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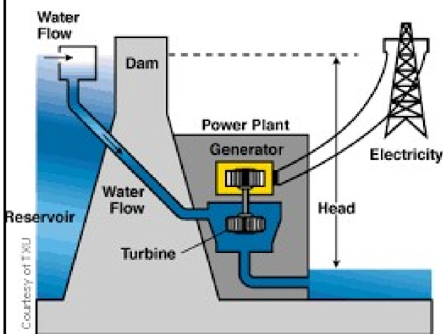
Hydropower Engineering



Internal Evaluation (20 marks)
1. Class Attendance – 3 marks
2. Assignment/Tutorial – 4 marks
3. Field Trip & Report (1+2) – 3 marks
4. Tests – 10 marks

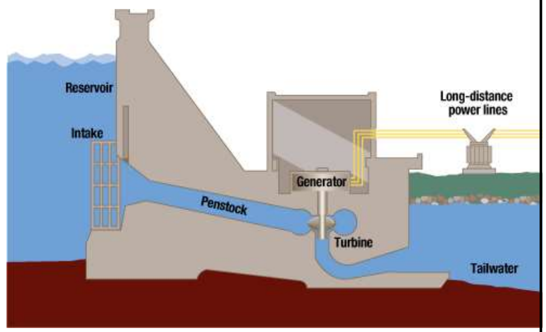
Text Books:
1. Dandekar and Sharma: Water Power Engineering
2. Narayan Prasad Gautam: Principles of Hydropower Engineering
3. Sanjeeb Baral: Fundamentals of Hydropower Engineering

INTRODUCTION

Hydropower

Hydropower is defined as the source of renewable energy formed by the movement of flowing mass of water on the surface of the earth. It gives number of times energy production without change of its physical properties.

In hydropower plant the water is utilized to move the turbines which in turn run the electric generators. The potential energy of the water stored in the dam gets converted into the kinetic energy of the moving water in the penstock.



And this kinetic energy gets converted into the electrical energy with the help of turbine and generator combination.

Power, $P = \rho_w \, g \, Q \, H \, \eta$ Watts

Energy:

Energy is defined as the total power consumed over a certain period, measured in Kilowatt-hours (kWh).

1 Kwh = 1 unit = Energy consumed when power is utilized at the rate of 1 kW for 1 hour.

Classification of Energy

1. Renewable Energy:

The source of energy which could be reused after its utilization is termed as renewable energy. Hydro, wind, tidal and solar energies are renewable energy. They are available in plenty and by far most the cleanest sources of energy available on this planet.

2. Non-Renewable Energy:

The source of energy which is converted into the unusable form after its utilization is termed as non-renewable energy. Fossil fuels, natural gas, oil and coal are non-renewable energy. Non-renewable sources are not environmental friendly and can have serious affect on our health.

Energy Sources:

1. Conventional energy source
- ➢ Thermal power
- ➢ Nuclear power
- ➢ Hydropower

2. Non-conventional energy source
- ➢ Solar power
- ➢ Wind power
- ➢ Tidal power
- ➢ Geo-thermal power

1. Conventional energy source

a) Thermal power:
- Fuel (e.g. fossil fuel) is burned to produce steam to drive a turbine
- Converts 30 → 40 % of the energy content of the fuel to electric energy.
- Plants may be designed to operate on coal, natural gas, oil or combination of fuels.
- Plants require several hours for start up.
- Require 4-6 weeks of maintenance each year (specially those run by coal)
- Forced outage rate of 10 → 20 %
- Maximum possible plant factor ranges from 65 → 85 %
- Relatively high capital cost
- Suitable for base and peak load.

b) Nuclear power:
- Nuclear fission produces the heat required to generate the steam
- Efficiency about 33% i.e, slightly lower than that of coal plants.
- Fuel costs are lower (8 – 10 times less than the fossil oil) compared to thermal power.
- High capital cost
- Used almost exclusively for base load service.
- Normally out of service for about 8 weeks for maintenance and refueling each year.
- Forced outage rates ~ 15%
- overall availability (maximum possible plant factor) of 65 – 70%

c) Hydropower:
i) Conventional Hydropower:
- Quantity of fuel (i.e; water) available at any given time is fixed.
- Techniques such as seasonal storage (Kulekhani HEP) or daily/weekly pondage can be used
- High capital cost
- Energy conversion efficiency at 80 – 90%
- Fast response to the change in demand
- Forced outage rate very low (2–4%)
- Require 3 weeks for maintenance per year
- Average availabily including scheduled maintenance ~ 95%

ii) Pump storage Hydropower:
- A form of energy storage
- Relatively low cost energy (secondary energy and energy from cheaper sources) is used to pump water into an upper storage reservoir during periods of low power demand
- During high demand period (when energy is most valueable) water is released to produce power, i.e., normally used for peaking service
- overall efficiency is about 65–75%
- Construction cost is moderately high
- Forced outage rates about 5%

2. Non-Convention energy source

a) Tidal power:
- Tidal amplitudes attain considerable magnitudes along certain costal stretches.
- Favorable site → where the sea is encroached to the coast with a narrow strip with shallow depth
- A barrier (dam) creats head and separates water body (reservoir) and sea
- A powerhouse is often accommodated in the barrier itself.
- Amplitudes of 6–8 m recorded in the coasts of Pacific and French Atlantic regions.
- Amplitudes of 13.5 m and 10 m reported in the coast of Canadian Atlantic Coast and Bristol channel
- Power plant constructed in La Rance, U.K. (240 MW capacity with the installation of 24 nos. bulb turbines) is the successful example.

d) solar power:

A huge potential for electric power generation is solar energy. This is abundant and available all over the world. Research in developed and developing countries are going on to utilize it efficiently. The utilization of solar power is limited only to micro scale uses.

e) wind power:

- High wind velocity is utilized to drive the propeller, the shaft which is coupled with the generator.
- In the Netherlands, mechanical power from the windmills was utilized to lift the water from the low land to the sea or river in the past.
- Considering strong wind velocity prevailing in Kaligandaki basin of Nepal in Mustang area a windmill was installed in Kagbeni, however due to poor design and installation, it went out of order after few months of installation.

d) Geo-thermal power:

The heat prevailing in different part of the earth can also be used for power production.

Comparision between Hydropower and Thermal power:

Hydropower	Thermal power
- Developed by coupling a generator to a water turbine.	- Developed by coupling a generator to a steam turbine.
- Water turbine are run by hydraulic energy.	- Steam turbines are run by fuels, such as oil or natural gas.
- Environmentally clean.	- No eco-friendly.
- Cheapest energy source.	- Costly energy source.
- Renewable source.	- Non-replenishable source.
- outages are relatively less.	- More outages.
- Low maintenance cost.	- High maintenance cost.
- overhaul and maintenance need shut-down of very short durations.	- Require longer shut-downs for overhaul and maintenance.

- simple plant equipment and operation.	- complecated plant equipment and generation.
- High efficiency (Turbine efficiency 90 – 95 %)	- Low efficiency (Turbine efficiency 30 – 40 %)
- High extra cost on dam and appurtenant works.	- Construction of dam and appurtenant works is not involved.
- Reliable energy source.	- Low reliability energy source.
- Greater life expectancy, 50 – 100yrs.	- Low life expectancy, 25 yrs.
- Longer gestation period.	- Shorter gestation period.
- Generating sets can run and pick up load in almost no time to start and to stop.	- Considerable time is required to run and pick up load.
- High cost on long transmission lines to carry the energy and hence high transmission losses.	- Low cost on short transmission lines and hence less transmission losses.
- Large area is submerged in the reservoir	- Land required for power plant is less but large additional area is required for disposal of ash and mining of coal.

Advantages of Hydropower:

- Renewable source of energy
- Economical source of power
- Environment friendly
- Reliable energy source (≃ 90 % availability)
- Low generation cost
- Helps in management and regulation of water resources
- Provide employment opportunities being labour intensive
- Lead to development of remote areas, socioeconomic benefits
- Long span of life (50 to 100 years)
- Technically more reliable, 2 to 3 months stabilisation period
- Low operation and maintenance cost
- Better service operation flexibility
- Possible to build power plants of high capacity
- Quick to start and stop
- Higher efficiency (turbine efficiency 90 – 95%)

Disadvantages of Hydropower:

- Susceptible to vagaries of nature such as drought
- Longer construction period
- High initial cost
- Long term flow data is essential
- Loss of large land due to submergence in the reservoir
- Suitable sites for the construction of dam is difficult to available
- Displacement of large population from reservoir area and their rehabilitation
- High cost of transmission system for remote sites.

Hydropower Potential:

1. Gross potential:

The power which can theoritically be possible to generate is known as gross potential power.

A river basin is divided into several cascades. Based on the head and hydrograph of the particular cascade the power can be calculated. Then the total power in the river basin can be calculated using the following relation.

$$P = \sum_{i=1}^{n} \eta \gamma Q H / 1000$$

where, P = Power in KW

Q = Discharge in cumecs

H = Net head in m

γ = sp. wt. of water in KN/m^3 ($9.81\ KN/m^3$)

η = Plant efficiency.

and n = Number of cascades.

- The estimated gross potential of Nepal ≈ 83,000 MW.

2. Technical potential:

The power which is technically viable to produce is known as technical potential.

All the theoritically possible power in nature can't be produced due to various constraints like unfavorable geology, topography, climatic conditions, accessibility etc.

- Technical potential power in Nepal ≈ 44,000 MW.

3. Economic potential:

The power which is economically feasible to produce is known as economic potential.

It is not always economical to utilize the discharge available during a short period of the year. The projects are usually considered economically feasible if the internal rate of return is higher than the prevailing interest rate and the benefit cost ratio is higher than unity.

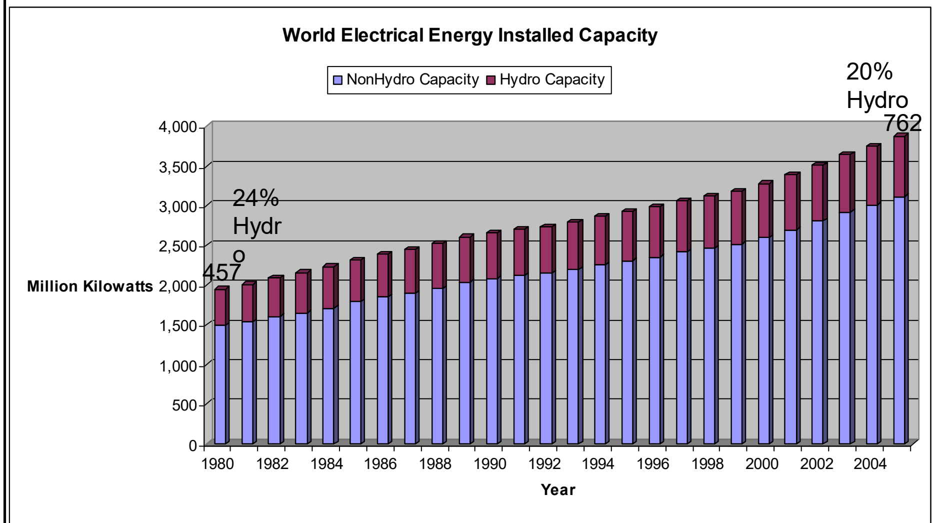
- Estimated economic potential power in Nepal ≈ 42,000 MW.

Hydropower potential of Nepal:

- Huge potential of water resoures compared to country size
- Annual surface runoff ≈ 0.5% of total surface runoff of the world
- Mean annual runoff formed only inside Nepalese territory ≈ 174 billion cubic meter
- Total annual surface runoff including the flow from Tibetian drainages ≈ 200 billion cubic meter
- The geographical constitution of Nepal offers tremendous energy potential for generating hydropower
- The major river basins are Koshi, Gandaki, Karnali and Mahakali
- The average precipitation ≈ 1500 mm (80% experienced during monsoon)
- Theoritical potential ≈ 83,000 MW
- Technical potential ≈ 44,000 MW
- Economic potential ≈ 42,000 MW

Theoritical hydro potential of Nepal:

River Basins	Small river courses CA of 300-1000 km^2	Major river courses CA > 1,000 km^2	Total potential (GW)	Economically Feasible (GW)
Sapta Koshi	3.6	18.75	22.35	10.86
Sapta Gandaki	2.7	17.95	20.65	5.27
Karnali and Mahakali	3.5	32.68	36.18	25.1
Sourthen Rivers	1.04	3.07	4.11	0.88
Country Total	10.84	72.45	83.29	42.11



World's Largest Hydroelectric Plants
(over 4,000 MW capacity)

Name of dam	Location	Rated capacity (MW) Present	Rated capacity (MW) Ultimate	Year of initial operation
Itaipu	Brazil/Paraguay	12,600	14,000	1983
Guri	Venezuela	10,000	10,000	1986
Grand Coulee	Washington	6,494	6,494	1942
Sayano-Shushensk	Russia	6,400	6,400	1989
Krasnoyarsk	Russia	6,000	6,000	1968
Churchill Falls	Canada	5,428	5,428	1971
La Grande 2	Canada	5,328	5,328	1979
Bratsk	Russia	4,500	4,500	1961
Moxoto	Brazil	4,328	4,328	n.a.
Ust-Ilim	Russia	4,320	4,320	1977
Tucurui	Brazil	4,245	8,370	1984

Sept 9, 2008

New #1: Three Gorges Dam in China

Beginning in 2009, 26 turbines with a capacity of 18,200 megawatts will produce on average 84.7 terawatt hours a year, which corresponds to the performance of 16 atomic power plants, and will negate the burning of 40-to- 50 million tons of coal annually.

Sources of Hydropower Energy:

1. Surface flow → Rivers, Ponds, Lakes

The energy of the flow is trapped either in form kinetic energy or combination of kinetic energy and pressure energy due to possession of potential energy of the flow by virtue of geological advantage.

2. Subsurface source → Ground water accumulation

The energy of the flow is trapped in the form of steam energy by the virtue of possession of heat energy inside the earth crust.

3. Sea / ocean / Bay → Tidal rise

The energy of the flow is trapped either from kinetic energy or pressure energy from the possession of potential energy of the flow by virtue of tidal rise in the sea / ocean / bay due to gravitation attraction between moon and earth.

HISTORY OF HYDROPOWER DEVELOPMENT IN NEPAL

- Pharping Hydropower Plant is one of the oldest hydropower plants of Asia and the first hydropower plant of Nepal.
- Established in the year 1911 while the first hydropower plant in China was established in 1912.
- Ironically, we have lagged behind in hydropower generation ever since and have faced seemingly perpetual load shedding hours in the recent years
- This is despite the fact that Nepal is among the richest country in the world in terms of water resources.

STATUS OF HYDROPOWER DEVELOPMENT IN NEPAL

- Presently, the total installed capacity of Nepal's power plants is roughly 787 MW including thermal and solar plants (NEA, 2015).

Source	Capacity (KW)
Total Small Hydro (NEA)-Isolated	4,536
Total Hydro (NEA)	477,930
Total Hydro (IPP)	255,647
Total Hydro (Nepal)	733,577
Total Thermal (NEA)	53,410
Total Solar (NEA)	100
Total Installed Capacity (NEA and IPP)	787,087

STATUS OF HYDROPOWER DEVELOPMENT IN NEPAL

- Except 92 MW Kulekhani reservoir project, all of the hydropower projects in Nepal are of run-of-river (ROR) type
- Huge power generation difference between rainy and dry season
- Peak demand for 2015 was estimated as 1291.80 MW

3 distinct stages of Hydropower development in Nepal

- Donor assisted till 1995
- Independent Power Producers (IPPs) oriented till 1995-2001
- Open and Liberal policy since than

First Five-Year Plan (1961-1961)

- Electricity development highly prioritized
- Main objective was to generate 20 MW of electricity

- In 1962, Nepal Electricity Corporation (NEC) was established and in 1985 it was restructured to Nepal Electricity Authority (NEA)
- Nepal embarked on a market led liberalized economy after the restoration of democracy in 1990.
- Since then a number of hydropower development policies have been formulated.

Eighth Five Year Plan (1992- 1997)

- First plan by the democratic government formed after *JanaAndolan* of 1990.
- Hydropower Development Policy 1992, Water Resources Act 1992, Electricity Act 1992 and Foreign Investment and One Window Policy 1992 were formulated to attract foreign as well as domestic investment from private sectors

Tenth Five Year Plan Period (2002-2007)

- Laid emphasis on construction of small, medium, large and reservoir type hydropower projects
- To promote integrated development of water resources involving private and public sector with emphasis on rural electrification and control of unauthorized leakage of electricity

Ten Years Hydro Development Plans

- 10,000 MW in 10 Years
- Reserving small hydropower projects up to 50 MW for domestic investors
- Building cost effective projects under Public-Private Partnership (PPP)

Twenty Years Hydro Development Plans

- 25,000 MW in 25 Years and projects divided into 5, 10, 15 and 20 years time frames
- Domestic consumption and export oriented
- Also emphasized Public-Private Partnership (PPP)

ISSUES IN HYDROPOWER DEVELOPMENT IN NEPAL

1. Political constraints including the state reconstruction
2. Technical constraints
3. Financial constraints
4. Policy constraints
5. Climate change

1. Political constraints

- Lack of Political Will
- Persistent political instability
- Aspiration of local people
- State Reconstruction

2. Technical Constraints

- Technical constrains for the development of hydropower related to geological, hydrological and topographical settings of the country.
- Also, lack of manpower specialized in hydropower development and lack of long term hydrological and sediment logical data are other technical constraints
- Lack of adequate transmission lines and insufficient capacity of existing and planned cross-border transmission lines
- Absence of Storage-type Projects

3. Financial Constraints

- Hydropower projects are more capital intensive
- Nepal doesn't have the necessary financial resources to develop the hydropower in its own and have to be reliant upon investment form international financial institution and donor agencies
- Pricing Issue of electricity

4. Policy Constraints

- Issue of License and institutional constraints
- Monopoly of NEA over transmission and distribution of power
- Overlapping responsibilities among governmental ministries and departments
- Inconsistency among various hydropower policies

5. Climate Change

- Water resources and hydropower ranks among the most vulnerable resources

- With only 1-2% of its potential currently developed, it will be quite some time before the opportunities to expand hydropower energy are constrained by climate change in Nepal.
- This does not mean that the existing facilities might not be seriously affected by Climate change. The ways in which climate change can affect hydropower resources include:
  - **Run off variability**
  - **Glacial retreat**
  - **Glacial Lake Outburst Flood (GLOF)**
  - **Sediment load and Evaporation losses**
  - **Financial constraints and disincentives**

Legal and Policy Environment

- Government has adopted the Hydropower Development Policy of 2001 and encourages both local as well as foreign investment, especially for the development of SHP

Highlights of the Hydropower Development Policy, Nepal 2001

- Development at an affordable price
- Uplift the living standard of the rural community
- Efforts to reduce the risk of investment
- To open market for sale of electricity both at national and international level.
- Easy acess for the expatriates to work in the country in relation to the project implementation

Legal and Policy Environment

Government Agencies in the power sector

i. Ministry of Water Resources (MoWR)
ii. Water and Energy Commission Secretariat (WECS) – planning and policy research.
iii. Department of Electricity Development (DoED) – licensing, facilitation, promotion, compliance monitoring and project study *(regulating body under Ministry of Water Resources).*
iv. Nepal Electricity Authority (NEA) – public utility for generation, transmission and distribution of electricity *(Government of Nepal undertaking company under Ministry of Water Resources, Formed in 1985).*
v. Electricity Tariff Fixation Commission (ETFC) – tariff setting

Legal and Policy Environment

Moreover, for the promotiion of hydropower projects, the DoED has been designated as 'One Window' under the MoWR, with following responsibilities:

- Issuance of survey and Project (generation) licenses.
- Providing concessions and incentives.
- Facilitating the import of the plant, equipments and goods required for the project.
- Facilitating in the acquisition of government land required for the project.
- Facilitating in obtaining various permits and approvals.

The Hydropower Development Policy 2001

The Hydropower Development Policy 2001 (HDP) addresses issues including private sector demand, the need for reasonable pricing, rural electrification, the need to raise the level of employment, hydro power exports and investor friendly practices.

Objective
- Keep electricity costs low by using least cost generation
- The delivery of electricity with reasonable quality and price
- The need to combine electrification with the economic activities
- The expansion of rural electrification
- Need to make hydropower an exportable commodity

GoN's Commitments
- Survey license: term of 5 years
- Generation license term: 35 years for domestic supply and 30 years for export oriented projects
- Additional maximum five years for hydrological risks

The Hydropower Development Policy 2001

- Projects turned over free of cost on good operating condition at the end
- Water rights guaranteed
- No nationalization
- Foreign exchange and repatriation facility

Procedure
- Projects to be developed by way of competitive bidding
- BOOT (build-own-operate-transfer) model for private investment
- Respect for high standards for environment protection
- GoN to assist in land acquisition
- Royalty structure fix rate up to 1000 MW, export projects negotiable rate above 1000 MW
- Separate agreement for developers and GoN

Planning of Hydropower Projects

Types of Hydropower plants:

1. Classification based on installed capacity:

a) Napalese context

i) Micro Hydropower plant - up to 100 KW
ii) Mini Hydropower Plant – 100 KW to 1000 KW
iii) Small Hydropower Plant – 1 MW to 25 MW
iv) Medium Hydropower Plant – 25 MW to 100 MW
v) Large Hydropower Plant - greater than 100 MW

b) According to Prof. Mosonyi

i) Midget hydropower plant – upto 100 KW
ii) Low capacity hydropower plant – 100 to 1,000 KW
iii) Midium hydropower plant – 1,000 to 10,000 KW
iv) High capacity hydropower plant – 10,000 to 50,000 KW

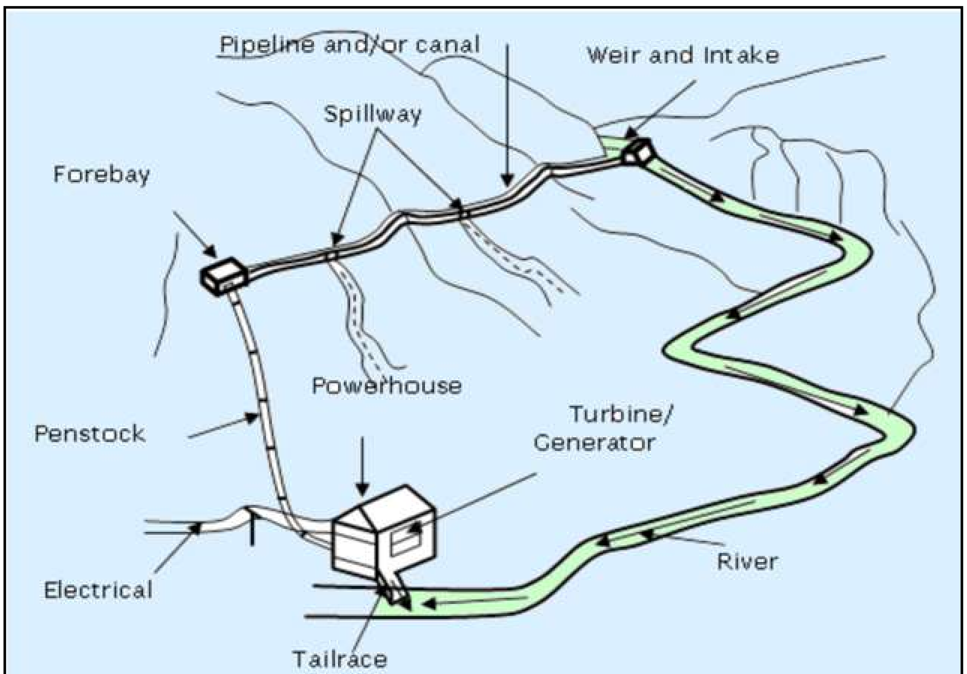
c) According to Dandekar and Sharma

i) Micro hydropower plant – upto 5 MW
ii) Medium capacity hydropower plant – 5 MW to 100 MW
iii) High capacity hydropower plant – 100 to 1,000 MW
iv) Super capacity hydropower plant – > 1,000 MW

2. Classification based on storage:

a) Run of River plants (ROR)

Those plants, which do not regulate the hydrograph of the source river in seasonal term, are known as ROR plants. Keeping view the increased load during peak hours, ROR plants may be constructed with pondage, which can regulate daily hydrograph or weekly hydrograph, store water (full or partial volume) to run the plant under full capacity. These types of plants are known as peaking Run of River (PROR) plants. Those plants which do not have such capability are known as ROR plants. Plants like Marsyangdi, Kaligandaki, Sunkoshi and Panauti HEP fall into the category of PROR, whereas Khimti - I, Bhotekoshi, Indrawati - III, into ROR.



Layout of a Run-off-the-River Hydro Power Project

b) Storage Plants

Those plants which can regulate the hydrograph of the river by, one or more seasons are usually known as storage plants. The storage reservoir is provided by constructing a dam which is a major item of capital expenditure and increases the initial investment of the project greatly. At the same time, it usually implies a much more efficient and controlled use of the available water. The storage plants may be of following types:

i) seasonal storage

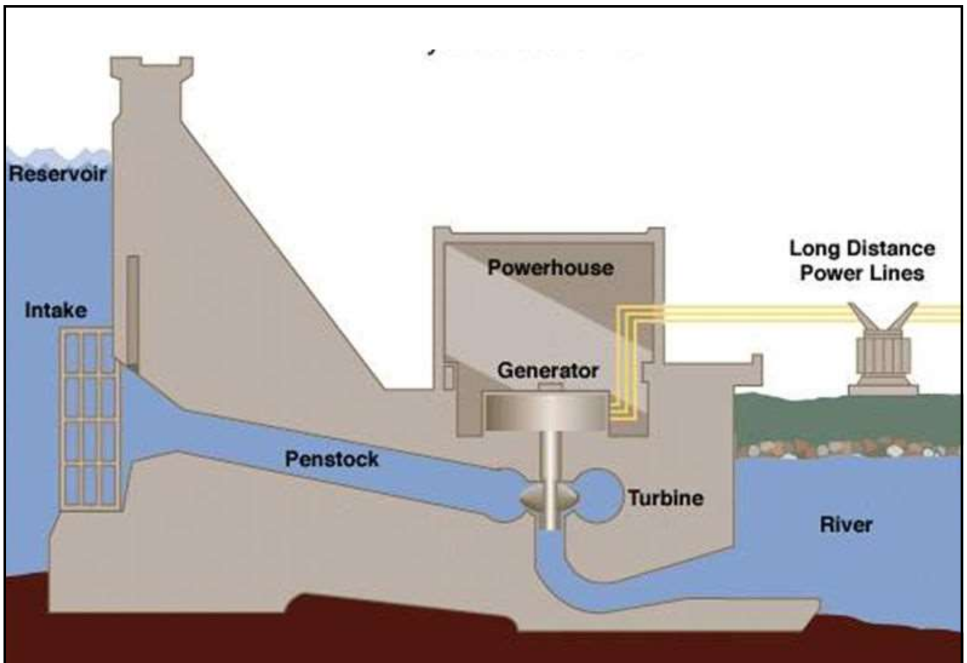
Plants with reservoir which can regulate the discharge of summer season to run the plant in enhanced capacity during winter.

ii) Annual storage

Plants with reservoir which can fully regulate the average hydrograph of the source river.

iii) Pluri annual storage

Plants with reservoir which can regulate the hydrograph of the source river for several years.



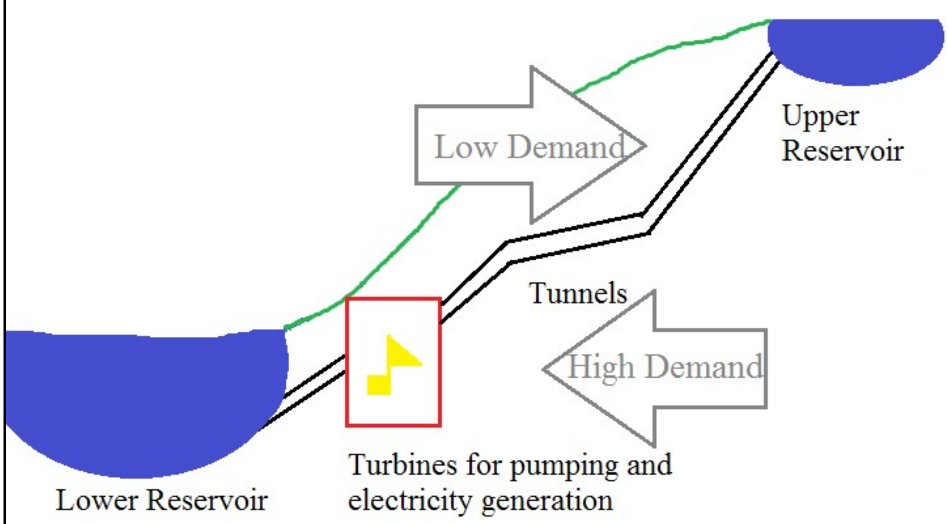
(ii) Pumped storage

Plants having water bodies / reservoir at upstream as well as downstream of power house which collects the water in the upstream reservoir by pumping from down-stream reservoir using cheaper energy during off peak hours and utilizing this to generate power during peak hours.

3. Classification based on construction feature :

a) Valley type

In a valley type plant, a dam is dominant feature, creating a storage reservoir behind it. Power house is generally located at the toe of the dam. Diversion of water away from the source is not entertained.



b) Diversion type

In a diversion type plant, the water from the source is diverted through canal or tunnel to the powerhouse, situated sufficiently away from the diversion point, in the same or neighbouring river basin.

4. Classification based on head:

i) Low head plant – $H < 15$ m
ii) Medium head plant – $H = 15$ to 70 m
iii) High head plant – $H = 70$ to 250 m
iv) Very high head plant – $H > 250$ m.

Assessment of water power potential:

Let, Q = discharge in cumecs
H = height of fall in metres

Then, the rate of work done

$= Q \gamma H$ kg-m/sec

$[\because 1\ HP = 75\ kg\text{-}m/s]$

$\therefore$ Horse power generated $= \frac{\gamma Q H}{75}$

Taking $\gamma = 1000\ kg/m^3$

Horse power generated $= \frac{1000\ QH}{75}$

$= 13.33\ QH$

If η_t = overall efficiency of turbines, the mechanical power available from the shaft of the turbine

$= 13.33\ Q H \eta_t$ H.P.

Since 1 H.P. = 736 watts,

Electrical power $= 13.33\ QH \eta_t \eta_g * 0.736$ KW

$$= 9.8\, Q H \eta_t \eta_g \text{ KW}$$
$$= 9.8\, Q H \eta$$

where, η = overall efficiency of the system.

Stages of Hydropower Development

1. Reconnaissance
2. Pre-feasibility Study
3. Feasibility Study

1. Reconnaissance

Reconnaissance studies are normally the first step of hydropower studies. The reconnaissance is of preliminary nature. Being the first step in the project planning reconnaissance studies are concerned with identification as well as investigation of projects which are suitable for the stated purpose. Reconnaissance studies are carried to follow pre-feasibility and feasibility studies but it will require less accuracy and less detail. In this stage of studies, it is necessary to rely on very experienced hydropower planner in order to formulate the well balanced projects.

The main objectives of the reconnaissance studies are:-

1.To recognize appropriate projects for the declared function.

2.To investigate apparent alternative solution for inclusion in the plans or rejection.

3.To evaluate nominated projects and work out the best project for the given function.

4.Compare the projects and formulate the project best suited for the stated purpose.

5.To record the lower rank projects and projects alternatives for the future experience.

6.To provide preliminary cost figures in the performance of reconnaissance studies.

7.To record, screen, rank the project alternatives.

Steps and Activities in reconnaissance study

i) Data and information collection

Work starts with gathering and evaluation of all relevant data and information about power markets, hydrology, geology, topography, environment, geotechnical aspects and socio-economy of the projects and its surrounding area.

ii)Desk Study

It involves planning elements and parameters (power market, flow, head, environment). Tentative layout of the project showing all elements of the layout of the plants is also prepared in this stage.

iii) Field work and Design

It involves field visit for the verification of layouts. Reselection of permissible alternatives and other related studies will be conducted.

iv) Estimates and schedules

Preliminary cost estimate and implementation schedules are prepared based on the reconnaissance level of study.

v) Environmental and social studies

IEE (Initial Environmental Examination) or EIA (Environmental Impact Assessment) shall be prepared based on the legal requirements. Secondary data shall be analyzed to get the social and environmental information.

vi) Economic Assessment

The power unit cost and unit cost of annual generation capability of the project are prepared.

vii) Reconnaissance study Report

Report generally includes:-
- Documentation of the all available data and information
- Drawing of layout
- Report on main project features
- Reports on discarded solutions and alternatives
- Project reports cover implementation cost and time aspects.

2. Pre-feasibility Study

The second phase of hydropower planning after reconnaissance study of the project is pre-feasibility study.

The main objectives of the prefeasibility studies are:-

1. Establish the need and justification of the project
2. Formulate a plan for the development of the project.
3. Determine the technical, economical and environmental practicability of the project.
4. Define the limit of the project
5. Ascertain the local interest and desire for the project.
6. Make recommendations for further action
7. To choose and approve possible projects for the further consideration.

Steps and Activities in pre-feasibility study

i) Data and information collection

It involves the collection of the available hydrological, meteorological data. Topographical mapping, geological mapping, walkover survey along the project alignment will be carried out.

ii)Desk Study

The report which is prepared during reconnaissance study shall be reviewed and verified.

iii) Field work and Design

Topographical survey and geological survey is carried out. Hydrological study is also performed in this stage.

iv) Financial analysis

Based on the preliminary component sizing; preliminary cost estimates is prepared. Cost per KW is also calculated.

v) Environmental and social studies

IEE, the ToR of IEE works is prepared. If the project requires EIA, the ToR and scoping works is carried out. The field trip of the proposed area is carried out to identify the potential social issues from the project.

vi) Pre-feasibility study Report

The pre-feasibility report should close with firm statements on the suitability of the project in development context and its practicability for the stated purpose. A concise recommendation on the project's further development role is also required.

3. Feasibility Study

The feasibility investigation is a comprehensive analysis and detailed study of the contemplated project, directed towards its ultimate authorization, financing, design and construction. The feasibility study is carried out in order to determine the engineering(technical), economical and environmental feasibility of the project. The feasibility study report will provide the necessary information from which the owners can decide whether or not to go for the implementation of the project.

The main objective of the feasibility studies is to stabilize the economic

The main objectives of the feasibility studies are:-

1. To stabilize the economic analysis and to analyze dispute issues
2. To estimate the net economic value to be produce as well as estimates the cost of development, construction, operation, maintenance and replacements

Steps and Activities in pre-feasibility study

i) Data collection

- Supplementary data and information
- Hydrological data
- Topographical data
- Geological and geotechnical data
- Detailed socio-economic data from social survey

ii) Review of desk study and prefeasibility report

The desk study report and reconnaissance level study report and other relevant study report related to the project shall be reviewed.

iii) Field work and Design

- Field visit for layouts for the proposed layout during prefeasibility study
- Field trip for additional survey
- Field visit for geological data collection, pit sampling, ERT survey etc.
- Establishment of gauging station at headworks and powerhouse site
- Detailed investigation for structure location
- Installed capacity optimization
- Design of each component

iv) Estimates and schedules

- Detailed cost estimate is prepared
- Detailed implementation schedules and planning also prepared

v) Environmental and social studies

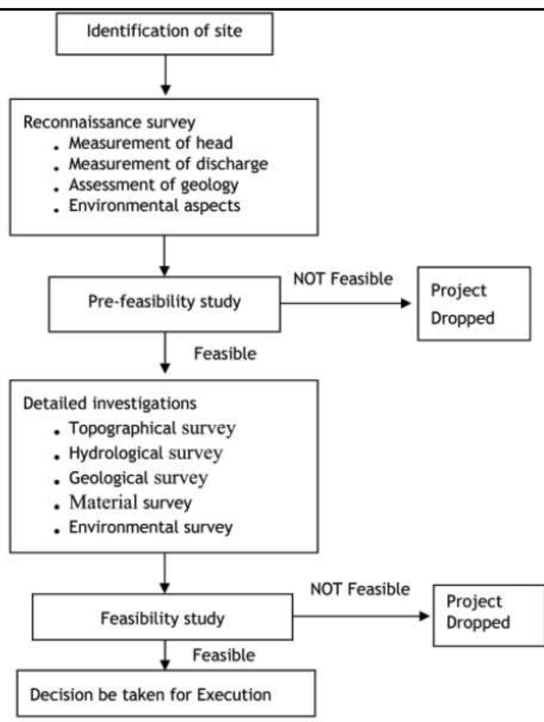
- Detailed IEE or EIA report shall be prepared
- Consultation with stakeholders
- Environmental management/mitigation plan
- Building harmony with project affected family for smooth execution of the project

vi) Economic assessment and financial evaluation

- Financial evaluation and sensitivity analysis shall be done
- FIRR (Financial Internal Rate of Return), B/C ratio, RoE (Return on Equity) and NPV (Net Present Value) calculation shall be done

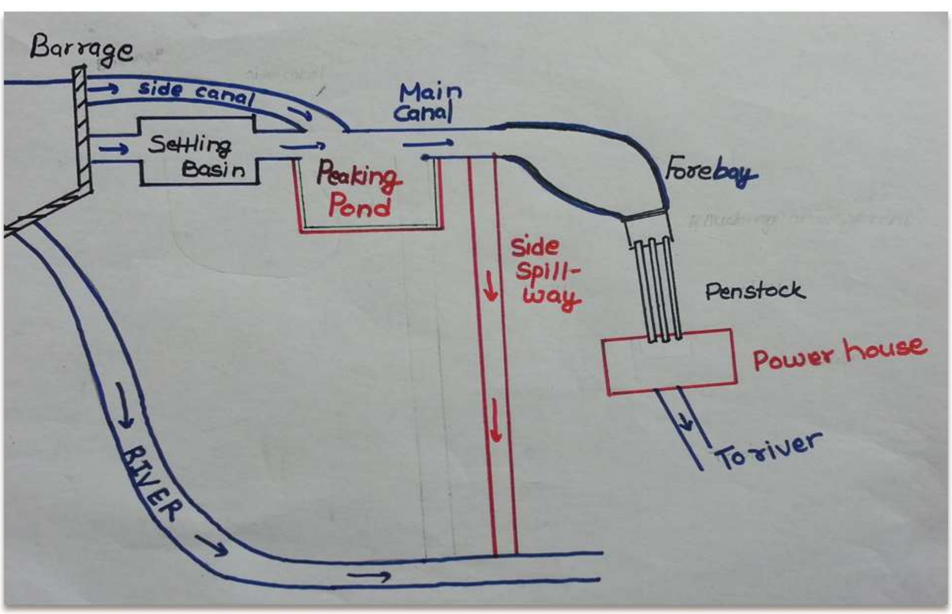
vii) Feasibility study report

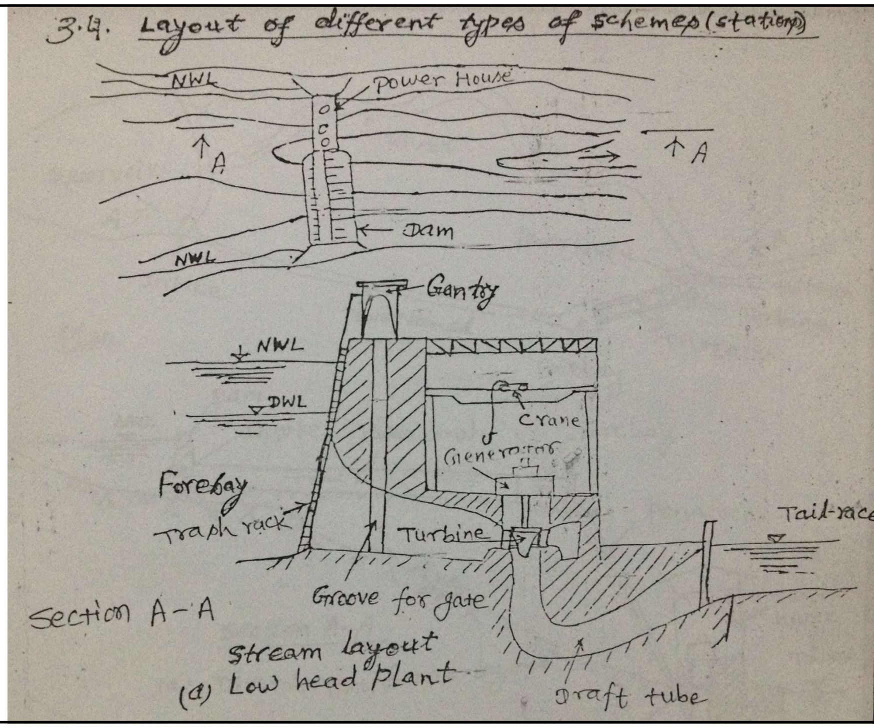
This feasibility study report is sometimes called as DPR(Detailed Project Report). This feasibility report is also bankable document. This document is necessary for PPA (Power Purchase Agreement) of the project, financial closure and other requirement of DoED. It should include the conclusion an recommendation for further action.

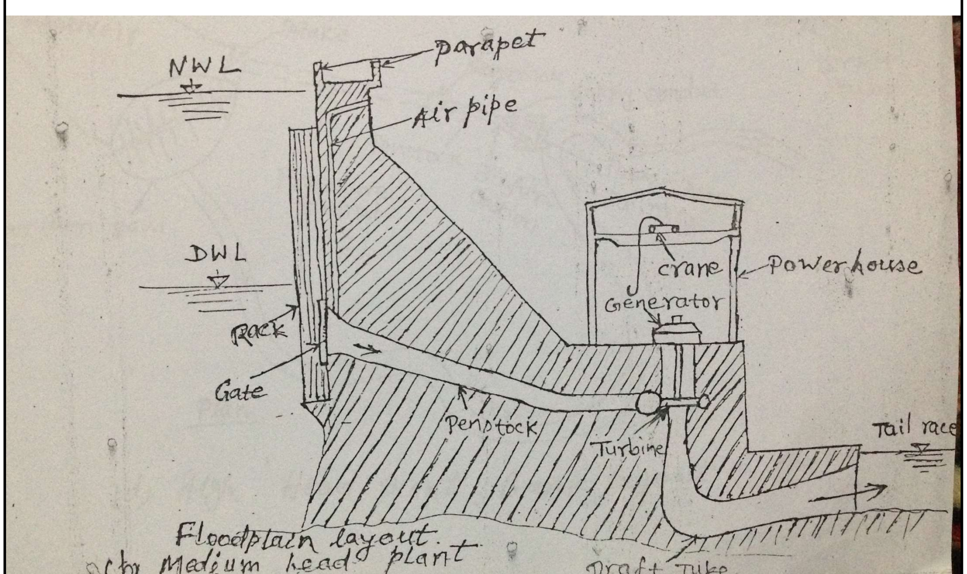


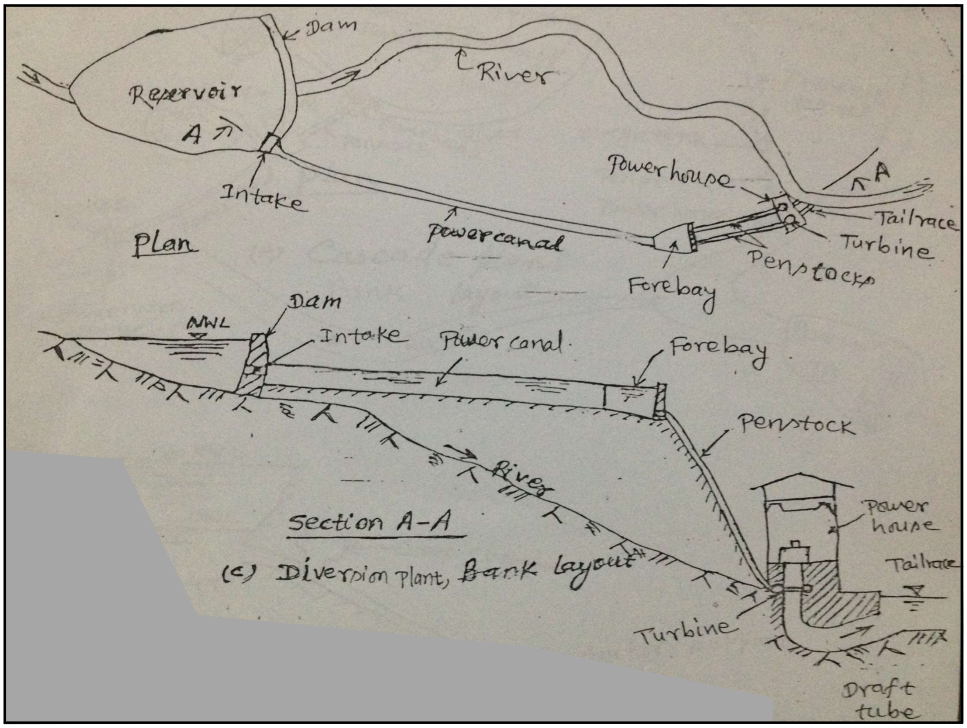
Hydropower Development cycle:

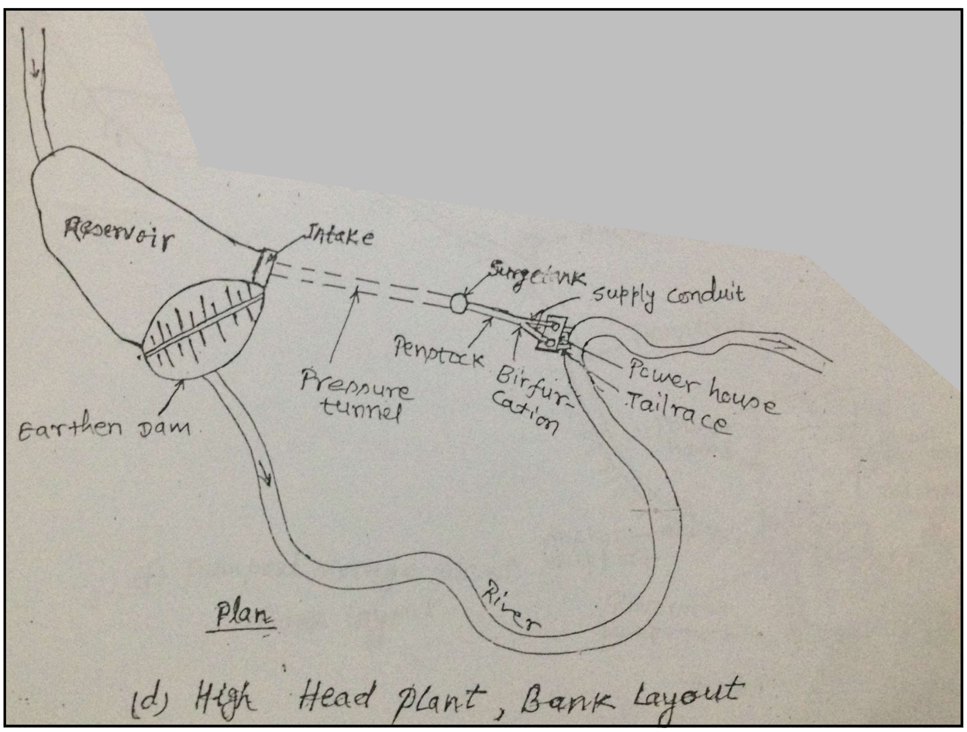
1. Pre construction phase:
   - Reconnaissance studies
   - pre-feasibility study
   - Feasibility study
2. Implementation phase:
   - Design and procurement
   - Detailed design and work drawings
   - construction of civil, structural and transmission works,
   - Manufacture, erection of electromechanical and hydro mechanical equipments
   - commissioning and start up installations
3. Operation phase:
   - project starts for revenue generation
   - ~~training~~ Research and training works

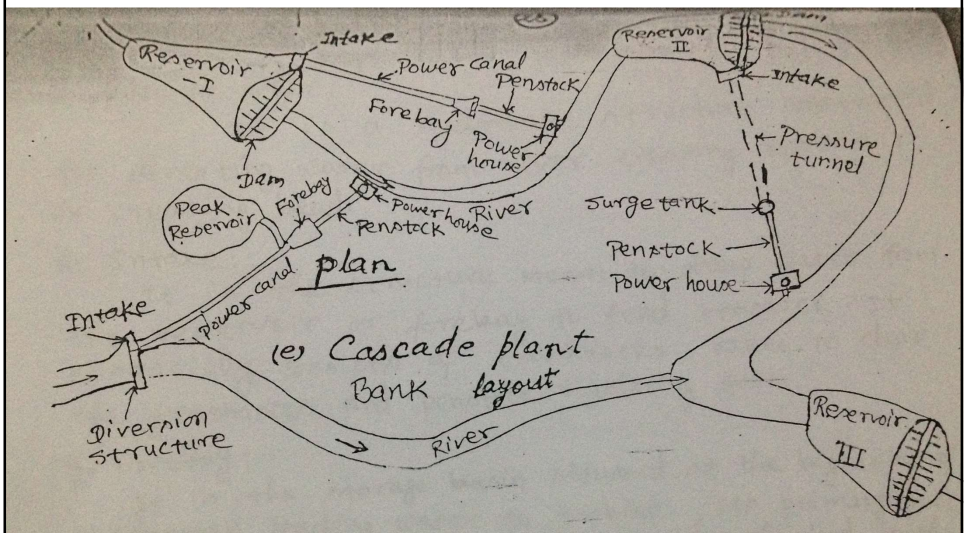


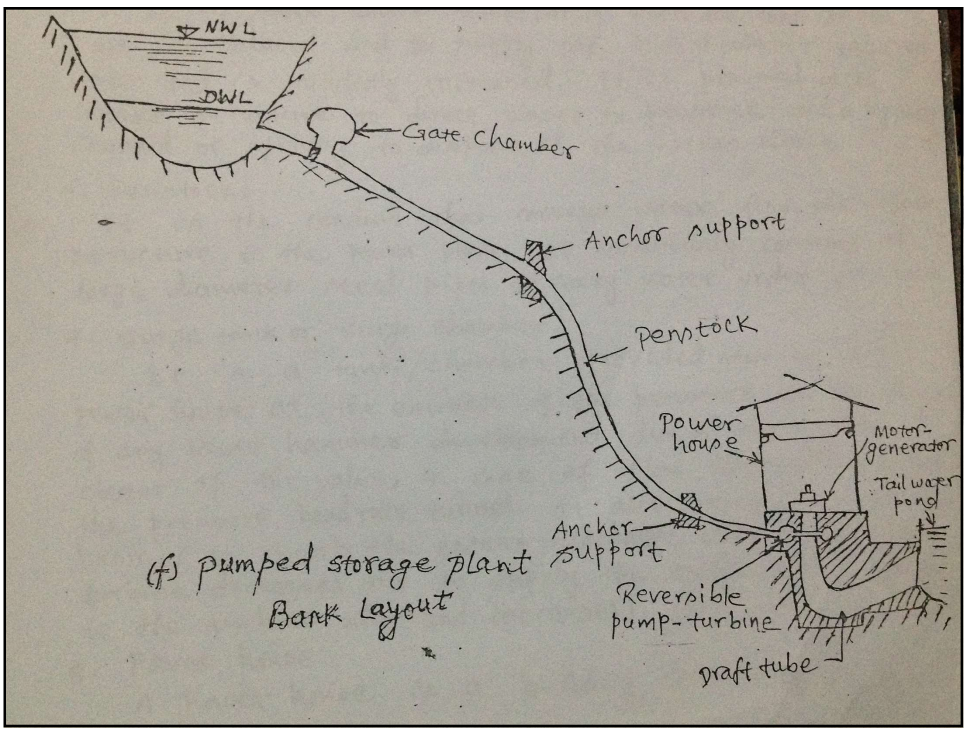

3.4. Layout of different types of schemes (stations)
NWL
Power House
A
A
Dam
NWL
Gantry
NWL
DWL
Crane
Generator
Forebay
Trash rack
Turbine
Tail-race
Section A-A
Groove for gate
Stream layout
(a) Low head plant
Draft tube


parapet
NWL
Air pipe
DWL
crane
Power house
Generator
Rack
Gate
Penstock
Turbine
Tail race
Floodplain layout
(b) Medium head plant
Draft Tube

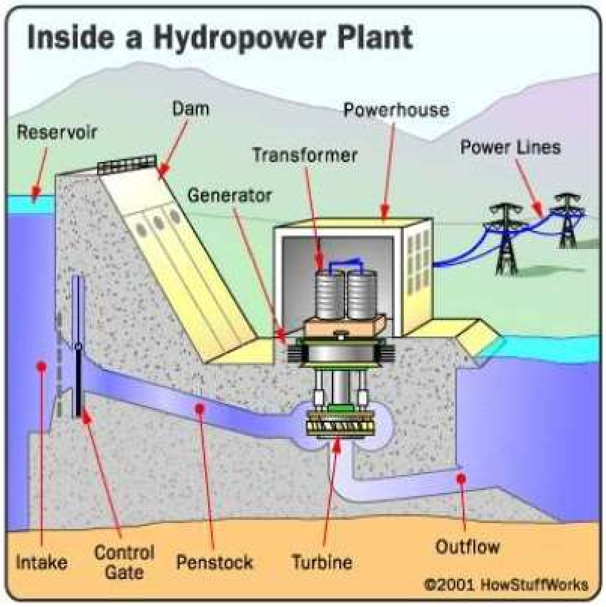

Dam
River
Reservoir
A
Intake
Power house
A
Tailrace
Turbine
Plan
Power canal
Penstocks
Forebay
Dam
NWL
Intake
Power canal
Forebay
Penstock
River
Power house
Section A-A
Tailrace
(c) Diversion plant, Bank layout
Turbine
Draft tube


Reservoir
Intake
Surge tank
supply conduit
Penstock
Birfurcation
Power house
Pressure tunnel
Tailrace
Earthen Dam
River
Plan
(d) High Head plant, Bank Layout


Reservoir -I
Intake
Power canal
Penstock
Fore bay
Power house
Dam
Reservoir II
Intake
Pressure tunnel
Peak Reservoir
Forebay
Power house
River
Penstock
Surge tank
Penstock
Power house
plan
power canal
Intake
(e) Cascade plant
Bank layout
Diversion structure
River
Reservoir III

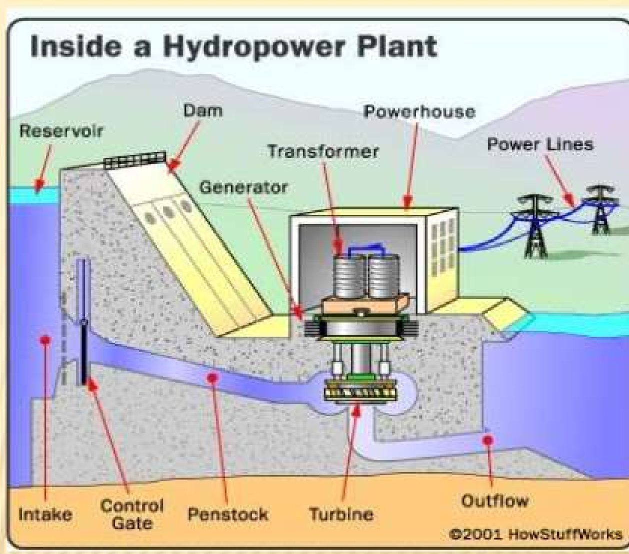

NWL
DWL
Gate chamber
Anchor support
Penstock
Power house
Motor-generator
Tail water pond
Anchor support
(f) pumped storage plant
Bank layout
Reversible pump-turbine
Draft tube

BASIC ELEMENTS OF HYDEL POWER PLANT



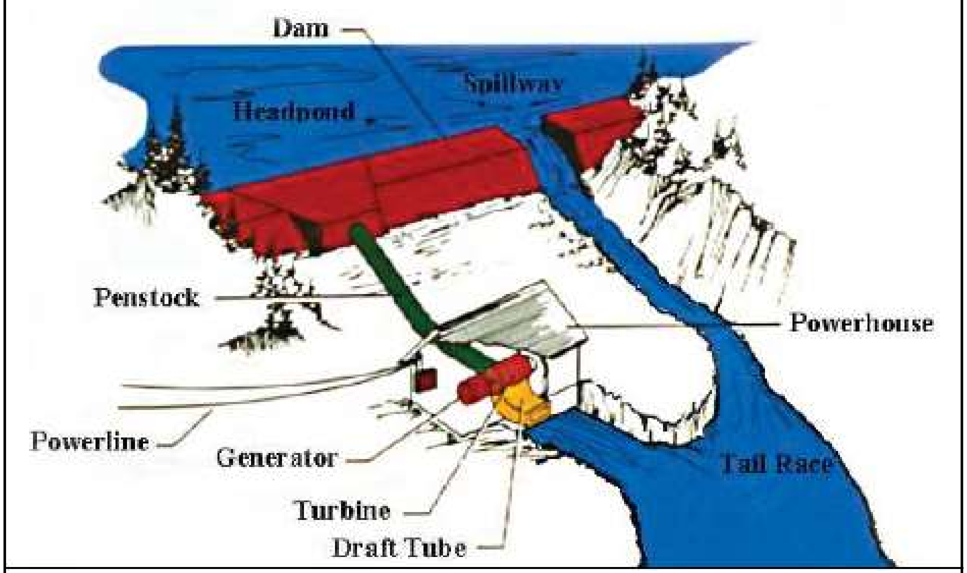
- Reservoir
- Dam
- Trace rack
- For bay
- Surge tank
- Penstock
- Spillway
- Turbine
- Powerhouse

1. Dam and Reservoir
2. Spillway
3. Fore bay
4. Surge tank
5. Penstock
6. Turbine
7. Power house
8. Draft tube



COMPONENTS

COMPONENTS OF A HYDRO SYSTEM



Components of hydropower plants:

1. Dam or weir:

It is a hydraulic structure constructed for diverting water from river creating reservoir or creating head.

2. Intake:

It is the structure meant to draw water from the reservoir or forebay to feed penstock. It essentially consists of trashracks, rakes to clear the trashracks and penstock regulating gates.

3. Headrace canal or Power canal:

It is a canal constructed for the conveyance of maximum discharge from intake to forebay.

4. Pressure tunnel or Headrace tunnel:

It is the structure constructed for the conveyance of pressure flow from intake to surge tank.

5. Forebay:

It is the storage basin situated at the beginning of a penstock leading water to turbine. Its primary function is to store water temporarily when the load on the plant is reduced and to supply the initial water required when load is suddenly increased. It is provided with intake structure to direct water to penstock and a bypass channel or spillway to dispose of the excess water.

6. Penstock:

It is the conduit that conveys water from the intake structure to the power plant. It essentially consists of large diameter steel pipes to carry water under pressure.

7. Surge tank or surge chamber:

It is a tank/chamber provided near by the power house at the entrance of the penstock for the release of any water hammer development due to the sudden closer of the valve, in case of flow conveyed through the pressure headrace tunnel. It also serves as a storage basin (to absorb the excess discharge when load on the turbine decreases and to supply the extra needed water to the turbine when load increases).

8. Power house:

A power house is a building consisting of a substructure extending from foundation to turbine level, intermediate structure extending from top of draft tube to top of generator foundation and superstructure extending above generator floor level to house generator, exciters, control room, service bay, overhead traveling crane, etc.

9. Scroll casing:

It is a spiral shaped steel or concrete conduit which surrounds the turbine runner and guide mechanism. It distributes the water, that is delivered by the penstock, evenly around the guide vanes and at the same time avoid formation of eddies.

10. Turbine :

It is the machine which converts hydraulic energy into mechanical energy which in turn is used in running the electrical generator by directly coupling the shaft of the turbine with the generator.

11. Draft tube :

It is a conduit which connects the outlet of the reaction turbine runner to the tailrace. Water, as it emerges out of the runner, flows through this pipe of gradually increasing diameter and comes to the tailrace level. It serves the function to achieve recovery of velocity head at runner outlet and to allow the turbine to be set at higher elevation to facilitate maintenance.

12. Tailrace :

It is the exit channel carrying water from draft tube to river. It is designed so that silting or scouring of the channel does not take place.

Power and Energy Potential Study

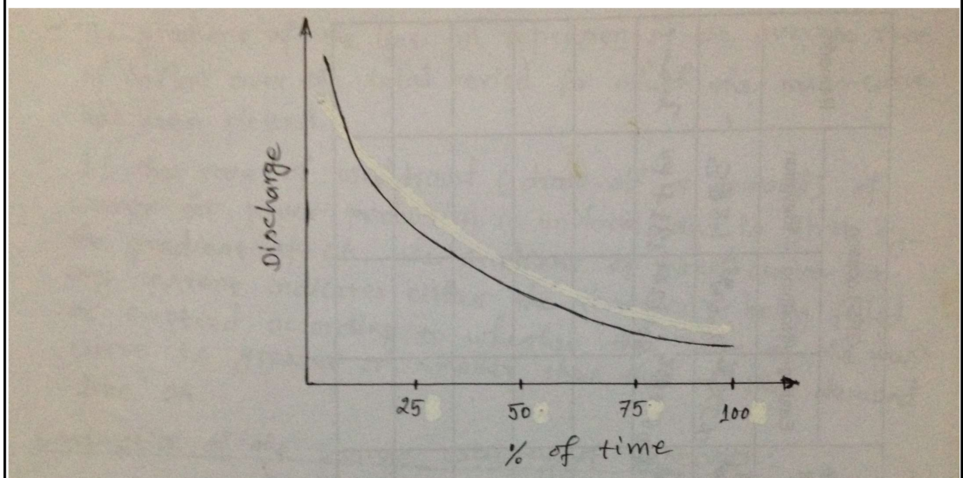
Processing site flow data:

Flow duration curve:

It is the curve showing the percentage of occurance of given discharge during the given span of time. This graph will help to understand the frequency of occurance of particular discharge for the given time.

Usually the run of river hydropower project should be designed with the discharge that will be occuring sufficient period of the time i.e, at least 50% of the time.

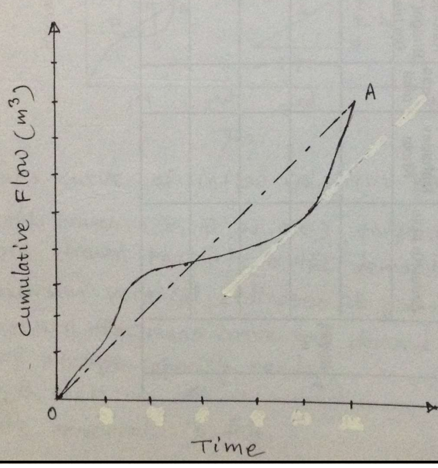
The graph will allow to estimate the firm energy as well as the secondary energy that could be produced with or without Pondage in run of river plants



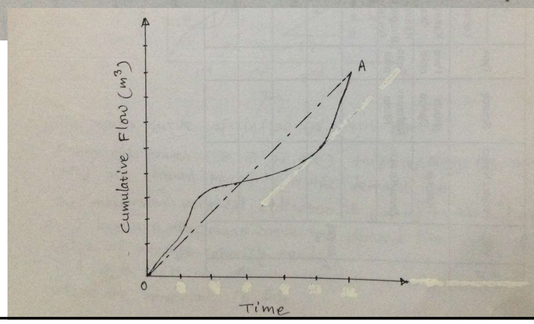
Mass curve or Ripple diagram:

The mass curve is a plot of cumulative flow (volume) against time throughout the period of record.

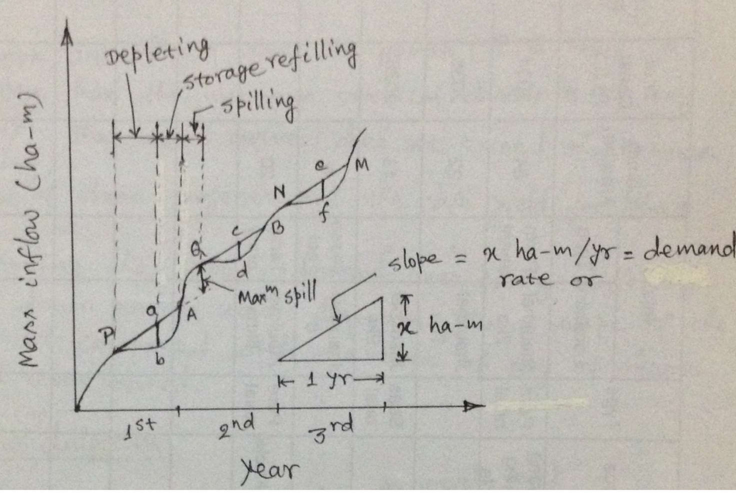
It is useful tool in calculating the storage capacity of the reservoir and safe yield from a reservoir of given capacity. The slope of the curve at any point indicates the rate of inflow at that particular time.



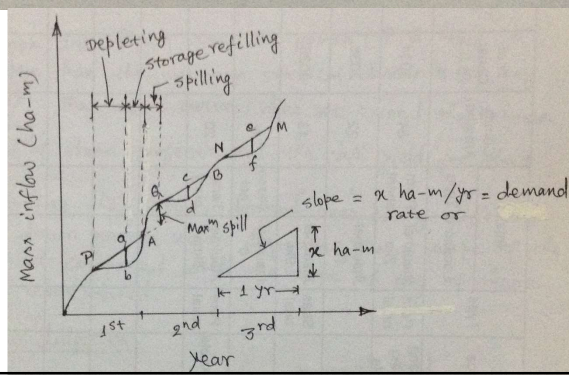
- The gradient of the line OA represents the average rate of inflow over the total period for which the mass curve has been plotted.
- If the rate of withdrawl (draw-off or demand) of water for power production is uniform, and is given by the gradient of OA, the gradient of mass curve at any instant indicates either the reservoir being filled or emptied according to whether the slope of the mass curve is greater or smaller than that of the demand line OA.



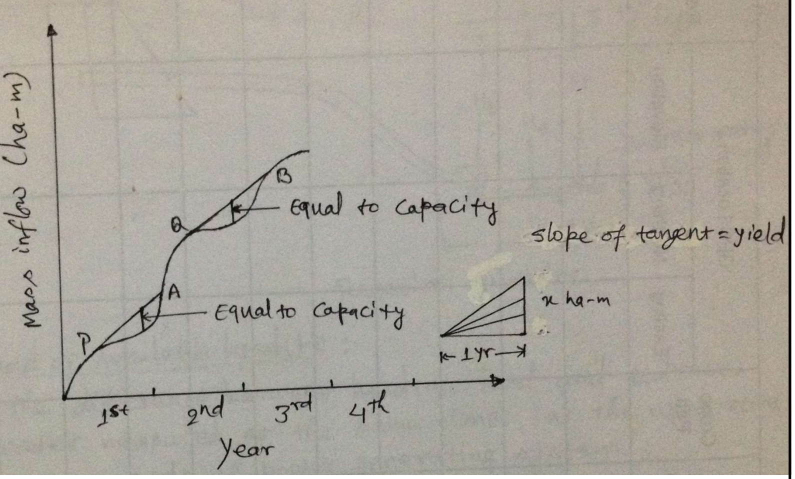
Estimation of the Storage Capacity of Reservoir:



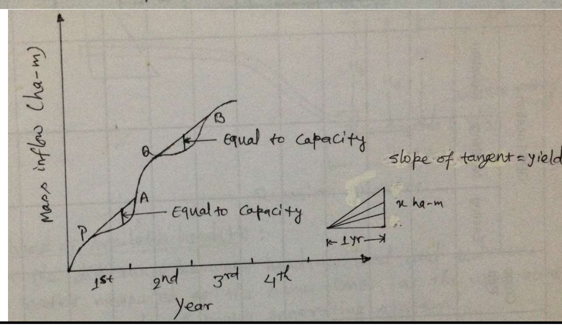
- The mass curve of inflow is first plotted
- From the peaks (P, Q, N, etc.), tangents (PA, QB, NM, etc.) are drawn parallel to the demand line
- The maximum vertical ordinate ab (amongest ab, cd, ef, etc.) between the mass curve and demand line gives the value of storage capacity needed.
- P, A, Q, B, N, M, etc. are assumed to be the points where the reservoir is full.



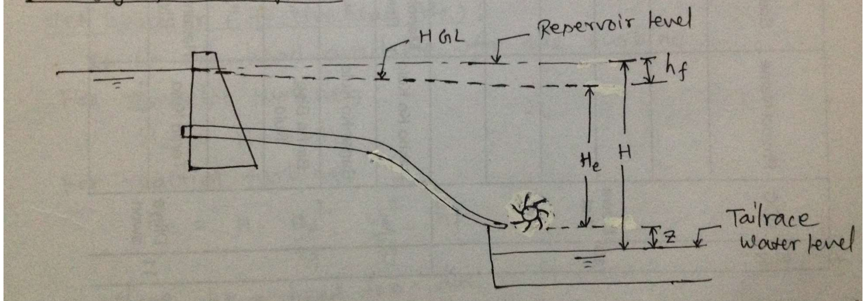
Estimation of the yield from the reservoir:



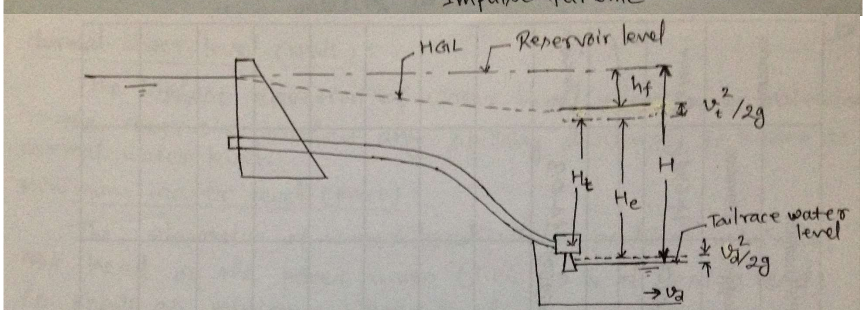
- The tangents are drawn from the peaks (P, Q, etc.) in such a way that the maximum vertical distance of any tangent from the mass curve does not exceed the reservoir capacity.
- The slopes of these tangents give the safe yield for that period.
- For measuring the slopes, demand lines having different rates are drawn on the same diagram. Then, slope of the tangents are compared with the slope of any of these lines and interpolated.



Energy flow diagram:



Impulse turbine



Reaction turbine

Gross head or Available head (H) :

It is the difference between headrace level and tailrace level of water measured at the same time, at the upstream and downstream ends of power generating system.

In storage plant,

H = Difference of water level in the reservoir and water level in the tailrace.

In run of river plant,

H = Difference of water level at the point of diversion of water and the water level at the point where water is returned back to the river.

Operating head (H_0)

It is the difference of water level between forebay/entry to penstocks and tailrace.

H_0 = TEL at entry to penstock − TEL at tailrace exit.

Net head or Effective head (He) :

It is the head available for the turbine.

For impulse turbine,

$$H_e = H - z - h_f$$

For reaction turbine,

$$H_e = H - \frac{v_d^2}{2g} - \frac{v_t^2}{2g} - h_f$$

where, h_f = head loss due to friction

v_t = velocity at turbine

and v_d = exit velocity

Normal water level (NWL) :

The highest elevation of water level that can be maintained in the reservoir without any spillway discharge is known as normal water level.

Minimum water level (MWL) :

The elevation of water level which produces minimum net head on the power units (i.e., 65 % of design head) is known as minimum water level.

Weighted average level (WAL):

The level above and below which equal amounts of power are developed during an average year (i.e., 50% units between NWL and WAL and 50% units between WAL and MWL) is called weighted average level.

Design head (H_d):

It is the net head under which the turbine reaches peak efficiency at synchronous speed.

Rated head (H_r):

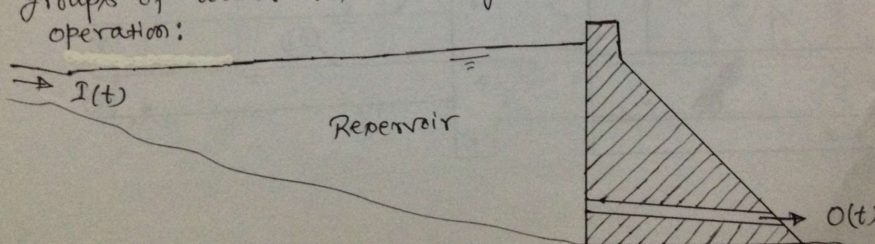
It is the head at which the turbine functioning at full gate opening will produce a power out-put, equal to that specified in the name plate of the turbine.

For maximum over all plant efficiency, H_r should be equal to H_d.

Reservoir Regulation:

It is defined as the rational distribution of river flow in time and space among the different groups of water resources systems.

operation:



Continuity eq^n,

$$I(t) - O(t) = \frac{ds}{dt}$$

where, $I(t)$ = Inflow

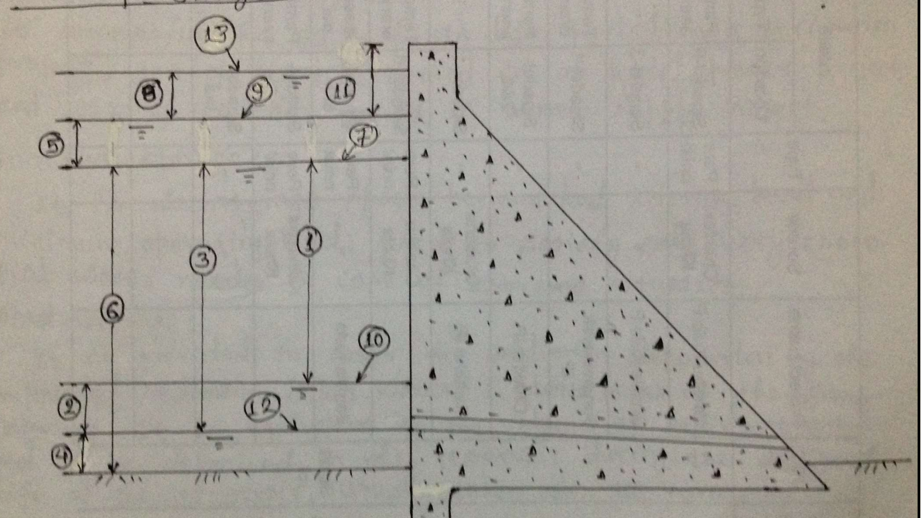
$O(t)$ = out flow

and $\frac{ds}{dt}$ = Change in storage

Inflows are time dependent hydrograph including
- Discharge from upstream
- Direct rainfall on reservoir area
- Ground water seepage into reservoir

Out-flows are time dependent demands including
- Water supply, Irrigation, Hydropower, Navigation, etc.
- Environmental requirement
- Evaporation
- Leakage from reservoir
- Aquifer recharge

Zones of storage in Reservoir:



1→ Active storage or Effective storage or Useful storage
2→ Inactive storage
3→ Live storage
4→ Dead storage
5→ Flood storage
6→ Reservoir capacity or Gross capacity of reservoir or Gross storage or Storage capacity
7→ Retention water level or Top water level or Normal top water level or Full supply level or Normal water level or Normal operation level.
8→ Flood surcharge or Surcharge or Surcharge storage
9→ Maximum water level or Maximum operation level
10→ Minimum operating level or Top of inactive storage
11→ Freeboard
12→ Dead storage level
13→ Exceptional water level

Active storage :

It is the storage available for project purposes; usually between the full supply level and minimum operation level. Active storage has to be sufficient so that the project is successful for 75% of its life period in a irrigation project, 90% of its life period in an hydel power project and 100% of its life period in water supply project.

Inactive storage :

It is the storage space between dead storage level and minimum operating level; used to satisfy only very essential water needs in case of extreme situation.

Dead storage:

It is provided to cater for sediment deposition by the impounded sediment-laden waters; about 15-25% of the gross capacity. It is equivalent to the volume of sediment expected to be deposited in the reservoir during the designed life of the reservoir, usually taken as 100 years.

Flood storage:

It is the storage space between normal operation level and maximum operation level. It is provided in a reservoir for storing flood water temporarily for absorbing high flows for alleviating downstream flood damages.

Flood surcharge:

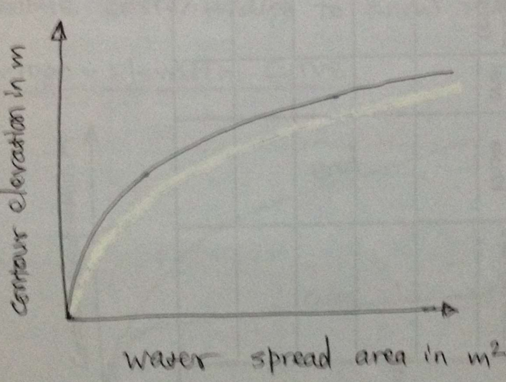
It is the storage space between maximum operation level and exceptional water level which may induced by regulating the outlet gates after the reservoir is filled up to full reservoir level.

Freeboard:

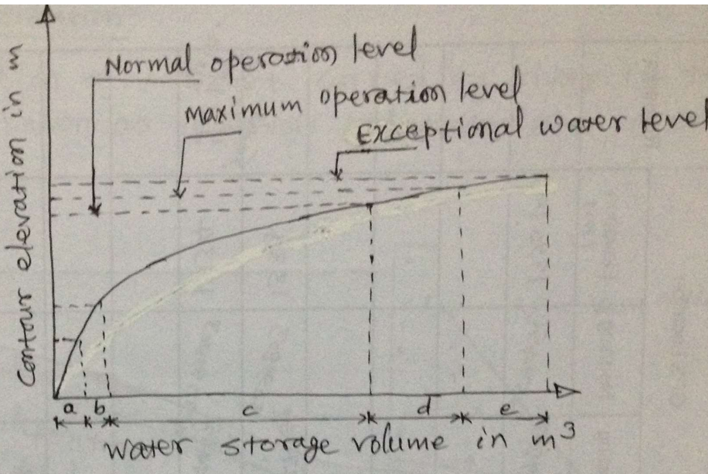
It is the margin between maximum water level and top of the dam. It is provided to avoid the possibility of water spilling over the dam top due to wave action, wind effect and flood surcharge.

Area-Elevation curve and Storage-Elevation curve:

Area-elevation curve also called water spread area curve is a plot of contour elevation against water spread area.



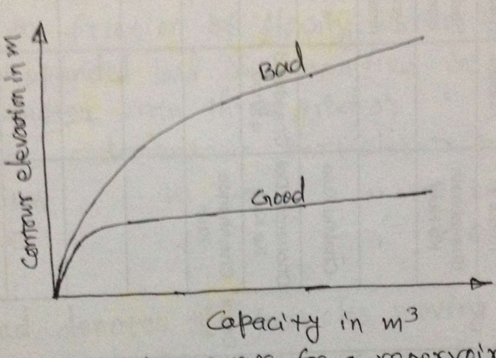
Storage elevation curve also called capacity-elevation curve or capacity curve is a plot of contour elevation against water storage volume.



a → Dead storage
b → Inactive storage
c → Active storage
d → Flood storage
e → Flood surcharge

* Area-elevation curve determines the water spread area corresponding to known reservoir level.
* Storage-elevation curve determines reservoir storage volume corresponding to know reservoir level.

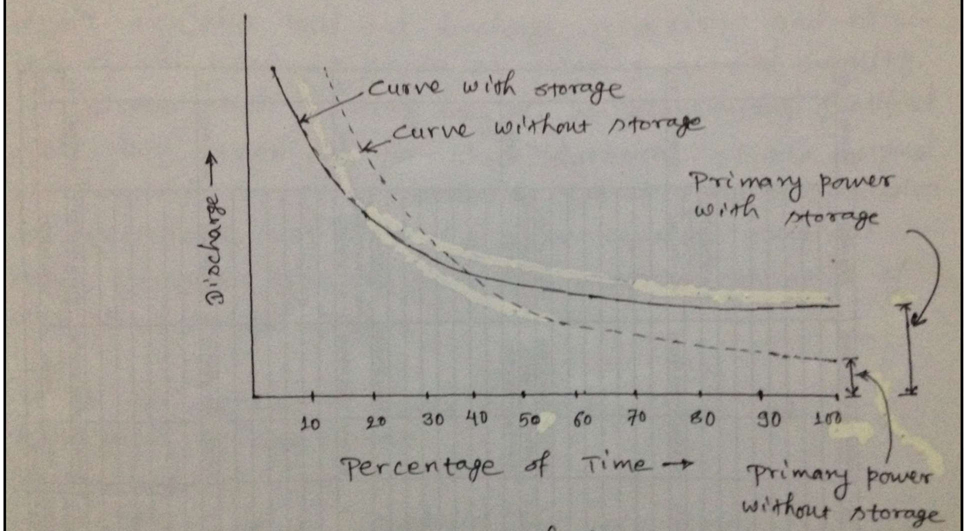
Best Storage - Elevation Curve:



The best capacity curve for a reservoir is the one in which the rise to the straight line is the quickest. It results from a cup-shaped catchment, with gentle longitudinal slope.

Power Regulation

Firm (or Primary) and secondary power:



The firm power is the power which is available throughout the year. It corresponds to the minimum stream flow with or without consideration of storage system. After analysing hydrograph of a river for a series of year the discharge available for more than 95% of the time is determined. The power corresponding to this discharge is usually called firm power. Firm power can be increased by providing storage facility, as it flattens the flow duration curve. The unit price of the firm power is much higher than the secondary power.

The secondary power is the excess power, which is available in the plant exceeding the firm power. 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 inter-connected stations thus affecting economy.

Plant or Installed capacity:

The maximum power which can be produced inaccordance to the available head and discharge in a river and efficiency of the plant is known as plant or installed capacity.

However such capacity may not be economically justified as the plant runs only for short duration. If the marginal cost [variable cost (i.e, installation cost of unit) + operation and maintenance cost of unit] just equal to marginal benefit (revenue from the sales of energy) installation of such plant is justified.

Load:

It is the amount of power delivered or received at a given point at any instant.

Average load:

It is the load produced per unit time.

In other words, it is the hypothetical constant load over a given period of time that would produce the same energy output as the actual loading produced.

Peak load or demand:

The highest instantaneous value of the demand is known as peak load or peak demand.

Load factor (LF)

It is the ratio of the average load over a certain period to the peak load during the same period.

i.e, $\text{Load factor} = \dfrac{\text{Average load}}{\text{Maximum load}}$

$$\text{Daily load factor} = \frac{\text{Average load during 24 hrs}}{\text{Maximum demand}}$$

$$= \frac{\text{Energy consumed during 24 hrs}}{(\text{Maximum demand}) * \text{24 hrs.}}$$

$$\text{Annual load factor} = \frac{\text{Average load during 8760 hrs}}{\text{Maximum demand}}$$

$$= \frac{\text{Energy consumed during 8760 hrs}}{(\text{Maximum demand}) * \text{8760 hrs}}$$

Capacity factor (CF) or Plant factor (PF) or Plant use factor:

It is the ratio of the average load to the plant capacity i.e., the ratio of the average output to the installed capacity of the power plant.

$$\text{CF or PF} = \frac{\text{Average load}}{\text{Plant capacity}} \text{ or } \frac{\text{Average output}}{\text{Installed capacity}}$$

- when maximum load = plant capacity,
  Capacity Factor = Load Factor
- Range of PF = 0.25 to 0.75

Utilization factor (UF):

It is the ratio of quantity of water actually utilized for power production to that available in the river.

If the head is constant,

$$UF = \frac{\text{Power utilized}}{\text{Power available}}$$

- Range of UF = 0.4 to 0.9

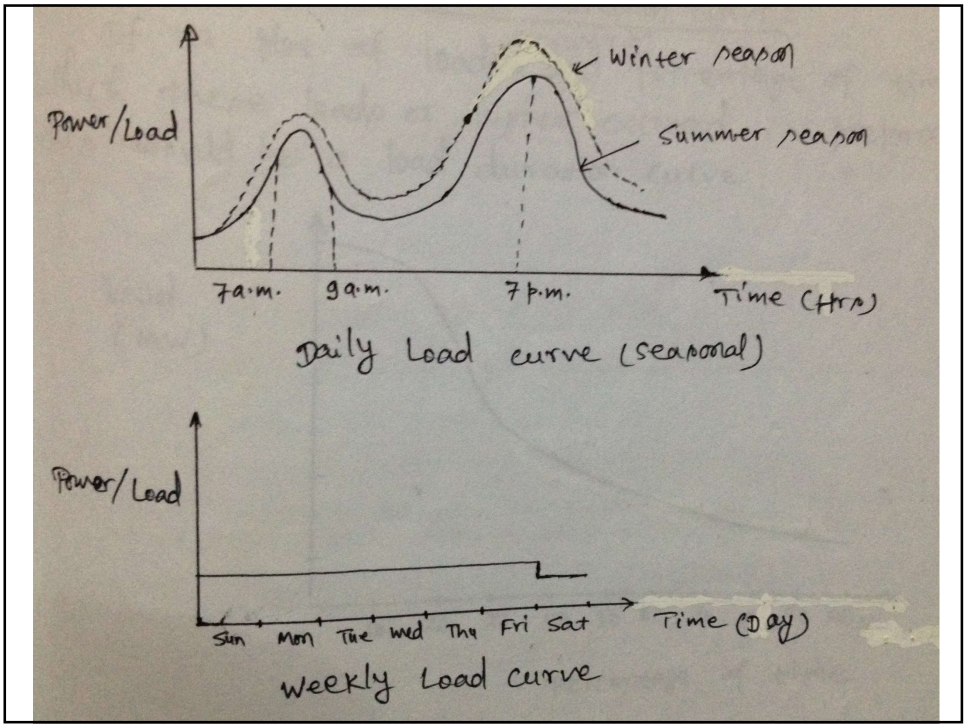
Diversity factor (DF):

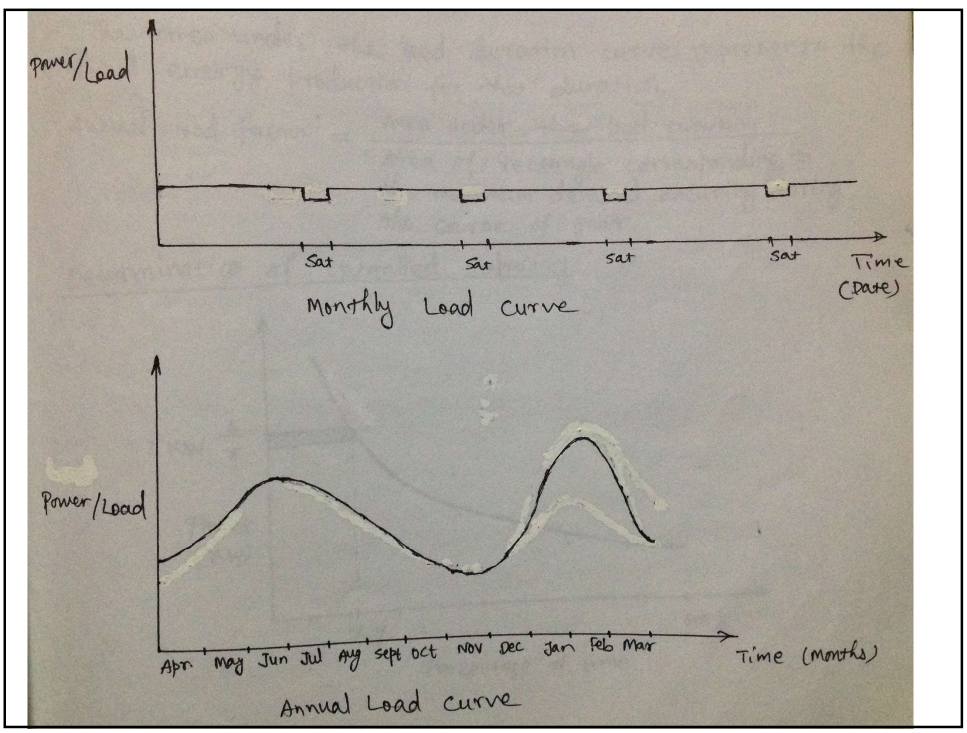
It is the ratio of sum of maximum demands of the individual consumers to the simultaneous maximum demand of all the consumers during a particular time.

$$DF = \frac{\text{Sum of individual maximum demands}}{\text{Simultaneous maximum demand}}$$

Load curve / Power curve:

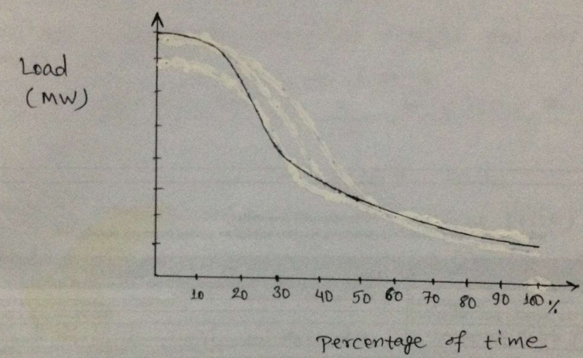
A load curve is a graph of load consumption with respect to time and directly gives an indication of the power used at any time


Winter season
Power/Load
Summer season
7a.m.
9a.m.
7p.m.
Time (Hrs)
Daily Load curve (seasonal)
Power/Load
Sun Mon Tue Wed Thu Fri Sat
Time (Day)
Weekly Load curve


Power/Load
Sat
Sat
Sat
Sat
Time (Date)
Monthly Load curve
Power/Load
Apr. May Jun Jul Aug Sept Oct Nov Dec Jan Feb Mar
Time (Months)
Annual Load curve

Load duration curve / Power duration curve:

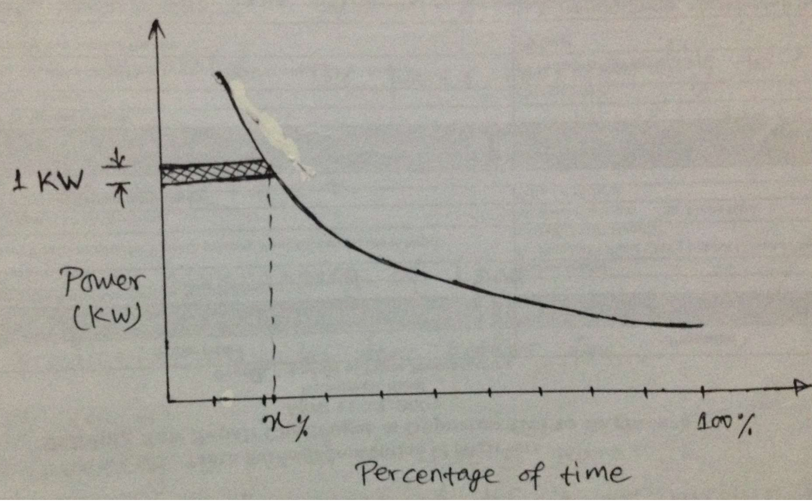
If a plot of loads (powers) and percentage of time during which these loads or higher occured is prepared, then this would be a load duration curve.



- The area under the load duration curve represents the total energy production for that duration.

$$\text{Annual load factor} = \frac{\text{Area under the load curve}}{\text{Area of rectangle corresponding to the maximum demand occuring during the course of year}}$$

Determination of installed Capacity:



Let, variable cost = U\$ V/KW (Capital cost) = Investment
= Electro-mechanical installation cost
Energy price = U\$ E / MWh

while analyzing installed capacity cost of civil works is assumed to be of constant nature

Let us analyse the marginal benefit and cost of 1 KW installation.

Total energy generated in x% of time = $\frac{1}{1000} * 8760 * x$ MWh

$= 8.76\,x$ MWh

Marginal benefit = U\$ $8.76x * E$ (selling price)

Total cost (C) = Civil work cost + variable cost + operation and maintenance cost

Marginal cost = Annual cost (Ac) + O & M cost (≈ 2% of V)

Annual cost (Ac) = U\$ $V * \frac{i}{\left[1 - \frac{1}{(1+i)^N}\right]}$

where, i = Internal rate of return in %
N = Economic life of the project in years

Equating marginal cost and marginal benefit, value of x can be obtained. The power corresponding to this x% of time in power duration curve is the best installed capacity of the project.

INSTALLED CAPACITY OPTIMIZATION

The power generated by a hydropower plant is a function of head and discharge, which is given by the following equation:

$$P = \eta \gamma Q H$$

Where P =power, γ = specific weight of water, Q = discharge, H = Head, and η = overall efficiency

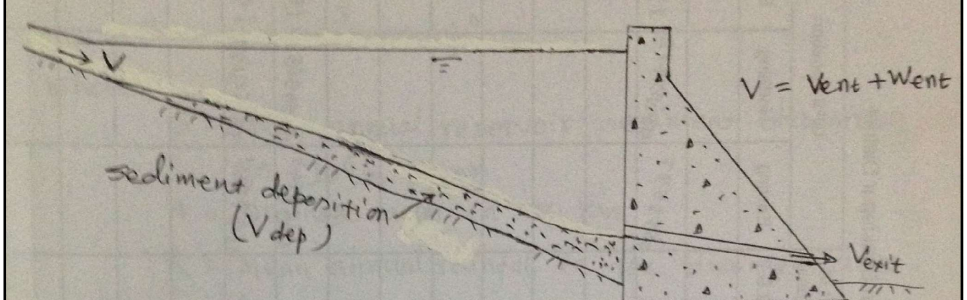
Energy = P*time

Revenue = Energy*rate

The power can be calculated for different percentile of available flow. Increasing the percentile of available flow, the design discharge reduces. Decreasing the design discharge, the size of hydropower components such as intake, settling basin, conveyance system, penstock etc. decreases. Hence, the project cost reduces. But, the project capacity as well as energy also reduces due to the decrease in design discharge, thereby decreasing annual revenue. Hence the revenue and cost is traded to get optimum benefit. For this generally, the flow with 25% available to 70% available with certain interval is calculated from the flow duration curve, and power and revenue is calculated. The cost for each option is also calculated and then optimization study is done. The optimum capacity is taken as installed capacity.

Reservoir sedimentation:

The deposition of sediment carried by river in the reservoir is known as reservoir sedimentation or reservoir silting.



- sediment concentration of yellow river ≈ 1600 gm/lit.
- " " " many Asian rivers ≈ 500 mg/lit. (ppm)

Sediment load consists of
i) suspended load, and
ii) Bed load.

i) suspended load:

The suspended load is very fine and generally kept in suspension because of vertical component of the eddies formed due to friction of flowing water against the bed. The suspended load is classified in terms of sediment diameter into three grades;

- Coarse sediments → particles > 0.20 mm
- Medium " → " 0.20 to 0.075 mm
- Fine " → " < 0.075 mm

ii) Bed load:

Bed load denotes the particles moving on or near the bed. Movement of particles takes place by rolling, sliding and hopping depending on the velocity of flow.

The bed load is generally much smaller, 10 to 15 % of the suspended load.

Estimation of suspended sediments:

i) AIT:

$$S = K * 10^{-3} * (RA)^n$$

where,

S = Mean annual reservoir sediment deposition in $10^6\ m^3/year$

A = Drainage area in sq. km.

R = Mean annual rainfall in the drainage area in mm

K = Drainage area coefficient = 4.5

and n = Rainfall characteristics coefficient = 0.945

ii) RS varshini:

$$S = \frac{1.534}{A^{0.264}} \text{ ; Catchment Area} > 5{,}000 \text{ sq. km.}$$

where, S = Mean annual reservoir sediment deposition in $10^6 m^3/100$ sq. km

and A = Drainage area in sq. km.

iii) Dr. JN Nayak:

$$f_{tr} = C(AR)^n \text{ , Nepalese drainage area.}$$

where, f_{tr} = suspended sediment transportation capacity of the river in kg/m^3 of flow

A = Drainage area in sq. km.

R = Mean annual rainfall in the drainage area in mm

C = Coefficient of drainage area

= 1.02 for drainage laying at 70 – 2000 m above m.s.l.

= 1.15 " " " " 2000 – 4000 m " "

= 1.28 " " " " 4000 – 6000 m " "

and n = Rainfall characteristics coefficient

= 0.052 for drainage laying at 70 – 2000 m above m.s.l.

= 0.039 " " " " 2000 – 4000 m " "

= 0.031 " " " " 4000 – 6000 m " "

Estimation of bed sediments:

$$V = V_{ent} + W_{ent} = V_{dep} + V_{exit}$$

where, V = Mean annual volume of sediments at the entrance of reservoir in m^3/year.

V_{ent} and W_{ent} = Mean annual volume of suspended and bed sediments at the entrance of reservoir respectively in m^3/year.

V_{exit} = Mean annual volume of suspended sediments at the exit of reservoir in $m^3/year$

and V_{dep} = Mean annual volume of deposited sediments in the reservoir in $m^3/year$.

Measurement of sediment load:

- The samples of water carring silt, at various depths, is taken.
- The samples are then filtered and the sediment is removed and dried.
- Then the sediment load is measured in parts per million (ppm) of water.
- No accurate devices to measure bed load and is estimated to about 15% of the suspended load.

Factors affecting sedimentation:

i) Nature of soil of the catchment area:
- If the soil is soft, there is possibility of sheet erosion. Hard soil carry lesser silt.

ii) Topography of the catchment area:
- Steep slopes give rise to high velocities and erode the surface soil.

iii) Vegetation cover:
- Sufficient vegetation catchment area checks velocities and reduces erosion.

iv) Intensity of rainfall:
- Higher rainfall intensity causes greater runoff and more erosion.

Distribution pattern of sediments in reservoirs:

- Sediment deposites in lower reaches of the reservoirs, if located in steeper slope rivers and higher elevations if located in flatter slope rivers.
- Large sediment deposites in lower reaches in reservoirs of shorter length but at higher elevations in reservoirs of longer length.
- If there are constrictions in reservoirs, the sediment deposition is more in upper reaches.
- If large particles are more, more deposition takes place at higher elevations. If fine particles are more, more deposition takes place near the dam.
- Small reservoirs in large rivers discharges almost all fine particles downstream whereas large reservoir may deposite almost all suspended sediment.
- Vegetal growth helps in trapping more sediments at higher elevations.

- In regular shaped reservoirs, suspended sediment deposites uniformly along the direction of flow with decreasing depth away from the reservoir.
- Outlets of adequate capacity with lower elevations reduce the sediment deposition near the dam.

Effects of sedimentation:

- Reduction of active storage capacity (i.e., reduction in power supply)
- May block outlet intakes, especially not frequently operated outlet intakes.
- Increase in flood levels upstream may cause additional submergence.
- River regime at entry to the reservoir is affected (delta formation and braided river pattern).
- Deterioration on reservoir water quality
- Retrogression of river bed d/s of the reservoir.

Sediment control in Reservoirs:

1. Pre-constructing measures:
   - Selection of dam site in such a way as to exclude the runoff from the easily erodible catchment.
   - Construction of the dam in stages
   - Construction of check dams across the river streams contributing major sediment load.
   - Promotion of vegetation growth at the entrance of the reservoir as well as in the catchment.
   - Construction of under-sluices in the dam.
2. Post-constructing measures:
   - Removal of post flood water
   - Mechanical stirring of the sediment
   - Erosion control and soil conservation.

Life of a reservoir:

The term life of reservoir denotes the period during which whole or specified fraction of its total or active capacity is lost.

i) Service life of reservoir:

It is the period for which a reservoir is expected to provide a part of full planned benefits inspite of sedimentation.

ii) Economic life of reservoir:

It is a period for which benefits likely to accure in further operation of the reservoir with the further costs involved in the operation and maintenance of the system but excluding any element to cover the past costs incurred.

iii) Feasible service life of reservoir:

It is the period for which a reservoir is expected to provide a part of full planned benefits in respect of storage in reservoir being imparied by sedimentation.

Trap efficiency (η):

It is the percentage of sediment deposited in the reservoir even inspite of taking precautions and measures to control its deposition.

$$\eta = \frac{\text{Total sediment deposited in the reservoir}}{\text{Total sediment flowing in the river}}$$

- Most of the reservoirs trap 95 to 100% of sediment load flowing into them.
- It has not possible to reduce trap efficiency below 90% or so.

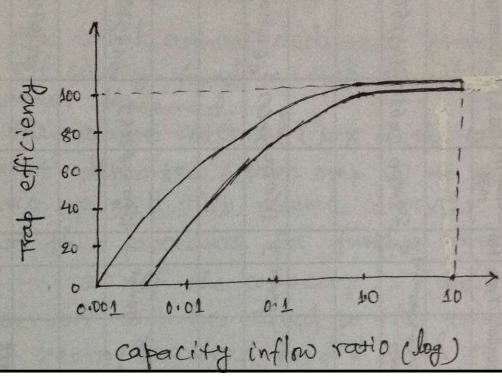
Capacity inflow ratio:

It is the ratio of the reservoir capacity to the total inflow of water in it.

i.e, $\text{Capacity inflow ratio} = \frac{\text{Capacity}}{\text{Inflow}}$

It has found that the trap efficiency is the function of capacity inflow ratio.

$$\eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right)$$



- If capacity reduces with constant inflow, trap efficiency reduces; lesser sediment is trapped.
- For small reservoirs on large rivers, the trap efficiency is extremely low.
- For large reservoirs on small rivers, almost complete deposition of sediment may take place.

Procedure for calculation of life of a reservoir:

i) Knowing the river inflow rate calculate the capacity inflow ratio and obtain the trap efficiency from the trap efficiency curve for the full capacity of the reservoir.
ii) Divide total capacity of the reservoir into any suitable interval (say 10%). Assuming that capacity (10% capacity) has been reduced due to sediment deposit, find the trap efficiency for the reduced capacity (i.e, 90% of the original) and the capacity inflow ratio.

iii) For this interval (10% capacity), find the average trap efficiency by taking the average of trap efficiency found in steps i) and ii).
iv) Determine the sediment inflow rate by taking water samples and drying the sediment.
v) Multiply the total annual sediment transported by the trap efficiency found in step iii), which finds the annual sediment deposited.
vi) Divide the volume interval (10% of the capacity) by the sediment deposited (from step v) to get the number of years to fill this volume interval.
vii) Repeat this procedure for further intervals (i.e, 80%, 70%, ---- 20% of the capacity). The total life of the reservoir will be equal to the total of the number of years required to fill each of the volume intervals.

Service life of reservoirs (J. N. Nayak) :

Degree of the sediment transporting capacity of the river flow mainly determines service life of the reservoir, on which depends the economic effectiveness of the project to be constructed in the country.

Service life of the reservoir in years is given by

$$T = \frac{V}{\frac{\rho_{tr} V_0}{\gamma_1}\left(1 + \frac{\gamma_1}{\gamma_2}\right) + V_b - V_{exit}}$$

where,

ρ_{tr} = suspended sediment transporting capacity of the river flow in kg/m^3

V_0 = Mean annual flow of river in m^3/year.

γ_1 and γ_2 = volume weight of suspended and bed sediments respectively in kg/m^3.

V_b = Mean annual volume of the sediments deposited in the reservoir due to erosion of its banks in m^3/year.

V_{exit} = Mean annual volume of the sediments outleting through the pressure flushing tunnels in m^3/year.

V = Storage capacity of the reservoir in m^3.

Prediction or Forecasting of Load:

Load prediction may be done either for

i) short term
- covering a period of 4 to 5 years
- done for the areas of deficit or surplus power for operation - planning

ii) Medium term
- Covering a period of 8 to 10 years
- Basis for the expansion programme for power generation transmission facilities

iii) Long term
- Covering a period of 20 years or more
- Helps in the formulation of the country's perspective plan for power generation, country's power resources and modes of transmission of voltages

Prediction methods:

The methods of prediction considers the following
- class-wise consumption
- mathematical formulae
- Historical trends
- Correlation between power development and economic development

a) Scheer formula

$$\log_{10} G = c - 0.15 \log_{10} U$$

or, $$G = \frac{10^c}{U^{0.15}}$$

where, G = Annual growth in generation in percentage

U = Per capita generation

and c = Constant = 0.02 (population growth rate) + 1.33

b) Belgium formula

$$E = K M^{0.6}\, 2^{0.465t}$$

where, E = Electricity consumption

M = Index of manufacture of production
t = Time for which consumption is to be projected
K = Adjustment factor

Significance of prediction:

- For the installation of new power project or for the expansion of the existing power plant
- Necessiates to estimate the total amount of load that would be required to be met for various purposes
- Necessiates that the hydro-electric station should be able to cater at least 15 years demand in future from the planning stage

The followed practice is that the full potential of the project is developed in stages

→ First stage – power production corresponding to the immediate demand

→ second and third stages – remaining potential is developed.

Power system and its components:

Power system is the system of such subsystems or components which generate electricity from the naturally available source of energy viz water and transmit it to the place of consumption safely and efficienty.

The power system is said to be composed of following three subsystems.

1. Generation system
2. Transmission system
3. Distribution system

1. Generation system:

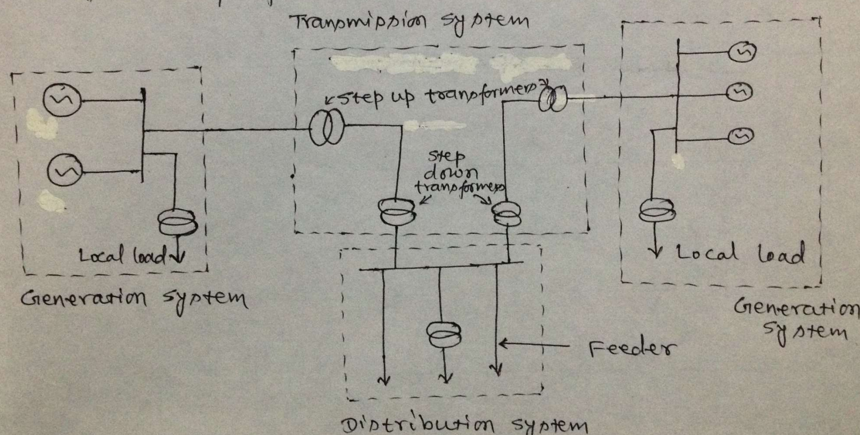
It is the place from where electricity is tapped from the sources of energy available. It consists of turbines, generators and measuring, protecting and controlling equipments

2. Transmission system:

The function of transmission system is to transmit the power generated at generation station to the load centres or distribution sub-station. It consists of step up transformer at generation end and step down transformer at distribution substation, which is end of transmission system.

3. Distribution system:

The function of distribution system is to supply the power to the consumers from distribution substation. It consists of feeders and distribution transformers.



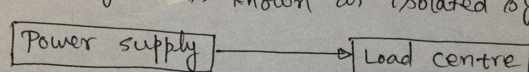
Power grid system:

Power system inter-connection is known as power grid system.

1. Isolated system
2. Inter-connected system

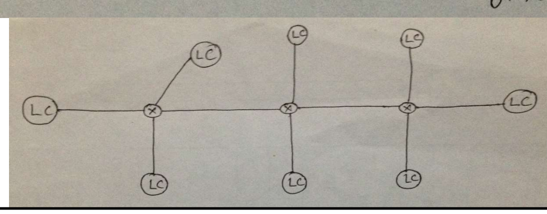
1. Isolated system:

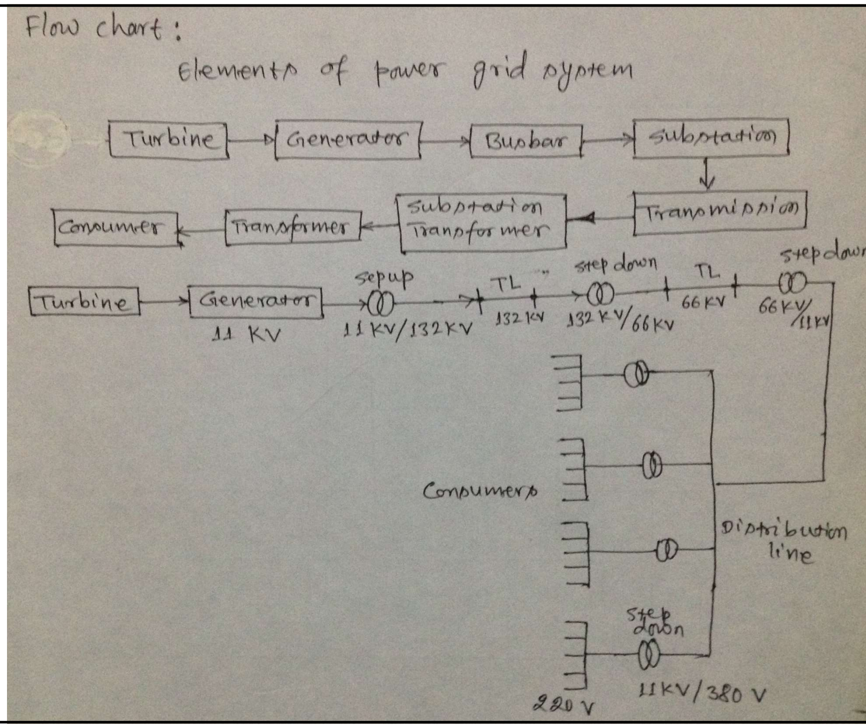
A system of connection in which there is no linkage with the national grid is known as isolated system.



2. Inter-connected system:

A system of connection in which there is linkage with the national grid is known as inter-connected system.





Example 24.1. *The load on a hydel plant varies from a minimum of 10,000 kW to a maximum of 35,000 kW. Two turbo-generators of capacities 22,000 kW each have been installed. Calculate :*

(a) total installed capacity of the plant ;

(b) plant factor ;

(c) maximum demand ;

(d) load factor ;

(e) utilisation factor.

Solution. (*a*) Since two generators, each of capacity 22000 kW are installed, we have

the total installed capacity $= 2 \times 22000\,\text{kW} = 44000\,\text{kW}$. **Ans.**

(*b*) *Plant factor* (*i.e.* capacity factor)

$$= \frac{\text{Energy actually produced in time } t}{\text{Max. energy that can be produced in time } t}$$

$$= \frac{\dfrac{(10000 + 35000)}{2} \times t}{44000 \times t} = \frac{22500}{44000} = 0.511, \textit{ i.e. } 51.1\%. \textbf{ Ans.}$$

(*c*) *Maximum demand* = 35000 kW (as given). **Ans.**

(*d*) *Load factor* $= \dfrac{\text{Average load over a certain period}}{\text{Peak load during that period}}$

$$= \frac{\dfrac{10000 + 35000}{2}\ \text{kW}}{35000\ \text{kW}} = \frac{22500}{35000} = 0.643 \textit{ i.e. } 64.3\%. \quad \textbf{Ans.}$$

(*e*) *Utilisation factor* $= \dfrac{\text{Max. power utilised}}{\text{Max. power available}}$

$$= \frac{35000\ \text{kW}}{44000\ \text{kW}} = 0.795, \textit{ i.e. } 79.5\%. \quad \textbf{Ans.}$$

Example 24.2. *A common load is shared by two hydel stations ; one being a base load station with 20 MW installed capacity, and the other being a stand-by station with 25 MW capacity. The yearly output of the stand-by station is 10×10^6 kWh and that of the base load plant as 110×10^6 kWh. The peak load taken by stand-by station is 12 MW and this station works for 2500 hours during the year. The base load station takes a peak of 18 MW. Find out :*

(*a*) *Annual load factors for both stations.*
(*b*) *Plant use factors for both stations.*
(*c*) *Capacity factors for both stations.*

Solution. We will work out all the required three factors for the base load plant first, and then for the other stand-by plant.

For Base Load Plant. For this base load station, data is given as :

Installed capacity $= 20\ \text{MW} = 20 \times 10^3.\text{kW}$
Yearly output $= 110 \times 10^6\ \text{kWh}$
Peak load taken $= 18\ \text{MW} = 18 \times 10^3\ \text{kW}.$

(*a*) Now, *Annual load factor*

$$= \frac{\text{Total energy (kWh) generated per year}}{\text{Max. power demand in kW} \times 365 \times 24}$$

$$= \frac{110 \times 10^6}{(18 \times 10^3) \times 365 \times 24} = \mathbf{0.698}, \textit{ i.e. } \mathbf{69.8\%}. \quad \textbf{Ans.}$$

(*b*) *Plant use factor* $= \dfrac{\text{Max. power utilised}}{\text{Max. power available}}$

$$= \frac{18000\ \text{kW}}{20 \times 10^3\ \text{kW}} = \mathbf{0.9}, \textit{ i.e. } \mathbf{90\%}. \quad \textbf{Ans.}$$

(*c*) *Capacity factor* $= \dfrac{\text{Av. energy utilised in a period (year)}}{\text{Max. energy that can be produced in that period}}$

$$= \frac{110 \times 10^6}{(20 \times 10^3) \times (24 \times 365)} = \mathbf{0.628}, \textit{ i.e. } \mathbf{62.8\%} \quad \textbf{Ans.}$$

For stand by plant. The data given for this plant is :

Installed capacity $= 25 \text{ MW} = 25 \times 10^3 \text{ kW}$

Yearly output in 2500 hrs $= 10 \times 10^6 \text{ kWh}$

Peak load $= 12 \text{ MW} = 12 \times 10^3 \text{ kW}$.

No. of working hours during the year = 2500 hr.

(*a*) *Annual load factor*

$$= \frac{\text{Total kWh generated per year}}{\text{Max. power demand in kW} \times (24 \times 365)}$$

$$= \frac{10 \times 10^6 \text{ kWh}}{(12 \times 10^3 \text{ kW}) \times (24 \times 365 \text{ h})} = \mathbf{0.095}, \textit{i.e.}\ \mathbf{9.5\%}. \quad \textbf{Ans.}$$

(*b*) *Plant use factor* $= \dfrac{\text{Max. power utilised}}{\text{Max. available power}}$

$$= \frac{12 \times 10^3 \text{ kW}}{25 \times 10^3 \text{ kW}} = \mathbf{0.48}, \textit{i.e.}, \mathbf{48\%}. \quad \textbf{Ans.}$$

(*c*) *Capacity factor* $= \dfrac{\text{Av. output}}{\text{Station capacity}}$

$$= \frac{10 \times 10^6 \text{ kWh}}{(25 \times 10^3 \text{ kW}) \times 2500 \text{ h}} = \mathbf{0.16}, \textit{i.e.}\ \mathbf{16\%}. \quad \textbf{Ans.}$$

Example 24.3. *A run-off river plant is to be constructed across a river at a site where a net head of 22 m is available on the turbines. The river carries a sustained minimum flow of 26 cumecs as dry weather flow. Behind the power station, sufficient water pondage has been provided to supply daily peak load of demand with a load factor of 70%. Assuming the plant efficiency of 58%, determine :*

(*i*) *The maximum generating capacity of the generators to be installed at the power house.*

(*ii*) *The volume of pondage to be provided to supply the daily demand, assuming that the daily load pattern consists of average load for 21 hours and of peak load for 3 hours.*

Solution. The power produced (at dry weather flow) is given by eqn. (24.2), as

$$P = 9.81\,\eta \cdot Q \cdot H \text{ (in kW)}$$

Here $\eta = 0.58$

$Q = 26$ cumecs

$H = 22$ m

$\therefore \quad P = 9.81 \times 0.58 \times 26 \times 22 \text{ kW} = 3251 \text{ kW}$

Load factor $= \dfrac{\text{Av. load (Av. power)}}{\text{Peak load (Peak power)}}$

$\therefore \quad 0.70 = \dfrac{3251}{\text{Peak load}}$

$\therefore \quad \text{Peak load} = \dfrac{3251}{0.7} = 4645 \text{ kW}.$

(*i*) Assuming there is no reserve capacity, we have the *Maximum capacity of the generators to be installed* = 4645 kW. **Ans.**

(*ii*) Excess water from pondage is drawn in order to meet the excess demand (demand in excess of average) for 3 hours.

Excess power reqd. to be developed during 3 hours

$$= \text{Peak demand} - \text{Av. demand}$$
$$= 4645 - 3251 = 1394 \text{ kW}$$

Excess discharge required for developing this excess power is given as

$$P = 9.81\,\eta \cdot Q \cdot H$$

$$\therefore \quad 1394 = 9.81 \times 0.58 \times Q \times 22$$

$$\therefore \quad Q = \frac{1394}{9.81 \times 0.58 \times 22} = 11.15 \text{ m}^3/\text{sec.}$$

This excess discharge is required for 3 hours each day.

$\therefore$ Required pondays per day

$$= 11.15 \times 3 \times 60 \times 60 = 12.04 \times 10^4 \text{ m}^3 \quad \textbf{Ans.}$$

Example 24.4. *A run-off river plant with an installed capacity of 15,000 kW operates at 28% load factor when it serves as a peak load station :*

(*a*) *What should be the minimum discharge in the stream, so that it may serve as a base load station ? The plant efficiency may be assumed to be 80% when working under a head of 20 m.*

(*b*) *Also calculate the maximum load factor of the plant when the discharge in the stream is 35 cumecs.*

Solution. Installed capacity = 15,000 kW

$$\text{Load factor} = 28\%$$

$$\text{Load factor} = \frac{\text{Av. Load}}{\text{Peak Load}}$$

The installed capacity plant serving as peak load plant means that the Installed capacity = Peak load.

$$\therefore \quad \text{Peak load} = \text{Installed capacity} = 15000 \text{ kW}$$

$$\therefore \quad 0.28 = \frac{\text{Av. load}}{15000}$$

$$\therefore \quad \text{Av. load} = 15000 \times 0.28 = \mathbf{4200\ kW.}$$

(*a*) Now, when this plant has to serve as base load plant, it should develop average load, *i.e.* 4200 kW.

Hence, Q reqd. to develop 4200 kW is given by :

$$P = 9.81\,\eta \cdot Q \cdot H$$

or
$$4200 = 9.81 \times 0.8 \times Q \times 20$$

$$\therefore \quad Q = \frac{4200}{9.81 \times 0.8 \times 20} = \mathbf{26.78\ cumecs.} \quad \textbf{Ans.}$$

(*b*) When discharge in the stream is 35 cumecs, then power developed

$$= 9.81\,\eta \cdot QH$$
$$= 9.81 \times 0.8 \times 35 \times 20 \text{ kW} = \mathbf{5488\ kW.}$$

$$\text{Load factor} = \frac{\text{Av. power}}{\text{Peak power}} = \frac{5488 \text{ kW}}{15000 \text{ kW}} = 0.366, \textit{ i.e. } 36.6\%.$$

Hence, maximum load factor at 35 cumecs flow in the stream = **36.6 %.** **Ans.**

Example 24.5. *A run-off river plant is installed on a river having a minimum flow of 15 m^3/sec. If the plant is used as a peak load plant operating only for 6 hours daily, compute the firm capacity of the plant :*

(a) without pondage ;

(b) with pondage but allowing 8% water to be lost in evaporation and other losses. Head at the plant is 16 m, and the plant efficiency may be assumed as 80%.

Solution. (*a*) Power developed by 15 m^3/sec discharge is given by

$$P = 9.81\,\eta \cdot Q \cdot H$$
$$Q = 9.81 \times 0.8 \times 15 \times 16 = \mathbf{1882\,kW.}$$

This represents the firm capacity of the plant without pondage. **Ans.**

(*b*) With pondage, the total volume of water stored during 18 hours when the plant is not operating is

$$= 15 \times (18 \times 60 \times 60)\ m^3 = 9.72 \times 10^5\ m^3$$

The loss of water due to evaporation, etc., per day

$$= 10\% \times 9.72 \times 10^5\ m^3 = 0.972 \times 10^5\ m^3$$

Net amount of available pondage per day

$$= (9.72 - 0.972)\ 10^5\ m^3 = 8.75 \times 10^5\ m^3$$

This daily pondage can supply a uniform Q, (reqd. for 6 hrs. only)

$$= \frac{8.75 \times 10^5}{6 \times 60 \times 60}\ m^3/sec = 40.51\ m^3/sec.$$

∴ Total flow available for power generation (reqd. only for 6 hrs)

$$= (15 + 40.51)\ m^3/sec = 55.51\ m^3/sec$$

∴ Power developed due to this discharge (*i.e.* firm power, because discharge is firm)

$$= 9.81 \cdot \eta \cdot Q \cdot H = 9.81 \times 0.8 \times 55.51 \times 16 = 6963\ kW.$$

Hence, the firm power developed by the plant (if pondage during rest hours is allowed) = **6963 kW. Ans.**

Example 24.6. *During a low water week, a river stream gets an average daily flow of 35 m^3/sec with daily fluctuations requiring a pondage capacity of about 16% of the daily flow.*

A hydroelectric plant is to be located on this river, which will operate for 5 days a week, 24 hours a day, but will supply power at varying rates, such that the daily load factor is of 60%, corresponding to which the pondage required is 0.2 times the meanflow to the turbines. On saturdays and sundays, all the flow is ponded for use on rest of the days.

If the effective head on the turbine, when the pond is full is to be 26 m, and the maximum allowable fluctuation in pond level is 1.2 m, determine :

(*i*) *the surface area of the pond to satisfy all the operating conditions ;*
(*ii*) *the weekly output at the switchboards in kWh.*
Assume the turbine efficiency as 82% and generator efficiency as 90%.

Solution. $Q = 35 \text{ m}^3/\text{sec.}$

Now, Daily flow $= 35 \times 24 \times 60 \times 60 = 3.024 \times 10^6 \text{ m}^3$

Pondage reqd. for discharge fluctuations

$$= 16\% \text{ of daily flow} = \frac{16}{100} \times 3.024 \times 10^6 \text{ m}^3$$

$$= \mathbf{0.484 \times 10^6 \text{ m}^3} \quad ...(i)$$

The discharge at the turbines is caused by the daily flow of 35 cumecs plus the discharge caused by the water stored during the two weekly rest days, *i.e.* saturday and sunday.

Now, *total water stored in 2 rest days*

$$= 2 \times \text{Daily flow} = 2 \times 3.024 \times 10^6 \text{ m}^3$$

$$= \mathbf{6.048 \times 10^6 \text{ m}^3} \quad ...(ii)$$

This stored water is utilised during the 5 working days in the week. Hence, the discharge obtained per day from this storage

$$= \frac{6.048 \times 10^6}{5} \text{ m}^3 = 1.2096 \times 10^6 \text{ m}^3.$$

The streamflow caused by this storage

$$= \frac{1.2096 \times 10^6}{24 \times 60 \times 60} \text{ m}^3/\text{sec} = 14 \text{ m}^3/\text{sec.}$$

∴ Total average flow to the turbines

$$= 35 + 14 = \mathbf{49 \text{ m}^3/\text{sec.}}$$

Hence, pondage required for load fluctuations

$$= 0.2 \times \text{Mean daily flow to turbines}$$

$$= 0.2 \times 49 \times (24 \times 60 \times 60) = \mathbf{0.847 \times 10^6 \text{ m}^3} \quad ...(iii)$$

Hence, the total ponding required

$$= (i) + (ii) + (iii)$$

$$= [0.484 \times 10^6 + 6.048 \times 10^6 + 0.847 \times 10^6] \text{ m}^3 = 7.379 \times 10^6 \text{ m}^3.$$

(*i*) With maximum variation of 1.2 m in pond level, we have the min. surface area of pond reqd. to accommodate $7.379 \times 10^6 \text{ m}^3$ of water

$$= \frac{7.379 \times 10^6}{1.2} \text{ m}^2 = 6.15 \times 10^6 \text{ m}^2$$

$$= \frac{6.15 \times 10^6}{10^4} \text{ hectares} = \mathbf{615 \text{ hectares.}} \quad \textbf{Ans.}$$

Since the allowable max. fluctuation in pond level is 1.2 m, and the head at full level is 26 m, we have the average head on turbines

$$= \frac{26 + (26 - 1.2)}{2} = \frac{26 + 24.8}{2} = \frac{50.8}{2} = 25.4 \text{ m.}$$

(*ii*) $\therefore$ Max. Power generated = $9.81\,\eta \cdot Q \cdot H$.

where η is the combined efficiency of turbines and generators

$$= 0.82 \times 0.90 = 0.738$$

$$Q = 49 \text{ m}^3/\text{sec}$$

$$H = 25.4 \text{ m}$$

$$\therefore \quad P_{max} = (9.81 \times 0.738 \times 49 \times 25.4) \text{ kW} = 9001.4 \text{ kW.}$$

With a daily load factor of 0.60, we have the average power developed = $0.6 \times$ Max. power

$$= 0.6 \times 9001.4 = 5400.8 \text{ kW.}$$

$\therefore$ Weekly power developed at the switchboard

$$= (5400.8 \times 5 \text{ days}) \text{ kW day}$$

$$= (5400.8 \times 5 \times 24) \text{ kWh} = 6{,}48{,}100 \text{ kWh}$$

Hence, weekly output at switchboard

$= \mathbf{6.481 \times 10^5}$ **kWh. Ans.**

Example 24.7. *A run-of-stream station with an installed capacity of 15000 kW operates at 15% load factor when it serves as a peak load station. What should be the lowest discharge in the stream so that the station may serve as the base load station ? It is given that the plant efficiency is 75% when working under a head of 20 m.*

(b) Also calculate the maximum load factor of the plant when the discharge in the stream rises to 20 cumecs.

Solution. Installed capacity = 15000 kW

Load factor = 15%

Since the plant acts as peak load station, the installed capacity would be equal to the peak load.

$$\text{Now,} \quad \text{Load factor} = \frac{\text{Average Load}}{\text{Peak Load}}$$

$$\therefore \quad 0.15 = \frac{\text{Average Load}}{15000}$$

$\therefore$ Average load = $15000 \times 0.15 =$ **2250 kW.**

In order that the plant can act as a base load station, it must supply 2250 kW average power. Using

$P = 9.81\,\eta \cdot Q \cdot H$, we have

$$P = 2250 \text{ kW}$$

$$\eta = 0.75 \quad \text{(given)}$$

$$Q = ?$$

$$H = 20 \text{ m}$$

$$\therefore \quad 2250 = 9.81 \times 0.75 \times Q \times 20$$

$$\therefore \quad Q = \frac{2250}{9.81 \times 0.75 \times 20} = \mathbf{15.31.\ m^3/sec.}$$

Hence, the stream must carry a minimum discharge of 15.31 cumecs, in order to make the plant work as base load station. **Ans.**

(*b*) When $Q = 20$ cumecs, then power developed by the plant is given as :

$$P = 9.81\, \eta \cdot QH$$
$$= 9.81 \times 0.75 \times 20 \times 20 = 2940 \text{ kW}$$

$$\text{Load factor} = \frac{\text{Av. Load}}{\text{Peak Load}} = \frac{2940 \text{ kW}}{15000 \text{ kW}} = 0.196, \textit{ i.e. } \mathbf{19.6\%}. \quad \textbf{Ans.}$$

Example 22.10. *It is observed that a run-of-river plant operates as peak load plant with a weekly load factor of 25%, all this capacity being firm capacity. Determine the minimum flow in river so that power plant may act as a base load plant. The following data is supplied : Rated installed capacity of generating plant = 10 MW; operating head = 16 m; Plant efficiency = 86%.*

If the stream flow is 15 m³/s, find the daily load factor of the plant.

Solution. Weekly load factor = 25%

Rated installed capacity of generating plant = 10 MW (= 10000 kW)

Operating head $\quad H = 16$ m

Plant efficiency, $\quad \eta_0 = 86\%$

Minimum flow in river in m³/sec, Q :

$$\because \quad \text{Load factor} = \frac{\text{Average load}}{\text{Maximum demand}}$$

$$\therefore \quad \text{Average load} = \text{Load factor} \times \text{maximum demand}$$
$$= 0.25 \times 10000 = 2500 \text{ kW}$$

$$E = \text{Total energy generated in one } \textit{week}$$
$$= 2500 \times 24 \times 7 = 42 \times 10^4 \text{ kWh}$$

Now, Power developed, $\quad P = \eta_0 wQH \text{ kW}$

$$= 0.86 \times 9.81 \times Q \times 16 \text{ kW} = 134.98\, Q \text{ kW}$$

$$\therefore \quad E_1 = \text{Total energy generated in one } \textit{week}$$
$$= 134.98\, Q \times 24 \times 7 = 22676.6\, Q \text{ kWh}$$

Now, $E = E_1$

$$42 \times 10^4 = 22676.6\,Q$$

$$\therefore \quad Q = \frac{42 \times 10^4}{22676.6} = 18.52 \text{ m}^3/\text{s}$$

Hence, minimum flow rate = **18.52 m³/s. (Ans.)**

Power developed whe stream flow is 15 m³/s,

$$P_1 = 134.98 \times 15 = 2024.7 \text{ kW}$$

Energy generated per day,

$$E_2 = P_1 \times \text{time} = 2024.7 \times 24 = 48592.8 \text{ kWh}$$

$$\therefore \quad \textit{Daily load factor} = \frac{\text{Average load}}{\text{Maximum load}}$$

$$= \frac{48592.8}{10000 \times 24} = \mathbf{0.2025 \text{ or } 20.25\%} \text{ (Ans.)}$$

Example 22.11. *Calculate the firm capacity of a run-of-river hydro-power plant to be used as 8 hours peak plant assuming daily flow in a river to be constant at 15 m³/s. Also calculate pondage factor and pondage if the head of the plant is 11 m and overall efficiency is 85%.*

Solution. Discharge, $Q = 15 \text{ m}^3/\text{s}$

Plant head, $H = 11 \text{ m}$

Overall efficiency, $\eta_0 = 85\%$

Specific weight of water, $w = 9.81 \text{ kN/m}^3$

P = Firm capacity without pondage

$$= \eta_0 \times wQH = 0.85 \times 9.81 \times 15 \times 11 = 1375.8 \text{ kW}$$

$$PF = \text{Pondage factor} = \frac{t_1}{t_2}$$

where, t_1 = Total hours in one day = 24, and

t_2 = Number of hours for which plant runs = 8

[***Pondage factor*** *is the ratio of total inflow hours in a given period to the total number of hours for which plant runs during the same period.*]

$$PF = \frac{24}{8} = \mathbf{3.} \text{ (Ans.)}$$

$$Q_1 = 15 \times 3 = 45 \text{ m}^3/\text{s}$$

P_1 = Firm power with pondage

$$= 1375.8 \times 3 = 4127.4 \text{ kW}$$

Pondage (*magnitude*) = (24 – 8) = 16 hours flow

$$= 16 \times 60 \times 60 \times 15 = \mathbf{8.64 \times 10^5 \text{ m}^3}. \text{ (Ans.)}$$

Solved Examples 4-3

From the mean monthly flow for a Nepalese river, the power duration curve is calculated which are as shown below.

% time	8.33	16.67	25	33.33	41.67	50	58.33	66.67	75	83.33	91.67	100
Power (MW)	703	588	537	305	256	183	147	107	82	72	59	52

Determine the best installed capacity of the power plant with the following data.

Interest Rate = 10%

Energy price = U$30/MWH

Fixed cost = U$2000/kW

Variable cost = U$600/kW (electromechanical)

O and M = 2% of variable cost

Economic Life of plant = 40 yrs

Solution :

Letus analyze marginal benefit and cost for 1kW installation.

Assuming X% of the duration corresponds to the best-installed capacity for 1 kW power generation, then,

Total annual energy generation = $\frac{1\text{KW} * \text{X\%} * 365 * 24 \text{ MWh}}{1000}$ = 8.76 X% MWh (Annual)

= 8.76 X /100 = 0.876 X MWh per KW

Marginal benefit = U$ 30 * 0.876x = U$ 2.628x

Total cost = Variable cost + O & M Cost

= Actual cost of installed electromechanical equipment + O & M Cost

Here, fixed cost is civil cost and not taken in total cost.

Annual variable cost $= V_{Cost}\left[\left(\frac{(1+i)^n * i}{(1+i)^n - 1}\right]\right.$

Total cost = $V_{Cost}\left[\left(\frac{(1+i)^n * i}{(1+i)^n - 1}\right]+\%\ O\&M * V_{cost}$

$$= 600*\left[\frac{(1.1)^{40} * 0.1}{(1.1)^{40} - 1}\right] + 2\%\ \ of\ 600 = \text{U\$73.36}$$

Equating marginal cost and benefit

$2.628\ X = 73.36$

$\therefore X = 27.91\%$

Hence, the advantageous duration is 27.91%. Now the power plant capacity can be taken as the one that can generate for 27.91% of time in a year. Hence from power duration curve, for given percentile of time, the power can be calculated as

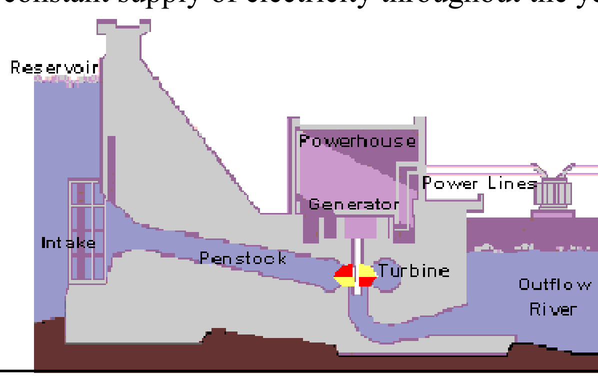
$$P = 537 + \frac{27.91 - 25}{33.33 - 25} * (305 - 537)$$

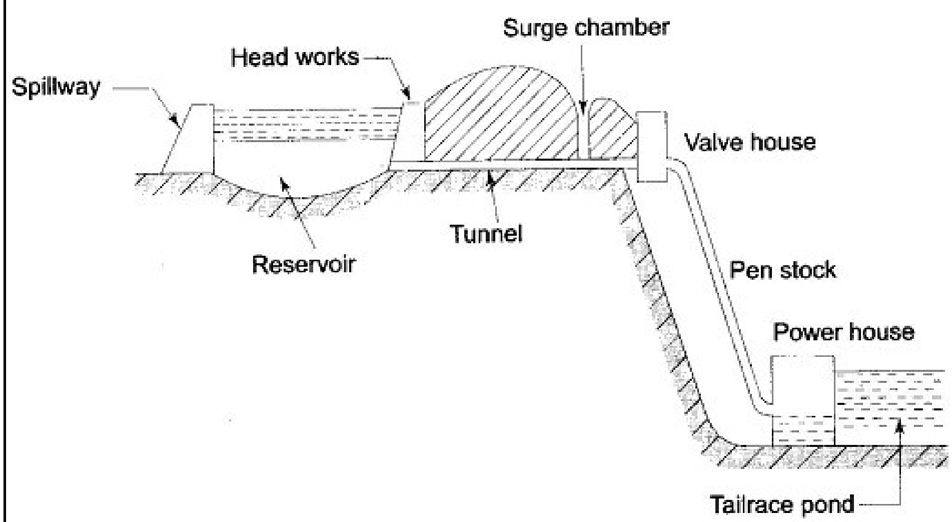
$$P = 455.95KW$$

HEADWORKS OF STORAGE PLANTS AND ROR PLANTS

Storage or Impoundment power Plants

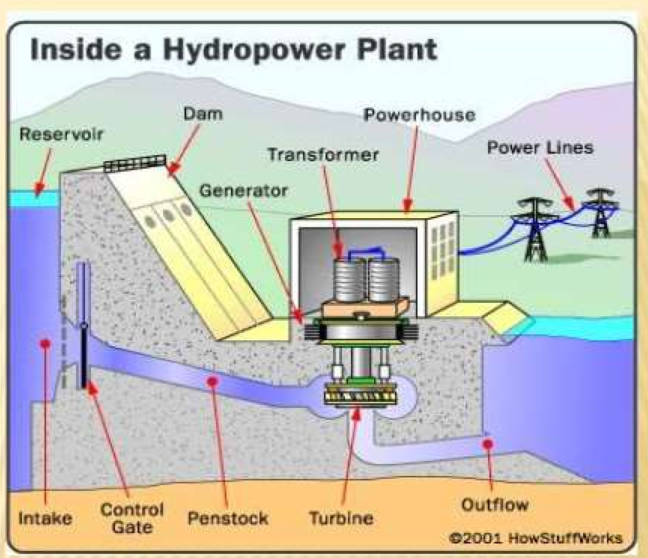
They are the most common and are typically large. They use a dam to store river water in a reservoir and water is released from the reservoir to flow through a turbine(s) which turns a generator to produce electricity. The water level behind the dam can be managed to either maintain a constant reservoir level or released to meet changing electrical needs.
These types of plants have enough storage capacity to off set seasonal fluctuations in water flow and usually that water storage capacity is used to provide a constant supply of electricity throughout the year.





A typical layout for a storage type hydro plant

1. Dam and Reservoir
2. Spillway
3. Trace rack
4. Surge tank
5. Penstock
6. Turbine
7. Power house
8. Draft tube



COMPONENTS

Dam

A dam is a hydraulic structure of fairly impervious material built across a river to create a reservoir on its upstream side for impounding water for various purposes. These purposes may be Irrigation, Hydropower, Water-supply, Flood Control, Navigation, Fishing and Recreation. Dams may be built to meet the one of the above purposes or they may be constructed fulfilling more than one. As such, Dam can be classified as: Single-purpose and Multipurpose Dam.

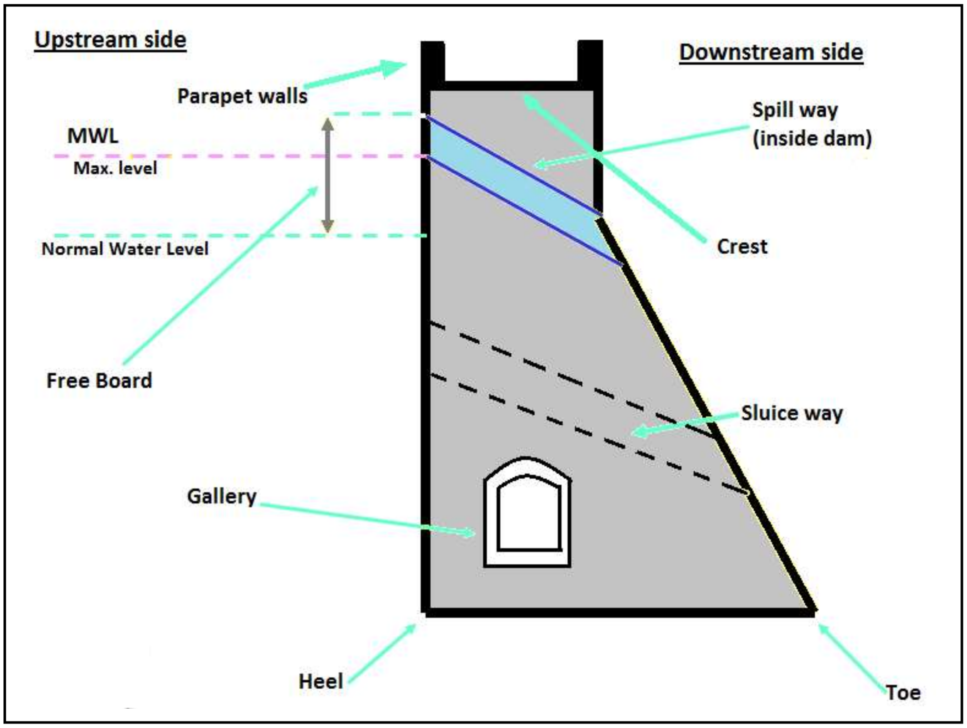
Different parts & terminologies of Dams:

Crest: The top of the Dam. It may in some cases be used for providing a roadway or walkway over the dam.

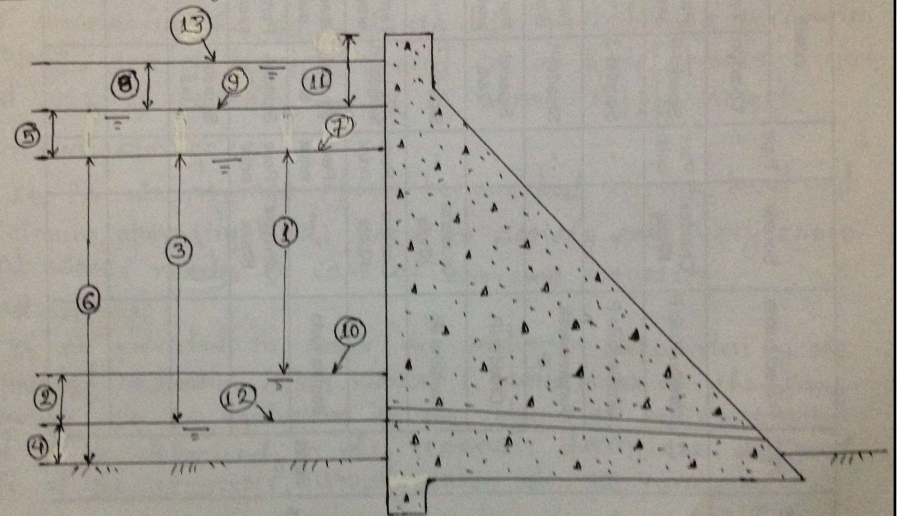
Parapet walls: Low Protective walls on either side of the roadway or walkway on the crest.

Heel: Portion of Dam in contact with ground or river-bed at upstream side.

Toe: Portion of dam in contact with ground or river-bed at downstream side.



Zones of storage in Reservoir:



1 → Active storage or Effective storage or Useful storage
2 → Inactive storage
3 → Live storage
4 → Dead storage
5 → Flood storage
6 → Reservoir capacity or Gross capacity of reservoir or Gross storage or storage capacity
7 → Retention water level or Top water level or Normal top water level or Full supply level or Normal water level or Normal operation level.
8 → Flood surcharge or Surcharge or Surcharge storage
9 → Maximum water level or Maximum operation level
10 → Minimum operating level or Top of inactive storage
11 → Freeboard
12 → Dead storage level
13 → Exceptional water level

Spillway: It is the arrangement made (kind of passage) near the top of dam for the passage of surplus/ excessive water from the reservoir.
Abutments: The valley slopes on either side of the dam wall to which the left & right end of dam are fixed to.
Gallery: Level or gently sloping tunnel like passage (small room like space) at transverse or longitudinal within the dam with drain on floor for seepage water. These are generally provided for having space for drilling grout holes and drainage holes. These may also be used to accommodate the instrumentation for studying the performance of dam.
Sluice way: Opening in the dam near the base, provided to clear the silt accumulation in the reservoir.
Free board: The space between the highest level of water in the reservoir and the top of the dam.
Dead Storage level: Level of permanent storage below which the water will not be withdrawn.
Diversion Tunnel: Tunnel constructed to divert or change the direction of water to bypass the dam construction site. The dam is built while the river flows through the diversion tunnel.

Purpose of Dam

- Dam is generally most suitable in hilly area where deep valleys are available which gives a deep storage of water. The stored water on its upstream side serves various purpose such as:
- Flood Mitigation
- Irrigation
- Water Supply
- Navigation
- Fishery and wild life Preservation
- Hydro-electric Power Generation
- Recreation

Classification of Dams

Classification as per function and use

Storage dam

- This is the most common type of dam normally constructed to store excess flood water which can be utilized later when demand exceeds the flow in river. The Storage dams may be constructed for various purposes such as **irrigation, water supply, hydro-power generation etc.** they may be made of concrete, stone or earth or rock fill etc.

Storage Dam

Classification of Dams

Detention Dams

- This type of dams are mainly constructed to control flood. This type of dam stores water temporarily and releases it gradually at a safe rate when the flood recedes. Detention dam provides safeguard against possible damage due to flood on the downstream side of it. Sometimes a detention dam may also be used as storage dam.

Detention Dams

Classification of Dams

Diversion Dam

- The purpose of diversion dam is necessarily different. It is constructed to divert the river water into canal, conduit etc. For this purpose, mostly a weir or low level dam is constructed across the river to raise the water level which can be diverted as per the needs. This type of dam may be used for water supply, irrigation or some other purposes.

Diversion Dam

Debris or Check Dam

It is a low height dam across a stream channel for soil conservation and retention of sand, gravel, driftwood or other debris to lower reaches.

Coffer Dam

It is a temporary dam constructed to segregate the dam construction area (working area) from the river flow.

Classification of Dams

Classification as per hydraulic design

Overflow Dam: An overflow dam is built to allow the overflow of surplus discharge above the top of it. They are generally built of masonry or concrete and they are gravity type of dam. Usually dams are not designed as overflow for their entire length. Only few meters of its length is kept as overflow section.

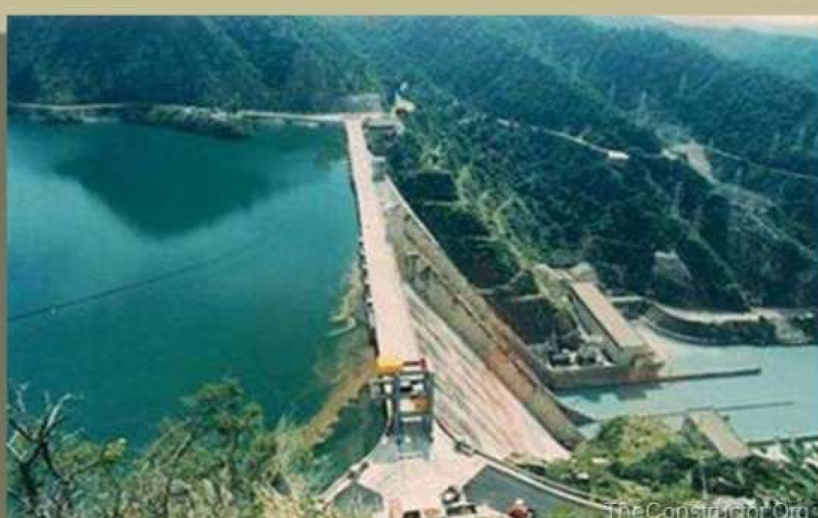
Overflow Dam

Classification of Dams

Non-Overflow Dam:

- In this type of dam, water is not allowed to overtop the dam. The top of the dam is fixed at a higher elevation than the expected maximum flood level. Since water is not allowed to overtop, it can be constructed of large variety of materials such as earth, rock fill, masonry, concrete etc.

Non-Overflow Dam



Classification as per Structural Design

Gravity Dam:

- It is a solid concrete or masonry dam that all external forces are resisted by its own weight or gravity forces.

Arch Dam:

- An arch dam is curved masonry or concrete dam which has convex portion facing upstream. It resists major portion of water pressure by arch action.
- The self weight of the dam is comparatively lesser than gravity dam.

Gravity Dam & Arch Dam

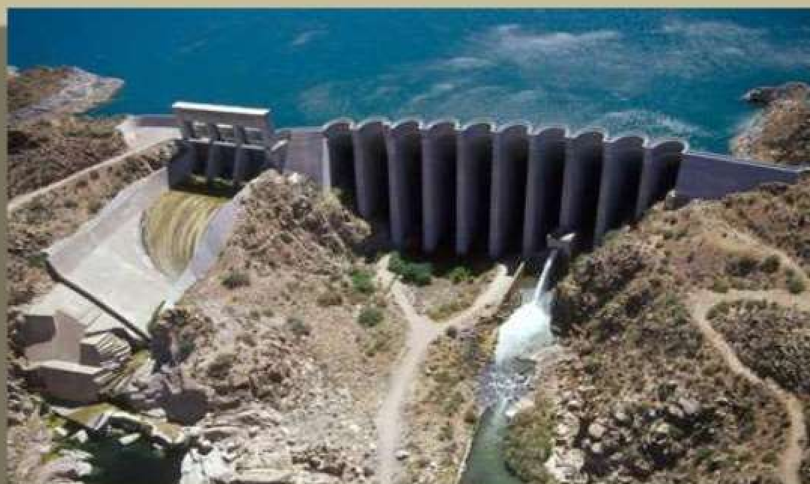
Classification as per Structural Design

Buttress Dam:

- It consists of sloping membrane or deck on upstream which is supported by number of buttress or piers. These buttress are constructed of reinforcements concrete and supported by struts or bracings.

Buttress Dam

Buttress Dam



Classification as per Structural Design

Embankment Dams

- They are constructed of locally available soils, gravels and sands, which resists all external forces by it shear strength. These types of dams are more suitable up to moderate height. They are generally trapezoidal in section. Earth dam, earth and rock fill dam are the example of this type of dam.

Embankment Dams

Earthen Dams

Rock Fill Dam

Classification as per size

1. ***Large (Big) dam***
2. ***Small dam***

- International Commision on Large Dams, (ICOLD) assumes a dam as big when its height is bigger than 15m.

- If the height of the dam is between 10m and 15m and matches the following criteria, then ICOLD accepts the dam as big:

  - If the crest length is bigger than 500m
  - If the reservoir capacity is larger than 1 million m^3
  - If the flood discharge is more than 2000 m^3/s
  - If there are some difficulties in the construction of foundation

Classification as per height

- *High Dam or Large Dam*
  - If the height of the dam is bigger than 100m
- *Medium Dam*
  - If the height of the dam is between 50m and 100m
- *Low Dam or Small Dam*
  - If the height of the dam is lower than 50m

Classification as per material of construction

Rigid Dams

- These dams are built of rigid materials such as masonry, concrete, steel or timber. In earlier times dams were mostly built of stone masonry which have been replaced now-a-days by concrete.

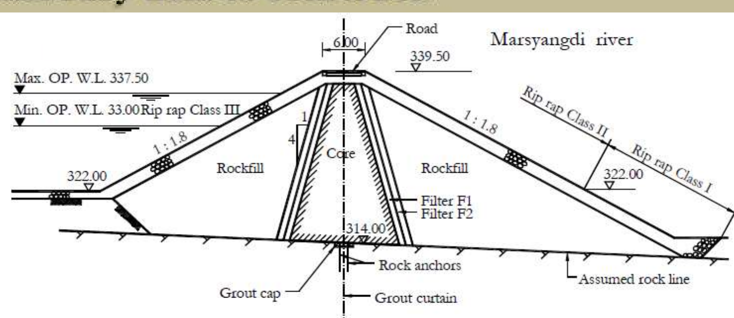
Non-Rigid Dams

- These dams are built of non-rigid materials such as earth, rock fill etc. Earth Dam, Rock fill Dam etc are the examples of this type.

Rigid Dams & Non-Rigid Dams

Embankment Dams

- They are the most ancient type of dams that can be build by naturally available materials with minimum of processing. These dams are not suited for sites where good foundations is not available at a reasonable depth for concrete or masonry dam to construct.



Embankment Dam at Marsyangdi Hydroelectric Project, Nepal (NEA, 1983)

Embankment Dams

TYPICAL EMBANKMENT DAM PROFILE

Embankment dams shall have a trapezoidal cross-section composed of soil. It shall typically be composed of the following elements

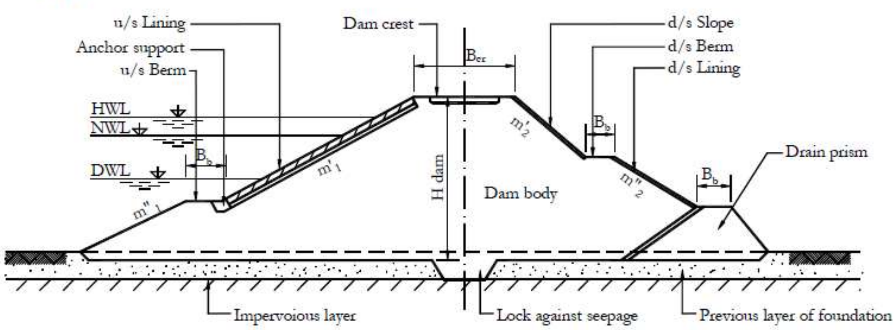
a. Dam body.

b. Dam crest.

c. Upstream and downstream berms.

d. Upstream and downstream lining.

e. Drains.



Terms and abbreviations

Axis of dam Vertical plane or curved surface, chosen by a designer, appearing as a line in plan or in cross-section to which the horizontal dimensions of the dam are referenced.

Berm A nearly horizontal step in the sloping profile of an embankment dam.

Composite earth dam Earth dam consisting essentially of an inner or enclosed impervious section supported by two or more outer sections of relatively pervious material.

Dam crest Upper (top) part of the dam body.

Embankment dam
A dam with the main section composed principally of gravel, sand, silt and clay. It is also called earthen dam.

Homogeneous earth dam Earth dam composed of a single type of material, except for protective material on the exposed faces.

Length of dam The length along the top of the dam.

Rock-fill dam Dam composed of rock, either dumped in the lifts or compacted in layers ,as a major structural element.

Rock toe The downstream toe of an earth dam or other structure constructed of rock materials.

Seepage Interstitial movement of water that may take place through a dam, its foundation or its abutments.

Toe drain Drain constructed at the downstream toe of an earth dam to collect and drain away the seepage through the dam and its foundation.

Embankment Dams

Earth Dam:

- Earth dams are made of locally available soils, sands and gravel with trapezoidal in section. They are economical and suitable for almost all type of available foundation.

Homogeneous Type

- This type of dam is constructed of a single kind of materials with stone pitching at upstream side to safeguard against erosion. Also have a rock toe at downstream to drain out the seepage water through the body of the dam.

Earth Dam

Zoned Type (Non-homogeneous Type)

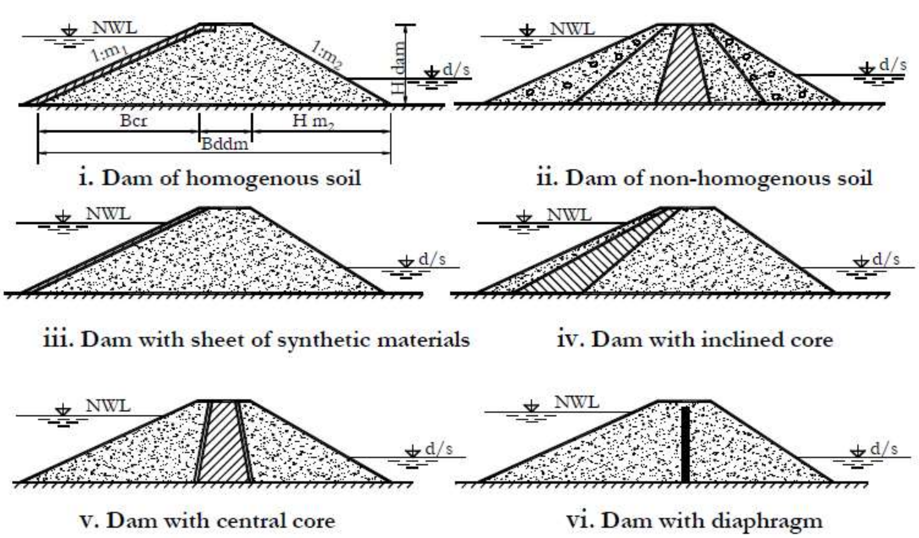
- The zoned type of dam consists of more than one kind of material. It consists of a central impervious core made of clay and outer pervious zone made of mixtures of earth and gravel. It also has rock toe at downstream side and stone pitching on upstream side.

Earth Dam

Diaphragm Type:

- This type of section is used when impervious material is available in lesser quantity at site. It consists of thin impervious core of diaphragm made of clay, cement concrete or bituminous concrete which is surrounded by earth or rock fill. They are also called sometimes as thin core dams. The construction of this type of dam is limited to small dams only. It has upstream stone pitching and downstream stone blanket.

Based on the structural composition of their dam body, embankment dams may belong to the following categories:



Types of embankment dams (Zhurablov, 1975)

Characteristics of Earth Dam

- They are normally constructed when huge quantity of material e.g. soils, gravels are available locally.
- They are suitable for almost all type of available foundation.
- They resist all external forces acting mainly by shear strength of soil.
- They can be built rapidly with relatively unskilled labour because they use locally available material in large quantity.
- They are comparatively cheaper than concrete and arch dam.
- They allow easy increase in their height if needed, without much difficulties.

Characteristics of Earth Dam

- They require a separate spillway away from the main dam.
- They require heavy maintenance cost and constant supervision
- They are more susceptible to be damaged by floods than any other type of dam.

Advantages

(i) Earth dams are usually cheaper than gravity dams if suitable earth for construction is available near the site.

(ii) Earth dams can be constructed on almost all types of foundations, provided suitable measures of foundation treatment and seepage control are taken.

(iii) Earth dams can be constructed in a relatively short period.

(iv) The skilled labour is not required in construction of an earth dam. Earth dams can be raised subsequently.

(vi) Earth dams are aesthetically more pleasing than gravity dams.

(vii) Earth dams are more earthquake-resistant than gravity dams.

Disadvantages

(i) Earth dams are not suitable for narrow gorges with steep slopes.

(ii) An earth dam cannot be designed as an overflow section. A spillway has to be located away from the dam.

(iii) Earth dams cannot be constructed in regions with heavy downpour, as the slopes might be washed away.

(iv) The maintenance cost of an earth dam is quite high. It requires constant supervision.

(v) Sluices cannot be provided in a high earth dam to remove slit.

(vi) An earth dam fails suddenly without any sign of imminent failure. A sudden failure causes havoc and untold miseries.

Rock fill Dam

- It is the type of embankment dam which uses various sizes of materials to provide stability. It also has impervious membrane on upstream face to provide water tightness.
- Impervious membrane is usually made of concrete. Rock fill dam is preferred when plenty of rocks are available from nearby quarry.
- Rock fill dams require foundation which will result in a minimum settlement. The foundation should be free from all foreign materials like silt, clay, sand etc. The upstream and downstream slopes of a rock fill dams depend on the type of impervious membrane and its location. They are cheaper then concrete dam and can be built rapidly if proper rock is available.

Rock fill Dam

(a) **Advantages**

(i) Rockfill dams are quite inexpensive if rock fragments are easily available.

(ii) Rockfill dams can be constructed quite rapidly.

(iii) Rockfill dams can better withstand the shocks due to earthquake than earth dams.

(iv) Rockfill dams can be constructed even in adverse climates.

(b) **Disadvantages**

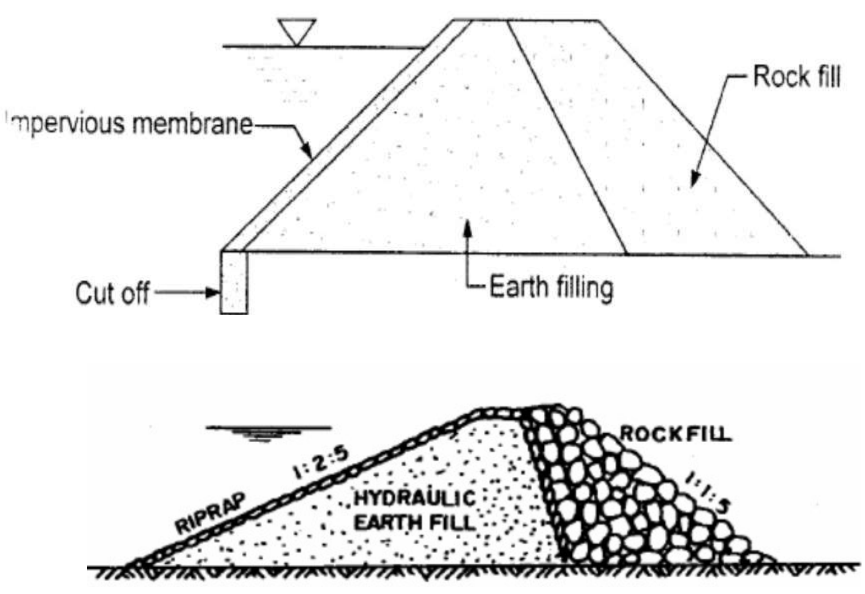
(i) Rockfill dams require more strong foundations than earth dams.

(ii) Rockfill dams require heavy machines for transporting, dumping and compacting rocks.

Combined Earth and Rock fill Dam

- It is a composite embankment dam. In this upstream consists of soil where as downstream portion is filled with rock. Upstream has a riprap. With cement grouted core wall to check seepage. Riprap makes upstream slope stronger against seepage and damage due to wave action.

Combined Earth & Rock fill Dam



Concrete Dams

- They are Categorized as rigid dams because they are constructed of rigid material like concrete. They may be either straight or curved in plan. These types of dams are normally best suited on solid rock foundations. The construction of such dams requires heavy mechanized plants, concrete, aggregate, cement and sand.

Concrete Dams

Gravity Dams

- It is a solid concrete or masonry dam which resists all external forces by its own weight. It needs a sound rock foundation because it transmits all the forces including self weight to the foundation. Most of the gravity dams are provided with an overflow portion known as spillway within the body of the dam

Gravity Dams

Gravity Dams

Advantages

- It is Stronger and more stable than any other type of dam
- It can house an overflow spillway to pass excess flood water safely.
- It can built of any height provided suitable foundation is available to bear all the loads coming on it.
- The failure of a gravity dam is not sudden at all. It gives sufficient time for evacuation of area downstream of it.

Gravity Dams

Disadvantages

- Its construction is possible only on sound rock foundation.
- Initial cost is higher.
- It needs skilled labor and mechanized plants for construction.
- It may take more time in construction, if manufacturing and transporting equipments are not available.

Arch Dam

- It is curved Concrete Dam. The self weight of this dam is quite less compared to gravity dam it transmit major portion of water load to the abutments.

Advantages

- It is particularly suited in Deep Georges where length is same compared to its height.
- Very small portion of water pressure is transmitted to foundation hence it can be built on moderate or weak foundation.
- It has less initial cost as compared to Gravity Dam.

Arch Dam

Arch Dam

Disadvantages

- It needs skilled labour, sophisticated formwork and specialized design.
- Construction time is normally Large.
- It needs very strong abutments of solid rock to resist arch thrusts
- It is not suitable if solid rocks are not available.

Buttress Dams

- They may be considered as lightened version of the gravity dam. A buttress dam consists of a continuous inclined upstream face supported by downstream buttresses at regular intervals.

Buttress Dams

Buttress Dams

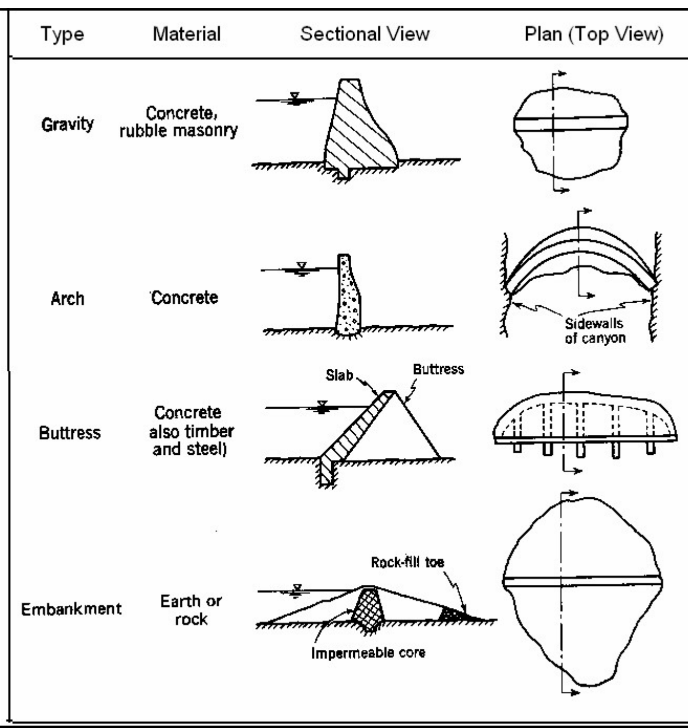
Advantages

- It requires less materials for construction
- It can be constructed on even weak foundation as pressure on foundation is quite less.
- The water pressure acts normal to the inclined deck. Hence the vertical components of water pressure stabilizes the dam against overturning and sliding.

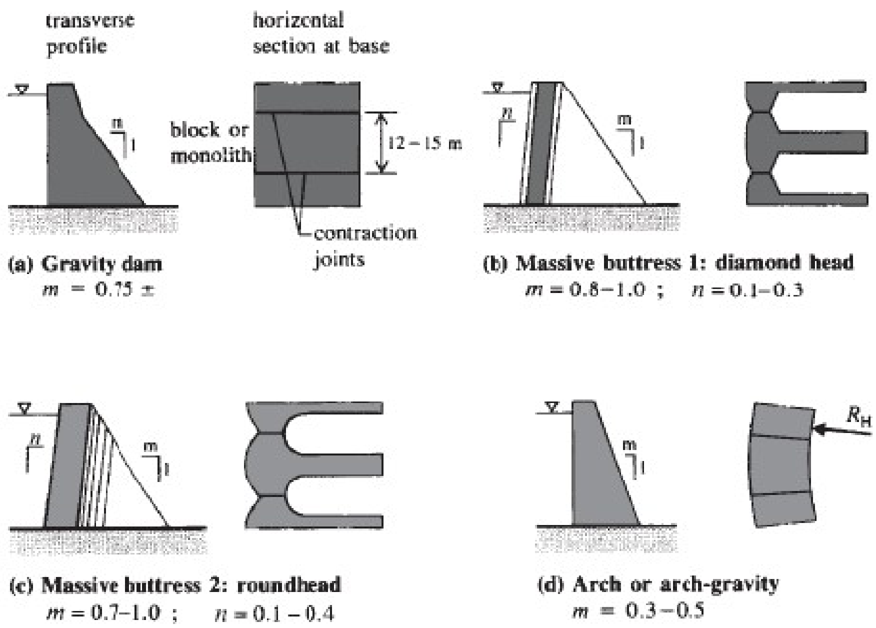
Buttress Dams

Disadvantages

- It requires more formwork than solid concrete dams.
- As Thickness of upstream concrete surface is less, it is more liable to get deteriorated
- It requires constant maintenance and supervision.
- Life of dam is less as compared to other dams.



Principal variants of concrete dams (values of m are indicative only)



Principal variants of concrete dams (values of m and n are indicative only)

(e) Cupola or double-curvature arch

Fig. Principal variants of concrete dams (values of m and n indicative only; in (e) R_H and R_V generally vary over dam faces)

Principal variants of concrete dams (values of m and n are indicative only)

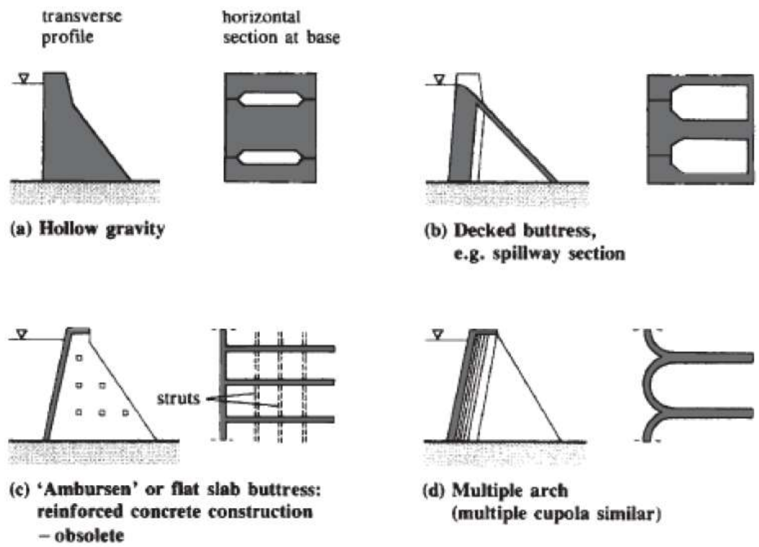
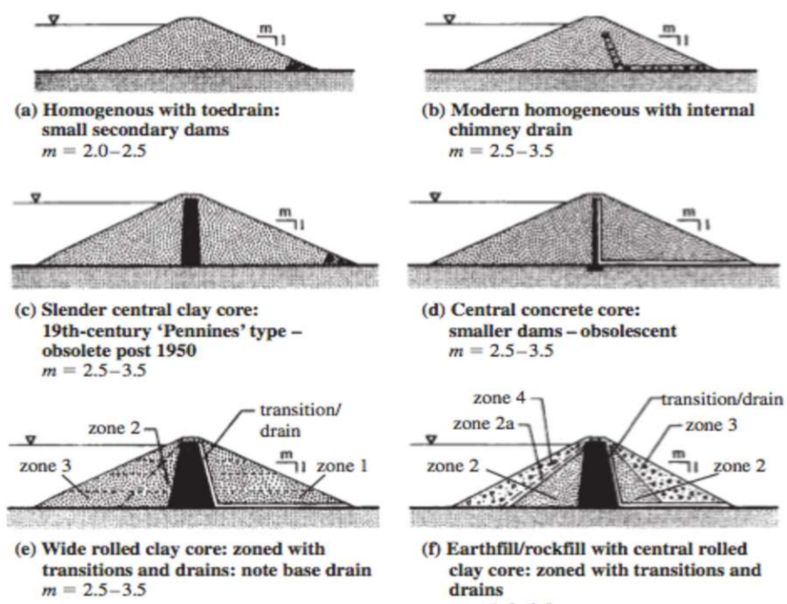
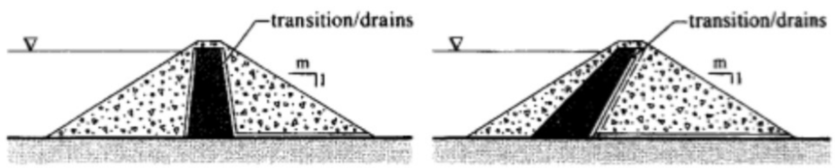


Fig. Further variants of concrete dams

Principal variants of earthfill and earthfill–rockfill embankment dams (values of m are indicative only)

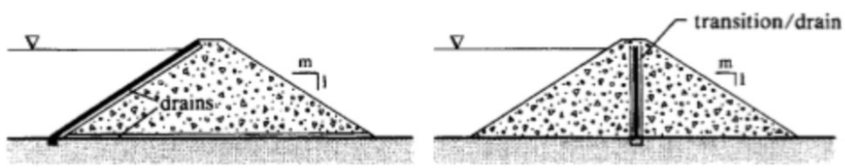


Principal variants of rockfill embankment dams (values of m are indicative only)



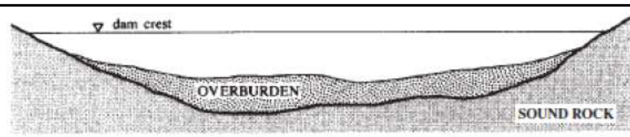
(a) Central rolled clay core
m = 1.6–2.0

(b) Inclined rolled clay core
m = 1.6–2.0

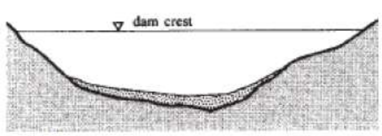


(c) Decked: upstream asphaltic or concrete membrane
m = 1.6–2.0

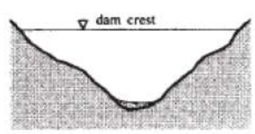
(d) Central asphaltic membrane
m = 1.6–2.0



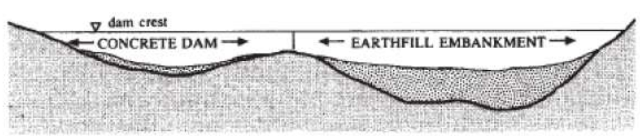
(a) Wide valley with deep overburden:
fine-grained deposits, e.g. Fine glacial soils, tills etc., over 5m deep favour earthfill embankment dam



(b) Valley with little overburden:
suitable for embankment, gravity, or buttress dam



(c) Narrow valley, steep sides, little overburden:
suitable for arch, cupola, or rockfill embankment dam



(d) Valley with irregular depth of overburden:
possible composite solution as shown; spillway on concrete dam

Fig. Illustrative examples of dam type in relation to valley profile

Selection of Type of Dam

Construction of a dam required

1- Political decision
2- Arranging project funding
3- Construction feasibility
4- Environmental impact
5- Social impact

And the construction feasibility required a number of investigations include:-

(i) Site reconnaissance (include survey, geological, and geotechnical investigation, availability of construction material).

(ii) Hydrological data and records.

(iii) Stability and integrity with respect to water retention.

In summary, dam site investigation required careful planning and considerable investigation of time and resources. Wherever possible, in situ and field test techniques should be employed to supplement lab testing program.

Proper interpolation of geological and geotechnical data demands the closest cooperation between the engineering geologist the geotechnical specialist and the dam engineers.

The optimum type of dam for a specific site is determined by estimates of cost and construction programme for all design solutions which are technically valid. Where site circumstances are such that viable alternatives exist it is important that options are kept open, assessing the implications of each with respect to resources, programme and cost, until a preferred solution is apparent.

It may also be necessary to take account of less tangible socio-political and environmental considerations in the determination of that solution.

Dam selection:

Embankment

Earthfill

- Suited to either rock or compressible soil foundation and wide valleys
- Can accept limited differential settlement given relatively broad and plastic core
- Cut-off to sound, i.e. less permeable horizons required
- Low contact stresses
- Requires range of materials, e.g. for core, shoulder zones, internal filters etc.

Rockfill

- Rock foundation preferable; can accept variable quality and limited weathering
- Cut-off to sound horizons required
- Suitable for all-weather placing
- Requires material for core, filters etc.

Concrete

Gravity

- Suited to wide valleys, provided that excavation to rock < 5 m
- Limited weathering of rock acceptable
- Check discontinuities in rock with regard to sliding
- Moderate contact stress
- Requires imported cement.

Buttress

- As gravity dam, but higher contact stresses require sound rock
- Concrete saved relative to gravity dam 30–60%.

Arch and Cupola

- Suited to narrow gorges, subject to uniform sound rock of high strength and limited deformability in foundation and most particularly in abutments
- High abutment loading
- Concrete saving relative to gravity dam is 50–85%

Selection of Site for Dam

(1)Foundation:

- Suitable foundations must be available at the selected site for a particular type of dam. The foundation should be free from seams and faults. It is however possible to improve the foundation conditions by adopting suitable foundation treatments.

(2) Topography:

- Dam site should have a narrow valley to reduce its length it should store maximum volume of water. A major portion of dam should be located on high ground as compared to river basin. This will reduce the cost of dam and facilitates easy drainage of dam section as well.

Selection of Site for Dam

(3) Reservoir:

- Dam site should form deep reservoir with small water surface to reduce (i) evaporation loss and (ii) Submergence area (iii)and control on weed growth. The quantity of leakage through the sides and bed of selected site should be minimum. Reservoir site with the presence of permeable rocks reduces the water tightness of the reservoir.
- For larger storage in reservoir, dam site should be located at the confluence of two rivers.

Selection of Site for Dam

Catchment Area

- The geological conditions of catchment should be such that it yields maximum runoff. It should also have minimum percolation losses. The catchment area should avoid or exclude water from tributaries carrying high percentage of silt in water.

Selection of Site for Dam

Spillway

- A suitable site for spillway should be available near dam site when spillway is to be located separately from dam. E.g. for earth or rock fill dam. There is no special site requirement for the spillway if it is to be built inside the dam.

Spillway

Selection of Site for Dam

Construction Materials

- Huge amount of materials is required for the construction of dam. Therefore construction materials should easily be available either locally or near vicinity of the site so as to reduce the transportation cost.

Selection of Site for Dam

Communication

- The dam Site should be easily approachable so that it can economically be connected to the important towns, cities etc. by rails or roads.

Environmental Conditions:

- Healthy environmental conditions must be available at dam site to set up colonies, residential quarters for labours and other staff members

Factors Governing the selection of Type of Dam

- Selection of the kind of dam is the first task. The choice and selection of dam at a particular place in the river may depend on the following factors.

Topography:

- This is the first factor which governs the choice of dam for a site
- (a) A low rolling plain topography gives choice of an earth dam with a separate site of spillway
- (b) A low narrow V-shaped valley with sound rock in the abutment suggests an arch dam.

Factors Governing the selection of Type of Dam

(2) Geology and Foundation Conditions:

- Next Important factor for the choice of Dam.
- (a) Solid rock foundation with no fault or fissures, any type of dam can be constructed.
- Rocks like granite, gneiss and schist provide good foundation for a gravity dam.
- Poor rock or gravel foundation suggests choice of an earthen dam, rock fill dam or low concrete gravity dam.
- Silt and fine sand foundation pose the problem of seepage settlement etc. Hence such foundations are suitable only for earth dam or low concrete gravity dam. They are not suitable for rock fill dam.

Factors Governing the selection of Type of Dam

(3)Availability of material for Construction

- The Construction materials must be available locally o near the dam site in order to achieve economy in dam construction. The local availability of sand, gravel, crushed stone suggests concrete gravity dam. However, if coarse and fine grained soils are available locally, an earth dam may be suitable.

(4) Spillway Size and Location

- Spillway is required for safe disposal of of flood water. When separate site of spillway is available earthen dam may be preferred. In case of large capacity spillway, an overflow concrete gravity dam having overflow section in the middle will be the best choice.

Factors Governing the selection of Type of Dam

(5) Roadway

- The provision of roadway at the top of dam requires the choice of earth dam or gravity dam.

Loads/Forces on Dams

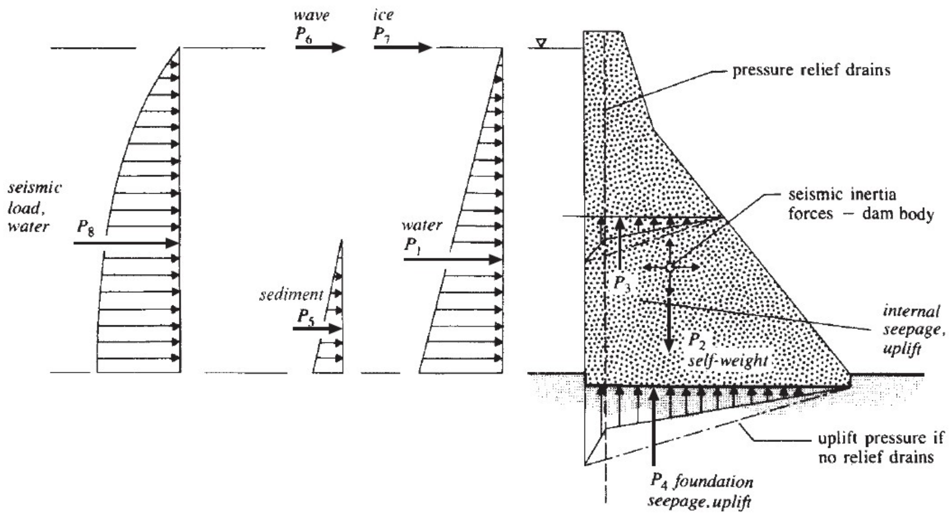
- Loads can be classified in terms of applicability/relative importance as ***primary loads***, ***secondary loads***, and ***exceptional loads***.

1. *Primary loads* are identified as universally applicable and of prime importance to all dams, irrespective of type, e.g. water and related seepage loads, and self-weight loads.

2. *Secondary loads* are generally discretionary and of lesser magnitude (e.g. sediment load) or, alternatively, are of major importance only to certain types of dams (e.g. thermal effects within concrete dams).

3. *Exceptional loads* are so designated on the basis of limited general applicability or having a low probability of occurrence (e.g. tectonic effects, or the inertia loads associated with seismic activity).

Loading Diagram on Gravity Dams



(a) Primary loads

1. *Water load.* This is a hydrostatic distribution of pressure with horizontal resultant force *P*1. (Note that a vertical component of load will also exist in the case of an upstream face batter, and that equivalent tailwater loads may operate on the downstream face.)

2. *Self-weight load.* This is determined with respect to an appropriate unit weight for the material. For simple elastic analysis the resultant, *P*2, is considered to operate through the centroid of the section.

3. *Seepage loads.* Equilibrium seepage patterns will develop within and under a dam, e.g. in pores and discontinuities, with resultant vertical loads identified as internal and external uplift, *P*3 and *P*4, respectively.

(b) Secondary loads

1. *Sediment load.* Accumulated silt etc. generates a horizontal thrust, considered as an equivalent additional hydrostatic load with horizontal resultant P_5.

2. *Hydrodynamic wave load.* This is a transient and random local load, P_6, generated by wave action against the dam (not normally significant).

3. *Ice load.* Ice thrust, P_7, from thermal effects and wind drag, may develop in more extreme climatic conditions (not normally significant).

4. *Thermal load (concrete dams).* This is an internal load generated by temperature differentials associated with changes in ambient conditions and with cement hydration and cooling (not shown).

(b) Secondary loads

6. *Abutment hydrostatic load.* This is an internal seepage load in the abutment rock mass, not illustrated. (It is of particular concern to arch or cupola dams.)

(c) Exceptional loads

1. *Seismic load.* Oscillatory horizontal and vertical inertia loads are generated with respect to the dam and the retained water by seismic disturbance. For the dam they are shown symbolically to act through the section centroid. For the water inertia forces the equivalent static thrust, P_8, is shown.

2. *Tectonic effects*. Disturbance following deep excavation in rock, may generate loading as a result of slow tectonic movements.

Load Combinations

A concrete dam should be designed with regard to the ***most rigorous adverse groupings*** or ***combinations of loads***, which have a ***reasonable probability of simultaneous occurrence***.

–***Three nominated load combinations*** are sufficient for almost all circumstances.

–In ascending order of severity they may be designated as ***normal, unusual, and extreme load combinations***, denoted as ***NLC, ULC and ELC, respectively***

Load combination A *(construction condition or empty reservoir condition):* Dam completed but no water in the reservoir and no tail water

Load combination B *(Normal operating condition):* Full reservoir elevation (or top of gates at crest), normal dry weather tail water, normal uplift, ice and uplift (if applicable)

Load combination C *(Flood Discharge condition):* Reservoir at maximum flood pool elevation, all gates open, tail water at flood elevation, normal uplift, and silt (if applicable)

Load combination D - Combination A, with earthquake

Load combination E - Combination B, with earthquake but no ice

Load Combination F - Combination C, but with extreme uplift (drain inoperative)

Load Combination G - Combination E, but with extreme uplift (drain inoperative)

. Forces Acting on Gravity Dam

The various external forces acting on a gravity dam may be :

(1) Water Pressure

(2) Uplift Pressure

(3) Pressure due to earthquake forces

(4) Silt Pressure

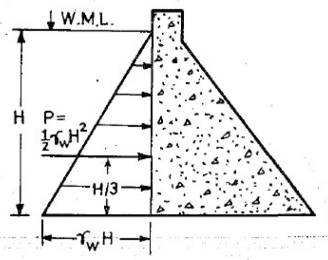
(5) Wave Pressure

(6) Ice Pressure

(7) The stabilising force is the weight of the dam itself.

An estimation and description of these forces is given below :

(1) **Water Pressure.** Water pressure (P) is the most major external force acting on such a dam. The horizontal water pressure, exerted by the weight of the water stored on the upstream side on the dam can be estimated from rule of hydrostatic pressure distribution ; which is triangular in shape, as shown in Fig. 19.2 (a) and (b). When the upstream face is vertical, the intensity is zero at the water surface and equal to $\gamma_w H$ at the base ; where γ_w is the unit weight of water and H is the depth of water : as shown in Fig. 19.2 (a). The resultant force due to this external water $= \frac{1}{2}\gamma_w H^2$, acting at $H/3$ from base.



Where γ_w = unit weight of water
9.81 kN/m^3 = 1000 kgf/m^3
Fig. 19.2. (a)

When the upstream face is partly vertical and partly inclined [Fig. 19.2 (b)], the resulting water force can be resolved into horizontal component (P_h) and vertical component (P_v). The horizontal component $P_h = \frac{1}{2}\gamma_w H^2$ acts at $\frac{H}{3}$ from the base ; and the vertical component (P_v) is equal to the weight of the water stored in column $ABCA$ and acts at the c.g. of the area.

Similarly, if there is tail water on the downstream side, it will have horizontal and vertical components, as shown in Fig. 19.2. (b).

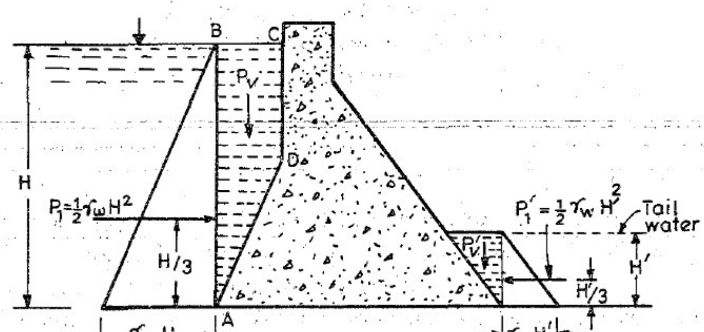


Fig. 19.2 (b)

(2) **Uplift Pressure.** Water seeping through the pores, cracks and fissures of the foundation material, and water seeping through dam body and then to the bottom through the joints between the body of the dam and its foundation at the base ; exert an uplift pressure on the base of the dam. It is the second major external force and must be accounted for in all calculations. Such an uplift force virtually reduces the downward weight of the body of the dam and hence, acts against the dam stability.

The amount of uplift is a matter of research and the present recommendations which are followed, are those suggested by United States Bureau of Reclamation (U.S.B.R.). According to these recommendations, the uplift pressure intensities at the heel and the toe should be taken equal to their respective hydrostatic pressures and joined by a straight line in between, as shown in Fig. 19.3 (*a*). When drainage galleries are provided to relieve the uplift, the recommended uplift at the face of the gallery is equal to the hydrostatic pressure at toe ($\gamma_w \cdot H'$) plus $\frac{1}{3}$rd the difference of the hydrostatic pressures at the heel and the toe ; as shown in Fig. 19.3 (*b*) ; *i.e.* $\left[\gamma_w \cdot H' + \frac{1}{3}(\gamma_w \cdot H - \gamma_w \cdot H')\right]$. It is also assumed that the uplift pressures are not affected by the earthquake forces.

The uplift pressures can be controlled by constructing cut-off walls under the upstream face, by constructing drainage channels between the dam and its foundation, and by pressure grouting the foundation.

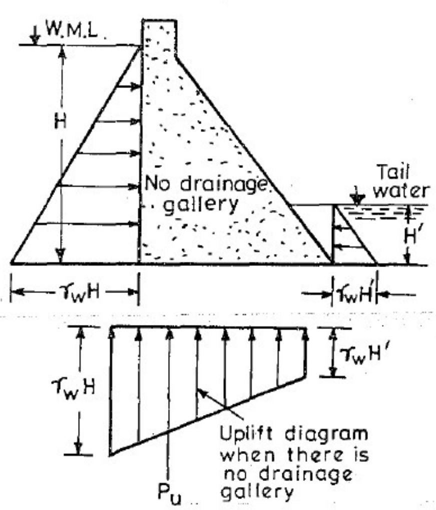


Fig. 19.3 (*a*) Uplift pressure (*U*) diagram, when no drainage gallery is provided.

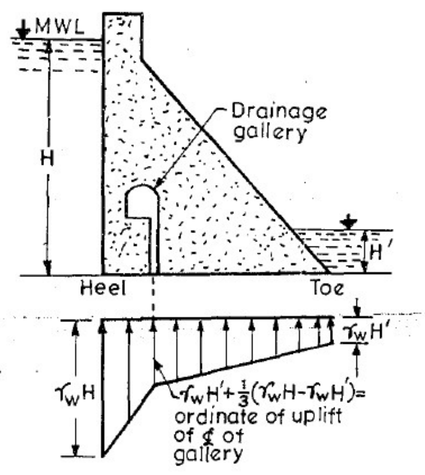


Fig. 19.3 (*b*) Uplift pressure (*U*) diagram, when drainage gallery is provided.

(3) **Earthquake Forces.** If the dam to be designed, is to be located in a region which is susceptible to earthquakes, allowance must be made for the stresses generated by the earthquakes.

An earthquake produces waves which are capable of shaking the Earth upon which the dam is resting, in every possible direction.

The effect of an earthquake is, therefore, equivalent to imparting an acceleration to the foundations of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction, and for design purposes, it has to be resolved in vertical and horizontal components. Hence, two accelerations, *i.e.* one horizontal acceleration (α_h) and one vertical acceleration (α_v) are induced by an earthquake. The values of these accelerations are generally expressed as percentage of the acceleration due to gravity (g), *i.e.* $\alpha = 0.1\,g$ or $0.2\,g$, etc.

On an average, a value of α equal to 0.1 to 0.15 g is generally sufficient for high dams in seismic zones.

In areas of no earthquakes or very less earthquakes, these forces may be neglected. In extremely seismic regions and in conservative designs, even a value upto 0.3 g may sometimes be adopted.

Effect of vertical acceleration (α_v). A vertical acceleration may either act downward or upward. When it is acting in the upward direction, then the foundation of the dam will be lifted upward and becomes closer to the body of the dam, and thus the effective weight of the dam will increase and hence, the stress developed will increase.

When the vertical acceleration is acting downward, the foundation shall try to move downward away from the dam body ; thus reducing the effective weight and the stability of the dam, and hence is the worst case for designs.

Such acceleration will, therefore, exert an **inertia force** given by

$$\frac{W}{g}\,\alpha_v \text{ (i.e. force = Mass × Acceleration)}$$

where W is the total weight of the dam.

$\therefore$ The net effective weight of the dam $= W - \frac{W}{g}\cdot\alpha_v$.

If $\qquad \alpha_v = k_v \cdot g$

[where k_v is the fraction of gravity adopted for vertical acceleration, such as 0.1 or 0.2, etc.].

Then, the net effective weight of the dam

$$= W - \frac{W}{g} \cdot k_v \cdot g = W[1 - k_v].$$

In other words, vertical acceleration reduces the unit weight of the dam material and that of water to $(1 - k_v)$ times their original unit weights.

Effects of horizontal acceleration (α_h). Horizontal acceleration may cause the following two forces :

(*i*) Hydrodynamic pressure ; and
(*ii*) Horizontal inertia force.

Both these forces are discussed below :

(*i*) *Hydrodynamic pressures.* Horizontal acceleration acting towards the reservoir causes a momentary increase in the water pressure, as the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as *hydrodynamic pressure*.

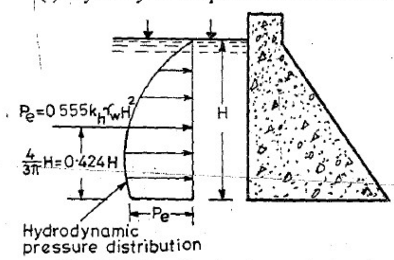


Fig. 19.4. Showing development of Hydrodynamic pressure by a horizontal earthquake moving towards the reservoir. A similar pressure will be developed on d/s tail water when the earthquake is reversed.

According to **Von-Karman**, the amount of this hydrodynamic force (P_e) is given by.

$$P_e = 0.555 \cdot k_h \gamma_w \cdot H^2 \quad ...(19.1)$$

and it acts at the height of $\frac{4H}{3\pi}$ above the base, as shown in Fig. 19.4.

where k_h is the fraction of gravity adopted for horizontal acceleration, such as 0.1, 0.2 etc.

γ_w = unit wt. of water

Moment of this force about base

$$= M_e = P_e\left(\frac{4H}{3\pi}\right) = \mathbf{0.424\, P_e \cdot H} \quad ...(19.2)$$

Zanger has given certain big formulas for evaluating the amount of this force and its position, etc. on the vertical as well as on an inclined faces. The results of these big formulas are quite comparable to those given by Von-Karman equation and hence, for average ordinary purposes, the Von-Karman equation (19.1) is sufficient.

Zanger's formula for hydrodynamic force. According to Zanger

$$P_e = 0.726\, p_e \cdot H \quad ...(19.3)$$

$$\text{where } p_e = C_m\, k_h \cdot \gamma_w \cdot H \quad ...(19.4)$$

$$\therefore \quad P_e = 0.726\, C_m \cdot k_h \cdot \gamma_w \cdot H^2 \quad ...(19.5)$$

where C_m = Maximum value of pressure co-efficient for a given constant slope

$$= 0.735\left(\frac{\theta^\circ}{90}\right); \quad ...(19.6)$$

where θ is the angle in degrees, which the u/s face of the dam makes with the horizontal.

k_h = fraction of gravity adopted for horizontal acceleration (α_h) such as $\alpha_h = k_h \cdot g$

γ_w = unit w.t. of water

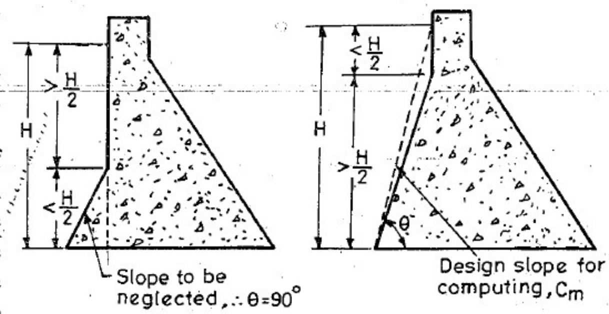
The moment of this force about the base is given as :

$$M_e = 0.299\, p_e \cdot H^2 \quad \text{...(19.7)}$$

$$= 0.299 \frac{P_e}{0.726 \cdot H} \cdot H$$

or $$M_e = 0.412\, P_e \cdot H \quad \text{...(19.8)}$$

It was further stated, that if the upstream face is partly inclined (Fig. 19.5 *a*), which does not extend to more than half the depth of the reservoir, it can be taken as vertical. If the slope extends to more than half the depth (Fig. 19.5 *b*), the overall slope up to the whole height may be taken as the value of θ in equation (19.6) above.



(*a*) (*b*)

Fig. 19.5

(*ii*) *Horizontal Inertia Force.* In addition to exerting the hydrodynamic pressure, the horizontal acceleration produces an inertia force into the body of the dam. This force is generated in order to keep the body and the foundation of the dam together as one piece. The direction of the produced force will be opposite to the acceleration imparted by the earthquake.

Since an earthquake may impart either upstream or downstream acceleration, we have to choose the direction of this force in our stability analysis of dam structure, in such a way that it produces most unfavorable effects under the considered conditions.

Say for example, when the reservoir is *full*, this force would produce worst results if it additive to the hydrostatic water pressure, thus acting towards the *downstream* (*i.e.* when upstream earthquake acceleration towards the reservoir is produced). When the reservoir is *empty*, this force would produce worst results, if considered to be acting *upstream* (*i.e.* when earthquake acceleration, moving towards downstream, is produced).

Under reservoir empty conditions, earthquake forces produce effects, which may cause slight tension near the toe ; and hence stability analysis for reservoir empty case may be carried out only on the basis of wt. of the dam by ignoring earthquake forces and keeping the section free from any tension. However, for all precise designs, these forces must be fully considered, as we have done in example 19.2.

The amount of this horizontal inertial force is equal to the product of the mass of the dam and the acceleration.

∴ This horizontal Inertia force

$$= \left(\frac{W}{g}\right)\alpha_h = \frac{W}{g} \cdot k_h \cdot g = W \cdot k_h \quad \text{...(19.9)}$$

(where k_h is the fraction of gravity adopted for horizontal acceleration, such as 0.1, or 0.2, etc.).

This force should be considered to be acting at the centre of gravity of the mass, regardless of the shape of the cross- section, and it acts horizontally downstream in worst cases, for reservoir full case.

(4) **Silt Pressure.** It has been explained under 'Reservoir Sedimentation' in chapter 18 that silt gets deposited against the upstream face of the dam. If h is the height of silt deposited, then the force exerted by this silt in addition to external water pressure, can be represented by Rankine's formula as :

$$P_{silt} = \frac{1}{2} \cdot \gamma_{sub} \cdot h^2 K_a \text{ and it acts at } \frac{h}{3.} \text{ from base} \quad ...(19.10)$$

where K_a is the coefficient of active earth pressure of silt $= \frac{1 - \sin\phi}{1 + \sin\phi}$ where ϕ is the angle of internal friction of soil, and cohesion is neglected.

γ_{sub} = submerged unit weight of silt material.

h = height of silt deposited.

If the upstream face is inclined, the vertical weight of the silt supported on the slope also acts as a vertical force.

In the absence of any reliable data for the type of silt that is going to be deposited, U.S.B.R. recommendations may be adopted. In these recommendations, deposited silt may be taken as equivalent to a fluid exerting a force with a unit wt. equal to 3.6 kN/m^3 in the horizontal direction and a vertical force with a unit wt. of 9.2 kN/m^3. Hence, the total horizontal force will be $3.6 \frac{h^2}{2} = 1.8\, h^2$ kN/m run, and vertical force will be $9.2 \cdot \frac{h^2}{2} = 4.6\, h^2$ kN/m run.

In most of the gravity-dam designs, the silt pressure is neglected. The basis for neglecting this force is that :

Initially, the silt load is not present, and by the time it becomes significant, it gets consolidated to some extent and, therefore, acts less like a fluid. Moreover, silt deposited in the reservoir is somewhat impervious and, therefore, will help to minimise the uplift under the dam.

(5) **Wave Pressure.** Waves are generated on the surface of the reservoir by the blowing winds, which causes a pressure towards the downstream side. Wave pressure depends upon the wave height. Wave height may be given by the equation,

$$h_w = 0.032\sqrt{V \cdot F} + 0.763 - 0.271\,(F)^{1/4} \textbf{ for F < 32 km,} \text{ and} \quad ...(19.11)$$

$$h_w = 0.032\sqrt{V \cdot F} \textbf{ for F > 32 km} \quad ...(19.12)$$

where h_w = height of water from top of crest to bottom of trough in metres.

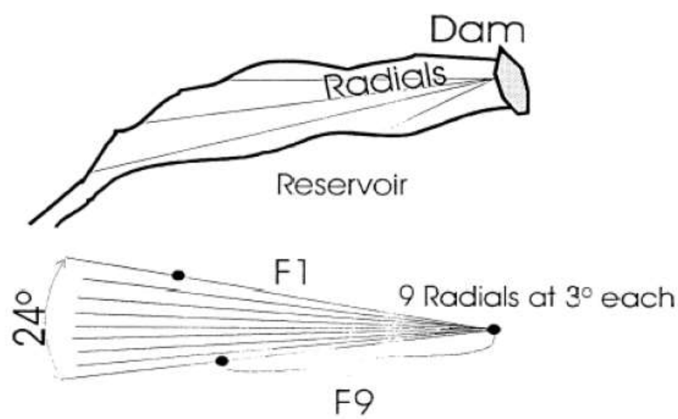
V = wind velocity in km/hr.

F = Fetch or straight length of water expanse in km.

The maximum pressure intensity due to wave action may by given by

$$p_w = 2.4\, \gamma_w \cdot h_w \text{ and acts at } \frac{h_w}{8} \text{ metres above the still water surface.}$$

The pressure distribution may be assumed to be triangular, of height $\frac{5h_w}{3}$, as shown in Fig. 19.6.



Fetch length (*fetch – continuous area of water over which the wind blows in a constant direction)*

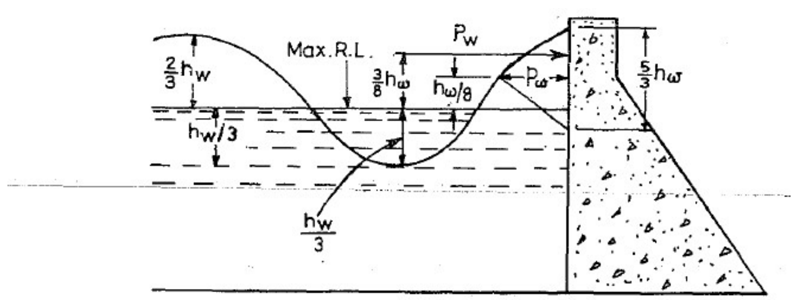


Fig. 19.6

Hence, the total force due to wave action (P_w)

$$= \frac{1}{2}(2.4\,\gamma_w \cdot h_w) \cdot \frac{5}{3} \cdot h_w$$

or $$P_w = 2 \cdot \gamma_w \cdot h_w^2 = 2 \times 9.81\, h_w^2 \text{ kN/m}$$
$$= 19.62\, h_w^2 \text{ kN/m} \qquad \ldots(19.14)$$

This force acts at a distance $\frac{3}{8}h_w$ above the reservoir surface.

(6) **Ice Pressure.** The ice which may be formed on the water surface of the reservoir in cold countries, may sometimes melt and expand. The dam face has then to resist the thrust exerted by the expanding ice. This force acts linearly along the length of the dam and at the reservoir level. The magnitude of this force varies from 250 to 1500 kN/m^2 depending upon temperature variations. On an average, a value of 500 kN/m^2 may be allowed under ordinary conditions.

(7) **Weight of the Dam.** The weight of the dam body and its foundation is the major resisting force. In two dimensional analysis of a gravity dam, a unit length of the dam is considered. The cross-section can then be divided into rectangles and triangles. The weight of each along with their *c.gs.*, can be determined. The resultant of all these downward forces will represent the total weight of the dam acting at the *e.g.* of the dam.

Combination of forces for Designs. The design of a gravity dam should be checked for two cases, *i.e.* (*i*) when Reservoir is full ; and (*ii*) when Reservoir is empty.

(*i*) **Case I.** Reservoir full case :

When reservoir is full, the major forces acting are : weight of the dam, external water pressure, uplift pressure, and earthquake forces in serious seismic zones. The minor forces are : silt pressure, ice pressure and wave pressure. For the most conservative designs, and from purely theoretical point of view, one can say that a situation may arise when all the forces may act together. But such a situation will never arise and hence, all the forces are not generally taken together. U.S.B.R. has classified the 'normal load combinations' and 'extreme load combination, as given below :

(*a*) *Normal Load Combinations*

(*i*) Water pressure upto normal pool level, normal uplift, silt pressure and ice pressure. This class of loading is taken when ice force is serious.

(*ii*) Water pressure upto normal pool level, normal uplift, earthquake forces, and silt pressure.

(*iii*) Water pressure upto maximum reservoir level (maximum pool level), normal uplift, and silt pressure.

(*b*) *Extreme Load Combinations*

(*i*) Water pressure due to maximum pool level, extreme uplift pressure without any reduction due to drainage and silt pressure.

Case II. Reservoir empty case :

(*i*) Empty reservoir without earthquake forces to be computed for determining bending diagrams, etc. for reinforcement design, for grouting studies or other purposes.

(*ii*) Empty reservoir with a horizontal earthquake force produced towards the upstream has to be checked for non- development of tension at toe.

19.4. Modes of Failure and Criteria for Structural Stability of Gravity Dams

A gravity dam may fail in the following ways :

(1) By overturning (or rotation) about the toe.

(2) By crushing.

(3) By development of tension, causing ultimate failure by crushing.

(4) By shear failure called sliding.

The failure may occur at the foundation plane (*i.e.* at the base of the dam) or at any other plane at higher level.

(1) **Over-turning.** If the resultant of all the forces acting on a dam at any of its sections, passes outside the toe, the dam shall rotate and overturn about the toe. Practically, such a condition shall not arise, as the dam will fail much earlier by compression. The ratio of the righting moments about toe (anti clockwise) to the over turning moments about toe (clock-wise) is called the factor of safety against overturning. Its value, generally > 1.25 may be acceptable, but ≥ 1.5 is desirable.

(2) **Compression or crushing.** A dam may fail by the failure of its materials, *i.e.* the compressive stresses produced may exceed the allowable stresses, and the dam-material may get crushed. The vertical direct stress distribution at the base is given by the equation :

$$p = \text{Direct stress} + \text{Bending stress}.$$

$$\therefore \quad p_{\frac{max}{min}} = \frac{\Sigma V}{B} \pm \frac{M}{I} y = \frac{\Sigma V}{B} \pm \frac{\Sigma V \cdot e}{B^2/6} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

or

$$p_{\frac{max}{min}} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \quad ...(19.15)$$

where e = Eccentricity of the resultant force from the centre of the base.

ΣV = Total vertical force.

B = Base width.

Note. Resultant is nearer the toe and hence, maximum compressive stress is produced at the toe (Reservoir full case)

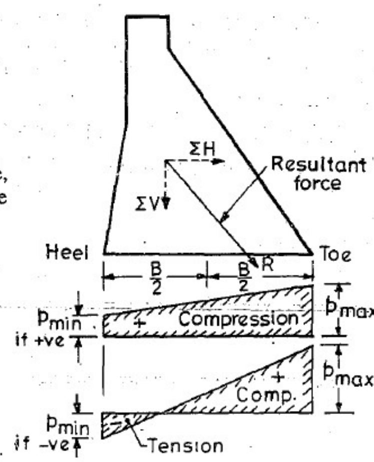


Fig. 19.7. (*a*) Vertical Stress Distribution for Reservoir Full case.

The maximum stress, *i.e.* p_{max}, will be produced on the end which is nearer to the resultant, as shown in Fig. 19.7 (*a*) and (*b*).

Note. The resultant is nearer the heel and hence, maximum compressive stress (+ve stress) is produced at the heel (Reservoir empty with horizontal earthquake wave moving away from reservoir-case).

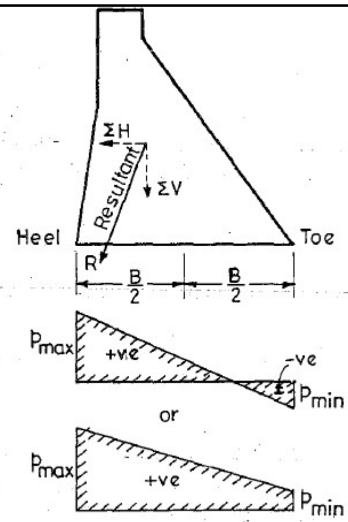


Fig. 19.7. (*b*) Vertical Stress Distribution for Reservoir Empty case.

If p_{min} comes out to be negative, it means that tension shall be produced at the appropriate end.

If p_{max} exceeds the allowable compressive stress of dam material [generally taken as 3000 kN/m^2 (30 kg/cm^2) for concrete]; the dam may crush and fail by crushing.

(3) **Tension.** Masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere, because these materials cannot withstand sustained tensile stresses. If subjected to such stresses, these materials may finally crack. However, for achieving economy in designs of very high gravity dams, certain amount of tension may be permitted under severest loading condition. This may be permitted because of the fact that such worst loading conditions shall occur only momentarily for a little time and would neither last long nor occur frequently. The maximum permissible tensile stress for high concrete gravity dams, under worst leadings, may be taken as 500 kN/m^2 (5 kg/cm^2).

Effect produced by tension cracks. In a dam, when such a tension crack develops, say at the heel, crack width (or strictly speaking crack-area) looses contact with the bottom foundations, and thus, becomes ineffective.

Hence, the effective width B (considering unit length) of the dam base will be reduced. This will increase p_{max} at the toe.

Moreover, the uplift pressure diagram gets modified due to crack formation, as shown in Fig. 19.8, resulting in an increase in the uplift. Since the uplift increases and the net effective downward force reduces, the resultant will shift more towards the toe and thus further increasing the compressive stress at the toe and further lengthening the crack due to further tension development. The process continues ; the effective base width goes on reducing and compressive stress at the toe goes on increasing ; finally leading to the failure of the toe by direct compression. *Hence, a tension crack by itself does not fail the structure, but it leads to the failure of the structure by producing excessive compressive stresses.*

ABC = old uplift diagram
A'B'C' = New uplift diagram after the crack AA', has devloped.

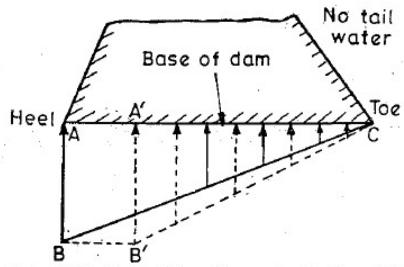


Fig. 19.8

In order to ensure that no tension is developed anywhere, we must ensure that p_{min} is at the most equal to zero.

Since $$p_{\substack{max\\min}} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \quad ...(19.15)$$

$$p_{min} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right]$$

If $$p_{min} = 0,$$

$$\frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right] = 0$$

or $$1 - \frac{6e}{B} = 0$$

or $$e = \frac{B}{6}.$$

Hence, maximum value of eccentricity that can be permitted on either side of the centre is equal to $\frac{B}{6}$; which leads to the famous statement : *the resultant must lie within the middle third.*

(4) **Sliding.** Sliding (or shear failure) will occur when the net horizontal force above any plane in the dam or at the base of the dame exceeds the frictional resistance developed at that level.

The friction developed between two surfaces is equal to $\mu\Sigma V$., (Fig. 19.9) where ΣV is the algebraic sum of all the vertical forces whether upward or downward, and μ is the coefficient of friction between the two surfaces. In order that no sliding takes place, the external horizontal forces (ΣH) must be less than the shear resistance $\mu \cdot \Sigma V$.

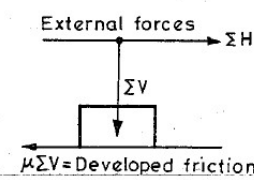


Fig. 19.9

or $$\Sigma H < \mu\Sigma V.$$

or $$\frac{\mu\Sigma V}{\Sigma H} > 1$$

$\frac{\mu \cdot \Sigma V}{\Sigma H}$ represents nothing but the factor of safety against sliding, which must be greater than unity.

$\therefore$ F.S.S. (Factor of safety against sliding) = $\frac{\mu \cdot \Sigma V}{\Sigma H}$.

In low dams, the safety against sliding should be checked only for friction, but in high dams, for economical precise designs, the shear strength of the joint, which is an additional shear resistance, must also be considered. If this shear resistance of the joint is also considered, then the equation for factor of safety against sliding which is measured by shear friction factor (S.F.F.) becomes

$$\text{S.F.F.} = \frac{\mu\Sigma V + B \cdot q}{\Sigma H} \quad ...(19.16)$$

where B = width of the dam at the joint,

q = Average shear strength of the joint which varies from about 1400 kN/m^2 (14 kg/cm^2) for poor rocks to about 4000 kN/m^2 (40 kg/cm^2) for good rocks.

The value of μ generally varies from 0.65 to 0.75.

Attempts are always made to increase this shear strength (q) at the base and at other joints. For this purpose, foundation is stepped at the base, as shown in Fig. 19.10 and measures are taken to ensure a better bond between the dam base and the rock-foundation.

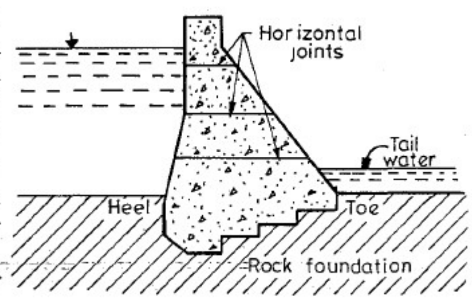


Fig. 19.10

During the construction of a dam, horizontal joints have to be left as shown in Fig. 19.10. The shear strength of these joints should be made as good as possible by ensuring better bond between the two surfaces. For this purpose, the lower surface must be thoroughly cleaned and a layer of neat cement or rich cement mortar should be spread before pouring the standard concrete mix for the upper layer. If these precautions of quality control are not adhered to in the filed, the assumption made in accounting for this shear strength in the design, will not be justified. That is why, for small dams, where quality control is less, this shear strength of the joint is not taken into account at all, while determining the shear friction factor or factor of safety against sliding.

19.4.1. Principal and Shear Stresses. The vertical stress intensity, p_{max} or p_{min} determined from the equation (19.15) is not the maximum direct stress produced anywhere in the dam. The maximum normal stress will, in fact, be the major principal stress that will be generated on the major principal plane. When the reservoir is full, the vertical direct stress [given by equation (19.15), and represented by p_v in future] is maximum at the toe as the resultant is nearer to the toe. To study the principal stresses that will develop near the toe, let us consider a small element ABC [See Fig. 19.11 (*a*) and (*b*)] near the toe of

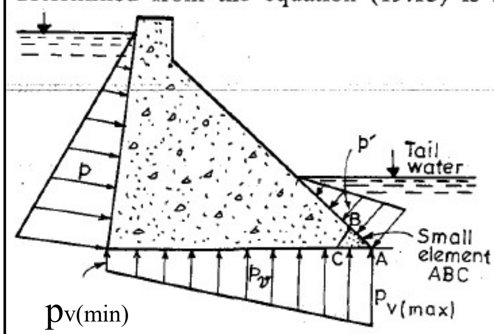


Fig. 19.11 (*a*)

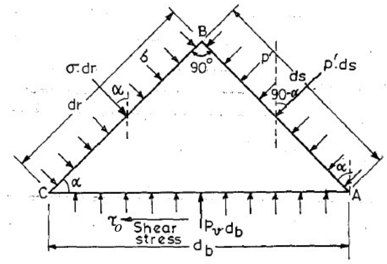


Fig. 19.11. (*b*) Enlarged view of small element *ABC* of Fig. 19.11 (*a*).

the dam. The element is so small that the stress intensities may be assumed to be uniform on its faces.

Let the downstream face of the dam be inclined at an angle α to the vertical.

This face of the dam will act as a principal plane because the water pressure p' acts at right angles to the face, and also there is no shear stress acting on this plane. Since the principal planes are at right angles to each other ; the plane BC drawn at right angles to the face AB will be the second principal plane. Let the stress acting on this plane be σ.

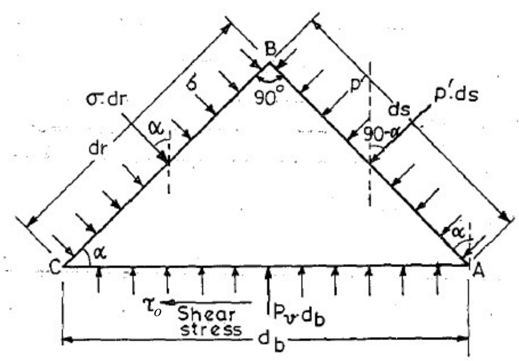


Fig. 19.11. (*b*) Enlarged view of small element *ABC* of Fig. 19.11 (*a*).

Let ds, dr and db be the lengths of AB, BC and CA respectively. p' is the intensity of water pressure on face AB and p_v is the intensity of vertical pressure on face AC, and σ is the intensity of normal stress (principal stress) on face BC. Considering unit length of the dam, the forces acting on the faces AB, BC and CA are $p'\,ds$, $\sigma\,dr$ and $p_v \cdot db$ respectively.

Resolving all the forces in the vertical direction, we get

$$p' \cdot ds \cdot \sin\alpha + \sigma \cdot dr \cdot \cos\alpha = p_v \cdot db,$$

Now $\dfrac{ds}{db} = \sin\alpha, \quad \text{or } ds = db \cdot \sin\alpha.$

$$\frac{dr}{db} = \cos\alpha, \quad \text{or } dr = db \cdot \cos\alpha.$$

$\therefore \quad p' \cdot (db \cdot \sin\alpha) \cdot \sin\alpha + \sigma \cdot (db \cdot \cos\alpha)\cos\alpha = p_v \cdot db$

or $\quad p' \cdot \sin^2 a + \sigma \cdot \cos^2\alpha = p_v.$

or $\quad \sigma = \dfrac{p_v - p' \cdot \sin^2\alpha}{\cos^2\alpha}$

or $\quad \boxed{\sigma = p_v \cdot \sec^2\alpha - p' \tan^2\alpha}$ ...(19.17)

For σ to be maximum, p' should be zero, *i.e.* when there is no tail water ; then in such a case

$$\sigma = p_v \cdot \sec^2\alpha \qquad \text{...[19.17 }(a)]$$

Since $\sec^2\alpha$ is always more than 1, it follows, that σ will be more than p_v. *This value of normal stress, which is the maximum produced anywhere in the body of the dam, must be calculated and should not be allowed to exceed the maximum allowable compressive stress of dam material.*

If the hydrodynamic pressure (p_e') exerted by the tail water during an earthquake moving towards the reservoir is also considered, then the net pressure on the face AB will be $(p' - p_e')$, because the effect of this earthquake will be to reduce the tail water pressure.

The principal stress (σ) can then be given by

or $\quad \boxed{\sigma_{at\ toe} = p_v \cdot \sec^2\alpha - (p' - p_e')\tan^2\alpha}$ ...(19.18)

The equation for σ, derived above for the element at the toe is also applicable to the element at the heel. The equation at the heel is, therefore, given as :

$$\sigma_1 = \sigma_{at\ heel} = p_v \cdot \sec^2\phi - (p + p_e)\tan^2\phi \qquad \text{...(19.19)}$$

where ϕ is the angle which the u/s face makes with vertical.

But at the heel, the pressure of water p is always more than σ, and hence, σ will be the minor principal stress at the heel.

Shear stress on the horizontal plane near the toe. A shear stress τ will act on the face CA on which the vertical stress is acting. Resolving all the forces [Fig. 19.11 (b)] in the horizontal direction, we get

$$\sigma \cdot dr \sin\alpha - p' \cdot ds \cdot \cos\alpha = \tau_0 \cdot db$$

or $\quad \sigma \cdot (db \cdot \cos\alpha)\sin\alpha - p'(db \cdot \sin\alpha)\cos\alpha = \tau_0 \cdot db$

or $\quad \sigma \cdot \sin\alpha\cos\alpha - p'\sin\alpha\cos\alpha = \tau_0$

or $\quad \tau_0 = (\sigma - p')\sin\alpha\cos\alpha$

Substituting the value of σ from equation (19.17), we get

$$\tau_0 = \left[\overline{p_v \sec^2\alpha - p' \tan^2\alpha} - p'\right]\sin\alpha\cos\alpha$$

$$\tau_0 = \left[p_v \sec^2\alpha - p'(1 + \tan^2\alpha)\right]\sin\alpha\cos\alpha = \left[(p_v - p')\sec^2\alpha\right]\sin\alpha\cos\alpha$$

or $= [(p_v - p') \sec^2 \alpha \cdot \sin \alpha \cdot \cos \alpha]$

or $\boxed{\tau = (p_v - p') \tan \alpha}$ ...(19.20)

Neglecting tail water, shear stress is given by

$$\tau_0 = p_v \cdot \tan \alpha \quad \text{...[19.20 (a)]}$$

If the effect of hydrodynamic pressure produced by an earthquake moving towards the reservoir, is also considered, the equation for shear stress on a horizontal plane near the toe becomes,

$$\tau_0 = \left[p_v - (p' - p_e')\right] \tan \alpha \quad \text{...(19.21)}$$

Similarly, shear stress at heel

$$\tau_{o(heel)} = -\left[p_v - (p + p_e)\right] \tan \phi$$

–ve sign shows that the direction is reversed.

Gravity Method or Two Dimensional Stability Analysis

The various assumptions made in the two dimensional designs of gravity dams are summarised below :

(i) The dam is considered to be composed of a number of cantilevers, each of which is 1 m thick and each of which acts independent of the other.

(ii) No loads are transferred to the abutments by beam action.

(iii) The foundation and the dam behave as a single unit ; the joint being perfect.

(iv) The materials in the foundation and body of the dam are isotropic and homogeneous.

(v) The stresses developed in the foundation and body of the dam are within elastic limits.

(vi) No movements of the foundations are caused due to transference of loads.

(vii) Small openings made in the body of the dam do not affect the general distribution of stresses and they only produce local effects as per St. Venant's principle.

Procedure: Two-dimensional analysis can be carried out analytically or graphically.

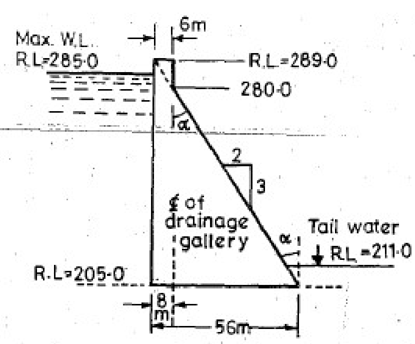
(a) Analytical Method: The stability of the dam can be analysed in the following steps:

(i) Consider unit length of the dam.

(ii) Work out the magnitude and directions of all the vertical forces acting on the dam and their algebraic sum, i.e. $\sum V$.

(iii) Similarly, work out all the horizontal forces and their algebraic sum, i.e. $\sum H$.
(iv) Determine the lever arm of all these forces about the toe.
(v) Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments, i.e. $\sum M$.
(vi) Find out the location of the resultant force by determining its distance from the toe.
(vii) Find out the eccentricity (e) of the resultant (R). It must be less than B/6 in order to ensure that no tension is developed anywhere in the dam.
(viii) Determine the vertical stresses at the toe and heel.
(ix) Determine the maximum normal stresses, i.e. principal stresses at the toe and the heel. They should not exceed the maximum allowable values. The crushing strength of concrete varies between 1500 to 3000 kN/m2 depending upon its grade M15 to M30.
(x) Determine the factor of safety against overturning.
(xi) Determine the factor of safety against sliding, using Sliding factor and Shear friction factor (S.F.F.). Sliding factor must be greater than unity and S.F.F. must be greater than 3 to 5. The analysis should be carried out for reservoir full case as well as for reservoir empty case.

Given figure shows the section of a gravity dam (non-overflow portion) built of concrete.



Calculate (neglecting earthquake effects)

(i) The maximum vertical stresses at the heel and toe of the dam.

(ii) The major principal stress at the toe of the dam.

(iii) The intensity of shear stress on a horizontal plane near the toe.

Assume weight of concrete = 23.5 kN/m^3; and unit length of dam. Allowable stress in concrete may be taken 2500 kN/m^2

Solution. Assuming $\gamma_w = 9.81$ kN/m^3; the various forces acting on the dam are drawn in Fig. 19.12 (*b*).

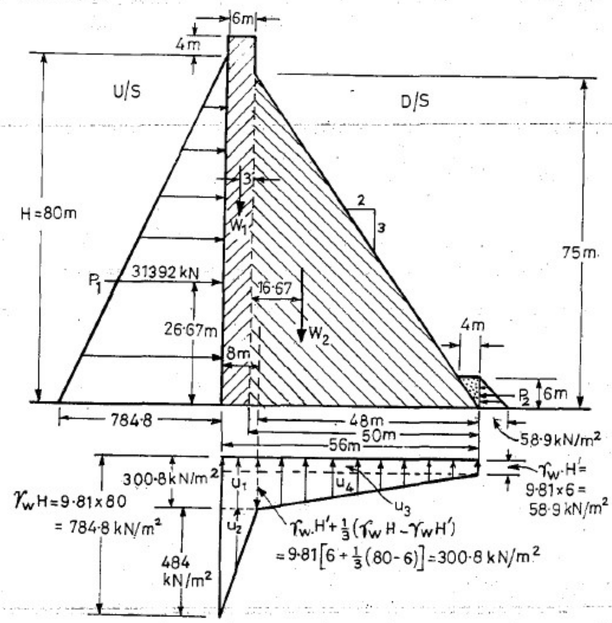


Fig. 19.12 (*b*)

Consider 1 m length of the dam.

The various forces and their moments about the toe are then calculated and tabulated in Table 19.1. From this table, we have

Distance of resultant from the toe

$$(\bar{x}) = \frac{\Sigma M}{\Sigma V}$$

$$= \frac{7{,}77{,}639 \text{ kN} \cdot m}{43050 \text{ kN}} = 18.06 \text{ m}$$

$$\text{Eccentricity} = e = \frac{56}{2} - 18.06 = 28 - 18.06 = 9.94 \text{ m}$$

Vertical stress p_v is given as :

$$p_v = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\therefore \quad p_v = \frac{43{,}050 \text{ kN}}{56 \text{ m}}\left[1 \pm \frac{6 \times 9.94}{56}\right] = 768.8\,(1 \pm 1.065)$$

Table 19.1

Name of the force	*Designation if given*	*Magnitude in kN*	*Lever arm in m*	*Moments about toe in kN.m*
Vertical forces				
Downward weight of the dam	W_1	(+) $84 \times 6 \times 1 \times 23.5 = 11{,}844$	53.0	(+) 6,27,732
	W_2	(+) $\frac{1}{2} \times 50 \times 75 \times 1 \times 23.5 = 44{,}063$	33.33	(+) 14,68,620
Weight of water supported on d/s face	—	(+) $\frac{1}{2} \times 4 \times 6 \times 1 \times 9.81 = 118$	1.33	(+) 157
		$\Sigma V_1 = 56{,}025$		$\Sigma M_1 =$ (+) 20,96,509
Uplift pressures	U_1	(–) $300.8 \times 8 \times 1 = 2406$	52.0	(–) 1,25,112
	U_2	(–) $\frac{1}{2} \times 484 \times 8 \times 1 = 1936$	53.33	(–) 1,03,247
	U_3	(–) $58.9 \times 48 \times 1 = 2827$	24.0	(–) 67,848
	U_4	(–) $\frac{1}{2} \times 241.9 \times 48 \times 1 = 5806$	32.0	(–) 1,85,792
		$\Sigma V_2 =$ (–) 12,975		$\Sigma M_2 =$ (–) 4,81,999
		$\Sigma V = 56{,}025 - 12{,}975 = 43050$		
Horizonal *Water pressure*				
On u/s face	P	$\frac{1}{2} \times 784.8 \times 80 \times 1 = 31{,}392$	26.67	(–) 8,37,225
On d/s face	P'	(–) $\frac{1}{2} \times 58.9 \times 6 =$ (–) 177	2.0	(+) 354
		ΣH (towards downstream) = 31,215		$\Sigma =$ (–) 8,36,871
$\Sigma M =$ Net (+) moment = 20,96,509 – 4,81,999 – 8,36,871 = 7,77,639 kN-m				

$\therefore$ Max. vertical stress $= p_{max}$ at toe $= 768.8 \times 2.065 = \mathbf{1587.6\ kN/m^2}$

$\therefore$ Min. vertical stress $= p_{min}$ at heel $= 768.8 \times (-)\ 0.065 = (-)\ \mathbf{49.97\ kN/m^2}$ **Ans.**

(*ii*) Major principal stress at toe (σ) is given by Eq. (19.17) as :

$$\sigma = p_{v\,(toe)} \sec^2 \alpha - p' \cdot \tan^2 \alpha$$

here $p_{v\,(toe)} = 1587.6\ \text{kN/m}^2$

$p' = 58.9\ \text{kN/m}^2$

$\tan \alpha = \dfrac{2}{3}$

$\sec \alpha^2 = 1 + \tan^2 \alpha = 1 + \dfrac{4}{9} = \dfrac{13}{9}$

$$\therefore \quad \sigma = 1587.6 \times \frac{13}{9} = 58.9 \times \frac{4}{9}$$

$$= \mathbf{2267\ kN/m^2 < 2500\ kN/m^2\ (OK)} \quad \textbf{Ans.}$$

(*iii*) Intensity of shear stress on a horizontal plane near toe is given by equation (19.20)

$$\therefore \quad \tau_0 = \left[p_{v(toe)} - p'\right] \tan \alpha$$

$$= 1587.6 - 58.9)\ \tfrac{2}{3} = \mathbf{1019.1\ kN/m^2}. \quad \textbf{Ans.}$$

19.6. Elementary Profile of a Gravity Dam

The elementary profile of a dam, subjected only to the external water pressure on the upstream side, will be a right-angled triangle, having zero width at the water level and a base width (B) at bottom *i.e.*, the point where the maximum hydrostatic water pressure acts. In other words, the shape of such a profile is similar to the shape of the hydrostatic pressure distribution (Fig. 19.14).

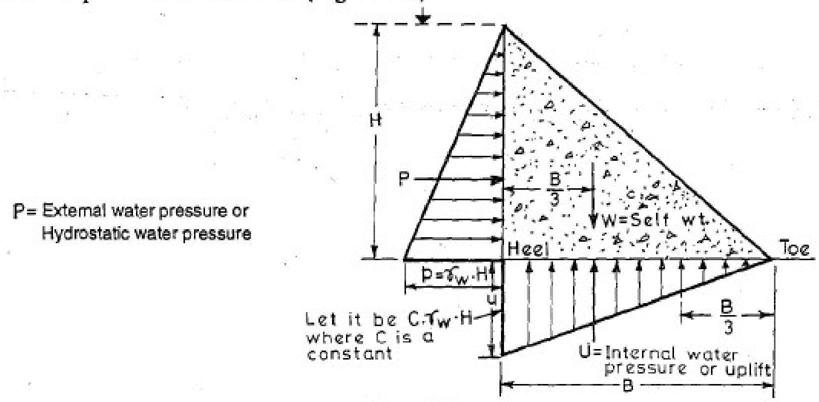


Fig. 19.14

When the reservoir is empty, the only single force acting on it is the self-weight (W) of the dam and it acts at a distance $B/3$ from the heel. This is the maximum possible innermost position of the resultant for no tension to develop. Hence, such a line of action of W is the most ideal, as it gives the maximum possible stabilising moment about the toe without causing tension at toe, when the reservoir is empty. *The vertical stress distribution at the base, when the reservoir is empty,* is given as :

$$P_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Here $\Sigma V = W$

$$e = \frac{B}{6}.$$

$$\therefore \qquad p_{max/min} = \frac{W}{B}\left[1 \pm \frac{6 \cdot B}{B \cdot 6}\right]$$

or $\qquad p_{max} = \dfrac{2W}{B}$

and $\qquad p_{min} = 0.$

Hence, the maximum vertical stress equal to $\frac{2W}{B}$ will act at the heel ($\because$ the resultant is nearer the heel) and the vertical stress at toe will be zero.

When the reservoir is full, the base width is governed by :

(*i*) The resultant of all the forces, *i.e.* P, W and U (Fig. 19.14) passes through the outer most middle third point (*i.e.* lower middle third point).

(*ii*) The dam is safe in sliding.

(*i*) For the 1st condition to be satisfied, we proceed as follows : Taking moments of all the forces (Fig. 19.14) about the lower middle third point (*i.e.* the point through which resultant is passing), we get

$$W\left(\frac{B}{3}\right) - U\left(\frac{B}{3}\right) - P\frac{H}{3} = R \times 0$$

or

$$(W - U)\frac{B}{3} - P\frac{H}{3} = 0$$

But

$$W = \frac{1}{2} \times B \times H \times 1 \times S_c \times \gamma_w$$

where S_c = Sp. gravity of concrete, *i.e.* that of the material of the dam.

γ_w = unit wt. of water = 9.81 kN/m^3

Let the uplift at the heel be $C \cdot \gamma_w \cdot H$., where C is a constant which according to U.S.B.R. recommendation is taken equal to 1.0 in calculation and will be equal to zero when no uplift is considered.

$$\therefore \quad U = \left(\frac{1}{2} C \cdot \gamma_w \cdot H\right) B$$

and

$$P = \frac{1}{2}\gamma_w \cdot H \cdot H = \frac{\gamma_w H^2}{2}$$

$\therefore$ Equation $(W - U)\frac{B}{3} - P\frac{H}{3} = 0$, becomes

$$\left[\frac{1}{2} B \cdot H \cdot S_c \cdot \gamma_w - \frac{1}{2} C \cdot \gamma_w \cdot H \cdot B\right]\frac{B}{3} - \frac{\gamma_w H^2}{2} \cdot \frac{H}{3} = 0$$

or

$$\frac{B}{3} \times \frac{1}{2} B \cdot H \cdot \gamma_w \cdot [S_c - C] = \frac{\gamma_w H^3}{6}$$

or

$$B^2 (S_c - C) = H^2$$

or

$$\boxed{B = \frac{H}{\sqrt{S_c - C}}} \qquad \text{...(19.22)}$$

Hence, if B is taken equal to or greater than $\frac{H}{\sqrt{S_c - C}}$, no tension will be developed at the heel with full reservoir,

when $\quad C = 1$

$$B = \frac{H}{\sqrt{S_c - 1}} \qquad \text{...[19.22 (a)]}$$

If uplift is not considered, $B = \frac{H}{\sqrt{S_c}}$ $\quad (\because C = 0)$ $\qquad$ ...[19.22 (*b*)] (*ii*) For the II condition (*i.e.* dam is safe in sliding) to be satisfied ; the frictional resistance $\mu\Sigma V$ or $\mu\,(W-U)$ should be equal to or more than the horizontal forces $\Sigma H = P$.

or $$\mu\,(W-U) \geq P$$

or $$\mu\left(\frac{1}{2}BH\cdot S_c\cdot\gamma_w - \frac{1}{2}C\cdot\gamma_w\cdot H\cdot B.\right) \geq \frac{\gamma_w H^2}{2}$$

or $$\mu\,(S_c - C)\,\frac{1}{2}\cdot B\cdot H\cdot\gamma_w \geq \frac{\gamma_w H^2}{2}$$

or $$\mu\,(S_c - C)\,B \geq H$$

or $$B \geq \frac{H}{\mu\,(S_c - C)}$$

Under limiting condition

or $$\boxed{B = \frac{H}{\mu\,(S_c - C)}} \qquad \text{...(19.23)}$$

If $C = 1$ $$B = \frac{H}{\mu\,(S_c - 1)} \qquad \text{...[19.23}(a)\text{]}$$

If $C = 0$, *i.e.* no uplift is considered, then

$$B \geq \frac{H}{\mu S_c} \qquad \text{...[19.23}(b)\text{]}$$

The value of B chosen should be greater of the two values given by Equations (19.22) and (19.23).

Using $S_c = 2.4$ and $\mu = 0.7$ and $C = 0$, we get

$$B\text{ (by Equation 19.22)} = \frac{H}{\sqrt{2.4-0}} = \frac{H}{\sqrt{2.4}}$$

$$B\text{ (by Equation 19.23)} = \frac{H}{0.7\times(2.4-0)} = \frac{H}{0.7\times 2.4} = \frac{H}{1.68}$$

But $\frac{H}{1.68}$ is less than $\frac{H}{\sqrt{2.4}}$

$\therefore$ For all practical purposes, the base width may be taken as $\frac{H}{\sqrt{S_c}}$

The vertical stress distribution when reservoir is full is given as :

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

where $\Sigma V = W - U$

$$= \left(\frac{1}{2}B\cdot H\cdot 1\cdot S_c\cdot\gamma_w - \frac{1}{2}C\cdot\gamma_w\cdot H\cdot B\right)$$

$$= \frac{1}{2}B\cdot\gamma_w\cdot H\cdot[S_c - C]$$

$$e = \frac{B}{6}$$

$$\therefore \quad p_{max/min} = \frac{\frac{1}{2} \cdot B \cdot \gamma_w \cdot H(S_c - C)}{B}\left[1 \pm \frac{6B}{6B}\right]$$

maximum stress will occur at toe, because the resultant is near the toe.

$$\therefore \quad p_{max} \text{ at toe} = \frac{1}{2}\gamma_w \cdot H(S_c - C)\, 2.0 = \gamma_w H(S_c - C)$$

$$\boxed{p_v \text{ at toe} = \gamma_w H(S_c - C)} \qquad \text{...(19.24)}$$

$$P_{\min} \text{ at heel} = 0.$$

The principal stress near the toe (σ) which is the maximum normal stress in the dam, is given by Equation (19.17)

$$\sigma = p_v \sec^2 \alpha - p' \tan^2 \alpha$$

when there is no tail water *i.e.*, $p' = 0$

$$\sigma = p_v \sec^2 \alpha$$

σ at toe, with full reservoir in elementary profile

$$= \gamma_w H(S_c - C) \sec^2 \alpha$$

$$= \gamma_w H(S_c - C)[1 + \tan^2 \alpha]$$

$$= \gamma_w H(S_c - C)\left[1 + \frac{B^2}{H^2}\right]$$

But $B = \frac{H}{\sqrt{S_c - C}}$ from Eq. (19.22)

$$\text{or} \quad \frac{B^2}{H^2} = \frac{1}{S_c - C}$$

$$\therefore \quad \sigma = \gamma_w H(S_c - C)\left[1 + \frac{1}{S_c - C}\right]$$

$$\text{or} \quad \boxed{\sigma = \gamma_w H(S_c - C + 1)} \qquad \text{...(19.25)}$$

when $C = 1$, $\quad S_c = 2.4$

$$\sigma = \gamma_w H\left(\frac{2.4 - 1 + 1}{2.4 - 1}\right) = 2.4\,\gamma_w H$$

The shear stress τ_0 at a horizontal plane near the toe is given by the equation (19.20) as :

$$\tau_0 = (p_v - p') \tan \alpha$$

If $\quad p' = 0$

$$\tau_0 = p_v \tan \alpha$$

But $p_v = \gamma_w H(S_c - C)$ $\qquad$ from Eq. (19.24)

$$\therefore \quad \tau_0 = \gamma_w H(S_c - C) \tan \alpha$$

$$\text{or} \quad \tau_0 = \gamma_w \cdot H(S_c - C)\frac{B}{H}$$

$$= \gamma_w H (S_c - C) \frac{1}{\sqrt{S_c - C}}$$

or $$\boxed{\tau_0 = \gamma_w H \sqrt{S_c - C}} \qquad ...(19.26)$$

19.7. High and Low Gravity Dams

The principal stress calculated for an elementary profile is given by Equation (19.25), *i.e.* $\sigma = \gamma_w H (S_c - C + 1)$. The value of principal stress calculated above varies only with H, as all other factors are fixed.

To avoid dam failure by crushing, the value of σ should be less than or at the most equal to the maximum allowable compressive stress of dam material. If f represents the allowable stress of the dam material, then the maximum height (H_{max}) which can be obtained in an elementary profile, without exceeding the allowable compressive stresses of the dam material, is given as :

$$f = \gamma_w H (S_c - C + 1)$$

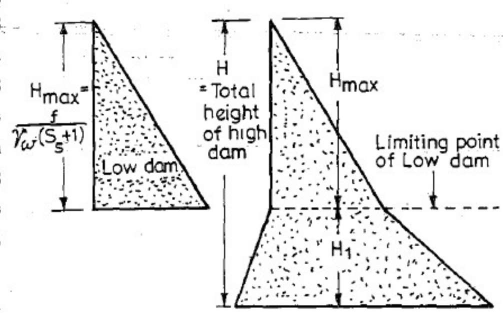
or $$H = \frac{f}{\gamma_w (S_c - C + 1)}$$

The lowest value of H will be obtained when $C = 0$, *i.e.* when uplift is neglected. Hence, for determining the limiting height and to be on a safer side, uplift is neglected.

H_{max}, *i.e.* maximum possible height is given as :

$$H_{max} = \frac{f}{\gamma_w (S_c + 1)} \qquad ...(19.27)$$

Hence, if the height of a dam having an elementary profile of a triangle, is more than that given by the Equation (19.27), the maximum compressive stress generated will exceed the allowable value. In order to keep it safe within limits, extra slopes on the upstream as well as on the downstream, below the limiting height will have to be given, as shown in Fig. 19.15.



Low gravity dam High gravity dam
Fig. 19.15

This limiting height (H_{max}) given by Equation (19.27), draws a dividing line between a low gravity dam and a high gravity dram, which are purely technical terms to differentiate between them.

Hence, a low gravity dam is the one whose height is less than that given by Equation (19.27). If the height of the dam is more than this, it is known as a high gravity dam.

The limiting height of a low concrete gravity dam, constructed in concrete having strength equal to 3000 kN/m^2 is thus given :-

$$H_{max} = \frac{f}{\gamma_w (S_c + 1)}$$

where $\gamma_w = 9.81\ kN/m^3$

$S_c = 2.4$

$f = 3000\ kN/m^2$

$$\therefore \quad H_{max} = \frac{f}{\gamma_w (S_c + 1)} = \frac{3000}{9.81\ (2.4 + 1)} = \mathbf{90\ m}$$

19.8. Profile of a Dam from Practical Considerations

The elementary profile of a gravity dam, (*i.e.* a triangle with maximum water surface at apex) is only a theoretical profile. Certain changes will have to be made in this profile in order to cater to the practical needs. These needs are : (*i*) providing a straight top width, for a road construction over the top of the dam ; (*ii*) providing a free-board above the top water surface, so that water may not spill over the top of the dam due to wave action, etc.

The additions of these two provisions, will cause the resultant force to shift towards the heel. The resultant force, when the reservoir is empty, was earlier passing through the inner middle third point. This will, therefore, shift more towards the heel, crossing the inner middle third point and consequently, tension will be developed at the toe. In order to avoid the development of this tension, some masonry or concrete will have to be added to the upstream side, as shown in Fig. 19.16, which shows the typical section along with the possible dimensions that can be adopted for a low gravity dam section. It should, however, be checked for stability analysis.

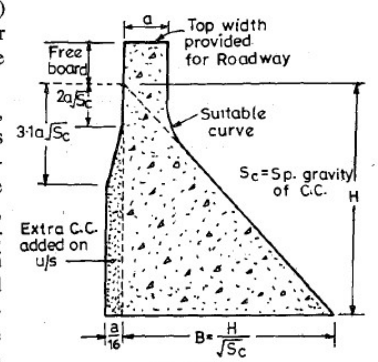


Fig. 19.16.
Typical section of a low gravity dam.

- **Top width** The top width of dam is generally dictate by the requirement of Roadway to be provided. The most economical top width is **14 % of the dam height** It is also taken to **0.55 $H^{1/2}$** is the maximum water depth. Usually the width varies from **6 to 10 m.**
- **Free Board:** The Upstream parapet of the roadway above the top of the dam is usually a solid wall but does not form a part of free board. The free board provide is maximum of (i) **1.33 h_w (≈1.5 h_w)**

  (ii) **3 to 4 %, usually 5 % of the dam height, whichever is more,**

Foundation Treatment for Gravity Dams

The material underlying the base of a dam, *i.e.* the foundations of the dam, must be strong enough and capable to withstand the foundation pressure exerted on it under various conditions of loading and in dry as well as wet condition. Most of the failures of the dams have occurred because of the failure of their underlying strata.

The foundation treatment commonly adopted for all foundations can be divided into two steps :

(i) Preparing the surface ; and

(ii) Grouting the foundation

(i) Preparing the surface

The surface preparation consists in removing the entire loose soil till a sound bed rock is exposed. The excavation should be carried out in such a way that the underlying rock is not damaged. The final surface obtained above is stepped, so as to increase the frictional resistance of the dam against sliding. The stepping of the foundation and provision of a shear key is shown in Figure below. The shear key may sometimes be provided in the centre but is generally provided at the heel.

If faults, seams, or shattered rock zones are detected in the exploratory geological investigations, special steps and remedies must be taken to ensure their removal. They may have to be entirely excavated and back-filled with concrete grouting. The treatment will depend upon the specific needs.

The top foundation surface is thoroughly cleaned with wet sand blasting and washing before the concreting for dam section is started to be laid.

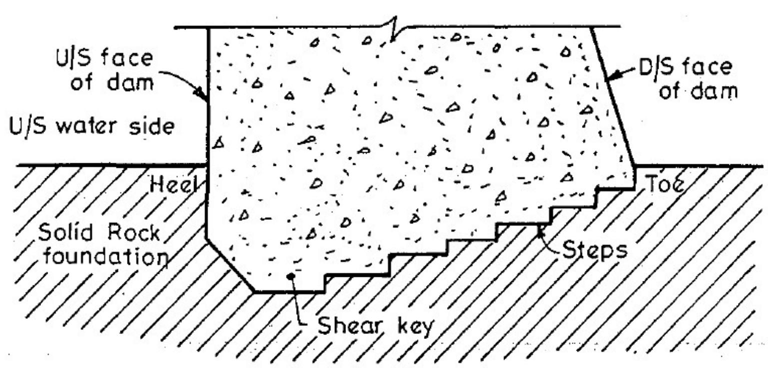


Fig. 19.36. Stepping of dam foundation and provision of shear key.

(ii) **Grouting the Foundation**
The foundation grouting can be divided into :
(a) Consolidation grouting ; and
(b) Curtain grouting.

(a) Consolidation grouting

The entire foundation of the dam is consolidated by grouting. For this purpose, shallow holes (called *B* holes) are drilled through the foundation rock. The depths of these holes generally vary between 10 to 15 m. They are situated at about 5 to 20 m apart, in the general area of the heel of the dam. After the holes have been drilled, mixtures of cement and water (called grout) is forced into the holes at low pressure of about 30 to 40 N/cm2. This is accomplished before any concreting for the dam section is laid. This low pressure grouting will result in a general consolidation of the foundations. These low pressure grout holes will later serve the purpose of a cut-off against leakage of high pressure grout which is to be used after some concreting of the dam has taken place.

(b) Curtain Grouting

It helps in forming the principal barrier or a curtain against the seepage through the foundations, and thus reduce the uplift pressures. To accomplish this high pressure grouting, relatively deeper holes (called *A* holes) are drilled near the heel of the dam. The spacing of the holes may vary from 1.2 to 1.5 m. Holes are first of all, drilled and grouted at about 10 to 12 m apart, and then the intermediate holes are drilled and grouted. The depths of the holes vary from 30 to 40% of the total upstream water head for strong rock foundations, and may be as much as 70% of the water pressure head for poor rocks. After the holes have been drilled, a mixture of cement and water *(i.e.* grout) is forced into the holes under high pressure. The grouting pressure may be kept as high as possible without lifting the foundation strata. Usually, the foundation pressure used in this high pressure grouting is equal to 2.5 *D* N/cm2, where *D* is the depth of grouting in metres below the surface. This grouting is generally done in stages of depth equal to 15 m or so, and carried out only after some portion of the dam section has been laid.

This grouting may have to be accomplished from the foundation gallery or from other galleries within the dam. It may also be done from the upstream face of the dam, if possible. In certain special cases, this grouting may have to be accomplished from tunnels driven into the foundation rock below the dam.

Earthen Dams

- **Earthen Dams have been constructed from long past**. They are constructed with the natural materials. **The construction of earthen dam, up to 1930,** was based mostly on experience. But now with the advance knowledge of soil mechanics, these dams are designed and constructed on scientific basis. With the increased knowledge of the **behavior of soils and the development of earth moving machinery earth dams can be constructed economically even up to the height of 250m to 300 m.**

Earthen Dams

Materials of Earthen Dams

- **Earthen dam require very large quantity of materials**. It is necessary to utilize the soils available in **large quantities near the site**. In general earth dams can be designed to fulfill its **function satisfactorily with any type of material available**.

Materials of Earthen Dams

Materials of Earthen Dams

The following material are commonly used.

- Clayey material
- Black cotton soil, silty clayey loam, for heating and cutoff
- Sandy Material
- Murum, soft rock, sandy silt, for casting
- Rock
- For pitching and riprap, rock masonry, etc.
- Sand
- For filters, seepage drain and masonry
- Cement, steel, lime, and other building materials in small quantities for the construction of spillway, outlets, etc.

Materials of Earthen Dams

Types of Earthen Dams

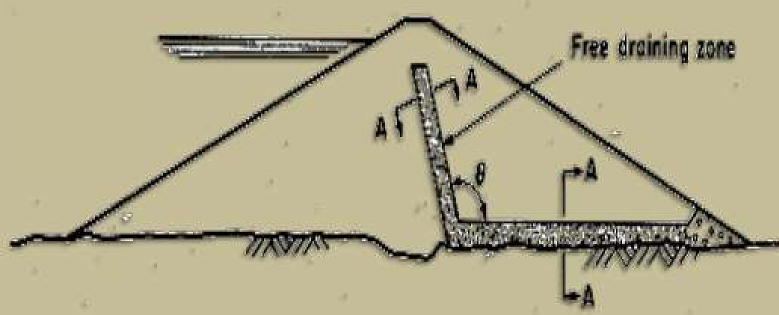
- Depending upon the mode of construction, earthen dams are classified as
- **Homogeneous Type**
- **Zone Type**

Types of Earthen Dams

Homogeneous Type

- **Homogeneous sections are constructed with one type of soil**. The soil should have frictional resistance as well as low permeability and should be available in required quantities near site. **Such sections are not in common practice.**

Homogeneous Type

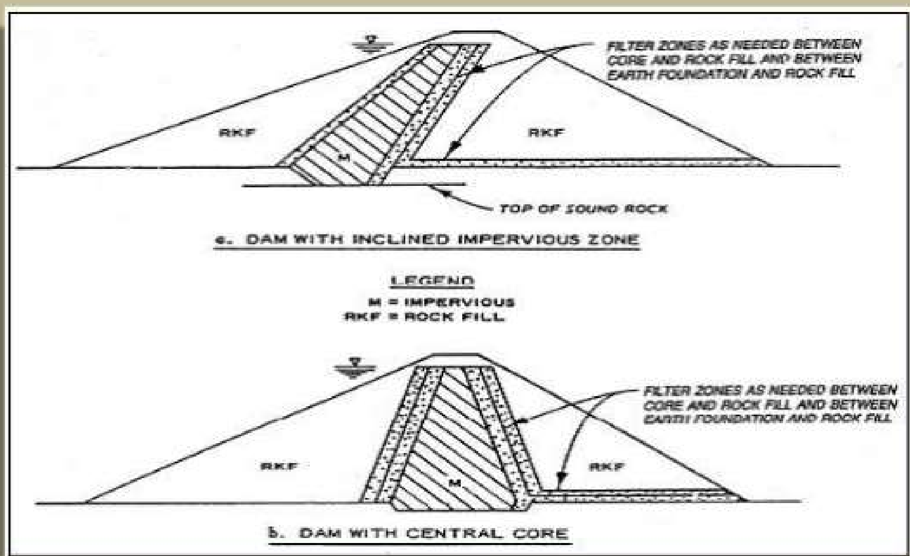


Types of Earthen Dams

Zone Type

- This type of dams are generally used
- The dam is divided into two parts:
- **Hearting or Core forming the central impervious Zone**
- Casing or outlet shell forming the upstream and downstream casing zone and covering the hearting.
- The **hearting is made of clayey soil such as black cotton soil.** It provides water tightness to the dam against seepage.

Types of Earthen Dams



Types of Earthen Dams

- The **casing is made of sand and gravel or murum**, soft rock, etc. It provides water tightness to the dam against seepage.
- The casing is made of sand and gravel or murum, soft rock etc. **It provides stability to the dam section.**
- **In the most of the dam sites, clayey silts, murum, gravelly deposits are available and all such local materials** can be used in zoned type embankments.

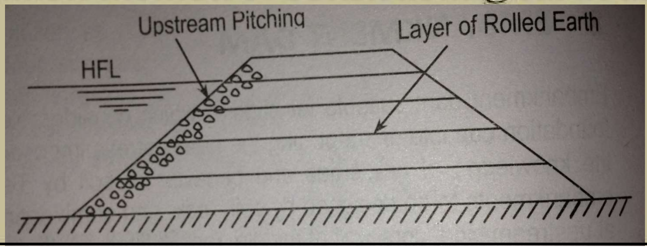
Types of Earthen Dams

- Depending on the method of construction, earthen dams are also classified as
- **Rolled fill Type**
- **Hydraulic fill Type**

Types of Earthen Dams

Rolled fill type

- In this type, the earth moving machinery is used for excavating the soils, placing in layers of 20 cm thickness and compacting at optimum moisture content. This is very common method of constructing earth dams

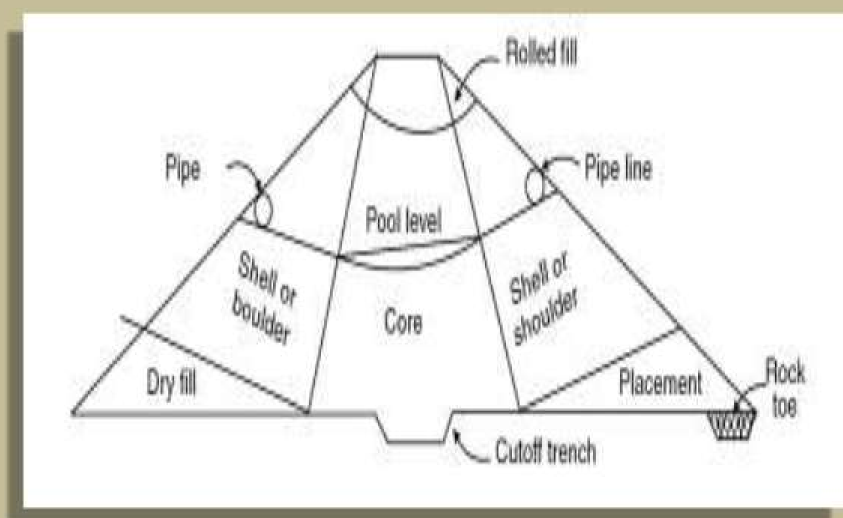


Types of Earthen Dams

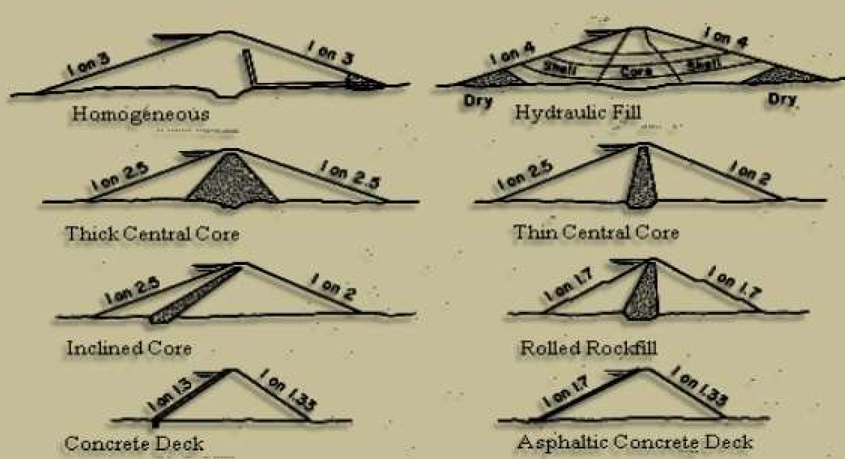
Hydraulic fill type

- In this type of dam construction, excavation, transporting and placing of soils is done by hydraulic method. No compaction by roller or sheep foot rollers is required as the soil gets consolidated during the hydraulic operations

Hydraulic Fill Type



Types of Earthen Dams

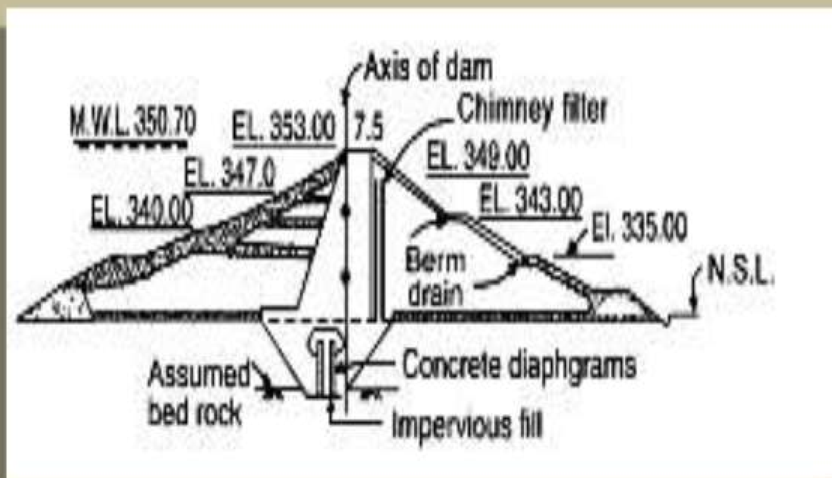


Types of Earthen Dams

Diaphragm Dam

- **It consist of a central impervious thin diaphragm of earth, cement concrete or bituminous concrete surrounded by earth or rock fill.** The diaphragm extends through the cutoff to the impervious foundation to check seepage through the dam. **It may be placed either centrally or at the impervious foundation to check seepage through the dam. It may be placed either centrally or at the upstream face as blanket. The inclined diaphragm provides slightly better stability against earthquake.**
- When the diaphragm is of earth, by definition, it has thickness of the impervious core at any elevation less than 3 m or less than the height of the embankment above the corresponding elevation.
- However, if the diaphragm thickness equals or exceeds these limits, it is considered to be zoned embankment type.
- The core has base width of 0.3 to 0.5 times the water head.

Diaphragm Dam



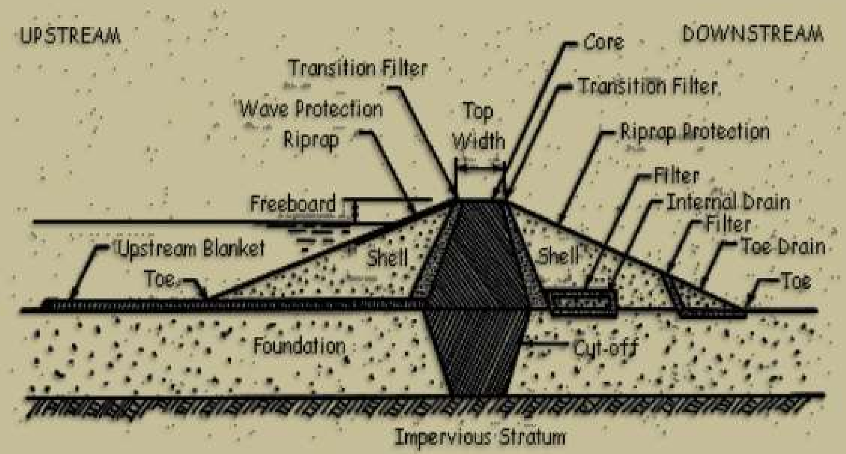
Component parts of earthen dams

Component parts of earthen dams

Hearting (Core)

- It forms the central impervious section constructed with clayey soil, silt clay loam, etc. It is compacted at O.M.C it provides water tightness to the dam and adequate shear resistance against slipping. It controls the seepage flow through the body of the dam.

Component parts of earthen dams



Component parts of earthen dams

Casing

- It forms the outer portion of the dam. It is constructed with murum soft rock, or sand and gravel etc. It is compacted at its **O.M.C Casing provides a cover to the hearting protecting it from cracking.** It develops shear resistance against slip, and provides stability to the dam. It also helps in drainage.

Component parts of earthen dams

Rock Toe

- It is constructed from the rock pieces or boulders larger than 20 cm size, **it helps to prevent slogging of the toe due to the seepage flow and increase the stability of dam.**

Component parts of earthen dams

Pitching

- Pitching of 30 cm to 45 cm thickness is provided by laying stones of 30 cm size, and 40 kg to 50 kg weight on a dressed upstream slope. **It prevents the erosion of material on the upstream face caused due to wave action and protects the slope from sudden drawdown.**

Component parts of earthen dams

Turfing

- It is **planting of special type of grass called harali on the downstream face of the dam**
- It **protects the downstream slope from eroding action of rain water.**

Berms

- Berms are offsets provided on downstream at 8 to 10 m vertical intervals from 3 to 5 m width.
- The **object of berm is to collect rain water and dispose it off safely.**
- To provide **roadways for vehicles**
- To **reduce the velocity of rain** water falling on slope
- To provide minimum cover of 2 m above the seepage line.

Component Parts of Earthen Dams

Component parts of Earthen Dams

Component parts of earthen dams

Drains

- A network of drains is provided with longitudinal drains (L-drains) cross-drains and toe drain on downstream side of the embankment.

Transition filters

- It is graded filter placed in between clayey core and sandy shells (along d/s slope of hearthing and help reduce pore pressure.

Conditions of stability of Earthen Dams

- The dam should not be **overtopped** by flood waters.
- The **seepage line should be well within the d/s face of the dam**
- The **u/s and d/s face** should be **stable** under the worst conditions.
- There **should be no opportunity for free flow of water from u/s and d/s**
- The **foundations shear stresses should be within the safe limits**
- The u/s slope should be **protected from wave action** and burrowing animals.
- The dam and foundation should be safe against piping.

Conditions of stability of Earthen Dams

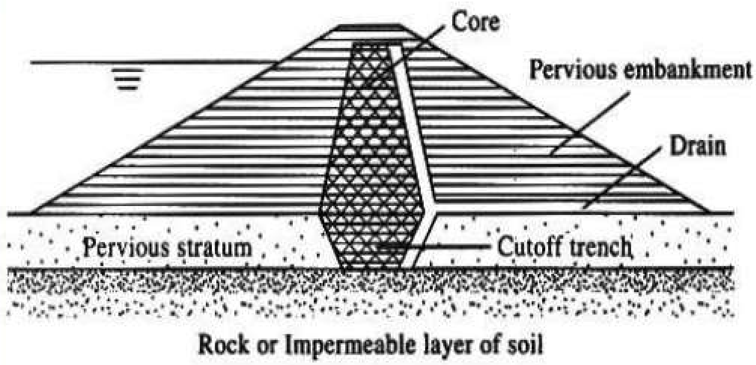


Figure 6-3 Cutoff trench and core of an earth dam

Seepage Line (Phrathic Line)

- It is also called **saturation line or Hydraulic gradient line.** It is defined as the line within a dam section below which there are positive hydrostatic pressures in the dam. **On the line itself the hydrostatic pressure is equal to atmospheric pressure or zero. Above the saturation line** there will be a zone of saturation in which the hydrostatic pressure is negative. The saturation line should nor strike the downstream face of the dam. **Minimum cover over the seepage line should be 2m.**

Cause of failure of Earthen Dams

- The main cause of earthen dam can be classified as under
- **Hydraulic Failures**
- **Seepage Failures**
- **Structural Failures**
- **Earthquake Failures**

Cause of failure of Earthen Dams

Cause of failure of Earthen Dams

(a) Hydraulic Failure

- Above 40 % of earthen dam failures are due to this reason only.
- Hydraulic failures are due to the following reasons.
- (i)By **over topping** The overtopping of dam may cause due to insufficient capacity of spillway and insufficient free board or its spillway gates are not properly operated.
- (ii) **Erosion of u/s slope**
- Erosion is caused due to wave action on the upstream slope and leads to its slip. The slope should be properly protected by providing pitching
- (iii) **Cracking due to frost action**
- Cracks in the upper portion are developed due to frost. It leads to profuse seepage and consequent failure. In areas of low temperature additional free board about 1.0 m should be provided to guard against such failure.

Hydraulic Failure

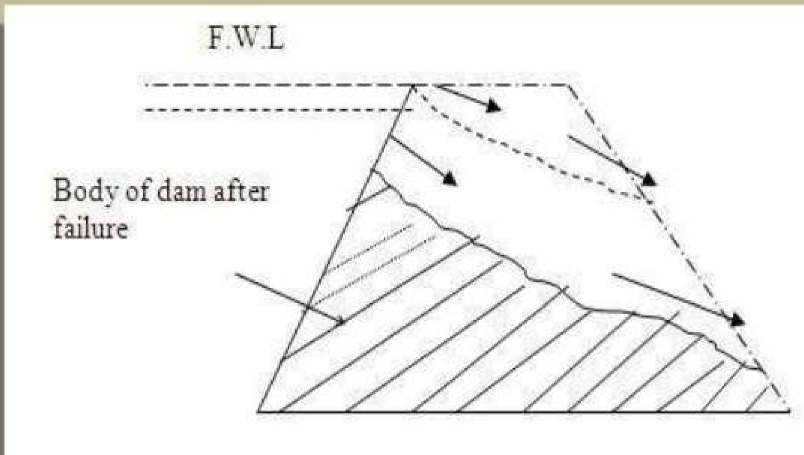


Fig. Dam failure by overtopping

Cause of failure of Earthen Dams

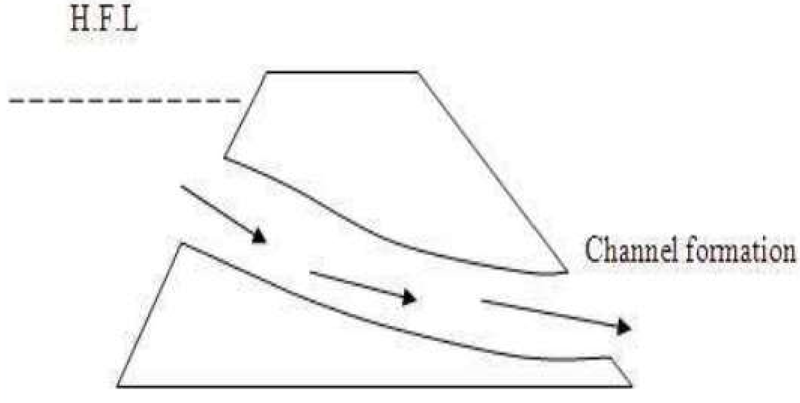
- (iv) **Erosion of d/s slope** Erosion occurs on the d/s slope due to rain action. **If unchecked, it forms gullies on the d/s face, ultimately leading to dam failure. This can be avoided by planting harali on d/s face by proper maintenance.**
- (v) **Erosion of d/s toe**
- The **toe of the dam may be eroded due to heavy cross-current coming from spillway bucket or tail water.** The d/s slope should be protected by providing stone pitching or riprap

Cause of failure of Earthen Dams

Seepage Failure

- **More than 33 % of earthen dam failure are due to seepage. Seepage always occur in earth dams.** It does not harm its stability if it is within the design limits. But excessive seepage will lead to failure of the dam.
- **(i) Piping through the body of the dam**
- **It is due to transport of soil particles with seepage flow.** It results in gradual formation of drain from u/s to d/s through which water flows and thus the dam fails.

Seepage Failure



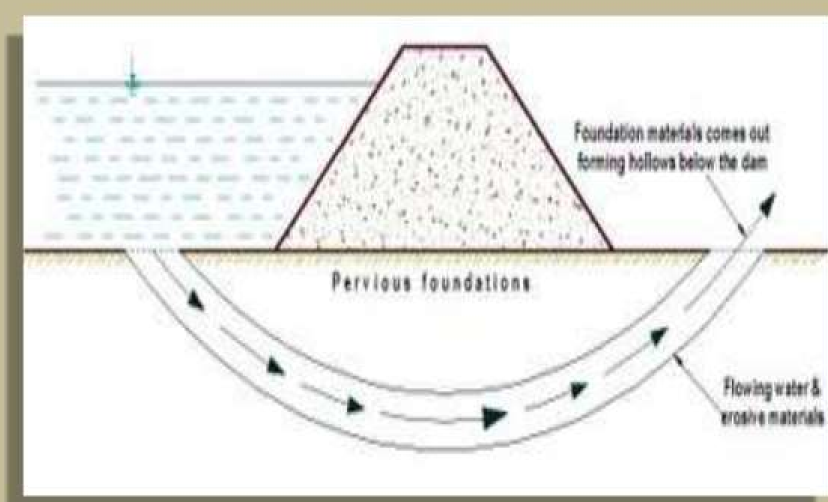
Failure of dam due to piping through dam body

Cause of failure of Earthen Dams

(ii) Piping through foundations

- **When highly permeable strata of gravel, sand or cavities are present in the foundation of dams**, it permits heavy seepage of water through it causing erosion of soil which will result in the formation of piping. **Hence, the dam will sink down causing its failure. Careful investigation of foundations soil and proper will help in avoiding such failures.**

Piping through Foundations



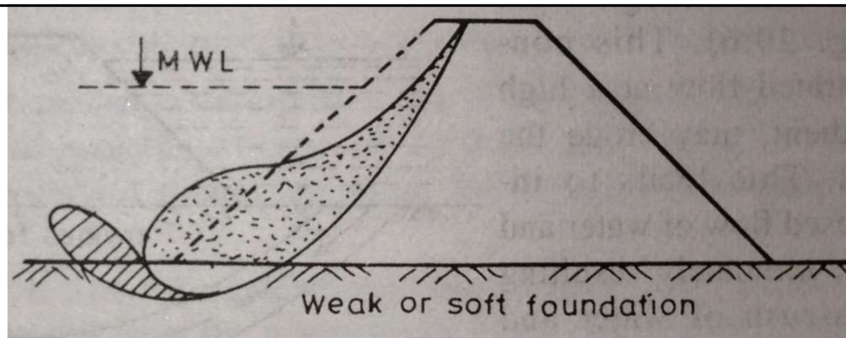
Cause of failure of Earthen Dams

Cause of failure of Earthen Dams

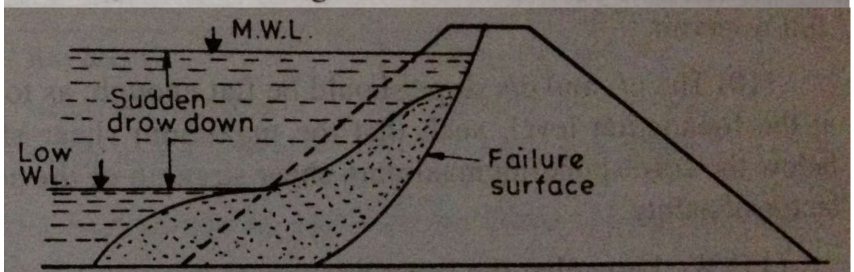
Structural Failures (Shear failures)

About 25 to 30 % of the dam failure are due to this reason

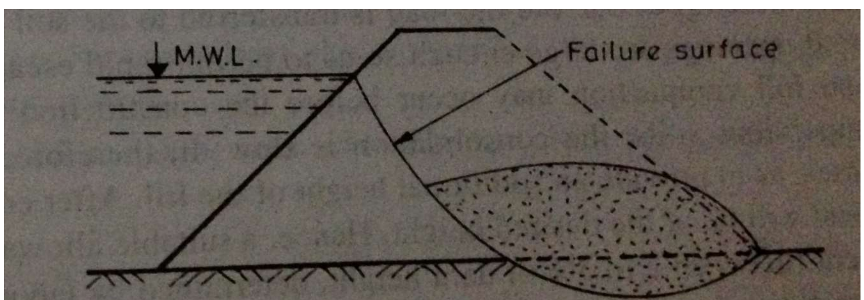
- **(i) U/S and D/S slopes slide**
- The slopes being steeper than required, leads to slips due to stress strength. The slopes should, therefore, be flat as required from structural point of view.
- **(ii) Sudden draw-down**
- The sudden draw- down in water level of the reservoir causes slips of u/s slope. The slope should be flat enough to be stable under sudden drawdown.



Sliding due to soft or weak foundation.

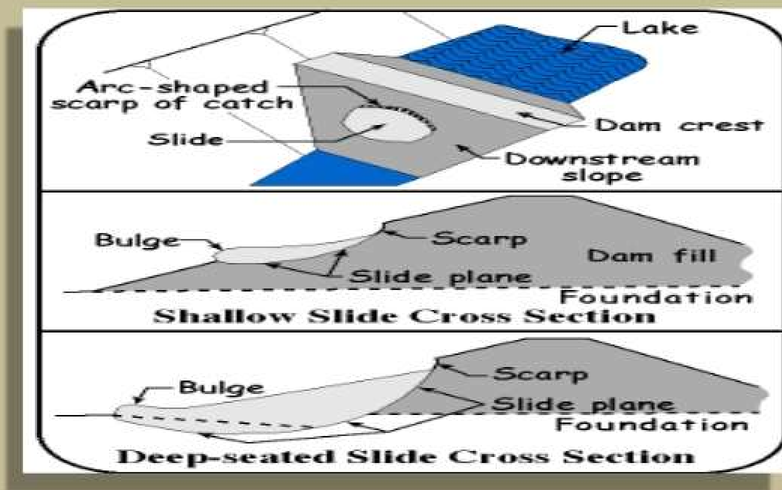


U/S slope slide due to sudden draw-down.



D/S. slope slide during full reservoir condition.

Structural Failures (Shear failures)



Cause of failure of Earthen Dams

- **(iii) Faulty construction and improper maintenance**
- Wrong placement of material in different zones
- Under compaction or over compaction
- Blind drains due to mixing of soil.
- Timely repair of gullies, rain cuts, settlement, pitching will help for better health of the dam

Failure by Earthquakes

- The potential hazard to a dam from earthquake depends on how large the earthquake is and how near to dam site it is. **Main hazards to a dam from an earthquake are surface faulting under the dam**, strong ground shaking, water waves in the reservoir produced by earthquake ground motions or land slides and rock falls and **pervasive ground deformation associated with nearby faulting which may manifest in cracking at dam top and central core, settlement of dam, crest, shear failure at the base of dam, liquefaction of loose and structured soil mass in the lower portions of the dam, and overtopping due to high waves generated in the reservoir.**

Failure by Earthquakes

Failure by Earthquakes

Seepage Control in Earth Dams

- The **Water seeping through the body of the earthen dam or through the foundation of the earthen dam, may prove harmful to the stability of the dam** by causing softening and sloughing of the slopes due to development of pore pressures. **It may also cause piping either through the body or through the foundation, and thus resulting in the failure of the dam.**

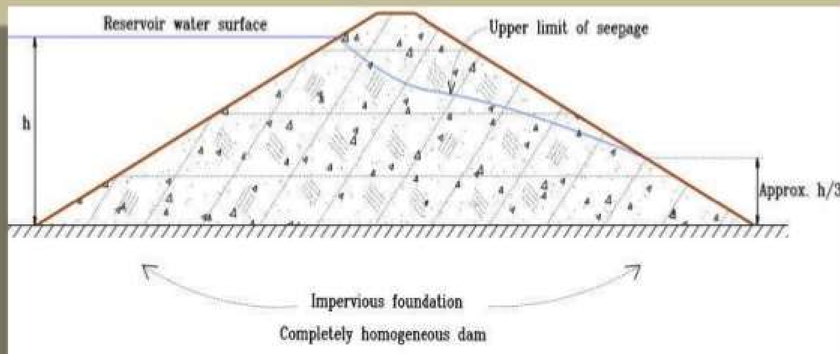
Seepage Control in Earth Dams

Seepage Control in Earth Dams

Seepage Control through Embankment

- Drainage filters called 'Drains' are generally provided in the form of
- **(a) Rock toe (b) horizontal blanket (c) Chimney drain, etc**. in order to **control the seepage water**. The provision of such filters reduces the **pore pressure in the down stream portion of the dam and thus increases the stability of the dam, permitting steep slopes** and thus affecting economy in construction. It also checks piping by migration of particles. These drains, consist of **graded coarse material in which the seepage is collected and moved to a point where it can be safely discharged.**
- **A multi layer filter, generally called inverted filter or reverse filter is provided.**

Seepage Control in Earth Dams



Seepage flow through earth dam with no filter at the dam toe

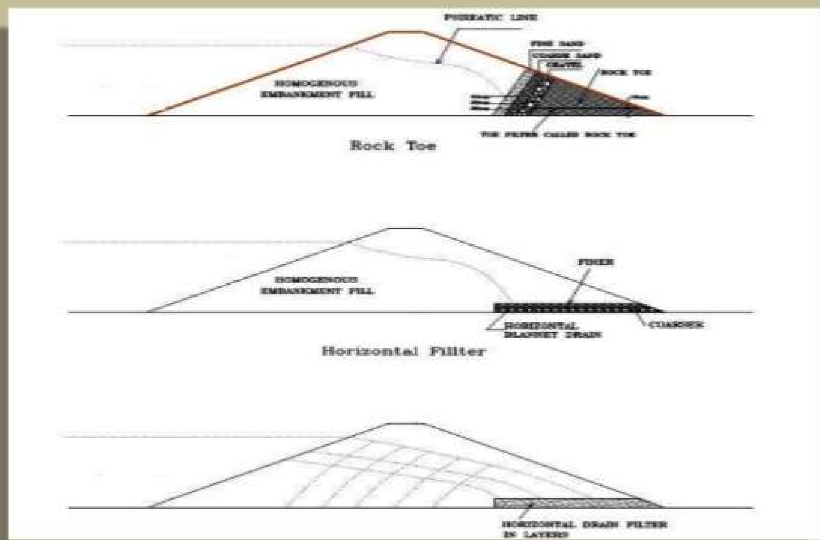
Seepage Control in Earth Dams

- The various kinds of drains which are commonly used are as shown below:

Rock Toe or Toe Filter

- The **'rock toe' consists of stones of size usually varying from 15 to 20 cm**. a toe filter is provided as a transition zone, between the homogeneous embankment fill and rock toe.
- **Toe filter generally consists of three layers of the fine sand, coarse sand and gravel.**
- **The height of the rock toe is kept between 25 to 35 % of reservoir head.** The top of the rock toe must be **sufficiently higher than the tail water depth, so as to prevent the wave action of the tail water.**

Rock Toe or Toe Filter

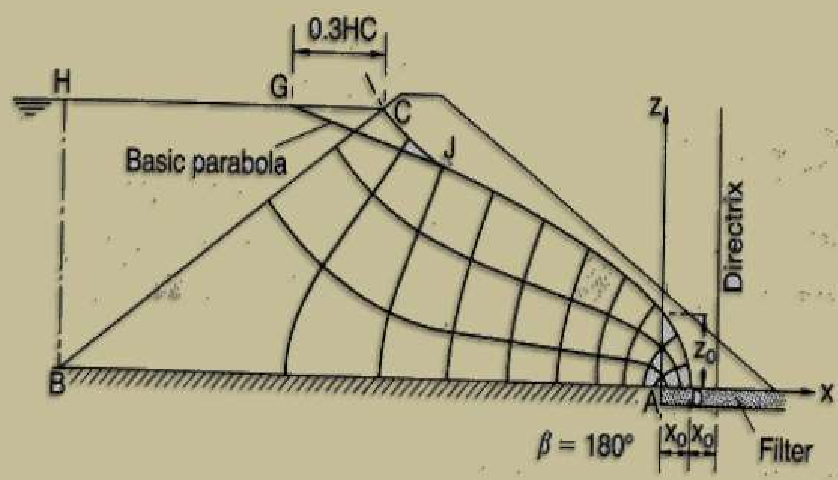


Seepage Control in Earth Dams

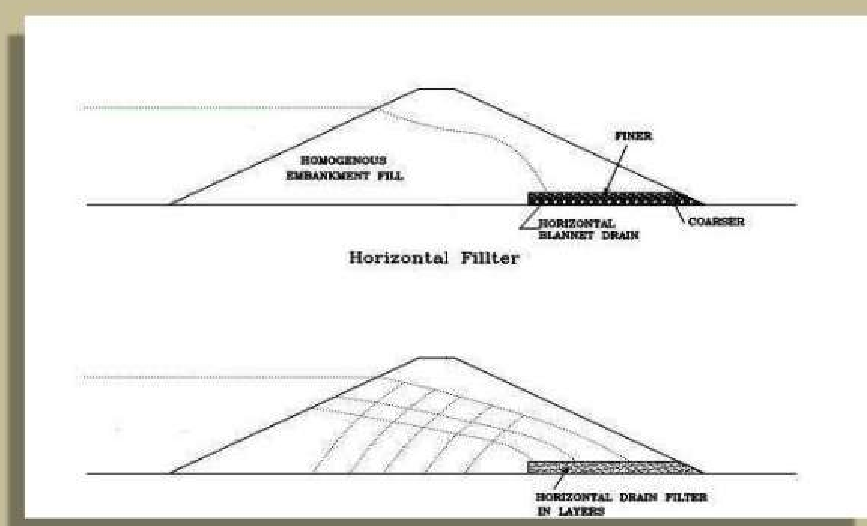
Horizontal Blanket or Horizontal Filter

- The **horizontal filter extends from the toe (d/s end) of the dam**, inward, **up-to a distance varying from 25 % to 100 % of the distance of the toe from the centre line of the dam.** Generally, a length equal to three times the height of the dam is sufficient. **The blanket should be properly designed as per the filter criteria, and should be sufficiently pervious to drain off effectively.**

Horizontal Blanket or Horizontal Filter



Horizontal Blanket or Horizontal Filter

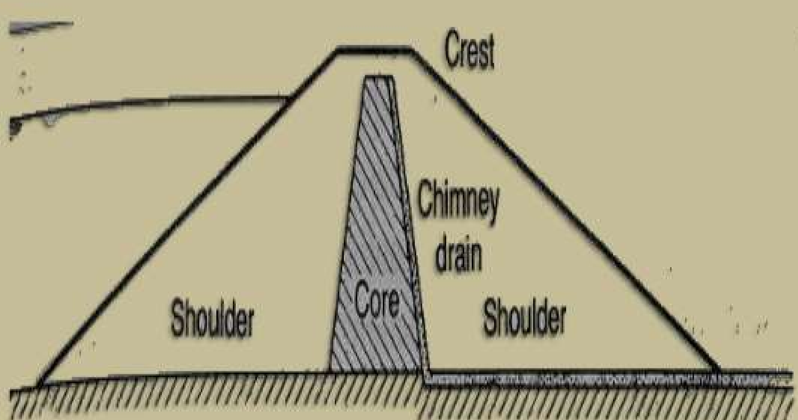


Seepage Control in Earth Dams

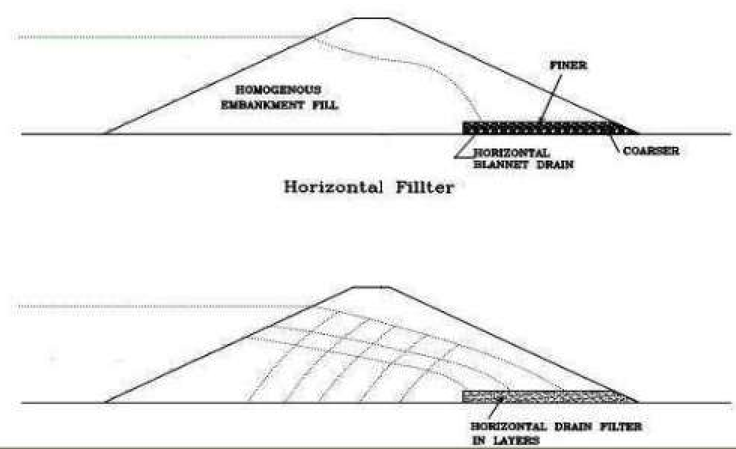
Chimney Drain

- The **horizontal filter, not only helps in bringing the phreatic line down in the body of the dam but also provides drainage of the foundation and helps in rapid consolidation**. But the **horizontal filter tries to make the soil more pervious in the horizontal direction and thus causes stratification.** When large scale **stratification occurs, such a filter becomes inefficient**. In such a possible case, **a vertical filter is placed along with the horizontal filter,** so as to intercept the seepage **such an arrangement is called chimney drains.** Sometimes a horizontal filter is combined and placed along with a rock toe.

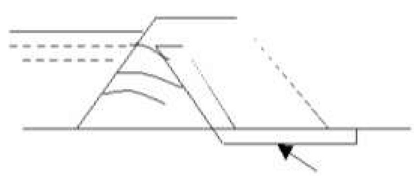
Seepage Control in Earth Dams



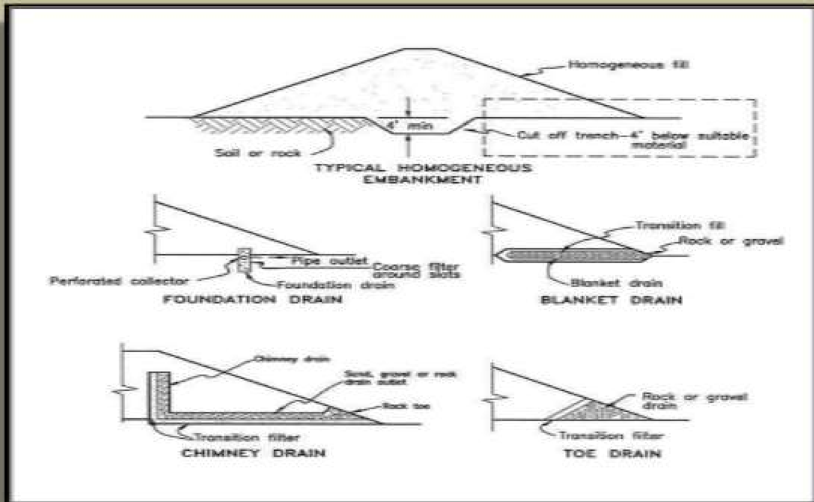
Horizontal Blanket or Horizontal Filter



Seepage Control in Earth Dams



Seepage Control in Earth Dams



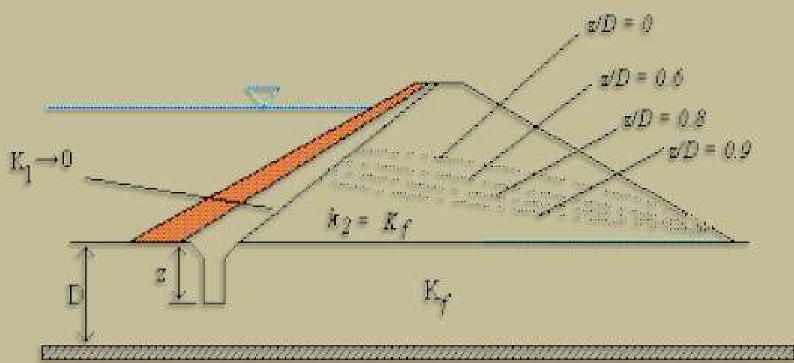
Seepage Control in Earth Dams

Seepage Control through foundations

Impervious Cutoffs

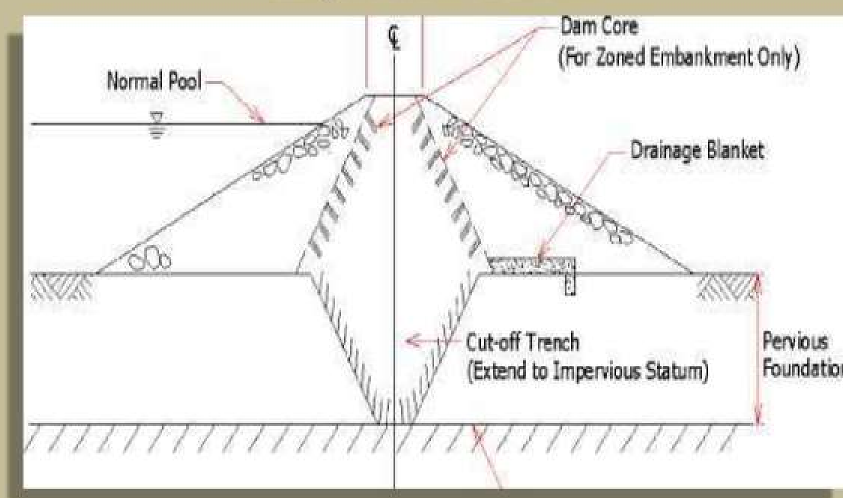
- Various **impervious cutoffs made of concrete or sheet piles may be provided at the upstream end (i.e..... at heel) of the earthen dam.** These **cutoffs should be extending through the entire depth of the pervious foundations so as to achieve effective control on the seeping water.** When the depth of the pervious **foundation strata is very large**, a cutoff, up to a lesser depth may be provided. **Such a cutoff reduces the seepage discharge by a smaller amount. So much so, that a 50 % depth reduces the discharge by 25 % and 90 % depth reduces the discharge by 65 %.**

Seepage Control through foundations Impervious Cutoffs



Effects of Partial Cutoff on Position of Line of Seepage

Seepage Control through foundations Impervious Cutoffs

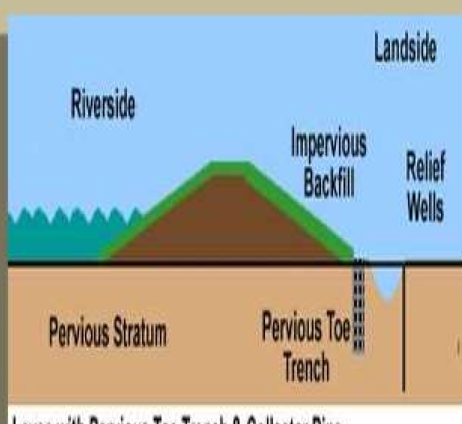


Seepage Control in Earth Dams

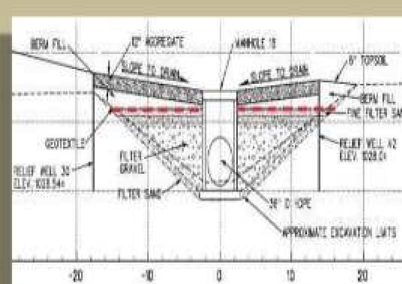
Relief wells and Drain Trenches

- When **large scale seepage takes place through the pervious foundation**, overlain by thin pervious layer, there is a **possibility that the water may boil up near the toe of the dam**
- Such a possibility can be controlled by **constructing relief wells or drain trench through the upper impervious layer**. So as to permit **escape of seepage of water**. The possibility of sand boiling may also be controlled by providing d/s beams beyond the toe of the dam**. The weight of overlying material, in such a case, is sufficient to resist the upward pressure and thus preventing the possibility of sand boiling.**

Relief wells and Drain Trenches



Levee with Pervious Toe Trench & Collector Pipe



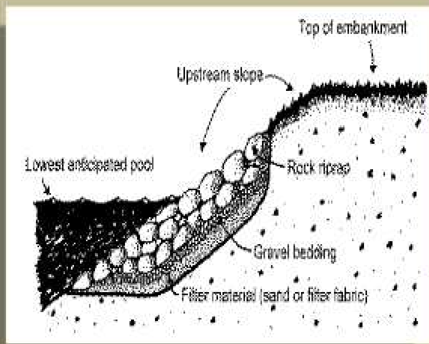
Slope protection

Protection of Upstream Slope

- The **upstream slope of the earth dam is protected against the erosive action of waves by stone pitching or by stone dumping.** The thickness of the **dumped rock should be about 1 m and should be placed over a gravel filter of about 0.3 m thickness.** The filter prevents the washing of fines from the dam into the riprap. The provision of such a dumped rip-rap has been found to the most effective and has been to fail only in 5 % of cases.

Slope Protection

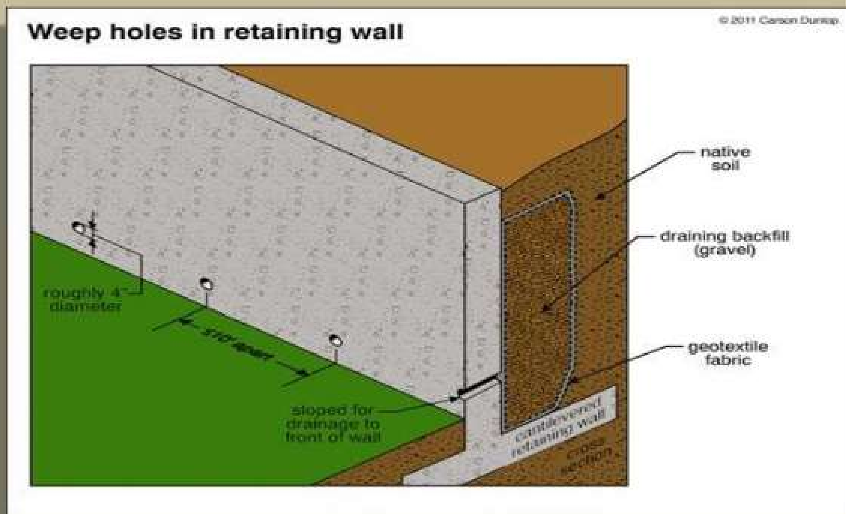
Slope Protection



Slope Protection

- The **stone pitching, i.e...... the hand packed rip rap requires a lesser thickness and may prove more economical if suitable rock is available only in limited quantity.** However, when provided in smaller thickness it is more susceptible to damage and has been founded to fail in about 30 % of cases.
- **Concrete slabs may also be laid over the slope of the earth dam.** When **such slabs are constructed, they must be laid over a filter and weep holes should be provided so as to permit escape of water when the reservoir is drawn down.** If the filter is not provided, the fines from the embankment may get washed away from the **joints creating hollows beneath the slab and causing slab protections have been found to fail in about 36 % cases.**

Slope Protection



Slope Protection

Protection of Downstream Slope

- The **Downstream slope of the earthen dam is protected against the erosive action of waves up to and slightly above the water depth, in a similar manner as is explained above from u/s slope.**
- More, the d/s slope should be protected against the erosive action of rain and its run-off by providing horizontal **berms at suitable interval say about 15 m or so as to intercept the rain water and discharging it safely. Attempts should be made so as to grass and plant the d/s slopes, soon after the construction.**

Slope Protection

Slope Protection

Safety Measures

- The dam safety can be ensured if the following aspects are taken care of:

Hydraulic Failure

- Such type of failure can be averted by providing
- (i) Adequate spillway capacity.
- (ii)Adequate freeboard so that dam safety is not endangered by overtopping during high floods
- (iii) Proper maintenance of gates so that they are always operative and do not get clogged.

Safety Measures

Seepage Failure

- Seepage failure can be taken care by
- (i) Providing filter at the toe to minimize movement of the material
- (ii) Seepage line is well within the body of the dam
- (iii) Provision of settlement, after the composition of the dam, be made from a normal 1 % of height to a maximum of 6 %

Safety Measures

Structural Failure

- In spite of best geological and foundation investigations done prior to dam construction, geological problems may arise such as induced seepage, earth tremors, slides, gougy seams and sloughing in the vicinity of dam and reservoir area surface during the construction or several years after the reservoir filling.
- periodic geotechnical inspection is essential for early detection and resolution of potential problems, besides provision of adequate rip-rap and its maintenance and drawdown within permissible limits.

Safety Measures

Earthquake Failure

- Necessary provisions shall be made in the design of a dam to account for additional forces due to earthquakes.

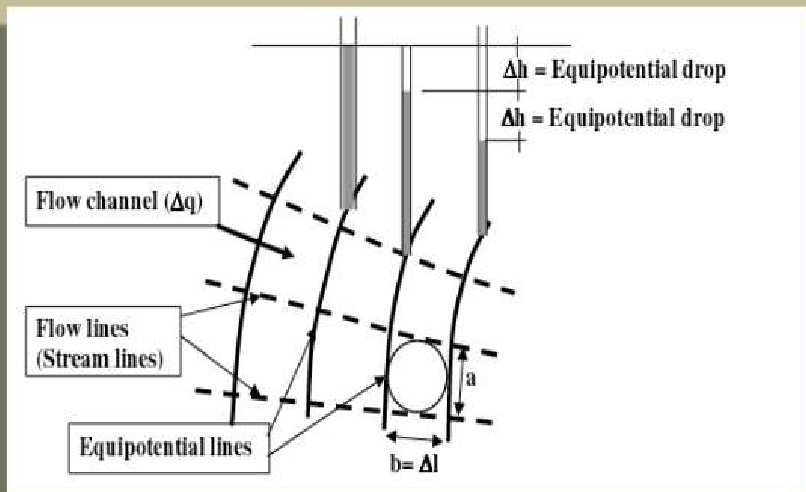
Flow-Net

- A **flownet** is a graphical representation of two-dimensional steady-state groundwater flow through aquifers.
- The **method is often used in civil engineering, hydrogeology or soil mechanics** as a first check for problems of flow under hydraulic structures like dams or sheet pile walls. **As such, a grid obtained by drawing a series of equipotential lines is called a flownet.**

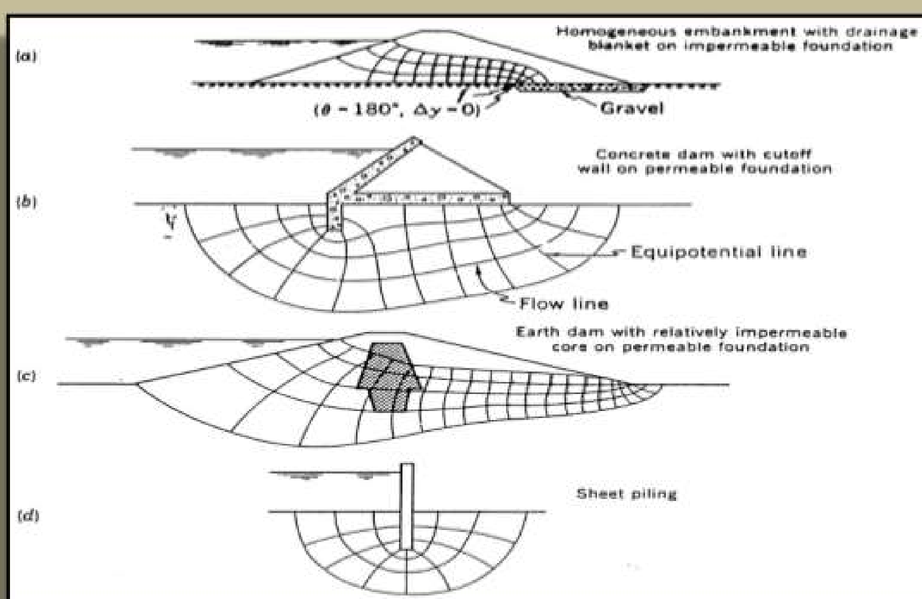
Characteristics of Flow -Nets

- Flow lines & Stream lines **represents flow path of particles of water**
- Flow lines and equipotential lines are **perpendicular to each other.**
- The area between two flow lines is called **flow channel**.
- The rate of **flow in a flow channel is constant.**
- Flow **cannot occur across flow lines**.
- An **equipotential line is a line joining points with same head**
- The **velocity of flow is normal** to the equipotential lines.
- A flow line **cannot intersect another flow line**.
- An **equipotential line cannot intersect another equipotential line.**

Characteristics of Flow -Nets



Flow-Net



Seepage Analysis

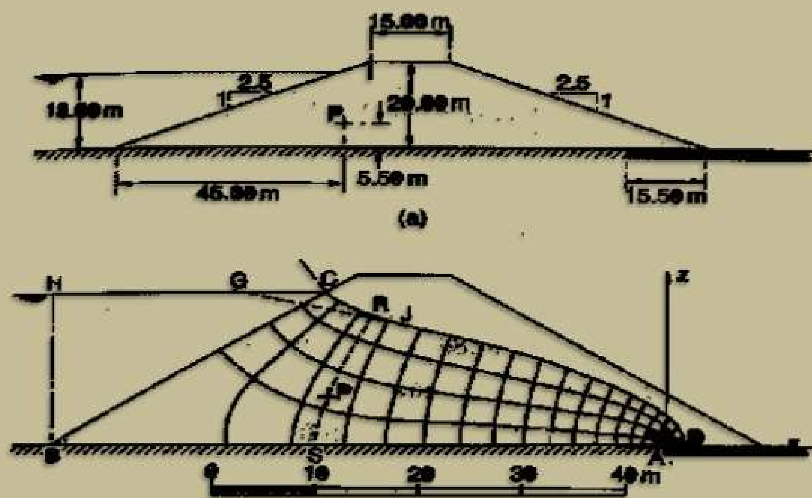
- Seepage occurs through the body of all earthen dams and also through their pervious foundations. The amount of seepage has to be controlled in all conservation dams & the effects of seepage has to be controlled for all dams in order to avoid failures.
- The seepage through a pervious soil material, for two dimensional flow, is given by Laplacian eq

$$\frac{\partial^2\phi}{\partial x^2} + \frac{\partial^2\phi}{\partial y^2} = 0$$

Seepage Analysis

- Where ϕ = K.h= Velocity potential
- K= Permeability of the Soil
- h= Head Causing flow
- The above eq^n is based on the following assumptions:
- Water is **incompressible**
- The **soil is incompressible and porous.** The size of pore space do not change with time regardless of water pressure.
- The **quantity of water entering the soil in any given time is the same as the quantity flowing out of the soil.**
- Darcy's law is valid for the given soils.
- The hydraulic boundary conditions at the entry and exit are known.

Seepage Analysis



Seepage Analysis

- Seepage discharge through the Isotropic Soils
- The amount of seepage can be easily computed for the flow net. Let us assume that the soil is isotropic i.e. its permeability is constant in all the directions, or $K_h = K_v$
- The flow net is drawn by free hand sketch by making suitable adjustments and corrections until the flow lines and equipotential lines are at right are at right angle.

Seepage Analysis

- The seepage rate (q) can be computed from the flow net, using Darcy's law. Applying the principal of continuity between each pair of flow lines. It is evident that the velocity must vary inversely with the spacing.
- $q = \frac{K.H\,N_f}{N_d}$

K= Permeability of the Soil
N f= Number of flow Channels
Nd= Total number of drops in complete flow net.

Seepage Analysis

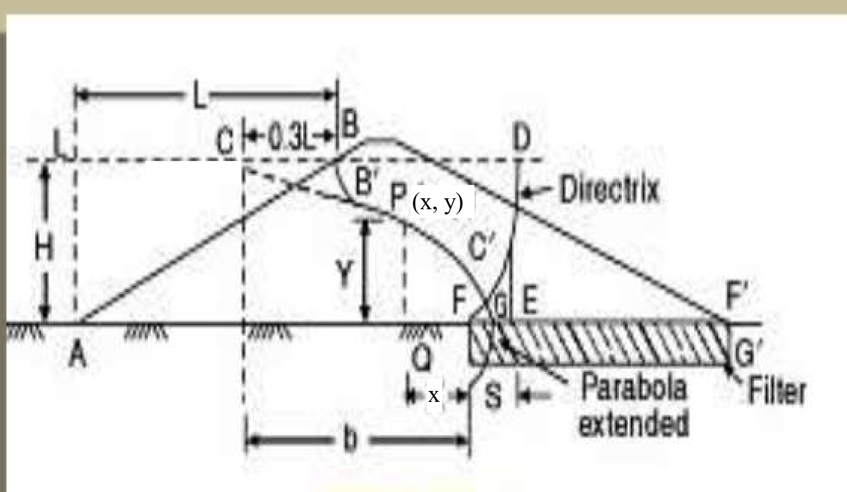
- Seepage Discharge for Non-isotropic Soil
- If the permeability of the soil is different in the horizontal direction than that in the vertical direction;
- The discharge can be computed by the equation
- $Q = (K_H . K_v)^{1/2} \frac{H.N_f}{N_d}$

Determination of Phreatic Line

Dam with a Filter

- In Fig AB is the upstream face and BL (Equal to L) is the horizontal projection. Locate a Point C, on water surface, at a distance equal to 0.3 L from B, i.e. BC= 0.3 L.
- F, the Starting point of the filter is the focus of basic parabola.
- The filter Length usually kept 25 % of the distance of toe of dam to the centre of dam crust.

Determination of Phreatic Line

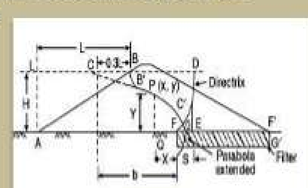


Determination of Phreatic Line

- With point **C** as centre and **CF** as radius draw an arc cutting the line **LB** extended in **D**.
- Draw a vertical tangent to the curve **FD** at **D** such that DE is the Directrix, **CD** being equal to **CF**, point G is located midway between **F & E.**
- Thus a basic parabola is drawn **C C' G** is drawn with focus at F.
- The Base of the parabola is now corrected by eye, a short transition curve **BB'** being drawn to connect point B with base parabola.
- The eq^n of base parabola- with F as Focus and any point P (x,y) on the parabola, is as under.

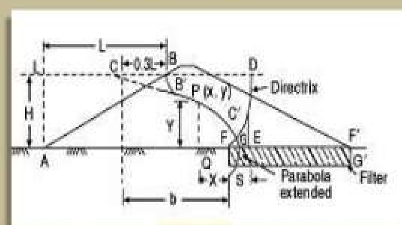
Determination of Phreatic Line

- PF = QE
- $(X^2 + Y^2)^{1/2} = QF + FE$
- $(X^2 + Y^2)^{1/2} = x + S$ (S = focal Distance FE)......................................(1)
- Squaring both sides
- $X^2 + Y^2 = X^2 + 2Sx + S^2$
- $X = \dfrac{Y^2 - S^2}{2S}$
- $Y^2 = 2Sx + S^2$
- Which is the eqn of base parabola, considering point C with coordinates (b,H) on the parabola such that CF=b, substitute in eqn (1) above
- $b^2 + H^2 = b + S$
- $S = (b^2 + H^2)^{1/2} - b$
- At C', x = 0 , y=S, thus vertical ordinate FC' at F is S



Determination of Phreatic Line

- For Determining discharge through the dam consider unit width of dam and let q is the seepage discharge per unit width of the dam
- From Darcy's law
- q = K.I.A
- $q = K \frac{dy}{dx} (y.1)$



A= Saturated depth y x Width

Determination of Phreatic Line

- $q = K \frac{dy}{dx} . y$ ....(2)

But eq^n of Parabola

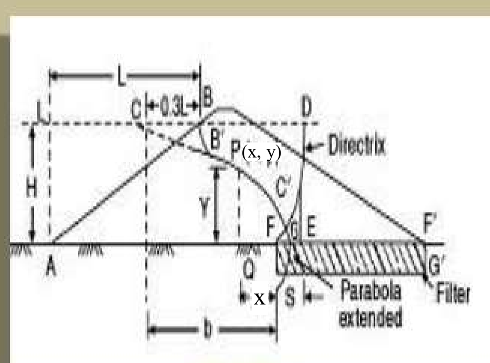
$x^2 + y^2 = (x+S)^2$

$y^2 = (x+S)^2 - x^2$

$y = \{(x+S)^2 - x^2\}^{1/2}$

$y = (S^2 + 2Sx)^{1/2}$

$\frac{dy}{dx} = \frac{S}{(2Sx+S^2)^{1/2}}$

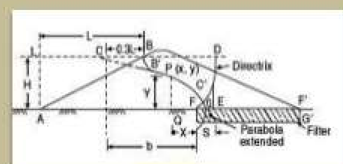


Determination of Phreatic Line

- Substituting the value of y in eq[n] 2 above

$$q = K \cdot \frac{S}{(2Sx + S^2)^{1/2}} \cdot (S^2 + 2Sx)^{1/2}$$

- **$q = K.S$**
- Thus the discharge can be calculated from the known coefficient of permeability K and focal distance S
- For determining q in terms of distance b and height H,
- $S = (x^2 + y^2)^{1/2} - x$
- At C , x=b and y=H
- Therefore $S = (b^2 + H^2)^{1/2} - b$
- Hence $q = K.S$
- **$q = K[(b^2 + H^2)^{1/2} - b]$**



20.12.2. Determination of Phreatic Line when the Dam Section is Homogeneous (without Filter). The phreatic line can be determined on the same principles as was done for dam with a filter case. The focus (*F*) of the parabola, in this case, will be the lowest point of the downstream slope as shown in Fig. 20.13. The base parabola *BIJC* will cut the downstream slope at *J* and extend beyond the dam toe up to the point *C* (*i.e.* the vertex of the parabola).

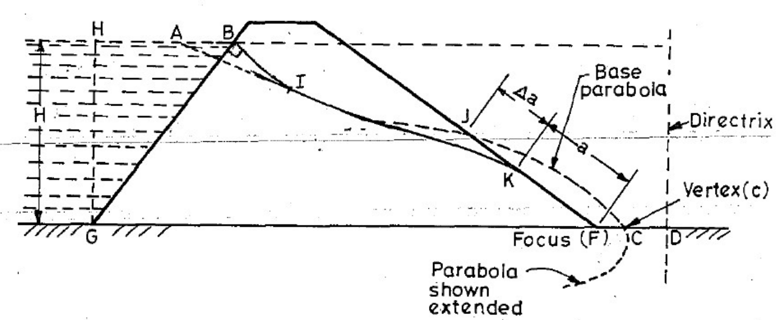


Fig. 20.13

The seepage line will, however, emerge out at K, meeting the downstream face tangentially there. *The portion KF is known as discharge face and always remains saturated.* The correction JK (say Δa) by which the parabola is to be shifted downward can be determined as follows :

(A) Graphical general solution. Cassagrande has given a general solution to evaluate Δa for various inclinations of discharge face. Let a be the angle which the discharge face makes with the horizontal. The various values of $\frac{\Delta a}{a + \Delta a}$ have been given by Cassagrande, as shown in table 20.4.

Table 20.4

α in degrees	$\frac{\Delta a}{a+\Delta a}$	*Remarks*
30°	0.36	*Note.* Intermediate values can be interpolated, or read out from a graph between α and $\frac{\Delta a}{a+\Delta a}$ plotted with the values given here.
60°	0.32	
90°	0.26	
120°	0.18	
135°	0.14	
150°	0.10	
180°	0.0	

$(a + \Delta a)$ is the distance FJ (*i.e.* the distance of the focus from the point where the parabola cuts the d/s face) and is known. Δa can then be evaluated. a and Δa can be connected by a general equation.

$$\Delta a = (a + \Delta a)\left[\frac{180° - \alpha}{400°}\right] \qquad ...(20.21)$$

The value of α will be equal to 180° for a horizontal filter case and may be equal to or more than 90° in case a rock toe is provided at the downstream end, as shown in Fig. 20.14 (*a*). α will be less then 90° when no drainage is provided.

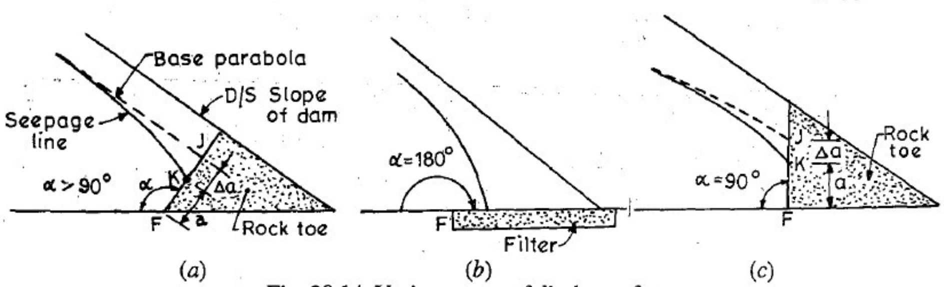


Fig. 20.14. Various types of discharge faces.

(B) Analytical solutions for determining the position of point k, *i.e.* the point at which the seepage line intersects the d/s slope.

Case (*a*) when $\alpha < 30°$

Schaffernak and Van Iterson have derived an equation for determining the value of 'a' (and thus fixing the position of point K) in terms of H, b' and α. Their final equation is

$$a = \frac{b'}{\cos\alpha} - \sqrt{\frac{b'^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}} \quad \text{...(20.22)}$$

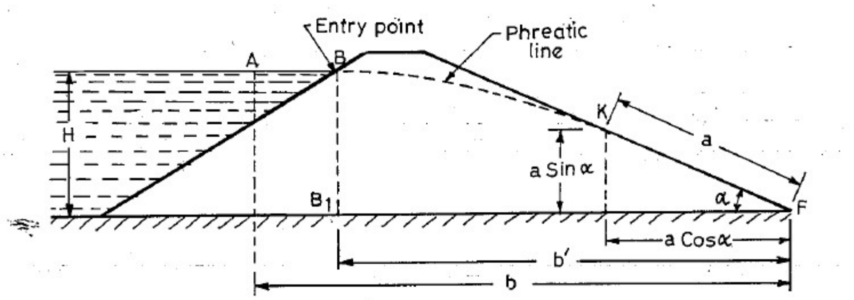


Fig. 20.15

The above equation has been obtained on the assumption that the hydraulic gradient (i) is equal to the slope of the phreatic line. This assumption is nearly true so long as the downstream slope is sufficiently flat (*i.e.* $\alpha < 30°$).

This equation can be easily derived on the basis of the above assumption as follows:

$$i = \text{Hydraulic gradient} = \frac{dy}{dx}$$

$$A = y \cdot 1$$

$$q = KiA = K \cdot \left(\frac{dy}{dx}\right) \cdot (y \cdot 1)$$

$$\text{But } \frac{dy}{dx} = \tan\alpha$$

$$\text{and} \quad y = a\sin\alpha$$

$$\therefore \quad q = K \cdot \tan\alpha \cdot a \cdot \sin\alpha$$
$$= K \cdot a \cdot \sin\alpha \cdot \tan\alpha$$

or $$K \cdot \frac{dy}{dx}(y) = K \cdot a \cdot \sin\alpha \tan a$$

or $$\frac{dy}{dx}(y) = a\sin\alpha\tan\alpha$$

or $$dy \cdot y = a\sin\alpha\tan\alpha\, dx$$

or $$y \cdot dy = a\sin\alpha\tan\alpha\, dx.$$

Integrating both sides between the limits

$$x = a\cos\alpha \quad \text{to} \quad x = b'$$

and $$y = a\sin\alpha \quad \text{to} \quad y = H,$$

we get $$\int_{y=a\sin\alpha}^{y=H} y\cdot dy = a\sin\alpha\cdot\tan\alpha\int_{x=a\cos\alpha}^{x=b'} dx$$

or $$\left|\frac{y^2}{2}\right|_{a\sin\alpha}^{H} = a\sin\alpha\tan\alpha\,\left|\,x\,\right|_{a\cos\alpha}^{b'}$$

or $$\frac{H^2 - a^2\sin^2\alpha}{2} = a\sin\alpha\tan\alpha\left[b' - a\cos\alpha\right]$$

or $$\frac{H^2}{2} - \frac{a^2}{2}\sin^2\alpha = ab'\sin\alpha\tan\alpha - a^2\cdot\sin\alpha\cos\alpha\cdot\tan\alpha$$

or $$\frac{H^2}{2} - \frac{a^2}{2}\sin^2\alpha = ab'\sin\alpha\tan\alpha - a^2\cdot\sin^2\alpha$$

or $$\frac{H^2}{2} = ab'\sin\alpha\tan\alpha - \frac{a^2}{2}\sin^2\alpha$$

or $$\frac{a^2}{2}\cdot\sin^2\alpha - ab'\sin\alpha\tan\alpha + \frac{H^2}{2} = 0$$

or $$a^2\sin^2\alpha - 2ab'\sin\alpha\tan\alpha + H^2 = 0$$

or $$a^2 - \frac{2ab'}{\cos\alpha} + \frac{H^2}{\sin^2\alpha} = 0$$

or $$a = \frac{\dfrac{2b'}{\cos\alpha} \pm \sqrt{\left(\dfrac{2b'}{\cos\alpha}\right)^2 - \dfrac{4H^2}{\sin^2\alpha}}}{2}$$

Ignoring unfeasible +ve sign, we have

$$\boxed{a = \frac{b'}{\cos\alpha} - \sqrt{\frac{b^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}}}$$

This is the required equation.

Case (*b*)·When α lies between 30° and 60°

Cassagrande has derived an equation for determining the value of '*a*' in terms of *b*, *H* and α. His final equation is

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2 \cot^2 \alpha} \qquad \text{...(20.23)}$$

where *b* is defined in Fig. 20.16,

H is the head causing flow and α is the angle which the d/s face makes with the horizontal (clockwise) as defined earlier.

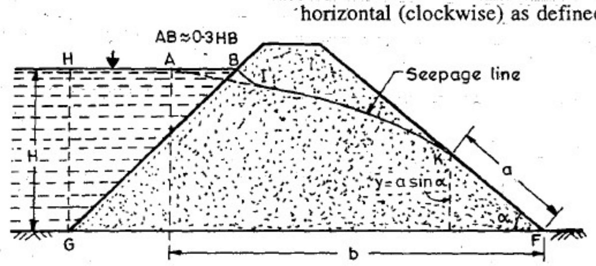


Fig. 20.16

Equation (20.23) gives satisfactory results for values of α less than 30°, but for steeper slopes, the deviations from the correct values become quite high and for such cases, Cassagrande has suggested the use of sin α in place of tan α.

In other words, the hydraulic gradient (i) is given by $\frac{dy}{ds}$ (*i.e.* sin α) of the seepage line and not by $\frac{dy}{dx}$ (*i.e.* tan α) as was taken in the previous case.

Therefore, $q = K \cdot i \cdot A$

$$q = K\,(dy/ds)y.1 = K\,(\sin\alpha)\,(a\sin\alpha) \qquad \because \frac{dy}{ds} = \sin\alpha$$

$$K\,(dy/ds)y = K \cdot a \cdot \sin^2\alpha$$

$$dy\,.\,y = a\sin^2\alpha \cdot ds.$$

Integrating between the limits

$$y = a\sin\alpha, \qquad s = a$$

$$y = H \qquad s = S_0$$

where S_0 is the total length of parabola from the point *A* to the point *F*.

or $$\int_{y=a\sin\alpha}^{y=H} y \cdot dy = a \cdot \sin^2\alpha \int_{s=a}^{s=S_0} ds$$

or $$\left|\frac{y^2}{2}\right|_{a\sin\alpha}^{H} = a\sin^2\alpha \cdot |s|_{s=a}^{s=S_0}$$

or $$\frac{H^2 - a^2\sin^2\alpha}{2} = a\sin^2\alpha\,[S_0 - a]$$

or $$\frac{H^2}{2} - \frac{a^2}{2}\sin^2\alpha = a\sin^2\alpha \cdot S_0 - a^2\sin^2\alpha$$

or $$\frac{a^2}{2}\sin^2\alpha - S_0\sin^2\alpha \cdot a + \frac{H^2}{2} = 0$$

or $$a^2 - 2S_0 \cdot a + \frac{H^2}{\sin^2\alpha} = 0$$

or $$a = \frac{2 \cdot S_0 \pm \sqrt{4 \cdot S_0^2 - 4 \cdot \frac{H^2}{\sin^2\alpha}}}{2}$$

Ignoring +ve sign, we get

$$a \doteq S_0 - \sqrt{S_0^2 - \frac{H^2}{\sin^2 \alpha}} \qquad ...(20.24)$$

The total length of the parabola S_0 can be approximately taken to be equal to $\sqrt{b^2 + H^2}$, then

$$S_0 = \sqrt{b^2 + H^2}$$

Substituting, we get

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 + H^2 - \frac{H^2}{\sin^2 \alpha}}$$

or

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2\left(\frac{1}{\sin^2 \alpha} - 1\right)}$$

$$\boxed{a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2 \cot^2 \alpha}}$$ which is the required eqn. (20.23).

Design Criteria of Embankment Dam

Embankment dams shall be designed to meet the following criteria:

- It shall be safe against excessive overtopping by wave action especially during high design flood flows.
- The embankment slopes shall be stable during all conditions of the reservoir operations, including rapid drawdown, if applicable.
- Seepage flow through the body of embankment dam, foundation and abutments shall be controlled so that no internal erosion (piping) takes place and there is no sloughing in areas where seepage emerges.
- The embankment dam shall not overstress the foundation.
- Slopes of the embankment dam shall be acceptably protected against erosion by wave action and from gullying as well as scour against surface runoff due to rain.
- The embankment dam, foundation, abutments and reservoir rim shall be stable and shall not develop unacceptable deformations under earthquake conditions.

Selection of Type of Embankment Dams

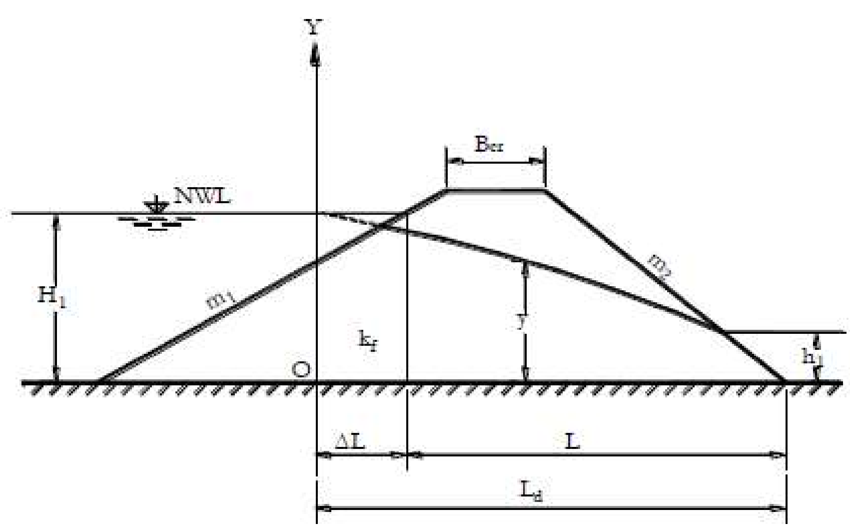
The type of embankment dam for a particular site shall be selected based on the following considerations:

- Engineering geological, hydrogeological and climatological conditions of the site.
- Local availability of construction materials.
- Availability of construction facilities and components of structures of the headworks.

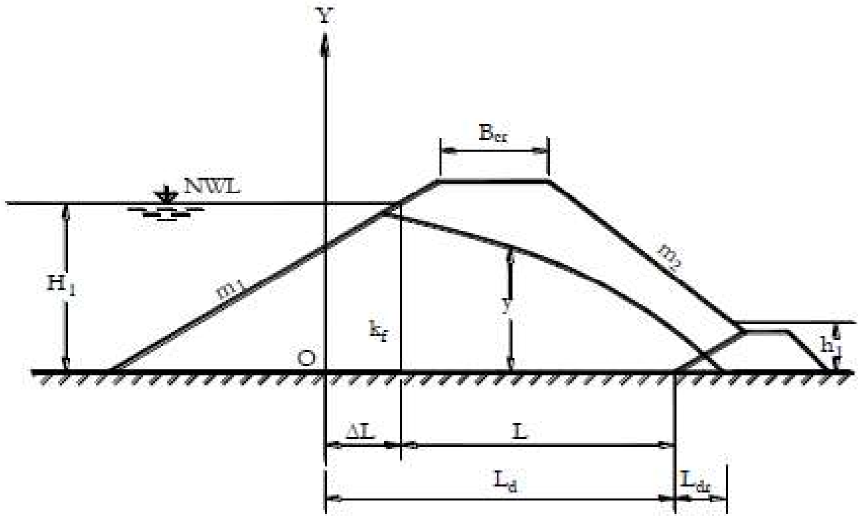
SEEPAGE FLOW DESIGN

1. Seepage Design of Dams on Impermeable Foundation

In recent years the widely used method of equivalent profile with simple calculations has been recommended for general use. In this method an adopted design sketch of the dam is replaced by the equivalent one in relation to seepage with vertical upstream slope, which lies at the distance ΔL from the vertical plane (line) drawn through the point of interaction of the water level with embankment slope. The magnitude ΔL is determined by the relationship (Zhurablov, 1975):



i. Homogenous dam without drain prism



ii. Homogenous dam with drain prism

Sketches of seepage calculation (Zhurablov, 1975)

$\Delta L = \beta H_1$ (1)

where, $\beta = m_1/(2m_1+1)$ (2)

Starting from the vertical plane, the phreatic curve is made and its part in approach to the upstream slope has to be corrected visually such that it could be perpendicular to the embankment slope and further be changed into the phreatic curve.

i) Homogenous Earthen Dam without Drain

Height of the seepage flow existing at the down stream slope is known as the leaking zone (Figure i) and its magnitude can be determined by the formula (Zhurablov, 1975):

$$h_1 = \frac{L_d}{m_2} - \sqrt{\left(\frac{L_d}{m_2}\right)^2 - H_1^2} \quad (3)$$

Specific seepage discharge is found by the formula:

$$q = k_f \frac{h_1}{m_2} \quad (4)$$

Taking initial coordinate at point O, the phreatic curve is drawn by Dupy's equation:

$$y^2 = H_1^2 - 2\frac{q}{k_f}x \qquad (5)$$

Giving the values of x from zero to $x = L_d - m_2h_1$, the phreatic curve is to be constructed by the help of the Eq. (3) and it has to be corrected by hand at the zone approaching to the upstream slope, as above explained.

ii) Homogenous Earthen Dam with Drain

In this case the seepage equation will be:

$$q = \frac{k_f H_1^2}{2(L_d + l_{dr})} \qquad (6)$$

where $L_d = L + \Delta L$ is design seepage length, m and l_{dr} is drain length.

If in the Eq. (6) length of the drain l_{dr} is to be neglected because of its very small value in the comparison with L_d, it will turn into:

$$q = \frac{k_f H_1^2}{2L_d} \qquad (7)$$

Ordinate of the phreatic curve at the beginning of drain becomes

$$h_1 = q/k_f \qquad (8)$$

From the initial co-ordinate at point O, phreatic curve has to be drawn by Eq. (3). For x=0, ordinate will be H_1 and for $x = L_d$ it equals to be h_1. For $x=L_d+l_{dr}$ ordinate will be zero and distance from the starting of the drain up to this point is determined by the relationship (Zhurablov, 1975):

$$l_{dr} = 0.5\frac{q}{k_f} \qquad (9)$$

iii) Earthen Dam with Central Core Wall

For the seepage calculation of such dams the method of virtual length is usually applied, for which a central core having mean thickness δ_m and permeability coefficient k_c is replaced by the prism with permeability coefficient k_f. Virtual length of the central core is to be determined by the expression:

$$L_c = \delta_m \frac{k_f}{k_c} \qquad (10)$$

After such replacement, calculation is done in the same way as for homogenous earthen dam without drain or with drain depending upon the achieved calculation sketches. Phreatic curve has to be drawn only in the zone of dam before and after the central core.

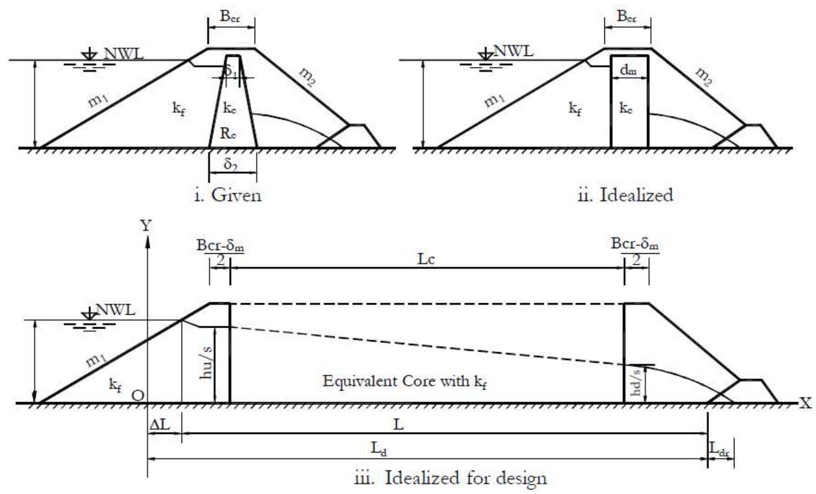


Figure: Sketches for seepage calculation with central core (Nayak, 1993)

iv) Earthen Dam with Inclined Core

Seepage calculation of such dams can be done by the different methods. One of them is the method of virtual length which is based on the replacement of inclined core having mean thickness δ_m and permeability coefficient k_c of equivalent prism with permeability coefficient k_f and its horizontal length:

$$L'_c = \delta_m \frac{k_f}{k_c} \sin\theta \qquad (11)$$

where θ is the angle formed between inclined core and horizontal line of the foundation in degree.

Thus, to the adopted sketch solution is applied for homogenous earthen dam with drain or without drain depending upon the provided structure of the dam. Losses of head in the range of loaded layer of the inclined core are neglected.

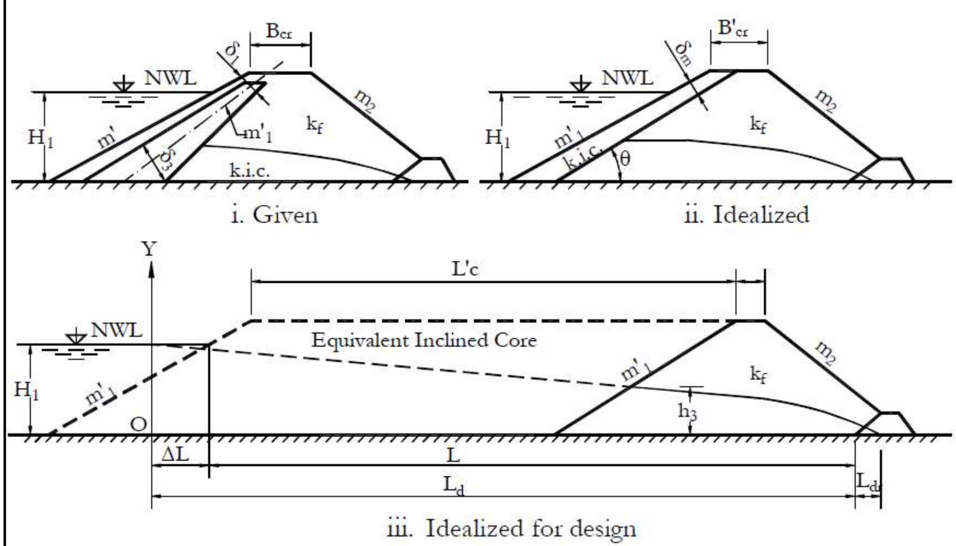


Figure: Sketches for seepage calculation of dam with inclined core (Nayak, 1993)

2. Seepage Design of Dams on Permeable Foundations

i) Earthen Dam with Screen, Apron and Drain

In the calculation of these dams the losses of head in the protected layer are not considered. Screen and apron are taken into account to be impervious. Slope of the screen is to be adopted along its mean line and fall of head along the length of apron is taken to be straight (linear).

Seepage equation for the different values of permeability coefficient of the dam body k_f and foundation k_0 takes the form (Zhurablov, 1975):

$$k_0 T \frac{H_1 - h_3}{n(L_a + m'_1 h_3)} = \frac{h_3}{L}\left(k_0 T + k_f \frac{h_3}{2}\right) \qquad (12)$$

Geometrical dimensions entering in the Eq. 12 are shown in Figure below. Here n is coefficient considering the length of seepage flow due to curvature: n = 1.15 for L/T=20; 1.18 for 5; 1.23 for 3; 1.44 for 2 and 1.87 for 1.

$$k_0 T \frac{H_1 - h_3}{n(L_a + m'_1 h_3)} = \frac{h_3}{L}\left(k_0 T + k_f \frac{h_3}{2}\right)$$

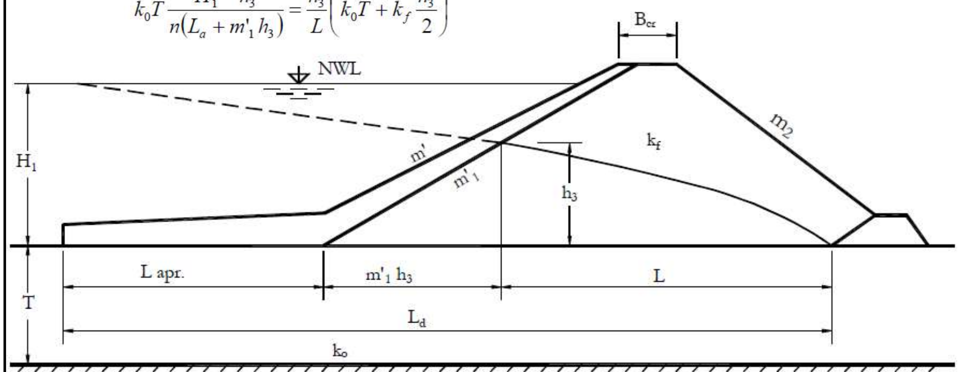


Figure: Sketches for seepage calculation of dam with screen and apron (Zhurablov, 1975)

Such equation is easily to be solved graphically, for which the curves for L.H.S. and R.H.S. parts of the equation are drawn for the different values of h3. The point of intersection of the curves gives actual root of this equation. Depression curve in the body of dam with drain is to be drawn by the use of Eq. (3), replacing H_1 by h_3 in it.

Homogenous Dams without Drain

Here is possibility that permeability coefficient of the dam body k_f and its foundation k_o may be the same or different. In the approximate method, the design of seepage flow shall be conducted with two independent assumptions. Initially, it is considered that the dam body is permeable and its foundation – impermeable, and for this case specific seepage discharge q_1 shall be found with construction of the phreatic curve in it. Then other case will be considered, in which the dam body shall be taken into account the impermeable and its foundation – permeable, and specific seepage discharge q_2 will be determined by the formula (Zhurablov, 1975):

$$q_2 = k_0 T \frac{H_1 - h_1}{nL} \qquad (13)$$

where T is the depth of the permeable foundation, L is the width of the dam, n is the numerical coefficient considering the length of seepage flow due to curvature to be adopted depending on the ratio L/T (see Eq. 10).

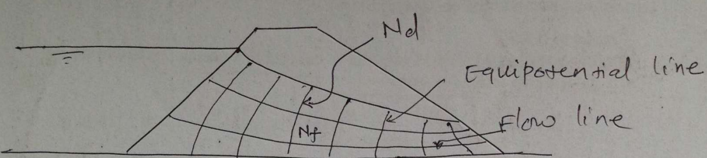
The total seepage flow discharge shall be obtained as the sum of discharges through the dam body and its foundation:

$$q = q_1 + q_2 \qquad (14)$$

It is essential to keep it in mind that in this approximate method, location of the phreatic curve shall be little bit higher and more higher with enough difference in the permeability coefficient of the dam foundation and its body for the design case with $k_0 < k_f$.

Seepage through earthen dam:

1. Flownet method (Graphical method)



The flownet

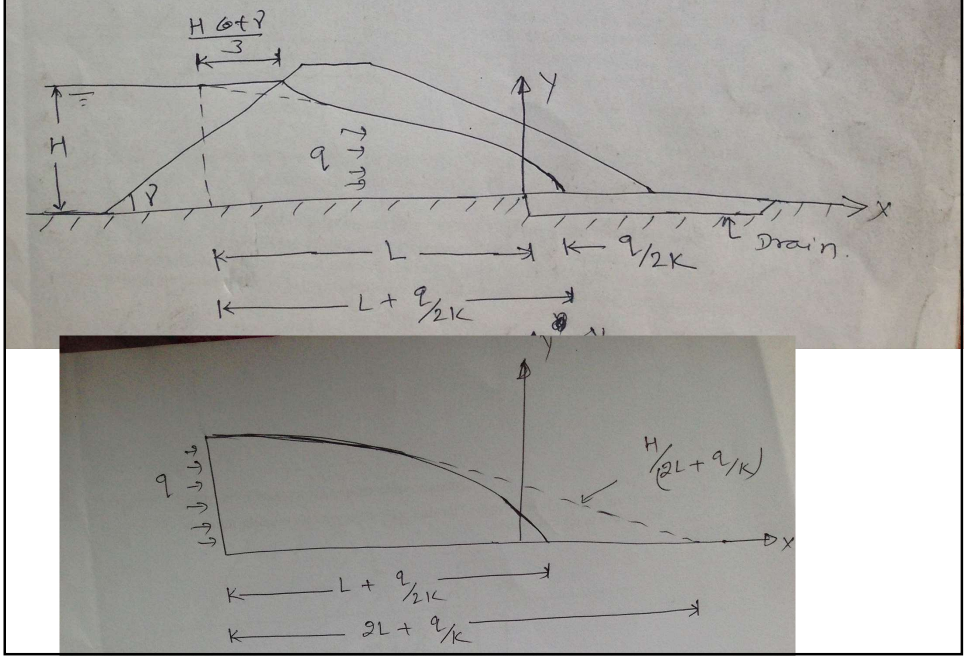
The distance between flow lines is made equal to that between equipotential lines, so that they form a set of approximately curvilinear squares.

If N_f = No. of flow channels
N_d = No. of ~~equi~~ potential drops
H = Total head causing flow
and K = Coefficient of permeability

Then the seepage discharge through the flownet per unit width,

$$q = KH\frac{N_f}{N_d}$$

2. Analytical method



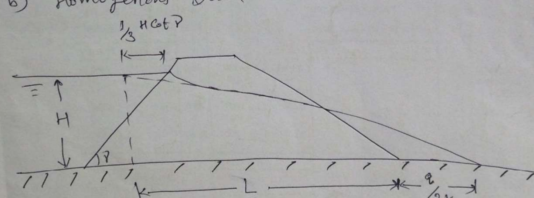
$$q = K H \frac{H}{(2L + q/K)}$$

or, $\boxed{q = \frac{K H^2}{2L}}$ Dupit's Formula [For $q/K \approx 0$]

a) Homogeneous Dam with Filter

$$q = \frac{K H^2}{2L + q/K} \approx \frac{K H^2}{2L}$$

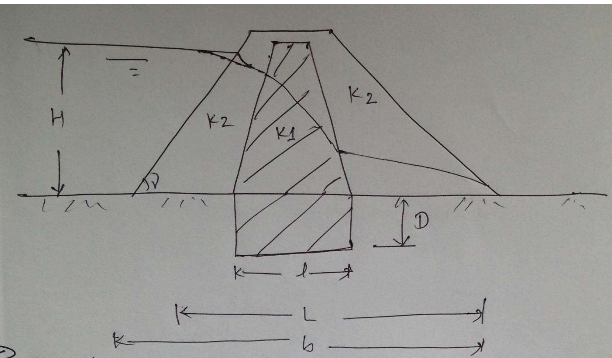
b) Homogenous Dam without Filter



c) Dam with core material

$$L' = L + \left(\frac{K_2}{K_1}\right) l$$

$$q = q_1 + q_2 = \frac{K_2 H^2}{2L'} + \frac{K_2 H D}{L'}$$



For homogeneous foundation

$$q_f = k_f \frac{H}{b} D$$

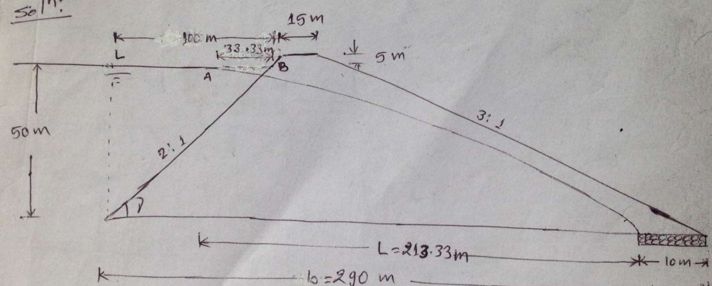
For foundation,

$$q_f = K_2 \frac{H}{L'} D$$

OS

* The U/S and D/S slopes of a homogenous earthen dam with 10 m long toe drain are 2:1 and 3:1 (H:V) respectively. The water depth U/S of the dam is 50 m. The dam has a crest width of 15 m and freeboard of 5.0 m. The coefficient of permeability, K for the fill material is 2cm/hr. Calculate the specific discharge passing through the dam.

Soln:



$LB = H \cot\theta = 50 * \frac{2}{1} = 100 \text{ m}$

$AB = \frac{1}{3} H \cot\theta = \frac{1}{3} * 100 = 33.33 \text{ m}$

Base width, $b = 2(H+FB) + 15 + 3(H+FB)$

$= 2*55 + 15 + 3*55 = 290 \text{ m}$

$L = b - 10 - \frac{2}{3} H \cot\gamma = 290 - 10 - \frac{2}{3} * 100 = 213.33 m$

$\therefore$ Specific discharge passing through the dam,

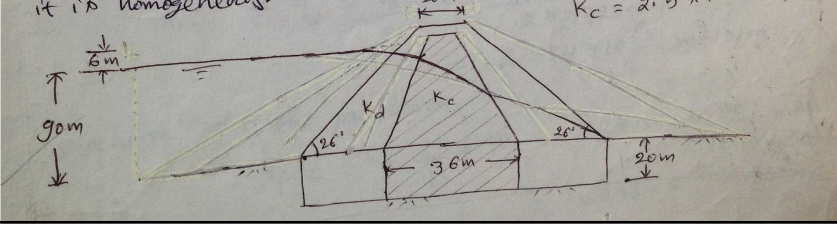
$$q = \frac{KH^2}{2L} \qquad \left[K = 2 cm/hr = 5.56 * 10^{-6} m/s \right]$$

$$= \frac{5.56 * 10^{-6} * 50^2}{2 * 213.33} = 3.258 * 10^{-5} \text{ cumecs/m}$$

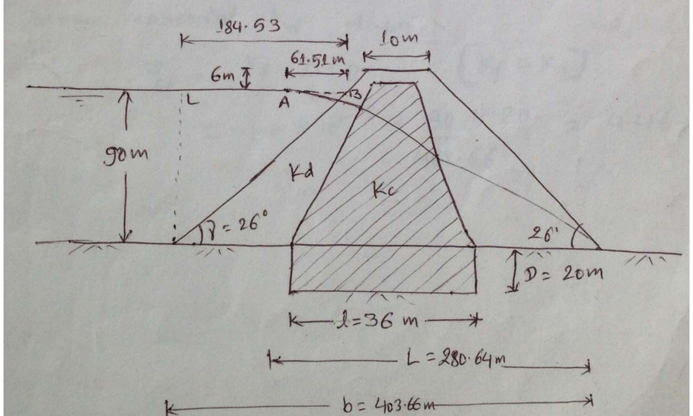
※ Find out the discharge passing through the dam body and foundation of an embankment dam as shown below. Also find out the discharge passing through the ~~foun~~ foundation if it is homogeneous.

$K_d = 1.0 * 10^{-5}$ m/s

$K_c = 2.5 * 10^{-7}$ m/s



Sol^n:



$LB = H \cot\gamma = 90 * \cot 26° = 184.53$ m

$AB = \frac{1}{3} H \cot\gamma = \frac{1}{3} * 90 * \cot 26° = 61.51$ m

Base width, $b = (H + FB) \cot 26° + 10 + (H + FB) \cot 26°$

$= 96 \cot 26° + 10 + 96 \cot 26° = 403.66$ m

$L = b - \frac{2}{3} H \cot\gamma = 403.66 - \frac{2}{3} * 90 \cot 26° = 280.64\,m$

$\therefore L' = L + \left(\frac{K_d}{K_c}\right) l = 280.64 + \left(\frac{1.0 * 10^{-5}}{2.5 * 10^{-7}}\right) * 36$

$= 1720.64\,m$

Discharge passing through the dam body,

$$q_d = \frac{K_d H^2}{2L'} = \frac{1.0 * 10^{-5} * 90^2}{2 * 1720.64}$$

$$= 2.354 * 10^{-5}\ m^3/s/m$$

Discharge through foundation,

$$q_f = K_d \frac{H}{L'} * D = 1.0 * 10^{-5} * \frac{90}{1720.64} * 20$$

$$= 1.046 * 10^{-5}\ m^3/s/m$$

If there is cutoff on the foundation, fill with same material K_d, then

$$q_f = K_f \frac{H}{b} D \qquad [K_f = K_d]$$

$$= 1.0 * 10^{-5} * \frac{90}{403.66} * 20 = 4.46 * 10^{-5}\ m^3/s/m$$

20.8. Design Criteria for Earth Dams

(1) A fill of sufficiently low permeability should be developed out of the available materials, so as to best serve the intended purpose with minimum cost. Borrow pits should be as close to the dam site as possible, so as to reduce the leads.

(2) Sufficient spillway and outlets capacities should be provided so as to avoid the possibility of overtopping during design flood.

(3) Sufficient freeboard must be provided for wind set-up, wave action, frost action and earthquake motions.

(4) The seepage line (*i.e.* phreatic line) should remain well within the downstream face of the dam, so that no sloughing of the face occurs.

(5) There is little harm in seepage through a flood control dam. if the stability of foundations and embankments is not impaired, by piping, sloughing, etc. : but a conservation dam must be as watertight as possible.

(6) There should be no possibility of free flow of water from the upstream to the downstream face.

(7) The upstream face should be properly protected against wave action, and the downstream face against rains and against waves upto tail water. Provisions of horizontal berms at suitable intervals in the d/s face may be thought of, so as to reduce the erosion due to flow of rain water. Ripraps should be provided on the entire u/s slope and also on the d/s slope near the toe and up to slightly above the tail water so as to avoid erosion.

(8) The portion of the dam, downstream of the impervious core, should be properly drained by providing suitable horizontal filter drain, or toe drain, or chimney drain, etc.

(9) The upstream and downstream slopes should be so designed as to be stable under worst conditions of loading. These critical conditions occur for the u/s slope during sudden drawdown of the reservoir, and for the d/s slope during steady seepage under full reservoir.

(10) The u/s and d/s slope should be flat enough, as to provide sufficient base width at the foundation level, such that the maximum shear stress developed remains well below the corresponding maximum shear strength of the soil, so as to provide a suitable factor of safety.

(11) We know that the consolidation of the soil does not take place instantaneously when the compaction is done by external leadings. It takes place slowly as the excess pore water goes out and the load is transferred to the soil grains. In coarse gravels, the void openings are large enough so as to permit rapid escape of confined water and air, and full compaction may occur before the construction is over. But in fine grained impervious soils, the consolidation is slow. It, therefore, becomes necessary in such cases, as to provide an additional height of the fill. After consolidation, the the embankment will be of the desired height. Hence, a suitable allowance in the height of embankment (between 2 to 3% of dam height, determined by laboratory tests) must be made in fine grained soils so as to account for the consolidation that may take place upto years after construction. Dewatering the foundations may sometimes be used to accelerate the process of consolidation.

(12) Since the stability of the embankment and foundation is very critical during construction or even after construction (*i.e.* during the period of consolidation), due to development of excessive pore pressures and consequent reduction in shear strength of soil, the embankment slopes must remain safe under this critical condition also.

All the above criteria must be satisfied and accounted for, in order to obtain a safe design and construction of an earth dam.

20.9. Selecting a Suitable Preliminary Section for an Earth Dam

The preliminary design of an earth dam is done on the basis of existing dams of similar characteristics and the design is finalised by checking the adequacy of the selected section for the worst loading conditions. Empirical rules are frequently used in these designs.

A few recommendations, for selecting suitable values of top width, free board, u/s and d/s slopes, drainage arrangements, etc. are given below for preliminary designs :

Freeboard. Freeboard or minimum freeboard is the vertical distance between the maximum reservoir level and the top of the dam (*i.e.* the crown or crest of dam). The vertical distance between normal pool level or spillway crest and the top of the dam is termed as normal freeboard.

The minimum height of the freeboard for wave action is generally taken to be equal to 1.5 h_w, where h_w is given by the equations (19.11) and (19.12). Most of the hydraulic failures of earth dams have occurred due to overtopping of dams. Hence, the freeboard must be sufficient enough, as to avoid any such possibility of overtopping. Values of freeboard, for various heights, recommended by U.S.B.R. are given in table 20.1.

EARTHERN DAMS AND ROCK...

Table 20.1. U.S.B.R. Recommendations for Freeboard for Earth Dams

Spillway Type	*Height of Dam*	*Minimum freeboard Over MWL*
Uncontrolled (*i.e.* Free) Spillway	Any height	Between 2 m to 3 m
Controlled spillway	Height less than 60 m	2.5 m above top of gates
Controlled spillway	Height more than 60 m	3 m above top of gates

An additional freeboard upto 1.5 m should be provided for dams situated in areas of low temperatures for frost action.

Width. The top width of large earthen dams should be sufficient to keep the seepage line well within the dam, when reservoir is full. It should also be sufficient to withstand earthquake shocks and wave action. For small dams, this top width is generally governed by minimum roadway width requirements.

The top width (A) of the earth dam can be selected as per the following recommendations :

$$A = \frac{H}{5} + 3 \text{ for very low dams} \quad \ldots(20.11)$$

$$A = 0.55\sqrt{H} + 0.2\,H \text{ for dams lower than 30 m} \quad \ldots(20.12)$$

$$A = 1.65\,(H + 1.5)^{1/3} \text{ for dams higher than 30 m} \quad \ldots(20.13)$$

where H is the height of the dam.

Upstream and Downstream slopes. The side slopes depend upon various factors such as the type and nature of the dam, and foundation materials, height of dam, etc. etc. The recommended values of side slopes as given by Terzaghi are tabulated in Table 20.2.

Table 20.2. Terzaghi's Side Slopes for Earth Dams

Type of Material	*U/S slope (H : V)*	*D/S slope (H : V)*
Homogeneous well graded	2.5 : 1	2 : 1
Homogeneous course silt	3 : 1	2.5 : 1
Homogeneous silty clay		
(*i*) Height less than 15 m	2.5 : 1	2 : 1
(*ii*) Height more than 15 m	3 : 1	2.5 : 1
Sand or Sand and gravel with a central clay core	3 : 1	2.5 : 1
Sand or Sand and gravel with R.C. diaphragm	2.5 : 1	2 : 1

The various dimensions of low earth dams for their preliminary sections, may sometimes be selected from the recommendations of Strange, as given in Table 20.3.

Table 20.3. Preliminary Dimensions of Low Earth Dams (Strange's recommendations)

Height of dam in metres	*Maximum freeboard in metres*	*Top width (A) in metres*	*U/S slope (H : V)*	*D/S slope (H : V)*
Up to 4.5	1.2 to 1.5	1.85	2 : 1	1.5 : 1
4.5 to 7.5	1.5 to 1.8	1.85	2.5 : 1	1.75 : 1
7.5 to 15	1.85	2.5	3 : 1	2 : 1
15 to 22.5	2.1	3.0	3 : 1	2 : 1

General Arrangement of RoR Headworks

Existing headworks of Kali Gandaki A HPP during wet season operation

The salient features of the power plant are given below:

Power plant type	:	Peaking-run-of-river
Installed capacity	:	144 MW
Annual average energy	:	842 GWh
Maximum gross head/net head:		130m / 115m
Design flow	:	141 m3/s
Catchment area	:	7,618km2 (Kaligandaki River) and 476km2 (Andhikhola River)
Intake	:	6 Nos. 10m x 10.63m (W x H) each
Spillway	:	3 radial gates of 15m x 19m (W x H) each
Settling basins	:	2 Nos. 187m x 80m x 14m (L x B x H) each, gravity flushing type
Headrace tunnel	:	5905m
Penstock	:	1 Nos., 243m long, Ø 5.25m, inclined steel lined

Turbine	:	3 Francis
		Rated discharge - 44.86 m3/s
		Rated output - 48MW
		Rated speed - 300 rpm
Generator	:	Rated output - 56.5MVA
		Rated voltage - 13.8kV
		Rated frequency - 50Hz
Power factor	:	0.85
Start of project construction	:	1997
Commercial operation	:	16 August, 2002

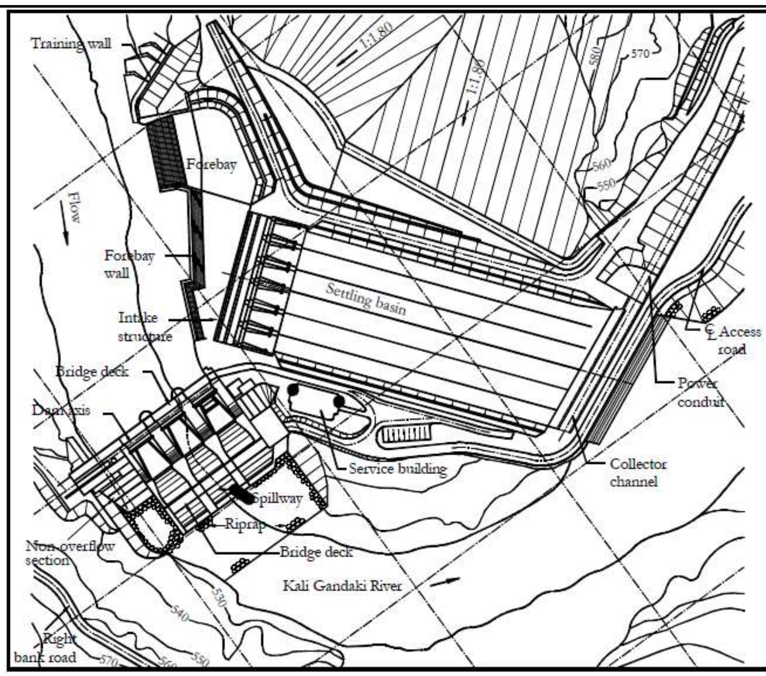


Figure 1: Headworks of Kali Gandaki "A" Hydroelectric Project, Nepal (Nepal Electricity Authority 2002)

Diversion Structure

The diversion structure shall generally be planned on a straight axis, but it may also be slightly curved if the upstream curvature locates that part of the diversion structure on higher bedrock foundation and thereby adds to economy and safety. As far as possible, the structure shall be aligned perpendicular to the river course to minimize its length and to ensure normal and uniform flow through its bays.

The overall length of the diversion structure shall be fixed based on the site topography and hydraulic requirements. Its crest level shall be determined considering the head needed to pass the available flow, less the environmental release, during the dry season.

Spillway

The spillway may be provided as an integral part of the diversion structure or as a separate structure. In concrete diversion structures, the spillway shall generally be located centrally, away from both abutments where erosion damage needs to be avoided. In embankments, the spillway shall be located on one side of the embankment at an erosion-resistant site.

Intake

To minimize sediment entry into the water conveyance system, the intake shall normally be aligned at an angle of 90º to 110º to the axis of the diversion structure. A bed-load sluice may be located below the intake to separate out smaller stones and gravel which may follow the abstracted flow.

Undersluice

The undersluice shall be provided close to the intake to flush out the sediments deposited in front of the intake and thus control the bed levels in its approach area. The opening of the undersluice shall be sized to pass the largest possible boulders brought along by the river. Based on the flow to be handled, the undersluice may consist of one or more bays; however, for small rivers flows, a single sluice bay may be preferred over two bays with smaller openings. Its crest and upstream floor levels shall generally be kept at the lowest bed level of the deep channel of the river, subject to the cost of foundation and the difficulty in dewatering.

Gravel Trap

In certain cases, a gravel trap may be provided in front of the intake below its invert level. It shall be proportioned based on the debris content and the size of the gravels present in the river water.

Settling Basin

Depending on the availability of sufficient space and flushing head, the settling basin may be integrated with the intake or be connected to it through an approach channel/conduit. It shall be provided with flushing channels and control arrangements for sediment sluicing. It may also be provided with side or end spillways to spill the excess water in it back to the river.

Depending on site conditions and project requirements, the settling basin may be a singlechamber or a multi-chamber basin. Considering the heavy silt conditions in Nepali rivers, a minimum of two chambers shall be considered.

Divide Walls

A divide wall shall normally be constructed to separate the undersluice bays from the other bays of the diversion structure. Divide walls shall be positioned at right angles to the axis of the diversion structure. On the upstream side, the divide wall shall extend from two-third to the full width of the diversion structure to obtain a pocket of comparatively still flow at the intake for sediment deposition.

On the downstream side, the wall shall generally extend to the end of impervious floor to ensure adequate tail water depth for jump formation and to prevent cross flows that could cause objectionable scours.

Fish Pass Structures

Fish ladders shall be located in areas with high flow of water. To increase their effectiveness, attraction flows or leaders shall be arranged to guide the fish to the fish ladder or the fish bypass system.

Intake:

It is a structure constructed at the entrance of the canals or tunnels or pipes through which water is conveyed to the power plant.

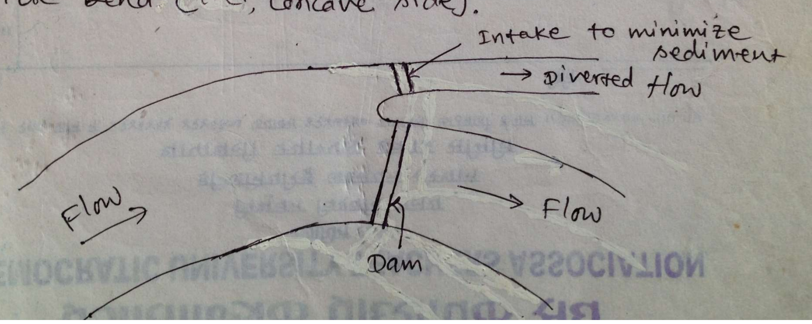
Functions / Importance:

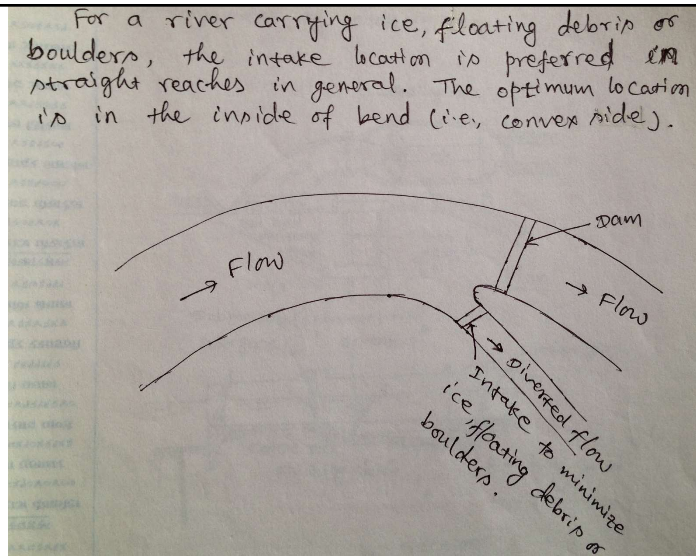
- Supply of desired amount of flow to headrace conveyance system (Power canal / Pressure Tunnel).
- Prevention of bed load intrusion into the conveyance system.
- Smooth, easy and turbulance free entry of water into the conveyance passages.
- Deflection and flusing of bed load accumulated in front of the weir.
- Protection of the conveyance system from floating debris, ice, etc. if any carried by the river.

Location:

Intakes may be located on the straight as well as curved reaches of the river. Intakes located on the straight reaches are usually economically attractive.

For a river carrying heavy sediment loads, straight reaches of river are not desireable because sediment is likely to accumulate. The preferred intake location is in the downstream portion of the outside bend (i.e., concave side).





The main parts of an intake are:

1) the inlet section including the sill and coarse rack,
2) the inlet gate and transition section,
3) the settling basin and sand flushing canal.

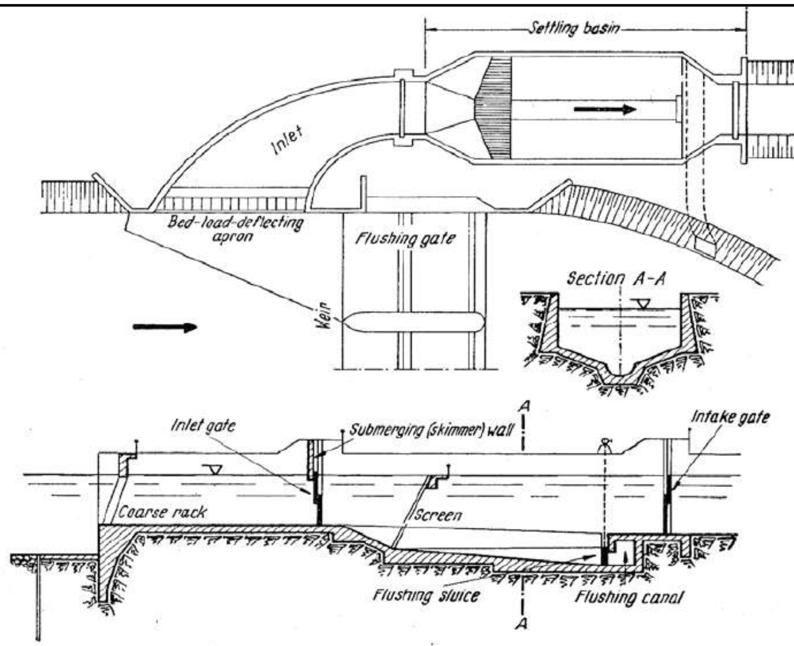


Figure. General arrangement of intake

Basic considerations during the design of good intake:

1. Hydraulic and structural consideration:
   - Vortex formation
   - sediment exclusion
   - Head loss consideration
   - Structural stability
   - strength to carry out various operations above it.
2. Operational consideration:
   - Diversion of required quantity of flow with the gate operation
   - Trash handling
   - Maintenance provision.
3. Environmental consideration
   - Fish diversion systems.

Types of intakes:

1. Based on hydraulic regime:

i) Non-pressure intake or surface intake:

Water from the rivers or reservoirs passes into the canal at the normal water level in them.

- Broad crested weir → $2B \geq H$
- Narrow crested weir → $2B < H$

These intakes are usually adopted on the parts of river having small banks, when the arrangement of deep excavation for canals is uneffective. They are also used on the unstable banks of river.

ii) Pressure intake or sub-surface intake:
- orifice type

These intakes are usually used on the hilly parts of rivers characterised by the following geological conditions:

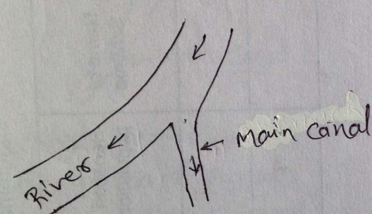
- Quick attack of the short time flood due to heavy rainfall.
- High flow velocities allowing to transport enough quantity of sediments
- Enough duration of flood with exceedence of the flood discharge 20 – 30 times with respect to the mean flow discharge.
- occurance of the mudy flows attacking suddenly.

These intakes make only few disterbance for their natural hydraulic and hydrological conditions.

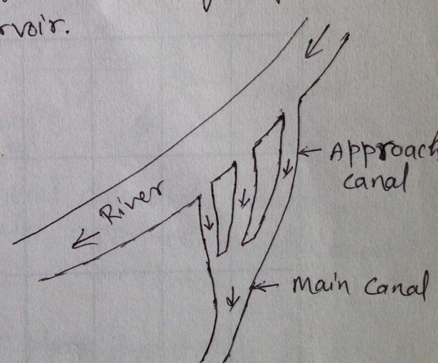
2. Based on control system:

i) Uncontrolled intake (or Intake without regulator):

In the uncontrolled intakes the water level in the main canal synchronously follows the change of water level in the river or reservoir.



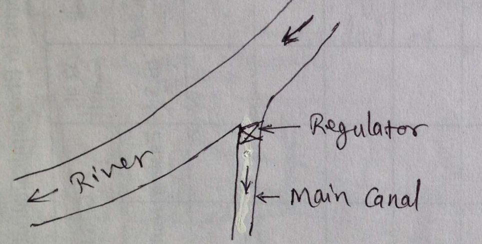
uncontrolled intake



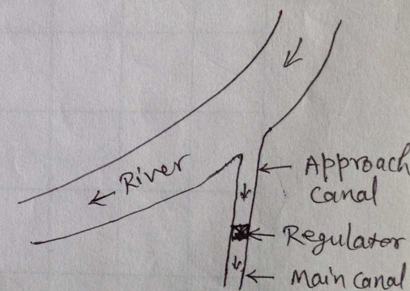
Uncontrolled intake with multi-approach canals

ii) Controlled intake (or Intake with regulator)

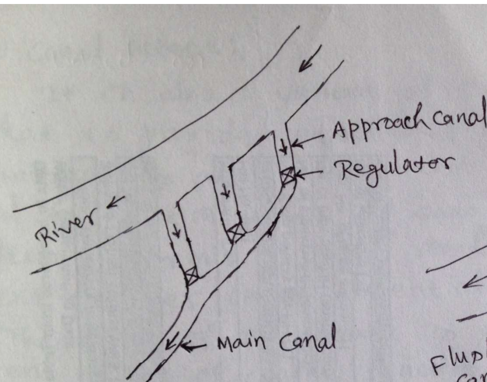
In the controlled intakes there is used head-regulator, with the help of which the flow of water is to be guaranteed in the main canal.



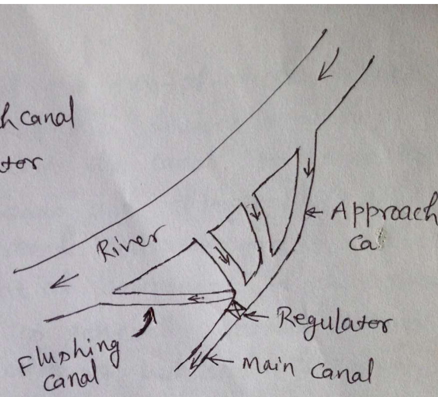
Controlled intake at river bank



Controlled intake apart from river bank



Controlled intake with multi approach canals apart from river bank

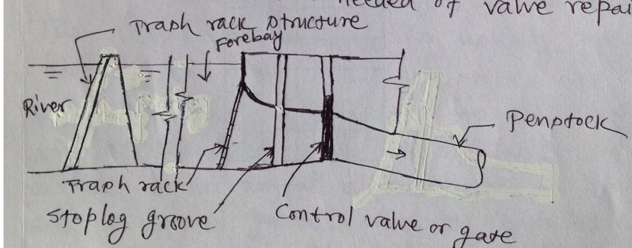


Central controlled intake with multi approach canals apart from river bank

3. Based on type of power plant layout:

i) Run-of-river intake:

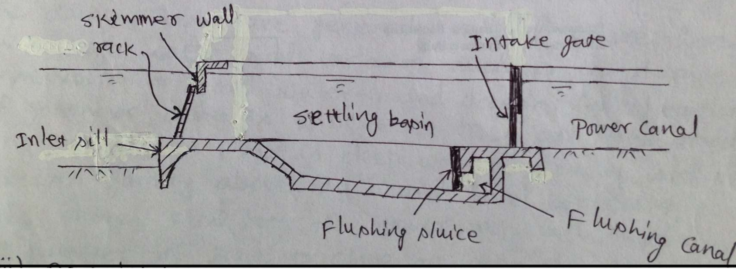
This is a low head intake provided for run-of-river plants and form an integral part of the power house. There is a bell-mouth entrance which is gaurded by an RCC or steel grid, forming the trashracks. The control valve or gate is situated immediately after the bell-mouthing. Stoplog groove is provided ahead of the valve for the needed of valve repairs.



In silty rivers special arrangement for desilting are usually provided.

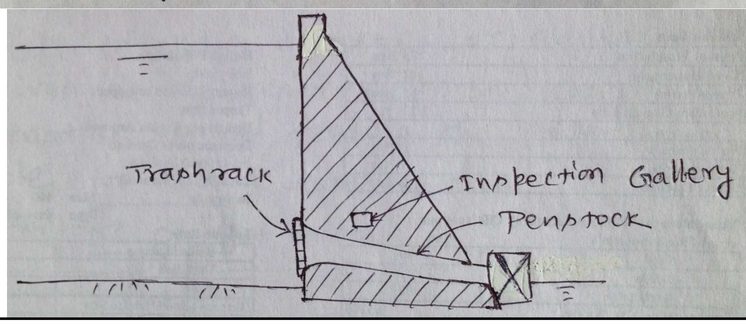
ii) Canal intake:

It is also a variant of the run-of-river intake, that is provided adjacent to the diversion weir/barrage to admit water into the canal. It is designed to function under low heads and topography and geology permit straight reach suitable for it. Sediment excluder is an essential component of the intake. The crest of the intake is generally raised to prevent entry of coarse fraction of bed load. Trash-rack is provided on the upstream of intake. In certain locations disilting tunnels/canal may also have to be provided upstream of the intake.



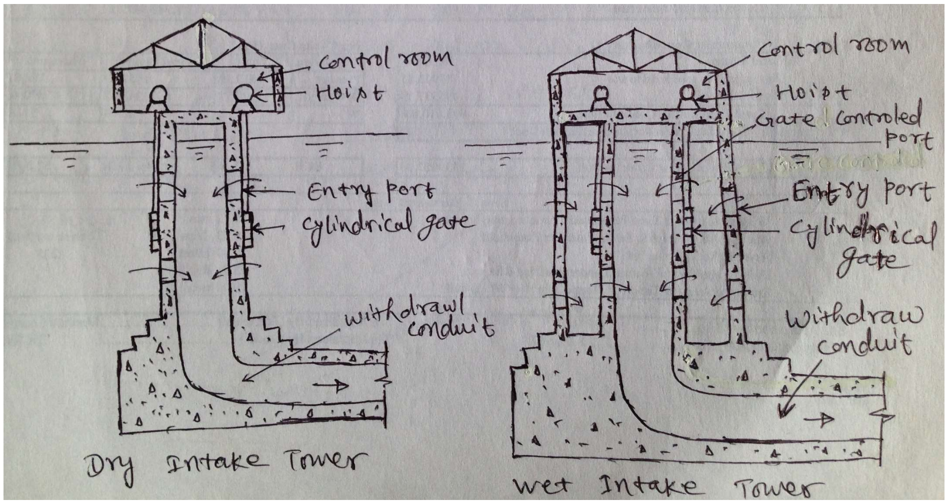
iii) Dam intake :

It is designed basically for high head power plants. The intake structure is usually provided in the body of the dam. The penstocks are embeded in the dam and feed water to the power house at the toe of the dam. A bell mouth inlet, which may have either a horizontal alignment or an inclined alignment and a control gate, that is installed either at the entrance or after the bell mouth section.



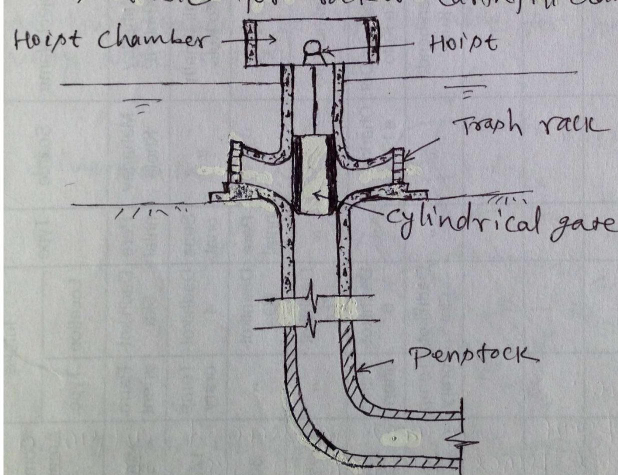
iv) Tower Intake :

Tower intakes are normally on hill sides, not far off from the dam, when it is not convenient to provide the simple intake directly on the upstream face of the dam. These are generally utilised where there is a large water level variation such as in storage reservoirs or tidal waters, and access for operation of gates or valves is essential. They are free standing structures, set in deep water, usually with an access gantry above top water level connecting to the shore. Flow into the tower may be controlled by a number of gates to close or open the gates at various levels.



v) Shaft Intake:

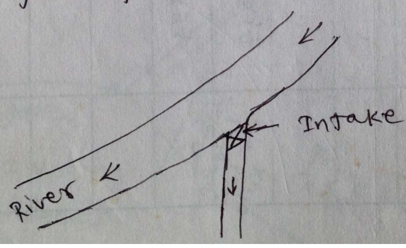
In shaft intake, a vertical or a near-vertical shaft, driven at the reservoir site, carries water to the penstock tunnel feeding the power house. It is suitable for rock or earthfill dam.



4. Based on river merphology and hydraulics (flow of river) :

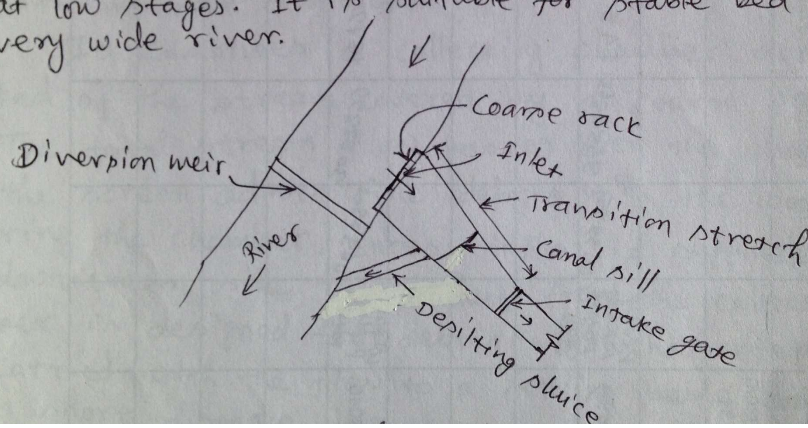
i) Bank Intake :

These are structures located on a river bank, side of reservoir or a coastal site. They are generally adopted for locations where only a small portion of flow passing the intake is to be abstracted and where fluctuations in water level are not large. The face of the intake is aligned with the bank.



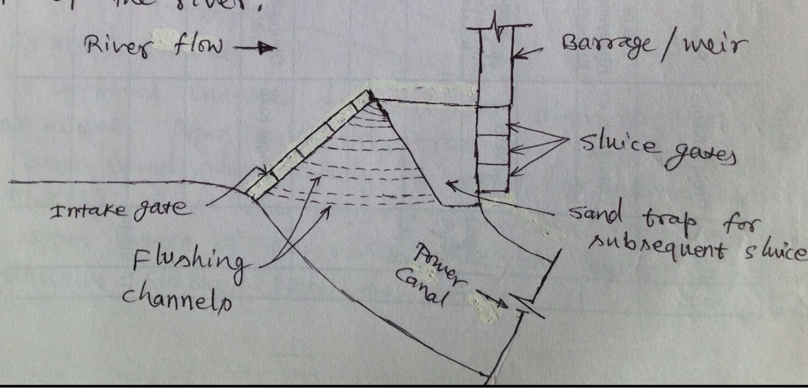
ii) Side Intake :

For rivers and streams where a substantial portion of the flow is to be diverted, a cross weir of some sort is an essential feature to ensure available water is not lost to the intake at low stages. It is suitable for stable bed and very wide river.



iii) Diversion type (weir / Barrage type) Intake:

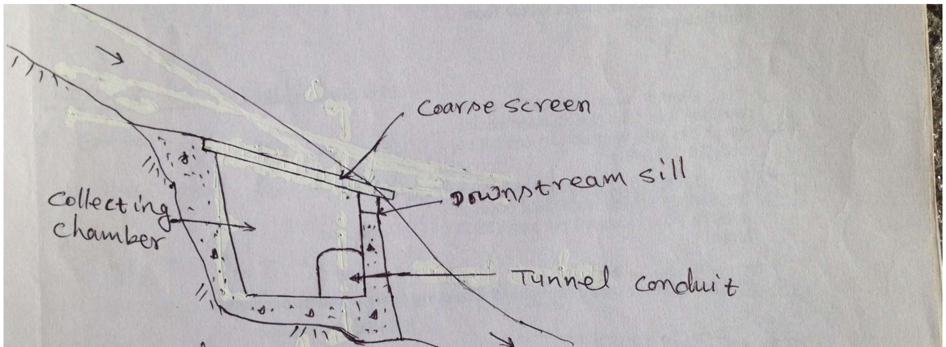
Intake to the power canal has a series of undersluices set in the face of the intake sill, which extract the lowest layer of flow with the highest concentration and coarsest sediment and return it continously to downstream of the river control structure. It is suitable for mild to flat slope of the river.



iv) Bottom / Drop / Tyrolean Intake:

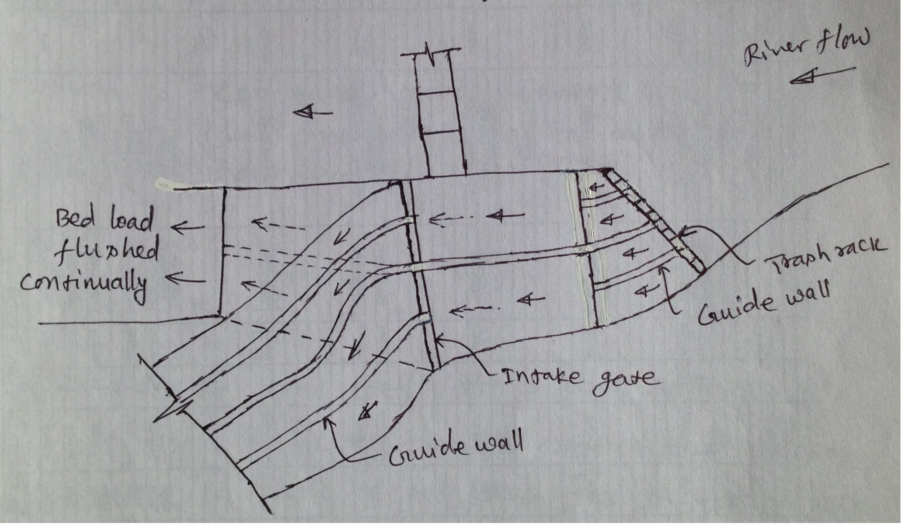
Bottom intakes have been developed for glaciers and mountain torrents, where site conditions may be extremely difficult for access and construction, and where boulders and rock debris have to be passed with minimum obstruction.

It comprises a collecting chamber across the bed of the stream covered by a coarse screen. The total stream flow passes over the chamber, and the screen admits fine debris with the water entering the chamber. Excess inflow is supplied at the downstream sill. The conduit from the collecting chamber is designed for debris which has entered to be carried with the flow to a settling basin constructed at a short distance downstream.



v) Frontal Intake:

A frontal intake designed for abstraction of clear water from mountain streams. This design has been used successfully in major systems mainly in Turkey. Since the abstracted water is taken from the upper layers of the river while the lower layer is continually flushed past the intake, it is particularly applicable where the majority of sediment carried by the river is bed load, and where large proportion of the flow continues down the original water course.

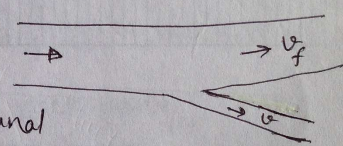


Losses in intakes:

1. Entrance losses:

i) The loss due to change in direction of flow.

$$h_{l_1} = \frac{v^2}{2g} - c\frac{v_f^2}{2g}$$



where, v = velocity in the canal

v_f = velocity of flow in the main stream

and c = constant depending on the angle of diversion = 0.8 to 0.4. According to Mosonyi higher values for intakes with lower offtake angles and lower values for near rectangular diversions.

ii) The loss due to sudden contraction of area at the diversion.

$$h_{l_2} = K\frac{v^2}{2g}$$

where, K = constant varies between 0.03 for rounded (bell-mouthed) entry to about 1.3 for sharp entry.

2. Rack losses:

i) $h_{lr} = K_t \frac{v^2}{2g}$

where, K_t = loss coefficient = $1.45 - 0.45R - R^2$

v = velocity through contracted opening

and R = ratio of net area through trash rack bars to gross area of racks and supports.

ii) Kirschmer's formula:

$$h_{fr} = K_r \left(\frac{t}{b}\right)^{4/3} \cdot \frac{V_b^2}{2g} \sin\phi$$

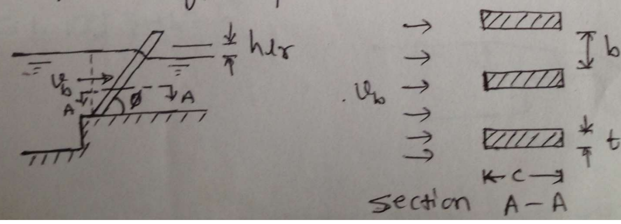
where, K_r = a factor depending on the cross-section of bars

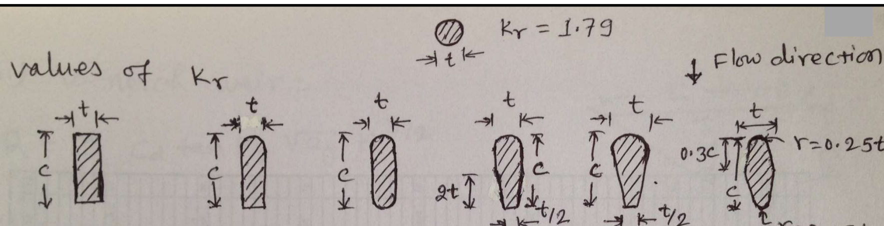
t = thickness of rack bars

b = spacing between bars

V_b = velocity of flow in front of bars/rack

ϕ = angle of bars with the horizontal.





$K_r = 2.42$ $K_r = 1.83$ $K_r = 1.67$ $K_r = 1.035$ $K_r = 0.92$ $K_r = 0.76$

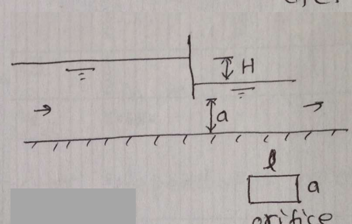
3. Other losses :→ Head gate loss, Fine rack loss, Bend (if any), etc.

Design of Intake:

1. Hydraulic Design
2. Structural Design

1. Hydraulic Design:

a) Submerged flow:



$Q = C_d \cdot A \sqrt{2gH}$

where, A = area of orifice
$= a * l$

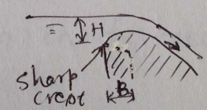
C_d = Coefficient of discharge

b) Surface flow:

i) Narrow crested weir → $2B < H$

$$Q = \frac{2}{3} C_d L \sqrt{2g} \, H^{3/2}$$

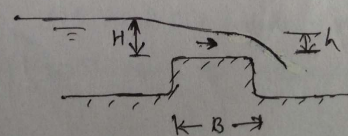
where, L = length of weir



Ogee weir

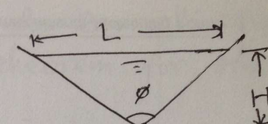
ii) Broad crested weir → $2B \geq H$

$$Q = C_d L \sqrt{2g} \sqrt{H h^2 - h^3}$$

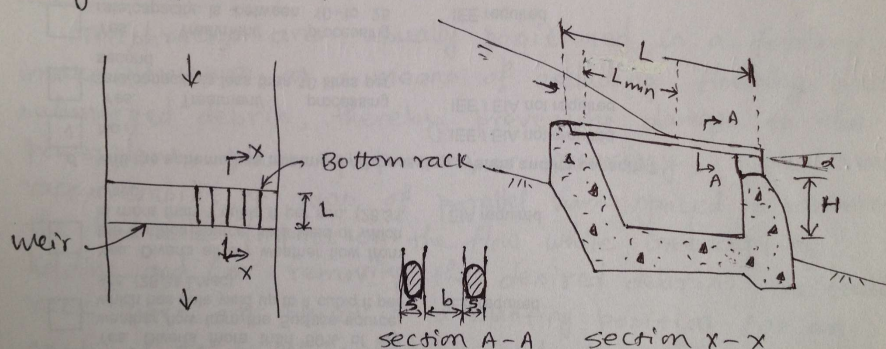


iii) V-notch weir:

$$Q = \frac{8}{15} C_d \tan \phi/2 \sqrt{2g} \, H^{5/2}$$



c) Tyrolean intake:



wetted length of rack,

$$L_{min} = \frac{0.85}{\psi m \sqrt{c}} \cdot \frac{q^{2/3}}{\cos^{3/2} \alpha} \text{ in m.}$$

where, φ = contraction factor, depends on rack bar cross-section.

$= 0.8$ for cross-section rounded at both ends.

$m = \dfrac{b}{b+s}$ = relative rack clearance

b = spacing of rack

s = thickness of rack

α = rack inclination in degrees.

$c = f(\alpha)$ and given by

$2c^3 \cos\alpha - 3c^2 + 1 = 0$, small positive value of c is taken.

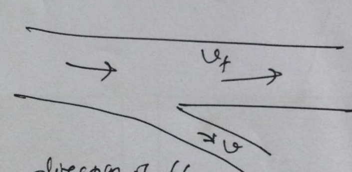
Designed length of rack, $L = 1.5$ to $2.0 * L_{min}$, keeping in view that the clogging of rack due to sediments and debris.

* For the flow velocity at inlet section of intake at 1 m/s, Calculate: i) maximum entrance loss in the intake if project is of reservoir type, and ii) minimum entrance loss in the intake if project is of ROR type with sharp angle of diversion.

Soln:

$v = 1\,m/s$

$h_{ent} = h_{l_1} + h_{l_2} = \left(\dfrac{v^2}{2g} - c\,\dfrac{v_f^2}{2g}\right) + K\,\dfrac{v^2}{2g}$



h_{l_1} = loss due to change in direction of flow

h_{l_2} = loss due to sudden contraction of area

$c = 0.8$ to 0.4 [depending on angle of diversion; higher value for lower offtake angle and lower value for higher offtake angle (i.e, near rectangular diversion)].

$K = 0.03$ for rounded bell mouth and 1.3 for sharp entry.

i) $h_{l_1} = 0$, for reservoir type project.

$$\therefore h_{ent} = h_{l_2} = K\frac{v^2}{2g} = 1.3\,\frac{1^2}{2*9.81} = 0.0663\text{ m}$$ //

ii)

$$h_{ent} = \left(\frac{v^2}{2g} - c\,\frac{v_f^2}{2g}\right) + K\,\frac{v^2}{2g}$$

$$= \frac{1^2}{2*9.81} - 0.4*\frac{1^2}{2*9.81} + 0.03*\frac{1^2}{2*9.81} \quad [v \simeq v_f]$$

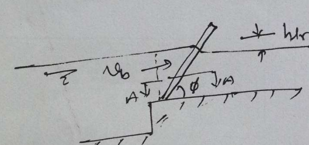
$= 0.032$ m //

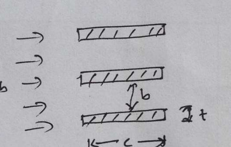
Calculate the resistance of a screen with an inclination of 75°, where the thickness of bars is 6.2 mm, the spacing between bars is 19.2 mm. The bars are rectangular cross-section (~~factor~~ section factor = 2.4), velocity of flow in front of the bars is 1.12 m/s.

Soln:

Using Krishmer's formula,

$$h_{tr} = k_r\left(\frac{t}{b}\right)^{4/3}\frac{v_b^2}{2g}\sin\phi$$





Secon as A-A

$k_r = 2.4$, $t = 6.2$ mm
$\phi = 75°$, $b = 19.2$ mm
$v_b = 1.12$ m/s

$$h_{tr} = 2.4*\left(\frac{6.2}{19.2}\right)^{4/3}*\frac{1.12^2}{2*9.81}*\sin 75°$$

$= 0.033$ m//

* Compute the necessary length of a rack of bottom intake. The intercepted flow is $4\ m^3/s$, width of the rack is 5 m, slab (bar) thickness 10 mm, clearance 15 mm, inclination 30° and the contraction factor is 0.8

soln:

$Q_r = 4\ m^3/s$, $B = 5\ m$, $s = 10\ mm$, $b = 15\ mm$, $\alpha = 30°$
$\psi = 0.8$.

Now,

specific discharge, $q = \frac{Q_r}{B} = \frac{4}{5} = 0.8\ m^2/s$

Relative rack clearance, $m = \frac{b}{b+s} = \frac{15}{15+10} = 0.6$

$\cos\alpha = \cos 30° = 0.866$

For calculation of c,

$$2c^3 \cos\alpha - 3c^2 + 1 = 0$$

or, $2c^3 * 0.866 - 3c^2 + 1 = 0$

or, $c = 1.462,\ 0.778,\ -0.508$

Take small positive value

$\therefore c = 0.778$

$$\therefore L_{min} = \frac{0.85}{\psi\, m \sqrt{c}} * \frac{q^{2/3}}{\cos^{3/2}\alpha}$$

$$= \frac{0.85}{0.8 * 0.6\sqrt{0.778}} * \frac{(0.8)^{2/3}}{(0.866)^{3/2}} \simeq 2.15\ m$$

Hence, $L = (1.5 \text{ to } 2.0) * L_{min}$

$= (1.5 \text{ to } 2.0) * 2.15$

$= 3.2 \text{ to } 4.3\ m$ //

Trashracks:

Trashracks are usually positioned in a forebay or intake structure as a means of excluding floating and submerged debris, thereby preventing damage to the powerplant, plant equipment, or waterway. Typical trash-racks consist of rows of parallel bars spaced to minimize the hydraulic impact on the flow while intercepting, holding and / or removing the desired debris. The trash-rack bars are placed in a slanting position (at an angle of 50° to 80° with horizontal).

Spacing of bars:

- Fine racks or screens → 1.5 cm to 10 cm
- Trashrack bars → 10 cm to 50 cm
- For high discharge, low head Kaplan turbine → 25 cm to $\frac{1}{30}$th of the runner diameter, whichever is greater.
- For Francis turbines clear spacing → distance between the runner vanes.
- The bars, if long enough, may need intermediate supports consisting of I-beams.
- Unsupported length of bars $\ngtr$ 70 times the bar thickness.
- Usually made of mild steel flats with rounded edges, both upstream and downstream having sections 7.5 cm * 0.65 cm to 10 cm * 1.2 cm.

Pressure on racks:

- Water pressure and dynamic pressure of the floating material apart from the dead weight.

- Unbalanced load (generated by partial or total clogging of the racks)

Design of racks:

According to Mosonyi, the racks are generally designed for a differential head of

- 1 to 2 m under normal circumstances
- 4 to 5 m under exceptional circumstances.

Cleaning of racks:

- For small stations where depth of the racks $\ngtr$ 4 or 5 m and floating material is small → manual cleaning
- For more trashrack height and large floating material → mechanical cleaning

Velocity through trashrack:

- According to Zowski, $v > 90$ cm/sec may cause the rack structure to vibrate and even collapse.
- Justin and creager's formula,

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

- Mosonyi's formula to eliminate eddies and vortices,

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

- For small plants where manual cleaning is restored to, velocity in front of screen should be of the order of 60-75 cm/sec.

Control requirement for the good intake design:

1. Vortex free flow
2. Sufficient aeration

1. Vortex free flow:

Vortex formation at the front of intake causes

- Non-uniform flow conditions

- Introduce air into the flow, with unfavourable results on the turbines (vibration, cavitation, unbalanced loads, etc.)
- Increase head loss and decrease efficiency
- Draw trash into the intake.

According to the ASCE Committee on Hydropower Intakes; disturbances, which introduce non-uniform velocity, can initiate vortices. These include:

- Asymmetrical approach conditions
- Inadequate submergence
- Abrupt changes in flow direction cause separation and eddy formation
- Approach velocities $\not> 0.65$ m/s

Lack of sufficient submergence and asymmetrical approach seem to be the commonest cause of vortex formation.

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

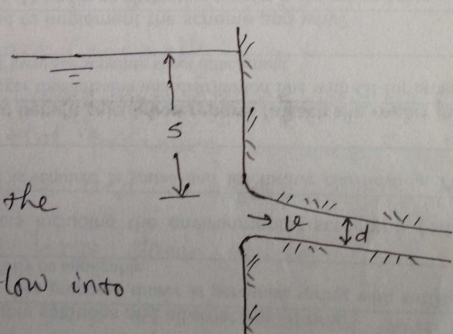
$$\frac{s}{v\sqrt{d}} > 0.3 \text{ for symmetrical approach}$$

and $$\frac{s}{v\sqrt{d}} > 0.4 \text{ for lateral approach}$$

where, s = depth of submergence for the intake

d = diameter of the conduit

and v = velocity of flow into the conduit.

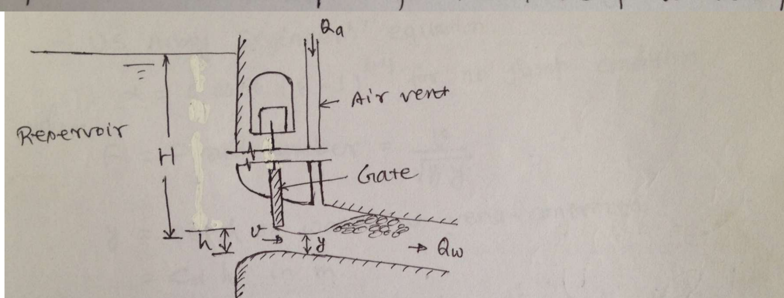


2. Sufficient aeration:

Air vent is necessary

- to release the air formation behind the gate of pressure intake, which if accumulated will decrease the discharge carrying capacity of the conduit.
- when the flow is stopped at the intake and if the penstock is required to be drained out, then in such case negative pressure (suction pressure) will be developed at the pipe and may cause the collapse of the pipe and vibration of the gates.

Emperical relationships for the size of air vent pipes:



i) $Q_a = 400\, c\, a \sqrt{p}$

where, Q_a = Discharge of air in cumec

c = A constant (normally 0.7)

a = Area of vent pipe in m^2

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

ii) Fourth congress on large dams guide:

Area of air vent = 10 % of the control gate area.

iii) USBR design guide:

Capacity of air vent = 25% of conduit discharge

iv) $Q_a = Q_w \alpha$

where, Q_a = Discharge of air in cumec

Q_w = Discharge of water in cumec

and $\alpha = \frac{Q_a}{Q_w}$

Various formulas to determine the quantities are:

- Kalinske and Roberson equation:

$\alpha = 0.03 (F-1)^{1.06}$ when hydraulic jump forms at the downstream gate.

- US Army Engineers' equation:

$\alpha = 0.0066 (F-1)^{1.4}$ for no jump condition

where,

F = Froud number = $\frac{v}{\sqrt{gy}}$

y = Depth of water at vena-contracta

$= C_d h$ in m.

C_d = Coefficient of discharge (0.8 for 45° gate bottom and 0.6 for sharp bottom lip)

h = Height of gate opening in m.

v = Spouting velocity = $\sqrt{2gH}$

H = Head across valve in m for short conduits with small losses and difference in head from reservoir surface to the top of the vena-contracta for long conduits.

$Q_w = v y b$

b = Gate width in m.

V_a = Allowable air velocity = 45 to 90 m/sec.

∴ vent area, $\boxed{A = \frac{Q_a}{V_a}}$

- vent loss should be < 1.5 m of water head.

Spillway:

A spillway is hydraulic structure designed to pass excess water downstream safely when the reservoir is full.

Purpose / Function of spillway:

- To release the excess flood flows from the reservoir to the channel downstream without damaging its components like gates, chamber and its end structures.

Types / classification:

A) According to hydraulic regime:

i) Pressure spillway → shaft spillway, syphon spillway and submerged type spillway

ii) Non-pressure spillway → others.

B) According to function / purpose:

i) Main or service spillway

ii) Auxiliary spillway

iii) Emergency spillway

i) Main or service spillway:

It is the spillway designed to pass the design flood. This spillway is necessary for all dams and in most of the dams, it is the only spillway.

ii) Auxiliary spillway:

In some dams, where the site conditions are favourable, an auxiliary spillway is usually constructed in conjunction with a main spillway. In such a case, the main spillway is usually designed to pass floods which are likely to occur more frequently.

The capacity of the main spillway is kept less than required for the design flood. When the floods exceed the design capacity of the main spillway, the auxiliary spillway comes into operation.

The favourable site conditions for the adoption of an auxiliary spillway are:

- when there is a saddle or depression along the rim of the reservoir which leads to natural drainage.
- when there is a gently-sloping abutment where an excavated channel can be carried sufficiently beyond the dam so that there is no possibility of the erosion of the dam or its appurtenant works.

iii) Emergency spillway:

An emergency spillway is an additional safety valve of the dam provided in addition to the main spillway. It comes into operation only during an emergency which may arise at any time during life of the dam. Some of the conditions which may lead to emergency are as follows:

- when actual flood exceeds the design flood
- when there is an enforced shut down of the outlets.
- when there is a malfunctioning of spillway gates.
- when there is damage or failure of some part of the main spillway.
- when high flood occurs before the previous flood has been evacuated by the main spillway.

c) According to regulation / control:

i) Controlled or gated spillway

ii) Uncontrolled or ungated spillway.

i) Controlled spillway:

A controlled spillway is one which is provided with the gates over the crest to control out flow from the reservoir. The full reservoir level (FRL) is usually kept at the top level of the gates. The water can be released from the reservoir even when the water level below the FRL.

ii) Uncontrolled spillway:

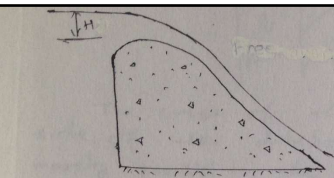
An uncontrolled spillway is one which is not provided with the gates over the crest. The FRL is the crest level of the spillway. The water escapes automatically when the water level rises above the crest level.

①) According to pertinent feature:

i) overflow or overfall spillway
ii) side channel spillway
iii) chute or Trough spillway
iv) shaft or Morning Glory spillway
v) Siphon spillway
vi) Tunnel or Conduit spillway
vii) Orifice or submerged type spillway

i) overflow spillway:

Such spillways are generally constructed as a part of the main dam itself by using a portion of the dam as an overflow section with crest as FRL. Such type of spillways are suitable for concrete gravity dam in the valley which have sufficient width to accomodate the required crest length.



$$Q = \frac{2}{3} C_d L \sqrt{2g} H^{3/2}$$

$C_d = 0.62$

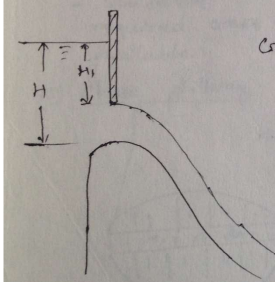
Free over fall, $Q = C L_e H_e^{3/2}$

$c = 2.2$

$H_e = H + H_a$

L_e = effective length of crest

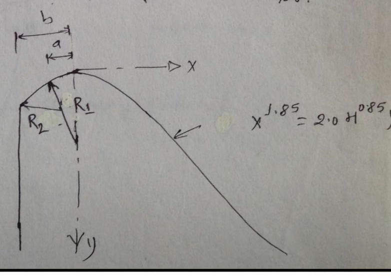
$= L - 0.1 n H_e$



Gated controll over fall

$$Q = \frac{2}{3} C_d \sqrt{2g}\, b \left(H^{3/2} - H_1^{3/2}\right)$$

shape of the crest:



$$x^{1.85} = 2.0 H^{0.85} y$$

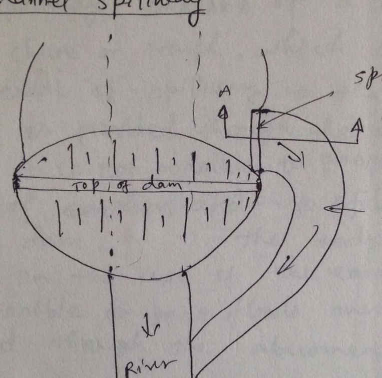
$a = 0.175 H$

$b = 0.282 H$

$R_1 = 0.50 H$

$R_2 = 0.20 H$

ii) side channel spillway

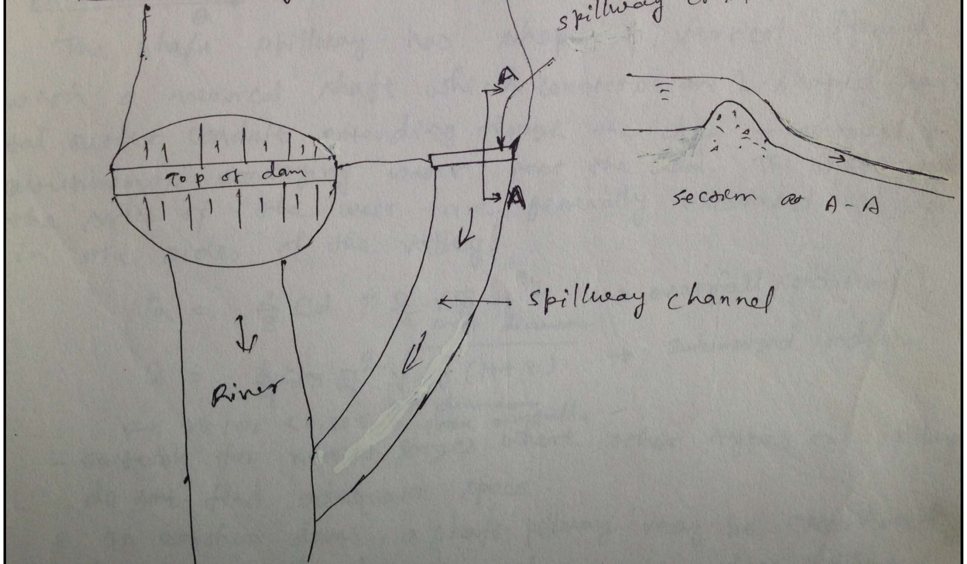


section at A - A

The crest of the spillway is placed along the side of the discharge channel. The crest is approximately parallel to the spillway channel.

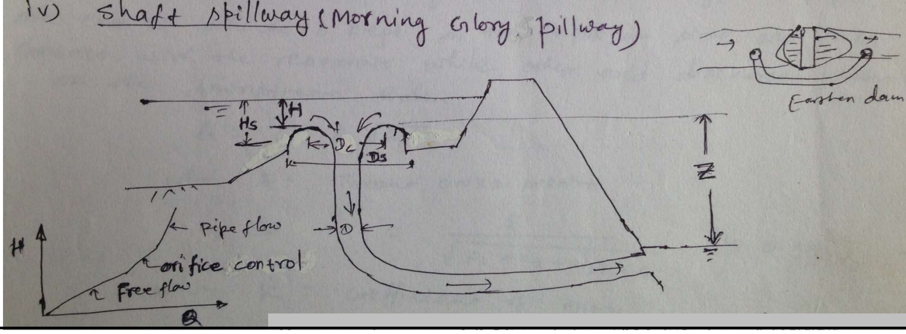
- suitable for earthen, rockfill dams (i.e. embankment dams)
- suitable in a narrow valley, where the required crest length of the overflow spillway is not available.

iii) Chute spillway :



Chute spillway consists of a steep-sloped open channel called a chute or trough, which carries the water passing over the crest of spillway to the river downstream. This type of spillway is provided through the abutments of the of the dam when it is not possible to pass floods over the dam as in the case of earthen and rockfill dams. In the case of concrete dam, the body of dam is used as a spillway. In the case of the earthen and rockfill dams, it is not possible to pass floods over the dam and so it is provided through the abutments of the dam.

iv) Shaft spillway (Morning Glory spillway)



The shaft spillway has shape of vertical funnel with a vertical shaft which connects an L-shaped horizontal outlet conduit extending through the dam or through the abutments conveying water past the dam. The shaft and the sides of the weir are generally excavated in rock in the sides of the valley.

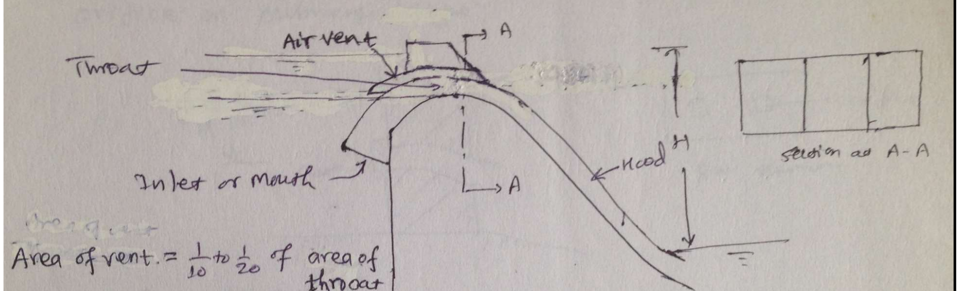
$$Q = \frac{2}{3} C_d \, \pi \, \underbrace{D_c}_{\text{crest diameter}} \sqrt{2g} \, H^{3/2} \rightarrow \text{overfall condition}$$

$$Q = \frac{1}{4} C_d \pi \underbrace{D}_{\text{shaft diameter}}^2 \sqrt{2g(H+Z)} \rightarrow \text{submerged condition.}$$

For, $H_s / D_s < 0.225 \rightarrow$ free overfall.

- Suitable for narrow gorges where other types of spillways do not find adequate space.
- In earthen dams, a shaft pillway may be excavated through the foundation or flank of the river valley.

v) Syphon spillway:



A syphon spillway consists of a syphon pipe, one end of which is kept on the upstream side and is in contact with the reservoir while other end discharges water on the downstream side.

$$Q = C_d \, A \sqrt{2gH}$$

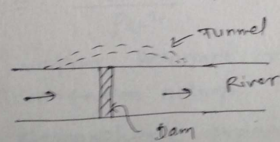
where, A = Throat cross-section

$$C_d = \frac{1}{\sqrt{K_1 + K_2 + K_3 + K_4}} \simeq 0.90$$

where, K_1 = coefficient of entry
K_2 = " " exit
K_3 = " " bend
K_4 = " " friction

- Suitable for limited width gorges having less space for spillway.
- Suitable when there are small fluctuations in water level of the reservoir to maintain constant water level.
- Also suitable for forebays where the surges created by the change in turbine loads are to be equilized.

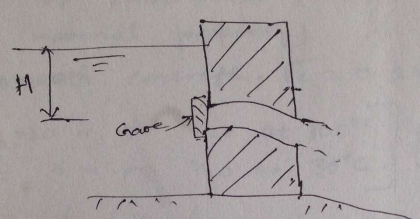
vi) Tunnel (or conduit) spillway:

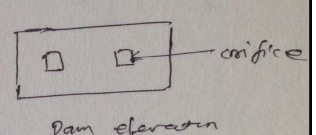


It consists of a closed conduit to carry the flood discharge to the downstream channel. It is constructed in the abutment or under the dam.
- Suitable for dam sites in narrow canyons with steep abutments.

Other type:

Orifice or submerged type:





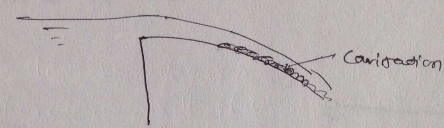
$$Q = C_d A \sqrt{2gH}$$

Factors governing choice of the spillway:

- economy
- reliability of flood estimation accuracy.
- reliability of gate operation
- duration and amount of spillway discharge.
- Seismicity of project site.
- Topography and geology.

Cavitation:

Cavitation is the hydraulic phenomenon which occur whether the pressure in the flow of water drops to the value of the saturation vapour pressure.



Checking for cavitation:

Cavitation index or cavitation number or Euler's number,

$$\sigma = \frac{2(p - p_v)}{\rho v^2}$$

where, p_v = vapour pressure or sub-atmospheric pressure in which cavitation occurs.

p = atmospheric pressure

v = velocity of flow

ρ = density of water.

- Cavitation occurs when $\sigma < \sigma_c$

where, σ_c = critical value depends upon the material and its smoothness (i.e., function of geometry and material property).

For smooth concrete, $\sigma_c = 0.25$

$$\boxed{\begin{array}{l} p_v = p_a = 10 \text{ m for } H_2O \text{ at } 100^\circ C \\ p_v = 5 \text{ m for } H_2O \text{ at } 30^\circ C. \end{array}} \; ?$$

* Check whether cavitation occur in spillway surface.

$p_v (t = 20^\circ C) = 2.5$ m, $\sigma_c = 0.25$, $v = 20$ m/sec.

Soln: we have, $\sigma = \dfrac{2(p - p_v)}{\rho v^2} = \dfrac{2 * (9810 * 10 - 9810 * 2.5)}{1000 * 20^2}$

$= 0.37 > 0.25$

Hence no cavitation. //

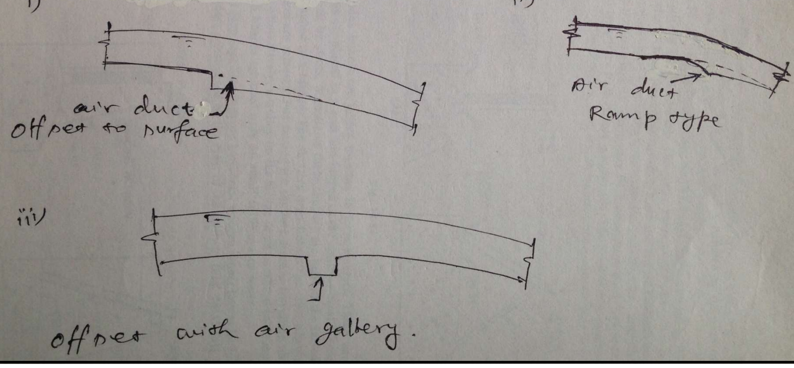
Control measures:

a) Before construction → Decrease specific discharge 'q' by widening of spillway length or opening.

b) During construction :→ Aeration to flow (ducts below the spillway surface to be constructed which should be connected to atmospheric pressure)

c) Artificial aeration → if the provision made in (b) is not enough.

b) i) ii)



Gates: Types and their location

A spillway with control mechanism is almost invariably provided for release of waters during excess flood inflows. Releases of water may also be carried out by control devices provided in conduits in the body of the dam and tunnels. In order to achieve flow control, a gate or a shutter is provided in which a leaf or a closure member is placed across the waterway from an external position to control the flow of water.

The different types of gates used in water resources projects may be broadly classified as either the Crest or Surface type, which are intended to close over the flowing water and the Deep-seated or Submerged type, which are subjected to submergence of water on both sides during its operation. The different types of gates falling under these categories are as follows:

1. Crest or Surface type gates

i) Stop-logs/flash boards

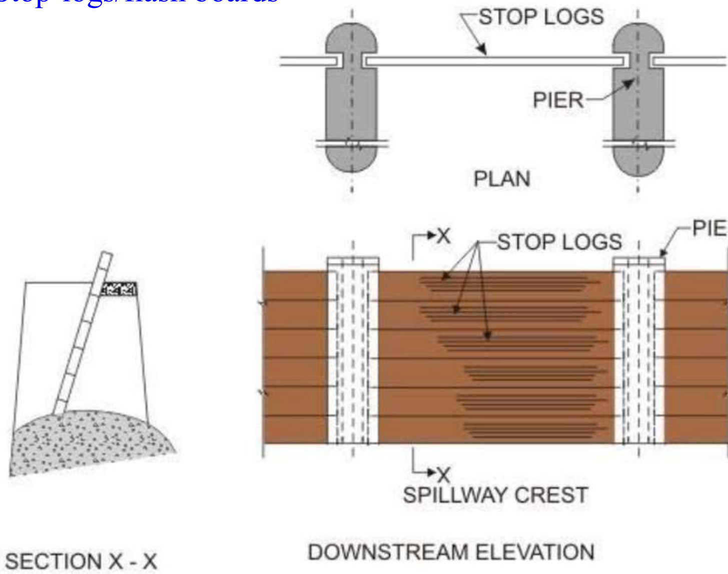


Figure: Stoplogs (Needles) inserted within groves of adjacent piers

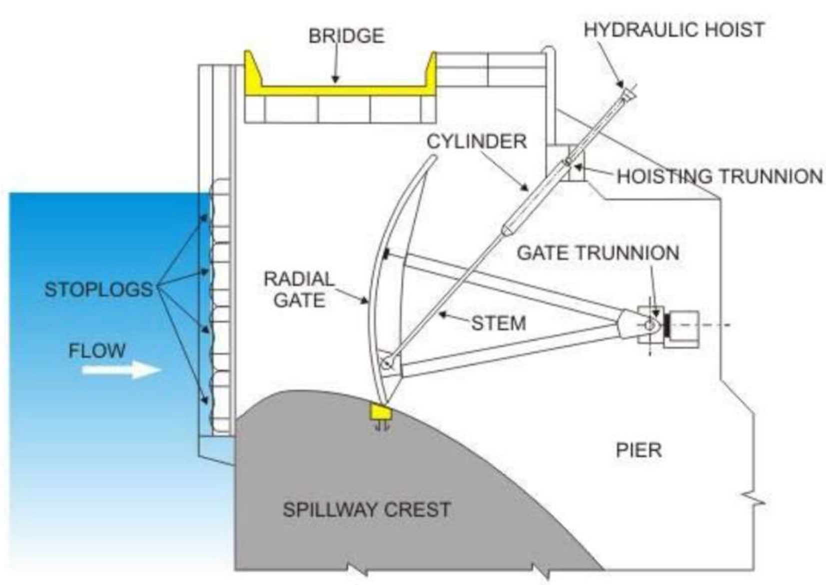


Figure: Modern day stoplogs shown in combination with a radial gate and hydraulic hoist

A log, plank cut timber, steel or concrete beam fitting into end grooves between walls or piers to close an opening under unbalanced conditions, usually handled or placed one at a time. Modern day stop-logs consist of steel frames that may be inserted into grooves etched into piers and used during repair / maintenance of a regular gate. The stop logs are inserted or lifted through the grooves using special cranes that move over the bridge.

ii) Vertical lift gates

These are gates that moves within a vertical groove incised between two piers. The vertical lift gates used for controlling flow over the crest of a hydraulic structure are usually equipped with wheels, This type of gate is commonly used for barrages but is nowadays rarely used for dam spillways. Instead, the radial gates are used for dams. This is mostly due to the fact that in barrage spillways, the downstream tail-water is usually quite high during floods that may submerge the trunnion of a radial gate.

ii) Vertical lift gates

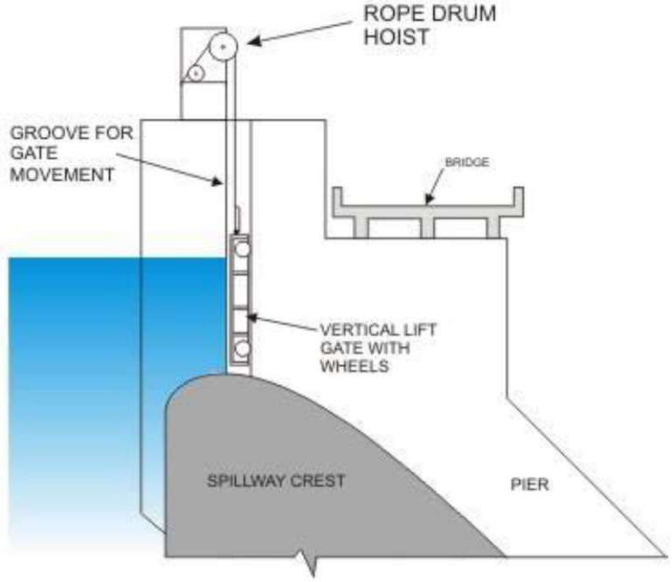


Figure: Vertical lift gate arrangement for dam spillway

iii) Radial gates

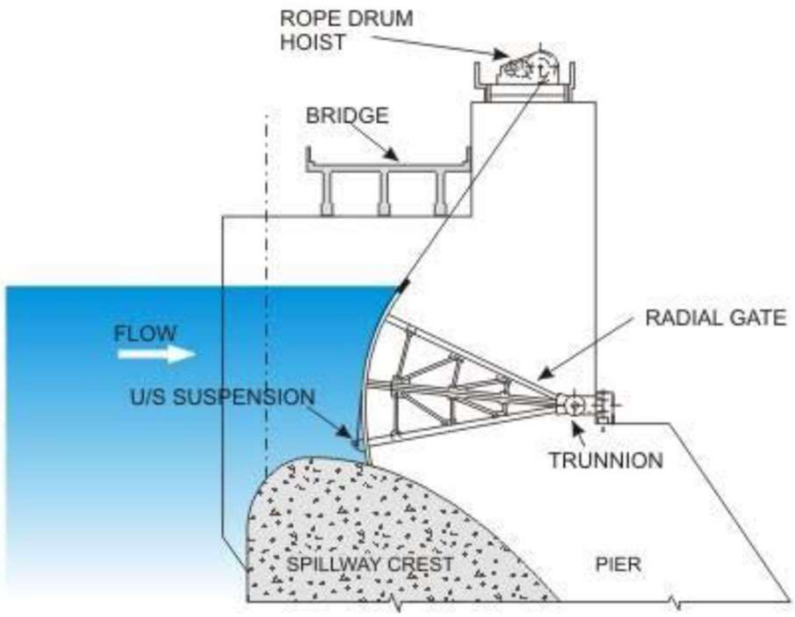


Figure: Radial gate shown with rope drum hoisting mechanism

iii) Radial gates

These are hinged gates, with the leaf (or skin) in the form of a circular arc with the centre of curvature at the hinge or trunnion. The hoisting mechanism shown is that using a cable that is winched up by a motor placed on a bridge situated above the piers. Another example of radial gate may be seen in Figure above (Stop-logs/flash boards), where a hydraulic hoisting mechanism is shown.

iv) Ring gates

A cylindrical drum which moves vertically in an annular hydraulic chamber so as to control the peripheral flow of water from reservoir through a vertical shaft.

iv) Ring gates

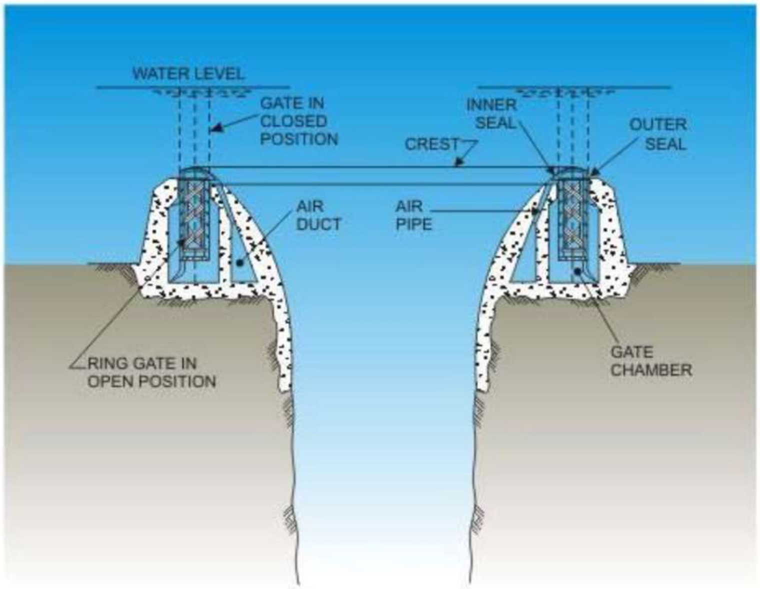


Figure: Ring gate arrangement for morning glory spillway

v) Stoney gates

A gate which bears on roller trains which are not attached to the gate but in turn move on fixed tracks. This type of gate is not much in use now.

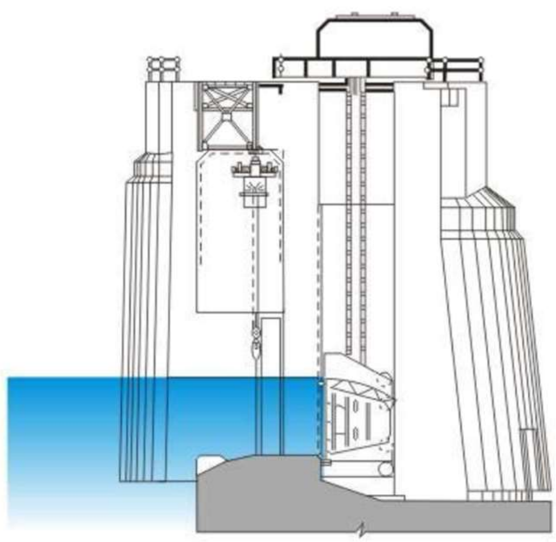


Figure: Typical arrangement of stoney gate

vi) Sector gates

A pair of circular arc gates which are hinged on vertical axis in a lock. These gates are used in navigation locks where ships pass from a reservoir with a higher elevation to one with a lower elevation.

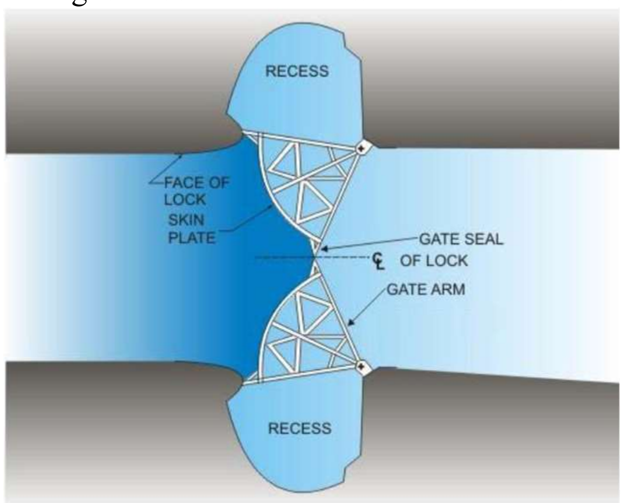


Figure: Plan of a sector gate, left side contains water at higher elevation

vii) Inflatable gates

These are gates which has expandable cavities. When inflated either with air or water it expands and forms an obstruction to flow thus effecting control. It is used quite often in many other countries because of its simplicity in operation. However, they suffer from possible vulnerability from man-made damages.

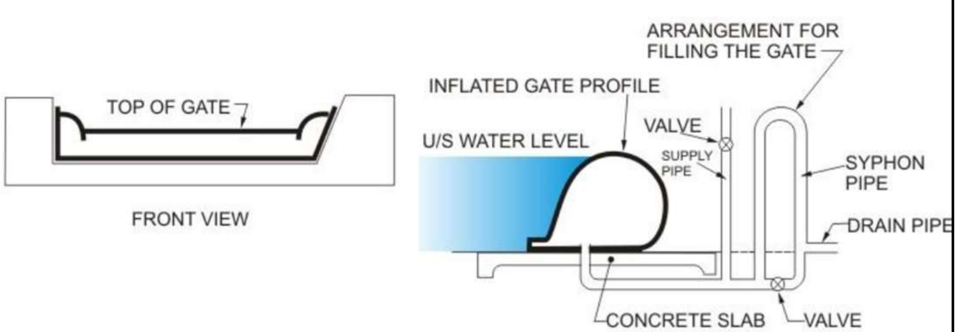


Figure: Inflatable gate

viii) Falling Shutters

Low head gates installed on the crest of dams, barrages or weirs which fall at a predetermined water level. Generally these are fully closed or fully open, that is, fallen flat, which are shown to operate using a hoist. However, in some weirs, falling shutters have been provided earlier that are manually operated. In many of the older weir installations constructed during the pre-independence period were equipped with falling shutters, some of which are still in use today.

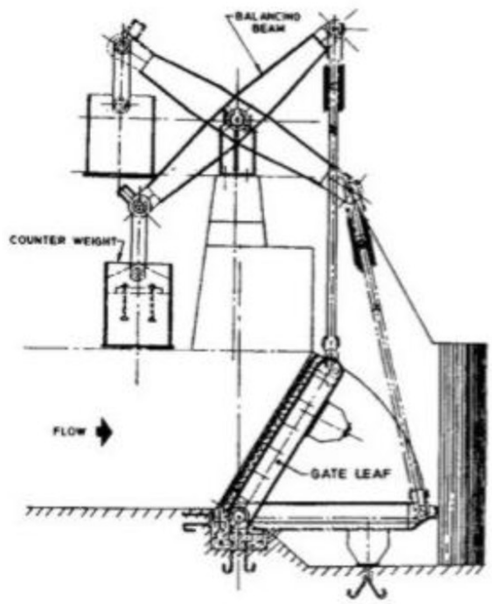


Figure: Automatic Falling Shutter

viii) Falling Shutters

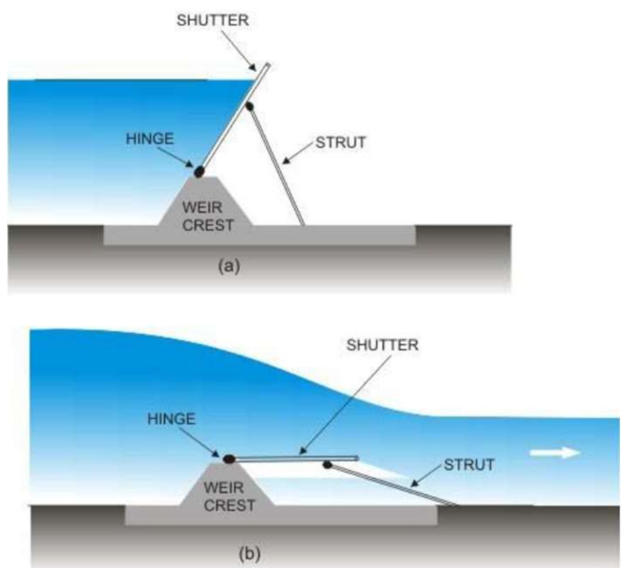


Figure: Manually operated falling shutter (a. closed position, b. open position)

ix) Float operated gates

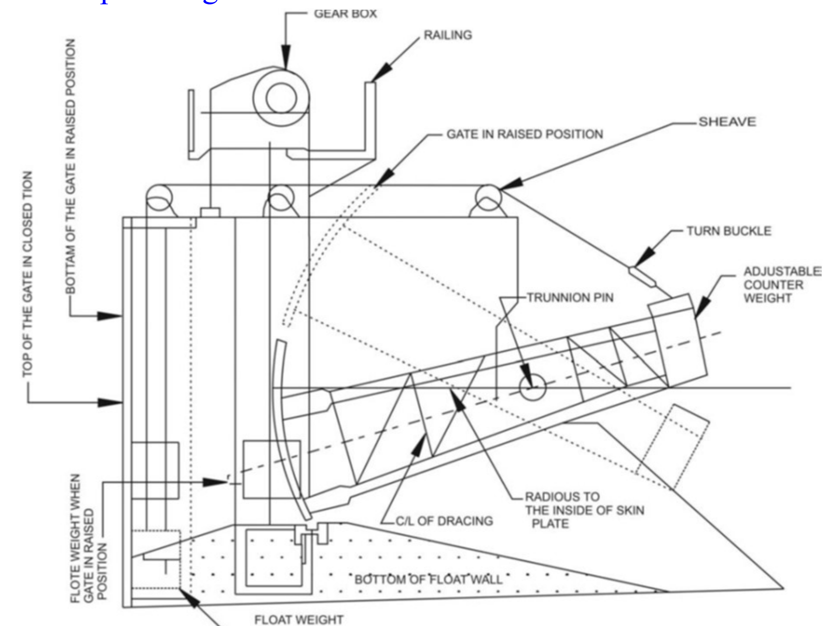


Figure: Automatic float operated radial gate

ix) Float operated gates

A gate in which the operating mechanism is actuated by a float that is pre-set to a predetermined water level. These may be used as escape in canals or even in dams to release water if it goes above a certain level considered dangerous for the overall safety of the project.

x) Two-tier gates

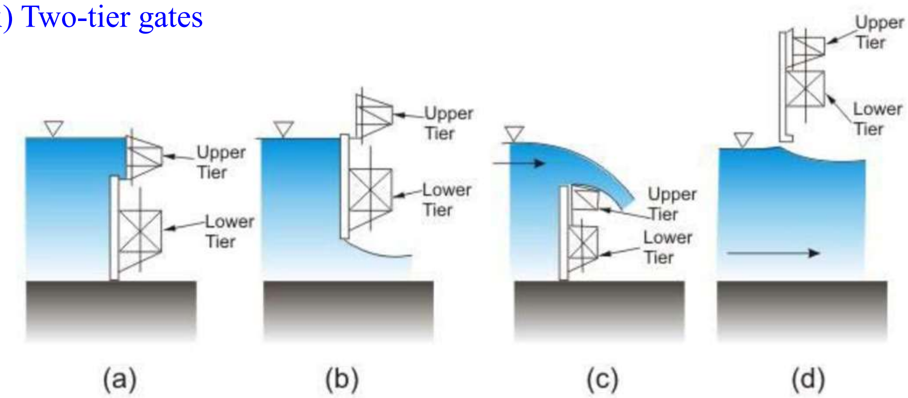


Figure: Two tier gate at different positions (a. closed, b. under flow, c. over flow, d. free flow)

x) Two-tier gates

A gate used in two leaves or tiers which can be operated separately, but when fully closed act as one gate. These types of gates are used to reduce the hoist capacity or the lift of the gate.

2. Deep seated gates

i) Vertical gate

Similar to that used for crest type gates, but usually for deep-seated purposes like controlling flow to hydropower intake either the ones with roller wheels, or the sliding-type without any wheels, are used.

i) Vertical gate

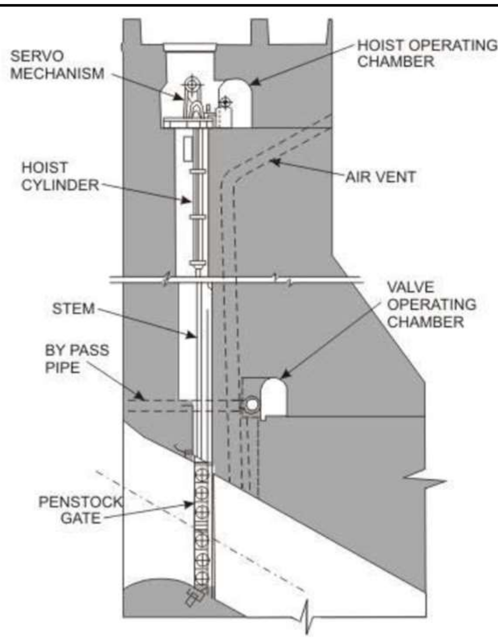


Figure: Typical arrangement of vertical lift gate with wheels with hydraulic gate

i) Vertical gate

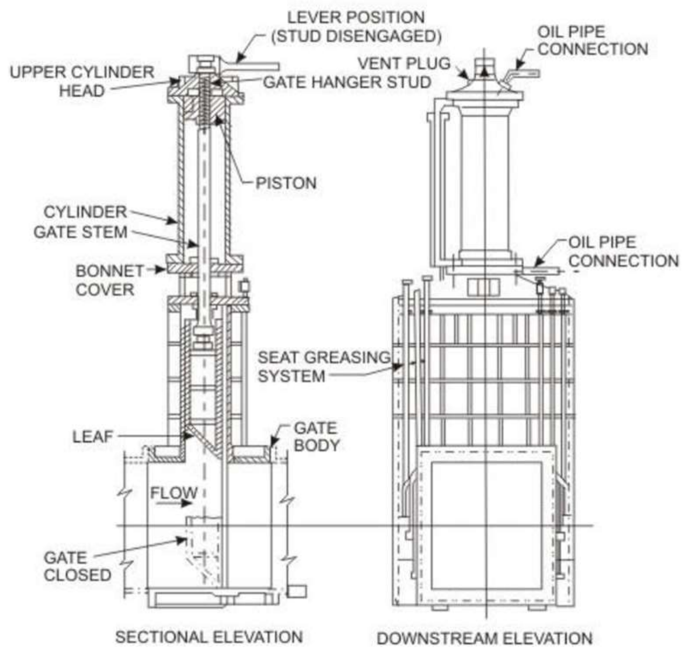


Figure: Slide gate (vertical lift) with hydraulic hoist mechanism

ii) Deep-seated radial gates

These are low level radial outlet gates. These gates have sealing on top apart from on all sides. They are located at sluices in the bottom portion of dam. The hoisting arrangement is usually at the top but could also be provided near the elevation of top seal to reduce hoist stroke.

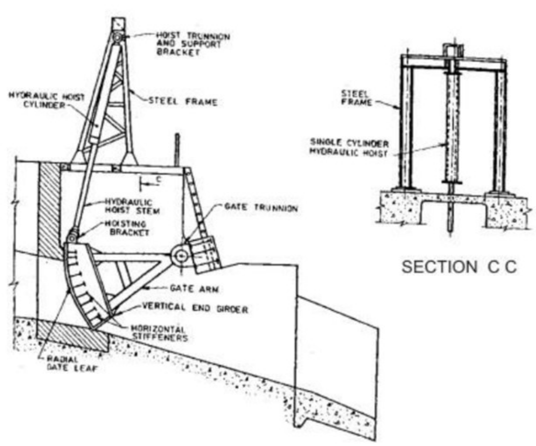


Figure: Deep-seated radial gate with hydraulic hoist

iii) Disc gates

A gate, which is in the form of disc, and rotates about an axis of its plane to control the flow of water.

iv) Cylindrical gate

A gate in the form of a hollow cylinder placed in a vertical shaft. These gates are used usually for intake towers, upstream of dams for shutting off the water to penstocks and control values. These may also be used in outlet works

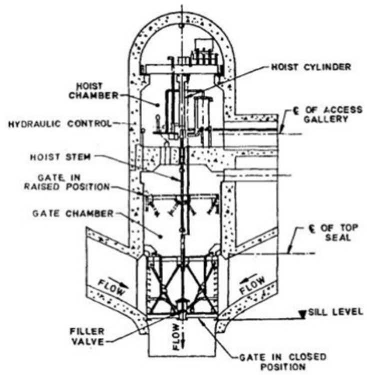


Figure: Cylindrical gate with hydraulic hoist

v) Ring follower gates

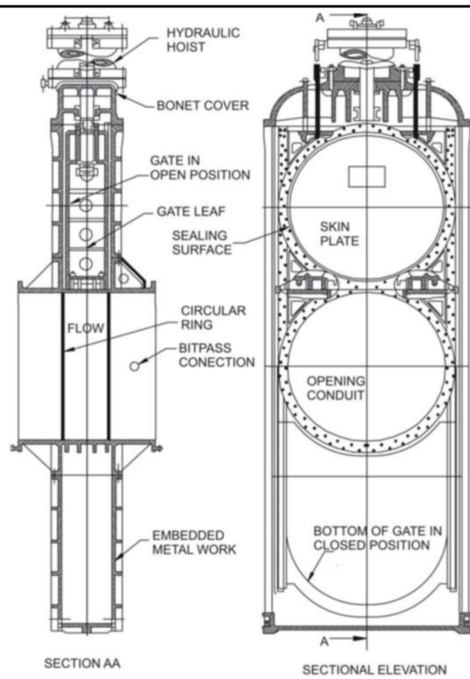


Figure: Ring follower gate

v) Ring follower gates

These are gates with a slide gate with a circular ring (a leaf with a circular hole) extending below the gate leaf. The diameter of the circular hole is equal to the diameter of the conduit. When the gate leaf is raised above the conduit, the circular hole forms an unobstructed passage for the flow of water in the conduit. When the gate is lowered to shutoff the flow, the circular ring fits into a recess below the invert of the conduit. It is used as emergency gate upstream of a regulating or service gate and is operated either in fully closed or fully open position.

vi) Jet flow gates

A high pressure regulating gate in which the leaf and the housing are so shaped as to make the water issue from the orifice in the form of a jet which skips over the gate slot without touching the downstream edge of the slot. They are adopted when very fine control of discharge is desired.

vi) Jet flow gates

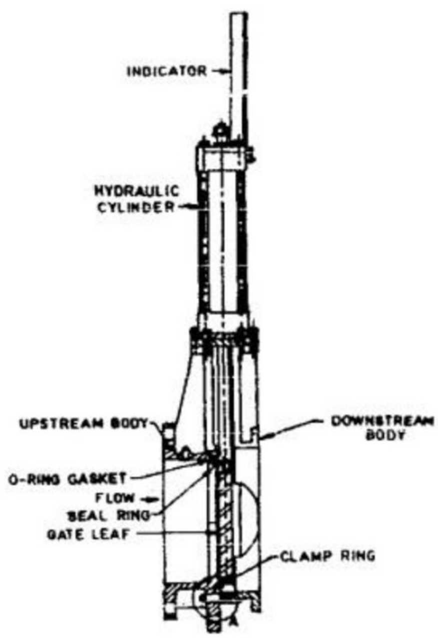


Figure: Jet flow gate

vii) Ring seal gates

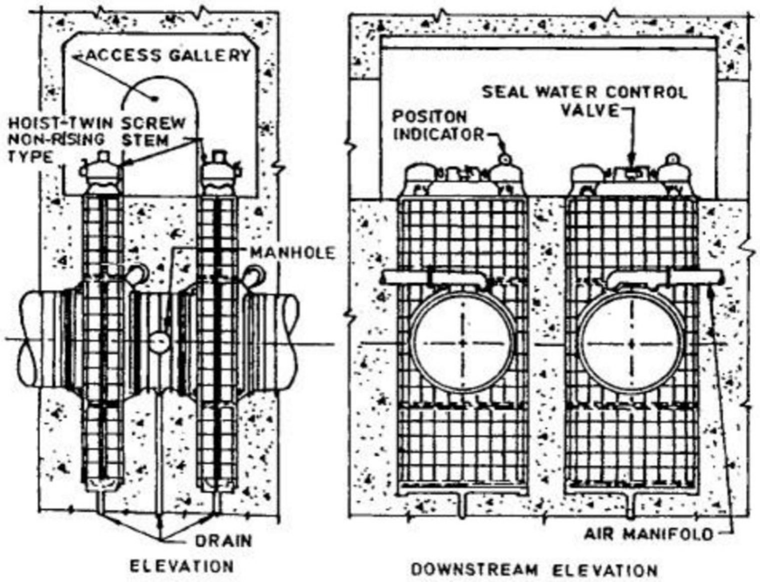


Figure: Ring seal gate

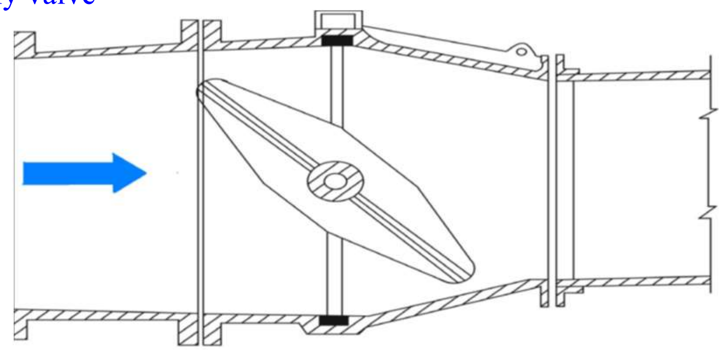
vii) Ring seal gates

A roller or wheel mounted gate in which the upper portion of the gate leaf forms a bulkhead section to stop the flow of water and the lower portion forms a circular opening of the same size as the conduit so as to produce as unobstructed water passage with the leaf in the open position. Complete closure of the leaf in the lower position is made by extending a movable ring seal actuated hydraulically from the water pressure in the conduit to contact a seat on the leaf. This type of gate is usually used as either service or emergency gates in the penstocks or other conduits.

Valves: Types and Suitability

Different types of valves used in water resources engineering are mostly used to control flow in the high pressure conduits like penstocks conveying water to turbines for generation of hydroelectricity. The Bureau of Indian Standards code IS: 4410 (Part 16, Section 2) – 1981 mentions a list of valves in use for various purposes. The valves that are commonly used for water resources projects are mentioned below:.

1. Butterfly valve

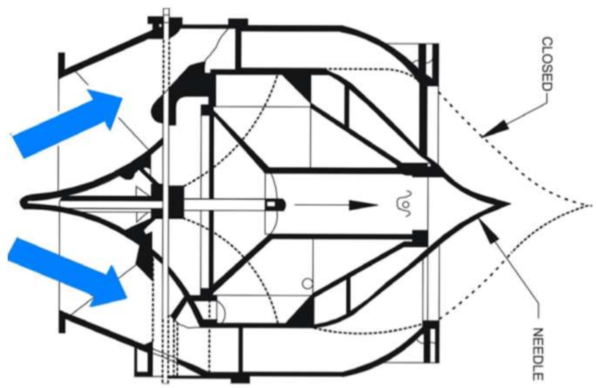


1. Butterfly valve

A valve in which the disk is turned about 90 degrees from the close to the open position, about a spindle supported on the body of the valve on an axis transverse to that of the valve.

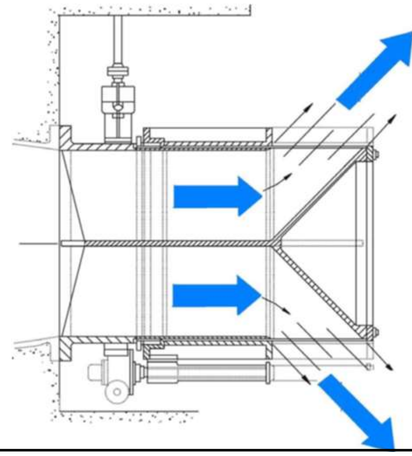
2. Hollow jet valve

A high pressure valve wherein a needle, which, when moved downstream to open the valve, releases water in the form of a hollow jet.



3. Howell-Bunger (Cylindrical) valve

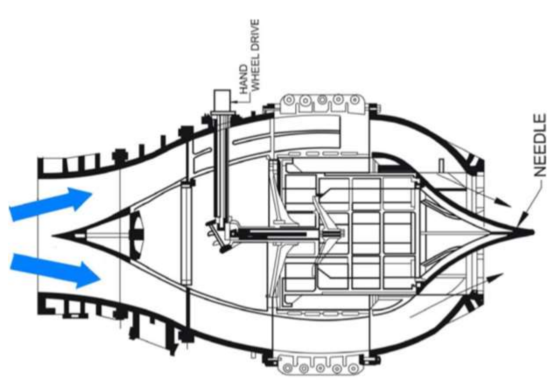
A valve having two telescopic cylinders with a streamline dispersing cone secured to the inner cylinder by radial ribs. The outer cylinder closes the sideway opening between the cone and the inner cylinder when it is slid in position. In its open position, the water is discharged on the sides of the cylinder in the form of a highly diverging hollow inside in the shape of a cone.



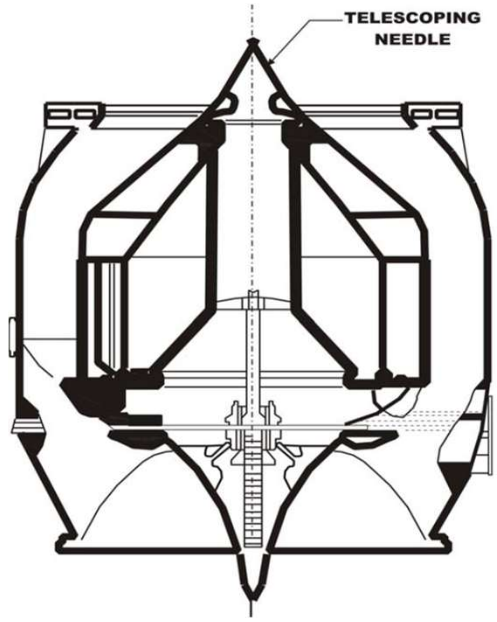
4. Needle valve

A valve with a circular outlet through which the flow is controlled by means of a tapered needle which extends through the outlet, reducing the area of the outlet as it extrudes, and enlarging the area as it retreats.

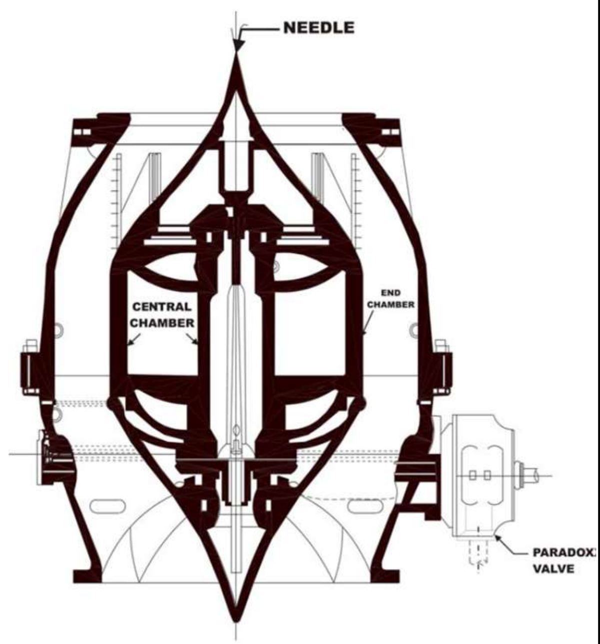
Balanced Needle Valve - A needle valve of improved design in which the needle is moved by water pressure from the outlet conduit, which acts on interior chambers in the valve. The movement is controlled by a hand wheel installed above the valve, with the motion transmitted through shafting and gearing to a poke positioning device located inside the valve.



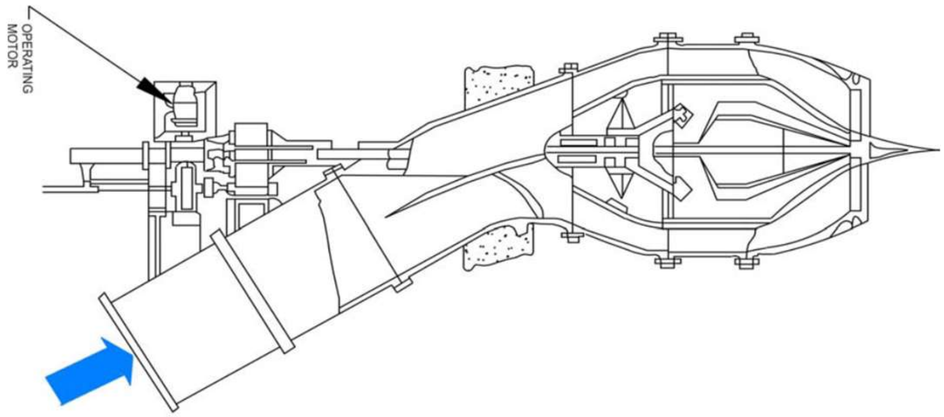
Interior Differential Needle Valve- A differential needle valve with a needle that telescopes over a member fixed to the valve body instead of moving within the valve body as in the case of an internal differential needle valve.



Internal Differential Needle Valve - This is an improved type of balanced needle valve with three chambers in the needle. The two end chambers are connected. The valve is operated by the differential thrust resulting from the changes in pressure in the end chambers with respect to that in the central chamber through a valve paradox.

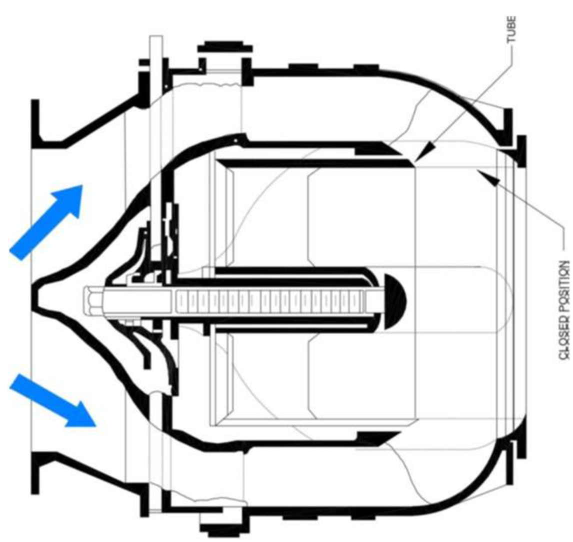


Motor-operated Needle Valve- This is a needle valve in which the position of the needle is controlled by a motor-operated rod.



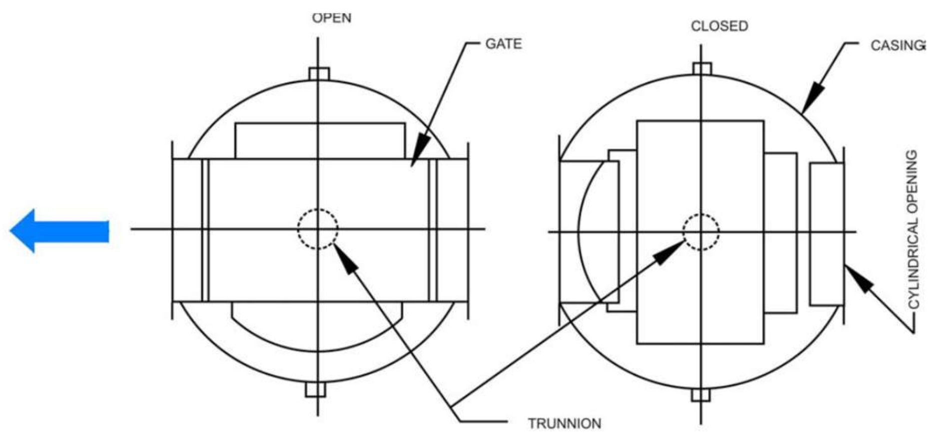
5. Tube Valve

An improvement over the needle valve. The water passages are similar to the internal differential valve, except that the downstream end of the needle is omitted. A tube or hollow cylinder similar to that of the cylinder gate, instead of a needle, comprises the moving part of the valve. This is actuated by a hydraulic cylinder and piston and a pressure pump or by a screw with an electric motor or by manual control.



6. Spherical or Rotary Valve

A valve consisting of a casing more or less spherical in shape, the gate turning on trunions through 90 degrees when opening or closing, and having a cylindrical opening of the same diameter as that of the pipe it serves.



Energy Dissipation: Water flowing over a spillway has very high kinetic energy. If the water flowing such a high velocity is discharged directly into the channel downstream, serious scour of channel bed may occur. If the scour is not properly controlled, it may extend backward and may endanger the spillway and the dam. In order to protect the channel bed against scour, the kinetic energy of the water should be dissipated before it is discharged into downstream channel.

the most common methods of energy dissipation below spillways are:
(1) Hydraulic jump type stilling basin. (2) Roller bucket type energy dissipator. (3) Ski-jump bucket energy dissipator. These are briefly described below:

(1) Hydraulic Jump Type Stilling Basin

Figure 9.18 shows a typical hydraulic jump type stilling basin. The water flowing down the face of the spillway slope enters over the horizontal apron of the basin with a supercritical small depth y_1 and high velocity V_1. Since the tail water depth y_2 is in a subcritical range, a hydraulic jump occurs on the apron as shown in Fig. 9.18 and water suddenly enters from a lower stage y_1 to a higher stage y_2 accompanied by considerable eddy turbulence. Due to this, a major part of the initial flow energy is dissipated. The formation of hydraulic jump is possible only if the flow is supercritical before the jump (Froude Number $F_1 = \frac{V_1}{\sqrt{g y_1}} > 1.0$) and subcritical after the jump ($F_2 < 1.0$). Secondly, for the formation of the jump, the tailwater depth must match with the post-jump depth requirements as given by the equation:

$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}$

$q = v_1 y_1$

$$\boxed{\frac{y_2}{y_1} = \frac{1}{2}\left\{\sqrt{1+8F_1^2}-1\right\}} \quad \ldots (9.16)$$

If the tailwater depth is greater than y_2, then the jump gets submerged and hence little loss of energy takes place. On the other hand, if the tailwater depth is less than y_2, the jump gets swept off downstream. In such a case, the super-critical high-velocity flow continues for some distance on the river bed. This creates problems of river bed erosion. Hence, the success of the stilling basin of this type lies in ensuring the formation of the hydraulic jump at different conditions of depth, discharge and Froude Number before the jump. The formation of the hydraulic jump is facilitated by some appurtenances (structures) on the stilling basin. These are: chute blocks at the toe of the spillway, friction blocks, or baffle piers on the horizontal apron and solid or notched (dentated) end-sill at the end of the apron as shown in Fig. 9.18. Various standard designs of the stilling basins and the appurtenances are available in literature. USBR-type or SAF-basin (Saint Anthony Falls) are some recognised types. The amount of energy dissipated in such stilling basins depends upon the Froude Number of flow. For strong jumps ($F_1 > 9.00$) the

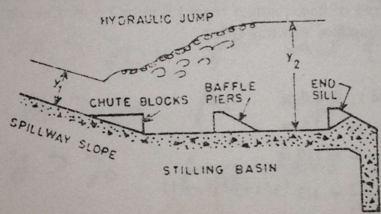


Fig. 9.18. Hydraulic jump.

energy dissipation efficiency is fairly high and can reach the value of as much as 80 per cent under certain circumstances.

2) **Roller Bucket Type Energy Dissipator**

This type is employed where the tailwater condition is not favourable for adopting hydraulic jump type basin. The roller bucket is a spoon type structure at the toe of the spillway as shown in Fig. 9.19(a). When the high velocity sheet

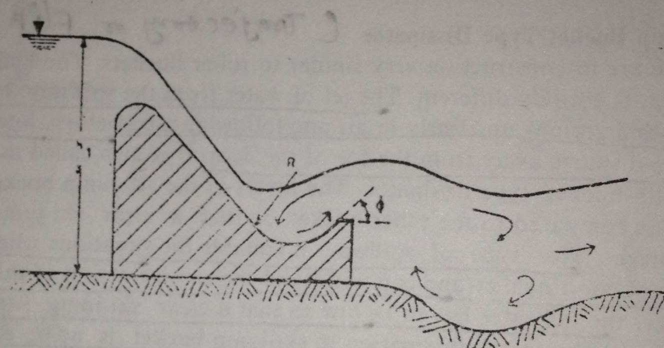


Fig. 9.19 (a). Roller Bucket type dissipator.

of water slides down the spillway, it gets arrested by the tail water. This gives rise to a surface roller as well as bottom roller action and eddy turbulence, accompanied by the energy dissipation. Bucket type energy dissipator has a relatively short structure as compared to hydraulic jump-type stilling basin. For successful roller action, the tailwater depth has to be slightly greater than that needed by the hydraulic jump type basin. The main variables in the design of the bucket are the radius of the bucket R and the lip angle ϕ. The radius varies between 15m to 25m and the lip angle between 20° to 40°. The optimum dimensions are decided after model studies.

Roller Buckets were first used for Grand Coulee dam (USA) and have become quite popular since then. In India, a number of dams have used roller buckets. Following table gives some relevant data for some dams.

- Energy dissipation upto ≃(70-80)% if jets collides with each other.

3) **Ski-Jump Bucket Type Dissipator** (Trajectory or Flip bucket type dissipator)

These are in construction very similar to roller buckets. The hydraulic action, however, is entirely different. The jet of water from the spillway flows over the bucket and springs up clearly in air and following a trajectory, hits the river bed at some distance away from the toe of the dam. It is also called as a trajectory or a flip-bucket type dissipator. The invert of the ski-jump bucket is relatively higher as compared to the roller bucket, so that a clear ski-jump action can take place. This type of spillway is suitable for situations where foundation rock is of good quality and can withstand the erosive action of the plunging jet. Secondly the tailwater has to be low so that a clear ski-jump formation could take place. The energy dissipation in ski-jump bucket is achieved due to the combined action of air resistance, viscous effects and the turbulence due to impact on the river bed. Ski-jump bucket dissipator is particularly suited for arch dams. , Lip angle generally varies betn 30° to 40°

Chastang and L'Aigle dams in France have typical ski-jump installations. In India, the proposed Jakham dam has been provided with a ski-jump type dissipator.

A curious feature of the ski-jump bucket is that many a times, the dissipator acts as ski-jump type at certain discharges and as roller bucket at lower discharges. A typical example is Rihand dam in India where the bucket acts as a ski-bucket at discharges of 33m³/s/m and above, but as a roller bucket below this value of the discharge.

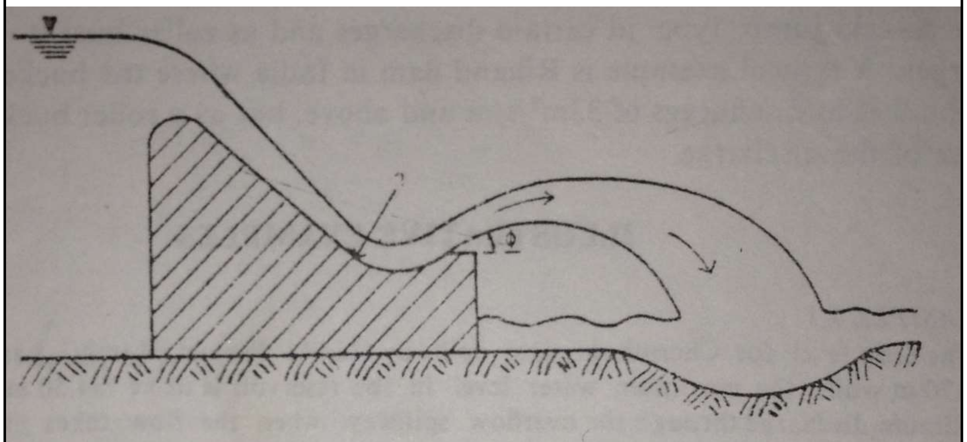
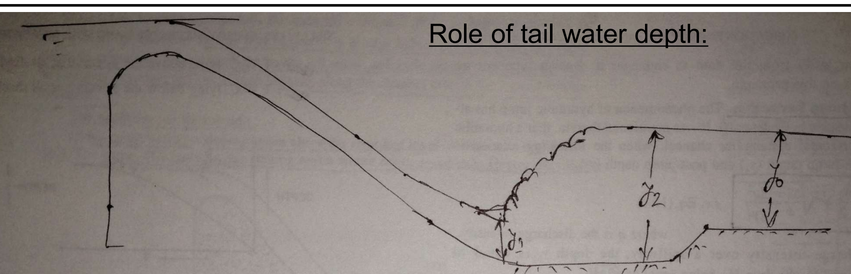


Fig. 9.19 (b). Ski-jump dissipator.

Wind effect on jet trajectory:

- $v < 20\ m/s$ → No effect
- $v > 40\ m/s$ → 30% energy reduced through distance

Role of tail water depth:



Role of tailwater depth:

- determines whether stilling basins required or not.
- river bed protection is determined by tailwater depth.

Different cases:

a) $y_2 > y_0$ for all q → Requires stilling basin

b) $y_2 = y_0$ for all q → S/B requires for safety

c) $y_2 < y_0$ for all q → not requires S/B

d) $y_2 > y_0$ for high discharges → Requires stilling basin

e) $y_2 > y_0$ for low discharges → " " "

21.11.1. Hydraulic Jump Formation. The phenomenon of hydraulic jump has already been explained in details in Chapter 10. It was mentioned therein, that a hydraulic jump can form in a horizontal rectangular channel, when the following relation is satisfied between the pre-jump depth (y_1) and post-jump depth (y_2).

$$\boxed{y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}} \quad i.e. \text{ Eq. (10.4)}$$

where q is the discharge intensity.

For a given discharge intensity over a spillway, the depth y_1 is equal to q/V_1 ; and V_1 is determined by the drop H_1, being equal to $\sqrt{2gH_1}$.

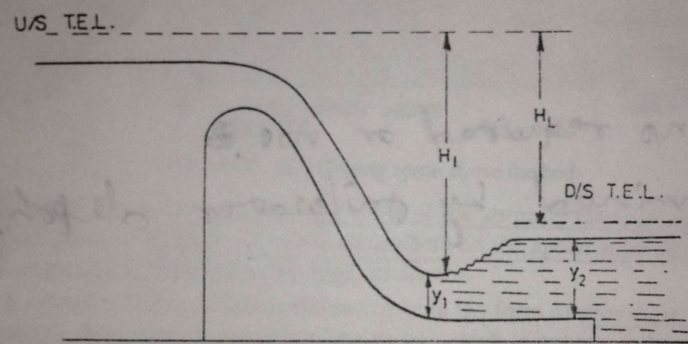
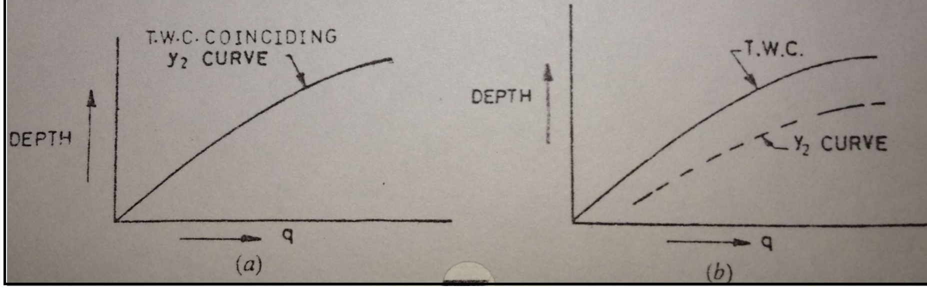


Fig. 21.29

Hence, for a given discharge intensity and given height of spillway, y_1 is fixed and thus y_2 (*i.e.* the depth required for the formation of hydraulic jump) is also fixed. But the availability of a depth equal to y_2 in the channel on the d/s cannot be guaranteed as it depends upon the tail water level, which depends upon the hydraulic dimensions and slope of the river channel below. The problem should, therefore, be analysed before any solution can be found. Hence, for different discharges, the tail water depth is found by actual gauge discharge observations and by hydraulic computations. The post jump depths (y_2) for all those discharges, are also computed from equation (10.4). If a graph is now plotted between q and tail water depth, the curve obtained is known as the *Tail Water Curve (T.W.C.)*. Similarly, if a curve is plotted on the same graph, between q and y_2, the curve obtained is known as the *Jump Height Curve (J.H.C.)* or *y_2 curve*.

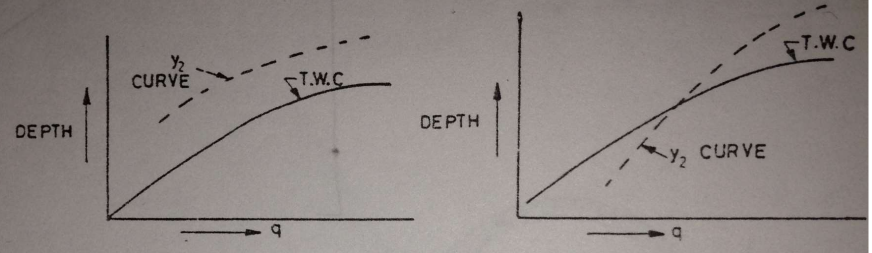
Now there are five possibilities

(*a*) T.W.C. coinciding with y_2 curve at all discharges [Fig. 21.30 (*a*)].



(b) T.W.C. lying above the y_2 curve at all discharges [Fig. 21.30 (b)].

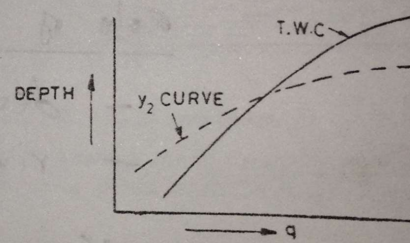
(c) T.W.C. lying below the y_2 curve at all discharges [Fig. 21.30 (c)].



(c) (d)

(d) T.W.C. lying above the y_2 curve at smaller discharges and lying below the y_2 curve at larger discharges [Fig. 21.30 (d)].

(e) T.W.C. lying below the y_2 curve at smaller discharges and lying above the y_2 curve at larger discharges [Fig. 21.30 (e)].



(e)
Fig. 21.30

Depending upon the relative positions of T.W.C. and y_2 curve, the energy dissipation arrangements can be provided below the spillway, as explained below for all these five cases.

21.11.1.1. Energy dissipators for case (*a*) : *When T.W.C. coincides with y_2 curve at all discharges.* This is the most ideal condition for jump formation. The hydraulic jump will form at the toe of the spillway at all discharges. In such a case, a simple concrete apron of length $5(y_2 - y_1)$ is generally sufficient to provide protection in the region of hydraulic jump, as shown in Fig. 21.31 (*a*).

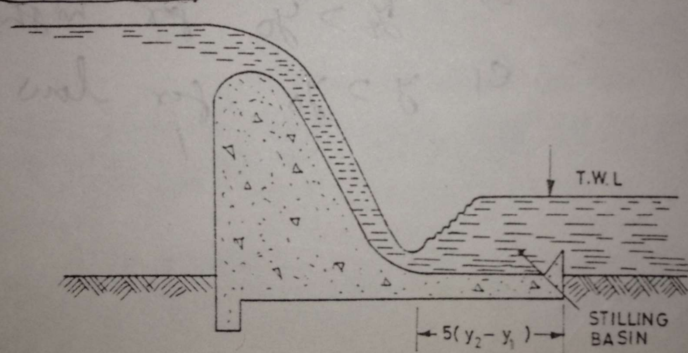


Fig. 21.31. (*a*) Simple horizontal apron.

21.11.1.2. Energy dissipators for case (b) : *When T.W.C. is lying above the y_2 curve at all discharges.* In this case, when y_2 is always below the tail water, the jump forming at toe will be drowned out by the tail water, and little energy will be dissipated. Water may continue to flow at high velocity along the channel bottom for a considerable distance.

The problem can be solved :

(*i*) by constructing a **sloping apron above the river bed level** as shown in Fig. 21.31 (b_1). The jump will form on the sloping apron where depth equal to y_2 (lesser than the

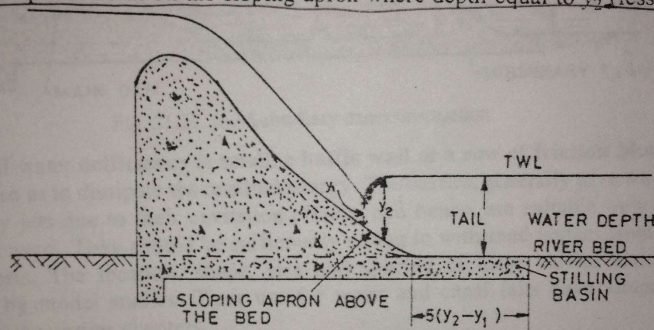


Fig. 21.31. (b_1) Sloping apron above the bed.

tail water depth at toe) is available. The slope of the apron is made in such a way that proper conditions for a jump will occur somewhere on the apron at all discharges. A lot of extra concreting is required to be done, as shown.

(*ii*) A second solution of this problem can be in the form of providing a **roller bucket** type of energy dissipator. It consists of an apron, which is upturned sharply at ends, as shown as in Fig. 21.31 (b_2).

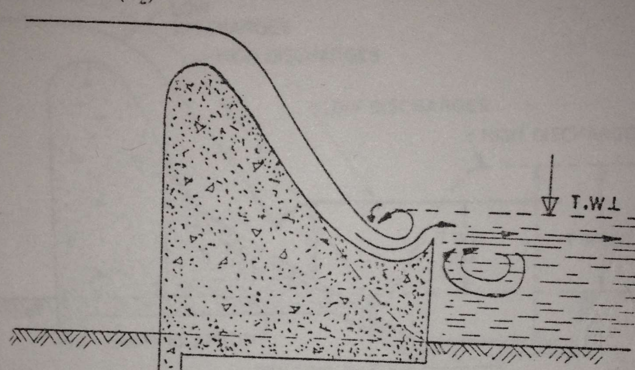


Fig. 21.31. (b_2) Roller Bucket.

Two main rollers are formed which dissipate the energy due to internal turbulence.

The roller which is formed downstream of the bucket, tends to move the scoured bed material towards the dam, thus, preventing serious scour at toe of the dam. Sometimes, the scoured material may enter the bucket under the action of u/s roller, and may cause severe abrasion. A *dentated bucket lip* may, therefore, have to be provided, so as to permit removal of material caught in the bucket.

21.11.1.3. Energy dissipators for case (*c*) : *When T.W.C. lies below the* y_2 *curve at all dischages.* (*i*) If the tail water is very low, the water may shoot up out of the above bucket, and fall harmlessly into the river at some, distance downstream of the bucket. *This bucket is then known as* **ski jump bucket** and can be used for energy dissipation in case (*c*) : *i.e.* when the tail water depth is insufficient or low at all discharges. The ski jump bucket type of an energy dissipator requires sound and rocky river bed, because a part of the energy dissipation takes place by impact, although some of the energy is dissipated in air by diffusion and aeration.

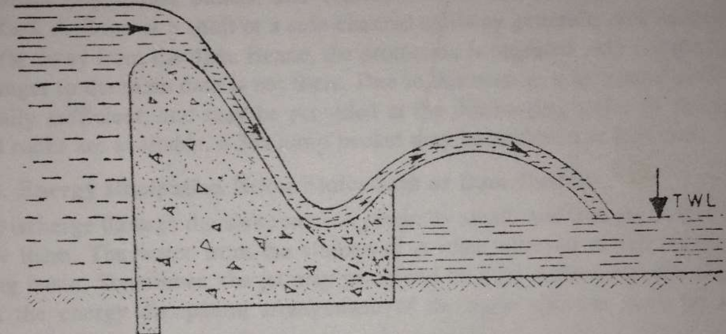


Fig. 21.31. (c_1) Ski jump bucket.

(*ii*) The second solution to the problem can be the *provision of a* **sloping apron** as *in case* (*b*) *but* **below the river bed,** as shown in Fig. 21.31 (c_2). The required depth y_2

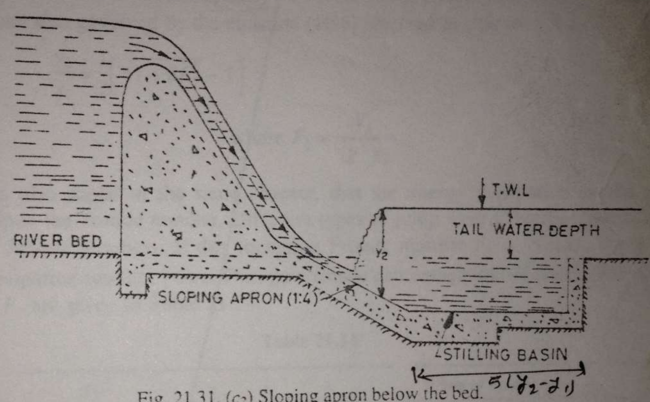


Fig. 21.31. (c_2) Sloping apron below the bed.

which is greater than T.W. depth, can thus be made available by letting the jump form on this sloping apron as shown. This sloping apron and the horizontal cistern of length $5(y_2 - y_1)$ shall be entirely in cutting and may be expensive, though otherwise quite satiactory.

(*iii*) The third solution to this problem may be the construction of a **subsidiary dam** below the main dam, so as to increase the tail water depth and cause a jump to form at the toe of the main dam, as shown in Fig. 21.31 (c_3).

(*iii*) The third solution to this problem may be the construction of a subsidiary dam below the main dam, so as to increase the tail water depth and cause a jump to form at the toe of the main dam, as shown in Fig. 21.31 (c_3).

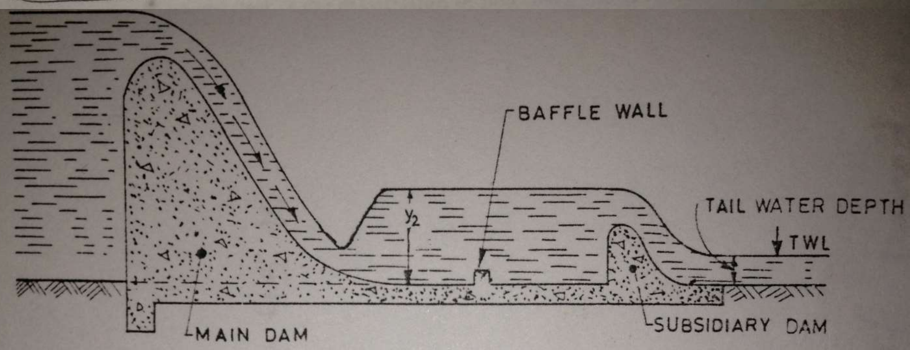


Fig. 21.31. (c_3) Subsidiary dam construction.

If the tail water defficiency is small, a **baffle wall** or a **row of friction blocks** may be provided so as to dissipate the residual energy. The baffles, generally give way, under high velocity jets due to their cavitation effects, and hence, are suitable only for low spillways or weirs. They should be sufficiently strong to withstand impact from ice and floating debris. The location, shape, size and spacing of these baffles can be best determined by model studies. Their use for weirs and canal falls have already been explained in the earlier chapters.

21.11.1.4. Energy dissipators for case (*d*). *When T.W.C. lies above the y_2 curve at low discharges and lies below the y_2 curve at high discharges.* In this case, at low discharges, the jump will be drowned and at high discharges, tail water depth is insufficient. The solution to the problem lies in providing a **sloping apron partly above and partly below the river bed** as shown in Fig. 21.31 (*d*). The horizontal apron and end sill should also be provided.

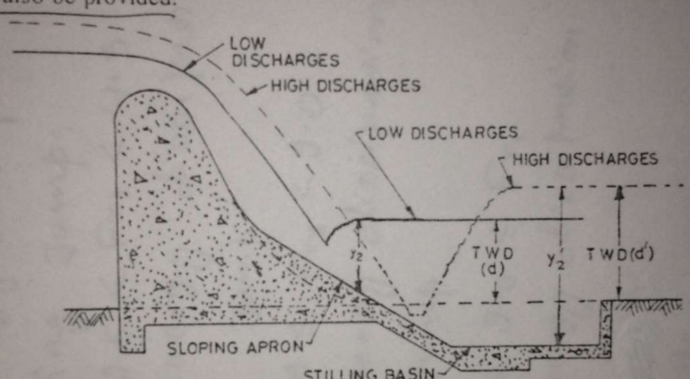


Fig. 21.31. (*d*) Sloping apron partly above and partly below the ground level.

At low discharges, the jump will form on the apron above the river bed, where the available depth is equal to the required depth and less than the T.W. depth. Similarly, at high discharges, the jump will form on the apron below the river bed, where the available depth is more than the T.W. depth and equal to the depth required for jump formation.

21.11.1.5. Energy dissipators for case (e). *When tail water depth is insufficient at low discharges and is greater at high discharges.* This case is just the reverse of case (d) and the same arrangement which was made in case (d) will serve the purpose. The only difference will be that at low discharges, the jump will form on the apron below the bed; and at high discharges, the jump will form on the apron at a point above the bed.

6.0 Design of stilling basin:

Types of Jump:

i) Undular Jump:
- $1 \le Fr_1 < 1.7$
- energy dissipation is about 5%

ii) weak Jump:
- $1.7 \le Fr_1 < 2.5$
- energy dissipation is about 20%

iii) Oscillating Jump:
- $2.5 \le Fr_1 < 4.5$
- energy dissipation between 20 – 45%.

iv) Steady Jump:
- $4.5 \le Fr_1 < 9.0$
- energy dissipation 45 – 70%

v) Strong Jump
- $9.0 \le Fr_1$
- energy dissipation 70 – 80%

Stilling basin:

A stilling basin is a short length of paved channel placed at the end of spillway or any other source of supercritical flow. The aim of the designer is to make a hydraulic jump form within the basin, so that the flow is converted to subcritical before it reaches the exposed and unpaved riverbed downstream.

Appurtenances usually associated with stilling basins are:

i) chute blocks
ii) sill → (Dentated or solid)
iii) Baffle piers (or Baffle blocks or Basin blocks)

i) chute blocks:

Chute blocks are used in the approach section to channalize the flow and lift part of the flow off the bed. The purpose of these blocks is to shorten the length of the jump and stabalize it.

ii) sill:

The dentated or solid sill occurs at the end of the basin. The function of this appurtenance is to further reduce the length of the jump and control scour. In large stilling basins, the sill is usually dentated to aid in the diffusion of the high velocity jet that may reach the end of the basin.

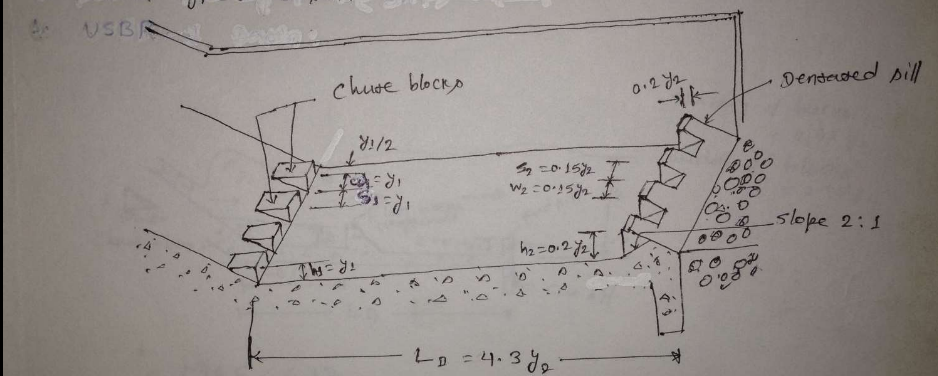
iii) Baffle piers:

Baffle piers are placed at intermediate locations in the basin, and the primary function is the dissipation of energy by impact. When the approach section velocities are low, baffle piers can be very effective, however, when incoming velocities are high, this type of appurtenance may be inappropriate because of the possibility of Cavitation. In some situations, baffle piers must be designed to withstand the impact of debris and ice.

Stilling Basin Design:

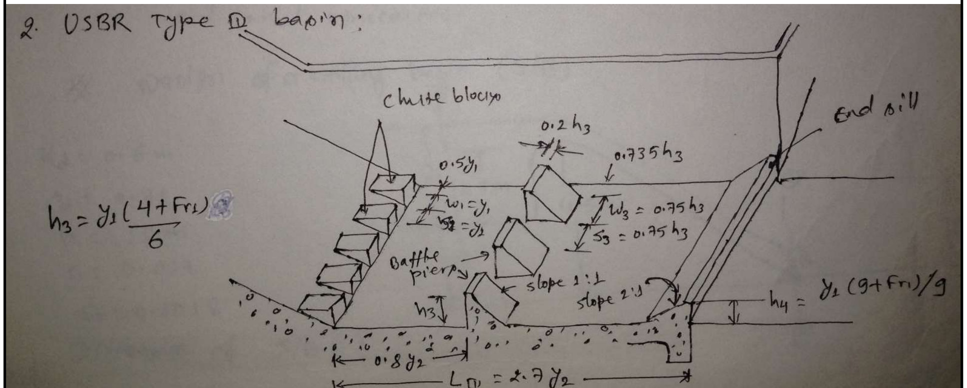
A number of generalized designs have been developed based on model studies, experience, and the observation of existing stilling basins. The generalized designs are

1. USBR Type II basin:



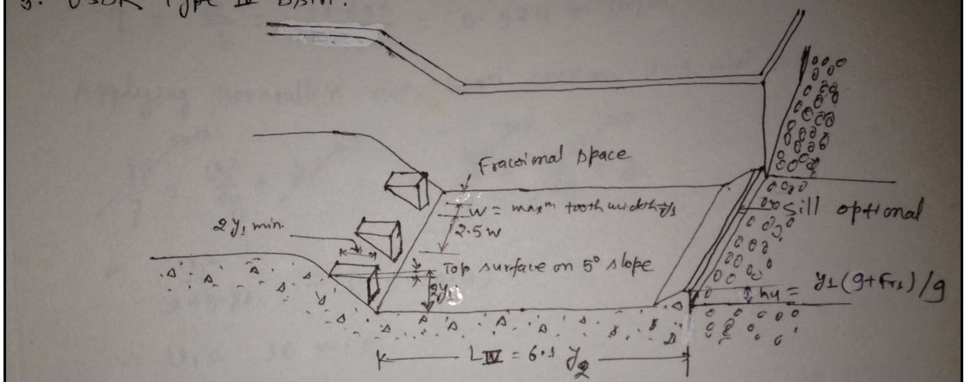
$Fr_1 \geq 4.5$, $v_1 \geq 20$ m/s, $TW = 0.97 y_2$

2. USBR Type III basin:



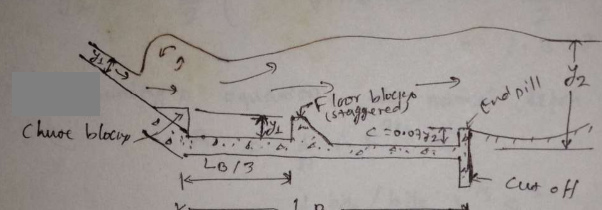
$Fr_1 \geq 4.5$, $v_1 \leq 20$ m/s, $TW = 0.83 y_2$

3. USBR Type IV basin:



$2.5 \le Fr_1 \le 4.5$, $TW = y_2$

4. Saint Anthony Falls (SAF) basin:



spacing of blocks $= 0.75 y_1$

width of blocks $= 0.75 y_1$

- $1.7 \le Fr \le 17$
- used in conjunction with small spillways, outlet works, and canal structures.

※ Design of a stilling basin (S/B):

$H_d = 0.6\ m$

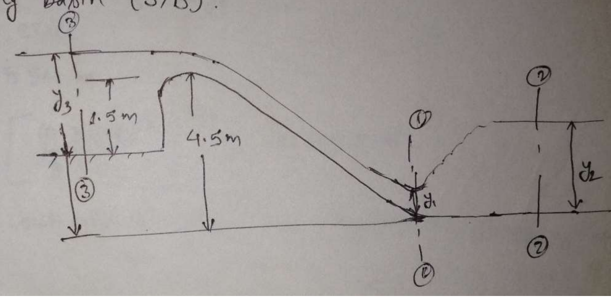
$C_d = 0.71$

$b = 12\ m$

$n = 0.014$

$S_0 = 0.0012$

Dimension of S/B = ?



$$Q = \frac{2}{3} C_d\, b \sqrt{2g}\, H_d^{3/2} = \frac{2}{3} * 0.71 * 12 * \sqrt{2*9.81} * 0.6^{3/2}$$

$$= 11.693\ m^3/s$$

$$\therefore q = \frac{Q}{b} = \frac{11.693}{12} = 0.974\ m^3/s/m$$

Applying Bernoulli's eqn. betn sections 1-1 and 3-3.

$$\cancelto{0}{\frac{p_1}{\gamma}} + \frac{v_1^2}{2g} + \cancelto{0}{y_1} = \cancelto{0}{\frac{p_3}{\gamma}} + \cancelto{0}{\frac{v_3^2}{2g}} + y_3$$

or, $$\frac{v_1^2}{2*9.81} = (4.5 + 0.6)$$

$$\therefore v_1 = 10\ m/s$$

$$\therefore y_1 = \frac{q}{v_1} = \frac{0.974}{10} = 0.097\ m$$

$$Fr_1 = \frac{v_1}{\sqrt{g y_1}} = \frac{10}{\sqrt{9.81*0.097}} = 10.25$$

$$\therefore y_2 = -\frac{y_1}{2}\left(1-\sqrt{1+8Fr_1^2}\right) = \frac{-0.097}{2}\left(1-\sqrt{1+8*(10.25)^2}\right)$$

$$= 1.358 \text{ m}$$

Using Manning's equation, the normal depth y_0 is given by

$$Q = \frac{1}{n} A R^{2/3} S_0^{1/2}$$

$$= \frac{1}{n} b y_0 \left(\frac{b y_0}{b+2y_0}\right)^{2/3} S_0^{1/2}$$

or, $$11.693 = \frac{1}{0.014} * 12\, y_0 \left(\frac{12\, y_0}{12+2y_0}\right)^{2/3} * (0.0012)^{1/2}$$

or, $$y_0 = 0.394\left(\frac{12+2y_0}{12 y_0}\right)^{2/3}$$

By trial and error

$$y_0 = 0.594 \text{ m}$$

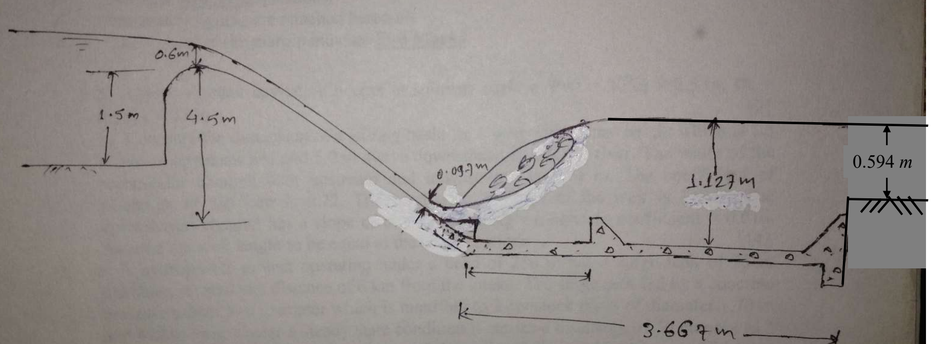
$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left[\frac{(0.974)^2}{9.81}\right]^{1/3} = 0.459 \text{ m}$$

As $y_0 > y_c \Rightarrow$ subcritical flow i.e. hydraulic jump occurs.

As $\left.\begin{array}{l} V_1 < 20 \text{ m/s} \\ Fr_1 > 4.5 \text{ m} \end{array}\right\}$ select USBR Type-III basin.

Now,

$$\left.\begin{array}{l} TW = 0.83\, y_2 = 0.83 * 1.358 = 1.127 \text{ m} \\ y_0 = 0.594 \text{ m} \\ L_B = 2.7\, y_2 = 2.7 * 1.358 = 3.667 \text{ m} \end{array}\right\} \text{Ans.}$$



Settling Basin / Desilting Basin / Desander

Settling basin is the hydraulic structure constructed to settle down the sediment particles which are objectionable to hydromechanical equipment (harmful to crops incase of irrigation projects) and flush the settled particles back to the sources.

Classification of sediment particles:

According to grain size

1. Boulders → > 250 mm
2. Cobbles → 250 – 60 mm
3. Gravel → 60 – 2 mm
   i) Coarse gravel → 60 – 20 mm
   ii) Medium " → 20 – 6 mm
   iii) Fine " → 6 – 2 mm
4. Sand → 2 – 0.06 mm
   i) Coarse sand → 2 – 0.6 mm
   ii) Medium " → 0.6 – 0.2 mm
   iii) Fine " → 0.2 – 0.06 mm
5. Silt → 0.06 – 0.002 mm
   i) Coarse silt → 0.06 – 0.02 mm
   ii) Medium " → 0.02 – 0.006 mm
   iii) Fine " → 0.006 – 0.002 mm
6. Clay → < 0.002 mm.

Effects of sediment on Hydro-electric projects:

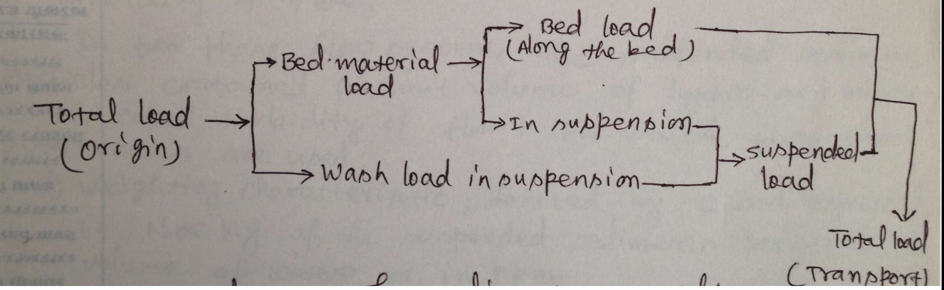
→ wear and tear of hydromechanical equipments such as turbine blades, guide vanes, guide vane rings, steel pipes / lining, valves and other hydraulic structures.

→ chocking on valves, flushing channels, inlet, monitoring devices, sediment samplers, current meters, turbidimeter, etc.

* Wear and tear depends on shape, size and hardness of sediment particles.

* The effect of sediments is most severe in high head plants.

Modes of sediment transport:



Necessary degree of sediment removal:

1. According to Dr. Haakon Stole,
   Most of the particles bigger than 0.15 - 0.30 mm must be excluded to minimize the cost related to turbine wear and tear and generation losses during operation and maintenance of turbines.
2. According to Mosonyi,
   - For medium head plants (H = 50 - 300 m), particles > 0.2 to 0.5 mm should be excluded.
   - For high head plants (H > 300 m), particles > 0.1 to 0.2 mm should be excluded.
   - For plant with very high heads, of several 100 m, particles > 0.01 to 0.05 mm should be excluded.

* Lower limit should be considered critical if the fine sediment fractions include sharp-edged ~~quart~~ quartzite grains.

Characteristics of suspended sediments:

1. Settling velocity:

The falling velocity of suspended sediments in the settling basin is called settling or sinking velocity of the sediments and denoted by ω in cm or mm per second.

2. Turbidity of flow:

In two phase flow system, the suspended sediment particles contained in unit volume of hydro-soil mixture is called turbidity of flow. For which, two characteristics are used:

i) weighting characteristic; denoted by **c** and expressed in N or Kg of the suspended sediments per unit volume of water or in ppm.

ii) Volumetric characteristics; denoted by μ and expressed in litres of suspended sediments per unit volume of flowing water.

3. Transporting capacity of flow:

The limited weighted quantity of sediments to be moved by the flow in suspended condition is known as transporting capacity of flow. It is denoted by C_{tr} and expressed in the same unit of measurement as the flow turbidity.

4. Lifting velocity of flow:

Movement of sediment particles in suspended condition due to the vertical component of the flow velocity is called lifting velocity of flow

Parameters of settling basin design:

1. Particle fall velocity (ω):

Average terminal settling velocity (ω) of a particle falling in quiescent, distilled water of infinite extent depends on particle size, weight, shape and viscosity of water (temperature).

$$\omega = \left[\frac{4 g d \Delta}{3 C_d}\right]^{1/2}$$

where, C_d = drag coefficient $= \frac{24}{Re}$

Re = Reynold's number $= \frac{\omega y}{\nu}$

y = depth of flow

ν = Kinematic viscosity

$= 1.003 * 10^{-6}\ m^2/s$ at $20^\circ C$ temperature.

$\Delta = \frac{\rho_s - \rho}{\rho}$

ρ_s = density of sediment particle

ρ = " " water.

and d = particle size

For $d < 0.1$ mm, Stoke's law is valid.

According to Stoke's law,

$$\omega = \frac{1}{18} * \frac{\gamma_s - \gamma}{\gamma} * g \frac{d^2}{\nu}$$

2. Concentration of sediment or Turbidity (C):

In general, ω decreases with the increase of c due to interference of other particles. However, Tobson et al observed that under certain conditions ω of a particle increases due to group settling (due to flocculation of higher concentration of finer materials).

when $c > 2000$ ppm, interference effect become significant which reduce the velocity of coarse silt upto 10 % (average particles) and generally the effects are not significant in terms of ranges of sediment concentration and degree of accuracy of settling basin design.

3. Turbulence:

Turbulance is defined as irregular flow motion resulting from eddies that are carried by the flow and swirling in an irregular manner. Turbulence decreases the trap efficiency of basin as it retards the ω.

Methods of Design:

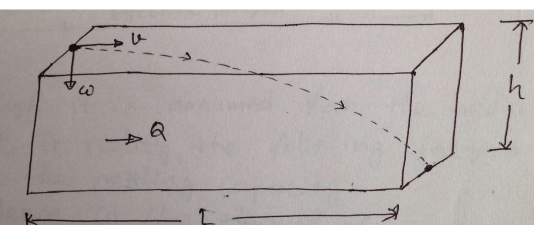
1. Particle approach:

Basin is designed assuming that the time required for settling and that required for travelling along the basin for the designed particle size are equal.

In this approach ideal basin is designed.

Assumptions:

- Flow is steady
- Flow is quiescent (no turbulance)
- Particles once settled do not come into suspension again.



Governing equations:

Settling time, $t_s = \frac{h}{\omega}$

Travelling time, $t_r = \frac{AL}{Q} = \frac{AL}{Av} = L/v$

Since Settling time = Travelling time

$$\frac{h}{\omega} = \frac{L}{v}$$

i.e., $\boxed{L = \frac{hv}{\omega}}$

Flow through velocity, $v \ngtr 30$ cm/sec.

According to Camp, the critical velocity

$$v = a\sqrt{d_{mm}} \quad \text{cm/sec.}$$

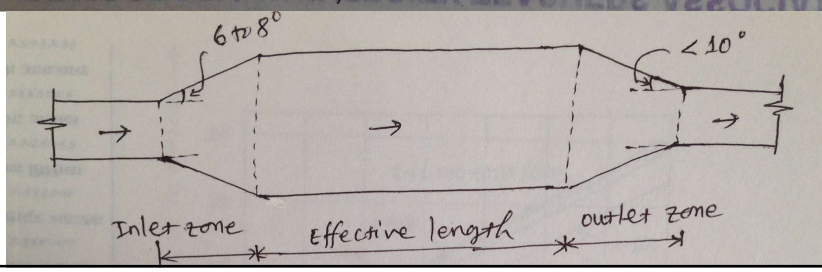
where, d = diameter of particles in mm

and a = constant

= 36 for $d > 1$ mm

= 44 for $1\text{ mm} > d > 0.1$ mm

= 51 for $0.1\text{ mm} > d$



Although it is assumed that the media is quiescent, in reality, the following factors mainly influence the settling capacity:

→ Turbulence in flow through the basin

→ Short-circuiting and currents within the basin caused by

- current set up caused by poor inlet and outlet conditions
- Expansion and contraction zones which generates recirculating eddies
- Boundary friction which retards some flow zones and causes faster flow
- wind induced surface currents
- Thermal effect or extremely high sediment concentration.

Effect of turbulence :

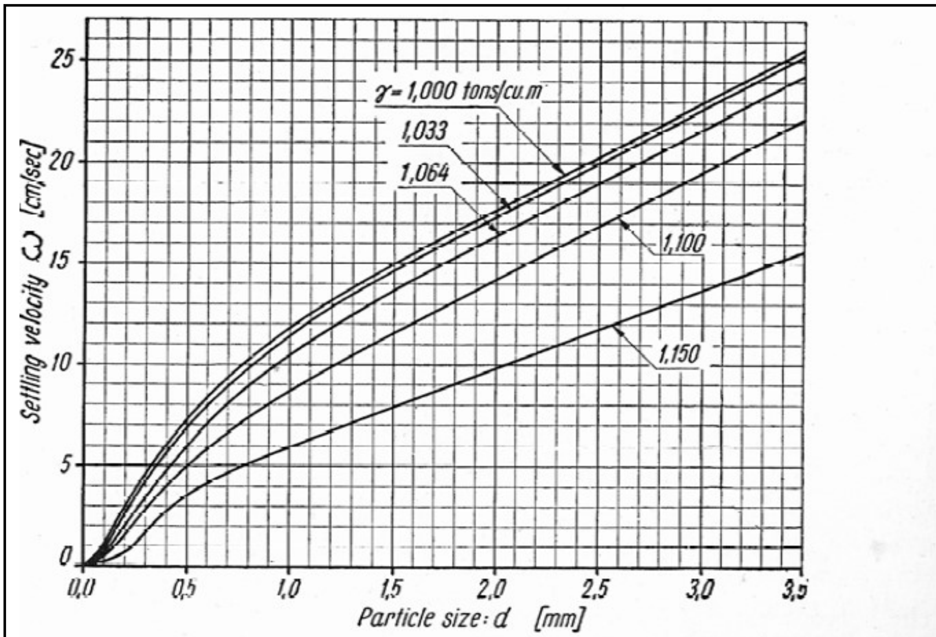
Turbulance is considered by using lower settling velocity i.e, $\omega - \omega'$

where, $\omega' = \alpha v$ with $\alpha = \frac{0.132}{\sqrt{h}}$

where, h = depth of flow.

$$\therefore \text{ Length, } L = \frac{hv}{\omega-\omega'} = \frac{hv}{\omega - \alpha v} = \frac{hv}{\omega - \frac{0.132}{\sqrt{h}} v}$$

i.e., $$\boxed{L = \frac{h^{3/2} v}{\sqrt{h}\,\omega - 0.132 v}}$$



Settling velocity in stagnant water plotted against the density of silty water and the particle diameter. (After *L. Sudry*)

Check:

i) Check for width: $B \geq 4.75\sqrt{Q}$

ii) Check for length: $L \geq$ Velikanov's L

iii) Check L and B ratio: $L/B \approx 4$ to 10

Velikanov's method:

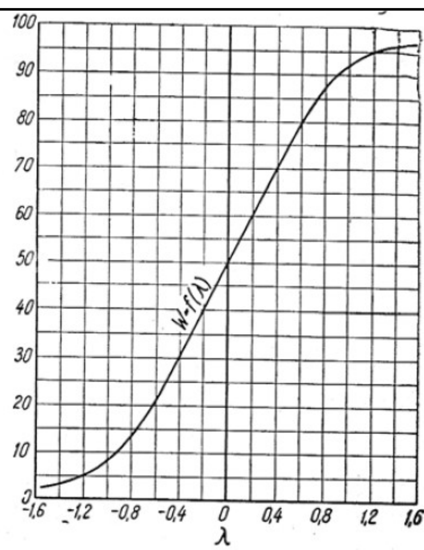
Based upon the calculation of probabilities, the necessary settling length for turbulent flow is computed from the settling velocity in stagnant water ω and the flow through velocity v.

The settling length,

$$l = \frac{\lambda^2 v^2 (\sqrt{h} - 0.2)^2}{7.51\,\omega^2} \quad \text{in m.}$$

where, $\lambda = f(W)$

$$W = \frac{\text{Settled sediment}}{\text{Total load entering with the flow}}$$



Velikanov's relationship
$w = f(\lambda)$ for designing settling basins

* As indicated by experience, satisfactory values can be obtained by using coefficients pertaining to a 95 to 98% removal of the limit particle size.

2. Concentration approach (Trap efficiency computation methods)

The optimum width and depth of a settling basin would be that which produces a flow velocity below the value necessary to initiate movement of the bed material. Favorable settling conditions need to be obtained where the mean water velocity in the basin is in the range of 0.1 to 0.4 m/s. Generally, a velocity of 0.2 m/s is recommended at the initial stage of settling basin design in Himalayan Rivers, based on net cross-sectional area for sedimentation (Støle, H, 1993). Enough space for the deposition of sediment also needs to be provided. The basin must be able to exclude the targeted particle size. Various combinations of depth and width together with length can be considered to achieve the targeted efficiency and the best combination is finally adopted.

There are several methods adopted for the design and computation of trapping efficiency of settling basins used for hydropower, water supply and irrigation projects. Some of the methods commonly used are mentioned below with brief introduction of each.

i) Camp's method
ii) Hazen's method
iii) Vetter's method
iv) Physical model
v) Three dimensional numerical analysis

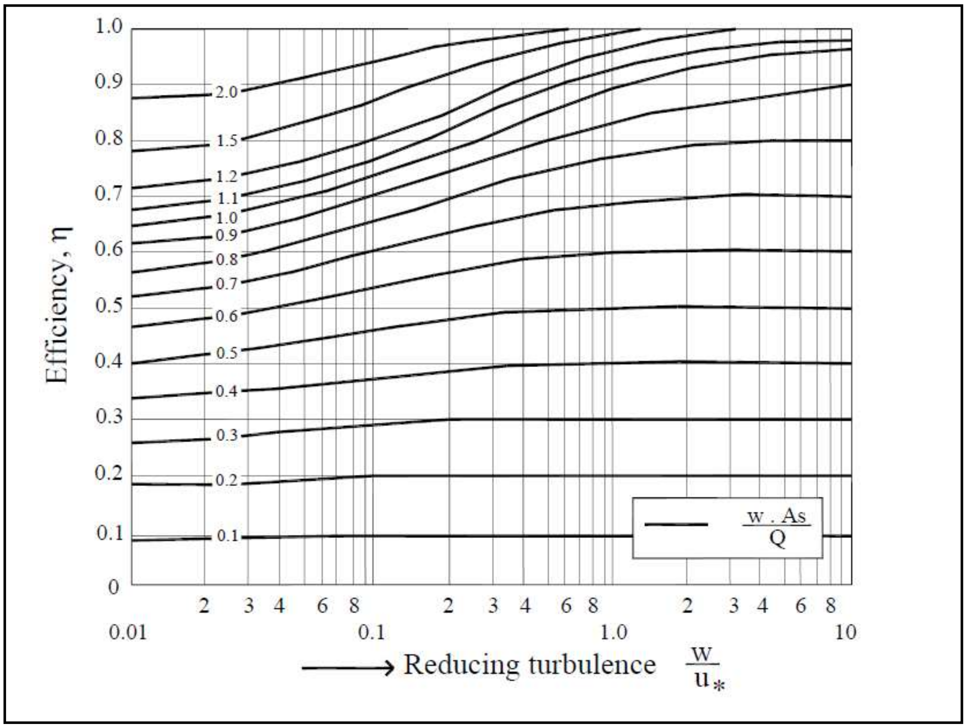
i) Camp's method

Camp's method is based on the classic approach to settling basin design. In this design Camp has made assumptions that; fluid velocity and the turbulent mixing coefficient are the same throughout the fluid and derived a relation as follows:

$$\eta = f\left\{\frac{wA_s}{Q}, \frac{w}{v_*}\right\}$$

where; η is the **t**rapping efficiency, A_s is the basin surface area (m^2), v_* is the shear velocity $=\sqrt{(\boldsymbol{\tau}_o/\rho)} = \sqrt{(gRS_e)}$, R is the hydraulic depth (m), S_e is the hydraulic gradient, $S_e=[Q/(MAR^{2/3})]^2$ and M is the Manning's number (1/n).

w/v_* is regarded as a dimensionless indicator of the effect of the fluid turbulence on a given particle size. The trapping efficiency is read in the figure shown below for the computed values of w/v_* and wA_s/Q.



ii) Hazen's method

This method of design of a settling basin accounts for the effect of both turbulence and imperfect flow distribution, which in real situation, is true by a general classification of basin performance.

The formula proposed by Hazen is given by;

$$1-\eta=\left[1+\frac{mwA_s}{Q}\right]^{-\frac{1}{m}}$$

where; m = performance parameter (m=0 for best and m=1 for very poor).

The drawback of this equation is that several different physical effects are combined into a single parameter 'm'. It is therefore better for the designer to consider each effect separately, where possible.

iii) Vetter's method

Vetter's method of computing efficiency of a settling basin is virtually identical to the equation proposed by USBR (Vanoni, A. 1977) which is given by the formula;

$$\eta = 1 - e^{-\frac{wA_s}{Q}}$$

In other words this is simply the best performance solution of Hazen's equation i.e. curve for m = 0. Vetter's equation is also corresponds to the turbulent side of Camp's solution and thus to implicit conditions of turbulence.

iv) Physical model test

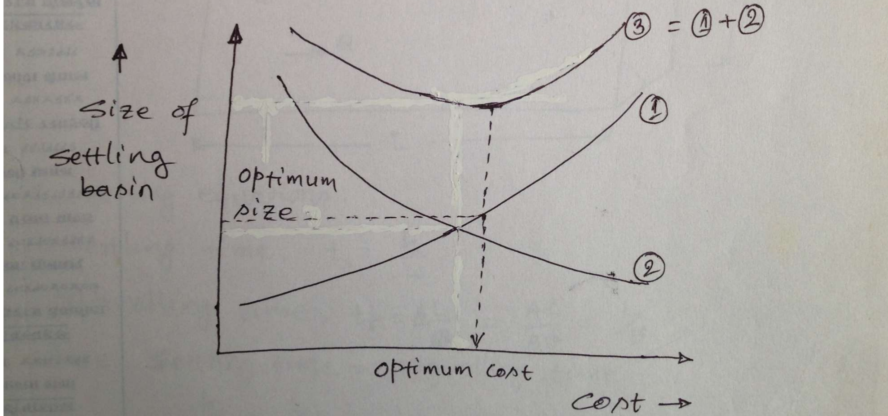
In order to simulate the hydraulic conditions of the flow in the inlet as well as in the basin a well accepted method of design basis is the physical hydraulic model study. The model provides opportunity to check the flow pattern and to adjust the structure if necessary, which in turn helps in getting good efficiency with low cost. It is almost impractical to construct a full scale structure for the testing purpose. So, a model is prepared in a laboratory, which represents the real structure called the prototype. The relationship between model and prototype performance is governed by the laws of hydraulic similarity, i.e. geometric, kinematic and dynamic similarity (Webber, NB, 1995). There are various similarity laws such as Eular law, Froude law, Reynolds law, and Weber law.

Settling Basin Design

It is impossible to design a settling basin which can remove all the suspended sediments coming into it from economic point of view. The best combination of the following items needs to be analysed with respect to cost and benefit to obtain the optimum efficiency of the basin during the design.

- Construction cost of the settling basins.
- Initial cost of the turbines and auxiliaries with an objective of wear resistance depending upon the type of turbine and quantity of erosion due to sediments.
- Cost of overhaul and replacement of the components.
- System for monitoring and operation of the power plant with an objective of reducing the sediment exposure by partial or full close-down during periods of high sediment concentration.
- Cost of generation loss during flushing if the plant is to be designed for power plant close down during flushing.

Design criteria of settling basin:



① → Cost of basin

② → i) Cost of overhead and replacement and
ii) Cost of turbine and its accessories with respect to wear and tear

Design criteria

The design of a settling basin aims to meet the following criteria:

- Determination of the maximum size of particles which can enter to the turbine without causing major damages and facilitate exclusion of 95 to 100% of this and larger size particles.
- To manage with as minimum width as possible.
- To use the length that is available depending on the topography and intake location (topographical limitations).
- To optimize sediment exclusion with respect to the cost parameters. Cost parameter in this context are; cost of down time i.e generation loss, cost of repair and maintenance of hydraulic machinery as well as civil, initial cost of the underwater machinery and the construction cost of the basin itself.
- To secure as high generation regularity as possible. To achieve this, the power plant should be in operation most of the time and the basin should have adequate facility of flushing out the sediments with minimum loss of generation.

Design for Sediment Flushing

Design of chamber flushing of the settling basin shall involve determination of the transporting capacity of the flow for sediment flushing and the time of flushing.

The flushing capacity shall be determined as (Zhurabov, 1975)

$$\rho_{tr} = \frac{(V_f - 0.35)^3}{h_f^2}$$

where ρ_{tr} is the mass of sediments transported in kg/m^3, V_f is the flushing velocity in m/s and h_f is the flow depth during flushing in m. The above equation may also be used to determine the flushing velocity if the transporting capacity of the flow, ρ_{tr}, is known, and then the flushing gallery bed slope of the settling basin may be obtained from Manning's equation below.

$$S_{ch} = n^2 V_f^{\,2}/R^{4/3}$$

where n is the Manning's coefficient and R is the hydraulic mean radius of the flow section. The flushing duration of the settling chamber shall be determined by the expression

$$t_f = (16.7\,\gamma_s\,0.6\,V_{dead}) \,/\, [(\rho_{tr} - \rho_o)\,Q_f]$$

where t_f is the flushing time in minutes, γ_s is the unit weight of sediment to be flushed, V_{dead} is the dead volume of chamber of the settling basin, Q_f is the adopted flushing discharge in m^3/s, ρ_o is the sediment concentration of the flow entering the chamber during flushing, usually taken to be the sediment concentration of river flow.

Storage volume:

sediment storage volume in settling basin is given by

$$V = \eta\, c\, Q\, t$$

where, η = trap efficiency

c = sediment concentration

Q = discharge

t = sediment storage time.

Usually storage volume corresponding to 7 days storage due to monsoon flow is allocated in the settling basin.

Flushing of settled particles

In order to ensure the production regularity to an acceptable limit the particles settled into the basin has to be flushed out frequently. Settling basins are designed for a certain capacity of storing sediments. When the capacity exceeds then the deposited material will tend to reduce the cross-sectional area of the basin causing the increase in transit velocity and ultimately decreasing the trapping efficiency of the basin. The deposition rate in the settling basin is very much dependent on the suspended sediment concentration in incoming water and the particle size distribution for a certain hydraulic conditions. Hence, the storing capacity of the basin decreases with increase in the concentration, which demands a higher flushing frequency.

There are various types of flushing system designed for different projects depending on the economic criteria. Basically, flushing system can be divided on the basis of plant operation point of view which is given below.

i) *Power plant close down during flushing.*
- conventional gravity flushing.
- mechanical removal
- manual removal (small scale project only)

ii) *Power plant in operation during flushing*
- continuous flushing
- intermittent flushing, this includes; hopper system, the Beri system, the S4 system, the slotted pipe ejectors, dredging, scrapers, etc.

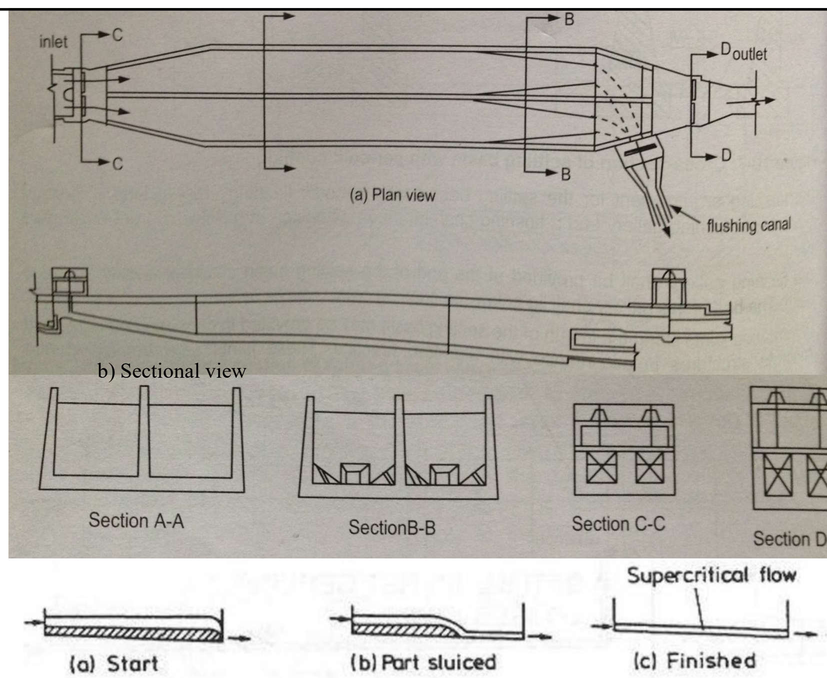
Types of Settling Basin

1. Conventional Type of Settling Basin
2. Hooper Type of Settling Basin
3. Bieri Type of Settling Basin
4. Serpent Sediment Sluicing System (4S) Type of Settling Basin
5. Spilt and Settle Type of Settling Basin
6. Slotted Pipe Type of Settling Basin

1. Conventional Type of Settling Basin

Before removal of deposits operation of basin is stopped and it is dewatered. Deposited sediments are removed mechanically by scrapers or by manually. So to keep continuous sediment removal one spare desilting basin is required. Some times due to power requirement such a situation arise when power has to be generated without flushing, this temporarily fulfill the power requirement but in long run turbine maintenance cost increase.

Also by generating a swift current inside the basin during conventional gravity flow, scouring of deposits can be achieved. This involves operation of flushing gates in addition to the gates at the both end of the basin.

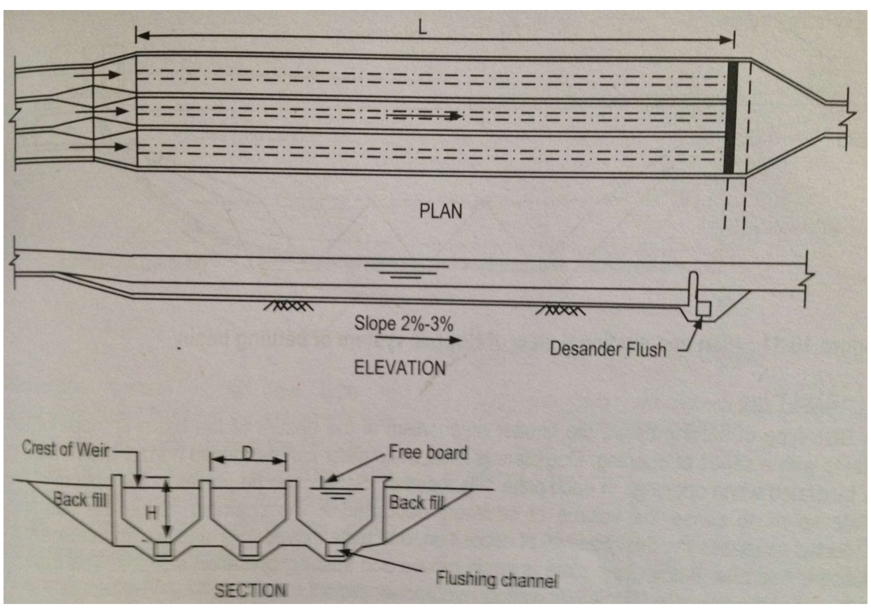


Sediment sluicing (Diagrammatic)

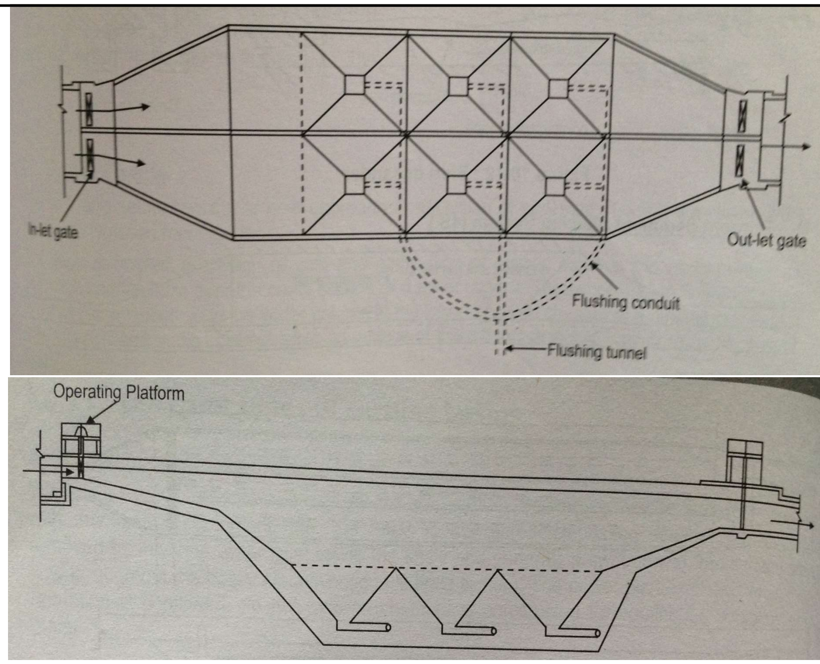
2. Hooper Type of Settling Basin

It is one of the most common type with longitudinal hoppers with a flushing conduit running parallel to hopper. There are evenly spaced openings, oriented perpendicular to the longitudinal axis of basin, connecting the hopper to flushing conduit. Flushing conduit increase in size in the direction of flow, also some time number of flushing conduits are provided in parallel, where every conduit carries part of flushing discharge, from a portion of desilting basin.

A constant velocity is maintained in flushing conduit. It provides a constant pressure difference between the basin and conduit, thus even abstraction of water from basin along the bottom of the hopper. A flushing gate is provided at the downstream end of flushing conduit. This gate has to be kept always open. The main problem with this system is that if deposition has occurred than flushing system can not be revitalized with normal operation of the flushing gate.



Plan and Section of Hooper Type Desander

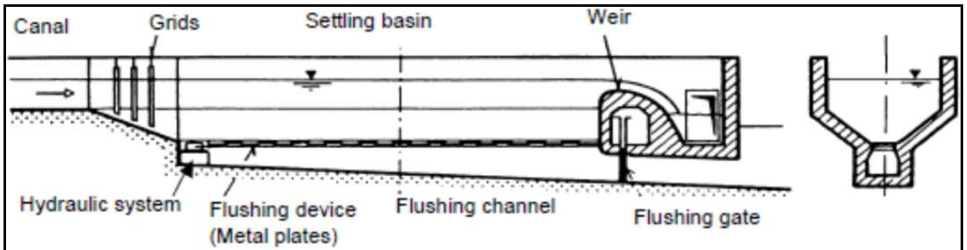


Plan and Sectional view of Hooper System Settling Basin

When flushing operation is intermittent there is no loss of water between two flushing operations. A hopper system with sediment ejection pipe at the bottom of each hopper is most common. Four sides of hoppers which are at bottom of desilting basin has slope of 40 to 45º. The system is consist of number of valves, bends etc,. so once it is chocked, it has to be kept out of operation for some time. Gravels in sediment or mal-operation may also chock it.

3. Bieri Type of Settling Basin

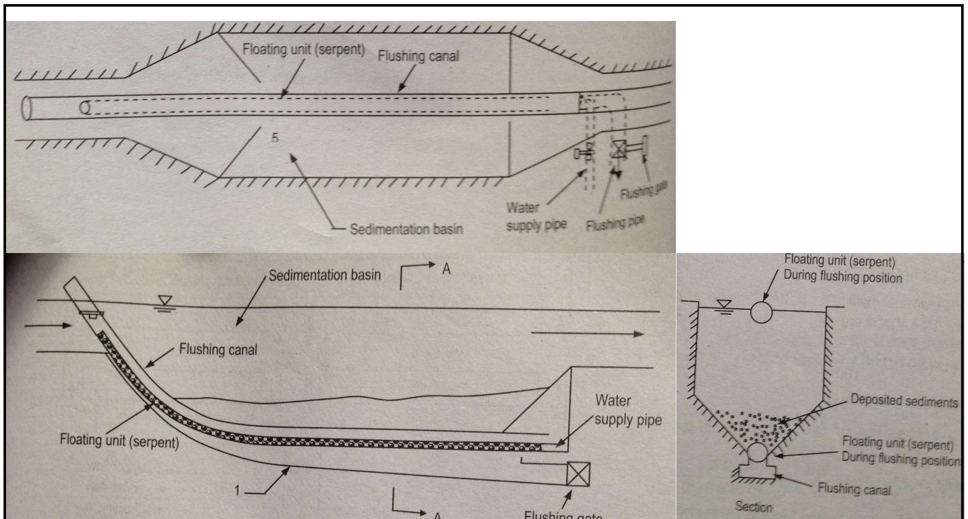
Bieri system have shutter mechanism at the bottom of the basin with two plates having series of openings. One plate is fixed and the other one is moveable with the help of servo motor. Flushing is facilitated when openings in each plate falls together. Sensor is placed at the bottom of the plate so as to sense the deposited volume of sediment in chamber. Once attaining the desired level of deposition, the moveable plate moves over fixed plate to flush the sediment. After flushing, the moveable plate again move to original position to stop flushing. When the sediment contains much quartz, wear and tear of plates are high. This type of settling basin is employed in Middle Marsyangdi Hydropower Project.

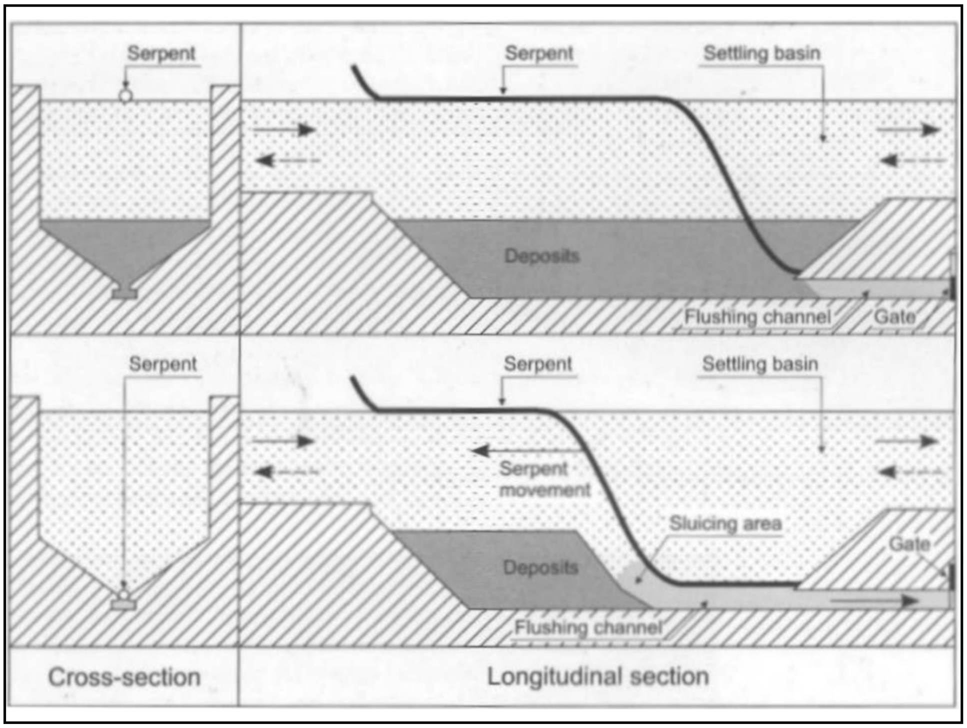


4. Serpent Sediment Sluicing System (4S) Type of Settling Basin

The serpent is a hollow heavy-duty rubber tube, which seals a longitudinal slit between the settling basin and a flushing canal along the bottom of the basin, when it is filled with water. A flushing gate at the downstream end of flushing conduit and a operational valve can fill up or empty the serpent.

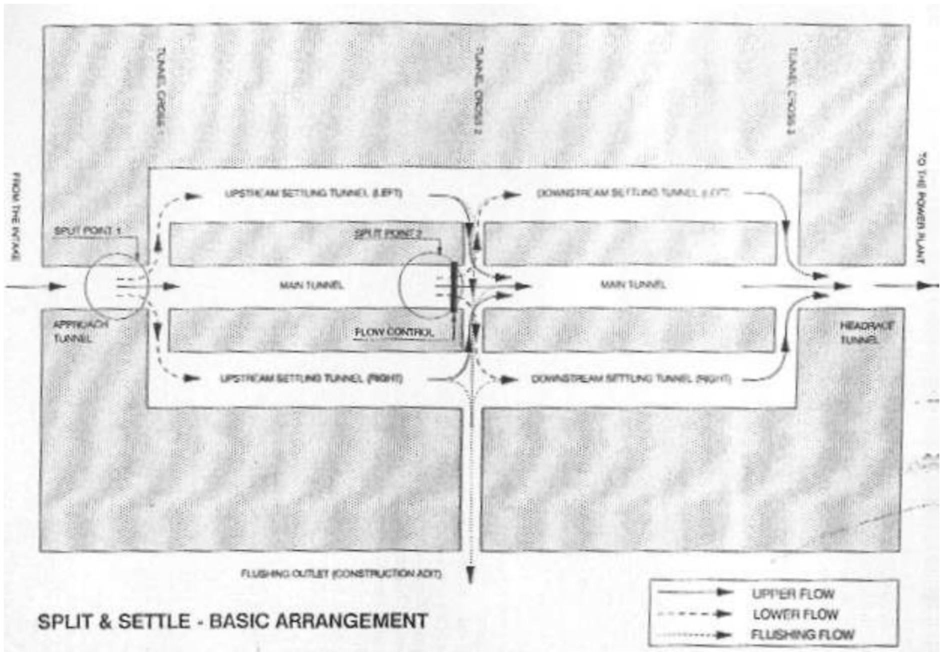
Sediments can be removed in two mode one opening mode, another closing mode. In first one serpent is gradually lifted from the slit (over flushing conduit), along the bottom of the basin, over to the surface, in second mode the serpent is gradually closing the slit, when serpent is filled up with water. One advantage is that flushing water consumption is 10% only during flushing.

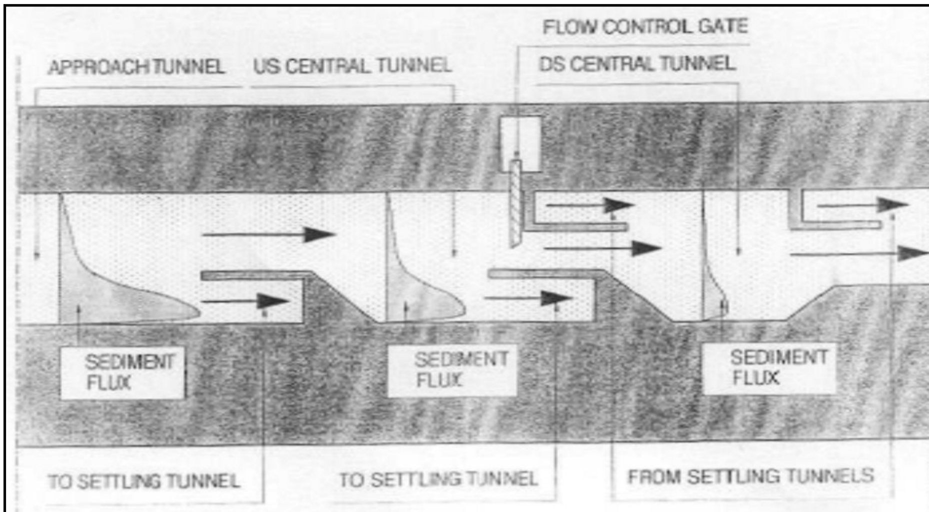




5. Spilt and Settle Type of Settling Basin

The flow in the tunnel upstream of settling basin is split horizontally at the first tunnel cross. The bottom water contains relatively more sediments than the higher up. The bottom water (say 20-40% of the total flow) is therefore diverted to the upstream settling tunnels running parallel to main tunnel. The settling tunnels are processing the dirtiest part of the water flow. The transit velocity is reduced in order to facilitate settling of a major part of the suspended load in relatively small caverns. The cleanest water flows in the main tunnel where the transit velocity has been reduced somewhat (60-80% of the velocity in the approach tunnel). Suspended sediments will therefore continue to accumulate in the lower segments of flow. The dirtiest water in the main tunnel may therefore be diverted once more in the second tunnel cross just upstream of the section where the cleaned water from the upstream settling tunnels are returned to the main tunnel. This water is then processed in the downstream desilting basin before the water in these basins are returned to the main tunnel in the third tunnel cross.

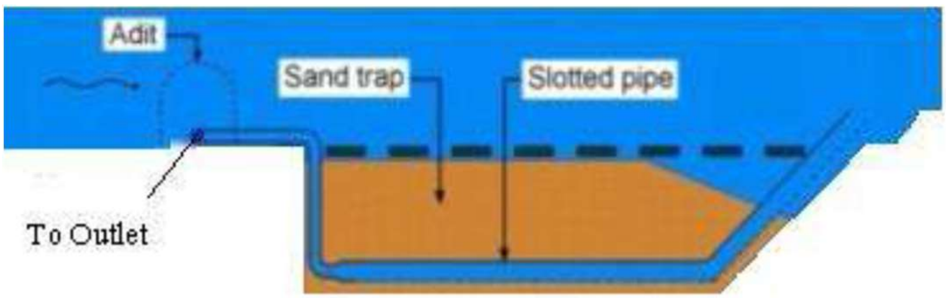




This concept removes the sediments without reducing the transit velocity of the entire flow to the settling level. The settling basins are located in serial pattern compare to conventional parallel pattern. Process can be repeated several times, if necessary. System is in use at Jhimruk, Nepal

6. Slotted Pipe Type of Settling Basin

Slotted pipe sediment excluder removes sediments from open and pressurized sand traps without interrupting water supply and power production. Sediments are removed without any input of power and without any movable parts. It suck blend of sediment and water and the available head between the sand trap and the outlet is normally used as driving force, thus no external energy input is needed, and the system can operate entirely without movable parts. System has been installed at Khimati Hydropower Project, Nepal is in use.



Example - 1: Design a settling basin for a high-head power plant by using the settling theory. The basin should serve to remove particles greater than 0.5 mm diameter from the water conveying mainly sand. The design discharge is 5 m3/sec and assume an initial value of 3.20 m for the depth of the basin.

Solution:

Determine first the permissible velocity flow velocity. Owing to economical considerations this should equal the critical velocity for which,

$v = 44\sqrt{d} = 44\sqrt{0.5} = 31.2\ cm/\text{sec}$

In designing the basin, v = 30 cm/sec flow velocity will be used. The following step is to determine the settling velocity according to the limit particle size of 0.5mm to be removed.

From the settling velocity – particle size Figure, w = 6 cm/sec (for γ = 1.064). The required length of the basin is,

$l = hv/w = 3.20 \times 30/6 = 16\ m$

And the width,
$b = Q/(hv) = 5/(3.2\times0.3) = 5.21\ m$

Checking: The settling time is, $t = h/w = 3.20/0.06 = 53.4$sec
The discharge conveyed during this period is, $V = Qt = 5\times53.4 = 267m3$
Should be equal to the capacity of the basin;
$V = hbl = 3.20\times5.21\times16.0 = 267m3$

Determine the length of the basin using identical basic values by the method of Velikanov's Figure for a removal ratio of 97% (W=0.97).
The Figure yields $\lambda = 1.50$ for W = 0.97. The length of the basin is,

$$l = \frac{\lambda^2 V^2 \left(\sqrt{h} - 0.2\right)^2}{7.51 w^2}$$

$$l = \frac{1.5^2 \times 0.3^2 \times \left(\sqrt{3.2} - 0.2\right)^2}{7.51 \times 0.06^2} \cong 19m$$

Example - 2: Compute for the conditions of the preceding example the settling length by considering the retarding effect of turbulent.

Solution:
The coefficient governing the reduction of settling velocity is,
$\alpha = 0.132/\sqrt{h} = 0.132/\sqrt{3.20} = 0.0737$
And thus the velocity decrement, $w' = \alpha v = 0.0737\times0.30 = 0.0221m/$sec
The settling length,
$l = hv/(w- w') = 3.20\times0.30/(0.060\text{-}0.0221) = 25.30\ m$
The unchanged width of the basin is,
$b = Q/(hv) = 5.0/(3.20\times0.30) = 5.21\ m$
And its capacity,
$V = hbl = 3.20\times5.21\times 25.30 = 422\ m3$

Example - 3: Compute the modified dimensions for a reduced depth of 2.40 m.

Solution:
$\alpha = 0.132/\sqrt{h} = 0.132/\sqrt{2.40} = 0.0851$
$l = hv/(w-\alpha v) = 2.40\times0.30/(0.06-0.0851\times0.30) = 20.90\ m$
Width of the basin is,
$b = Q/(hv) = 5.0/(2.40\times0.30) = 6.95\ m$
And the reduced capacity,
$V = lbh = 20.9\times6.95\times2.40 = 348m3$

Example - 4: A power plant is fed by a river carrying very coarse suspended sediment load. As indicated by the gradation curve obtained for the sediment, 70% are held on the 1 mm screen. Design a basin for a discharge of 12 m3/sec with the retarding effect of turbulent and a depth of 2.80 m will be taken.

Solution:
In order to protect the turbines the entire over 1 mm diameter should be settled.

$100\ Cp/C = 30\%$

The critical velocity is, $v = 44\sqrt{d} = 44\times\sqrt{1} = 44\ cm/\text{sec} > 30\ cm/sec$
Adopt $v = 30\ cm/sec$
The settling velocity in stagnant water is obtained from the Figure (for $\gamma = 1.064$) $w = 10.4$ cm/sec. The settling velocity decrement due to the turbulent, $w' = \alpha v = (0.132/\sqrt{h})v = (0.132/\sqrt{2.80})\times0.30 = 0.0237\ m/\text{sec}$
The settling length,
$l = hv/(w- w') = 2.80\times0.30/(0.104-0.0237) = 10.46\ m$
The required width of the basin is,
$b = Q/(hv) = 12.0/(2.80\times0.30) = 14.29\ m$
Compute the length of the basin also by the equation of Velikanov (W = 0.97), $\lambda = 1.50$,

$$l = \frac{1.50^2 \times 0.30^2 \times \left(\sqrt{2.8} - 0.2\right)^2}{7.51 \times 0.104^2} = 5.41\ m$$

Adopt $l = 10.5\ m$, $b = 14.5\ m$ and $h = 2.8\ m$.

Water Conveyance Structures

Tunnel:

Tunnel is the structure constructed for the conveyance of flow or for transportation purpose. Usually in hydro technical practice, tunnel is used for the conveyance of flow.

Classification of tunnels:

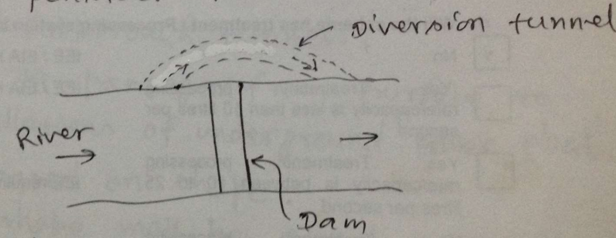
1. classification based on function served:

a) water carrying tunnels

These are the tunnels which serve for conveying water. They are further classified as

i) Diversion tunnels:

These are the tunnels constructed to divert the river flow away from the construction area. On completion of the dam these are converted into spillway tunnels / penstock tunnels.



ii) Headrace tunnels/Power tunnels / Pressure tunnels:

These are the tunnels which convey water from dam/ barrage to the power plant.

iii) Tailrace tunnels:

These are the tunnels which carry away the water from power plant. These are usually free flow tunnels. When the turbine setting is below tail water level, these are pressure tunnels.

iv) Spillway tunnels:

These are the tunnels which act as spillway to dispose of flood water, usually called shaft spillway.

b) Non-water conveying tunnels:

These are the dry tunnels also called service tunnels which serve the purposes other than conveying water. These are further classified as

i) Cable tunnels:

These are meant to convey cables from the power house to switch yard in underground power plants.

ii) Ventilation tunnels:

These are the tunnels provided to convey fresh air to the underground tunnels.

iii) Adit tunnels:

These are provided to facilitate starting tunnelling at various heads along main tunnel.

iv) Access tunnels / Approach tunnels:

These serve as passage from surface to underground installations of underground power plant.

2. Classification based on shape:

The tunnels shape may be circular, rectangular or any combination of these forms such as horse shoe shape, D-shapped, egg shaped, etc. Circular tunnels are structurally more suitable and more stable. Horse-shoe shape tunnels are very convenient from the construction point of view.

3. Classification based on protective surface:

i) Lined tunnel:

Lining is a protective layer of concrete, RCC or steel on the inner surface of the tunnel. The power tunnels are usually lined to economise on loss of head, smaller cross-section and hydraulically efficient.

ii) Unlined tunnel:

Tunnels in good sound rock can be left unlined.

4. Classification based on alignment:

i) Tunnel:

The ~~term~~ term tunnel implies here the one with a slight bed slope or vertually a horizontal tunnel.

ii) Shaft:

It is a tunnel with alignment vertical. Inclined shaft is one which is steeply inclined to the horizontal.

5. Classification based on characteristics of flow:

i) Low pressure tunnel:

It is a tunnel with low internal water pressure i.e, water head less than 10 m. Diversion tunnels are usually low pressure tunnels.

ii) Medium pressure tunnel:

It is a tunnel with water head of 10 to 100 m.

iii) High pressure tunnel:

High pressure tunnel has high internal pressure corresponding to a head of over 100 m.

iv) Drainage tunnel:

It is a tunnel provided in the abutments of an earthen dam to collect seepage water from rainfall, reservoir for safe disposal.

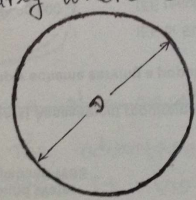
Advantages of tunnels:

- Provides direct and short route for the water passage resulting in saving in head thereby results in saving in project cost.
- Tunneling work can be started simultaneously at many points, facilitating quicker completion.
- Natural landscape of the area is not distrubed
- Tunneling is getting easier and faster with the development of latest tunnel boring techniques.
- The concept of rock mechanics and the latest tool of stress analysis by using finite element method, has established structural stability of tunnels.

Shape of tunnel / Geometric Design:

i) Circular section:

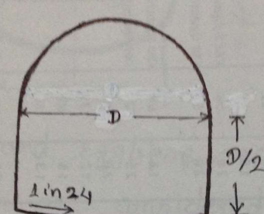
It is the most suitable and stable shape from structural considerations where the tunnel is subjected to high inward or outward radial pressures. For poor quality rock and/or inadequate rock cover around, it is most suitable from the excavation point of view; particularly where cross-sectional area is small.



- Max^m flow occurs, when $\frac{\text{depth}}{\text{diameter}} = 0.93$
- when depth = 0.8 * dia., flow is same as for full circle.

ii) D-shaped section:

It is suitable in good quality, intact sedimentary rocks and massive external igneous hard, compacted, metamorphic rocks where the external pressure due to rock and water are not very large and also where the lining is not designed against internal pressure. Its flatter width of invert gives more working floor space in the tunnel and eliminates the tendency of wet concrete to slump.

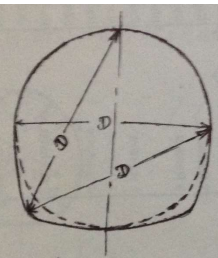


Area = $0.905\,D^2$

Perimeter = $3.58\,D$

iii) Horse-shoe section:

It is a compromise between circular and D-shaped sections. It is structurally strong to withstand external rock and internal water pressures. It is suitable in a moderately good rock. The maximum flow occurs when ratio of depth to diameter is 0.94 and the flow is same as for full section when depth = 0.82 * diameter.

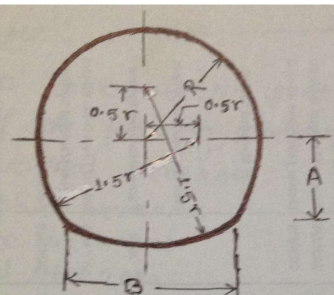


Area = $0.8293\ D^2$

Perimeter = $3.267\ D$

iv) Modified horse-shoe section:

It provides the advantage of a flatter invert for constructional ease. It also affords easy change over to circular section with minimum additional cost in reaches where rock quality is poor or rock cover is inadequate.



Area = $3.254\ r^2$

Perimeter = $6.426\ r$

$A = 0.7808\ r$

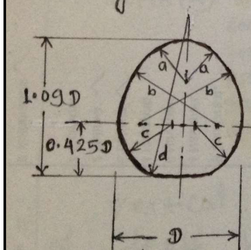
$B = 1.562\ r$

$r = 0.987580\ R$

v) Egg-shaped and Egglipse section:

These sections are considered where the rock is stratified, soft and very closely laminated (as laminated sand, stone, slates, micaceous schists, etc.) and where the external pressures and tensile forces in the crown are likely to be high so as to cause serious rock falls. These are suitable for tunnel carrying sediments.

Egglipse shape has advantage over egg shaped section as it has a smooth curvature and is hydraulically more efficient.



Egg shaped section

a = 0·384 D

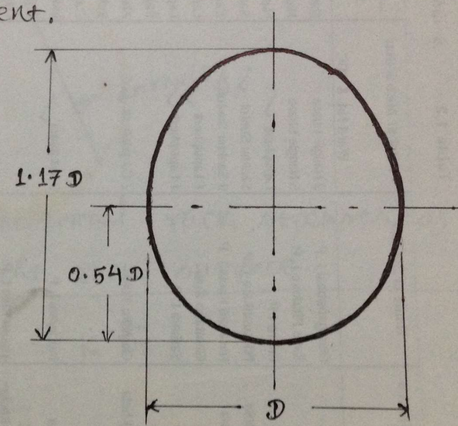
b = 0·78 D

c = 0·45 D

d = 1·3 D

Area = $0.864\,D^2$

Perimeter = 3·317 D



Egglipse section

Loads on tunnel:

- Earth pressure
- Reaction force due to the elastic deformation of the rock
- Over burden pressure
- Hydrostatic pressure
- Earthquake pressure

Tunnel lining:

The power tunnels which connect directly to the penstock are essentially lined with cement concrete or reinforced or steel liner.

Tunnel lining is restored to when

- internal pressure is high (i.e, > 100 m water head).
  → in sound rock formations, ulined upto 200 m water head.

- in strata of low strength and where the rock is anisotropic
- the hydraulic resistance is desired to be low.

Advantages:

- Provides strength and stability and facilitates tunneling in weaker strata.
- Distributes internal loads uniformly in the surrounding rock mass.
- Saves energy by reducing hydraulic resistance.
- Saving in cost with the reduction in cross-sectional area of tunnel.
- Minimises the danger of accidental rock falls in the tunnel cavity
- Seepage across the lining becomes negligible
- Rock trap provision on the upstream of turbines is not necessary.

Types of lining:

1. PCC lining:

Tunnels are lined with PCC when the tunnel walls are to support outside pressure due to the rock, but low pressure inside.

2. RCC lining:

Tunnels are lined with RCC when the internal pressures are high, PCC lining is inadequate and tension cracks are developed in lining.

3. Steel lining:

The steel lining consists of a steel plate of adequate thickness conforming to boiler and pressure vessel steel provided to the inner surface of the tunnel.

The main function of steel lining is to protect the concrete and to stop leakage of water from the tunnel.

It is essentially required where

- The tunnel has to withstand high pressure
- The rock cover is inadequate
- Cavitation of lining is expected due to high velocity of water
- Erosion can occur as in silt flushing tunnel.

4. Shotcrete lining:

Shotcrete lining is provided to protect and support zones of fractured, crushed, disintegrated or spalling rock, and to preserve and prevent further deterioration caused by action of water or atmosphere or aging. Shotcrete lining offers an effective, economical and safe alternative where conventionally placed concrete is not possible.

Structural Analysis:

Because of the applied external rock pressure, hydrostatic pressure, hydrostatic pressure from inside the tunnel and self weight, bending stress as well as compressive stress is developed in the tunnel. Due to that reason, tunnel lining cross-section will be deformed and elastic reaction may be developed.

Calculation of deformation of the tunnel membrane could be done through the known value of the bending moment and the compressive force. With the known value of the deformation, the elastic reaction could be estimated with the following relationship,

$\sigma_{elastic} = K * \delta$

where, δ = deformation in m.

and K = elastic resistance of the rock in KN/m^2, which could be established through in-situ test or from Galerkin's relationship.

Galerkin's relationship,

$$K = \frac{E}{r(1+\mu)}$$

where, E = modulus of elasticity of rock

r = radius of tunnel

and μ = Poisson's ratio.

For the approximate calculation value of K for f_h ranging from 1.5 – 20, following emperical relationship can be used.

$$K_o = 500\,\alpha\, f_h \;(kgf/cm^2)$$

$$K = K_o \frac{100}{r_e}$$

where, r_e = radius of tunnel

α = crack intensity coefficient

= 0.8 for highly fractured rock (i.e, $M_q = 5 - 15$)

= 1 for fractured rock (i.e, $M_q = 1.5 - 5$)

= 1.2 for less fractured rock (i.e, $M_q < 1.5$)

and f_h = hardness coefficient of rock.

Hydraulic Design:

Losses:

a) Friction losses:

i) Manning's formula

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

where, n = Manning's roughness coefficient

= 0.040 – 0.060 for very rough surface

= 0.025 – 0.035 for surface trimmed

= 0.020 – 0.030 for surface trimmed and

where, n = Manning's roughness coefficient
= 0.040 – 0.060 for very rough surface
= 0.025 – 0.035 for surface trimmed
= 0.020 – 0.030 for surface trimmed and invert concreted.
v = mean velocity of flow
l = length of tunnel
and, R = hydraulic radius or hydraulic mean depth

ii) Darcy weisbach formula

$$h_f = \frac{f l v^2}{2 g D}$$

where, f = Darcy's friction factor
l = length of tunnel
v = mean velocity of flow
and D = diameter of tunnel.

b) Trash rack loss:

$$h_t = K_t \frac{v^2}{2g}$$

where, K_t = loss coefficient for trash rack
and v = velocity through contracted opening

$$k_t = 1.45 - 0.45 \frac{a_n}{a_t} - \left(\frac{a_n}{a_t} \right)^2$$

where,
a_n - net area through the trash rack bars
a_t - gross area of the opening

c) Entrance loss :

$$h_e = K_e \frac{v^2}{2g}$$

where, K_e = loss coefficient for entrance and assumed to vary from 0.1 for gradual contraction to 0.5 for abrupt contraction
and v = velocity of flow

d) Bend loss :

$$h_b = K_b \frac{v^2}{2g}$$

where, K_b = bend loss coefficient

$$= \frac{2\alpha}{\pi^2 \left(\ln \frac{R_c}{D} + \alpha \right)}$$

α = deflection angle
R_c = radius of curvature
and v = velocity of flow.

e) Gate loss :

$$h_g = K_g \frac{v^2}{2g}$$

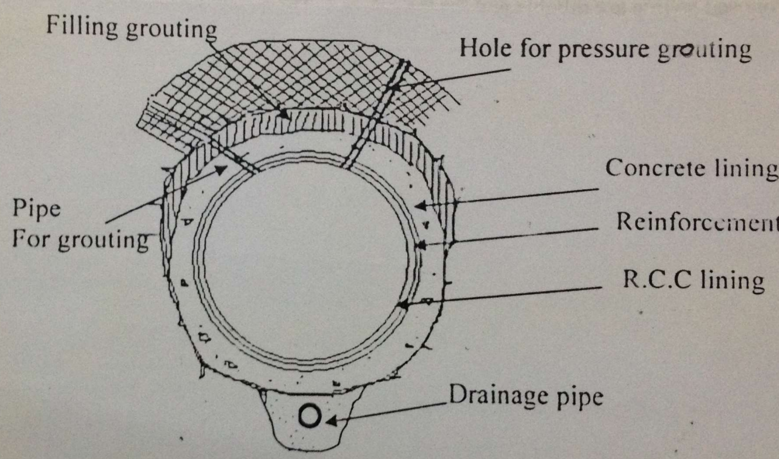
where, K_g = gate loss coefficient
= 1.0 for full gate opening
= 1.15 for 3/4 " "
= 5.6 " 1/2 " "
= 24.0 " 1/4 " "
and v = flow velocity

f) Exit loss :

$$h_{exit} = K_{exit} \frac{v^2}{2g}$$

where, K_{exit} = exit loss coefficient ≈ 1.0
and v = flow velocity.

Typical section of pressure tunnel:



Pressure Tunnel

Economic Dimensioning of tunnel:

1. Economical diameter of tunnel:

According to 'Handbook of Hydroelectric Engineering' – P.S. Migam.

$$D^{7.33} = \frac{19.35\ Q^3 n^2\ e\ u * 10^5}{(E + 0.36 L) * O}$$

where,

Q = Discharge

n = rugosity coefficient

e = overall efficiency

u = unit cost of power ($\simeq$ Rs. 7/unit say)

L = unit price of lining ($\simeq$ Rs. 4000/m^3 say)

E = unit price of tunnel excavation

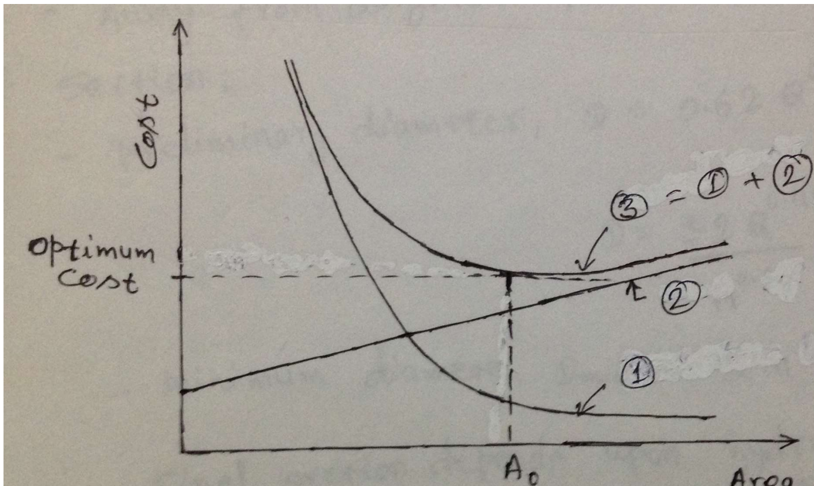
O = Annual operation and maintenance cost ≃ 8% of total cost.

2. Cost benefit analysis:

$$\frac{dC_T}{dA} + \frac{dC_B}{dA} = 0$$, will give economical area

C_T = cost of tunnel per metre

C_B = Capitalized value of annual head loss



Optimisation of tunnel cross-section

① → Annual power loss

② → Annual cost of tunnel

③ → Total cost

Considering head loss and energy regained, the economic section of the tunnel can be determined based on the optimization of cost of the tunnel and minimizing the energy losses due to friction in tunnel as shown in graph above.

Design considerations:

1. Alignment:
   - As short as economically possible length.
   - should follow the location of intake on one hand. the surge shafts on the other hand.
   - Availability of convenient adit points.
   - Good and sufficient rock cover.
   - Away from dangerous fault planes.

2. Section:
   - Preliminary diameter, $D = 0.62\,Q^{0.48}$ for concrete lining

     $D = \dfrac{1.2\,Q^{0.45}}{H^{0.12}}$ for steel lining

   - Minimum diameter, $D_{min} \approx 2\,m$ (Masonyi $\approx 1.8\,m$)
   - Final section depends upon hydraulic considerations, convenience of construction and the overall economy of the operation.

3. Slope:
   - slope is fixed upon the basis of dewatering requirements.
   - Power tunnel with a gentle slope has relatively low internal pressure.

4. velocity:
- Limiting velocity in unlined tunnels → 2 to 2.5 m/s
- " " " concrete lined " → 4 to 5 m/s
- " " " steel-lined " → 9 m/s
- Greater velocities in diversion tunnels (Mossy rock, USA, planned for max^m velocity of 7.5 m/s)
- Steel lined pressure shafts have higher velocity than power tunnel (normal range of 5 to 8 m/s).

5. Overburden:
- Based on elementary statical equilibrium,

$$\gamma h \leq \gamma_r h_r$$

where, γ and γ_r = unit weights of water and rock respectively

h and h_r = internal water pressure head and desireable height of overburden respectively.

Taking $\gamma = 10\ kN/m^3$ and $\gamma_r = 25\ kN/m^3$

$10h \leq 25\ h_r$ $[\gamma_r \simeq 24-32\ kN/m^3]$

i.e. $h \leq 2.5\ h_r$

Taking factor of safety = 2.5

$\boxed{h \leq h_r}$ → Thumb rule.

Practical value of factor of safety = 4 to 6 with smaller value for greater depth of overburden.

- Mosonyi recommends

$\boxed{h = 0.4 \text{ to } 0.8\ h_r}$, depending upon the rock formation.

If $h > 0.4$ to $0.8\ h_r$, lining is required.

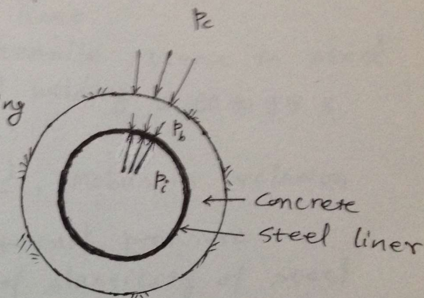
6. Lining:
- Thickness depends upon the load shared by the lining.

- Total internal pressure = pressure transferred to rock + pressure resisted by the lining itself.

P_c = Pressure transferred to the rock

$P_b - P_c$ = pressure taken by the concrete lining

$P_i - P_b$ = pressure taken by the steel lining



Stress development in the lining

- when the tunnel is running full, the lining and rock must be able to resist internal pressure of water
- when the tunnel is empty, the lining must be able to resist external load transferred to it.
- Thickness of PCC lining = 20 to 60 cm, poorer the rock, greater will be the thickness.

- Thickness of steel liner (t),

i) Thumb rule

$$t_{min} = \frac{d + 50}{400} \text{ in cm}$$

where, d = internal dia. of tunnel in cm.

$t = t_{min}$ + 2 mm (allowance for corrosion)

ii) Estimating actual internal or external pressures:

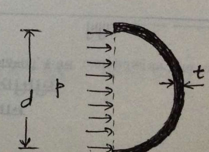
a) Internal pressure:

Internal pressure force = $p.d$

Resisting force = $\sigma_{st} * 2t * \eta$

Equating these,

$$t = \frac{p\,d}{2\,\sigma_{st}\,\eta}$$



where, p = the part of the internal pressure to be resisted by liner

d = internal diameter of the tunnel

t = thickness of liner

σ_{st} = permissible tensile stress in steel

η = efficiency of welding ≈ 90 to 95 %

b) External pressure:

$$P_{ext} = \frac{8EI(K-1)}{d^3}, \text{ Mobussin relation}$$

where, P_{ext} = design external pressure

E = modulus of elasticity of steel

I = moment of inertia per unit length of the liner $= \frac{t^3}{12}$

t = thickness of liner

d = internal diameter of liner

and K = a factor depending upon the angle of buckling, for an angle of $30°$, $K = 8.62$.

Buckling due to external loads can be prevented to a large extent by

→ using dense concrete between the rock mass and liner.

→ grouting deep into rock mass.

→ providing anchorage for the steel liner into concrete backing.

Tunneling Methods:

1. Conventional method
2. NATM (New Austrian Tunneling Method)
3. TBM (Tunnel Boring Machine) method

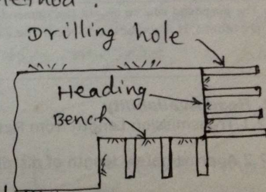
1. Conventional method:

i) Full face method:

In this method, the entire cross-sectional area of the tunnel to be excavated is attacked simultaneously. It is generally adopted for small size tunnels in good rock conditions where major rockfalls are not anticipated.

ii) Top heading and benching method:

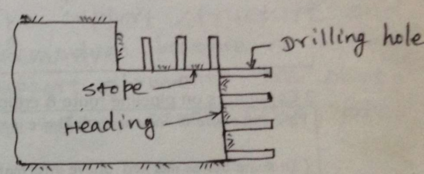
In this method, a top heading is excavated first, either to full length or part length of the tunnel, and is supported simultaneously. The benching is then removed



slowly. This method is used where the tunnel has large cross-sectional area or where the rock is not of good quality.

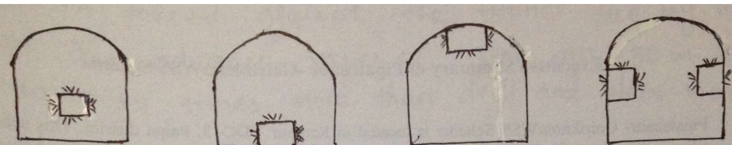
iii) Bottom heading and stoping method:

In this method a bottom heading is made first and the overhead stope is removed later. It is used where the rock is consistent and sound and the tunnel section is very large.



iv) Drift method:

This method is used where in a large tunnel it is economical to drive a small tunnel, called drift or a pilot tunnel, prior to excavating a full face. The drift may be excavated in the centre, bottom, top or side.



Central drift Bottom drift Top drift Side drift

Specially the side drift is adapted to large size tunnels through bad rock which require support before mucking.

v) Multiple drift method:

It is a combination of the side drifts and top drift. It is suitable for large openings in crushed rock formations in fault zones. In this method, a side drift is driven first on each side.

2. NATM:

It is based on the principle to take maximum advantage of rock's capacity to support itself by careful measures and deliberate guidance of the force during the readjustment process from primary to the secondary state of stress around the excavations.

The NATM constitutes a method by which the surrounding rock or soil formation of a tunnel are integrated into an overall ring like support structure and as such these formations themselves become a part of the primary support. The support elements are shotcrete, welded wiremesh, rock bolts, steel ribs, forepoling, steel lagging.

3. TBM method:

A full face TBM consists of a wheel cutter head fitted with teeth or rollers to cut or spall the rock. The wheel may consist of spokes or of a solid disc with slots to allow the muck to pass through. The rotating speed of the wheel is 4 to 10 rpm. The

wheel is forced against the tunnel face by hydraulic jacks. The advance rates of TBM are 50 m a day i.e., 10 to 15 times more than drill and blast method.

Tunneling sequence:

The full-face tunneling procedure adopted may be carried out in the following sequence:

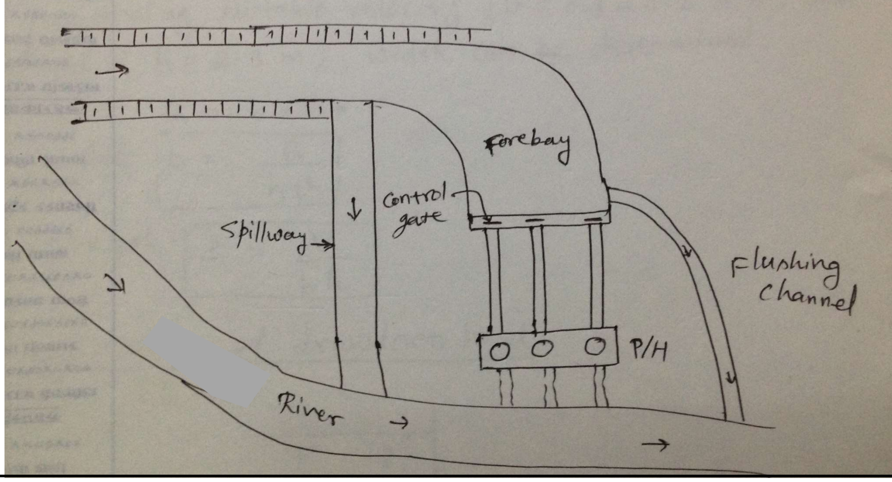
1. Drilling
2. Charging and blasting
3. Ventilation for the exhaust of the fumes cushion
4. Scaling
5. Mucking and Haulage of rock pieces
6. Ribbing (fixing the steel supports)
7. Initial concreting (concreting the space between rock and steel supports.

For the first, a fully mechanised drilling jumbo is required. However, all the first four operations may also be done by the same machine. Ventilation may be achieved through fans or air ducts. Sometimes a mixture of compressed air and water blown through the nozzles, brings down the fumes quickly and cleans the air.

After the blasting is over and the fumes are exhausted, any loose pieces are scaled down and the blasted rock is mucked with the help of muckers and mine cars. Then ribbing is done, depending upon the size of the tunnel and the quality of rock material. The gap between the ribs and the rock is then filled with concrete at pressure (shotcrete).

Forebay or Headpond:

It is the pondage at the beginning of a penstock leading water to turbine or the enlarged end of hydel channel in case of a canal as a headrace conveyance.



Main purposes of forbay :

- To distribute discharge evenly among the penstocks / pressure shafts
- To regulate the flow entering into the penstocks.
- To dispose excess water
- To provide daily storage (in some cases) eg, Sunkoshi

Storage capacity of a forebay:

The flow in the canal is fully accelerated only after sometime of turbine startup. Therefore, enough storage is required in the forebay to generate power during the time taken by the flow for full acceleration. Usually, time depends on site conditions and layout of the system. In usual considerations duration of 3-4 minutes is considered for the estimation of storage capacity of forebay.

Storage volume, $V = Q * t$

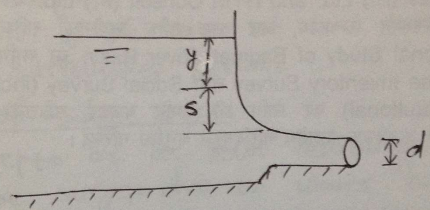
Assuming suitable velocity ($v = 0.1 - 0.2$ m/s) and depth ($h = 2-3$ m), width can be determined.

$$Q = b * h * v$$

$\therefore$ width; $\boxed{b = \dfrac{Q}{hv}}$

Length, $\boxed{l = \dfrac{V}{bh}}$

Estimation of drawdown level:



The inlet section of the penstocks / pressure shafts should remain fully submerged during the sudden startup of the units to avoid entrapment of air.

Drawdown, $y = v * \sqrt{\frac{l}{g} * \frac{A_p}{A_f}}$

where, v = velocity in penstock
A_p = Area of "
A_f = surface area of forebay
and l = Length of penstock.

Minimum submergence,
$s = a v \sqrt{d}$ m.
where, $a = 0.545$ for symmetrical flow
$= 0.725$ for asymmetrical flow.

Vortex free conditions:

Relative submergence, $\frac{s}{d} > 0.5 + 0.4 F_r$

where, F_r = Froud number $= \frac{v}{\sqrt{gd}} < 0.5$.

※ Design a forebay for supplying 25 m^3/s discharge to turbine through a 200 m long penstock of 3 m diameter.

Soln:

Assuming time taken by the flow for full acceleration $t = 4$ minutes and velocity of flow in forbay $v = 0.2$ m/s with depth $h = 3$ m

width, $b = \frac{Q}{hv} = \frac{25}{3*0.2} = 41.67$ m

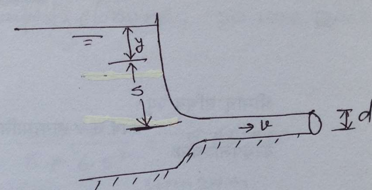
length, $l = \frac{V}{bh}$

where, V = storage volume = $Q*t = 25 * 4 * 60 = 6000\ m^3$

$\therefore l = \frac{6000}{41.67*3} = 48$ m.//

Now,

Draw down, $y = v\sqrt{\frac{L_p}{g}\frac{A_p}{A_f}}$



where, $v = \frac{Q}{A_p} = \frac{25}{\pi * \frac{3^2}{4}}$

$= 3.54$ m/s

$A_p = \pi d^2/4 = \pi * \frac{3^2}{4} = 7.07\ m^2$

$A_f = l\,b = 48 * 41.67\ m^2$

$\therefore y = 3.54\sqrt{\frac{200}{9.81} * \frac{7.07}{48*41.67}} = 0.95$ m //

Minimum submergence,

$s = a\,v\sqrt{d}$ [Let the flow is symmetrical]

$= 0.545 * 3.54 * \sqrt{3} = 3.34$ m.//

Vortex free conditions,

$\frac{s}{d} > (0.5 + 0.4\,F_r)$

$F_r = \frac{v}{\sqrt{gd}} = \frac{3.54}{\sqrt{9.81*3}} = 0.65 \nless 0.5$,

which is not so trouble in operation.

i.e., Required s/d = 0.5 + 0.4 ×0.65 = 0.76

Calculated s/d = 3.34/3.0 = 1.113 > Required s/d (O.K.)

Since y < h, additional depth is not required.

Surge tank or Expansion chamber or Surge Chamber:

Surge tank is a hydraulic structure located between slightly inclined pressure conduit (tunnel) and steeply sloping penstock (pressure shaft) constructed to control hydraulic transient in a hydropower system.

It is designed as a chamber excavated in the mountain in most of the cases or as a tower raising high above the surrounding terrain.

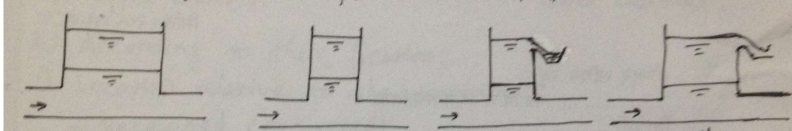
Purpose of Surge Tanks:

- Intercepts or at least redically reduces the pressure surges due to water hammer and exempts thereby the pressure tunnel from excessive internal pressures.
- Provides protection to the penstock itself against the detrimental effects of water hammer if no bypass valve is installed or if bypass valve fails to operate.
- Improves the regulation of the governing system
- Provides additional water supply to the turbines in the case of starting up.

Type of Surge Tanks:

A) According to the hydraulic design:

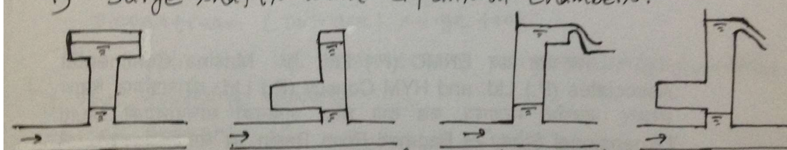
1. Simple surge tanks:
   - Designed as shafts or as basins



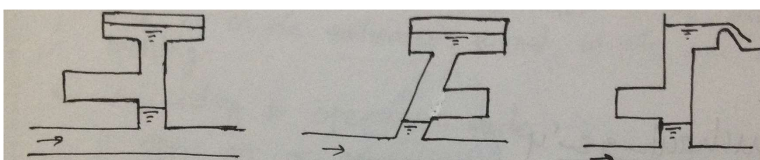
Simple tank — simple shaft — spilling shaft — overflow type tank

2. Special surge tanks:

i) Surge shafts with expansion chambers:

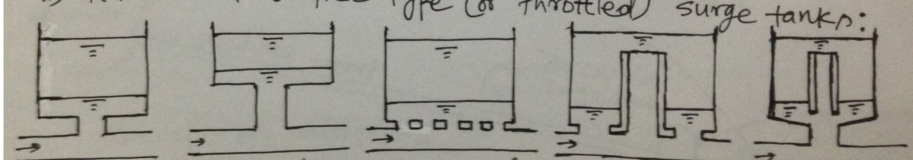


Shaft with upper expansion chamber — shaft with lower expansion chambers — shaft with overflow type upper chamber — Overflow type shaft with lower chamber



Shaft with two chambers — Inclined shaft with two chambers — Overflow type tank with two chambers

ii) Restricted orifice type (or throttled) surge tanks:



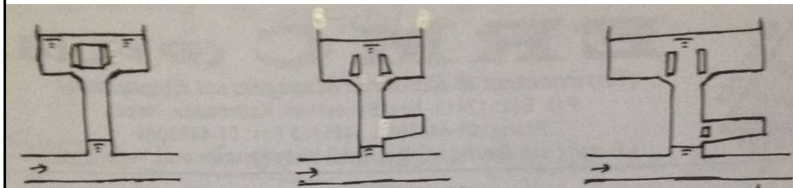
Simple restricted orifice type tanks — Tank with gate type throttling — Differential (Johnson type) tanks

Simple surge tanks: The simple surge tank is of uniform cross-section and is open to the atmosphere, acting as a reservoir. It is directly connected to the penstock so that water flows in and out with small head losses when load variations occur. It is usually large in size with expensive proportions and sluggish in responding to damping surges. These are very rarely used in modern practice except in installations where load changes are either small or very gradual.

Throttled tank: In the throttled tank (restricted orifice type tank) the restricted entry to the surge tank creates retardation and acceleration conditions of flow in the tunnel upstream of it, thus reducing the storage requirement and minimizing the maximum up and down surges. Although this type of surge tank is economical (because of its smaller size) compared with the simple tank section, the rapid creation of retarding and accelerating heads complicates the governing mechanism, requiring additional inertia in the turbo-generator units.

Surge tank with expansion chambers: This type of surge tank consists of a narrow riser (main surge shaft); attached to it at either end are large expansion chambers. The narrow riser reacts quickly, creating accelerating or decelerating heads, and at the same time the expansion chambers minimize the maximum up- and down-surge levels, thus limiting the range of surge levels (i.e. easier governing). In order to reduce the costs of the structure, spilling arrangements may sometimes be provided either to wastage (if water is not scarce) or back to the penstock.

Differential surge tank: This type (also known as Johnson's differential tank) consists of an internal narrow riser shaft with an orifice entry to the larger outer shaft at the bottom. As the central riser is narrow it responds instantaneously during the upward phase; at the same time the maximum amplitude is restricted to its top level, any excess water spilling back into the outer chamber. Similarly, during the downward phase water spills into the narrow riser while the riser itself responds quickly to maintaining the desired level. The differential tank with an extended penstock, which acts as a central riser.



Differential tank with upper chamber — Double chamber differential tank — Double chamber differential tank with throttled lower chamber

B) According to their location:

i) Location relative to the terrain
- Recessed (Excavated)
- Free-standing (Exposed)

ii) Location relative to the powerhouse
- Upstream (Headrace) surge tank
- Downstream (Tailrace) surge tank

The application of downstream surge tank is justified by the following reasons:

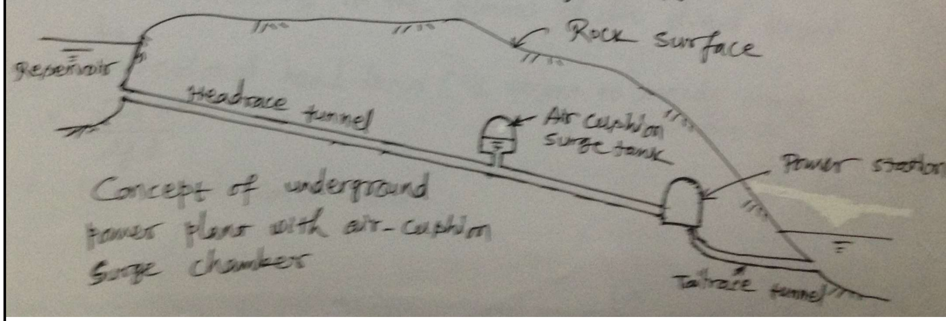
→ To prevent the separation of the water column in the draft tube and/or in the tailwater tunnel, in the case of rapid closure.

→ To prevent excessive pressures in the draft tube and or in the tailwater tunnel, in the case of rapid opening

C) According to operation mode:

i) Open air or atmospheric surge tank

ii) Enclosed or air-cushion surge tank



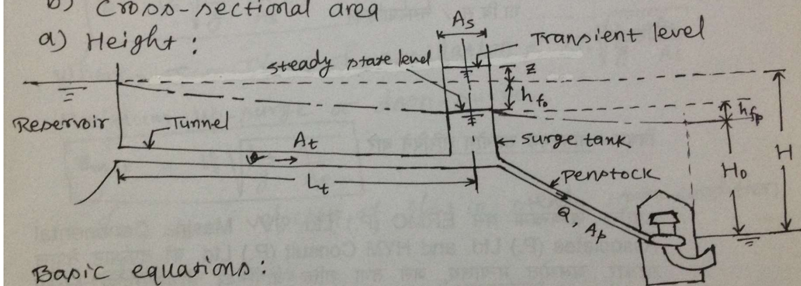
Concept of underground power plant with air-cushion surge chamber

D) According to material of construction:
- Concrete
- Steel

Surges in surge chambers (Hydraulic Design):

The hydraulic design of surge tank concerns itself with two main aspects
a) Height
b) Cross-sectional area

a) Height:



Basic equations:

$$v A_t = A_s \frac{dz}{dt} + Q \rightarrow \text{Continuity equation}$$

$$\frac{L_t}{g} \frac{dv}{dt} + z \pm h_f = 0 \rightarrow \text{Momentum equation}$$

Where,

Q = Rate of flow to the power house at any time t

$\frac{dz}{dt}$ = Rate at which water rises in the surge tank

A_s = cross-sectional area of surge tank

A_t = cross-sectional area of headrace tunnel

v = Flow velocity in the tunnel of dia. d and length L_t.

h_f = Frictional head loss [h_{f_0} refers to steady state condition]

$= \frac{f L_t v^2}{2gd}$

f = Frictional coefficient

$\pm$ve sign = +ve sign for the flow direction from the reservoir and -ve sign for the flow direction to the reservoir.

The solution of the above equation gives the value of z at any time t.

i) For the simplest case of full closure and negligible friction,

$$\boxed{z = v_0 \sqrt{\frac{L_t}{g} \frac{A_t}{A_s}} \sin 2\pi \frac{t}{T}}$$

where, T = Time of oscillation = $2\pi \sqrt{\frac{L_t}{g} \frac{A_s}{A_t}}$

Maximum up-surge or down-surge,

$$\boxed{z_{max} = v_0 \sqrt{\frac{L_t}{g} \frac{A_t}{A_s}}}$$

where, v_0 = velocity of flow in steady state condition.

According to Novak,

$$z_{max} = \frac{Q_0}{A_s \cdot r} \quad \text{with } r = \sqrt{\frac{g A_t}{L_t A_s}}$$

ii) In cases where friction is taken into account,

$$\boxed{\frac{z}{z_{max}} = 1 - \frac{2}{3} K_0 + \frac{1}{9} K_0^2}$$, According to Jaegar for $K_0 < 0.7$

where, z = Maximum upsurge with friction taken into account

z_{max} = Maximum upsurge with negligible friction

$K_0 = \frac{h_{f_0}}{z_{max}}$

h_{f_0} = Frictional head loss in the steady state condition.

$$\boxed{\frac{z}{z_{max}} = -1 + 2K_0}$$, According to Calame and Gaden for $K_0 < 0.8$

[Max^m downsurge when 100% opening i.e, 100% load demand]

where, Z = Maximum downsurge with friction taken into account

Z_{max} = Maximum downsurge with negligible friction

$$k_0 = \frac{h_{f_0}}{Z_{max}}$$

* Upsurge to be determined considering maximum reservoir level and downsurge to be determined considering minimum reservoir level for the design of surge tank.

b) Cross-sectional area:

For the stability consideration, D. Thoma analysed assuming the following

- The turbine governer maintains a constant power output
- The surgetank oscillations are small
- The turbine efficiency is constant
- The friction losses correspond to steady state condition, and
- The governer response is instantaneous

He linearized the basic differential equation by neglecting the higher order non-linear terms. For damped oscillations

$$\boxed{A_{sc} \geqq \frac{v_0^2}{2g} \frac{A_t}{h_{f_0}} \frac{L_t}{H_0}}$$

where, A_{sc} = critical cross-sectional area.

For actual practice, $A_s = FOS * A_{sc} = 1.5\ A_{sc}$.

For smooth concrete finishing,

$1/n = 85$ i.e, $n \simeq 0.012$

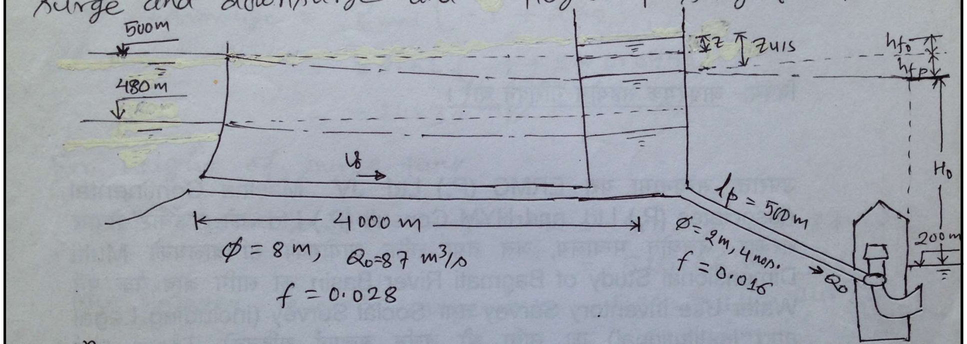
$$h_{f_0} = \frac{n^2 v_0^2 L_t}{R^{4/3}} \rightarrow \text{Manning's equation.}$$

Also, $R = d/4$, where d = dia. of tunnel

$$\therefore \quad A_{sc} \geq \frac{v_0^2}{2g} \cdot \frac{\pi d^2}{4} \cdot \frac{(d/4)^{4/3}}{n^2 \, v_0^2 \, L_t} \cdot \frac{L_t}{H_0}$$

i.e., $$\boxed{A_{sc} \geq 45 \frac{d^{10}}{H_0}}$$

※ Determine cross-sectional area, Maximum up-surge and downsurge and Height of surge tank.



Sol^n:

Frictional head loss from intake to surge tank

$$h_{f_0} = \frac{f l v^2}{2gD} = \frac{0.028 * 4000}{2 * 9.81 * 8} * \left(\frac{87}{\pi * \frac{8^2}{4}}\right)^2 = 2.14 \text{ m}$$

Frictional head loss from surge tank to power house

$$h_{f_p} = \frac{f l v^2}{2gD} = \frac{0.016 * 500}{2 * 9.81 * 2} * \left(\frac{87/4}{\pi * 2^2/4}\right)^2 = 9.78\ m$$

$$\therefore\ H_0 = 500 - 200 - h_{f_0} - h_{f_p} = 300 - 2.14 - 9.78 = 288.08\ m$$

Now, $$A_{sc} = \frac{v_0^2 L_t A_t}{2g h_{f_0} H_0} = \left(\frac{87}{\pi * \frac{8^2}{4}}\right)^2 * \frac{4000 * \left(\pi \frac{8^2}{4}\right)}{2 * 9.81 * 2.14 * 288.08}$$

$$= 50\ m^2$$

$$\therefore\ A_s = 1.5\ A_{sc} = 1.5 * 50 = 75\ m^2 //.$$

Also,

$$Z_{max} = v_0 \sqrt{\frac{L_t}{g} \frac{A_t}{A_s}} = \left(\frac{87}{\pi * \frac{8^2}{4}}\right) \sqrt{\frac{4000}{9.81} * \frac{\pi * 8^2/4}{75}}$$

$$= 28.61\ m$$

$$K_0 = \frac{h_{f_0}}{Z_{max}} = \frac{2.14}{28.61} = 0.0748$$

$$\therefore\ Z_{upsurge} = Z_{max}\left[1 - \frac{2}{3} K_0 + \frac{1}{9} K_0^2\right]$$

$$= 28.61\left[1 - \frac{2}{3} * 0.0748 + \frac{1}{9} * 0.0748^2\right]$$

$$= 27.20\ m\ //.$$

and $$Z_{downsurge} = Z_{max}\left[-1 + 2K_0\right]$$

$$= 28.61\left[-1 + 2 * 0.0748\right)$$

$$= -24.33\ m //.$$

For height of surge tank,

Max^m water level in surge tank $= 500 - 2.14 + 27.20$

$= 525.06\ m$

Min^m water level in surge tank $= 480 - 2.14 - 24.33$

$= 453.53\ m$

Assuming FB = 2 m and water cushion at the bottom = 3 m

$\therefore$ Ht. of surge of tank $= (525.06 - 453.53) + 2 + 3$

$= 76.53\ m //.$

discharge

Penstocks:

Penstocks are the conveyance conduits which supply water from forebay / surgetank to turbines. These are high pressure conduits, designed to withstand water hammer pressure due to the change in boundary conditions such as starting and closing of turbine, sudden load rejection / acceptance, etc.

Classification of penstocks:

1. Based on material of fabrication

i) Steel
ii) Cast iron
iii) Plastic (for very low head)
iv) wooden
v) RCC

2. Based on method of support:

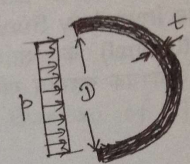
i) Buried or embeded → Trench size 1 to 1.5 m deep
ii) Exposed above the ground
iii) Partly buried.

3. Based on rigidity of connections and support:

i) Rigid → Anchorage throughout the length i.e., no longitudinal movement
ii) Semi-rigid → Each member anchoraged (fixed) at one end only
iii) Flexible → Expansion joint throughout the length.

Thickness of penstocks:

Acting force per unit length of penstock $= p * D * 1$ --- ①



Resisting force per unit length of penstock $= \sigma_{st} * 2 (t * 1) * \eta_j$ ---- ②

where, D = Internal diameter of penstock

p = Internal pressure

σ_{st} = Permissible tensile stress in steel

η_j = Efficiency of welding at joints.

Equating equations ① and ②

$$p * D = \sigma_{st} * 2t * \eta_j$$

i.e; $$\boxed{t = \frac{pD}{2\sigma_{st}\eta_j}}$$

or, $$\boxed{t = \frac{pR}{\sigma_{st}\eta_j}}$$

where, R = Internal radius of penstock

Add 1 to 3 mm generally for corrosion allowance.

ASME [American society of Mechanical Engineer] Recommendation:

$$\boxed{t = \frac{pR}{\sigma_{st}\eta_j - 0.6p} + 0.15 \text{ cm}}$$

where, p = Internal pressure in kg/cm^2

R = Internal radius in cm

σ_{st} = Permissible tensile stress in steel in kg/cm^2

η_j = Efficiency of joint (0.9 to 1.0)

Diameter of penstocks:

1. Empirical formulae:

a) USBR (United States Bureau of Reclamation) formula:

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

where, v = optimum velocity in m/s
and H = maximum working head in m

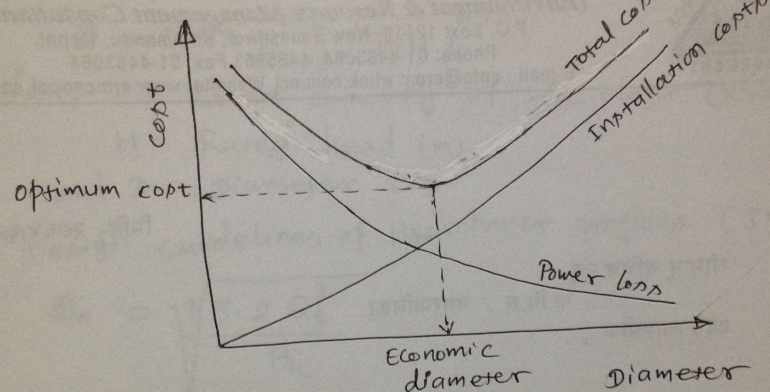
Then from continuity equation adopt diameter of penstock.

b) G. Sarkaria formula:

$$D = 0.62\,\frac{P^{0.35}}{H^{0.65}}$$

where, D = Penstock diameter in m
P = HP transmitted by the pipe
H = Maximum net head at the end of the penstock in m.

2. Graphical solution:



Economic Diameter of Penstock

The energy of the flow through a penstock is inevitably reduced owing to entry and friction losses.

Although the friction losses can be minimized by careful selection of the pipe diameter, and its entrance losses can be minimized by a bell-mouthed entrance, an economical penstock diameter may be determined from a study of the annual charges on the cost of the installed pipe compared with the lost of revenue due to this power loss. Energy losses decrease with the increasing diameters while construction cost increase. A diameter which minimizes the total annual costs can be determined from the sum of the two costs.

Fahlbush (1982) reformulated the objective of the economic analysis in terms of the amount of the invested capital and the capitalized value of the lost energy, and arrived at the conclusion that the most economical diameter can be computed within an accuracy of about ± 10% from

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

where, P = Rated capacity of the plant (KW)

H = Rated head (m)

and D = Diameter (m)

3. Design Guidelines of Hydropower projects (JNN):

$$D_e = \sqrt[7]{\frac{5.2\, Q_t^3}{H_h}}$$

where, D_e = Economic diameter of penstock (m)

H_h = Dynamic head on the penstock (m)

Q_t = Flow discharge of the turbine (m^3/s)

4. Based on Economic analysis:

$$d \le \sqrt[7]{\frac{\lambda\, \sigma_a\, t\, C_2\, Q^3}{1000\, C_1\, H}}$$

where, d = economic diameter of penstock in cm
λ = friction coefficient
σ_a = allowable stress in kg/cm²
t = annual duration of operation of HEP in hrs.
C_2 = unit cost of power generation
C_1 = annual cost of 1 kg wt. of penstock
H = design head in m
and Q = Discharge through penstock in m³/s

※ A penstock of discharging capacity of 5 m³/s is to be functioned in a hydropower station with a design head of 50 m. Determine the economic diameter of the penstock. Take overall efficiency of the plant as 85 %.

Soln:

$Q = 5\ m^3/s$, $H = 50\ m$, $\eta = 85\%$

$P = \gamma Q H \eta = 9.81 * 5 * 50 * 0.85 = 2084625$ watt

$= \frac{2084625}{746} HP = 2794.4\ HP$

Using USBR formula:

$v = 0.125\sqrt{2gH} = 0.125\sqrt{2 * 9.81 * 50} = 3.915\ m/s$

$A = \frac{Q}{v}$

$\therefore D = \sqrt{\frac{4A}{\pi}} = \ldots = 1.28\ m$ //

Using Sarkaria formula

$D = 0.62\frac{P^{0.35}}{H^{0.65}} = 0.62 * \frac{(2794.4)^{0.35}}{50^{0.65}} = 0.784 m$

Using formula based on Graphical solution

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

$$= 0.52 * 50^{-0.17} * \left(\frac{2084.625}{50}\right)^{0.43} = 1.51\ m$$

※ A penstock carries $8\ m^3/s$ of water ~~at~~ head of 25 m. The cost of pipe line in place is given by $35hd^2$ rupees per metre length, where h is the head and d is the diameter of the pipe. Annual fixed charges are 8% of the pipe line cost. The head lost in friction is $0.025Q^2/12.1d^5$ m per m length of the pipe. Efficiency of the turbine is 80% and the selling price of the power generated is 70 rupees per kW per annum. Calculate the most economical diameter of the penstock.

Soln:

Consider 1 m length of the penstock.

Cost of pipe = Rs. $35 * 25\, d^2$ = Rs. $875\, d^2$

Annual fixed charges per annum

$= 0.08 * 875\, d^2$ = Rs. $70\, d^2$

Head loss $= \frac{0.025\, Q^2}{12.1\, d^5} = \frac{0.025 * 8^2}{12.1 * d^5} = \frac{0.132}{d^5}$

∴ Power lost $= 9.81 * 8 * \frac{0.132}{d^5} * 0.80$ $[\gamma Q H \eta]$

$= \frac{8.29}{d^5}$

∴ Cost of power lost = Rs. $\frac{8.29}{d^5} * 70$ = Rs. $\frac{580.30}{d^5}$

∴ Total cost = Rs. $\left[70\, d^2 + \frac{580.30}{d^5}\right]$

Differentiating this total cost with respect to d and equating to zero.

$$140\, d - \frac{2901.5}{d^6} = 0$$

or, $140\, d^7 = 2901.5$

$\therefore d = 1.54\ m$

Water hammer phenomena (Hydraulic Transient)

What is water hammer?

Water hammer is a phenomena that occurs in pressurized pipe systems when the flowing liquid is suddenly (instantaneously) obstructed by (for example) closing a valve or by pump stoppage (failure). A laud noise similar to hammer knocking noise occurs due to the collision of the liquid mass with the obstruction body (valve or pump) and the internal walls of the pipe.

The magnitude of pressure rise depends on:

(*i*) *The speed at which valve is closed,*
(*ii*) *The velocity of flow,*
(*iii*) *The length of pipe, and*
(*iv*) *The elastic properties of the pipe material as well as that of the flowing fluid.*

The rise in pressure in some cases may be so large that the pipe may even burst and therefore it is essential to take into account this pressure rise in the design of the pipes.

What is the problem with water hammer?

Hydraulic transient (water hammer) may cause disasters in pressurized liquid systems such as:

1. Rupture of pipes and pump casings.
2. Vibration and high noise.
3. Excessive pipe displacements.
4. Vapor cavity formation.
5. Environmental pollution.
6. Life and economical losses.

When water hammer occurs?

The most common cases where water hammer occurs are:

1. Turbine/pump failure.
2. Turbine/pump start up.
3. Sudden closure a valve.
4. Failure of flow or pressure regulators.

12·12·4. Time required by Pressure Wave to travel from the Valve to the Tank and from Tank to Valve.

Time taken, $t = \dfrac{\text{Distance travelled from valve to tank and back}}{\text{Velocity of pressure wave}}$

$$= \frac{L+L}{C} = \frac{2L}{C} \quad i.e., \quad t = \frac{2L}{C} \qquad \text{...(12·32)}$$

where, L = Length of the pipe, and

C = Velocity of pressure wave.

(i) The closure of valve is said to be *gradual* when $t > \dfrac{2L}{C}$

(ii) The closure of valve is said to be *instantaneous* when $t < \dfrac{2L}{C}$

where, C = velocity of the pressure wave.

12·12·1. Gradual Closure of Valve

Consider a long pipe carrying liquid (Fig. (12·45)) and provided with a valve which is closed gradually.

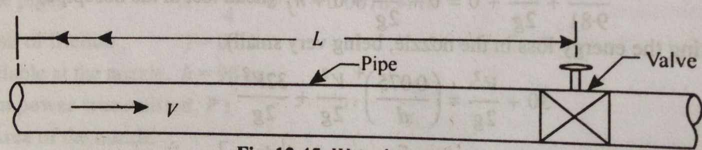


Fig. 12.45. Water hammer.

Let, A = Area of cross-section of the pipe,

L = Length of the pipe,

V = Velocity of flow of water in the pipe,

t = Time required to close the valve (in seconds), and

p = Intensity of pressure wave produced.

The mass of liquid contained in the pipe is $= \rho AL$

Assuming that the rate of closure of the valve is so adjusted that the liquid column in the pipe is brought to rest with a uniform retardation; from an intial velocity V to zero in time t seconds, we have,

Retardation of water $= \dfrac{V-0}{t} = \dfrac{V}{t}$

$\therefore$ The axial force available for producing retardation

$$= \text{Mass} \times \text{retardation}$$

$$= \rho AL \times \frac{V}{t} \qquad \text{...}(i)$$

Also force due to pressure wave is $= p.A$ ...(ii)

Equating the two forces given by eqns. (i) and (ii), we have

$$\rho AL \times \frac{V}{t} = p \times A$$

or, $$p = \frac{\rho LV}{t} \qquad \text{...(12·26)}$$

$\therefore$ Head of pressure, $$H = \frac{p}{w} = \frac{\rho LV}{w \times t} = \frac{\rho LV}{\rho \cdot g \cdot t} = \frac{LV}{gt}$$

$i.e.,$ $$H = \frac{LV}{gt} \qquad \text{...(12·27)}$$

Instantaneous Closure of Valve in Rigid Pipes

Consider a pipe of length L and area of cross-section A (Fig. 12·45) carrying water which is flowing through it at a velocity V. When the valve is closed instantaneously the K.E. of the flowing water is converted into strain energy of water (neglecting effect of friction and assuming the pipe wall to be perfectly rigid).

$$\text{Loss of } K.E. = \frac{1}{2} mV^2 = \frac{1}{2}\rho AL \times V^2 \qquad (\because m = \rho \times A \times L)$$

$$\text{Gain of strain energy} = \frac{1}{2}\left(\frac{p^2}{K}\right) \times \text{volume} = \frac{1}{2}\frac{p^2}{K} \times AL$$

$$\left[\begin{aligned}\text{where, } k &= \text{Bulk modulus of elasticity of water, and}\\ p &= \text{Intensity of perssure wave produced.}\end{aligned}\right]$$

Equating the loss of $K.E.$ to the gain of strain energy, we get

$$\frac{1}{2}\rho AL \times V^2 = \frac{1}{2}\frac{p^2}{K} \times AL$$

or,
$$p^2 = \frac{1}{2}\rho ALV^2 \times \frac{2K}{AL} = \rho KV^2$$

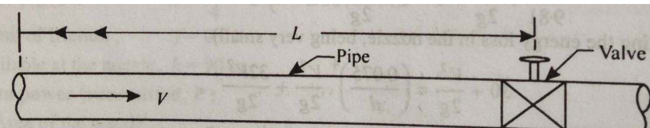


Fig. 12.45. Water hammer.

$$\therefore \quad p = \sqrt{\rho KV^2} = V\sqrt{\rho K} = V\sqrt{\frac{K\rho^2}{\rho}}$$

or,
$$p = V\rho C \qquad \text{...(12·30)}$$

$$\left(\text{where, } C = \sqrt{\frac{K}{\rho}}, C \text{ being the } \textit{velocity of pressure wave.}\right)$$

12·12·3. Instantaneous Closure of Valve in Elastic Pipes

As shown in Fig. 12·45, consider a pipe of length L, diameter D, thickness t (small compared to diameter).

Let, p = Increase of pressure due to water hammer,

E = Modulus of elasticity of pipe material, and

$\frac{1}{m}$ = Poisson's ratio for pipe material.

When the valve is closed intantaneously, rise of pressure takes place due to which circumferential and longitudinal stresses are produced in the pipe wall; these stresses are given as (from knowledge of strength of materials)

$$\sigma_c = \frac{pD}{2t} \quad \text{and} \quad \sigma_l = \frac{pD}{4t}$$

where, σ_c = Circumferential stress, and

σ_l = Longitudinal stress.

Also, strain energy stored in the pipe material per unit volume is

$$= \frac{1}{2E}\left(\sigma_c^2 + \sigma_l^2 - \frac{2\sigma_c\,\sigma_l}{m}\right)$$

$$= \frac{1}{2E}\left[\left(\frac{pD}{2t}\right)^2 + \left(\frac{pD}{4t}\right)^2 - \frac{2\times\frac{pD}{2t}\times\frac{pD}{4t}}{m}\right]$$

$$= \frac{1}{2E}\left[\frac{p^2D^2}{4t^2} + \frac{p^2D^2}{16t^2} - \frac{p^2D^2}{4mt^2}\right]$$

Assuming $\frac{1}{m} = 1/4$, we have

Strain energy per unit volume $= \frac{1}{2E}\left[\frac{p^2D^2}{4t^2} + \frac{p^2D^2}{16t^2} - \frac{p^2D^2}{16t^2}\right] = \frac{p^2D^2}{8Et^2}$

Total strain energy stored in pipe material

$$= \frac{p^2D^2}{8Et^2} \times \text{total volume of pipe material}$$

$$= \frac{p^2D^2}{8Et^2} \times \pi\, Dt \times L$$

$$= \frac{p^2 \times \pi D^2 \times DL}{8Et} = \frac{p^2 ADL}{2Et} \qquad \left[\because A \text{ (area of the pipe)} = \frac{\pi}{4}\times D^2\right]$$

Loss of K.E. of water $= \frac{1}{2}mV^2 = \frac{1}{2}\rho AL \times V^2$

Gain of strain energy in water $= \frac{1}{2}\left(\frac{p^2}{K}\right) \times \text{volume} = \frac{1}{2}\frac{p^2}{K} \times AL$

Also, the loss of K.E. of water = gain of strain energy in water + strain energy stored in material.

$$\therefore \quad \frac{1}{2}\rho AL \times V^2 = \frac{1}{2}\frac{p^2}{K} \times AL + \frac{p^2 ADL}{2Et}$$

Dividing both sides by $\frac{AL}{2}$, we get

$$\rho V^2 = \frac{p^2}{K} + \frac{p^2 D}{Et} = p^2\left(\frac{1}{K} + \frac{D}{Et}\right)$$

$$\therefore \quad p^2 = \frac{\rho V^2}{\left(\frac{1}{K} + \frac{D}{Et}\right)}$$

or, $$p = \sqrt{\frac{\rho V^2}{\left(\frac{1}{K} + \frac{D}{Et}\right)}} = V \times \sqrt{\frac{\rho}{\left(\frac{1}{K} + \frac{D}{Et}\right)}} \qquad \text{...(12·31)}$$

Example 12.51. *In a pipe 600 mm diameter and 3000 m length, provided with a valve at its end, water is flowing with a velocity of 2 m/s. Assuming velocity of pressure wave C = 1500 m/s, find :*

(*i*) *The rise in pressure if the valve is closed in 20 seconds, and*

(*ii*) *The rise in pressure if the valve is closed in 2·5 seconds. Assume the pipe to be rigid one and take bulk modulus of water as 2 GN/m².*

Solution. Diameter of the pipe, $D = 600$ mm $= 0{\cdot}6$ m

Length of the pipe, $L = 300$ m

Velocity of water, $V = 2$ m/s

Velocity of pressure wave, $C = 1500$ m/s.

(*i*) **Rise in pressure, p :**

Time taken to close the valve, $t = 20$ s

Now, the ratio, $\frac{2L}{C} = \frac{2 \times 3000}{1500} = 4$

The close of valve is said to be *gradual* if,

$$t > \frac{2L}{C} \qquad \text{...[Eqn. (12·28)]}$$

Hence, the valve is closed *gradually*.

The rise in pressure (p), for gradual closure of valve, is given by

$$p = \frac{\rho L V}{t} \qquad \text{...[Eqn. (12·26)]}$$

$$= \frac{1000 \times 3000 \times 2}{20} = 300 \times 10^3 \text{ N/m}^2 \text{ or } \mathbf{300 \text{ kN/m}^2 \text{ (Ans.)}}$$

(*ii*) **Rise in pressure, p :**

Time taken to close the valve, $t = 2{\cdot}5$ s

Bulk modulus of water, $K = 2$ GN/m²

Velocity of pressure wave is given by,

$$C = \sqrt{\frac{K}{\rho}} = \sqrt{\frac{2 \times 10^9}{1000}} = 1414{\cdot}2 \text{ m/s}$$

The ratio, $\frac{2L}{C} = \frac{2 \times 3000}{1414{\cdot}2} = 4{\cdot}24$ s

$$\therefore \quad t < \frac{2L}{C}$$

Thus, the valve is closed *intantaneously* [From eqn. (12·29)]

When pipe is rigid, the rise in pressure due to instantaneous closure of the valve is given by (eqn. 12·30),

$$p = V\rho C = 2 \times 1000 \times 1414{\cdot}2 \text{ N/m}^2 \text{ or } \mathbf{2828{\cdot}4 \text{ kN/m}^2 \text{ (Ans.)}}$$

Example 12·52. *Water is flowing in a pipe of 150 mm diameter with a velocity of 2·5 m/s when it is suddenly brought to rest by closing the valve. Find the pressure rise assuming pipe is elastic, E = 206 GN/m², Poisson's ratio = 0·25 and K for water = 2·06 GN/m². Pipe wall is 5 mm thick.*

Solution. Diameter of the pipe, $D = 150$ mm $= 0{\cdot}15$ m

Thickness of the pipe, $t = 5$ mm $= 0{\cdot}005$ m

Velocity of water, $V = 2{\cdot}5$ m/s

Modulus of elasticity, $E = 206$ GN/m²

Bulk modulus of water, $K = 2{\cdot}06$ GN/m²

Poisson's ratio, $\frac{1}{m} = 1/4$

Pressure rise, p :

Using the relation :

$$p = V\sqrt{\frac{\rho}{\left(\frac{1}{K} + \frac{D}{Et}\right)}} \qquad \text{...[Eqn. (12·31)]}$$

$$= 2{\cdot}5\sqrt{\frac{1000}{\left(\frac{1}{2{\cdot}06 \times 10^9} + \frac{0{\cdot}15}{2{\cdot}06 \times 10^9 \times 0{\cdot}005}\right)}} = 2{\cdot}5\sqrt{\frac{10^{12}}{0{\cdot}485 + 0{\cdot}1456}}$$

$= \mathbf{3148\ kN/m^2}$ **(Ans.)**

Example 12·53. *In a pressure penstock 4500 m long water is flowing at 4 m/s. If the velocity of the pressure wave travelling in the pipe due to sudden complete closure of a valve at the downstream end is given as 1500 m/s, find :*

(*i*) *The maximum pressure rise, and*

(*ii*) *The period of oscillation.*

Solution. Length of the penstock, $L = 4500$ m

Velocity of water, $V = 4$ m/s

Velocity of the pressure wave, $C = 1500$ m/s

(*i*) **The maximum pressure rise :**

Maximum pressure is given by

$$p = V\rho C \qquad \text{...[Eqn. (12·39)]}$$

$= 4 \times 1000 \times 1500$ N/m² or **6 MN/m² (Ans.)**

(*ii*) **The period of oscillation :**

The period of oscillation $= \frac{2L}{C} = \frac{2 \times 4500}{1500} =$ **6 seconds (Ans.)**

※ Determine the rise in pressure that will occur in a penstock leading to a power plant if the turbine gates are closed in 1.2 sec.

Initial head = 50 m

length = 1000 m

velocity = 3.6 m/s

Pressure wave velocity, c = 1429 m/s.

Soln:

we have, $T_c = \frac{2L}{c} = \frac{2 * 1000}{1429} = 1.4 \text{ sec} > 1.2 \text{ sec.}$

So it is the case of rapid ~~closer~~ closure.

i) In a rigid pipe:

$$P_w = v\sqrt{\rho K} = v\sqrt{\rho^2 \frac{K}{\rho}} = v c \rho = 3.6 * 1429 * 1000$$

$$= 5144400 \text{ kg/ms}^2 \text{ or N/m}^2$$

$$= 5144.40 \text{ kN/m}^2 //$$

ii) In a elastic pipe:

$$\left[c = \sqrt{\frac{K}{\rho}}\right]$$

$$P_w = v\sqrt{\frac{\rho}{\frac{1}{K} + \frac{d}{Et}}}$$

※ A steel penstock 0.6 m in diameter has a shell thickness of 12 mm. The modulus of elasticity of the pipe shell material is $2.1 * 10^5$ N/mm², the volume modulus of elasticity is $2.1 * 10^3$ N/mm². The pipe is designed to discharge the water through the pipe of mean velocity of 2.1 m/s. Determine the water hammer pressure rise caused by sudden closure of the value at the d/s a) neglecting the elasticity b) considering the elasticity.

Soln:

6

a) Neglecting the elasticity.

$$P_w = v\sqrt{\rho K} = 2.1 * \sqrt{1000 * 2.1 * 10^9}$$

$$= 3.043 * 10^6 \text{ N/m}^2 //$$

$$= 3.043 * 10^3 \text{ kN/m}^2 //$$

b) Considering the elasticity.

$$p_w = v\sqrt{\frac{\rho}{\frac{1}{K}+\frac{d}{Et}}} = 2.1 * \sqrt{\frac{1000}{\frac{1}{2.1*10^9}+\frac{0.6}{2.1*10^{11}*12*10^{-3}}}}$$

$$= 2.485 * 10^6 \ N/m^2 \ //.$$

$$= 2.485 * 10^3 \ KN/m^2 \ //$$

※ what is the maximum permissible velocity in a cast iron pipe line 100 mm ϕ and 15 mm thick, which can be suddenly stopped by a valve at the outlet end of the pipe ~~wh~~ without letting the ripe in pressure exceed $1.545 * 10^3$ KN/m².

$E = 2.1 * 10^5$ N/mm²

$K = 2.1 * 10^3$ N/mm²

Soln:

$$p = v\sqrt{\frac{\rho}{\frac{1}{K}+\frac{d}{Et}}}$$

$$\therefore v = p\sqrt{\frac{1/K+d/Et}{\rho}}$$

$$= 1.545 * 10^6 \sqrt{\frac{\frac{1}{2.1*10^9}+\frac{0.10}{2.1*10^{11}*15*10^{-3}}}{1.000}}$$

$$= 1.101 \ m/s. \ //.$$

EXAMPLE 10.1

A penstock, with an internal diameter of 1.20 m, supplies water at a head equivalent to 17.6 kg/cm^2. There is a possibility of 20 per cent increase in the pressure due to transient conditions. The design stress and the efficiency of the joint may be assumed to be 1020 kg/cm^2 and 85 per cent respectively. Calculate the approximate wall thickness of the penstock required.

SOLUTION

Here total pressure intensity, including over pressure $\qquad p = 17.6 + 3.52 = 21.12 \text{ kg/cm}^2$

Design stress $\qquad s = 1020 \text{ kg/cm}^2$

Joint efficiency $\qquad \eta = 85\%$

Internal radius $\qquad R = 60 \text{ cm}$

Wall thickness

$$t = \frac{pR}{s\eta - (0.6\,p)} + 0.15 \text{ cm}$$

$$= \frac{21.12 \times 60}{1020 \times 0.85 - (0.6 \times 21.12)} + 0.15 \text{ cm}$$

$$= \frac{1267.2}{866 - 12.67} + 0.15 \text{ cm}$$

$$= 1.485 + 0.15 \text{ cm}$$

or $\qquad t = 1.635 \text{ cm}$

Hydro-electric Machines

Hydro-mechanical Equipment

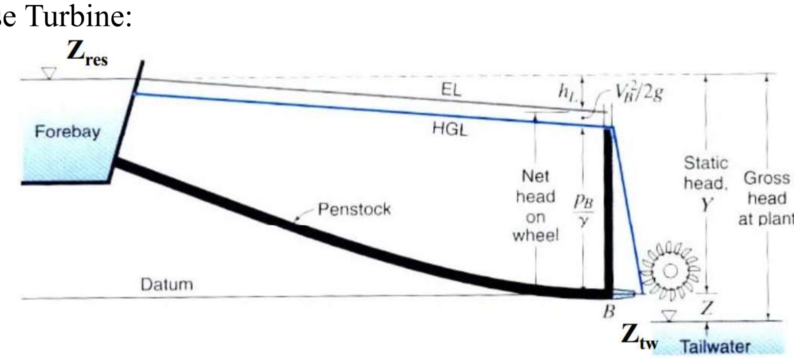
Hydraulic Turbines:

- Mechanical device that converts the potential energy contained in an elevated body of water (a stream or reservoir) into rotational mechanical energy
- Primary function is to drive a electric generator

Types of Hydraulic Turbines:

A) According to the energy conversion

1. Impulse Turbine:



Hydro-mechanical Equipment

1. Impulse Turbine:

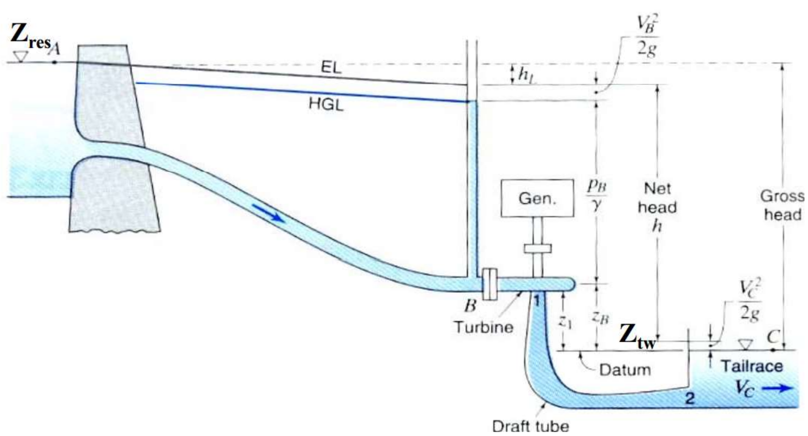
- The flow energy is completely converted to kinetic energy before transformation in the runner
- The impulse forces being transformed by the direction changes of the flow velocity vectors when passing the buckets create the energy converted to mechanical energy on the turbine shaft
- The flow enters the runner from jets spaced around the rim of the runners; and the jet hits momentarily only a part of the circumference of the runner.
- Pelton turbine and Turgo turbine

Hydro-mechanical Equipment

Types of Hydraulic Turbines:

According to the energy conversion

2. Reaction Turbine:



Hydro-mechanical Equipment

2. Reaction Turbine:

- Two effects cause the energy transfer from the flow to the mechanical energy on the turbine shaft:
  - Firstly, it follows from a drop in pressure from inlet to outlet of the runner. This is denoted as the reaction part of the energy conversion.
  - Secondly the changes in the directions of the flow velocity vectors through the runner blade channels transfer impulse forces. This is denoted as the impulse part of the energy conversion.
- The pressure drop from inlet to outlet of the runners is obtained because the runners are completely filled with water.
- Francis turbine, Kaplan turbine and Propeller turbine

Hydro-mechanical Equipment

Difference between Impulse and Reaction turbine

Aspects	Impulse	Reaction
Conversion of fluid energy	Converted into KE via nozzle	Partly transformed into KE before it enters the runner of turbine
Changes in pressure and energy	The pressure remains atmospheric throughout the action of water in runner	After entering the runner with an excess pressure, water undergoes changes both in velocity and pressure while passing through runner
Water tight case	Not required	Essential
Interaction with water	Wheel does not run full and air has free access to buckets	Water completely fills and all the passages between blades and while flowing between inlet and outlet sections does work on blade
Install of units	Always installed above TWL	Can be installed below TWL
Draft tube	Not used	Necessary to recover lost energy
Flow Regulation	By means of spear valve or deflector fitted into the nozzle	By the adjustment of wicket gates (guide vanes)
Specific speed	Low (7-20)	Moderate to high (50-300:Francis; 240-920: Kaplan)

B) According to Direction of Flow

1. Tangential flow turbine:
- The flow of water strikes the runner in the direction of tangent to the wheel
- Example: Pelton wheel turbine

2. Axial flow turbine:
- The flow of water is in the direction parallel to the axis of the shaft
- Example: Kaplan turbine and Propeller turbine

3. Radial flow turbine:
- The flow of water strikes in the radial direction
- Example: Old Francis turbine

4. Mixed flow turbine:
- The water enters the runner in the radial direction and leaves in axial direction
- Example: Modern Francis turbine

Hydro-mechanical Equipment

Types of Hydraulic Turbines:

C) According to name of Inventor

1. Pelton wheel turbine
2. Francis turbine
3. Kaplan turbine
4. Deraiz turbine

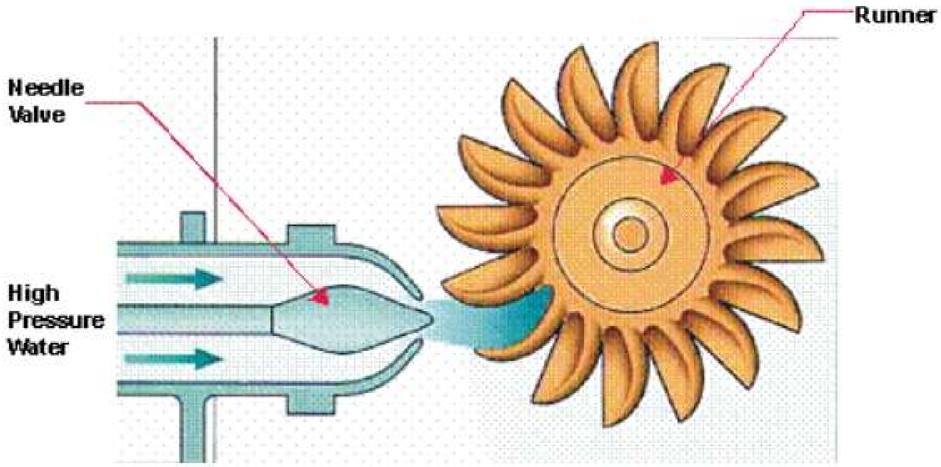
D) According to Head

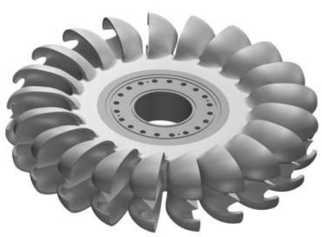
1. Very high head:
   > 500 m – Pelton wheel turbine
2. High head:
   71-500 m – Pelton wheel turbine and Francis turbine
3. Medium head:
   16-70 m – Francis turbine and Kaplan turbine
4. Low head:
   2-15 m – Kaplan series

Hydro-mechanical Equipment

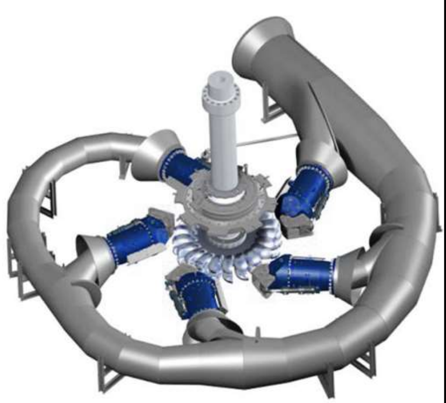
Types of Hydraulic Turbines:

1. Pelton wheel turbine:





Runner



5-jet Pelton turbine

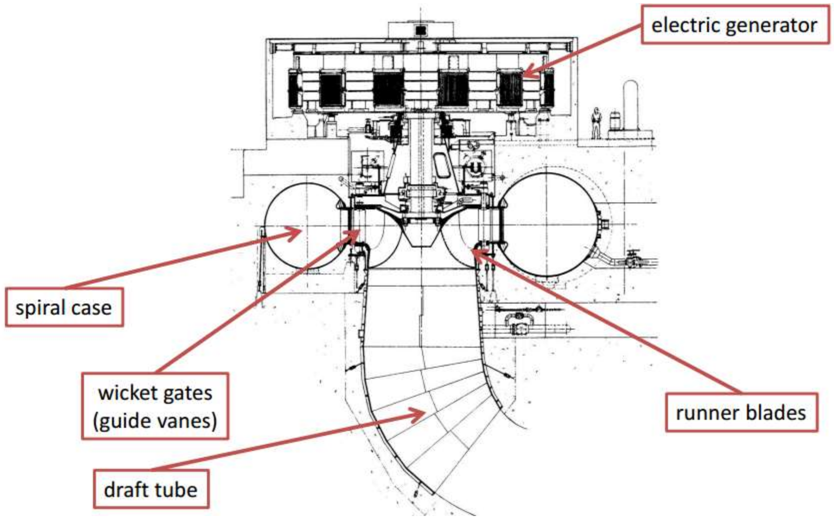
Pelton Turbine

5-jet turbine

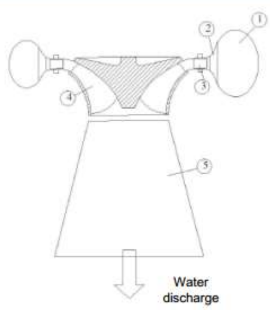
- Invented by Lester Allan Pelton in the 1870s
- An impulse type water turbine
- Extracts energy from the impulse of moving water
- The pressure all over the wheel is constant and equal to atmospheric pressure
- The water flows along thetangent to the path of the runner
- Nozzles direct forceful streams of water against a series of spoon-shaped buckets mounted around the edge of a wheel
- As water flows into the bucket, the direction of the water velocity changes to follow the contour of the bucket
- When the water-jet contacts the bucket, the water exerts pressure on the bucket and the water is decelerated as it does a "U-turn“ and flows out the other side of the bucket at low velocity
- Casing is provided to prevent splashing of the water and to discharge water to tailrace
- Suitable for high head plants
- Used in 60 MW Khimti HPP (H=650m), 60 MW Kulekhani I (H=550m) and 32 MW Kulekhani II (H=284.1 m)

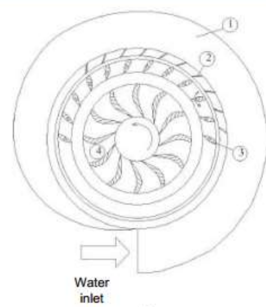
Hydro-mechanical Equipment

2. Francis turbine:

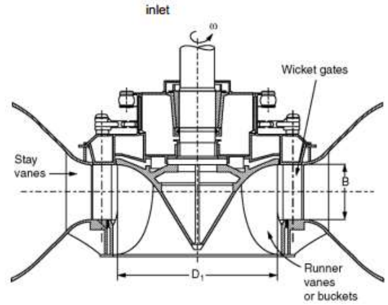


Main components

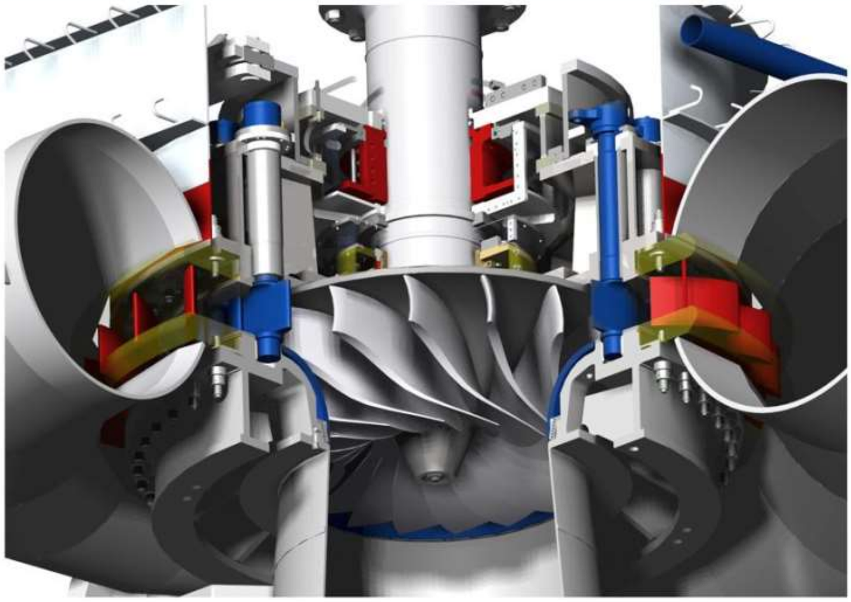




1. spiral case
2. stay vanes
3. wicket gates (guide vanes)
4. runner
5. draft tube



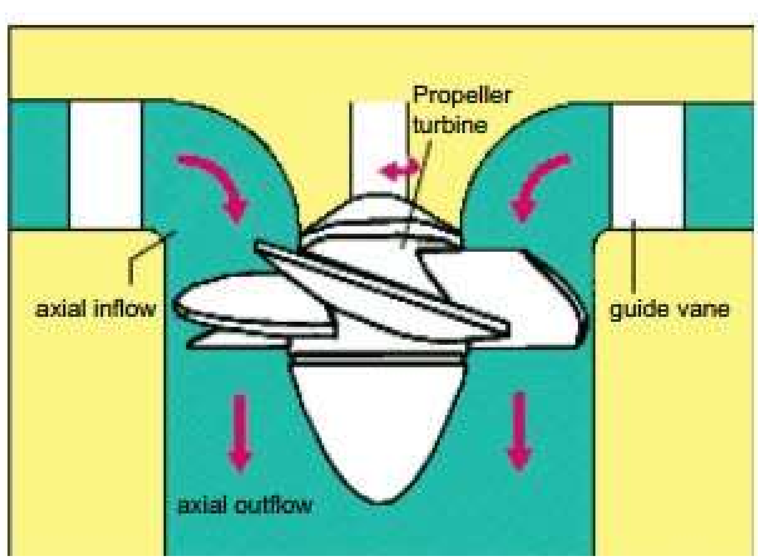
Main components



- Developed by James B. Francis in 1984
- It is an inward-flow reaction turbine that combines radial and axial flow concepts
- Operates at its best completely filled with water at all times
- The most common water turbine in use today
- Operates in a water head from 40 to 600 m
- Power output ranges just a few kilowatts up to 800 MW
- The speed range of the turbine is from 75 to 1000 rpm
- Wicket gates around the outside of the turbine's rotating runner control the rate of water flow through the turbine for different power production rates
- The runner consists of 12-22 nos of curved blades
- Water is discharged to the tailrace through a closed tube of gradually enlarged section, called draft tube
- Used in 144 MW Kali Gandaki HPP (H=115 m), 69 MW Marsyangdi (H=95 m); both have three units of Francis turbine

Hydro-mechanical Equipment

3. Propeller turbine:



➢Propeller turbines have non-adjustable propeller vanes. They are used in where the range of flow / power is not large. Larger propeller turbines produce more than 100 MW

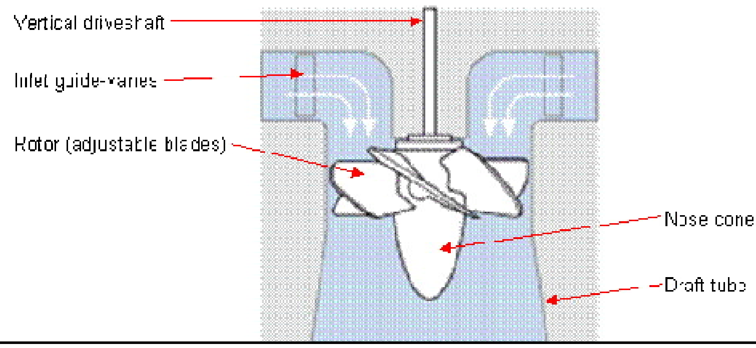
➢Axial flow turbine

➢No of blades: 3-8

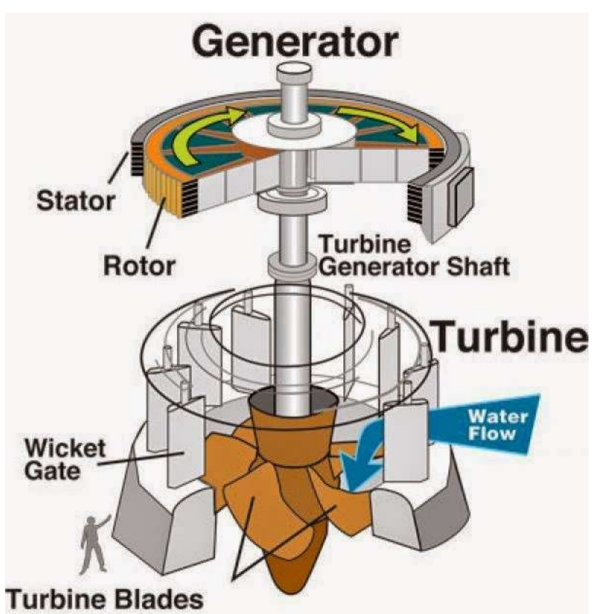
➢The head ranges from 4–80 m

➢Runner diameter: 2-11 m

4. Kaplan turbine:



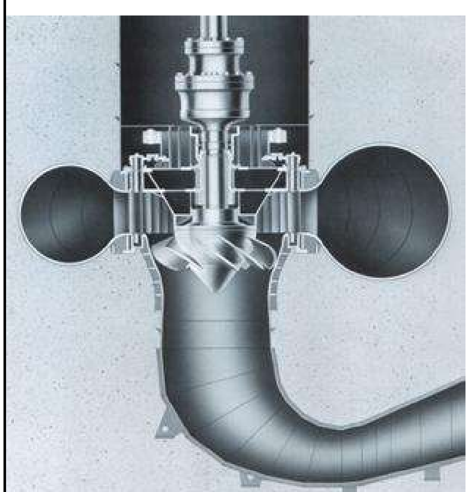
Hydro-mechanical Equipment

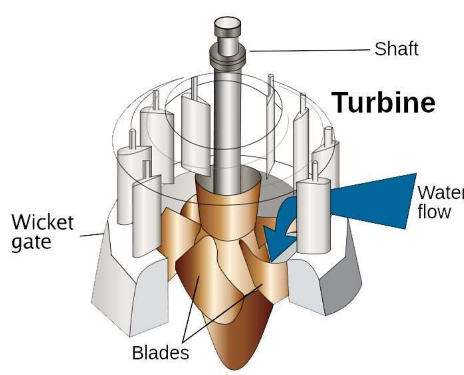


- Axial flow turbine
- No of blades: 3-8
- Kaplan turbine is developed by Viktor Kaplan in 1913
- Kaplan turbine is the propeller-type water turbine which has adjustable blades; combined automatically adjusted propeller blades with automatically adjusted wicket gates to achieve efficiency over a wide range of flow and water level
    - Large range of load (50% under load - 50% over load)
- The Kaplan turbine was an evolution of the Francis turbine; allowed efficient power production in low-head applications that was not possible with Francis turbine
    - The head ranges from 10–70 m and the output from 5 to 200 MW
    - Runner diameter: 2-11 m
- Kaplan turbines are now widely used throughout world in high-flow, low-head power production

Hydro-mechanical Equipment

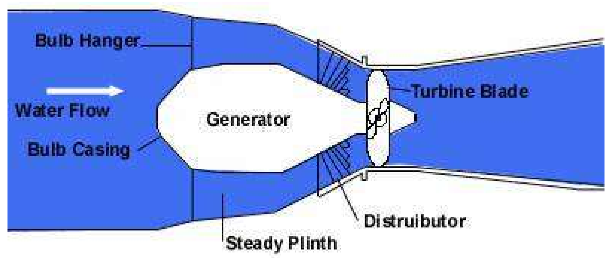
5. Deriaz turbine:





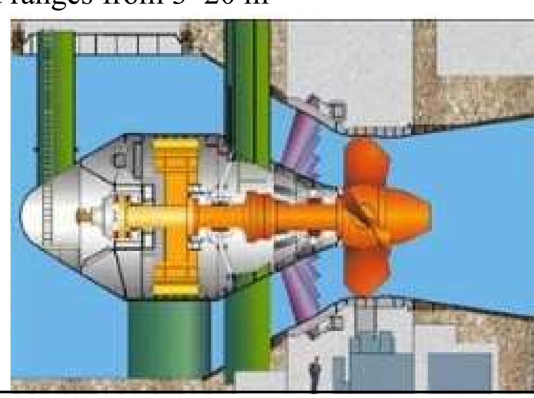
➢Invented by Paul Deriaz in 1956
➢Mixed-flow reaction turbine
➢Similar to a Kaplan turbine but inclined blades (flow over the runner is at 45°) to make it more suitable for higher heads
➢Particularly suitable for the head range between 20 - 100 m, in between the ranges of Francis and Kaplan turbines
➢No of blades: 10-12

6. Bulb turbine:



Hydro-mechanical Equipment

➢Generator is housed in enclosed bulb-shaped casing
➢Axial flow reaction turbine
➢immersed in the water channel, the flow enters and exits the turbine with minor changes in direction
➢The draft tube is straight flaring tube
➢Compact arrangement of components and less danger of cavitations
➢The head ranges from 3–20 m



Turbine Efficiency

Hydraulic Efficiency: The hydraulic efficiency of the turbine is the ratio of the power developed by the runner to the net power supplied by the water at the entrance to the turbine.

$$\eta_h = \frac{\text{power developed by runner}}{\text{net power supplied at the runner}}$$

Mechanical Efficiency: The mechanical efficiency of the turbine is the ratio of the power available at the turbine shaft to the power developed by the runner. These two differ by an amount of the mechanical losses viz. bearing loss.

$$\eta_m = \frac{\text{power available at the turbine shaft}}{\text{power developed by the runner}}$$

Volumetric Efficiency: The volumetric efficiency is the ratio of quantity of water actually striking the runner and the quantity of the water supplied to the turbine. These two quantities differ by an amount of water that slips directly to the tailrace without striking runner.

$$\eta_v = \frac{Q}{Q + \Delta Q}$$

Overall Efficiency: The overall efficiency of the turbine is the ratio of the power available at the turbine shaft to the power supplied at the entrance to the turbine.

$$\eta_o = \eta_h \times \eta_v \times \eta_m$$

Turbine Speeds

1. Runway speed

If the external load on the machine suddenly drops to zero (sudden rejection) and the governing mechanism fails at the same time, the turbine will tend to race up the maximum possible speed, known as runway speed. The suggested runway speed of the various runners for their appropriate design, and acceptable head variations of such turbines are given in following table.

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

2. Specific speed (Ns)

The specific speed of a turbine is defined as a speed of geometrical similar turbine that would develop unit power under unit head. All geometrical similar turbine (irrespective of size) will have same specific speed when operating under same head.

$$N_S = \frac{P^{1/2} N}{H^{5/4}}$$

where,

N = Rotational speed of turbine in rpm

P = Power out-put

H = Head of turbine

Specific speed (Ns) for different type of turbines

Given below the specific speed of different type of turbines that would develop 1 HP under 1 m head.

$$N_s = \frac{2400}{\sqrt{H}} \quad \text{for Francis Turbine}$$

$$N_s = \frac{1080}{H^{\frac{1}{4}}} \quad \text{for Fixed Blade Propeller Turbine}$$

$$N_s = \frac{1475}{H^{\frac{1}{3}}} \quad \text{for Adjustable Blade Propeller (Kaplan) Turbine}$$

3. Synchronous speed

If the turbine is directly connected to the generator, the turbine speed η must be a synchronous speed. For the turbine speed n to be synchronous, the following equation must be fulfilled.

$$n = \frac{120f}{N_p}$$

Where n= rotational speed

f = electrical frequency in Hertz, Hz (50Hz in Nepal)

N_p = number of generator pole.

(divisible by 4 for head up to 200 m or by 2 for heads above 200 m)

4. Speed factor or speed ratio (ø)

It is the ratio of peripheral speed 'v' of the buckets or vanes at the nominal diameter 'D' to the theoretical velocity of water under the effective head 'H' acting on the turbine.

$$\text{Ø} = v/\sqrt{(2gH)} = \pi D N/(60\sqrt{(2gH)})$$

N= Angular velocity in rpm

Range of ϕ values, specific speeds and heads

Type of Runner	ϕ	N_s	H(m)	Efficiency (%)
Impulse	0.43-0.48	8-17 17 17-30	>250	85-90 90 90-82
Francis	0.60-0.90	40-130 130-350 350-452	25-450	90-94 94 94-93
Propeller	1.4-2.0	380-600 600-902	<60	94 94-85

18·10. Unit Quantities

Let us consider a *single unit*. When the head on the unit is changed/varied then the speed of an ungoverned turbine changes. The velocities at various points do not change direction but their magnitudes vary in proportion to the *square root of the head*.

At a given point in the turbine under a head H, let

V = Absolute velocity,

V_r = Relative velocity,

u = Peripheral velocity, and

V', V_r', u' = Corresponding values at a different head H', then as velocity is proportional to $\sqrt{H}$, we have

$$\frac{u}{u'} = \frac{V_r}{V_r'} = \frac{V}{V'} = \frac{\sqrt{H}}{\sqrt{H'}} \quad \text{...(18·34)}$$

If the discharges are Q and Q' then,

$$\frac{Q}{Q'} = \frac{V}{V'} = \frac{N}{N'} = \frac{\sqrt{H}}{\sqrt{H'}} \quad \text{...(18·35)}$$

If the power outputs are P and P' then,

$$\frac{P}{P'} = \frac{QH}{Q'H'} = \frac{\sqrt{H}}{\sqrt{H'}} \times \frac{H}{H'} = \left(\frac{H}{H'}\right)^{3/2} \quad \text{...(18·36)}$$

$$\left(\because \frac{Q}{Q'} = \frac{\sqrt{H}}{\sqrt{H'}}\right)$$

The hydraulic efficiency of the turbine under these two heads may be considered to be nearly same, as the velocity triangles at these heads are similar at a point.

If the various quantities are *reduced to a theoretical one metre head* the comparison of performance data and computations of experimental values on a single unit are *considerably simplified*.

Then, $$N_u = \frac{N}{\sqrt{H}} \quad ...(18{\cdot}37)$$

$$Q_u = \frac{Q}{\sqrt{H}} \quad ...(18{\cdot}38)$$

$$P_u = \frac{P}{H^{3/2}} \quad ...(18{\cdot}39)$$

The above quantities are called **unit quantities** of a turbine. *Unit speed is the hypothetical speed of the turbine operating under one metre head.* Similarly, other proportionality constants in Eqns. 18·38 and 18·39 are defined.

For presenting the performance of geometrically similar turbines independent of the actual head, discharge and power output the ***unit characteristics*** prove quite helpful. **Geometrically similar turbines will have the same unit characteristics under similar operating conditions.** Thus with the help of a model the performance of a prototype can be predicted within certain limits.

If a turbine is working under different heads the behaviour of the turbine can be easily known from the values of the *unit quantities* as follows :

Let, H_1, H_2 = Heads under which a turbine works,

N_1, N_2 = Corresponding speeds,

Q_1, Q_2 = Corresponding discharges, and

P_1, P_2 = Corresponding powers developed.

Then using eqns. (18·37), (18·38), (18·39), respectively, we obtain

$$N_u = \frac{N_1}{\sqrt{H_1}} = \frac{N_2}{\sqrt{H_2}} \quad ...(18{\cdot}40)$$

$$Q_u = \frac{Q_1}{\sqrt{H_1}} = \frac{Q_2}{\sqrt{H_2}} \quad ...(18{\cdot}41)$$

$$P_u = \frac{P_1}{H_1^{3/2}} = \frac{P_2}{H_2^{3/2}} \quad ...(18{\cdot}42)$$

Example 18·49. *A turbine is to operate under a head of 25 m at 200 r.p.m. The discharge is 9 m³/s. If the efficiency is 90 per cent determine the performance of turbine under a head of 20 m.* [M.U.]

Solution. Head under which turbine works, $H_1 = 25$ m

Speed of the turbine, $N_1 = 200$ r.p.m.

Discharge through the turbine, $Q_1 = 9$ m³/s

Efficiency (overall), $\eta_0 = 90\%$

Performance of turbine under a head of 20 m; N_2, Q_2, P_2 :

Performance of turbine under a head, $H_2 = 20$ m means to find speed (N_2), discharge (Q_2), and power generated (P_2) by the turbine when working under a head of 20 m.

Overall efficiency, $$\eta_0 = \frac{\text{Shaft power}}{\text{Water power}} = \frac{P}{wQH} = \frac{P_1}{wQ_1H_1}$$

$$\therefore \quad P_1 = \eta_0 \times wQ_1H_1 = 0{\cdot}9 \times 9{\cdot}81 \times 9 \times 25 = 1986{\cdot}5 \text{ kW}$$

Now, $$\frac{N_1}{\sqrt{H_1}} = \frac{N_2}{\sqrt{H_2}}$$ ...[Eqn. (18·40)]

$$\therefore \quad N_2 = \frac{N_1\sqrt{H_2}}{\sqrt{H_1}} = \frac{200 \times \sqrt{20}}{\sqrt{25}} = 178{\cdot}88 \text{ r.p.m. } \textbf{(Ans.)}$$

and, $$\frac{Q_1}{\sqrt{H_1}} = \frac{Q_2}{\sqrt{H_2}}$$ ...[Eqn. (18·41)]

$$\therefore \quad Q_2 = \frac{Q_1\sqrt{H_2}}{\sqrt{H_1}} = \frac{9 \times \sqrt{20}}{\sqrt{25}} = 8{\cdot}05 \text{ m}^3\text{/s } \textbf{(Ans.)}$$

and, $$\frac{P_1}{H_1^{3/2}} = \frac{P_2}{(H_2)^{3/2}}$$ ...[Eqn (18·42)]

$$\therefore \quad P_2 = \frac{P_1 \times (H_2)^{3/2}}{(H_1)^{3/2}} = \frac{1986{\cdot}5 \times (20)^{3/2}}{(25)^{3/2}} = 1421{\cdot}4 \text{ kW } \textbf{(Ans.)}$$

Performance of Turbines under unit quantities

The unit quantities give the speed, discharge and power for a particular turbine under a head of 1m assuming the same efficiency. Unit quantities are used to predict the performance of turbine.

1. Unit speed (N_u) - Speed of the turbine, working under unit head
2. Unit power (P_u) - Power developed by a turbine, working under a unit head
3. Unit discharge (Q_u) - The discharge of the turbine working under a unit head

18·13. Performance Characteristics of Hydraulic Turbines

The turbines are normally designed for specific values of head, speed, discharge, power and efficiency (known as the *designed conditions*). But oftenly turbines may be required to operate under conditions different from those for which these have been designed. Thus, to know about their exact behaviour under varying conditions it becomes necessary to conduct tests either on the actual turbines at the site or on their small scale models in a research laboratory. The results so obtained are usually represented graphically and the curves obtained are known as "*Characteristic curves*". These curves are usually plotted in terms of unit quantities (for sake of convenience). The characteristic curves are of the following types :

1. Main or constant head characteristic curves.
2. Operating or constant speed characteristic curves.
3. Constant efficiency or iso-efficiency or Muschel curves.

18·13·1. Main or Constant Head Characteristic Curves

- *Head and gate opening are maintained constant.*
- *Speed is varied* by allowing a variable quantity of water to flow through the inlet opening.
- The brake power (P) is then measured mechanically by means of a dynamometer.
- The overall efficiency and unit quantities are then calculated by using the basic data; these are then plotted *against unit speed as abscissa.*

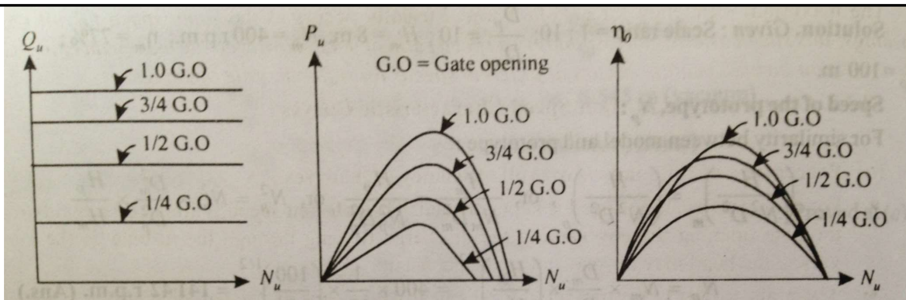


Fig. 18.53. Main characteristic curves of Pelton wheel.

Figs. 18·53, 18·54 and 18·55 show the main characteristic curves of Pelton wheel, Francis turbine and Kaplan turbine respectively.

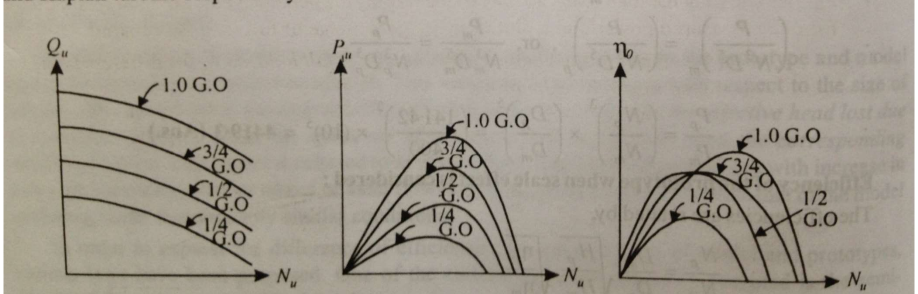


Fig. 18.54. Main characteristic curves of Francis turbine.

The main characteristic curves yield the following information :

- The discharge Q_u for a *Pelton wheel* depends only upon the gate opening and is independent of N_u; the curves for Q_u are horizontal.
- The curves between Q_u and N_u for a *Francis turbine are falling curves*. This is due to the fact that a *centrifugal head develops which* acts outwards and *opposes* the external head causing flow, eventually decreasing the discharge as the speed increases.
- The curves between Q_u and N_u for a *Kaplan turbine* are *rising curves*; the discharge increases with the increase in speed.

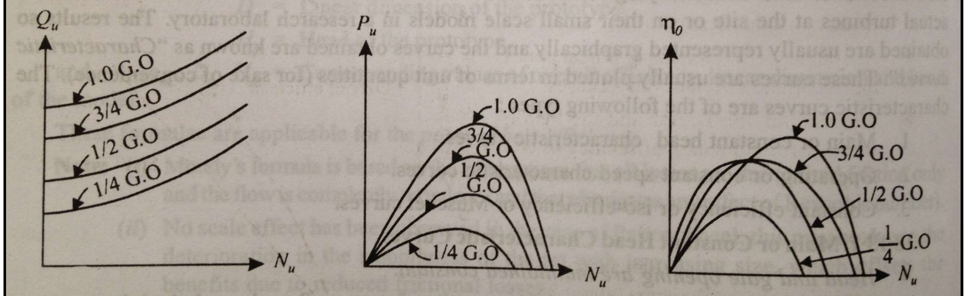


Fig. 18.55. Main characteristic curves of Kaplan turbine.

- The curves between P_u and N_u and those between η_0 and N_u indicate that at a particular speed the efficiency is maximum.

The maximum efficiency for a *Pelton wheel* usually occurs at the *same speed for all gate openings; this speed usually corresponds to a speed ratio of 0·45 . However, the maximum efficiency for a reaction turbine usually occurs at different speeds for different gate openings.*

18·3·2. Operating or Constant Speed Characteristic Curves

These curves are obtained as follows :

(*a*) **Percentage of full load *v/s* overall efficiency (η_0) curves :**

- For each gate opening speed is kept constant. The constant speed is attained by regulating the gate opening thereby varying the discharge flowing through the turbine as the load varies; the head may or may not remain constant.
- The brake power (*P*) is measured mechanically by means of a dynamometer.
- The overall efficiency (η_0) is then calculated from the measured values of discharge, head and power.
- Further knowing the total load capacity of the turbine the percentage of full load is computed from the measured power and a plot of η_0 *v/s* percentage of full load is prepared.

Fig. 18·56 shows the graphs plotted between *percentage of full load v/s* η_0 for different types of turbines. The following *points are worth noting* :

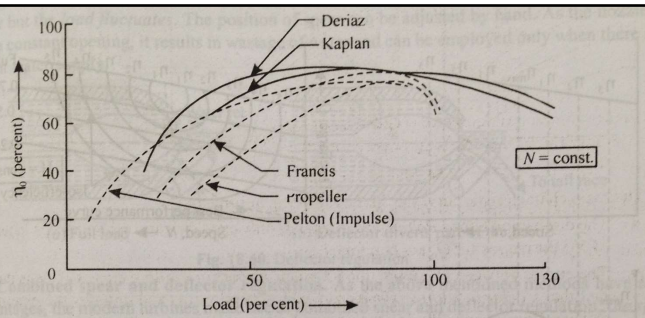


Fig. 18.56. Percentage of full load v/s η_0 curves for hydraulic turbines.

- As the percentage full load increases η_0 also increases (In other words, at reduced loads η_0 is also less).
- At 100 per cent full load η_0 is near about the maximum efficiency in all cases.
- The Kaplan, the Deriaz and the Pelton wheel maintain a high efficiency over a longer range of *part load* as compared with either the Francis or the fixed blade propeller turbine.
- The maximum overall efficiency of all the turbines is almost the same (about 85%).

(*b*) **Overall efficiency (η_0) and output (shaft) power (P) v/s discharge (Q) curves:**

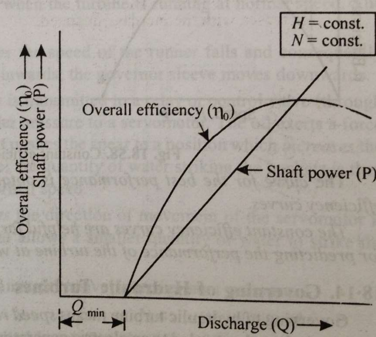


Fig. 18.57. η_0 and P v/s discharge curves.

Fig. 18·57 shows overall efficiency (η_0) and shaft power (P) *v/s* discharge curves. Q_{min} is the minimum discharge required to set the turbine runner into motion from its state of rest. These curves yield the following information :

- Shaft power or output power (P) is a straight line, since $P \propto Q$ if H (head) is constant.
- η_0 *v/s* discharge (Q) graph is curvilinear and η_0 increases with Q and remains *nearly* constant beyond a particular value of discharge.

18·13·3. Constant efficiency or iso-efficiency or Muschel curves

Refer to Fig. 18·58. As η–N curve is of parabolic nature, there exits two speeds for one value of efficiency except for maximum efficiency which occurs at one speed only. Corresponding to these values of speeds there are also two values of discharge for each value of efficiency (Q-N curve). Hence on Q-N curve we can plot two points for each value of efficiency and one point for maximum efficiency. By adopting this procedure for different gate openings or heads we can get number of Q-N curves and we can plot on them efficiency points (as described above). The points denoting the same efficiency can now be joined to get constant iso-efficiency curves or Muschel curves (The German word 'Muschel' means shell, indicating shape of curve). The diagram showing these curves is also called Hill diagram (since it looks like top view of a hill). In actual practice unit speed and unit discharge are taken along the co-ordinate axes.

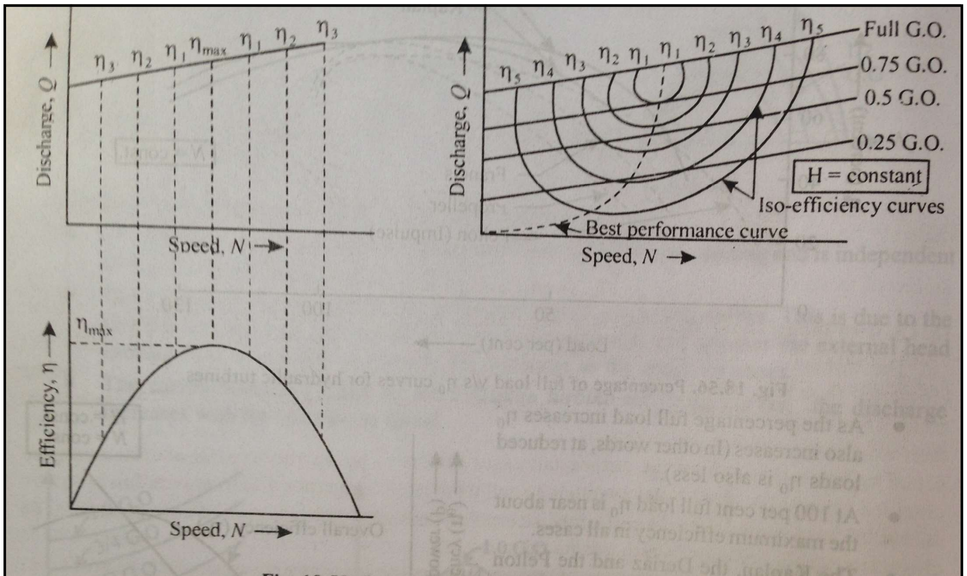
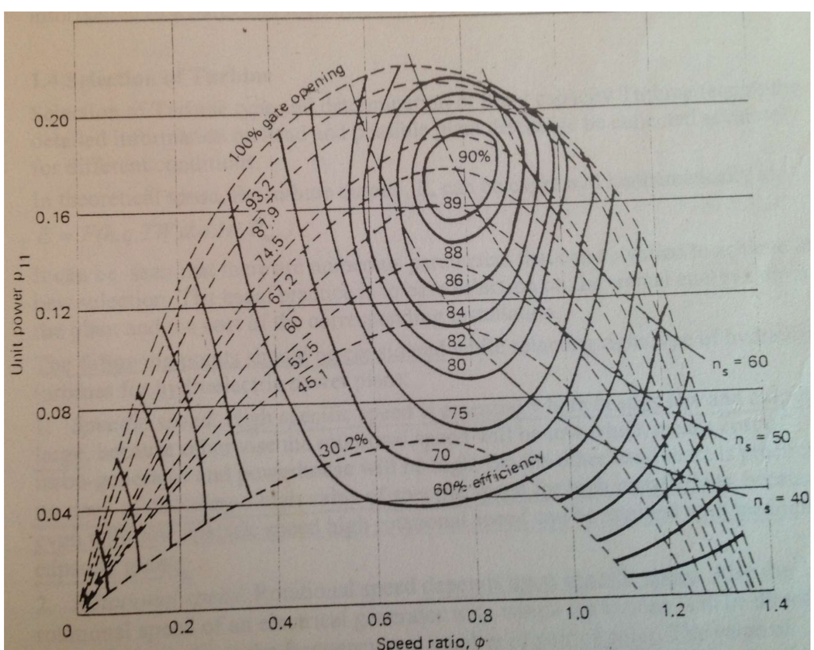


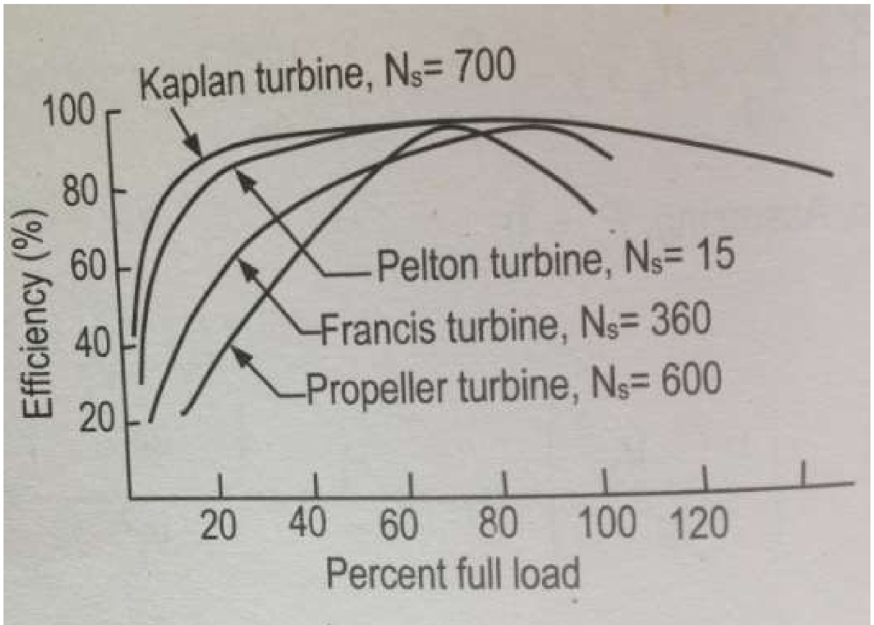
Fig. 18.58. Constant efficiency curves for turbines.

The *curve for the best performance is obtained by joining the peak points of the various efficiency curves.*

The constant efficiency curves are helpful for determining the zone of constant efficiency and for predicting the performance of the turbine at various efficiencies.

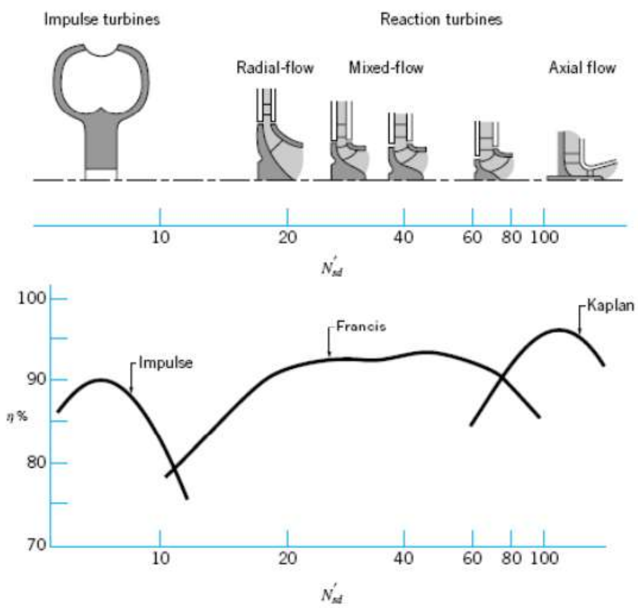


Typical turbine hill curve



Efficiency Vs Percent full load Curves

Efficiency vs Specific Speed



Draft Tube

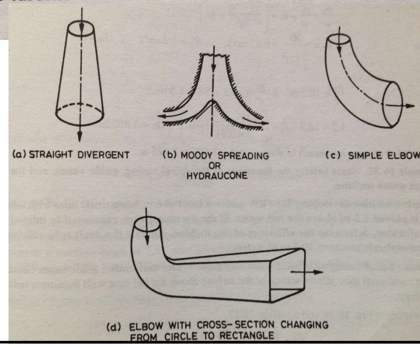
Draft tube is an integral part of the low head turbines with large through flow, *i.e.*, for reaction turbines of the Francis and Kaplan type. The unit consists an airtight diverging conduit with cross-sectional area increasing along its length. One end of this diverging tube is connected to the runner exit and the other is located below the level of tail race.

A draft tube has the following functions to perform :

- decrease the pressure at the runner exit to a value less than atmospheric pressure and thereby increase the working head.
- recover a portion of the exit kinetic energy which otherwise goes waste to the tail race.

These two factors help the turbine to develop more power and thereby the efficiency of the turbine increases. The unit also serves to fix the turbine above the tail race and that ensures proper inspection of the turbine.

Types of Draft Tube

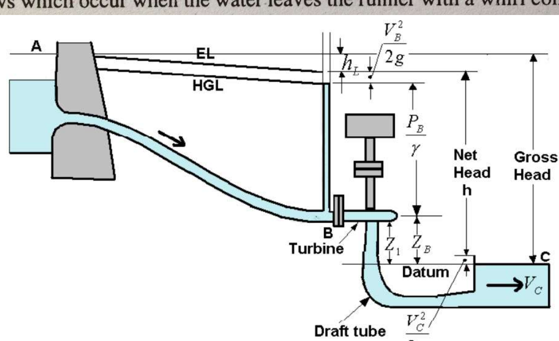


* *Straight conical type* which takes the form of the frustrum of a cone with central angle less than 8-degree ($\alpha = 4^o$) so as to prevent the possibility of flow separation. This draft tube has an efficiency of about 90% and is generally employed for low specific speed and vertical shaft turbines.

* *Simple elbow type* finds application where the length of the shaft has to be minimum so as to cut short the volume of excavation. Because of bend there is a loss of head and so the elbow type draft tube has a low efficiency of the order of 60%.

* *Elbow type* having a circular cross-section at the inlet and a square or rectangular section at the outlet. The change from circular section in the vertical leg to rectangular section in the horizontal leg takes place in the bend. The draft tube efficiency is about 85%.

* *Moody's spreading draft tube* is provided with a solid central core of conical shape which reduces whirling action of discharge water. The draft tube has an efficiency of about 85% and is suited particularly for helical flows which occur when the water leaves the runner with a whirl component.



Energy Equation Applied to Draft Tube

$$\frac{P_B}{\gamma} + Z_B + \frac{V_B^2}{2g} + h_L$$

- The velocity V_2 can be reduced by having a diverging passage.
- To prevent cavitation, the vertical distance z_1 from the tail water to the draft tube inlet should be limited so that at no point within the draft tube or turbine will the absolute pressure drop to the vapour pressure of water.

Absolute velocity at inlet of draft tube for Francis turbine (m/s): $V = 8.74 + \frac{248}{N_s}$

Absolute velocity at inlet of draft tube for Kaplan turbine (m/s): $V = 8.42 + \frac{250}{N_s}$

Draft-tube theory

Consider a conical draft tube as shown in figure.

Let H_s - Vertical height of draft tube above the tail race.

y - Distance of bottom of draft tube from tail race.

Applying Bernoulli's equation to section 1-1 and 2-2

$$(H_s + y) + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + h_f$$

h_f loss of energy between 1-1 & 2-2

here $\frac{P_2}{\gamma} = \frac{P_a}{\gamma} + y$

or $$(H_s + y) + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} = \frac{P_a}{\gamma} + y + \frac{V_2^2}{2g} + h_f$$

or $$\frac{P_1}{\gamma} = \frac{P_a}{\gamma} + \frac{V_2^2}{2g} + h_f - \frac{V_1^2}{2g} - H_s$$

$$\frac{P_1}{\gamma} = \frac{P_a}{\gamma} - H_s - \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} - h_f\right)$$

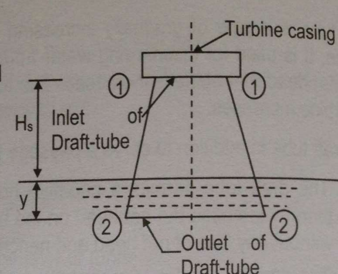


Figure: Draft tube theory

Hence in this equation $\frac{P_1}{\gamma}$ is less than atmospheric pressure. There is the limiting value for the setting of the runner with respect tail water due to cavitations effect.

Efficiency of draft tube

$$\eta_d = \frac{\text{Atual conversion of kinetic head into pressure head}}{\text{Kinetic head at the inlet of draft tube}}$$

Theoretical conversion of kinetic head into pressure head in draft tube = $(\frac{V_1^2}{2g} - \frac{V_2^2}{2g})$

Actual Conversion of kinetic head into pressure head = $(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}) - h_f$

Draft tube efficiency, $\eta_d = \frac{(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}) - h_f}{V_1^2 / 2g}$

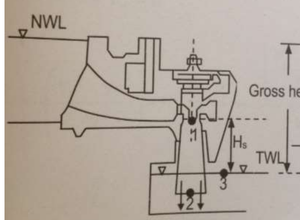


Fig: Vertical Draft Tube

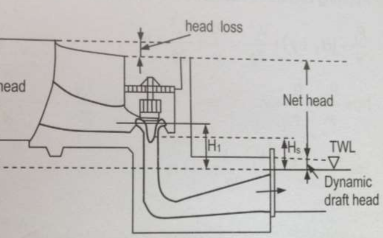


Fig: Elbow type Draft

Cavitation in Hydraulic Turbine

Cavitation is formation of vapor bubbles in the liquid flowing through any Hydraulic Turbine. Cavitation occurs when the static pressure of the liquid falls below its vapor pressure. Cavitation is most likely to occur near the fast moving blades of the turbines and in the exit region of the turbines.

Causes of Cavitation

The liquid enters hydraulic turbines at high pressure; this pressure is a combination of static and dynamic components. Dynamic pressure of the liquid is by the virtue of flow velocity and the other component, static pressure, is the actual fluid pressure which the fluid applies and which is acted upon it. Static pressure governs the process of vapor bubble formation or boiling. Thus, Cavitation can occur near the fast moving blades of the turbine where local dynamic head increases due to action of blades which causes static pressure to fall. Cavitation also occurs at the exit of the turbine as the liquid has lost major part of its pressure heads and any increase in dynamic head will lead to fall in static pressure causing cavitation.

Detrimental Effects of Cavitation

The formation of vapor bubbles in cavitation is not a major problem in itself but the collapse of these bubbles generates pressure waves, which can be of very high frequencies, causing damage to the machinery. The bubbles collapsing near the machine surface are more damaging and cause erosion on the surfaces called as cavitation erosion. The collapses of smaller bubbles create higher frequency waves than larger bubbles. So, smaller bubbles are more detrimental to the hydraulic machines.

Smaller bubbles may be more detrimental to the hydraulic machine body but they do not cause any significant reduction in the efficiency of the machine. With further decrease in static pressure more number of bubbles is formed and their size also increases. These bubbles coalesce with each other to form larger bubbles and eventually pockets of vapor. This disturbs the liquid flow and causes flow separation which reduces the machine performance sharply. Cavitation is an important factor to be considered while designing Hydraulic Turbines.

Avoiding Cavitation

To avoid cavitation while operating Hydraulic Turbines parameters should be set such that at any point of flow static pressure may not fall below the vapor pressure of the liquid. These parameters to control cavitation are pressure head, flow rate and exit pressure of the liquid. The control parameters for cavitation free operation of hydraulic turbines can be obtained by conducting tests on model of the turbine under consideration. The parameters beyond which cavitation starts and turbine efficiency falls significantly should be avoided while operation of hydraulic turbines.

In a reaction turbine, the point of minimum pressure is usually at the outlet end of the runner blades, i.e., at the inlet to the draft tube. For the flow between such a point and the final discharge into the trail race (where the pressure is atmospheric), the Bernoulli's equation can be written, in consideration of the velocity at the discharge from draft tube to be negligibly small, as

$$\frac{p_e}{\rho g} + \frac{V_e^2}{2g} + z = \frac{p_{atm}}{\rho g} + hf \qquad (1)$$

where, *pe* and *ve* represent the static pressure and velocity of the liquid at the outlet of the runner (or at the inlet to the draft tube). The larger the value of *ve*, the smaller is the value of *pe* and the cavitation is more likely to occur. The term *hf* in Eq. (1) represents the loss of head due to friction in the draft tube and *z* is the height of the turbine runner above the tail water surface. For cavitation not to occur *pe* ≥ *pv*, where *pv* is the vapour pressure of the liquid at the working temperature.

An important parameter in the context of cavitation is the available suction head (inclusive of both static and dynamic heads) at exit from the turbine and is usually referred to as the net positive suction head 'NPSH' which is defined as

$$NPSH = \frac{p_e}{\rho g} + \frac{V_e^2}{2g} - \frac{p_v}{\rho g} \qquad (2)$$

with the help of Eq. (1) and in consideration of negligible frictional losses in the draft tube (*hf = 0*), Eq. (2) can be written as

$$NPSH = \frac{p_{atm}}{\rho g} - \frac{p_v}{\rho g} - z \qquad (3)$$

A useful design parameter σ known as Thoma's Cavitation Parameter (after the German Engineer Dietrich Thoma, who first introduced the concept) is defined as

$$\sigma = \frac{NPSH}{H} = \frac{(p_{atm}/\rho g) - (p_v/\rho g) - z}{H} \qquad (4)$$

For a given machine, operating at its design condition, another useful parameter σ_c, known as critical cavitation parameter is define as

$$\sigma_c = \frac{(p_{atm}/\rho g) - (p_e/\rho g) - z}{H} \qquad (5)$$

Therefore, for cavitation not to occur $\sigma \geq \sigma_c$ (since, $pe \geq pv$)
If either z or H is increased, σ is reduced. To determine whether cavitation is likely to occur in a particular installation, the value of σ may be calculated. When the value of σ is greater than the value of σ_c for a particular design of turbine cavitation is not expected to occur.

In practice, the value of σ_c is used to determine the maximum elevation of the turbine above tail water surface for cavitation to be avoided. The parameter value of σ_c increases with an increase in the specific speed of the turbine. Hence, turbines having higher specific speed must be installed closer to the tail water level.

Note: Eq. (4) can be written as, $\sigma_c = (Ha - Hv - Hs)/H$
Hs = Turbine setting = z

Cavitation in Turbines

Traveling bubble cavitation in Francis turbine

Inlet edge cavitation in Francis turbine

Leading edge cavitation damage in Francis turbine

Critical Plant sigma values σ_c

	Francis Runners					Propeller Runners			
Specific Speed	75	150	225	300	375	375	600	750	900
σ_c	0.025	0.1	0.23	0.4	0.64	0.43	0.8	1.5	3.5

$\sigma_c = 0.625\ (Ns/380.78)^2$ for Francis runners

$= 0.28 + (Ns/380.78)^3 / 7.5$ for Propeller runners

$= 1.10\ (0.28 + (Ns/380.78)^3 / 7.5)$ for Kaplan runners

Ns = Specific speed (r.p.m., kW, m)

Methods to avoid cavitation :

The following methods may be used to *avoid cavitation* :

1. Runner/turbine may be *kept under water*. But it is not advisable as the inspection and repair of the turbine is difficult. The other method to avoid cavitation zone without keeping the runner under water is *to use the runner of low specific speed*.
2. The *cavitation free runner* may be designed to fulfil the given conditions with extensive research.
3. It is possible to reduce the cavitation effect by *selecting materials which resist better the cavitation effect*. The cast steel is better than cast iron and stainless steel or alloy steel is still better than cast steel.
4. The cavitation effect can be reduced by *polishing* the surface. That is why the cast steel runners and blades are coated with stainless steel.
5. The cavitation may be avoided by selecting a runner of proper specific speed for given head.

18·16. Selection of Hydraulic Turbines

The following points should be considered while selecting right type of hydraulic turbines for hydroelectric power plant :

1. **Specific speed.** High specific speed is essential where head is low and output is large, because otherwise the rotational speed will be low which means cost of turbo-generator and power-house will be high. On the other hand, there is practically no need of chosing a high value of specific speed for high installations, because even with low specific speed high rotational speed can be attained with medium capacity plants. Refer to Table 18·2.

2. **Rotational speed.** Rotational speed depends on specific speed. Also the rotational speed of an electrical generator with which the turbine is to be directly coupled, depends on the frequency and number of pair of poles. The *value of specific speed adopted should be such that it will give the synchronous speed of the generator.*

3. **Efficiency.** The turbine selected should be such that it gives the *highest overall efficiency for various operating conditions.*

4. **Partload operation.** In general the efficiency at partloads and overloads is less than normal. For the sake of economy the turbine should always run with maximum possible efficiency to get more revenue.

When the turbine has to run at part or overload conditions *Deriaz turbine* is employed. Similarly, for low heads, Kaplan turbine will be useful for such purposes in place of propeller turbine.

5. **Cavitation.** The installation of water turbines of reaction type over the tail race is affected by *cavitation*. The critical value of cavitation factor must be obtained to see that the turbine works in *safe zone*. Such a value of cavitation factor also affects the design of turbine, especially of Kaplan, propeller and bulb types.

6. **Disposition of turbine shaft.** Experience has shown that the *vertical shaft* arrangement is better for large-sized reaction turbines, therefore, it is *almost universally adopted*. In case of *large size impulse turbines, horizontal shaft arrangement* is mostly employed.

7. **Head.** (*i*) *Very high heads* (*350 m and above*). For heads greater than 350 m, Pelton turbine is generally employed and there is practically no choice except in very special cases.

(*ii*) *High heads* (*150 m to 350 m*). In this range either Pelton or Francis turbine may be employed. *For higher specific speeds Francis turbine is more compact and economical than the Pelton turbine* which for the same working conditions would have to be much bigger and rather cumbersome.

(*iii*) *Medium heads* (*60 m to 150 m*). A Francis turbine is usually employed in this range. Whether a high or low specific speed unit would be used depends on the selection of the speed.

(*iv*) *Low heads* (*below 60 m*). Between 30 and 60 m heads both Francis and Kaplan turbines may be used. The latter is more expensive but yields a higher efficiency at partloads and overloads. It is therefore preferable for *variable loads. Kaplan turbine is generally employed for heads under 30 m. Propeller turbines are however, commonly used for heads up to 15 m. They are adopted only when there is practically no load variations.*

(*v*) *Very low heads.* For very low heads bulb turbines are employed these days. Although Kaplan turbines can also be used for heads from 2 m to 15 m but they are *not economical*.

Table 18·2. Criteria for Selection of Turbines

S.No.	Type of turbine	Head H(m)	Specific speed (N_s) (rpm., kW, m)	Speed ratio (K_u)	Maximum hydraulic efficiency (%)	Remarks
1.	*Pelton*: 1 jet	up to 2000	12 to 30	0·43 to 0·48	89	Employed for very high head.
	2 jets	up to 1500	17 to 50			
	4 jets	up to 500	24 to 70			
2.	*Francis*:					Full load efficiency high; partload efficiency lower than Pelton wheel.
	High-head	up to 300	80 to 150	0·6 to 0·9	93	
	Medium head	50 to 150	150 to 250			
	Low head	30 to 60	250 to 400			
3.	*Propeller and Kaplan*	4 to 60	300 to 1000	1·4 to 2	93	High part load efficiency; high discharge with low head
4.	*Bulb or tubular turbines*	3 to 10	1000 to 1200	6 to 8	91	Employed for very low head–tidal power plants.

Design procedure of the Pelton Wheel Turbine

Design outcome should be

i) Diameter of nozzle (d)
ii) Diameter of runner with buckets (D)
iii) Number of buckets(N_b)
iv) Spacing of buckets (s)
v) Width of the buckets (w)
vi) Design speed(N)
vii) Specific speed (N_s)

The following points need consideration while designing a pelton turbine.

i) Velocity of jet: the theoretical velocity of the jet is given by Vth = $\sqrt{2gH}$ and the actual velocity is given by V = $C_v\sqrt{2gH}$

Where H= net head on the turbine and

C_v = coefficient of velocity (the value of C_v = 0.97 to 0.99)

ii) Calculate the diameter of nozzle (d) , Q = AV = $\frac{\pi d^2}{4} \times V$

iii) Speed ratio or peripheral coefficient ($\varnothing$) : it is the ratio of the peripheral velocity of the jet bucket at their mean diameter to the theoretical velocity of the jet.

$$\varnothing = \frac{u}{\sqrt{2gH}}$$

Peripheral velocity (u) = $\frac{\pi ND}{60}$

Speed ratio has average value 0.45 to 0.47

N = rotational speed in rpm

D = diameter of the bucket or pitch diameter

iv) Jet ratio(m) : it is the ratio of the pitch circle diameter to the jet diameter

ie m = $\frac{D}{d}$

For maximum hydraulic efficiency, jet ratio lies between 11 and 15 then the diameter of the bucket can be calculated by the jet ratio

v) Number of buckets: the empirical formulae for calculating the desirable number of buckets $N_b = 0.5\frac{D}{d} + 15$

Bucket spacing $S = \frac{\pi D}{N_b}$

vi) Width of the bucket (w) it is calculated in terms of jet diameter (d)

Axial width =3d to 4d

Radial length = 2d to 3d

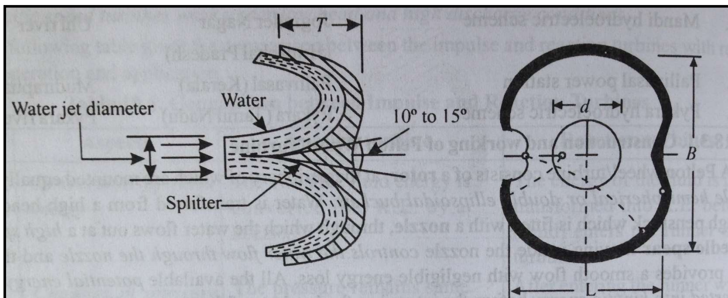
Depth = 0.8d to 1.2 d

vii) Initial rotational speed $N = \frac{60u}{\pi D}$

viii) Number of poles = $\frac{120f}{N}$, recalculate rotational speed N , using corrected number of poles

ix) Calculate power by p= $\eta\gamma QH$

x) Calculate specific speed $N_s = \frac{NP^{1/2}}{H^{5/4}}$



$\frac{L}{d} = 2 \text{ to } 3; \frac{B}{d} = 3 \text{ to } 4; \frac{D}{d} = 11 \text{ to } 16; \frac{T}{d} = 0.8 \text{ to } 1.2$; notch (width) = $1.1d + 5$ mm

The angle of deflection of jet to bucket is limited to about 165° to 170°

15.6.2 Design procedure of the Francis turbine

i) calculate specific speed for the given type of the turbine

ii) calculate rotational speed of the turbine (N)

$$N_s = \frac{NP^{1/2}}{H^{5/4}}$$

iii) calculate number of poles

p= $\frac{120f}{N}$ where p= no of poles

iv) calculate corrected synchronous speed N= $\frac{120f}{P}$

v) calculate corrected specific speed $N_s = \frac{NP^{1/2}}{H^{5/4}}$

vi) calculate diameter of the turbine

$$D= \frac{84.6\varnothing\sqrt{H}}{N}$$

Where $\varnothing$= peripheral coefficient is given by

$\varnothing = 0.019N_s^{2/3} + 0.0275$ for Francis turbine

$\varnothing = 0.0242N_s^{2/3}$ for propeller turbine

vii) setting of turbine $H_s = H_a - H_v - \sigma H$

Numbers of turbine units

It is most effective to have a minimum number of units at a given installation. Increase in turbine size increases the efficiency of the plant. However, multiple units may be necessary to make the most efficient use of water where flow variation is great. Factors as space limitation by geological characteristics of existing structure may dictate large or small units. The difficulty in transportation or large runners sometimes makes it necessary to limit their size. Isolation system may require more number of units. The percentage of load covered by the plant is another important factor if the contribution of the plant to system is more; the number of units should be more. For the power plant it is better to make the units are identical also somehow dictates the number of plants. If the power generation from the plant is fluctuating, the number of units will be increased and vice versa.

Governing of Hydraulic Turbines

➢N = 120f/p implies speed of the generator can be maintained at a constant level only when the speed of the turbine is constant.

➢Load is increased => speed tends to decrease and vice versa.

➢The function of the governor is to regulate the quantity of water flowing through the runner in proportion to the load. Thus the governing mechanism maintains the speed of the runner at a constant level at all loads.

➢For reaction turbines, the governor controls the guide vanes and wicket gates. For impulse turbines, the governor controls the spear and nozzle.

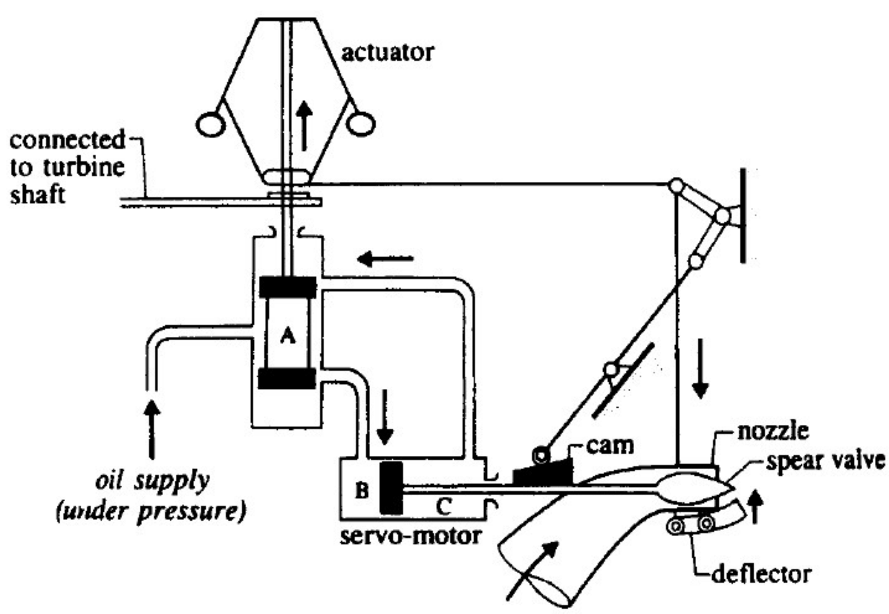
Governor

The governor is a mechanism controlling the rotational speed of the turbo-generator unit; constant speed must be maintained in order to obtain the a.c. supply with constant frequency. As the turbine and hence its interconnected generator tend to decrease or increase speed as the load varies, the maintenance of an almost constant speed requires regulation of the amount of water allowed to flow through the turbine by closing or opening the gates (or nozzles) of the turbine automatically, through the action of a governor.

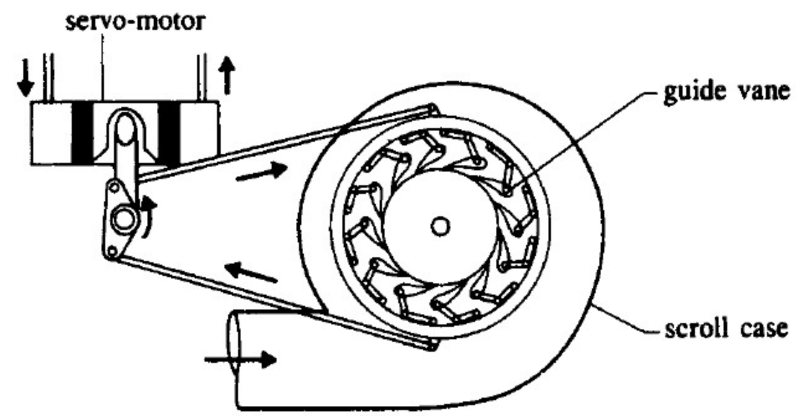
A simple governing mechanism for turbines is shown in figure below. Increase in the rotor speed raises piston A, permitting oil to enter chamber B, thus closing the gates slightly. The operation is reversed if the speed drops.

A rapid closing or opening of the nozzle or guide vanes (gates) is undesirable, as serious water hammer problems may result in the penstocks.

(a) Governing of impulse turbine



(b) Governing of reaction turbine



Sudden changes may be avoided in the case of a Pelton turbine if a deflector is activated in front of the jet, thus diverting part of the flow away from the turbine. Similarly, in the case of a reaction-type turbine a relief valve may allow a part of the discharge to flow directly to the tail race without entering the runner.

17.12 Governor and its mechanism

The governing of a turbine is defined as the operation by which the speed of turbine is kept constant under all working condition. It is done automatically by means of governer, which regulates the rate of flow through turbine, according to the changing load condition of the turbine. Governing of the turbine means regulating the speed of the turbine.

Governing of a turbine is necessary as a turbine is directly coupled to an electric generator, which is required to run at constant speed under fluctuation load condition. The frequency of power generation by a generator of a constant number of pair of poles under varying condition should be same.

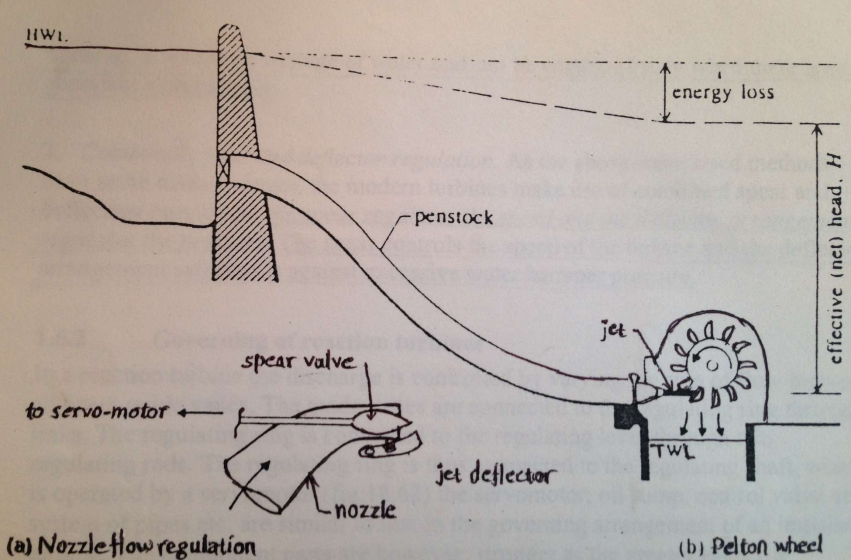
When the load on the generator decreases the speed of the generator increases beyond the normal speed. If the turbine or the generator is to run at constant speed, the rate of flow of water to the turbine should be decreased till the speed becomes normal. The process by which the speed of the turbine is kept constant under varying condition of load is called governing.

The governor of a Pelton wheel turbine decreases or increases the outlet area of the nozzle by moving the needle valve. In the case of Francis or Kaplan turbine, the governor decreases or increases the wicket gate.

The function of the governor is to detect any error in speed between actual and desired values and to effect a change in the turbine output. This is done so that the system load is in equilibrium with the generating unit output at the desired speed. The governor system of the turbine acts as opening, closing and gate-setting system for starting, stopping and synchronizing the turbine which allows for matching output to the system load to maintain the system frequency.

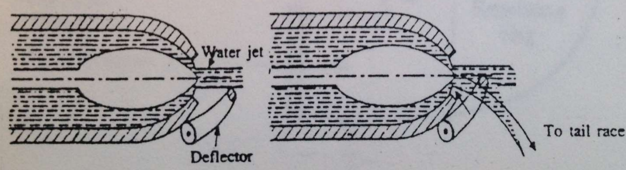
1.6.1 Governing of impulse turbines

In order to regulate the quantity of water rejected from the turbine nozzle and from striking the buckets one of the following methods of regulation may be adapted: 1.Spear regulation. 2.Deflector regulation. 3.Combined spear and deflector regulation



Impulse turbine (Pelton wheel).

1. *Spear regulation.* In this method the rate of flow is regulated by altering the cross-sectional area of stream by moving the spear to and fro inside nozzle. This method of speed regulation is suitable when the fluctuation of load is small and a relatively large penstock feeds a small turbine. The disadvantage of this method is that when a load falls all of sudden, the turbine nozzle has to close suddenly which may cause water ~~haunter~~ hammer in the penstock.



(*a*) Full load (*b*) Deflector diverts part of the jet

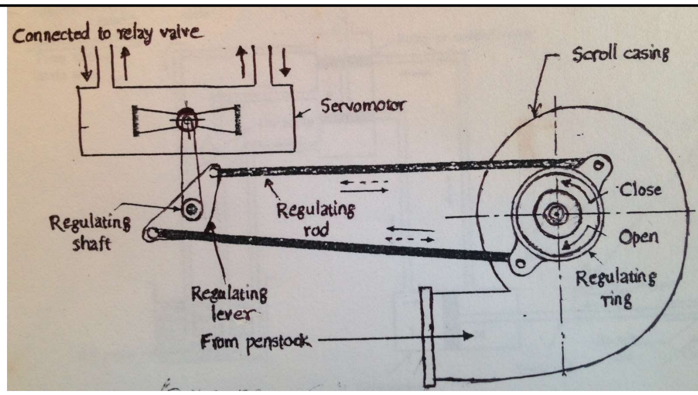
Deflector regulation.

2. *Deflector regulation.* The deflector is generally a plate connected to the oil pressure governor by means of levers. When necessity arises to deflect the jet the plate can be brought in between the nozzle and buckets, thereby diverting the water away from the runner and directing into the tailrace. The use of deflector regulation is restored to when the supply of water is constant but the load fluctuates. The position of spear can be adjusted by hand. As the nozzle has always a constant opening, it results in wastage of water and can be employed only when there is an abundant water supply.

3. *Combined spear and deflector regulation.* As the above-mentioned methods have some disadvantages, the modern turbines make use of combined spear and deflection regulation; *the spear regulates the speed and the deflector arrangement regulates the pressure*. The spear controls the speed of the turbine and the deflector arrangement safeguards against excessive water hammer pressure.

1.6.2 Governing of reaction turbines

In a reaction turbine the discharge is controlled by varying the area of flow between adjacent guide vanes, The guide vanes are connected to the regulating ring through links. The regulating ring is connected to the regulating lever through two regulating rods. The regulating ring is thus connected to the regulating shaft, which is operated by a servomotor (fig.18.62) the servomotor; oil pump, control valve and system of pipes etc. are similar to that in the governing arrangement of an impulse turbine. The component parts are however, stronger as the greater energy is required to move the gates as compared to spear in the nozzle of a Pelton turbine.



1.6.3 Governor Controlling Systems

Mainly Governor system can be classified as:

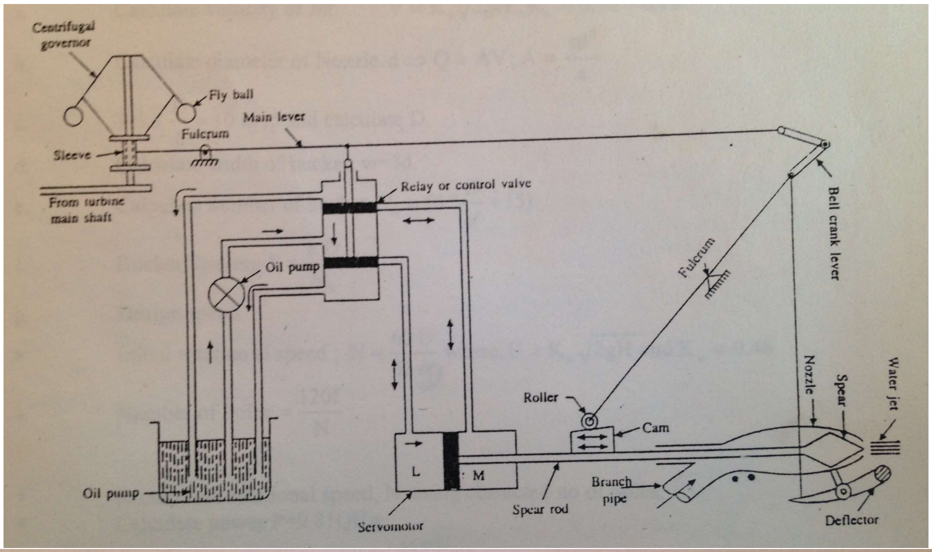
- Mechanical Hydraulic Governor
- Electro Hydraulic Governor

In any type of governor the basic three elements of operation are:

- The speed responsive system for detecting changes in speed
- The power component for operating wicket gates/spear valve etc.
- The stabilizing element that prevents runaway speed in the turbine and holds the servomotor in position which the turbine output and generator load are equalized.

Mechanical Hydraulic Governor : The working of the system is as follows:

- When the load on the turbine increases the speed of runner falls and consequently balls of the centrifugal governor move inward; the governor sleeve moves downward.
- The downward movement of the sleeve is transmitted to a relay or control valve (through suitable linkages), which admits oil under pressure to a servomotor. The oil exerts a force on the piston of servomotor, and that puss the spear to a position that increases the annular area of the nozzle flow passage; the quantity of water striking the buckets is then increased and the turbine regains its normal speed.



Electric Governing : The electric governor uses the electronic circuits in place of mechanical devices.

Governing of Turbines

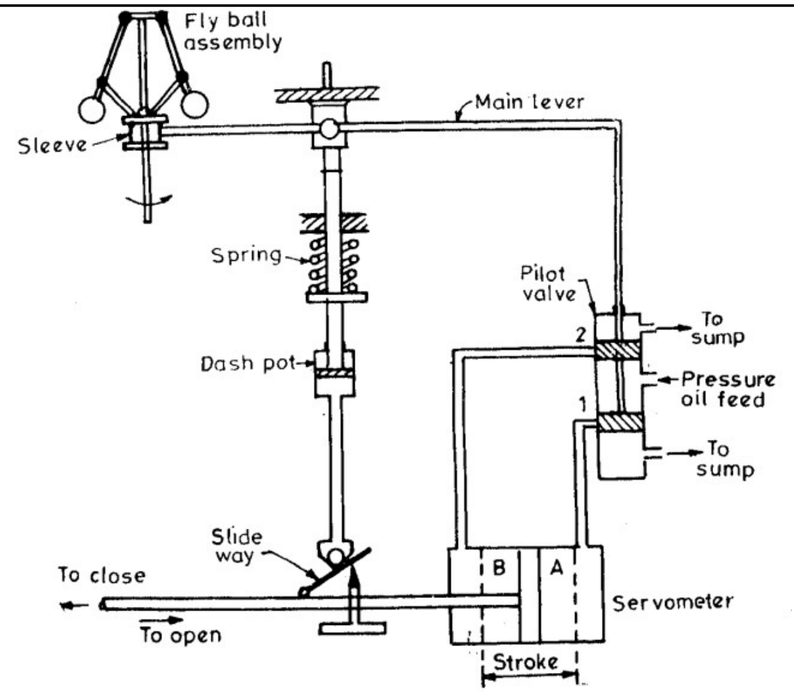
Governing of turbine means speed regulation. The turbine governor is an essential piece of equipment of the unit and works to regulate the speed of the turbine upon changes in load, by controlling the flow of water through the runner.

The various components of the governor are :

(*i*) a speed sensitive device (fly ball assembly), (*ii*) a mechanical amplifier to increase the effect of the above device, and (*iii*) the follow up system which produces the actual control on the water flow. This actual control comprises of operating a gate mechanism in the case of Francis turbine, the gate and blade adjusting mechanism in the case of Kaplan turbine, and the nozzle, needles and deflectors in an impulse turbine (Pelton wheel).

The most commonly used system of governing in turbines is through oil pressure. The oil pressure governor is described here.

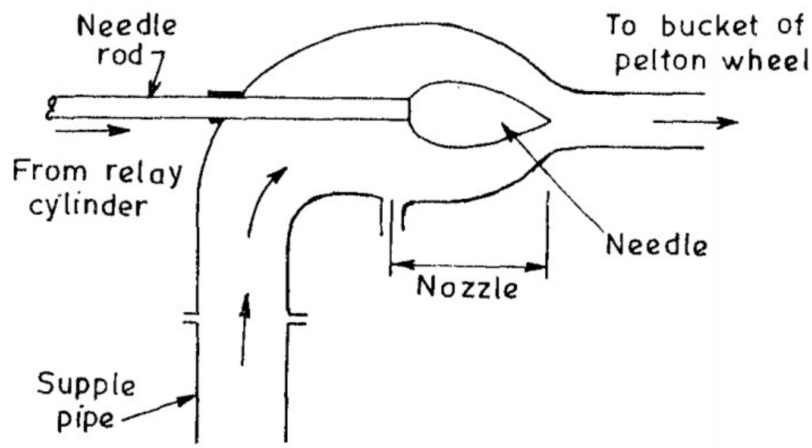
The governor system includes the governer itself, the energy supply equipment (pump supplying oil under constant pressure), the piping and the connections to the turbine's flow control mechanism.



Turbine flow control mechanism.

The governor in its simple form consists of the governor head (speed sensitive device, usually flyball assembly), the control or pilot valve and the servomotor. At normal speed the ports 1 and 2 of the control valve are both closed and the pressure in chambers *A* and *B* of the servomotor being the same, the servomotor piston is stationary in position, maintaining the constant rated discharge. Increase in the turbine speed rotates (through a belt) the governor head faster. The flyballs move outwards lifting the sleeve up and opening the port 1. Oil pressure is applied in chamber *A*, which moves the piston to the left and reduces the discharge until normal speed is restored. Any decrease in the turbine speed brings the sleeve downwards, opening the port 2, thereby applying pressure in chamber *B* and pushing the piston to the right to increase the discharge until the speed is stepped up to the normal. This apparently is simple process, however is rendered unsatisfactory because of the inertia effects, which cause the over travel of the controls and the consequent 'hunting' of governor.

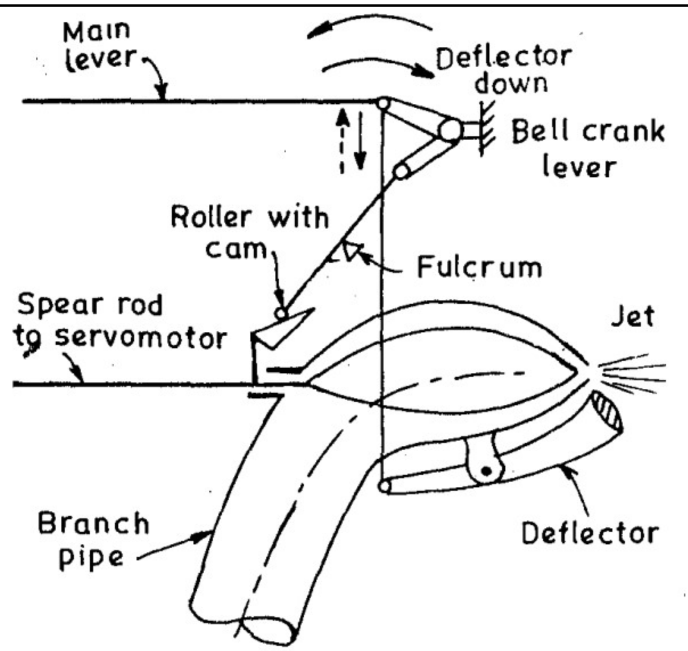
As has been stated, the governing of an impulse turbine is effected by regulating the quantity of water ejected through the nozzles and striking the turbine buckets. Two common ways of doing this are : Spear or needle regulation and the deflector regulation A third method involves the combination of these two arrangements. Fig. below shows the guide mechanism for an impulse turbine



Guide mechanism for an impulse turbine.

using needle regulation. The governor closes the nozzle opening partially by pushing the needle in the nozzle thus allowing less water to enter the turbine, which, therefore, will run slow. Needle regulation is suitable where a relatively large penstock supplies water to a small turbine and where the fluctuations in load are small, while the deflector is generally employed where supply of water is constant but the load on the turbine fluctuates.

The combination of spear and deflector control is quite popular in modern installations since double regulation is a feature of all modern turbines. The double operation involves simultaneous operation of two elements. This uses both the needle regulation effected through the direct connection with governor servomotor piston rod, and the deflector regulation which is effected through a linkage arrangement from the main lever and the servomotor piston rod. The deflector is thus, actuated directly by the governor and works quicker than the needle/ spear regulator which is permitted to act gradually, and thus avoid serious water hammer in the piping system. The double regulation arrangement of turbine governing is shown in Fig. below .



Double regulation system of governing impulse turbine

Double regulation system is also employed in reaction turbines by control the water flow through guide blades. This consists of guide blade control and a relief valve in Francis turbine and guide blade control and runner blade adjustment in case of Kaplan turbine. The guide blade control comprises of the used control mechanism, and blade adjusting mechanism are additional features. Relief valve which fitted in the penstock at the turbine inlet, opens and direct the water to the tailrace by passing the runner, when the power demand drops suddenly and the guide blades are closed suddenly.

Electro-mechanical Installation

Generator:

Electric Generator is a machine that converts mechanical energy to electric energy. Usually this energy is obtained from a rotating shaft that is also called the armature of the generator. The electric energy then produced can be used for power transmission to commercial, industrial or even domestic level. Generators supply current which usually has a frequency of 50 Hz, which is used here. An electric generator has two parts: i)Stator and ii) Rotor

The stator comprises of the stationary magnetic poles, whereas the rotating armature is included in the rotor.

Hydropower generator:

Hydropower generator is the main power equipment to making electricity energy in a hydropower station. It is a water-to-wire generator with the turbine as the prime motor. When the water flow goes through turbine, it changes the water power into mechanical energy.

Electro-mechanical Installation

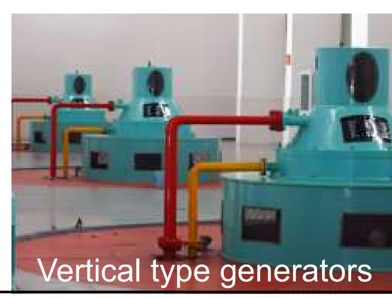
Types of hydropower generator:

According to its axis location, hydro generator is usually divided into two types: i) Horizontal and ii) Vertical

Large and medium-sized units usually adopt the vertical type layout, whose maximum speed can reach 750 rpm. While horizontal type usually is used for medium and small capacity units, whose maximum speed can reach 1500 rpm.

Usually, the horizontal hydro generator drives by Pelton turbine, and the vertical hydro generator drives by Francis or Kaplan turbines.

Horizontal type generators



Vertical type generators

Generators transform mechanical energy into electrical energy. Although most of early hydroelectric systems were of direct current variety to match early commercial electrical system now a days only 3-phase alternating current generators are used in normal practice. Depending upon the practice of network supply, there are 2 types of generators

a) Synchronous Generator: Such generators are equipped with a DC excitation systeem (rotating or static) associated with a voltage regulator to provide voltage and phase angle control before the generators is connected to the grid. Synchronous generators can run isolated from the grid and produce power since excitation is not grid dependent. These are more expensive than asynchronous generator.

b) Asynchronous Generator: These are simple squirrel cage induction motors with no possibility of voltage regulation and running at a speed directly related to system frequency. They draw their excitation current from the grid, absorbing reactive energy by their own magnetism. They can not generate when disconnected from the grid because they ar incapable of providing their own excitation current.

Pump:

A pump is a device that moves fluids (liquids or gases), or sometimes slurries, by mechanical action. Pumps operate by some mechanism (typically reciprocating or rotary), and consume energy to perform mechanical work by moving the fluid.

19·2. Classification of pumps

On the basis of transfer of mechanical energy the pumps can be broadly *classified* as follows

1. *Rotodynamic pumps*

(*i*) Radial flow pumps (*ii*) Axial flow pumps (*iii*) Mixed flow pumps.

2. *Positive displacement pumps.*

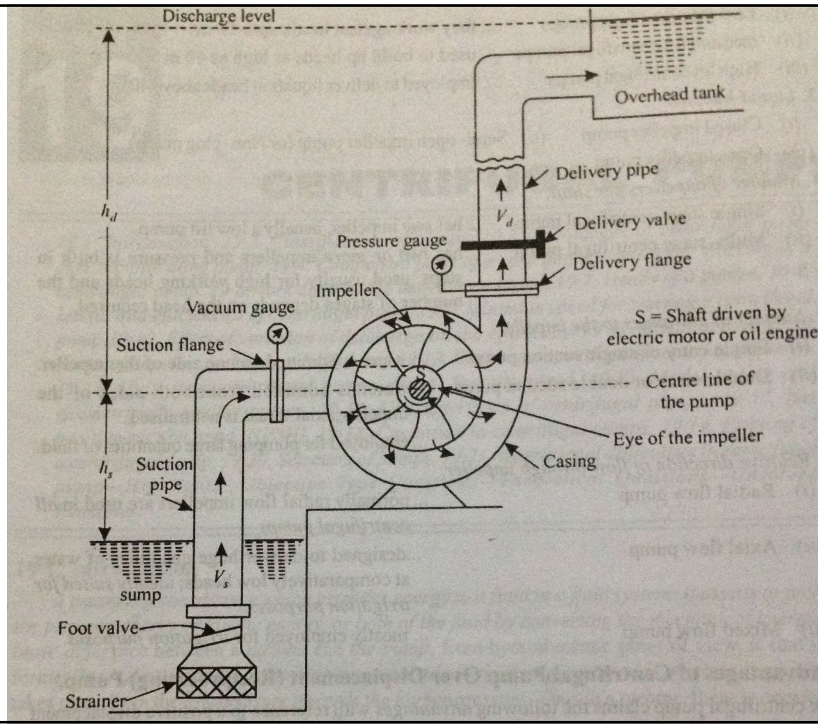
In rotodynamic pumps, *increase in energy level is due to a combination of centrifugal energy, pressure energy, and kinetic energy.*

19·4. Component Parts of a Centrifugal Pump

Refer. Fig. 19·1. A centrifugal pump consists of the following *main components*:

1. Impeller 2. Casing 3. Suction pipe 4. Delivery pipe.

1. **Impeller.** An *impeller* is a wheel (or rotor) with a series of backward curved vanes (or blades). It is mounted on a shaft which is usually coupled to an electric motor.



2.**Casing.** The casing is an *airtight chamber surrounding the pump impeller.* It contains suction and discharge arrangements, supporting for bearings, and facilitates to house the rotor assembly .It has provision to fix stuffing box and house packing materials which prevent external leakage. The essential *purposes* of the casing are:

(*i*) To guide water to and from the impeller, and

(*ii*) To partially convert the kinetic energy into pressure energy.

3. **Suction pipe.** The pipe which connects the centre/eye of the impeller to sump from which liquid is to be lifted is known as *suction pipe.* In order to check the formation of air pockets the pipe is laid *air tight.* To prevent the entry of soid particles, debris etc. into the pump the suction pipe is provided with a strainer at its lower end.The lower end of the pipe is also fitted with a *non - return foot valve* which does not permit the liquid to drain out of the suction pipe when pump is *not working*; this also helps in *priming.*

4. **Delivery pipe.** The pipe which is connected at its lower end to the outlet of the pump and it delivers the liquid to the required height is known as *delivery pipe* . A *regulating valve* is provided on the delivery pipe to regulate the supply of water.

19·5. Working of a Centrifugal pump

A centrifugal pump works on the *principal that when a certain mass of fluid is rotated by an external source, it is thrown away from the central axis of rotation and a centrifugal head is impressed which enables it to rise to a higher level.*

The *working /operation* of a centrifugal pump is explained *step-wise* below:

1. The delivery valve is closed and the pump is *primed* that is, *suction pipe, casing and portion of the delivery pipe upto the delivery valve are completely filled with the liquid (to be pumped) so that no air pocket is left.*

2. Keeping the delivery valve still closed the electric motor is started to rotate the impeller. The rotation of the impeller causes strong suction or vaccum just at the eye of the casing.

3. The speed of the impeller is gradually increased till the impeller rotates at its normal speed and develops normal energy required for pumping the liquid.

4. After the impeller attains the normal speed the delivery valve is opened when the liquid is continuously sucked (from sump well) up the suction pipe , it passes through the eye of casing and enters the impeller at its centre or it enters the impeller vanes at their inlet tips. This liquid is impelled out by the rotating vanes and it comes out at the outlet tips of the vanes into the casing. Due to impeller action the *pressure head as well as velocity heads of the liquid are increased* (some of this velocity heads is converted into pressure head in the casing and in the diffuser blades/vanes if they are also provided).

5. From casing , the liquid passes into pipe and is lifted to the required height (and discharged from the outlet or upper end of the delivery pipe).

6. So long as motion is given to the impeller and there is supply of liquid to be lifted the process of lifting the liquid to the required height remains continuous.

7. When pump is to be stopped the delivery valve should be first closed, otherwise there may be some backflow from the reservoir.

19·16. Characteristics of Centrifugal Pumps

Ordinarily a centrifugal pump is worked under its maximum efficiency conditions. However, when the pump is run at conditions different from the design conditions, it performs differently. Therefore, to predict the behaviour of the pump under varying conditions of speeds,heads, discharges or powers, tests are usually conducted. The results obtained from these tests are plotted in from of *characteristic curves;* these curves delineate useful information about the performance of a pump in its installation.

The following *four types of characteristic curves* are usually prepared for centrifugal pumps:

1. Main characteristic curves,
2. Operating characteristic curves,
3. Constant efficiency or Muschel curves, and
4. Costant head and constant discharge curves.

1. **Main characteristic curves:**

The main characteristic curves are obtained as follows:

- The pump is run at a *constant speed* and the *discharge is varied* over the desired range (by delivery valve)
- Measurements are taken for manometric head (H_{mano}) and shaft power (P) for each discharge(Q).
- Calculations are made for the pump overall efficiency, η_0
- The curves are plotted between Q and H_{mano}; Q and P; and Q and η_0 for that speed.
- The same procedure is repeated by running the pump at another speed.
- A family of curves is obtained as shown in Fig. 19·25.

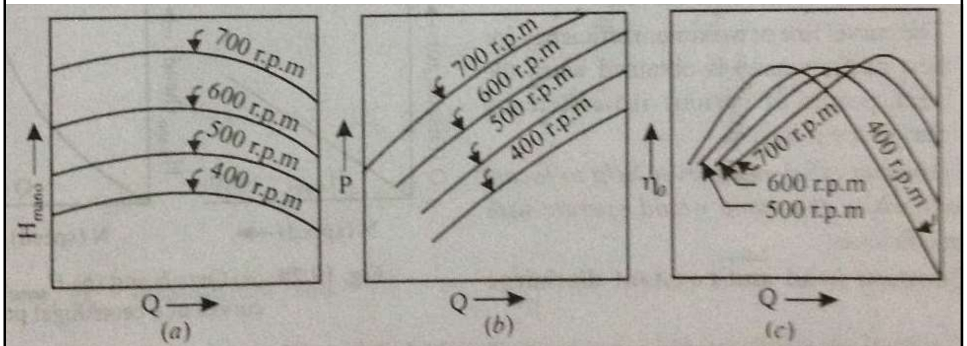


Fig. 19.25. Main characteristic curves.

2. Operating characteristic curves:

When a centrifugal pump operates at the *design speed* (same as speed of driving motor) the *maximum* efficiency occurs. Evidently for *optimum performance,* the pump needs to be operated at the design speed. To obtain *operating characteristic curves* the pump is run at the design speed and the discharge is varied, as in the case of main characteristic curves. The operating characteristic curves are shown in Fig. 19·26 .The design discharge and head *are obtained from the corresponding* curves where the efficiency is maximum,

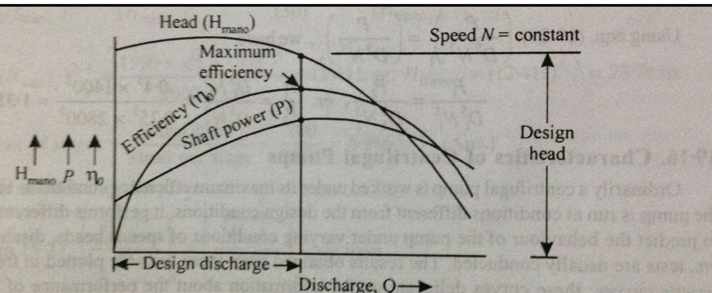


Fig. 19.26. Operating characteristic curves of a centrifugal pump.

3. Constant efficiency or Muschel curves:

The constant efficiency curves (also called iso-efficiency curves), depict the performance of a pump over its entire range of operations. These curves are obtained from main characteristic curves as follows:

- For a given efficiency, the values of discharges are obtained from Fig. (19·25) (*c*). These points are projected on the head (H_{mano}) v/s discharge (Q) for that speed in Fig. 19·25 (*a*).
- Similarly, for another value of efficiency and speed, the points are obtained and projected.
- The points corresponding to one efficiency are joined.

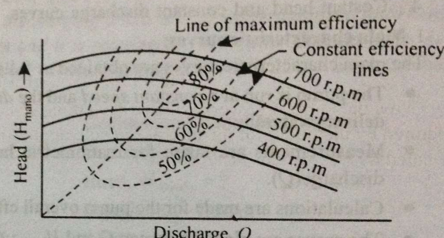


Fig. 19.27. Constant efficiency or Muschel curves.

- The curves so obtained are the constant efficiency or Muschel curves.
- The curve/ line of maximum efficiency (or best performance) is obtained when the peak points of various iso-efficiency curves are joined.

The constant efficiency curves help to locate the regions where the pump would operate with maximum efficiency.

4. **Constant head and constant discharge curves:**

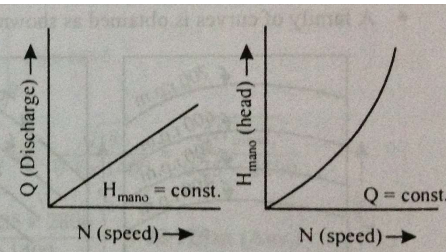


Fig. 19.28. (*a*) Qv/s *N* and (*b*) H_{mano} v/s *N* curves of a centrifugal pump.

The performance of a variable speed pump for which the *speed constantly varies* can be determined by these curves. When the pump has a variable speed, the plots between *Q* and *N*, and H_{mano} and *N* may be obtained. In the first case H_{mano} is kept constant and in the second case, *Q* is kept constant. The curves are shown in Fig. 19·28.

Reciprocating Pump

The reciprocating pump is a *positive displacement pump* as it sucks and raises the liquid by actually displacing it with a piston/plunger that executes a reciprocating motion in a closely fitting cylinder. The amount of liquid pumped is equal to the volume displaced by the piston.

The pumps designed with disk pistons create pressures upto 25 bar and the plunger pumps built up still higher pressures. Discharge from these pumps is almost wholly dependent on the pump speed.

The total efficiency of a reciprocating pump is about 10 to 20% higher than a comparable centrifugal pump.

Reciprocating pumps for industrial uses have almost become *obsolete owing to their high capital cost as well as maintenance cost as compared to that of centrifugal pumps*. However, small hand-operated pumps such as cycle pumps, football pumps, kerosene pumps, village well pumps and pumps used as important parts of hydraulic jack etc. still find wide applications. *The reciprocating pump is best suited for relatively small capacities and high heads*. This type of pump is very common in *oil drilling operations*.

20·3. Main Components and Working of a Reciprocating Pump

Refer Fig. 20·1. The **main parts** *of a reciprocating pump are:*

1. Cylinder
2. Piston
3. Suction valve
4. Delivery valve
5. Suction pipe
6. Delivery pipe
7. Crank and connecting rod mechanism operated by a power source e.g. steam engine, internal combustion engine or an electric motor.

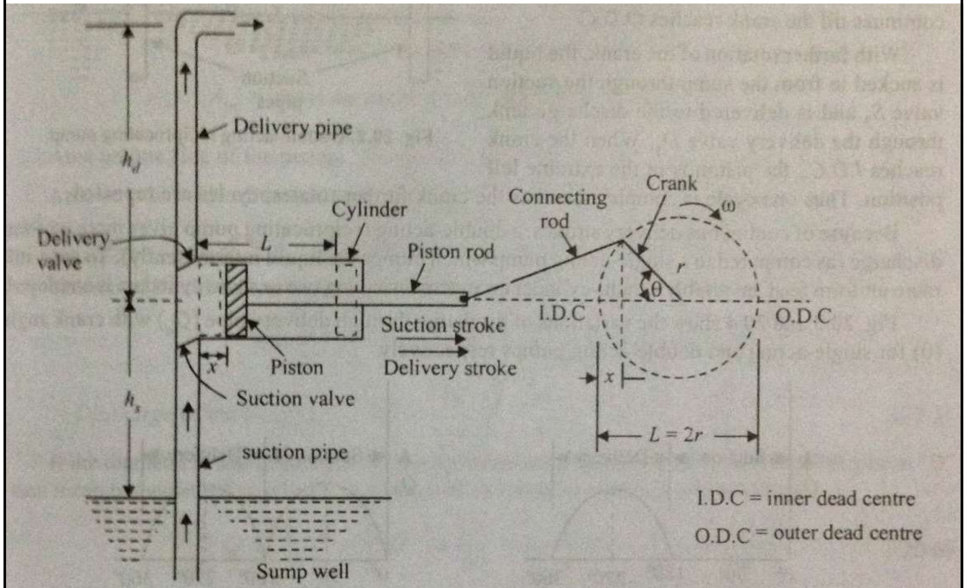


Fig. 20.1. Schematic view of single-acting reciprocating pump.

Working of a single-acting reciprocating pump:

As shown in Fig. 20·1 a single acting reciprocating pump has one suction pipe and one delivery pipe. It is usually placed above the liquid level in the sump. When the crank rotates the piston moves backward and forward inside the cylinder. The pump operates as follows:

— Let us suppose that initially the crank is at the inner dead centre (I.D.C.) and crank rotates in the clockwise direction. As the crank rotates, the piston moves towards right and a vacuum is created on the left side of the piston. This vacuum causes suction valve to open and consequently the liquid is forced from the sump into the left side of the pistor When the crank is at the outer dead centre (O.D.C) the suction stroke is completed and the left side of the cylinder is full of liquid.

When the crank further turns from O.D.C to I.D.C., the piston moves inward to the left and high pressure is built up in the cylinder. The delivery valve opens and the liquid is forced into the delivery pipe. The liquid is carried to the discharge tank through the delivery pipe. At the end of delivery stroke the crank comes to the I.D.C and the piston is at the extreme left position.

Working of a double-acting reciprocating pump:

Refer Fig. 20·2. In a double-acting reciprocating pump, suction and delivery *strokes occur simultaneously*. When the crank rotates from *I.D.C.* in the clockwise direction, a vaccum is created on the left side of piston and the liquid is sucked in from the sump through value S_1. At the same time, the liquid on the right side of the piston is pressed and a high pressure causes the delivery valve D_2 to open and the liquid is passed on to the discharge tank. This operation continues till the crank reaches *O.D.C.*

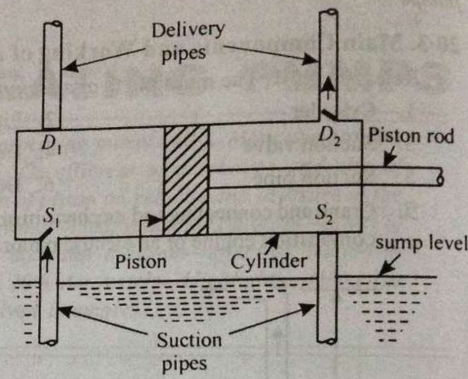
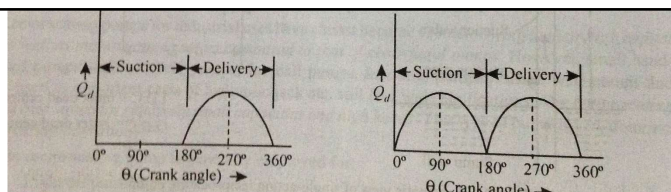


Fig. 20.2. Double-acting reciprocating pump.

With further rotation of the crank, the liquid is sucked in from the sump through the suction valve S_2 and is delivered to the discharge tank through the delivery valve D_1. When the crank reaches *I.D.C.*, the piston is in the extreme left position. Thus one cycle is completed and as the crank further rotates, cycles are repeated.

Because of continuous delivery strokes, a double-acting reciprocating pump gives more uniform discharge (as compared to a single-acting pump which pumps the liquid intermittently). To get a still more uniform feed, invariably a multi-cylinder arrangement having two or more cylinders is employed.

Fig. 20·3 and 20·4 show the variations of discharge through delivery pipe (Q_d) with crank angle (θ) for single-acting and double-acting pumps respectively.



19·3. Advantages of Centrifugal Pump Over Displacement (Reciprocating) Pump.

The centrifugal pump claims the following *advantages* with reference to a positive displacement (reciprocating) pump.

1. The cost of a centrifugal pump is less as it has fewer parts.
2. Installation and maintenance are easier and cheaper.
3. Its discharging capacity is much greater than that of a reciprocating pump.
4. It is compact and hads smaller size and weight for the same capacity and energy transfer.
5. Its performance characteristics are superior.
6. It can be employed for lifting highly viscous liquid such as paper pulp, muddy and sewage water ,oil, sugar molasses etc.
7. It can be operated at very high speeds without any danger of separation and cavitation.
8. It can be directly coupled to an electric motor or an oil engine.
9. The torque on the power source is uniform,the output fron the pump is also uniform.

However , because of higher efficiency the reciprocating pumps are still employed for high heads and small discharges. A reciprocating pump can build up very high pressures (as high as 700 bar or even more) and as such these pumps are used for lifting oils from very deep oil wells.

Example 18·11. *The following data relate to a Pelton wheel :*

Head	*... 72 m*
Speed of the wheel	*... 240 r.p.m.*
Shaft power of the wheel	*...115 kW*
Speed ratio	*... 0·45*
Co-efficient of velocity	*... 0·98*
Overall efficiency	*... 85%*

Design the Pelton wheel.

Solution. Effective head, $H = 72$ m

Speed of the wheel, $N = 240$ r.p.m.

Shaft power, $P = 115$ kW

Speed ratio, $K_u = 0{\cdot}45$

Co-efficient of velocity, $C_v = 0{\cdot}98$

Overall efficiency, $\eta_0 = 85\%$

Design of the Pelton wheel means to find diameter of the wheel D, diameter of jet (d), width and depth of buckets and number of buckets on the wheel.

(*i*) **Diameter of wheel, D:**

Velocity of jet, $V_1 = C_v\sqrt{2gH} = 0{\cdot}98\sqrt{2 \times 9{\cdot}81 \times 72} = 36{\cdot}8$ m/s

$\therefore$ Bucket velocity, $u\,(= u_1 = u_2) = K_u \times V_1 = 0{\cdot}45 \times 36{\cdot}8 = 16{\cdot}56$ m/s

But, $u = \dfrac{\pi DN}{60}$, or, $D = \dfrac{60u}{\pi N} = \dfrac{60 \times 16{\cdot}56}{\pi \times 240}$ = **1·32 m (Ans.)**

(*ii*) **Diameter of jet, d:**

Overall efficiency, $\eta_0 = \dfrac{\text{Shaft power}}{\text{Water power}} = \dfrac{P}{wQH}$

$$0.85 = \frac{115}{9.81 \times Q \times 72}$$

or, $$Q = \frac{115}{0.85 \times 9.81 \times 72} = 0.1915 \text{ m}^3/\text{s}$$

But, Q = Area of jet × velocity of jet

$$0.1915 = \frac{\pi}{4} \times d^2 \times V_1 = \frac{\pi}{4}\, d^2 \times 36.8$$

$\therefore$ $$d = \left(\frac{0.1915 \times 4}{\pi \times 36.8}\right)^{1/2} = 0.0814 \text{ m or } \mathbf{81.4\ mm\ (Ans.)}$$

(*iii*) **Size of buckets:**

Width of the bucket, B = 3 to 4 times jet diameter (d)

$\simeq 3{\cdot}5\, d = 3{\cdot}5 \times 81{\cdot}4 =$ **285 mm (Ans.)**

Radial length of bucket, L = 2 to 3 times jet diameter (d)

$\simeq 2{\cdot}5\, d = 2{\cdot}5 \times 81{\cdot}4 =$ **203·5 mm (Ans.)**

Depth of bucket, T = 0·8 to 1·2 times jet diameter (d)

$\simeq 1{\cdot}0\, d =$ **81·4 mm (Ans.)**

(*iv*) **Number of buckets on the wheel, Z:**

$$Z = 15 + \frac{D}{2d} = 15 + \frac{1{\cdot}32 \times 1000}{2 \times 81{\cdot}4} = \mathbf{23\ (Ans.)}$$

Example 18·43. *A conical draft tube having inlet and outlet diameters 1·2 m and 1·8 m discharges water at outlet with a velocity of 3 m/s. The total length of the draft tube is 7·2 m and 1·44 m of the length of draft tube is immersed in water. If the atmospheric pressure head is 10·3 m of water and loss of head due to friction in the draft tube is equal to 0·2 × velocity head at outlet of the tube, determine :*

(i) *Pressure head at inlet, and*

(ii) *Efficiency of the draft tube.*

Solution. Inlet diameter of the draft tube, $d_i = 1\cdot2$ m

Outlet diameter, $d_0 = 1\cdot8$ m

Velocity at outlet, $V_3 = 3$ m/s

Total length of draft tube, $H_s + y = 7\cdot2$ m

Length of draft tube in water, $y = 1\cdot44$ m

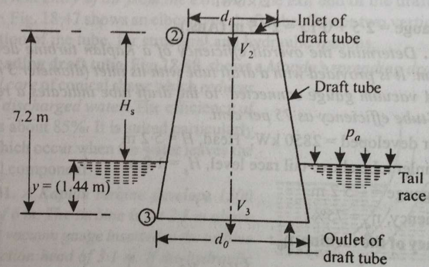


Fig. 18.49

$\therefore \quad H_s = 7\cdot2 - 1\cdot44 = 5\cdot76$ m

Atmospheric pressure head, $\dfrac{p_a}{w} = 10\cdot3$ m

Loss of head due to friction,

$$h_f = 0\cdot2 \times \text{velocity head at outlet}$$

$$= 0\cdot2\,\frac{V_3^2}{2g}$$

(i) **Pressure head at inlet,** $\dfrac{\mathbf{p_2}}{\mathbf{w}}$ **:**

Discharge through the draft tube,

$$Q = A_3V_3 = \frac{\pi}{4} \times d_0^2 \times V_3 = \frac{\pi}{4} \times 1\cdot8^2 \times 3 = 7\cdot634 \text{ m/s}$$

Velocity of inlet, $V_2 = \dfrac{Q}{A_2} = \dfrac{7\cdot634}{\frac{\pi}{4}d_i^2} = \dfrac{7\cdot634}{\frac{\pi}{4}\times 1\cdot2^2} = 6\cdot75$ m/s

Using eqn. (18·32),

$$\frac{p_2}{w} = \frac{p_a}{w} - H_s - \left(\frac{V_2^2 - V_3^2}{2g} - h_f\right) = \frac{p_a}{w} - H_s - \left(\frac{V_2^2 - V_3^2}{2g} - 0\cdot2\,\frac{V_3^2}{2g}\right)$$

$$= 10\cdot3 - 5\cdot76\left(\frac{6\cdot75^2 - 3^2}{2\times 9\cdot81} - 0\cdot2 \times \frac{3^2}{2\times 9\cdot81}\right)$$

or, $$\frac{p_2}{w} = 4\cdot54 - (1\cdot863 - 0\cdot092) = \mathbf{2\cdot769\ m\ (abs)\ (Ans.)}$$

(ii) **Efficiency of the draft tube, η_d :**

$$\eta_d = \frac{\left(\dfrac{V_2^2 - V_3^2}{2g} - h_f\right)}{\dfrac{V_2^2}{2g}} = \frac{\dfrac{V_2^2 - V_3^2}{2g} - 0\cdot2\,\dfrac{V_3^2}{2g}}{\dfrac{V_2^2}{2g}} = \frac{\dfrac{V_2^2}{2g} - \left(\dfrac{V_3^2}{2g} + 0\cdot2\,\dfrac{V_3^2}{2g}\right)}{\dfrac{V_2^2}{2g}}$$

$$= 1 - 1\cdot2\left(\frac{V_3}{V_2}\right)^2 = 1 - 1\cdot2\left(\frac{3}{6\cdot75}\right)^2 = 0\cdot763 \quad \text{or} \quad \mathbf{76\cdot3\ \%\ (Ans.)}$$

Example 18·45. *A turbine is to operate under a head of 25 m at 200 r.p.m. The discharge is 9 m³/s. If the overall efficiency is 90 per cent, determine :*

(*i*) *Power generated*; (*ii*) *Specific speed of the turbine;*

(*iii*) *Type of turbine.* **[AMIE]**

Solution. Head, $H = 25$ m; Speed, $N = 200$ r.p.m.;

Discharge, $Q = 9$ m³/s; Overall efficiency, $\eta_0 = 90\%$.

(*i*) **Power generated, *P*:**

$$P = \eta_0 \times wQH = 0{\cdot}9 \times 9{\cdot}81 \times 9 \times 25 = \mathbf{1986{\cdot}5\ kW\ (Ans.)}$$

(*ii*) **Specific speed of the turbine, N_s:**

$$N_s = \frac{N\sqrt{P}}{H^{5/4}} = \frac{200 \times \sqrt{1986{\cdot}5}}{(25)^{5/4}} = \mathbf{159{\cdot}4\ r.p.m.\ (Ans.)}$$

(*iii*) **Type of Turbine :**

As the specific speed lies between 80 and 400 (Refer to table 18·2), the turbine is a **Francis turbine. (Ans.)**

Example 18·48. *In a hydroelectric station, water is available at the rate of 175 m³/s under a head of 18 m. The turbines run at a speed of 150 r.p.m. with overall efficiency of 82%. Find the number of turbines required if they have the maximum specific speed of 460.* **(GATE)**

Solution. *Given* : $Q = 175$ m³/s; $H = 18$ m; $N = 150$ r.p.m.; $\eta_0 = 82\%$; $N_s = 460$.

Number of turbines required :

Specific speed of the turbine,

$$N_s = \frac{N\sqrt{P}}{H^{5/4}} \qquad \text{...[Eqn. (18.33)]}$$

$$460 = \frac{150\sqrt{P}}{(18)^{5/4}} \quad \text{(where, P is in kW and H is in metres.)}$$

or, Power available at turbine shaft, $P = \left[\frac{460 \times (18)^{5/4}}{150}\right]^2 = 12927{\cdot}5$ kW

Power available from turbines $= wQH \times \eta_0 = 9.81 \times 175 \times 18 \times 0.82 = 25339.23$ kW

No. of turbines required $= \dfrac{25339{\cdot}23}{12927{\cdot}5} = 1{\cdot}96$ say **2 (Ans.)**

Example 18·58. *A Francis turbine works under a head of 25 m and produces 11800 kW while running at 120 r.p.m. The turbine has been installed at a station where atmospheric pressure is 10 m of water and vapour pressure is 0·2 m of water. Calculate the maximum height of the straight draft tube for the turbine.*

Solution. Head under which the turbine works, $H = 25$ m

Power output, $P = 11800$ kW

Speed of the turbine, $N = 120$ r.p.m.

Atmospheric pressure, $p_a = 10$ m of water

Vapour pressure, $H_v = 0{\cdot}2$ m.

Maximum height of the draft tube, H_s:

Specific speed, $N_s = \dfrac{N\sqrt{P}}{H^{5/4}} = \dfrac{120\sqrt{11800}}{(25)^{5/4}} = 233{\cdot}2$ r.p.m.

Critical value of Thoma's cavitation factor for a Francis turbine

$$\sigma_c = 0{\cdot}625\left(\frac{N_s}{380{\cdot}78}\right)^2 \quad \text{...[Eqn. (18·49)]}$$

$$= 0{\cdot}625\left(\frac{233{\cdot}2}{380{\cdot}78}\right)^2 = 0{\cdot}2344$$

Also, $$\sigma_c = \frac{H_c - H_v - H_s}{H} \quad \text{...By definition}$$

or, $$0{\cdot}2344 = \frac{10 - 0{\cdot}2 - H_s}{25}, \text{ or, } 0{\cdot}2344 \times 25 = 10 - 0{\cdot}2 - H_s$$

$\therefore$ $$H_s = 10 - 0{\cdot}2 - 0{\cdot}2344 \times 25 = 3{\cdot}94 \text{ m}$$

Hence, *maximum permissible height of the draft tube* = **3·94 m (Ans.)**

Solved Examples 17-1

A conical draft tube having inlet and output diameters 1.2 and 1.8 m discharges water at output with a velocity of 3 m /sec. The total length of the draft tube is 7.2 m and 1.44 m of length of draft tube is immersed in water. If atmospheric pressure head is 10.3 m of water and loss of head due to friction in the draft tube is equal to 0.2 times velocity head at output of the tube. Determine a) pressure head at input and b) Efficiency of draft tube.

Solution

Inlet diameter, $d_1 = 1.2m$. $A_1 = \dfrac{\pi d^2}{4} = \dfrac{\pi \times 1.2^2}{4} = 1.131\text{m}^2$

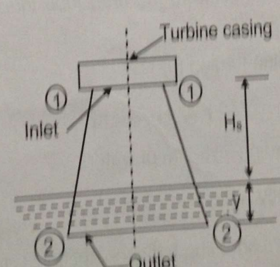
Output diameter, d2 =180 m, so $A_2 = \dfrac{\pi \times 1.8^2}{4} = 2.545\text{m}^2$

$V_2 = 3m/\sec$

From continuity equation, $Q_1 = Q_2$

$A_1V_1 = A_2V_2$

$V_1 = \dfrac{Q}{A_1} = \dfrac{7.634}{1.131} = 6.75m/\sec$



Head lost in draft tube $= 0.2 \times \frac{V_2^2}{2g} = \frac{0.2 \times 3^2}{2 \times 9.81} = 0.092\,m$

Applying Bernoulli's equation between section 1-1and 2-2

$$\frac{P_1}{\gamma} + (H_s + y) + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + h_f$$

Now $\frac{P_2}{\gamma} = \frac{P_a}{\gamma} + y.$

So, $\frac{p_1}{\gamma} = \frac{P_a}{\gamma} - \left(\frac{V_1^2 - V_2^2}{2g}\right) + h_f - H_s$

$$= 10.30 - \left(\frac{6.75^2 - 3^2}{2 \times 9.81}\right) + 0.092 - 5.76 = 2.77m.$$

(ii) Efficiency of draft tube (h_d)

$$h_d = \frac{\frac{V_1^2 - V_2^2}{2g}}{\frac{V_1^2}{2g}} x100\%$$

$$= \frac{V_1^2 - V_2^2}{V_1^2} \times 100 = \frac{6.75^2 - 3^2}{6.75^2} \times 100 = 80.25\%$$

Solved Examples 17-2 :

Determine the overall efficiency of Kaplan turbine developing 2850 KW under a head of 5.20 m. It is provided with a draft tube with its inlet (diameter 3 m) set 1.8 m above the tailrace level. A Vaccum gauge connected to the draft tube indicates a reading of 5.20 m of water. Assume draft tube efficiency as 75%.

Efficiency of Kaplan turbine (η) =?

Output power (P) =2850 KW, H =5.2 m

Vacuum table reading = -5.2 m of water

Draft tube efficiency $(\eta_d) = 75\%$

Applying Bernoulli's equation between section 1-1 & 2-2

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + H_s + y = \left(\frac{P_a}{\gamma} + y\right) + \frac{V_2^2}{2g} + h_f$$

So, Assuming $P_a = 0$

$$\frac{P_1}{\gamma} = -\left(\frac{V_1^2 - V_2^{\ 2}}{2g}\right) - H_s \Rightarrow \frac{P_1}{\gamma} = -\frac{\left(\frac{V_1^2 - V_2^{\ 2}}{2g}\right)}{\frac{V_1^{\ 2}}{2g}} x \frac{V_1^{\ 2}}{2g} - H_s$$

Now, $\dfrac{\frac{V_1^2 - V_2^2}{2g}}{\frac{V_1^{\ 2}}{2g}}$ = efficiency of draft tube $= \eta_d$.

$$\frac{P_1}{\gamma} = -\eta_d x \frac{V_1^{\ 2}}{2g} - H_s$$

$$-5.2 = -0.75 x \frac{V_1^{\ 2}}{2 \times 9.81} - 1.80.$$

$$\Rightarrow V_1 = 9.43 m / \text{sec}.$$

$$Q = A_1 V_1 = \frac{\pi \times 3^2}{4} \times 9.43 = 66.66 m^3 / \text{see}.$$

$\therefore$ Efficiency of turbine $= \dfrac{\text{Power Output}}{\gamma Q H} = \dfrac{2850}{9.81 \text{ x } 66.66 \text{ x } 5.20} = 83.80\,\%$

Solved Examples 17-3

A proposed hydropower development having net head of 150 m design discharge of 25 m³/sec in going to use Francis turbine. Taking turbine efficiency 0.81, Calculate specific speed and turbine diameter and setting of the turbine.

Solution,

Net head (H) = 150 m .

$Q_d = 25 m^3 / \text{sec},\ \eta = 0.81$

Specific speed (N_s) = ?

Turbine diameter (D) = ?

Power (P) $= \eta \gamma Q H$

$$= 0.81 \times 9.81 \times 25 \times 150 = 29.797.87 \text{ KW}$$

$$= \frac{29.797.87}{0.746} = 39943.52 \text{ HP}$$

$$N_s = \frac{2400}{\sqrt{H}} = \frac{2400}{\sqrt{150}} = 195.96 rpm.$$

$$\text{Rotational speed (N)} = N_s \frac{H^{5/4}}{\sqrt{P}}$$

$$= \frac{195.96 \times 150^{5/4}}{\sqrt{39943.52}} = 514 \text{ r.p.m.}$$

Number of poles $P = \frac{120f}{N} = \frac{120 \times 50}{514} = 11.67 = 12.$

Corrected synchronous Speed, $N = \frac{120f}{p}$

$$= \frac{120 \times 50}{12} = 500\ rpm.$$

Corrected $N_s = \frac{N\sqrt{P}}{H^{5/4}} = \frac{500 \times \sqrt{39943.52}}{(150)^{5/4}} = 190.36\,\text{rpm}.$

Diameter of turbine, $D = \frac{84.60\phi\sqrt{H}}{N}$

Where, $\phi = 0.0127\ N_s^{2/3} + 0.0275$

$$= 0.0127 \times (190.36)^{2/3} + 0.0275 = 0.0679$$

$$D = \frac{84.60 \times 0.679 \times \sqrt{150}}{500} = 1.407 = 1.50\,m.$$

Setting of turbine H_s

$$\sigma_c = 0.625\left(\frac{N_s}{38.78}\right)^2$$

Where, $N_s = \frac{N\sqrt{P}}{H^{5/4}}$ where, P in KW

$$= \frac{500 \times \sqrt{29727.87}}{(150)^{5/4}} = 164.42\ \text{rpm.} = 165\ \text{rpm}$$

$$\sigma_c = 0.625\left(\frac{165}{380.78}\right)^2 = 0.117$$

Again,

$$\sigma_c = \frac{H_a - H_v - H_s}{H}$$

$$-H_s = \sigma_c H - H_a + H_v$$

$$H_s = (H_a - H_v) - \sigma_c H$$

$$= 10 - 0.117 \times 150 = 7.60m$$

Solved Examples 17-5

Select the size of Francis turbine for a site where the net head is 110 m and discharge is 140 m³/sec having efficiency if 94%. Determine also the elevation of turbine with reference to the water surface in tailrace. Assume the turbine will have to drive a 50 cycle generator.

Size of frames turbine (D) =?

Head, H=110 m

Design discharge, Q=140 m³/Sec

η =94%

Elevation of turbine with respect to tail water surface. Assume the turbine will have to drive 50 cycle Generator, f = 50Hz,

Setting of Turbine, H_s = ?

$P = \eta\gamma QH$

$= 0.94 \times 9.81 \times 140 \times 110 = 142009.56$ KW

$= \frac{14009.56}{0.746} = 190261.34 HP$

Specific speed $N_s = \frac{2400}{\sqrt{H}}$ for Francis turbine

$= \frac{2400}{\sqrt{110}} = 228.831 = 229$ rpm

Synchronous speed $(N) = \frac{N_s H^{5/4}}{\sqrt{P}}$ Where P = Power in HP

$= \frac{299 \times (110)^{5/4}}{\sqrt{190361.34}} = 187$ rpm

No of pole $(N_p) = \frac{120f}{N} = \frac{120 \times 50}{187} = 32.08 = 32$ Nos

Corredted $(N) = \frac{120 \times 50}{32} = 187.50$ rpm

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

$= \frac{187.80\sqrt{190361.34}}{(110)^{5/4}} = 229.64$ rpm $= 230$ rpm

Diameter of turbine $D = \frac{84.6\,\phi\sqrt{H}}{N}$ H

Where, $\phi = 0.0197 N_s^{2/3} + 0.0275$

$= 0.0197 \times (230)^{2/3} + 0.0275$

$= 0.767$

$D = \frac{84.6 \times 0.767 \times \sqrt{110}}{187.500} = 3.63 = 3.75m$

Setting of turbine

$$\sigma_c = 0.625\left(\frac{N_s}{380.78}\right)^2 \quad \text{For Francis turbine}$$

Where N_s for P in KW

$$N_s = \frac{N\sqrt{P}}{H^{5/4}} = \frac{187.5\sqrt{142009.56}}{(110)^{5/4}} = 198.35$$

$$\sigma_c = 0.625\left(\frac{198.35}{380.78}\right)^2 = 0.17$$

Setting of turbine, $H_s = (H_a - H_v) - \sigma_c H = (10 - 0.17 \times 110) = -8.70$

Solved Examples 17-6

Design Pelton wheel turbine for a hydropower plant which have a net head 312.5m and discharge of 5 m³/sec . Take the efficiency of the turbine is 85%, head H= 312.5, Design discharge, Q=5 m³/sec, Overall efficiency, η =85 %

For Pelton turbine, velocity of jet $V_1 = C_d\sqrt{2gH}$.

$(C_v = 0.98)$

$$= 0.98 \times \sqrt{2 \times 9.8 \times 312.50} = 76.74 m/\text{sec}.$$

$Q = A.V$

$$A = \frac{Q}{V} = \frac{5}{76.74} = 0.065 m^2$$

So, diameter of jet $d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{0.065 \times 4}{\pi}} = 0.288m$

Diameter of runner (D) (Diameter of turbine)

$$take, \frac{D}{d} = 15 \Rightarrow D = 15 \times 0.288m = 4.32m$$

Number of bucket of runner $(Z) = 15 + \frac{D}{2d} = 15 + \frac{4.32}{2 \times 0.288} = 22.50 = 23 Nos$

Bucket spacing $= \frac{\pi \times 4.32}{23} = 0.59m = 59cm$

Design speed or Rotational speed (or speed of runner)

$N = \frac{60\,\overline{V}}{\pi D}$

Where, $\overline{V}$ = trangentail speed.

$\overline{V} = Kv.\sqrt{2gH} = 0.46\sqrt{2 \times 9.81 \times 312.50} = 36.02 m/sec$

So, $N = \frac{60 \times 36.02}{\pi \times 4.32} = 159.24$ rpm.

No of poles $N_p = \frac{120f}{N} = \frac{120 \times 50}{159.24} = 37.68 = 38$

So, Corrected $N = \frac{120f}{N_P} = \frac{120 \times 50}{38} = 158 rpm$

Power (P) $= \eta 9.81QH = 0.85 \times 9.81 \times 5 \times 312.50$

$= 13028.90\,KW = \frac{13028.90}{0.746} HP = 17465.02\,HP$

Corrected $Ns = \frac{N\sqrt{P}}{H^{5/4}} = \frac{158\sqrt{17465.02}}{(312.50)^{5/4}} = 15.89 = 16$ rpm

Solved Examples 17-7

A hydroelectric plant built for design of H=20 m should develop power output 2000 HP. Determine the numbers of turbine units, if turbine having specific speed of N_s= 600 are to be used at a normal operating speed of N=230 rpm.

Solution,

H = 20 m.

Normal operating speed (N) = 230 rpm.

Specific speed (N_s) = 600 rpm

Power output (P) = 2000 HP.

No of turbine = ?

$$N_s = \frac{N\sqrt{P}}{H^{5/4}}, \quad \text{Where P in HP.}$$

$$\therefore \quad P = \left(\frac{N_s\, H^{5/4}}{N}\right)^2 = \left[\frac{600 \times 20^{5/4}}{230}\right]^2 = 12173.68\,HP$$

Number of turbine required $= \frac{3000\,HP}{12173.68\,HP} = 0.16 \simeq 1$

Solved Examples 17-8

Select turbine type and design with a power output 10 MW, head is 60 m and load variation up to minimum of 40%.

Here,

$P = 10MW = 10000KW = 13404.83HP.$

Load variation upto a minimum of 40%

Type of turbine =?

Corrected of turbine =?

For 60 m head both Kaplan and Francis turbine can be used. But due to the load variation upto a minimum of 40 %, Kaplan turbine must be selected because of the facility of the movable blades. The efficiency of the Kaplan turbine remains high over large range of load from 50% under load to 50% over load.

$$N_s = \frac{1475}{H^{1/3}} = \frac{1475}{60^{1/3}} = 378\,\text{rpm}$$

$$N = \frac{N_s H^{5/4}}{\sqrt{P}} = \frac{378 \times 60^{5/4}}{\sqrt{13464.83}} = 545\,rpm$$

$$\text{No of poles } N_P = \frac{120f}{N} = \frac{120 \times 50}{545} = 11.01 = 12Nos$$

$$\text{Corrected } N = \frac{120 \times 50}{12} = 500\,\text{rpm}$$

$$\text{Corrected } N_s = \frac{500\sqrt{13464.83}}{60^{5/4}} = 347\,\text{rpm}$$

Solved Examples 17-9

For a hydropower project with installed capacity 20MW, head 500m and specific speed, N_s = 25, Design a turbine (diameter of nozzle, diameter of turbine).

Diameter of Nozzle (d) = ?

Diameter of turbine (D) = ?

$$\text{Here, } N = \frac{60\bar{V}}{\pi d}$$

For head, H=500 m, pelton turbine is used.

$$\text{Where, } \bar{V} = K_u\sqrt{2gh} = 0.46\sqrt{2 \times 9.81 \times 500}$$

$$\text{So } N = \frac{870.15}{d}$$

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

Where, P in HP

$$\text{so, } P = \frac{20000}{0.746} = 26809.65\,HP$$

$$\text{So,} \quad 25 = \frac{870.15\sqrt{26809.65}}{d\ (500)^{5/4}}$$

$$d = 2.41m$$

$$\text{Taking, } \frac{D}{d} = 10$$

$$D = 10d = 2.41 \times 10 = 24.10m.$$

* Determine the power of a pump lifting pulp with density of 1200 kg/m^3, discharge of 1100 l/sec, head of 60 m and efficiency of 70 %.

Soln:

$\gamma = \dfrac{9.81 * 1200}{1000} = 11.772$ kN/m^3

$Q = 1100$ l/sec $= 1.1$ m^3/s

$H = 60$ m

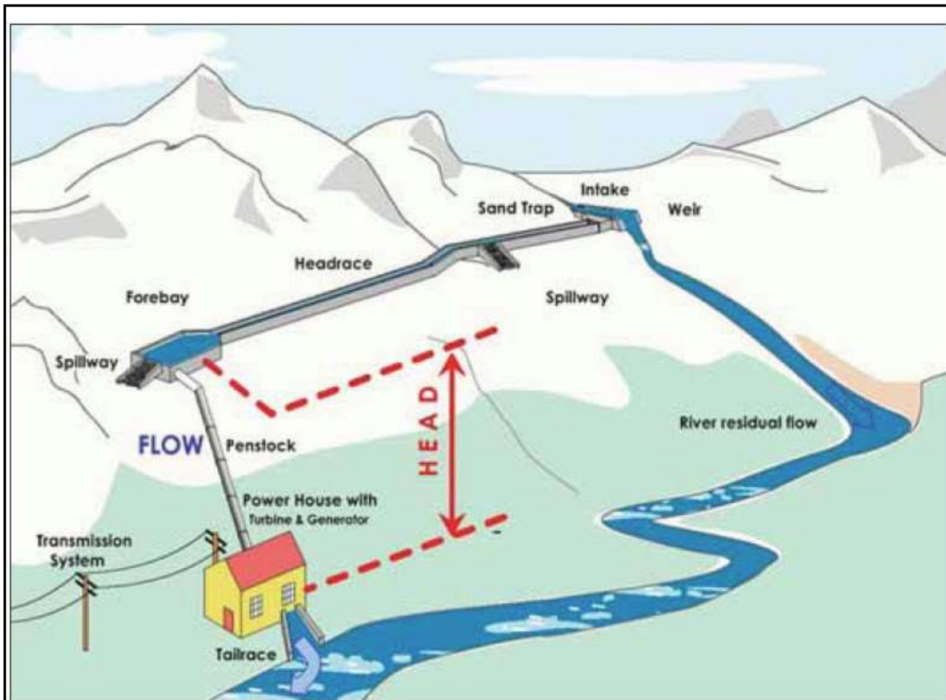
$\eta = 70\% = 0.70$

Neglecting losses,

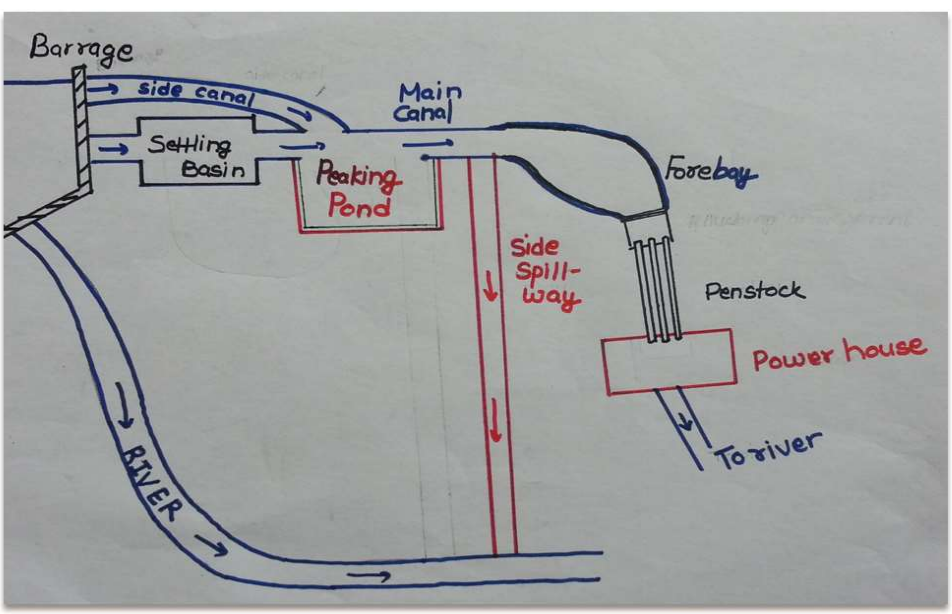
hydraulic power $= \gamma Q H = 11.772 * 1.1 * 60 = 776.952$

$\therefore$ the power of pump lifting pulp $= \dfrac{\text{hydraulic power}}{\eta}$

$= \dfrac{776.952}{0.70} = 1109.93$ KW //.

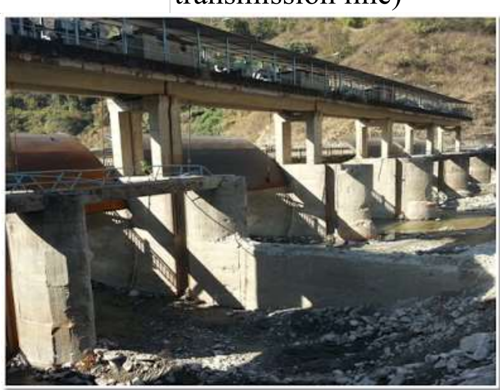


Sunkoshi Hydropower Station



Type	Peaking Run of River
Location	Sindhupalchowk
Installed capacity	10.05 MW
Design Discharge	39.9 m3/s
Maximum Net head	30.5 m
Length of Canal	2.653 km
Diameter of Penstock	2.54 m, 3 Nos.
Turbine Generator Set	3
Shaft Configuration	Vertical
Turbine	• Type: Francis • Output: 3530 kW • Speed: 300 rpm
Generator	• Type: Synchronous, 3 phase) • Capacity: 4000 kVA • Rated Voltage: 6.3 kV • Rated Current: 361 A • Rated Power Factor: 0.85

Transmission Line	66 kV, Single Circuit
Project Inception Date	End of 1968
Project Placed in Service	January 1972
Project Financed by	People's Republic of China and Government of Nepal
Project Cost	NRs. 109.37 million (including transmission line)



Sunkoshi Barrage

Sunkoshi barrage with hoisting room above

Canal

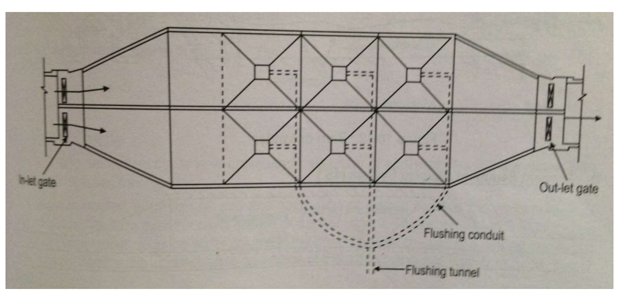
There are two canals:

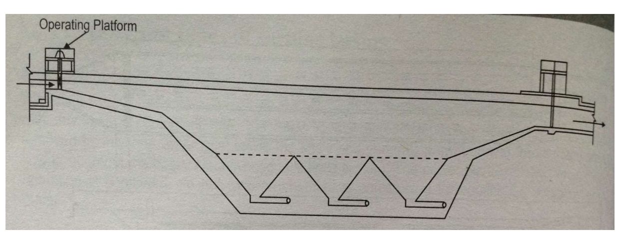
1. Main canal
2. Side canal

Under normal condition, main canal operates. However, whenever the main canal is not functioning due to the dredging work in settling basin, side canal takes its place.

Settling Basin

Hopper type settling basin is present in main canal. It is a continuous flushing system.





Trash Rack

Trash rack is present in both Main and Side canals' entrance.
It functions in checking the entry larger sediments and the floating debris in the river

Peaking Pond

Size of Peaking pond : 67000 cu m
It is used for satisfying the peaking demand of one and half day.

Forebay

Size of Forebay is 18000 cu.m. It is energy realising structure. It protects the 3 Penstocks below. It is also provided with trash rack.

Penstock

There are 3 Penstocks. Its length is 87m each and the diameter is 2.4 m each.

Turbine

The are 3 turbines receiving flow form 3 penstocks. Turbine type: Reaction turbine- Francis turbine, which is the desirable turbine for medium head of 30.5m. The speed of rotation (optimum)= 300rpm.

Powerhouse

18 POWER HOUSE PLANNING

18.1 Introduction

The structural complex where all the equipments for providing electricity are suitably arranged is a powerhouse. Two basic requirements of powerhouse planning are functional efficiency and aesthetic beauty. One of the first choices is whether to locate the powerhouse in a building above ground called as surface powerhouse or to locate it as the underground powerhouse situated in caverns, excavated below the ground.

Depending upon the location, the powerhouse can be

A. Surface powerhouse

Powerhouse complex is constructed above ground so the surface powerhouse has less space restrictions then the underground power stations. But the foundation analysis of surface powerhouse should be carefully examined. If solid bed rock isnot available in surface powerhouse option, special foundation treatment is essential such as pile foundation, mat foundation etc. architectural planning of surface powerhouse is essential to fit and for optimum design of available landscape.

B. Underground powerhouse

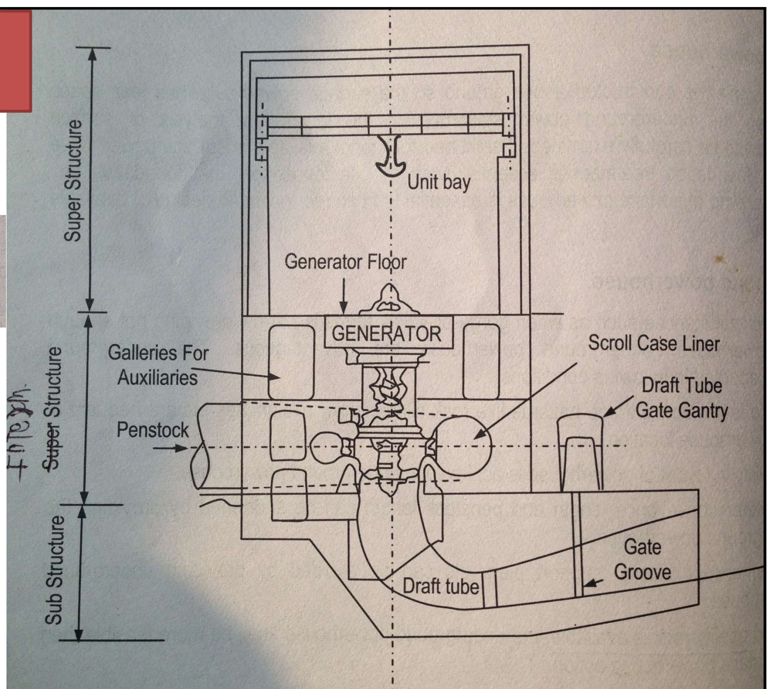
Under special circumstances such as when gorge or valley forms is narrow providing not enough space for powerhouse, underground powerhouse are advantageous.

Powerhouse Structure

Sub structure,

Intermediate structure

Super structure



Divisions of a power house

Powerhouse

In general, a powerhouse in hydropower plant may be divided into three areas:

1.The main powerhouse structure: housing the generating units and having either separate or combined generator and turbine room,

2.Erection bay, and

3.Service areas

1. Main powerhouse structure

The ***generator room*** is the main feature of the powerhouse about which other areas are grouped. It is divided into ***bays*** or ***blocks*** with one generating unit normally located in each block. The width (*upstream-downstream dimensions*) of the generator room for the indoor type should provide for a passageway or aisle with a minimum width of 10 feet between the generators and one powerhouse wall.

The ***height of the generator room*** is governed by the maximum clearance height required for ***dismantling and/or moving major items of equipment***, such as parts of generators and turbines; location of the crane rails due to erection bay requirements; the crane clearance requirements; and the type of roof framing.

All clearances should be adequate ***to provide convenient working space*** but should not be excessive.

2. Erection bay

In general, ***the erection bay*** should be located at ***the end of the generator room***, preferably at the same floor elevation and with a length equal to at least one generator bay.

The above length should be increased sufficiently ***to provide adequate working room*** if railroad access is provided into the erection bay at right angles to the axis of the powerhouse.

However, no additional space should be required if the access railroad enters from the end of the powerhouse.

3. Service area

Service areas include: offices, control and testing rooms, storage rooms, maintenance shop, auxiliary equipment rooms, and other rooms for special uses.

18.4 Powerhouse dimensioning

Three essential constituents (bay) of superstructure of powerhouse are

(a) Machine hall or the unit bay (b) Erection or the loading bay, and (c) Control bay

Machine hall

Length: Length of machine hall depends upon the number of units, the distance between the units, sizes of machines, and clearance. For the vertical alignment unit, the center to center distance between the units is controlled by the total width of the scroll casing layout. The standard distance of scroll casing is about 4.5D to 5D, where D is the outlet diameter of turbine. The minimum clearance of about 2 to 3m is added to this distance. So the center to center distance between the unit is taken as (5D+2.5) m. For higher specific speed, this distance can be reduced upto (4D+2.5) m. The total length of the machine hall can be calculated by knowing the total number of unit required for particular project. One extra unit is placed for maintenance purpose, so space required for this also considered.

Width: width of the machine hall is also determined by the size and clarance space from the walls needed as gangway. Since gangway requirements are of the order of 2.5m, as a first approximation, the width of the machine hall can be presumed to be (5D+2.5) m. The width is kept as less as possible to keep small span of the girder and roof structure. The generator is not placed at the center of the width; it is shifted to one side so as to provide adequate operating space.

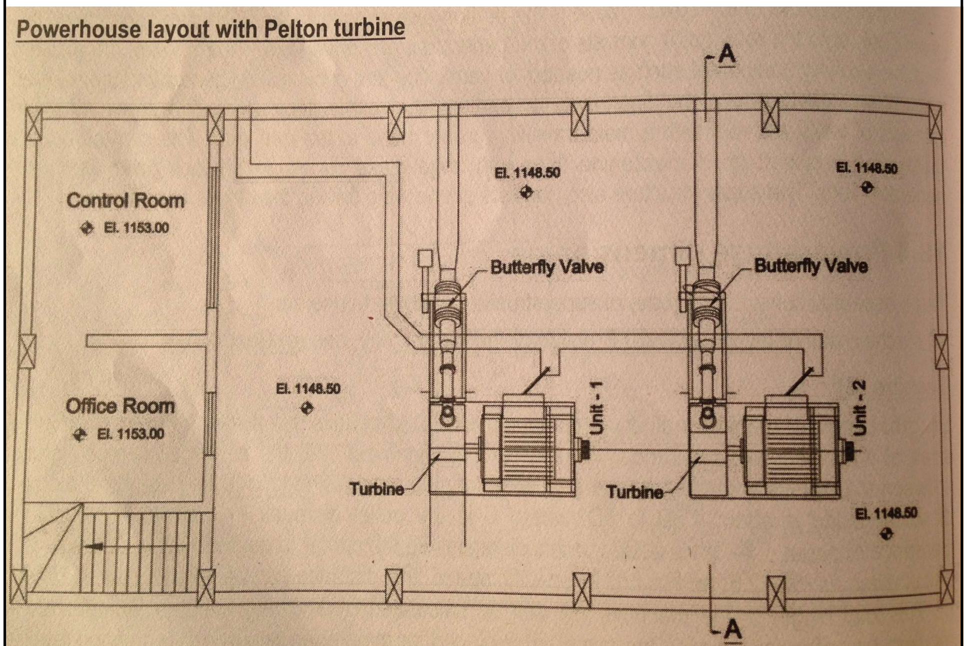
Height: The height of the machine hall is fixed up by head room requirement of crane operation. In general 2 to 2.5m head requirement is for crance operation. The hall must have a height which will enable the cranes to lift the rotor of the generator of the runner of the turbine clear off the floor without any other machine sets forming obstruction. To this clearance space is to be added the depth of crane grinder and head room for the operating cabin.

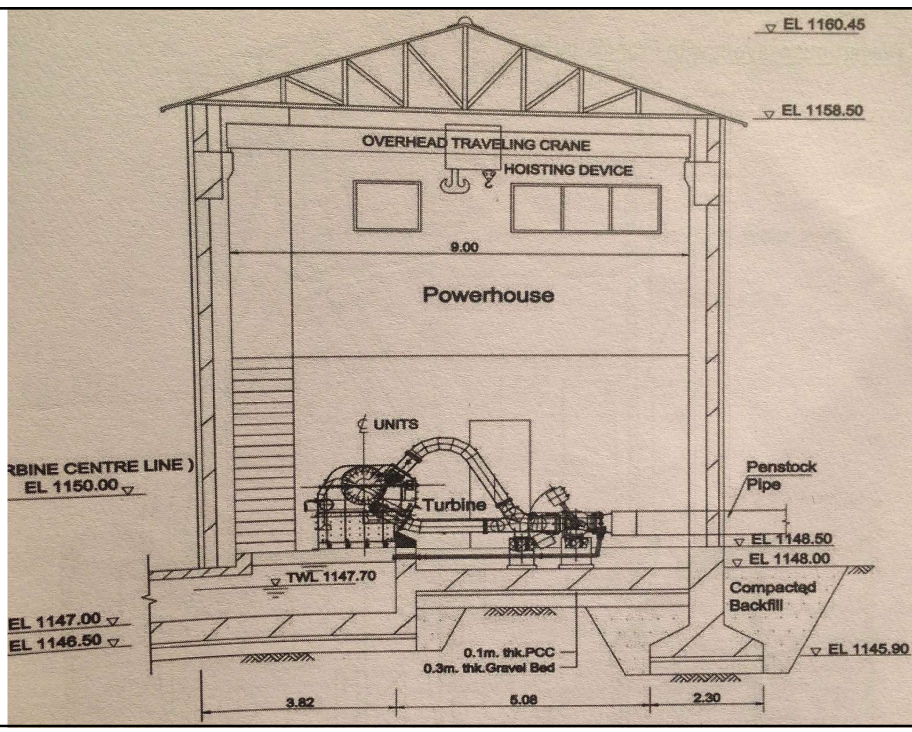
Loading bay

Loading bay, also known as erection or service bay, is a space where the heavy vehicles can be loaded and unloaded, the dismantled parts of the machines can be placed and where small assembling of the equipments can be done. The loading bay should be of sufficient to receive the large parts like the rotor and runner. The loading bay floor will be having a width at least equal to the centre distance between the machines.

Control bay:

Control bay is the main room and control other equipments like runner, gates valves, generator etc. it may be adjacent to the unit bay i.e. machine halls as it sends instructions to the operation bay from where the operation control is achieved.





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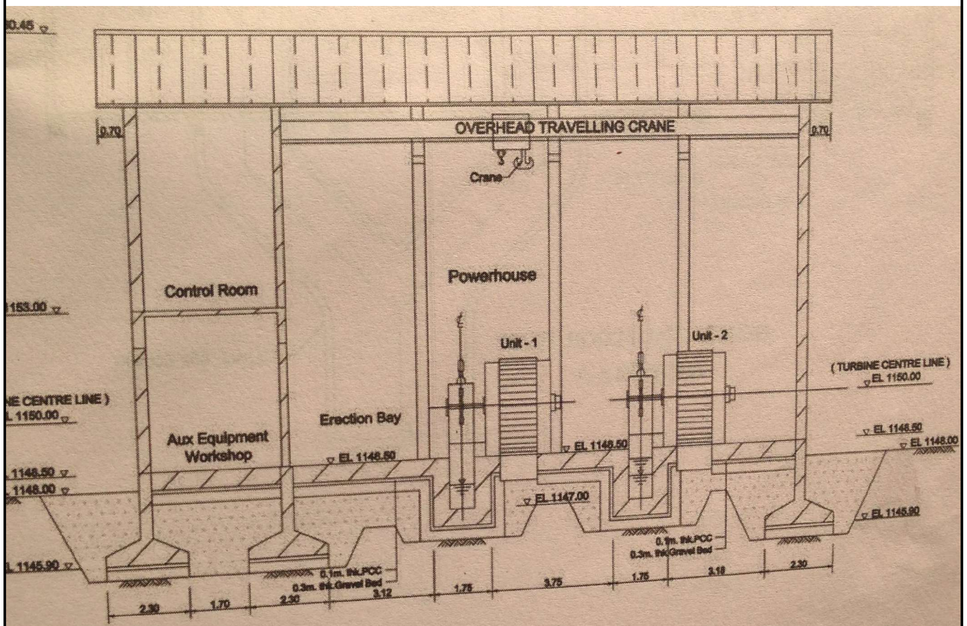
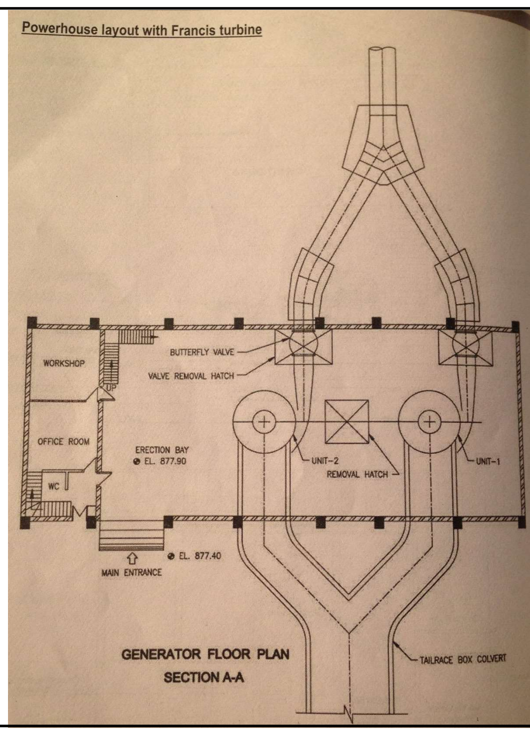
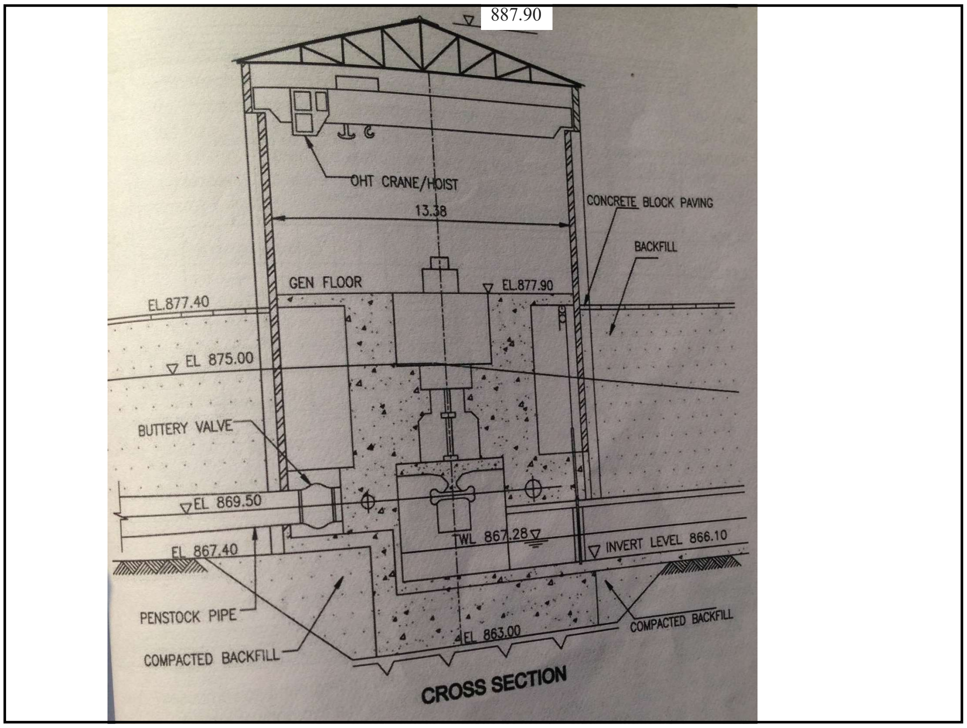
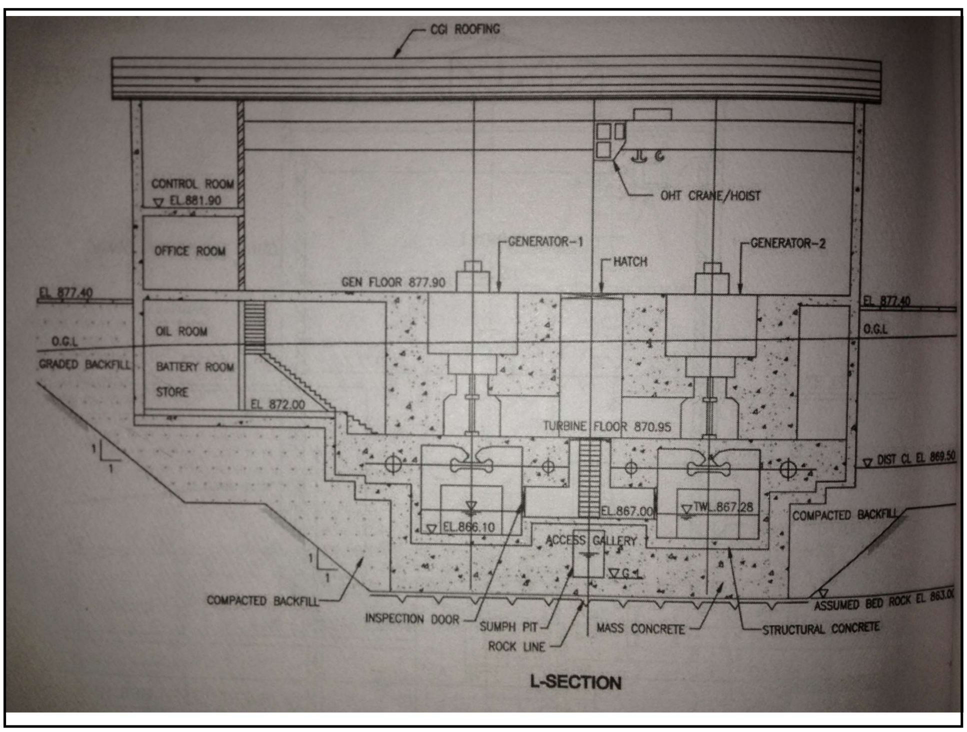


OVERHEAD TRAVELLING CRANE
Crane
Control Room
Powerhouse
Unit - 1
Unit - 2
(TURBINE CENTRE LINE)
EL 1150.00
Aux Equipment Workshop
Erection Bay
EL 1148.50
EL 1147.00
EL 1148.00
EL 1145.90
0.1m. thk.PCC
0.3m. thk.Gravel Bed



Powerhouse layout with Francis turbine
WORKSHOP
BUTTERFLY VALVE
VALVE REMOVAL HATCH
OFFICE ROOM
ERECTION BAY
EL. 877.90
WC
UNIT-2
UNIT-1
REMOVAL HATCH
EL. 877.40
MAIN ENTRANCE
GENERATOR FLOOR PLAN
SECTION A-A
TAILRACE BOX COLVERT


887.90
OHT CRANE/HOIST
13.38
CONCRETE BLOCK PAVING
BACKFILL
GEN FLOOR
EL.877.90
EL.877.40
EL 875.00
BUTTERY VALVE
EL 869.50
EL 867.40
TWL 867.28
INVERT LEVEL 866.10
PENSTOCK PIPE
COMPACTED BACKFILL
EL 863.00
COMPACTED BACKFILL
CROSS SECTION


CGI ROOFING
CONTROL ROOM
EL 881.90
OFFICE ROOM
OHT CRANE/HOIST
GENERATOR-1
HATCH
GENERATOR-2
GEN FLOOR 877.90
EL 877.40
EL 877.40
O.G.L
O.G.L
OIL ROOM
GRADED BACKFILL
BATTERY ROOM
STORE
EL 872.00
TURBINE FLOOR 870.95
DIST CL EL 869.50
EL 867.00
TWL 867.28
EL 866.10
COMPACTED BACKFILL
ACCESS GALLERY
ASSUMED BED ROCK EL 863.00
COMPACTED BACKFILL
INSPECTION DOOR
SUMPH PIT
MASS CONCRETE
STRUCTURAL CONCRETE
ROCK LINE
L-SECTION

Switchyard

The switchyard transforms electricity voltage from low to high and performs several other functions; it is the gateway from the generating unit (HPP) to the distribution network. Switchyard location is determined by local topographic conditions and available space, but it should be adjacent or close to the powerhouse.

Switchyard elevation should be above designed tail water level; for instance above the level corresponding to a flood event with a reoccurrence period of 100 years.

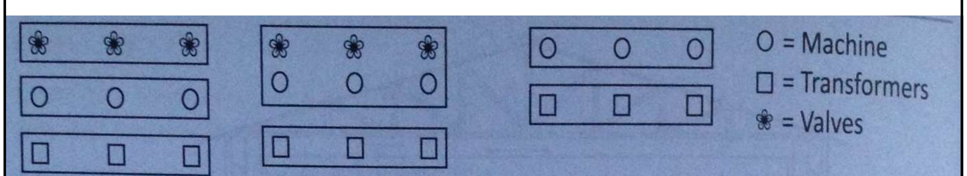
Layout of the Powerhouse

Generally the power house are carried out in one of the following ways

i. Parallel hall arrangement
ii. Linear arrangement
iii. Composite arrangement
iv. Perpendicular and skew arrangement

i. Parallel hall arrangement

In this type of layout the erection bay and control room are located in line with the machine hall and the value house machine hall and transformer hall are located in independent parallel hall.



This type of layout is modified by combing the values house and the machine hall in ore rooms and the transformers are located in other separated room with parallel to the machine hall (fig 16.5 (ii)) and another type of layout the machine hall and transformers are located in separate halls with parallel each other and the values have been omitted altogether (fig 16.5 (iii)). This type of arrangement have some advantages than other parallel arrangement

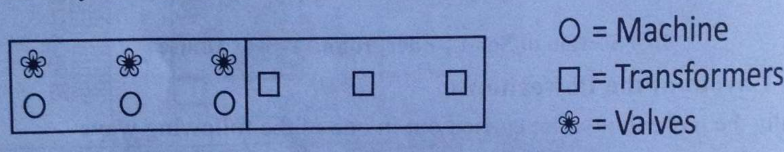
- It is located in limited dimensions of machine hall cavity.
- Greater safety in case of fine hazards or pipe burst.

Disadvantages

- Require separate crane for erecting transformers and control values.

Linear arrangement

In this type of arrangement all the machine hall, transformers, control bay one erection bay are located in a single hall

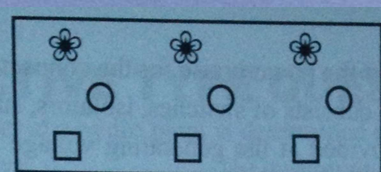


The following advantages are envisaged in the linear arrangement.

- A common access serves all the component units
- Single gentry crane can be used for handing the transformers and other equipment.
- No separate bus bar galleries are required

Composite Arrangement

This type of arrangement, the transformers generators and control values and located in the machine hall. The control values are located on the upstream side and the transformers on the downstream side of the machine hall.



(i) Composite Arrangement

In composite arrangement single crane is used for operation and maintenance of control values, transformers and generator units. It also advantageous that a common access gallery serves all the comment units It also modified by locating value house separately parallel to the machine hall and transformer hall room.

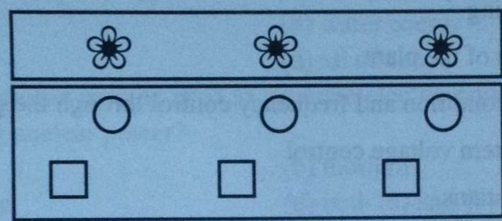
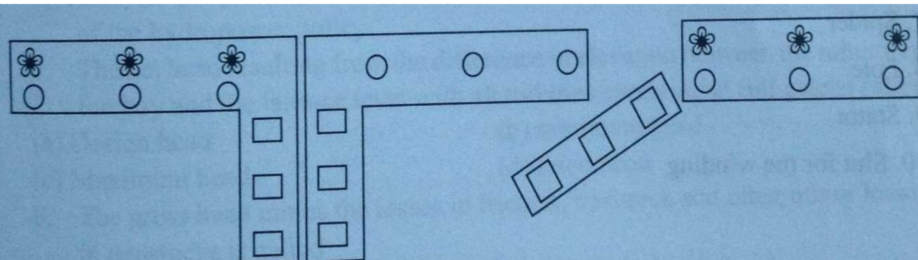


Fig. 16.6 (ii) Composite Arrangement

Perpendicular and Skew Arrangement

This type of arrangement are the slightly variation with other types of arrangement which are mentions above. The transformers are located at perpendicular or skew to the machine hall but control and service bays are located nearby the machine hall and transformers hall. in this arrangement single gentry crane is used for operator and maintenance of the transformers.



(i) (ii) (iii)

Perpendicular arrangement Skew arrangement

Switchgear

It is the system provided at the powerhouse for the connection and breaking of the circuit when necessary. It consists of switches, isolators, surge arrestor and circuit breaker. Switches are provided at the generating voltage before the transformer and at the transmission voltage after the transformer. Usually the switchgear at the generated voltage is located inside the powerhouse and for transmission voltage located at outside and termed as outside switchyards.

Figure below shows the section of a gravity dam built of concrete. Examine the stability of this section at the base. The earthquake forces may be taken as equivalent to 0.1g for horizontal forces and 0.05 g for vertical forces. The uplift may be taken as equal to the hydrostatic pressure at the either ends and is considered to act over 60% of the area of the section. A tail water depth of 6 m is assumed to be present when the reservoir is full and there is no tail water when the reservoir is empty. Also indicate the values of various kinds of stresses that are developed at heel and toe. Assume the unit wt. of concrete as 24 kN/m^3; and unit wt. of water as 10 kN/m^3.