect of upward force due to internal pressure. The required depth of overburden may vary for lined and unlined tunnels.

In the case of ***unlined tunnels***, the entire internal water pressure is resisted by the overburden rock pressure. Where a steep valley side constitutes the overburden above the tunnel, the rule of thumb equation, H= (0.4 to 0.8)*hr has to be modified and given by:

$$h_w = \frac{1}{\eta}\frac{\gamma_r}{\gamma_w} L\cos\beta$$

Where L is the shortest distance between the ground surface and the studied point of the tunnel (or shaft) and β is the average inclination of the valley side with the horizontal *(see figure below).*

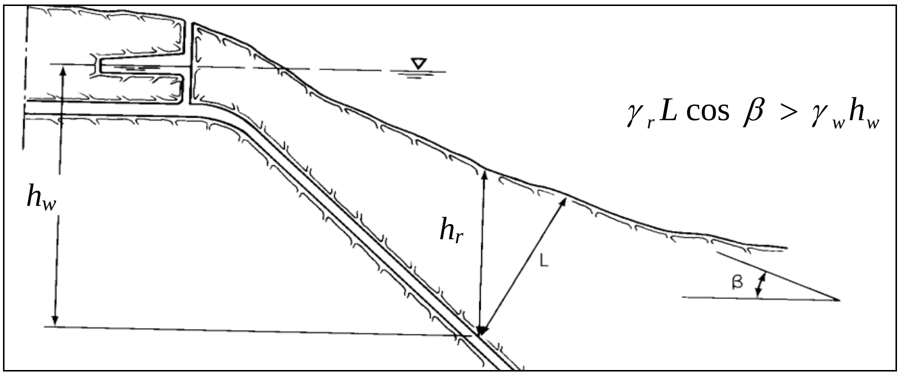


Figure 4.7: Overburden requirement in a steep valley side

In the case of concrete or steel ***lined tunnels***, the linings are usually designed to carry part or full load of the internal water pressure, and the above equations, given for unlined tunnels, are modified accordingly in determining the required overburden depth.

Head Loss: head losses in tunnels can be computed using Manning's, Darcy-Weisbach, or Hazen-Williams formulas.

Manning formula: $$h_f = n^2 \frac{lv^2}{R^{4/3}}$$

Darcy-Weisbach formula: $$h_f = \lambda \frac{l\,v^2}{2g\,D_{eq}}$$

Hazen-Williams formula (rarely used): $$h_f = 6.84 \frac{l\,v^{1.85}}{C^{1.85}\,D_{eq}^{\;1.17}}$$

Where, h_f is head loss due to friction, L is tunnel length, V is mean velocity of flow, R is hydraulic radius, D_{eq} is equivalent diameter ($D_{eq} = \sqrt{4A/\pi}$), A is area of the tunnel x-section, n is Manning's roughness coefficient, λ is Darcy-Weisbach friction factor (can be obtained from Moody diagram), and C is Hazen-Williams roughness coefficient.

Optimum X-section: the optimum x-section of a tunnel or a shaft is one for which the sum of tunnel construction cost and the economic loss due to head loss is minimum.

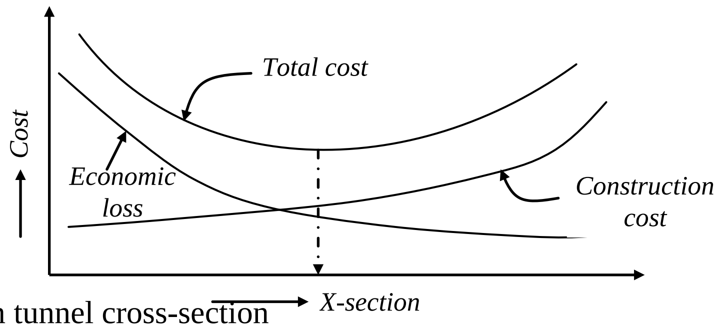


Figure 4.8: Optimum tunnel cross-section

For a quick initial estimate of the diameter of pressure tunnels, the empirical formula suggested by *Fahlbusch* can be used:

For concrete-lined tunnels: $D = 0.62\,Q^{0.48}$

For steel-lined tunnels: $D = 1.12\,\dfrac{Q^{0.45}}{H^{0.12}}$

4.11 Rock Stresses

Most stress related problems are caused by stresses which are greater than the critical limit for the rock mass in question. However, the problem may also be caused by too low stresses. Generally, the stresses surrounding underground openings are defined are:

1. the stress situation prior to excavation (the "Virgin" stresses)
2. the geometry of the opening

When the virgin stresses and the geometry are known, it is in theory easy to evaluate the magnitude and the direction of the stresses surrounding a tunnel or rock cavern. If the rock mass properties are known, it is theoretically also relatively simple to analyze potential stability and leakage problems caused by stresses, the need for rock support, the possibilities for optimizing the excavation geometry, etc.

Nevertheless, empirical methods are mainly used for analyzing rock stress problems. The reason for this is primarily the problem in obtaining reliable input parameters which are needed for the more advanced analyses. The following parameters have been the most difficult to quantify.

- the magnitudes and directions of the virgin rock stresses
- the properties of the in-situ rock mass, in particular the elasticity parameters
- the failure criterion of the in-situ rock mass

Origin of Rock Stresses: the virgin stress generally represents the resultant of the following components:

- Gravitational stresses
- Topographic stresses
- Tectonic stresses
- Residual stresses

Gravitational Stresses: this component is a result of the gravity alone. When the surface is horizontal, the vertical gravitational stress at a depth Z is:

$$\sigma = \rho.g..Z$$

When **ρ.g =** specific gravity of the rock

The magnitude of the total vertical stress is often identical to the magnitude of the gravitational vertical component. However, at great depths, particularly, there are considerable deviations from this trend.

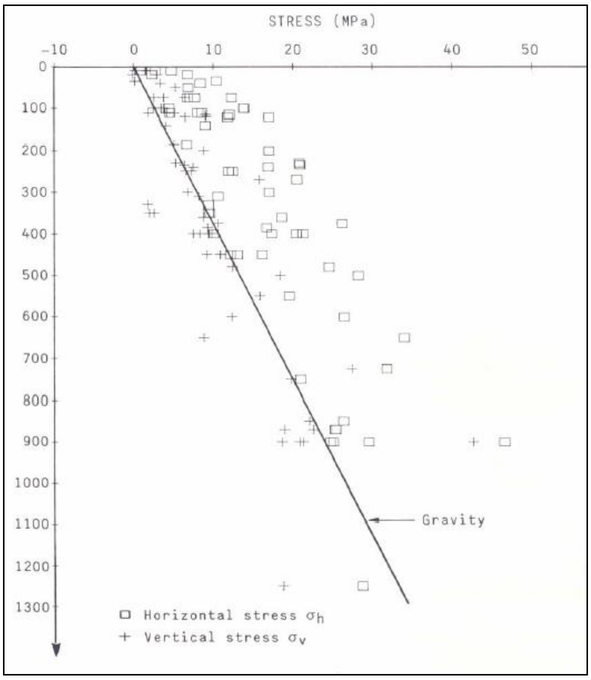


Figure 4.9: Vertical and horizontal rock stresses as a function of depth below the rock surface (Scandinavians Case)

In anelastic rock mass with Poisson's ratio of *v*, the horizontal stresses induced by gravity are:

$$\sigma_x = \sigma_y = \frac{\nu}{1-\nu}\sigma_Z$$

For a Poisson's ratio of 0.25, which is fairly common for rock masses, this means that the horizontal stress induced by gravity is approximately 1/3 of the vertical stress.

Topographic stresses: when the surface is not horizontal, the topography will affect the rock stress situation. Stresses caused by topographic effects are generally referred to as topographically induced stresses or simply topographic stresses.

In high valley sides, where hydropower plants are often located, the stress situation is totally dominated by the topographic effects. In such cases the major principal stress (**σ_1**) near the surface will be more or les parallel to the slope of the valley, and the minor principal stress (**σ_3**) will be approximately perpendicular to the slope.

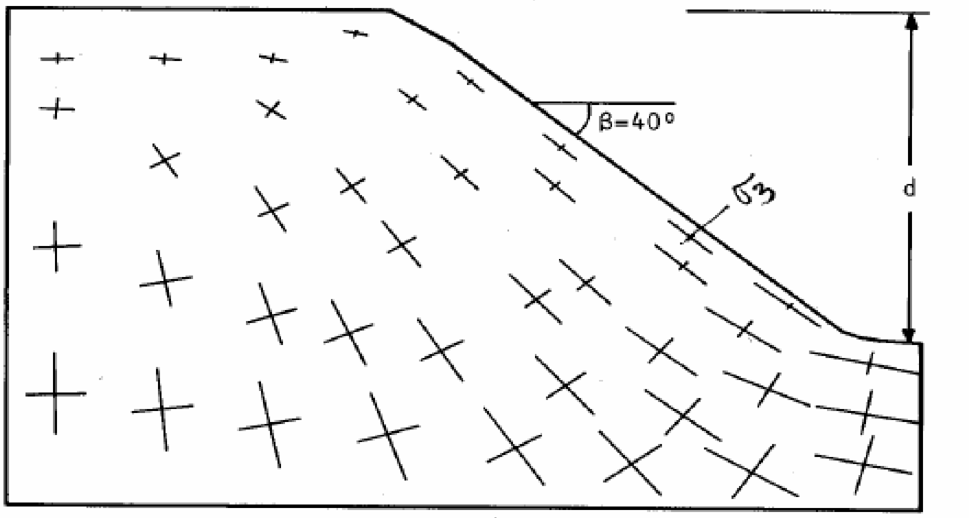


Figure 4.10: Magnitudes and directions of directions of the major and minor principal stresses in a valley side as computed by a Finite Element Analysis

Tectonic Stresses: evidence of tectonic stresses is represented by dramatic incidents like faulting and folding and also by movements such as heave of the Scandinavian Peninsula after the last glaciations. The main cause of faulting and folding as well as tectonic stress is the action of plate tectonics; drifting and tectonic activity along the margins of some 20 rigid plates that constitute the earth's outer shell.

Because of tectonic stresses the total horizontal stress is often much higher than the horizontal stress which is induced by gravity alone. This is particularly the case at shallow and moderate depths.

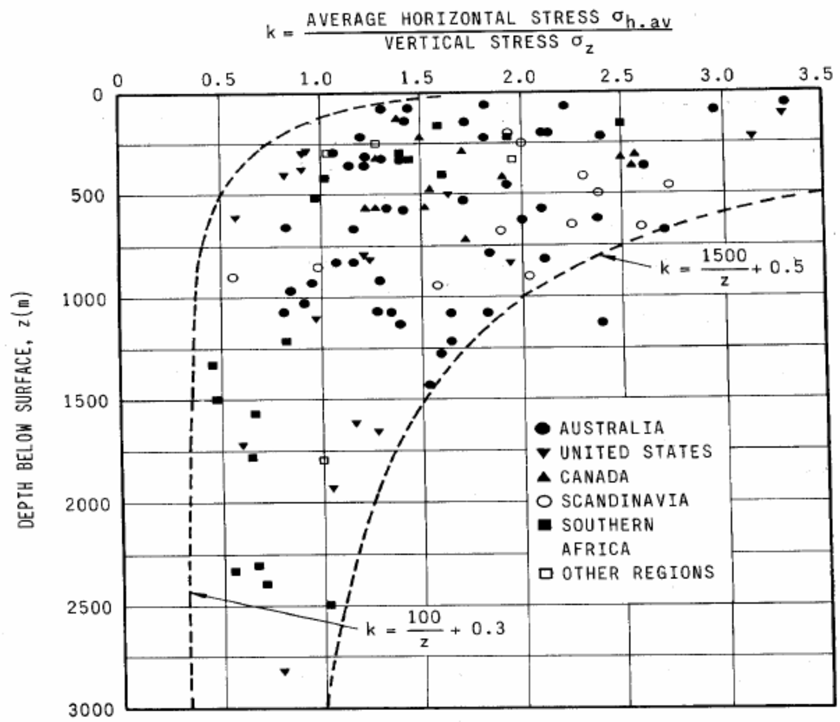


Figure 4.11: Variation of ratio of average horizontal stress to vertical stress with depth below surface

Residual Stresses: residual (or remnant) stresses are generally defined as stress which has been locked in to rock material during earlier stages of its geological history. Stress caused by contraction during cooling of a rock melt (magma) is probably the most relevant example of this category. Vertical stresses which are abnormally high are often explained as being caused by residual stress.Stresses Surrounding Underground Openings

When analyzing potential problems due to rock stresses, the stress situation close to the contour of the tunnel or rock cavern is of particular interest.

- Stresses surrounding Circular opening

The simplest case is represented by the following idealized conditions:

- Homogeneous and isotropic, elastic material
- Isotropic virgin stresses (**$\sigma_1 = \sigma_2 = \sigma_3 = \sigma$**)

If the radius of the opening is a, the radial and tangential stresses of a cross section (σ_r and σ_t, respectively) will be the following as function of the distance **r** from the circle center.

$$\sigma_r = \sigma(1 - \frac{a^2}{r^2})$$

$$\sigma_t = \sigma(1 + \frac{a^2}{r^2})$$

In the figure below these equations are shown graphically. It is particularly important to notice the rapid increase in tangential stress close to the contour. Generally, in a case like this, a tangential stress with a magnitude of twice the magnitude of the isostatic stress will be induced all around the periphery.

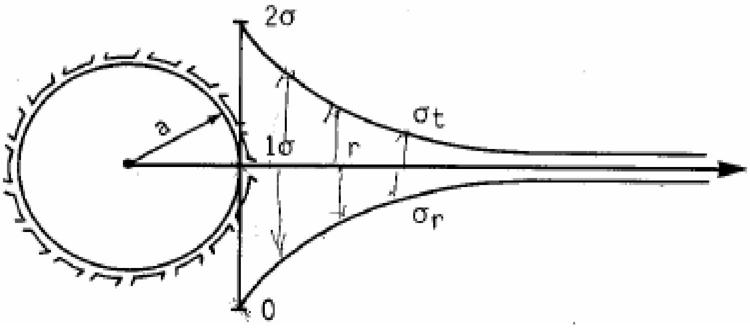


Figure 4.12: Tangential and radial stress surrounding a circular opening in isostatic stress field.

For an anisotropic stress condition the so-called Kirsch's equations are used for evaluating the tangential stresses. According to Kirsch the tangential stress will reach the maximum value ($\sigma_{t(max)}$) where the σ_1 direction is tangent to the contour, and its minimum value ($\sigma_{t(min)}$) where the σ_3 direction is tangent. The actual values will be:

$$\sigma_{t(\max)} = 3\sigma_1 - \sigma_3$$

$$\sigma_{t(\min)} = 3\sigma_3 - \sigma_1$$

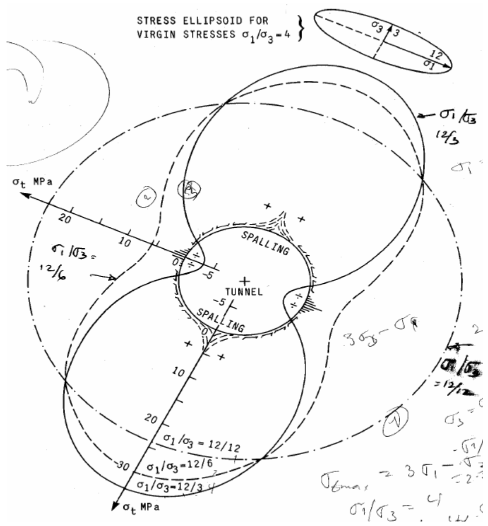


Figure 4.13: The influence of rock anisotropy on the stress surrounding of a circular opening. Potential problems due to the stresses are indicated

The above figure illustrates that the distribution of tangential stress is strongly influenced by the degree of stress anisotropy. If the stresses are very anisotropic the minimum tangential stress, as illustrated, may even be negative, i.e. tensional

4.11.1 Stresses near Corners

Non symmetrical geometry and sharp corners in particular, will strongly affect the magnitude of the tangential stress as described by Jaeger & Cook and others.

When the curvature radius is reduced, the magnitude of the tangential stress will increase. This means, for instance, that the sharper the corner between the wall and the roof of a cavern, the higher the stress concentration will be in that corner. In extreme cases such stress concentration may reach magnitudes of more than 10 times the major principal stress value

In cases with benches or protruding corners the stress situation will be the opposite. Here the stabilizing stresses, or the confinement, will be reduced, and stability problems will often result.

Influence of the Rock properties:

In a TBM-bored tunnel or in a carefully blasted tunnel the tangential stress will have a distinct maximum at the tunnel contour as illustrated figure below. As a result of blasting damage to the rock, however, the situation in most drill and blast tunnel will be very different from this idealized picture. In such case the extra joining close to the contour caused by blasting reduces the capability of the rock mass to transfer stress, and the distribution of tangential stress will in principle be as shown in the figure with a maximum value some distance from the contour.

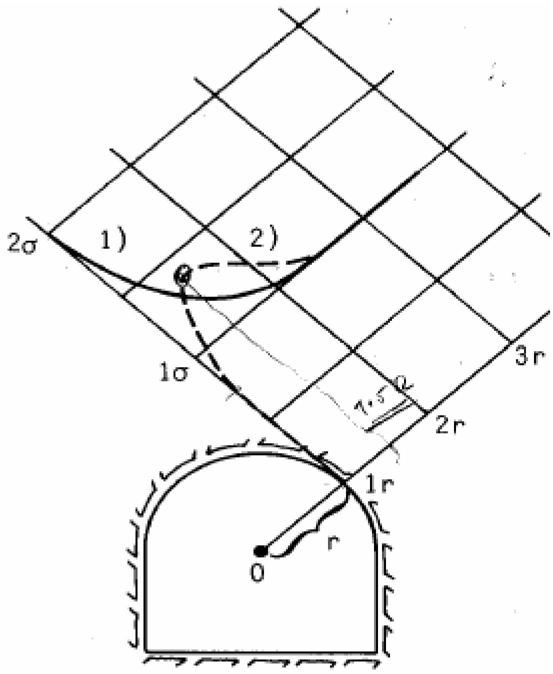


Figure 4.14: Principle sketch illustrating the concentration of tangential stress in a tunnel when:

1) the contour rock is undisturbed
2) it is fractured as a result of blasting damage (the virgin stress is assumed isostatical, $\boldsymbol{\sigma_1 = \sigma_2 = \sigma_3 = \sigma}$)

The detailed distribution of the tangential stress will depend mainly on the deformation properties of the rock mass. In very jointed rock the stress peak is relatively flat, and the maximum stress value is located relatively far from the tunnel contour. This is also the case in typical soft rocks. In hard and elastic rocks the stress peak is much steeper, and the maximum stress value is located close to the contour.

The magnitude of the maximum tangential stress depends in theory on the shape of the underground opening, and not on its size. The zone of influence however will increase with increasing size. In-situ rock stress measurements indicate that the stresses stabilize at a constant level at a distance from the tunnel contour corresponding to approximately half the tunnel width. The constant level corresponds to the actual virgin stress.

Stability problems Due to stresses:

In the contour of an underground opening, there are normally two diametrically opposed areas of tangential stress concentration and two areas of minimum tangential stress as illustrated in Figure 8 when rock stresses are causing problems. The problems are normally confined to the areas of maximum tangential stress. However, if the minimum tangential stress is very low, this may also be a problem.

Problems Due to Tensile stress:

Due to its discontinuous character, a rock mass can resist little tensile stress. Hence even a very small tangential tensile stress may cause radial jointing as indicated in figure 8.

In most cases a tensile jointing will not have much influence on the rock stability. For high pressure tunnel it is more important that secondary jointing and opening of existing joints may increase the risk of water leakages out of the tunnel.

Problems Due to High Compressive Stress:

If the compressive tangential stress exceeds the strength of the rock, fracturing parallel to the tunnel contour will be the result in hard rock as shown in figure 8. The situation has a certain similarity of fracturing in point load testing, in which the fracture is also induced by a compressive stress in the direction of fracturing.

The fracturing process is often accompanied by loud noises from the rock. A phenomenon commonly referred to as rock burst. At moderate stress level the fracturing will result in a loosening of thin rock slabs, often referred to as rock slabbing or spalling. If the tangential stress is very high, the rock burst activity may be quite dramatic. In extreme cases it may have the character of popping of large rock slabs with considerable force and speed.

When the stresses are very high, rock bursts may be a major thereat to safety if the right type of rock support is not installed at the right moment. In such cases extensive rock support is necessary.

Rock burst activity is most intensive at the working face immediately after excavation. Experience shows that the most difficult area is the section 10-20 m closest to the working face.

In soft rocks the stress problems will not be characterized by spalling. Because of the plastic nature of such rocks the potential problem here will be squeezing. In extreme cases reductions of the original tunnel diameter of several tens of centimeters due to squeezing have occurred in Central Europe.

The Influence of Rock Mass properties:

The character of the rock stress problem will largely depend on the rock mass properties. Important aspects such as primary jointing and strength properties have already been discussed. However, anisotropy and elastic properties may also be influential.

The orientation of the major principal stress relative to the direction of major joints sets and important structural features, such as bedding and schistosity, will have a major influence on rock burst activity. Severe problems may occur if the schistosity runs parallel to the tunnel axis, and the major principal stress acts perpendicular to the axis and in the dip direction of the schistosity.

Along a tunnel there will be a certain variation in stresses, rock type and elastic properties, and therefore also a variation in rock burst activity. Generally, there will be a concentration of stresses in stiff rocks and considerably lower stress in softer rocks. In gneisses, for instance it is commonly experienced that tunnel sections particularly rich in mica are often characterized by stress relief, while the rock burst is confined to more quartz and feldspar rich sections.

Major weakness zones may also affect the rock stress situation. As many such zones are only able to transform shear stress to a minor extent, the principal stresses will often be parallel and perpendicular to the zones. Hence a tunnel through a major weakness zone may experience extensive rock spalling on one side of the zone, while the stresses are reduced to a moderate or low level on the other side

4.11.2 Norwegian Experience, Examples

An old Norwegian rule of thumb states that if heights above the tunnel of 500m or more are reached at an angle of 25^0 or steeper in a valley side, one should always be prepared for stress induced stability problems. Although this simple rule does not consider the influence of for instance tectonic stress, it still reflects general experience from the majority of Norwegian hydropower plants.

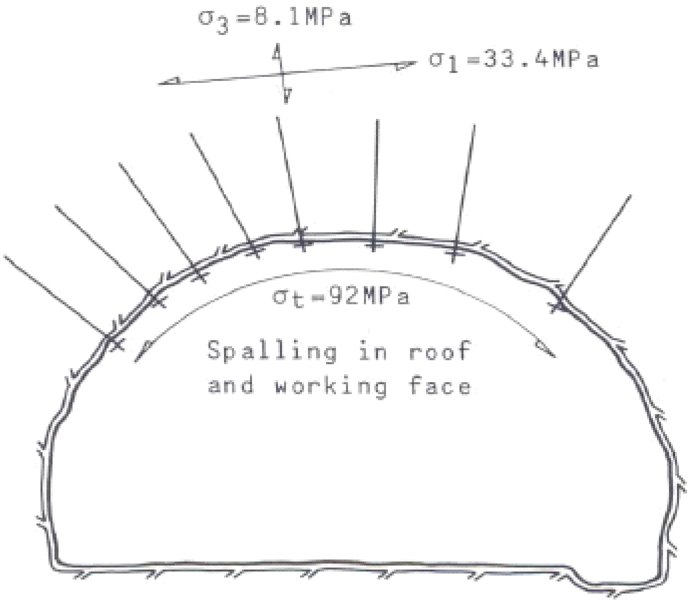


Figure 4.15: The Consequence of high horizontal stress and an isotropic condition.

As an example, the major principal stress in figure 10 is approximately horizontal and perpendicular to tunnel axis. The vertical stress (σ_3) is only one quarter of the value of the major principal stress. And according to Kirsch's equation, the tangential stress is therefore 2.75 times the maximum horizontal stress.

In water tunnels it has occasionally been experienced that spalling may continue for many years. This long-term effect is probably caused by a combination of high stresses, a reduction of rock strength due to water saturation, creep effects and hydraulic pressure variations.

4.11.3 Rock Stress Measurements

To be able to analyze the potential problems due to rock stresses, it is necessary to obtain information about magnitudes and directions of the principal stresses. Reliable information on this issue can be obtained only by carrying out rock stress measurements.

Methods:

Throughout the years a considerable variety of different equipment for in-situ rock stress measurements has been developed. However, for hydropower projects the following methods are most relevant.

- Triaxial stress measurements by drill hole over coring
- Hydraulic fracturing

The drill hole over coring technique has the longest tradition, and there are several versions of this method. Figure 11 illustrates the principles of the version which is most commonly used. As can be seen, what is actually being recorded are the strains. To be able to compute the stresses, laboratory analyses of the elastic properties have to be carried out.

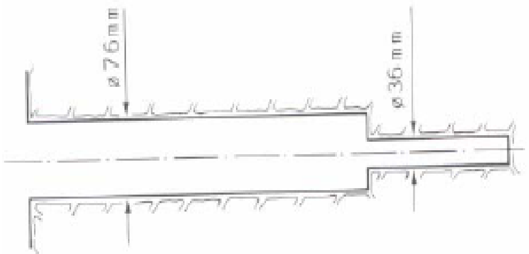
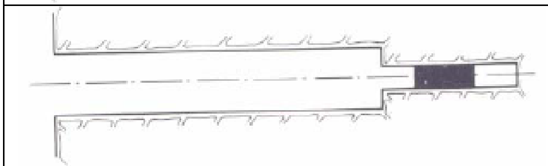
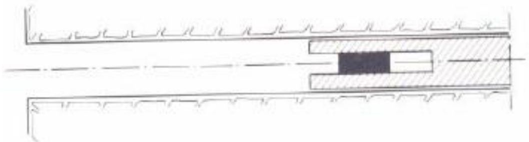
	1. A diamond drill hole is drilled to the required depth. A concentric hole with a smaller diameter is drilled approximately 30 cm further.
	2. A measuring cell containing three strain rosettes is inserted, and the rosettes are glued to the walls of the small hole.
	3. The Small hole is over cored by the larger diameter bit, thus stress relieving the core. The Corresponding strains are recorded by the rosettes. When the elastic constants are known, the triaxial state of stress can be computed.

Figure 4.16: The principle of three dimensional rock stress measurements by overcoming

The basic principle of hydraulic fracturing is to isolate a section of a drill hole and, by gradually increasing the pressure of water which is pumped into the hole, to obtain fracturing of the surrounding rock. By recording water pressure and flow, the principle stress situation can be evaluated.

4.11.4 Modeling

For analyzing rock stresses and deformation, numerical models may be a valuable tool .Because of the large quantities of data involved in such analyses, powerful computers have to be used.

Principles:

Generally, there are two main categories of numerical models:

- Discontinuous
- Continuous

In discontinuous models (or" block models") the rock mass is modeled as a system of single blocks which interact along their edges. One fairly well known example of a method belonging to this category is the Universal Distinct Element Code (UDEC). Obviously; the concept of discontinuity represents certain advantages. However, as this is quite a new category of numerical models, it has had little application up to now.

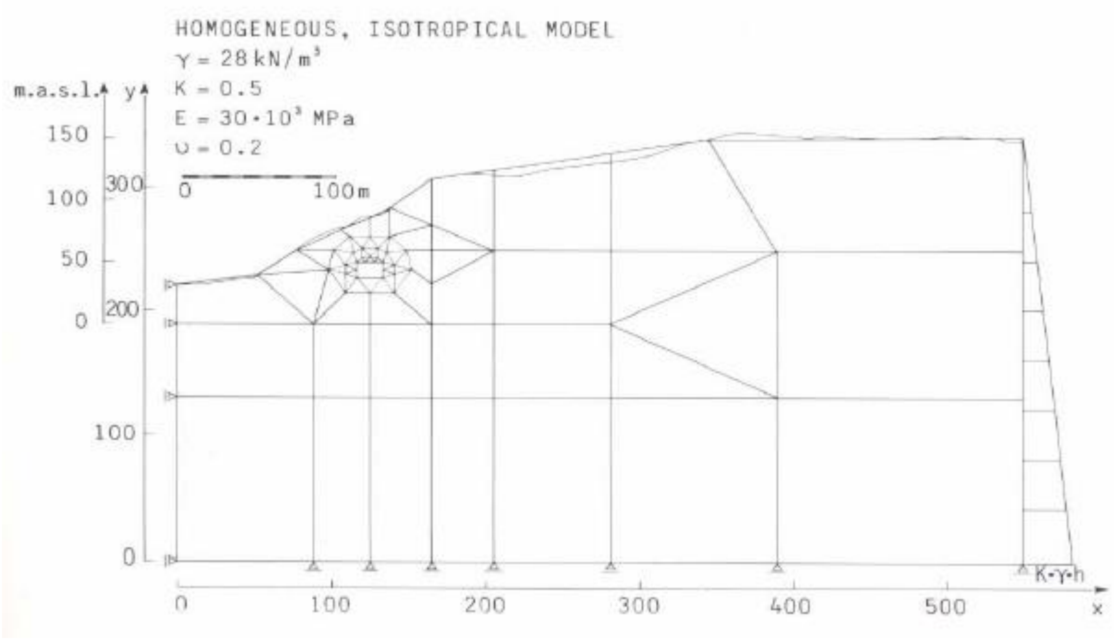
Continuous models, on the other hand, have been used on many occasions, in this model category the rock mass is modeled as a basically continuous medium. Limited number of discontinuities (joints .faults, etc) may also be included. Common methods in this category are the Finite Element Method (FEM) and the Boundary Element method ("BEM")

The initial step of FEM-analyses is to define a geological model of the actual area. The next step is to generate the element mesh. Based on in put of rock properties and boundary conditions the magnitudes and directions of stresses for all nodal points of the element mesh are finally competed.

Examples:

The basic principle of a Simple FEM- model (homogeneous and isotropic conditions) is shown in Figure 12. In this model the size of the elements becomes generally smaller close to the contour of the rock cavern which is being modeled. This is simply because this is the area of prime interest when analyzing stability and planning rock support. A special feature of this model is the possibility of excavation elements in the roof, thus permitting analyses for a cavern with a curved roof as well as one with a flat roof.

The relevant mechanical parameters are given, i.e. specific gravity ((γ), modulus of elasticity (E) and position's ratio (V) need to be known. The nodal points at the bottom of the model are free to move horizontally only, while the nodal points at the left hand side are free to move vertically only.



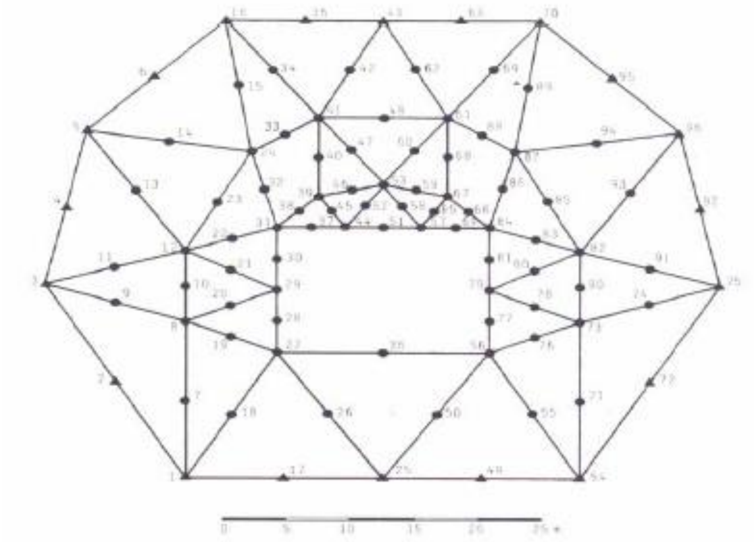


Figure 4.17: Example of Finite Element model for analyzing the stresses surrounding a planned rock cavern.

Vertically the model is loaded with gravity forces; γ.h. Horizontal load is applied on the right hand side of the model, and is given as k.γ.h. This is both force resulting from elastic deformation and tectonic force.

The computed directions and magnitudes of principal stresses from such model analysis are as illustrated in Figure 13. Here, the magnitudes and directions of the major and minor principal stresses are given by the vector lengths and directions, respectively, of each of the crosses.

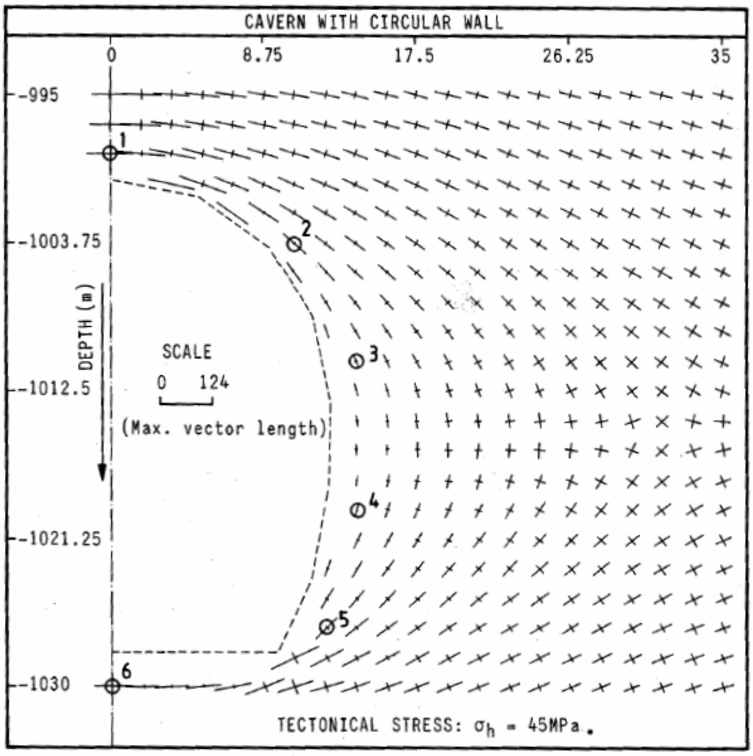


Figure 4.18: Directions and magnitudes of principal stresses surrounding a rock cavern as computed by boundary element analysis

Numerical modelling as shown by the above examples may be very useful during the design of underground openings as well as during the planning of rock support. However, one should be a ware that there are certain restrictions in connection with such modeling. The main restriction is the difficulty in obtaining reliable input parameters. Greatest uncertainty is connected to the boundary stress data and to the elasticity parameters of the rock mass.

When applying numerical analyses one should always keep in mind that the reliability of the analysis will never be better than the reliability of the input parameters.

The main use of numerical analyses in hydropower tunneling is on planning and design of high pressure tunnels and air cushion chambers.

4.11.5 Tunneling Methods

There are two commonly used types of tunneling techniques:

- Conventional "Drill and Blast"
- Use of tunnel boring machines (TBM)

Conventional "Drill and Blast" Method:

In this method of tunneling, the main equipment used is the so-called *drilling jumbo* or *drilling rig*, which performs the main operation. The following are the main sequences to be followed during excavation of each round:

i) Drilling
ii) Charging
iii) Blasting
iv) Ventilating
iii) Scaling
iv) Mucking and hauling
v) Tunnel supporting

Drilling is carried out by the percussion method. The principle is to force a drill rod with a suitable drill bit against the tunnel face, generate a stroke in order to obtain rock spalling, and then rotate the rod before it is once again forced against the hole and a new stroke is generated. The power for the drilling thrust and rotation is transmitted by hydraulic systems.

In tunneling, blast hole diameters of 45 to 50 mm are most common. Medium size tunnels (about 40 m^2) may require about 60 to 70 drill holes. The length of the drill holes usually varies from 3 to 5 m.

Once the drilling operation is completed, ***charging*** of the drill holes with explosives will be performed. There are different types of explosives, the most common one being dynamite.

If the holes close to the planned contour of the tunnel are too heavily loaded with explosives, a considerable "over-break" and a rough, uneven contour may result. This over-break greatly increases the need for scaling and tunnel support. If the tunnel is unlined, it will also greatly increase the head loss. In order to minimize the over-break in the walls and roof of the tunnel, reduced charges are used close to the contour.

In rock ***blasting*** the main principle is to break the rock and push the rock fragments towards a free surface. In a tunnel the degree of confinement of the blast volume is far higher than in a quarry. In order to obtain a satisfactory result from a tunnel blast it is, therefore, necessary to include the so-called "***cut***" in the blast hole design. These consist of holes of larger diameter than the blast holes and are usually left unloaded.

After blasting the round, ***ventilation*** has to be carried out to lower the concentration of blasting fumes to a satisfactory level. The fans are usually started just after the explosion. It is very seldom possible to enter the working face area until 15-20 minutes after the blast, but this depends on the ventilation equipment.

After each blast round, ***scaling*** (removing loose rocks from the roof and walls of the tunnel) is done for the sake of safety. For small tunnels the scaling is made directly from the muck pile, in larger it is often carried out from the wheel loader.
The selection of equipment for ***mucking and hauling*** largely depend on the cross-sectional area and the gradient of the tunnel. For tunnels with cross-sections smaller than 16 m^2, the only alternative for transportation is *rail transport* system. In larger tunnels, *wheel transport* system shall be used. For wheel transport in cross-sections between 16 and 30 m^2 "***niches***" are required every 100 to 150 m for the purposes of loading and turning trucks.

A major restriction for the rail transport alternative is that the maximum gradient has to be less than 2%. For wheel transport gradients up to 15% may be tolerated.

Use of Tunnel Boring Machines (TBM):

A tunnel boring machine (TBM) is a complex and very advanced piece of machinery designed to excavate the entire cross-section in a single operation without the use of explosives. Tunnels with diameters of about 1.8 m to more than 11 m have been excavated with tunnel boring machines.

TBM consists of a wheel cutter head fitted with teeth or rollers to cut or spall the rock. The wheel is slightly smaller than the bore of the tunnel and is equipped with disc-cutters to produce the designed bore. The wheel is forced against the tunnel face by hydraulic jacks and is made to rotate. As excavation proceeds, the rock-cuttings are picked up in buckets attached around the rim of the wheel and are discharged on to a conveyor belt incorporated with the machine as shown in Figure 14.

The diameter of the cutters is normally within the range of 45 to 50 cm, and the total number of cutters varies from 20 for smallest machines to more than 70 for the largest.

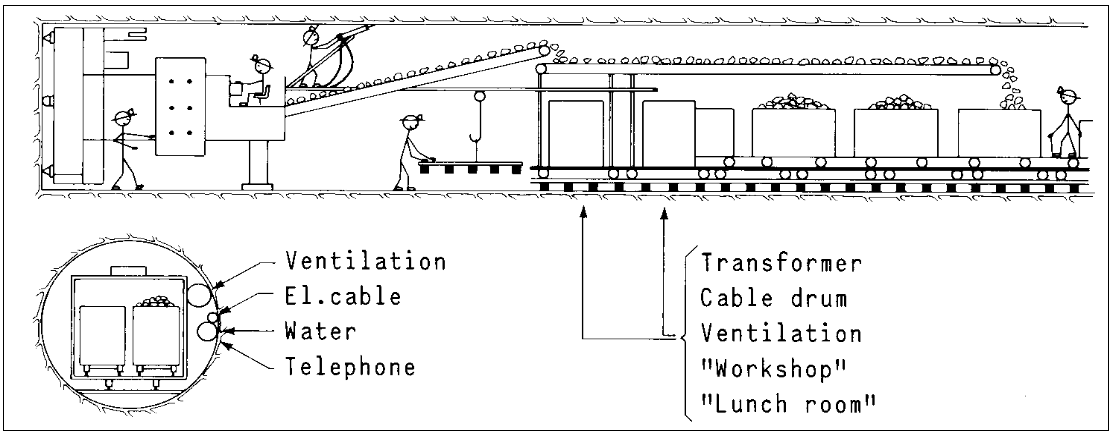


Figure 4.19: Typical sketch for TBM tunneling

Compared to conventional D & B tunneling technique, TBM tunneling has the following advantages:

- For long tunnels (> 3 km) the excavation time and the costs in many cases are considerably lower due to a higher advance rate combined with reduced requirement for tunnel support and ventilation.

- The tunnel will have a circular profile with a smooth contour, which is of particular importance in reducing head loss in unlined water tunnels.
- Potential problems due to blast vibrations are eliminated, especially in populated areas.
- Less number of "adits" is required.

There are, however, some limitations in connection with the use of TBM in tunneling. These include:

- Initial cost of the machine is high
- Requires detail geological investigation than the D & B alternative
- Less flexible than Conventional D & B technique in tackling stability problems
    - o maximum gradient is restricted to approximately 2% for railroad transport
    - o the maximum curve radius is restricted to 150-450 m

4.11.6 Tunnel Supports

A basic philosophy in tunneling is that the extent of installed tunnel support should reflect the actual rock conditions. In good quality rock the *self-supporting* capacity of the rock mass should be used to its advantage, and the amount of tunnel support kept at a minimum. In poor quality rock the design of support should be based on a sound understanding of the character and extent of the stability problem.

The various geological factors which may influence the stability are:

- The strength and quality of the intact rock
- The degree of jointing and their character
- Weakness zones and faults
- Rock stresses
- Water inflow

Tunnel support may be installed either at the working face *(immediate support)*, or behind the face *(permanent support)*. Whenever possible the design for the immediate support should be chosen which makes it possible to act later as permanent support.

The following support methods are the most commonly used in hydropower tunneling today:

- Rock bolting
- Shotcreting
- Grouting
- Concrete lining

Rock Bolting:

A rock bolt is a steel bar, which is inserted into a hole drilled in a rock to improve the rock competency. The distant end has a device which permits it to firmly anchored in the hole and the projecting end is fitted with a plate which bears against the rock surface (*see Figure 15*). The bolt is placed in tension between the anchor and the plate, thereby exerting a compressive force on the rock.

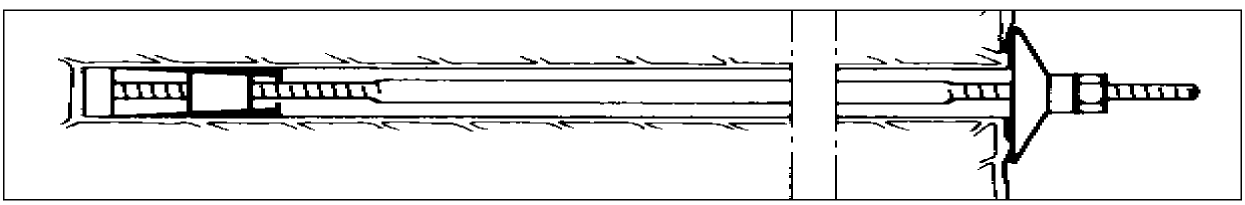


Figure 4.20: Principle of rock bolting

Rock bolting in tunnels is carried out according to one of the following two main principles:

- Spot bolting of individual, unstable blocks

- Systematic bolting of a section of the tunnel or the cavern in a defined pattern

On average, the length of rock bolts in water tunnels is 1.5 to 4 m and the diameter is 16 to 25 mm. Rock bolting is usually used for an immediate support near the tunnel face.

Shotcreting:

A shotcrete is a quick-setting concrete plaster shot at rock surfaces pneumatically. It creates the best possible rock support condition, which makes it an economical, rapid, and effective means of providing tunnel support. In practice the shotcrete is placed in 5 cm layers until a desired thickness is attained.

The use of steel fibres in the concrete mix has an effect of increasing the strength of the shotcrete. For an immediate support in areas of heavily jointed rock masses or in areas of high rock stresses, steel fibre reinforced shotcrete is commonly used.

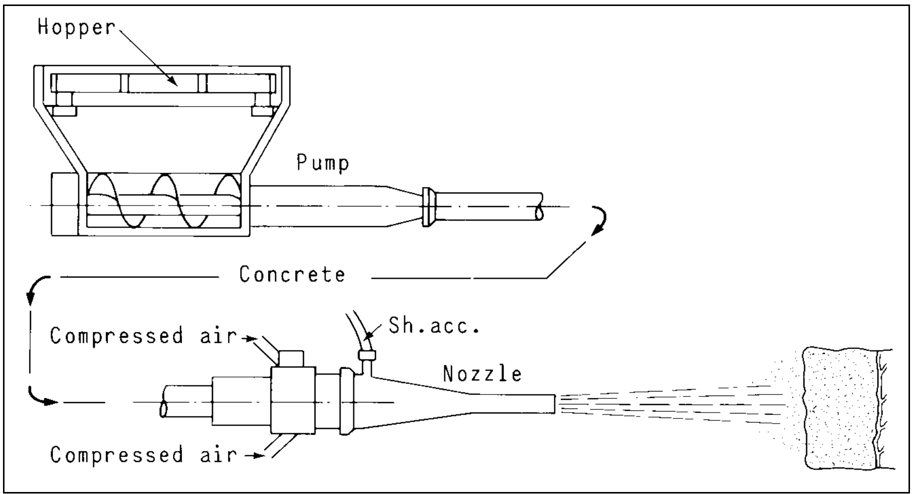


Figure 4.21: Principle of shotcreting

In many cases, the shotcrete is combined with rock bolting for use as a permanent support. A combination of steel fibre reinforced shotcreting and systematic rock bolting can replace concrete lining alternative, provided that water inflow and active gouge material in the discontinuities are minimal or absent.

A general restriction in the use of shotcrete is in areas with water leakage. The main restriction, however, is where weakness zones contain swelling clay (smectite). If shotcrete is applied on such zones, there will be no room for expansion of the swelling clay, and high swelling pressure will be activated when the zones are exposed to water. This may easily destroy the shotcrete lining.

Grouting: A grout is a mixture of cement and water forced in to rocks around the tunnel periphery. Grouting may be performed ahead of the tunneling face (*pre-grouting*) or behind the tunneling face (*post-grouting*). Pre-grouting is necessary in areas where groundwater inflow makes tunnel driving difficult. Probe holes are drilled ahead of the tunnel face to perform permeability testing before deciding the necessity of pre-grouting. Post-grouting is done to improve the stability of the rock mass behind the tunnel face.

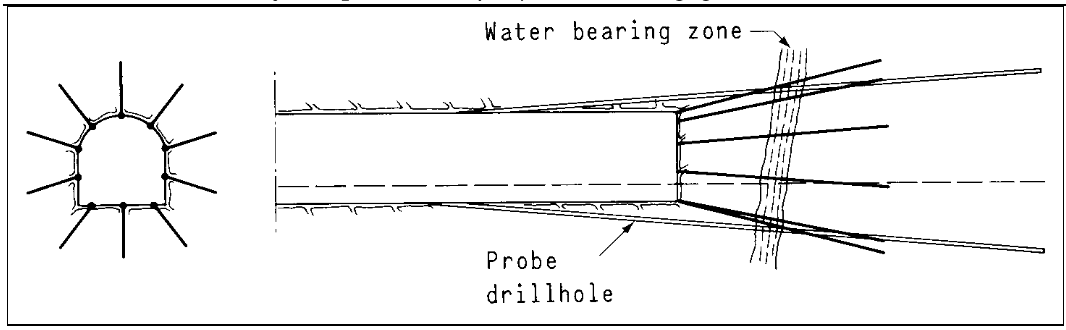


Figure 4.22: Principle of pre-grouting in a water-bearing zone

4.11.7 Tunnel Logging

	Tunnel Length Pos.	50 m	100 m	150 m	200 m
	Tunnel orientation	N03W	N36W	N36W	N36W
Weak ness Zones	zone description Strike/Dip	Metasandstone Sand stone reasonably at favorable angle	Sandstone mixed with quartz Bedding perp. to tunnel direction, cross joints parallel (at the roofs)	Sandstone Has open joints, leaking water, small shear zone little bit of clay	Sand stone
	Width: Right side Roof Left side				10 to 15 m at the roof
	Gouge matr. Colour Side rock Specimen No				
Rock mass/joints	Rock type Water	Metasandstone	Sandstone	Sandstone Leaking water	Sandstone
	Joints Strike/Dip: I II III Spacing Roughness	Bedding joint N20 – 50E 30-150cm Rough	Bedding joints N70-90E	Bedding joint N70-90E Rough	Bedding joint N70-90E Rough
Reinforcement	Type of reinforcement	None	None	None	None
	Extra scaling 10- H/round 5-				
	Remarks				
	HYDROPOWER SCHEME. TUNNEL SECTION			Date : 10/09/03	Sign: GROUP B

	Tunnel Length Pos.	250 m	300 m	350 m	400 m
	Tunnel orientation	N36W	N36W	N36W	N36W
Weak ness Zones	zone description Strike/Dip	Sandstone Small particles fallen from the roof, small band of phyllitic rock	Sandstone	Sandstone	Sand stone Long cross joints run along the longitudinal axis of the tunnel, wire mesh support
	Width: Right side Roof Left side				10 to 15 m at the roof
	Gouge matr. Colour Side rock Specimen No				
Rock mass/joints	Rock type Water	Sandstone	Sandstone	Sandstone Leaking water	Sandstone
	Joints Strike/Dip: I II III Spacing Roughness	Bedding joint/cross joint N70-90E Few centimeters Rough	Bedding joint/cross joint N70-90E Few centimeters Rough	Bedding joint/cross joint N70-90E Few centimeters Rough	Bedding joint/cross joint N70-90E Few centimeters Rough
Reinforcement	Type of reinforcement	None	None	None	None
	Extra scaling H/round				
	Remarks				
	HYDROPOWER SCHEME. TUNNEL SECTION			Date : 10/09/03	Sign: GROUP B

	Tunnel Length Pos.	450 m	500 m	550 m	600 m
	Tunnel orientation	N36W	N36W	N36W	N36W
Weak ness Zones	zone description Strike/Dip	Sandstone Sand stone reasonably at favorable angle, wire mesh support	Sandstone and some phyllitic type Shotcrete lining/ concrete lining (20), weak zone	Sandstone Weak zone, concrete lining	Sand stone / Lime stone
	Width: Right side Roof Left side		About 20 m shot crete 10 m 11 m 12 m	11 m 12 m 12 m	10 to 15 m at the roof
	Gouge matr. Colour Side rock Specimen No				
Rock mass/joints	Rock type Water	Sandstone	Sandstone and some phyllitic type	Sandstone	Sand stone / Lime stone
	Joints Strike/Dip: I II III Spacing Roughness	Bedding joint N70-90E Rough	Bedding joints N70-90E/N50-64E 20 cm to 2 m Rough	Bedding joint N70-90E Rough	Bedding joint N70-90E Rough
Reinforcement	Type of reinforcement	None	Steel fiber	Steel fiber	
	Extra scaling H/round				
	Remarks				
	HYDROPOWER SCHEME. TUNNEL SECTION			Date : 10/09/03	Sign: GROUP B

	Tunnel Length Pos.	650 m	700 m	750 m	800 m
	Tunnel orientation	N36W	N36W	N36W	N36W
Weak ness Zones	zone description Strike/Dip	Limestone Shotcretthrough out	Limestone Roof is shotcreted, calcite material is quite frequent	Limestone /amphibolitic schist Roof is shotcreted , contact zone between limestone and amphibolitic schist	Amphibolitic Schist Shotcreted is continued
	Width: Right side Roof Left side		9 m	30 cm – 1m	
	Gouge matr. Colour Side rock Specimen No				
Rock mass/joints	Rock type Water		limestone	Limeston/amphibolitic schist	
	Joints Strike/Dip: I II III Spacing Roughness	N70-90E/ 80-85NE	N70-90E/ 80-85NE	N64-80E/80-85SE	N64-80E/80-85SE
Reinforcement	Type of reinforcement				
	Extra scaling H/round				
	Remarks				
	HYDROPOWER SCHEME. TUNNEL SECTION			Date : 10/09/03	Sign: GROUP B

	Tunnel Length Pos.	850 m	900 m	950 m	1000 m
	Tunnel orientation	N36W	N36W	N36W	N36W
Weak ness Zones	zone description Strike/Dip	Amphibolitic Schist Shotcreted is continued , highly weatherd/ concrete lined	Amphibolitic Schist Shotcreted is continued , highly weatherd/ concrete lined	Amphibolitic Schist Shotcreted is continued , highly weatherd/ concrete lined	Amphibolitic Schist /Limestone/Green Gneiss Grayish with white bands of quartz, shear zone alond the contact zone
	Width: Right side Roof Left side				5-150cm
	Gouge matr. Colour Side rock Specimen No				
Rock mass/joints	Rock type Water				
	Joints Strike/Dip: I II III Spacing Roughness	N64-80E/80-85SE	N64-80E/80-85SE	N64-80E/80-85SE	N70-73E/ 80-90SE
Reinforcement	Type of reinforcement				
	Extra scaling H/round				
	Remarks				
	HYDROPOWER SCHEME. TUNNEL SECTION			Date : 10/09/03	Sign: GROUP B

5. TRANSMISSION OF ELECTRIC POWER

If the power plant and the load centers are close to each other, the costs of electric power transmission and maintenance are minimum. In most cases power plants are located in remote areas and inside gorges which demands high cost of for transmission of electric power to the load centers.

A design criterion of transmission lines considers:

- The maximum allowable voltage variation from no load to full load
- The maximum economic power loss
- Protection from lightning and other damages
- Structural stability in high winds (or, in temperate areas, in ice and snow)
- Safety for people living and working near the lines

Underground or over ground?

Over ground lines are used most often because, by using air as the cable insulator, the cable is less expensive. Insulation can be cheap and simple. In most developing countries, uninsulated cable is more readily available than underground.

Uninsulated cables are exposed to lightning and to falling trees. The land close to the lines has to be cleared of trees, and this has to be carried out periodically. The poles may also have a finite life, and so may need replacing, perhaps every 15 years. Further, overhead lines are less efficient than underground for a given conductor size because the wide spacing of the conductors gives rise to inductive losses.

Underground lines have to be insulated, and protected against ground movement, ploughing, new buildings, etc. Once installed, however, the line should run without maintenance until the insulting material deteriorates.

High Voltage (HV) or Low Voltage (LV) lines?

If transformers are used to step up the voltage to high values, the currents in the conductors are smaller and cables are smaller. The lower cost of the cables is offset by the cost of the two transformers needed, one at the start of the transmission line, and one at the end, to step the voltage back down to the standard value. Additional costs for checking of ventilation or cooling level and insulators for attachment of the cables to support poles are needed. By contrast low voltage (LV) lines without transformers are more easily erected and maintained by the local users of power.

5.1 Transmission and Distribution (T&D) system planning

Planning of T&D in principle should satisfy the socio-economic premises, technical requirements depending on the availability of investment, cost level and other political measures. Hence, power system planning is based on specific objective: seeking a plan that contributes to minimize the total socio-economical supply cost, keeping all relevant restrictions during the period of analysis, typically 20-30 years.

The planning process may have the following phases:

i Establishing the database

- Electrical system data, i.e. description of existing system and suggested extensions
- Load data, i.e. historic and present energy consumption, description of heavy/light load situations as well as prognoses for energy and power

ii Determine the main principles for system layout/renovation strategy

- Decision whether to follow earlier practice(system layout) of the utility or if the system philosophy is to be considered e.g. by use of optimization calculation based on simplified system description

iii Technical analysis of different system alternatives

- Load flow analysis, short circuit calculations, reliability analysis, contingency analysis and stability analysis

iv Establishing investment costs and operation costs

- The investment costs (including interest) and operation costs for qualified system solutions are evaluated.

v Cost minimization

- When all the fixed and operation costs for all qualified are calculated for the period of analysis, the task in this phase is to determine the system plan that contributes to minimizing total costs. The quantities under investigation here are: *which investments are to be made in the system (type), the size of the investment (size) and when are investments to take place (year)*

vi Decision of investment plan

- This is an evaluation phase where a selection is made among the economically most favorable plan. The evaluation is accounting for: uncertainties in the database *(possibly by sensitivity analysis), parameters not directly represented in the model of economic analysis (not all relations can be cost evaluated), how flexible the different plans related to uncertainty in the database.*

5.2 Design philosophy of overhead lines

The main parts of a power line, as roughly shown in the figure below, are the conductors (1), the supports (towers or poles) (2) which hold the bare conductors, insulators (3) needed between the conductors and the support and shield wires (4) attached to tower extensions. Towers keep the conductors at suitable distance from the ground and other objects (external clearances) and mutually apart (internal clearances). The three elements: conductors, supports and insulators constitute the main types of components of an overhead power line. In addition, supports need either foundations, or the lower part is buried in the ground, to keep them in a fixed position, and hardware and clamps are used to fix the insulators between supports and conductors. Shield wires are provided in some power lines.

Figure 5.1: Parts of Power Line

The clearance has to be large enough to avoid discharge. The components must have the mechanical strength to resist the stresses they are exposed to.

Standards and regulations are required to layout and design over head power lines, which out line the criteria for electro technical and mechanical aspects. The electro technical aspects will be to determine the voltage stresses acting on the line and to determine the required resistance voltage. The mechanical aspects will be to determine the loads acting on the line and to determine the strength of the various components that will resist the stresses created by the loads.

Design philosophy and standard of overhead lines should aim in explaining the general frame work of a power line (standard) that can be improved in line with practical and theoertical experience.

Figure 5.2: Stylized Power System

5.3 Framework of a standard

i Probabilistic methods :

- based on statistical Knowledge of an event

e.g. a climate load, that can be quantified by its yearly maximum value or the properties of a component that can be quantified by e.g. its ultimate strength

ii Loads (Analyzing loads)

- Mechanical aspects: Analyzing the loads acting on the line
  - Basic loads: due to the weight of towers, conductors and hardware and to vertical and horizontal changes to line direction
- Additional loads:
  - Climate loads: due to wind, ice and temperature, either separately or in combination (statistical basis and experience)
    - Special loads: to meet situations that can occur occasionally during construction and operation
    - Security loads: to withstand and satisfy requirements to the security of the over head line, e.g. a failure can occur due to unpredictable event, longitudinal loads acting on a suspension tower may be due to broken conductor under normal tension (deterministic basis)
    - Safety loads: to withstand with a good safety margin for personnel working on the transmission line
- Electro technical aspects: concerning the mechanical aspects, the electrical stresses acting on the line have be analyzed

  The loads are classified as the following voltages:
  - Continuous power frequency voltages
  - Temporary over voltages
  - Slow front over voltages
  - Fast front over voltages

With all loads the minimum required insulator string length and electric clearance distances will be determined and certain measures will be taken to ensure the operation security for safety for humans and animals

5.4 Right Way of planning

Laying to Transmission lines needs extensive work of panning: The planning process may include:

- Early clarification of possibilities and alternatives
- Close contact with local interests and users of the areas in question
- Recording of all important interests connected with the actual alternatives
- Consultations where all justifiable feasible alternatives are included

In the process of planning it is very important that the planners do not choose their own favorite alternative before all relevant information is brought forward.

i *Adaptation to use of land*

As a main rule, avoiding the most valuable and conflict filled areas, where satisfactory alternatives are available should be the aim

Aim at avoiding:

- Pristine areas and areas of high protection value
- Large continues tracts of nature and out door activity areas
- Land cape gems

- Routes with towers in cultivated fields but crossing cultivated field is preferable to going through forest

Evaluate Border zones:

- Between forest and cultivated fields
- Between residential areas and other areas
- Along roads (but carefully)

ii Landscape

The main rule should be that wherever possible and where solutions are otherwise acceptable, the aim should be to find right of ways adapted to, and subordinated to the landscape.

- avoid silhouettes
- avoid barrier effects
- avoid conspicuous effects
- avoid strand zones or crossing over lakes
- stay low, search for routes where the line and especially the towers get cover or back ground in topography or vegetation
- follow existing curves in the landscape, e.g. border zones in topography and vegetation
- Crossing rivers, roads and traffic routes should, as a main rule be perpendicular to them.
- Take care of forest screens when crossing or going along side rivers and traffic routes
- Choose the upper side or inner curve when going along side traffic routes

iii Health Impacts of electric and magnetic fields

In recent years grater attention has been focused on electric and magnetic fields, both among the general public and experts, as a result of the fear that these field can constitute a health risk. The suspicion is mainly directed at the power frequency (50c/s) alternate fields since these can induce electric currents in different materials, among them living tissue. All appliances connected to the electric network as well as electrical component in the electricity supply; develop such fields in their vicinity.

Figure 5.3: Aim at avoiding

Electric and magnetic fields in relation to power lines are important in this combination even if such fields usually are weak compared to what is found in other electric sources. However, power lines extend over larger areas and thus the public is regularly, and in some cases permanently, exposed to the fields.

a) The electric field (E-field)

The electric filed is designated with the latter E and is a measure of the rate of change of the voltage when moving in a certain direction. It is measured in volts per meter (V/m).

The electric field is proportional to the voltage of the line, it is reduced with increased distance to the line and it is increased with increased diameter of the conductor. The E field is also characterized by being affected by objects in the field. Trees, buildings, terrain etc. can therefore intentionally be used to reduce the electric fields. Objects placed in the field will be charged and this can cause a jolt when touched due to the electric discharge. Earthing objects in the field normally solves this problem.

b) The magnetic field (B – filed)

The magnetic induction is designated with the letter B and gives the strength of the magnetic field in the unit Tesla (T).

The B- filed is not directly relative to the voltage of the line, but is proportional to the current. It is reduced with increased distance from the line and with large distance the filed is approximately reduced inversely with the square of the distance. The influence of physical obstacles on B–filed is minor.

iv risk of Bird Habitat:

Power lines affect bird life. The power lines may have an impact indirectly on bird life by disturbing the birds' habitat, e.g., by reducing their access to food, nesting possibilities and destroying their territory. The power lines also affect birds directly as they can be killed or injured in collision with the conductors or by discharges.

Figure 5.4: Reducing the risk of bird collision

The risk of bird collision can be reduced by:

- Choosing right of way out side the best isotopes
- Keeping away from natural migration routes
- Leading over head lines of the same dimension conductors heights in parallel

- Adapting the choice of right of way so that the conductors are shielded by vegetation or terrain to avoid conductors just above tree tops.

5.5 Tower spotting

Tower spotting is done with the help of land surveying. During pegging of the route centre line all necessary information including measurement of crossing lines, communication lines, houses, buildings, roads, rivers and other objects along the route and property boundaries have to be recorded.

The pegging in principle is carried out as a polygon mesh to be able to look between each survey instrument and back to the former. At each new survey instrument position back sight is taken to the former station. Each station is marked with a peg with the station number. As the pegging proceeds, reference points with known coordinates have to be used, to obtain the required accuracy of the direction pages.

Measuring points are taken where the route centre line changes direction. The distance between the direction pegs should not exceed 50 meters. A direction peg is located with suitable distance between the stations, depending on the terrain type.

Side terrain is measured to both sides of the centre line where the side terrain is at a higher elevation than the centre line. How far out from the center line it should be measured is determined e.g. from phase distance and clearance required from the ground. Usually the side terrain will be measured approximately to the horizontal projection of the outer phase. In sloping terrain it has to be measured further out to the side to allow the conductors to swing under wind loads. The width of the route to be measured is largely dependent on the span lengths used, snce conductors in longer span have large sags and will thus swing much more than those with shorter spans.

5.6 Sag calculations and drawing of catenaries

Catenary adoption is to draw the catenaries onto the route profile to check the distance to the ground; it is draw for the highest temperature according to relevant regulations or standard.

Figure 5.5: Definition Sketch

If C is the lowest point on the curves, they forces acting on arc length CB are the tensions T at B, T_o at C and the mass (ws) of portion CB, where w is the mass per unit length and S is the arc length CB. Thus CB must be in equilibrium under the action of the forces:

Resolving Vertically $\quad T\sin\theta = \omega_s$

Resolving Horizontally $\quad T\cos\theta = T_o$

Therefore $\tan\theta = \frac{\omega s}{To}$

For a small increment of the cable:

$$\frac{dx}{ds} = \cos\ \theta\ \left(1t\tan\theta^2\right)^{-1/2} = \left(1+\frac{\omega^2{}_s{}^2}{To^2}\right)^{-1/2} = \left(1-\frac{\omega^2 s^2}{2To^2}\right)$$

$$:.x = \int\left(1-\frac{\omega^2 s^3}{2To^2}\right)ds$$

$$= S - \frac{\omega^2 s^3}{6To^2} + K$$

When x = 0, s = 0, k = 0 X = $s - \frac{\omega^2 s^3}{6To^2}$

The say correction for the whole span ACB

$$C_s = 2(s-x) = 2\left(\frac{\omega^2 s^3}{6To^2}\right)$$

ABC = $but \quad s = 1/2 \ \Rightarrow\ s = \frac{\omega^2 L^3}{24 To^2} = \frac{\omega^2 L^3}{24\ T^2} \ For\ \ Small\ \ values\ of\ \ \theta$

i.e $T\cos\ \theta \approx\ T\ \approx To$

where w = mass per unit length (kg/m)
T = Straining length (m)
L = recorded length (m)

As $\omega\ = \omega/L$

$$\Rightarrow\ \ Cs = \frac{\omega^2 L}{24\ \ T\ \ 2}\ldots\ldots\ldots\ldots\ldots\ldots(b)$$

Egns (a) and(b) apply only to takes standardized on the flat and are always negative

The sag , y is calculated as :

$$\frac{dy}{ds} = \sin\ \theta\ \approx\ \tan\ \theta\ = \frac{\omega s}{To} \quad when \quad \theta\ is\ mall$$

$$:.\ \ y\ \ = \int\frac{\omega s}{To}ds \qquad = \frac{\omega s^2}{2To}$$

If y is the maximum say at the centre of the cable, then s = $S = 1/2 \quad and$

$$y\ \max\ \ = \frac{wL^2}{8T}\ldots\ldots\ldots\ldots\ldots(c)$$

Hence the total length of the cable required due to sag is

$$L^1 = L + correction \quad due\ \ to\ \ say$$

$$= L + \frac{\omega^2 L^3}{24T^2}\ldots\ldots\ldots\ldots\ldots\ldots(d)$$

The elastic elongation of the conductor due to the horizontal tension is approximately

$$\frac{LT}{AE}$$

Where A = the sectional area of the conductor

E = the elasticity Modulus (Young's Modulus) of the conductor

The elongation of the conductor between temperatures t_1 and t_2 is $\pm L(t_2 - t_1) \quad \Sigma_t$

Where $\Sigma_t = -$ thermal elongation coefficient of the conductor

Hence, the overall length the cable b/n tow towers

$$L'' = L' + L(t_2 - t_1)\ \Sigma_t$$

$$= L + \frac{\omega^2 L^3}{24T^2} \pm L(t_2 - t_1)\ \Xi_t$$

5.7 Conditions influencing the Tower spotting

Tower Spotting is used for determining the location and height of towers on the route profile. Several factors can be listed.

- Conductor type
- Tower type
- Terrain type
- Climatic loads
- Crossings
- Clearances to adjacent objects
- Building conditions, etc.

The conductor type used will influence span lengths as well as heights of towers. The breaking strength of the specific conductor type will limit the capacity for long spans, especially for smaller conductors. In connection with spans that means have to be long, e.g. across valleys, a special conductor has to be inserted.

The conductor type is evaluated in relation to the electric power it is going to transmit and from the needed mechanical strength ACSR (Aluminum Conductor Steel Core Reinforced) conductors combine good conductivity with high mechanical strength. They consist of a steel core with a certain number of steel strands with one or several layers aluminum strands outside.

The choice of tower type is very important. When building wood towers, the height of the towers is limited and so as the span length since the lengths of wood poles is a limiting factor.

Terrain type and climatic loads will influence both span lengths and towers locations, for instance in high mountainous areas where there often is heavy wind load. Too long length should be avoided due to the risk of clashing between the conductors. Where very long spans can not be avoided in such areas, the distance between the conductors (phase) could be evaluated carefully to avoid clashing.

5.8 Tower Design: (Static Analysis and dimensioning based on agreed standards)

Towers and foundations should be designed to resist the forces transmitted to the towers from the conductors through their suspension points in the tower. In addition to this comes the net weight of the tower as well as wind load on the tower body.

Over head line supports with wood are used up to 132KV power lines. These conductors use up to ACSR Conductor, (overall diameter 27.7mm). For larger conductors steel towers are used. The Most commonly used wood tower for high power line is the **H-frame** with two legs of round timber. The foundation is made by burying (digging down) the lower part in to the ground. The digging depth depends on the height of the tower and diameter of the towers at the ground surface.

Figure 5.6: tension Tower

When designing the towers, they have to be checked against bending and buckling.

Wood towers are usually built of timber, pressure impregnated by creosote or salt. A well built construction, with an impregnated tower that largely prevents water from penetrating in to the wood can attain longer life time under normal conditions.

The towers can roughly be divided in to two types: tension towers (separating the sections) and tangent towers. The tension towers should be rigid and able to prevent loads from one section being transmitted to the next. The tangent towers should support the conductors within the section.

Longitudinal loads, e.g. due to a broken conductor, will as a rule be determining for the design of the tension towers. For tangent towers transverse wind loads are usually decisive for the designs. When designing transmission supports, attention should be give to the fact that different load cases can be decisive for the dimension of different parts of the tower. In principle therefore, parts of a tower must be controlled for all load cases, and the various components must act so that force from the conductor can be transferred through the construction from the conductor clamps to the foundation.

Tension towers are usually formed as so called ***A-towers with 2 or 3*** legs. Top and base connections must be designed for the anticipated shear forces and are carried out with the necessary number of bolts. Alternatively, tension can be attained with guyed towers, but this gives a less stiff construction. There are many types of tangent towers, depending on function. Most often the so called ***H-frame*** with suspended insulator chain is used. The tower has proved the needed flexibility for longitudinal and transverse load, as well as torsion. The connection between cross arms and tower legs should be formed so that it allows for moments with out giving rise to wear.

Figure 5.7: Tangent Towers

In tension towers as well as in tangent towers with angle or up lift, tension chains are most often used. The mechanical design of these, i.e. clamps, insulators and suspense are implemented according to the regulations or standards.

Steel Towers:
Computer programs are most often used to analyze the loads on steel towers (forces and displacements)

Testing of Towers:
Even through it can be documented by means of the design that towers in a power line have the necessary strength, it is customary to test the different tower types of a large power line in a testing station. At the testing station the towers are erected on foundations and checked for the relevant loads cases in full scale. Loads are imposed on the towers in the suspension points for conductors/insulators. To simulate wind loads on the tower body, loads can be applied elsewhere in the tower.

The loads can be applied using simple weights or pulleys and hydraulic systems. During the testing of the different load cases the loads can be applied steadily and controlled. With the help of measuring equipment the deflection of the towers can be recorded and checked against the loads the tower are designed for.

Figure 5.8: Tangent towers with guy wires anchored in buried foundation

Concrete poles

5.9 Design of foundations

The foundations of the towers may be a separate construction upon which the tower is placed in the case of a conventional wood pole the poles themselves are dug down in to the ground.

With regard to foundations as separate parts of the tower they are usually built on steel reinforced concrete. This type of foundation may be divided in to:

i *Foundation designed to resist compression only*
ii *Foundation designed to resist uplift*
iii *Foundation designed to resist both compression and uplift*
iv *Foundation designed to resist toppling overturning moment*

An example of type ***i*** and ***ii*** is foundations of guyed steel towers.
The columns in this case are exposed to axial compression and may rest on foundations designed for compression only. The guy wires for this type of towers are anchored in buried foundations (anchors) designed for tension forces only.

In connection with self supporting lattice steel supports, they usually have separate foundation for each leg which can be exposed to both compression and tension forces. In addition to these forces these foundation also have to resist shear forces that the towers are exposed to.

Towers exposed to overturning are generally self supporting towers placed on a single foundation only. These foundations have also to resist the axial and lateral forces transferred from the towers.

Figure 5.9: Tower Foundations

5.10 Conductors

These carry the electrical power from one end to other for transmission and distribution.
Requirements of good conductor:

- Good conductivity or low specific resistance
- High tensile strength to withstand mechanical stresses
- Not brittle
- Not too expensive
- Low specific gravity for low weight

Materials may be:

i. Copper
ii. aluminum
iii. ACSR (Aluminum conductor steel core reinforced
iv. Galvanized steel
v. Phosphor bronze
vi. Cadmium copper

To increase the flexibility, all conductors are stranded in which case the central layer has successive layers 6, 12, 18, 24 wires.

Copper conductors have high electrical conductivity and tensile strength. It is non corrosive and free from electrolytic trouble. But it is expensive.

Although the conductivity of aluminum conductor is 60 percent and strength is 75% to that of copper, aluminum conductors are usually employed for carrying heavy currents for being cheaper and lighter in weight. The disadvantage is the greater coefficient of expansion, greater sag, low melting point, difficulty in jointing, etc. ACSR conductors are good for larger span in general.

5.11 Insulators

These are provided so that there is no leakage of current to the earth through the support poles. Porcelain, glass and steatite are used as insulators. Porcelain is (excessively widely) used as insulator. It is dielectric strength is 60 KV per cm of thickness and compressive strength is 70,000 kg/cm^2. But tensile strength is low which us about 500kg/cm^2. The dielectric strength of glass is high (140kv/cm of thickness) and has high compressive strength but dirt deposits easily and during rain there may be surface leakage. Its limit is up to 25kv under ordinary condition and 59kv for dry condition.

Steatite insulators are used in tension towers and transmission lines with sharp turn.
The types of insulators are:

- Pin type
- Suspension type
- Strain type
- Shackle type
- Stay or Egg type

Corona

It is a phenomenon in high voltage transmission lines due to portal breakdown of air in the vicinity of energized line. This is characterized by a violet glow around the conductor and a hissing noise produced along the glow.

6. CONSTRUCTION FEATURES OF HYDROPOWER PROJECTS

Main feature of hydropower project:

- Storage
- Conveyance
- Power house

6.1 Reservoirs

Purpose: to stabilize the flow of water in order to satisfy a varying demand from consumers or of regulates water supplied to a river course.

Investigation of reservoir sites:

In an investigation of a potential reservoir site, consideration must be given to the amount of rainfall, runoff, infiltration, and evapotranspiration which occurs in the catchments area. The climatic, topographical and geological conditions are therefore important, as is the type of vegetation cover.

Basic data for reservoir design studies:

- Topographical Map
- Hydrological records

Leakage from reservoirs:

The most attractive site for a large impounding reservoir is a valley constricted by a gorge at its out fall with steep banks upstream so that a small dam can impound a large volume of water with a minimum extent of water spread. However, two other factors have to be taken in to consideration:

i. The water tightness of the basin and
ii. Bank stability

The question of whether or not significant water loss will take place is chiefly determined by the groundwater conditions, more specifically by the hydraulic gradient. Accordingly, once the ground water conditions have been investigated an assessment can be made of water tightness and possible ground water control measures.

Leakage from reservoirs takes the form of sudden increases in stream flow downstream of the dam site with boils in the river and the appearance of springs on the valley sides. It may be associated with major defects in the geological structure such as solution channels, fault zones, or buried channels through which large and essentially localized flows takes place. Seepage is more discrete flow, spread out over a larger area, but may be no less in total amount.

Apart from the conditions in the immediate vicinity of the dam, the two factors which determine the retention of water in reservoir basins are the piezometeric conditions in, and the natural permeability of, the floor and flanks of the basin.

For ground water condition (Knell, 1971)

i. The groundwater divide and piezometeric level are at a higher elevation than that of the proposed top water level. In this situation no significant water loss takes place.

ii. The ground water divide, but not the piezometeric level, is above the top water level of the reservoir. In these circumstances seepage can take place through the separating ridge in to the adjoining valley.

iii. Both the ground water divide and piezometeric conditions are at a lower elevation than the top water level but higher than that of the reservoir floor. In this case the increase in groundwater head is low and the flow from the reservoir may be initialed under condition of low piezometeric pressure in the reservoir flanks.

iv. The water table is depressed below the base of the reservoir floor. This indicates deep drainage of the rock mass or very limited seepage.

Troubles from seepage can usually be controlled by exclusion or drainage techniques.

Exclusions:

- Cut of trenches
- Grouting
- Impervious lining – Asphalt membrane
- Clay blankets

Grouting:

- Curtain grouting
- Consolidation grouting

The depth of grouting hole should be more or equal to the dam height.

Drainage gallery:

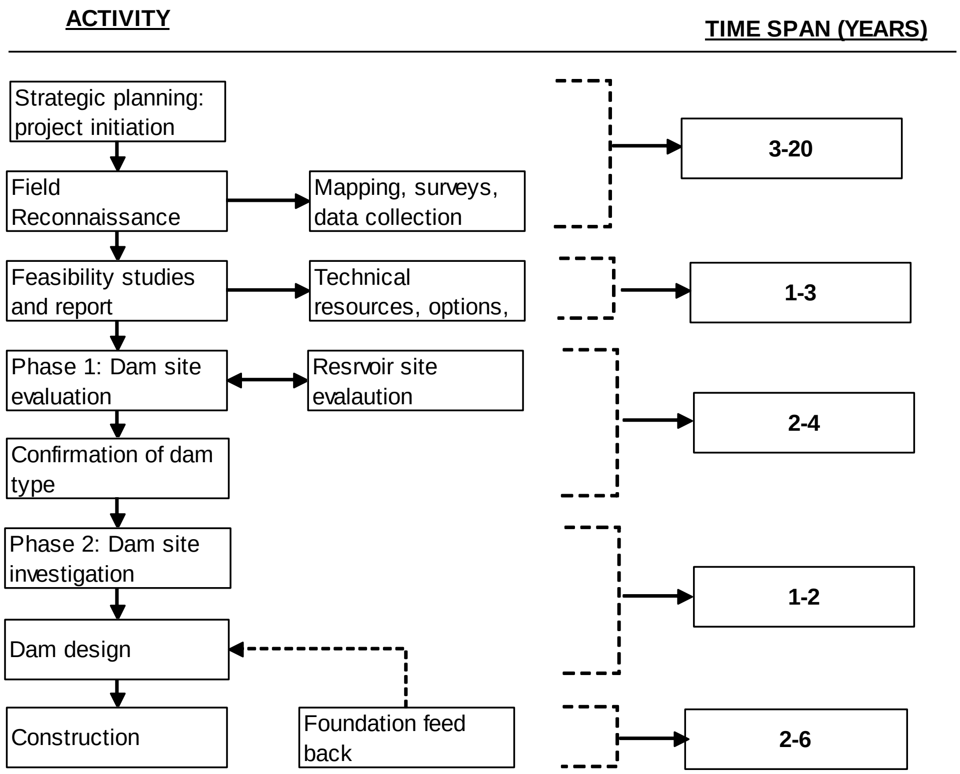
Any seepage water through the foundation will be intercepted by the drain hole and brought up to a collection drain from where water is pumped out.

6.2 Concrete Dam Construction

i. Inspection galleries
ii. Transverse joints (Inter block joints): vertical contraction joints are formed at regular intervals of 12-15m along the dam axis. The joints are made necessary be the shrinkage and thermal characteristics of mass concrete. They permit minor differential movements between adjacent blocks, and in their absence major transverse cracks will develop. To control seepage along the plane of the joints a water barrier is formed close behind the upstream face.

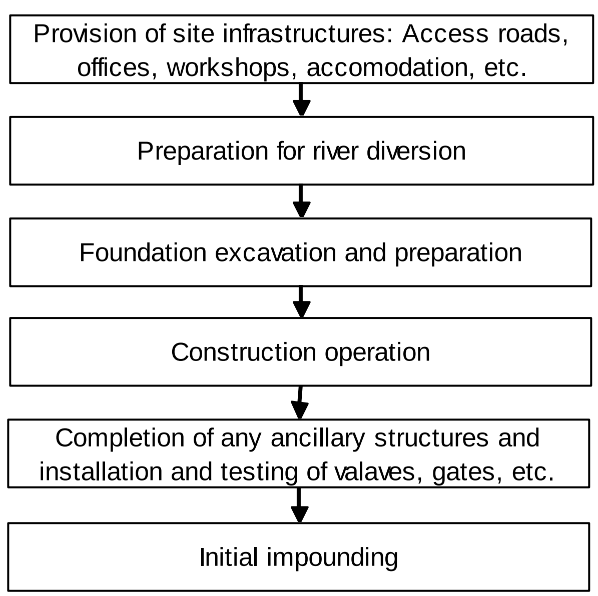
iii. Construction joints (inter life joint): individual concrete pours within each monolith must be limited in volume and in height to reduce post construction shrinkage and cracking. Concrete pours are therefore restricted by the regular formation of near horizontal construction or "lift" joints. Lift height is generally limited to 1.5 -2.0m. The lift surface is generally constructed with a stepped or uniform fall of 5-10% towards the upstream face to improve the notional resistance to sliding on that potentially weaker plane.

Stages in dam site appraisal and project development Activities:

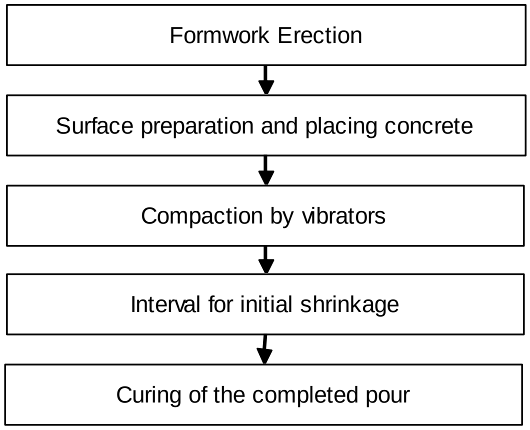


Construction of monolith can be done on either the 'alternate block' or the 'shrinkage slot' principle. In either method the objective is to maximize shrinkage before pouring abutting lifts of concrete in adjacent blocks.

Phases of Project Excution



Sequences of Concret dam Construction



a) Alternate block construction adjacent pours phased to accommodate shrinkage- lag time approximately 30-60 days.

b) Construction with contraction gaps or shrinkage slots: gaps concreted approximately 30-60 days after adjacent lifts completed.

Mass concrete in dams is not subjected to compressive stresses comparable with those developed in most other major structures. The volume of concrete with in a dam is relatively great, however, requiring large pours and high placing rates. Several other properties therefore rank equally with strength as indices of quality and fitness for purpose. The desirable characteristics for a mass concrete for use in dam can consequently be summarized as follows.

- Satisfactory density and strength
- Durability
- Low thermal volume change
- Resistance to cracking
- Economy

Cement Requirement (kg/m3):

Zoning of concretes in high dams (H>100m)

Constituent Materials:

Cement:

Ordinary Portland cement is not recommended in dam construction. The resulting temperature rise and heat gain in large pours is unacceptable in relation to consequent problems of shrinkage, heat dissipation and cracking. It is therefore preferable to employ a ***low heat or modified Portland cement***. Thermal problems can be alleviated by the use of Pozzolana blended Portland cements. In the absence of special cements, partial replacement with pulverized fuel ash (PFA) and /or cooling are also effective in containing heat build up.

Aggregate :(Cheap, inert)

A maximum size of coarse aggregate of 75-100 mm is considered the optimum with rounded or irregular natural gravels generally preferable to crushed rock aggregates. In fine aggregate range, i.e. <4.67mm size natural sands are similarly preferable to crushed fines. Aggregates should be clean and free from surface weathering or impurities. A smooth well-graded particle size distribution curve for the combined aggregate will ensure maximum packing density for the compacted concrete mix.

Water:

Water for use in concrete should be free of undesirable chemical contamination, including organic contaminates. A general standard is that the water should fit for human consumption.

Admixtures:

Air entraining agent, (AEA) is added to the concrete which helps in reducing the water contents, and handling becomes easy. Water reducing admixture (WRAs) is sometimes employed to cut the water requirement, typically by 7-9%. They are also effective in delaying setting time under conditions of high ambient temperatures.

Concrete mix parameter:

The parameters which are principally responsible for controlling the properties of concrete manufactured with specific cement and aggregates are cement content, C (kg/m^3); water content, w (kg/m^3)and water: cement ratio (by weight). Some further influence can be exerted through the addition of PFA and and/or the use of other admixtures such as AEA and WRA.

The in-situ properties of the mature concrete are dependent upon attaining maximum density through effective compaction. The ability to achieve this is largely controlled by the physical characteristics of the fresh concrete, notably its cohesiveness and workability which is related to the mix proportions, principally in terms of the water, cement and fines contents.

Characteristics of Mass concrete for dams

Characteristics	Unit	Concrete mix	
		Hearting	Facing
Cement(C) +PFA(F)	(kg/m^3)	150-230	250-320
F/(C+F)	%	20-35	0-25
Water : (C+F) ratio	-	0.5-0.70	0.45-0.65
90-days compressive strength, σ_c	MN/m^2	18-30	25-40
Tensile strength, σ_t / Compressive strength, σ_c	-	0.10-0.15	0.07-0.10
Unit weight, γ_c	KN/m^3	23-25	
Modulus of elasticity E,	GN/m^2	30-45	
Poisson ratio	-	0.15-0.22	
Shrinkage (at 1 year)	%	0.02-0.05	
Coefficient of thermal expansion	X10^{-6}/C	9-12	

Handling and placing of concrete:

For lower lifts it may be possible to carry the concrete by trucks but for higher lifts, the concrete is to be carried by crane arrangement, traveling overhead cable ways and conveyor systems. Concrete lifts are normally formed in at least two layers, and compacted by poker vibrators. The cost efficiency and effectiveness of the compaction may be improved by the use of immersion vibrators mounted on suitable tracked plant running on the surface of the concrete pour.

Uniformity and consistency has to be ensured during concrete production and placing over the period of the construction.

Controlling concrete temperature:

During placing, the concrete temperature has to be maintained low (12-15^{0}C). The temperature can be brought down either by pre-cooling of coarse aggregate and use of ice chilled water during concrete production reduces the concrete temperature. Pre-cooling of the coarse aggregate is done by spraying cool water.

Post Cooling:

Depending on the ambient temperature, post cooling may be needed. High density polyethylene pipes are laid between 1.0 to 1.5m interval in the lifts and ice cooled water (3-4^{0}C) is circulated through the pipes. The period of post cooling could be as high as 6 months.

Roller Compacted concrete dam. Construction (RRCC dam):

The construction of concrete gravity dam consumed long construction time due to the slow curing process of mass concrete to avoid thermal shrinkages. A new technology, RCC dam construction was introduced in 1970s which offers a potential of financial benefits associated with shortening of construction period by up to 35% combined with a lower-cost variant of concrete for large dams.

Three approaches:

RDLC- Rolled Dry Lean Concrete

RCD- Rolled –Concrete Dam (Japan) – lean hearting

RCC- Roller – Compacted Concrete – high paste content material and known to have high PFA content

a) Rolled Dry Lean Concrete (RDLC)

b) High paste Roller Compacted Concrete(RCC)

In the construction of RCC dam the concrete is handled as an earth fill, and compacted at or near its optimum moisture content in thin layers.

Construction in RDLC and some other RCCs permits an intensively mechanized construction process, with concrete delivery and compaction plant. Construction joints, if considered, may be sawn through each successive layer of concrete after placing.

The RCC approach is best suited to wide valley; giving scope for unobstructed 'end-to-end' continues placing. The construction saving realized are at a maximum for high-volume dams and arise from a 25-35% reduction in construction time as well as reduced unit costs for the RCC. In its low-cost 'geotechnical' format (e.g. RDLC). RCC is particularly suited to more remote sites where importation of cement and/or PFA is difficult or expensive.

Characteristics of RCCs for dams

Characteristics	Unit	RCC type			
		Lean RCC (RDLC)	RCD	High pasted RCC	Convention Lean Nearing concrete
Cement (C) + PFA(F)	Kg/m^3	100-125	120-130	>150	150-230
F/(L+F)	%	0-30	20-35	70-80	20-35
Water : (C+F)ratio	-	1.0-1.1	0.8-0.9	0.5-0.6	0.5-0.7
90-days compressive Strength, σ_c	MN/m^2	8-12	12-16	20-40	18-40
Unit weight, γ_c	KN/m^3	23-25	23-25	23-25	22-25
Layer thickness	m	0.3	Lifts = 0.7-1.0	0.3	Lifts = 1.5-2.5
Contraction joint		Sawn	Sawn	Sawn or formed	Formed

Figure: Soil compaction relation ships.

Number of passes of roller = compaction magnitude.

Construction of Embankment Dams:

The construction operations of embankment dams fall in to four principal groups relating to:

i. Material source development: opening out of borrow areas or quarries, installation of fixed plants, e.g. crushers, and conveyors, construction of access and haulage roads, etc.
ii. Foundation preparation and construction: river diversion, removal of top soil and weathered surface.
iii. Fill construction: placing to materials and compaction.
iv. Ancillary works construction: construction of spillways, stilling basins, culverts, tunnels and outlet works.

Figure: schematic of the variation of embankment stability parameters during the construction and operation

Geosynthetics in embankment dams:

Geosynthetics (geotextile and geomembranes) have considerable potential in dam engineering given that issues of durability in specific applications can be resolved. A range of geosynthetics have been employed in a number of different applications both in new construction and in rehabilitation projects.

Geosynthetics can be employed to fulfill several different functions in embankment dams:

i. Impermeable membranes (upstream or internal): Polyvinyl Chloride (PVC) and High Density Polyethylene (HDPE) upstream membranes have been successfully employed in dams up to 40m height. The membranes, typically 3-4mm thick, are laid in 4-6m wide strips on a prepared sand bed and drainage layer, and anchored at crest and toe.

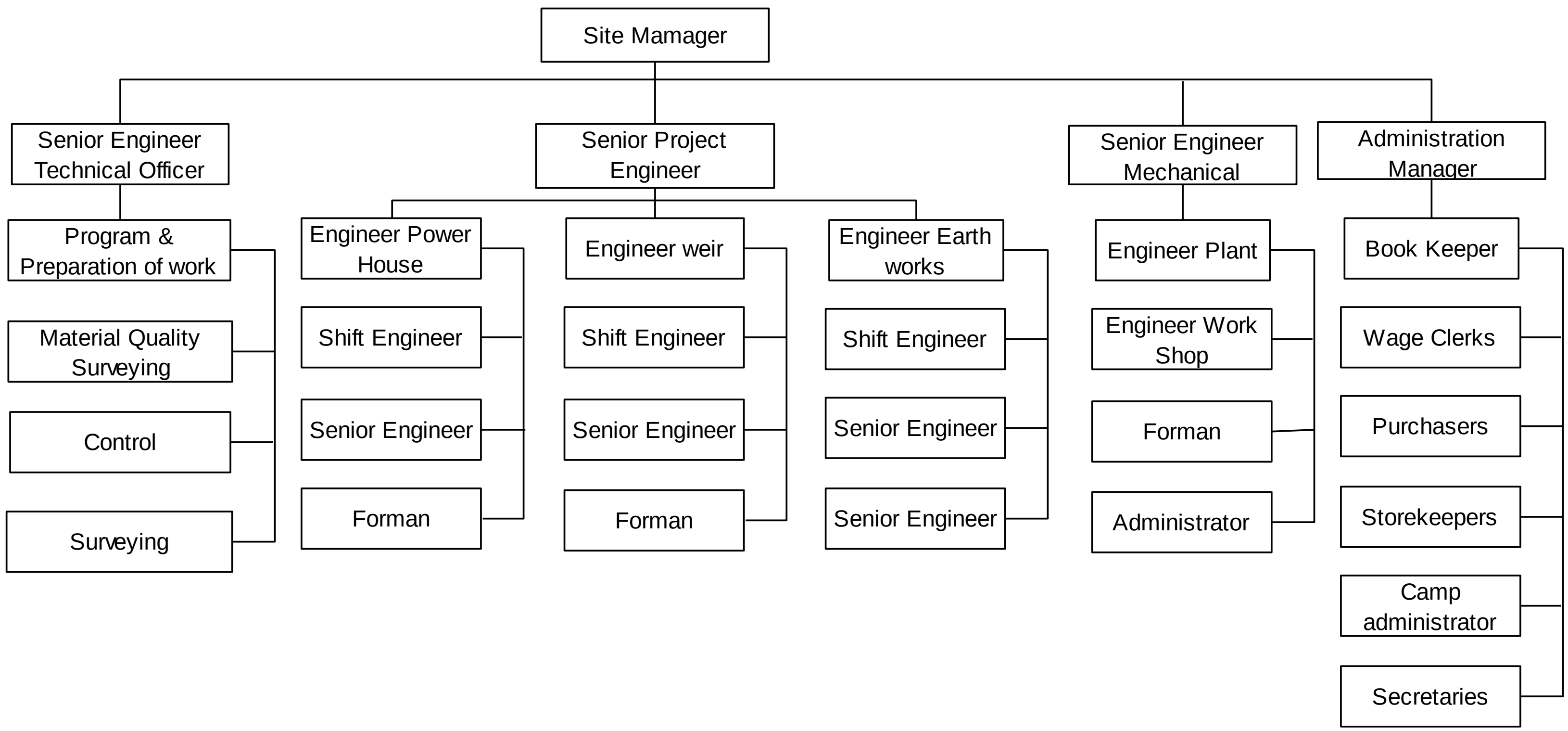
ii. Fitter and drainage layers (seepage control): Relatively thick geosynthetics with high internal transmissivity are suitable for filters or drainage layers.

iii. Earth reinforcement (Stability of slopes, etc.): Geosynthetics reinforcement materials, e.g. geogrids, can be used to permit construction of steeper face slopes or to help to contain lateral deformation and spread within the embankment or on a soft foundation.

iv. Control of surface erosion (precipitation or limited overtopping flows). The use of geogrids and mats in conjunction with natural vegetation has proved for erosion resistance.

v. Separation interlayer: geosynthetics can be used to act as an interlayer to ensure positive separation of fill materials, at an interface.

Organization chart of project construction operation:



6.3 Instrumentation Objectives and Dam safety

The principal objectives of a geo-technical instrumentation plan may be generally grouped into four categories:

- Analytical assessment
- Prediction of future performance
- Legal evaluation and
- Development and verification of future research and designs

Instrumentation achieves these objectives by providing quantitative data to assess groundwater pressure, deformation, total stress, temperature, seismic events, leakage, and water levels. Total movements as well as relative movements between zones of an embankment and its foundation may also need to be monitored. A wide variety of instruments may be utilized in a comprehensive monitoring program to ensure that all critical conditions for a given project are covered sufficiently.

Analytical assessment:

Analysis of data obtained from geo-technical instrumentation may be utilized to verify design parameters, verify design assumptions and construction techniques, analyze adverse events, and verify apparent satisfactory performance.

Prediction of future performance:

Instrumentation data should be used in such a manner that informed valid predictions of future behavior of an embankment can be made. Such predictions may vary from indicating continued satisfactory performance under normal operating conditions to an indication of potential future distress which may become threatening to life or safety, and necessitate remedial action. Often earth and rock fill embankments constructed for flood-control purposes remain dry, or maintain only very low level conservation or recreation pools, except during infrequent flood events.

As a result, these embankments may have existed for years without ever experiencing maximum design conditions. However, instrumentation data obtained during intermediate flood events can be projected to predict performance during potential maximum flood stage or reservoir levels.

Legal evaluation:

Valid instrumentation data can be valuable for potential litigation relative to construction claims. It can also be valuable for evaluation of later claims relative to changed groundwater conditions downstream of a dam or landward of a levee project. In many cases, damage claims arising from adverse events can be of such great monetary value that the cost of providing instrumentation can be justified on this basis alone.

Instrumentation data can be utilized as an aid in determining causes or extent of adverse events so that various legal claims can be evaluated.

Development and verification of future research and designs:

Analysis of the performance of existing dams and levees, and instrumentation data generated during operation, can be used to advance the state-of-the-art of design and construction. Instrumentation data from existing projects can promote safer and more economical design and construction of future earth and rock fill embankments.

Operation and Maintenance Program:

The people responsible for dam operation and maintenance should become involved with the dam during the design and construction stages. This will give O&M (operation and maintenance) personnel an opportunity to become familiar with design and construction considerations and to become aware of problems that may require special attention during the operation and maintenance of the dam. An inspection should be made at construction completion by design, construction, and operations personnel to ensure that all items are complete or deficiencies are identified for later completion. During this inspection, problems, unique operations, general maintenance requirements, etc. should be discussed and procedures established for their proper handling. Requirements for initial filling should be available and should be agreed upon. During this time extra precautions and procedures for operation should be established because unpredictable situations may occur. During the first filling the facility should be attended continuously.

Routine maintenance and inspection of dams and appurtenant facilities should be an ongoing process. All unusual conditions that may adversely affect the operation, maintenance, or safety of the dam should be reported promptly using predetermined written procedures.

In addition to ongoing routine maintenance and inspection, periodic in-depth inspections should be made on every dam at least every 5 years. The depth and frequency of these inspections should depend on dam size, hazard, complexity, and the previous problems encountered. A qualified team, usually headed by an engineer not directly involved in the operation and maintenance of the facility, should perform these inspections. The engineer should be accompanied by operations personnel familiar with all feature of the operation and maintenance of the dam. Inspections should be scheduled, if possible, during alternate periods of high and low water to observe conditions unique to these situations. Special inspections should be scheduled when there is reason to believe that significant damage has occurred or has potential to develop. Deficiencies noted during the inspection should be identified and documented in the report, and procedures should be established for correction in a timely manner. The responsibility for correcting problems should be clearly documented. Funding schedules should be considered to ensure adequate and timely funding to accomplish the work.

Underwater inspections of facilities not normally observable, such as stilling basins, upstream face, etc., should be scheduled periodically to ensure continued performance. An underwater inspection every 6 years is recommended; however, the inspection frequency can be adjusted depending on the findings. Inspections should be scheduled during low water periods to the maximum extent possible. Underwater divers and photography may be used to good advantage in some cases; however, dewatering may be required to better evaluate the condition of facilities. A report of all such inspections should be prepared, describing the condition of facilities and citing identified deficiencies.

Written instructions should be available for use by O&M personnel to operate the dam. These instructions furnished by designers and manufacturers should include the procedures for routine servicing and the requirements for special operation and maintenance of equipment. The procedures, generally referred to as SOP's (Standing Operating Procedures) should also include emergency preparedness plans and inundation mapping, the extent and nature of inspections, hydrologic and reservoir operations, and other pertinent aspects of dam O&M. The operation and maintenance of the dam should be carried out according to these procedures. Significant deviations from these procedures by O&M personnel should not be made without the approval of higher management or engineering personnel. A copy of these instructions should be accessible to the dam operator both during routine operation and during abnormal conditions at the dam. A log should be kept for each dam to record all significant actions or information, such as releases, seepage, maintenance, emergencies, etc. This book should be kept at the dam or other accessible convenient place for ready reference and use. It should become a part of the permanent records for the dam. Dam O&M personnel should be trained before their independent operation of a dam. The degree and complexity of training should depend on the conditions and hazards at and below the dam.

6.4 Dam safety principle and concepts

New Dams:

i. Planning and Design

A new dam should be developed in accordance with state-of-the-art design techniques and construction practices and in a manner commensurate with its size, function, geologic setting, and potential hazard classification. Careful attention must be given to the following planning and design considerations.

- Selection of the dam site
- Estimation of the PMF and selection of the IDF
- Identification of earthquake source are and structure, estimation of MCE's (Maximum Credible Earthquake) and identification of earthquake related safety concerns
- Development of a site-specific geotechnical exploration program
- Design of the foundation, dam, and appurtenant structures
- Design of a system of instrumentation to monitor the performance of the dam, foundation, and appurtenant structures
- Development of an initial reservoir-filling and surveillance plan and of reservoir drawdown criteria
- Preparation of designer's operating criteria and identification of special considerations to be observed during construction and operation
- Provisions for the automatic, independent review by competent individuals of all design decisions, methods, procedures, and results related to dam safety
- Provisions to revise the design to make it compatible with conditions encountered during construction

ii. **Construction**

Quality construction is critical to dam safety. Construction personnel must be constantly alert to recognize and recommend the possible need for adjustments in the design, construction materials, and construction practices to properly provide for actual conditions encountered. The essential aspects of the construction program include:

- Keeping construction engineers and inspectors informed of the design philosophies, assumptions, and intent of the designer with regard to foundation excavation and treatment, to the usage and processing of construction materials, and to the design concepts associated with the construction of embankments and concrete structures and with the installation of mechanical and electrical equipment
- Keeping construction engineers and inspectors informed of the field control measures and tests required to ensure quality construction
- Maintaining an adequately staffed and equipped materials laboratory at the dam site to meet the field testing requirements
- Providing a formal plan for construction inspection to ensure that each facet of essential work is accomplished in multi shift operations

- Giving the Project Construction Engineer the authority to suspend work until all site conditions different from those anticipated are evaluated and the necessary design or construction changes are implemented
- Inspection and acception of critical work stages, by the appropriate engineers or geologist (design and/or technical review personnel)
- Keeping a job diary and documentation that provides a complete history of the work
- Providing mapping and photographic documentation of the construction progress and of significant events; e.g., geologic maps and photographs of final treated foundations.

Existing Dams:

i. Operation and Maintenance

The operation and maintenance procedure implemented should ensure the safe operation of the dam and provide for timely repair of facilities. The essential procedures include:

- Preparing SOP's (Standing Operating Procedures); information on the preparation of SOP's is contained in chapter 12
- Training personnel in both normal and emergency operation and maintenance responsibilities and in problem detection
- Maintaining a written record of reservoir, waterway, and mechanical equipment operations and of maintenance activities
- Testing full operation of spillway and outlet works gates on a regular basis, using both primary and auxiliary power systems
- Providing for public safety and for security against vandalism of essential operating equipment
- Establishing and maintaining communication links with local governmental agencies and authorities
- Preparing and maintaining current EPP's (Emergency Preparedness Plan)

ii. Periodic Examinations and Evaluations

The periodic examination and evaluation of dams and reservoirs is of considerable importance for public safety. The intent of conducting periodic examinations and evaluations is to disclose conditions that can disrupt operations or threaten dam safety early enough for these conditions to be corrected.

Documentation on Dams:

All significant design data, computations, and engineering and management decisions should be documented and retained throughout the life of a dam. The documentation should cover investigations and design, construction plans and specifications, construction history, operation and maintenance instructions and history, instrumentation monitoring instructions, structural behavior history, damage, repairs and improvements, and periodic examinations and evaluations. Memoranda, reports, criteria, computations, drawings and records of all major decisions regarding the design, construction, operation and maintenance, and safety of the dam should be permanently retained and accessible in central file.

7. Mini Hydropower project Development considerations

7.1 Definition

Small Hydropower may be classified according to different criteria such as head, powerhouse layout, and installed capacity. The definition may vary at different times and in different countries implying that it has no strict definition. According to UNDO an installed capacity between 101KW and 1000KW is defined as Mini Hydropower (MHP) development.

7.2 Energy supplies in Rural Areas

The main prerequisite fir socio-economic development in an area is the acquisition of economic and reliable energy. According to statistics from the United Nations, a total installed capacity of 85 GW should be newly added in the world's rural areas so that the un electrified rural areas inhabited by 1.7 billion people will have electricity for basic needs (exclusive of industrial and agricultural loads). However due to the limitation of conventional energy resources and a shortage of funds and expertise etc, only a few millions of rural people in the world can be energized in a year. Therefore, the lack of electricity becomes a great constraint to the rural and the national economic development of a country.

At the heart of rural electrification is the development of commercial energy owing to some historic factors, vast rural areas are completely cut off from the national economy. Most energy consumption in rural areas is still from biomass and electricity occupies only a small portion of the energy consumed.

In our country more than 80% of the population is scattered in the country side consuming 88.4% of the Biomass energy out of 94.5% of Biomass energy consumption in the country (1996-statistics). On the other hand 751.128 metric tone of fuel oil was consumed out of which only about 8% of the fuel oil was consumed by rural energy consumption. This shows that the imposition on the financial balance of the country is high but urban and industrial centers are using large proportion of imported energy sources. Such disproportionate energy allocation leads to an increase in fire wood consumption in rural areas resulting in soil erosion and loss as well as a decrease in soil fertility and damage to the environment. Therefore, the promotion of rural commercial energy is a critical decision for our nation.

Those who are in favor of using conventional energy think that it all the total fire wood consumption in rural areas of the world is replaced by oil, about 0.2 billion tons of oil will be needed annually occupying only 7% of the total oil production in the world. So shortage of energy in rural areas is actually is an issue of poverty rather than an energy issue.

However, past energy crises and escalation in oil prices clearly show that this strategy is neither realistic nor cost effective Moreover; the large scale burning of Hydrocarbons would exacerbate the green house effect, making a serious ecological impact on the environment. Thus it is necessary to set up a clean rural energy structure.

Those who are in favor of a centralized energy supply think that MHP plants are neither economically feasible nor technically viable and the energy demand in rural could be better solved by the extension of large grids. Again this approach is not the case in reality. More over many rural areas are rich in MHP resources and many remote areas can not be economically energized by the extension of large grids. In reality a flexible or diversified strategy of rural electrification should be considered based on local conditions. In china diversity and decentralization of energy supply has brought effective rural economic development.

So far only few small Hydropower plants (SHP) have been developed with SCS (Self Contained System) Soar, Denbi and Yadot generate 5.00, 0.80 and 0.35 MW of electricity respectively. Out of which Debi and Yadot are Mini Hydropower plants according to the UN definition of hydropower classification

7.3 The Mini Hydropower development

In new and renewable energy sources, SHP is mature in technology. Long ago human beings learnt how to make use of water for power. In the country it is still possible to find primitive Hydraulic Devices (Water Mills). Nowadays, SHP is well developed, with the application of new technology and design to shorten its construction period and the initial cost being reduced by full use of local labour and materials as well as a series of preferential policies from government.
The main advantages of MHP are:

- its suitably for decentralized development, fully using local materials and appropriate technology with the participation of local people,
- its mature technology and small investment risk,
- its low operating costs easy maintenance and reliable power supply
- little environmental impact during construction with some positive impact on the environment
- the obvious social benefit to a developing local economy and improvements in the material and spiritual life of local residents

Hence, it is pointed out in a United Nations report that as a clean and renewable energy SHP or MHP ought to be developed as a priority for its maximum economic benefits as well as its multi purposes, such as irrigation water supply, fish breeding and ecological effects.

For developing countries, the maximum capacity of the rural industrial equipment is generally less than 100KW and rural industries can be energized by MHP if MHP resources are abundant in the region. For instance in China the unit cost of MHP is around $650– 00 and its M&O cost is much less than that of diesel or coal fueled plants. Therefore China has gone to great efforts to develop SHP and MHP, and ''to get richness by constructing MPH'' has become the common experience in hilly regions of china.

7.4 Factors of MHP development

On the basis of the experience of some countries, the following factors are required for the development of MHP:

- Rich MHP resources and certain loads
- Sufficient funds for the construction of MHP stations
- Expertise in its economic exposition
- Preferential policies from central and local governments

7.5 Preferential policy for MHP development

For instance in china, the government has stipulated a series of preferential policies to promote SHP development as follows:

- The "three self policy", namely self construction, self management and self consumption; which means that the people who invested in and constructed SHP stations have the right to manage the plant to use, to use the output of SHP plant and to obtain benefits from the station

- "Further developing SHP with benefits from existing stations which means that the benefits of SHP should be reinvested to further develop SHP should be reinvested to further develop SHP plants or local grids
- Local grids can have their own supply area and unified management system of generation, distribution and power supply and be connected to and mutually aided by large (or national) grids
- The government gives preferential loans and exemption to SHP developers

7.6 Funding

Generally speaking the unit cost of SHP or MHP is greater than that of medium and large hydropower plants and its initial investment is a great burden for local developers. The funding of SHP or MHP should mainly be self generated and be based on the particular conditions of a country. In any case a feasibility study of the project is first required for the developer or owner so as to make the right decision.
The funds for SHP or MHP can be gathered from:

- some subsidies or preferential loans from central and local governments
- loans from banks
- investment from industrial consumers and local people

7.7 Appropriate technology for MHP

- typical designs are available gates, pre-stressed concrete penstocks and pre-cast concrete poles
- many micro hydropower plants have been packaged and commercialized
- electro mechanical equipment in SHP and MHP plants have been standardized and serialized thus reducing thus SHP or MHP unit cost
- some practical devices, such as ELC (Electric Load Control), a simplified govern or (operator), auto-valves with counterweight and automatic controllers have been invented which reduces the operating cost and improved operation

7.8 Benefits of MHP

MHP has economic, social and environmental benefits such as:

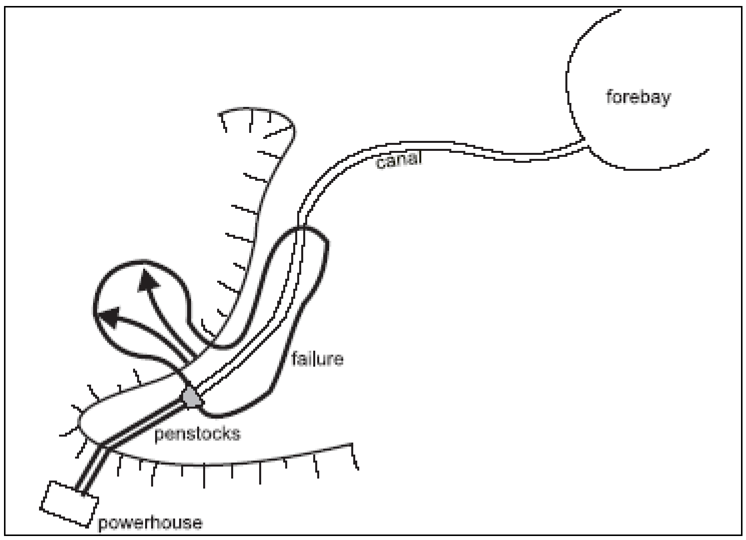
- Providing cheap power for local industry and agro-by-product processing
- MHP development can be combined with irrigation, water log control and flood prevention, thus promoting crop yields and agricultural modernization
- Increasing revenue for local government and income for local people
- Creating more jobs and reducing the migration of rural people in to cities
- Invigorating rural cultural life and improving the living standards of the local people
- MHP can be used in hilly areas for cooking, instead of firewood, hence conserving the environment
- Developing tourism in rural areas
- Benefiting social developments and stability

For example with the economic development of rural areas, there would be large numbers of the rural population moving to other industries. MHP development will help to establish more township-run enterprises, providing more employment opportunities for the rural people, who will leave the farm land bit not the rural area. It is effective in preventing the rural population from moving to cites.

Substituting electricity for fire wood gives positive effect by reducing deforestation and, hence, conserving the ecological environment as well as improving the hygiene of rural people

8. ENVIRONMETAL, SOCIAL AND POLITICAL FEASIBILITY OF HYDROPOWER PROJECTS

Impacts of hydropower schemes are highly location and technology specific. A high mountain diversion scheme, being situated in a highly sensitive area is more likely to generate impact than an integral low-head scheme in a valley. Diversion projects in mountains use the large change in elevation of a river as it flows downstream. The tail water from the power plant then reenters the river, and entire areas of the river may be bypassed by a large volume of water, when the plant is in operation. Given below is a description of possible impacts. However it is not certain that all or most of this list of descriptions will be applicable to a specific project. In the list are identified the event, persons or things affected, impact and priority at local and national levels.



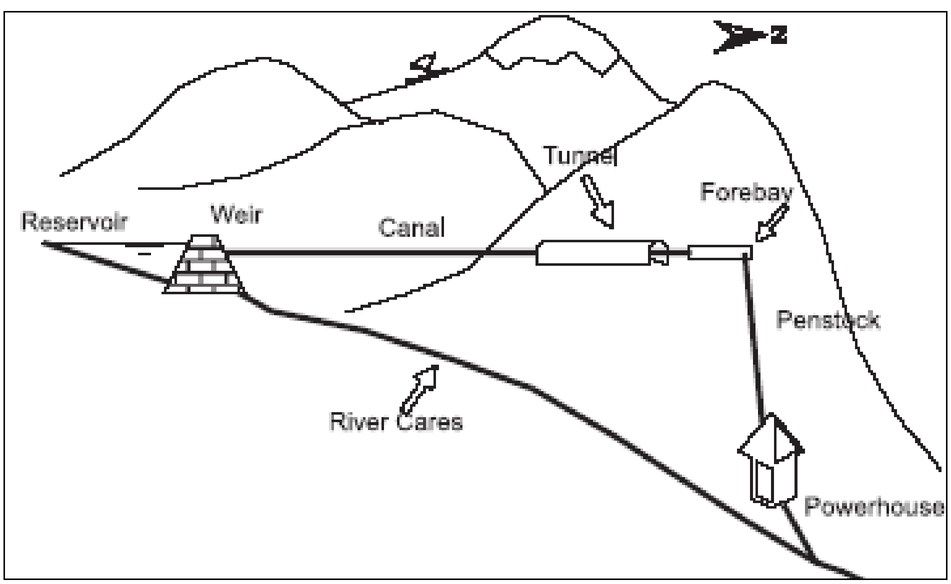


Figure 8.1: Layout of surface power house

Table 8.1 Environmental impact of a hydropower project

Event and Persons or things affected	impacts	Priority
During construction:		
Road Construction		
• General public	Noise	Low
	Accidents	Low
	Emissions	Low
• Wildlife	Noise disturbance	Low
	Collision's accidents	Medium
• Forest	Better access	Medium
	Future production loss	Medium
Accidents		
• Workers	Minor injuries	Medium
	Major injuries	High
	Death	High
Job creations		
• General publics	locally	High
	National	Medium
In operation:		
Flow alteration		
• Fish	Loss of habitat	High
• Plants	Loss of habitat	Medium
• Birds	Loss of habitat	Medium
• Wildlife	Loss of habitat	Medium
• Water quality	Contaminant dilution	Low
• General public	Loss of water falls ***(e.g. Tis Abay Fall)***	high
	Loss of recreation activities	medium
	Aesthetic effects	Medium
Excessive noise		
• Workers	On health	Medium
• General public	On health	Medium
Dams and Reservoirs		
• Agriculture	Loss of grazing area	Low
• Forestry	Loss of future production	Low
	Loss of settlement	High
	Loss of roads and crossings	High
	Loss of heritage and cultural lands	High
Aquatic ecosystem		
• Change of habitat	Local climate change	High
• General public	Global climate change	Negligible
	By methane	Not proven

• Water quality • Culture and archeological effects	Eutrophication Loss of objects & Features	Low High
Electricity transmission: ***On the construction:*** Accidents • Workers • Workers • Workers	Minor injuries Major injuries Death	Medium High High
Jobs created and increased income • General public	Local and national employment benefits	High
On the operation: Physical presence • Forestry • General public • Birds	Loss of future production Visual intrusion Injury, death	Low Medium Medium
Electromagnetic fields • General public	Cancers	Medium
Nonexistent accidents • General public	Major injuries Death	Negligible Negligible
Accidents on maintenance of transmission lines • Workers	Minor injuries Major injuries Death	Negligible Negligible Negligible
Jobs created and increased local income • General public	Local and national employment benefits	Medium

8.1 Reservoirs

The impacts generated by the construction of a dam and the creation of the adjoining reservoir include the loss of ground, the construction and opening of construction roads, working platforms, excavation works, blasting which are dependent on the dam size. Other non-negligible impacts are the barrier effect and the alteration of flow consequent to a river regulation that did not exist before.

8.2 Water intakes, open canals, penstocks, tailraces, etc.:

The impacts generated by the construction of these structures have been described in the table above e.g. noise affecting the life of the animals; danger of erosion due to the loss of vegetation consequent to the excavation work and affecting the turbidity of the water; downstream sediment deposition, etc. To mitigate such impacts it is strongly recommended that the excavation work should be undertaken in the dry season and the disturbed ground restored as soon as possible. In any case these impacts are always transitory and do not constitute a serious obstacle to the administrative authorization procedure.

In view of its protective role against riverine erosion it is wise to restore and reinforce the river bank vegetation that may have been damaged during construction of the hydraulic structures. It should be noted that the ground should be re-vegetated with indigenous species, better adapted to the local conditions.

The impact assessment study should take count of the effects of excess excavated material in the stream. To mitigate the impacts the traffic operation, avoiding of excavated material should be carefully planned prior to construction.

On the positive side it should be noted that the increase in the level of activity in an area usually economically depressed, by using local manpower and small local subcontractors during the construction phase is to be welcomed.

8.3 Impacts arising from the operation of the scheme

Sonic impacts:

The allowable level of noise depends on the local population or on isolated houses near to the powerhouse. The noise comes mainly from the turbines and, when used, from the speed increasers. Nowadays noise inside the powerhouse can be reduced, if necessary, to levels of almost unnoticeable outside.

To minimize the noise the following measures could be taken:

- Insulation of the machine hall, the noisiest room, from the adjacent rooms by means of double walls with different mass, with a layer of glass wool in between.
- Soundproofing doors
- False ceiling with noise killing characteristics
- Heavy trapdoors to the ground floor, fitted with soundproof counter trapdoors and sealing gaskets.
- Vibration damping joints between fans and ventilation ducts
- Low air velocity (4 m/sec) ducts
- Turbine rotating components dynamic balanced
- Water-cooled brushless synchronous generator
- Precision manufactured gears in the speed increaser
- Turbine casings and speed increaser casings strongly stiffened to avoid resonance and vibrations
- Anchoring of the turbine by special anti-shrinking concrete to ensure the monolithic condition between hydro unit and foundation block
- Turbine ballasting with large masses of concrete to reduce to a minimum the vibrations amplitude

8.4 Landscape impact

The quality of visual aspects is important to the public, who are increasingly reluctant to accept changes taking place in their visual environment, such things may be rejected by a part of the population, even if, in many ways they improve the environment including landscaping. The problem is particularly acute in the high mountain hydropower schemes or in schemes located in an urban area with remarkable historical character. This concern is frequently manifested in the form of public comments and with legal challenges to those developers seeking to change a well-loved landscape by developing a hydropower facility.

Each of the components that comprise a hydro scheme - powerhouse, weir, spillway, penstock, intake, tailrace, and substation and transmission lines - has potential to create a change in the visual impact of the site by introducing contrasting forms, lines, colour or textures. The design, location, and appearance of any one feature may well determine the level of public acceptance for the entire scheme.

The penstock is usually the main cause of annoyance. Its layout must be carefully studied using every natural feature - rocks, ground, and vegetation - to cover it and painting it if there is no other solution so as to minimize contrast with the background. If the penstock can be buried, this is usually the best solution. Expansion joints and concrete anchor blocks can then be reduced or eliminated; the ground is returned to its original state and the pipe does not form a barrier to the passage of wild life.

The powerhouse, with the intake, the penstock tailrace and transmission lines must be skillfully inserted into the landscape. Any mitigation strategies should be incorporated in the project, usually without too much extra cost to facilitate permit approval.

8.5 Preliminary Questions

In assessing the feasibility of hydro power developments, it is important to consider early the social, political, and environmental feasibility at a proposed site or in a resource area that has potential sites. The purpose of such an evaluation is to determine whether there are restraints due to social concerns such as disruption of peoples' lives or the existing economy, institutional or legal restraints: and/or environmental concerns that will make proceeding with development unwise. Further, it is important to quantify the restraints to determine whether more time should be devoted to the study of social, political, or environmental acceptability and whether mitigation can be provided so that a hydro plant can be economically installed and operated.

Two questions need to be asked and answered:

First, when should the evaluation be done?
Second, who should make the evaluation?

Assessment of social, political, and environmental feasibility should proceed concurrently with the hydrologic studies and inventorying of other pertinent physical data as well as in time sequence with the economic analysis. Necessary information to make an evaluation will often be incomplete and the evaluator will want to collect more information to make a better evaluation. Evaluators should be cautioned that collecting impact data can take several years in some cases. The decision maker may want and need to make a determination before the data collection can be completed. ***Who should do the evaluation?*** This is normally not a technological or engineering type of evaluation. ***However, the engineer is often responsible for this evaluation in the planning process.*** The engineer must depend on the judgment of professionally qualified people in the various disciplines involved, such as biologists, social scientists, and legal experts who have relevant experience qualifications.

These assessments of social, political, and environmental feasibility need to be made to screen various alternatives in certain political subdivisions, river basins, and government jurisdictions. The assessments, due to limits on time and funds, and the nature of the evaluations, often become ***subjective*** and depend on ***indexed representations*** of the various factors involved. Unlike the economic evaluation, there are no common units of measurements.
At present there is no established methodology that is universally accepted by planners and decision makers.

8.6 Checklist of Considerations

In referring to the assessment of social, political, and environmental feasibility, the words used to refer to the variables in the appraisal include such words as ***factors, parameters, issues, and considerations.*** Important in the evaluation is first to develop a comprehensive checklist of the considerations that need to be assessed. This hopefully will ensure that none of the considerations will be overlooked. The degree of sophistication with which one weighs and determines the impact of hydropower development on various factors being considered will be quite site specific and depend on time and funding limitations. The following is a comprehensive checklist that might be used in developing and using methodologies.

i. Natural considerations:

a. Terrestrial
- Soils
- Landforms
- Seismic activity

b. Hydrological
- Surface water levels
- Surface water quantities
- Surface water quality
- Ground water levels
- Groundwater quantities
- Groundwater quality

c. Biological
- Vegetation
- Fish and aquatic life
- Birds
- Terrestrial animals

d. Atmospheric
- Air quality
- Air movement

ii. Cultural and human considerations:

a. Social
- Scenic views and vistas
- Open-space qualities
- Historical and archaeological sites
- Rare and unique species
- Health and safety
- Ambient noise level
- Residential integrity

b. Local economy
- Employment (short-term)
- Employment (long-term)
- Housing (short-term)
- Housing (long-term)
- Fiscal effects on local government
- Business activity

c. Land use and land value
- Agricultural
- Residential
- Commercial
- Industrial
- Other (public domain, public areas)

d. Infrastructure
- Transportation
- Utilities
- Waste disposal

- Government service
- Educational opportunity and facilities

e. Recreation

- Hunting
- Fishing
- Boating
- Swimming
- Pick-nicking
- Hiking/biking

8.7 Evaluation Methodologies

Numerous approaches have been used to systematize and quantify the assessment process. Two techniques are presented, an ***impact matrix and a factor profile approach.***

Impact matrix approach technique requires the development of a matrix in which certain activities or actions are arrayed against the various considerations. If the environmental impact appraisal is very broad, it can include the social, political, and economic issues that must be weighed. The actions or activities for planning, development, and operating a hydropower development are arrayed on the vertical scale of a matrix table and the various social, political, and environmental considerations are arrayed on the horizontal scale.

The practice is to enter into the matrix table a symbol to indicate the extent, to which a specific activity or sub activity will affect the particular consideration or sub factor. The entry can be qualitatively expressed in a scaling or rating approach by assigning the symbols, indicating the impacts have significant, limited and insignificant impacts on the resources area.

This implies the evaluator has good understanding of the base considerations as they exist or are expected to exist before construction and development proceeds. Naturally, this takes on a ***subjective weighing*** because it is not always easy to document why a particular entry was made. ***It implies a weighing of impact before and after development and even at stages during construction.***

Another technique that has been used in siting highways (Oglesby, Bishop, and Willike, 1970), in a water resource planning effort (Bishop, 1972), and in an appraisal of recreational water bodies (Milligan and Warnick, 1973) is a ***factor profile analysis.*** This is a ***graphical representation of subjective scaling*** of the impact or importance of various considerations on the overall feasibility of development. Feasibility should be considered from four principal areas of concern: ***(1) engineering and technological feasibility, (2) social acceptability, (3) environmental acceptability, and (4) economic feasibility.*** Figure 8.3 arrays the considerations environmental evaluation in just three main categories and thirteen sub factors. In Figure B, a bar graph has been developed for each of the sub factors of the major considerations. This requires the subjective scaling of impact the hydropower development will have either during construction or during operation, or both. A magnitude representation from 0 to -10 and 0 to +10 is made of each of the sub factors in the factor profile. This scaling is here referred to as an ***attribute number.*** Note that it can be either negative or positive, or both. For instance, a hydropower development might disrupt fish habitat by decreasing flows during certain times and cause a valuation of a negative entry in the factor profile. At the same time the flow release might improve the flows at other times, making a positive entry on the factor profile. Guidelines and ways of consistently arriving at the attribute number is the challenging problem. Here is where it is important to call on the help of professionals to develop the guidelines or scaling the attribute number and actually making the assessment.

Considerations of Impact / Devpt Activity		Natural Environmental Considerations									Cultural and Human considerations					
		Terrestrial			Hydrological			Biological	Atmospheric		Local Economy		Land use		Infrastructures	
		Soils	Land forms	Etc	Surface W. levels	G. Water levels	Veg.	Aquatic life	Air quality	Air movement	Employment	Business Activity	Agriculture	Residential	Transportation	Utilities
Construction Activity	Earth Work, Clearing		◙													
	Disposal		◙				◙									
	Coffer dam/Dewatering			□				◙								
	Equipment use			□												
	Material Transport			□												
	Temp. Structures & facilities		◙													
	Workers & Equip. access			□												
	Work Influx					◙					●	□		◙	◙	
Physical plant and Operation	Permanent facilities							◙	□	□			□			
	Electrical distribution								□	□			□			
	Electrical generation								□	□			□			◙
	Flood control								□	□			□			
	Impoundment control				◙								□			
	Worker & equipment access												□			
	Security & safety												□			

Figure 8.2: ***Impact matrix*** approach

● Significant Impact
◙ Limited Impact
□ Insignificant Impact

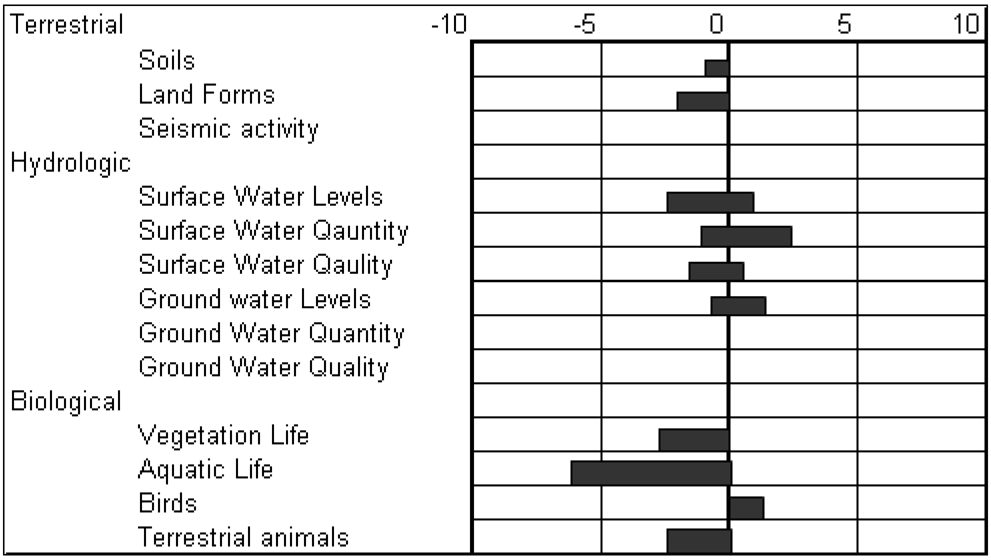


Figure 8.3: Example of factor profile for evaluating impact of hydropower development on environmental acceptability

To illustrate the technique more fully, a factor profile for just one category of the cultural and human considerations has been developed and presented in Figure 8.4. This is the social category with thirteen sub factors. Guidelines for assigning numerical value for the attribute numbers of two of the considerations are given below.

E.g. for scenic Views:

- If a major scenic vista or attraction such as waterfall would be inundated and destroyed, a -10 could be assigned.
- If a white-water cascading reach of stream would be inundated, a -7 could be assigned.
- If the attractive stream bank vegetation will be partially destroyed, a -5 could be assigned.
- If there appears to be negligible effect, a 0 could be assigned.
- If a barren, ravaged stream channel is replaced with a mirrored lake, a +4 could be assigned.

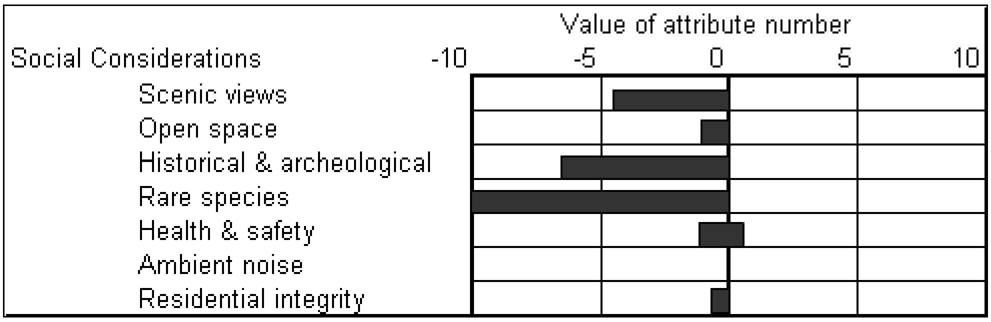


Figure 8.4: Example factor profile for evaluating impact of hydropower development on social conditions.

For open-space qualities:

- If several thousand acres of open space is inundated and penstock and canals cross and mar the open nature of the area, a-10 could be assigned.
- If a large area of open space is inundated, a -7 could be assigned.
- If a limited area of open space is disrupted, a -3 could be assigned.
- If no apparent change will occur in the open-space area, a 0 could be assigned.
- If impoundment and control of stream allows use of open space and new vegetation creates a more open and attractive area, a +5 could be assigned.

The factor profile can give a Visual representation of restraints. If desired, it is possible to sum the various values of attribute numbers. It is also possible to give added weight to certain of the considerations by giving a weighting factor to a given consideration or sub factor.

8.8 Other Social and Political Considerations

Land Ownership:

In hydropower feasibility studies, land ownership is an important consideration. In many cases the site with the best development potential presents a problem because the entity that wants to develop the energy does not have ownership of the land. The land is in government ownership, or there are certain legal restraints on the land. Land ownership problems need early attention in planning and may take on an inordinate importance in the feasibility determination and the implementation of a hydropower development.

Legal Considerations:

Legal considerations are important in the appraisal of social and political feasibility of hydropower developments: water rights, regional state regulatory permits, and federal licensing.

Depending on the state involved, there are other legal requirements that must be met and require attention even at the feasibility study level. Typical of these requirements are stream channel alteration permits, public utility certificates for study of need and convenience, state environmental impact statements, and proof of compliance with state water quality standards. Because of the direct impact of hydropower developments on the stream's fishery resources, there always should be requirements and political acceptance that must be sought from the legal authority. These problems must be addressed as the planning proceeds.

One of the challenging problems facing the engineer is responding political, institutional problems and helping to decide whether the development should be made by private entities, private utilities, rural electric cooperatives, public power entities, state governments, or the federal government. In many cases the social, political, and environmental feasibility will depend on which type of entity gains the opportunity to proceed with study, design and development.

9. ECONOMIC AND FINANCIAL EVALUTION OF HYDROPOWER PROJECTS

Objectives:

When you have completed this chapter you should be able to:

- Prepare for an economic and financial appraisal by assembling information documents for use during the appraisal and by deciding up on the degree to which this information
- Differentiate between an economic and a financial appraisal and discuss the criteria for economic and financial feasibility.
- Define time-value related terms that lead to the basic equations required for an economic and financial analysis
- Define capital costs, annual costs, direct benefit and indirect benefit.
- Use the net present value (NPV), benefit-cost ratio (B/C) and internal rate of return (IRR) methods in economic appraisal.
- Carry out a financial cash-flow analysis, a financial balance analysis and a generation cost profit analysis for financial appraisal
- Carry out uncertainty analysis.

9.1 Introduction

After our technical deliberations, we arrive at the economic and financial appraisal.

The objective of an economic and financial appraisal are first to ***provide an economic basis for deciding whether or not to implement a projec***t, and secondly ***to examine promising development alternative in an economic respect to determine which is the mist attractive***

An economic appraisal is based on the benefits and costs from the viewpoint of society as a whole, while a financial appraisal is viewed from the perspective of the project sponsor, and states whether the tangible value of the output of the project will be sufficient to amortize (pay back) the project loan, pay operation and maintenance cost, and meet the interest on other financial obligations.

A series of information documents must be assembled in an orderly fashion before the appraisal can be conducted as follows:

- Information on the Input of the Project : the capital cost of civil engineering and electro mechanical equipment; operating and maintenance costs; overhaul costs; useful life of the civil engineering and electro mechanical equipment and their rate of amortization.
- Information on project construction: the period of construction, its progress and date of putting into operation.
- Information on Hydropower plant parameters: firm power, peaking power (or operating power), installed capacity and annual generation of the power plant.
- Information from Market Analysis: the energy purchaser; rate of capacity and energy (tariff); market price of materials and equipment; labor costs and their shadow prices obtained from the planning department of the government.
- Information from financing: fund sources and funding; its yearly installment during construction; interest rate; the basic economic and financial discount rate and rate of escalation.
- Information on Alternative Energy Sources: construction costs; energy costs, operation and maintenance costs, fuel prices, etc.
- Information on Socio-Economic elements: institutions; codes; policies and other socio-economic factors concerning the environment industrial and agricultural productivities, etc.
- Information on Other Cost Rates, e.g. fees necessary for a license and low producers; categories of taxes and their rates; rates of insurance etc.

9.2 Economic and financial feasibility Basic Equations concerning time value

9.2.1 Economic and financial feasibility

Project feasibility includes two aspects: ***technical and economic***; the two aspects complement one another. Economic feasibility is based on technical feasibility. It is prohibited to sacrifice project safety in order to gain a fabricated feasibility. So, economic feasibility is always decisive when considering the engineering safety of an alternative project scheme.

Economic and financial appraisals are two aspects defining the project feasibility in respect of economics. The project sponsor pursuing maximum profits may ignore the environmental impact or may harm the national interest, which will lead to the project would be financially infeasible on one hand, on the other hand the project would be financially infeasible owing to the large discrepancy between theoretical energy costs and actual tariffs. Therefore the justification for financial feasibility becomes difficult. The project sponsor should take measure to strive to reduce investment costs, seek loans with a low interest, change the ways of funding, etc.

Economic analysis deals primarily with the development and applications of benefit cost analysis, which is the most frequently used procedure for project economic evaluation. In principle, the benefits include tangible and intangible benefits, even the social benefits evaluated in terms of money; shadow prices are introduced in benefit cost calculation, so adjustments should be made in salaries, labor, materials, and equipment, as well as energy and fuel prices taxes and subsidies should be deducted from construction and operation maintenance cost; and when we use the time value of money to discount the cash flow, we should use the social opportunity cost instead of replacing the financial interest rate. However, in most regions of the developing countries the condition is limited to shadow price adjustment which can only be replaced by market price or import price, and only tangible benefits and costs can be dealt with accurately from an accounting point of view. ***Economic feasibility is considered from the stand point of the sponsor. When total benefits accruing from the project exceed the total costs incurred, the project is regarded as economically feasible.***

Financial feasibility may be defined as a project's ability to obtain funds for implementation and repay these funds on a self-liquidating basis with acceptable risks at realistic interest rates. Financial analysis can be simply interpreted as a cash-flow analysis.

9.2.2 Basic equations concerning time value

The following time value related terms must be defined first before we establish the basic equations for the economic and financial analysis.

Economic Life (useful life) and the Calculation Period:

Economic life is the time, during which the project can be operated normally. In general, that is 30-40 years for civil engineering and 15-25 years for turbine generator sets (10-15 years for diesel sets.) Renewal of the main parts of the equipment or capital repair in civil engineering is needed after that period. In cash flow calculation we sometimes take the calculation period to be that which may equal the economic life of equipment, in this case the residual value of the civil engineering should be considered as a future benefit in cost benefit analysis; or we take the calculation period to be that which equals the economic life of the civil engineering, in this case the expenses in the renewal of the main parts of the equipment must be considered as future capital investment.

Discount Rate:

The discount rate is the cost of money reflecting the time value of money. The proper rate to use for testing economic feasibility is the opportunity cost of capital to society. This is the rate of return that could be earned by investing the capital cost of the project in a venture of similar risk or an alternative marginal project. The social discount rates are different in different countries; usually it takes around 10 per cent.

Interest Rate:

This is the fee that must be paid by the user for the lender's capital. It is used to ascertain financial feasibility. The interest rate is set in the capital market and fluctuates with changes in the health of the economy and government fiscal and monetary policies.

Present Value:

This is the value obtained by discounting all future costs and revenues into the present timeframe so that they can be compared on a current monetary basis. The sum of these values represents the net present value.

Annual Equivalent Value:

This is the capital value of an annuity, the cumulated present values of which (in n years) equals the total initial capital cost, or the capital is recovered in n years by an annual equivalent value under a given discount rate.

If we ser **P** = present value; **F** = Future expenditure; **A** = annual equivalent value; **i** = discount rate or interest rate; **n** = economic life or calculation period, then we get the following useful equations:

1. The single–payment future–value equation

$$F = P(1+i)^n, \qquad (9.1)$$

   Where $(1+i)^n$ the present–value factor with a single payment

2. The single–payment present–value equitation

$$P = F/(1+i)^n, \qquad (9.2)$$

   Where $1/(1+i)^n$ the present–value factor with a single payment

3. The uniform payment future–value equation

$$F = A[(1+i)^n - 1]/i, \qquad (9.3)$$

   Where $[(1+i)^n - 1]/i$ the future–value factor with uniform payments

4. The sinking-fund equation

$$A = F * i/[(1+i)^n - 1], \qquad (9.4)$$

   Where $i/(1+i)^n - 1$ the sinking – fund factor

5. The present value of an annuity equitation

$$P = A[(1+i)^n - 1]/i\ (1+i)^n, \qquad (9.5)$$

   Where $[(1+i)^n - 1]/i(1+i)^n$ the present value of an annuity factor

6. The capital – recovery equitation

$$A = P * i(1+i)^n / [(1+i)^n - 1], \qquad (9.6)$$

   Where $i(1+i)^n/[(1+i)^n - 1]$ the capital–recovery factor

Figure 9.1 shows the cash flow and the relationship between P, A, *n* and F.
Equations (9.2) and (9.5) are also suitable for the present–value calculation of benefit.

9.3 Costs and Benefits

9.3.1 Costs

Capital costs: This is the sum of money invested in a project (including its interest during construction) before its completion. Accordingly, the project sponsor will return the money from the energy sales to pay back the initial expenditure and operating costs, and at the same time retain the remaining profit for himself.

Figure 9.1: The Relation between P, A, *n* and F

In general the capital cost of preliminary design is classified by the following items:

- Civil engineering;
- Electro-mechanical equipment and its installation;
- Equipment such as the gate, hoist, penstock and its installation;
- Temporary engineering;
- Compensation for filling the reservoir;
- Other expenses, e.g. administration of the construction unit, operation preparation, scientific research, exploration and design, construction monitoring , establishment of the base of the construction enterprises, legal procedures, certificates, taxes and insurance,
- Reservation for unforeseen expenditures;
- Interest during the construction period;

Table 9.1: An example for estimating the total investment

Description	**Cost (10^3 US$)**
Direct costs	
• Civil	4636 .50
• penstock	155.90
• turbine–generator and its accessories	450.00
• Substations (step-up and step-down)	240.00
• Transmission line	472.00
• Total contingencies	5954.00
Contingencies	
• 15% for electromechanical equipment	174.00
• 20% for civil and penstock	958.00
• Total contingencies	1132.00
Engineering costs	
• 15% for direct costs and contingencies	1063.00
Administration and others	
➢ 10% for direct costs and contingencies	708.00
Total	8859.00
Interest over 2 years' period of construction	1373.00
Total Capital costs of project	10232.00

In a feasibility study, the items can be roughly divided as shown in Table 9.1

Annual costs:

Annual costs include the annual capital cost (the financial costs for loan amortization and interest) and the annual operation and maintenance costs, the latter involving salaries, material expenses, water fees, overhaul expenses, insurance, interim replacement and administration, etc. If the capital cost of the transmission line is included in the total investment, then the annual cost will have two parts: power generation and power supply.

The rate of the annual capital cost equals the capital recovery factor; the rate of the annual operation and maintenance costs.

9.3.2 Benefits

There are two kinds of benefit: direct benefit and indirect benefit

The direct benefit is mainly from the benefits of the energy sale, as in the following expression

$$B_e = E_e(1-\beta)\ (1+\gamma)p, \qquad (9.7)$$

Where

B_e = benefit from energy sale;

E_e = effective annual energy generation, i.e. the total net energy output given out by the generator of the hydropower plant during the year after the deduction of energy loss in outage;

$\beta =$ Plant use factor

$\gamma =$ grid loss factor

P = energy price

In Equation (9.7) $E_e(1-\beta)(1-\gamma)$ represents the amount of electric energy on sale. The project sponsor should decide which energy purchaser will be willing to purchase the energy output from the hydropower plant and what selling price of the energy can be obtained in the market.

Besides the benefit from energy sales, there would be a benefit from multipurpose utilization

Indirect benefit involves tangible and intangible benefits; the former can be calculated in money terms, e.g. pumping irrigation will increase the yield of the grain harvest, electric lighting may save kerosene expenses; the latter is uncountable e.g. to raise the standard of living of the society, also reduces deforestation, increase the opportunity of employment.

Economic analysis should consider the social benefit as far as possible, while financial analysis deals only with direct cash flow.

9.4 METHODS OF ECONOMIC APPRAISAL

The net present value (NPV), benefit–cost ratio (B/C) and internal rate of return (IRR) are methods generally used in an economic appraisal.

9.4.1 The net present value method

This method is useful for ranking multiple projects. If we set the first year of construction as the base year the procedure is to discount the net benefit (i.e. benefits minus cost) from each year to the base year, then to obtain their cumulative sum:

$$NPV = \sum_{j=0}^{n} \frac{B_j - C_j}{(1+i)^j} \qquad (9.8)$$

A diagram of equation (9.8) is shown Figure 9.3; when *m* = the construction period; A = annual operating and maintenance cost, B = annual benefit, P = annual investment. C in equation (9.8) involves P and A.

If we set the first year of operation of the hydropower plant as the base year, and the annual capital input, annual operating and maintenance costs, and annual benefit are uniform in distribution i.e., $p_o = P_l = P_2 .... = P,\ A_{n+1} = A_{n+2} = A_{n+3},\ B_{n+1} = B_{n+2} + B_{n+3} = B,,$ then the NPV can be directly calculated by Equation (9.9)

$$NPV = (B - A)\frac{(1+i)^n - 1}{i(1+i)^n} - \frac{p}{m}\left[\frac{(1+i)^{m+1} - 1}{i} - 1\right] \qquad (9.9)$$

If any residual values **R** exist at the end of the calculation period they should join the benefit flow to be discounted.

Figure 9.2: Simple diagram if Equation (9.8)

Example: Given a cash flow as shown in Table 9.2, calculate the NPV when

(a) ***i*** = 10%, price escalation = 0;

(b) ***i*** = 10%, price escalation = 7%

From the above calculation in Table 9.2 we find that NPV < 0 when the rate of the price escalation = 0 and NPV > 0 when the rate of the price escalation = 7%; hence the price escalation has a large influence on the result.

If we set the first year of commissioning as the base year, as shown in Figure 9.4, and the capital cost is uniformly invested over two years, then according to Equation (9.9) we get

Figure 9.3: simple diagram of Equation (9.9)

$$NPV' = (B-A)\frac{(1+i)^n - 1}{i(1+i)^n} - \frac{p}{m}\left[\frac{(1+i)^{m+1}-1}{i} - 1\right]$$
$$= (24500-4500)\frac{(1+0.1)^{15}-1}{0.1(1+0.1)^{15}} - \frac{150000}{2}\left[\frac{(1+0.1)^{2+1}-1}{0.1} - 1\right]$$
$$= -20500(US\ dollars).$$

Table 9.2: Calculation of NPV (in $US)

(a) ***i*** = 10% rate of price escalation = 0

Year	Capital Costs	O&M Cost	Annual Benefit	Net Annual Benefit (4)-(2)-(3)	Present Value Factor	Net Present Value (5)x (6)
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**
0	60 000			-60000	1.0	-60 000
1	90 000			-90000	0.909	-18 818
2		4500	245000	20000	0.826	16 529
3		4500	245000	20000	0.751	150.26
4		4500	245000	20000	0.683	1.660
5		4500	245000	20000	0.620	12418
6		4500	245000	20000	0.564	11289
7		4500	245000	20000	0.513	10263
8		4500	245000	20000	0.466	9330
9		4500	245000	20000	0.426	8482
10		4500	245000	20000	0.385	7711
11		4500	245000	20000	0.350	7010
12		4500	245000	20000	0.318	6327
13		4500	245000	20000	0.289	5793
14		4500	245000	20000	0.263	5266
$\sum$:. NPV	150 000 = - 12 666	54000				-12666

(b) ***i*** = 10% rate of price escalation = 7%

Year	Capital Costs	OM cost	Annual benefit	Net annual benefit (4)-(2)-(3)	Present value Factor	Net Present value (5)x (6)
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**
0	60000			-60000	1.000	-60 000
1	96300			-96300	0.909	-87 536
2		5152	28050	22898	0.826	18924
3		5512	30013	24501	0.751	18408
4		5898	32114	26216	0.683	17906
5		6311	34363	28051	0.620	17417
6		6753	36768	30015	0.564	16942
7		7226	39341	32115	0.513	16480
8		7732	42095	34363	0.4666	16031
9		8273	45042	3969	0.424	15594
10		8852	48195	39343	0.385	15168
11		9472	51569	42097	0.385	14755
12		10135	55178	45043	0.318	14352
13		10844	59041	48197	0.289	13961
14		11603	63174	51571	0.263	13563
$\sum$:. NPV	156 300 = 60 377	103736	564942			+60 377

9.4.2 The benefit – cost ratio method

The benefit–cost ratio is the ratio of the present value of benefit to that of cost. The criterion $B/C \geq 1$:

$$B/C = \frac{\sum_{j=0}^{n} B_j / (1+i)^j}{\sum_{j=0}^{n} C_j / (1+i)^j} \qquad (9.10)$$

In the above example with an escalation rate of 7 per cent, if we multiply columns (2), (3) and (4) by column (6) in Table 9.2(b), and add each of the said columns, we then obtain that the present value of the escalating stream of benefit is \$256700 and that of the escalating stream of costs is \$194700. The B/C ratio is then 1.32 indicating an economically feasible project

The B/C ratio can also be attained by converting the capital cost and its interest during the construction period to an annuity value, then

$$B/C = \frac{B}{(P+I)\dfrac{i(1+i)^n}{(1+i)^n - 1} + A} \qquad (9.11)$$

Where
B = annual benefit
A = annual operation and maintenance costs
P = total investment
I = interest during the construction period
n = calculation period from the first year of commissioning
$\dfrac{i(1+i)^n}{(1+i)^n - 1}$ capital recovery factor
The B/C ratio does not give the amount of net benefit. A project having the largest ratio may not yield the largest benefit

9.4.3 The internal rate of return method

The internal rate of return (IRR) is that discount rate at which the net present value is equal to zero. All projects that have an internal rate of return less than the opportunity cost of capital should be rejected. IRR is calculated through an iterative process, which is suitable for spread sheet computer processing.

Like the NPV, IRR incorporates all the pertinent economic data, but the criterion does not reflect any information on project scale and, consequently it cannot be used as the sole ranking criterion. The IRR expression is:

$$NPV = \sum_{j=0}^{n} \frac{B_j - C_j}{(1+IRR)^j} = 0, \qquad (9.12)$$

Where IRR – ***i*** on the condition that NPV = 0
If the investment, annual operating maintenance cost, and annual benefit are uniform, we can find the IRR by equation (9.9) through iterative calculation.

9.5 Methods of financial appraisal

The criteria for financial appraisal consist of the financial net present value, financial internal rate of return, financial B/C ratio, and the payback period of investment and payback period of the loan through a financial cash–flow analysis, a financial balance analysis and a cost–profit analysis. In addition, unit investment per kilowatt–hour and unit cost of energy are all economic indices subordinate to the main criteria.

9.5.1 Financial cash-flow analysis

A specifically designed format may be necessary to proceed with a cash-flow analysis. The out flowing cash includes the investment in fixed assets, annual operating and maintenances costs, financing for the renewal of electromechanical equipment during the calculation period, tax and insurance. The inflowing cash includes revenue from energy sales, returns on the residual value of fixed assets and others. By a similar approach to that of economic cash flow calculation, we obtain the net present value (using a basic financial discount rate), the financial internal rate of return, the financial B/C ratio, and the static payback period of investment (the total of the years when the cumulative net cash equals or is greater than the total investment without discounting).

In Table 9.3 a kind of financial cash–flow calculation is illustrated.
The result is:
NPV = $US 207,333 (from column (10);
IRR = 19.8% (from a tentative calculation of ***i*** = IRR = 19.8%, we obtain NPV = 0)

The static payback period = 6 years (from column (9) in Table 9.3, the sum of the net benefit > 0 in the sixth year).

Table 9.3: A financial cash – flow calculation (10^3 $US)

Year	Capital Cost	O&M(10% escalation)	Total Cost (2)+(3)	Benefit (10% escalation)	Net benefit (3)-(4)	Present value factor (12.5%)	Net present value (6)x(7)	Sum of Net benefit Σ (6)	Sum of Net present valueΣ (8)
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**
0	375000		375000		-375000	1.0000	-375000	-375000	-375000
1		16 500	16500	67375	50785	0.8889	45223	-324125	-329777
2		18 150	18150	74112	55962	0.7901	44216	-268163	-285561
3		19 965	19965	81524	61559	0.7023	43235	-206604	-202326
4		21 961	21961	89676	67715	0.6243	42270	-138089	-200048
5		24 158	24158	98644	74486	0.5549	41334	-64403	-158714
6		26 573	26573	108508	81935	0.4933	40416	17532	-118298
7		29 231	29231	119359	90128	0.4385	39517	107660	-73731
8		32 154	32154	131295	99141	0.3897	38639	206801	-40142
9		35 369	35369	144424	109055	0.3464	37781	315856	-2361
10		38 906	38906	158867	119961	0.3079	36942	435817	34581
11		42 797	42797	174753	131956	0.2737	36120	567773	70701
12		47 076	47076	192229	145153	0.2433	35318	712926	109019
13		51 784	51784	211452	159668	0.2163	34533	872594	140556
14		56 962	56962	232597	175635	0.1923	33766	1048229	174318
15		62 659	62659	255856	193197	0.1709	33 015	1241426	207333

9.5.2 Financial balance and the payback period of the loan

Financial balance aims to balance the source of income and expenditure by a year series in order to see the surplus and deficit of every year, then find out the payback period of the loan.

The sources of income include the finance from the loan, bonds and credit, energy and power values.

Expenditure involves project investment, interest during construction, debt service, operating and maintenance costs, fund drawing, tax and insurance etc.
In the calculation of the payback period, the debt should be liquidated by annual profits and revenues. The government would provide policies to accelerate the liquidation e.g. income tax is not payable during the pack back period, to pay back the loan through the partial amortization fund, etc.

Table 9.4 shows an example of a finical balance of a project.

From that balance we have found that the negative cash flows occur in 1981, 1982, 1984, and 1985, which must be covered from other financial resources. However, there is a positive cash flow during the first complete year of operation

Table 9.4: An example of a financial balance

Expenditure and incomes	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**	**1987**	**1988**
Expenditure								
1. Engineering, administration, Financial and legal	73	94	173	104	62			
2. Construction and contingencies				562	563			
3. Interest during construction				157	79			
4. Debt service on loan and bonds					129	249	249	249
5. O&M escalated at 10%					10	24	27	29
Total expenditure	**73**	**94**	**173**	**823**	**843**	**273**	**276**	**278**
Income								
1. Loan and bond sale	45		1812					
2. Interest returned on bond money				121	60			
3. Capacity credit					7	15	15	15
4. Power generation revenue escalated at 10%				-	207	455	501	551
Total income Balance	**45**	**0**	**1812**	**121**	**247**	**470**	**516**	**566**
Balance	-28	-94	1639	-702	-569	197	240	288
Expenditure and incomes	**1989**	**1990**	**1991**	**1992**	**1993**	**1994**	**1995**	
Expenditure								
1. Engineering, administration, Financial and legal								
2. Construction and contingencies								
3. Interest during construction								
4. Debt service on loan and bonds	249	249	249	249	249	249	248	
5. O&M escalated at 10%	32	35	39	43	47	52	52	
Total expenditure	**281**	**284**	**288**	**292**	**296**	**301**	**292**	
Income								
1. Loan and bond sale								
2. Interest returned on bond money								
3. Capacity credit	15	15	15	15	15	15	15	
4. Power generation revenue escalated at 10%	606	667	732	807	887	976	976	
Total income Balance	**621**	**682**	**748**	**822**	**902**	**991**	**991**	
Balance	340	398	460	530	606	690	699	

If the sponsor obtains a short-term loan from the bank with an interest rate of 13 per cent to fill the deficit in 1981 and 1982 and will pay back the loan in 1983, then we list the payback-period calculation as shown in Table 9.5 (assume that the amortization fund is paid for the debt, and taxes are exempt, so that only O&M expenses are deducted from the generation benefit).
From the calculation in Table 9.5, we can see that the project sponsor will liquidate the debt during the six years after putting the project into operation (in table 9.4 the planner gave the debt service until 1995, which is safe).

For brief estimation in a feasibility study, the method of annual equivalent cost can be applied to calculate the payback period. Let us take the same example as shown in Table 9.3. If we assume 8.5 years for the payback period, ***i*** = 12.5% percent, the total capital cost $375000. Then the capital recovery factors equals 0.1976, and the annual capital cost is $74124. From the calculation in Table 9.6, we find that the debt can be paid back at the ninth year (from column (7)

Table 9.5: The payback period calculation ($10^3$$ US)

Items	**1981**	**1982**	**1983**	**1984**	**1985**	**1986**
1. Cumulative borrowed capital at the beginning of the year	0	77.7	187.9	2024.3	2166.4	2184.0
2. Borrowed capital of this year	73.0	94.0	1812.0	-121.0	-67.0	-15.0
3. Interest of this year	4.7	16.2	142.2	263.1	281.6	283.9
4. Sum of capital and interest	77.7	187.9	2024.3	2166.4	2381.0	2452.9
5. Revenue from generation with 10% escalation	0	0	0	0	207	455
6. O&M expenses with 10% escalation	0	0	0	0	10	24
7. Sinking fund	0	0	0	0	197	431
8. Debt at the end of the year	77.7	187.9	2024.3	2166.4	2184.0	2021.9
Items	**1987**	**1988**	**1989**	**1990**	**1991**	
1. Cumulative borrowed capital at the beginning of the year	2021.9	1795.7	1492.1	1097.1	592.7	
2. Borrowed capital of this year	-15.0	-15.0	-15.0	-15.0	-15.0	
3. Interest of this year	262.8	233.4	194.0	142.6	77.1	
4. Sum of capital and interest	2269.9	2014.1	1671.1	1224.1	654.7	
5. Revenue from generation with 10% escalation	501.0	551.0	606.0	667.0	732.0	
6. O&M expenses with 10% escalation	27.0	29.0	32.0	35.0	39.0	
7. Sinking fund	474.0	522.0	574.0	632.0	693.0	
8. Debt at the end of the year	1795.7	1492.1	1097.1	592.7	-38.3	

Note: (1) the figure –121 is interest returned on the bond money; -67 is that plus capacity Credit; -15 is capacity credit.
(2) Half interest is considered for the borrowed capital of the present year.
(3) All the data are transcribed from Table 9.4

9.5.3 Generation cost and profit analysis

Generation costs are the sum of the annual operating and maintenance costs plus the amortization cost. The amortization cost depends on the amortization rate, which equals the inverse of the amortization period. However, the amortization period can be shortened in order to accelerate amortization. Unit generation cost in an important index on which the energy price is mainly based. Unit generation cost is the generation cost divided by the annual energy output. Here the annual energy output is the net energy given out to the grid or user after deducting the plant use and outage losses (see Equation (9.7) of Sub section 9.3.2)

Table 9.6: Annual equivalent cost used for the payback period calculation $US

Year	Debt service	O&M (10% escalation	Total cost	Benefit (10% escalation)	Net cash flow (50-(4)	Sum of net cash flow
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**
1	74124	16500	90624	76375	-14249	-14249
2	74124	18150	92274	74112	-18162	-32411
3	74124	19965	94089	18524	-12565	-44976
4	74124	21961	96085	89676	-6409	-51385
5	74124	24158	98284	98644	360	-51025
6	74124	26573	100697	108508	7811	-43214
7	74124	29231	103355	119359	16004	-27210
8	74124	32154	106278	131259	25017	-2193
9	74124	35369	109493	144244	34931	+32 738

The electrical energy must be sent to consumers for sale. The cost of energy sales equals the generation cost plus the energy supply cost. The unit cost of energy sales equals the cost of energy sales divided by the annual energy for sale. Here the annual energy for sale is the net energy sold to consumers after deducting the transmission losses (see equation (9.7) of Sub section 9.3.2).

The profit obtained from power generation is the net benefit from the energy sales, which equals the total benefit of energy sales from which the cost of energy sales and taxes (or other terms of expenses) is deducted. The profit is first used to repay debts, then for private or group benefit.

9.5.4 Uncertainty Analysis

For hydropower projects there would be a lacking of certainty about capital cost estimates, future annual costs and the future value of energy. Uncertainty analysis aims to analyze the capability to ensure an outcome unfavorable to the project sponsor. This risk should be analyzed and minimized as much as is feasible.

Uncertainty analysis includes sensitivity analysis and risk analysis.