# CENG 5604

## Hydropower Development

Lecture note

Contents


Chapter 1 Introduction ..... 1

1.1. Definition: ..... 1

1.2. History of Water Power ..... 1

Water Wheels ..... 1

Pitch-back ..... 2

Over-shot ..... 2

Breast-shot ..... 2

Under-shot ..... 2

High-Speed Commercial Hydraulic Turbines ..... 3

1.3. Types of developments ..... 3

Operational features ..... 4

Basis of operation ..... 5

Purpose ..... 5

Basis of uses ..... 5

Hydraulic feature ..... 6

Plant capacity and head ..... 7

1.4. Hydropower Development Cycles ..... 7

Reconnaissance studies ..... 8

Pre-feasibility study (Preliminary Design) ..... 9

Feasibility study ..... 10

Implementation Phase ..... 11

1.5. Hydropower in Ethiopia ..... 12

Energy Policy of Ethiopia ..... 14

Problems related to power production in Ethiopia ..... 15

Chapter 2: Hydraulics and Hydrology of Hydropower ..... 16

2.1. Hydraulic theory ..... 16

2.2. Hydrology of hydropower ..... 18

Flow duration analysis ..... 18

Discharge capacity of a plant ..... 21

Water Power Potential ..... 21

Other hydrologic considerations ........ 22
2.3. Energy and power analysis using FDC ........ 23
Power duration curve ........ 23
Load terminologies ........ 28
Load Duration Curve ........ 31
Chapter 3: Turbine selection and capacity determination ........ 41
3.1. Turbine types ........ 41
Turbine types: Reaction ........ 42
Turbine types: Impulse (or Velocity Turbines) ........ 45
3.2. Limits of use of turbine types ........ 48
3.3. Turbine selection criteria ........ 49
Rotational speed ........ 49
Specific Speed ........ 50
Maximum Efficiency ........ 52
3.4. Determination of number of units ........ 53
3.5. Power House ........ 55
Substructure ........ 56
Intermediate structure ........ 56
Superstructure: ........ 57
Power House Dimensions ........ 57
Underground power house ........ 57
Chapter 4: Water Passages ........ 58
4.1. Power Canal/tunnel ........ 58
Head Race: ........ 58
Tunnels ........ 62
4.2. Forebay ........ 65
4.3. Penstock ........ 66
Safe Penstock Thickness ........ 67
Size Selection of Penstocks ........ 67
Optimization of Penstock Diameter ........ 68
Anchor blocks ........ 68
Number of penstocks ........ 69

4.4. Spiral casing ..... 69
4.5. Draft tubes ..... 72
Types of draft tubes ..... 72
Turbine Setting ..... 73
Draft tube and turbine setting ..... 74
Cavitation and turbine setting ..... 77
Chapter 5 Pressure Control and Speed Regulation ..... 79
5.1. Water hammer theory and analysis ..... 79
Elastic Water column (EWC) theory ..... 81
Rigid water column (RWC) theory ..... 85
5.2. Pressure control systems ..... 86
Surge Tanks ..... 86
5.3. Speed terminology ..... 90
5.4. Speed control and governors ..... 91
Exercises ..... 92
Sample Exam ..... 97

# Chapter 1 Introduction

**Contents**


1. *Definition*
2. *History of water power*
3. *Types of Developments: Classification of hydro-electric power plants*
4. *Hydropower development cycles*
5. *Hydropower in Ethiopia: Status, Potential and Prospects*


## 1.1. Definition:

**Hydropower engineering** refers to the technology involved in converting the potential energy and kinetic energy of water into more easily used electrical energy.

The prime mover in the case of hydropower is a water wheel or hydraulic turbine which transforms the energy of the water into mechanical energy.

**Thought question: What are the Sources of water power?**

## 1.2. History of Water Power

- Greek poet Antipater (400 B.C.) refers to energy of falling water
- ~200 B.C., Egyptians were grinding grain with horizontal water mills
- By the First Century, the wheels were turned to operate vertically (horizontal axis) at much better efficiency
- About 1800, water mills were common in Europe
- In 1820s, Benoit Furneyron invented the turbine
- First electric power of 12 kW on Fox River, Appleton Wisconsin, 1882

## Water Wheels

Types of water wheels are based upon where the water strikes it

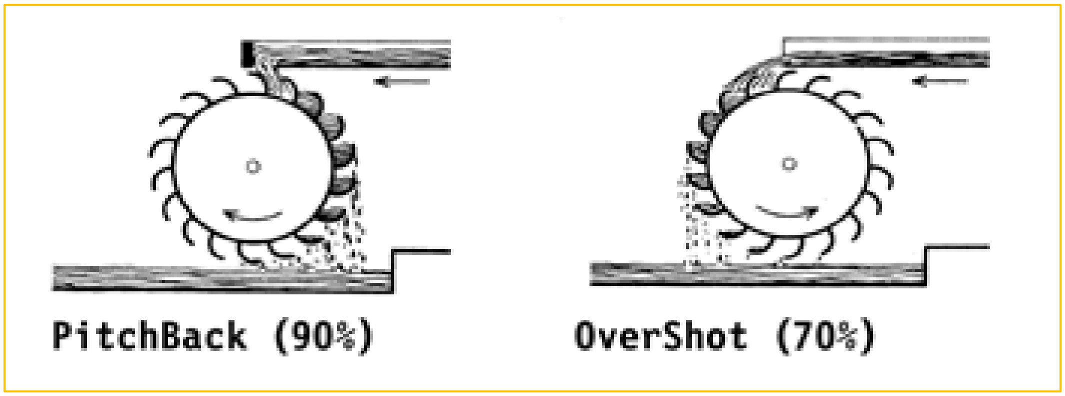
- Pitch-back – water drops from top and is deflected backwards to fall back towards the dam/river
- Over-shot – shoots over the top onto the wheel; the usual kind
- Breast-shot – strikes about 50% to 80% of height of the near side of the wheel
- Under-shot – pushes underneath and need not be more than immersed in a stream

Waterwheels turn slowly compared with turbines

- One - Fifty rpm

### Pitch-back

- A containing surround structure could force the water against the wheel as it falls and increase the weight of the water in the wheel



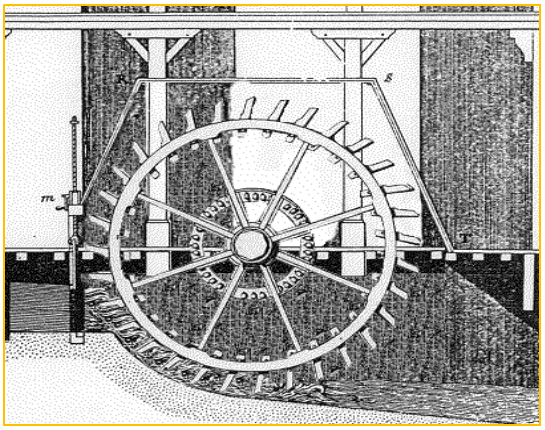

Note the difference in direction of the water flow (efficiency in bracket)

### Over-shot

- The water flows across the top of the wheel, pushing it forward, but also partially filling the buckets so that the weight pushes downward to turn the wheel
- The inertia of the water helps turn the wheel only slightly since it doesn't flow very fast. A very fast flow would be needed to get kinetic energy

### Breast-shot

The water strikes the wheel about mid-way up so the inertia and the weight of the water push the wheel around

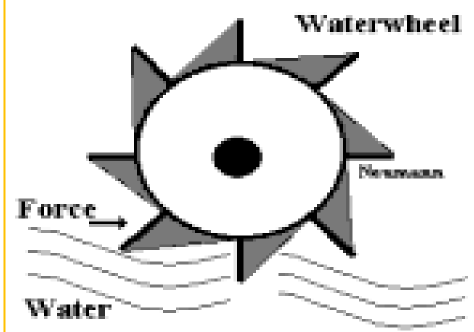


Note the contoured channel or surround at the bottom of the wheel that holds the water into the wheel.

### Under-shot

- The undershot wheel is simply placed in a stream with the bottom of the wheel pushed by the current
- Works well where there is little depth and no head
- Inefficient, but works where others won't
- Can be on a small boat anchored in a stream

Figure 4




Undershot Waterwheel

### High-Speed Commercial Hydraulic Turbines

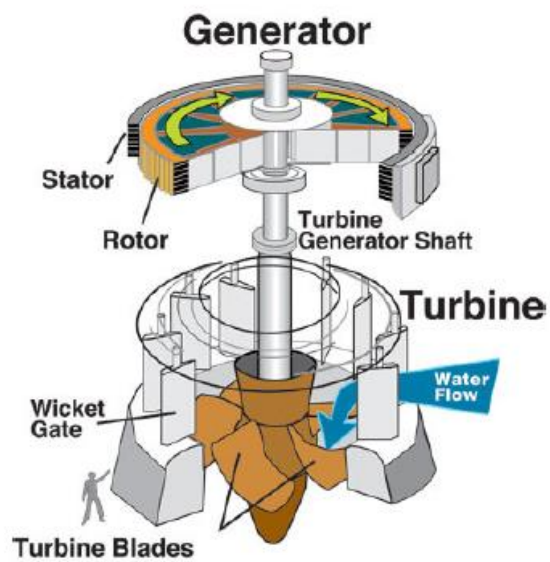
The turbine is made to convert hydraulic energy (potential and kinetic) into rotational mechanical energy on the turbine shaft.

The flow discharge is controlled by an aperture mechanism just in front of the turbine runner.

The rotating part of the turbine or water wheel is often referred to as the *runner*.

The shaft is directly connected to an electric generator that further converts the mechanical energy into electric energy.

The figure shows the Major Parts of a Turbine




## 1.3. Types of developments

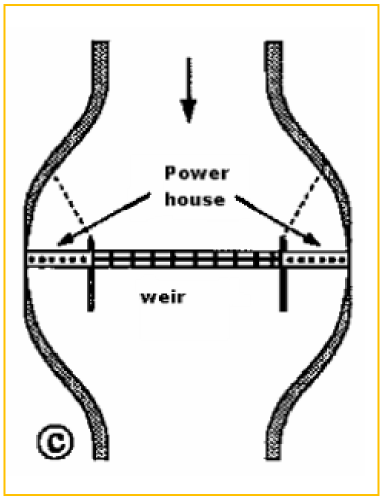
In studying the subject of hydropower engineering, it is important to understand the different types of development. The following classification systems are commonly used:

- Operational feature (Regulation of water flow)
- Basis of operation
- Purpose of development
- Uses to meet the demand for electrical power
- Hydraulic feature
- Plant capacity
- Operational head

## Operational features

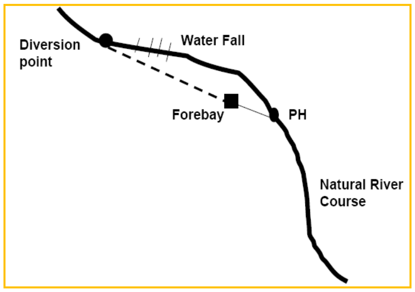
**Run-of-river developments:**

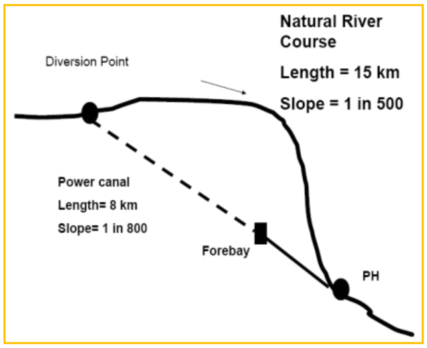
- The normal flow of the river is not disturbed
- There is no significant storage
- A weir or barrage is built across a river and the low head created is used to generate power
- Power house is in the main course of river
- Preferred in perennial rivers with moderate to high discharge, flat slope, little sediment and stable reach of a river.




**Diversion and canal developments:**

- Power canal or tunnel diverts water from main stream channel
- Powerhouse is provided at suitable location along the stretch of canal or tunnel
- Water from power house is returned to main stream by tailrace channel
- Short term pondage requirement is met through a pool called Forebay in the case of diversion canals and by means of a surge tank in case of diversion tunnel



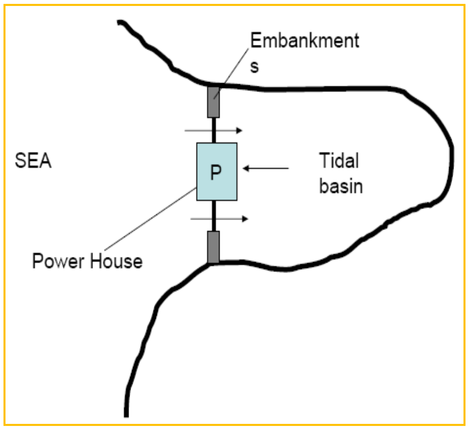

**Storage regulation developments:**

An extensive impoundment at the power plant or at reservoirs upstream of the power plant permits changing the flow of the river by storing water during high-flow periods to augment the water available during the low-flow periods.

Valley Dam Types of Hydropower Plants are storage regulation development type that have their powerhouse immediately at the toe of the dam(Ex. Grand renaissance hydropower project). The head difference between the reservoir water surface and the tail water level is characterized by its variability depending on the reservoir's storage conditions.

**Tidal power developments:**

In some estuaries, tidal power can be economically harnessed to develop electric energy.




## Basis of operation

- **Off-grid** (isolated) plant operating independently

- **In a grid system:** Plant operating as part of the interconnected grid system. In this system, a particular power plant may serve as a base load plant or as a peak load plant. Hydropower plants are best suited as peak load plants, because hydropower plants can start relatively quickly and can thus accept load quickly.

## Purpose

- **Single-purpose developments:** The water is used only for the purpose of producing electricity.

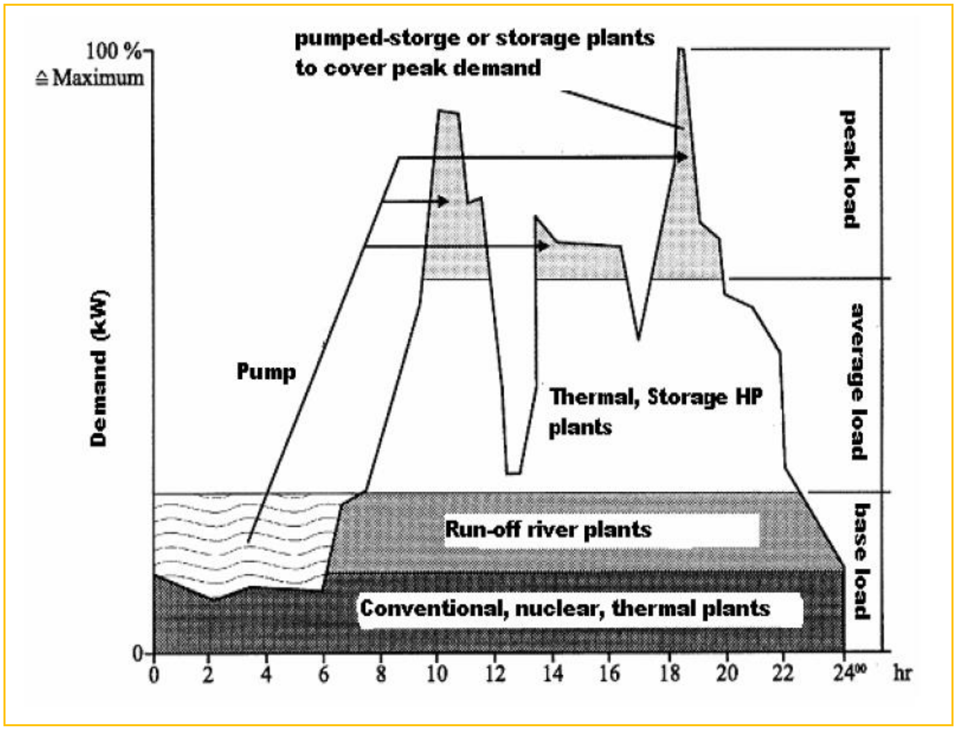
- **Multipurpose developments:** Hydropower production is just one of many purposes for which the water resources are used. Other uses might include, for example, irrigation, flood control, navigation, municipal, and industrial water supply.

## Basis of uses

Uses to meet the demand for electrical power:

- **Base-load developments:** When the energy from a hydropower plant is used to meet all or part of the sustained and essentially constant portion of the electrical load or firm power requirements, it is called a base-load plant. Energy available essentially at all times is referred to as firm power.

- **Peak-load development:** Peak demands for electric power occur daily, weekly, and seasonally. Plants in which the electrical production capacity is relatively high and the volume of water discharged through the units can be changed readily are used to meet peak demands. Storage of the water supply is necessary.




### Hydraulic feature

- **Conventional Hydro-plants**
  - Use normally available hydraulic energy of the flow of river
  - Run-off-river plant, diversion plant, storage plant
- **Pumped-storage plants**
  - Use the concept of recycling the same water
  - Normally used in areas with shortage of water
  - It has a function of indirect energy storage
- **Unconventional Hydro-plants**
  - Tidal power plant (Use the tidal energy of seawater)
- **Depression power plants**

- Energy generated by diverting water into a low lying depression
- Tailwater to be absorbed by evaporation

### Plant capacity and head

- Plant capacity: Usually this type of classification is arbitrary: for example:
  - Pico Hydro < 500 W
  - Micro hydro < 100 kW
  - Mini hydro < 1000 kW
  - Small to Medium < 60 MW
  - Large Hydro > 60 MW
- Classification based on head too arbitrary:
  - Low head plants < 15 m
  - Medium head plants 15 – 50 m
  - High head plants 50-250 m
  - Very high head plants > 250 m

## 1.4. Hydropower Development Cycles

Hydro power development involves a long process of assessing technical, commercial, environmental and social aspects, including understanding risks and opportunities for sustainable outcomes

- The studies to be carried out are:
- Resources studies
  - Preparation/updating of resources inventories
  - Preparation/updating of resources rankings
- Site specific studies
  - Preliminary/reconnaissance studies
  - Pre-feasibility studies
  - Feasibility studies
- The main purpose of resource inventory investigation is to identify register and catalogue the hydropower resource existing in a river basin; areas; districts and provinces.
- Flow data and data on topography is sufficient to establish the production and generating capability of a site.

- The identified project sites are ranked according to size, cost, electric demand, etc.
- Preparation of resources inventories and their updating is a continuous process and should not be stopped at any time.

### Reconnaissance studies

- The details and data requirements of these studies are regional in nature.
- Accuracy of these data as a requirement is less.
- Carried out for specific purpose such as: to establish the available potential in a district.
- They are concerned with project selection from inventories of resources.

The main objectives may be such as:

- Assessment of demand or define electric power need
- Selection of candidate projects from the resources inventories which will meet the electric power demand
- Investigation of candidate projects and project alternatives to the best technical level
- Technical ranking of candidate projects should be prepared and well recorded
- Selection of a suitable project from the list of investigated candidate projects.
- Estimation of preliminary cost and implementation schedule.

Main activities to be done in this stage:

- Data collection
- Office studies
- Field work and final reconnaissance report

**Data Collection:**

- Infrastructure information
- Power market and demand forecast
- Hydrology
- Topography
- Geology and geo technical engineering
- Environmental studies
- Socio-economic set up

**Office studies**:

- Power demand forecast
- Flow regulation
- Head
- Environmental constraints

**Field work:** the following issues should be recorded properly

- Terrain features such as location and placement of structures
- Infrastructures such as access to the project, transmission lines,
- Settlement and resettlement issue
- Availability of construction material
- Environmental issues such as diversion of flow from one catchment to the other, deforestation, etc.
- Multipurpose uses
- Diversion of flow during construction of Headwork and/or coffer dams
- In case of reservoir and tunnel projects special attention shall be given to the geological and geo technical properties.
- Appraisal of discharge available
- Study of existing and future water uses such as drinking, irrigation, etc.
- Verification of estimated head
- Powerhouse type, location and equipment

**Report:** Any reconnaissance report must conclude with a statement on the viability and sustainability of the project under consideration. Data requirement for feasibility study should be indicated.

## Pre-feasibility study (Preliminary Design)

In this study one or more project alternatives are proposed and studied before selection. The main purpose of pre-feasibility is to:

- Establish demand for the project
- Formulate a plan for developing this project
- Assess if the project is technically, economically and environmentally acceptable
- Make recommendation for future action

The following aspects are to be investigated during pre-feasibility study:

1. Hydrologic study:
    - Source extent amount, occurrence and variability of water.

- Present, past and future needs of water
- Include opportunities for control and development of water.
- Quality of water in terms of its physical and chemical properties
- Sediment quality and quantity
- Existing water rights should e recognized for each and every stakeholder.

2. **Power studies:** this considers a balance between power supply and demand.
3. **Layout Planning**: a comprehensive layout plan will be prepared and should be supplemented with sufficient number of drawings, which will be used for preparation of the bill of quantities.
4. Geology and foundation engineering
5. Seismic studies
6. Environmental studies
7. Estimation of cost
8. Economic and financial studies
9. Future investigation plan

**Pre-feasibility report**: A clear statement should be made in respect of technical, economical and environmental feasibility of the project. It should give clear indication whether or not to study the project in more detail.

### Feasibility study

Feasibility studies are carried out to determine the technical, economical and environmental viability of **a** project. This phase of investigation consists of a detailed study which is directed towards the ultimate permission, financing, final design and construction of the project under investigation.

The main part of feasibility studies include:

**1. Data Collection**:

- Socioeconomic data
- Population
- Income distribution
- Power market
- Tariffs
- Hydrology
- Topography
- Geology
- Seismic
- Environment
- Meteorology
- Infrastructure

**2. Project parameter estimation**

- Power and energy estimation
- Power system studies

- Water resources studies
- Geology and foundation conditions
- Seismic studies
- Construction materials
- Existing infrastructure

3. **Layout Optimization**

- Project layout
- Sediment and control measures
- Number and size of units
- Auxiliary equipment
- Transmission planning

4. **Environmental studies**

- Assessment of environmental disturbance and their mitigation measures

5. **Engineering design:**

- Intake structure and sediment excluder
- Headrace and tailrace
- Powerhouse
- Dimensioning and preparation of specification for hydro turbine and electromechanical equipment
- Construction facilities

6. **Estimation of project cost**

- Project cost
- Operation, maintenance and replacement
- Environmental cost
- Construction planning and budgeting
- Contingencies and other costs

7. **Economical and financial analysis**

**8. Future steps to be taken for the project implementation**

**Feasibility report**: A clear statement should be made in respect of technical, economical and environmental feasibility of **a** project. It should give clear indication whether or not to design and implement the project.

## Implementation Phase

Project implementation is a multidisciplinary job which includes:

- Approval and appropriation of funds
- Pre-qualification and hiring of consultants
- Detailed design
- Preparation of tender/contract documents

- Pre-qualification of contractors
- Preparation of construction design and engineering design
- Preparation of operation manual
- Construction supervision
- Construction of civil works
- Supply and erection of equipment
- Testing, commissioning and commercial operation
- Preparation of completion report

## 1.5. Hydropower in Ethiopia

Presently there are two different power supply systems,

- The Interconnected System (ICS), which is mainly supplied from hydropower plants,
- The Self-Contained System (SCS), which consists of mini hydropower plants and a number of isolated diesels generating units that are widely spread over the country.
- The ICS has a total installed generation capacity of about **3326.1** MW.

**Storage regulated hydropower development status in Ethiopia (Source EEPCO, MoWIE, 2017).**

| **1. Operational** | | | | | | |
|---|---|---|---|---|---|---|
| **No.** | **Name** | **River Basin** | **Region** | **UTME** | **UTMN** | **Installed Capacity (MW)** |
| 1 | Amerti Neshe | Abay | Oromia | 309105.2 | 1078089.9 | 97 |
| 2 | Fincha | Abay | Oromia | 316215 | 1058323 | 134 |
| 3 | Gibe-III | O. Gibe | Oromia/SNNR | 312200 | 757200 | 1870 |
| 4 | Gilgel Gibe I | O. Gibe | Oromia | 319025.8 | 870669.51 | 183.9 |
| 5 | Koka | Awash | Oromia | 518144.6 | 936460.7 | 43.2 |
| 6 | Melkawakena | W. Shebele | Oromia | 547907.5 | 792277.08 | 153 |
| 7 | Tana Beles | Abay | Amhara | 284013.4 | 1313351.9 | 460 |
| 8 | Tekeze-1 | Tekaze | Tigray/Amhara | 472125 | 1475533 | 300 |
| 9 | Tis-Abay-I | Abay | Amhara | 338393.1 | 1272689 | 12 |
| 10 | Tis-Abay-II | Abay | Amhara | 345289 | 1270370 | 73 |
| | **Total Functional** | | | | | **3326.1** |
| **2. Under Construction** | | | | | | |

| No. | Name | River Basin | Region | UTME | UTMN | Installed Capacity (MW) |
|---|---|---|---|---|---|---|
| 1 | Aba Samuel | Awash | Oromia | 467144 | 971116 | 6.6 |
| 2 | Genale Dawa - 3 | G. Dawa | Somali /Oromia | 610000 | 624000 | 254 |
| 3 | GERD | Abay | B. Gumuz | 73336.68 | 1242348.8 | 6450 |
| 4 | Koysha | O. Gibe | SNNPR | 204439 | 722611 | 2160 |
| | **Total Under Construction** | | | | | **8870.6** |

3. **Feasibility**

| No. | Name | River Basin | Region | UTME | UTMN | Installed Capacity (MW) |
|---|---|---|---|---|---|---|
| 1 | TAMS | B. Akobo | Gambela | 41390.34 | 908798.98 | 1700 |
| 2 | Mandeya | Abay | B. Gumuz/Amhara | 245592.5 | 1141383.4 | 1700 |
| 3 | Baro-1 | B. Akobo | SNNR /Oromia | 99020.54 | 892498.3 | 180 |
| 4 | Baro-2 | B. Akobo | SNNR /Oromia | 85931.19 | 898974.32 | 500 |
| 5 | Geba I | B. Akobo | Oromia | 123775.7 | 905581.6 | 180 |
| 6 | Geba II | B. Akobo | Oromia | 89023.23 | 924725.7 | 157 |
| 7 | Genale Dawa - 6 | G. Dawa | Somali | 650476.9 | 594391.62 | 246 |
| 8 | Hallele Werabesa I | O. Gibe | Oromia | 311000 | 948000 | 96 |
| 9 | Hallele Werabesa II | O. Gibe | Oromia | 322894 | 928453 | 326 |
| 10 | Gojeb | O. Gibe | Oromia | 267193.1 | 799702.34 | 153 |
| 11 | Chemoga – Yada | Abay | Amhara | 385726 | 1136182 | 162 |
| | **Total Feasibility** | | | | | **5400** |

**4. Pre-Feasibility**

| No. | Name | River Basin | Region | UTME | UTMN | Installed Capacity (MW) |
|---|---|---|---|---|---|---|
| 1 | Beko-Abo | Abay | Oromia/Amhara | 123623.2 | 1114738.3 | 5800 |
| 2 | Karadobi | Abay | Oromia/ Amhara | 353384.8 | 1090800.9 | 1600 |

| No. | Name | River Basin | Region | UTME | UTMN | Installed Capacity (MW) |
|---|---|---|---|---|---|---|
| 3 | Wabishebele (WS)-18 | W. Shebele | Somali/ Oromia | 844070.2 | 824634.32 | 87.5 |
| 4 | Mabil | Abay | Oromia/Amhara | 315603 | 1109013 | 1650 |
| 5 | Beshilo | Abay | Amhara | 445441 | 1213597 | 700 |
| 6 | Tekeze -2 | Tekeze | Tigray | 381997.8 | 1533150.2 | 450 |
| 7 | Gibe-4 | O. Gibe | SNNPR | 230736.2 | 728351.61 | 1472 |
| 8 | Gibe 5 | O. Gibe | SNNPR | 173976.9 | 697557.96 | 436 |
| 9 | Upper Dabus | Abay | B. Gumuz/Oromia | 50206.29 | 1101645.3 | 326.28 |
| 10 | Lower Dedessa | Abay | Oromia | 167256.2 | 1049164.6 | 301 |
| 11 | Lower Dabus | Abay | B. Gumuz/Oromia | 50206.29 | 1101645.3 | 326.28 |
| 12 | Birbir A | B. Akobo | Oromia | 80961.22 | 946311.39 | 97.4 |
| 13 | Birbir-R | B. Akobo | Oromia | 79927.78 | 942897.9 | 467 |
| | **Total Pre-Feasibility** | | | | | **13713.46** |

## Energy Policy of Ethiopia

Ethiopia's national energy policy identifies hydropower as the backbone of its sectoral development strategy. In 2005, the Government of Ethiopia released an aggressive, 25-year national energy master plan. The plan is updated annually and allows EEPCO great flexibility to easily integrate new projects. Besides its 25-year master plan, EEPCO is also undertaking an aggressive five-year plan called the Universal Rural Electrification Access Program (UREAP) to expand the domestic grid. The UREAP's goal is to increase access to electricity to 50% of the population. The Energy policy that corresponds to demand and consumption envisages to meet the following broad objectives:

- Giving high priority to Renewable Energy Development and follows climate resilient green economy strategy
- Considers Hydropower as the backbone of the country's energy generation and maximize its utilization;
- Enhancing regional and global cooperation in the energy sector to ensure exchange of know-how, information and transfer of technologies
- Strengthening cross boarder energy trade.
- Increasing access to affordable and adequate modern energy.

- Promoting efficient, clean, and appropriate energy technologies and conservation measure.
- Improving the energy efficiency of systems and operations.

### Problems related to power production in Ethiopia

- Lowest income (550 USD, in 2014?)
- High rate of population growth (2.8%)
- Scattered settlement pattern (>80% rural, 63 person/km$^2$)
- High investment cost (800 to 3000 USD/KW)
    - Too low domestic investors ability
    - Low credit availability (most of the project construction materials could easily be obtained in Ethiopia)
- Higher cost and time-consuming study and design phase.
- Higher risk (commercial, political, construction and hydrological risks)
- Lack of integrated water resources management
- Low domestic capacity building… etc.

# Chapter 2: Hydraulics and Hydrology of Hydropower

***Contents***


1. *Hydraulic Theory*
2. *Hydrologic analysis for hydropower*
    1. *Flow Duration Analysis*
    2. *Other Hydrologic Considerations*
3. *Energy and Power Analysis Using Flow Duration Approach*
    1. *Power duration curve*
    2. *Load terminologies*
    3. *Load duration curve*

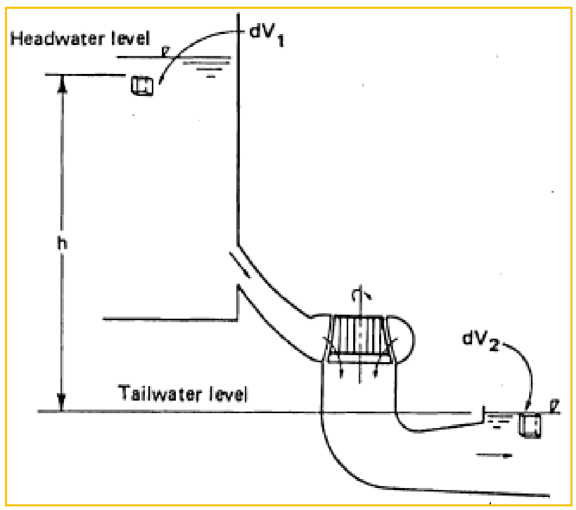

## 2.1. Hydraulic theory

**Energy-work approach:**

- Work (W) = Force x Distance in the direction of force
- Work = weight of water x the distance it falls $W = \rho_w V_w g h$

Where: $\rho_w$ is density of water; g- acceleration due to gravity; $V_w$- volume of water falling; $h$- the vertical distance the water falls.

It is conventional in hydropower computations to treat $h$ as the effective head that is utilized in producing power. Effective head (h) is the difference between energy head at the entrance to the turbine and the energy head at the exit of the draft tube.




The $h$ has been purposely designated as slightly below the headwater or Forebay level. Hence, in the Figure, the losses of head in the water moving through the penstock to the entrance of the turbine have been accounted for in positioning the elemental cube.

Power (P) = Work / time

$$P = \frac{W}{t} = \frac{\rho_w V_w g h}{t} = \rho_w Q g h$$

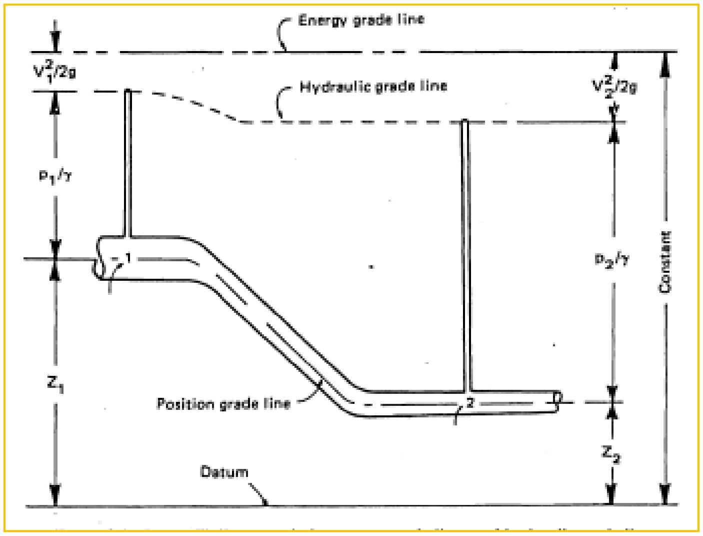
Where Q is discharge.

Note $Q = \frac{V_w}{t}$ P is in watt. To compare kilowatts and horsepower remember that: $P_{kw} = 0.746 P_{hp}$

**<u>Estimating h</u>**

Mathematical development in terms of energy grade lines and hydraulic grade lines, using the Energy Equation:

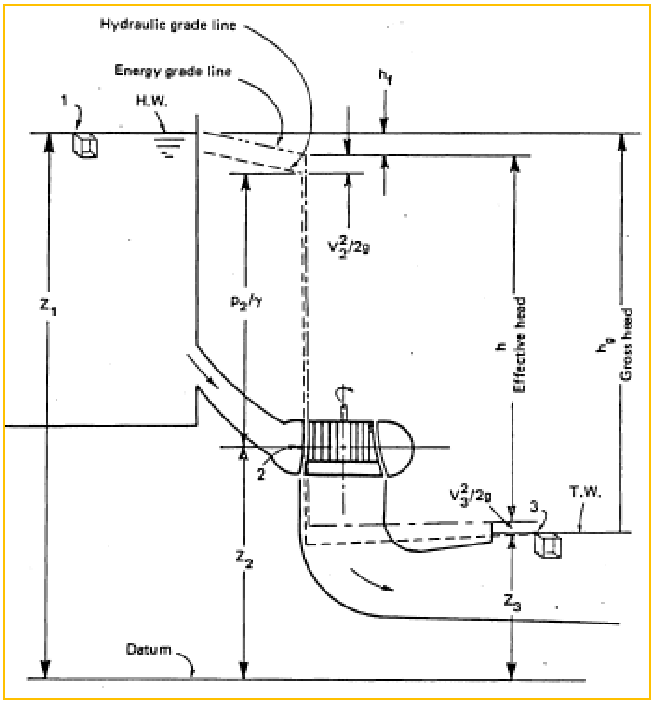
$$\frac{v_1^2}{2g}+\frac{p_1}{\gamma}+z_1=\frac{v_2^2}{2g}+\frac{p_2}{\gamma}+z_2+h_f=Cons.$$




where

- $V_1$ = *water velocity at point 1*
- $p_1$ = *pressure at point 1*
- $\gamma=\rho g$ = *specific weight of water*
- $Z_1$ = *potential head at point 1 referenced to the datum*
- $V_2$ =*water velocity at point 2*
- $p_2$ =*pressure at point 2*
- $Z_2$ =*potential head at point 2*
- $h_f$ = *head loss in flow passage between points 1 and 2*

The same could be seen in natural hydropower installation shown in the next figure. Referring to the Figure, the Energy equation for a hydropower installation is first written between point 1 at the surface of the Forebay and point 2 at the entrance to the turbine as




$$\frac{v_1^2}{2g}+\frac{p_1}{\gamma}+z_1=\frac{v_2^2}{2g}+\frac{p_2}{\gamma}+z_2+h_f...(1)$$

Then the Energy equation is written between points 2 and 3, the surface of the water at the exit to the draft tube:

$$\frac{v_2^2}{2g}+\frac{p_2}{\gamma}+z_2=\frac{v_3^2}{2g}+\frac{p_3}{\gamma}+z_3+h...(2)$$

Where $h$ is effective head on the turbine

Recognizing that for practical purposes $v_1$, $p_1$, *and* $p_3$ *are equal to zero, then solving* for $p_2/\gamma$ *in Eq. 1, the result is:*

$$\frac{p_2}{\gamma} = z_1 - \frac{v_2^2}{2g} - z_2 - h_f \dots(3)$$

$$h = \frac{v_2^2}{2g} + \frac{p_2}{\gamma} + z_2 - \frac{v_3^2}{2g} - z_3 = \frac{v_2^2}{2g} + \left( z_1 - \frac{v_2^2}{2g} - z_2 - h_f \right) + z_2 - \frac{v_3^2}{2g} - z_3$$

$$h = z_1 - z_3 - h_f - \frac{v_3^2}{2g} \dots(4)$$

Because the Energy equation defines terms in units of Kilogram –meter per Kilogram of water flowing through the system, it should be recognized that the Kilograms of water flowing through the turbine per unit of time by definition is ***ρgq***.

Now recognizing that energy per unit of time is power, it is simple to calculate power by multiplying Eq. (4) by ***ρgq*** *or* ***γq*** *to obtain the theoretical power delivered by the water to* the turbine as ***γqh*** which is the theoretical power

## 2.2. Hydrology of hydropower

- Hydrology is the study of the occurrence, movement and distribution of water on, above, and within the earth's surface.

- Parameters necessary in making hydropower studies are water discharge (Q) and hydraulic head (h). The measurement and analyses of these parameters are primarily hydrologic problems.

- Determination of the head for a proposed hydropower plant is a surveying problem that identifies elevations of water surfaces as they are expected to exist during operation of the hydropower plant.

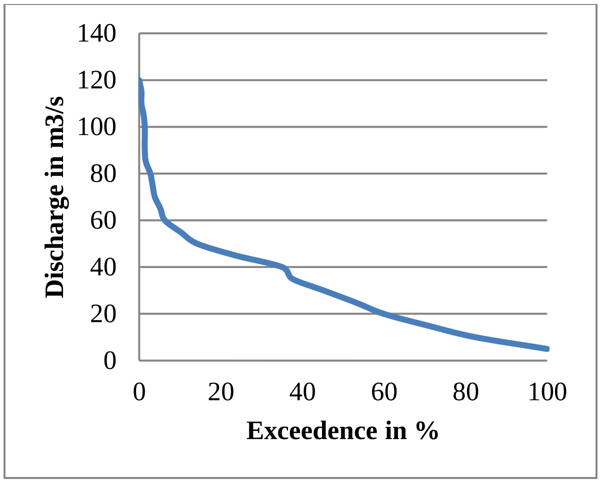
- In some reconnaissance studies, good contour maps may be sufficient to determine the value for the hydraulic head.

- Because the headwater elevation and tailwater elevations of the impoundment can vary with stream flow, it is frequently necessary to develop headwater and tailwater curves that show variation with time, river discharge, or operational features of the hydropower project.

### Flow duration analysis

**Flow Duration Curves:** is a plot of flow versus the percent of time a particular flow can be expected to be **equaled or** exceeded. A flow duration curve merely reorders the flows in order of magnitude instead of the true time ordering of flows in a flow versus time plot.

- Two methods
    - The rank ordered technique and
    - The class-interval technique.




**The rank-ordered technique**

Considers a total time series of flows that represent equal increments of time for each measurement value, such as mean daily, weekly, or monthly flows, and ranks the flows according to magnitude.

The rank-ordered values are assigned individual order numbers, the largest beginning with order 1. The order numbers are then divided by the total number in the record and multiplied by 100 to obtain the percent of time that the mean flow has been equaled or exceeded during the period of record being considered.

The flow value is then plotted versus the respective computed **exceedance percentage**. Naturally, the longer the record, the more statistically valuable the information that results.

**The class-interval technique**

It is slightly different in that the time series of flow values are categorized into class intervals. The classes range from the highest flow value to the lowest value in the time series. A tally is made of the number of flows in each, and by summation the number of values greater than a given upper limit of the class can be determined.

- The number of flows greater than the upper limit of a class interval can be divided by the total number of flow values in the data series to obtain the **exceedance percentage**.
- The value of the flow for the particular upper limit of the class interval is then plotted versus the computed exceedance percent.

**Characteristics of Flow Duration Curves**

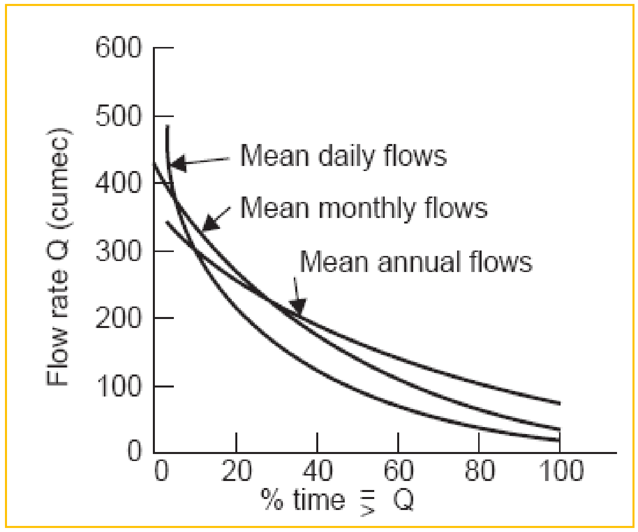
The flow duration curve (FDC) shows how flow is distributed over a period (usually a year). A steep flow duration curve implies a *flashy catchment* – one which is *subject to extreme floods and droughts*.

Factors which cause a catchment to be flashy are:

- Rocky, shallow soil,
- Lack of vegetation cover,
- Steep, short streams,
- Uneven rainfall (frequent storms, long dry periods).

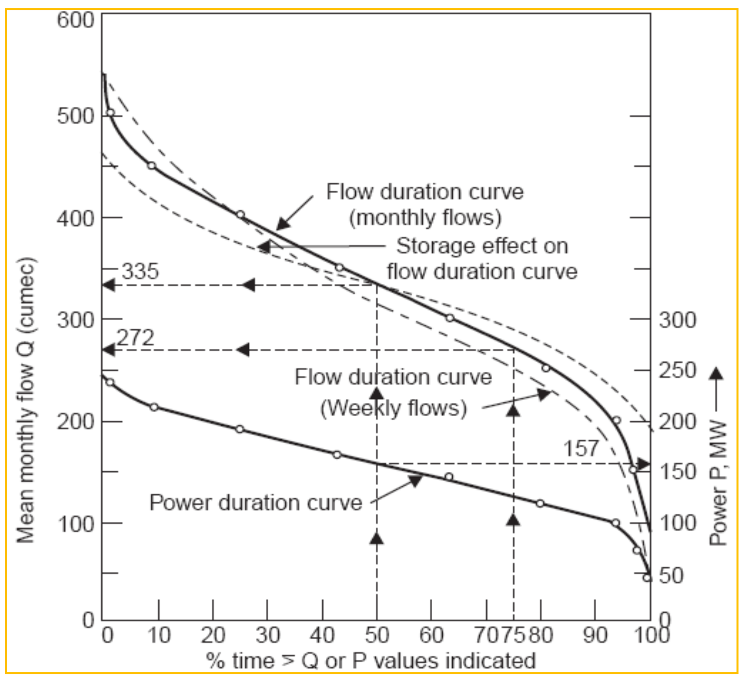
Such type of FDC (i.e. steep) is bad for hydropower development (especially run-of-river type). A flat flow duration curve is good because it means that the total annual flow will be spread more evenly over the year, giving a useful flow for longer periods, and less severe floods.

The other factor which makes the FDC flatter is the time interval used in collecting the flow data. The selection of the time interval for FDC depends on the purpose of the study. As the time interval increases the range of the curve decreases (see Fig behind.).




While daily flow rates of small storms are useful for the pondage studies in a run-off river power development plant, monthly flow rates for a number of years are useful in power development plants from a large storage reservoir

The flow duration curve is actually a river discharge frequency curve and the longer the period of record, the more accurate is the indication of the long-term yield of a stream. Since the area under the curve represents the volume of flow, the storage will affect the flow duration curve as shown by the dashed line in the Figure below; *i.e.*, reducing the extreme flows and increasing the very low flows.




Flow duration curve, very often, plotted using the average monthly values of the flow. The capacity estimate for firm power (power available in the river throughout the recorded period) is then made by using the entire recorded flow data and plotting in a single flow duration curve. In such a case two different methods are in use.

(i) The *total period method*, and

(ii) The *calendar year* method.

Both methods utilize the flow data available for the entire period for which records are available.

***Total period method:*** the entire available record is used for drawing the FDC. Thus, ten years' record would produce 120 values of monthly average flows.

- These are first tabulated in the ascending order starting from the driest month in the entire period and ending with the wettest month of the ten-year duration.
- The FDC would then be drawn with the help of 120 values.

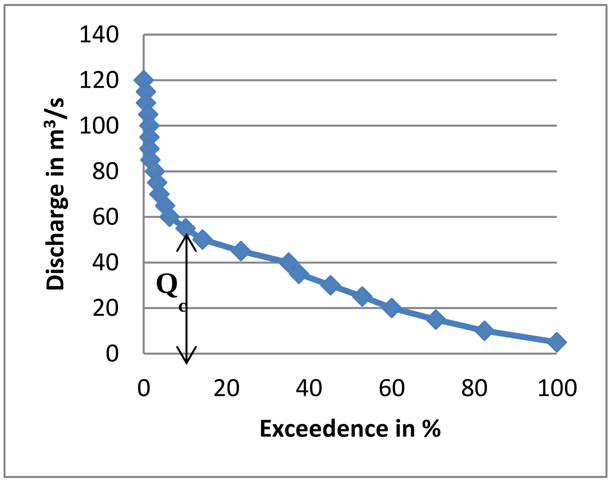
***Calendar year method***: each year's average monthly values are first arranged in ascending order.

- Then the average flow values corresponding to the driest month, second driest month, and so on up to the wettest month are found out by taking arithmetic mean of all values of the same rank. These average values are then used for plotting flow duration curve.
- Such a curve would have only twelve points.

***The total period method gives more correct results than the calendar year method which averages out extreme events.***

### Discharge capacity of a plant

Discharge capacity ($Q_c$) of a plant is the discharge the plant can pass at its full gate opening of the runner(s) of the turbine(s) under design head. A flow duration curve is used to explain discharge capacity ($Q_c$) as labeled in the Figure. Even though to the left of that point on the duration curve the stream discharge is greater, it is not possible to pass the higher discharges through the plant.




### Water Power Potential

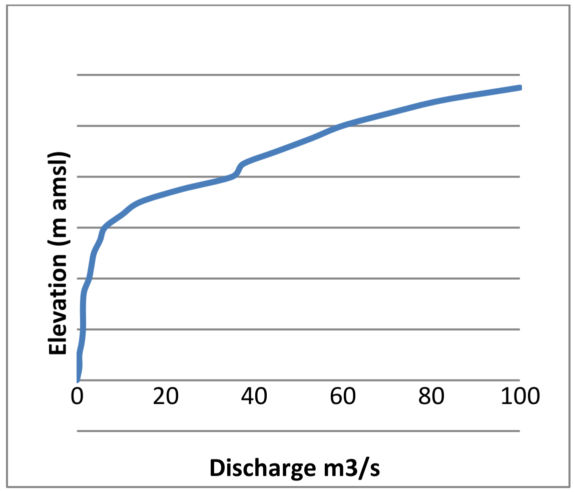
Before any power plant is contemplated, it is essential to assess the inherent power available from the ***discharge*** of the river and the ***head available at the site.***

- The gross head of any proposed scheme can be assessed by simple surveying techniques, whereas
- Hydrological data on rainfall and runoff are essential in order to assess the quantity of water available.
- The hydrological data necessary for potential assessment are:
    - The daily, weekly, or monthly flow over a period of several years, to determine the plant capacity and estimate output,
    - Low flows, to assess the primary, firm or dependable power.

### Other hydrologic considerations

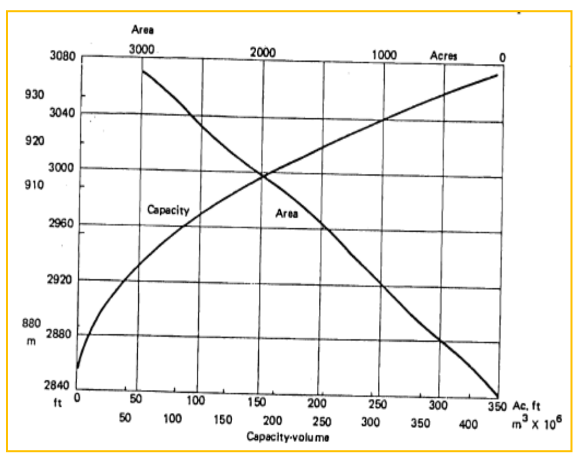
**Tailwater Relationships:**

As releases of water over spillways and any other releases into the stream immediately below a hydropower plant are made, the tailwater elevation below the outlet to the turbines will fluctuate. Therefore, it is important to develop a tailwater elevation versus river discharge curve over the complete range of flow that is to be expected.




**Area capacity curves**

Most hydropower developments involve an impoundment behind a dam. As the water in storage in the impoundment is released the headwater elevation changes and this will influence the design of the plant and the pattern of operation. Therefore, it is necessary to have a storage volume versus impoundment surface elevation curve.




**Reservoir rule curves**

When releases from reservoirs 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 downstream use dictate 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.

**Evaporation Loss Evaluation:** Where there is an impoundment involved in a hydropower development there is need to assess the effect of evaporation loss from the reservoir surface.

**Spillway Design Flood Analysis:** Many hydropower developments require a dam or a diversion that blocks the normal river flow. This then requires that provisions 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. It is customary on larger 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.

## 2.3. Energy and power analysis using FDC

### Power duration curve

Remember: From section 2.1

$$P = \rho_w Q g h$$

The above equation is for theoretical conditions. The actual output is diminished by the fact that the turbine has losses in transforming the potential and kinetic energy into mechanical energy. Thus an efficiency term *($\eta$), usually called overall efficiency*, must be introduced to give the standard power equation:

$$P = \eta \rho_w Q g h$$

If the river course is divided into a number of stretches, the total power can be described by

$$P = \rho_w g \sum Qh$$

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. **How?**

The actual use of the equation for estimating the potential (P); however, is made difficult due to the fact that the discharge of any river varies over a wide range. High discharges are available only for short durations in a year. Thus, the corresponding available power would be of short duration.

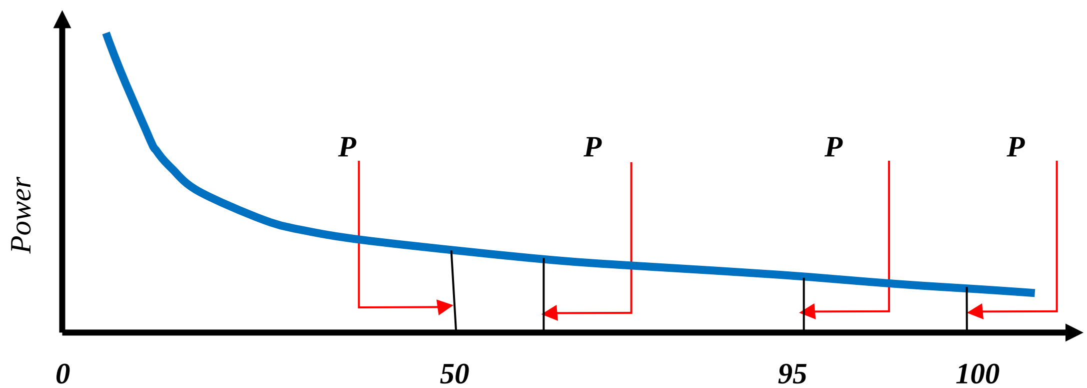
- If the discharge rate and the percentage duration of time for which it is available are plotted, a ***flow-duration curve*** result.

- ***Power duration curve*** can also be plotted since power is directly proportional to the discharge and available head.

***Discharge/Power duration curve*** indicates discharge or power available in the stream for the given percentage of time. The available power from a run-of-river plant could be represented by a power duration curve exactly on lines analogous to a FDC.

Generally, the head variation in a run-of-river plant is considerably less than the discharge variation. If the head is presumed to be constant at an average value, power duration curve would exactly correspond to FDC. This is very often the procedure in elementary rough calculations. If, however, a precise power duration curve is desired, then the head corresponding to any discharge is required to be known.

**Some common terminologies**




- ***Minimum potential ($P_{100}$) power computed from the minimum flow available for 100 % of the time (365 days or 8760 hours).***

- ***Small potential ($P_{95}$) power computed from the flow available for 95 % of time (flow available for 8322 hours).***

- ***Average potential** power **($P_{50}$)*** computed from the flow available for 50% of the time (flow available for 6 months or 4380 hours).

- ***Mean potential** power **($P_m$)*** computed from the average of mean yearly flows for a period of 10 to 30 years, which is equal to the area of the flow-duration curve corresponding to this mean year. This is known as '*Gross river power potential*'.

- It would be more significant to find out the technically available power from the potential power; According to Mosonyi, the losses subtracted from the P values present an upper limit of utilization;

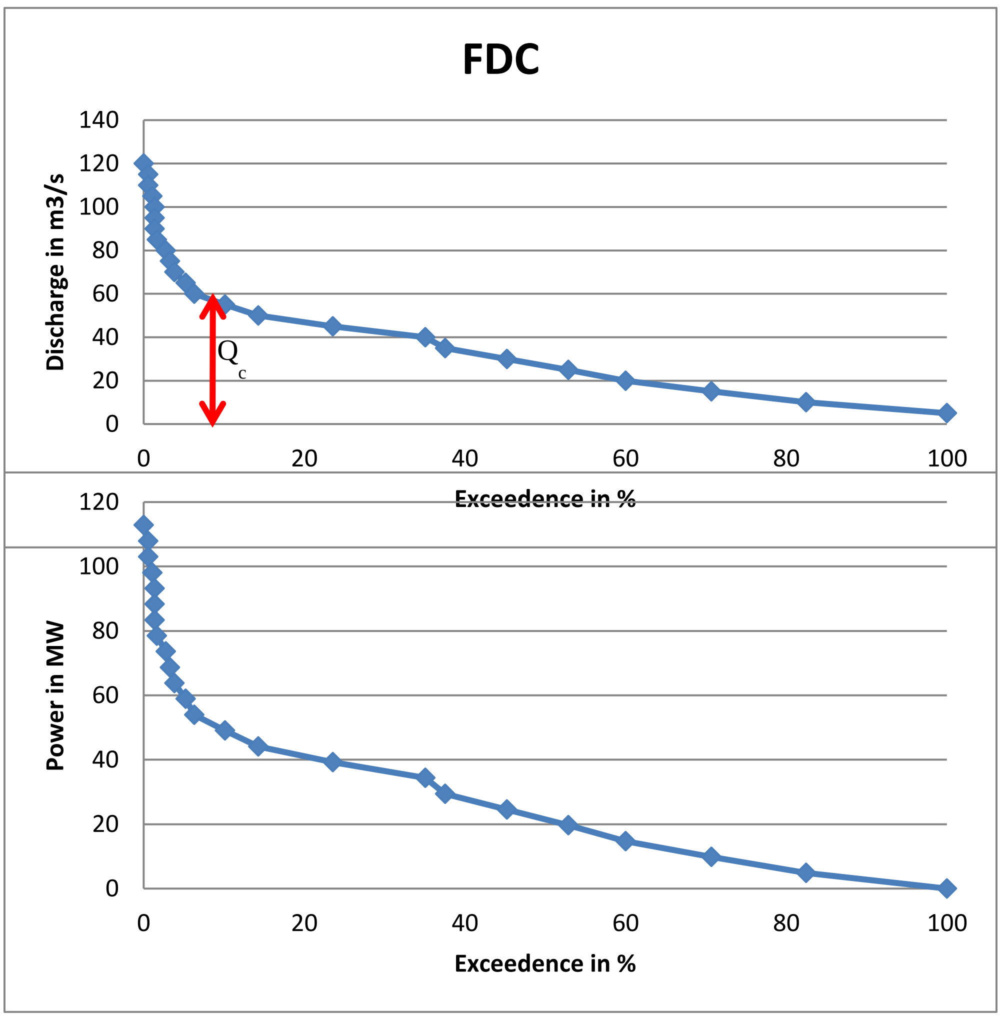
- ***Technically available power***: With conveyance efficiency of 70% and overall efficiency of the plant as 80%, a combined multiplying factor of 0.56 should be used with the average potential power, $P_{50}$; $P_a = 0.56 P_{50}$

The value of net water power capable of being developed technically is also computed from the potential water power by certain reduction factors to account for losses of head in the conveyance and losses associated with energy conversion. This factor is usually about 0.75 or 0.80, i.e. $P_{m\,net} = (7.4 \ to \ 8.0) Q_m h$ ; Where $Q_m$ = the arithmetic mean discharge

The maximum river energy potential is given by $E_{\max net} = 8760 P_{m\,net} (Kwh)$

Energy production for a year or a time period is the product of the power ordinate and time and is thus the area under the power duration curve multiplied by an appropriate conversion factor.

**Example 1:** What is wrong with the PDC?



**Example 2:** The following is the record of average yearly flow in a river for 15 years. If the available head is 15 m, construct the FDC and power duration curve for the river.

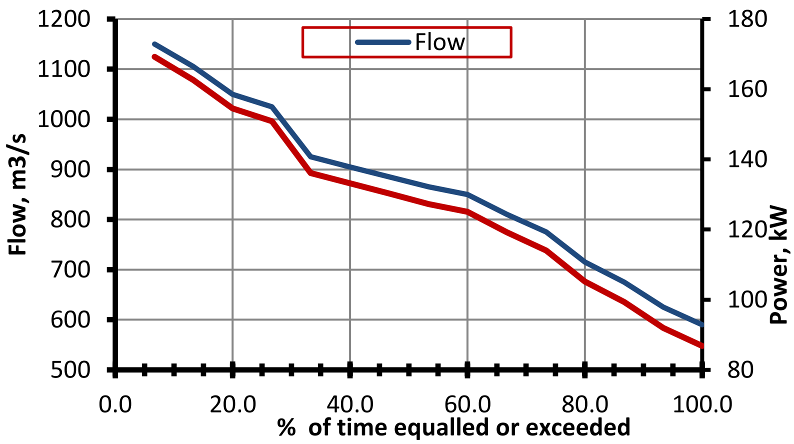
| **Year** | **1956** | **1957** | **1958** | **1959** | **1960** | **1961** | **1962** | **1963** | **1964** |
|---|---|---|---|---|---|---|---|---|---|
| **Flow ($m^3/s$)** | **905** | **865** | **1050** | **1105** | **675** | **715** | **850** | **775** | **590** |
| **Year** | **1965** | **1966** | **1967** | **1968** | **1969** | **1970** | | | |
| **Flow ($m^3/s$)** | **625** | **810** | **885** | **1025** | **1150** | **925** | | | |

**Solution:** The yearly flow values are arranged in ascending order (see table below). The power corresponding to each flow values are calculated assuming the head (=15 m) to be constant. Then, FDC and power duration curves are plotted on the same graph.

**Row1: Rank; Raw 2: Flow in ascending order ($m^3/s$); Raw 3: Power (=9.81 QH) [kW]; and Raw 4: Percentage of time exceeded = ( 15 +1 – n )*100/15**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| R1 | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| R2 | 590 | 625 | 675 | 715 | 775 | 810 | 850 |
| R3 | 86819 | 91969 | 99326 | 105212 | 114041 | 119192 | 125078 |
| R4 | 100 | 93.4 | 86.7 | 80 | 73.4 | 66.7 | 60 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| R1 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
| R2 | 865 | 885 | 905 | 925 | 1025 | 1050 | 1105 | 1150 |
| R3 | 127285 | 130228 | 133171 | 136114 | 150829 | 154508 | 162601 | 169223 |
| R4 | 53.4 | 46.7 | 40 | 33.3 | 26.7 | 20 | 13.3 | 6.7 |

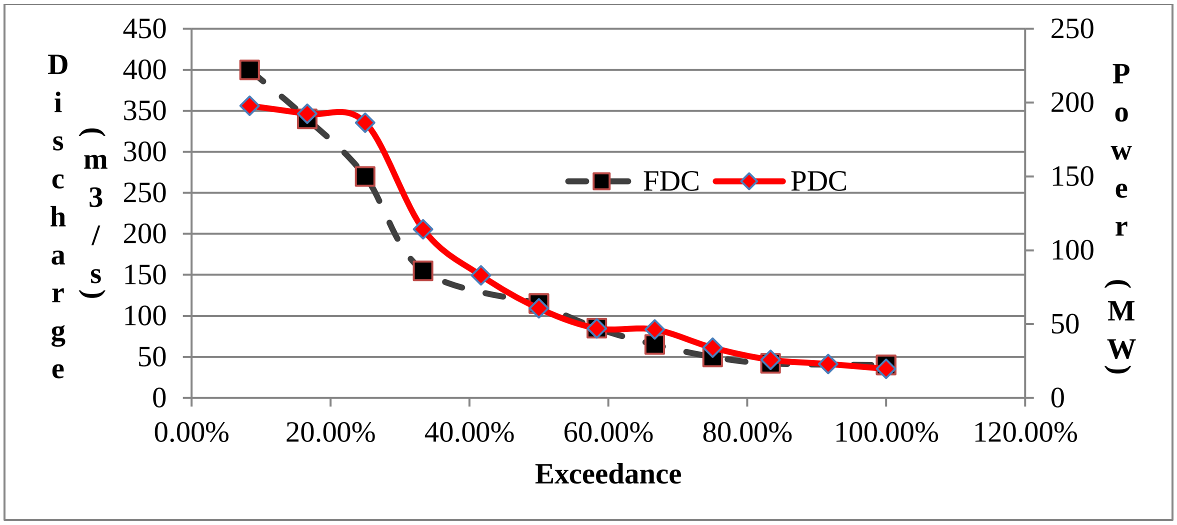



Example 3: Draw the flow and power duration curve if the discharge capacity $Q_c = 270\ m^3/s$

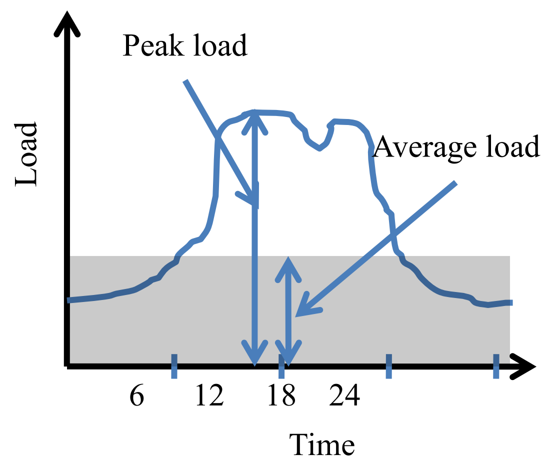
| Month | Flow ($m^3/s$) | Head(m) | Efficiency |
|---|---|---|---|
| **January** | 65 | 83.5 | 0.87 |
| **February** | 50 | 83.5 | 0.83 |
| **March** | 42 | 83.5 | 0.75 |
| **April** | 40 | 83.5 | 0.70 |
| **May** | 40 | 83.5 | 0.60 |
| **June** | 115 | 83.5 | 0.50 |
| **July** | 400 | 80 | 0.88 |
| **August** | 340 | 81.6 | 0.89 |
| **September** | 270 | 83 | 0.90 |
| **October** | 155 | 83.5 | 0.90 |
| **November** | 115 | 83.5 | 0.88 |
| **December** | 85 | 83.5 | 0.87 |

Solution

| | | | | | FDC | | | | | | PDC | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mon | Q Sorted | Head | EFF | Rank Q | Exce. % | Q for P | P (MW) | | Mon | Rank P | Exce. % | P sorted |
| Jul | 400 | 80 | 0.88 | 1 | 8.3 | 270 | 186.47 | | Sep | 1 | 8.3 | 197.86 |
| Aug | 340 | 81.6 | 0.89 | 2 | 16.7 | 270 | 192.36 | | Aug | 2 | 16.7 | 192.36 |
| Sep | 270 | 83 | 0.9 | 3 | 25.0 | 270 | 197.86 | | Jul | 3 | 25.0 | 186.47 |
| Oct | 155 | 83.5 | 0.9 | 4 | 33.3 | 155 | 114.27 | | Oct | 4 | 33.3 | 114.27 |
| Jun | 115 | 83.5 | 0.5 | 6 | 50.0 | 115 | 47.10 | | Nov | 5 | 41.7 | 82.90 |
| Nov | 115 | 83.5 | 0.88 | 6 | 50.0 | 115 | 82.90 | | Dec | 6 | 50.0 | 60.58 |
| Dec | 85 | 83.5 | 0.87 | 7 | 58.3 | 85 | 60.58 | | Jun | 7 | 58.3 | 47.10 |
| Jan | 65 | 83.5 | 0.87 | 8 | 66.7 | 65 | 46.32 | | Jan | 8 | 66.7 | 46.32 |
| Feb | 50 | 83.5 | 0.83 | 9 | 75.0 | 50 | 33.99 | | Feb | 9 | 75.0 | 33.99 |
| Mar | 42 | 83.5 | 0.75 | 10 | 83.3 | 42 | 25.80 | | Mar | 10 | 83.3 | 25.80 |
| Apr | 40 | 83.5 | 0.7 | 12 | 100 | 40 | 22.94 | | Apr | 11 | 91.7 | 22.94 |
| May | 40 | 83.5 | 0.6 | 12 | 100 | 40 | 19.66 | | May | 12 | 100 | 19.66 |




### Load terminologies




- **Load** is the amount of power delivered or received at a given point at any instant.
- **Average Load** is the total load produced divided by the number of hours in the time period of interest.
- **Peak Load** is the maximum instantaneous load or a maximum average load over a specified period of time.
- ***Base load*** is the total load continuously exceeded;

- ***Power demand*** is defined as the total load, which consumers choose, at any instant of time, to connect to the supplying power system.
- The highest instantaneous value of the demand is, strictly speaking, the ***peak load or peak demand***. Generally, peak load is defined as that part of the load carried at intensity greater than 4/3 times the mean load intensity.

### *Load Factor*

- The degree of variation of the load over a period of time is measured by the **load factor**, which may be defined as the average load divided by the peak load within the given time range.

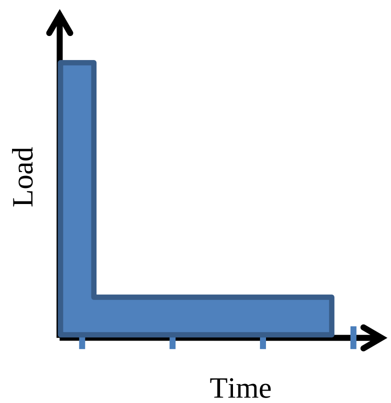
- The load factor measures variation only and does not give any indication of the precise shape of the load-duration curve.

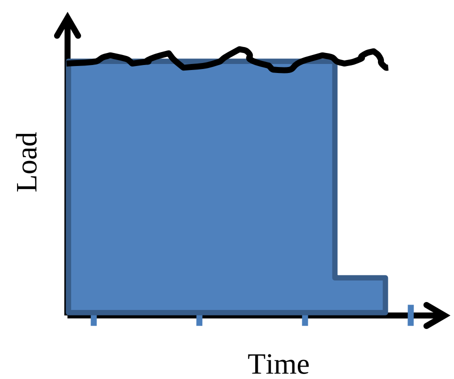
- The area under the load curve represents the energy consumed in kWh; Thus, a daily load factor may also be defined as the ratio of the actual energy consumed during 24 hours to the peak demand assumed to continue for 24 hours.

- Load factor gives an idea of degree of utilization of capacity;

- Thus, an annual load factor of 0.4 indicates that the machines are producing only 40% of their yearly production capacity.

- As the load factor approaches zero, the duration curve will approach a narrow L shape, indicating a peak load of very short duration with very low or no load during the major portion of the time. As the load factor approaches unity, the duration curve will be somewhat rectangular in appearance, indicating high sustained loads.




### *Capacity factor*

- The capacity factor is the ratio of the energy actually produced by the plant for any given period of time to the energy it would be capable of producing at its full capacity for that period of time.

- The extent of use of the generating plant is measured by the capacity factor, frequently also termed **plant factor**. If during a given period a plant is kept fully loaded, it is evident that it is used to the maximum extent, or operated at 100% capacity factor.

- The factor is equal to the average load divided by the rated capacity of the plant.

- Capacity factor and load factor become identical when the peak load is equal to the capacity of the plant. The relationship between the two factors is evidently

$$Capacity\ Factor = \frac{Peak\ Load \times Load\ factor}{Rated\ capacity\ of\ the\ plant}$$

***Plant use factor:***

**Capacity factor** = (actual energy produced)/(energy which could have been produced had the plant run at full rated output). Both numerator and denominator are taken over the same time. **Plant Use factor** = (actual energy produced)/(energy which could have been produced had the plant been run). Again both are taken over the same time. These are not the same because the plant's capacity may have been reduced for many possible reasons (e.g. shortage of water) and if it had run it would have been at below rated output. This makes the capacity factor normally a lower number than the use factor.

$$Plant\ Use\ \ factor = \frac{Actual\ energy\ used\ in\ a\ \ period}{Maximum\ energy\ produced\ in\ the\ same\ period}$$

***Utilization Factor***

The utilization factor measures the use made of the total installed capacity of the plant. It is defined as the ratio of the peak load and the rated capacity of the plant.

***Utilization Factor***: is the ratio of the quantity of water actually utilized for power production to that available in the river. If the head is assumed to be constant, then the utilization factor would be equal to the ratio of power utilized to that available.

The factor for a plant depends upon the type of system of which it is a part of. A low utilization factor may mean that the plant is used only for stand-by purposes on a system comprised of several stations or that capacity has been installed well in advance of need.

In the case of a plant in a large system, high utilization factor indicates that the plant is probably the most efficient in the system. In the case of isolated plants a high value means the likelihood of good design with some reserve-capacity allowance.

The value of utilization factor varies between 0.4 and 0.9 depending on the plant capacity, load factor and storage.

***Diversity factor***

Diversity factor (DF) is the summation of the different types of load divided by the peak load. If there be four different types of load $L_1$, $L_2$, $L_3$ and $L_4$ and the peak load from the combination of these loads is $L_P$, then the diversity factor is expressed as:

$$(L_1 + L_2 + L_3 + L_4)/L_P$$

Note that the diversity factor has a value which is greater than unity. Its value could be 1 *which indicates the maximum demand of the individual sub-system occurs simultaneously.*

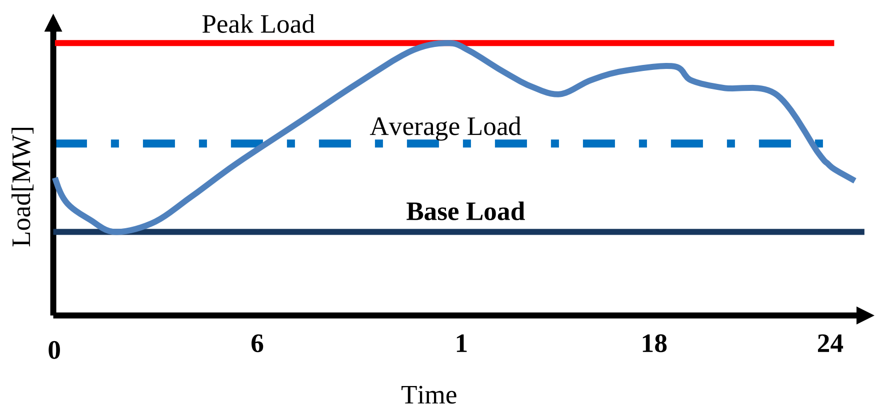
For n load combination: $DF = \sum_{i=1}^{i=n} L_i \Big/ L_p$

An area served by a power plant having different types of load, peaking at different times, the installed capacity is determined by dividing the total of maximum peak load by diversity factor.

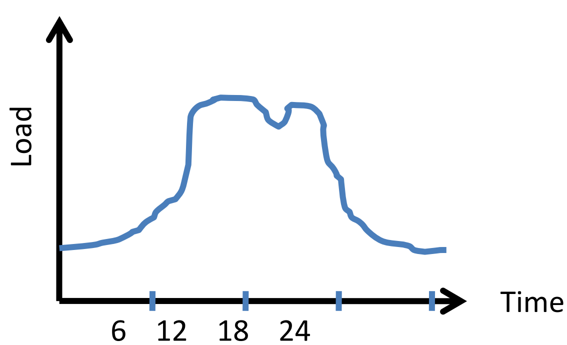
### Load Duration Curve

**Load Curve:** A load curve is a graph of load consumption with respect to time and directly gives an indication of power used at any time (daily, weekly, monthly, annually, etc.)

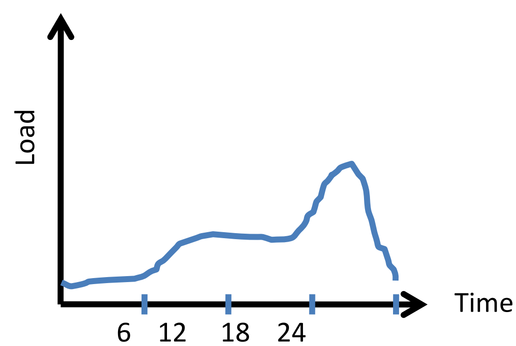
***Daily Load Curve*** is a curve drawn between load as the ordinate and time in hours as the abscissa for one day.




The area under the curve of a daily chronological load curve measures the total energy consumed by the load during the day. This energy is evaluated by: $E = \int_{t=1}^{t=24} kwdt$



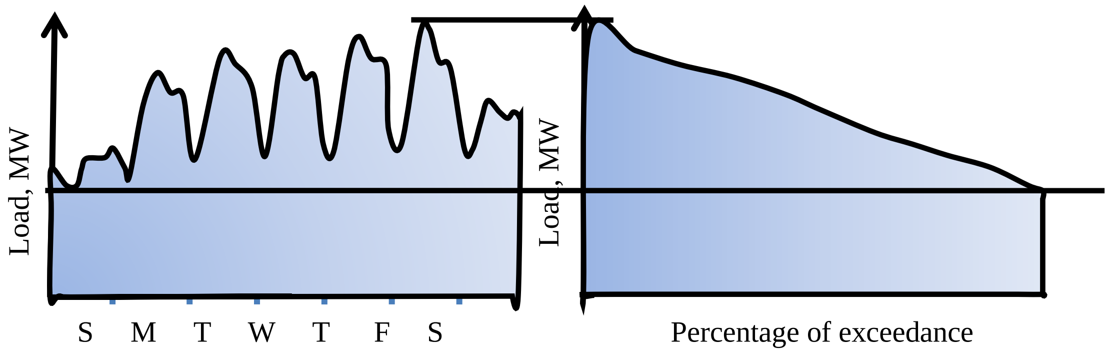

Industrial load with one shift operation




Residential load

**Draw the load curve for Street Light?**

It will be necessary for system planning and operating estimates to express the variation in, and the integration of, the total energy requirements for a period of time in some concise form; the load-duration curve does this.

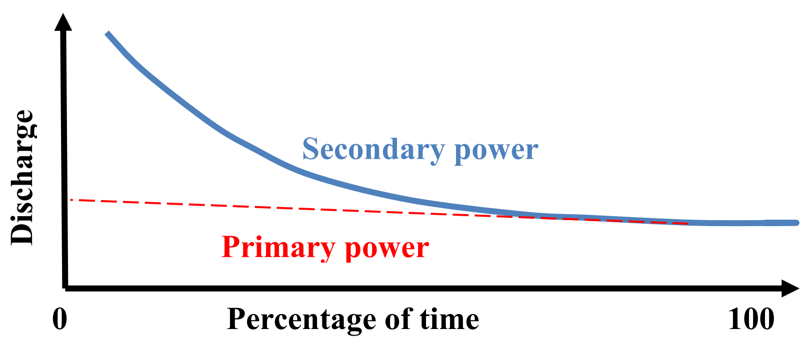



Fundamentally the load-duration curve (right) is nothing more than a rearrangement of all the load elements of a chronological curve (called load curve (left)) in the order of descending magnitude. The areas under the load-duration and corresponding chronological curves are equal. Since it is impractical to determine the equation of load curve, the area or energy is determined graphically.

It is usually prepared for a longer duration such as a year. The area under load duration curve for time duration would be the same as that of a load curve for the same particular period of time.

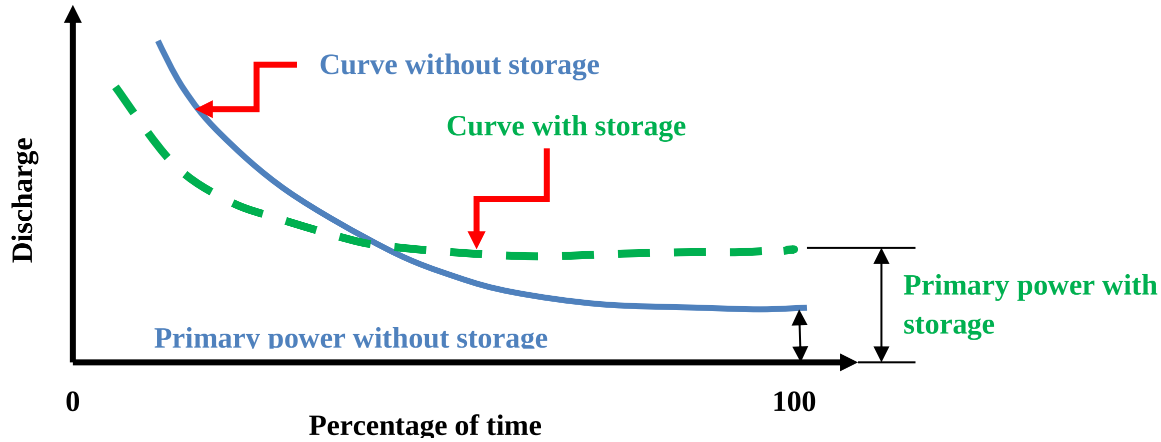
The area under a load duration curve represents the total energy production for the duration. Thus, annual load factor is given by the ratio of the area under the curve to the area of the rectangle corresponding to the maximum demand occurring during the course of the year.

***Firm and secondary power***




**Firm Power**: The firm or **primary power** is the power which is always ensured to a consumer at any hour of the day and is, thus, completely dependable power. Firm power would correspond to the minimum stream flow and is available for all the times. The firm power could be increased by the use of pondage (storage).

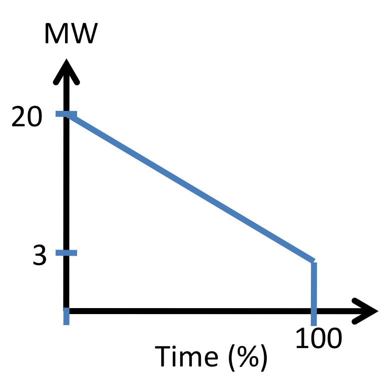
**Secondary power**: *Also known as surplus or non-firm power*, is the power other than the primary power and is, thus, comparatively less valuable.




The secondary power is useful in an interconnected system of power plants. At off-peak hours, the secondary power may be called upon to relieve the interconnected stations thus affecting economy. The secondary power may also be used to take care of the current demand by following a load-duration plan

**Example 4**: Consider the yearly load duration curve for a certain load center to be a straight line from 20 to 3 MW. To meet this load, three turbines, two rated at 10 MW each and one at 5 MW are installed. Determine: Load factor (LF); Capacity factor (CF); Utilization factor (UF)

Solution:




$$LF = \frac{\frac{3+20}{2}\times 100\% \times t}{20\times 100\% \times t}\times 100\% = 57.5\%$$

$$CF = \frac{\frac{3+20}{2}\times 100\% \times t}{(10+10+5)\times 100\% \times t}\times 100\% = 46\%$$

$$UF = \frac{20\times 100\% \times t}{25\times 100\% \times t}\times 100\% = 80\%$$

**Example 5:** A power station has to meet the following demand

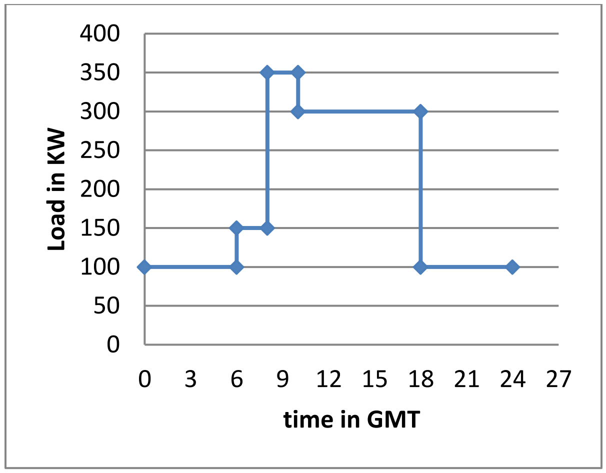
- Group A: 200KW between 8AM and 6 PM
- Group B: 100KW between 6AM and 10AM
- Group C: 50KW between 6AM and 10AM

- Group D: 100KW between 10AM and 6AM

Plot the daily load curve and determine the (i) diversity factor (ii) Power generated per day (iii) Load factor

Solution: The given load cycle can be tabulated as under:

| **Time** | **0-6** | **6-8** | **8-10** | **10-18** | **18-24** |
|---|---|---|---|---|---|
| Group A | - | - | 200 | 200 | - |
| Group B | - | 100 | 100 | - | - |
| Group C | - | 50 | 50 | - | - |
| Group D | 100 | - | - | 100 | 100 |
| **Total** | **100** | **150** | **350** | **300** | **100** |




It is clear from the curve that the maximum demand is 350 KW

Sum of individual maximum demands would be:

$$200 + 100 + 50 + 100 = 450 \text{ KW}$$

(i) Diversity factor = 450/350 = **1.286**

(ii) Power generated per day = Area below the load curve: 100*6 + 150*2 + 350*2 + 300*8 + 100*6 = **4600 KWh**

Average load = 4600/24 = 191.67 KW

Load factor = Average load/Peak Load = 191.67/350 = **0.548 (54.8%)**

**Example 6:** A generating station has a maximum demand of 25 MW, a load factor of 60%, a plant capacity factor of 50% and a plant use factor of 72%. Find (i) The reserve capacity of the plant (ii) The daily energy produced and (iii) Maximum energy that could be produced daily if the plant, while running as per the schedule, were fully loaded.

Solution

$$L.F. = \frac{Average\ Demand}{Maximum\ Demand} = 0.6 \Rightarrow Average\ Demand = 0.6 \times 25 = 15MW$$

$$Plant\ C.F. = \frac{Average\ Demand}{Plant\ Capacity} = 0.5 \Rightarrow Plant\ Capacity = \frac{15}{0.5} = 30MW$$

$$\therefore \text{Re}\,serve\ capacity = Plant\ capacity - Maximum\ demand = 30 - 25 = 5MW$$

Daily energy produced = Average demand * 24hr = 15 * 24 = 360MWh

Maximum energy that could be produced (MEP)

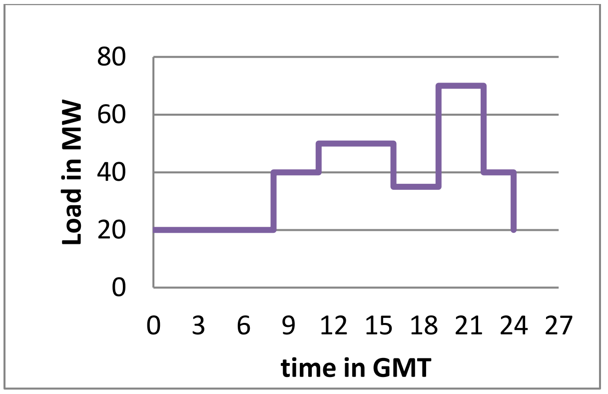
$$MEP. = \frac{Actual\ energy\ produced\ in\ a\ day}{Plant\ Use\ \ factor} = \frac{360}{0.72} = 500MWh$$

Example 7: A proposed station has the following daily load cycle

| **Time (hr)** | **6-8** | **8-11** | **11-16** | **16-19** | **19-22** | **22-24** | **24-6** |
|---|---|---|---|---|---|---|---|
| **Load (MW)** | 20 | 40 | 50 | 35 | 70 | 40 | 20 |

Draw the load curve and select suitable turbine units from the 8, 16, 20, and 24 MW. Prepare the operation schedule for the turbines selected and determine the load factor from the curve.

**Solution:** The load curve of the power station can be drawn to some suitable scale as shown below.




Units generated per day = area (in KWh) under the load curve.

$$= 10^3 \begin{bmatrix} 20 \times 8 + 40 \times 3 + 50 \times 5 + \\ 35 \times 3 + 70 \times 3 + 40 \times 2 \end{bmatrix} = 925 \times 10^3 KWh$$

$$Average\ Load = \frac{925 \times 10^3 KWh}{24h} = 38541.7KW$$

L.F. = 38541.7/70000 = 0.55

The generating units available are 8, 16, 20, and 24 MW. There can be several possibilities. However, while selecting the size and number of units, one has to (i) provide one set of highest capacity as a stand by unit. (ii) make the units meet the maximum demand (70MW in this case). (iii) see overall economy.

For example, if four set of 24MW each may be chosen. Three sets will serve to meet the maximum demand 70 MW and one unit may serve as a standby.

Operational schedule:

- Set No. 1 will run for 24 hrs
- Set no. 2 will run from 8 to the mid night
- Set no. 3 will run from 11 to 16 and again from 19 to 22 hrs

**Example 8:** The following data are obtained from the records of the mean monthly flows of a river for 10 years. The head available at the site of the power plant is 60 m and the plant efficiency is 80%.

| *Mean monthly flow range* ($m^3/s$) | *No. of occurrences (in 10-yr period)* |
|---|---|
| 100-149 | **3** |
| 150-199 | **4** |
| 200-249 | **16** |
| 250-299 | **21** |
| 300-349 | **24** |
| 350-399 | **21** |
| 400-449 | **20** |
| 450-499 | **9** |
| 500-549 | **2** |

1. *Plot the FDC and PDC*
2. *Determine the mean monthly flow that can be expected and the average power that can be developed.*
3. *Indicate the effect of storage on the FDC obtained.*
4. *What would be the trend of the curve if the mean weekly flow data are used instead of monthly flows?*

**Solution**

**1.** The mean monthly flow ranges are arranged in the ascending order as shown in Table below.

**Table:** Flow duration analysis of mean monthly flow data of a river in a 10 yr period

| ***Mean monthly flow C.I.*** **($m^3/s$)** | ***No. of occurrences (in 10-yr period)*** | **No. time the lower CI is equaled or exceeded (m)** | **% of time lower value of CI equaled or exceeded = (m/n) x 100%** | **Monthly P = 9.81x60x0.8xQ (MW); Q is lower value of CI** |
|---|---|---|---|---|
| **100-149** | **3** | **120** | **100** | **47.2** |
| **150-199** | **4** | **117** | **97.5** | **70.8** |
| **200-249** | **16** | **113** | **94.2** | **94.4** |
| **250-299** | **21** | **97** | **80.8** | **118** |
| **300-349** | **24** | **76** | **63.3** | **142** |
| **350-399** | **21** | **52** | **43.3** | **165** |

| | | | | |
|---|---|---|---|---|
| **400-449** | **20** | **31** | **25.8** | **189** |
| **450-499** | **9** | **11** | **9.2** | **212** |
| **500-549** | **2** | **2** | **1.7** | **236** |
| **Total n =120** | | | | |

The number of times that each mean monthly flow range (class interval, C.I.) has been equaled or exceeded (*m*) is worked out as cumulative number of occurrences starting from the bottom of the column of number of occurrences. Since the C.I. of the monthly flows, are arranged in the ascending order of magnitude.

It should be noted that the flow values are arranged in the ascending order of magnitude in the flow duration analysis, since the minimum continuous flow that can be expected almost throughout the year (*i.e.*, for a major percent of time) is required particularly in drought duration and power duration studies, while in flood flow analysis the CI may be arranged in the descending order of magnitude and *m* is worked out from the top as cumulative number of occurrences since the high flows are of interest.

(*i*) The flow duration curve is obtained by plotting $Q$ vs. percent of time in the Fig. ($Q$ = lower value of the CI.).

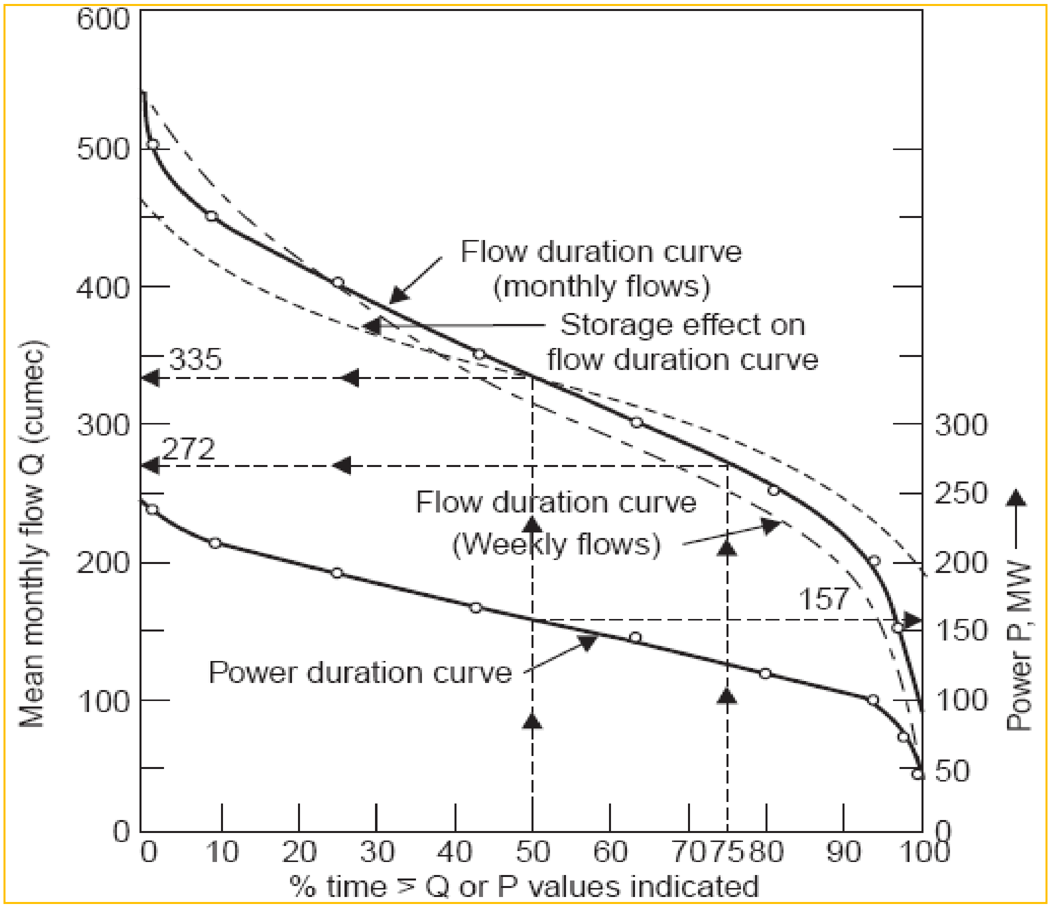
(*ii*) The power duration curve is obtained by plotting $P$ vs. percent of time, see the Figure below.

2. The mean monthly flow that can be expected is the flow that is available for 50% of the time *i.e.*, 335 $m^3/s$ from the FDC drawn.

The average power that can be developed *i.e.*, from the flow available for 50% of the time, is 157 MW, from the PDC drawn.

3. The effect of storage is to raise the flow duration curve on the dry weather portion and lower it on the high flow portion and thus tends to equalize the flow at different times of the year, as indicated in Fig. above.

4. If the mean weekly flow data are used instead of the monthly flow data, the flow duration curve lies below the curve obtained from monthly flows for about 75% of the time towards the drier part of the year and above it for the rest of the year as indicated in Figure above.




*In fact the flow duration curve obtained from daily flow data gives the details more accurately (particularly near the ends) than the curves obtained from weekly or monthly flow data but the latter provide smooth curves because of their averaged out values.*

**Example 9:** A run-of-river plant with an effective head of 22 m and plant efficiency of 80% supplies power to a variable load as given below: Draw the load curve and determine

(i) The minimum average daily flow to supply the indicated load

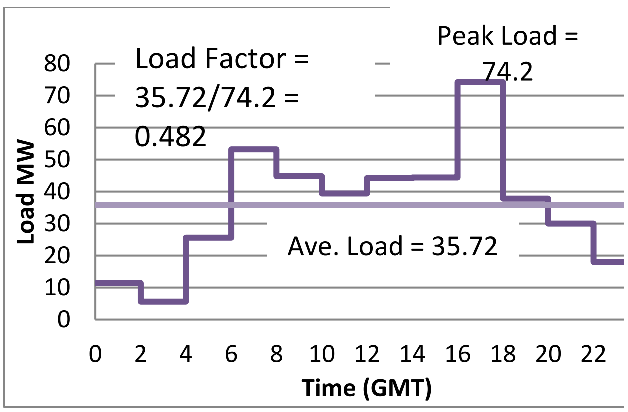
(ii) Pondage required to produce the necessary power at the peak

(iii) The plant load factor

| Time (hr) | Load (1000 kW) | Time (hr) | Load (1000 kW) |
|---|---|---|---|
| 0-2 | 11.4 | 12-14 | 44.2 |
| 2-4 | 5.6 | 14-16 | 44.4 |
| 4-6 | 25.6 | 16-18 | 74.2 |
| 6-8 | 53.2 | 18-20 | 37.8 |
| 8-10 | 44.8 | 20-22 | 30.0 |
| 10-12 | 39.4 | 22-24 | 18.0 |

**Solution**

(i) The load curve is shown below.




- Total sum of loads at 2-hr intervals = 428.6 x 1000 kW
- Average load = (428.6 x 1000 kW x 2hr)/24hr = 35.72 MW
- Flow, Q, required to develop the average load

$\rightarrow$ Q = 35.72MW/(9.81x22x0.8) = 207 $m^3/s$

(ii) Flow required to produce the required load/demand

Q = P in 1000 kW/(9.81x22x0.8) = **5.8 x Load in 1000 kW**

To determine the pondage capacity the table below is prepared. From the table

Total deficiency = Total excess = 510 $m^3/s$

Therefore, pondage capacity required = 510 $m^3/s$ for 2 hrs

= 510 x (2 x 60 x 60) = 3.67 x $10^6$ $m^3$ or 3.67 $Mm^3$

(iii) Plant load factor is the ratio of average load to peak load,

35.72/74.2 = 0.482

| Time (hr) | Load (MW) | Required flow ($m^3/s$) | Deviation from the average flow of 207 $m^3/s$ | |
|---|---|---|---|---|
| | | | Deficiency | Excess |
| 0-2 | 11.4 | 66.1 | | 140.90 |
| 2-4 | 5.6 | 32.46 | | 174.54 |
| 4-6 | 25.6 | 148.4 | | 58.60 |
| 6-8 | 53.2 | 308.2 | 101.2 | |
| 8-10 | 44.8 | 260.0 | 53.0 | |
| 10-12 | 39.4 | 228.5 | 21.5 | |
| 12-14 | 44.2 | 256.0 | 49.0 | |
| 14-16 | 44.4 | 257.4 | 50.4 | |
| 16-18 | 74.2 | 430.0 | 223.0 | |
| 18-20 | 37.8 | 219.4 | 12.4 | |
| 20-22 | 30.0 | 174.0 | | 33.0 |
| 22-24 | 18.0 | 104.3 | | 102.7 |
| Total | 428.6 | | **510.1** | **509.74** |

# Chapter 3: Turbine selection and capacity determination

Contents

1. Turbine types
2. Limits of Use of Turbine Types
3. Turbine selection criteria
    1. Rotational speed
    2. Specific speed
    3. Maximum efficiency
4. Determination of Number of Units
5. Power house

## 3.1. Turbine types

Hydraulic Turbines transfer the energy from a flowing fluid to a rotating shaft. Turbine itself means a thing which rotates or spins. Hydraulic Turbines have a row of blades fitted to the rotating shaft. Flowing liquid, mostly water, when passes through the Turbine it strikes the blades of the turbine and makes the shaft rotate.

For every specific use, a particular type of Hydraulic Turbine provides an optimum output. Thus a hydro turbine is usually tailor made in order to fit a particular net hydraulic head and a design flow discharge.

Turbines are classified based on Flow path or pressure change

**Turbines types: Based on flow path**

(i) **Axial Flow Hydraulic Turbines:** Hydraulic Turbines having the flow path of the water mainly parallel to the axis of rotation.

(ii) **Radial Flow Hydraulic Turbines:** Hydraulic Turbines having the water flowing mainly in a plane perpendicular to the axis of rotation.

(iii) **Mixed Flow Hydraulic Turbines:** Hydraulic Turbines having significant component of both axial and radial flows. Francis Turbine is an example of mixed flow type, in Francis Turbine water enters in radial direction and exits in axial direction.

**None of the Hydraulic Turbines are purely axial flow or purely radial flow**. There is always a component of radial flow in axial flow turbines and of axial flow in radial flow turbines.

**Turbine Types: Based on pressure change**

One more important criterion for classification of Hydraulic Turbines is whether the pressure of water changes or not while it flows through the runner of the Hydraulic Turbines. Based on the pressure change Hydraulic Turbines can be classified as of two types.

(i) **Impulse Turbine:** The pressure of water does not change while flowing through the runner. In Impulse Turbines pressure change occur only in the nozzles. One such example of impulse turbine is Pelton Wheel.

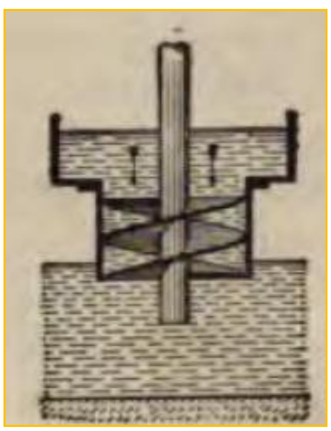
(ii) **Reaction Turbine:** The pressure of water changes while it flows through the runner. The change in water velocity and reduction in its pressure causes a reaction on the turbine blades; this is where from the name Reaction Turbine may have been derived. Francis and Propeller Turbines fall in the category of Reaction Turbines.

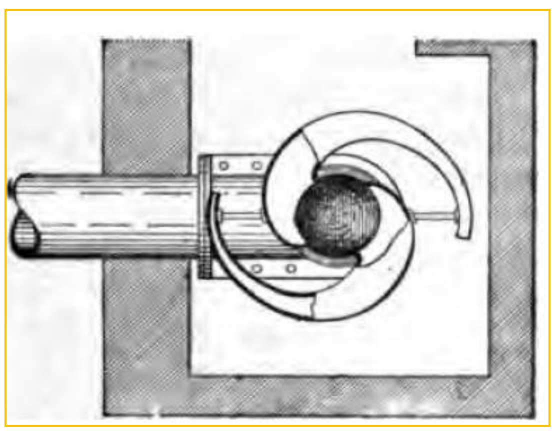
### Turbine types: Reaction

In reaction turbines only a part of the inlet hydraulic energy is converted into velocity energy in the stationary turbine parts. Thus, the conversion of hydraulic to mechanical energy in the runner can be divided into two:

- The impulse action caused by the change of velocity direction from the runner inlet to the outlet, and
- The reaction contribution caused by the pressure drop through the runner. The pressure drop is obtained because the runner is completely filled with water. In the draft tube (Connecting the outlet of the turbine runner to the tail race) some of the velocity energy at the runner outlet is converted to potential energy.

Francis, Deriaz and propeller turbines belong to this group.



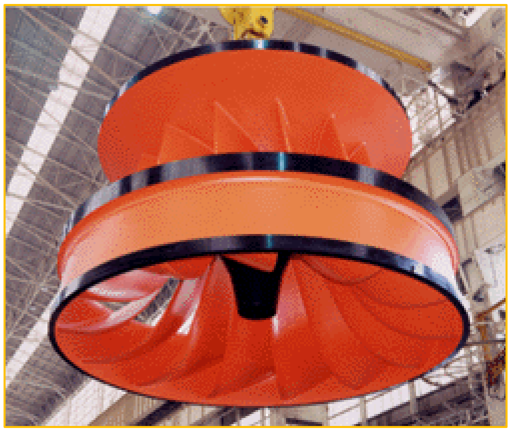
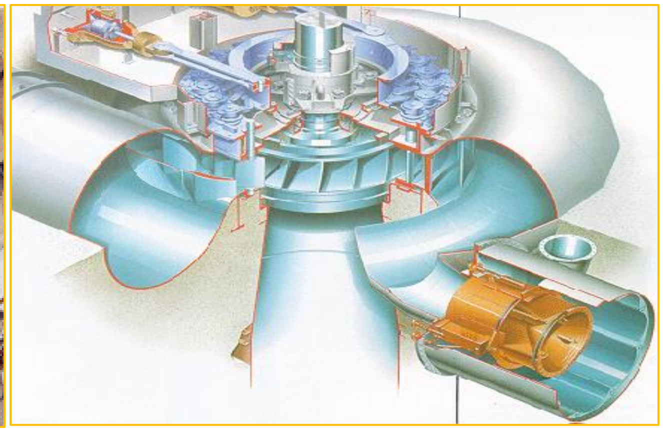


Screw turbine (Axial) Scotch turbine (Tangential)

**Screw turbine**: the water gliding over the blades forms the rotation of the shaft

**Scotch turbine**: where the exit water momentum will make the shaft to rotate.

**The Francis Turbine:**

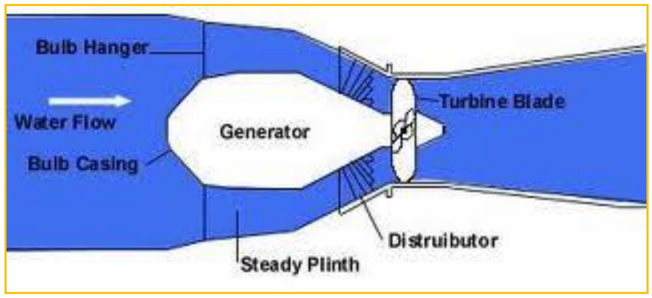
- It is a reaction turbine developed by Sir J.B. Francis.
- The water enters the turbine through the outer periphery of the runner in the radial direction and leaves the runner in the axial direction, and hence it is called '**mixed flow turbine**'.
- Only a part of the available hydraulic head is converted into the velocity head before water enters the runner. The pressure head goes on decreasing as the water flows over the runner.
- The pressure at the runner exit may be less than the atmospheric pressure and thus water fills all the passages of the runner.
- The change in pressure while water is gliding over the blades is called '**reaction pressure**' and is partly responsible for the rotation of the runner.
- A Francis turbine is suitable for high heads (70 to 500 m) and requires a medium quantity of water.

**Propeller turbine**

A propeller turbine generally has a runner with three to six blades in which the water contacts all of the blades constantly. The pitch of the blades may be fixed or adjustable. The major components besides the runner are a scroll case, wicket gates, and a draft tube. Types of propeller turbines:

- **Bulb turbine:** The turbine and generator are a sealed unit placed directly in the water stream.
- **Kaplan:** Both the blades and the wicket gates are adjustable, allowing for a wider range of operation.
- **Other include Straflo and Tube turbines**

**Bulb Turbine**


Used for very low head applications with a net head range from 0.5 to 30 m

The most important feature of a bulb turbine is the horizontal positioning of the shaft, which results in a reduced size compared to a vertical shaft arrangement. After the intake, the water passes through the moveable wicket gates, which are another device besides the runner for adjusting the water discharge and, as a result, the turbine output. After passing the runner, the water exits from the turbine through the draft tube. The generator is located in a watertight housing called the "bulb".

Small versions of the bulb turbine can be lowered into a stream by hand to power a remote home

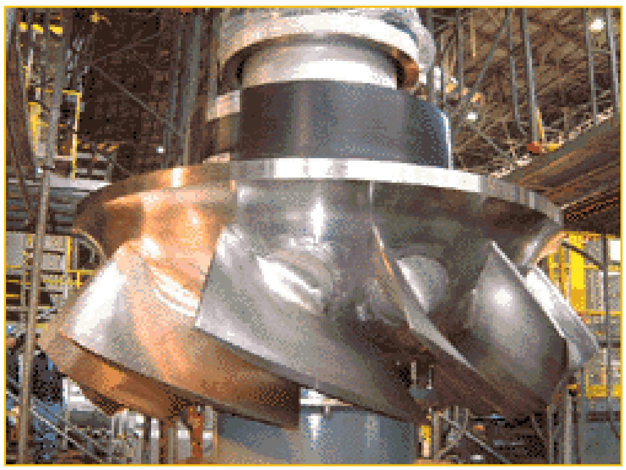
**Kaplan Turbine (Axial)**

- It is a reaction turbine developed by Victor Kaplan ,1919
- The water enters the turbine through the outer periphery of the runner in the axial direction and leaves the runner in the axial direction, and hence it is called '**axial flow turbine**'.
- Only a part of the available hydraulic head is converted into the velocity head before water enters the runner. The pressure head goes on decreasing as the water flows over the runner blades.
- The static pressure at the runner exit may be less than the atmospheric pressure and thus water fills all the passages of the runner blades.

- The ‘reaction pressure’ is partly responsible for the rotation of the runner.
- A Kaplan turbine is suitable for medium heads (16 to 70 m) and requires a relatively large quantity of water.

**Deriaz turbine**

- Deriaz turbine is a reaction turbine developed by Paul Deriaz. It was the first diagonal turbine, invented in 1956.
- Deriaz turbine is a mixed-flow turbine, which means that water enters the runner radially and leaves it axially.
- It is similar to Kaplan turbine but its runner blades are more inclined, and thus it is more suitable especially for medium heads, ranging between 20 and 100 meters.
- Deriaz turbine can act as pump too.



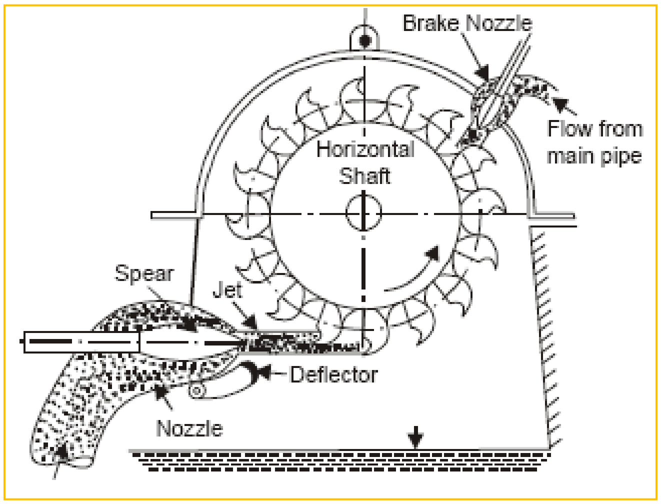
- The positive aspect of Deriaz turbine is that it can have fixed or adjustable blades. When they are adjustable, it enables the turbine to work with high efficiency in various loads and heads. Therefore, it is suitable for a power station with wide variation of head and discharge.
- The other positive aspect is its higher efficiency at part-load operation when compared to Kaplan turbine.
- The Deriaz turbine has reduced discharge at overspeed.
- It can be used for reversible pump-turbine service for heads above 90m.
- The hub contains the blade servomotor and operating linkages used for adjusting position of the runner blade.

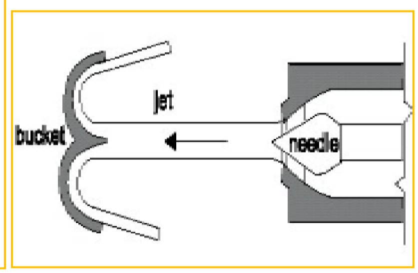
## Turbine types: Impulse (or Velocity Turbines)

- Where all the hydraulic energy entering the turbine is converted into velocity energy in the stationary parts in front of the runner. The runner is only partly filled with water, and operates in nearly atmospheric pressure.

- As the water enters/hits the turbine runner tangentially  they are called Tangential flow turbines

- All impulse turbines are tangential flow turbines
    - The Pelton Turbine used for the highest heads
    - Turgo impulse turbine
    - Cross flow turbine (used in substitution of Francis turbine).




The Pelton Turbine: Patented by Lester Pelton 1880

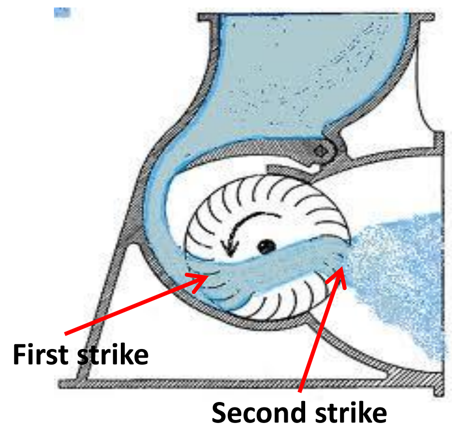
**Turgo Impulse turbine**

The turbine is designed so that the jet of water strikes the buckets at an angle to the face of the runner and the water passes over the buckets in an axial direction before being discharged at the opposite side. **It is invented by Eric Crewdson in 1920.**

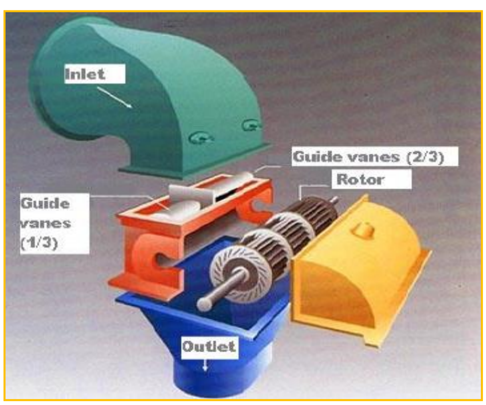
### The Cross-flow impulse turbine

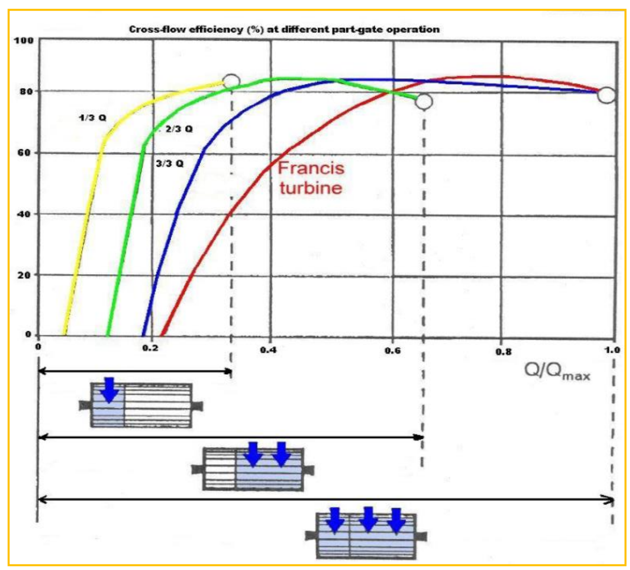
An impulse turbine also called the Banki or Michell turbine. The name "cross-flow" comes from the fact that the water crosses through the runner vanes twice in producing the rotation (see Figure).

The cross-flow principle was developed by Michell, an Austrian engineer, in 1903. Professor Banki, a Hungarian engineer, developed the machine further.




### Divided Cells of a cross-flow turbine




Advantage of Divided Cells of a cross-flow turbine

- During low flow 1/3 of runner width operates
- During medium flow 2/3 of runner width operates
- During high flow 3/3 or whole of runner width operates
- The turbine can be operated efficiently starting from 20% part-gate

### 3.2. 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.

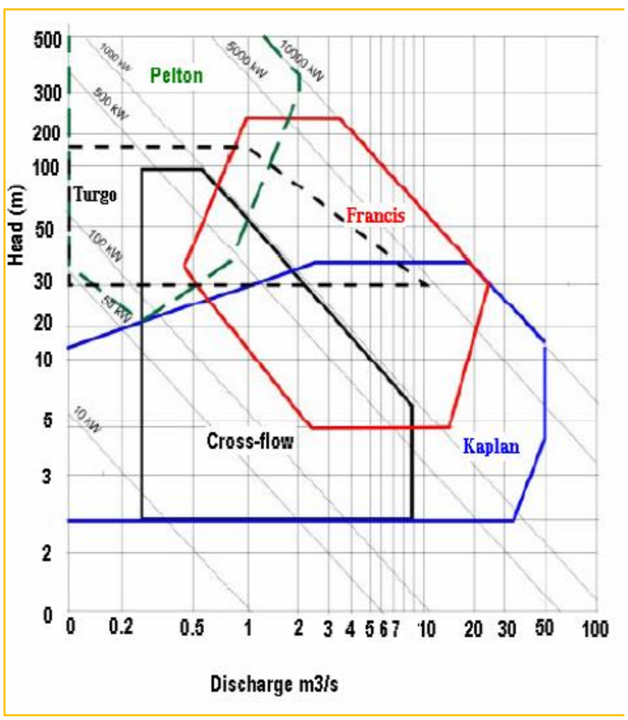
**Pelton Impulse turbines** normally have most economical application at heads above 300m, but for small units and cases where surge protection is important, impulse turbines are used with lower heads.

For **Francis turbines** the units can be operated over a range of flows from approximately 50 to 115% best-efficiency discharge. The approximate limits of head range from 60 to 125% of design head.

**Propeller turbines** have been developed for heads from 5 to 60m but are normally used for heads less than 30m. 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.

**Operational Envelopes:** The rated flow and the net head determine the set of turbine types applicable to the site and the flow environment. Suitable turbines are those for which the given rated flow and net head plot within the operational envelope




**Head:**

Low head, 1.5-15m, reaction-Propeller

Medium head, 16-70m, reaction-Kaplan

High head, 71-500m, reaction- Francis

Very high head, >500m, Impulse-Pelton

**Discharge:**

Low discharge, Impulse- Pelton

Intermediate discharge, Reaction-Francis

High discharge, Reaction-Kaplan

## 3.3. Turbine selection criteria

The usual practice is to base selection on the annual energy output of the plant and the least cost of that energy for the particular scale of hydropower installation. Generally, the selection shall be based on: Available head, Available discharge, Power demand fluctuation and Cost

For small head, the discharge requirement is high, requiring bigger turbines; thus costly. For larger head, the discharge requirement is low, requiring smaller turbines; thus cheaper. (**Why?**)

The choice of a suitable hydraulic prime-mover (Turbine) depends upon various considerations for the given head and discharge at a particular site of the power plant. The type of the turbine can be determined if the head available, power to be developed and speed at which it has to run are known to the engineer beforehand. The following factors have the bearing on the selection of the right type of hydraulic turbine:

I. Rotational Speed – Generator

II. Specific Speed;

III. Maximum Efficiency;

### Rotational speed

***Turbine or synchronous speed***: Since turbine and generator are fixed, the rated speed of the turbine is the same as the speed of the generator.

In all modern hydraulic power plants, the turbines are directly coupled to the generator to reduce the transmission losses. This arrangement of coupling narrows down the range of the speed to be used for the prime-mover. **The generator generates the power at constant voltage and frequency** and, therefore, the generator has to operate at its synchronous speed. The synchronous speed of a generator is given by $N = 60\frac{f}{p}$

Where*: N* speed rpm*; f*- frequency of the generator (usually 50 *hz* or 60 *hz*), *p*- **number of pair of poles** of the generator

*f and p* are constants thus *N* is constant.

Problems associated with the high speed turbines are the danger of cavitation and centrifugal forces acting on the turbine parts which require robust construction. No doubt, the overall cost of the plant will be reduced by adopting higher rotational speed as smaller turbine and smaller generator are required to generate the same power. The construction cost of the power house is also reduced.

The ratio of the peripheral speed, *v*, 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, $\phi$.

$$\phi = \frac{v}{\sqrt{2gH}} = \frac{\omega r}{\sqrt{2gH}} = \frac{\pi DN}{60\sqrt{2gH}} = \frac{DN}{84.6\sqrt{H}}$$

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

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

**Cavitation**

A reduced pressure under the blades (or buckets) of a turbine runner may lead to cavitation – phenomenon detrimental to the turbine. The term cavitation basically refers to the ability of cold water to boil under low pressure. Under a normal absolute barometric pressure of 1 bar water starts to boil at 100 $^oC$. However, when the pressure drops to 0.033 bar (which is called the critical pressure, $P_{cr}$) it may begin to boil at 25 $^oC$, that is, at normal river water temperature.

When the pressure under a runner approaches $P_{cr}$, the water in the stream starts boiling, giving rise to cavities (known as cavitation bubbles) filled with water vapor.

The boundary between the low-pressure zone immediately under the blades (or buckets) and the high-pressure zone in the stream above the runner follows an extremely unstable pattern. The cavitation bubbles find themselves from time to time in the high-pressure zone. As a result, the vapor instantly condenses and a cavitation bubble collapses. As this takes place, an enormous pressure develops at the bubble center, which spreads quickly in an explosion-like manner.

A series of such micro-explosions following one another at very short intervals causes a good deal of noise and vibration in the turbine and may provoke the runner blades into pitting.

## Specific Speed

The turbine specific speed is a quantity derived from dimensional analysis. For a specific turbine type (Francis, Kaplan, Pelton), the turbine efficiency will be primarily a function of specific speed.

Neglecting, the effect of hydraulic losses on the turbine power, the power will be given by: $P = \eta\gamma QH$

Obviously, the turbine should have as large an efficiency $\eta$ as possible. In general, η will depend on the specific geometrical configuration of the turbine system, as well as the flow rate $Q$, the head H, and the turbine rotation rate $N$.

For specific values of $Q$, $H$, and $N$, an optimum geometrical design would exist which would optimize the turbine efficiency. Determination of this optimum design would be performed using either experimental method.

Alternatively, given a specific turbine design (i.e., Francis, Kaplan, Pelton), one would anticipate that there would be a specific set of operating conditions Q, *H, and N which would optimize the turbine efficiency.*

The basic concept of the *turbine specific speed* is to identify the optimum operating conditions for a given turbine design. This identification process can be developed via simple dimensional analysis, coupled with an inviscid (i.e., ideal) model of fluid mechanics.

Say that we have developed an optimized turbine design (i.e., maximized *$\eta$) for the specific conditions $Q_1$, $H_1$, and $N_1$. This design would have associated it a characteristic* size *$D_1$ (*for example the turbine runner diameter).

Since the design is optimized for the specific conditions, we can state that $P_1 = \eta_{opt}\gamma Q_1 H_1$

where $\eta_{opt}$ is the optimum (i.e., maximized) efficiency.

Say we change the head to some new value $H_2$ we want to estimate the corresponding conditions $Q_2$ and $N_2$ which will maintain the optimum efficiency of the turbine. Or, perhaps, we scale the turbine to a new characteristic size $D_2$: what are the corresponding new values of $N_2$, $Q_2$, $H_2$ which maintain $\eta_{opt}$?

Such dimensional analysis would finally result in a quantity called specific speed of the turbine

$$\frac{N_1 (P_1)^{0.5}}{(H_1)^{1.25}} = \frac{N_2 (P_2)^{0.5}}{(H_2)^{1.25}} = const. = N_s$$

The quantity $N_s$ *is referred to as the specific speed of* the turbine. Understand that $N_s$, *as defined above*, is not a dimensionless quantity – we would need to appropriately include *$\rho$ and $g$ to cancel out the units*. Perhaps dividing it by $g^{5/4}\rho^{1/2}$.

**Meaning of specific speed**: Any turbine, with identical geometric proportions, even if the sizes are different, will have the same specific speed. If the model had been refined to get the optimum hydraulic efficiency, all turbines with the same specific speed will also have an optimum efficiency.

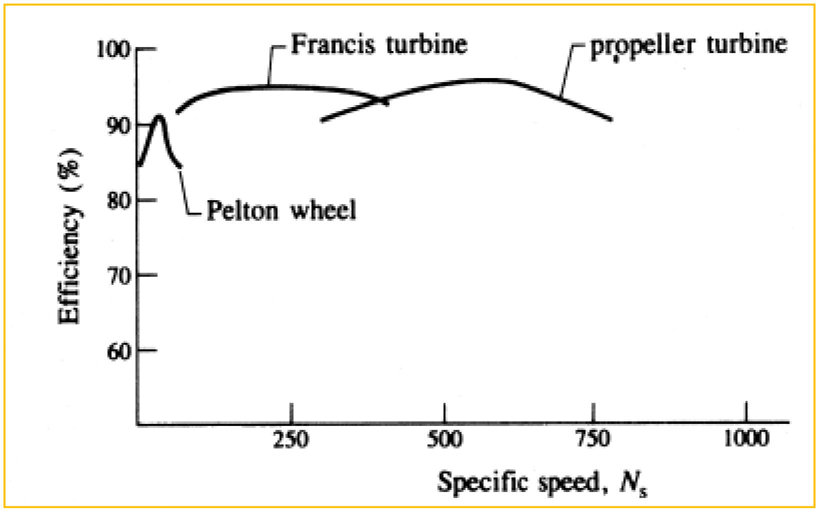
In all modern power plants, it is common practice to select a high specific speed turbine because it is more economical as the size of the turbo-generator as well as that of power house will be smaller.

Suppose generator of a given power runs at either 120 rpm or at 800 rpm and say available head is 200 meters. If the power developed in a single unit at 120 rpm is 60000 KW, the required specific speed of the runner will be 39.08 rpm. Now if the same power is developed at 800 rpm, the required specific speed of the runner will be 260.54 rpm.

The above calculations show that the required power can be developed either with one impulse turbine (Pelton) or reaction turbine (Francis).

### Maximum Efficiency

The maximum efficiency the turbine can develop depends upon the type of the runner used. In case of impulse turbine, low specific speed is not conducive to efficiency, since the diameter of the wheel becomes relatively large in proportion to the power developed so that the bearing tends to become too large.




The low specific speed of reaction turbine is also not conducive to efficiency. The large dimensions of the wheel at low specific speed contribute disc friction losses.

The leakage loss is more as the leakage area through the clearance spaces becomes greater and the hydraulic friction through small bracket passages is larger. These factors tend to reduce the efficiency as small values of specific speed are approached.

In most hydropower design problems, one would typically know beforehand the available head $H$ and the total available flow-rate $Q$. Then the power $P$ produced by the turbine, assuming an efficiency of 100% and no head losses, could be estimated from: $P = \eta\gamma QH \ldots(a)$

The figure above could then be used, in conjunction with the head H and the estimated power $P$, to determine the corresponding rotational speeds of Pelton, Francis, and Kaplan turbines operating at their maximum efficiency, i.e., $N = \frac{N_s H^{1.25}}{P^{0.5}}$

An alternative approach is to specify beforehand the desired rotation rate $N$ of the turbine. The power produced by the three turbine types, operating at optimum efficiency, would then be obtained by: $P = \frac{N_s^2 H^{2.5}}{N^2}$

Q could then be calculated from Eq. (*a*), and the number of required turbine units would be obtained from the total available flow rate divided by the flow through a single turbine.

### 3.4. Determination of number of units

It is cost effective to have a 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 great.

Factors governing selection of number of units:

- Space limitations by geology or existing structure.
- Transportation facility
- Possibility of onsite fabrication
  - Is costly and practical only for multiple units
  - Runners may be split in two pieces, completely machined in the factory and bolted together in the field. This is avoided as the integrity of the runner cannot be assured.

The current trend is to have small number of units having larger sizes, as larger sized units have a better efficiency.

Example 1: Installed capacity needed: 1500 MW, set the number of turbine required.

**Solution:** This could have a number of alternatives on the number of units' selection

- +1 units of 500 MW capacity
- +1 units of 300 MW capacity
- 8 +0 units of 200 MW capacity
- 1+1 unit of 1500 MW capacity
- Etc.

**Example 2:** A turbine is to operate under a head of 25 m at 200 rpm. The discharge is 9 m$^3$/s. If the efficiency is 90% determine the power generated; specific speed of the turbine and type of the turbine.

**Solution:**

$$P = \eta\gamma Qh = 0.9 \times 9.81 \times 9 \times 25 = 1986.523 KW$$

$$N_s = \frac{N(P)^{0.5}}{(H)^{1.25}} = \frac{200(1986.523)^{0.5}}{(25)^{1.25}} = 159.5$$

As the specific speed lies between 60 and 400 Francis turbine is selected

**Example 3:** What type of turbine would be used if the discharge is of 0.283 $m^3/s$ with a head of 75m? Assume an efficiency of 80% and rotational speed of 600rpm.

**Solution:**

$$P = \eta\gamma Qh = 0.9 \times 9.81 \times 0.283 \times 75 = 166.6 KW$$

$$N_s = \frac{N(P)^{0.5}}{(H)^{1.25}} = \frac{600(166.6)^{0.5}}{(75)^{1.25}} = 35.09$$

It would therefore be necessary to use a Fast Pelton Turbine. However, it might be possible to use a Pelton Wheel with two jets.

Power per jet = 166.6/2 = 83.3KW. Therefore, $N_s$ per Jet = 24.8 which is Medium speed Pelton wheel.

If a Pelton Wheel with six jets is used:

Power per jet = 166.6/6 = 27.76KW. Therefore, $N_s$ per Jet = 14.32 which is Slow speed Pelton wheel. This would be a practical proposition but would result in some loss of efficiency due to interference between the jets. Consequently, a better alternative would be to have two wheels on the same shaft with two jets per wheel.

**Example 4:** In a hydropower station, water is available at a rate of 175 $m^3/s$ under a head of 18 m. The turbines run at a speed of 150 rpm with overall efficiency of 82%. Find the number of turbines required if they have the maximum specific speed of 460.

**Solution**

Specific speed of the turbine:

$$N_s = \frac{N(P)^{0.5}}{(H)^{1.25}} = \frac{150(P)^{0.5}}{(18)^{1.25}} = 460$$

$\Rightarrow$ *Power generated by a turbine,* $P = 12927.5kw$

*Flow required by a turbine*

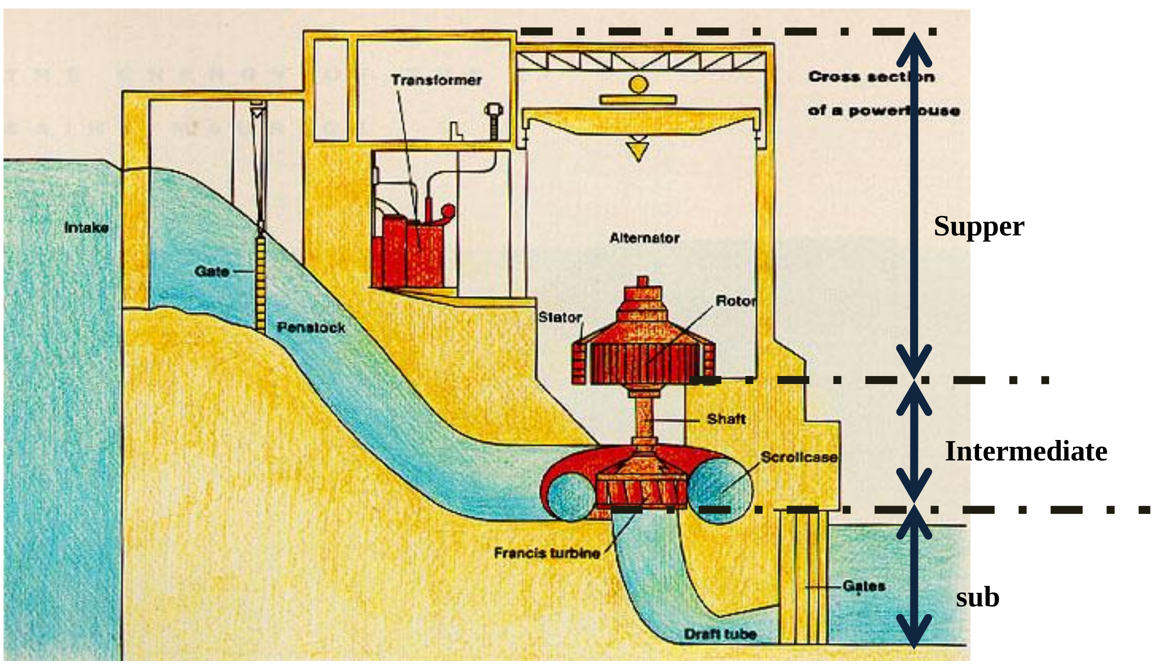
$$Q = \frac{P}{\eta\gamma h} = \frac{12927.5}{0.82 \times 9.81 \times 18} = 89.281\, m^3/s$$

$$No.\ of\ turbines = \frac{175}{89.281} = 1.96\ say\ 2\ turbines$$

## 3.5. Power House

The essential equipment needed in hydro-electric power generation are housed suitably in a structural complex called Power House. The major equipment in a power house: Turbines, Generators, Transformers, switch boards; shaft, ventilation, cranes, etc.

According to the location of the hydropower station, the power houses are classified as surface power house or underground power house. As the name implies, the underground power house is one which is built underground. A cavity is excavated inside earth surface where sound rock is available to house the power station. A surface power house is one which is founded on earth's surface and its superstructure rests on the foundation. The surface power house has been broadly divided into three sections which is separated from the intake: *Substructure; Intermediate structure; Super-structure*



## Substructure

The substructure of a power-house is defined as that part which extends from the bottom of the turbine to the soil or rock. Its purpose is to house the passage for the water coming out of the turbine.

**Hydraulic function**

In case of reaction turbines the substructure is used to provide a diverging passage (draft tube) where the velocity of the exit water is gradually reduced in order to reduce the loss in pushing out the water. In case of impulse turbine, such a draft tube is not required and only an exit gallery would serve the purpose.

**Structural function is dual:**

- To safely carry the superimposed loads of machines and other structures over the cavities.
- To act as transition foundation member that distributes heavy machine loads on the soil such that the obtainable ground pressures are within safe limits.

## Intermediate structure

The intermediate structure is part of the power house which extends from the top of the draft tube to top of the generator foundation.

This structure contains two important elements of the power house, one is the scroll case which feeds water to the turbine. The generator foundation rests on the scroll-case which is embedded in the concrete. Other galleries and chambers also rest on the same foundation.

Scroll or spiral case (for reaction turbine only) is a part of the turbine that distributes water coming from penstock uniformly and smoothly through guide vanes to the turbine. In case of impulse turbine **manifold** supplying water to the nozzles will serve this purpose.

The structural function of the generator foundation in the intermediate structure is to support the generator. Arrangements may be made either to transmit the load directly to the substructure through steel barrel or through a column beam or slab.

The structural function of the concrete around scroll case would depend upon the type of scroll case used:

If the scroll case is made of steel and strong enough to withstand internal loads including the water hammer effects, the surrounding concrete acts more or less as a space fill and a medium to distribute the generator loads to the substructure.

If it is a concrete scroll case then this concrete should be strong enough to withstand the internal hydrostatic and water hammer head as well as the external superimposed loads on account of the machine etc. Usually, a steel scroll case is used as water linear.

### Superstructure:

The part of the power house above the generator floor right up to the roof is known as superstructure. This part provides walls and roofs to power station and also provides an overhead travelling crane for handling heavy machine parts.

### Power House Dimensions

**Length**: depends on the number of units installed; the units shall be spaced at

- 5D + 2.5m where D is the turbine outer diameter
- 4D + 2.5m for high specific speed turbines
- Additional length is provided for loading/erection and control room is required

**Width**: equal to the center to center spacing of the units plus the clearance space from the wall

**Height:** fixed by the head room requirements of the crane operation.

### Underground power house

Depending on the rock quality, tunneling ease, overall economics, and the power house may be located:

- The plant totally underground
- In a pit: the PH is accessed from the surface
- Semi-underground: turbine in the ground while generator is on the surface
- In a rock cut

**Advantages:**

- Shorter underground conduit
- Cheaper penstock design
- Favorable construction conditions
- Preserve the landscape

**Disadvantages**

- Higher construction cost
- Higher operational cost
    - The lighting cost.
    - *The running cost of air-conditioned plant.*
    - *The removal of water seeping*

## Chapter 4: Water Passages

Contents

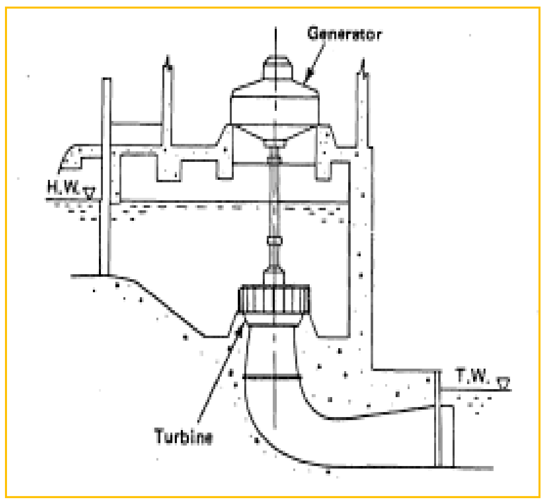
1. Power canal/tunnel
2. Forebay
3. Penstock
4. Spiral cases
5. Draft tubes
   1. Turbine setting

### 4.1. Power Canal/tunnel

Necessary components of the hydraulic turbines in a hydropower installation are specially designed water passages and gates for controlling and directing the water as it flows **to**, **through**, and **from** the turbines. The principal features to consider in engineering feasibility and design studies are the **Power canal/tunnel, penstocks, spiral cases, and draft tubes**.

**Power Canal:** In very simple low-head installations the water can be conveyed in an open channel directly to the runner. Power canal settings of turbines do require a protective entrance with a trash rack. The principal problem to be solved is to provide inlet conditions to the turbine that are relatively free from swirling and vortex flow as the water approaches the turbine runner.

A usual upper limit of the use of power canal settings for hydraulic turbines is that heads should not exceed 6 m. Flumes and canals are also used to convey water to penstocks for turbine installations with higher heads.




### Head Race:

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

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

***Canal Design:*** Involves determination of the following:

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

**i) Carrying Capacity and Velocity:**

The discharge is computed from continuity equation as **Q = VA**

The roughness coefficient is specified from the bed material type.

***Chezy's equation:*** $V = C\sqrt{RS}$

To determine the value of C we can use.

- Manning's Formula $C = \frac{1}{n} R^{1/6}$
    - Where n is Manning's roughness coefficient
- The Chezy-Manning equation
- Kutter Formula $V = \frac{1}{n} R^{2/3} S^{1/2} = M R^{2/3} S^{1/2}$
- The Agroskin formula

Apart from the hydraulic computations, the flow velocities in the canal or other water conduits in general are determined according to *economic point of views* (investments, head losses, wear and tear of material, danger of erosion and silting).

The velocity must be high enough to prevent sedimentation. It has to be low enough to prevent bed erosion for unlined-canals and wear by abrasion for lined-canals.

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

| | | | | |
|---|---|---|---|---|
| Clay | 2.0 | Concrete | 5.0 | |

Lowering the velocity keeps the head loss over the length of the canal to a minimum; however, it increases the cost necessary to construct the canal as the cross-sectional area increases when the velocity lowers.

In unlined canals flow velocities are limited by the resistance of the bed material to erosion. In lined canals flow velocities are limited by resistance against wear.

***Maximum Velocities***

Critical bottom velocity (w.r.t. erosion) is given by:

1. Strenberg: $V_b = \zeta\sqrt{2d}$ for d is particle size in meters, $\zeta = 4.43$
2. Maximum permissible mean velocity according to Bogardi and Yen is given by: $V = 22.9 d_m^{4/9}\sqrt{S_s - 1}$
    - Where $d_m$ is mean particle size and $S_s$ is specific gravity of particles.

***Minimum Velocities***

There are various recommendations for non silting velocity

1) According to Ludin
    - If $V_{min} > 0.3$ m/sec, there will be no silting (for silty sediments)
    - $V_{min} > 0.3$ to 0.5 m/sec, there will be no silting (for sandy sediments)
2) According to R.C. Kennedy
    - Non-scouring and non silting velocity is given by:
    - Where h is depth of water in meters and C is coefficient varying from 0.54 to 0.7, depending on silt load.

**ii) Roughness coefficient**

As water flows in a canal, it loses its energy in the process of sliding past the walls and bed material. The rougher the material, the more frictional loss and the greater the head drop or slope needed for a given velocity. The roughness coefficient, n, for various canal materials can be found in standard open channel books.

**iii) Power Canal Slopes**

In plain areas slope between 5 to 20 cm/km are in use. In mountainous areas slopes are as steep as 1 to 2 m/km. The canal bed slope can also be estimated using the Manning's equation:

$$S = \frac{n^2 V^2}{R^{4/3}}$$

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

**iv) Cross-sectional Profile:**

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

- A ***rectangular*** cross-section is often most appropriate when excavation is undertaken in firm rock. It is also commonly used when the canal incorporates properly constructed masonry walls. Use of a rectangular canal reduces the excavation required. For the most efficient rectangular cross-section, the width of the canal is twice the depth of the wetted area and, like a trapezoidal section, is a section in which a semi-circle can be inscribed.
- A ***semi-circular*** cross-section is the most efficient profile because, for a given canal slope and cross-sectional area, it conveys the maximum flow. However, this form is impractical to excavate. It is therefore used primarily with materials which lend themselves to this shape. Examples are prefabricated concrete, sheet metal, and wood-stave sections.
- A ***trapezoidal*** cross-section is the most widely used profile for both lined and unlined canals excavated in earth. If the canal is unlined, the maximum side slope is set by that slope at which the material will permanently stand under water. The magnitude of the side slope of a lined trapezoidal canal depends on the nature of the material on which the lining will rest, but usually steeper than unlined canals. In general, it should be nearly equal to the angle of repose of the natural soil so that no earth pressure is exerted on the back of the lining. The banks of a lined canal, resting on almost any free-draining material requires slopes not steeper than 1:1. For a trapezoidal canal with a given side slope, the most efficient cross-section is one in which a semi-circle can be inscribed in the wetted area. For this section, it can be shown that the length of either sloping side of the wetted area is half its top width.

***Freeboard Allowance:***

For earth canal the lower limit is 35 cm and the upper limit is 140 cm. Generally the free board = *[0.35+1/4h]* m. Where h is depth of flow. Allowances should be made for bank settlements.

For lined canals, the top of the lining is not usually extended for the full height of the free board. Usually it is extended to 15cm to 70cm above the design water level.

**Water Loss in Power Canals**

Water losses are due to

a) Seepage
b) Evaporation
c) Leakage at gates

Generally b) and c) are of minor importance.

## Tunnels

Tunnels are underground conveyance structures. A name tunnel indicates a very small bottom slope, i.e. tunnels are aligned nearly horizontal. Shaft is a tunnel with vertical alignment or inclined shaft when it is steeply inclined to the horizontal.

Tunneling is popular in hydropower because:

- It provides a direct and short route for the water passage thus resulting in considerable saving in cost
- Tunneling work can be started simultaneously at many points thus leading to quicker completion
- Natural land scape is not disturbed
- Development of new techniques of drilling & blasting, rock mechanics and machines.

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

***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*: this is a passage tunnel from surface to underground power house.

***Water carrying 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.

***Water carrying tunnels*** include:

- *Head race/power 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.

### Classification of Tunnels

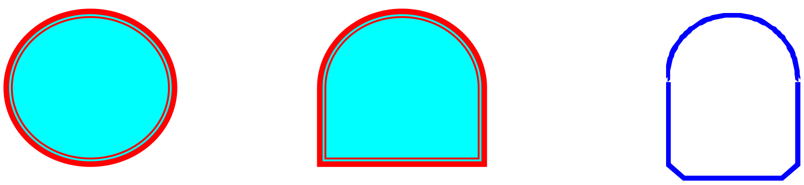
In addition to the above classification tunnels may be classified on the basis of **alignment, shape and design aspects**.

**Alignment:** Tunnels (nearly horizontal) and Shafts (nearly vertical)

***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 an arching roof. The horse-shoe shape is the most popular and convenient from the point of view of construction.

Commonly adopted shapes:



a) Circular shape b) Horseshoe shape C) D-shape

**Design Aspects of tunnels**

Aspects of **lining, pressure condition**, etc., can be considered to identify different types of tunnels.

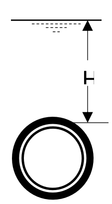
***Lining:*** Lining is a protective layer of concrete, R.C. or steel on the inner surface of the tunnel and it is an important aspect in classification of tunnels. Thus tunnels may be lined, unlined or partially lined.

Lining of tunnels is required:

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

***Pressure tunnels*:** are classified according to pressure head above the soffit of the tunnel. Accordingly:

- Low-pressure tunnels (H < 10 m)
- Medium pressure tunnels (10 m < H < 100 m)
- High-pressure tunnels (H > 100 m)



*Low Head Tunnels*

- The trimmed rock surface may be sufficient by only sealing visible fissure with concrete or cement mortar or granite layer.
- Full lining may be warranted only if external rock load or aggressivity or water head loss reduction justifies it.

*Medium head Tunnels*

- A water tight lining concrete is almost always needed since seepage is more likely to occur under increasing head.
- If the lining is only for water sealing purposes, and no load is carried by it, the permissible internal water pressure head is determined by the depth of overburden and the quality of the rock.
- If depth of overburden rock is $h_r$, Internal pressure head of water is H and $\gamma_r$ & $\gamma_w$ are specific weight of the rock and water, then for equilibrium: $\gamma_w H \leq \gamma_r h_r$; With $\gamma_w = 1$ ton/m$^3$, $H \leq \gamma_r h_r$;
    - Using a factor of safety of $\eta$, $H = \dfrac{\gamma_r h_r}{\eta}(m)$
    - Where factor of safety $\eta$ = 4 to 6.
    - With $\gamma_r$ = 2.4 t/m$^3$ to 3.2 t /m$^3$ and using lower $\eta$ values for good quality rock, one gets H = (0.4 to 0.8) $h_r$

*High Head Pressure Tunnels*

- Usually steel lining is used (R.C. Concrete lining not satisfactory)
- The steel lining is embedded in concrete filling the annular space b/n the steel lining & the rock. In order to provide proper contact b/n rock and concrete and b/n steel lining & concrete, all voids are filled by grouting with cement mortar.
- The profile of the Pressure tunnel should be such that the roof should always be at least 1 to 2m below the hydraulic grade line

In tunnel design:

- Saddles should be provided with dewatering provisions and summits should be provided with outlets or shafts.
- To reduce construction costs, relatively high velocities (higher than in open channels) are permitted in tunnels.
- The permissible velocity depends upon the sediment load carried by the water.
- Size of tunnels cannot be reduced arbitrarily. Requirements of passability limit the maximum size.
    - Minimum size of Tunnel: Circular, 1.8 mϕ ; Rectangular, 2m x 1.6m.

In addition to the general discussion above, as design features *alignment, geometric shape, longitudinal slope, flow velocity, head loss, rock cover (overburden), lining requirements (also coupled with stress analysis), and economic x-section* come in to play.

### 4.2. Forebay

A Forebay, also called a head pond, is a basin located at the end of a power canal just before the entrance to the penstock or pressure shaft. It acts as a transition section between the power canal and the penstock. It is formed simply by widening the power canal at the end.

Through Forebay, the water carried by a power canal is distributed among penstocks that lead to turbines. Water is temporarily stored in a Forebay in the event of rejection of load by turbine and is withdrawn from it when there is demand. Thus, it is a sort of regulating reservoir. It is located at the end of the canal.

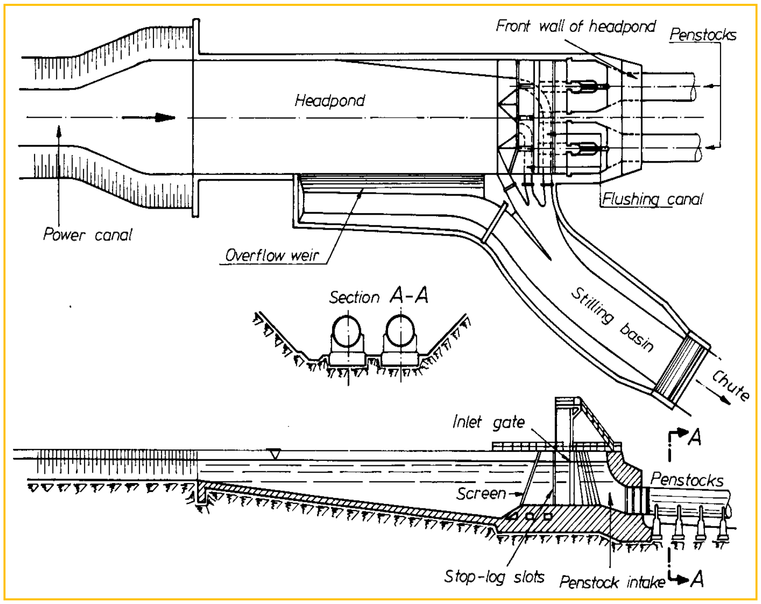
The Forebay is used:

- To decrease the distance to the power house so as to get the turbine on and off within a shorter period.
- To decrease the length of the penstock
- To halt the propagation of pressure waves to the power canal

Components of a Forebay

- The basin: Used to store water and sediment (if any)
- The spillway: Used to dispose excess water that might enter the Forebay
- The bottom outlet: Used for flushing out of the sediment stored in the basin as well as for de-watering the Forebay and the power canal for maintenance

***The penstock inlet:*** Serves in controlling flow into the pressure conduit and in preventing floating debris from entering the conduit. It also provides smooth transition between the basin and the conduit.



**Design Guidelines for a Forebay**

The location of the Forebay is primarily governed by topographic conditions and the geology of the site. The site of both the Forebay and the powerhouse should be selected simultaneously with a view of ensuring the shortest possible penstocks/pressure shafts.

The entire basin of the Forebay may be either excavated in rock or constructed above the terrain, enclosed by embankments and retaining walls. The size of a Forebay varies depending on the sediment content of the water conveyed in the power canal and whether it is to serve for storage.

It is not advisable to design the Forebay as a settling basin if the suspended sediment is fine to cause no damage to the turbines. A gradual transition section should be provided between the power canal and the Forebay basin. In the case of wide Forebay, **baffle piers** are usually constructed at the basin inlet in order to ensure even distribution of flow to the basin.

The bottom of the Forebay basin should be provided with a proper slope to enable periodical flushing of the silt deposited. A bottom lining of the Forebay basin is required in soils where large seepage is expected. The smoothed bottom of the basin is covered with plastic clay having thickness of 20 to 50 cm. The cover is compacted in several layers and is protected against disturbance due to soaking and wave action by a layer of gravel or crushed stone.

The spillway is usually an ogee type with stilling basin. If the discharge to be taken care of is great and if, at the same time, prevailing conditions do not permit the construction of a long overflow weir, water surface regulation within narrow limits can be attained by constructing a siphon type spillway. The spillway and the bottom outlet canal should be combined immediately at the foot of the basin. Water spilling over the spillway crest and through the bottom outlet can be either diverted into a suitable river bed (if any) in a nearby side valley or conveyed by a special chute.

In designing a Forebay tank, it is important to keep the entrance to the penstock fully submerged. This is to prevent air being drawn in to the penstock because of a vortex which can be formed if the penstock entrance is closer to the water surface in the basin.

The usual components of the intake such as trash racks, flow control devices (gates or valves), etc. must be provided at the penstock inlet. It is necessary to install an air vent behind the gate to prevent damage to the penstock if for some reason the penstock entrance is blocked or the gate is suddenly closed causing a low pressure inside the conduit which can make it collapse inwards. The air vent can also help remove air from inside the penstock during startup.

## 4.3. Penstock

A penstock is the conduit that is used to carry water from the supply sources to the turbine. This conveyance is usually from a canal or reservoir. Penstocks classified based operation conditions:

- **Pressure penstock:** requires that the water discharging to the turbine always be under a positive pressure (greater than atmospheric pressure).

- **Siphon penstock:** is constructed in such a way that at points in the penstock the pressure may be less than atmospheric pressure and sections of the conduit act as a siphon. This requires that a vacuum pump or some other means for initiating the siphon action must be used to fill the conduit with water and to evacuate the air in the conduit.

**Penstocks may be classified according to type of construction:**

- **Steel penstocks:** the most common type of installation, due to simplicity in fabrication, strength, and assurance that they will perform in a wide variety of circumstances. Normal practice is to use welded steel pipe sections.
- **Cast-in-place or precast reinforced concrete pipe:** Very large diameters are somewhat impractical. Cast-in-place concrete pipes are usually limited to heads of less than 35 m. According to Creager and Justin (1950), these penstocks can be used up to 4m in diameter and under heads up to 185 m by using a welded steel shell embedded in the reinforced concrete.
- **Fiberglass and polyvinyl chloride (PVC) plastic pipe:** A penstock at the Niagara Mohawk plant uses a fiberglass pipe 3 m in diameter.
- **Wood stave pipes** have been used in diameters ranging from 15 cm up to 6 m and utilized at heads up to 185 m with proper design.

## Safe Penstock Thickness

The thickness of the pipe shell (s) for penstocks should be determined by: $s = PD/(2\eta\sigma)$; where s = penstock shell thickness; P = internal pressure; D = pipe diameter; $\eta$ = joint efficiency of welded or riveted joint; $\sigma$ = allowable unit stress of hoop tension. The allowable equivalent unit stress for hoop tension will vary with the type of steel used in the penstock.

Minimum thickness (in inches), based on need for stiffness, corrosion protection, and strength requirements, is indicated by the U.S. Department of the Interior (1967) to be:

$$s_{min} = (D + 20)/400.$$

## Size Selection of Penstocks

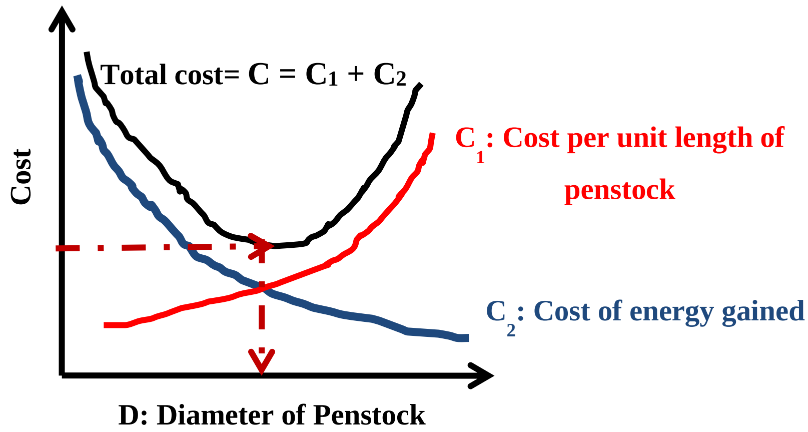
Various experience curves and empirical equations have been developed for determining the economical size of penstocks. Some of these equations use very few parameters to make initial size determinations for reconnaissance or feasibility studies. Other more sophisticated equations use many variables to obtain more precise results which may be necessary for final design. Economical size varies with type of installation and materials, as well as whether it is used above ground or buried.

- Gordon and Penman (1979) give a very simple equation for determining steel penstock diameter for small hydropower installations: $D_p = 0.72Q^{0.25}$
- Sarkaria (1979) developed an empirical approach for determining steel penstock diameter by using data from large hydro projects with heads varying from 57 m to 313 m and power

capacities ranging from 154 MW to 730 MW. He reported that the economical diameter of the penstock is given by the equation: $D = \frac{0.62 p^{0.35}}{h^{0.65}}$

- where D = economical penstock diameter, m; p = rated turbine capacity, hp; h = maximum net head at the end of the penstock, m.

### Optimization of Penstock Diameter




Differentiating the total cost with respect to D and setting the result equal to zero will give us the economic diameter.

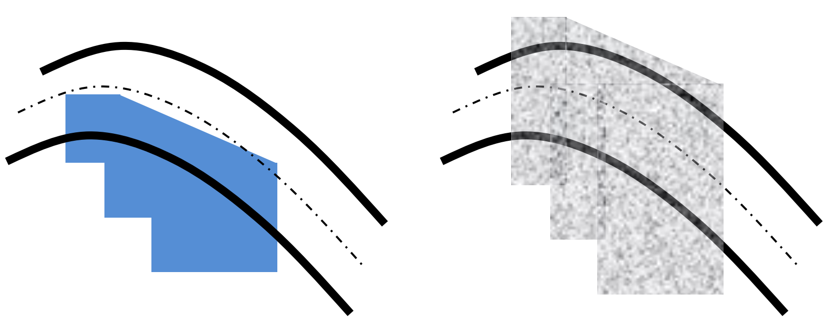
$$\frac{d}{dD}(C_1 + C_2) = \frac{dC_1}{dD} + \frac{dC_2}{dD} = 0 \;;\qquad \text{Note } \frac{dC_1}{dD} > 0; \frac{dC_2}{dD} < 0$$

### Anchor blocks

Anchor blocks are massive concrete blocks encasing the penstock pipe at intervals in order to anchor the pipe to the ground securely.

They are necessary at every horizontal and vertical bends of the penstock pipes. They are also provided at 150 m intervals on straight reaches.

The anchor blocks can either completely or partially incase the penstock



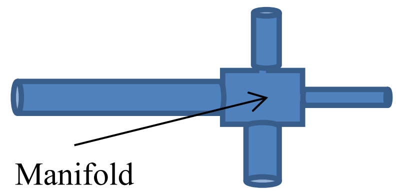
### Number of penstocks

A hydropower scheme having a battery of turbines has the following alternatives:

- To provide a single penstock for the complete power house. In such a case the penstock will have a manifold at its end with as many branches as the number of turbines.
- To provide as many penstocks as the number of turbines
- To provide multiple penstocks but each penstock supplying at least two turbines

The main considerations which lead to the best choice include:

- Economy
- Operational safeguard
- Transportation facilities



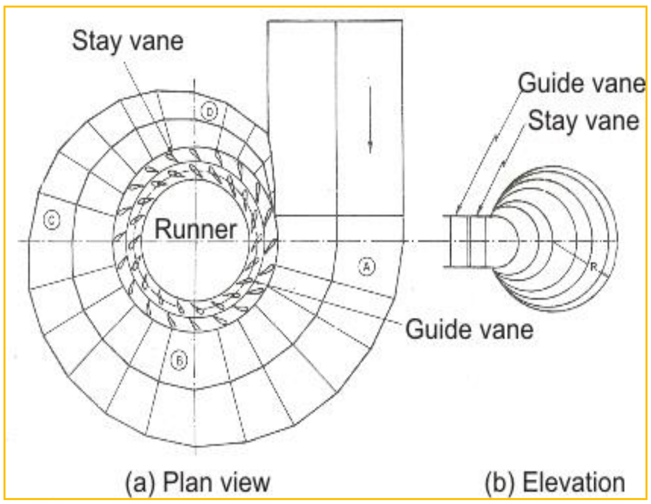

**Economy:** Cost /alternative penstock design/

**Operational safeguard**: in such case, a single penstock option is ruled out, as any damage to it will necessitate the total shut down of the entire turbine

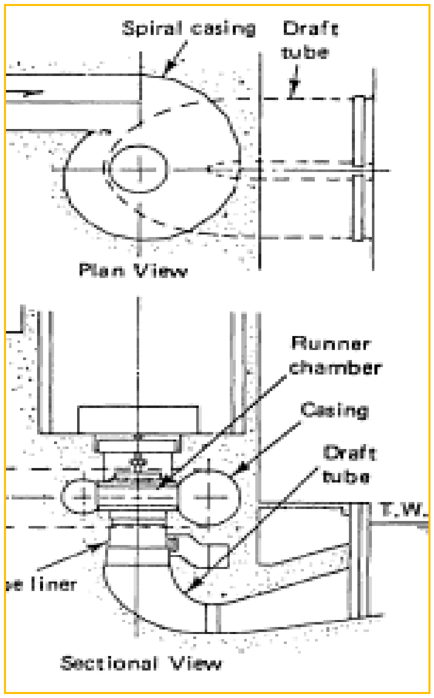
**Transportation facilities:** the penstock is shop welded in short section and then transported to the sites. The decision regarding the number of penstock automatically would influence the diameter of the penstock. It, therefore, has to be seen whether there are adequate facilities for the transportation of a given size of penstock.

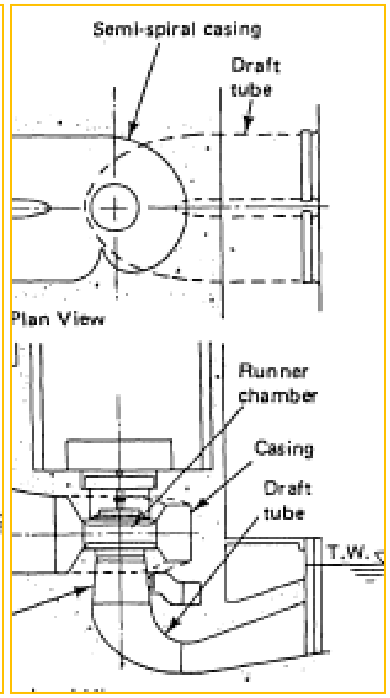
### 4.4. Spiral casing

Spiral case is that which conduits flow from the penstock to the runner in reaction type turbines. A spiral case of correct geometry will ensure evenly distribution of flow through the runner with minimized eddy formation. It is also known as scroll casing or volute. The cross-sectional area of this casing decreases uniformly along the circumference to keep the fluid velocity constant in magnitude along its path towards the guide vane.




Spiral casings can be semi or full spirals.

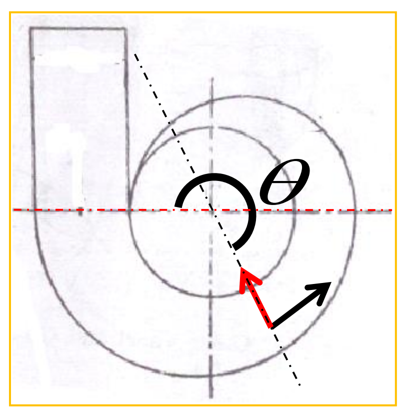





For large vertical-axis turbines, semi spiral or reinforced concrete spiral cases are used. The advantage of the semi spiral case is lower head loss. For low head schemes semi spiral scroll case is preferred. For medium to high head installations the full spiral scroll case used.

In design of the shape of the scroll cases, the following flow requirements need to be considered:

- A spiral case of constant height
- An evenly distributed flow in to the turbine
- No friction losses




Consider a discharge $Q_\Theta$ in a section of spiral case defined by an angle $\Theta$, thus: $Q_\Theta = Q\Theta/2\pi$; where Q total discharge from the penstock; $\Theta$ is guiding blade angle

The velocity at any point within the spiral case is divided:

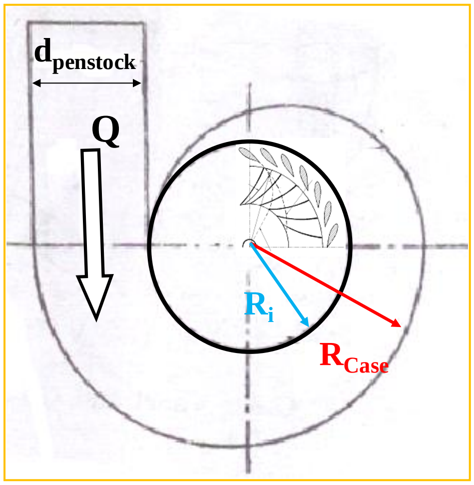
- Radial velocity, $\mathbf{v_r}$
- Tangential velocity, $\mathbf{v_t}$

$v_t = k/r$; Where k is the vortex strength; $k = 30\eta gh/(N\pi)$

Velocity at the entrance of the runner or exit of the penstock: $v_p = 0.2\sqrt{2gh}$

As the scroll case tries to retain this velocity throughout the scroll case and $Q_\Theta = Q\Theta/2\pi$, the x-sectional area of the scroll case at angle $\Theta$ will be computed as:

$$A_\theta = \frac{Q_\theta}{v} = \frac{Q\theta}{2\pi\left(0.2\sqrt{2gh}\right)} = 0.18\frac{Q\theta}{\sqrt{h}}$$




$$R_i = \frac{3.55\sqrt{Q}}{(Ns+100)^{1/3}h^{5/4}} \ for\ propeller$$

$$R_i = \frac{19\sqrt{h}}{N} \ for\ pelton$$

$$R_{Case} = R_i + \sqrt{\frac{4A_\theta}{\pi}}$$

Example 1: A runoff river plant uses a mean head of 15 m and generates approximately 60 MW. The load factor of the installation is 55%. Take the overall efficiency of 94%

- Determine the number type and specific speed of the turbine to be selected
- Design the scroll case for one of the turbines

**Solution:**

LF is average load to Peak load ratio

Peak load = 60/0.55 = 109.09 MW

Thus provide 6 units of 20 MW of which three works to satisfy the average demand the rest could be used to satisfy the peak demand and to replace one another during maintenance

For head less than 18 m $N_s = \frac{1475}{h^{1/3}} = 598.08 rpm$

Since 380 < 598 < 600, Propeller type turbine

Design of the scroll case:

$\eta$ = 94%; Q = P / (9.81 x h x $\eta$) = 20000/(9.81 x 15 x 0.94) = 144.6m$^3$/s

$Q_\Theta = Q\Theta/(2\pi) = 23.012\ \Theta$

Velocity at the entrance: $v = 0.2\sqrt{2gh} = 3.431$

$A_\Theta = Q_\Theta / v = 6.707\Theta$

For propeller turbine $R_i = \dfrac{3.55\sqrt{Q}}{(Ns+100)^{1/3} h^{5/4}} = 0.163m$

$$R_{Case} = R_i + \sqrt{\frac{4A_\theta}{\pi}} = 0.162 + 5.413\sqrt{\theta}$$

## 4.5. Draft tubes

The draft tube is a conduit discharging water from the runner to the tail race. Draft tubes are provided:

- To recover the velocity energy of the water leaving the runner as much as possible, 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.

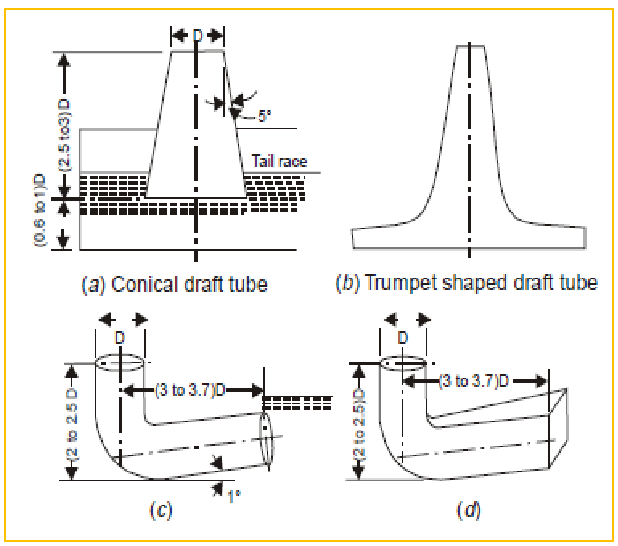
### Types of draft tubes

Based on cross-sectional shape

- See figure a-f

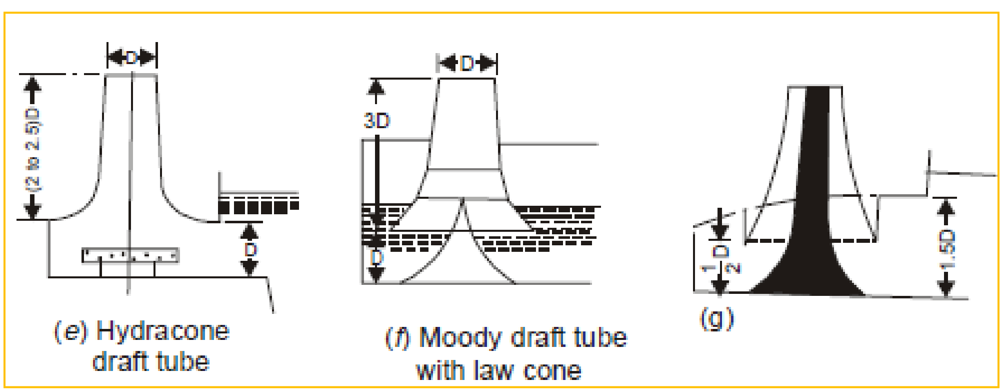
Based on alignment

- Elbow (Figure c and d)
- vertical (Figure a, b, e, f, g)




(*a*) Conical draft tube (*b*) Trumpet shaped draft tube

(*c*) (*d*)

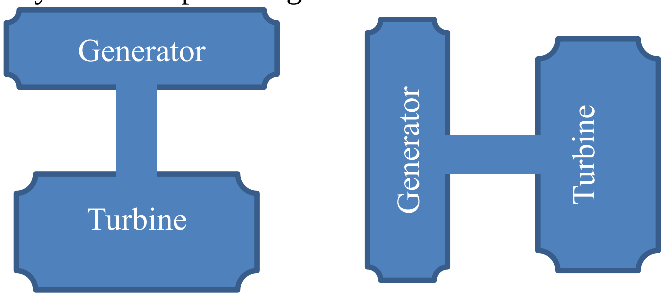



(*e*) Hydracone draft tube (*f*) Moody draft tube with law cone (g)

## Turbine Setting

Factors affecting the choice between horizontal and vertical setting of machines are

- Relative cost of plant,
- Foundations,
- Building space and layout of the plant in general.




Vertical

Horizontal

Vertical machines offer many advantages over horizontal especially when there are great variations in tail-race level.

Horizontal machines turbine-house should be above the tail-race level or the lower part of the house must be made watertight.

In vertical machines, the weight of rotating parts acts in the same direction as axial hydraulic thrust. This requires a thrust bearing capable of carrying considerable heavy load.

The efficiency of the vertical arrangement is 1 to 2% higher than for a similar horizontal arrangement. This is due to the absence of a suction bend near the runner.

In vertical turbine setting as the Generator being mounted above the turbine, it is completely free from flooding.

In horizontal machine setting:

- Two turbines could drive one generator and turbines would operate at a higher speed bringing about a smaller and lighter generator.
- Though the horizontal machines would occupy a greater length than the vertical, the foundations need not be so deep as required for vertical machines.
- The horizontal shaft machines require higher settings to reduce or eliminate the cost of sealing the generator, the auxiliary electrical equipment and cable ducts against water.

In actual cases, the overall cost will determine which arrangement to choose.

The majority of impulse turbines are of the horizontal shaft types, mainly because this setting lends itself readily to the use of multiple runner units.

The horizontal arrangement is simpler than vertical from constructional and maintenance point of view.

The overall height and width of the station will be relatively greater in case of vertical arrangement.

The floor space occupied by horizontal shaft units is in general greater than that required for vertical shaft machines.

Advantages of vertical arrangement:

- More compact and needs less floor area for the power house
- Design of hydraulic passages is simple
- In reaction turbines the wheels can be placed nearer to the tail water without disturbing the PH arrangement.

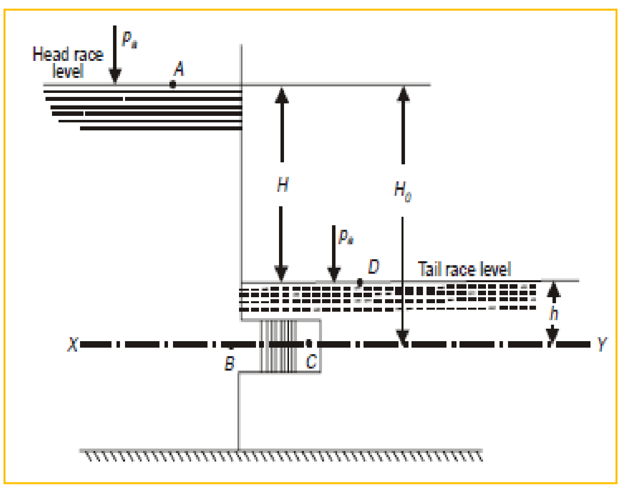
The horizontal arrangement is convenient:

- When two runners are mounted on the same shaft
- Where the vertical arrangement needs deep excavations
- In tidal power plants

## Draft tube and turbine setting

Output of reaction turbine when the tailrace level is above the turbine (submerged turbine). Considering the energies of unit weight of water at all points, we can write

$$E_a = E_b = H_o + \frac{p_a}{\rho g}$$



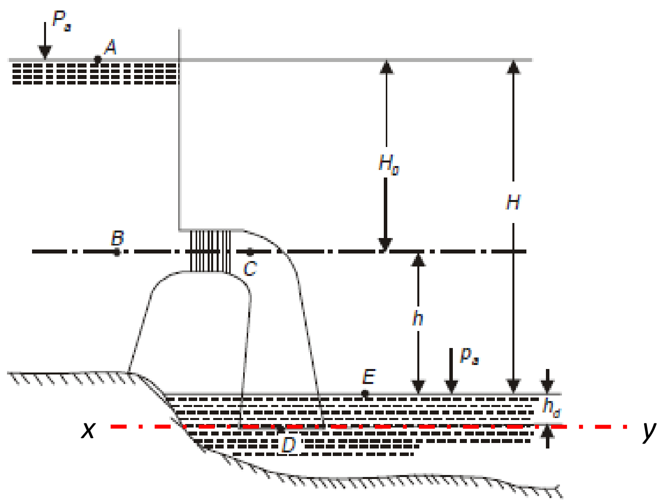

$$W_1 = E_b - E_c = \left(H_o + \frac{p_a}{\rho g}\right) - \left(\frac{p_c}{\rho g} + \frac{v_c^2}{2g}\right)$$
$$= \left(H_o + \frac{p_a}{\rho g}\right) - \left(\frac{p_a}{\rho g} + h + \frac{v_c^2}{2g}\right)$$

$$Note: \frac{p_c}{\rho g} = \frac{p_a}{\rho g} + h$$

$$\Rightarrow W_1 = H_o - h - \frac{v_c^2}{2g} = H - \frac{v_c^2}{2g} ....(a)$$

*$W_1$ (Work done per unit weight of water)*

Output of reaction turbine with draft tube: The arrangement of the turbine with draft tube is shown below and energies at all points are measured taking *x-y* as reference line.




$$E_a = E_b = H + h_d + \frac{p_a}{\rho g}$$

$$E_c = h + h_d + \frac{p_c}{\rho g} + \frac{v_c^2}{2g}$$

$$E_d = \frac{p_d}{\rho g} + \frac{v_d^2}{2g}$$

*$W_2$ (work done per unit weight of water passing through the turbine).* If $h_f$ is the head lost by water passing through the draft tube (friction and other losses).

$$W_2 = E_b - E_c = E_b - \left(E_d + h_f\right) = \left(H + h_d + \frac{p_a}{\rho g}\right) - \left(\frac{p_d}{\rho g} + \frac{v_d^2}{2g} + h_f\right)$$

$$= \left(H - \frac{v_d^2}{2g}\right) + \left(h_d + \frac{p_a}{\rho g} - \frac{p_d}{\rho g}\right) - h_f$$

The pressure at the point *D and E* must be same

$$\frac{p_d}{\rho g} = \frac{p_a}{\rho g} + h_d$$

Substituting this value in the above equation, we get

$$W_2 = H - \frac{v_d^2}{2g} - h_f \text{....}(b) \qquad\qquad W_2 = H - \frac{v_d^2}{2g};\ if\ \ h_f = 0$$

Comparing the equations *(a) and (b)* the extra work done per kg of water due to draft tube is given by

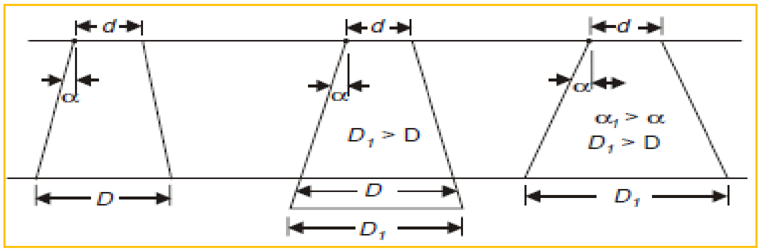
$$\Delta W = W_2 - W_1 = \left(H - \frac{v_d^2}{2g} - h_f\right) - \left(H - \frac{v_c^2}{2g}\right) = \frac{v_c^2 - v_d^2}{2g} - h_f$$

$$= \frac{v_c^2 - v_d^2}{2g},\ if\ \ h_f = 0$$

The head on the turbine *(H)* remains same as before, $\Delta W$ increases with the decrease in velocity $V_d$. The velocity $V_d$ can be decreased by increasing the outlet diameter of the draft tube.

The outlet diameter of the draft tube can be increased either by increasing the height or angle of the draft tube. The increase in height for increasing the diameter without increase in angle is limited by the pressure at the outlet of the runner (at point *C*). ***Why?***

An increase in draft tube angle (2α*)* for increasing the diameter without increase in height is limited by the losses in the draft tube.




The flow in the draft tube is from low pressure region to high pressure region. In such flow, there is a danger of water particles separating out from main stream and trying to flow back resulting in formation of eddies which are carried away in main stream causing losses. The maximum value of $\alpha$ is limited to $4^o$. The gain in work by increasing an angle α above $4^o$ will be lost in eddy losses and separated flow.

Sometimes in order to decrease the length of draft tube, the diverging angle has to be made more than 4° and under such cases to reduce the losses due to separation, the air is sucked from the inside surface of the draft tube.

By lining the draft tube and by proper designing the shape and size of the draft tube the work done by the draft tube is further increased by decreasing $h_f$.

The efficiency of the draft tube is given by

$$\eta = \frac{\Delta W}{v_c^2/2g} = \frac{v_c^2 - v_d^2}{v_c^2} = 1 - \left(\frac{v_d}{v_c}\right)^2$$

The advantages of using draft tube are:

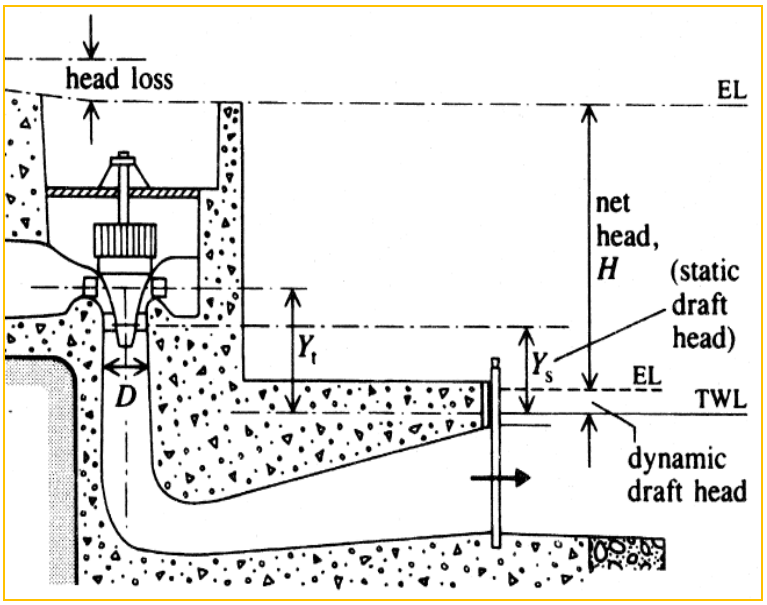
(1) It allows the turbine to be set above the tailrace water level where it is more accessible and yet does not cause any sacrifice in the head of turbine. It also prevents the flooding of generator and other equipment during flood period when the tailrace, water height goes up.

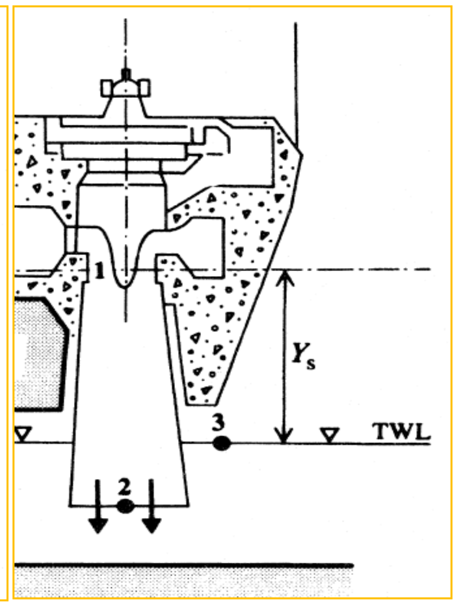
(2) It converts part of the velocity energy of the water leaving the turbine into the pressure energy and increases the overall efficiency of the plant.

## Cavitation and turbine setting

Cavitation results in pitting, vibration and reduction in efficiency and is certainly undesirable. Runners most seriously affected by cavitation are of the reaction type, in which the pressures at the discharge ends of the blades are negative and can approach the vapor pressure limits. Cavitation may be avoided by suitably designing, installing and operating the turbine in such a way that the pressures within the unit are above the vapor pressure of the water.

Turbine setting or draft head, $Y_s$ (*Figures below*), is the most critical factor in the installation of the reaction turbines.




The cavitation characteristic of a hydraulic machine is defined as the cavitation coefficient or plant sigma (σ), given by $\sigma = (H_a - H_v - Y_s)/H$

Where $H_a - H_v = H_b$, *is the barometric pressure head (at sea level and* 20°C, $H_b = 10.1$ *m), and H is the effective head on the runner.*

From the above equation the maximum permissible turbine setting *Ys, max* (elevation above tail-water to the centre line of the propeller runners, or to the bottom of the Francis runners) can be written as

$Y_{s,max} = H_b - \sigma_c H$ *(Thoma's formula)*

Where $\sigma_c$ is the minimum (critical) value of at which cavitation occurs (usually determined by experiments see table below). If *Ys is negative the runner must be* set below the tail-water.

| | **Francis Runners** | | | | | **Propeller Runners** | | | |
|---|---|---|---|---|---|---|---|---|---|
| $N_s$ | 75 | 150 | 225 | 300 | 375 | 375 | 600 | 750 | 900 |
| $\sigma_c$ | 0.025 | 0.1 | 0.23 | 0.40 | 0.64 | 0.43 | 0.8 | 1.5 | 3.5 |

The above recommended limiting values of plant sigma ($\sigma_c$) may also be approximated by

- $\sigma_c = 0.0432(Ns/100)^2$ *for Francis runners*
- $\sigma_c = 0.28 + 0.0024(Ns/100)^3$ *for propeller runners*
- With an increase of $\sigma_c$ by 10% for Kaplan turbines (Mosonyi, 1987).

The preliminary calculations of the elevation of the distributor above the tail water level ($Y_t$, *Fig. above) suggest the following empirical relationships* (Doland, 1957):

- $Y_t = Ys + 0.025DNs^{0.34}$ for Francis runners
- $Y_t = Y_s + 0.41D$ *for propeller runners*
- Where *D is the nominal diameter of the runner.*

# Chapter 5 Pressure Control and Speed Regulation

Contents

1. Water Hammer theory and Analysis
2. Pressure Control Systems
3. Speed Terminology
4. Speed Control and Governors

## 5.1. Water hammer theory and analysis

Sudden shutdowns of hydroelectric plants or changes in water flow through hydraulic turbines may cause problems ranging from rupture of penstocks due to water hammer to runner speed changes that cause the line current of the generators to vary from the desired frequency.

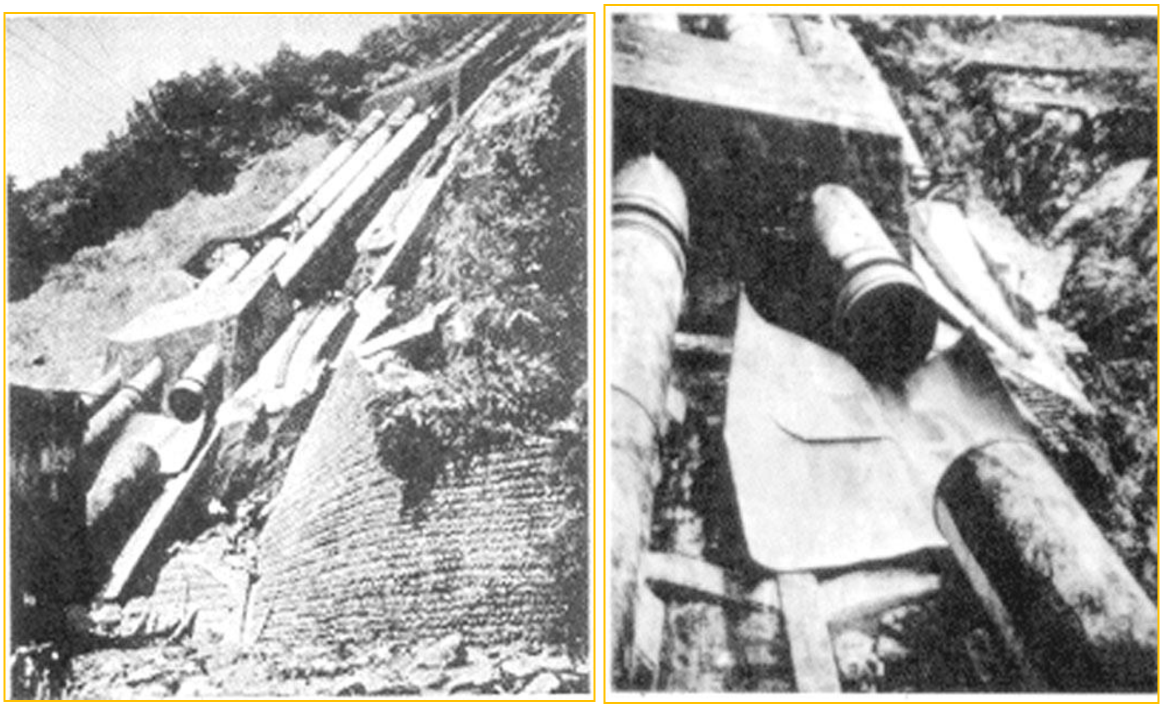
Regulating the water flow and coping with **sudden closure** of gates and valves require special equipment such as governors, pressure relief valves, and surge tanks. Solving the problems of pressure control and speed regulation requires a thorough understanding of the basic theory of water hammer.

Let us consider a simple case of fluid flowing with a certain velocity and brought to rest by closing a valve at the downstream end. If the fluid is entirely incompressible and the wall of the pipe is perfectly rigid, then all the fluid particles would have come to rest instantaneously. However, fluids are compressible to a certain extent, thus particles will not decelerate uniformly. Only those particles adjacent to the valve would stop instantaneously; other particles will come to rest later.

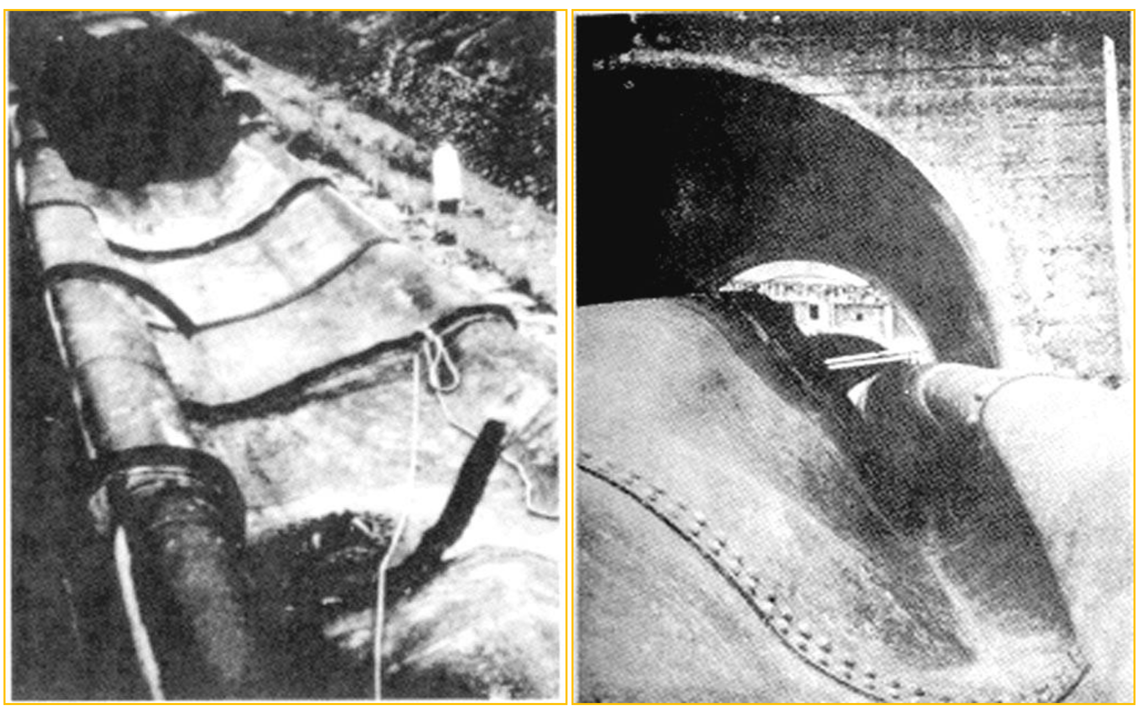
Recall Newton's second law: $F = m\,(dv/dt)$; the more rapid the deceleration the greater would be the force.

Water hammer is a phenomenon of pressure change in closed pipes caused when flowing water in a pipeline is decelerated or accelerated by closing or opening a valve or changing the velocity of the water rapidly in some other manner. The phenomenon is accompanied by a series of positive and negative pressure waves which travel back and forth in the pipe system until they are damped out by friction.

When a valve in a pipe or penstock carrying water is closed, the pressure head immediately upstream of the valve is increased, and a pulse of high pressure is propagated upstream to the nearest open water surface. On the downstream side of the valve a lowered pressure moves in a downstream direction to the nearest open water surface. Flow is changing from one steady-state to another over a very, very short time.



Effect of water hammer: Burst section of Penstock: Oigawa Power Station, Japan



Effect of water hammer: Collapsed section of Penstock: Oigawa Power Station, Japan

Causes of rapid changes in flow (acceleration or deceleration):

- Sudden opening or closing of control valves

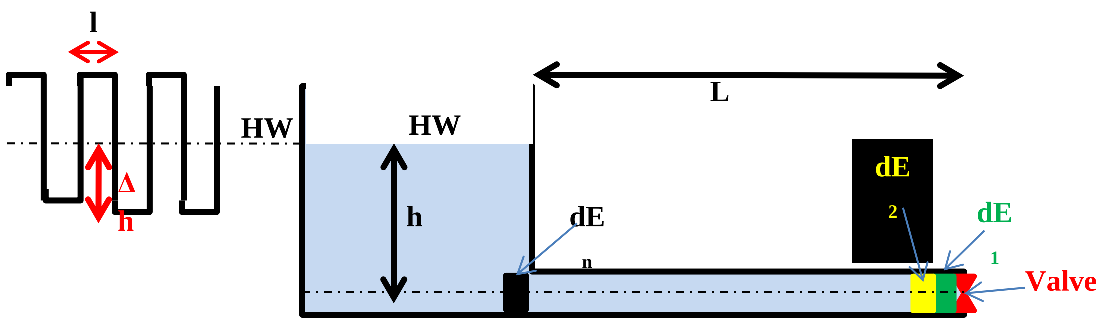
- Starting or stopping of pumps
- Rejecting or accepting load by hydraulic turbine

Rapid change in velocity results in change in momentum causing pressure fluctuations (waves) – Water Hammer. There are two theories in analyzing water hammer:

1. Elastic water column (EWC) theory
2. Rigid water column (RWC) theory

### Elastic Water column (EWC) theory

Propagation of pressure wave due to valve closure

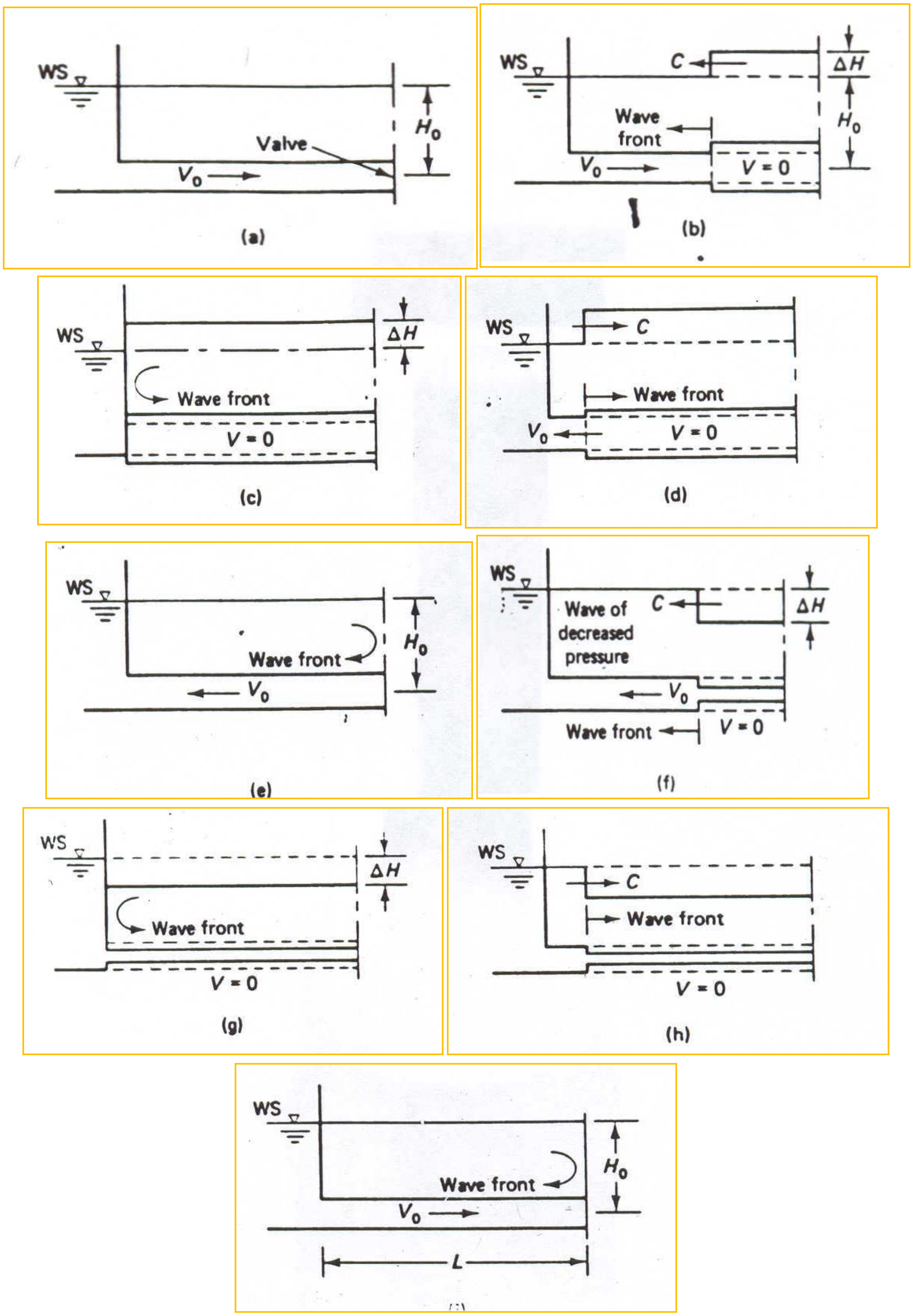



When a **valve** in a pipeline is suddenly closed, the element of water, **$dE_1$** nearest to the valve is compressed by the water flowing towards it and the pipe is stretched by the action. In the next time frame the element **$dE_2$** is stopped and it too is compressed. The water upstream of **$dE_2$** continues to move at the original velocity and successive elements of water are compressed. The action of compression moves upstream as a wave until it reaches the open water surface and the last element **$dE_n$** is compressed and the entire conduit of water has no movement thus under the increased pressure head, **$h + \Delta h$.**

The pressure pulse or wave moves at a velocity, ***c***, which is essentially the **velocity of sound in water**. Thus, the pressure wave reaches the open water surface in time, $t = L/c$. At that time the kinetic energy of the moving water has been converted to elastic energy in compressing the water and stretching the pipe.

At the open water surface the last element, **$dE_n$** expands to its original state, followed by other elements, causing a reverse or negative pressure wave. As this wave travels downstream, conditions change from zero velocity to a negative velocity and from the increased water pressure head, $h + \Delta h$, back to the normal pressure head, h. When the pressure wave reaches the valve, the pressure in the pipeline has returned to normal and a time $t = 2L/c$ has elapsed.

The water moving away from the valve now causes a reduction in pressure in the pipe and a negative pressure wave moves upstream to the open water surface.


WS
Valve
$H_0$
$V_0$
(a)
C
$\Delta H$
Wave front
$V = 0$
(b)
Wave front
(c)
(d)
Wave of decreased pressure
(e)
(f)
(g)
(h)
L

**a. Steady state prior to valve closure**

**b. Rapid valve closure – pressure increase, pipe walls expand, liquid compression; transient conditions propagate upstream**

**c. End of step 1 transient process @ L/c time**

**d. Pipe pressure > tank pressure water flows from pipe to tank relieving pressure in pipe**

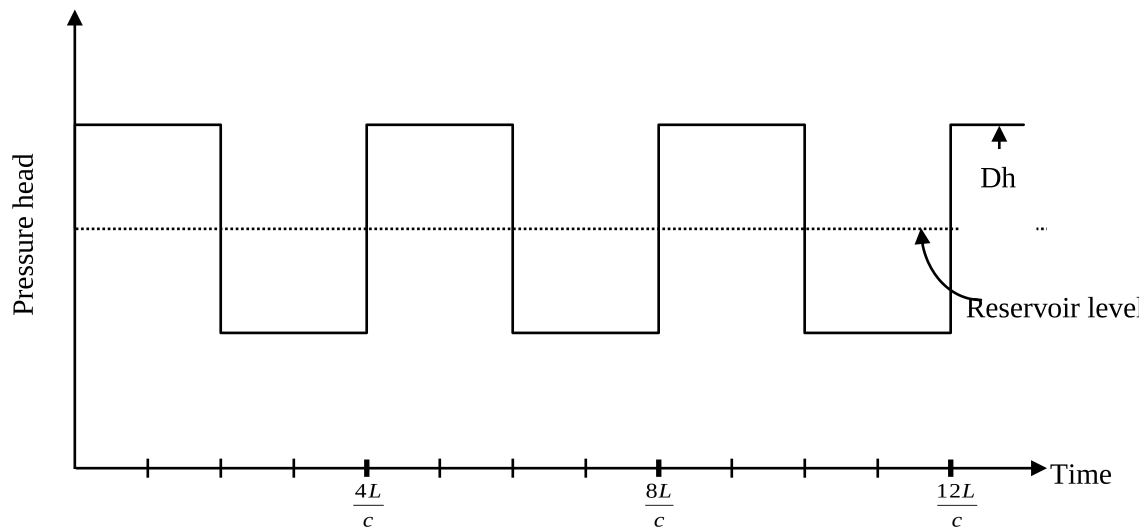
**e. Process starts at tank and continues to valve; time L/c, [total time 2L/c → water hammer period]**

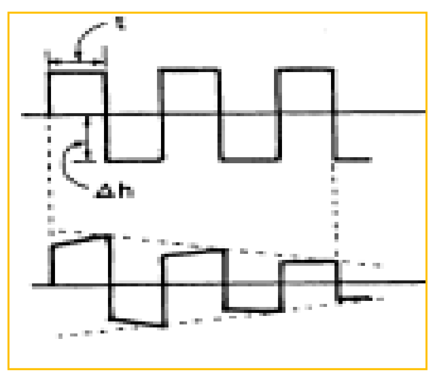
**f. Wave of backwater cannot go past the valve, starts wave of negative pressure toward tank**

**g & h. Pressure difference causes water to flow toward valve**

**i. One full cycle, 4L/c (the original steady state condition fig a)**

Pressure variation at the valve: velocity head and friction loss neglected would look like:



This periodic fluctuation at the reservoir water level look like figure A, if the water did not have friction acting. In reality, friction does act within the water and at the boundaries so that the pulses of pressure change have decreasing amplitude as shown in Figure B.

**Water hammer pressure is a function of … ?**

**Process Considerations**

- Pressure wave, resulting from compression of water, travels at speed of sound (sonic) in water
- Pressure change due to change in velocity
- Pressure increase moderated by expansion of the pipe wall due to increase in pressure

**Pipe Support Conditions**

1. Pipe supported at one end, pipe is allowed to move and pressure increase generates both axial and lateral stresses
2. Pipe anchored against axial movement throughout, no axial movement but axial stress still generated
3. No axial movement plus expansion joints at regular intervals, axial stress is taken up by play in joints

**Theory & terms**

$\Delta h$ = increase in head relative to original head *(assume frictionless flow)*

Assume $v_o$ is actual velocity in pipe based on friction, steady state prior to valve closure. Increase in pressure due to water hammer must be added to the steady-state pressure prevailing just before valve closure.

To find c (speed of pressure wave) we need to relate the volume of water entering the pipe to the increased pipe volume due to pipe expansion and the reduced volume of water because of compression. This approach needs the following parameters

- E = modulus of elasticity of pipe material
- K = bulk modulus of elasticity of fluid
- c = speed of pressure wave, celerity
- $\Delta h$ = increase in pressure head above initial conditions

According to Parmakian (1955), the velocity of the pressure wave, *c*, in a pipe is given by the following formula: $c = \left[\rho\left(\frac{1}{K} + \frac{C_1 d}{tE}\right)\right]^{-0.5}$

Where ρ = density of the water; K =volume modulus of water; d = diameter of pipe; t = thickness of pipe; E = pipe's Young's modulus of elasticity; $C_1$ = factor for anchorage and support of pipe

- $C_1$ = 0.95 for pipe anchored at upper end and without expansion joints
- $C_1$ = 0.91 for pipe anchored against longitudinal movement
- $C_1$ = 0.85 for pipe with expansion joints.

The maximum water hammer head is computed (Allievi) as: $h_{wm} = \frac{v_o c}{g}$

**Example:** How long does it take for a pressure wave to travel 500 m after a rapid valve closure in a 1 m diameter pipe without expansion joint, 1 cm wall thickness, steel pipeline? The initial flow velocity was 5 m/s. E for steel is 200 Gpa. K is 2.2 Gpa. What is the increase in pressure?

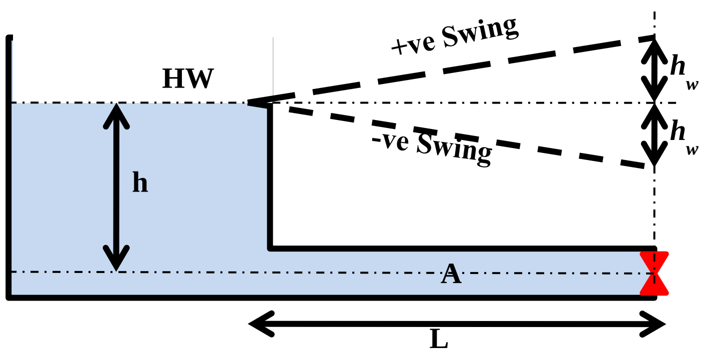
Solution:

$$c = \left[\rho\left(\frac{1}{K} + \frac{C_1 d}{tE}\right)\right]^{-0.5} = 1066.3 \text{ m/s}$$

The pressure wave takes 500/1066.3 = 0.47 seconds to travel the 500 m.

$$h_{wm} = \frac{v_o c}{g} = 543.5 \text{ m or increase in pressure} = 5331.5 \text{ KN/m}^2$$

## Rigid water column (RWC) theory




If the head losses in the pipe are neglected the velocity of flow is given by: $v_o = \sqrt{2gh}$. If the valve at the end is closed, the water in the pipe retards and hence there is a pressure increase. This pressure swings the normal hydraulic gradient to a position shown in the dotted lines. The pressure at the reservoir is atmospheric and hence constant. The +ve swing results from the pressure build up due to the retarded water flow.

As the water flows to the reservoir it creates partial vacuum conditions and the pressure in the pipe swings in the –ve direction. This induces the reservoir water to flow in to the pipe. But the valve being partially closed, much of this water again retarded giving rise to a +ve swing pressure again. Thus a valve closure brings about pressure oscillations. The maximum additional water hammer head $h_w$ can be worked out from Newton's second law:

$$F = p_w A = \frac{\gamma AL}{g}\left(-\frac{dv}{dt}\right);\; p_w = \gamma h_w \Rightarrow h_w = \frac{L}{g}\left(-\frac{dv}{dt}\right);\; v = \sqrt{2g(h+h_w)}$$

## 5.2. Pressure control systems

Hydraulic transients and pressure changes such as water hammer can be controlled in several ways. Gate controls and governor regulation can limit the gate or valve closure time so that there is no damaging pressure head rise. Pressure regulator valves located near the turbine can be used. The relief valve can be connected to the turbine spiral case and controlled by the turbine gate mechanism to prevent excessive pressure by maintaining a nearly constant water velocity in the penstock. The relief valve may be designed to close at a rate which limits pressure rise to an acceptable value.

**Design considerations**

- Specify a slow closure time
- Safety factor by doubling the minimum closure time
- Automatic valves programmed to prevent rapid closure
- Surge tanks

Valve Closure rate

Rapid Valve Closure means the time of closure $t_c < \frac{2L}{c}$ and full water hammer pressure is developed. $\Delta h = \frac{cv_o}{g}, or\ \Delta p = \rho c v_o$

Full pressure wave will not develop, if $\frac{2L}{c} < t_c < \frac{20L}{c}$ and the estimated water hammer head would be $\Delta h_{est} = \frac{cv_o}{g} \times \frac{2L/c}{t_c}$

Slow Closure after Lorenzo & Allievi will develop a pressure: $\Delta p = p_o\left(0.5N + \sqrt{0.25N^2 + N}\right) where, N = \left(\rho L v_o / p_o t_c\right)$

### Surge Tanks

Another way of pressure control can be accomplished with surge tanks. Surge tanks are vertical standpipes that act as a **Forebay** and shorten the distance for relief from the pressure wave of water hammer. A surge tank serves a threefold purpose:

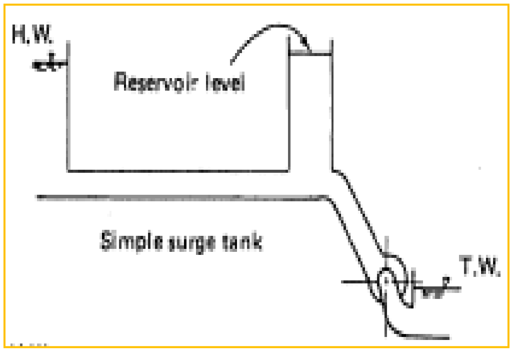
1. Flow stabilization to the turbine,
2. Water hammer relief or pressure regulation, and
3. Improvement of speed control.

In a practical sense, a rule of thumb that might be applied to determine **whether a surge tank or a relief valve may be needed** is that, extra caution should be exercised to evaluate pressure rise or decrease in systems where **the water conduit total length equals or exceeds the head by a factor of 3.**

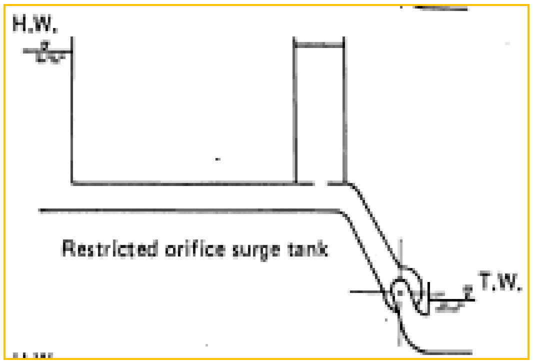
Surge tanks are usually not economical unless most of the drop in elevation in the penstock occurs near the turbine.

Three slightly different types of surge tanks are used.

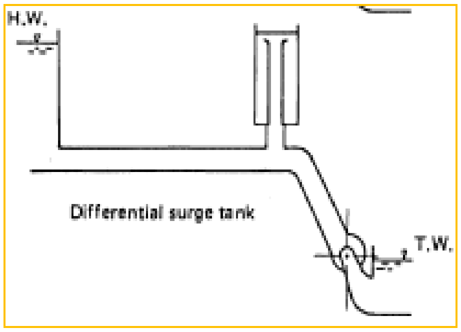
A ***simple surge tank*** is a vertical standpipe connected to the penstock with an opening large enough so that there is no appreciable loss in head as the water enters the surge tank. This is the most efficient surge tank to provide a ready water supply to the turbine when it is being accelerated, and especially when the initial loading is being applied. However, it is the most hydraulically unstable.

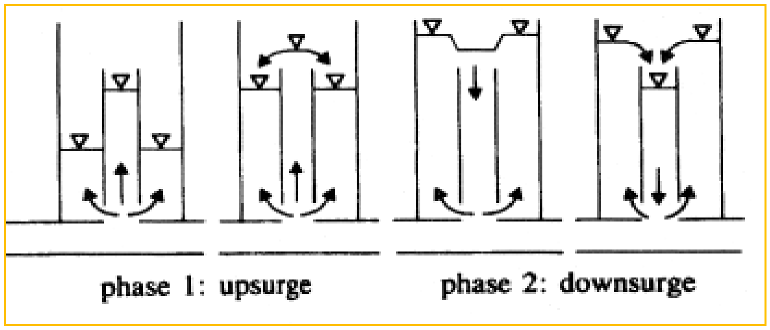



A ***restricted-orifice surge tank*** is connected in such a way that there is a restricted opening between the tank and the penstock that develops appreciable head loss in the water that flows into or out of the tank. Thus the orifice tank does not supply or accept excess penstock flow, but it is more hydraulically stable.

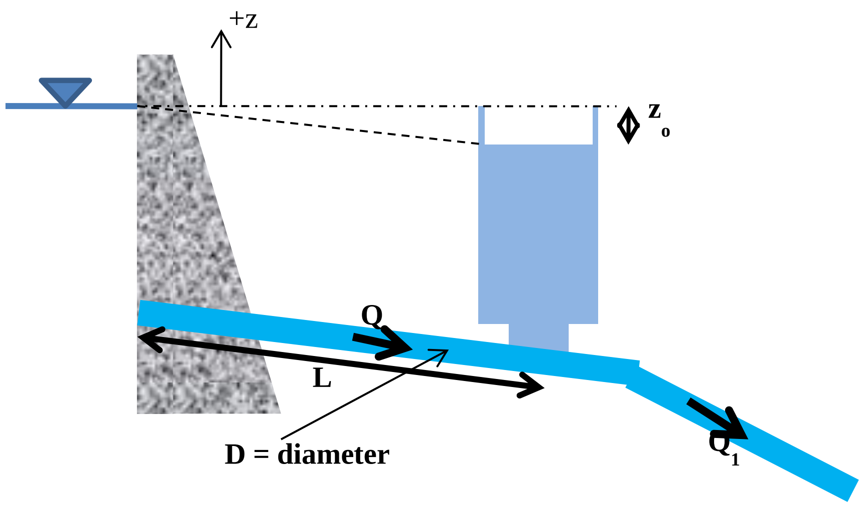



The ***differential surge tank*** is a combination of a simple tank and a restricted-orifice tank. An internal riser of smaller diameter than the full connection to the penstock is built to extend up through the tank while an outer tank is connected by a simple pipe connection to the penstock. The riser may also have a flow restrictor or orifice inside. Thus one part of the tank responds with a minimum of head loss while the outer tank offers resistance to rapid flow into the tank.




**Hydraulic design of simple surge tank**




Applying continuity equation: $Q = A_s \text{ x } (dz/dt) + Q_1;\ Q = v_t A_t$

$dz/dt = (v_t A_t - Q_1)/A_s$ … **(Eq. 1)**

Where $A_t$ and $A_s$ are the x-sectional area of the tunnel and surge tank

At steady state the water level in the surge tank will be at $-z = z_o$

Applying Energy Equation: $z_o = f \dfrac{L}{D}\dfrac{v_t^2}{2g}$

$$-F_t v_t^2 + z + \frac{L}{g}\frac{dv_t}{dt}\left(\frac{A_t}{A_s}\right)^2 = 0;\ Where\ F_t = f\frac{L}{2gD}\ \ ....Eq.2$$

If throated head loss in the throat is also considered:

$$-F_t v_t^2 - F_S v_s^2 + z + \frac{L}{g}\frac{dv}{dt}\left(\frac{A_t}{A_s}\right)^2 = 0$$

From Eq. 1, if $Q_1$ is zero, $dz/dt = v_t A_t /A_s \rightarrow d^2z/dt^2 = (A_t /A_s)dv_t/dt$ **(Eq. 3)**

For simplest case of full closure and negligible friction the solution to Eq. 2 has the following form**:** $z = z_{\max} \sin\left(\frac{t}{T}\right)$; $Where\ z_{\max} = v_t\sqrt{\frac{A_t L}{A_s g}}; T = \sqrt{\frac{A_t L}{A_s g}}$

Where $z_{max}$ is the maximum up surge or down surge.

Since the velocity is changing its sign after every half cycle, there is no general solution for the differential equation (Eq. 2). Usually a numerical solution is adopted. However; Jaeger has recommended use of the following approximate formula for the calculation of up surge in cases where friction is taken in to account:

$$\frac{z_{up}}{z_{\max}} = 1 - \frac{2}{3}k_o + \frac{k_o^2}{9};\ k_o = \frac{F_t v_t^2}{z_{\max}};\ z_{\max} = v_t\sqrt{\frac{A_t L}{A_s g}}$$

The above formula is applicable for conditions where $k_o$ is less than 0.7.

Calame and Gaden have given the following approximate formula suitable for computation of the lowest water reached after the first upward swing: $\frac{z_{down}}{z_{\max}} = -1 + 2k_o$

Where $z_{up}$ and $z_{down}$ are maximum and minimum water level in the surge chamber

**Example:** A surge chamber 100 m$^2$ in area is situated at the downstream end of a low pressure tunnel 10 km long and 5m in diameter. While, discharging 60 m$^3$/s the turbine inlet valves were stopped suddenly. Determine the maximum rise in level in the chamber and its time of occurrence. Use $f$=0.01

Solution: $A_s$ = 100 m$^2$; $A_t$ = 19.635 m$^2$; $v_t = Q/A_t$ = 3.056 m/s

At steady state the water level in the surge chamber: $z = -z_o = f\,\frac{L}{D}\frac{v_t^2}{2g} = -9.519m$; i.e. 9.519 m below the free water level of the reservoir

$$z_{\max} = v_t\sqrt{\frac{A_t L}{A_s g}} = 43.235m;\ k_o = \frac{F_t v_t^2}{z_{\max}} = 0.2202$$

$$\frac{z_{up}}{z_{\max}} = 1 - \frac{2}{3}k_o + \frac{k_o^2}{9} = 0.8586;\ z_{up} = 37.12m$$

At any time $t$ the velocity in the surge chamber $v_s = dz/dt;$ Applying continuity equation:

$$v_t A_t = A_s v_s + Q_1 = Q;\ But\ Q_1 = 0; \Rightarrow v_s = 0.6$$
$$v_s = \frac{dz}{dt} = 0.6 \Rightarrow z = 0.6t + c; \quad at\ t = 0\ z = -9.519m \Rightarrow c = -9.519m$$
$$z = 0.6t - 9.519; \quad thus\ \ for\ z = 37.12m \quad t = 77.732\sec$$

## 5.3. Speed terminology

A speed term that is important in the safety of hydropower plants is the **runaway speed**. It is the speed attained by turbine and generator (if directly connected)

- After a load rejection,
- If for some reason the shutdown mechanism fails to shut down the unit or
- If the rate of shutdown is not fast enough.

Runaway speed may reach 170 to 300% of normal operating speed. The magnitude of runaway speed is related to turbine design, operation, and setting of the turbine and will vary with the windage and friction that the runner and generator rotor offer as a revolving mass.

For some adjustable blade propeller turbines, the maximum runaway speed may be as high as 300% of normal operating speed.

Accurate numbers for defining runaway speed values must be based on model tests conducted by the turbine manufacturers.

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

The U.S. Department of the Interior (1976) gives empirical equations for estimating runaway speed based on several model tests of units installed by the Bureau of Reclamation. The equations are:

$$n_r = 0.85 n n_s^{0.2};\ n_s = 0.63 n N_s^{0.2}\ in\ metric\ units$$
$$n_{\max} = n_r \sqrt{h_{\max}/h_d}$$

Where

- $n_r$ = runaway speed at best efficiency head and full gate, rpm;
- n = normal rotational speed, rpm;
- $n_{max}$= runaway speed at maximum head;
- $n_s$ = specific speed based on full gate output at best efficiency head;
- $h_d$ = design head;
- $h_{max}$ = maximum head.

Another term is **over-speed**. It is the speed attained under transient conditions by a turbine after load rejection, while the governing and gate closure mechanisms are going into action. The rapidity with which the shutoff operates controls how much the over-speed will be? In very slow moving gate mechanisms over-speed can approach runaway speed.

## 5.4. Speed control and governors

Regulating the quantity of water admitted to the turbine runner is the usual means of regulating and maintaining a constant speed to drive the generator and to regulate the power output. This is done by operating wicket gates or valves. Such action requires a mechanism to control the wicket gates, which is the ***governor or governor system***.

The function of the governor is to detect any error in speed between actual and desired values and to effect a change in the turbine output. This is done so that the system load is in equilibrium with the generating unit output at the desired speed.

The function of the governor is to keep the speed of the prime mover constant irrespective of load variation. The governor is then controls the amount of water flowing through the turbine. Since the governor has to deal with water coming at very large force and in huge quantity, it should be very strong. To achieve this all types of turbines, use oil pressure governor.

**Speed droop**

If two or more generating units are operating to govern a system, it is impossible for them to maintain exactly the same speed. The governor with the higher speed will try to bring the system up to its speed, and take on load until it either achieves this or the turbine reaches full-gate position. If it is able to raise the system frequency, the other governor will sense too high speed and begin to drop load. The governors will therefore oppose each other. To avoid this situation, governors are built with a feature called **speed droop**.

Speed droop reduces the governor sensing speed as the gate opening or load increases. It is defined as the difference in speed (in percent) permitted when the units are operating between zero gate opening and 100% gate opening.

## Exercises

**Exercise-1:** Draw the flow and power duration curve given that $Q_c$ = 270 $m^3/s$, $\rho$= 1000 $kg/m^3$ and g= 9.81 $m/s^2$

| **Month** | **Flow ($m^3/s$)** | **Head(m)** | **Efficiency** |
|---|---|---|---|
| January | 65 | 83.5 | 0.87 |
| February | 50 | 83.5 | 0.83 |
| March | 42 | 83.5 | 0.75 |
| April | 40 | 83.5 | 0.70 |
| May | 40 | 83.5 | 0.60 |
| June | 115 | 83.5 | 0.50 |
| July | 400 | 80 | 0.88 |
| August | 340 | 81.6 | 0.89 |
| September | 270 | 83 | 0.90 |
| October | 155 | 83.5 | 0.90 |
| November | 115 | 83.5 | 0.88 |
| December | 85 | 83.5 | 0.87 |

**Exercise 2:** In an electrical district the following are some of the different kinds of demand:

- Domestic - 3 MW
- Irrigation - 5 MW
- Water supply – 2 MW
- Industries - ? Lp
- Offices - 2 MW
- Public utilities - 4 MW

Calculate the required installed capacity of the power station. Assume 15 % as loss, and 25 % of capacity as reserve. Assume diversity factor of 1.5.

**Exercise 3:** When a run-of-river plant operates as a peak load station with a weekly load factor of 20%, all its capacity is firm capacity. What will be the minimum flow in the river so that the station may serve as the base load station? It is given that

- Rated installed capacity of generator = 10 MW
- Operating head = 15 m
- Plant efficiency = 80%

Estimate the daily load factor of the plant if the stream flow is 15 $m^3/s$.

**Exercise 4:** A common load is shared by two stations, one being a base load plant with 25 MW installed capacity and other being a standby station with 30 MW capacity. The yearly output of the standby station is 10.5 x $10^6$ kWh and that of the base load plant is 125 x $10^6$ kWh. The peak load taken by the standby station is 15 MW and this station works for 2500 hours during the year. The base load station takes a peak of 22.5 MW. Find out:

- (i) Annual load factor for both stations;
- (ii) Capacity factor for both stations.

**Exercise 5:** The available flow for 97% of the time (i.e., in a year) in a river is 30 $m^3/s$. A run-of-river plant is proposed on this river to operate for 6 days in a week round the clock. The plant supplies power to a variable load whose variation is given below:

| *Period (hr)* | *0–6* | *6–12* | *12–18* | *18–24* |
|---|---|---|---|---|
| $\dfrac{\textit{Load during period}}{\textit{24-hr average load}}$ *ratio* | *0.6* | *1.4* | *1.5* | *0.5* |

*The other relevant data are given below:*

*Head at full pond level* = *16 m*

*Maximum allowable fluctuation of pond level* = *1 m*

*Plant efficiency* = *80%*

*Pondage to cover inflow fluctuations* = *20% of average daily flow*

*Pondage to cover wastage and spillage* = *10%*

*Determine:*

*(i) the average load that can be developed*

*(ii) daily load factor*

*(iii) plant capacity*

*(iv) weekly energy output*

***(v) Pondage required***

**Exercise 6:** The average weekly discharge as measured at a given site is as follows:

| Week | Flow (cumec) | Week | Flow (cumec) |
|---|---|---|---|
| 1 | 1000 | 14 | 1200 |
| 2 | 900 | 15 | 1000 |
| 3 | 909 | 16 | 900 |
| 4 | 800 | 17 | 800 |
| 5 | 800 | 18 | 500 |
| 6 | 600 | 19 | 400 |
| 7 | 500 | 20 | 400 |
| 8 | 500 | 21 | 300 |
| 9 | 800 | 22 | 300 |
| 10 | 800 | 23 | 400 |
| 11 | 1000 | 24 | 400 |
| 12 | 1100 | 25 | 500 |
| 13 | 1100 | 26 | 500 |

(a) Plot

(i) The hydrograph of weekly flow.

(ii) The flow-duration curve.

(iii) The power-duration curve if the head available is 50 m and the efficiency of the plant set is 85%.

(b) Determine the power that can be developed per $m^3$, the maximum power, average power, and total energy produced during 26 weeks.

**Exercise 6:** Average daily flows in a river in a typical low water week are given below. A run-of-river hydropower plant is proposed on this river to operate 6 days in a week round the clock. The full pond effective head on turbines is 20 m and the plant efficiency is 80%. Maximum allowable fluctuation of pond level is 1 m.

Determine:

- (i) the capacity of the pond required to give a maximum uniform output;
- (ii) the weekly energy output (kWh) from the plant.

| **Day** | **Su** | **Mo** | **Tu** | **We** | **Th** | **Fr** | **Sa** |
|---|---|---|---|---|---|---|---|
| **Flow ($m^3$/s)** | 26 | 35 | 40 | 50 | 45 | 40 | 30 |

**Exercise 7:** All the questions are to be done in a group of 7 to 10 students. Please hand in your solution by 19 Jan. 2013. Attempt each of the following problems and write down your solutions with all the necessary steps as neatly as possible. Please note that unreadable answers may not be corrected.

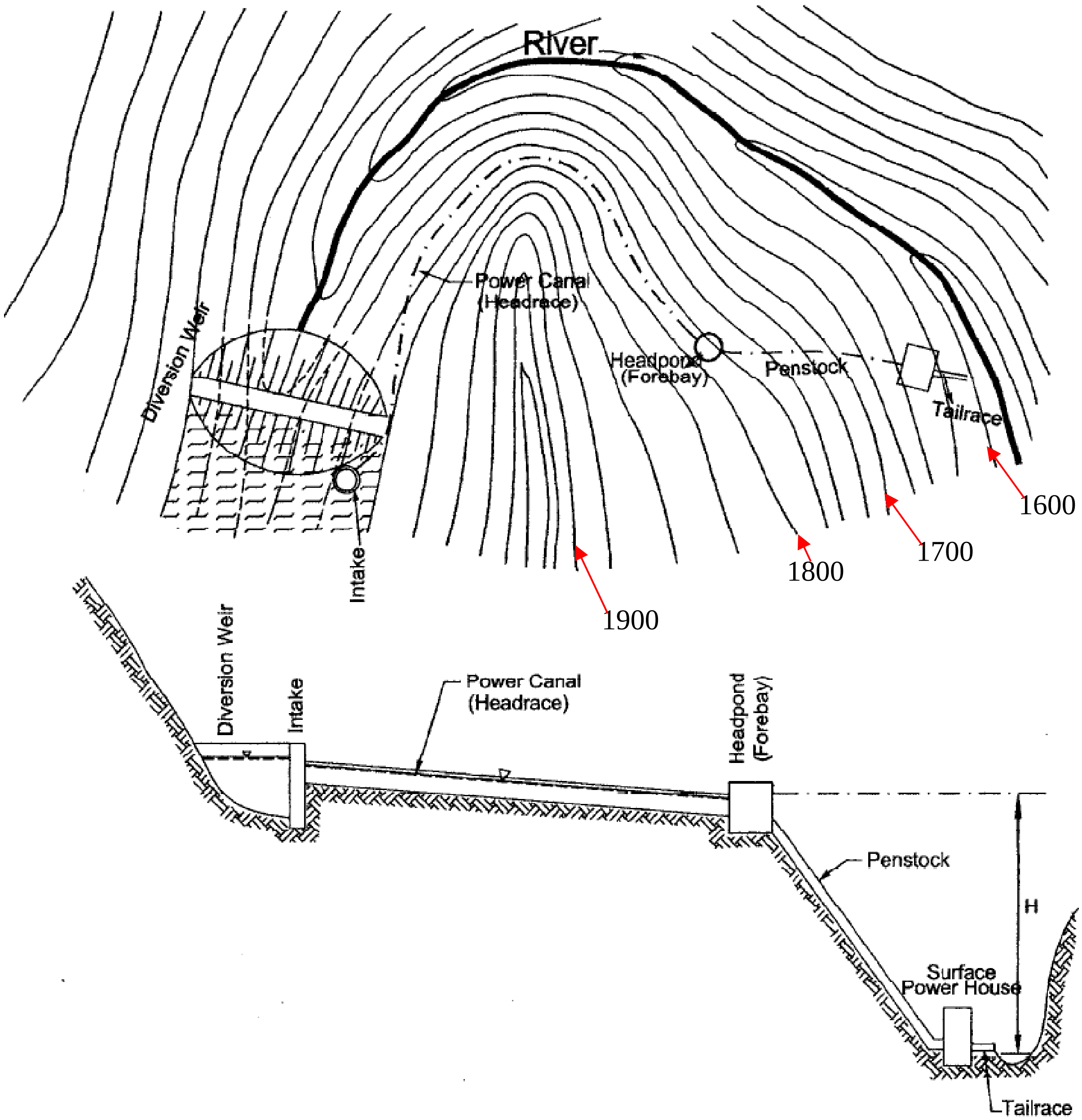



Figure Diversion canal plant

For the above figure the following information are given:

Elevation of the head water level at intake: 18**00 m above mean sea level**

Elevation of the tail water level: **1600 m**

Length of the power canal: **5000 m**

**The power canal traverses in high seepage geological setting.**

Steel Penstock Length = 200 m;

Coefficient of friction ***f** for* the penstock = 0.02.

The maximum discharge to be diverted from the head water for 14 hour power production is **30 $m^3$/s**.

Overall efficiency ($\eta_t$): **80%**

**Generator**: frequency **= 60 Hz**; Number of poles = 24.

Turbine type to be used: **reaction turbine.**

Unit weight of water = **9.81 KN/$m^3$**

Barometric head = **10.3 m**

Assume any other value if necessary.

1. Design the power canal
2. The Forebay storage capacity (if needed)
3. Decide on the number of Penstocks, diameter and thickness
4. Determine the maximum power output from the installation
5. Decide on the type of reaction turbine.
6. Decide on the number and capacity of reaction turbines for the installation.
7. Design a scroll casing for the turbine.
8. Determine the safe turbine setting relative to the tail water level
9. If a simple surge chamber 5 m in diameter is provided at **50 m from the Forebay** determine the maximum up surge and down surge in the surge chamber for sudden rejection by the turbine.
10. Comment on using the Forebay as a surge chamber or vice versa

## Sample Exam

**Answer all of the following questions. Note the mark each question possesses.**

1. State the types of hydropower projects when classified on the basis of operational feature (Regulation of water flow) **(8 %)**
2. What are depression hydropower plants? Mention (with justification) the possible site in Ethiopia. **(6%)**
3. Population growth rate being high is one of the problems related to Ethiopia's inability to harness its hydropower potential. Explain how this problem (population growth rate) is related to hydropower power production inability? **(6%)**
4. State factors that affect flow duration curve trend at a site. **(6%)**
5. Define the following load terminologies **(6%)**
   a. Plant use factor
   b. Utilization factor
   c. Diversity factor
6. Write at least three points that makes impulse and reaction turbines different. **(9%)**
7. Why it is usual practice to select high rotational speed turbines? What is/are the factor/s limiting the maximum rotational speed of a turbine to be used? **(6%)**
8. The following data are obtained from the records of the mean monthly flows of a river for 10 years. The head available at the site is 60 m and the plant efficiency and discharge capacity are 80% and 400 $m^3/s$, respectively. Draw the flow and power duration curve if the minimum average monthly flow observed was 50 $m^3/s$. **(20%)**

| Maximum monthly flow Q ($m^3/s$) | 100 | 150 | 200 | 250 | 300 | 350 | 400 | 450 | 500 | 550 |
|---|---|---|---|---|---|---|---|---|---|---|
| No. of occurrences below Q (in 10-yr period) | 220 | 330 | 440 | 550 | 660 | 770 | 990 | 1100 | 1110 | 1120 |

9. The daily power demand for a certain industry is approximated by $P = -2t^2 + 60t + 450$, where $P$ is the load in $KW$ needed at any time $t$ (hour); $t$ is in GMT (Greenwich Mean Time). If this industry is to be supplied by a hydropower plant having unlimited discharge with a head of **25m**, (Assume a generator having 60 Hz frequency and 12 poles.)
   a. Find the load factor **(8%)**
   b. Chose proper capacity and number of turbines **(3%)**
   c. Find the reserve capacity and plant capacity factor **(4%)**
   d. Estimate the specific speed and the type of turbine **(5%)**

10. A generating station is to supply four regions of load whose peak loads are 10 MW, 5MW, 8MW and 7MW. The diversity factor of the station is 1.5 and the average annual load factor is 60%. Calculate:
    a. The maximum demand on the station **(4%)**
    b. Annual energy supplied by the station **(4%)**
    c. Suggest the installed capacity and the number of units (assume 20% loss). **(5%)**
11. Define Forebay and why we need it? **(6 %)**
12. What is draft tube? Why we need a draft tube? **(6 %)**
13. Describe the structural and hydraulic functions of the three sections of a powerhouse **(9 %)**
14. What is water hammer pressure? Describe causes of water hammer pressure and ways to control water hammer. **(9 %)**
15. Describe/explain factors that are necessary in selecting a suitable Turbine for a site. **(6 %)**
16. Match List I with List II **(3 %)**

| List I | List II |
|---|---|
| A. Pelton wheel (single jet) | 1. Medium discharge, Low head |
| B. Francis Turbine | 2. High discharge, low head |
| C. Kaplan Turbine | 3. Medium discharge, medium head |
| | 4. Low discharge, high head |

17. At a particular hydroelectric power plant site, the discharge of water is 400 $m^3$/s and the head is 25 m. The turbine efficiency is 88%. The generator is directly coupled to the turbine having frequency of generation of 50 Hz and number of poles as 24. Calculate the least number of turbines required if :
    a) Francis turbine with specific speed of 300 is used **(4 %)**
    b) Kaplan Turbine with specific speed of 750 used **(4 %)**
    c) Comment on the result. **(3 %)**

**For questions no. 18 to 22 the following information/data are given use whichever is appropriate for your solution.**

Elevation of the head water level: 1400 m

Elevation of the tail water level: 1200m

**Tunnel:** Length = 1000 m; Diameter = 5 m; *f* = 0.03

**Steel Penstock:** Length = 200 m; Diameter = 2.5 m; thickness = 1 cm; *f* = 0.01

Anchorage factor for the penstock = 0.85

Young's modulus of elasticity for the steel as 200 Giga Pascal (GPa)

The bulk modulus of water as 2.2 GPa.

The maximum reservoir storage which can be utilized continuously for 24 hours: 2.5 million cubic meters.

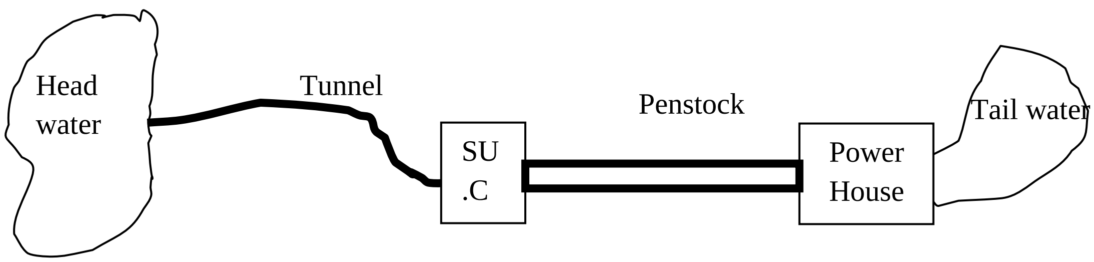
Overall efficiency of the installation ($\eta_t$): 80%

Unit weight of water = 9.81 $KN/m^3$

Borometric head = 10.3 m

Number of poles of the generator = 16

Frequency of the generator = 50 Hz




18. Determine the maximum power output from the installation (**14 %**)
19. Estimate the specific speed and the type of turbine (**9 pts**)
20. Determine the safe turbine setting relative to the tail water level (**6 %**)
11. If a simple surge chamber 5 m in diameter is provided at the end of the low-pressure tunnel estimate the maximum up surge in the surge chamber for sudden rejection by the turbine. (**12 %**)
12. Find the increase in pressure in the penstock for sudden rejection by the turbine **(9 %)**

**(Note: If you want to use rigid water column theory take the time of closure to be 0.55 second)**