ARBA MINCH UNIVERSITY

WATER TECHNOLOGY INSTITUTE

Hydraulic and Water Resources Engineering Faculty

HEng-3161 **Hydropower Engineering I** Lecture Note

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Contents

I. Course Objectives & Competences to be Acquired

Upon the completion of the course you will be able to:

- Acquire the basic concept of hydropower development
- Plan, design & analyze hydropower scheme components
- Construct and supervise hydropower schemes

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Chapter 1
Introduction

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1.1. Definition

What is Energy?

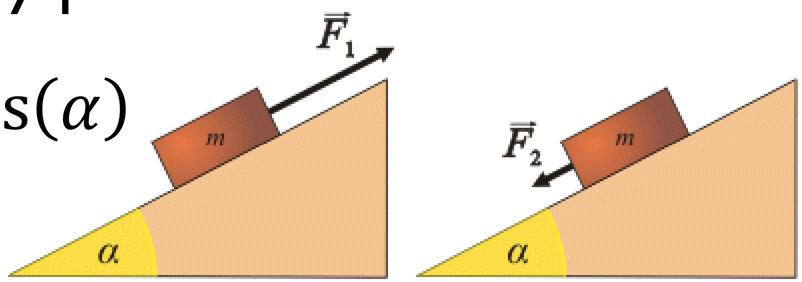
- Energy makes change
- It does things
- It moves cars along the road and boats over the water.
- It bakes a cake in the oven and keeps ice frozen in the freezer.
- It plays songs on the radio and lights our homes.
- Energy makes our body grow and allows our minds to think.

1.1. Definition

Energy is:

- the ability/capacity of the body to do work.
- the amount of work actually performed.

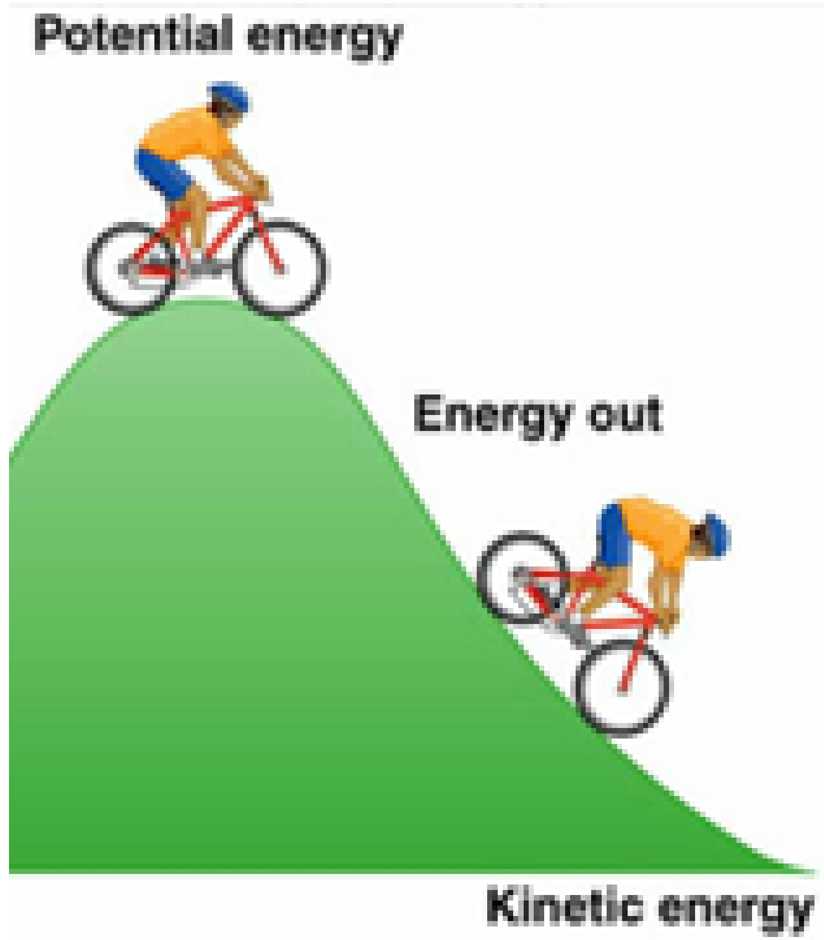
$$W = F \times S \times \cos(\alpha)$$



- the entity that neither created nor destroyed, but transforms from one form to the other

1.1. Definition

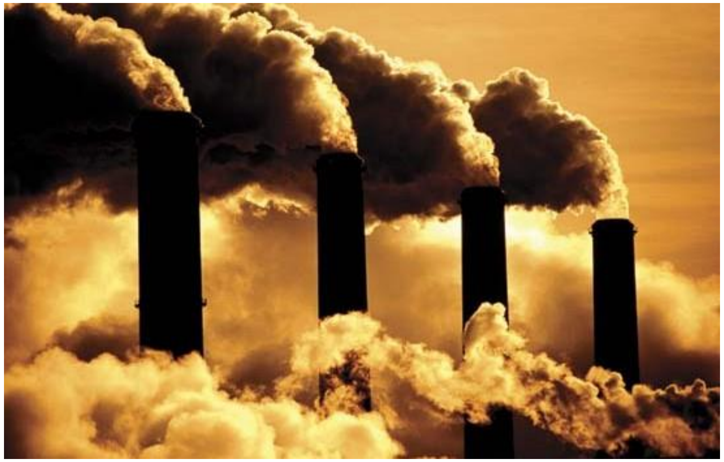
- Energy can be found in many things & takes many forms:
  - Potential Energy
  - Kinetic Energy
  - Heat Energy
  - Light Energy
  - Sound Energy
- How to generate?
- How to store?
- How to use?



1.2. Sources of Energy

- Basically the source of any energy is the sun
- Energy sources are classified into two:

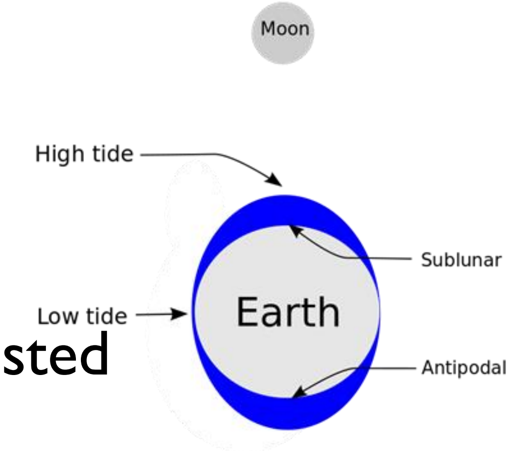
A. Conventional Energy Sources



  - Generally they are non-renewable
  - Replenish slowly or not at all
  - Toxic, exhausted, …….
  - Example:
    - Fossil fuels (petroleum, natural gas, coal, peat)
    - Nuclear fuels (uranium and plutonium)

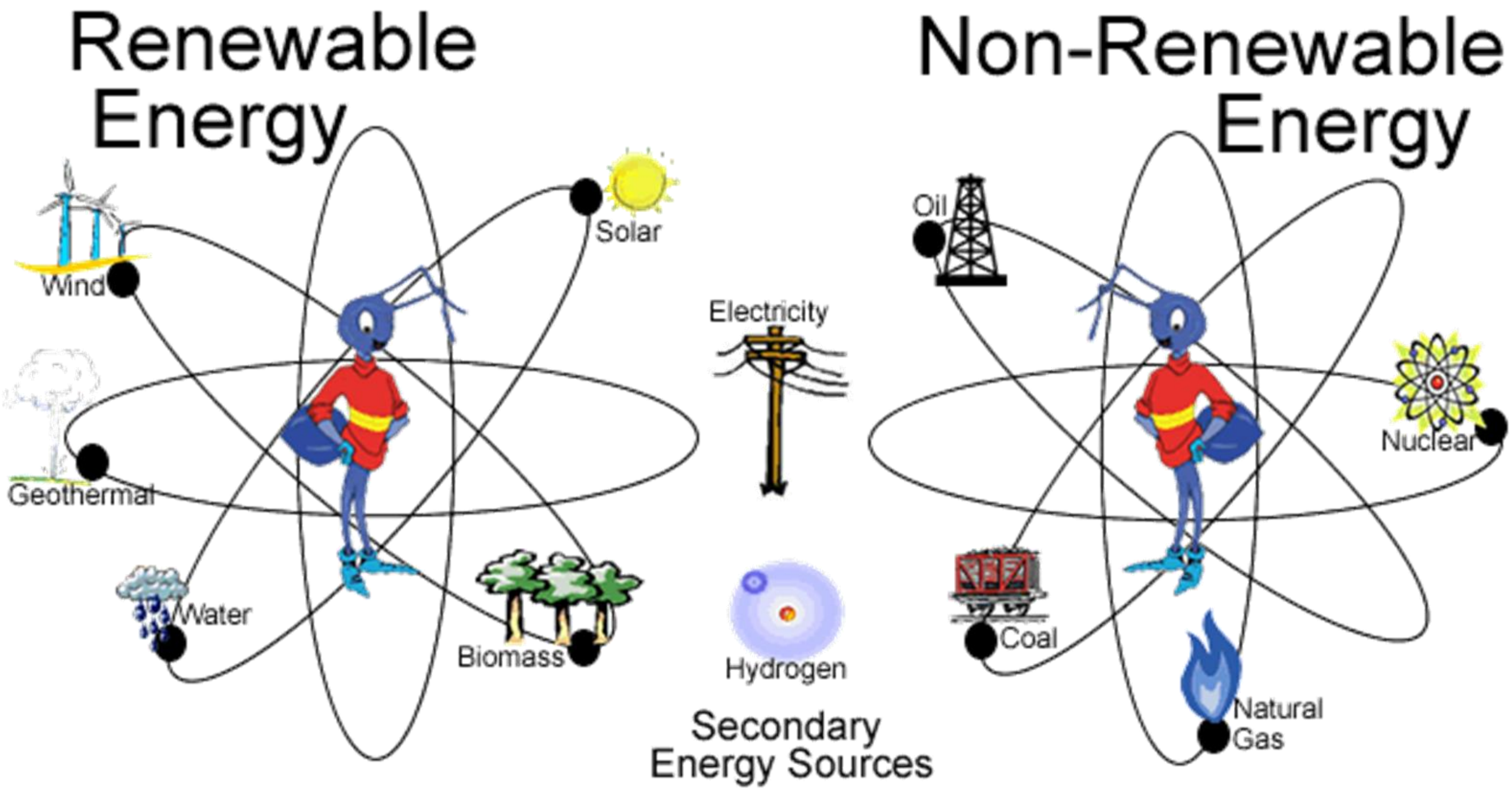
1.2. Sources of Energy

B. Unconventional Energy Sources



- Renewable
- Replenish rapidly
- Clean, Non-toxic, Non-exhausted
- Example:
  - Tidal energy
  - Solar energy
  - Wind energy
  - Geothermal
  - Hydropower

1.2. Sources of Energy



1.3. Pros & Cons of Energy Sources

Res.	Advantages	Disadvantages
1. petroleum	✓ Abundant, accessible & deposits are widespread in sedimentary areas ✓ Highly versatile ✓ High-grade fuel is obtained by refining and processing petroleum ✓ Petroleum and its by-products are used for: Transportation, Heating, Lighting, Cooling, Lubricating, Medical products, Animal protein, Fertilizer, etc.	✓ Non renewable ✓ Requires considerable capital investment ✓ Causes atmospheric pollution through combustion ✓ Cost of production from tar sand and oil shale is higher than from conventional sources ✓ Offshore exploration and drilling is more expensive than on land ✓ Extracting oil from deep-sea areas beyond the continental shelf involves technological and legal problems ✓ There are growing difficulties in maintaining equilibrium between supply and demand

1.3. Pros & Cons of Energy Sources

Res.	Advantages	Disadvantages
2. Natural Gas	✓ *Relatively cheap and abundant* ✓ *Relatively clean* ✓ *Virtually sulpher-free (except for sour gas)* ✓ *Versatile: used as a raw material for petro-chemicals*	✓ *Nonrenewable except when produced from organic waste or algae* ✓ *Expensive to transport when liquefied* ✓ *Risky to handle because of vapor clouds and danger of fire* ✓ *Causes some atmospheric pollution when used in power plants*

1.3. Pros & Cons of Energy Sources

Res	Advantages	Disadvantages
3. Coal	✓ Very abundant ✓ Deposits are widespread in sedimentary areas ✓ High-grade coal contains 70 – 80% of the energy per unit weight of oil ✓ Some kinds of coal are low in sulpher	✓ Nonrenewable ✓ Deep mining can be dangerous, expensive and hazardous to health ✓ Surface stripping damages the land and creates problems of soil erosion and unproductive land unless remedial work is undertaken, which may be expensive ✓ Causes atmospheric pollution through combustion releasing carbon dioxide, sulpher dioxide and fly ash

1.3. Pros & Cons of Energy Sources

Res.	Advantages	Disadvantages
4. Nuclear Fission	✓ *Uranium fairly widespread in nature* ✓ *Powerful energy source: uranium releases 20000 times as much heat as the equivalent weight of coal.* ✓ *Could provide unlimited energy resources through nuclear breeder reactors*	✓ *Installation of nuclear plants needs much capital and suitable sites are difficult to find* ✓ *All existing types of reactors consume more fissionable material than they produce* ✓ *Pollution: thermal waste (heated water dumped into river or sea) threatens aquatic life. radioactive waste is hazardous to health; plutonium used in breeder reactors is a dangerous poison* ✓ *Storage facilities of radioactive substances have short life to the life of radioactive materials* ✓ *Materials can be stolen or otherwise diverted for use in nuclear weapons*

1.3. Pros & Cons of Energy Sources

Res	Advantages	Disadvantages
5. Geothermal	✓ *Abundant* ✓ *Can generate electricity and provide heat economically in relatively small power units* ✓ *Where district heating or greenhouse heating is required, geothermal heat can be produced at very low cost*	✓ *Found principally in areas of tectonic activity* ✓ *Environmental pollution possible* ✓ *Release of sulphur components* ✓ *Highly mineralized hot waters* ✓ *containing toxic materials may have to be re injected into the field* ✓ *Thermal pollution may be created when used to generate electricity* ✓ *Technology for obtaining energy from geo-pressure zones and hot rocks not yet developed*

1.3. Pros & Cons of Energy Sources

Res	Advantages	Disadvantages
6. Wood (Bio-mas)	✓ *Renewable* ✓ *Provides heat for domestic purposes* ✓ *Less polluting than other fuels. Although the burning of a tree releases CO_2, an equal amount of CO_2 is removed from the atmosphere when the tree grows. Thus, so long as the trees that are burned are replaced by growing new trees, the net emission is zero.*	✓ *Provides less heat per unit of weight than other fuels, such as coal and oil* ✓ *Inefficient conversion causes smoke pollution* ✓ *Other industrial uses, such as construction and paper production, may yield a higher return than its use for energy* ✓ *Forests are far from industrial centers*

1.3. Pros & Cons of Energy Sources

Res	Advantages	Disadvantages
7. Waste products	✓ *Renewable* ✓ *Easily obtained* ✓ *Solves problems of waste disposal and related environmental pollution*	✓ *Organic municipal waste produces low-grade fuel* ✓ *Large-scale collection could be costly* ✓ *Technical problem to handle non-organic solids*
8. Tidal	✓ *Renewable* ✓ *Nonpolluting*	✓ *Possible only in areas where difference in tide levels is high enough to generate electricity* ✓ *Output is intermittent and depends on tide cycles* ✓ *Installations are complicated and costly*

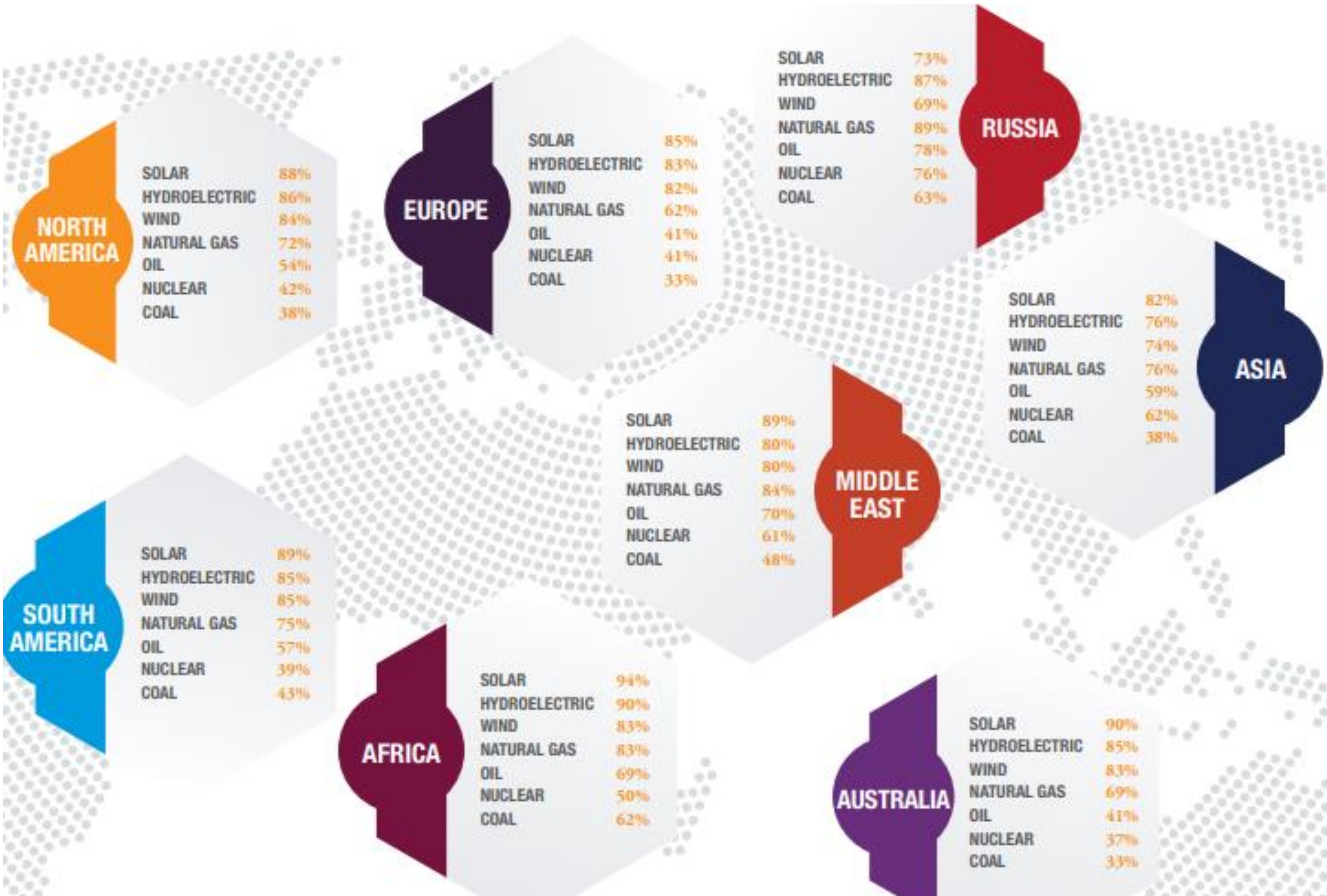
Pros & Cons of Energy Sources

Res	Advantages	Disadvantages
9. Solar	✓ *Easily obtained & Inexhaustible* ✓ *Direct use of heat for water and space heating, cooking, drying crops, desalination of water, evaporation to produce salt* ✓ *Nonpolluting and safe* ✓ *Considerable potential for space cooling and water pumping* ✓ *Possible use for extraction of hydrogen from water*	✓ *Limited hours of sunlight and variation of solar intensity* ✓ *Solar collectors can only provide low-grade heat on a small scale* ✓ *Technical difficulties in using on a large scale* ✓ *Power system entail high initial cost*

1.3. Pros & Cons of Energy Sources

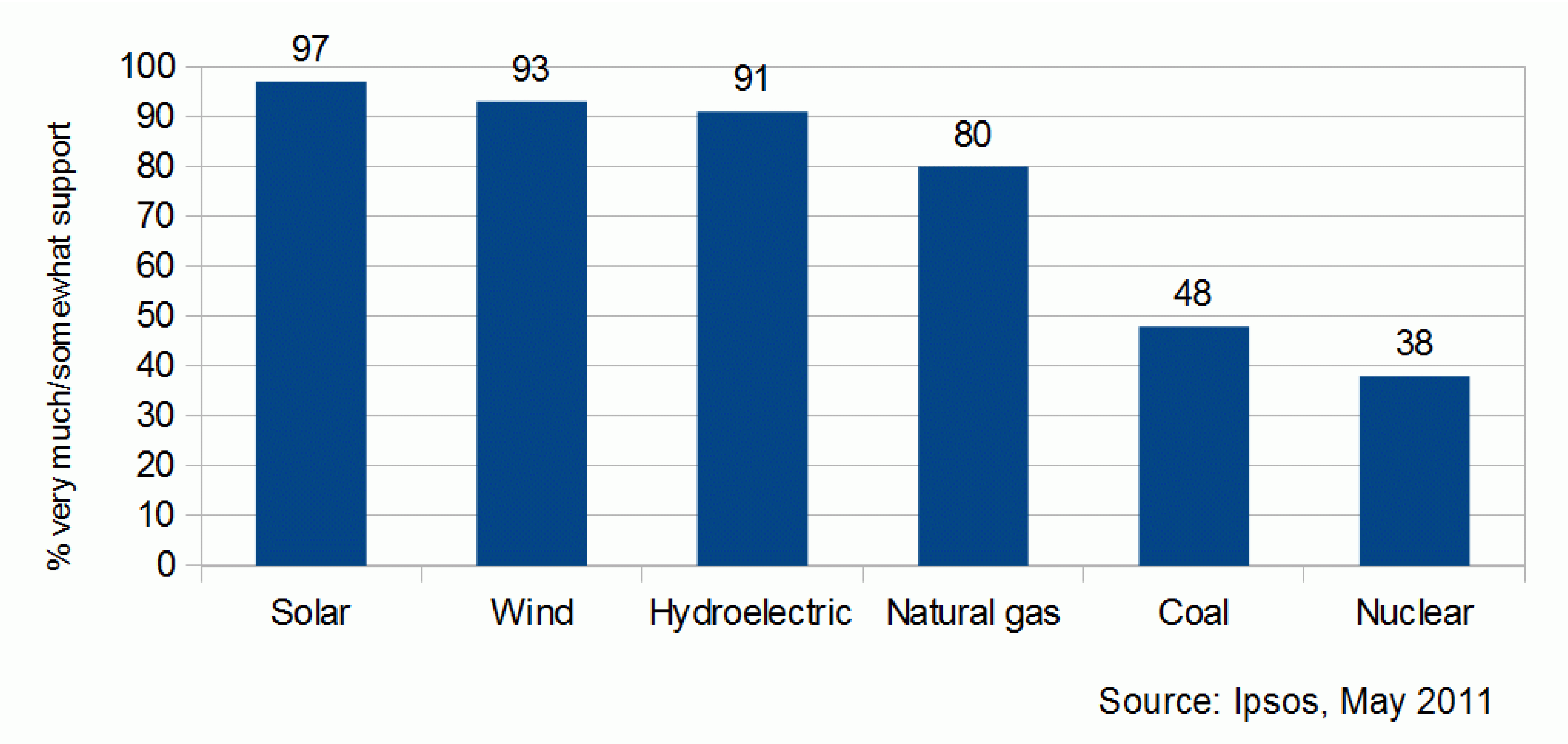
Res	Advantages	Disadvantages
10. Wind	✓ *Traditionally used in many rural areas. e.g. for pumping water and turning millstones* ✓ *Nonpolluting* ✓ *Inexhaustible* ✓ *Small wind generators can supply electric energy in isolated regions*	✓ *Variation in energy output according to duration and force of wind* ✓ *Storage of electricity when wind velocity changes is expensive* ✓ *For large scale production suitable sites with adequate wind power are hard to find*

Global Public Support for Energy Sources



Source: IHA 2019 status report

Global Public Support for Energy Sources



1.4. Energy – Physical Bases

Physical Term	Unit
Force = Mass ∗ Acceleration	$Kg*m/s^2$ = N (Newton)
Work = force ∗ Distance Work = power ∗ time	N∗m = J (joule) W∗s
Energy = available potential to work	J (joule)
Power = Work/time Power = Force ∗ velocity	joule/s = W (watt) $(Kg*m/s^2)*m/s$ = W(watt)

1.4. Energy – Physical Bases

	Unit	Application	Conversion
Energy	Joule	Metric SI-unit	1 J = 1 watt-second = 1 Newton meter (NM)
	Kilowatt-hour	Very common; disadvantage: mixing up the time units second and hour	1 kWh = 3.6×10^6 Ws = 3.6×10^6 J
	Calorie	Obsolete	1 cal = 4.1868 J
	Coal equivalent	Obsolete	1 kg SKE = 29.3×10^6 J
	British thermal unit	Non- metrical; used in the Anglo- American area. Various Btu are in use which differ only slightly	1 Btu = 1 055 J = 2.93×10^{-4} kWh
Power	Watt	Metric SI-unit	$1\,W = 1\frac{J}{s} = 1\,N\frac{m}{s}$
	Horse power (metric)	Obsolete	1 PS = 736 W
	Horse power (English)		1 HP = 746 W

Chapter 2
Hydropower Development

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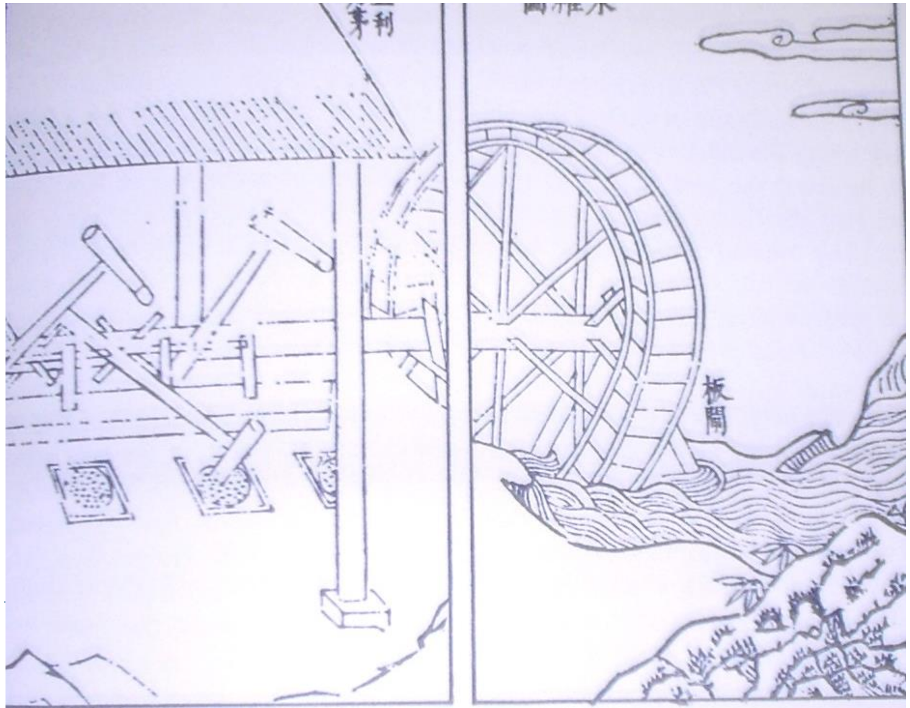
Contents of chapter two

- Hydropower Status in the World
- Hydropower potential & Status in Ethiopia
- Merits and Demerits of Hydropower

History of Hydropower

- Some of the earliest innovations in using water power were conceived in China during the Han Dynasty between 202 BC and 9 AD.
- Trip hammers powered by the vertical-set water wheel were used to pound and hull grain, break ore, and in early paper-making.



History of Hydropower

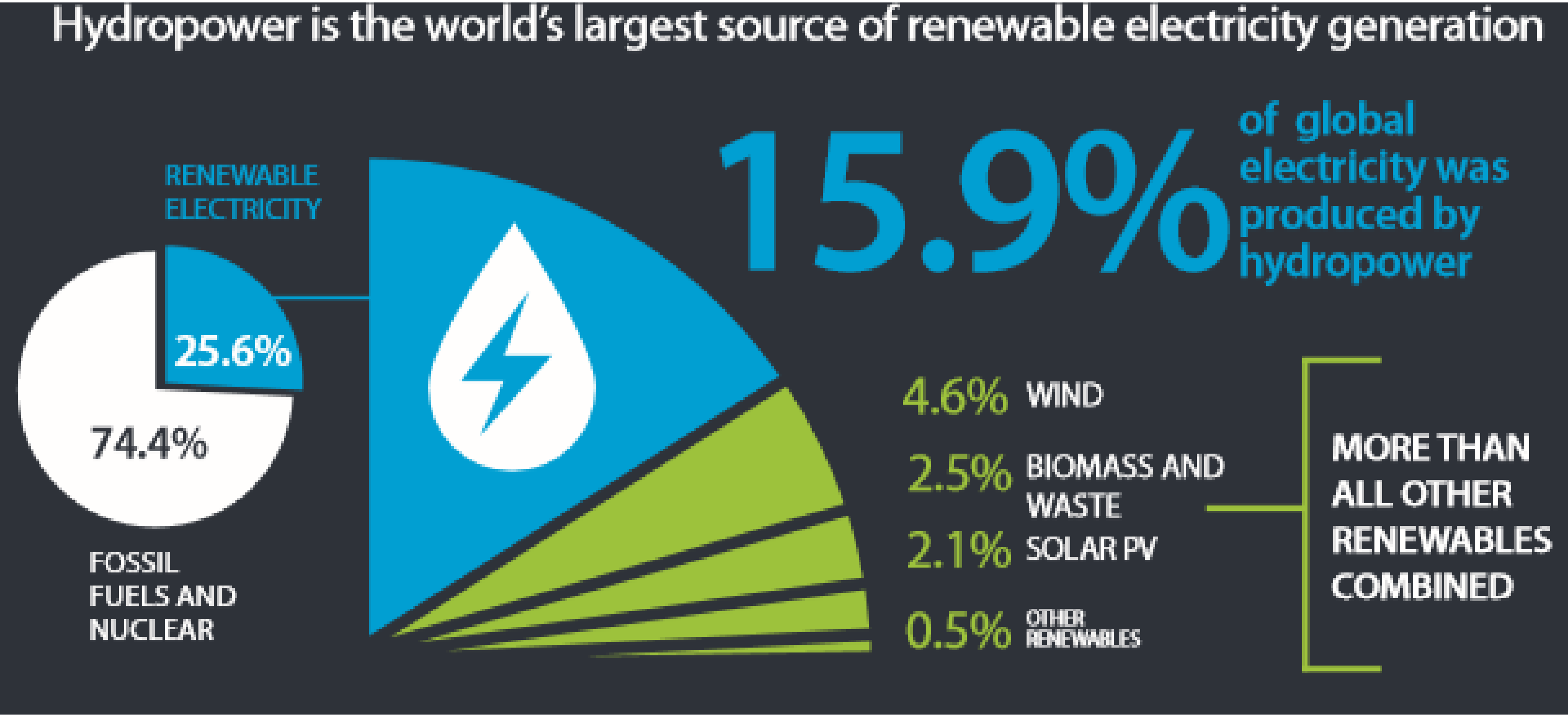
- Richard Arkwright was used hydropower to set up Cromford Mill in England's Derwent valley in 1771 to spin cotton, one of the world's first factory systems.
- The world's first hydroelectric project was used to power a single lamp in England, in 1878.
- In 1882 the first plant to serve a system of private and commercial customers was opened in Wisconsin, USA.
- Within a decade, hundreds of hydropower plants were in operation. They were used to supply mills and light some local buildings.

Hydropower Status in the World

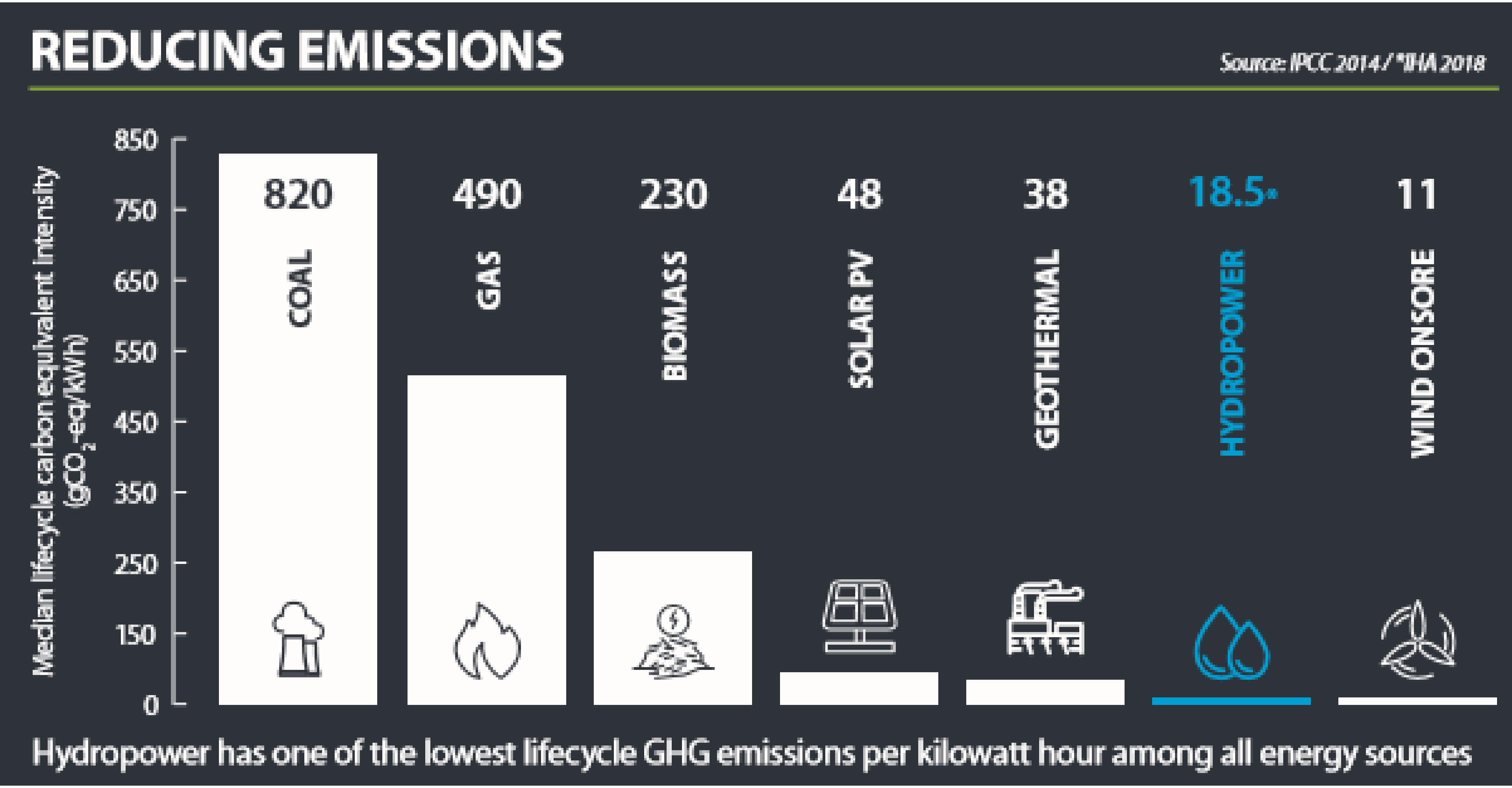
According to hydropower status report (IHA, 2019):

- Hydropower development today is most active in fast growing economies and emerging markets
- 16% of global electricity generation comes from hydropower
- At the end of 2018 the world hydropower installed capacity is 1,292 GW and total generation of 4,200 TWh/year

Hydropower Status in the World



Hydropower Status in the World



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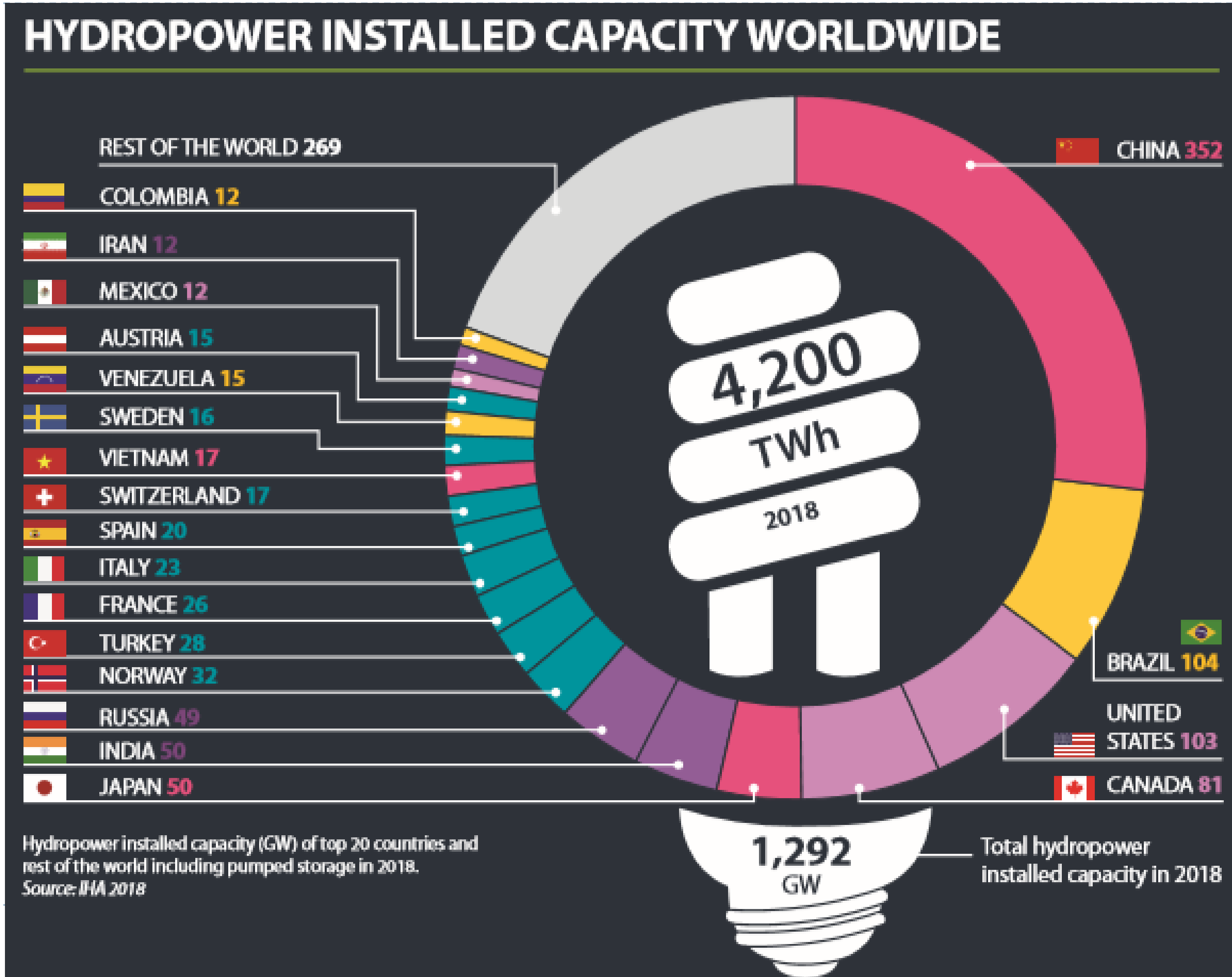
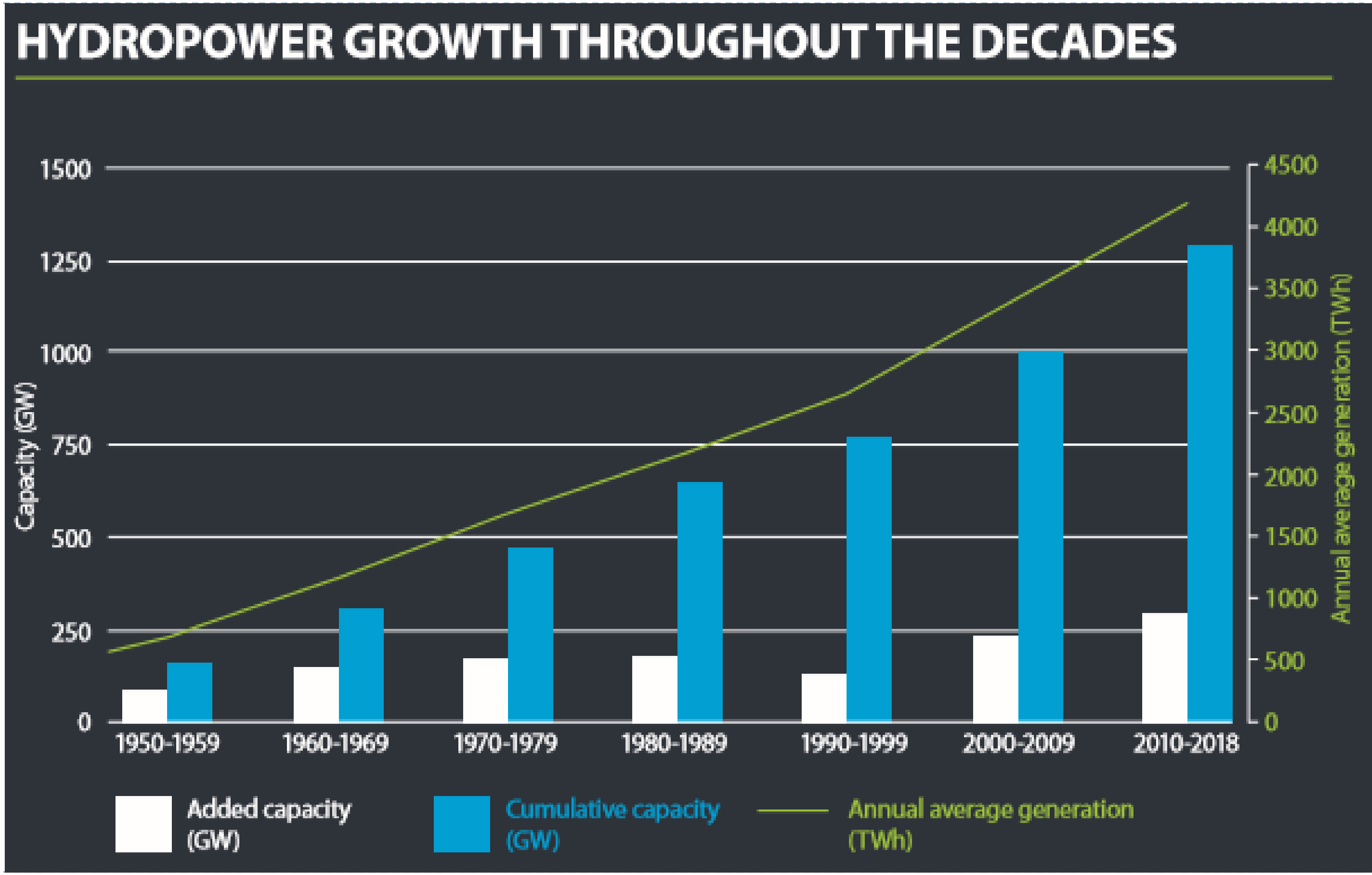


Hydropower Status in the World



Hydropower Status in the World

- This study by IHA analyses 9,266 hydropower stations worldwide, of which, according to ICOLD, 40% are multi-purpose and 60% single purpose. Figure 1 shows the evolution of single and multipurpose hydropower projects from 1900 to 2018.

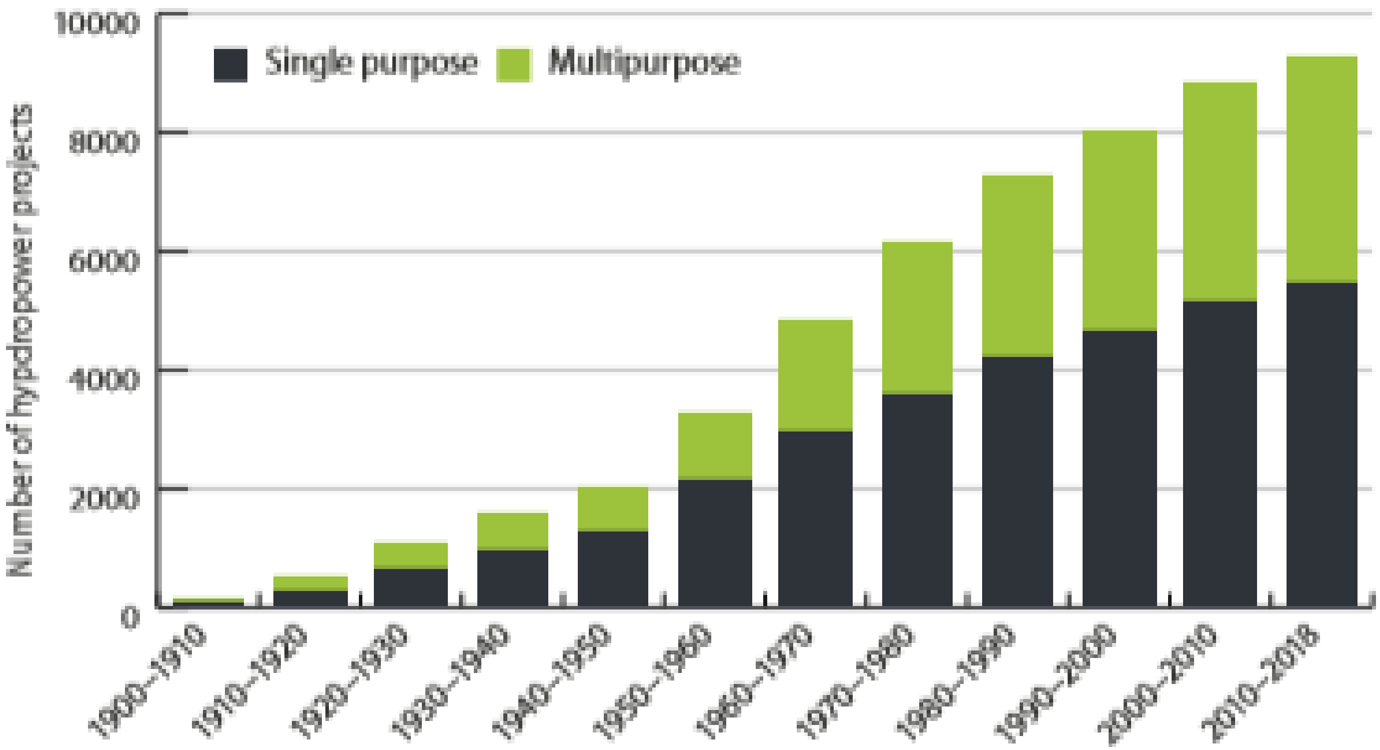
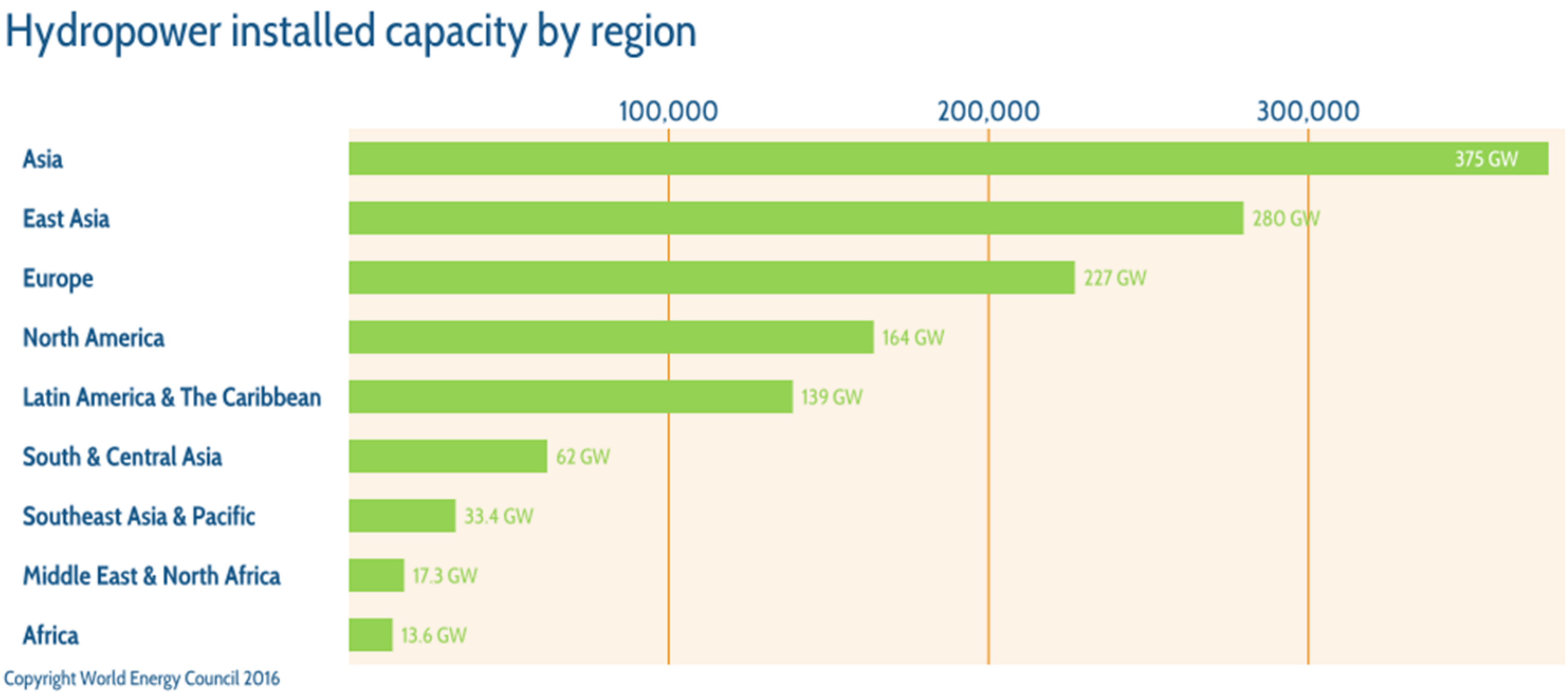


Figure Cumulative number of hydropower projects (data from IHA / ICOLD)

Hydropower Status in the World



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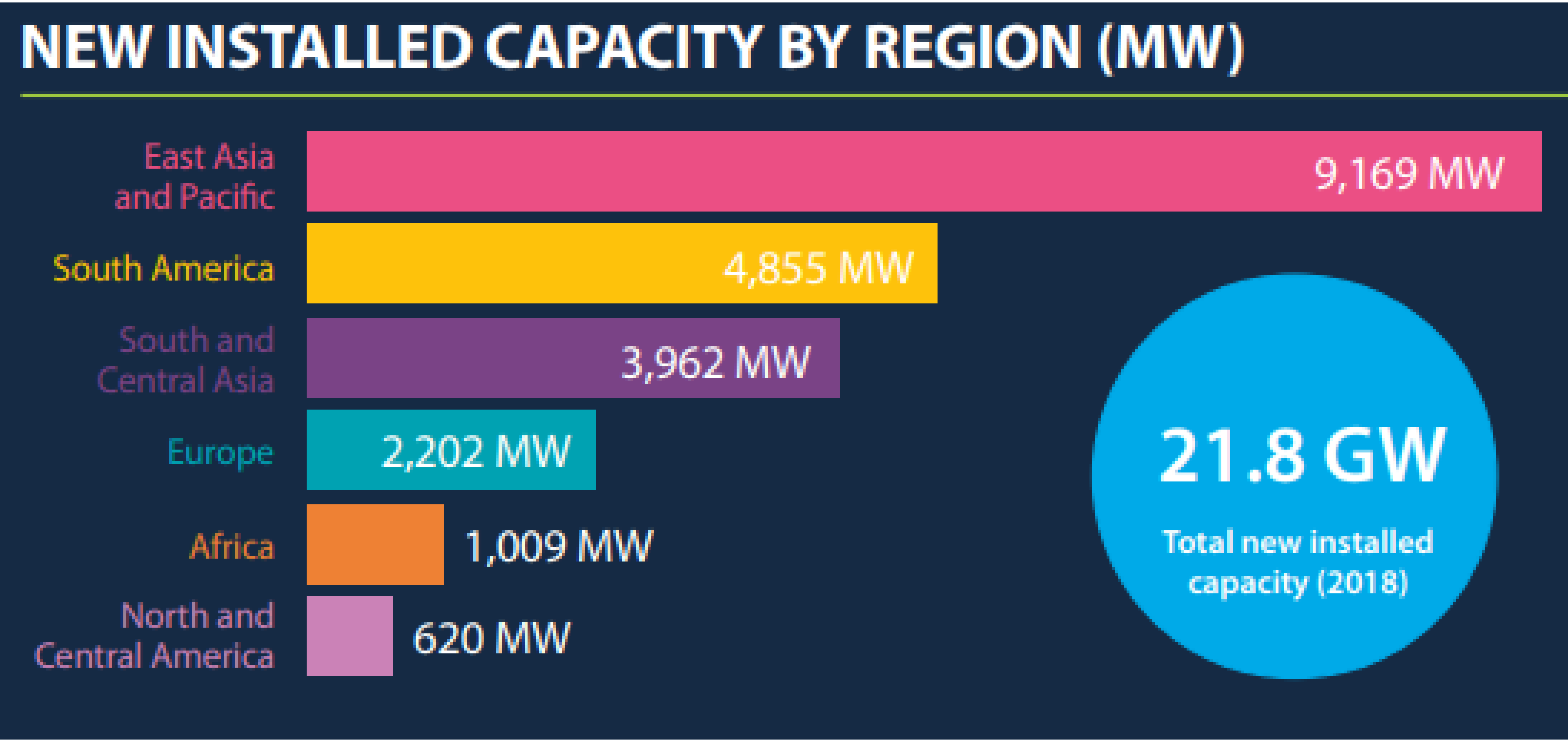

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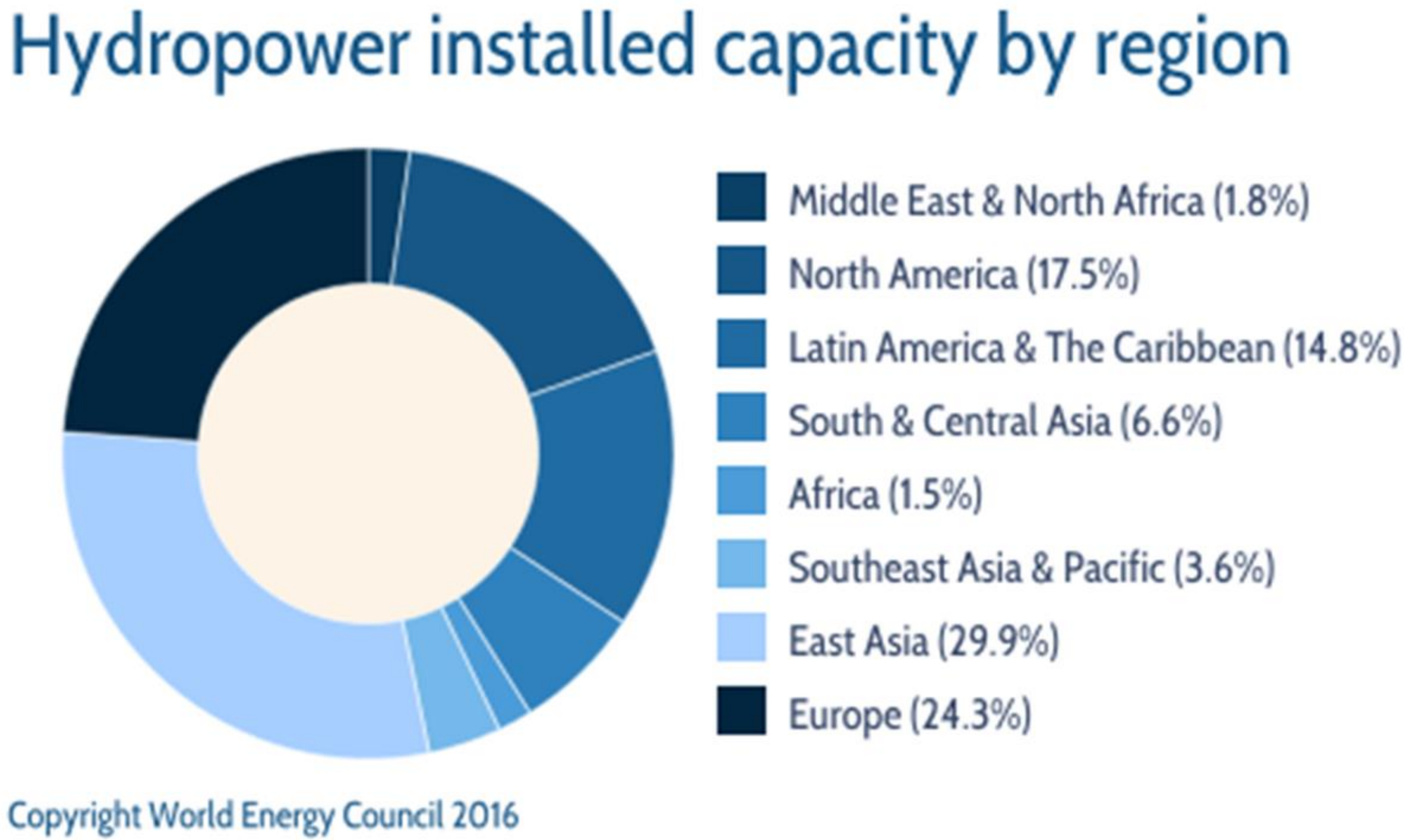


Hydropower Status in the World

AFRICA CAPACITY BY COUNTRY*

Rank	Country	Total installed capacity (MW)
1	Ethiopia	4,566
2	South Africa	3,595
3	Angola	3,083
4	Egypt	2,876
5	Democratic Republic of the Congo	2,704
6	Zambia	2,397
7	Mozambique	2,191
8	Nigeria	2,064
9	Sudan	1,923
10	Morocco	1,770

Hydropower Status in the World

NORTH AND CENTRAL AMERICA CAPACITY BY COUNTRY*

Rank	Country	Total installed capacity (MW)
1	United States	102,745
2	Canada	81,386
3	Mexico	12,117
4	Costa Rica	2,375
5	Panama	1,818
6	Guatemala	1,499
7	Honduras	656
8	Dominican Republic	543
9	El Salvador	472
10	Nicaragua	123

Hydropower Status in the World

SOUTH AMERICA CAPACITY BY COUNTRY*

Rank	Country	Total installed capacity (MW)
1	Brazil	104,139
2	Venezuela	15,393
3	Colombia	11,837
4	Argentina	11,288
5	Paraguay	8,810
6	Chile	6,753
7	Ecuador	5,072
8	Peru	4,995
9	Uruguay	1,538
10	Bolivia	658

Hydropower Status in the World

EUROPE CAPACITY BY COUNTRY*

Rank	Country	Total installed capacity (MW)
1	Norway	32,256
2	Turkey	28,358
3	France	25,519
4	Italy	22,926
5	Spain	20,378
6	Switzerland	16,948
7	Sweden	16,466
8	Austria	14,535
9	Germany	11,258
10	Portugal	7,347

Hydropower Status in the World

SOUTH AND CENTRAL ASIA CAPACITY BY COUNTRY*

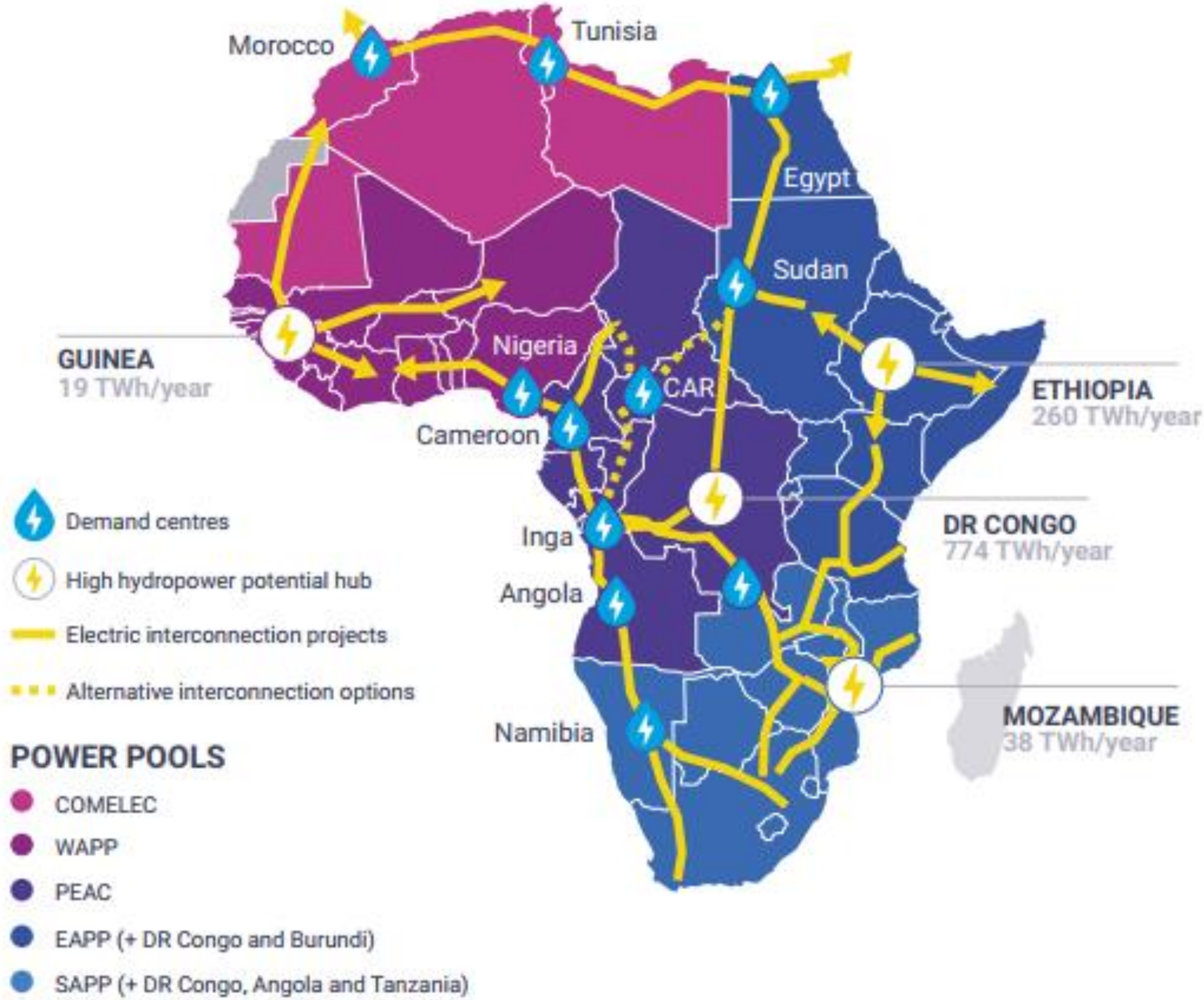
Rank	Country	Total installed capacity (MW)
1	India	49,917
2	Russia	48,506
3	Iran	11,951
4	Pakistan	9,827
5	Tajikistan	5,795
6	Georgia	3,221
7	Kyrgyzstan	3,091
8	Iraq	2,753
9	Kazakhstan	2,561
10	Uzbekistan	1,731

Hydropower Status in the World

EAST ASIA AND PACIFIC CAPACITY*

Rank	Country	Total installed capacity
1	China	352,260
2	Japan	49,905
3	Vietnam	16,679
4	Australia	8,790
5	South Korea	6,490
6	Malaysia	6,094
7	Indonesia	5,511
8	New Zealand	5,346
9	Laos	5,308
10	North Korea	5,010

Hydropower Status in the World



Hydropower in Ethiopia

- In Ethiopia the water power use came to existence when Abasamuel hydropower scheme is commissioned in 1932
- This station was capable of generating 6MW and operational up to 1970. Now its under rehabilitation.
- Ethiopia has a vast hydropower potential, which is estimated to be about 50,000 MW
- The current installed capacity is about 4566 MW
- By 2025 the country's total installed capacity will be 17,300 MW. *(source: GTPs documents)*

Hydropower in Ethiopia

- The demand for electricity is expected to grow at a rate of 12.7 % each year according to the EEPC
- Within the GTP periods, Ethiopia plans to commission a further new hydropower:
  - The cascaded Genale Dawa,
  - Geba I and II 385 MW.
  - Gibe IV and Gibe V (2,000 MW and 600 MW, respectively)
  - The Upper Dabus (326 MW) and
  - Halele Werabesa (436 MW)
  - etc.

Hydropower in Ethiopia

- Ethiopia also plans to begin construction on the following projects

No.	Renewable Energy source	Private Share IPP (MW)	Public Share EPC (MW)	Total Capacity
1	Hydro Power	3758	3821	7579
2	Geothermal	500		500
3	Wind	3600	1600	5200
4	Solar	5200		5200
5	Biomass	300	120	420

Hydropower in Ethiopia

There are two different power supply systems, viz.

The Interconnected System (ICS):

- supplied from hydropower plants
- has a total installed generation capacity of about 4566 MW

The Self-Contained System (SCS):

- consists of mini hydropower plants and a number of isolated diesel generating units
- has a total installed generation capacity of about 45.7 MW. *(outdated data)*

Hydropower in Ethiopia

S.No	Hydropower Plant name	River name	Capacity MW	Average Energy Production (GWh/yr)	Year of completion
1	Aba Samuel	Akaki river	6		1932
2	Koka	Awash river	42.3	131.12	1960
3	Awash II	Awash river	32	161.68	1966
4	Awash III	Awash river	32	174.81	1971
5	Fincha	Fincha river	134	912.29	1973
6	Melka Wakena	Shebele river	153	559.63	1989
7	Tis Abay I	Blue Nile river	11.4	48	1953
8	Tis Abay II	Blue Nile river	73	496.69	2001
9	Gilgel Gibe I	Gibe river	184	884.46	2004
10	Tekeze	Tekeze river	300	1069	2009
11	Gilgel Gibe II	Omo river	420	1886	2009
12	Tana Beles	Belesa river	460	2050	2010
13	Gilgel Gibe III	Omo river	1870		2015
14	Genale Dawa III	Genale river	254		2019

Hydropower in Ethiopia

S.No.	Name of the project	Capacity		Construction		Status
		MW	GWh	Launched	Completion	
1	Fincha-Amerti Nesh	97	215	2008		
2	Chemoga Yeda	278	1348	2011		New
3	Geba	336	1787	2011		New
4	Halele Werabesa	422	2030	2011		New
5	Genale Dawa VI	256	1475	2011		New
6	GER	6000		2011		Under construction
7	Gibe VI	1472				New
8	Gibe V	560				New

Hydropower in Ethiopia

- According to MoWIE, technical and economic exploitable power of Ethiopian river basins is shown below

No.	River Basin	Technical exploitable potential (MW)	Economic exploitable potential (MW)	Developed capacity (MW)
1	Abbay	23,868	11,459	7,315
2	Omo Gibe	7,960	5,226	4,634
3	BaroAkobo	7,744	2,988	0
4	Genela Dawa	3,246	881	254
5	Tekeze	2,377	750	300
6	Awash	689	127	107
7	WabiShebele	1,936	243	153
8	Rift Valley	209	0	0
		48,030	**21,674**	**12,763**

Source: MoWIE Conference August 27, 2019

Problems related to power production in Ethiopia

- Lowest income
- 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 here)
- 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

Pros and Cons of Hydropower

Advantages:

- Hydropower has a 'perpetual' source of energy
- It doesn't consume water
- Hydropower plants can be brought in to operation in few minutes and its controllable source of energy
- Running cost of hydropower plant is very low
- Very high Efficiency (90-95%)
- It provides secondary benefit such as recreation, fishing, flood control etc.
- No pollution.

Pros and Cons of Hydropower

Advantages:

- **Affordable and reliable energy:** Hydropower is the lowest cost source of electricity generation in many markets, with a global weighted average cost of USD 0.05 per kWh for new hydropower projects (IRENA 2018).
- **Enabling solar, wind and other renewables:** Hydropower supports growth in variable renewables such as wind and solar, meeting demand when these sources are unavailable.
- **Protecting from floods and drought:** The storage infrastructure provided by a hydropower reservoir mitigates against the risks posed by climate change, including extreme weather events such as floods and drought.

Pros and Cons of Hydropower

Advantages:

- Managing freshwater responsibly: Hydropower provides a vital means of safely managing freshwater, providing water supply for homes, businesses and agriculture.
- Avoiding pollutants and emissions: Hydropower is a low-carbon technology which helps to offset the carbon emissions and pollutants caused by fossil fuels.
- Improving infrastructure and waterways: Hydropower development delivers greater regional connectivity in distribution and transport networks

Pros and Cons of Hydropower

Advantages:

- Enhancing cooperation between countries: Long distance electricity transmission across national boundaries promotes strong inter-governmental cooperation.
- Community investments in rural areas: Hydropower development can bolster investment in local communities, including education, healthcare and other services.
- Recreational activities and tourism: Hydropower reservoirs can offer regional development through the creation of tourism, recreational activities and fisheries.

Pros and Cons of Hydropower

Disadvantages:

- It is capital intensive & therefore rate of return is low.
- The gestation period is long
- Hydropower is dependent on natural flow of streams
- The flooding of large areas of land, hence disturbance of habitat and serious geological damage
- Alteration of the natural water table level
- Dam breaching problems

Chapter 3
Classification & Types of Hydropower Development

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Contents of chapter three

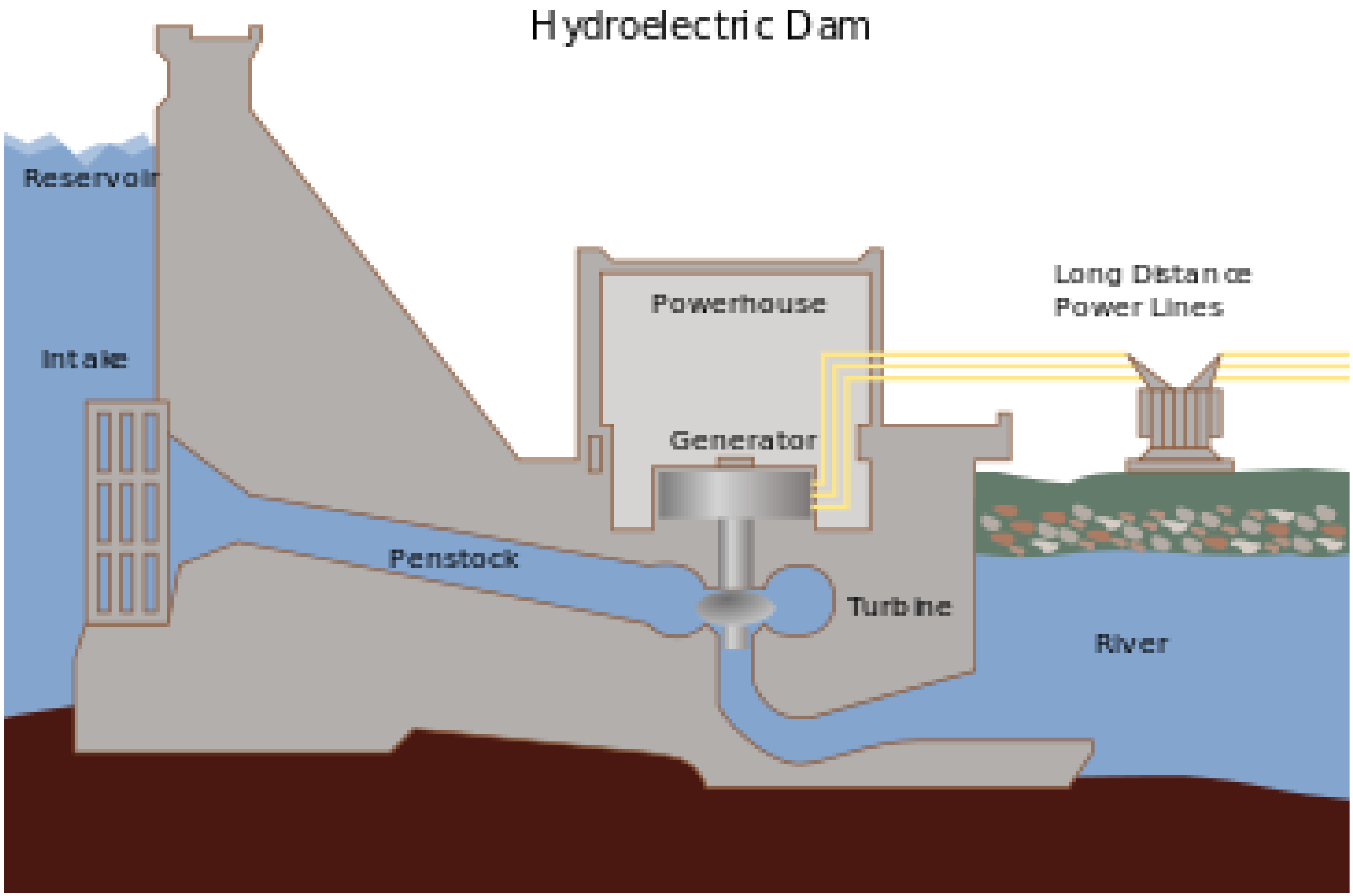
- Layout and types of hydropower developments
- Reservoir capacity determination
- Hydropower development cycles

LAYOUT OF HYDROPOWER DEVELOPMENTS

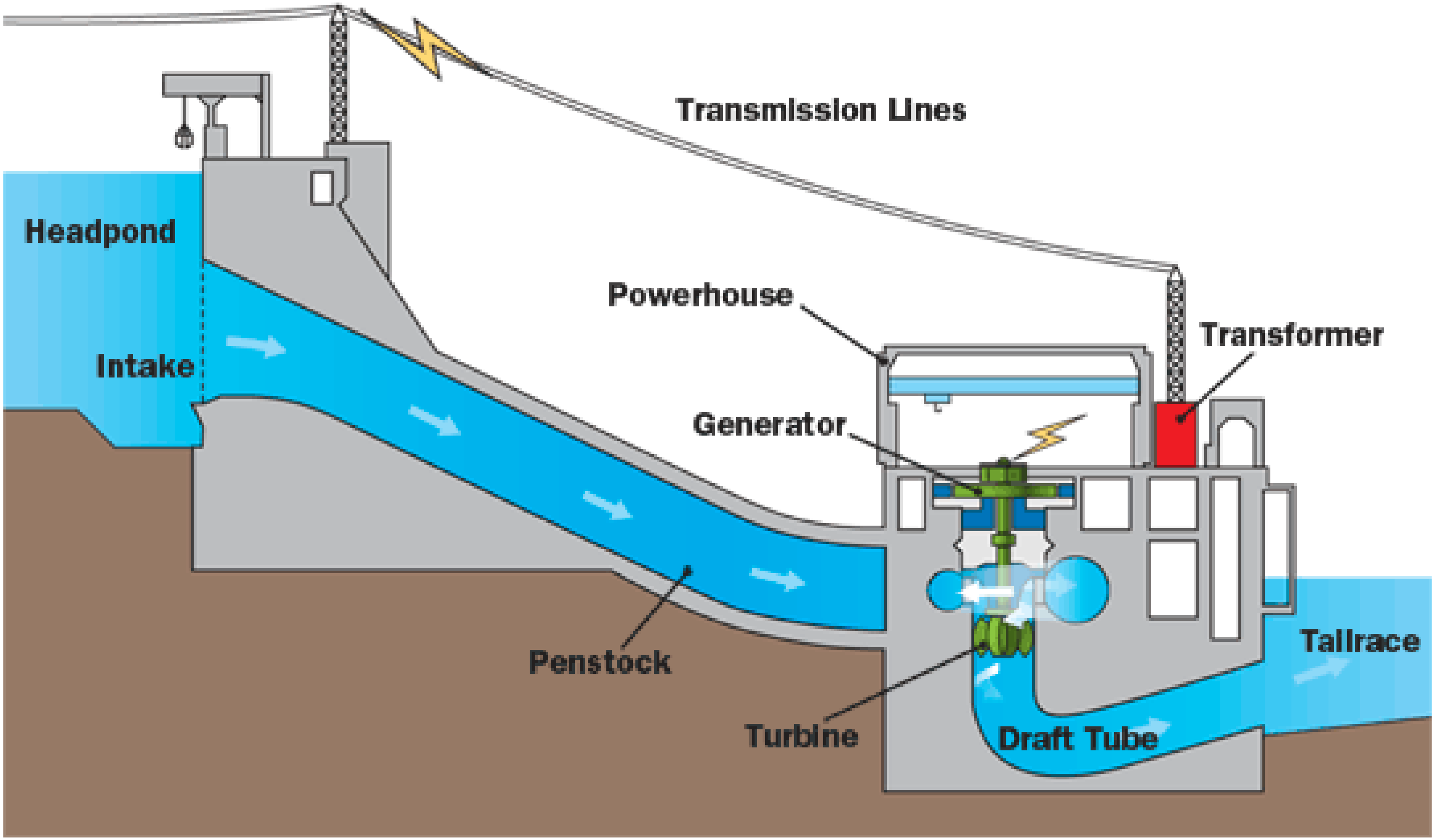
Typical components of a hydroelectric plant consist of the following:

1. Structure for water storage and/or diversion, like a dam or a barrage.
2. A head-race water conveying system like a conduit (penstock) or an open channel to transport water from the reservoir or head-water pool up to the turbines.
3. Turbines, coupled to generators
4. A tail race flow discharging conduit of open channel that conveys the water out of the turbine up to the river.

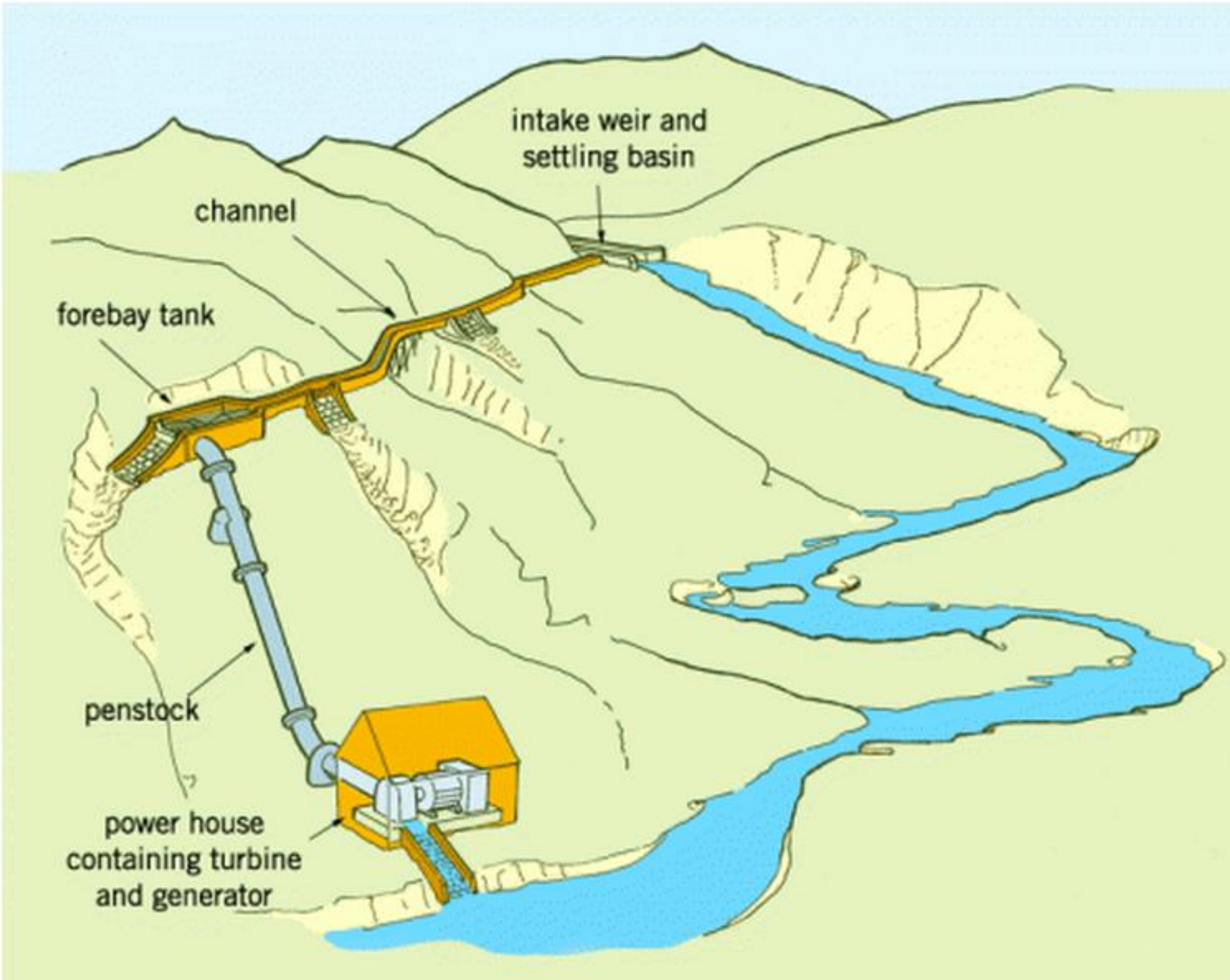
LAYOUT OF HYDROPOWER DEVELOPMENTS



LAYOUT OF HYDROPOWER DEVELOPMENTS



LAYOUT OF HYDROPOWER DEVELOPMENTS



Layout of hydropower developments

- Although the above components are common for all hydropower development schemes; the general arrangement are slightly different

- The arrangement of high and medium head power plants is more or less similar.

- The low head power plants, which are usually of run-of-power type schemes, have a slightly different arrangement

Layout of hydropower developments

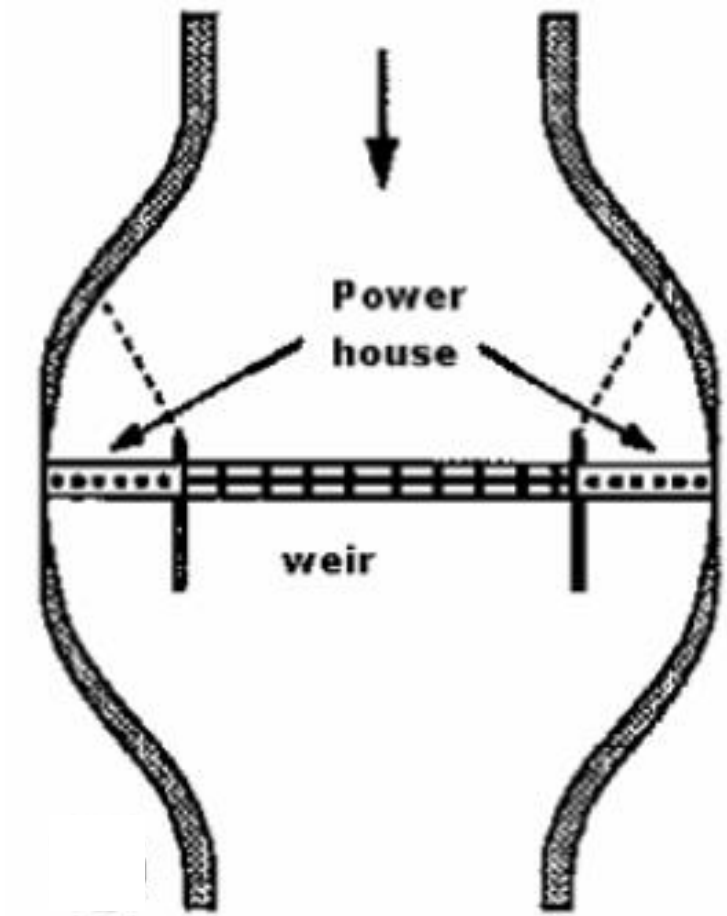
- In High and medium head development, usually, there could be two types of power scheme layout:
  - Concentrated fall schemes
  - Diversion schemes
- In the concentrated fall type projects, the powerhouse would be built at the toe of a concrete gravity dam.
  - The water is conveyed to the turbines via penstocks laid under, or bypassing, the dam.
  - It consists of a long system of water conduits.
- In the diversion type of layout, the diversion could be using a canal and a penstock or a tunnel and a penstock. The former is usually called the Open Flow Diversion System and the latter Pressure Diversion System.

TYPES OF DEVELOPMENTS

- In studying the subject of hydropower engineering, it is important to understand the different types of development.
- The following classification system are commonly used:
  - Operational feature
  - Basis of operation
  - Purpose of development
  - Uses to meet the demand for electrical power
  - Hydraulic feature
  - Plant capacity
  - Operational head

1. 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

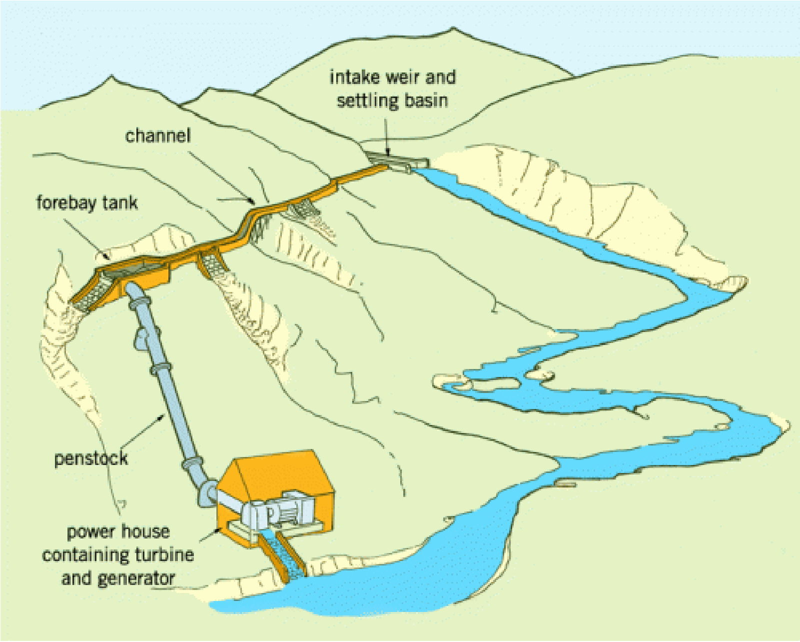


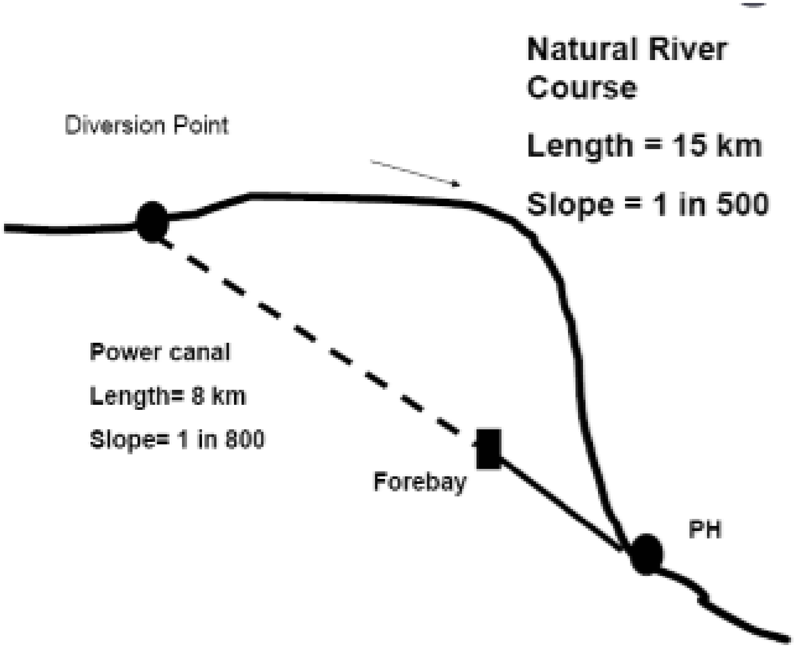
A. Operational feature

1. Run-of-river developments

2. Diversion and canal developments

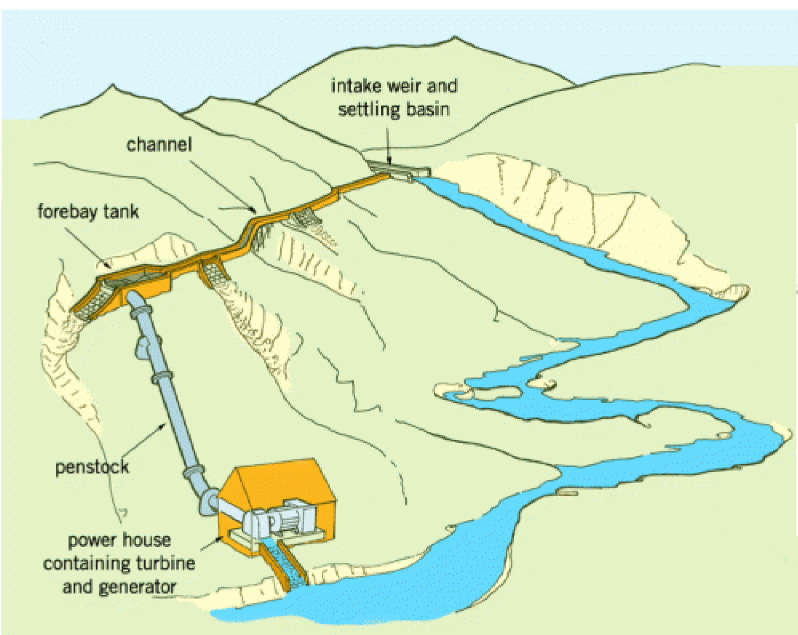
- Power canal or tunnel diverts water from main stream channel
- Powerhouse is provided at suitable location along the stretch of canal or tunnel

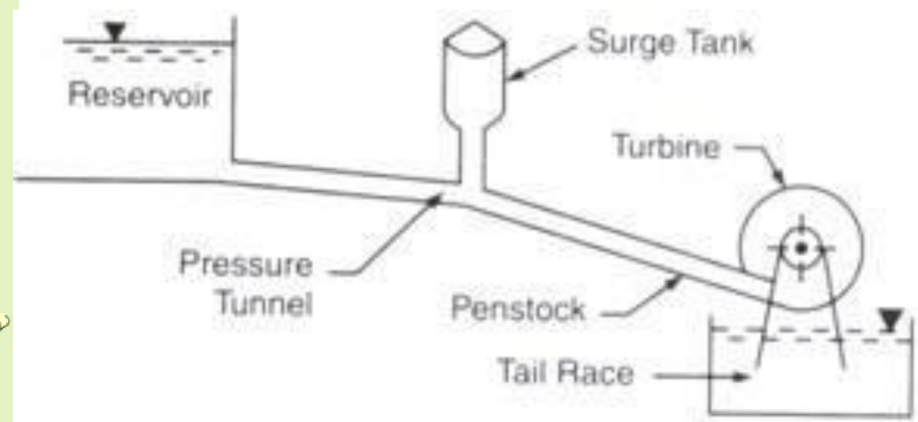




2. Diversion and canal developments

- 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



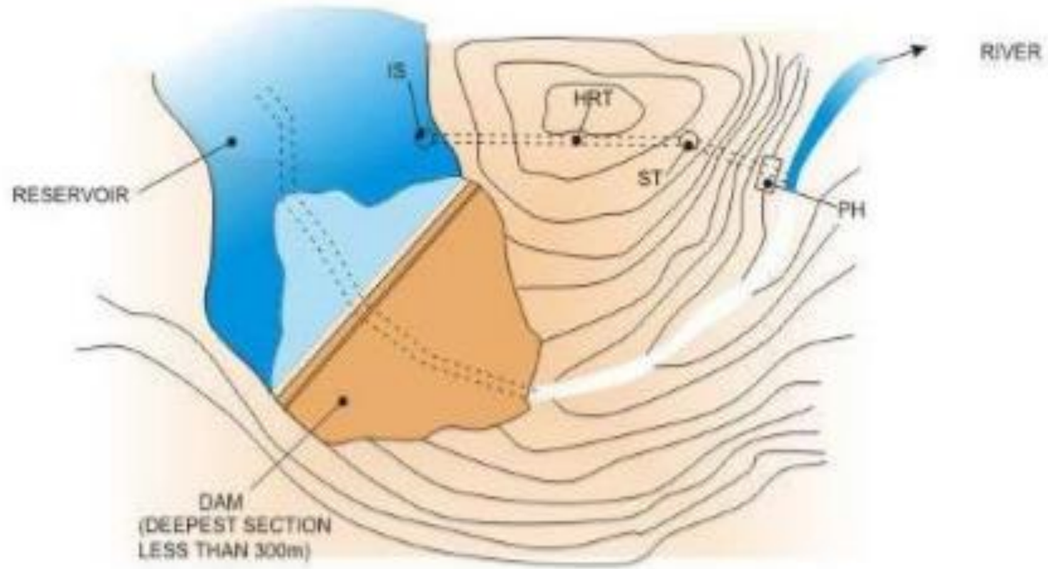


3. Storage regulation developments:

- Water stored during high-flow periods to augment the water available during the low-flow periods, thus supplying the demand for energy in a more efficient manner.

- Valley dam types of hydropower plants are storage regulation development type that have their powerhouse immediately at the toe of the dam.

3. Storage regulation developments:



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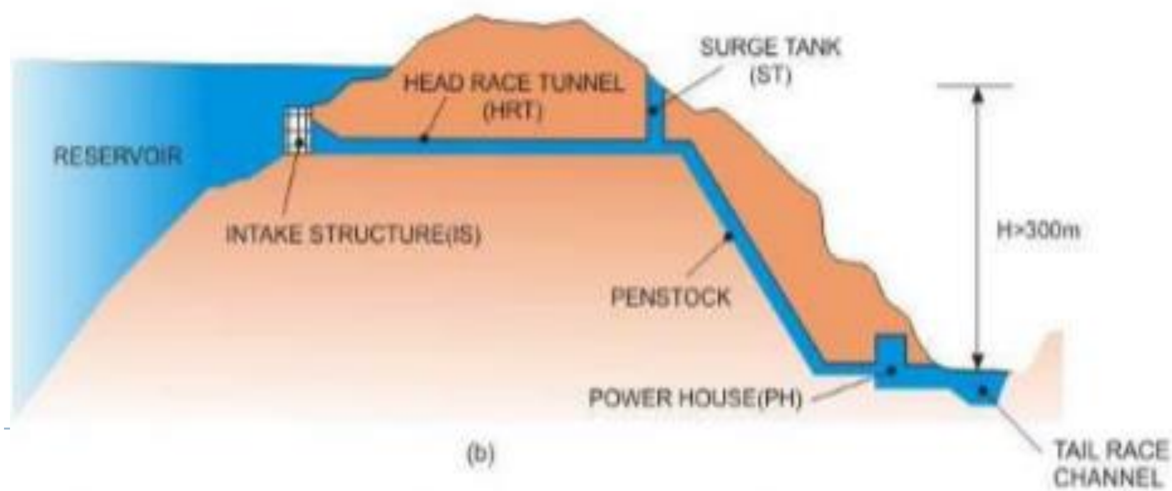
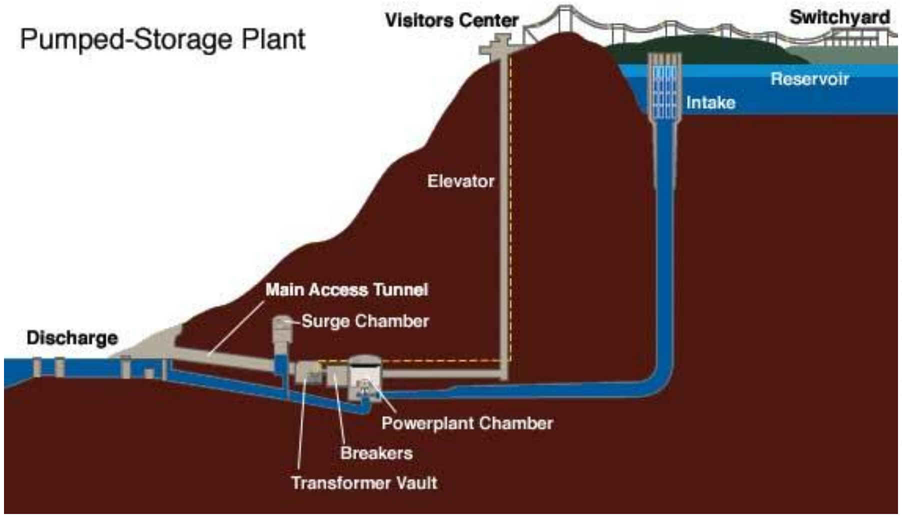


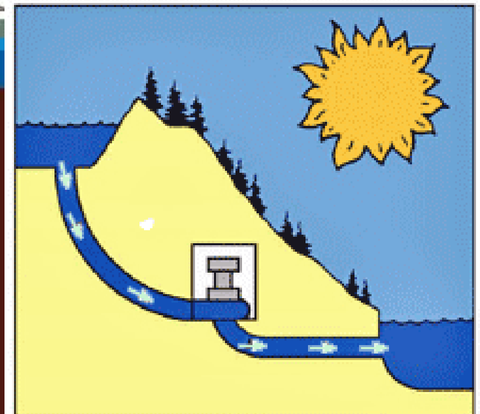
(b)

4. Pumped Storage Schemes

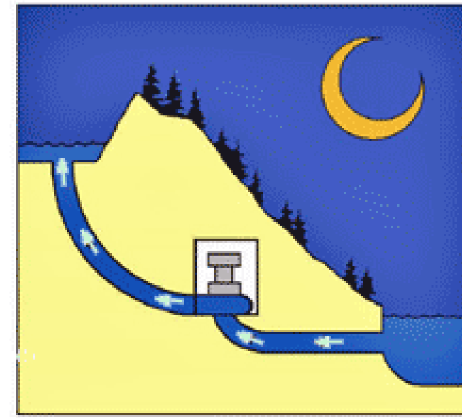
- Hydropower schemes of the pumped storage type are those which utilize the flow of water from a reservoir at higher potential (head-water pond) to one at lower potential (tail-water pond)
- During times of peak load, water is drawn from the head-water pond to run the reversible turbine-pump units in the turbine mode. The water released gets collected in the tailwater pond.
- During off-peak hours, the reversible units are supplied with the excess electricity available in the power grid which then pumps part of the water of the tail-water pond back into the head-water reservoir.

4. Pumped Storage Schemes

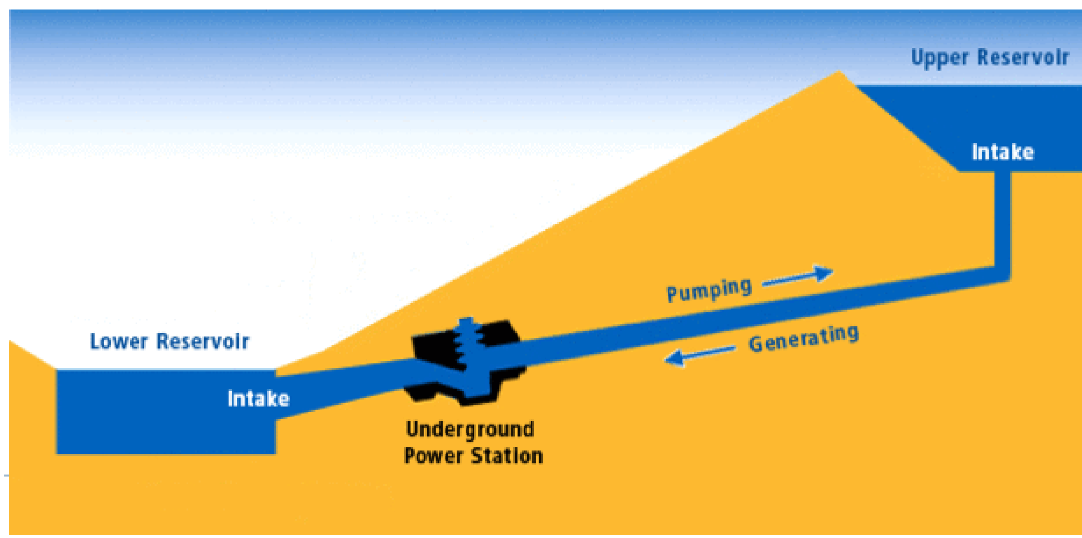




Daytime: Water flows downhill through turbines, producing electricity



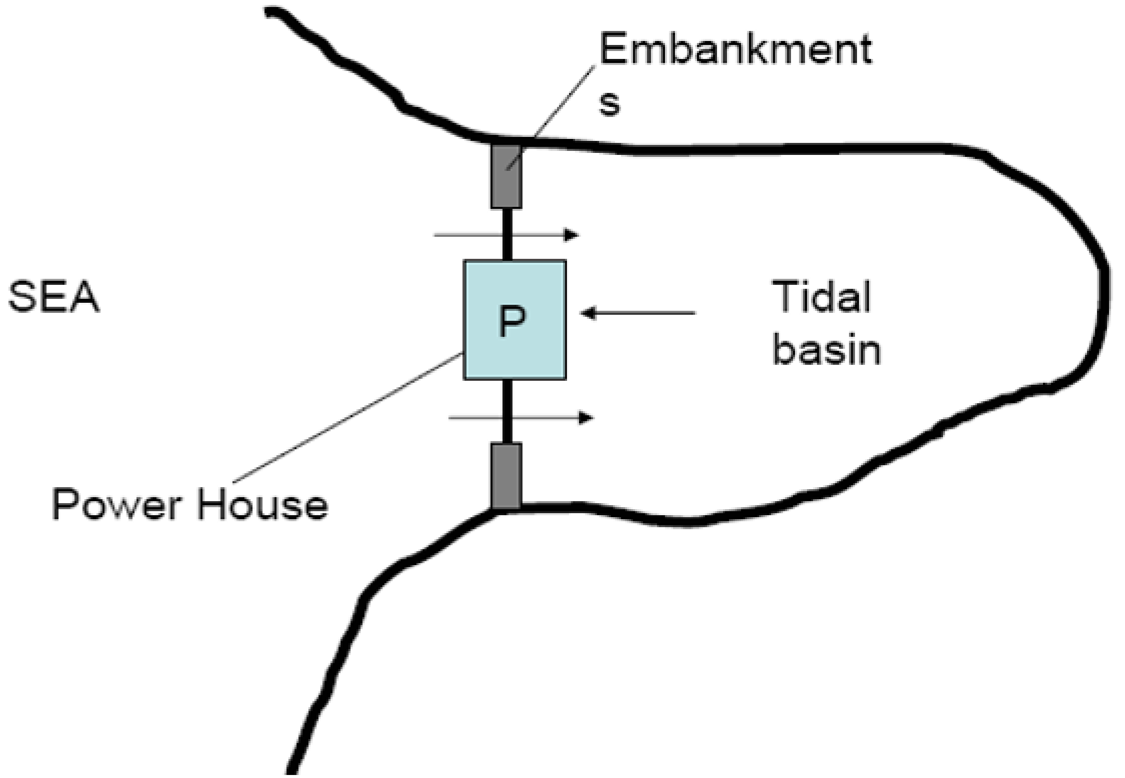
Nightime: Water pumped uphill to reservoir for tomorrow's use



4. Pumped Storage Schemes

5. Tidal power developments

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



B. 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.
- Note: Hydropower plants are best suited as peak load plants, because hydropower plants can start relatively quickly and can thus accept load quickly.

C. Purpose

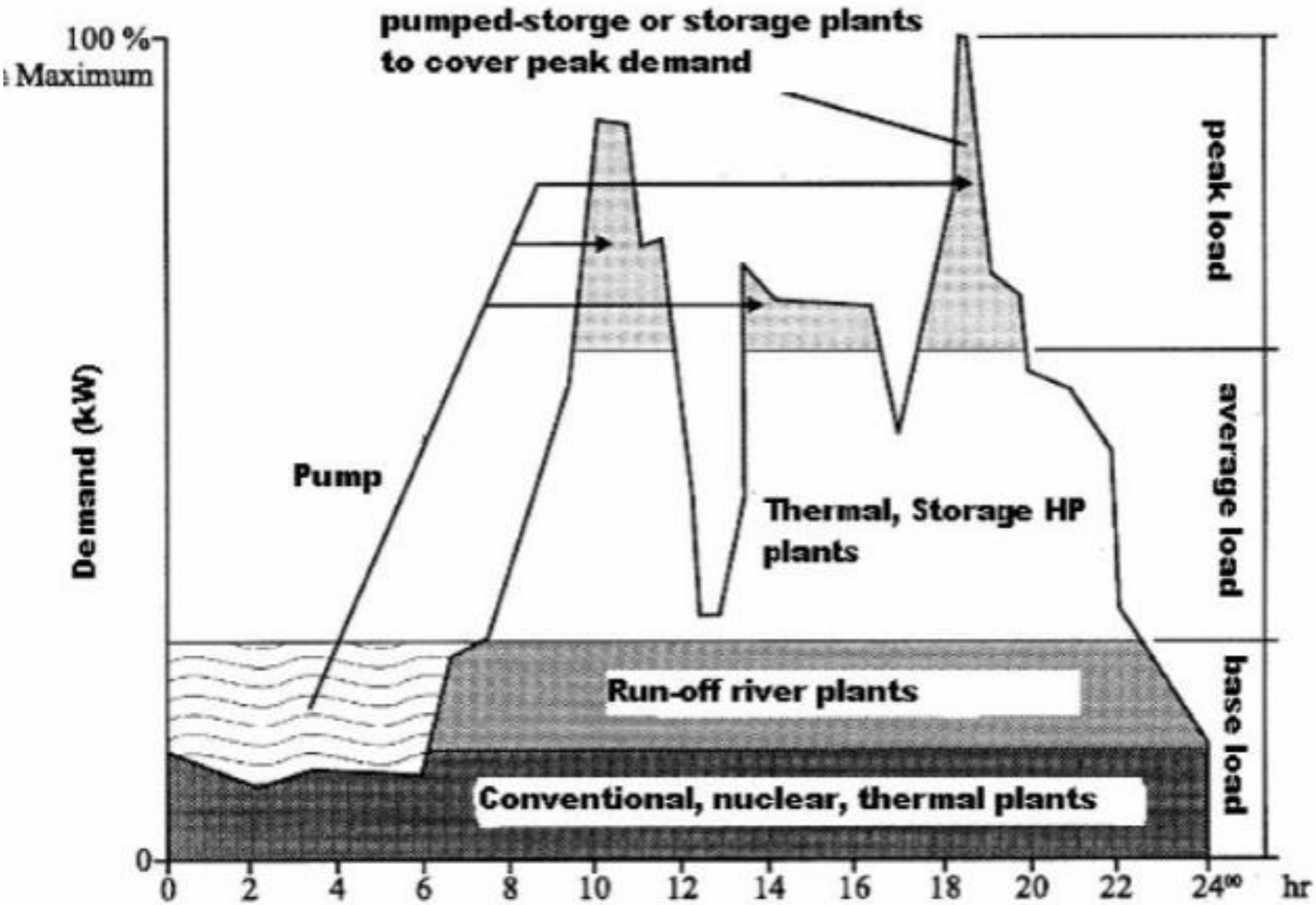
- **Single-purpose developments:** The water is used only for the purpose of producing electricity.

- **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.

D. 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

D. Basis of uses



E. Hydraulic feature

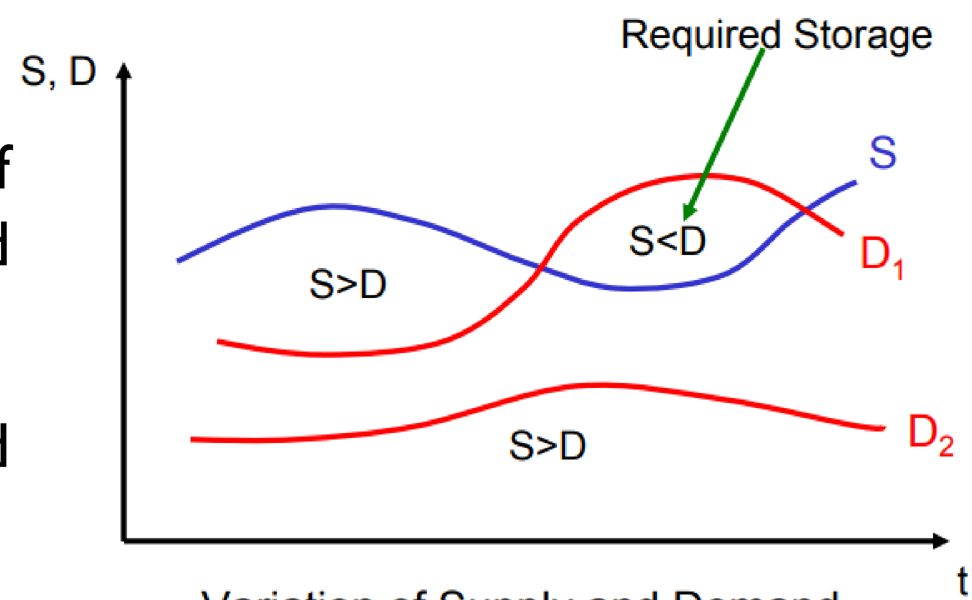
- Conventional Hydro-plants
  - Use normally available hydraulic energy of the flow of river
  - Run-off-river plant, diversion plant, storage plant
- Pumped-storage plants
  - Use the concept of recycling the same water
  - Normally used in areas with shortage of water
  - It has a function of indirect 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

F. Plant capacity and head

- Plant capacity: Usually this type of classification is arbitrary: for example:
  - 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

RESERVOIR (STORAGE) CAPACITY

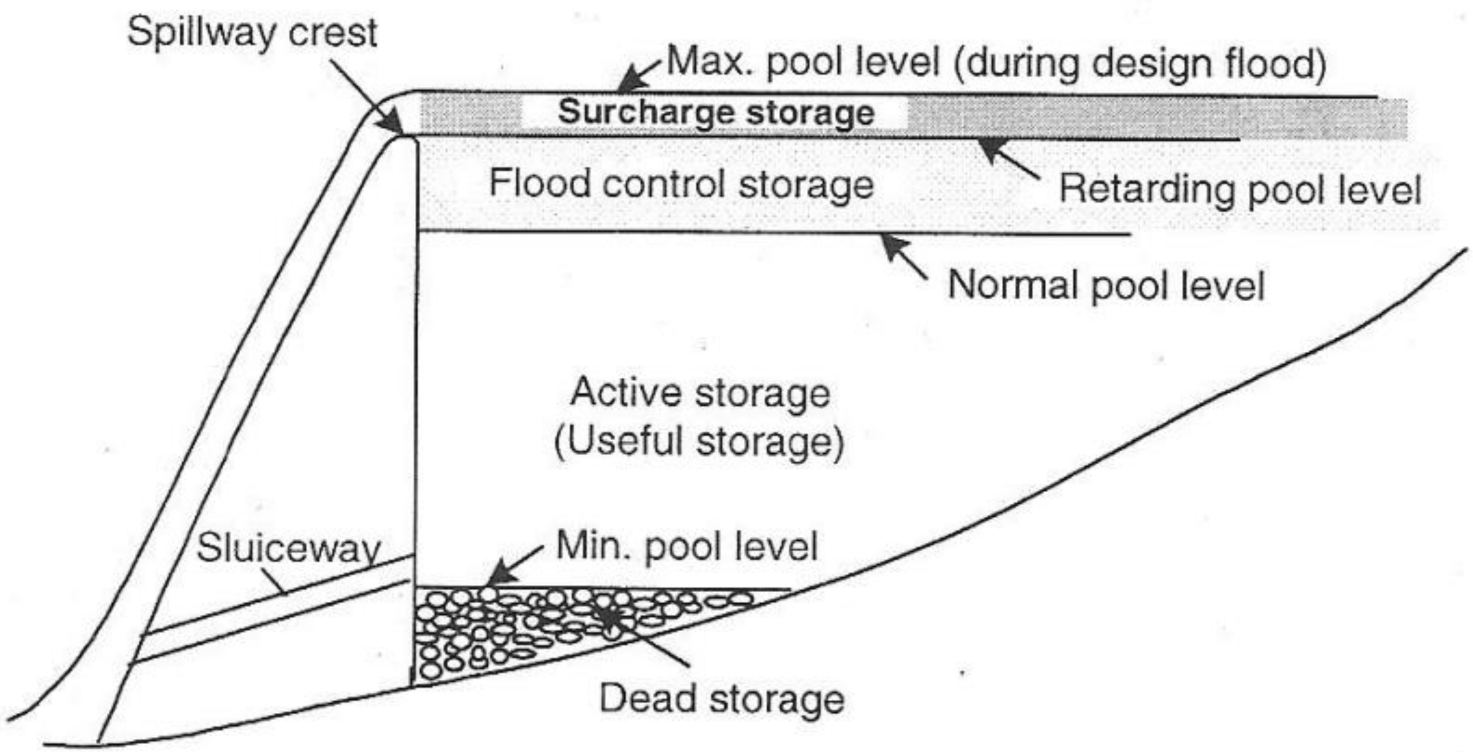
- **Reservoirs** are structures that store water.
- In general, we observe high flow in summer and low flow in winter
- On the other hand, the water demand is high in winter and low in summer.
- Therefore, the regulation of the streamflow is required to meet the demands.
- Reservoir capacity is depend on the inflow & demand.



Variation of Supply and Demand

RESERVOIR (STORAGE) CAPACITY

- Reservoir capacity has to be adjusted to account for the dead storage, evaporation losses and carry over storage.



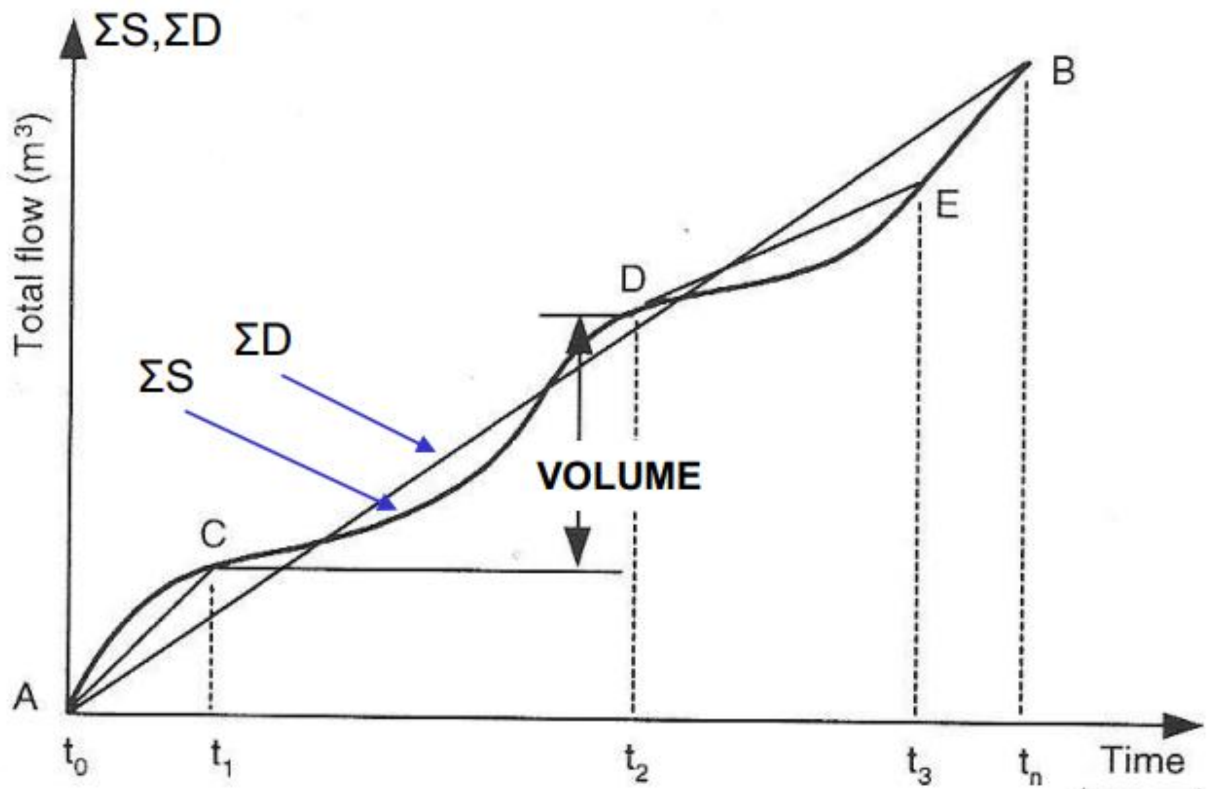
RESERVOIR (STORAGE) CAPACITY

The required capacity for a reservoir can be determined by the following methods:

- Mass Curve Analysis (Ripple diagram method)
- Sequent Peak Analysis
- Operation Study
- Other Approaches (Stochastic Methods and Optimization Analysis etc…)

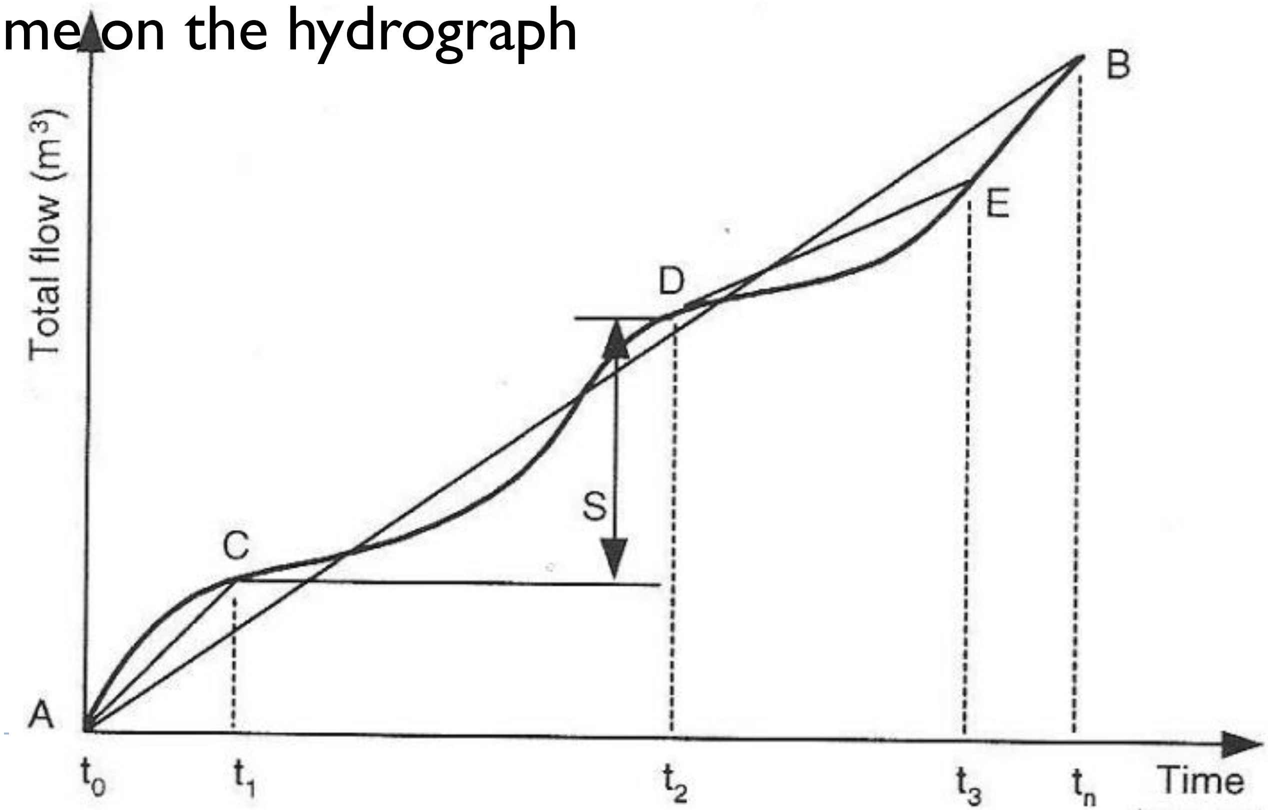
1. Mass Curve Analysis

- One of the most widely used methods.
- Assumptions:
  - Demand is constant, and
  - The year repeats itself continuously.



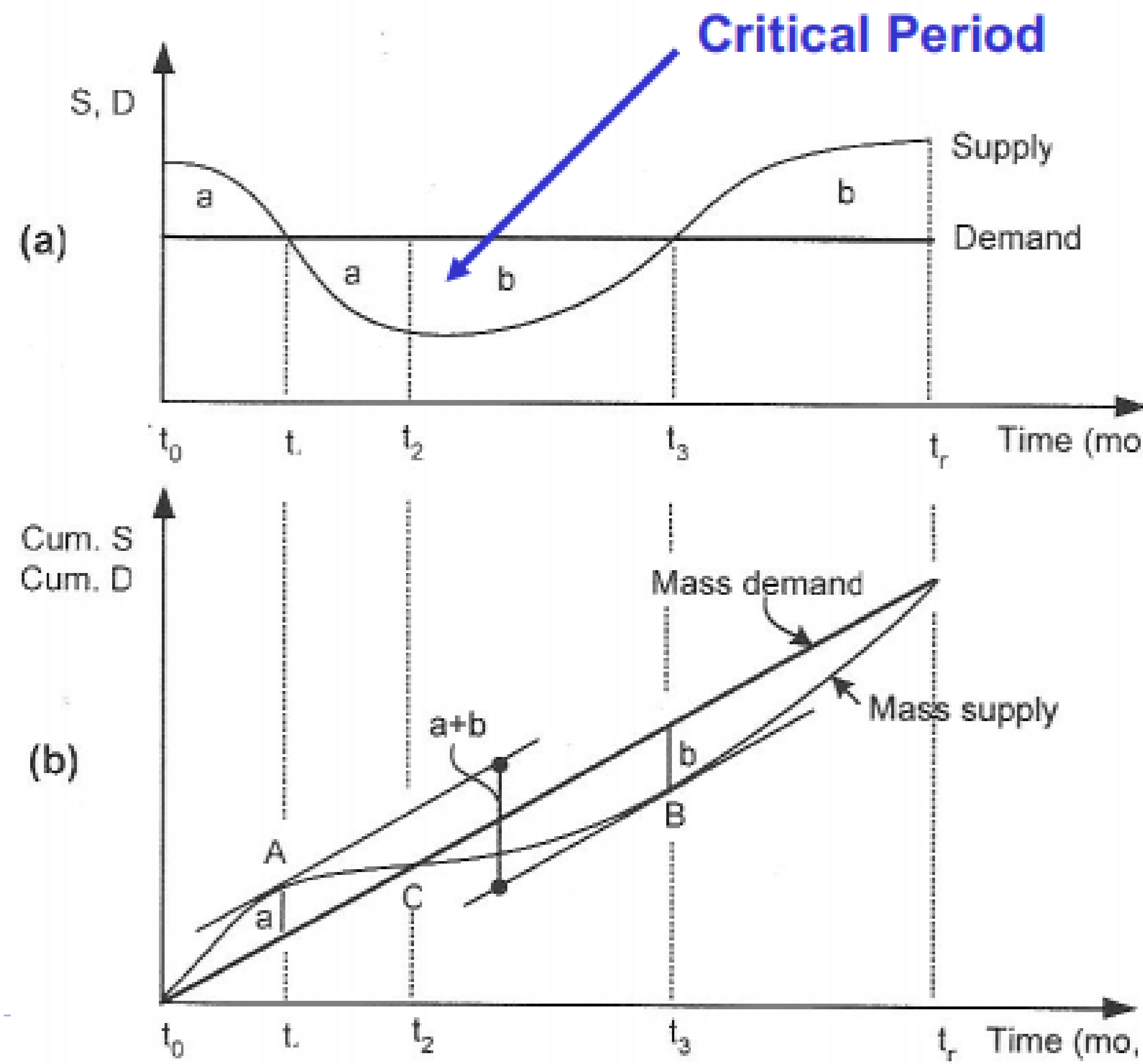
1. Mass Curve Analysis

- If the flow is the daily or monthly discharge then the area under the curve up to a certain time will be the volume of runoff for that period.
- The slope of the mass curve at a certain time gives the discharge at that time on the hydrograph



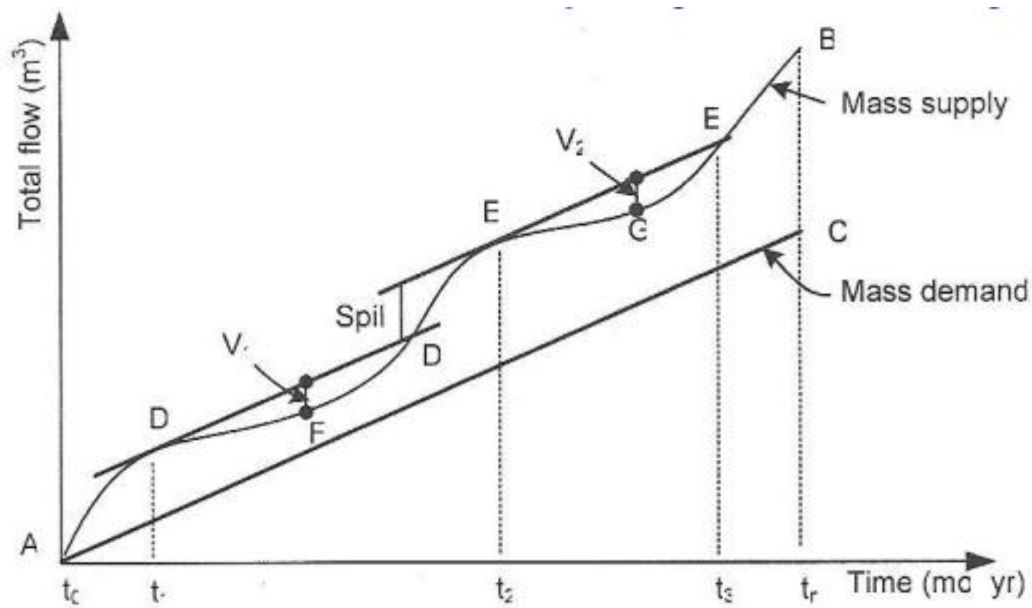
1. Mass Curve Analysis

- Required storage capacity of the reservoir is the vertical difference a+b



1. Mass Curve Analysis

A. Determination of capacity for a known yield



1. The tangents, which are parallel to the demand line, are plotted at the high points (D & E).

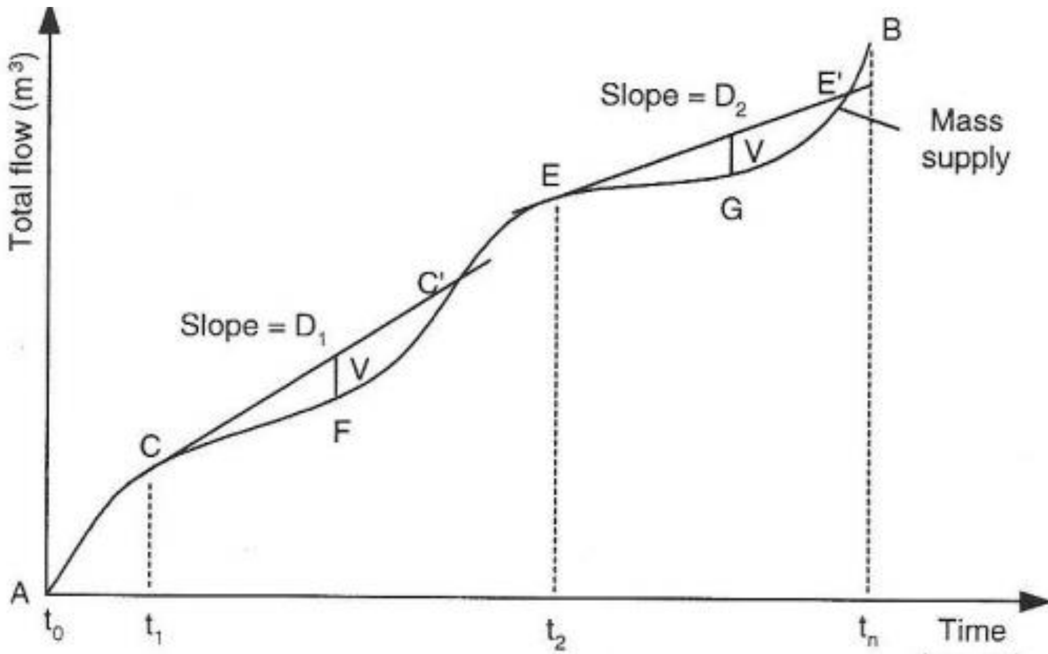
2. The maximum departures from the tangents to the following low points of the mass curve (F & G) determine the necessary storage amounts V_1 and V_2.

3. The largest one of the volumes will give the required capacity of the reservoir.

- The reservoir would be full at points D, D', E, & E'.
- The reservoir would be empty at points F & G.

1. Mass Curve Analysis

B. Determination of yield for a known **capacity**



- The plotted tangents must cut the mass curve when extended forward, as it is the case here with points C' and E'. Otherwise, the reservoir will not refill.

1. The value V of known reservoir capacity is placed vertically in all the low points in the mass curve and tangents are drawn to the previous high points.

2. The slope of these tangents (D_1 & D_2) indicate the yields that can be supplied for those critical periods with this given capacity.

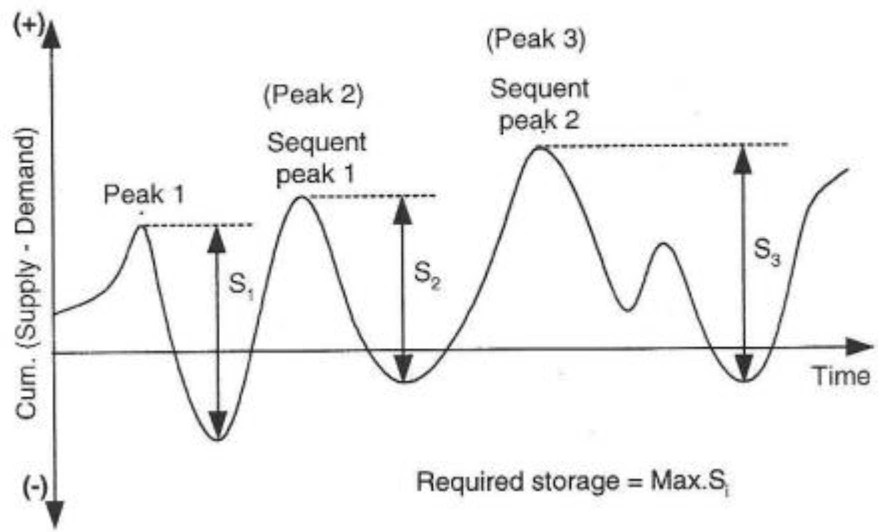
3. The smallest one of the yields can be supplied all the time.

1. Mass Curve Analysis

- The mass curve gives results if $\Sigma D < \Sigma S$ during the period of record.
- The graphical approach is quite satisfactory if the reservoir releases are constant during the period of analysis.
- When reservoir releases vary, the sequent-peak analysis is recommended

2. Sequent Peak Analysis

- Sequent Peak Analysis is more suitable when the data of long observation periods or long generated data are used, or when the demand is **not** constant



1. Differences between inflows (S) and demands (D) are calculated and their summations obtained.
2. Σ(S-D) values are plotted against time as shown in the figure.
3. On this plot the first peak value and next larger peak (sequent peak) are determined.
4. The storage required between these two points is the difference between the first peak and the lowest point in this period
5. This process is repeated for all the peaks in the record period as shown in the figure also. The maximum of the storage values is the required capacity.

2. Sequent Peak Analysis

- If the record period or generated data sequence is very long, the graphical solution may be time consuming
- In that case and analytical solution procedure may be applied for the analysis and it can be solved easily using a computer
- In this way, the required storage V_t at the end of a period t can be expressed as:

$$V_t = \begin{cases} D_t - S_t + V_{t-1} & \textit{if positive} \\ 0 & \textit{otherwise} \end{cases}$$

- At the beginning of the analysis, initially V_t-1 is set to zero and calculations continue to find V_t values for up to twice the length of the record period.
- The maximum of all the calculated values of Vt is the required storage capacity

3. Operation Study

- The reservoir storage is considered as adequate when the reservoir can supply all types of demands under possible losses like seepage and evaporation.
- In order to increase the operational performance of a reservoir, the evaporation and seepage must be controlled.
- The operation of a storage reservoir is also governed by the inflow.
- Rule curves indicating temporal storage requirements according to local conditions and project demands need to be used for effective operation purposes

3. Operation Study

Rules:

- During normal periods of river flow, the reservoir will be maintained at the normal pool level.
- If extremely high flows are expected, the normal pool level can be drawn to such an elevation that the maximum expected flood flow will be sufficient to restore the active storage to its maximum level.
- The operation study is based on the solution of the continuity equation

$$\frac{dv}{dt} = I - O$$

where dv: differential storage during time dt
I: instantaneous total inflow
O: instantaneous total outflow

3. Operation Study

- Since the information concerned the time variation of inflow and outflow is normally limited, then long term (e.g. one month) averaged quantities of inflow and outflow are considered in practice:

$$\frac{\Delta V}{\Delta t} = \bar{I} - \bar{O}$$

where ΔV: the change in storage during time interval Δt.

$\bar{I}$: the average inflow (runoff, precipitation etc…) during Δt

$\bar{O}$: the average outflow (evaporation, seepage, controlled outflows, mandatory releases, uncontrolled spills etc…) during Δt.

In general ....

1. Collect the stream flow data at the reservoir site during the critical dry period. Generally, the monthly inflow rates are required. However, for very large reservoirs, the annual inflow rates may be used.
2. Ascertain the discharge to be released downstream to satisfy water rights or to honor the agreement between the states or the cities.
3. Determine the direct precipitation volume falling on the reservoir during the month.
4. Estimate the evaporation losses which would occur from the reservoir
5. Find out the demand during various months.

In general ....

6. Determine the adjusted inflow during different months as follows:

   Adjusted inflow = Stream inflow + Ppt – Evap'n – d/s Discharge

7. Compute the storage capacity for each month

   Storage required = Adjusted inflow – Demand

8. Determine the total storage capacity of the reservoir by adding the storages required found in Step 7.

HYDROPOWER DEVELOPMENT CYCLES

The studies to be carried out are:

- Resources studies
  - Preparation/updating of resources inventories
  - Preparation/updating of resources rankings
- Site specific studies
  - Reconnaissance studies
  - Pre-feasibility studies
  - Feasibility studies

Resources Studies

- The main purpose of resource inventory investigation is to identify, register and catalogue the hydropower resource existing in a river basins
- 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.

Site Specific Studies

- 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.

Site Specific Studies

- 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 & 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:

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

Main activities to be done:

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
- Env'tal constraints

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.

Main activities to be done:

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
- Incase 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 like drinking, irrigation, etc.
- Verification of estimated head
- Powerhouse type, location and equipment

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

Pre-feasibility study (Preliminary Design)

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 be recognized for each and every stakeholder.
2. Power studies: considers a balance b/n power supply & 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.

Pre-feasibility study (Preliminary Design)

4. Geology and foundation engineering
5. Seismic studies
6. Environmental studies
7. Estimation of cost
8. Economic and financial studies
9. Future investigation plan
10. 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

Feasibility study

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 & control measures
- Number and size of units
- Auxiliary equipment
- Transmission planning

Feasibility study

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

Feasibility study

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
9. Feasibility Report

Implementation Phase

Project implementation is a multidisciplinary job which include:

- 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

Chapter 4
Estimation of Water Power Potential

HEng-3161 Hydropower Engineering I

Contents of chapter four

- Water power potential
- Power equation
- Hydrology of hydropower
  - Flow duration curve
  - Flow estimation for ungauged sites
  - Discharge capacity of HP plant
  - Tail-water Relationships
  - Area capacity curves
  - Reservoir Rule Curves
  - Evaporation Loss Evaluation
  - Spillway Design Flood Analysis
- Energy and Power Analysis Using Flow Duration Approach
  - Power duration curve
  - Load terminologies
  - Load duration curve

Water Power Potential

- When water flowing down from places of higher elevations to those with lower elevations, it loses its potential energy and gain kinetic energy

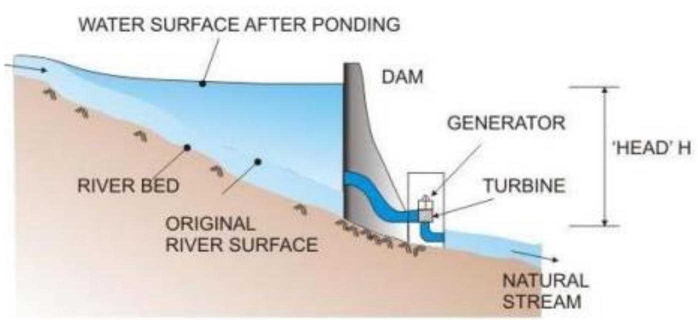
- This energy is quite high in many rivers.

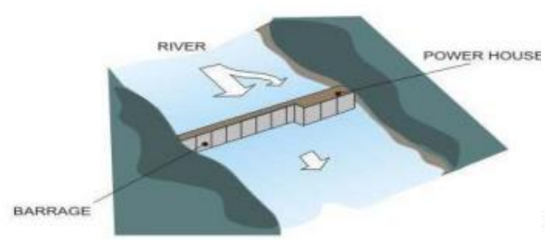
- Hydropower engineering tries to tap this vast amount of energy available in the flowing water and convert that to electricity

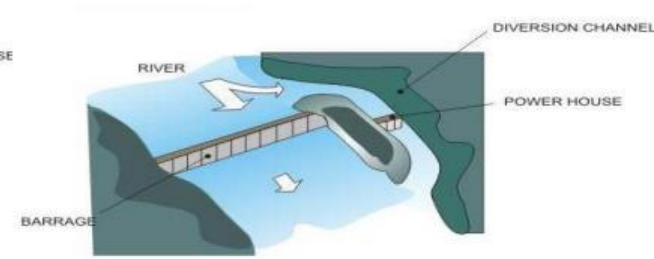
- To do so, it is necessary to create a head at a point of the stream and to convey the water through the head to the turbines which will transform the energy of the water into mechanical energy to be further converted to electrical energy by generators.

Water Power Potential

- The gross head can be assessed by surveying techniques.
- The necessary head can be created in different ways:
  - Building a dam across a stream
  - Divert a part of the stream by creating a low-head diversion structure like barrage.

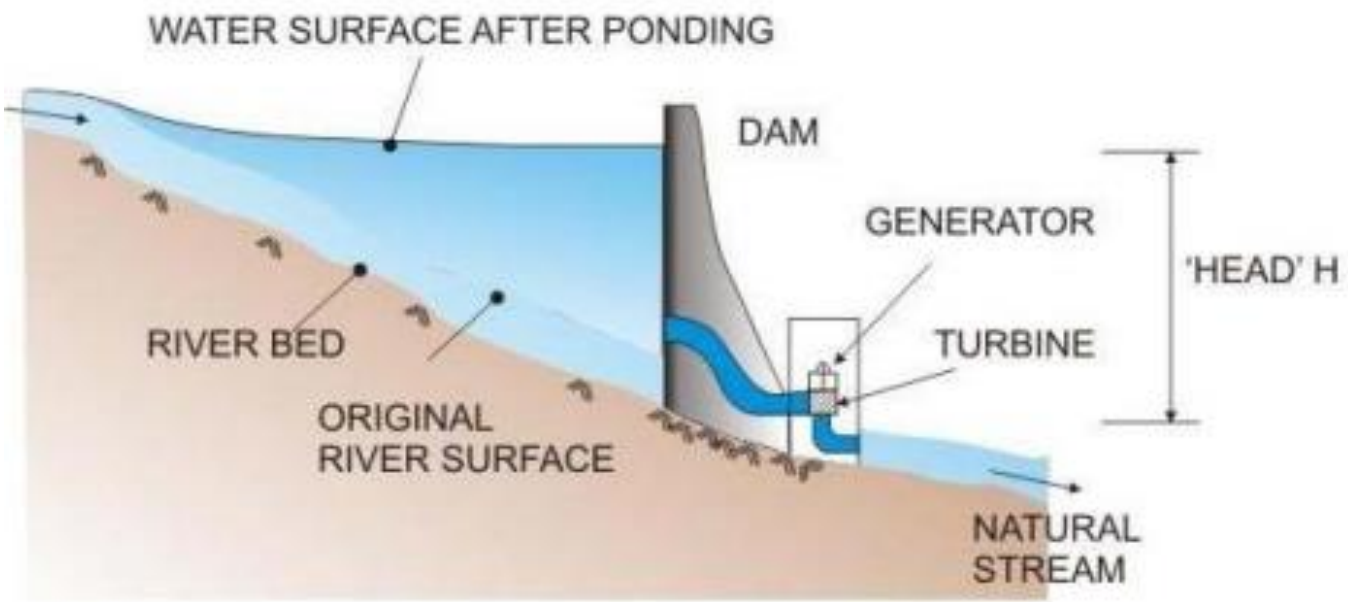






Water Power Potential

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



Water Power Potential

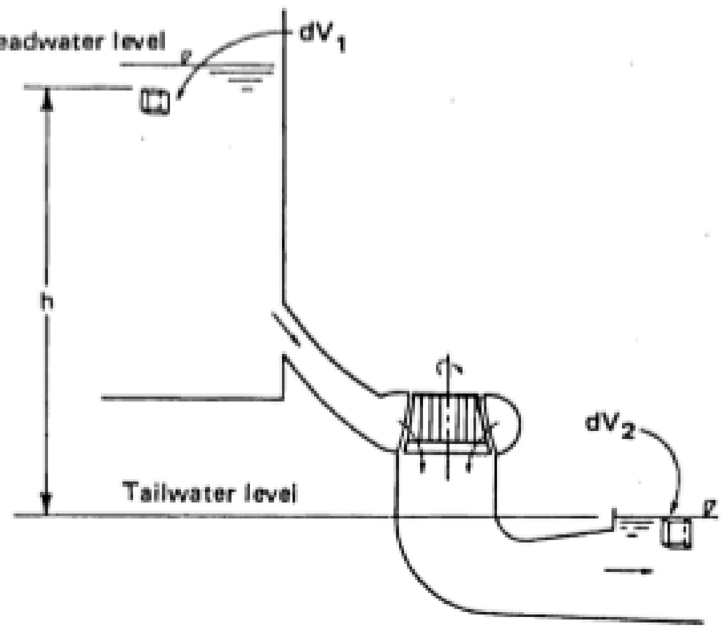
- The river flow rate can assessed through the study of hydrological data on rainfall and runoff
- The following hydrological data are necessary:
  a) The daily, weekly or monthly flow over a period of several years, to determine the plant capacity & estimated output.
  b) Low flows, to asses the primary, firm, or dependable power.
- Assumption: precipitation and stream flow conditions which have been observed in the past can be expected to occur, within reasonable limits of similarity, in the future

Power Equation

Energy-work approach

- Work (W) = Force x Distance in the direction of force
- Work (W) = Weight of water x the distance it falls

$$W = \rho_w V_w g h$$



- Where: ρ_w is density of water;
  g - acceleration due to gravity;
  V_w- volume of water falling;
  h - the vertical distance the water falls (effective head)

Power Equation

Energy-work approach

- Power (P) = Work / time

$$P = \frac{W}{t} = \frac{\rho_w V_w g h}{t} = \rho_w Q g h \qquad Note: Q = \frac{V_w}{\mathrm{t}}$$

$$P = \eta \gamma Q h \qquad P[kw]$$

- Where: Q is discharge, η is efficiency & γ is unit weight of water
- To compare kilowatts and horsepower remember that:

$$P_{kw} = 0.746 P_{hp}$$

Power Equation

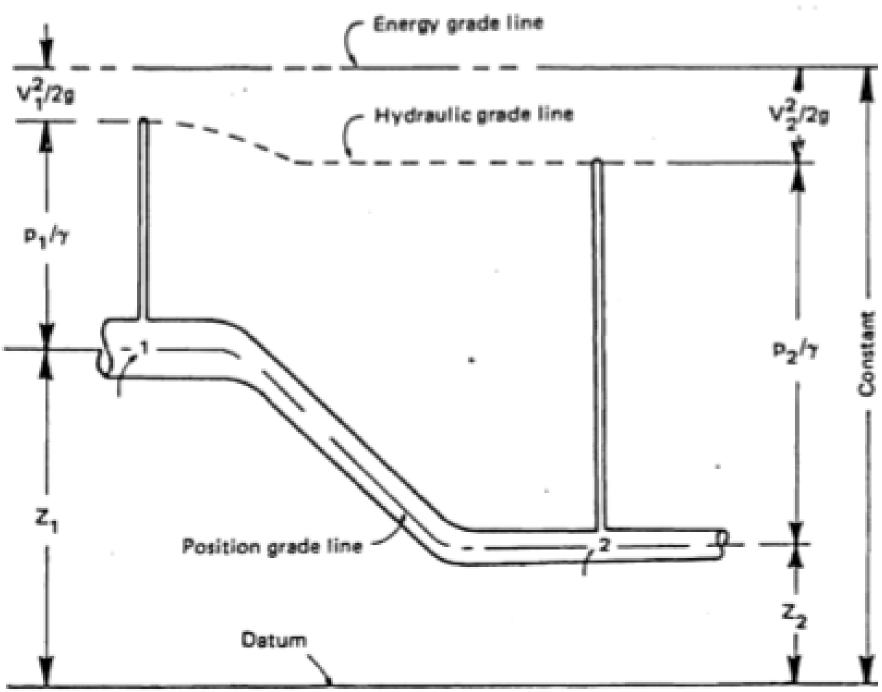
Energy Equation Approach

- 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*

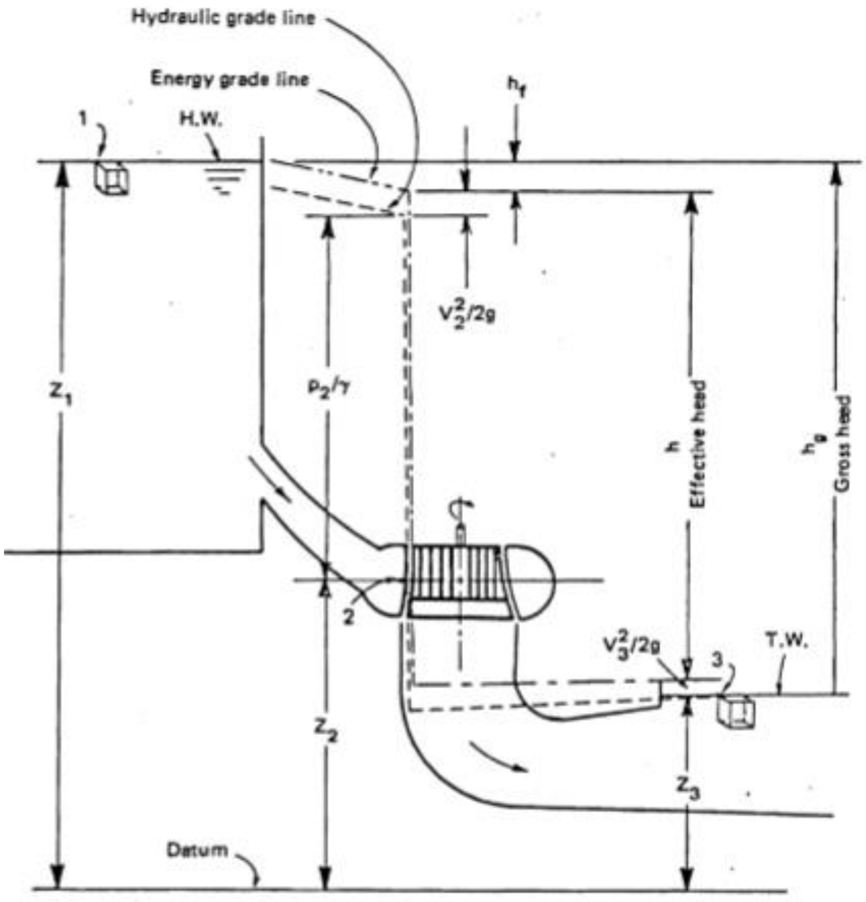
- p_1 = *pressure at point 1*
- $\gamma = \rho g$ = *specific weight of water*
- Z_1 = *potential head at point 1*
- V_2 = *water velocity at point 2*
- p_2 = *pressure at point 2*
- Z_2 = *potential head at point 2*
- h_f = *head loss in flow passage between points 1 and 2*



Power Equation

Energy Equation Approach

- 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:



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

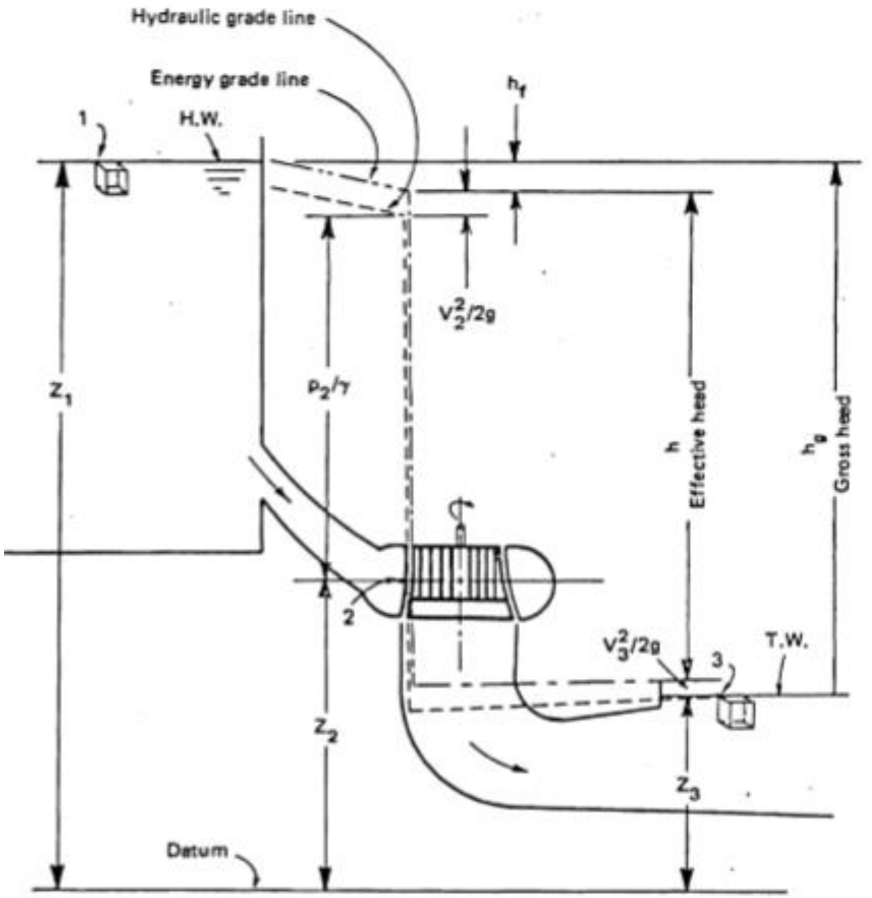
Power Equation

Energy Equation Approach

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

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

- Where h is effective head on the turbine



Power Equation

Energy Equation Approach

- Recognizing that for practical purposes V_1, P_1, and P_3 are equal to zero, then solving for P_2/γ in Eq. 1, the result is:

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

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

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

Power Equation

Energy Equation Approach

- 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 $\rho g q$.

- Now recognizing that energy per unit of time is power, it is simple to calculate by multiplying Eq. (4) and $\rho g q$ *or* γq to obtain the theoretical power delivered by the water to the turbine as $\gamma q h$ which is the theoretical power

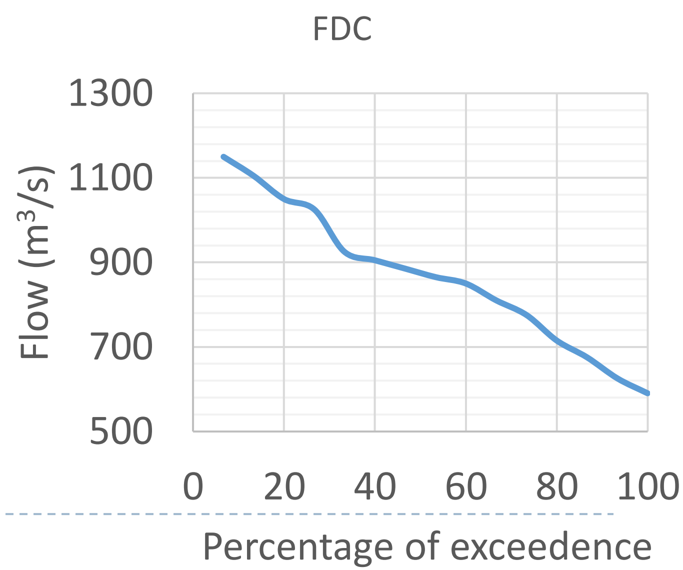
- $$P_{theoretical} = \gamma\, q\, h$$

Hydrology of hydropower

- Hydrological studies will provide data on the flow of water, one of the main parameters used in hydropower planning.
- Precipitation and hence water supply, varies widely between geographical locations, from season to season and from year to year.
- Each of these variations has a profound effect on the planning for the control and use of water resources
- The determination of the water requirement for power production is probably best accomplished by trial and error methods including incremental analyses and will require close coordination and integration of power studies and economic and social studies.

Flow duration studies

- A useful way of treating the time variability of water discharge data in hydropower studies is by utilizing flow duration curves.
- A flow duration curve is a plot of flow versus the percent of time a particular flow can be expected to be exceeded.
- A flow duration curve merely reorders the flows in order of magnitude instead of the time ordering of flows versus time plot.
- Two methods:
  - The Rank ordered technique and
  - The Class-interval technique.



Rank ordered 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.

Rank ordered technique

Example:

- The following is the record of average yearly flow (m^3/s) in a river for 15 years. Construct the FDC for the river.

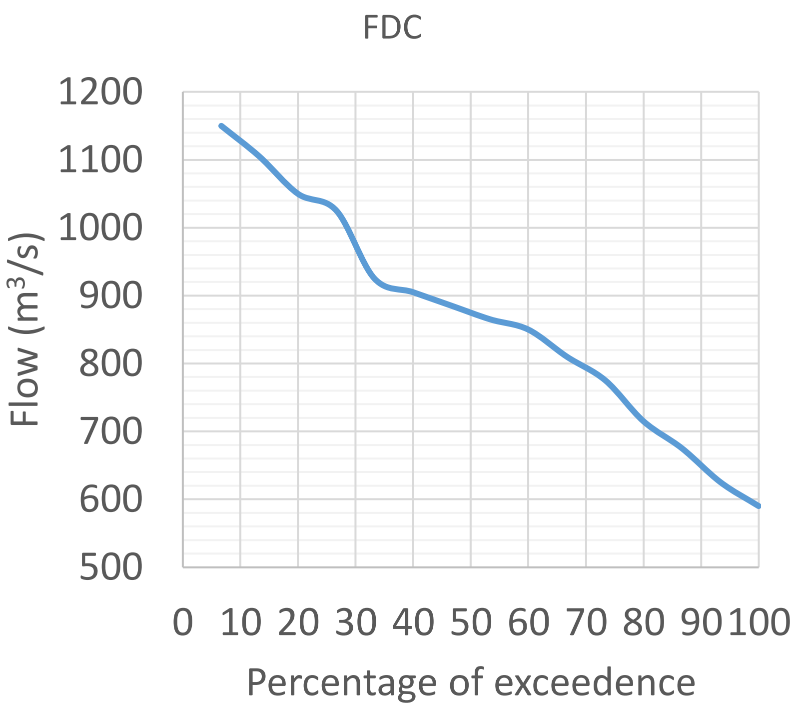
Year	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966
Flow	905	865	1050	1105	675	715	850	775	590	625	810

1967	1968	1969	1970
885	1025	1150	925

Rank ordered technique

Solution:

Flow (m^3/s)	Flow (Ascending)	Rank (m)	%ge of time exceeded $\frac{N+1-m}{N} \times 100$
905	590	1	100.00
865	625	2	93.33
1050	675	3	86.67
1105	715	4	80.00
675	775	5	73.33
715	810	6	66.67
850	850	7	60.00
775	865	8	53.33
590	885	9	46.67
625	905	10	40.00
810	925	11	33.33
885	1025	12	26.67
1025	1050	13	20.00
1150	1105	14	13.33
925	1150	15	6.67



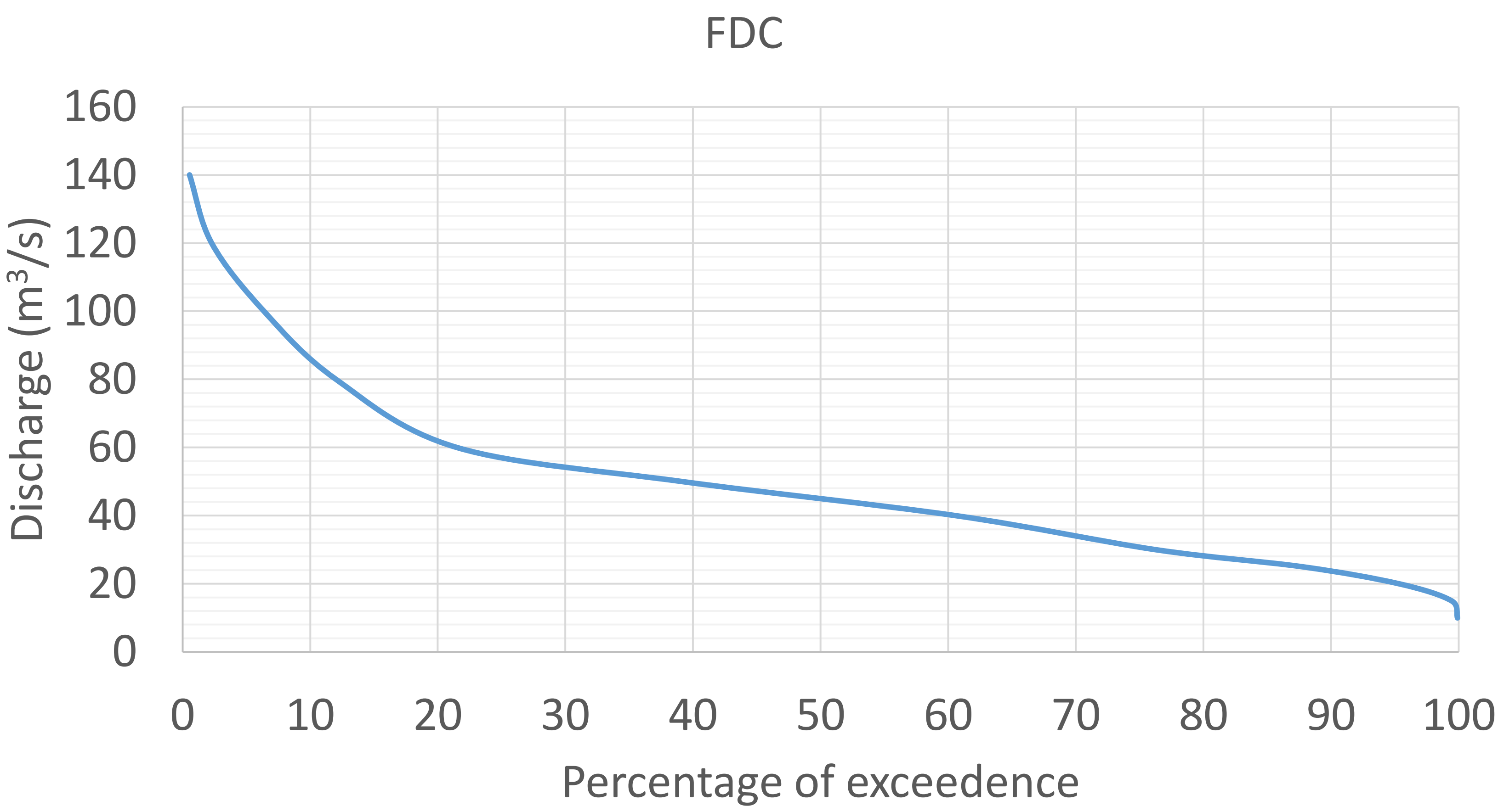
Class-interval technique

- The class-interval technique 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.

Class-interval technique

Daily mean discharge (m^3/s)	No. of days flow in each class interval			Total of columns 2, 3, 4		$P_p = \frac{m}{N+1} \times 100\%$
	1961-62	1962-63	1963-64	1961-64	Total m	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
140 - 120.1	0	1	5	6	6	0.55
120 - 100.1	2	7	10	19	25	2.28
100 - 80.1	12	18	15	45	70	6.38
80 - 60.1	15	32	15	62	132	12.03
60 - 50.1	30	29	45	104	236	21.51
50 - 40.1	70	60	64	194	430	39.19
40 - 30.1	84	75	76	235	665	60.62
30 - 25.1	61	50	61	172	837	76.30
25 - 20.1	43	45	38	126	963	87.78
20 - 15.1	28	30	25	83	1046	95.35
15 - 10.1	15	18	12	45	1091	88.45
10 - 5.1	5	--	--	5	1096	99.91
Total	365	365	366	N = 1096		

Class-interval technique



Extrapolation of flow duration data to ungauged sites

- If the project site have no recorded data (ungauged site), the representative data must be estimated from nearby sites having similar geomorphology.
- There are several methods to estimate flows from ungauged catchments :
    - Regional frequency analysis,
    - Sequential flow analysis and
    - Use of parametric flow duration curve etc.
  - *For detail please refer "Engineering Hydrology" course*

Characteristics of Flow Duration Curves

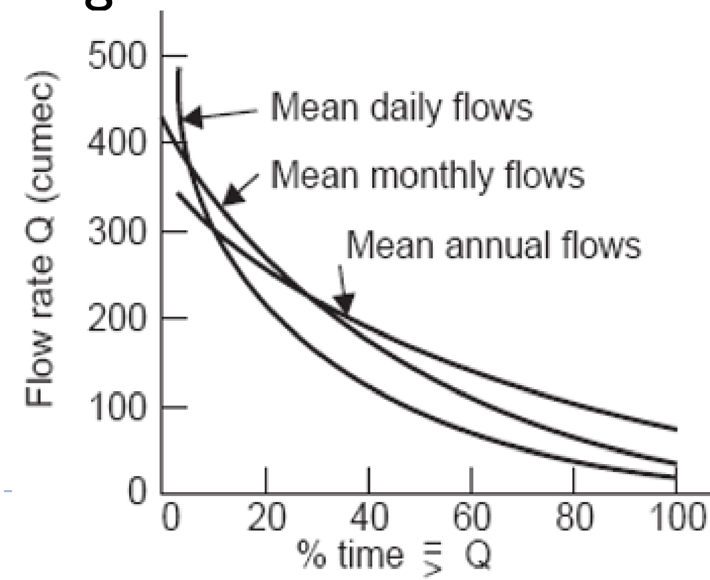
- The flow duration curve (FDC) shows how flow is distributed over a period (usually a year).
- A steep flow duration curve implies a flashy catchment – one which is subject to extreme floods and droughts.
- Factors which cause a catchment to be flashy are:
  - Rocky, shallow soil
  - Lack of vegetation 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).

Characteristics of Flow Duration Curves

- Steep FDC is bad for hydropower development, especially for run-of-river type.
- A flat FDC 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 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

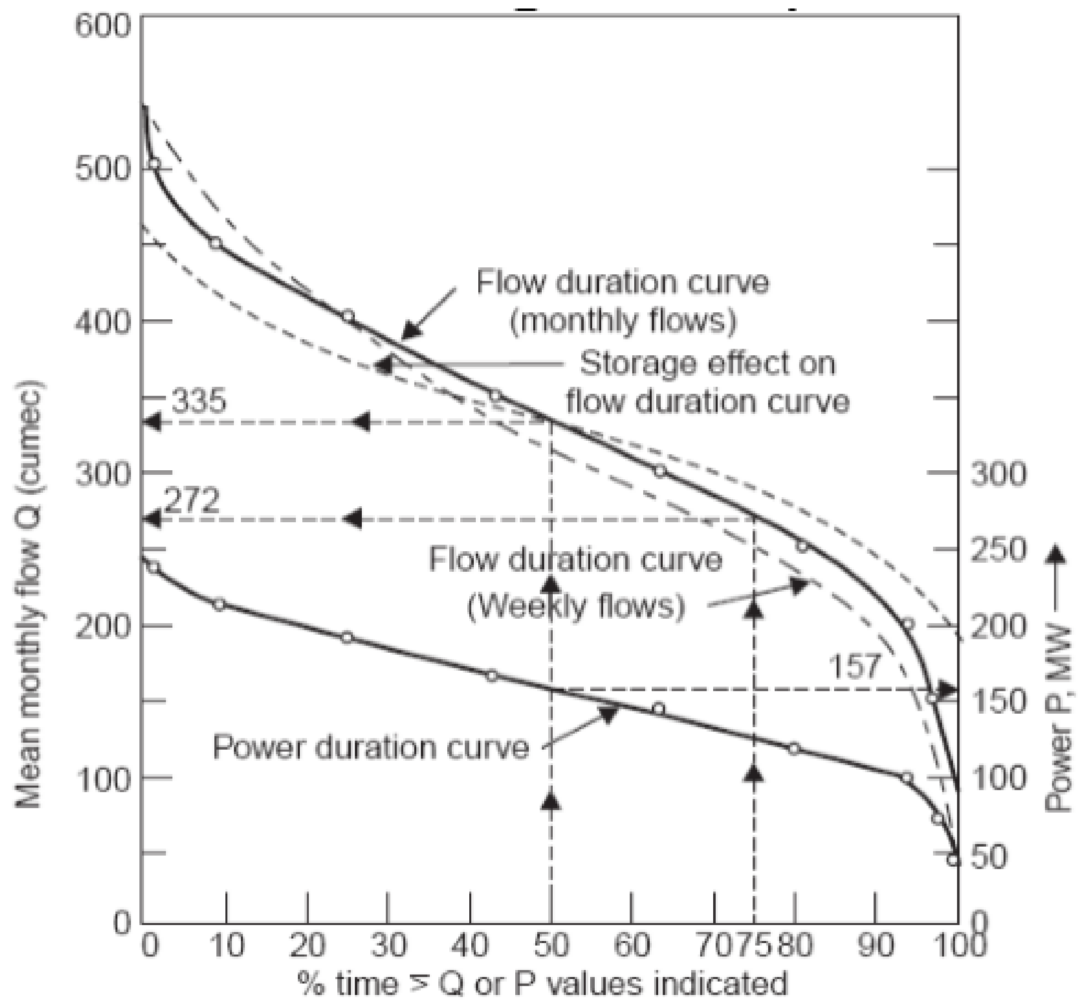
Characteristics of Flow Duration Curves

- The selection of the time interval for FDC depends on the purpose of the study.
- The 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
- As the time interval increases the range of the curve decreases (see Fig.).



Characteristics of Flow Duration Curves

- 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 Fig. i.e., reducing the extreme flows and increasing the very low flows.



Characteristics of Flow Duration Curves

- FDC, very often, plotted using the average monthly values of the flow.
- The capacity estimate for firm power is then made by using the entire recorded flow data and plotting in a single FDC.
- In such a case two different methods are in use.
  - The total period method, and
  - The calendar year method.
- The total period method gives more correct results than the calendar year method which averages out extreme events.

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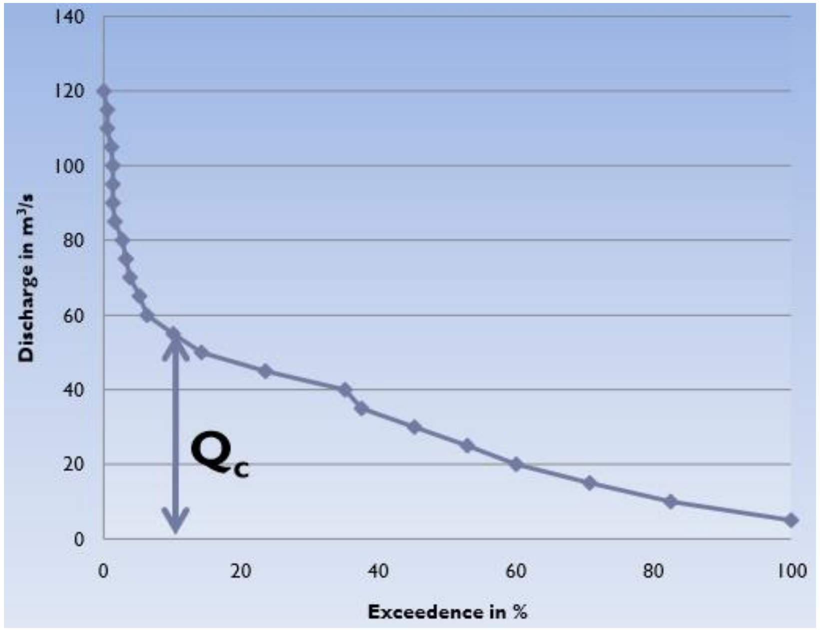
- The total period method gives more correct results than the calendar year method which averages out extreme events.

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.

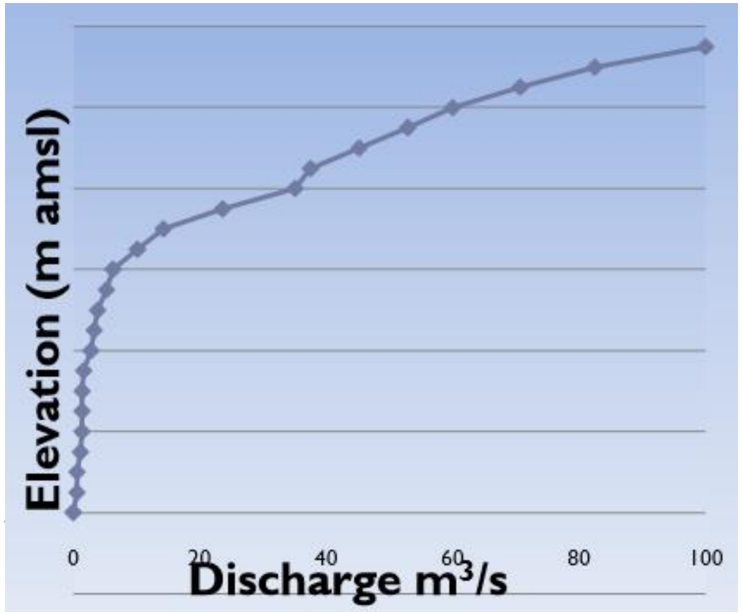


- A FDC is used to explain discharge capacity (Q_c) as labeled in the Figure. Even though to the left of that point on the duration curve the stream discharge is greater, it is not possible to pass the higher discharges through the plant.

Discharge capacity of a plant

- 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.

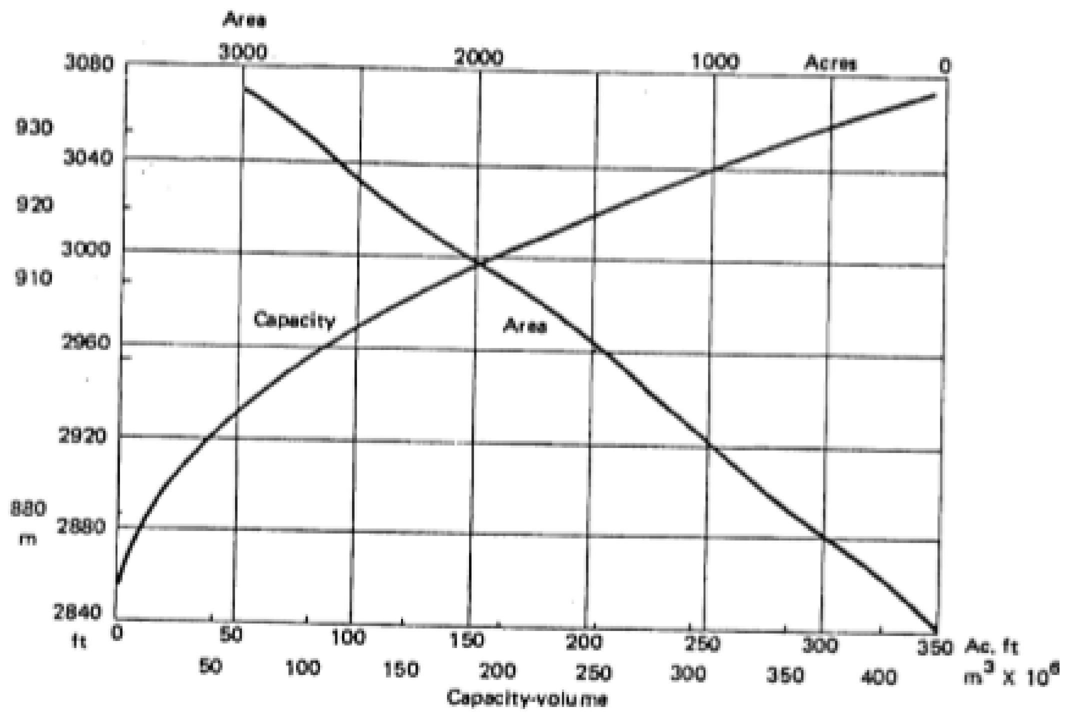


Discharge capacity of a plant

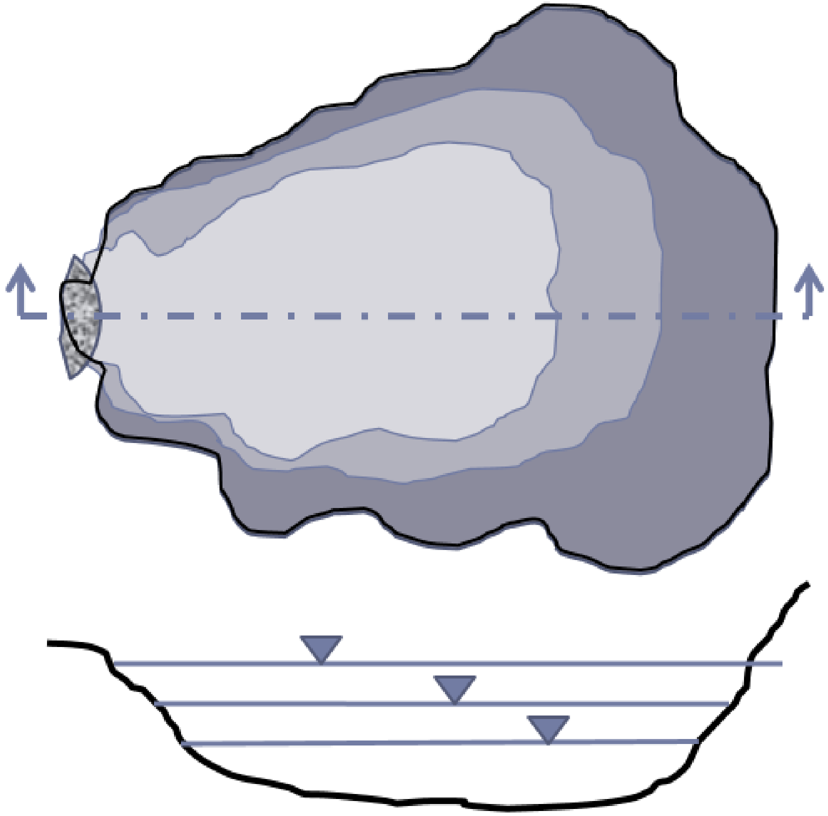
- Most hydropower developments involve an impoundment behind a dam.

- 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 elevation



Area capacity curves



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.

- This loss in warmer climate is considerable.

- The deeper the depth and the narrower the surface area the fewer the evaporation loss

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.

Energy and Power Analysis Using Flow Duration Approach

Heng-3161 Hydropower Engineering I

Power duration curve

- Remember: From previous section, $P = \rho_w Qgh$ ---- *theoretical power*

- 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 (η), usually called overall efficiency, must be introduced to give the standard power equation:

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

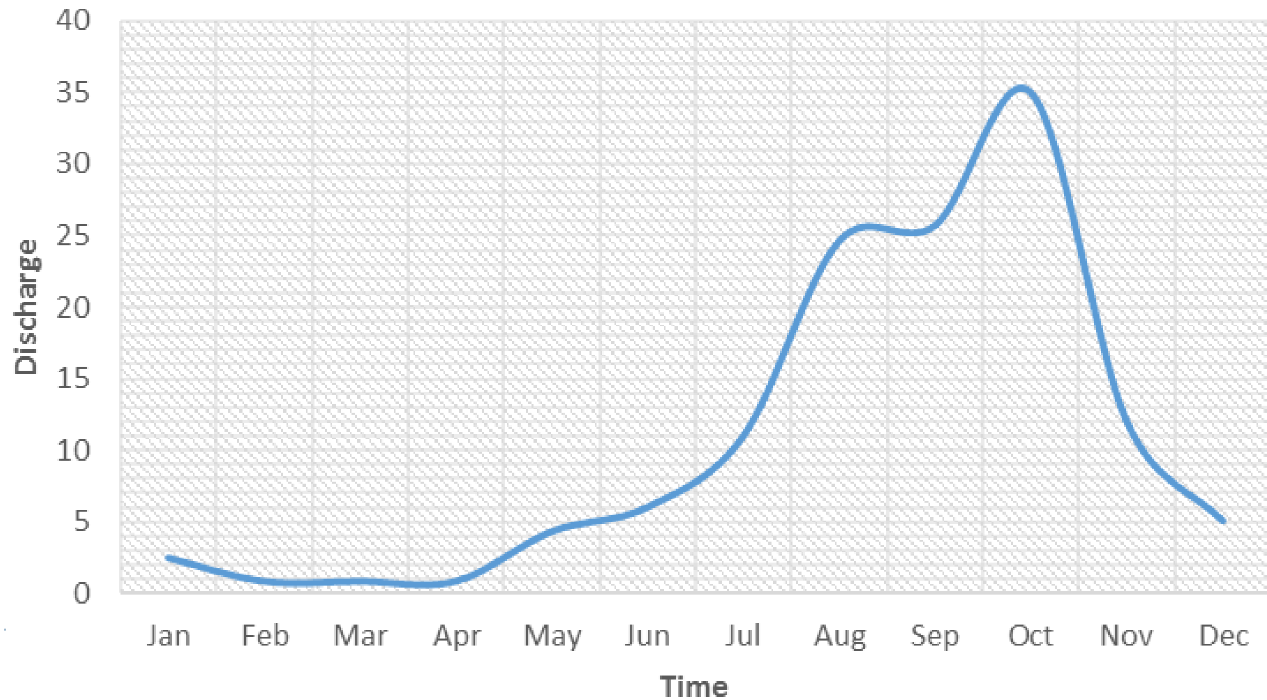
- If hydraulic head and the expected losses in the penstock are known, it is possible to generate a power duration curve from the flow duration curve. How?

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

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

Power duration curve

- 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.



Power duration curve

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

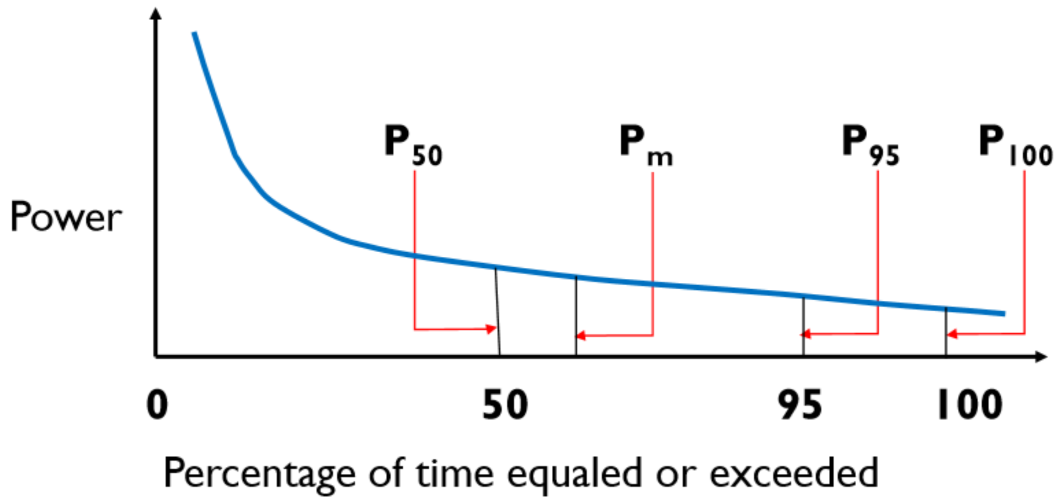
- **Power duration curve** can also be plotted since power is directly proportional to the discharge and available head.

- Discharge/Power duration curve indicates discharge or power available in the stream for the given percentage of time.

Power duration curve

- 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.

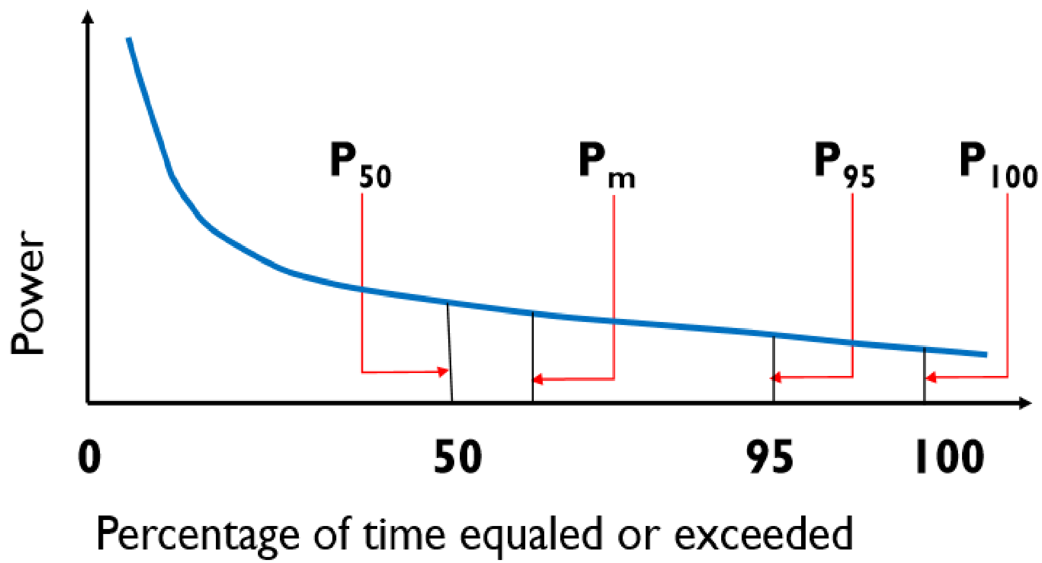
Power duration curve



- Minimum potential power computed from the minimum flow available for 100% of the time (365 days or 8760 hours). This is represented as P_{100}
- Small potential power computed from the flow available for 95% of time (flow available for 8322 hours). This is represented as P_{95}

Power duration curve

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



- Mean potential power 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' and is represented as Pm.

Power duration curve

- 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};

$$P_a = 0.56P_{50}$$

Power duration curve

- 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. EEC puts this factor to be about 0.75 or 0.80, i.e.

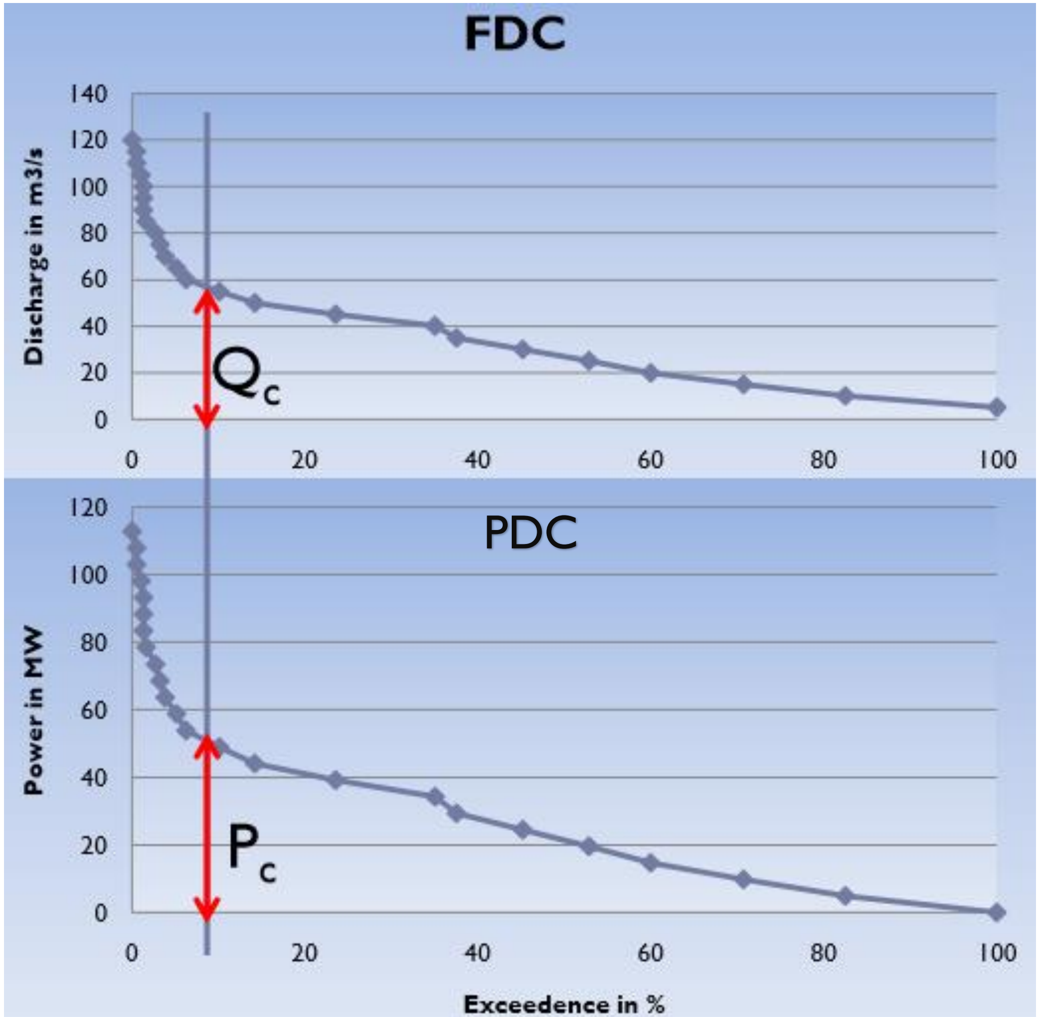
$$P_{m\ net} = (7.4 \text{ to } 8.0) Q_m h$$

Where Q_m is the arithmetic mean discharge

- The maximum river energy potential is given as

$$E_{\max net} = 8760 P_{m\, net} \ \ [kwh]$$

Example 1



- What is wrong with the PDC?
- 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 PDC multiplied by an appropriate conversion factor.

Example 2

- The following is the record of average yearly flow (m^3/s) in a river for 15 years. If the available head is 15m, construct the FDC and PDC for the river.

Year	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970
Flow	905	865	1050	1105	675	715	850	775	590	625	810	885	1025	1150	925

Solution:

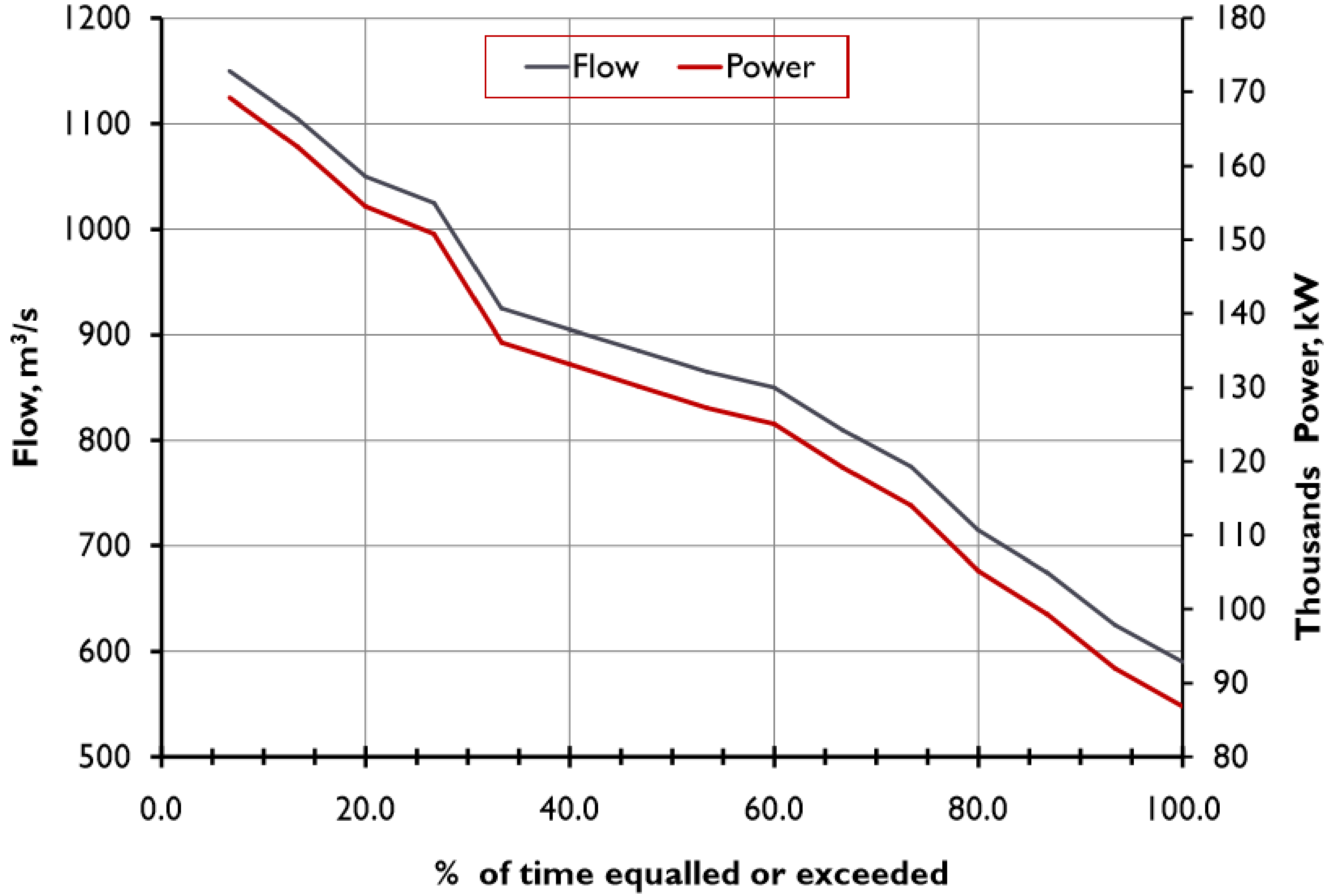
- The yearly flow values are arranged in ascending order (see table below). The power corresponding to each flow values are calculated assuming the head (=15 m) to be constant. Then, FDC and power duration curves are plotted on the same graph.

Example 2

Q (Ascending)	Rank (m)	Power $[P = 9.81 \times Q \times h]$	%ge of time exceeded $\left[\frac{N+1-m}{N} \times 100\right]$
590	1	86819	100
625	2	91969	93
675	3	99326	87
715	4	105212	80
775	5	114041	73
810	6	119192	67
850	7	125078	60
865	8	127285	53
885	9	130228	47
905	10	133171	40
925	11	136114	33
1025	12	150829	27
1050	13	154508	20
1105	14	162601	13
1150	15	169223	7

Power duration curve

Example 2



Example 3

- A turbine installation developing 7.5 MW under 27.5 m head with an overall efficiency of 0.83 is to be supplied from a reservoir. The estimated runoff for 12 consecutive months each 30 days (in Million Cusecs) were: 96.2, 101.8, 86.3, 74.9, 67.9, 80.6, 113.2, 90.5, 86.3, 93.4, 99.0, and 89.1.

- Assume the reservoir is full at the beginning of the first month.

  a. Determine the minimum capacity of the reservoir to ensure the required demand

  b. Find the discharge wasted

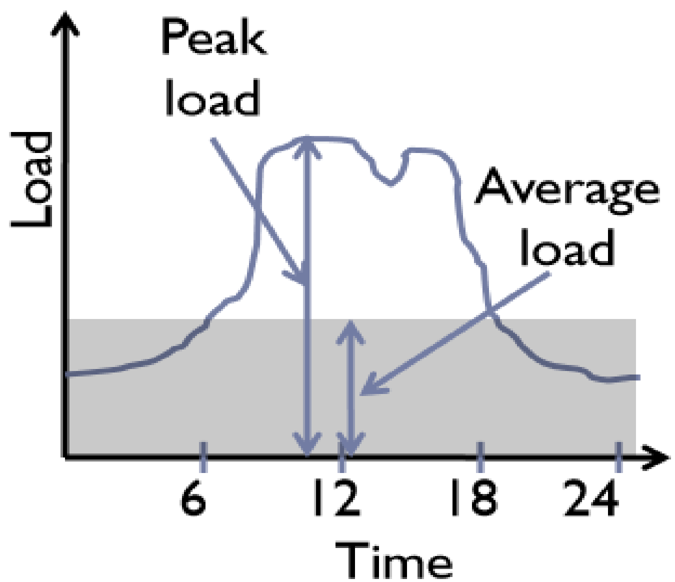
Power duration curve

Example 3

Q (10^6 m^3/ month)	Demand (10^6 m^3/ month)	Surplus (10^6 m^3/ month)	Inflow to resevoir	Deficit (10^6 m^3/ month)	Wastage (10^6 m^3/ month)	Total Wastage
96.2	86.8	9.4	0	0	9.4	9.4
101.8	86.8	15	0	0	15	24.4
86.3	86.8	-0.5	-0.5	0.5	0	24.4
74.9	86.8	-11.9	-11.9	12.4	0	24.4
67.9	86.8	-18.9	-18.9	31.3	0	24.4
80.6	86.8	-6.2	-6.2	**37.5**	0	24.4
113.2	86.8	26.4	26.4	11.1	0	24.4
90.5	86.8	3.7	3.7	7.4	0	24.4
86.3	86.8	-0.5	-0.5	7.9	0	24.4
93.4	86.8	6.6	6.6	1.3	0	24.4
99	86.8	12.2	1.3	0	10.9	35.3
89.1	86.8	2.3	0	0	2.3	**37.6**

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;

Load terminologies

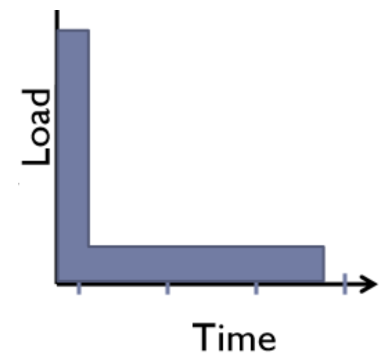
- **Power demand** is defined as the total load, which consumers choose, at any instant of time, to connect to the supplying power system.
- The highest instantaneous value of the demand, strictly speaking, is the peak load or peak demand.
- Generally, peak load is defined as that part of the load carried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

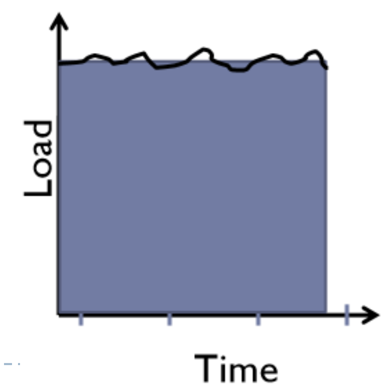
Load Factor

- 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

- 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 or use 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

- 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

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

- For example, if a plant with a capacity of 100MW produces 6,000,000 kWh operating for 100 hours, its capacity factor will be 0.6, i.e.

$$C.F. = \frac{6000000}{100000 \times 100} = 0.6$$

- The capacity factor for hydroelectric plants generally varies between 0.25 and 0.75.

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.

Utilization Factor

- 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.

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 hydropower units, two rated at 10 MW each and one at 5 MW are installed. Determine:

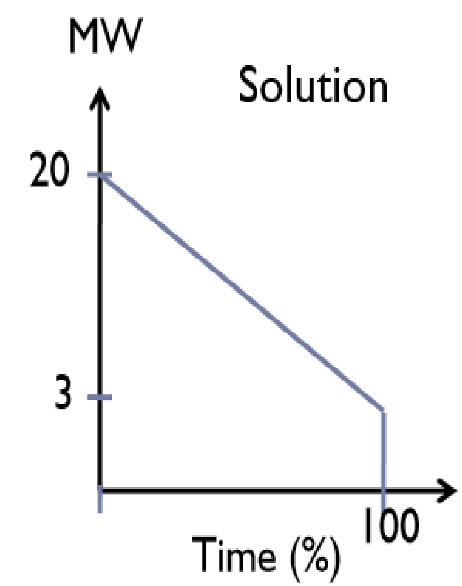
1. Load factor (LF) 2. Capacity factor (CF) 3.Utilization factor (UF)

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

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

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

Solution



Diversion Factor

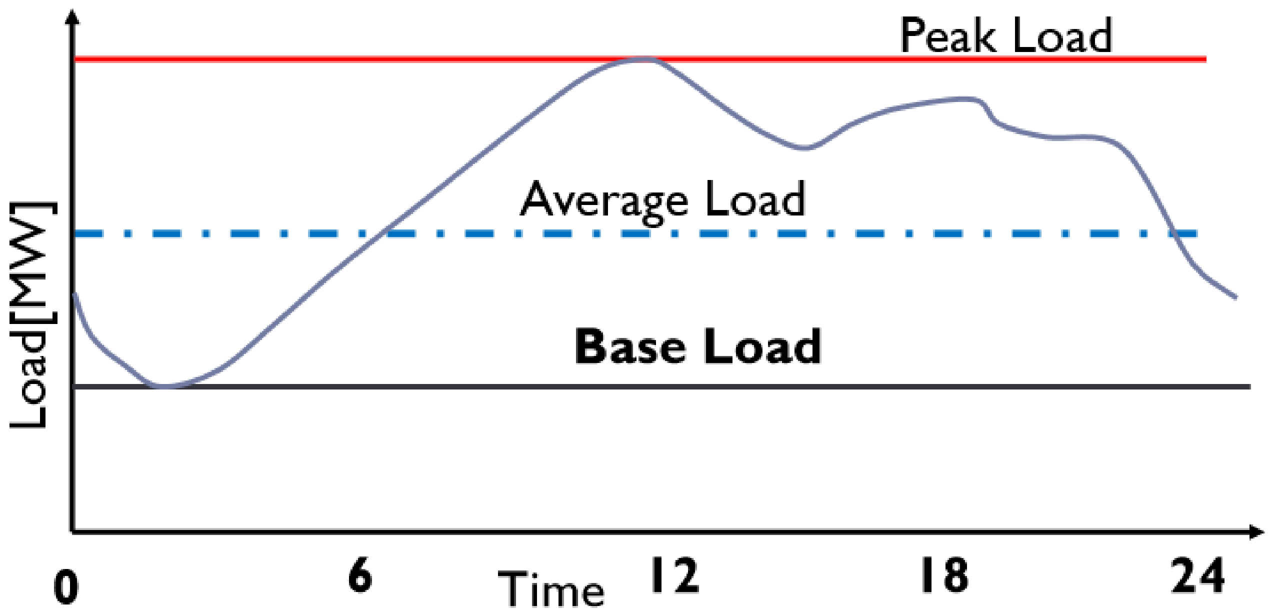
- Diversity factor (DF) is the summation of the different types of load divided by the peak load.
- If there be four different types of load L_1, L_2, L_3 and L_4 and the peak load from the combination of these loads is L_P, then the diversity factor is expressed as:

$$DF = \frac{L_1 + L_2 + L_3 + L_4}{L_p}$$

- Note that the diversity factor has a value which is greater than unity.
- For n load combination: $\dfrac{\sum_{i=1}^{i=n} L_i}{L_p}$
- 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.

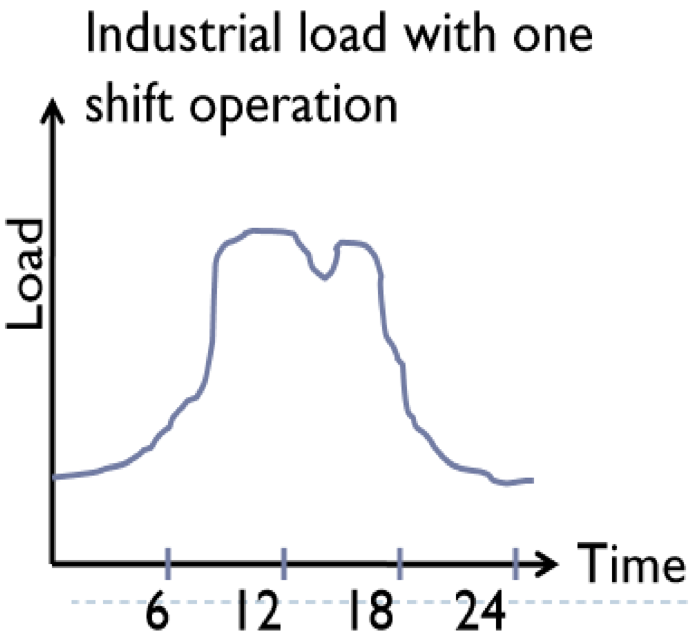
Load Duration Curve

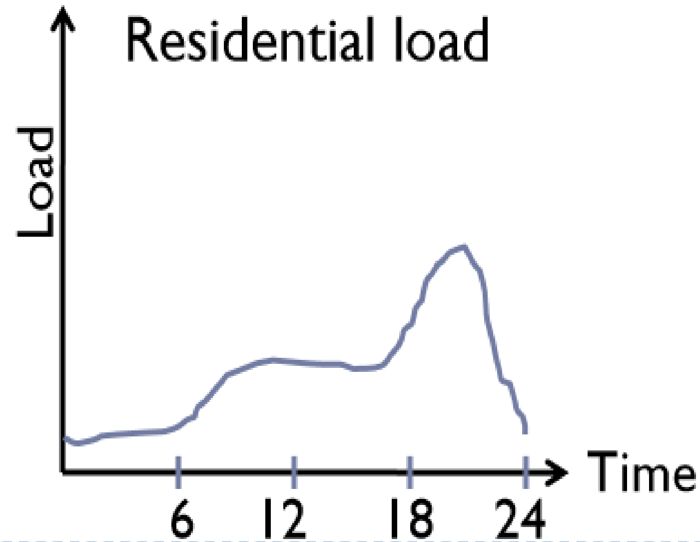
- **Load Curve:** A load curve is a graph of load consumption with respect to time and directly gives an indication of power used at any time (daily, weekly, monthly, annually, etc.)
- **Daily Load Curve** is a curve drawn between load as the ordinate and time in hours as the abscissa for one day.



Load Duration Curve

- 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: $E = \int_{t=1}^{t=24} kw dt$ *Street light?*

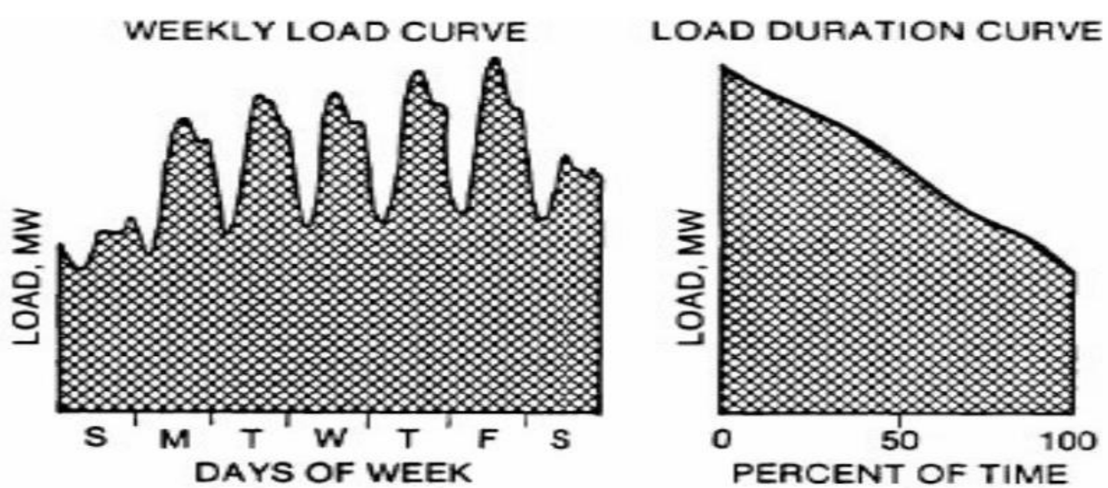




Load Duration Curve

- 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.

- **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

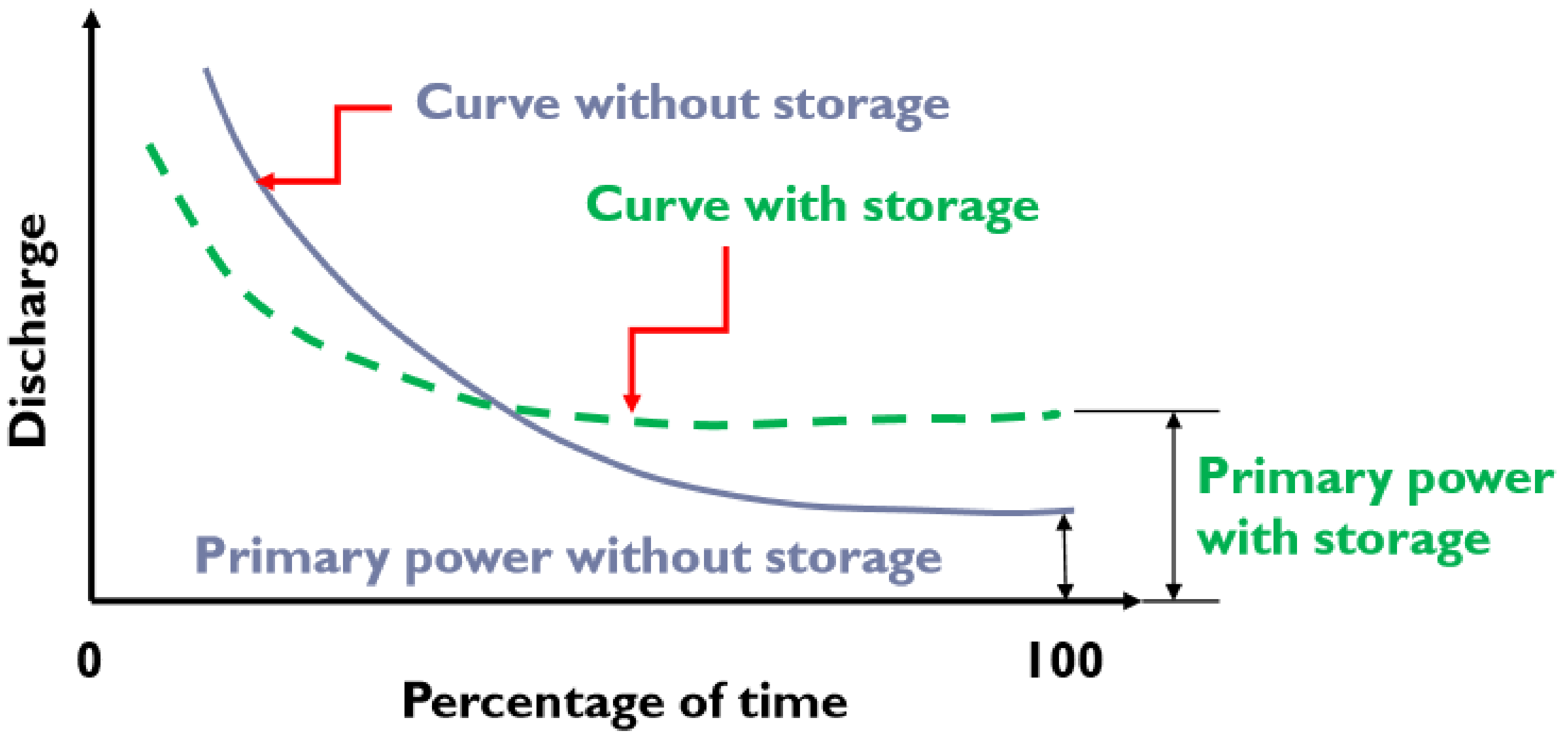


Load Duration Curve

- Fundamentally the load-duration curve is nothing more than a rearrangement of all the load elements of a chronological curve in the order of descending magnitude.
- The area under load duration curve for a given time 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 that 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.

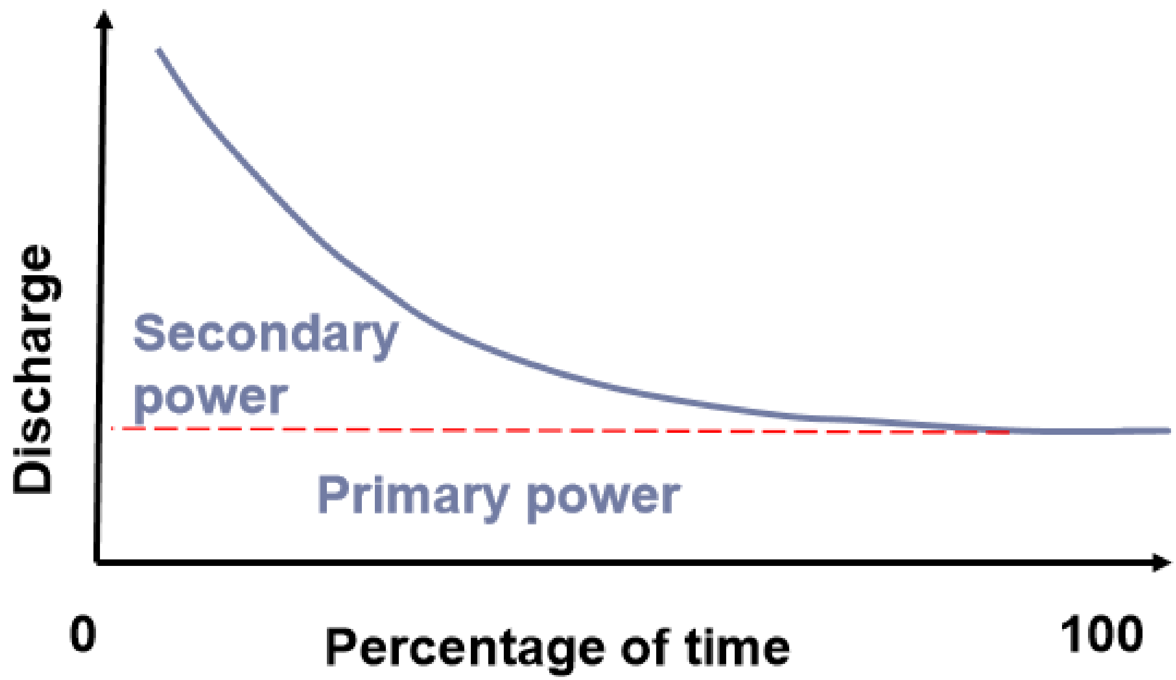
Load Duration Curve

- The firm power could be increased by the use of pondage (storage).



Load Duration Curve

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



Load Duration Curve

- 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 5

- 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 (Next slide). 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.

Solution

- Table: Flow duration analysis of mean monthly flow data of a river in a10 yr period (Example 5)

Mean monthly flow C.I. (m^3/s)	*No. of occurrences (in 10-yr period)*	No. time 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			

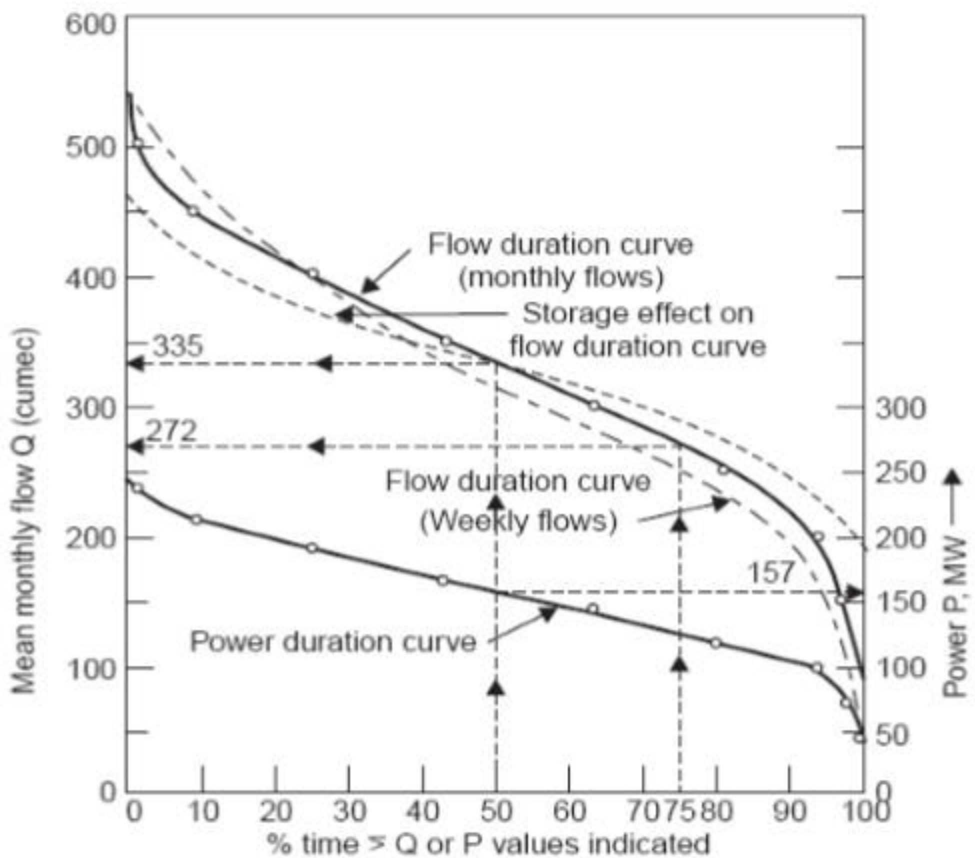
Solution

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 Fig.

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.



Solution

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 Fig. 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.

Pondage

- While storage refers to large reservoirs to take care of monthly or seasonal fluctuations in the river flow, pondage usually refers to the small storage at the back of a weir, in run-of-river plants, for temporarily storing water during non-working hours, idle days and low load periods for use during hours of peak load demand.

- Run-of-river plants are feasible for streams which have a minimum dry weather flow or receive flow as regulated by any storage reservoir upstream.

Pondage

- Pondage is needed to cover the following four aspects:

  a. To store the idle day flow.

  b. For use during hours of peak load.

  c. To balance the fluctuations in the stream flow.

  d. To compensate for wastage (due to leakage) and spillage.

Example 6

- A run-of-river hydroelectric 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

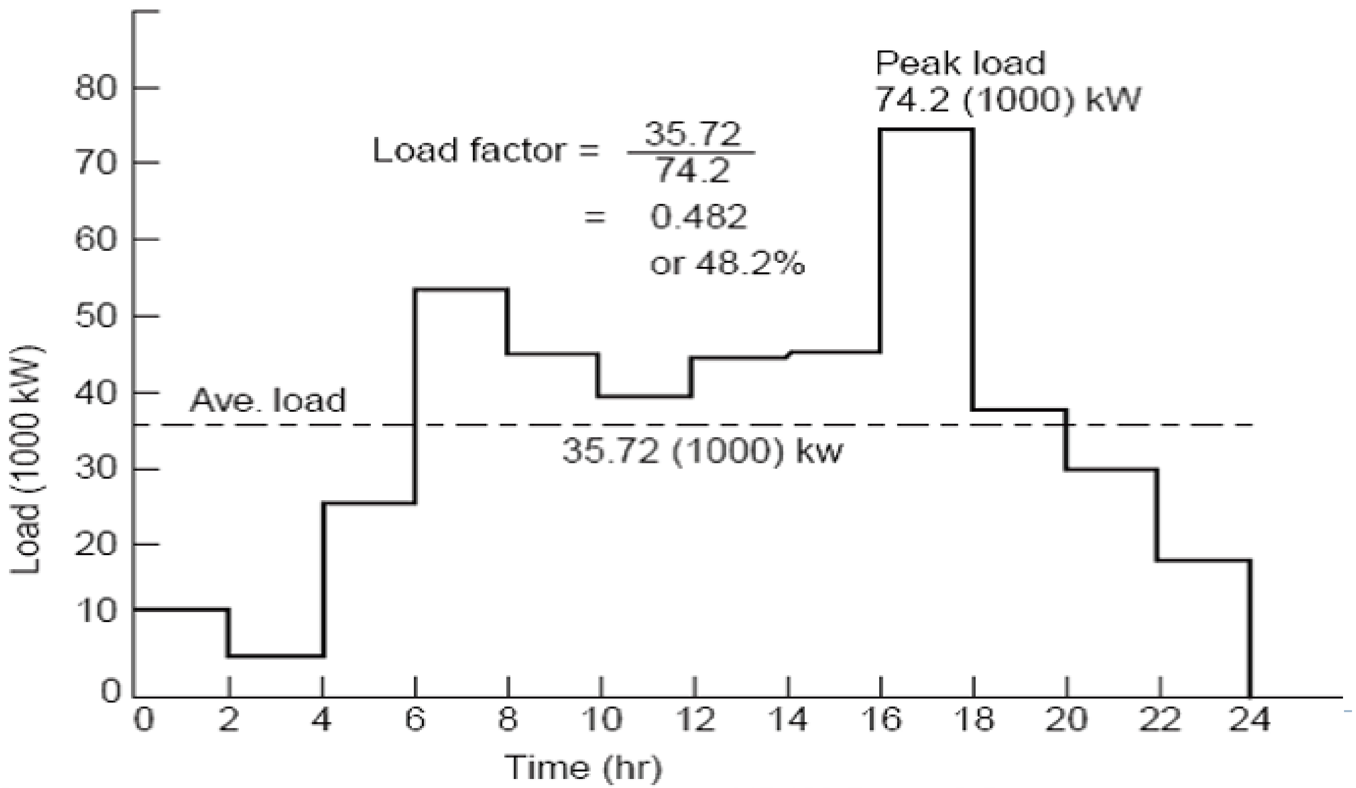
Time (hr)	*Load (1000 kW)*	*Time (hr)*	*Load (1000 kW)*
0-2	11.4	12-14	44.2
2-4	5.6	14-16	44.4
4-6	25.6	16-18	74.2
6-8	53.2	18-20	37.8
8-10	44.8	20-22	30.0
10-12	39.4	22-24	18.0

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:

$$Q = 35.72\text{MW}/(9.81 \times 22 \times 0.8) = 207\ \text{m}^3/\text{s}$$



Solution (ii)

- Flow required to produce the required load/demand at each time interval is given by:
- Q = P in 1000 kW/(9.81x22x0.8) = 5.8 x Load in 1000 kW
- To determine the pondage capacity the table in the next slide is prepared. From the table:

Total deficiency = 511.4 m^3/s

Therefore, pondage capacity required = 511.4 m^3/s for 2 hrs

= 511.4 x (2 x 60 x 60)

= 3.68 x 10^6 m^3 or 3.68 Mm^3

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

$$LF = \frac{35.72}{74.2} = 0.482$$

Load Duration Curve

Solution (ii)

Time (hr)	Load (MW) P	Required Flow (m^3/s) 5.8*P	Average Flow (m^3/s)	Deficiency (m^3/s)	Excess (m^3/s)
0-2	11.4	66.12	207		140.88
2-4	5.6	32.48	207		174.52
4-6	25.6	148.48	207		58.52
6-8	53.2	308.56	207	101.56	
8-10	44.8	259.84	207	52.84	
10-12	39.4	228.52	207	21.52	
12-14	44.2	256.36	207	49.36	
14-16	44.4	257.52	207	50.52	
16-18	74.2	430.36	207	223.36	
18-20	37.8	219.24	207	12.24	
20-22	30	174	207		33
22-24	18	104.4	207		102.6
Total	428.6			**511.4**	**509.52**

Example 7

- A run-off-river plant operates as a peak load plant with 20% weekly load factor, and all its capacity is firm capacity.
  - What will be the minimum flow in the river so that the station may serve as a base load station given that:
    - Installed capacity of generator = 10,000 KW
    - Operating head = 15m
    - Plant efficiency = 80%
  - Estimate the daily load factor of the plant if the stream flow is 15m^3/s.

Solution

- When the plant operates as a peak load with 20 load factor:

  The total energy generated for a week =
  $10{,}000 \times 0.2 \times (7 \times 24)$ Kwh = $\underline{33.6 \times 10^4}$ *[Kwh]*

- If Q is the minimum flow necessary for the plant as a base load

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

Power for the week, $P = 0.8 \times 9.81 \times 15 \times Q = 117.72Q$ *[Kw]*

Energy for the week $= 117.72Q \times (7 \times 24) = \underline{1.98 \times 10^4 Q}$

$$Q = \frac{33.6 * 10^4}{1.98 * 10^4} = 16.99 m^3 / s$$

Solution

- Power when the $Q=15m^3/sec$ is:
- $P=117.6\times Q=117.6\times 15=1764Kw$
- Total unit generated in 24 hrs $= 1764Kw\times 24hrs$
  $= 42,336Kwh$

$$\text{Hence, the daily load factor} = \frac{42336Kwh}{1000Kw * 24h}$$

$$= 17.65\%$$

Solution

OR

- For peak load station:

Installed capacity = Peak load capacity = 10Mw

Average Load = LF * Peak load = 0.2 *10 = 2MW

$Q = (2 * 10^6)/(0.8 * 9810 * 15) = 16.99 m^3/s$

Power when the stream flow is $15 m^3/s$ is:

$P = \eta \gamma h Q = 0.8 * 9810 * 15 * 15 = 1.7658MW$

$$\text{Hence, the daily load factor} = \frac{1.766}{10} = 17.65\%$$

Example 8

- Calculate the capacity factor of the hydropower plant having the following data:
  - $Q = 50\ m^3/s$
  - $H = 5\ m$
  - Operation hour = 7,000h/yr.
  - Overall efficiency = 0.8

Solution

Power, $P = \eta\ \rho_w\ g\ h\ Q = 0.8 \times 9810 \times 5 \times 50 = 1.962Mw$

Energy = P × time =1.962Mw×7000hr =13.734 Gwh/yr

Energy @ full Capacity = 1.962Mw×8760hr = 17.187Gwh/yr

CF = 13.734/17.187 = 79.91%

Example 9

- In an electrical district the followings are some of the different kinds of demand:-
  - Domestic = 3MW
  - Irrigation = 5MW
  - Water supply = 2MW
  - Industries =? L_P
  - Offices = 2MW
  - Public utilities = 4MW
- Calculate the required installed capacity of power station. Assume 15% as loss and 25% of capacity as reserve. Assume diversity factor as 1.5.

Example 9

$$DF = \frac{\sum Li}{Lp}$$

$$1.5 = \frac{3 + 5 + 2 + Lp + 2 + 4}{Lp}$$

L_P = 32MW

- For initial assumption, the installed capacity is the same as the peak demand required. Therefore installed capacity = 32 Mw
- Loss = 15% * 32 = 4.8Mw
- Then the capacity = 32+4.8 = 36.8Mw
- Reserve power = 25% * 36.8 = 9.2Mw
- Total installed capacity = 36.8 + 9.2 = 46MW

Example 10

- 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
$\dfrac{\text{Load during period)}}{\text{24-hr average load}}$	0.6	1.4	1.5	0.5

Example 10

- The other relevant data are given below:
  - Head at full pond level = 16m
  - Maximum allowable fluctuation of pond level = 1m
  - Plant efficiency = 80%
  - Pondage to cover inflow fluctuations = 20% of average daily flow
  - Pondage to cover wastage and spillage = 10%
- Determine:
  - The average load that can be developed
  - Daily load factor
  - Plant capacity
  - Weekly energy output
  - Pondage required

Solution

i. To calculate the average load that can be developed

- First, 7 days flow has to be used in 6 days Therefore, average flow available for power development/to the turbine

$$Q = 30 \times 7/6 = 35 \text{ m}^3/\text{s}$$

- Since maximum allowable fluctuation of pond level is 1m,

$$\text{Average Head} = \frac{16 + (16 - 1)}{2} = 15.5\text{m}$$

- The average load that can be developed
- P = 0.8 * 9810 * 35 * 15.5 = 4.27MW

Solution

ii. The daily load factor

$$LF = \frac{\text{Average Load}}{\text{Peak Load}} = \frac{1 * \text{Average Load}}{1.5 * \text{Average Load}} = \frac{1}{1.5} = 0.67$$

iii. Plant capacity By assuming no reservation of power:

Peak load = plant capacity

Hence, plant capacity = 1.5 * 4.27 = 6.4Mw

iv. Weekly energy output

Weekly energy output = Average load in kW x No. of working hours

= (4.27 x 1000) x (6 x 24)

= 6.15×10^5 kWh

It should be noted that the installed capacity has to be equal to the peak load and the number of units (kWh) generated will be governed by the average load.

Solution

v. Pondage required

a) To store the idle day's flow = 30 x 24 x 3600

= 2.592×10^6 m^3 = 2.592Mm^3

b) To store the excess flow during low loads to meet the peak load demand.

Since power developed is proportional to discharge (assuming constant average head of 15.5 m), flow required during peak load periods of 6:00 to 12:00 hrs. is

= (1.4 – 1) 35 m^3/s & from 12:00 to 18:00 hrs is (1.5 –1) 35 m^3/s

Therefore, pondage to meet the peak load demand

= (0.4 + 0.5) x 35 m^3/s for 6 hrs = (0.9 x 35) x (6 x 60 x 60)
= 6.81×10^5 m^3 = 0.681 Mm^3

Solution

c) Pondage to cover inflow fluctuations = (0.2*30)*(24*3600)

$= 0.518 Mm^3$

The total pondage of a, b and c

$= 2.592 + 0.681 + 0.518 = 3.791\ Mm^3$

d) Adding 10% of wastage and spillage

$= 0.1*3.791 = 0.3791\ Mm^3$

e) Hence, the total pondage required

$= 3.791+0.3791 = 4.170\ Mm^3$

Load Prediction

- For the installation of a new power plant or for the expansion of the existing power plant, it is necessary to estimate the total amount of load that would be required to be met for various purposes.

- Load forecasting may be done either for:
  - Short-term (< 5 years) – operation & planning
  - Medium-term (around 10 years) – expansion
  - Long-term (> 20 years) periods.

Load Prediction

- Basic load forecasting techniques:
  - Trend analysis
  - End-use analysis
  - Econometric analysis

Load Prediction

- Each forecasting method is distinctive in its handling of the four basic forecast ingredients:
  1. The mathematical expressions of the relationship between power demand and the factors which influence or affect it – the functions
  2. The factors which actually influence the power demand (popln., income, price, etc.) – the independent variables
  3. Power demand itself – the dependent variables
  4. How much power demand changes in response to population, income, price, etc., changes – the elasticities.

Trend Analysis

- Trend analysis extends past growth rates of power demand into the future. It focuses on past changes or movements in demand and uses them to predict future changes in the demand.
- Advantage:- simple, quick and inexpensive to perform. It is useful when there is no enough data to use more sophisticated methods or when time and funding do not allow for a more elaborate approach.
- Disadvantage:- it produces only one result (future power demand). It doesn‘t help analyze why power demand behaves the way it does, and it provides no means to accurately measure how changes in energy prices or government policies, for instance, influence the demand.

End-Use Analysis

- Concept: the demand for power depends on what it is used for (the end-use). For instance, by studying historical data to find out:

  [(Power used * No. of appliance) * Projected No of appliance] * Projected No. of houses

  Similarly for other industries…..And then sum-up.

- Advantage: it identifies exactly where power goes and how much is used for each purpose.
- Disadvantage: it assumes a constant relationship between power and end-use, for example, power used per appliance.* End-use analysis also requires extensive data.

Econometric analysis

- Uses economics, mathematics, and statistics to forecast power demand.
- Uses complex mathematical equations to show past relationships between demand and the factors which influence the demand.
- For instance, an equation can show how power demand in the past reacted to population growth, price changes, etc.
- For each influencing factor, the equation can show whether the factor caused an increase or decrease in a power demand.
- The equation is then tested and fine tuned to make sure that it is a reliable a representation as possible of the past relationships.

Econometric analysis

- Once this is done, projected values of demand-influencing factors (population, income, prices) are put in to the equation to make the forecast.
- Advantage: it provides detailed information on future levels of power demand, why future power demand increases or decreases, and how power demand is affected by all the various factors. In addition, it is flexible and useful for analyzing load growth under different scenarios.
- Disadvantage: the assumption that the changes in the power demand caused by changes in the factors influencing that demand remain the same in the forecast period as in the past. However, this constant elasticity assumption is hard to justify in reality.

Chapter 5
Water Conveyance Structures

HEng-3161 Hydropower Engineering I

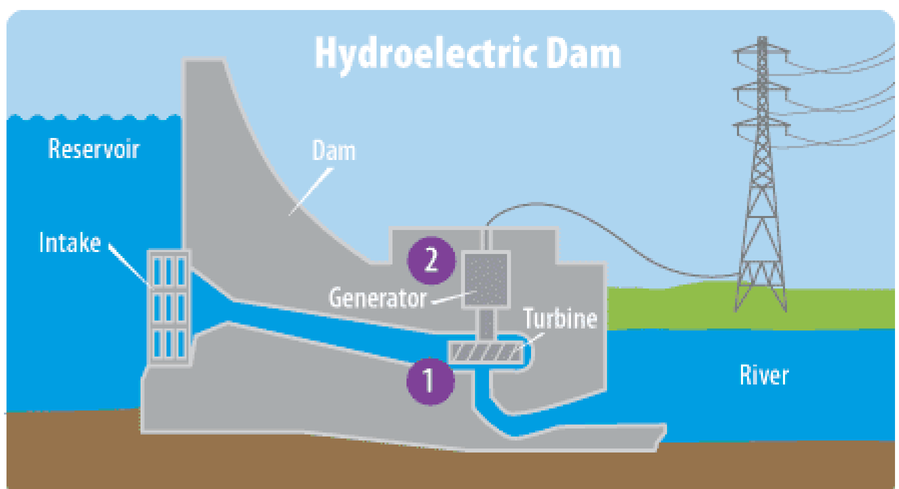
Contents of chapter five

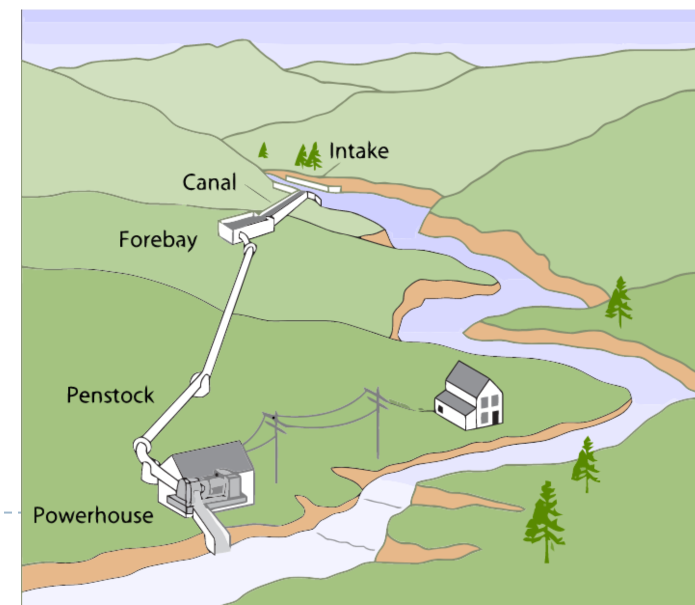
- Intakes and Head Race
- Water Conveyance System
  - Canals and Tunnels
  - Settling Basins
  - Water Hammer Analysis
  - Surge Tanks
  - Forebay
  - Penstock

Intakes and Head Race

Water Intake, Inlet Structures

- The intake is a structure constructed at the entrance of a power canal or tunnel or pipe through which the flow is diverted from the source such as a river or reservoir.
- It is an essential component of hydropower schemes and provided as an integral part or in isolation from the diversion, weir or dam.





Functions of Intakes

The main function are:

- To control flow of water in to the conveyance system. The control is achieved by a gate or a valve.
- To provide smooth, easy and vortex or turbulence free entry of water in the conveyance system which is to minimize head loss. This can be achieved through providing bell-mouth shaped entrance.
- To prevent entry of coarse river born trash matter such as boulders, logs, tree branches etc. Provision of trash racks at the entrance achieve this function.
- To exclude heavy sediment load of the river from interring the conveyance system. Special devices such as silt traps and silt excluders are used to control & trap the silt.

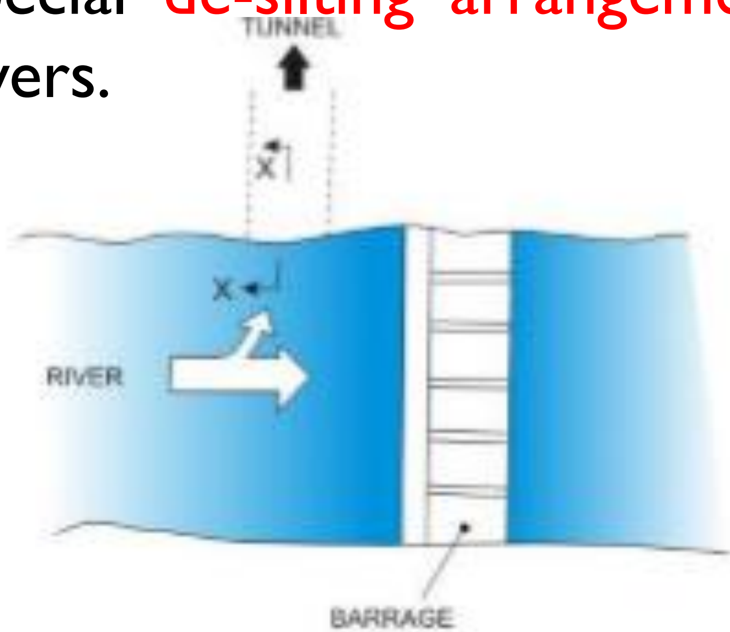
Types of Intakes

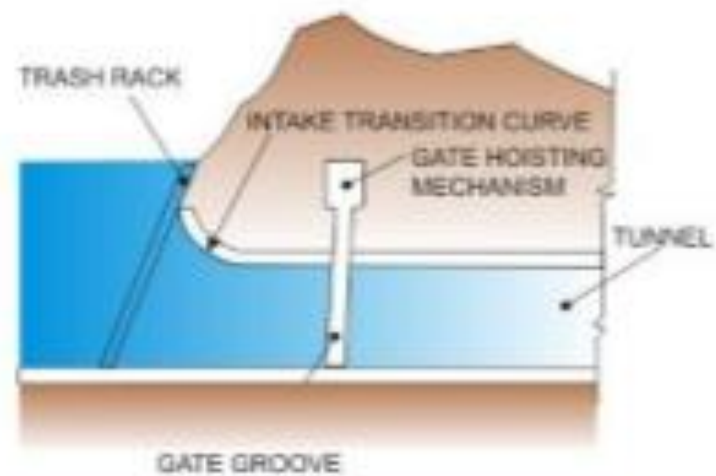
- Intakes are conveniently classified in to the following types depending on the power plant type and its layout.
  - Run - of - river intakes
  - Canal intakes
  - Dam intakes
  - Tower intakes
  - Shaft intakes
  - Intakes of special type

Run - of - river intakes

The component parts are

- Bell mouth entrance guarded by R.C or still grid forming the trash rack structure.
- Control gate situated immediately d/s of the bell mouth entry
- Special de-silting arrangement may be provided in silty rivers.

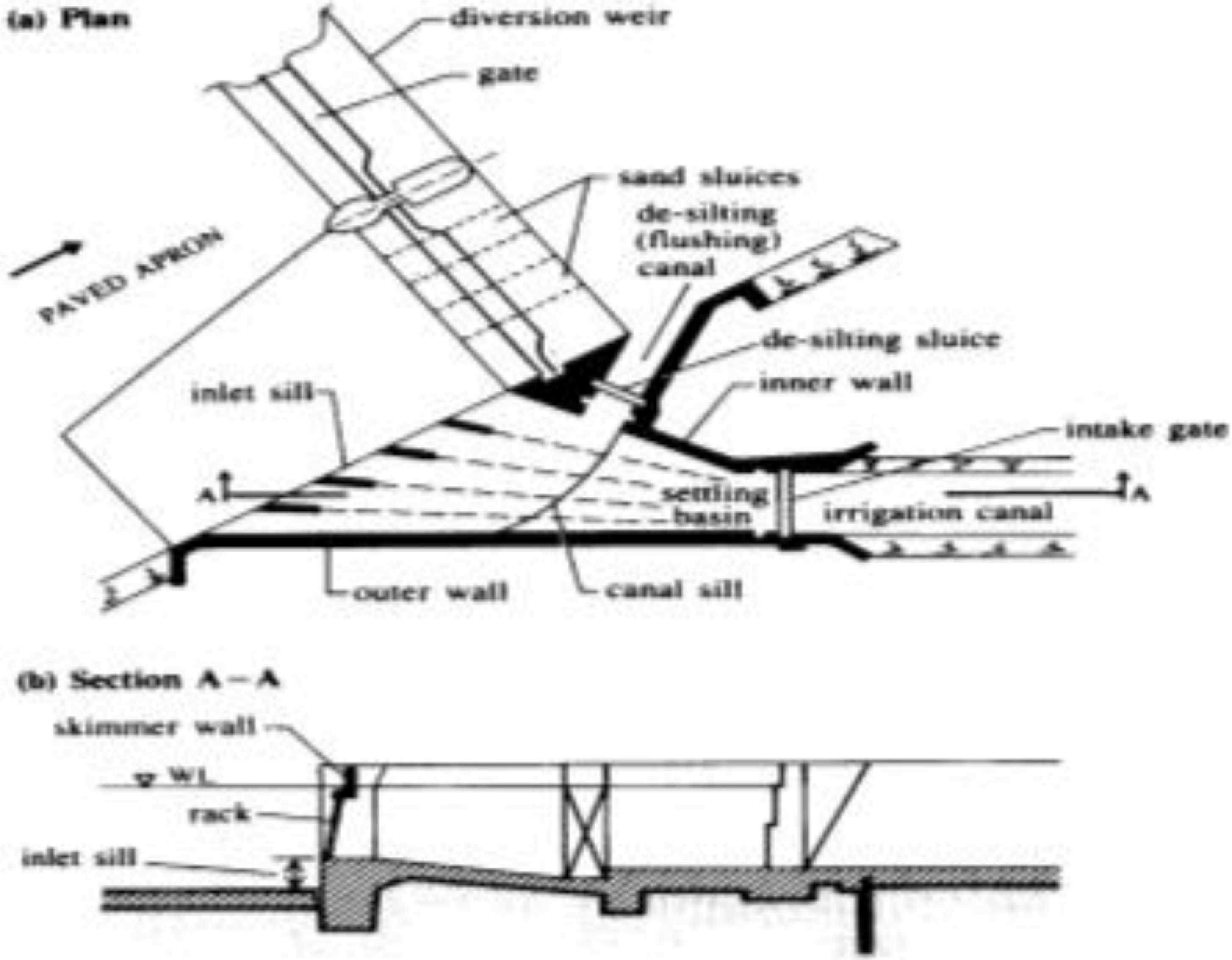




Canal Intakes

- Silt excluders or silt-traps are usually essential components of such intakes.
- The inlet invert level of the intake is raised to form a sill so as to prevent entry of rolling bed load.
- A skimmer wall (a diaphragm which extends below the water surface) abstracts the floating material from interring in to the canal.
- Trash racks are also fitted at the entrance.
- Vertical lift gate with motorized operation are used to control the flow.

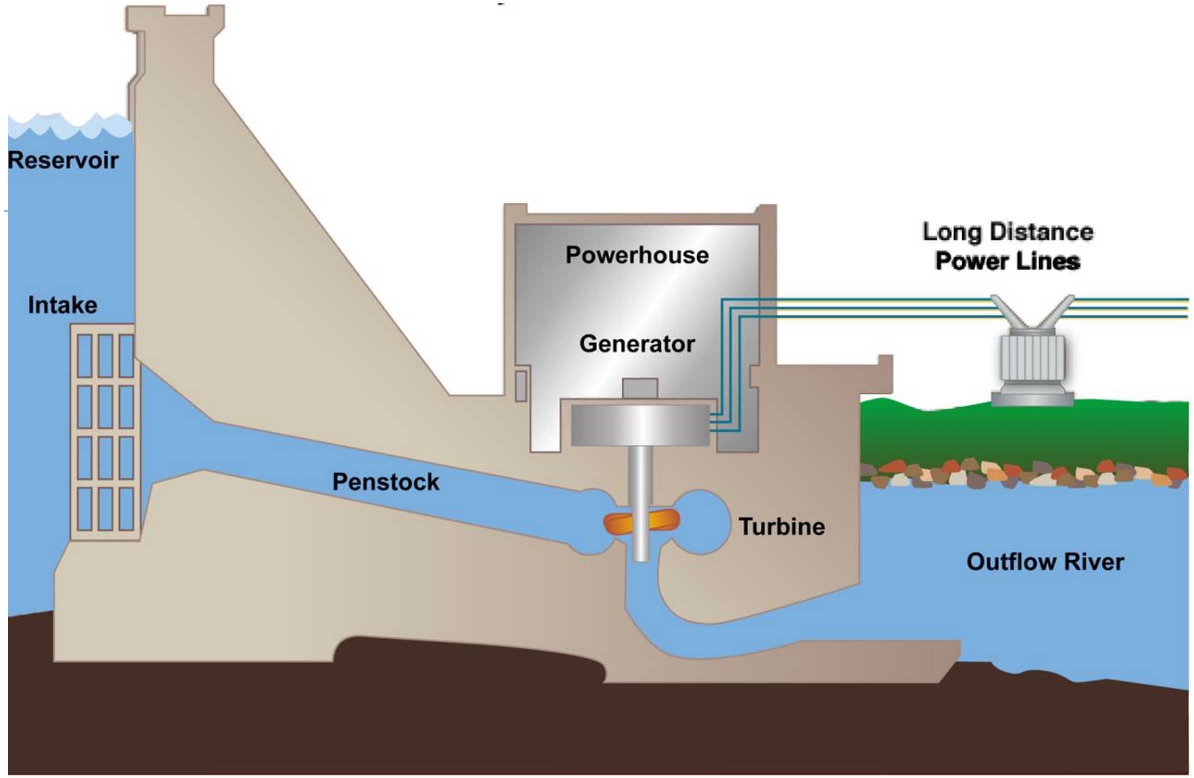
Canal Intakes

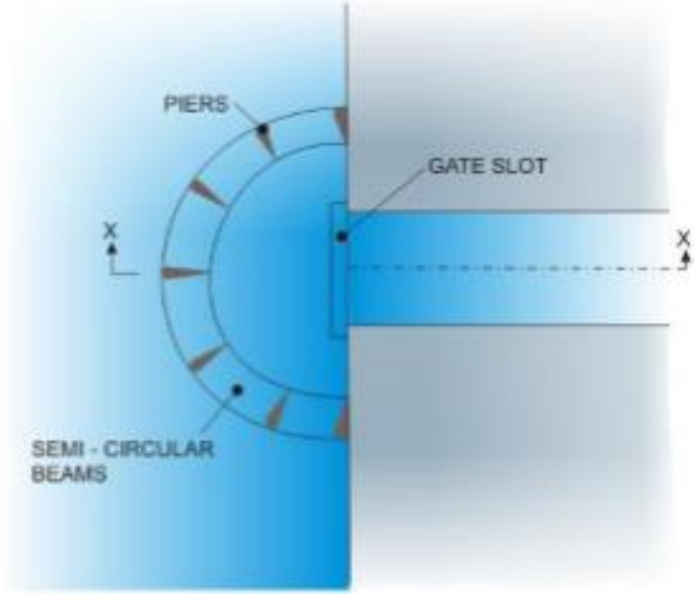


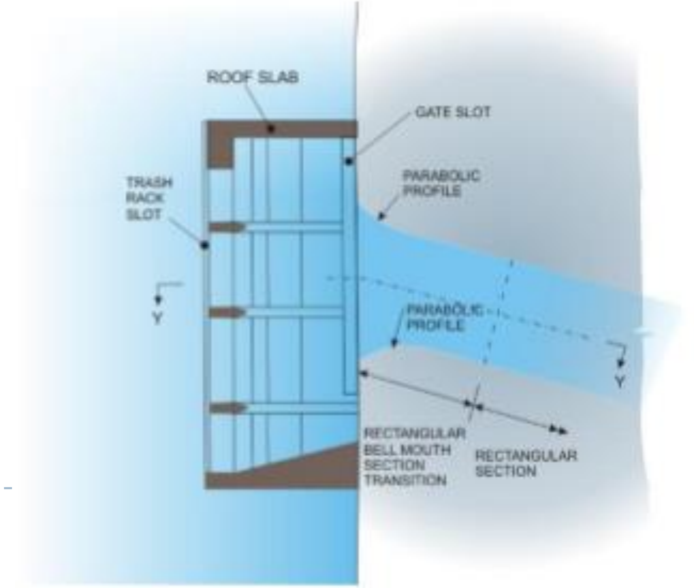
Dam intakes

- For valley dam plants, the intake structure is provided usually in the body of the dam.
- The penstocks are embodied in the dam.
- the main features of such an intake are
  - a trash rack structure in front of the dam.
  - a bell mouth inlet horizontal or inclined alignment
  - a control gate installed either at or after the bell mouth. Cage-shaped intakes resting against the face of the dam and supported on slab cantilevered from the dam provide larger area of entry than the penstock intake area, thus reducing entrance losses.
- Multi-level water are also some times used in dam intakes.

Dam intakes


Reservoir
Intake
Penstock
Powerhouse
Generator
Turbine
Long Distance
Power Lines
Outflow River

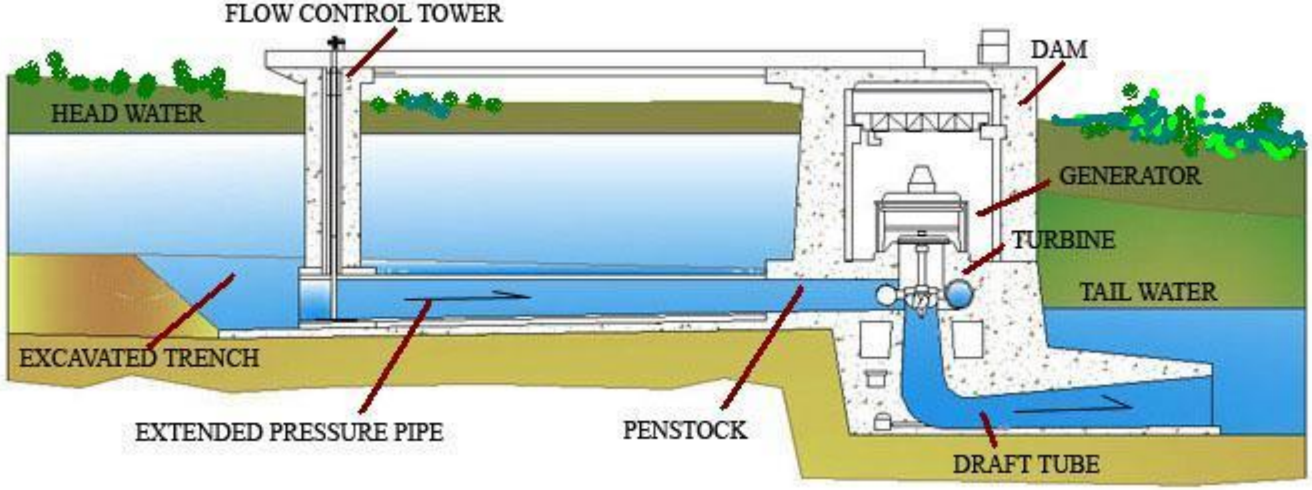

PIERS
GATE SLOT
X
X
SEMI - CIRCULAR
BEAMS

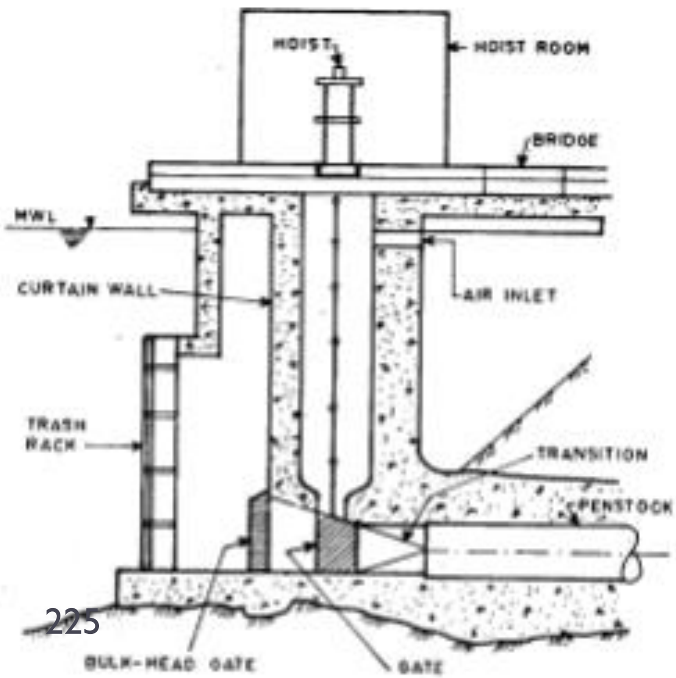

ROOF SLAB
GATE SLOT
TRASH
RACK
SLOT
PARABOLIC
PROFILE
PARABOLIC
PROFILE
Y
Y
RECTANGULAR
BELL MOUTH
SECTION
TRANSITION
RECTANGULAR
SECTION

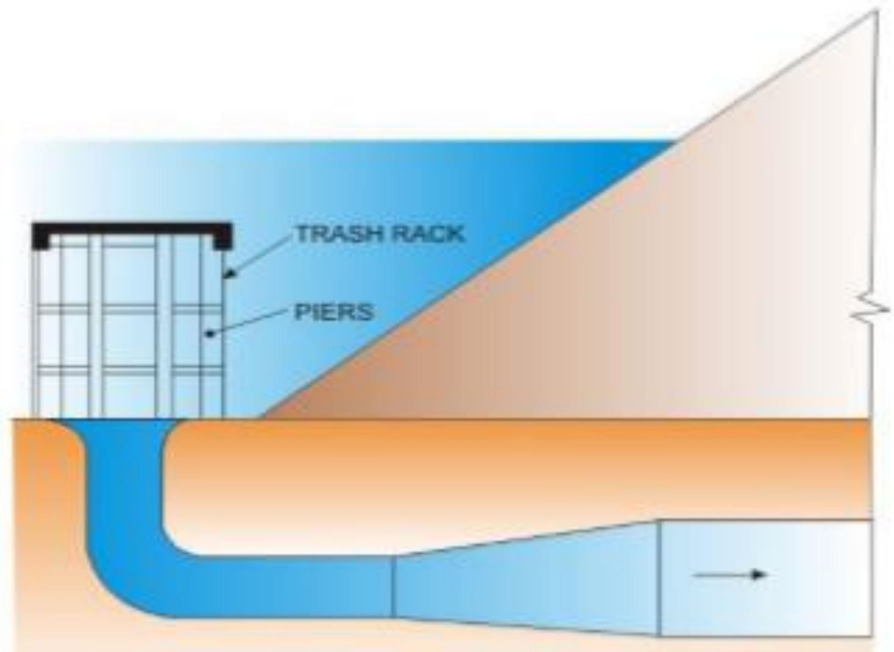
Tower Intakes

- Used when it is not convenient to provide the simple intake directly on the u/s face of the dam.
- Also used when there are wide fluctuations in water level.
- Tower may be connected with main dam through a bridge when the tower is near the dam
- Flow in to the tower is controlled by a number of gates to close or open the ports at various levels.
- Flow through the pressure conduit is controlled by vertical uplift gates.
- The structure should be strong enough to withstand hydrodynamic, earthquake, wind, etc.

Tower Intakes



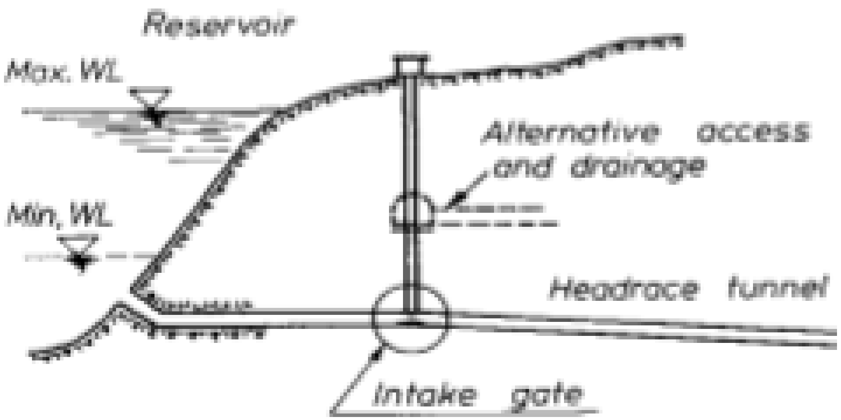


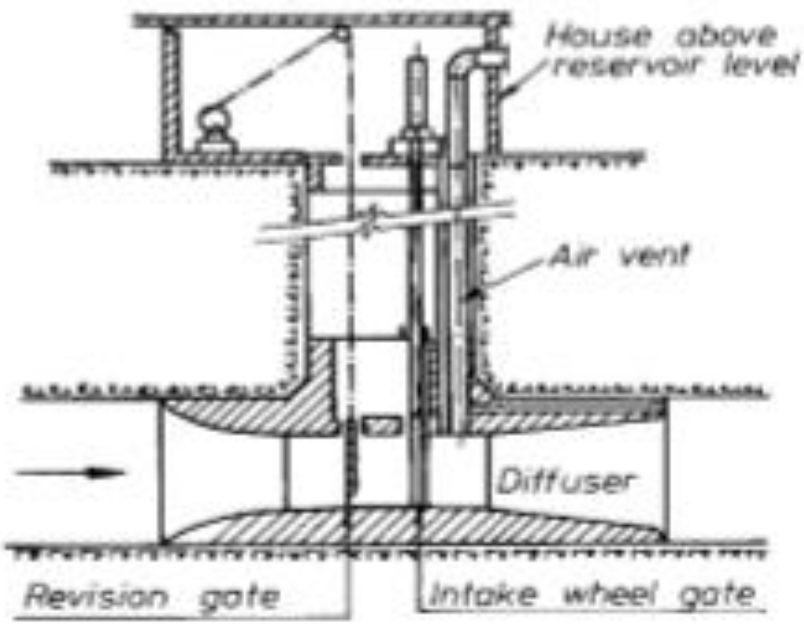


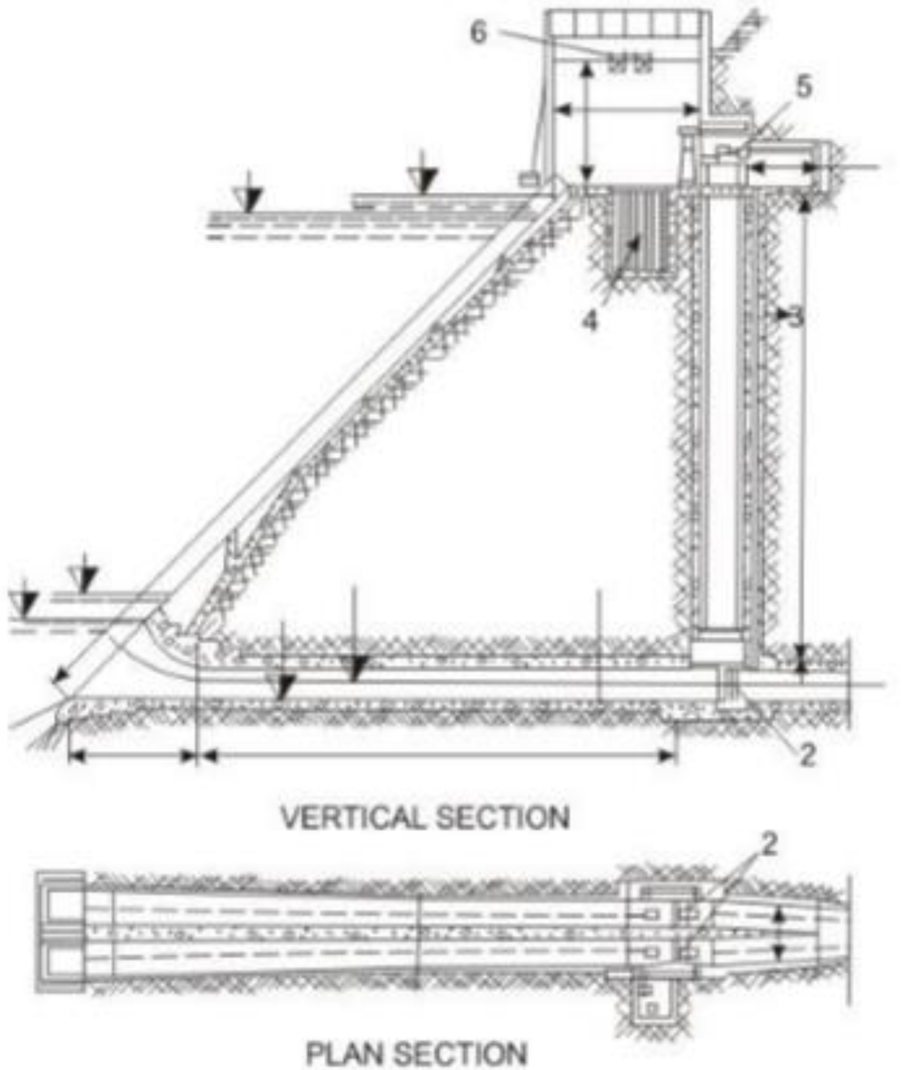
Shaft intakes

- This is a vertical shaft that carries water to the penstock tunnel.
- It consists of the following.
    - The entrance structure with trash rack and rounded inlet.
    - The vertical shaft followed by an elbow and transition connecting the shaft with the tunnel.
    - The intake gate (cylindrical) and sometimes a stop-log closure.

Shaft intakes







Trash racks and Skimmers

- Debris carried in the incoming water can have adverse impacts on a hydropower scheme in that:
- It can obstruct flow along the conveyance structures
- It can cause rapid deterioration of the penstock or turbine or cause a catastrophic failure
- Trash Racks:
  - intercepts the entire flow and removes any large debris, whether it is floating, suspended, or swept along the bottom.
  - Frequently, it is located in the intake structure to prevent debris from entering the water conveyance system.
  - It can also be placed just before the inlet to the penstock to remove smaller debris as well as other trashes which may have entered the water conveyance system downstream of the intake.

Trash racks

- A trash rack is made up of one or more panels, each generally fabricated of a series of evenly spaced parallel metal bars
- The approach velocity of flow should be kept within such limits that it will not cause damage to the rack structure. A design approach velocity of 0.5 m/s is usually used.
- If a trash rack is located immediately in front of the inlet to a penstock and the penstock velocities are significantly higher than 0.5 m/s, the trash rack can be built in a circular area to increase the area of the trash rack and correspondingly decreases velocity through it.
- Cleaning of the trash racks can be performed either manually (for small schemes) using manual rake or mechanically (for large schemes) using automatic cleaning machines.

Skimmers

- A skimmer wall is an obstruction placed at the water surface, usually at an angle to the stream flow which skims floating debris from the passing water.
- If the water level changes markedly as, for example, at the intake of stream, the skimmer can be a floating piece of timber secured at both ends. If changes in water level are small, a fixed skimmer can be used.
- Because some debris usually passes under the skimmer, a trash rack is still necessary. However, a skimmer reduces the frequency with which the trash rack has to be cleaned.
- Skimmer walls are made, for the most part, of reinforced concrete with a service bridge on top. They are designed usually for a horizontal pressure of 1000 kg/m^2 acting on the submerged surface.

Skimmers

Losses in Intakes

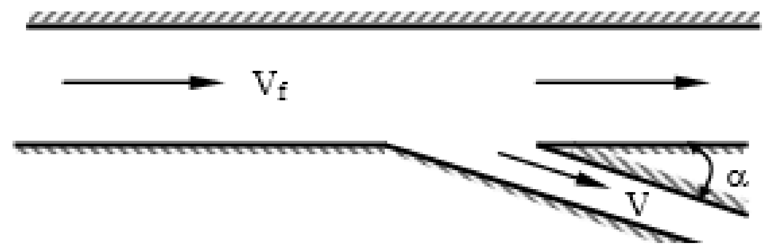
- The intake losses include entrance loss, trash rack loss and head gate loss.

1. Entrance Losses:

These comprises of:

a) Loss due to change in direction is given by:

$$\Delta h_e = \frac{V^2}{2g} - C\frac{V_f^2}{2g}$$



Where V is velocity in the diversion canal

V_f is velocity of flow in the main river

C is a constant which depends on the off-take angle of the diversion canal.

According to Mossonyi, C is equal to 0.8 for 30° off-take angle and 0.4 for 90° off-take angle.

Losses in Intakes

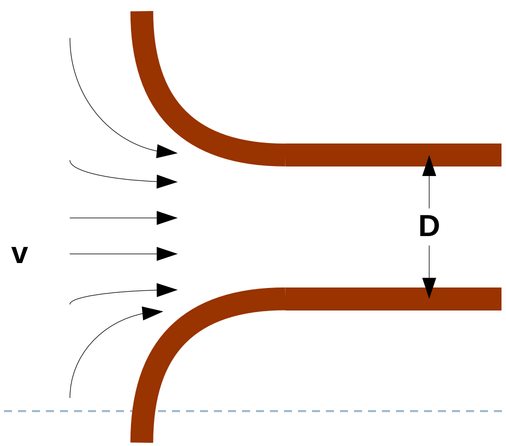
b) The losses due to sudden contraction of the area at the inlet section is given by:

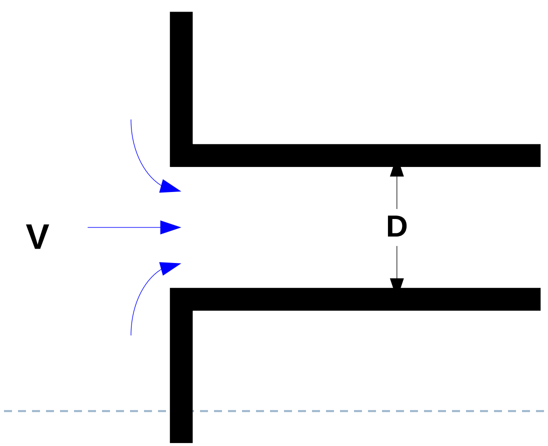
$$\Delta h_c = K\frac{V^2}{2g}$$

Where: K is a constant, which depends on the shape of the entry.

K=0.03 for bell-mouthed entry

K=1.3 for sharp cornered entry.





Losses in Intakes

2. Rack Losses:

There are numerous expressions available for predicting head loss across trash racks. One such expression (after Kirschmer's) is:

$$\Delta h_r = K_t \left(\frac{t}{b}\right)^{\frac{4}{3}} \frac{V_a^2}{2g} \sin\phi$$

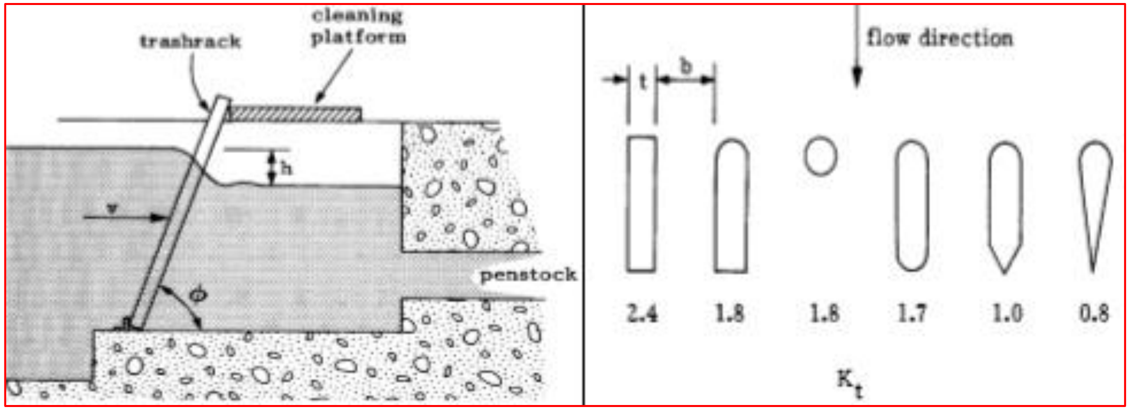
Where, K_t is trash rack loss coefficient (a function of bar shape),

t is bar thickness,

b is spacing between bars,

V_a is approach velocity, and

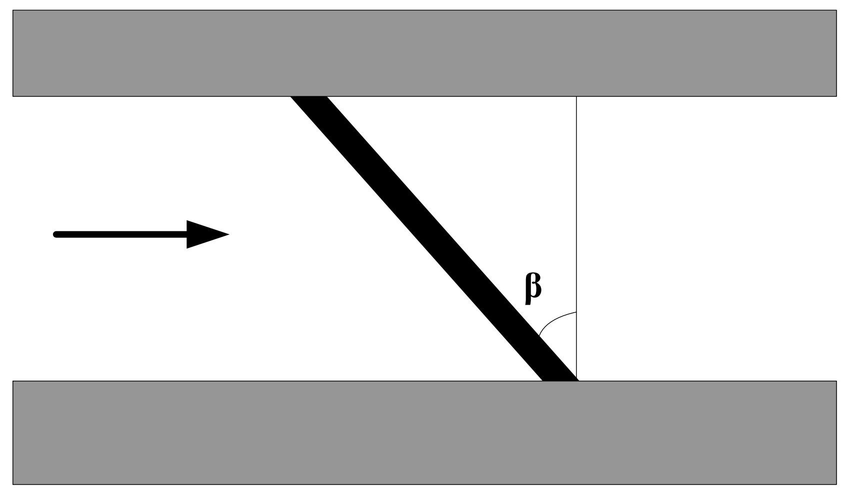
ϕ is angle of inclination of bars with the horizontal.



Losses in Intakes

- If the grill is not perpendicular but makes an angle β with the water flow (β will have a maximum value of 90° for a grill located in the sidewall of a canal), there will be an extra head loss, as by the equation.

$$\Delta h_{\beta} = \frac{V_a^2}{2g} \sin \beta$$



Losses in Intakes

3. Gate Losses

- Head loss due to gates (at part gate opening) is given by:

$$\Delta h_g = \frac{1}{2g}\left(\frac{Q}{C_d A}\right)^2$$

Where, Q is flow in the canal or conduit,

A is area of gate opening, and

C_d is discharge coefficient which varies between 0.62 and 0.83.

Velocity Through Trash Racks

- Velocity should be sufficiently low to avoid high head loss and should be sufficiently high to avoid large intake and trash rack cross section.
- The following are suggested limiting entrance velocities:

  i. Justin and Creager formula:

$$V \leq 0.12\sqrt{2gh}$$

  ii. Mossonyi's formula to eliminate eddies and vortices:

$$V \leq 0.075\sqrt{2gh}$$

  iii. U.S.B.R's criterion: permissible velocity in the range of 0.6 to 1.5 m/s. The trash rack is designed so the approach velocity (V_a) remains between 0.60 m/s and 1.50 m/s.

Air Entrainment & Vortex formation at Intakes

- Air entrainment is due to development of vortices and also due to partial gate opening that facilitates hydraulic jump formation.
- Effects of air entrainment are:
  - Additional head losses
  - Reduction in discharge and
  - Drop in efficiency of turbines.
- Minimizing vortex formation and avoiding hydraulic jump formation help in preventing air entrainment.

Air Entrainment & Vortex formation at Intakes

- A vortex which forms at the inlet to power conduit occasionally can cause troubles by itself.
  - It can induce loss of turbine efficiency,
  - Possible cavitation
  - Surging caused by the formation and dissipation of vortices,
  - Flow reduction as air replaces part of the water through the inlet.
  - It can also draw floating debris into the conduit.
- Vortices are formed due to the following factors:
  - Hydraulic jump formation
  - Velocities at intakes
  - Submergence at intakes
  - Geometry of approaching flow at intakes

Air Entrainment & Vortex formation at Intakes

- Designing for a low velocity into the conduit and increasing submergence of the inlet can help prevent the formation of vortices.

- Flow approaching the intake asymmetrically is more prone to vortex formation than symmetrical flow.

- It is therefore important that flows upstream of the inlet area be as straight and uniform as possible.

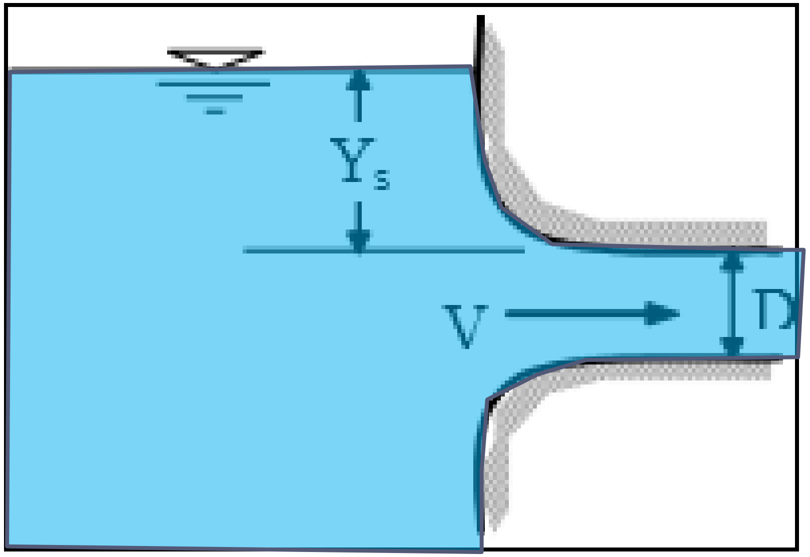
Air Entrainment & Vortex formation at Intakes

- For the condition of no vortices at intakes, the following empirical relations may be used (after J. B. Gardon):

$$Y_s > 0.545V\sqrt{D} \qquad \textit{for symmetrical approach}$$

$$Y_s > 0.725V\sqrt{D} \qquad \textit{for asymmetrical approach}$$

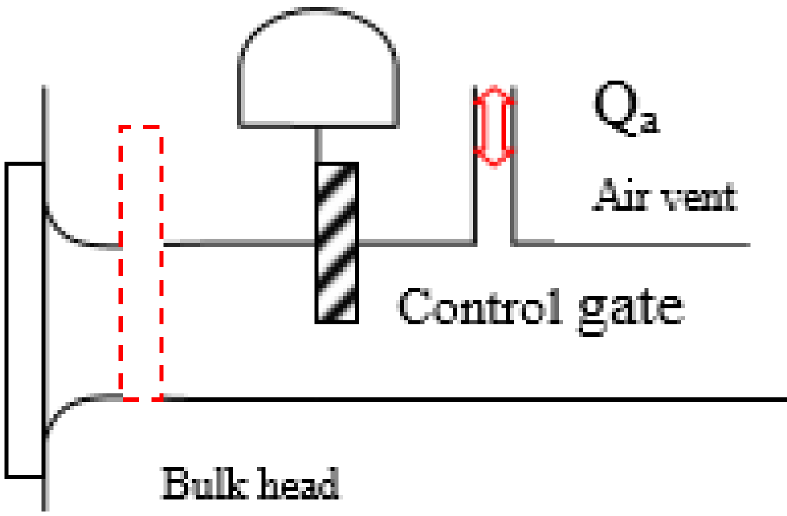
- Where: Y_s is necessary submergence depth
  V is velocity at inlet to the canal
  D is diameter of the conduit.



Another remedy to vortex formation is provision of a floating raft or baffle which disrupts the angular momentum of the water near the surface.

Inlet Aeration

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



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

b) Intake gates operate under conditions of balanced pressure on both sides of the gate.

Thus the conduit is required to be filled with water through a by-pass pipe. The entrapped air is therefore driven out through the air vent.

Inlet Aeration

Size of the air vent:

- There are several recommendations

1. $$Q_a = 400Ca\sqrt{p}$$

- Where Q_a = Discharge of air in cumecs

  a = Area of vent pipe in m^2 C = Constant ~0.7

  p = Pressure difference between the atmosphere and pressure in the penstock in kg/cm^2

2. 4th Congress on Large Dams (ICOLD)

   Area of air vent =10% of control gate area

3. USBR design guide:

   Capacity of air vent = 25% of conduit discharge

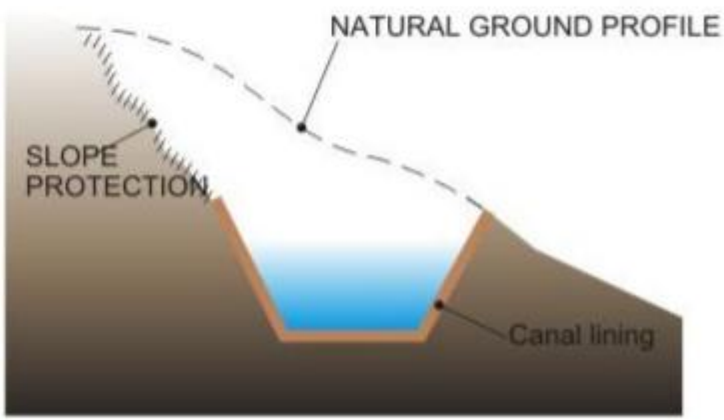
Water Conveyance System

Water Conveyance System

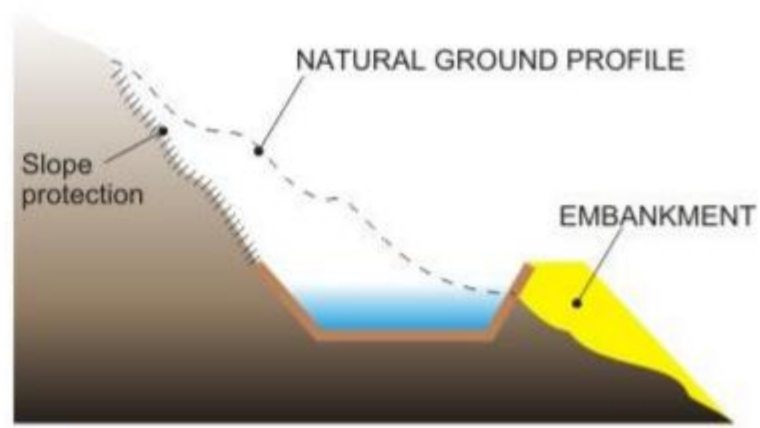
- After flowing through the intake structure, the water must pass through the water conveyance system may be either of:
  - Closed conduit type (tunnel off-taking from upstream of the river diversion) or
  - Open-channels
- High pressure intakes, for example as in the entry to penstocks would be either:
  - Reinforced concrete lined or
  - Steel lined.

Open channels

- These are usually lined canals to reduce water loss through seepage as well as to minimize friction loss.
- The design of canals for hydropower water conveyance follows the same rules as for rigid bed irrigation channels, and is usually termed as power canals.
- A power canal that off takes from a diversion structure has to flow along the hill



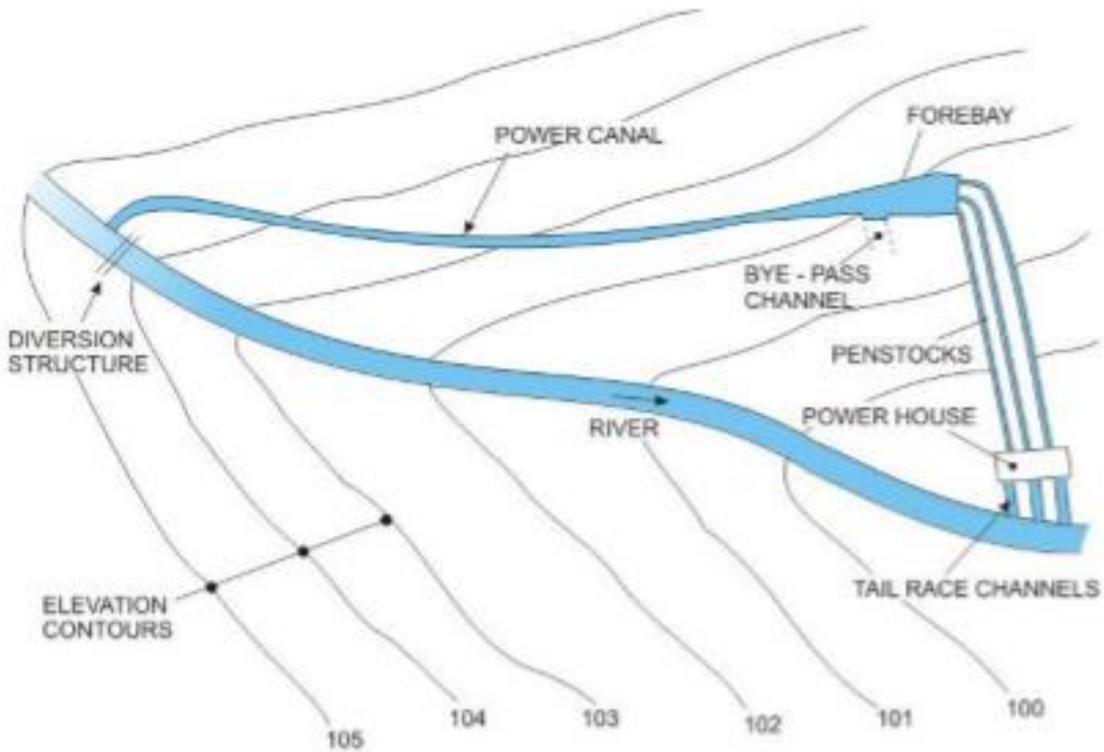
cross section of a power canal in cutting



Cross section of a power canal in partly cutting & partly filling

Open channels

- A power canal ends at a forebay, which is broadened to act as a small reservoir.
- From the forebay, intakes direct the water into the penstocks.



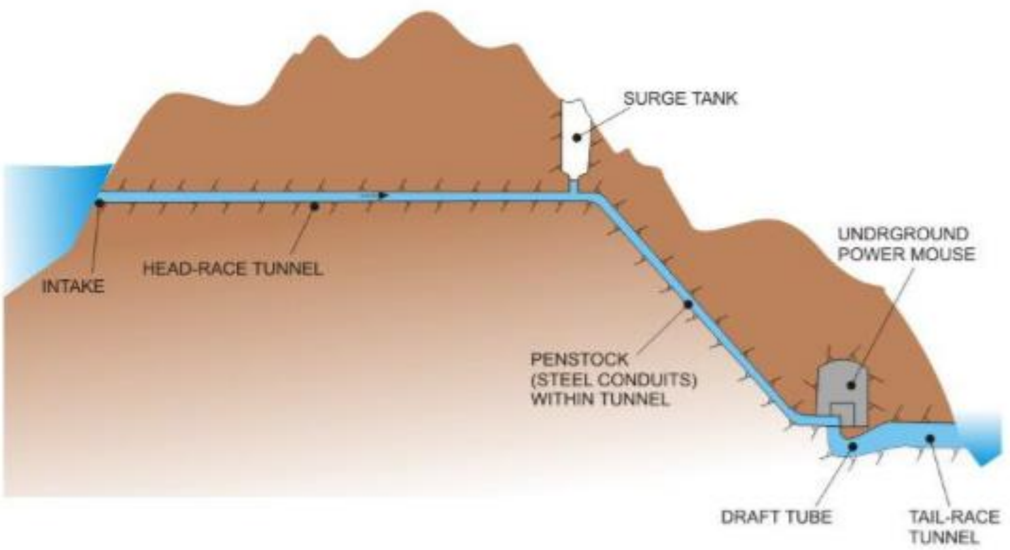
Open channels

- There usually is a bye-pass channel which acts as a spillway to pass on excess water in case of a valve closure in the turbine of the hydropower generating unit.
- If such an escape channel is not provided, there are chances that under sudden closure of the valves of the turbines, surge waves move up the power canal.
- Hence, sufficient free board has to be provided for the canals.

- *The design of power canal is same with that of irrigation canal.*

Tunnels

- Tunnels are underground conveyance structures constructed by special tunneling methods without disturbing the natural surface of the ground.
- The initial portion of the tunnel from the intake up to the Surge-Tank is termed as the Head Race Tunnel (HRT) and beyond that it houses the penstock or steel-conduits, which sustains a larger pressure than the HRT
- The surge tank is provided to absorb any surge of water that could be generated during a sudden closure of valve at the turbine end.



Tunnels

- In the headrace of water conveyance system, tunneling is popular because of the following reasons:

  a) It provides a direct and short route for the water passage thus resulting in considerable saving in cost

  b) Natural landscape is not disturbed

  c) Tunneling work has become easier with development techniques of drilling and blasting and new mechanical equipment (Tunnel Boring Machines)

  d) Development of rock mechanics and experimental stress analysis has given greater confidence to engineers regarding stability of tunnels.

Tunnels

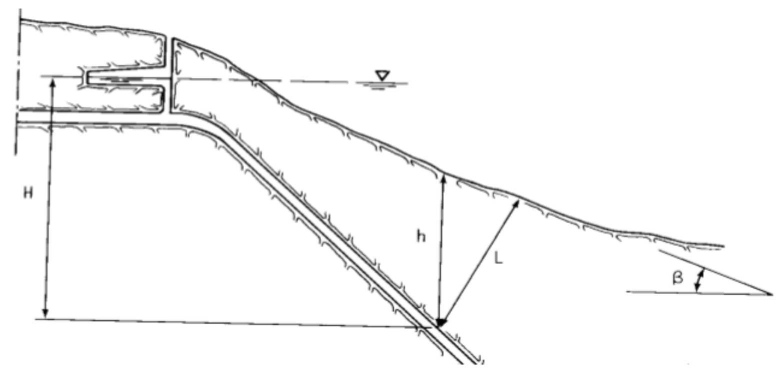
- The HRT may either be unlined (in case of quite good quality rocks) or may be lined with concrete.
- Lining of tunnels is required:
    i. For structural reasons to resist external forces particularly when the tunnel is empty and when the strata is of very low strength.
    ii. When the internal pressure is high
    iii. When reduction in frictional resistance and therefore the head loss is required for increasing capacity
    iv. For prevention or reduction of seepage losses
    v. For protection of rock against aggressive water

Tunnels

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

Tunnel Design Features

- Let h_r = depth of overburden rock
  γ_r = specific weight of the rock
  γ_w = specific weight of water.
  H = Internal pressure head of water.



- Then for equilibrium:

$$\gamma_w H \leq \gamma_r h_r$$

- With γ_w = 1 ton/m^3, we have $H \leq \gamma_r h_r$
- Using a factor of safety of η , $H = \dfrac{\gamma_r h_r}{\eta}(m)$
- Recommended factor of safety, η = 4 to 6.
- With γ_r = 2.4 t/m^3 to 3.2 t /m^3 and using lower η values for good quality rock, one gets

$$H = (0.4 \text{ to } 0.8)h_r$$

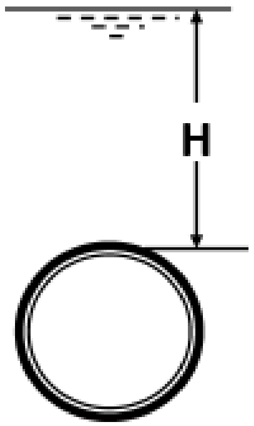
Tunnel Design Features

- The following features are most important in the design of tunnel:
  - Alignment
  - Geometric shape
  - Longitudinal slope,
  - Flow velocity
  - Head loss
  - Rock cover (overburden)
  - Lining requirements and
  - Economic x-section

Alignment

- A name tunnel indicates a very small bottom slopes, 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

- 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)



Alignment

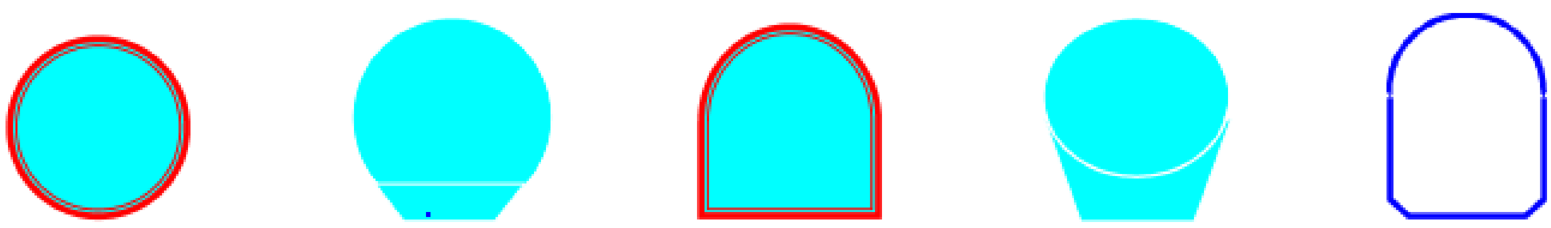
In aligning water tunnels, the following points should be taken in to account:

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

Geometrical Shape

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

- Hydraulic considerations: Circular is preferable
- Stability considerations: Circular is preferable
- Convenience for construction: Horseshoe is preferable
- Available tunneling equipment: Different shape



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

Longitudinal Slope

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

Flow Velocity

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

Rock Cover (overburden)

- For pressure tunnels, it is obvious that the overburden on the roof of the tunnel serves to balance the effect of upward force due to internal pressure.

- The required depth of overburden may vary for lined and unlined tunnels.

- In the case of unlined tunnels, the entire internal water pressure is resisted by the overburden rock pressure.

- Where a steep valley side constitutes the overburden above the tunnel, the rule of thumb equation, H= (0.4 to 0.8)h_r has to be modified and given by:

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

Head Loss

Head losses in tunnels can be computed using:

- Manning formula:

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

- Darcy-Weisbach formula:

$$h_f = \lambda \frac{lv^2}{2gD_{eq}}$$

- Hazen-Williams formula (ru):

$$h_f = 6.84 \frac{lv^{1.85}}{C^{1.85} D_{eq}^{1.17}}$$

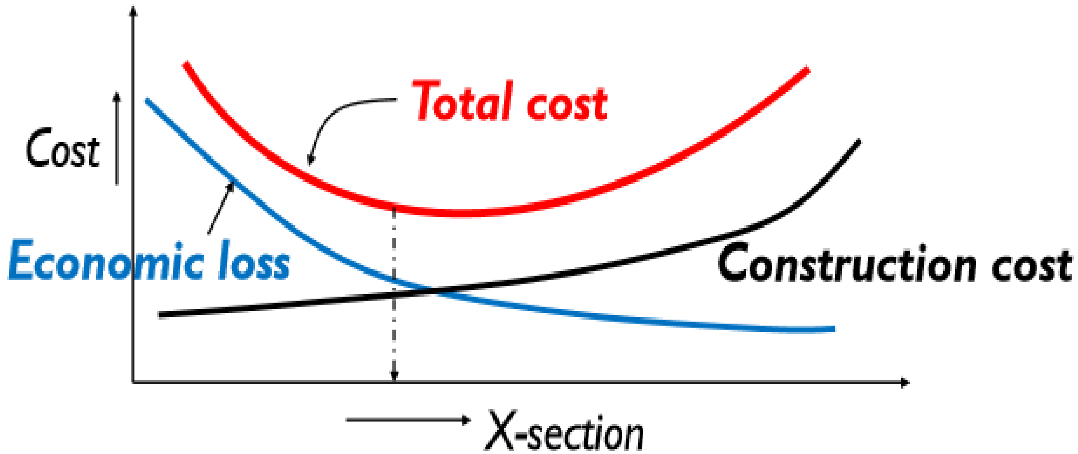
- Where: h_f is head loss due to friction
  - L is tunnel length
  - V is mean velocity of flow
  - R is hydraulic radius
  - D_{eq} is equivalent diameter

$$D_{eq} = \sqrt{4A/\pi}$$

  - A is area of the tunnel x-section
  - n is Manning's roughness coefficient
  - λ is Darcy-Weisbach friction factor (can be obtained from Moody diagram), and
  - C is Hazen-Williams roughness coefficient.

Optimum X-section

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



- For a quick initial estimate of the diameter of pressure tunnels, the empirical formula suggested by Fahlbusch can be used:
- For concrete-lined tunnels: $D = 0.62Q^{0.48}$
- For steel-lined tunnels: $D = 1.12\dfrac{Q^{0.45}}{H^{0.12}}$

Tunneling Methods

- There are two commonly used types of tunneling techniques:
  - Conventional "Drill and Blast"
  - Use of tunnel boring machines (TBM)
- The following are the main sequences to be followed during excavation of each round:
  - Drilling
  - Charging
  - Blasting
  - Ventilating
  - Scaling
  - Mucking and hauling
  - Tunnel supporting

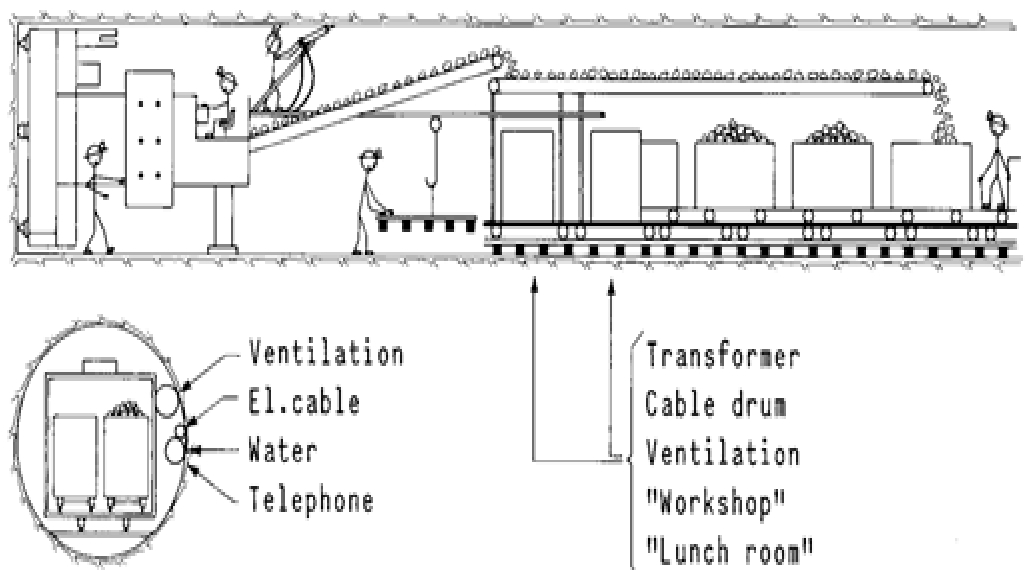
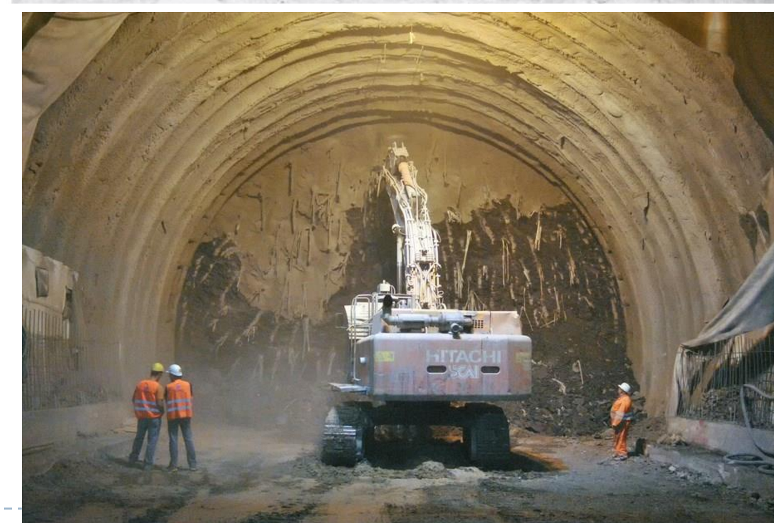


Fig. TBM

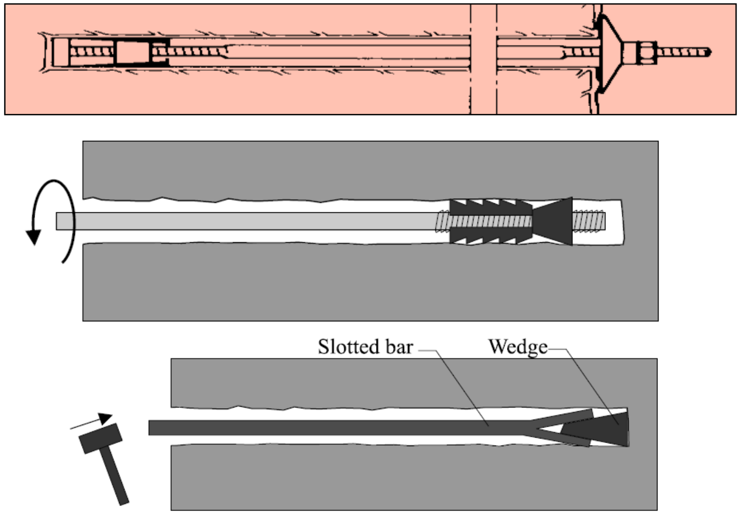
Tunneling Methods



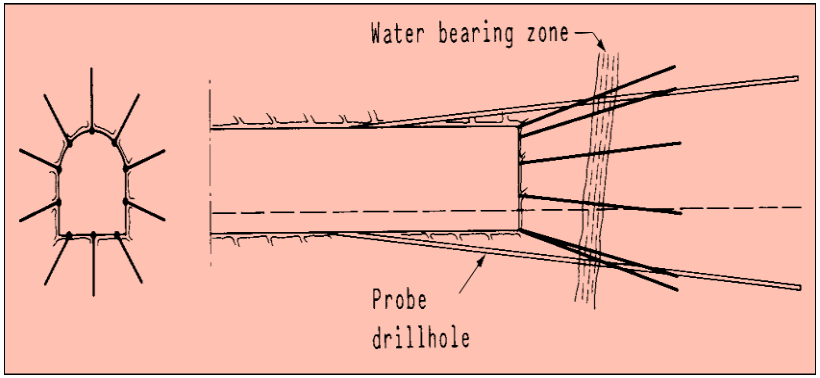
Tunnel Support

- Bolting
- Shotcreting
- Grouting
- Concrete lining

Shotcreting



Bolting



Grouting

Settling Basin

Settling Basin

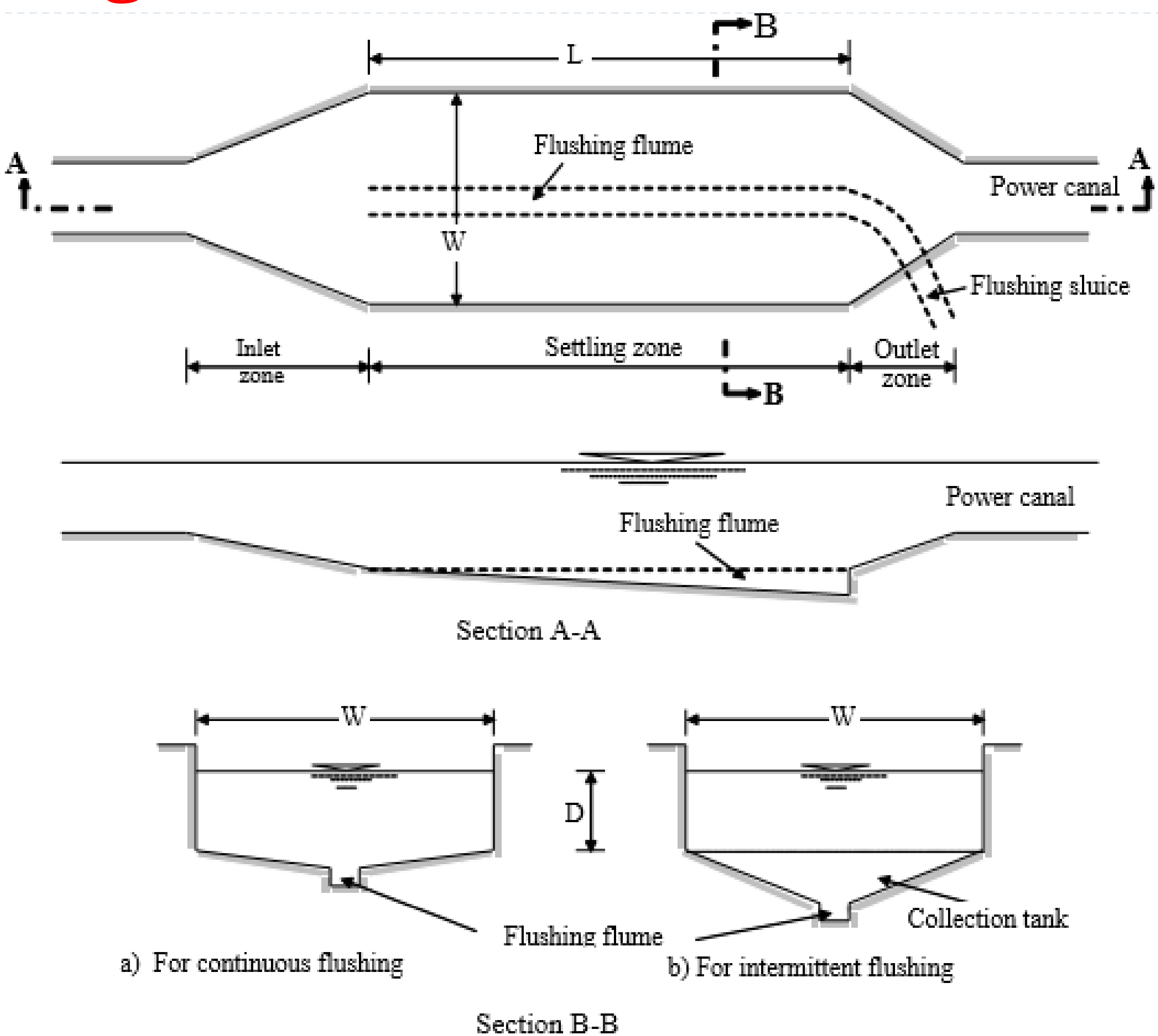
- The water drawn from a river for power generation may carry suspended sediment particles.
- This silt load may be composed of hard abrasive materials such as quartz and will cause damage or wear to the hydro-mechanical elements like turbine runners, valves, gates and penstocks.
- To remove this material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

Settling Basin

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

1. **Inlet Zone:** The main function of the inlet is to gradually decrease the turbulence and avoid all secondary currents in the basin.
   - This is achieved by decreasing the flow velocity through gradually increasing the flow cross-section, i.e., by providing gradual expansion of the width and depth.
   - To achieve optimum hydraulic efficiency and effective use of the settling zone, the inlet needs to distribute the flow uniformly over the cross-section of the basin.

Settling Basin



Settling Basin

- To achieve uniform flow distribution, the following techniques, in addition to the provision of gradual expansion, may be adopted at the inlet zone:
    - Use of submerged weir
    - Use of baffles
    - Use of slotted walls

Settling Basin

2. Settling Zone:

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

3. Outlet Zone:

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

Design of Settling Basin

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

1. The settling basin must have length and width dimensions which are large enough to cause settling of the sediments but not so large that the basin is over expensive and bulky.
2. It must allow for easy flushing out of deposits, undertaken at sufficiently frequent intervals.
3. Water removed from the flushing exit must be led carefully away from the installation. This avoids erosion of the soil surrounding and supporting the basin foundations.
4. It must avoid flow turbulence caused by introduction of sharp area changes or bends, and they must avoid flow separation.
5. Sufficient capacity must be allowed for collection of sediment.

Design of Settling Basin

- The hydraulic design of settling basins is broadly outlined in the following:

1. Exploration of sediment conditions, involving the quantitative and qualitative analysis of sediment carried by the river.
    - As regards to wear of the hydraulic machinery, suspended sediment is of significance, since the bulk of the bed-load moving along the bottom can be effectively prevented from entering the canal by a well-designed intake.

Design of Settling Basin

2. Degree of removal: Usually, the sensitivity of plant installations, particularly the hydraulic machines, requires that a marginal (critical) grain size d_{cr} is not exceeded.

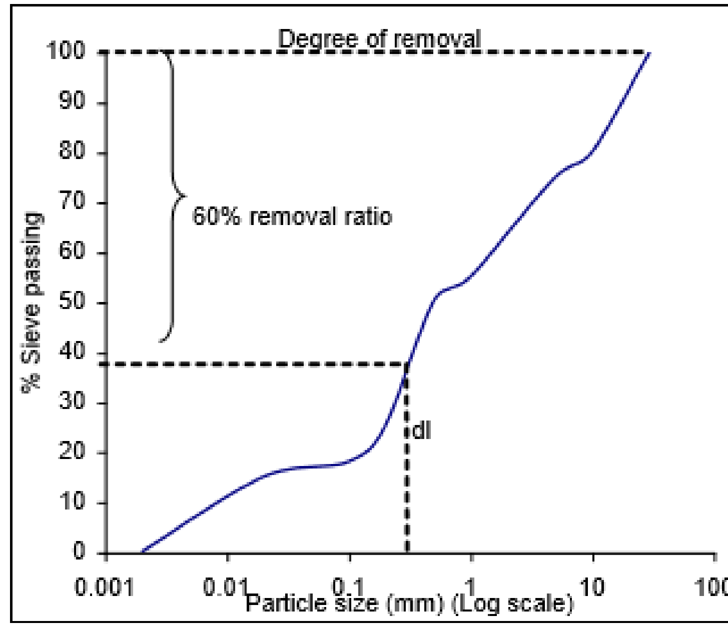
   - The settling basin must be dimensioned in such a way that grains with diameters bigger or equal to d_l (limit particle size) must be settled.

   - It should be noted, however, that no standard values or specifications have yet been developed

- For medium head = (15-50m); d_l = 0.2 to 0.5mm in diameter
  For head up to 100m; d_l = 0.1 to 0.2mm in diameter
  Very high head >100m; d_l = 0.01 to 0.025mm in diameter

Design of Settling Basin

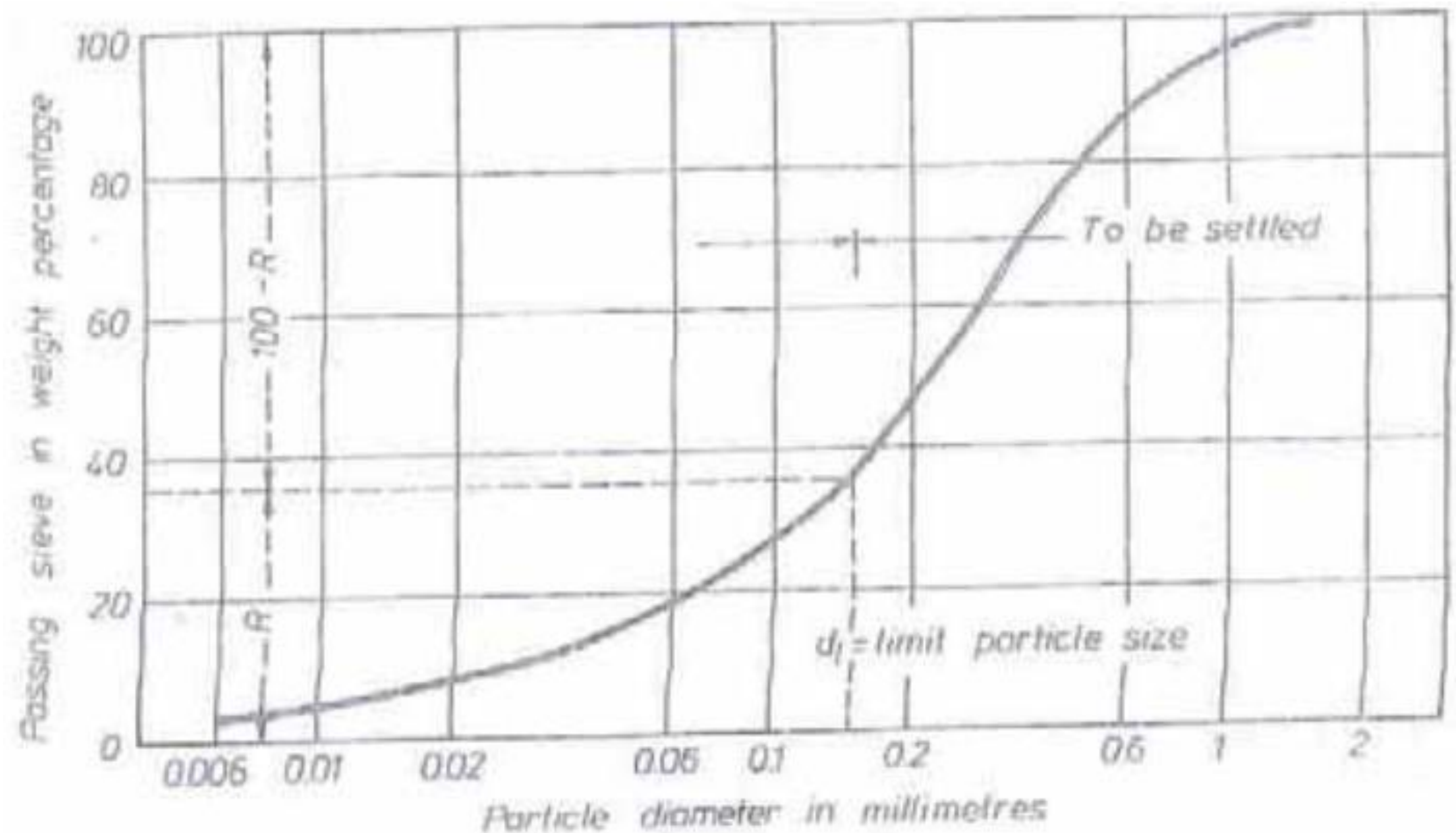
- For the limit particle sizes mentioned above, the lower limits should be used if the sediment fractions contain sharp-edged quartzite grains.



- Instead of using the limit particle size, the degree of removal is frequently defined by the removal ratio, which is the ratio of concentrations after and before settling, expressed in %ge.
- If the concentration of the raw water is C, and that of clarified water is specified as the permissible value C_p, the required removal ratio is obtained as:

$$R = \frac{C_p}{C} * 100(\%)$$

Design of Settling Basin



$$R = \frac{C_p}{C} * 100(\%)$$

Design of Settling Basin

3. Settling velocity of the smallest fraction (the limit particle size to be removed).

- This can be established theoretically (Stoke's law) or by experiments (Sudry graph).

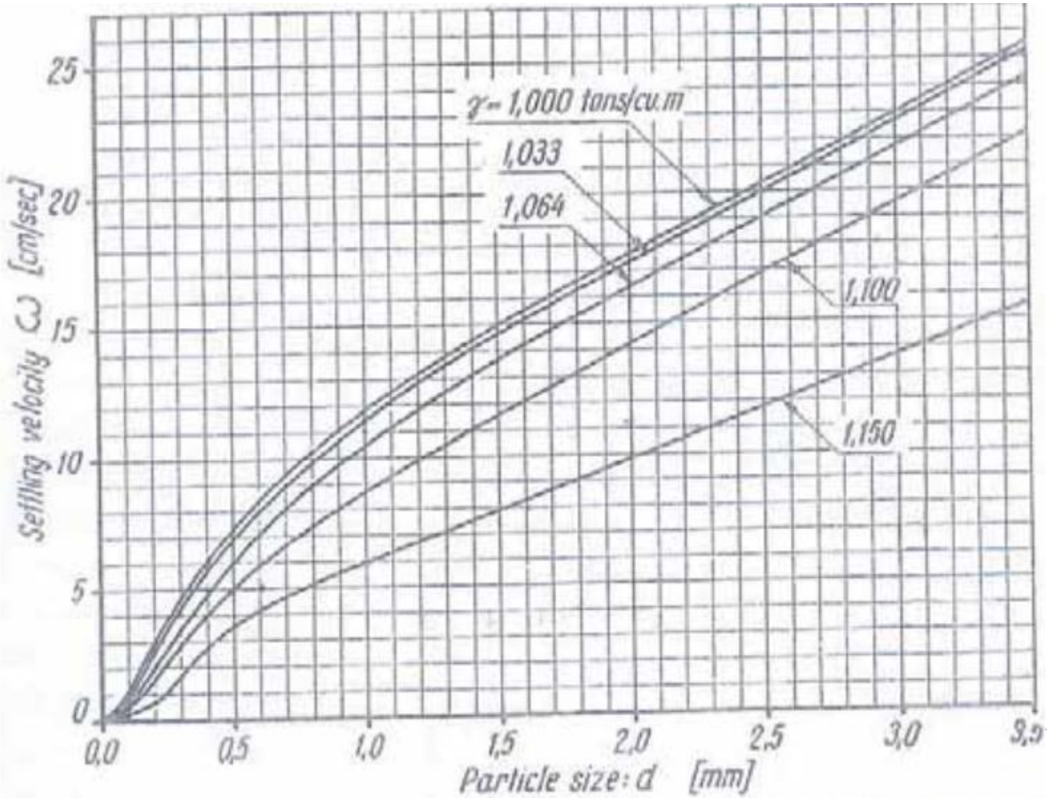
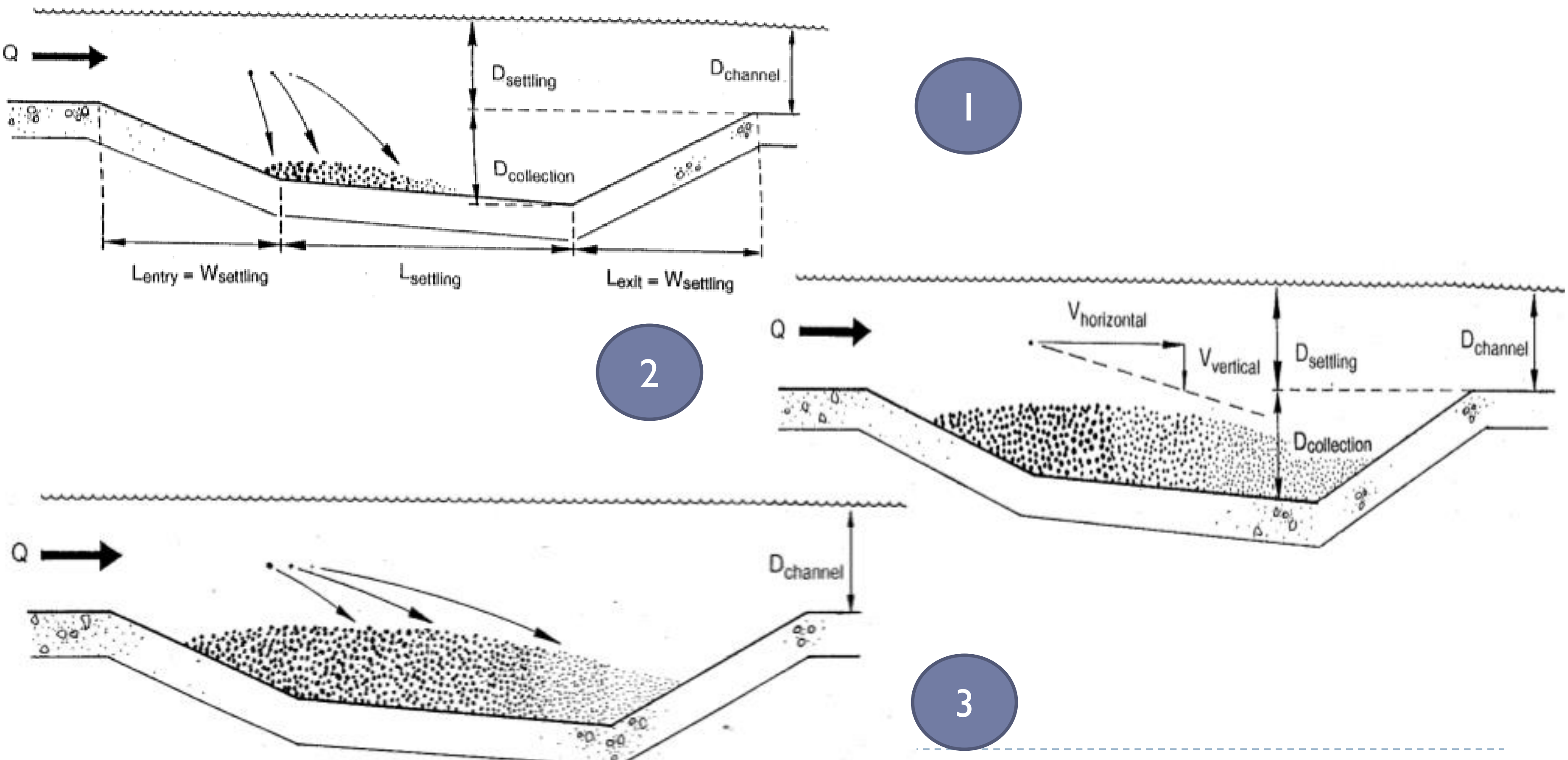


Figure: Settling velocity in stagnant water plotted against the density of silt loaded water and the particle diameter (After L.Sudry)

Design of Settling Basin

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



Design of Settling Basin

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

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

$$Q = WDV$$ V is the flow-through velocity.

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

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

Design of Settling Basin

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

$$L = V\,t$$

Design of Settling Basin

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

$$Q = WDV$$

$$L = \frac{DV}{\omega}$$

- Obviously a solution of the problem is not possible unless four of the six quantities are known.
    - The discharge Q is usually known.
    - The settling velocity ω is defined by the initially specified degree of removal and, as mentioned previously, can be established by calculation

Design of Settling Basin

- The highest permissible flow-through velocity V should also be specified, considering that particles once settled should not picked up again.
- According to Camp, the critical flow-through velocity is estimated from:

$$V = a\sqrt{d_1} \qquad [m/s]$$

- Where, d_1 is the equivalent diameter of the smallest sediment particle to be settled in mm and a is a constant given as:

  a = 0.36, for $d_1 > 1$mm

  a = 0.44, for 0.1mm< $d_1 < 1$mm

  a = 0.51, for $d_1 < 0.1$mm

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

Design of Settling Basin

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

$$from \quad Q = DWV \Rightarrow V = \frac{Q}{WD}$$

$$and \;\; from \;\; L = V\,t \;\Rightarrow V = \frac{L}{t}$$

$$therefore \frac{Q}{WD} = \frac{L}{t} \Rightarrow Q\,t = WLD$$

$$Water\ conveyed\ to\ the\ tank\,(\,Q\,t\,) = Volume\ of\ \ the\ tank\,(\,WLD\,)$$

Design of Settling Basin

B. Design considering the effect of turbulence:

- Owing to the retarding effect of turbulent flow on subsiding particles, settling is slower in flowing water.
- A more accurate investigation of the basin is thus by considering the retarding effect of turbulence into consideration.
- By using a lower settling velocity $\omega - \omega'$, yields greater values for the length of the basin. $L = \dfrac{DV}{\omega}$
- The reduction in the settling velocity ω' is related to the flow-through velocity by:

$$\omega' = \alpha V \quad [m/s] \qquad \alpha = \frac{0.132}{\sqrt{D}}$$

Design of Settling Basin

- Accordingly, the equation of the settling length can be modified as:

$$L = \frac{DV}{\omega - \alpha V} = \frac{D^{1.5}V}{D^{0.5}\omega - 0.132V} \quad [m]$$

- This shows a larger settling basin is required, when compared with simple settling theory.

- In the computation if the result provides negative value in the denominator, it indicates that no settling takes place in the basin; hence dimension should be modified.

Design of Settling Basin

- Theoretically, the following equation can be used to estimate the settling velocity:

$$\omega = \left[\frac{4}{3}\frac{gd}{C_d}\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{0.5}$$

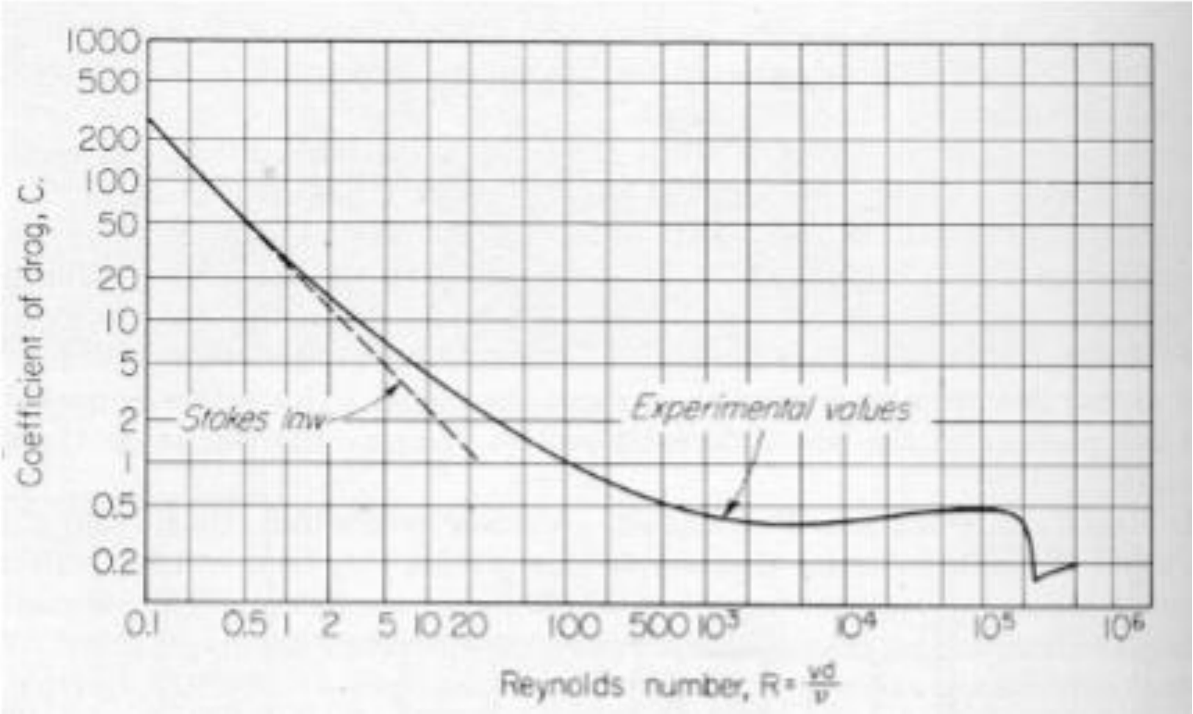
- Where, d = diameter of the sediment particle

  γ_s = specific weight of the sediment particle,

  γ_w = specific weight of water,

  C_d = the coefficient of drag and is a function of particle Reynolds number R = $\omega d/\nu$, ν being the kinematic viscosity of the water.

Design of Settling Basin



Drag coefficient of spheres as a function of particles Reynolds number (note ω= v)

The drag coefficient in the Stokes range (R < 0.1) is given by C_d = 24/R, and the above equation can be modified for Stokes range as:

$$\omega = \frac{gd^2}{18\upsilon}\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)$$

Removal of Sediments from Settling Basins

- There are different techniques for removing sediment deposits in settling basins:
  - Manual or mechanical removal of deposited sediments after the basin is dewatered.
  - Flushing of deposited sediments through an outlet provided at the bottom, often called flushing sluice.
- Continuous operation can be ensured by one of the following methods:
  a) Providing two or more parallel basins (some can be cleaned while others are operating).
  b) Adopting continuous flushing, by admitting excess water into the basin. An inflow exceeding the water demand by about 10% may be admitted continuously into the basin and used for flushing the sediment accumulating at the bottom.

Example

1. Design a settling basin for high-head power station using the simple settling theory. The basin should serve to remove particles greater than 0.5mm diameter from the water in which the sediment is mainly sand. Let the design discharge be $5m^3/s$ and assume an initial value of 3.2m for the basin depth. Take the water sediment mixture density to be 1064 ton/m^3

2. Compute the settling length by considering the retarding effect of turbulence.

Water Hammer Analysis

Water Hammer 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.

Water Hammer Analysis

- 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.

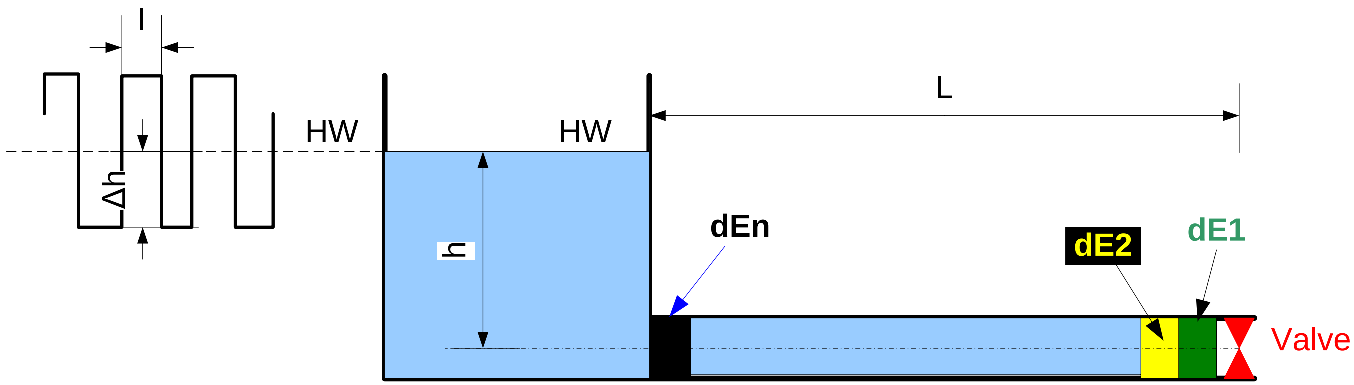
Water Hammer Analysis

- 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.

Water Hammer Analysis

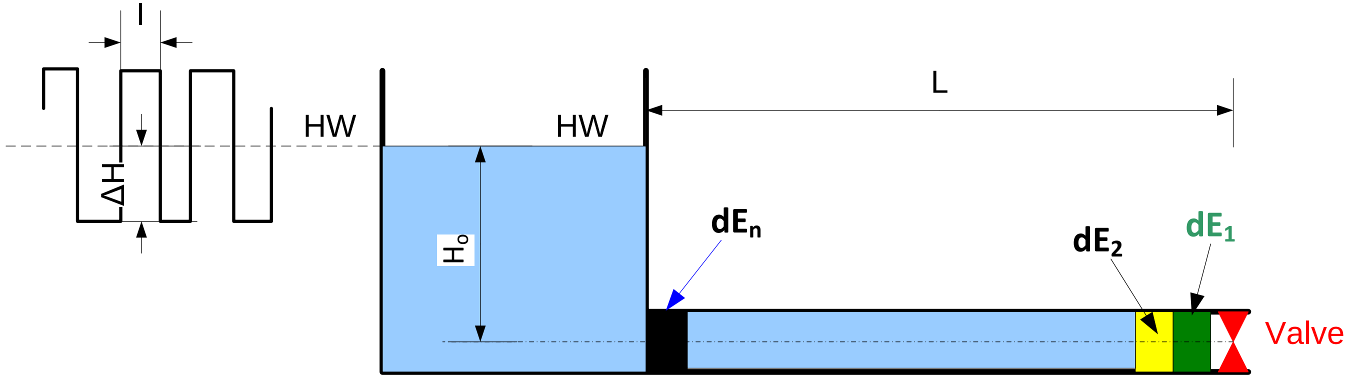
- Flow is changing from one steady-state to another over a very, very short time.
- Causes of rapid changes in flow (acceleration/deceleration):
  - Sudden opening or closing of control valves
  - Starting or stopping of pumps
  - Rejecting or accepting load by hydraulic turbine
- Rapid change in velocity results in change in momentum causing pressure fluctuations (waves) – Water Hammer
- There are two theories in analyzing water hammer:
  - Elastic water column (EWC) theory
  - Rigid water column (RWC) theory

Elastic water column (EWC) theory

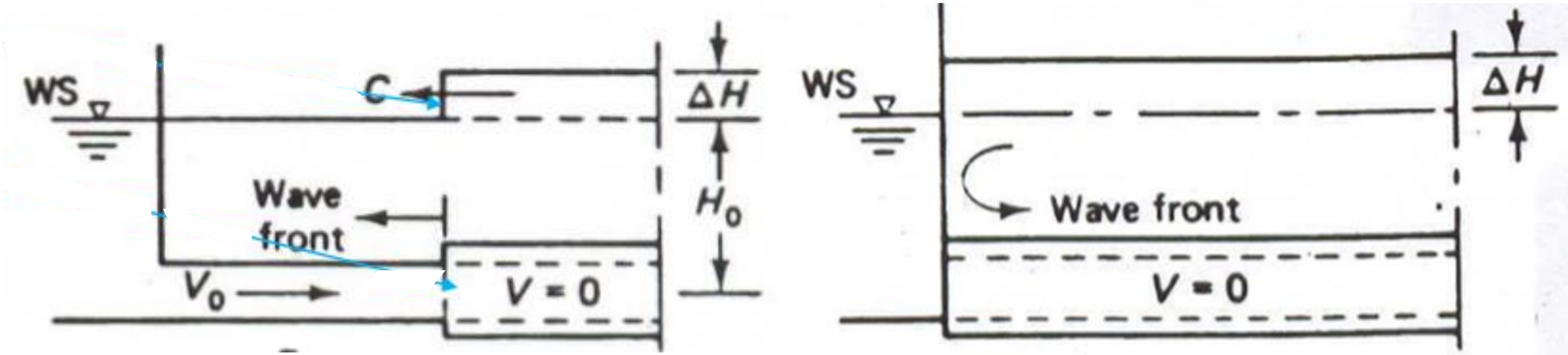


- When a valve in a pipeline is suddenly closed, the element of water, dE_l nearest to the valve is compressed by the water flowing towards it and the pipe is stretched by the action.
- In the next time frame the element dE_2 is stopped and compressed too.
- The water upstream of dE_2 continues to move at the original velocity and successive elements of water are compressed.

Elastic water column (EWC) theory

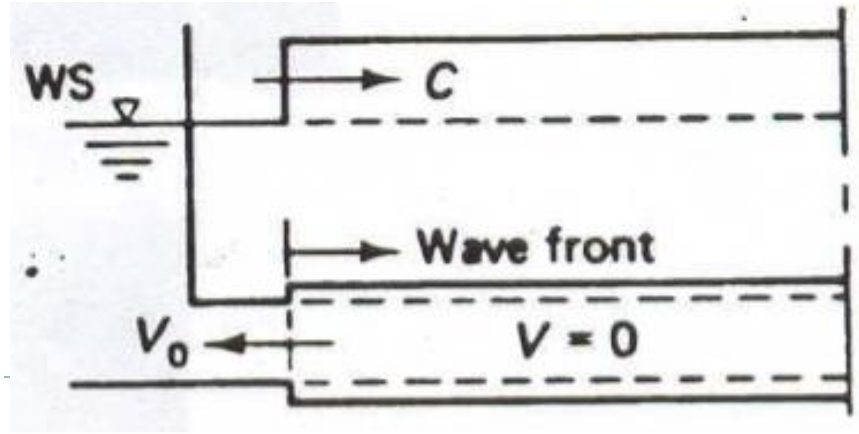


- The action of compression moves upstream as a wave until it reaches the open water surface and the last element dE_n is compressed and the entire conduit of water has no movement thus under the increased pressure head, $H_o + \Delta H$.



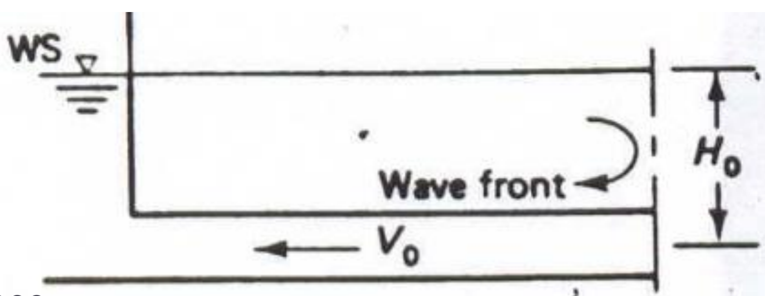
Elastic water column (EWC) theory

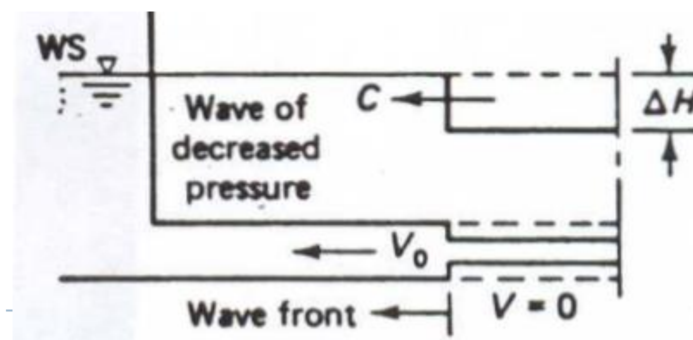
- The pressure pulse or wave moves at a velocity, c, which is essentially the velocity of sound in water.
- Thus the pressure wave reaches the open water surface in time, $t = L/c$.
- At that time the kinetic energy of the moving water has been converted to elastic energy in compressing the water and stretching the pipe.
- At the open water surface the last element, dEn expands to its original state, followed by other elements, causing a reverse or negative pressure wave.



Elastic water column (EWC) theory

- As this wave travels downstream, conditions change from zero velocity to a negative velocity and from the increased water pressure head, $H_o + \Delta H$, back to the normal pressure head, H_o.
- 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.



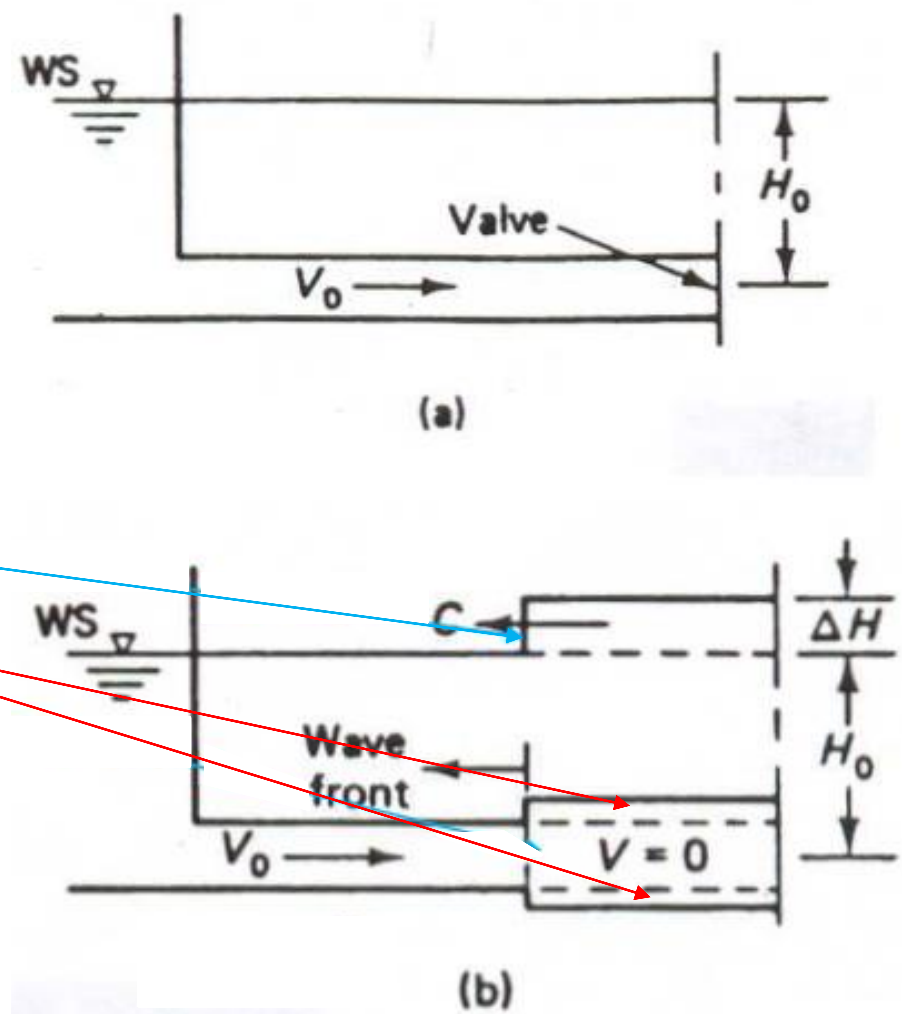


Elastic water column (EWC) theory

Propagation of pressure wave due to valve closure

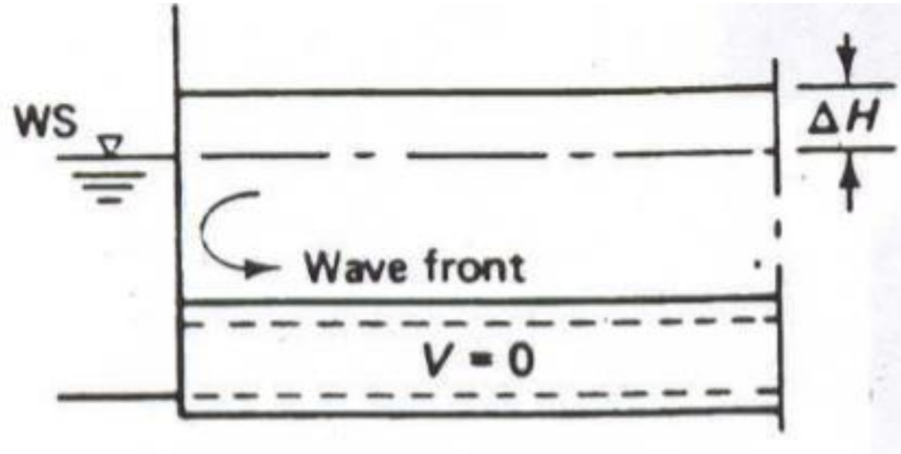
a. Steady state prior to valve closure

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



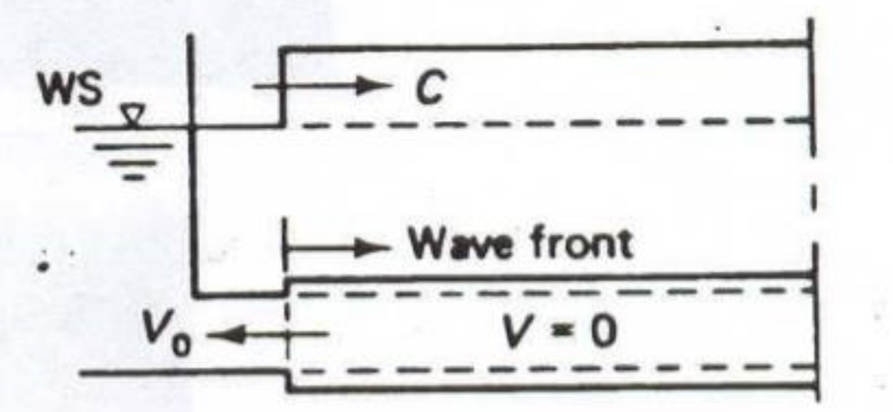
Elastic water column (EWC) theory

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



(c)

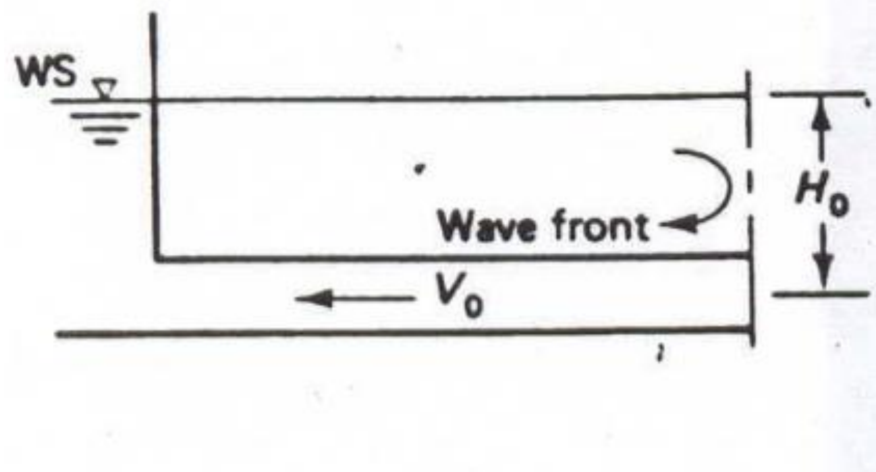
d. Pipe pressure > tank/resv. pressure water flows from pipe to tank/resv. relieving pressure in pipe



(d)

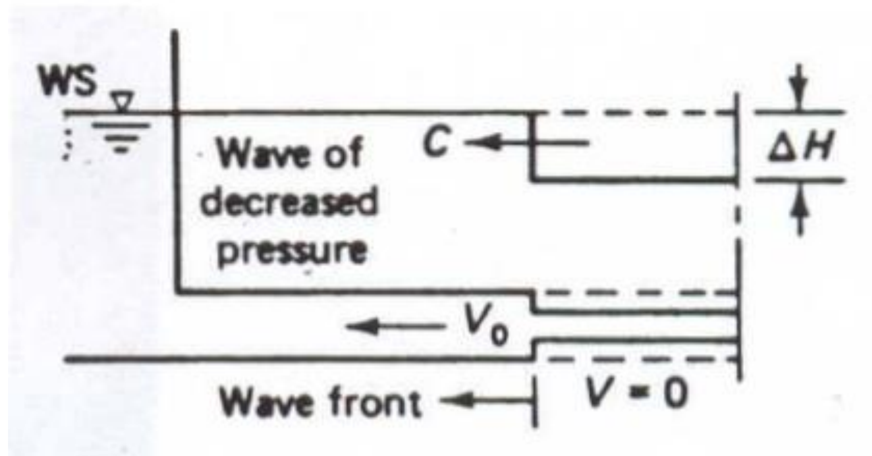
Elastic water column (EWC) theory

e. Process starts at tank and continues up to valve at time t = L/c,
at this moment total time = 2L/c → water hammer period



(e)

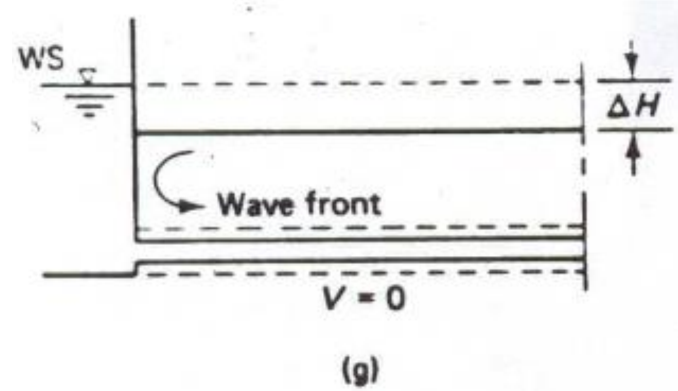
f. Wave of backwater cannot go past the valve, starts wave of negative pressure toward tank

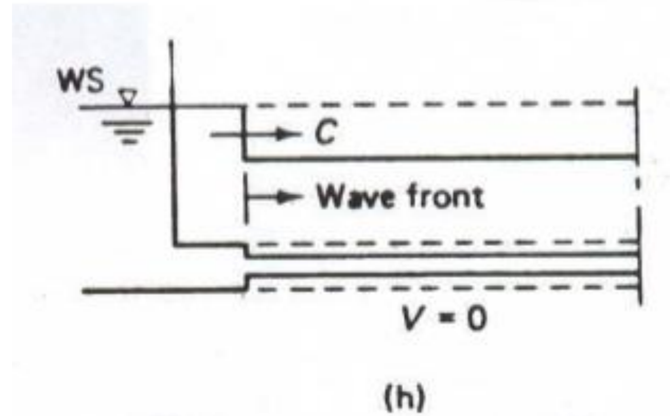


(f)

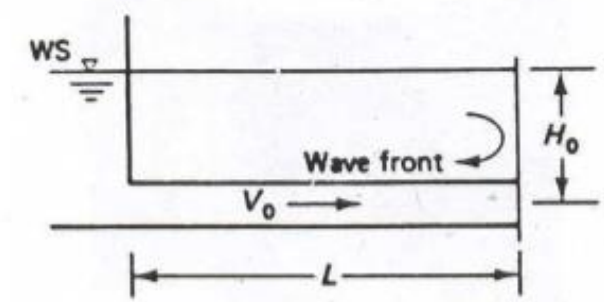
Elastic water column (EWC) theory

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

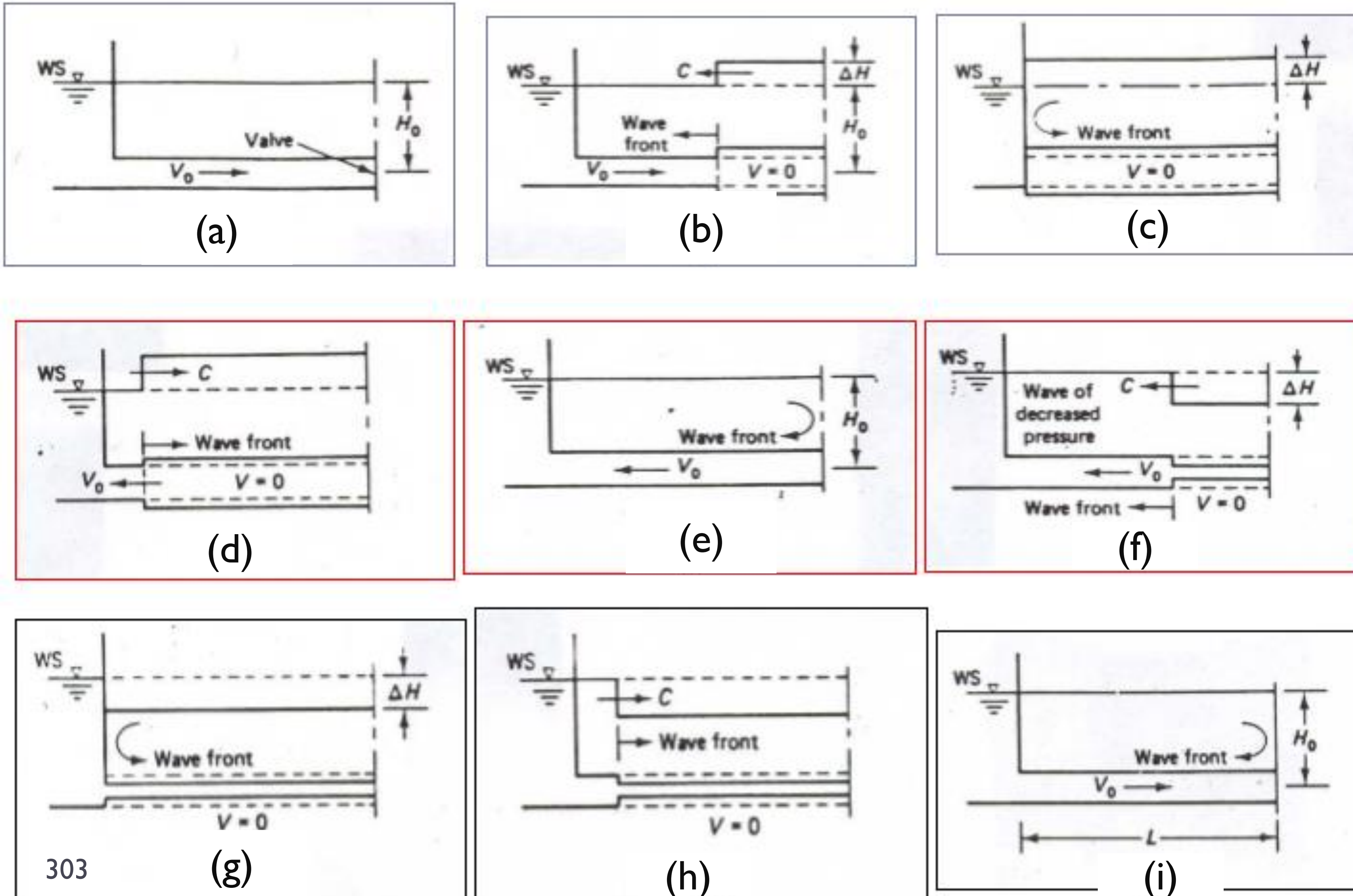




i. One full cycle, 4L/c

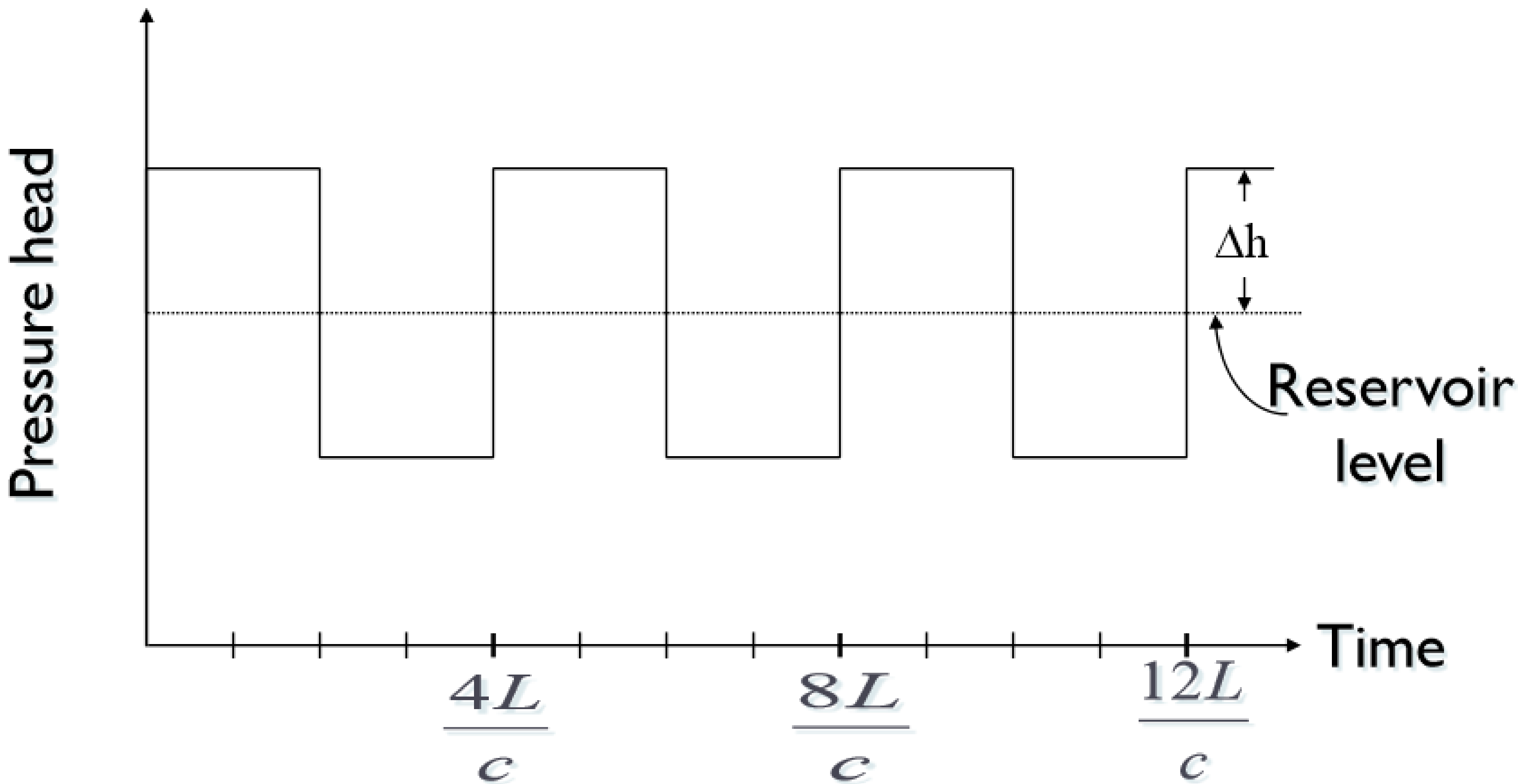


Elastic water column (EWC) theory



Elastic water column (EWC) theory

Pressure variation at valve: velocity head and friction losses neglected



Elastic water column (EWC) theory

- 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 a decreasing amplitude as shown in Figure B.

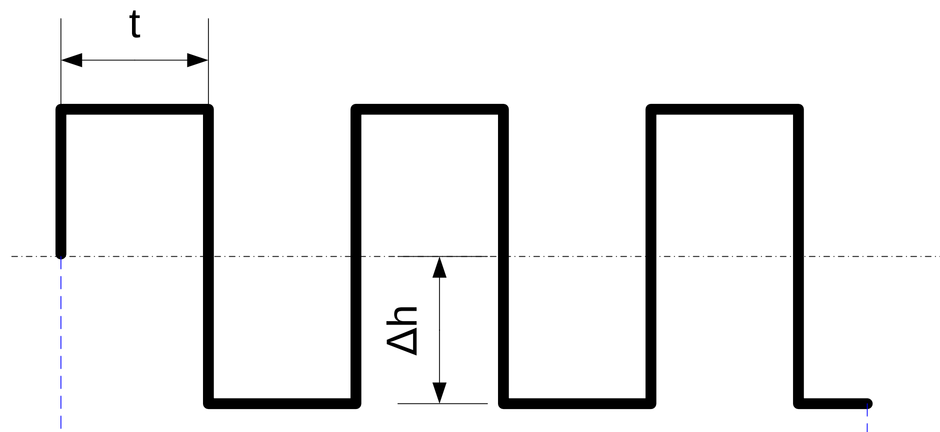


Figure: A

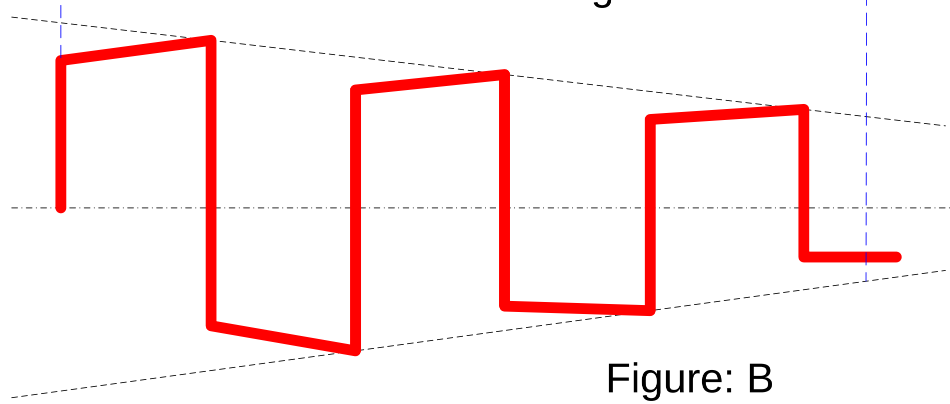


Figure: B

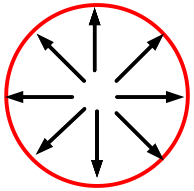
Elastic water column (EWC) theory

- Let Δh = increase in head relative to original head (assume frictionless flow)
- Assume v_o is the 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) 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
- E = modulus of elasticity of pipe material; K = bulk modulus of elasticity of fluid; c = speed of pressure wave, celerity

Elastic water column (EWC) theory

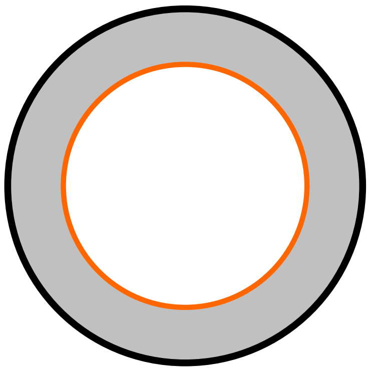
- Change in Pipe Volume due to added lateral stress

$$\sigma = \frac{\Delta pD}{2t} = \frac{\gamma \Delta hD}{2t}$$



$$\sigma = \frac{pr}{t}$$

- Lateral strain: Hooke's Law $\varepsilon = \frac{\sigma}{E} = \frac{\gamma \Delta hD}{2tE}$
- Generates elongation of diameter to D + εD
- Pipe area increase: $\Delta A = \frac{\pi}{4}(D + \varepsilon D)^2 - \frac{\pi}{4}D^2$



$$= \frac{\pi}{4}(D^2 + 2\varepsilon D^2 + \varepsilon^2 D^2) - \frac{\pi}{4}D^2$$

$$= \frac{\pi}{4}(2\varepsilon D^2) = 2\varepsilon(\frac{\pi}{4}D^2) = 2\varepsilon A$$

Elastic water column (EWC) theory

- The ε^2 term is dropped (very small) giving: Pipe area increase:

$$\Delta A = 0.5\pi D^2 \varepsilon = 2A\varepsilon$$

Over the length, L, the change in pipe volume is:

$$\Delta Vol_{pipe} = 2LA\varepsilon = \frac{LA\gamma\Delta hD}{tE}$$

- Reduction in the volume of water within the pipe is based on compressibility of water

$$K = \frac{dp}{\left(\frac{d\rho}{\rho}\right)} = -\frac{dp}{\left(\frac{dVol}{Vol}\right)}$$

Defined in terms of relative decrease in volume

Elastic water column (EWC) theory

$$\therefore \Delta V_{water} = \frac{V\Delta p}{K} = \frac{LA\gamma\Delta h}{K}$$

- Volume of water added = increase in pipe volume + increase in volume due to compression

$$\frac{LAV_o}{c} = \frac{LA\gamma\Delta hD}{tE} + \frac{LA\gamma\Delta h}{K} \Rightarrow \frac{v_o}{c} = \frac{\gamma\Delta h}{K}\left(1 + \frac{K}{E}\frac{D}{t}C_1\right)$$

C_1 coefficient added to enable evaluation of different pipe support systems

Elastic water column (EWC) theory

- Momentum equation: Now we need second equation relating c and Δ h. From control volume:
- Over time period Δt the pressure wave moves a distance Δx, mass of water stopped during this time is $c\Delta t A\rho$
- The velocity change is $-v_o$
- Therefore time rate of change in momentum is $\rho A v_o c$

$$-\gamma \Delta h A = (0 - \rho A v_o c) - \rho A v_o c \Rightarrow \Delta h = \frac{c v_o}{g}\left(1 + \frac{v_o}{c}\right)$$

$$for\ c >> v_o \qquad \Delta h = \frac{c v_o}{g} \quad or \quad \gamma \Delta h = \rho c v_o$$

Elastic water column (EWC) theory

- Substitute for Δh into volume balance

$$c = \frac{K}{\rho}\left[1+\left(\frac{K}{E}\right)\left(\frac{D}{t}\right)C_1\right]^{-0.5}$$

- C_1, pipe support conditions:

- Pipes free to move longitudinally (Gupta case 3)

$$C_1 = (1.25 - \varepsilon)$$

- Pipes anchored at both ends against longitudinal movement (Gupta case 2)

$$C_1 = (1 - \varepsilon^2)$$

- Pipes with expansion joints (Gupta Case 4)

$$C_1 = 1$$

- ε = Poisson's Ration, 0.25 for common pipe materials

Elastic water column (EWC) theory

- 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:

$$h_{wm} = \frac{v_o c}{g}$$

Elastic water column (EWC) theory

Example:

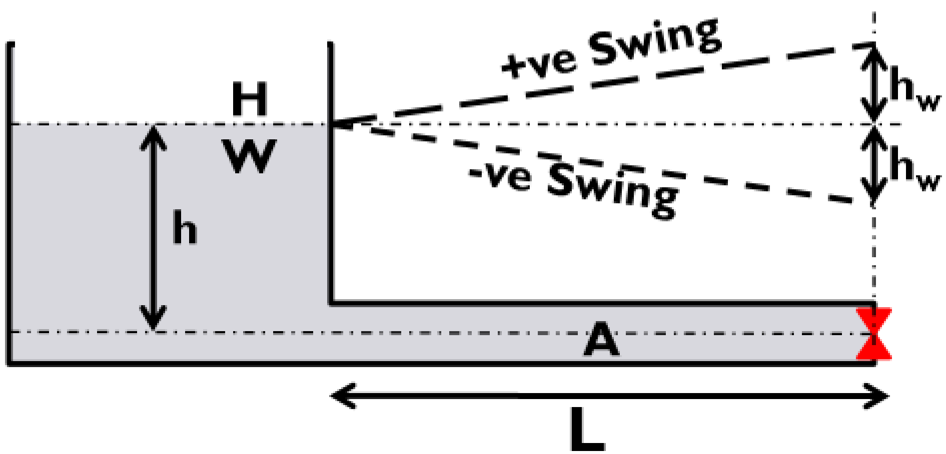
- How long does it take for a pressure wave to travel 500 m after a rapid valve closure in a 1m diameter pipe with expansion joint, 1cm wall thickness, steel pipeline? The initial flow velocity was 5m/s. E for steel is 200 Gpa. K is 2.2 Gpa. What is the increase in pressure?

Solution:

- $c = 1066.3$m/s
- $500/1066.3 = 0.47$ seconds to travel
- $h_{wm} = 543.5$ m or increase in pressure = 5331.5 KN/m^2

$$c = \left[\rho\left(\frac{1}{K} + \frac{C_1 d}{tE}\right)\right]^{-0.5} \qquad h_{wm} = \frac{v_o c}{g}$$

Rigid water column (RWC) theory



- If the head losses in the pipe is 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 doted 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.

Rigid water column (RWC) theory

- 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:

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

Rigid water column (RWC) theory

- If the total time of closure of the valve is T, then for complete closure, assuming uniform gate movement, the maximum water hammer head h_{wm} is given by:

$$\frac{h_{wm}}{h} = \frac{k_1}{2} \pm \sqrt{k_1 + 0.25k_1^2}\,; \qquad k_1 = \left(\frac{Lv_o}{ghT}\right)$$

- The velocity of the pressure wave, c, in a pipe according to RWC theory is given by the following formula:

$$c = \sqrt{\frac{K}{\rho}}$$

Where:

ρ = density of the water

K = Bulk volume modulus of elasticity of water

Rigid water column (RWC) theory

Limitations of RWC:

- Ignores elastic property due to sudden pressure rise
- In the above derivation friction effects were ignored and the conduit was assumed to have equal x-section.
- In case of variable x-sectioned conduit, it shall be converted in to an equivalent conduit:

$$L = \sum L_i\,; \qquad \frac{L}{A} = \sum \frac{L_i}{A_i}$$

Rigid water column (RWC) theory

Example:

- Find the speed of a pressure wave in a water pipeline assuming rigid walls.
  - K=2.2 Gpa (Bulk modulus of elasticity of water)
  - ρ = 1000 kg/m^3

$$c = \left(\frac{K}{\rho}\right)^{0.5} = \left(\frac{2.2\times10^9}{1000}\right)^{0.5} = 1483.24 m/s$$

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.
- 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

- Full pressure wave does not develop

$$\frac{2L}{c} < t_c < \frac{20L}{c}$$

$$\Delta h_{est} = \frac{cv_o}{g} \times \frac{2L/c}{t_c}$$

- Rapid Valve Closure

$$t_c < \frac{2L}{c}$$

$$\Delta h = \frac{cv_o}{g}, \quad or \quad \Delta p = \rho c v_o$$

- Slow Closure after Lorenzo & Allievi:

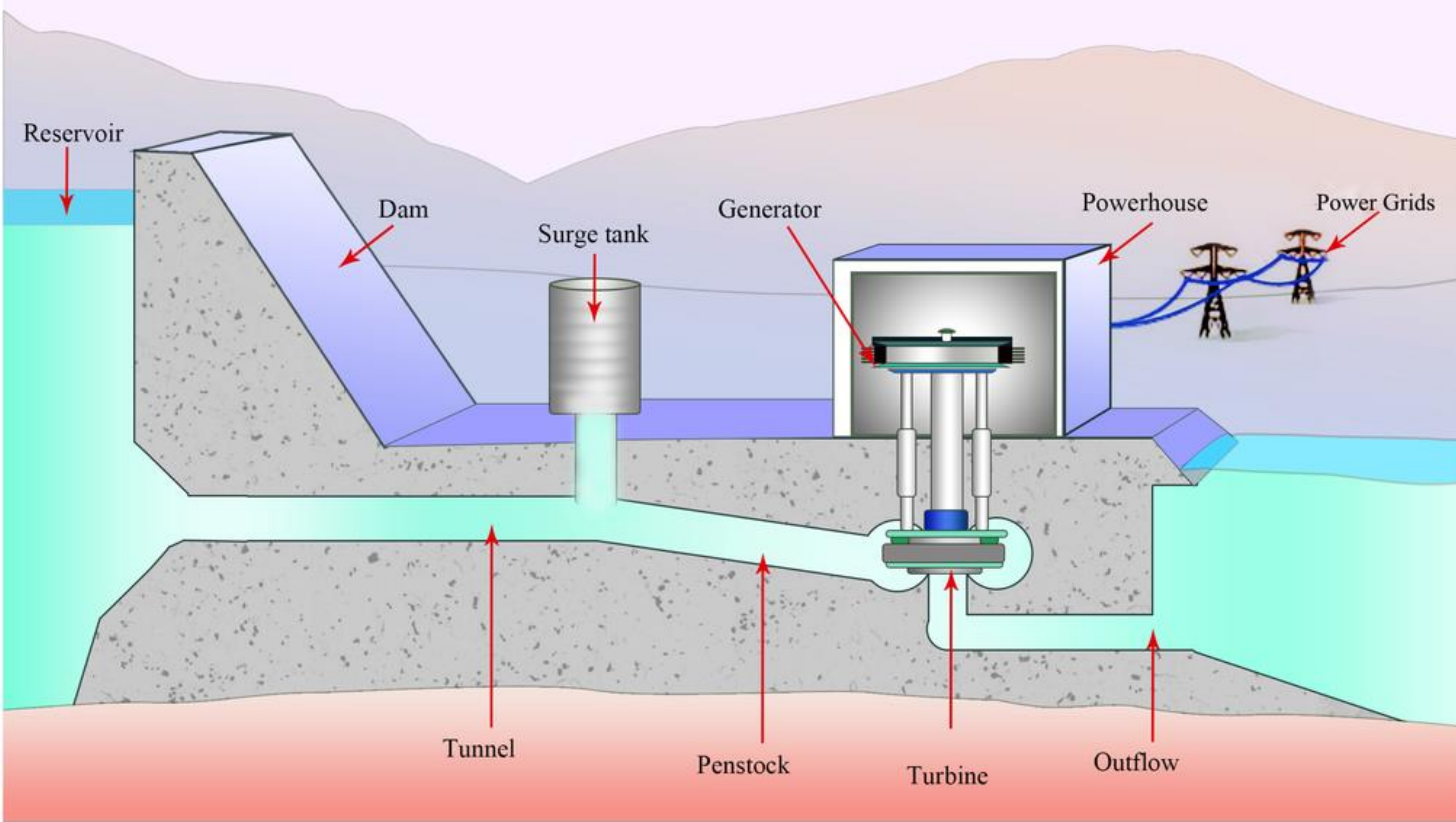
$$\Delta p = p_o\left(0.5N + \sqrt{0.25N^2 + N}\right) \qquad where\ N = \left(\frac{\rho L v_o}{p_o t_c}\right)$$

Positive pressure: Pipe explode

Negative pressure: Pipe collapse

Surge Tanks

Surge Tanks


Reservoir
Dam
Surge tank
Generator
Powerhouse
Power Grids
Tunnel
Penstock
Turbine
Outflow

Surge Tanks

Surge Tanks

Surge Tanks

- A surge tank (or surge chamber or expansion chamber) is a device introduced within a hydropower water conveyance system having a rather long pressure conduit to absorb the excess pressure rise in case of a sudden valve closure.

- It also acts as a small storage from which water may be supplied in case of a sudden valve opening of the turbine.

- If there is no surge tank, in case of a sudden opening of turbine valve, there are chances of penstock collapse due to a negative pressure generation

Surge Tanks

A surge tank serves a threefold purpose:

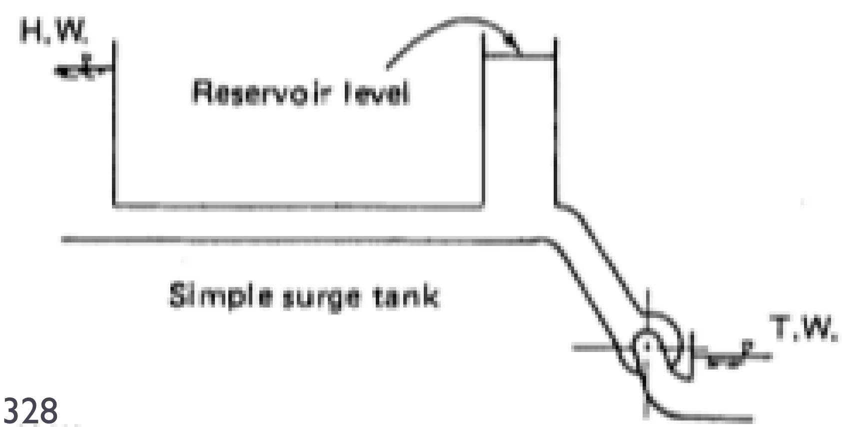
- Flow stabilization to the turbine,
- Water hammer relief or pressure regulation, and
- Storage function

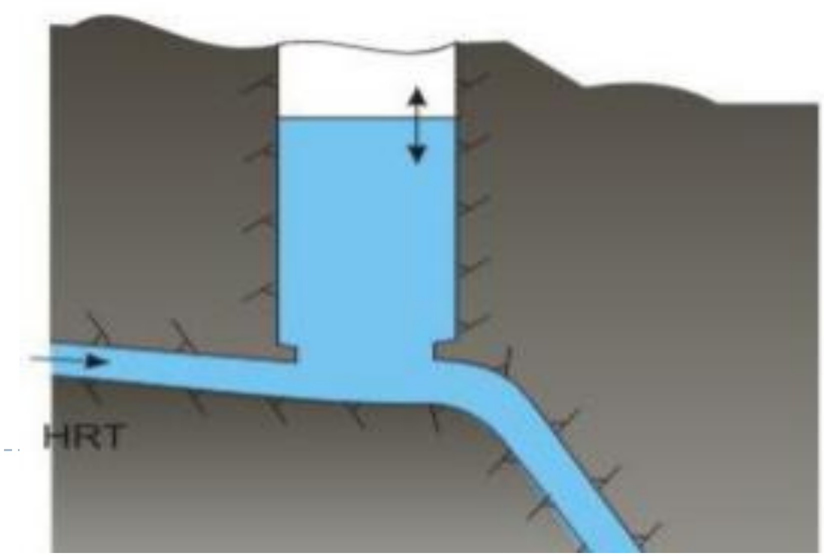
- In a practical sense, a rule of thumb that might be applied to determine whether a surge tank or a relief valve may be needed is that, extra caution should be exercised to evaluate pressure rise or decrease in systems where the water conduit total length equals or exceeds the head by a factor of 3.

Types

1. Simple Surge Tank: is a vertical standpipe connected to the penstock or pressure tunnel with an opening large enough (not less than the area of HRT) 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.

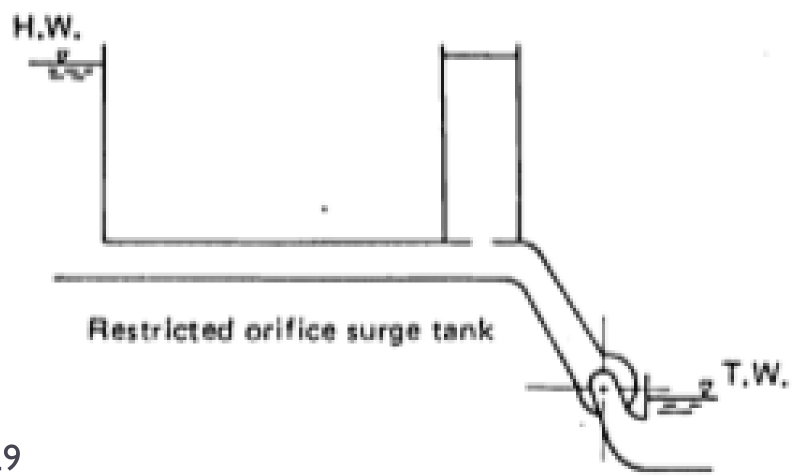


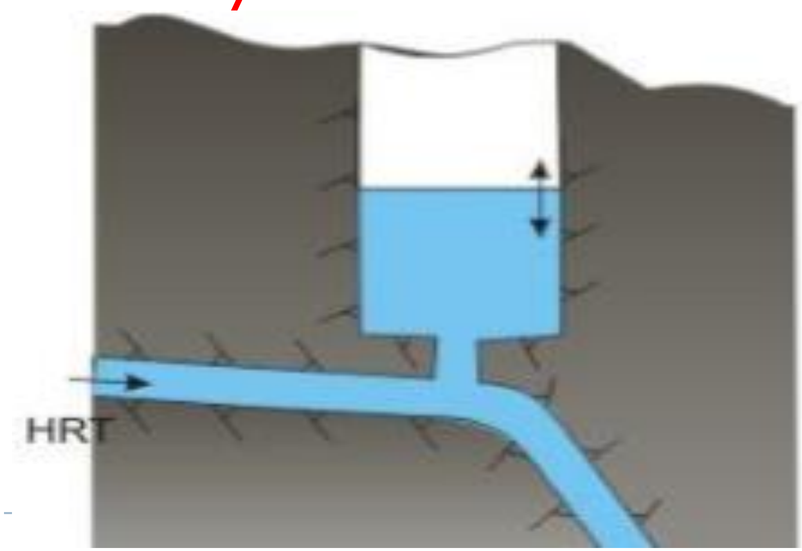


Types

2. Restricted Orifice Surge Tank: its a simple surge tank in w/c the inlet is throttled to improve damping of oscillations by offering greater resistance and connected to the head race tunnel with or without a connecting/communicating shaft

- A restricted opening b/n the tank and the penstock/HRT develops appreciable head loss in the water that flows into or out of the tank. It is more hydraulically stable

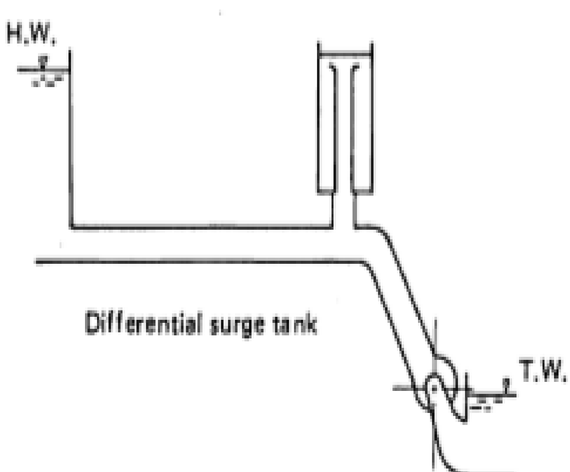


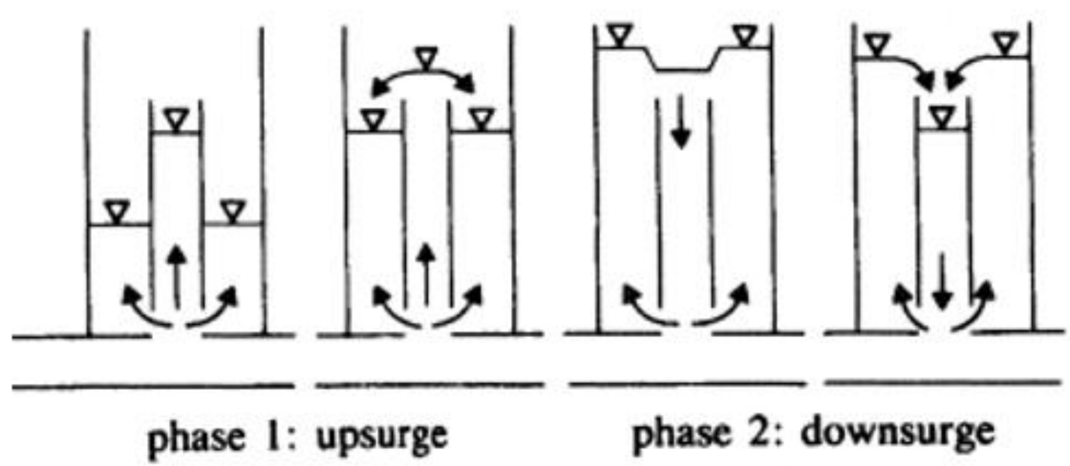


Types:

3. The differential surge tank: is a combination of a simple tank and a restricted-orifice tank.

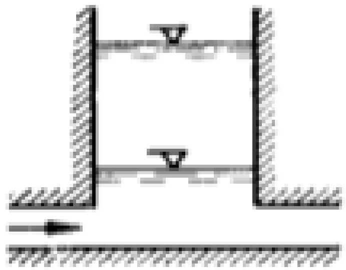
- An internal riser of smaller diameter than the full connection to the penstock is built to extend up through the tank while an outer tank is connected by a simple pipe connection to the penstock.
- The riser may also have a flow restrictor or orifice inside. Thus one part of the tank responds with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



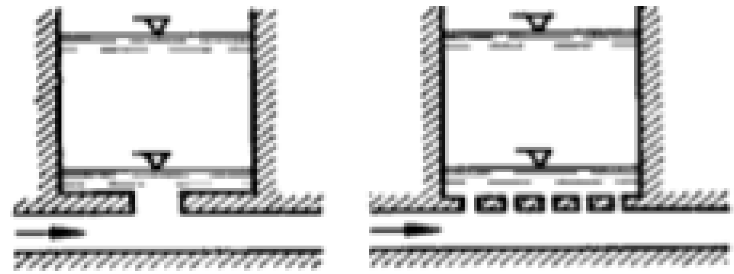


Surge Tanks

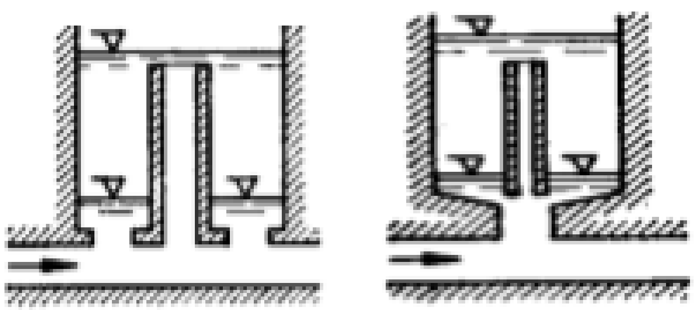
Types:



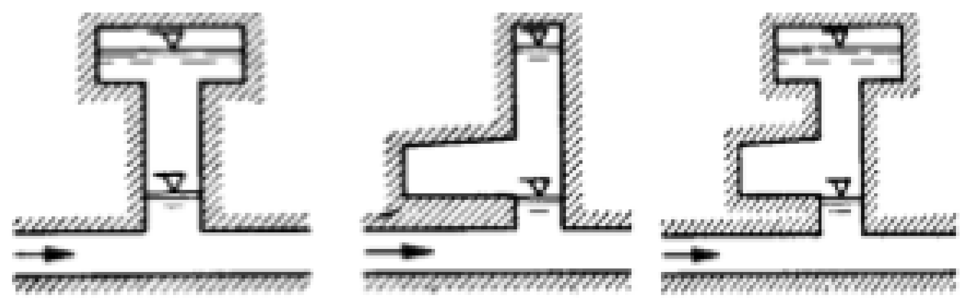
a) Simple surge tank



b) Restricted orifice surge tank



c) Differential surge tank

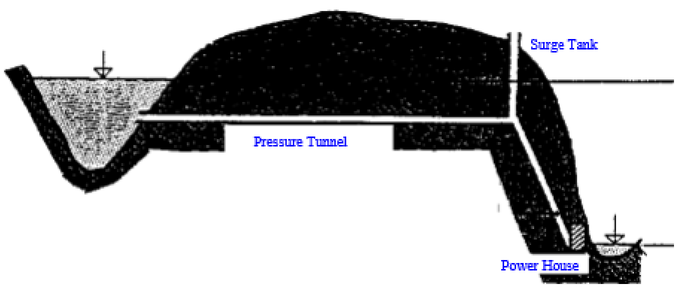


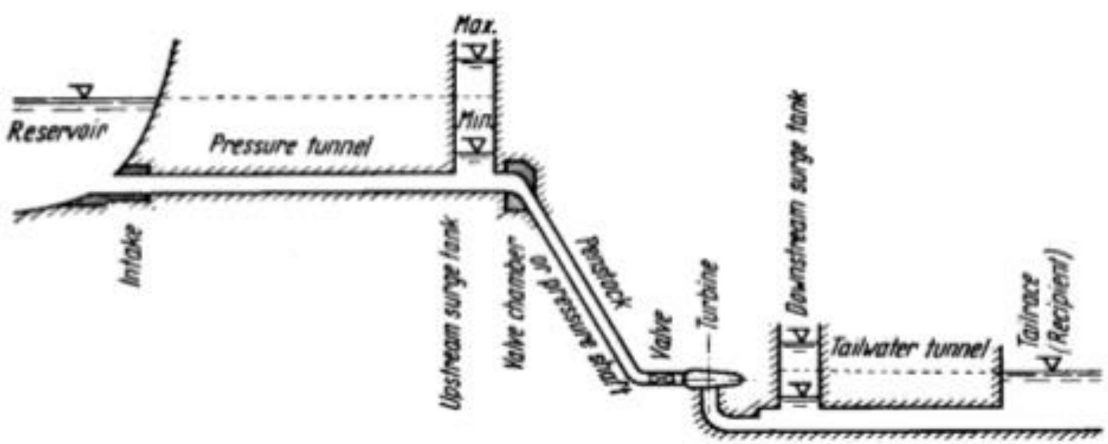
d) Surge tanks with expansion chambers

Types:

Surge tanks may be classified according to :

- Material of construction: Concrete or steel
- Location relative to terrain: Recessed (Excavated) & free-standing (exposed)
- Location in the hydraulic system: u/s & d/s





Hydraulic Design of Simple Surge Tank

- The hydraulic design of surge tank concerns with two main aspects: Height and Cross-sectional area.
- These aspects are decided up on with the view to fulfilling the following criteria:
  - The surge tank must be locates so that the positive & negative water hammer pressures are kept within acceptable limits.
  - The tank must be stable i.e. water surface oscillation must be damped out
  - The tank must accommodate maximum upsurge & lowest down surge

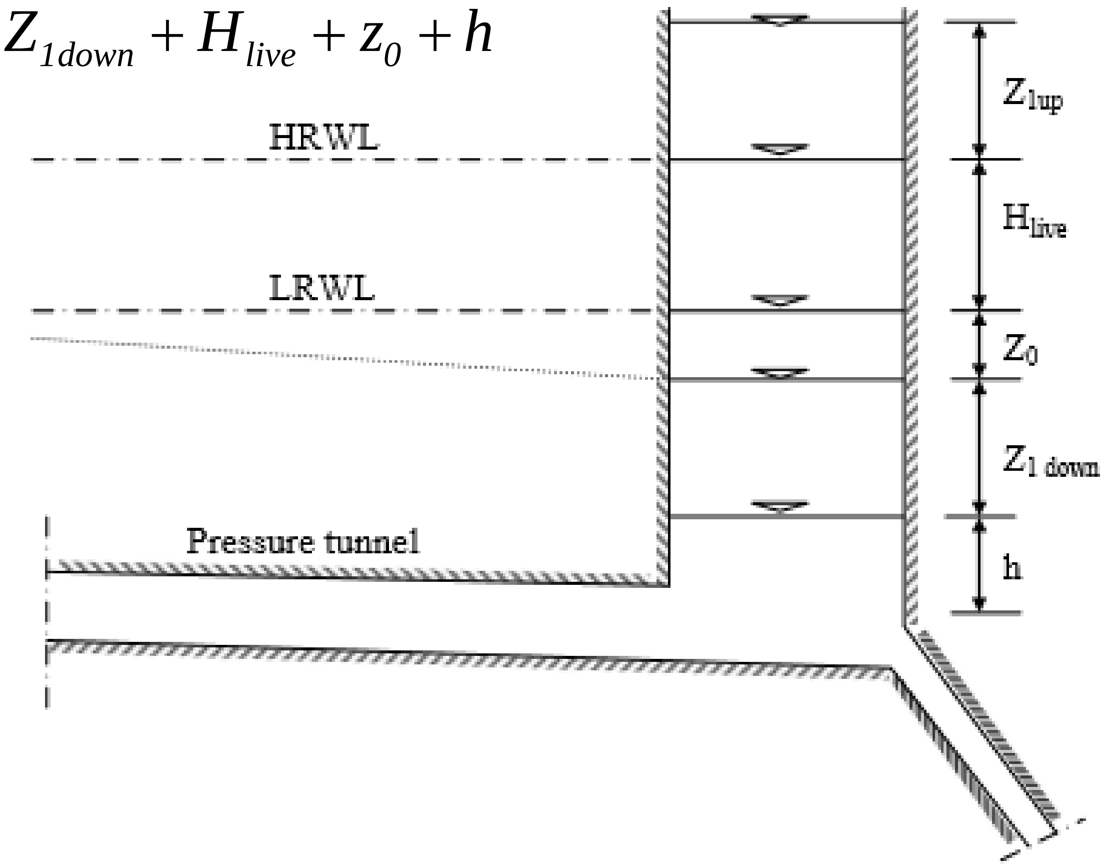
Hydraulic Design of Simple Surge Tank

Height

- The total height of the surge tank should be such that both the maximum upsurge and down-surge is contained within the surge tank height.
- Worst conditions should be considered to determine the height.
- For up-surge, the worst conditions are:
    - Instantaneous total closure
    - Reservoir level at its maximum
- For down-surge, the worst conditions are:
    - Instantaneous total opening
    - Reservoir level at its minimum

Hydraulic Design of Simple Surge Tank

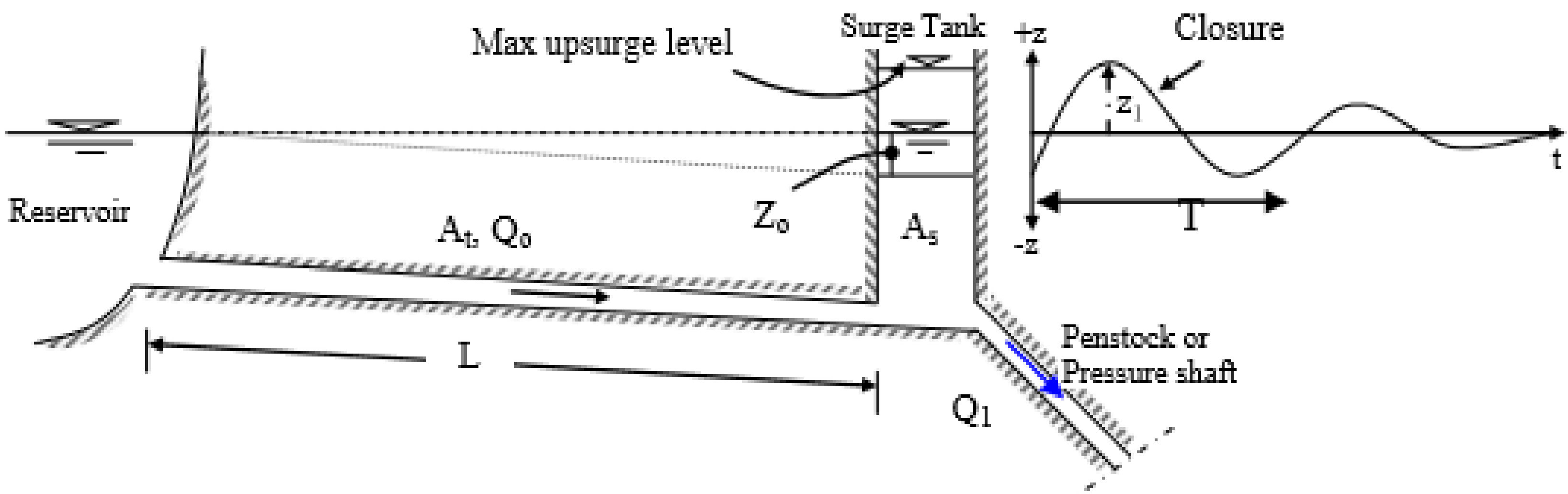
Height $$H_{tank} > Z_{1up} + Z_{1down} + H_{live} + z_0 + h$$



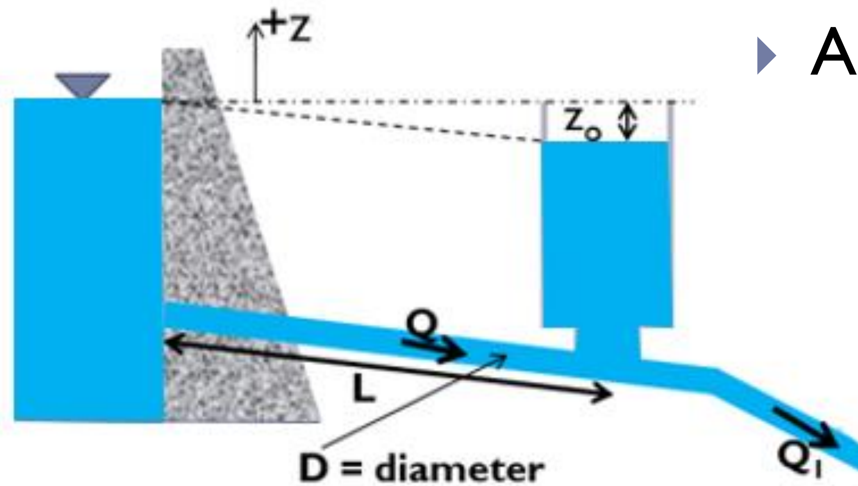
- The lowest possible level of down surge must be sufficiently above the conduit top level by a certain height, h in order to avoid vortex formation at entrance to the penstock/pressure shaft.

Hydraulic Design of Simple Surge Tank

- In order to determine the surge height and thus necessary tank height, etc., it is necessary to carry out water hammer analysis and determine corresponding surge heights under various closure and opening (load rejection and acceptance) conditions



Hydraulic Design of Simple Surge Tank



- Applying continuity equation:

$$-Q = A_s \frac{dz}{dt} + Q_1; \qquad Q = v_t A_t$$

$$-\frac{dz}{dt} = \frac{(v_t A_t - Q_1)}{A_s} - - - - - - - Eq.1$$

- Where A_t & 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

$$z_o = f \frac{L}{D} \frac{v_t^2}{2g}$$

Hydraulic Design of Simple Surge Tank

- Applying Energy equation:

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

$$Where \quad F_t = \frac{fLv_t^2}{2gD}$$

- If throated head loss in the throat is also considered:

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

- From Eq. 1 if Q_l is zero:

$$\frac{dz}{dt} = \frac{v_t A_t}{A_s} \Rightarrow \frac{d^2 z}{d^2 t} = \frac{A_t}{A_s}\frac{dv_t}{dt} --------- Eq.3$$

Hydraulic Design of Simple Surge Tank

- 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.

- For simplest case of full closure and negligible friction the solution to eq. 2 has the following form:

$$z = z_{\max} \sin\left(\frac{\mathrm{t}}{\mathrm{T}}\right); \quad Where \;\; z_{\max} = v_t \sqrt{\frac{A_s L}{A_t g}}; \;\; T = \sqrt{\frac{A_s L}{A_t g}}$$

- Where Z_{max} is the maximum up surge or down surge calculated by neglecting friction effect.

Hydraulic Design of Simple Surge Tank

- However; Jaeger has recommended use of the following approximate formula:

  i. For sudden 100% load rejection, maximum upsurge and down surge for $k_o < 0.7$ will be:

$$\frac{z_{up}}{z_{\max}} = 1 - \frac{2}{3}k_o + \frac{1}{9}k_o^2; \quad k_o = \frac{F_t v_t^2}{z_{\max}}; \quad z_{\max} = v_t\sqrt{\frac{A_s L}{A_t g}} \qquad \text{for } k_o < 0.7$$

$$\frac{Z_{down}}{Z_{max}} = \frac{-1}{1 + \frac{7}{3}k_o} \qquad \text{for } k_o < 0.7$$

  ii. For 100% load demand, the maximum down surge

$$\frac{Z_{down}}{Z_{max}} = -1 - 0.125k_o \qquad \text{for } k_o < 0.8$$

Hydraulic Design of Simple Surge Tank

- Calame and Gaden have given the following approximate formula suitable for computation of the lowest water reached after the first up ward swing.

$$\frac{Z_{down}}{Z_{max}} = -1 + 2k_o$$

Hydraulic Design of Simple Surge Tank

Cross-sectional Area (Stability Consideration)

- The required cross-sectional area of a surge tank is determined based on stability considerations for the surge oscillations in the tank.
- Stability conditions of the surge system were established by Thoma.
- He stated that in order to prevent the development of unstable oscillations the cross-section of the surge tank should exceed a certain critical magnitude.

Hydraulic Design of Simple Surge Tank

Cross-sectional Area (Stability Consideration)

- According to Thoma, the limiting x-sectional area for small oscillation is given by:

$$A_{s\,min} = \frac{v_t^2 \cdot A_t \cdot L_t}{2g \cdot h_f \cdot H_n} \quad (m)$$

h_f = head loss up to surge tank

$H_n = H - h_f$ net head on turbine

$$A_{s\,min} = \frac{m^2 \cdot R^{4/3} \cdot A_t}{2g \cdot H_n} = \frac{m^2 \cdot D^{10/3} \cdot A_t}{160 H_n} \quad (m)$$

$$m = \frac{1}{n}$$

n = Manning's coefficient

R = Hydraulic Radius

$$A_s = (1.5 \, to \, 1.8) A_{s\,min}$$

For tank stability

Hydraulic Design of Simple Surge Tank

Cross-sectional Area (Stability Consideration)

- For large amplitude of oscillation, the Thoma formula was modified by Ch Jaeger as:

$$A_s = \eta^* \times \frac{L \times A_t}{2g \times \beta \times H_n} = \eta^* \times \frac{m^2 \times D^{4/3} \times A_t}{2g \times H_n}$$

$$\eta^* = non\,constant\ factor\,of\ safety = 1 + 1.0482\frac{Z_{max}}{H_n}$$

$$\beta = \frac{1}{m^2 R^{4/3}}$$

or

$$A_s = \frac{\eta^* m^2 \times D^{4/3}}{160 \times H_n} = 170.482\frac{Z_{max}}{H_n}$$

Hydraulic Design of Simple Surge Tank

Example:

- A surge chamber 100m^2 in area is situated at the downstream end of a low pressure tunnel 10 km long and 5m in diameter. While, discharging 60m^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:

Given: A_s = 100m^2; A_t = 19.635m^2; v_t = Q/A_t = 3.056m/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

Hydraulic Design of Simple Surge Tank

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

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

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

$$v_t A_t = A_s v_s + Q_1 = Q; \;\; But \;\; Q_1 = 0 \Rightarrow v_s = 0.6$$

$$v_s = \frac{dz}{dt} = 0.6 \Rightarrow z = 0.6t + c$$

$$at \;\; t = 0, \;\; z = -9.519m \Rightarrow c = -9.519m$$

$$z = 0.6t - 9.519$$

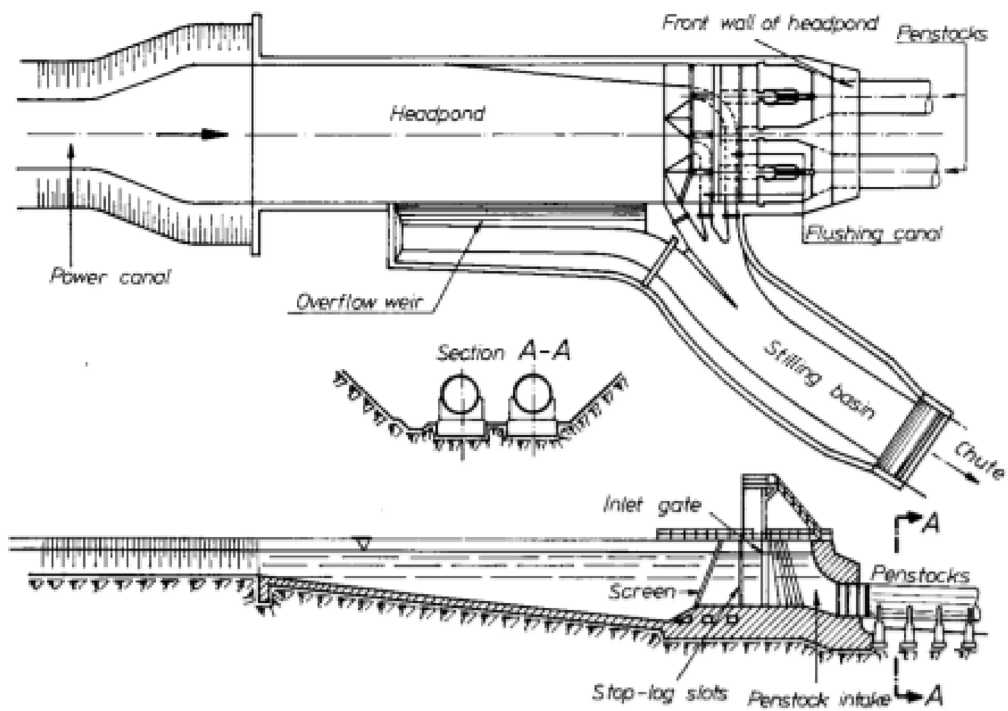
$$thus \; for \; z = 37.12, \;\; t = 77.732 \, sec$$

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.
- 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.



Forebay Design Guidelines

- 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 vary 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.

Forebay Design Guidelines

- 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.

Forebay Design Guidelines

- 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.

Penstock

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.

Penstock

Penstocks may be classified according to type of construction:

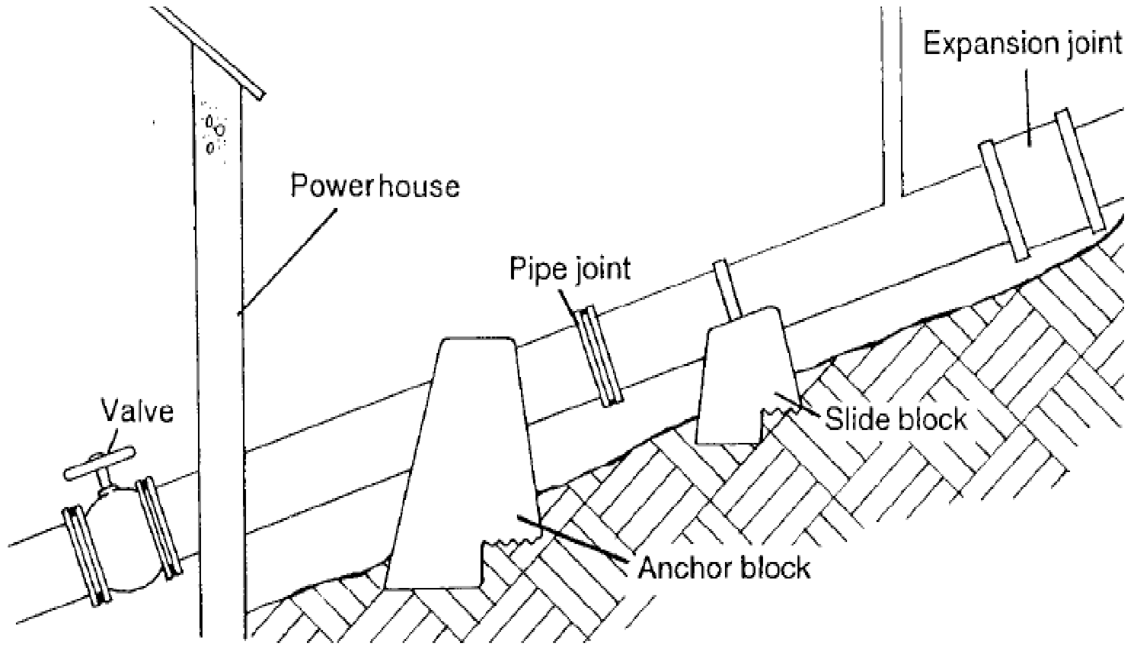
- **Steel penstocks:** the most common type of installation, due to simplicity in fabrication, strength, and assurance that they will perform in a wide variety of circumstances. Normal practice is to use welded steel pipe sections.

- **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.

Penstock


Expansion joint
Powerhouse
Pipe joint
Valve
Slide block
Anchor block

Safe Penstock Thickness

- The thickness of the pipe shell (s) for penstocks should be determined by:

$$s = \frac{PD}{2\eta\sigma}$$

Where:

s = penstock shell thickness;
P = internal pressure;
D = pipe diameter;
η = joint efficiency of welded or riveted joint;
σ = 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:

$$s_{min} = \frac{D + 20}{400}$$

Size Selection of Penstocks

- Various experience curves and empirical equations have been developed for determining the economical size of penstocks.
- 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:

$$D_p = 0.72Q^{0.25}$$

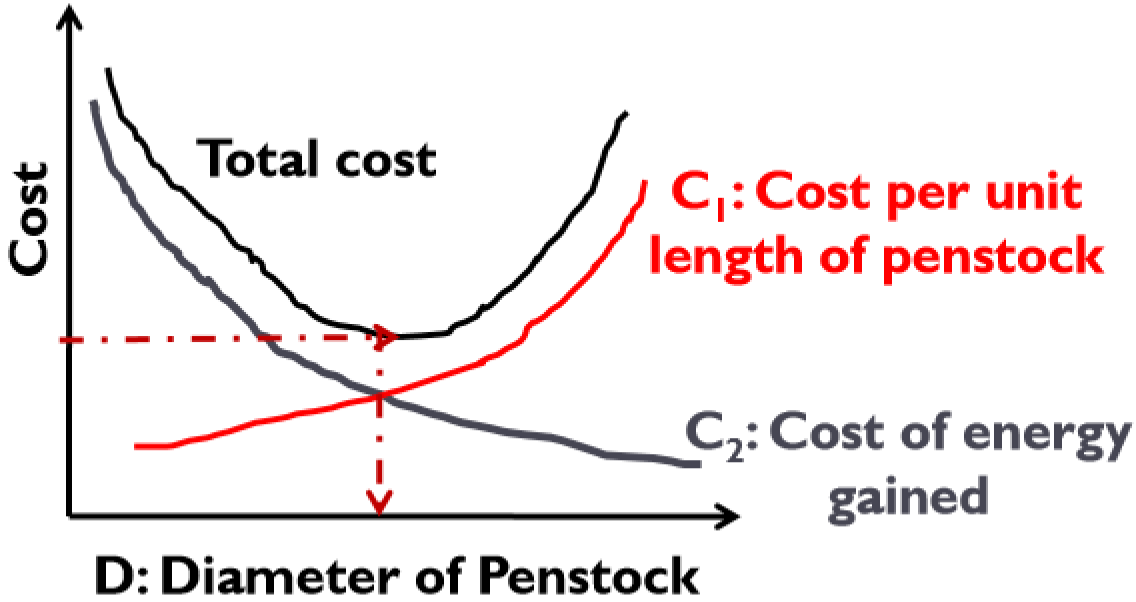
Size Selection of Penstocks

- 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.

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

Optimization of Penstock Diameter

- Thickness of steel pipe: $s = \dfrac{PD}{2\sigma} = \dfrac{(g.\rho.h)D}{2\sigma}$
- Weight of steel pipe: $G = g\rho_s.\pi.D.s.L$

$$G = g\rho.g\rho_s \frac{\pi.h.D^2.L}{2\sigma}$$ Substituting for s:

- Where
  - ρ_s – density of steel … 7850 kg/m^3
  - ρ - density of water …. 1000 kg/m^3
  - h - head acting on the penstock
  - σ - allowable strength of steel
  - L - length of penstock

Optimization of Penstock Diameter

- Substituting the values:

$$G = 9.81 \times 1000 \times 9.81 \times 7850 \times 3.14 \times 0.5 \times \frac{h \cdot D^2 \cdot L}{\sigma}$$

$$G = 1187 \times 10^6 \times \frac{h \cdot D^2 \cdot L}{\sigma} \quad [N]$$

$$G = 1187 \times 10^3 \times \frac{h \cdot D^2 \cdot L}{\sigma} \quad [KN]$$

- Adding 20% for water hammer pressure surges

$$G = 1424 \times 10^3 \times \frac{h \cdot D^2 \cdot L}{\sigma} \quad [KN]$$

Optimization of Penstock Diameter

- The cost of energy lost corresponds to the power calculated by considering head losses, where h_l stands for losses, is:

$$P_{lost} = 9.81 \cdot \eta \cdot Q \cdot h_l$$

- Taking efficiency of say, 77% (this value can be adjusted for particular cases)

$$P_{lost} = 9.81 \cdot 0.77 \cdot Q \cdot h_l = 7.55 \cdot Q \cdot h_l$$

- Major head loss in the penstock is given as:

$$h_l = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g} = f \cdot \frac{L}{D} \cdot \frac{16 \cdot Q^2}{\pi^2 D^4 \cdot 2g} \approx \frac{f \cdot L \cdot Q^2}{12 \cdot D^2}$$

- Where f is the friction coefficient for the pipe.

Optimization of Penstock Diameter

- For a given load factor of say λ given in decimal number, the total operation hours in a year would be = 8760 x λ.
- Therefore, the annual energy generation would amount to:

$$E_{lost} = p_{lost} \times t = 7.55 \times Q \times h_1 \times 8760 \times \lambda$$

- Taking the cost of energy per kWh at the existing tariff to be c_e

$$C_2 = \frac{7.55 \cdot Q^3 \cdot L \cdot f}{12 \cdot D^5} \cdot 8760 \cdot \lambda \cdot c_e = \left(0.63 \cdot \frac{Q^3 \cdot L \cdot f}{D^5} \right) \cdot 8760 \cdot \lambda \cdot c_e$$

- Now applying the differentiation:

$$\frac{dC_1}{dD} + \frac{dC_2}{dD} = 0$$

Optimization of Penstock Diameter

$$2848 \cdot 10^3 \cdot \frac{h \cdot L \cdot \alpha \cdot c_o}{\sigma} \cdot D + 0.63 \cdot (-5) \cdot \frac{f \cdot L \cdot Q^3}{D^6} \cdot 8760 \cdot \lambda \cdot c_e$$

$$2848 \cdot 10^3 \cdot \frac{h \cdot L \cdot \alpha \cdot c_o}{\sigma} \cdot D = 27594 \cdot \frac{f \cdot L \cdot Q^3}{D^6} \cdot \lambda \cdot c_e$$

$$D = \left[\frac{f \cdot Q^3 \cdot \sigma}{103.2 \cdot h \cdot \alpha \cdot c_o} \lambda \cdot c_e \right]^{1/7} = \left[0.0097 \frac{f \cdot Q^3 \cdot \sigma}{103.2 \cdot h \cdot \alpha \cdot c_o} \lambda \cdot c_e \right]^{1/7}$$

D – Optimum diameter of penstock (m);

f – friction coefficient in pipe;

σ – allowable strength of steel (N/m^2);

α – annual operating charges on the penstock including depreciation;

c_0 – unit cost of steel for finished penstock in USD per KN;

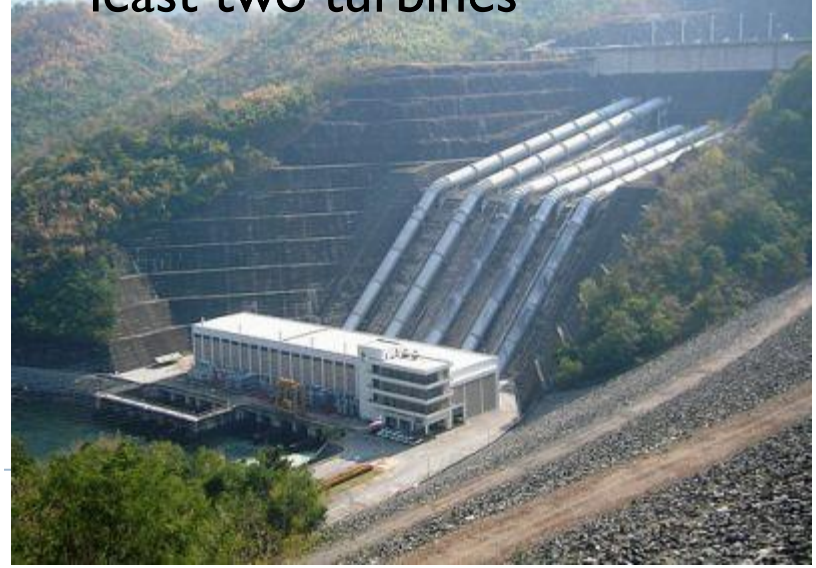
c_e – electric energy tariff in USD per kWh

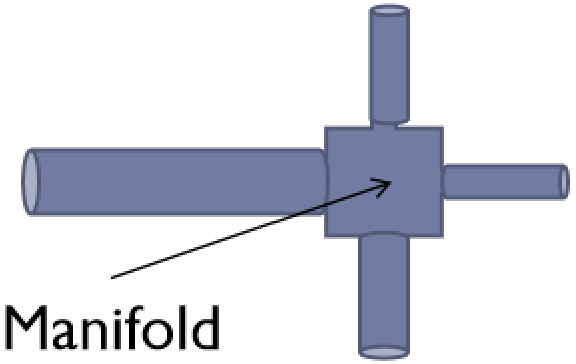
Q – discharge in m^3/s;

λ – load factor;

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 penstock as the number of turbines
  - To provide multiple penstocks but each penstock supplying at least two turbines





Number of penstocks

The main considerations which lead to the best choice include:

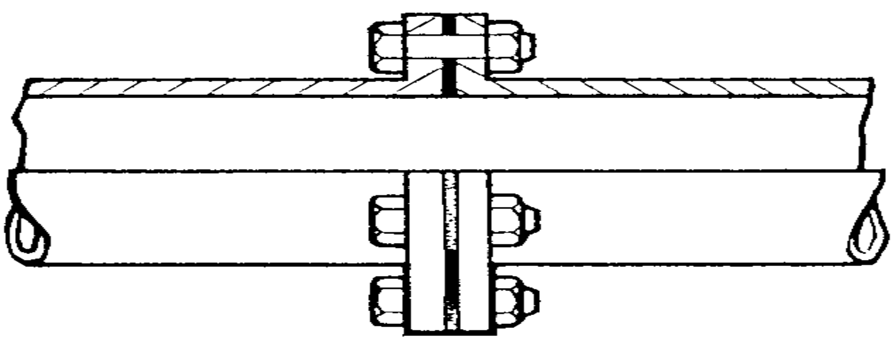
- **Economy:** Cost /alternative penstock design/
- **Operational safeguard:** in such case, a single penstock option is ruled out, as any damage to it will necessitate the total shut down of all the turbine
- **Transportation facilities:** the penstock are shop welded in short section and then transported 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

Penstock Joints

- The following factors should be considered when choosing the best jointing system.
  - Relative costs
  - Ease of installation
  - Suitability pipe material
  - Degree of joint flexibility
- Methods of pipe jointing:
  - Flanged joints
  - Spigot and socket joints
  - Mechanical joints
  - Welded joints

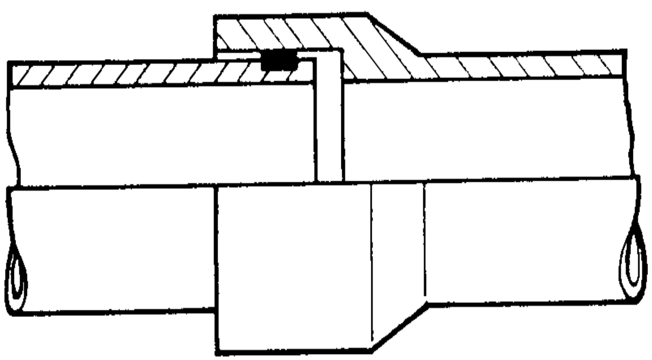
Flanged Joints

- Flanges are fitted to each end of individual pipes during manufacture, and each flange is then bolted to the next during installation.
- Flange jointed pipes are easy to install, but flanges can add to the cost of the pipe.
- Flange joints do not allow any flexibility.
- They are generally used to join steel pipes.



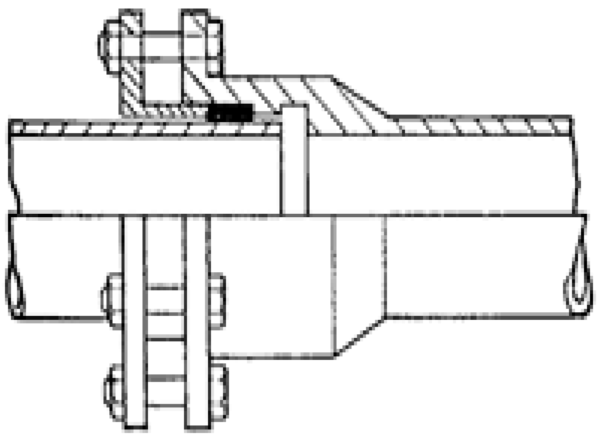
Spigot and Socket Joints

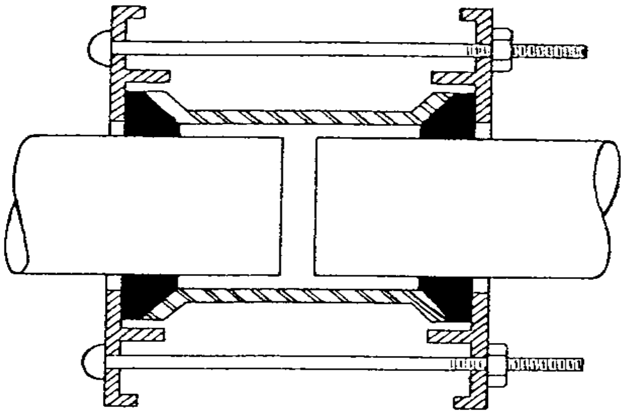
- Spigot and socket joints are made by increasing the diameter during manufacture of one end of each pipe
- The plain end of each pipe can be pushed into the collar or ‘socket’ in the next.
- Spigot and socket joints are generally used to join ductile iron, PVC, concrete and asbestos cement pipes



Mechanical Joints

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





Welded Joints

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

Penstock Supports and Anchors

- Slide blocks, anchors, and thrust blocks all serve the same basic function – to limit movement of the penstock.

Slide Blocks:

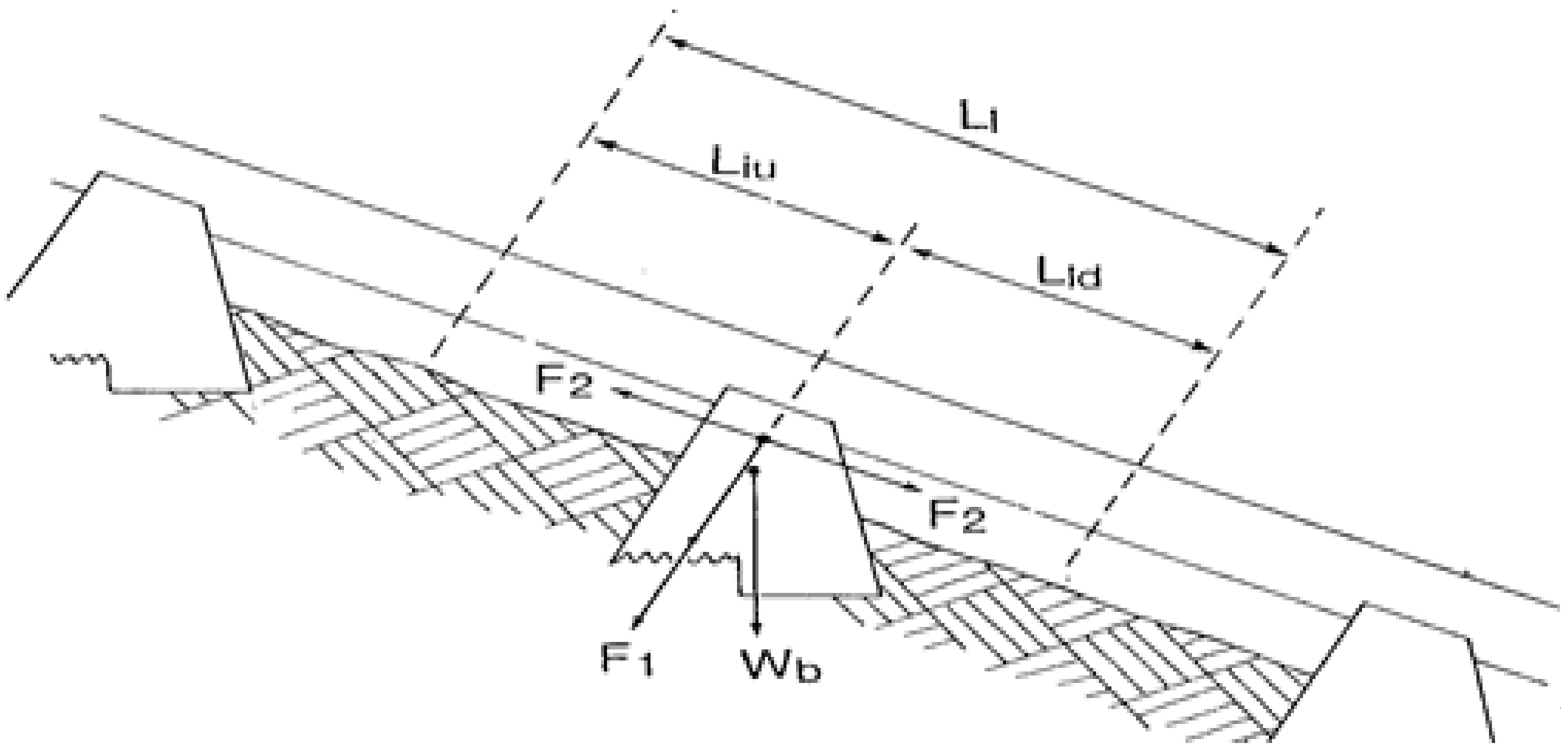
- A slide block, also called supporting pier, carries the weight of pipe and water, and prevent the pipe from upward and sideway movements, but allows it to move longitudinally.

Forces act on slide blocks include:

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

Penstock Supports and Anchors

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



Penstock Supports and Anchors

Anchor Blocks

- An anchor block consists of a mass of reinforced concrete keyed to the penstock so that the penstock cannot move in any direction relative to the block.
- It is designed to withstand any load of the penstock may exert on it.
- Anchors are often used at bends (horizontal and vertical).
- They can also be used along long straight sections of penstock, each one next to expansion joint.

Penstock Supports and Anchors

Anchor Blocks

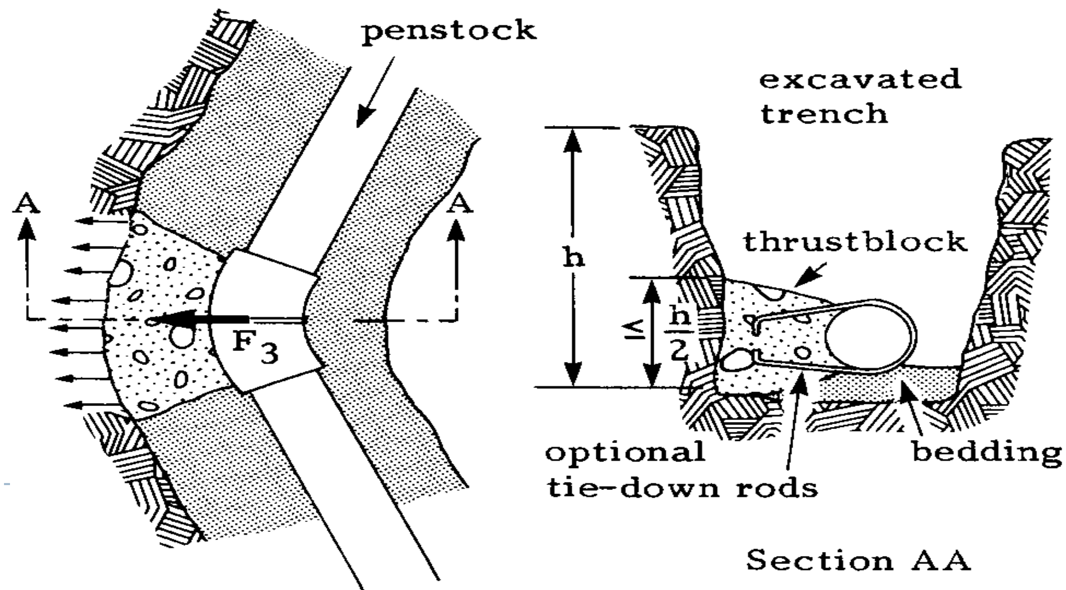
- The major forces which act on anchor blocks are the following:
  - Weight of the pipe and enclosed water
  - Hydrostatic force on a bend
  - Friction forces on slide blocks located between the anchor and expansion joint
  - Thermally induced stresses, when expansion joints are not incorporated
  -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

Penstock Supports and Anchors

Thrust Blocks

- Thrust Blocks are a special form of anchor whose only purpose is to transmit forces primarily caused by hydrostatic pressures at horizontal bends along a buried penstock to undisturbed soil.

- However, if the bend is vertical, an anchor block is still used.



Penstock Supports and Anchors

- For any penstock support or anchor to be stable and fulfill its intended purpose.
- The structure should be safe against sliding. $\sum H < \mu \sum V$

  Where, μ is the coefficient of friction between the structure and the ground often assumed as 0.5.

- The structure should be safe against overturning.
- For this condition to be fulfilled, the resultant force should act within the middle third of the base. $e < \frac{L_{base}}{6}$

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

Penstock Supports and Anchors

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

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

Penstock Valves

- Valves are the part used to control the flow through the penstock and usually installed at two places.
- The first valve is provided at the upstream end of the penstock, i.e., at the forebay or immediately after the surge tank, and is called penstock inlet valve,
- While the second is provided at the downstream end of the conduit/penstock, immediately a head of the turbine, and is named as turbine inlet valve.
- The main purpose of penstock inlet valve is for dewatering of the penstock in case maintenance of the penstock is required.

Penstock Valves

- The main purpose of turbine inlet valve is to close the penstock while the turbine is inoperative.
- It can also act as an emergency shut-off device.
- The number of turbine inlet valves required at a power station is governed by the number of turbine units installed, but not by the number of penstocks, as a single penstock can serve a number of units through a manifold at the end.
- There are varies types of valves: Gate valves, Butterfly valves, Spherical valves, Needle valves

Example

Given data

- Normal pool level of reservoir = 1376.00
- Maximum reservoir level = 1381.00
- Minimum reservoir level = 1351.00
- Dead storage level = 1343.00
- Tail water level = 1233.00
- Average discharge from demand curve = 34.5 m^3/sec
- Load factor of the plant (non industrialized area) = 0.5
- The proposed length of headrace tunnel depending upon the existence of overburden rock = 2800 m

Example

- The longitudinal slope of the head race tunnel based on the permissible velocity = 0.008
- The overburden at the end of the headrace tunnel is sufficient for the provision of underground surge tank.
- Total head loss at intake = 0.183 m and overall efficiency = 85%
- Total head loss in the penstock = 2.25 m
- Due to the existence of good sound rock, the head race tunnel is unlined (take 2.5 m/sec for this case)
- The shape of the tunnel is circular.

Determine:

a) Determine the design discharge for the conveyance structures.

b) Design the cross section of the tunnel

c) Calculate the net head of the plant. (Take Darcy Weisbach friction factor as 0.018 for unlined tunnel)

d) Design the cross sectional area and diameter of the surge tank

e) Design the height of the surge tank (Take a height of 3 m to prevent vortex formation)

f) Check and verify the location of the intake structure at the proper elevation. (It should be below the minimum water level and above the dead storage level.)

Solution:

A. Design discharge (Q_d):

- to find Q_d subtract 3 to 10% of the average discharge from the given discharge
- Assuming, 3% of residual flow:

$$Q_d = Q - 0.03Q = 34.5 - (0.03 \times 34.5) = 33.47 m^3/s$$

B. Cross section of the tunnel

- The tunnel is unlined and circular

$$Q_d = AV \quad \textit{from continuity}$$

$$A = \frac{Q_d}{V} = \frac{33.47}{2.5} = 13.39 m^2$$

$$A = \frac{\pi D_t^2}{4} \Rightarrow D_t = \left(\frac{4 \times A}{\pi}\right)^{0.5} = \left(\frac{4 \times 13.39}{\pi}\right)^{0.5} = 4.13 m$$

Solution:

C. Net Head (H_n) $$H_n = H_g - h_{loss}$$

- H_g = Max. Reservoir level – Tail water level
- H_g = 1381m - 1233m = 148m
- h_{losses} = (loss in intake + loss in penstock + frictional loss)
- Frictional loss, h_{lf} :

$$h_{lf} = f\frac{L}{D}\frac{v_o^2}{2g} = 0.018 \times \frac{2800}{4.13} \times \frac{(2.5)^2}{2 \times 9.81} = 3.89m$$

- h_{losses} = (loss in intake + loss in penstock + frictional loss)
- h_{losses} = 0.183+2.25+3.89 = 6.32m
- $H_{net} = H_g - h_{losses}$ = 148 - 6.32 = 141.68m

Solution:

D. Cross section of the surge tank (D_s)

$$A_{s\,\min} = \frac{v_t^2 L_t A_t}{2 g h_{loss} H_{net}} = \frac{2.5^2 \times 2800 \times 13.39}{2 \times 9.81 \times 6.32 \times 141.68} = 13.34 m^2$$

Note, the head loss should be up to the surge tank.

- Area of surge tank (A_s) = Factor of safety × A_{smin}

$A_s = 1.5 \times 13.34 = 20.01 m^2$

$$A_s = \frac{\pi \cdot D_s^2}{4} \Rightarrow D_s = \left(\frac{4 \times A_s}{\pi} \right)^{0.5} = \left(\frac{4 \times 20.01}{\pi} \right)^{0.5} = 5.05 m$$

Solution:

D. Height of the surge tank

$$H_{surge\,tank} = Z_{up} + Z_{down} + H_{net} + h$$

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

$$z_{\max} = 2.5\left(\frac{13.39 \times 2800}{20.01 \times 9.81}\right)^{0.5} = 34.55m$$

$$k_o = \frac{6.32}{34.55} = 0.183$$

$$\frac{z_{up}}{z_{\max}} = 1 - \frac{2}{3}(0.183) + \frac{1}{9}(0.183)^2 = 0.874$$

Solution:

D. Height of the surge tank

$$\frac{Z_{up}}{Z_{\max}} = 0.874 \Rightarrow Z_{up} = 34.55 \times 0.874 = 30.206m$$

$$Z_{down} = \left(-1 + 2k_o\right) \times Z_{\max} = \left(-1 + 2 \times 0.183\right) \times 34.55 = -21.905m$$

$$H_{\text{surge tank}} = Z_{up} + Z_{down} + H_{net} + h$$

$$H_{\text{surge tank}} = 30.206 + 21.095 + 141.68 + 3 = 196.791m$$

Example

- In a hydropower project, water is delivered from an impounding reservoir through a low-pressure tunnel and four high-pressure penstocks to the four turbine units. The elevation of the reservoir water level is 1500 m a.m.s.l, and the elevation of the tailwater is 1200 m a.m.s.l. The maximum reservoir storage which can be utilized continuously for a period of 48 hours is 15×10^6 m^3.

Example

The low-pressure tunnel is constructed as follows:

- Length, L_t = 4 km
- Diameter, D_t = 8 m
- Friction factor, f =0.028

The high-pressure penstocks (4 in no.) are constructed as follows:

- Length of each penstock, L_p = 500 m
- Diameter of each penstock, D_p = 2.0 m
- Friction factor, f = 0.016
- Turbine efficiency, η_t = 90%
- Generator efficiency, η_g =90%

Example

i. Determine the maximum power output from the installation

ii. If a simple surge tank 6m in diameter is provided at the end of the low-pressure tunnel, estimate:

   a. The maximum upsurge and downsurge in the surge tank for a sudden rejection of one unit, and

   b. The maximum downsurge for a sudden demand of one unit

,

Solution

- The discharge available: $Q = \frac{15\times10^6}{48\times60\times60} = 86.8\, m^3/s$

i. Power Output

- Velocity in tunnel: $V_t = \frac{Q}{A_t} = \frac{86.8}{\left(\frac{\pi\times8^2}{4}\right)} = 1.73\,\frac{m}{s}$
- Head loss in the tunnel:

$$h_{ft} = \frac{fL_tV_t^2}{2gD_t} = \frac{0.028\times4000\times1.73^2}{2\times9.81\times8} = 2.13\, m$$

- Discharge per penstock: $Q_p = \frac{86.8}{4} = 21.7\,\frac{m^3}{s}$
- Velocity in penstock: $V_p = 21.7\,\frac{21.7}{\left(\frac{\pi}{4}\times2^2\right)} = 6.91\, m/s$

Solution

- Head loss in penstock,

$$h_{fp} = \frac{fL_pV_p^2}{2gD_p} = \frac{0.016 \times 500 \times 6.91^2}{2 \times 9.81 \times 2} = 9.73\ m$$

- Gross head at the turbine = 1500 – 1200 = 300 m
- Hence, net head, H = 300 – 2.13 – 9.73 = 288.14 m
- Power output per turbine:

$$P = \eta_t \gamma Q_p H = 0.9 \times 9.81 \times 21.7 \times 288.17 = \mathbf{55.20\ \boldsymbol{MW}}$$

- Total power output:

$$P_{tot} = 4 \times 55.20 = \mathbf{220.80\ MW}$$

- The net output of the generator:

$$P_{net} = 0.9 \times 220.80 = \mathbf{198.72\ MW}$$

Solution

ii. Surge Tank

- Area of surge tank: $A_s = \pi \frac{D^2}{4} = \pi \frac{6^2}{4} = 28.27 m^2$
- Area of tunnel, $A_t = \pi \frac{D^2}{4} = \pi \frac{8^2}{4} = 50.27 m^2$
- Length of tunnel, $L_t = 4000\ m$

a) Sudden rejection of one unit:

$$Z_{max} = \frac{Q_o}{A_s}\sqrt{\frac{A_s L_t}{A_t g}} = \frac{21.7}{28.27}\sqrt{\frac{28.27 \times 4000}{50.27 \times 9.81}} = 11.62\ m$$

$$k_o = \frac{h_{fo}}{Z_{max}} = \frac{2.13}{11.62} = 0.183$$

Solution

- Maximum upsurge, Z, will be:

$$\frac{Z_{up}}{Z_{max}} = 1 - \frac{2}{3}k_o + \frac{1}{9}k_o^2 = 1 - \frac{2}{3} \times 0.183 + \frac{1}{9} \times 0.183^2 = 0.8817$$

$$Z_{up} = Z_{max} \times 0.8817 = 11.62 \times 0.8817 = 10.25\, m$$

- Maximum downsurge, Z,

$$\frac{Z_{down}}{Z_{max}} = \frac{-1}{1 + \frac{7}{3}k_o} = \frac{-1}{1 + \frac{7}{3} \times 0.183} = -0.70$$

$$Z_{down} = 11.62 \times (-0.70) = -8.14\, m$$

Solution

b) Maximum downsurge for sudden demand of one unit:

$$\frac{Z_{down}}{Z_{max}} = -1 - 0.125k_o = -1 - 0.125 \times 0.183 = -1.023$$

$$Z_{down} = 11.62 \times (-1.023) = -11.90\, m$$

Exercise 1. A power station is fed through a 10,000m long concrete lined tunnel of 5.0 m diameter operating under a gross head of 200 m. The discharge through the tunnel is $30m^3/s$. A surge tank of 300 m^2area has been provided at the end of the tunnel. Calculate:

a) The maximum upsurge in the tank,

b) The minimum downsurge in the tank.

Assume a friction factor f for the concrete lined tunnel as 0.016.

Q & A

What did you get from the course?

Are objectives of the course achieved?

End

Many Thanks!