1. Introduction (4 hours)

1.1 Historical Background of Power development in Nepal

1.2 Power Potential in Nepal and World, Cross, technical and economic potentials

1.3 Hydropower Development Policy of Nepal

Introduction:

- Hydropower is the power that can be generated in the form of electric power or other mechanical power on account of the energy head associated with the flowing or still water.
- Hydropower in the form of electrical energy is obtained from the generators coupled with turbines, which convert hydraulic energy into mechanical energy.
- High head required for running turbine which is created by constructing a dam across a river or diverting its flow into a location lower than the fore bay.

Advantages of Hydropower:

i) Renewable source of energy

ii) Economic source of Power

iii) Reliable energy source

iv) Long span of life

v) Low operation & maintenance cost

vi) Quick start & stop

Vii) High efficiency

Viii) Nonpolluting system

ix) Very efficient for peaking load

- **Disadvantages of Hydropower:**
- Takes large time for construction
- High initial cost
- Loss of large land due to submergence in reservoir
- Long term flow data is essential
- Displacement of large population from reservoir area and their rehabilitation
- Suitable site for the construction of dam is difficult to available
- High cost of transmission system for remote sites.

1.1 Historical Background of Power development in Nepal

- **Pharping Hydro** of 500 KW was the first hydropower plant in Nepal established in 1911.
- After a long interval of 25 and 29 years two other hydropower plants namely **Sundarijal 900 KW** (640 KW after interchanging of frequency from 60 Hz to 50 Hz) and **Panauti 2400 KW** came into operation.
- As the demand of electricity increased mainly onwards from 1960's bigger hydropower plants were constructed.
- In **ten years** period from 1965 to 1975 the installed capacity of hydropower increased almost **20 times**.

Chapter 1 : Introductions……

1.1 Historical Background of Power development in Nepal

- Establishment of *Small Hydel Development Board in 1975* to electrify remote laying district centers through isolated small scale hydropower projects led to categorization of hydropower plants in Nepal as follows:
- Plants less than **100 KW : Mini-micro** Hydropower
- Plants from **100 KW to 5 MW: Small** Hydropower
- Plants from **5 MW to 50 MW: Medium** Hydropower and
- Plants above **50 MW: Big** Hydropower
- *The objective of this classification is that*
  - mini-micro hydropower plants are meant for isolated rural areas
  - small hydropower plants are isolated for rural market places as well as grid connection
  - medium hydropower plants are envisaged for national demand of power and energy
  - large hydropower projects are planned for long term national demand and provision of export to neighboring countries.
- However, export of hydropower to neighboring countries other than India seems illogical in the present context.
- Government of Nepal (GoN) is trying hard in fulfilling the ever-increasing demand of electricity in the country- particularly in rural areas.

1.1 Historical Background of Power development in Nepal

- After the promulgation of Hydropower Development Policy 1992 and revised Hydropower Development Policy 2001, apart from the traditional public sector, private investors and local communities/co-operatives also have shown keen interest in generation and distribution of electricity in the country.
- In 1974, Small Hydel Development Board (SHDB) was established.
- To date, forty six small hydropower plants are in operation in Nepal with total capacity reaching nearly 50 MW.
- Also, there are about 1500 mini-micro hydropower plants - including portable type mainly for lighting purpose in the rural areas.

1.2 Power Potential in Nepal and World, Cross, technical and economic potentials

Estimation of Hydropower potential:

Converting potential energy in the kinetic energy (impulse turbine): Let us consider unit mass (m) of water is flowing in the river.

- If the height difference of the NWL and jet of the penstock pipe = h
- The potential energy possessed by the water = mgh
- This potential energy could be converted in to kinetic energy the flow through pipe conduit.

The kinetic energy of flow = $\frac{1}{2} * m * V^2$

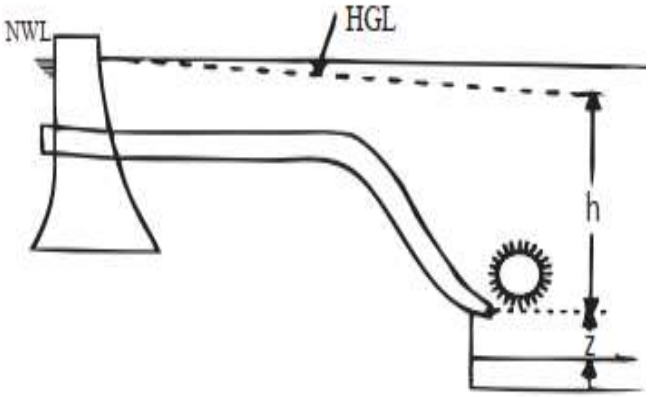

Fig 16.16 Power calculation from impulse turbine

1.2 Power Potential in Nepal and World, Cross, technical and economic potentials... Estimation of Hydropower potential.....

The velocity of the flow at the end of the conduit, V = $\sqrt{2gh}$

Kinetic energy of the flow (K.E.)

$= \frac{1}{2} * \rho * Volume * V^2$

$= \frac{1}{2} * \rho * Volume * (\sqrt{2gh})^2$

$= \frac{1}{2} * \rho * Volume * 2gh = \rho * Volume * gh$

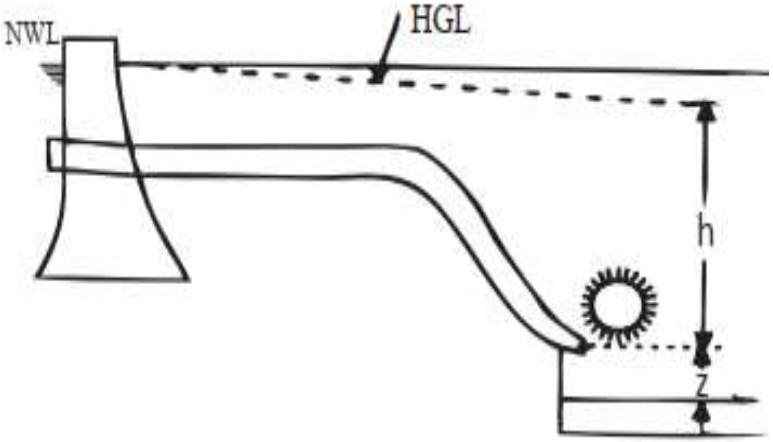

Fig 16.16 Power calculation from impulse turbine

The hydropower available

= Kinetic Energy Available / Time

$= \frac{\rho * Volume * gh}{t}$

$= \gamma * \frac{Volume}{t} * h$

Power (P) = γ*Q*h (Watt)

1) Gross potential:

- The power which is *theoretically possible* to generate is known as gross potential. 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 by using the following relation.

1.2 Power Potential in Nepal and World, Cross, technical and economic potentials…..

1) Gross potential:…..

P = $\frac{\eta * \gamma * Q * H}{1000}$ **,** Where, P= Power in KW, Q =discharge in m3/s, H= net head in (m) n = number of cascades

- The estimated gross potential of Nepal (according to Dr Hari Man Shrestha's thesis) is **83000 MW.**

2) Technical potential:

- All the theoretically possible power in nature can't be produced due to various constraints like unfavorable geology, topography, climatic condition, accessibility etc, so that the power which is technically possible to produce is known as technical potential.
- Out of 83000 MW gross potential, Nepal's technically potential power is about **44000 MW.**

3) Economic potential:

The technically feasible projects can't be implemented if they do not give a good return once the investment is made.

- The projects are usually considered economically feasible
    - if the internal rate of return (IRR) is higher than the prevailing interest rate.
    - If the benefit cost ratio is higher than unity (B/C >1).
    - Nepal's economic potential is estimated **42000 MW.**

1.3 Hydropower Development Policy of Nepal

- GoN approved on 15 October, 2001 a new "Hydropower Development Policy, 2058 (2001 AD)", Which implement earlier Hydropower Development policy, 2049.
- It has also dealt with issues such as the concept of multipurpose projects, roles of public and private sectors, use of domestic and foreign capital markets and transparency in GoN procedures in relation to private sector participation.
- Water Resources Act, 2049 BS : Umbrella act that takes care of all uses of water Resources Rules and Concerned regulation is Water Resources Rules, 2050 BS.
- Electricity Act, 2049 BS and Electricity Rules, 2050 BS covers the specific of the poser sector.

Objectives of HDP

- To utilize the existing water resources of the country and to produce electricity at a low cost.
- To ensure dependable, reliable and quality electricity services within the country at a reasonable rate.
- To tie – up electrification with the economic activities.
- To extend rural electrification to support rural economic development.
- To develop hydropower as an exportable commodity.

1.3 Hydropower Development Policy of Nepal….

Highlights of the HDP, 2058

- The policy provides guidelines and necessary institutional structure to develop the hydropower resources both for domestic consumption and export purposes.
- No license is required to generate hydropower up to **1 MW capacity**, but has to be registered with the District Water Resources Committee and the Depart of Electricity Development (DOED) has to be informed.
- License for hydropower production will be provided for 35 years and 30 years from the date of generation license issued for domestic consumption and for export.
- *In the case of storage projects*, the license period can be extended by up to 5 years in addition to the above stated periods depending on the construction period.

1.3 Hydropower Development Policy of Nepal…..

The policy covers the following topics/ provision to develop hydropower in the country to achieve the above stated objectives

1. Environmental Provision
2. Provision concerning Water Rights
3. Provision for Investment in Generation, Transmission and Distribution
4. Investment for Infrastructure Development of Rural Electrification
5. Provision relating to Transfer of Project
6. Provision relating to Power Purchase
7. Provision relating to Visa
8. Maximum Utilization of Local Resources and Means
9. Management of Investment Risks
10. Provision on Internal Electricity Market
11. Provision on Export of Electricity
12. Provision on License
13. Provisions relating to Fees

Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

2. Planning of Hydropower Projects (6 hours)

2.1 Types of Hydropower plants based on head, storage capacity and layout

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design

2.3 Layout of run-of-river and storage hydropower Projects, Components of Run-of River, Peaking Run-of River and Storage type projects.

2.1 Types of Hydropower plants based on head, storage capacity and layout

A) Classification based on head

i) Low head plant = (H< 15m)

ii) Medium head plant = (15M<H< 70 m)

iii) High head plant = (70 m<H< 250m)

iv) Very high head plant = (H≥250 m)

Salient features of the low, medium and high head plants are as follows

SN	Plant Features	Low Head Plants	Medium Head Plants	High Head Plants
1	Head	<15m	16-70m	70-250m
2	Structure for controls of water	Barrages with gates	Barrages or dam	High dam
3	Relative discharge	large	Moderate	small
4	Topography	Flat land	Hilly	Mountains
5	Turbine	Kaplan	Francis/ Kaplan	Impulse/Francis
6	Relative size of equipment	Large size turbine	Medium size	Small size
7	Supply of load	Base load plants	Base load plants, occasionally peak loads	Peak load plants
8	Character of energy	Fluctuating	Slightly fluctuating or occasionally continuous	Continuous as per load
9	Production cost	High	Moderate	Low
10	Character of storage	None or daily pondage	Daily or weekly pondage	Annually pondage
11	Character of foundation	Loose to clay soil	Rock	Rock
12	Arrangement of power house structure	Machine hall and sub structure in one unit	Arrangement of either low head or high head type	Separate power house structure

2.1 Types of Hydropower plants based on head, storage capacity and layout……

B) Classification based on Installed capacity

In Nepal, the following classification is adopted for hydropower projects.

i) Micro Hydropower plant: capacity ≤ 100 KW

ii) Mini hydropower plant: capacity between 101 KW to 1,000 KW

iii) Small hydropower plant: Capacity between 1 MW to 25 MW

iv) Medium hydropower plant: capacity between 25 MW to 100 MW

v) Large hydropower plant: capacity > 100 MW

C) Classification based on Storage capacity

Hydropower plants can be classified on the basis of whether they provides a storage reservoir or not as below

i) RoR / PRoR River plants

- Those plants, which do not regulates the hydrograph of the source river in seasonal term, are known as **Run – of – River (ROR) plants.**
- The ROR plants are generally preferred in Perennial River with moderate to high discharge, flat slope, little sediment or ice load and stable reach of the river.
- Keeping the consideration during peak hours, ROR plants may be constructed with pondage, which can regulates daily hydrograph or weekly hydrograph, store water (full or partial) to run the plant under full capacity. These type of plants are known as **peaking run of river plant (PROR).**
- ROR type: Khimti – I, Bhotekoshi, Indrawati II
- PROR type: Kaligandai – A, Middle – Marsyangdi, Sunkoshi and Panauti

C) Classification based on Storage capacity….

ii) Storage plants

- Those plants, which can regulates the hydrograph of river by one or more seasons, are usually known as **storage plants.**
- In such plants, a dam is constructed to create a large reservoir to permit carry over over – storage from rainy season to the dry season.
- Such plants are generally located on non – perennial rivers.
- These are generally medium head or high head plants.
- Storage plants involve high initial investment but provides a much more efficient and controlled use of the available water.
- Example: Kulekhani I (60 MW) and Kulekhani II (32 MW).

2.3 Layout of run-of-river and storage hydropower Projects, Components of Run-of River, Peaking Run-of River and Storage type projects.

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design

- Planning for water power development is very complex in view of the competing demand and various complicities involved during *investigation, planning, cost estimation and design* because of complexity of topography, geography, environment, resettlement and rehabilitations.
- For this purpose, various guidelines, manuals, Indian and Nepalese standards of code should be follows.
- The main objective of the planning is to *obtain maximum benefit* from the available water with least possible cost and least in concert with the environment.
- It is therefore essential to use *technologies, knowledge and skill approach* in planning, analysis, design, construction and operation of the hydropower projects.
- The *various factors should be considered for proper planning* of the hydropower projects. They are as follows:

i) Acceptable level of reliability

ii) Available technologies in the topographical and geological aspects

iii) Options for environment and ecological aspects

iv) Demand and supply side management

v) Policies of the government

vi) Global trend in fuel availability and its price

vii) Economic aspects

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design…….

The hydropower development cycle consist following of three main parts.

i) Pre construction

ii) Implementation

iii) Operation

- The major part of investigations, planning and design take place in the pre construction

Phase **(First Phase).**

- There may be many project possibilities and a large number of alternatives to be investigated.
- Each project has different physical properties and conditions which have to be considered in order to obtain a good performance on the basis for planning and designing.
- *Project investigation, planning and designing* are normally organized in several consecutive studies which are planned in increasing order of detail, magnitude and reliability. They are as follows

- Reconnaissance studies
- Pre – feasibility studies
- Feasibility

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design…….

The **second phase**, implementation, covers engineering and construction of the project. The main activities of the second phase are

a) Design and procurement – which consist of

- Definite plan study
- Final Design
- Tender / contract document
- Tendering and contracting

b) Detailed Design and work drawing – which consist of

- Detailed design and preparation of work drawings for civil, structural and transmission works
- Detailed design and preparation of shop drawing for electro mechanical equipment and hydro mechanical equipment

c) Construction of Civil, Structural and transmission works

d) Manufacture, erection of electromechanical and hydro mechanical equipment

e) Commissioning and start up installations

The last phase in the life of hydropower projects, and by far the longest phase is operation. In this phase, the project starts for revenue generation and in this stage research and training works shall be done in parallel to the operation of the project.

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design…….

1) Reconnaissance studies:

- The first stage of hydropower studies is the identification of projects.
- Reconnaissance studies are the first steps of project oriented planning and basic investigations, to identify and investigate the available Hydropower resources.
- The project identification includes selection of the project having least cost to fulfil the stated purpose.
- The specific purpose leading to reconnaissance investigations is usually defined needs for electric power and energy i.e. existing power markets or demand forecasts.
- Reconnaissance studies are carried to follow pre-feasibility and feasibility studies but it will be required less accuracy and lesser detail.
- In this stage of studies, it is necessary to rely on very experienced hydropower planner in order to formulate well balanced projects.

The main purposes of reconnaissance studies are as:

i) To identify suitable power projects for the stated purpose.

ii) To investigate apparent alternative solution for inclusion in plans or rejection of plans.

iii) To investigate and study the various projects and project alternatives to the confidence level required.

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

v) Record lower ranked projects and project alternatives for future reference.

vi) Provide preliminary cost figures and implementation schedules for the selected project.

Vii) To record, screen, rank the project alternatives.

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design…….

Steps and major Activities of Reconnaissance studies

1. Data Collection:

General data and information, power market, hydrology, topography, geology and geotechnical engineering, environment and socio economy.

2) Desk study

- Define and tentative layout of the project.
- The main elements of study are power demand, flow, head, regulation need and environmental constrains.

3) Field work and design

- Verification of layouts, care of the river should be given attention, reselection of permissible alternatives.

4) Estimate and schedules

- Preliminary cost estimate and implementation schedules are prepared.

5) Environmental and Social studies

- IEE (Initial Environmental Assessment) or EIA (Environmental Impact Assessment) shall be screened based on the legal requirements.

6) Economic assessment

- The power unit cost (cost per KW installed) and unit cost of annual generation capability (cost per KWh of annual generation) prepared. These are used for ranking of projects and comparisons with other similar purpose projects.

7) Report: Study report will conclude with a statement on the viability and suitability of the projects.

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design…….

2) Prefeasibility studies:

The main purposes of Prefeasibility studies are as:

- Establish the need and justification for the project.
- Formulate a plan for development of the project.
- Determine the technical, economical and environmental practicability of the project.
- Define the limitation of the project.
- Ascertain local interest and desire for the project.
- Make recommendations for further action.
- To select and apprise possible projects of further consideration.

Steps and major Activities of Prefeasibility studies

1. Data Collection:

Data collection of hydrological, metrological data, topographical map, geological mapping carried out.

2) Desk study

Reconnaissance study report shall be reviewed and verified.

3) Field work and design

Topographical survey, geological survey, hydrological study – correlation upto headwork site, preliminary hydraulic calculation using thumb rule, cost estimate.

4) Financial Analysis

Based on preliminary component sizing, preliminary cost estimates, cost/KW, FIRR (Financial Internal Rate of Return).

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design…….

2) Prefeasibility studies: (Steps and major Activities of Prefeasibility studies)…..

5) Environmental and Social studies

If the project required to conduct, IEE, the ToR of IEE works shall be prepared and if the project requires EIA, the ToR and scoping works shall be carried out.

6) Report

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

3) Feasibility studies:

- 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 owner can decide whether go or not to go for implementation of the project, i.e. to proceed with the definite plan studies, final design and construction of the project.
- It also serves as application documentation for the development license.
- The purpose of the investigation is to establish and define the specific engineering and operation plan and to determine whether the potential development has technical, economical and environmental feasibility and justification under anticipated economic conditions.

2.2 Stages of hydropower development: Reconnaissance, Prefeasibility, Feasibility studies and detailed Engineering design…….

3) Feasibility studies……

Feasibility investigations included analysis of resources:

i) Estimates of net economics values to be produced

ii) Estimates of cost of development and construction

iii) Estimation of cost of operation ,maintenance and replacement

iv) Assessment of the implementation impact of the project.

v) Assessment of the environment effect after implementation of the project and cost of mitigating the effects.

Steps and major Activities of Feasibility studies

1. Data Collection:

- Supplementary general data and information.
- Hydrological data – from the established gauging station and additional data if any
- Topography – supplementary survey if required.
- Geology and geotechnical engineering
- Environment – detailed socio – economic data from social survey.
- Socio economy

2) Review of Desk study and prefeasibility study

- The desk study report and Reconnaissance level study report and other relevant study report relevant to project shall be reviewed.

3) Feasibility studies……(Steps and major Activities of Feasibility studies)….

3) Field work and design

- Field trip for layouts for the proposed layout during prefeasibility study.
- Field visit for additional survey
- Field visit for geological data collection, pit sampling.
- Hydrological – establishment of gauging station at headwork and powerhouse sites.
- Detailed investigation for structure location.
- Design of each component (hydraulic design), component optimization.

4) Estimate and schedules

- Detailed cost estimate
- Detailed implementation schedules and planning.

5) Environmental and social studies

- Detailed IEE or EIA report shall e prepared.
- Environmental management / mitigation plan

6) Economic assessment and financial evaluation

- Financial evaluation
- FIRR, B/C Ratio, NPV calculation.

7) Report

The feasibility report is also bankable document. To get generation license comprehensive feasibility study report is must.

- Feasibility study report (sometimes also called DPR)
- Conclusion and recommendation for further action.

2. Diversion type

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

2.1 Run-of-river hydropower projects

In the run-of-river schemes the turbine generates electricity as and when the water is available and provided by the river. When the river dries up and the flow falls below some predetermined amount, the minimum technical flow of the turbine equipping the plant -, generation ceases.

Run-of-river (RoR) hydropower projects shall be designed to generate power using the flow in the river as it occurs. Due to the large seasonal variations in flow in Nepali rivers, these projects shall normally be designed to utilize the flows in the river during the dry season. The installed capacity of these schemes shall be based on the dependable flow available in the river throughout the year, with deductions for mandatory flow releases for downstream water needs.

Weir or barrage is constructed for diversion of water from river, which is conveyed through the conveyance system and delivered to powerhouse through penstock pipe.

The run-of river type projects have three possible layouts

- Run-of river projects with canal system
- Run-of river projects with pipe option
- Run-of river projects with tunnel option

Run-of river plants are designed with the discharge of certain percentile of available flow. When the flow available in the river is greater than the design discharge, the designed discharge is used for power generation. But when the flow available in the river is less than the design discharge, the available flow in the river deducting minimum downstream release as an environmental flow is used for power generation.

The suitability of different option and general layout of run-of-river project having different options are discussed below.

A. Run of river with canal option

This type of project is generally employed in small projects, where the gentle slope terrain with good geology is available from the diversion point of the river to the powerhouse location. Water is diverted by small height diversion weir and the conveyance system is canal upto the forebay location. Penstock pipe draws water from forebay to the powerhouse. In this type of scheme, the flow in the canal is open channel flow. Aqueduct, superpessage, siphon etc are constructed in the crossing as a cross drainage structures. In this type of scheme, the water spilled water from the forebay during rejection of load should be managed properly. Generally, the spilled water is managed to nearby kholsi or stream with necessary energy dissipating structures. Landslides may affect this type of scheme frequently, water loss from evaporation and seepage may be high. Water can be used by local people in unauthorized way within the canal section for irrigation and other purposes as the flow is open channel flow. This type of scheme is less costly as stone masonary or

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R.C.C canal can be constructed as a conveyance system. A typical layout of run-of-river scheme with canal scheme is shown in figure 2-1 below.

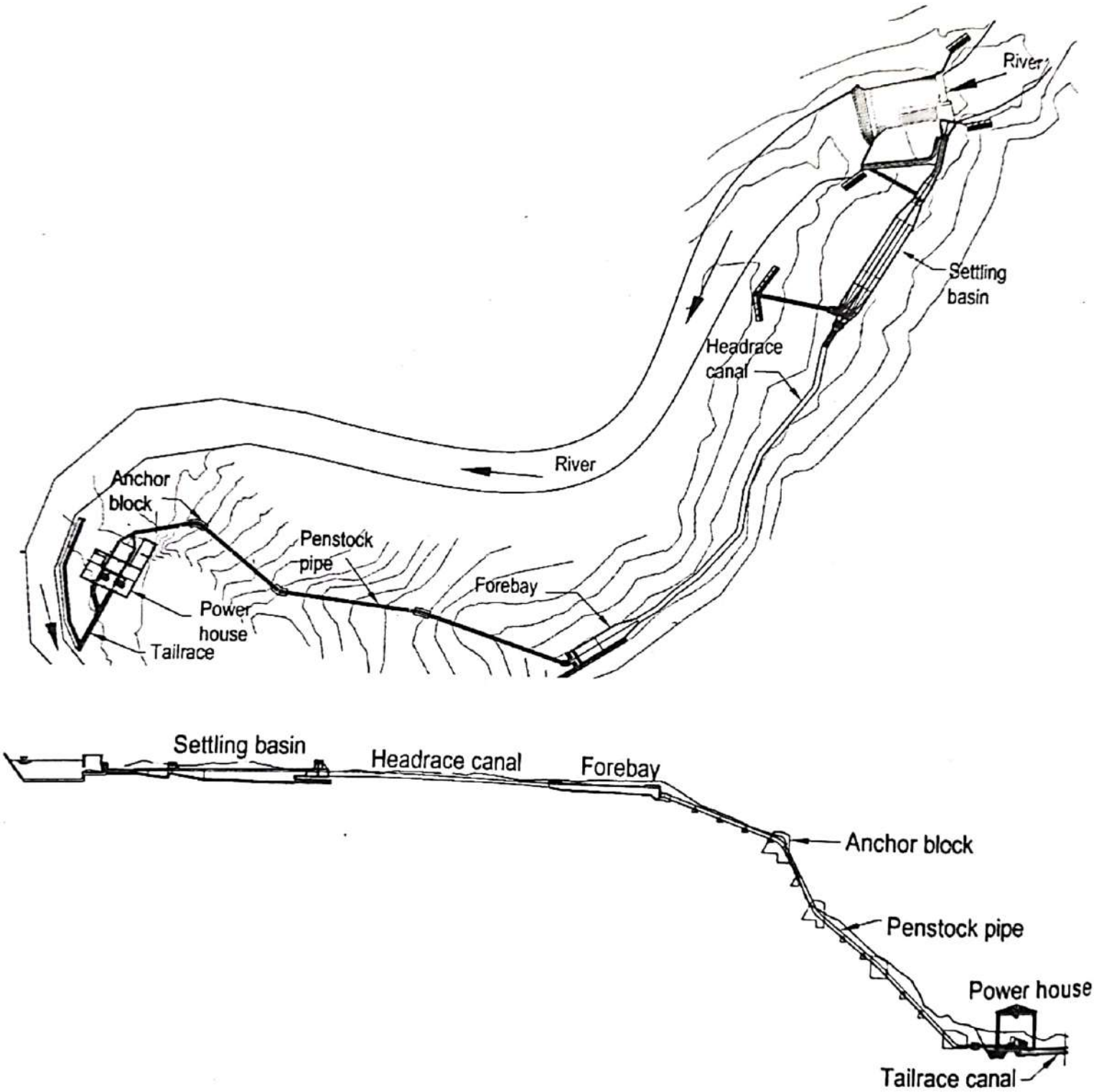

Figure 2-1 : Plan and section of RoR project with canal option

B. Run-of river projects with pipe option

If the conveyance alignment is difficult to construct canal, there are large number of loops which increase the length of the canal, there are several number of cross drainage structures, the conveyance alignment passed through the household areas, landslide areas the pipe is better than the canal option. In such type of option, after the diversion weir, settling basin / headpond is constructed and directly supplied water to powerhouse by pipe. The length of the conveyance

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system can be reduced significantly by using pipe alignment option. Surge tank may be provided near powerhouse if the pipe alignment is long. Sometime, forebay is constructed upto which water is delivered by low pressure pipe of open channel flow in pipe and then water is supplied to power house by penstock pipe from forebay. Typical plan and profile of RoR project with pipe alignment is shown in figure 2-2 below.

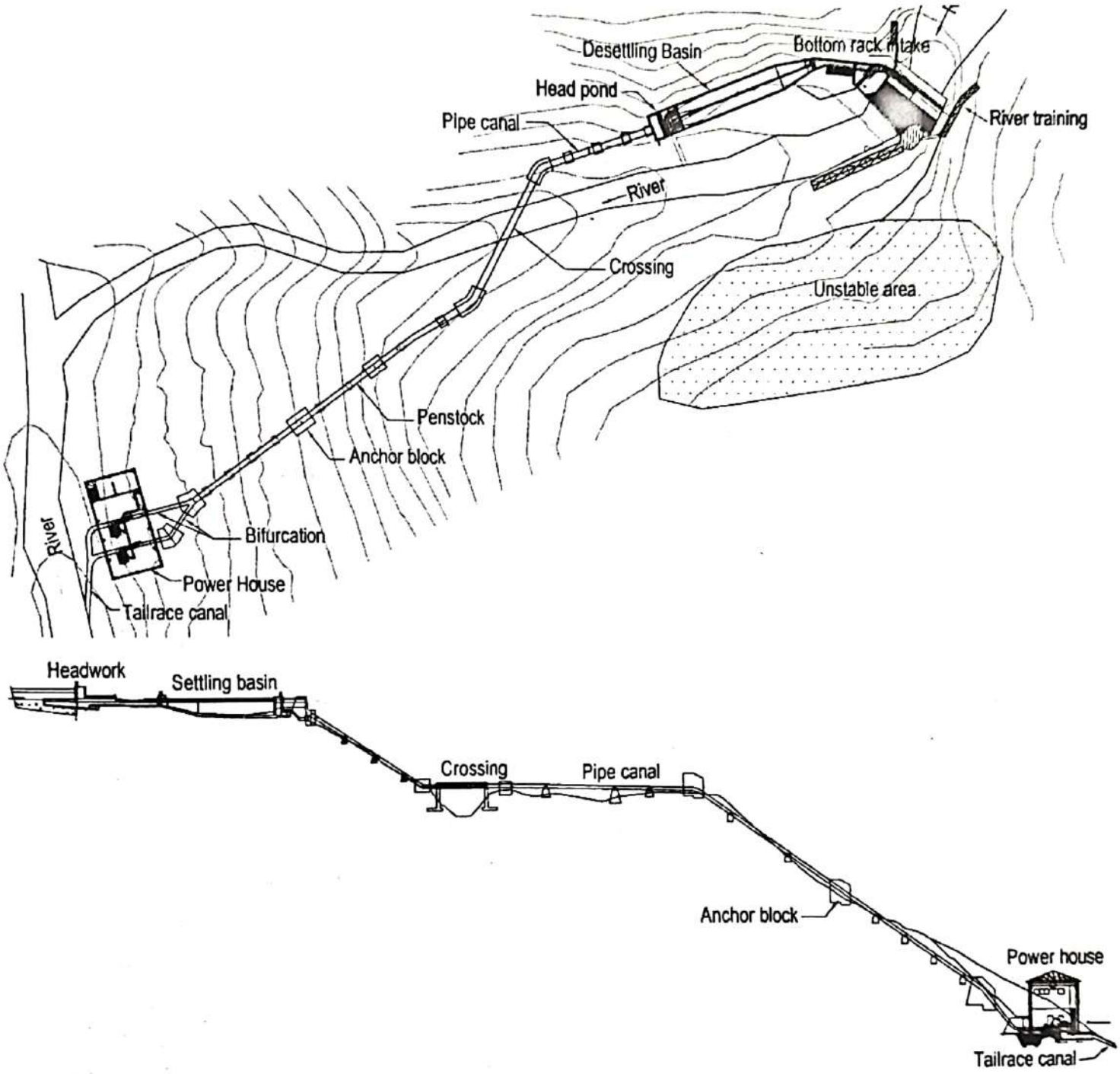

Figure 2-2 : Plan and section of ROR project with pipe option

C. Run-of river projects with tunnel option

In some location, if the river is in loop form and good geology is available, it is economical to use the tunnel system. The headloss can be reduced significantly in this system because of shorter length of conveyance system. The run-of river project with tunnel alignment is generally feasible for large projects. The land acquisition problem can be minimized by constructing underground

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structures. Surge shaft is constructed at suitable location near powerhouse. Typical plan and profile of RoR project with tunnel alignment is shown in figure 2-3 below.

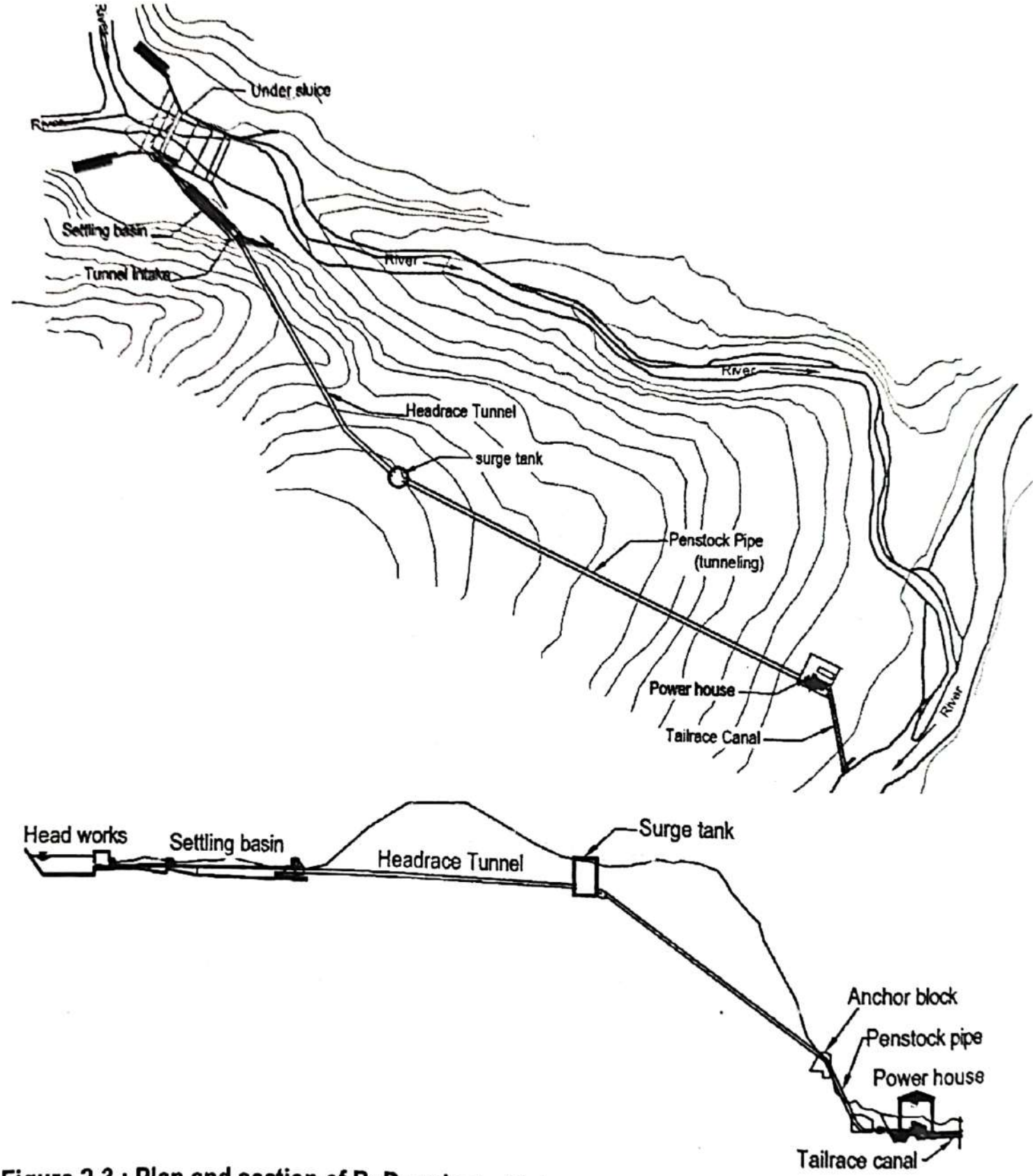

Figure 2-3 : Plan and section of RoR project with tunnel option

2.2 Peaking run-of-river hydropower projects (PRoR)

Peaking run-of-river (PRoR) hydropower projects shall be designed to generate power according to fluctuations in the power demand through hourly regulation of the daily flow available in the river. This capability shall be achieved through daily pondage of the water at the headworks during dry

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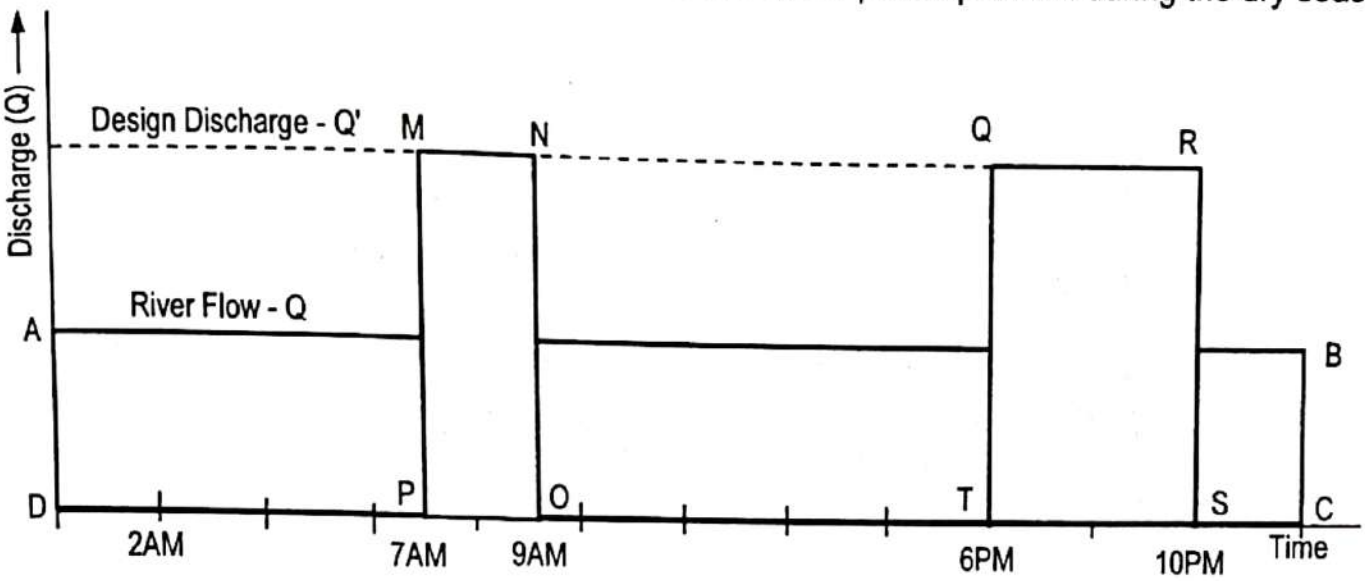
seasons when the flow in the river is less than the design discharge. During the rainy season when the river discharge exceeds the design discharge, the plant shall operate like a RoR plant to facilitate bed load flushing.

In order to provide peak load coverage, the installed capacity of PRoR schemes shall be sized for flows larger than the dependable river flow. Based on site conditions and optimization, peak requirements shall usually be fixed for about 4 to 6 hours of plant operation during the dry season.



The peak duration is from 7AM to 9AM in morning and 6PM to 10 PM in evening i.e 2 hour in morning and 4 hour in evening.

Area ABCD = MNOP + QRST

If 40 percentile exceedence flow in a river is $10m^3/sec$, then the designed discharge of a 6 hour peaking run of river plant is $40m^3/sec$. ($10m^3/sec \times 24hr = 40m^3/sec \times 6$ hr). Hence peak installed capacity of a power plant can be increased four times in peaking run of river plant then simple run of river plant.

The layout of peaking run-of-river projects are same as that of run-of river schemes i.e three possible options are

- Peaking run-of river projects with canal system
- Peaking run-of river projects with pipe option
- Peaking run-of river projects with tunnel option

2.3 Choice between RoR and PRoR schemes

Although both types of projects generate the same amount of daily energy under a given set of conditions, the choice between the two schemes shall be made considering the following factors:

- Owing to their greater flexibility in operation, the PRoR projects are capable of bridging the gap between water availability and power demand and are, as such, more suited to the power system in Nepal.
- The additional cost of storage facilities required for PRoR projects makes their energy costlier than that generated from RoR projects.

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- But for the private sector, there is no different energy rate within a day. (daily peak hour and off peak energy rate is same). For the development of PRoR project, it need additional cost but no incentive mechanism is introduced. So PRoR projects arenot financially justifiable. But for NEA, for better system operation, to address the morning and evening peak, PRoR project is must.

In Country like Nepal the energy is consumed for household level use (Cooking, lightening etc) only not for industrial uses. So the daily load variation is high. The evening peak is about 2 times the daytime load and the morning peak is about 1.3 times greater than the day time load. The morning peak continues for two hour and evening peak is continued for four hour. So for such load variation case, the peaking runoff river plant is quite useful/importance.

2.4 RoR and PRoR project components and their functions

The hydropower components for RoR and PRoR projects are

- Diversion weir
- Undersluice
- Intake
- Gravel trap
- Approach canal
- Settling basin
- Conveyance system (Canal, Pipe or Tunnel)
- Surge tank / Forebay
- Penstock pipe
- Anchor blocks and support piers
- Powerhouse
- Tailrace

The main functions of these components are:

1. **Diversion weir**: Structure placed across the river to divert a part of the river flow to the water conveyance system through the intake.
2. **Intake**: An opening to draw design flow from the river and provided with trash rack screen to prevent entry of floating debris and coarse bed load into the water conveyance system. It is equipped with a control gate to regulate flow down the conveyance system under various flow conditions in the main river.
3. **Gravel trap**: Structure to intercept bed load carried by the diverted flow. It is equipped with gravel flushing gate followed by a flushing canal to discharge the trapped gravel particles back to the river. A lateral side spillway also can be located in this structure to spill excess flow entering from the intake during high floods.
4. **Settling basin**: An approach canal conveys water from the intake/ gravel trap to the settling basin. It settles suspended sediment contained in the water. The settled sediment is subsequently discharge back into the river through a flushing arrangement consisting of flushing gate/ valve and canal.
5. **Headrace conveyance system**: Usually a canal/ pipe/ tunnel or a combination of these for the purpose of delivering design flow to the forebay or surge tank with a minimum energy loss.
6. **Forebay (head tank or surge tank)**: It stores water to compensate changes in design flow, stabilizes head on the penstock. A side spillway is provided in this structure to spill excess water coming from the conveyance system. The forebay is equipped with a fine

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screen/ trash rack to prevent floating debris into the penstock pipe and ultimately to the turbines. It serves as a secondary settling basin as well and a sediment sluice to remove settled suspended particles.

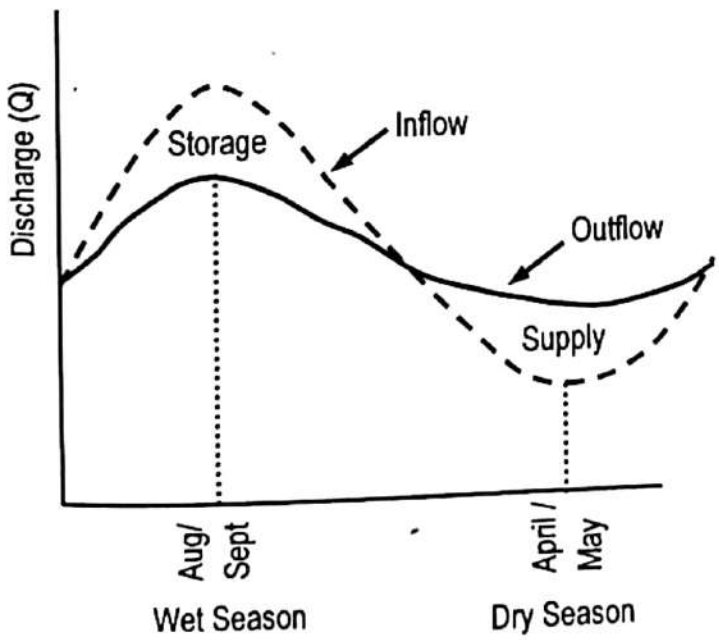
7. **Penstock**: Conveys design flow under pressure to the turbine/s with a minimum head loss. Anchor blocks and saddle supports are provided to stabilize forces acting on the penstock pipe.
8. **Powerhouse**: Houses electricity generating units, switch gear, control equipment and maintenance equipment.
9. **Tailrace**: Disposes the design flow discharged by hydraulic turbines after power generation.

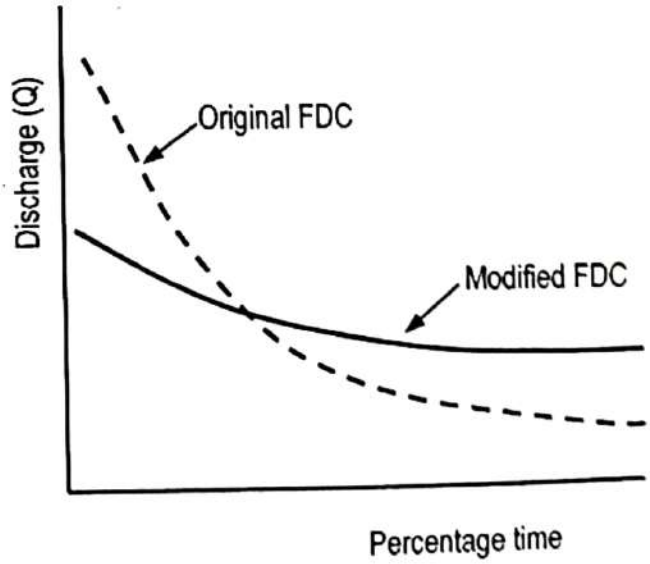
2.5 Storage project

In storage projects, dam is constructed across a river to create a reservoir upstream. Most of rivers of Nepal have huge amount of flow variation in rainy (monsoon) season and in dry season. Due to such variation, the power generation during dry season is quite low while designing run- of-river schemes. The excess water flowing in the river during rainy season is stored so that the scheme can run throughout the year. Main components of storage project are dam body of sufficient height to store required amount of water, spillway section of sufficient capacity to discharge excess flood, headrace tunnels, surge tanks, penstocks and powerhouse.

The storage plants may be of following types based on the regulation of flow:

Seasonal storage: Plants with reservoir, which can regulate the discharge of summer (monsoon) season to run the plant in enhanced capacity during winter. But there is spill in the monsoon period. The percentage storage of the annual flow available depends upon the dam height and valley type to store the water. The seasonal storage type of project is designed from the dam height optimization works and overall economical analysis for storage project.





Annual storage: Plants with reservoir, which can fully regulate the average hydrograph of the source river. The average outflow is calculated from the available long term mean monthly (or

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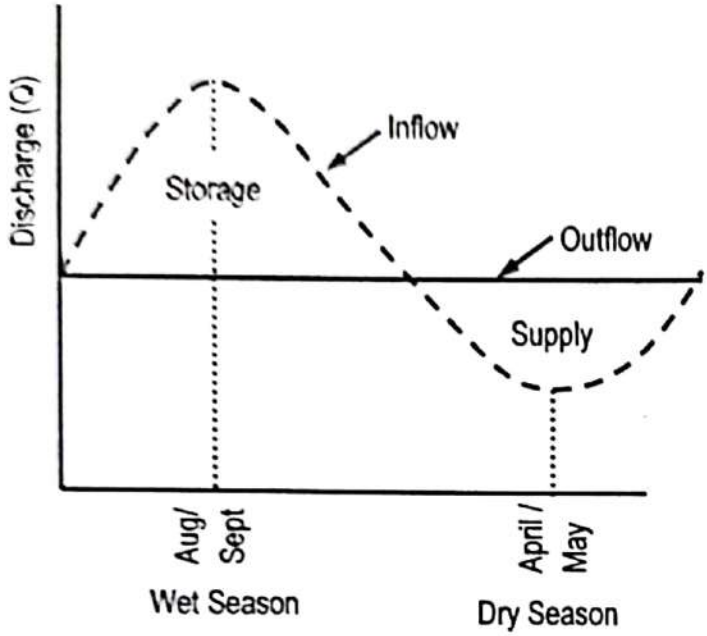
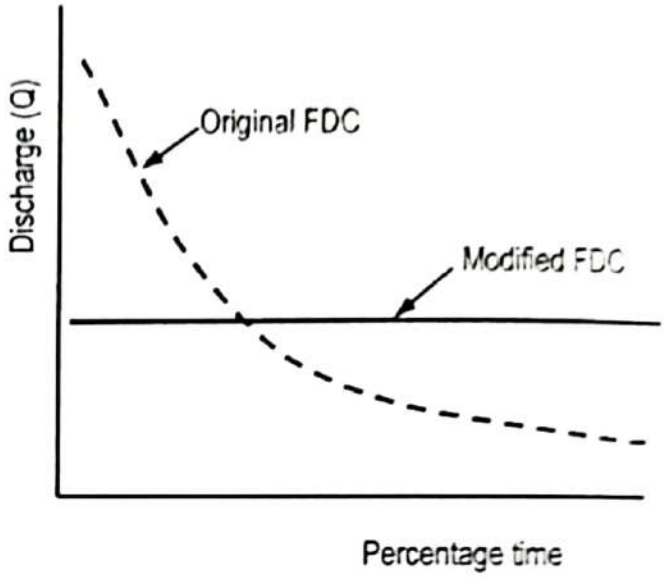
mean daily) flow. There will be no spill of water in normal year during monsoon season. Constant flow will continuously flow downstream of the reservoir.

$$Outflow = Q_{avg} = \frac{Q_{jan} + Q_{feb} + Q_{mar} + \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots + Q_{dec}}{12}$$





Pluri annual storage: Plants with reservoir, which can regulate the hydrograph of source river for several years. This type of storage project is generally constructed at the water resources scarce areas; otherwise such type of storage will not be financially feasible.

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

Based on the layout the storage project may be of two types:

- Storage project with powerhouse at the dam toe
- Storage project with powerhouse located certain distance downstream from dam

A. Storage project with powerhouse at the dam toe

In this type of storage project, powerhouse is constructed at the toe of the dam body. The main components in such schemes are, Dam, spillway, penstock and powerhouse. This type of project layout is preferred when there is wide river at the dam axis. The space for construction of required number of powerhouse units is available at the toe of the dam. Spillway is provided at the overflow section of the dam. The plan and section of high dam storage project is shown in figure 2-4 below.

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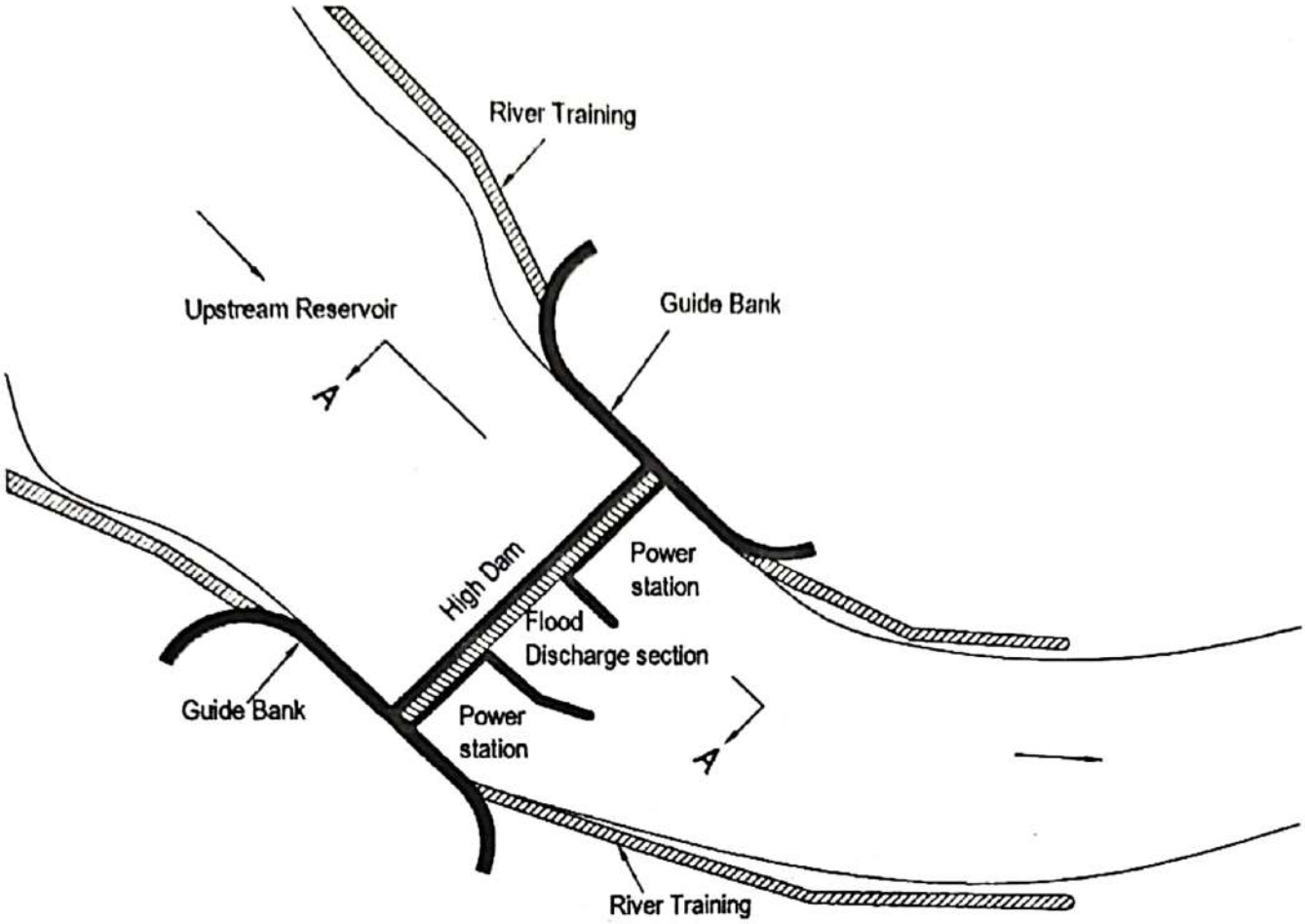
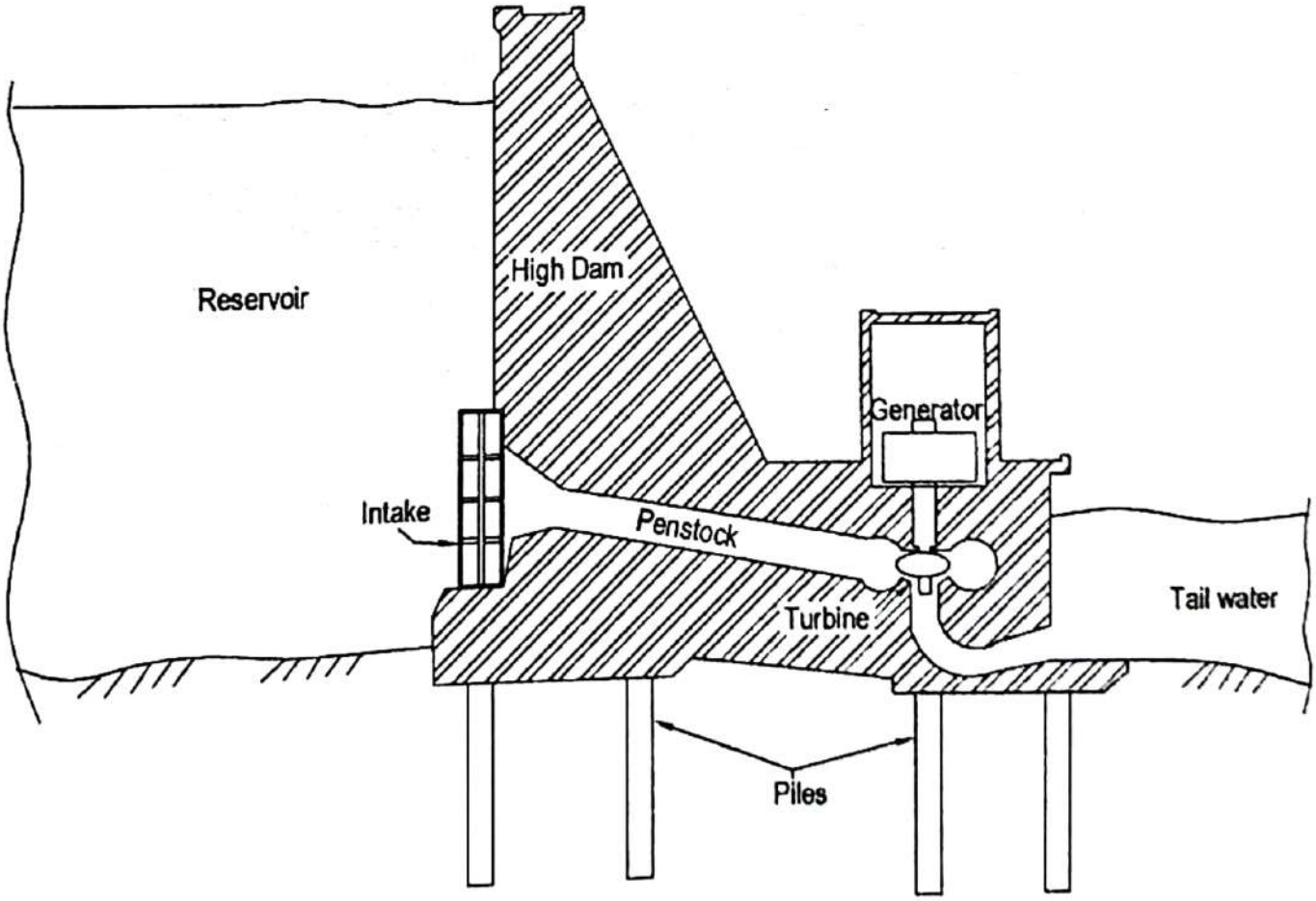


(a) Plan View



(b) Sectional View A-A

Figure 2-4 : Plan and section of high dam storage project with PH located at dam toe

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B. Storage project with powerhouse located certain distance downstream from dam

In natural bend of river is available from the dam axis, the powerhouse can be located certain distance downstream from the dam body. Additional head can be achieved by using this type of layout in storage project. Dam body, spillway, tunnel, surge tank, penstock and powerhouse are the main components of this type of project. Reservoir is constructed in front of main dam body, which is taken away from the dam body and powerhouse is constructed at suitable location to generate hydropower. The plan and section of the storage project with powerhouse located certain distance downstream from dam is shown in figure 2-5.

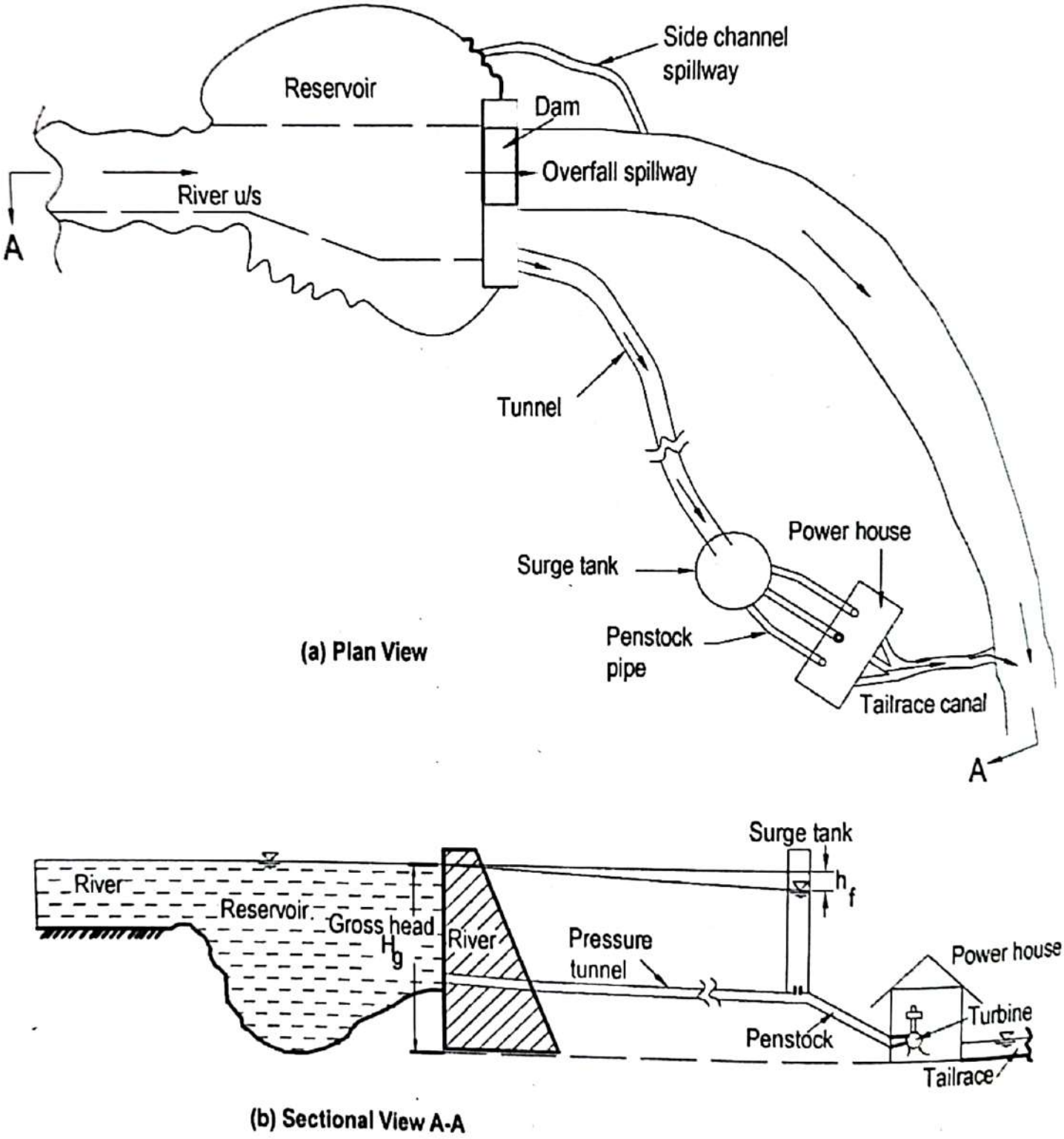

Figure 2-5 : Plan and section of storage project with PH located certain distance d/s

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2.6 Basin diversion project

The river located at higher elevation can be diverted to the river in lower elevation to get hydropower is called basin diversion project. The reservoir can be generated in higher river system for regulated flow in hydropower generation. Jhimruk and Andhikhola hydropower projects in Nepal are basin diversion type projects, in which the Jhimruk river is diverted to Madi river to get 12MW in Jhimruk hydropower project and the Andhikhola River is diverted to the Kaligandaki river to generate 5MW in Andhikhola hydropower project. The layout of Jhimruk hydropower project is shown in figure 2-6 below

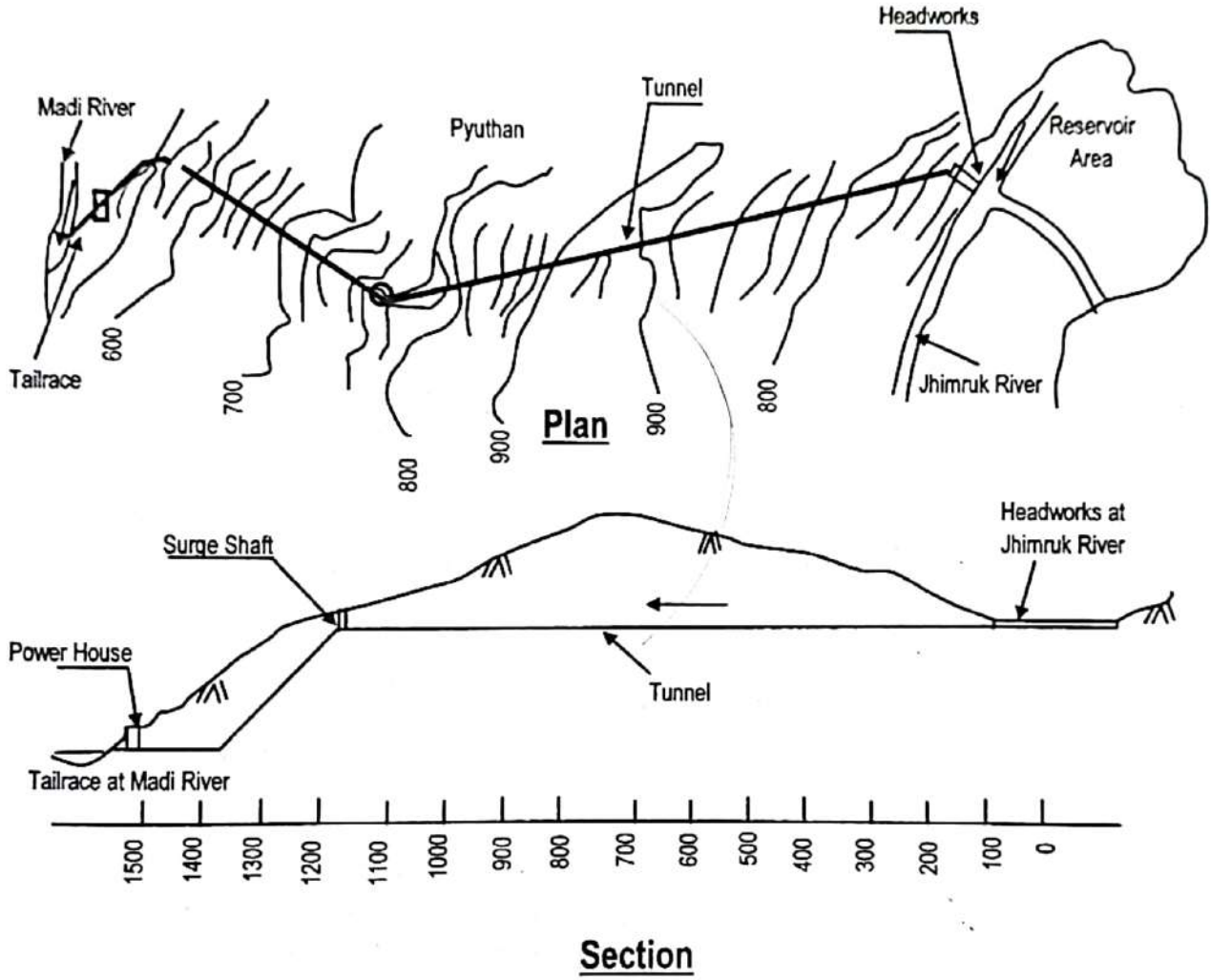

Figure 2-6 : Plan and section of basin diversion type project

2.7 Pumped storage projects

In pumped storage power plant, the hydropower is generated during peak time (during power deficit time) and same arrangement is used for pumping during off peak time (during excess power time). The optimum utilization of water resources can be achieved and peak load management can be easy by the development of pumped storage project. The schematic diagram of basic arrangement for pumping power plant is shown in figure 2-7.

The fundamental arrangement consists in having two pools, at higher level and other at lower level with the powerhouse occupying at intermediate position. The water passages from the higher level pool to the powerhouse and from the powerhouse to lower pool during hydropower generating

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phase but during pumping phase water flows from lower pond to powerhouse and powerhouse to upper pool.

Hence, during peak hours, the pumped storage plant works as a peak load plant i.e it generates power in peak hours. During off peak hours, water from the tail race reservoir (Lower reservoir) is pumped back by a pump to the upper reservoir through the same pipes as the plant remains off. The upper reservoir is maintained at the same level to run a peak load plant at peak hours again. When a developed machine designed to work as turbine as well as pump, additional cost of pumping can be saved. Similarly, generator cum motor is if provided instead of separate motor to run the pump and the generate electricity from the available mechanical energy of the pump, load factor of the plant can be increased to a major extent.

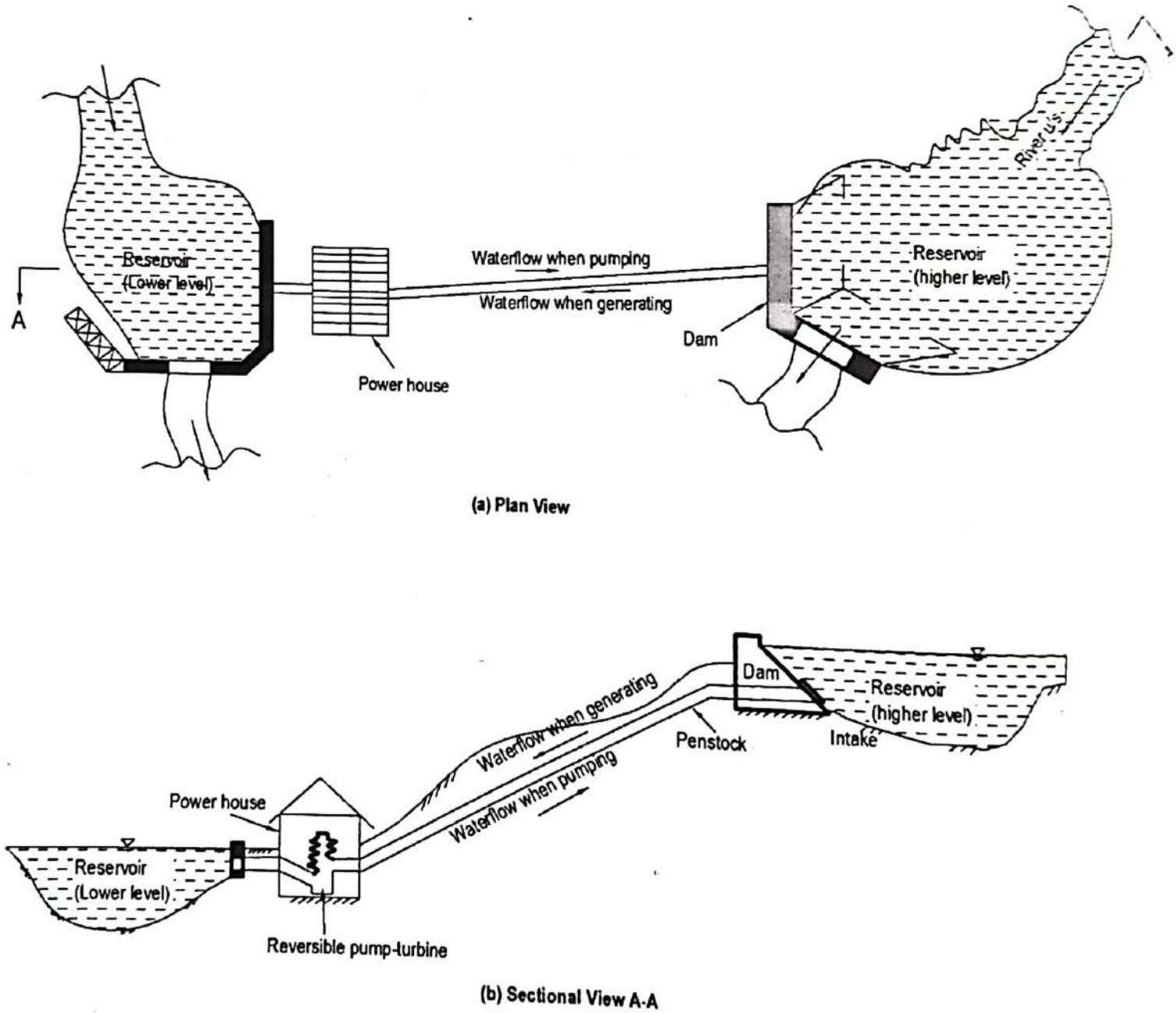

Figure 2-7 : Plan and sectional view of pumped storage plant

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Advantages of pumped storage plant

1. As compared to other peaking units, pumped storage plants have low capital cost and are thus an economical source of peaking capacity.
2. In the pumped storage plant, the pumping is done when there is spill energy (during night time) and power generation is done at peak time. The power required for pumping is available at a cheaper rate (slack hour's rate), while the power produced by the plant can be sold at a prime rate (peak hour rate). The relatively low hydraulic efficiency is partly compensated due to this fact.
3. Pumped storage plants allow a great deal of flexibility in the operational schedule of the system.
4. The pumped storage plant is a rugged and dependable as conventional hydropower station and can peak up load rapidly in a matter of few minutes.
5. Pumped storage plant is entirely free from effects of environmental pollution. (No air and water pollution)
6. Pumped storage plants are readily adaptable to automation as well as remote control.
7. The pumped storage plants allow the entire thermal and nuclear power generation to take up the base load. Thus the load factor of these units improves giving rise to overall greater system efficiency.

Types of pumped storage plants

According to the arrangement of upper and lower reservoir

1. Both the reservoir on a single river, in a tandom chain manner. Main dam create main reservoir and re-regulating dam downstream of the main dam creates the secondary storage.
2. Two reservoirs in two separate rivers close to each other and flowing in different elevation.
3. Higher reservoir an artificially constructed pool with the help of dykes all around, on a high level plateau or on a leveled hill top and lower level on a natural river.
4. The lower reservoir is a natural lake and while higher reservoir is artificial.

According to the mode of operation

1. Pure pumped storage plant
2. Mixed pumped storage plant

A pure pumped storage plant is a closed cycle plant with the volume of water flowing to the lower reservoir being equal to the volume pumped to the higher reservoir, while in mixed system partial volume is pumped to the higher reservoir.

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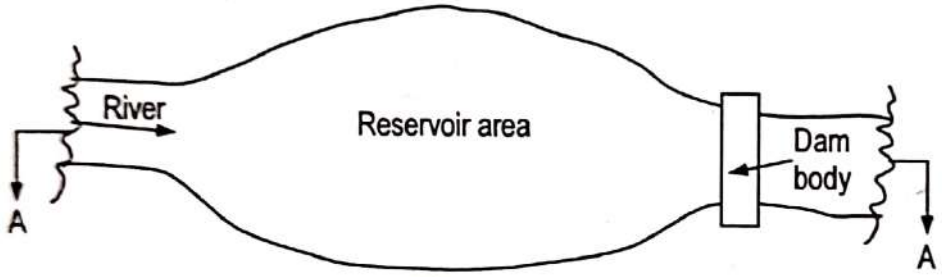
5 RESERVOIR PLANNING

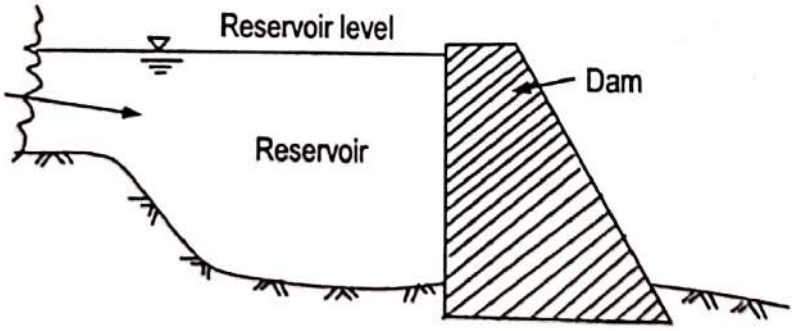
5.1 Introduction

When a barrier is constructed across some river in the form of a dam, water gets stored on the upstream side of the barrier, forming a pool of water, called a dam reservoir or an impounding reservoir or a river reservoir. Any collected water in form of pool or lake, may be termed as reservoir.

Dams and reservoirs are the most important elements of any multipurpose river basin development or development of large storage hydropower projects. Dams and reservoirs are complementary to each other. Reservoir can be developed only by constructing dam and conversely when a dam is constructed reservoir is bound to develop. Dam and reservoir planning such as selection of site for dam and reservoir, fixation of reservoir capacity, safe yield of storage and other connection works has very important role for development of multipurpose project. The plan and sectional views are shown in figure 5-1 below.



a. Plan



b. Section A-A

Figure 5-1 : Plan and section of reservoir upstream of dam

5.2 Classification of reservoir

It has two main purposes

a. Flood control storage
b. Storage or conservation storage

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The aim of flood control storage is to store some of the flood water of river when discharge rate at a stage likely to causes damage downstream of the valley and release this food storage slowly at a safe rate when flood subsides.

Conservation storage to store water for different purpose like

a. Hydropower
b. Irrigation
c. Public and industrial water supply
d. Navigation
e. Soil conservation and land reclamation
f. Fishery and wild life
g. Recreation
h. Growing useful aquatic life
i. Debris control
j. Sediment control of river etc

Thus a reservoir is called **single purpose reservoir** if it serves only one purpose and is called **multipurpose** if it is used for almost all the purposes mentioned above or more than one purpose.

A reservoir is called a **distribution reservoir** without a dam in which the water from river is pumped into some concrete tank constructed at suitable places. Water stored in those reservoirs is mainly used for supplying to the consumers though distribution networks.

The flood control storage discussed above may be retarding type or detention type. In retarding type, it simply retards the flood water without control like gates or spillway, whereas in detention type it detains major portion flood water with the help of gates, etc. and releases the extra water when flood subsides.

5.3 Location of site for reservoir

In selection the site for a reservoir the following points are to be considered:

a. It is located in the area of minimum percolation and maximum runoff (Sufficient inflow).
b. Leakage in the selected area should be minimum to minimize the grouting works.
c. It should not be located on highly permeable rocks like shale's, slates, gneisses, granite, etc.
d. Suitable dam site with water tight rock base should be available in the located area.
e. To reduce the length of the dam, narrow opening of the basin is essential.
f. Site should be easily accessible by road and if required to construct them, cost of construction should be minimum
g. Topography of the location should have adequate capacity without submerging excessive land and other properties.
h. Reservoir location should be free from objectionable minerals and salts.

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i. Site should exclude water from tributaries solution of minerals and salts.

j. Located area should provide sufficient water depth with smaller water area. Higher depth provides lower submerged area/unit capacity and decreases the possibility of weed growth. Smaller surface causes less evaporation losses.

k. Construction materials for the dam and other allied works should be locally available.

l. Suitable area near the location should be available for construction of staff quarters, labor colonies, godowns, stack yards, etc.

m. The site should be selected after investigating, so that cost of construction should be economic.

n. Heavily silt laden tributaries should not lead their discharge to the reservoir.

o. Less submergence of habitat area, fertile land etc

5.4 Determination of storage capacity

The following two methods are most reliable for determination of storage capacity reservoir:

5.4.1 By contour map

A contour map of entire area as shown in figure 5-2 is required to estimate the storage capacity. If the reservoir area is large enough, a ground survey requires a considerable time and expenditures. In such big reservoir, aerial survey is used. A contour map of a reservoir with areas A_1, A_2, A_3.....A_n between two contours is shown in figure 5-2.

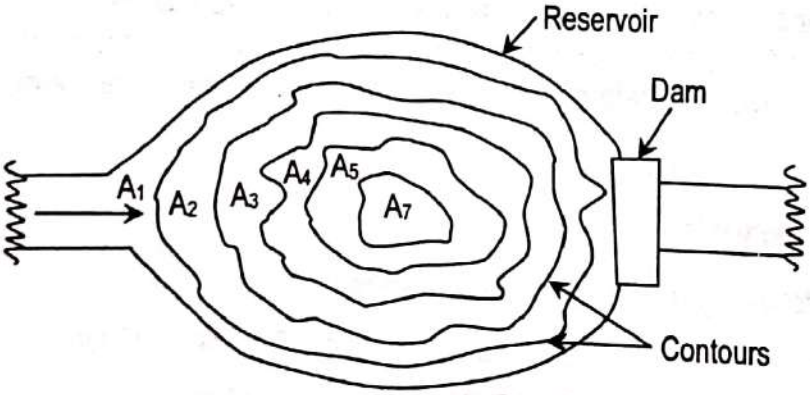

Figure 5-2 : Contours of reservoir

A contour interval of 0.5 m is taken for moderate reservoir and 1.5 m for a high dam. Thus, a very accurate triangulation survey is required with required contour interval to a scale of 1/1000 and preferable 1/500. From the contour map of reservoir area, water spread area of the reservoir at any elevation is directly determined by measuring the area at the contour by planimeter.

Elevation against water spread area may be plotted. Capacity may be measured by taking contour at equal intervals and totaling by the trapezoidal rule or by Simpson's rule or by cone formula.

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$$V = \frac{h}{2}(A_1 + A_2)$$ Trapezoidal rule

$$V = \frac{h}{6}(A_1 + A_2 + 4A_m)$$ Simpson's rule

where A_m, is the area of a contour line between midway A_1 and A_2

$$V = \frac{h}{3}\left[A_1 + A_2 + \left(\frac{A_1 + A_2}{2}\right)\right]$$ Cone formula

V is the volume between two adjacent contours at an interval of having areas A_1 and A_2. With the above formulae volumes between the successive contours A_1 and A_2, A_2 and A_3 ..... may be determined. Adding all these volumes calculated between successive contours, total capacity or volume of the reservoir can be determined.

5.4.2 Determination by mass curve of inflow and demand

Mass inflow curve (mass curve)

It is called Ripple mass curve after its originator W Ripple in 1983. Mass curve of inflow is a plot of accumulated flow volume against time, accumulated volume as ordinate and time as abscissa. The mass curve is used to determine

a. Reservoir capacity from uniform demand of flow
b. Safe yield from reservoir from reservoir capacity

A mass inflow curve is prepared from the inflow hydrograph of a river for a large number of consecutive years. From this flood hydrograph, ordinates of the mass curve (accumulated inflow volume) at any time can be obtained when accumulated inflow volumes (in cumec month, or ha-m or 10^6 m^3) are plotted against time, mass curve is obtained.

The discharge ordinates represent the annual average discharge (i.e. average discharge of the full year, obtained from the total, annual volume of stream flow by dividing it by (365 x 24 x 3600)

Mass curve is continuously rising curve as it shows the accumulated volume upto that time. If there is no flow during a certain period, the mass curve can be horizontal but it can never fall. The slope of the curvature at any point indicates the usable flow for power production. The slope of the curvature at any point indicates the rate of inflow at that particular time.

For many water resources investigations it is important to know the total volumes of water that have to be dealt with over long periods of time. For such purposes the most convenient means of presenting the runoff data is the mass curve. When it is desired to use water at uniform or nearly uniform rate greater than the minimum discharge in the stream, it is necessary to provide storage where water is impounded during periods of high flow for use during periods of low flows. Mass curve serves as a very useful tool to determine the required storage capacity for any uniform rate of demand.

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A graph of the cumulative value of any hydrologic quantity, generally as the ordinate, plotted against the time as abscissa is known as a mass curve. When cumulative volume of runoff is taken as the hydrologic quantity, the mass curve may be called of low mass curve or simply a mass curve. It is the integration of the hydrograph and therefore represents the area under the hydrograph from one time to another. Mathematically, the mass curve is expressed by

$$V(t) = \int_0^t Q(t).dt$$

Where, V (t) = cumulative volume of flow up to time t from the start of the record, that is, the ordinate of the mass curve at any time t.

Q (t) = the discharge as a function of time, i.e., the ordinate of the hydrograph, and

t = a dummy variable.

The difference in the ordinates of any two points on the curve is the summation of the flow during the intervening period of time.

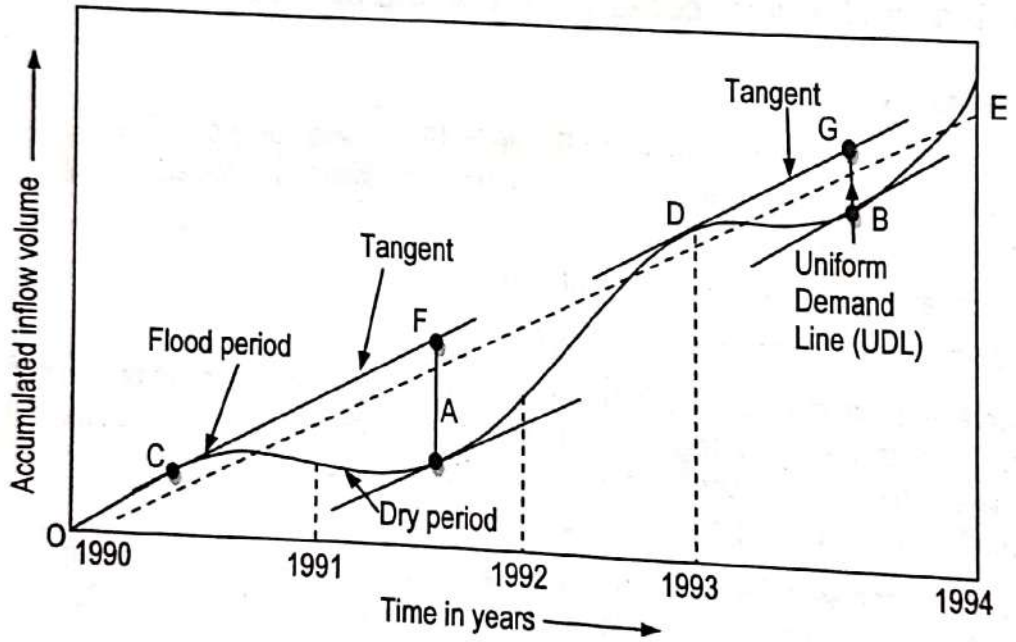

Figure 5-3 : Mass curve of inflow

Characteristics of mass curve

- The mass curve is a continuously rising curve as it shows the accumulated inflow volume with time.
- The slope of the curve at any point indicates the rate of inflow at that particular time.
- If the curve becomes horizontal at some point, it indicates that rate of flow at that particular time is zero. But the mass curve never fall down.
- If the curve rises sharply, it indicates the high rate of inflow within that period.

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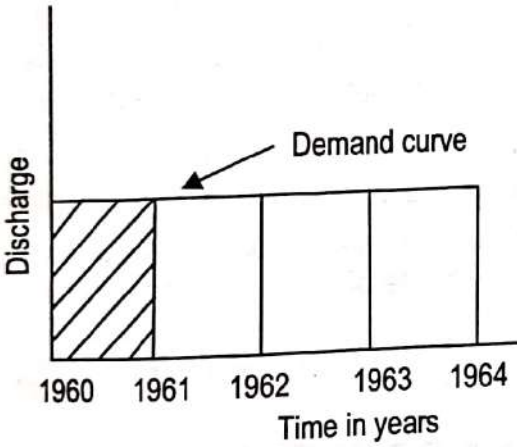
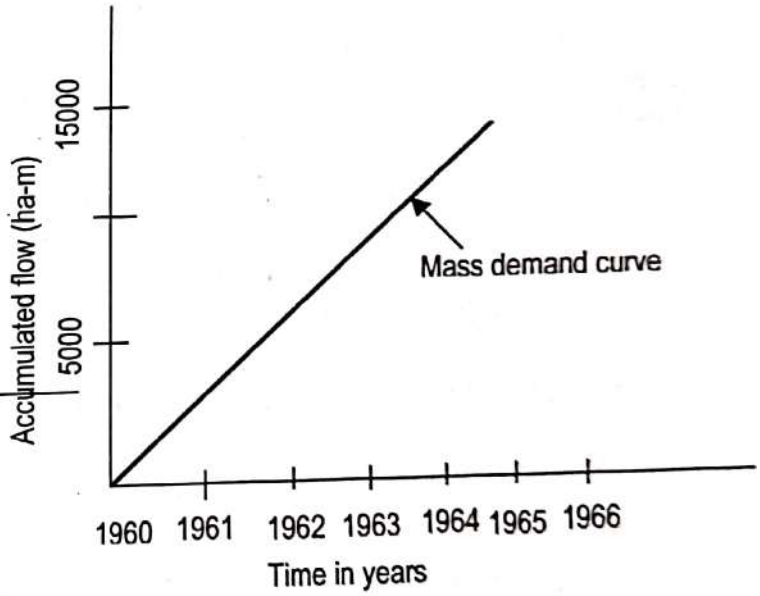
- Relatively convex rise and concave depression indicate flood and dry period respectively. The points A and B in mass curve (figure 5-3) indicate dry periods. Whereas points C and D indicate flood or high flow.

Mass demand curve

A demand curve is a plot between the demand rate as ordinate and the time as abscissa. If the demand is uniform, the demand curve is horizontal line, and the corresponding mass demand curve is straight line, having a slope equal to the demand rate. On the other hand, if the demand is variable, the mass demand curve is a rising curve with different slope at different time interval.

- The dotted line OE indicates uniform demand line (UDL), slope of which gives the uniform rate of demand.
- If OE is the uniform demand line, tangents at highest point C and D are drawn parallel to OE.
- The maximum vertical intercept between the mass curve (Here AF) gives the reservoir capacity.





Reservoir capacity by graphical method, using mass curve of inflow and demand

Using mass curve, the storage required for uniform flow can be calculated by the following steps

1. Draw the mass curve of inflow (mass curve)
2. Draw the demand line at the demand rate slope (mass demand curve)
3. From peak, draw tangent parallel to demand line
4. Maximum vertical ordinate between the mass curve and tangent gives the storage capacity of the reservoir that should be provided.
5. If the demand curve is not uniform it shows a non uniform demand
6. Determine the vertical intercepts AF, BG, etc which indicates the volume by which the

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inflow volume short of demand.

Determination of yield of a reservoir

Tangents are drawn from peaks such that the maximum vertical distance of any tangent from the mass curve does not exceed the given reservoir capacity. The slope of these lines gives the safe yield of the reservoir for that period.

The tangent when extended must intersect the mass inflow curve; otherwise the reservoir will not be full again. The flatest slope indicates the firm dependable yield from the reservoir.

5.4.3 Analytical method (Sequent peak algorithm)

In the mass curve analysis, the reservoir is assumed to be full at the beginning of the dry period and d the storage required to pass the dry period is estimated. If the mass curve contains only one ridge point and if there is no well defined subsequent trough point, it may be necessary to repeat the given data for one more cycle to arrive at the desired storage requirement. Also, the demand rate is invariably non-uniform due to seasonal variations in the demand. The sequent peak algorithm is a simple and straightforward analytical procedure leading to faster determination of the storage requirement. Unlike in the simple mass curve, it can easily take care of variable demands. Evaporation and seepage losses from the reservoir which are not considered so far can be easily taken care of by adding them to the demands when reasonable estimates of them are available.

a. Calculate the net inflow into the reservoir (V_i-D_i) for all the time periods i =1,2,.....,2N, where N is the number of periods of the data series and V_i for $i = N+1$, 1, to 2N are same as V_i for i=1 to N. That is, the given data is repeated once again to have a series of length 2N.V_i and D_i denote the volume of runoff and volume of Demand in the ith period.

b. Calculate the cumulative net inflow volumes, namely $\sum_{i=1}^{t}(v_i - D_i)\, for$ t = 1,2....., 2N.

c. Locate the first peak (local maximum) P_1, in the cumulative net inflows.

d. Locate the sequent peak P_2 which is the next peak of greater magnitude than the first.

e. Between this pair of peaks find the lowest trough (local minimum) T_1 and calculate $(P_1 - T_1)$.

f. Starting with P_2, find the next sequent peak P_3, that has magnitude greater than P_2.

g. Find the lowest trough T_2, between P_2, and P_3, and calculate (P_2- T_2).

h. Starting with P_3, find P_4 and T_3; calculate (P_3 - T_3).

i. Repeat the procedure for all sequent peaks available in the 2N period and compute (P_j- T_j) for all j values.

j. The required reservoir storage is given by S = Maximum of all (P_j - T_j) values. It may be noted that the sequent peak algorithm is nothing but the numerical adaptation of the graphical procedure of the residual mass curve using two cycles of the given data series.

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Solved Examples 5-1

The monthly runoff volumes in million m³ for a period of 24 months (two water year) recorded at a stream gauging site are 3, 6, 16, 30, 18, 15, 10, 8, 6, 4, 3, 12, 5, 17, 28, 20, 15, 12, 7, 5, 4, 3 and 2.
(a) Determine the size of the reservoir proposed at the gauging site if it is to maintain an assured supply of 5.33 million m³ per month. The water year may be taken as June to May.
(b) Obtain the storage requirement of reservoir without the use of the mass curve or (using sequent peak algorithm)

Solution

From the given data the cumulative runoff volumes are calculated as shown in Table. The mass curve is plotted as shown in figure.

Month	Runoff volume million m³	Cumulative runoff volume million m³
June	3	3
July	6	9
Aug	16	25
Sep	30	55
Oct	18	73
Nov	15	88
Dec	10	98
Jan	8	106
Feb	6	112
Mar	4	116
April	3	119
May	1	120
June	2	122
July	5	127
Aug	17	144
Sept	28	172
Oct	20	192
Nov	15	207
Dec	12	219
Jan	7	226
Feb	5	231
Mar	4	235
April	3	238
May	2	240

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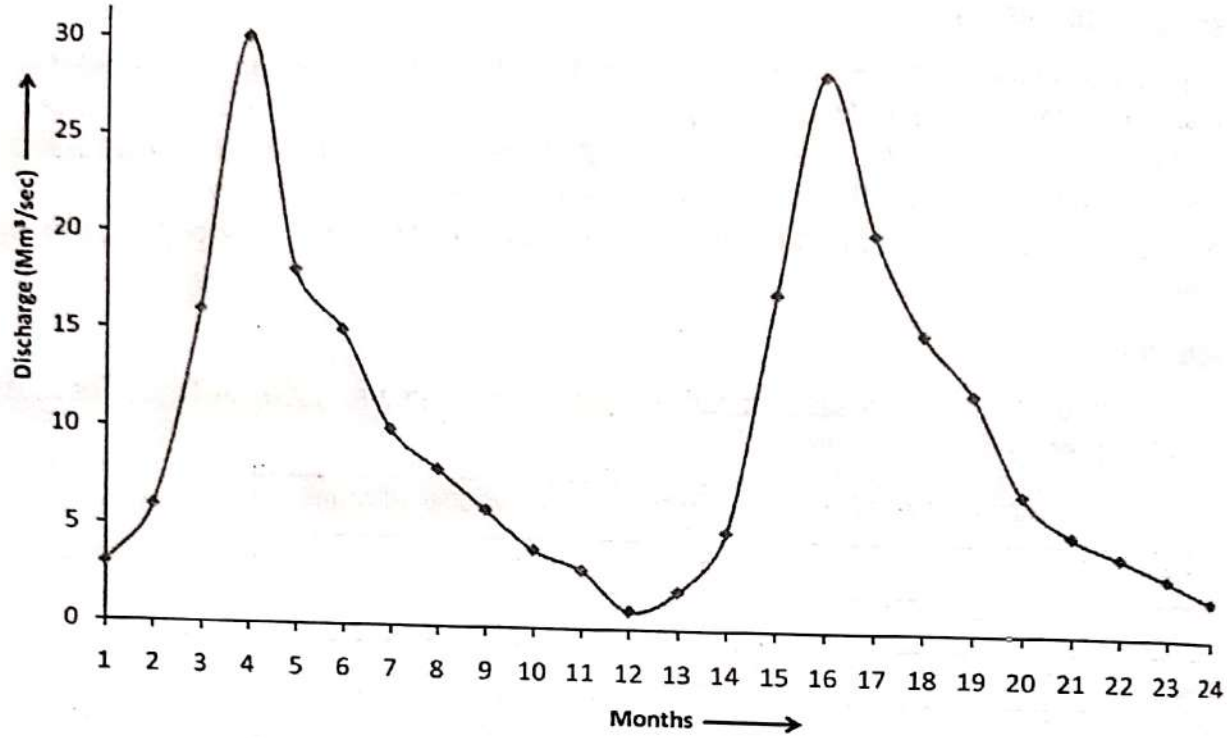

Figure 5-4 : Monthly hydrograph

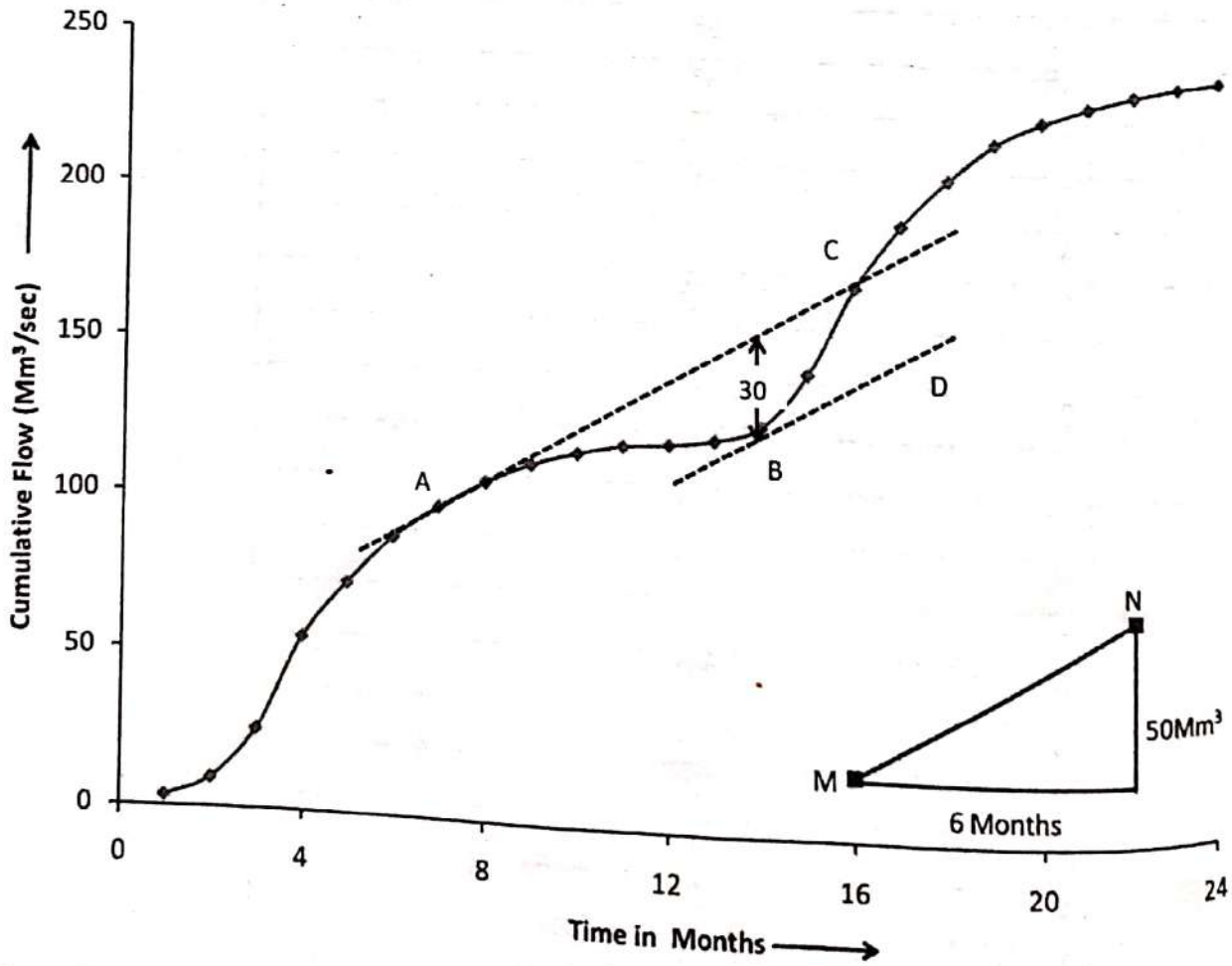

Figure 5-5 : Mass curve analysis

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The line MN in the inset indicates the draft rate of 50 million m^3 per 6 months or 8.33 million m^3/ month. Tangents AC and BD are drawn to the mass curve at the peak point A and the trough point B. These two tangents are parallel to MN. The vertical intercept between these two tangents, which is 30 million m^3 in the present case, is the required storage capacity.

(b) When the runoff volume in any time period is more than the demand in the same time period, there is surplus water. This surplus water either goes to build up the storage or it will be spilled if the reservoir is already full. Therefore nothing is dawn from the reservoir during the surplus period. When the runoff in the stream is less than the demand, then there is a deficit. Water is drawn from the storage to meet the deficits. Starting from the beginning of the dry period, if the deficit in each time period is found out and if these deficits are added, the calculative deficit at the end of dry period denotes the maximum amount of water drawn from the storage and the capacity of the reservoir must at least be equal to this.

Similarly, if the cumulative surplus is found out of the following surplus period it must exceed the cumulative deficit to ensure that the reservoir will be full again and we can also find out when this happens. Table below shows the calculation to determine the storage capacity by this method.

Storage determination by arithmetic calculations

Month	I	Runoff volume	Demand (V_i)	(V_i - D_i)	Cumulative deficit	Cumulative surplus
June	1	3	8.33	-5.33	-5.33	
July	2	6	8.33	-2.33	-766	
Aug	3	16	8.33	7.67		7.67
Sept	4	30	8.33	21.57		29.34
Oct	5	18	8.33	9.67		39.01
Nov	6	15	8.33	6.67		45.68
Dec	7	10	8.33	1.67		47.35
Ian	8	8	8.33	-0.33	-0.33	
Feb	9	6	8.33	-2.33	-2.66	
Mar	10	4	8.33	-4.33	-6.99	
April	11	3	8.33	-5.33	-12.32	
May	12	1	8.33	-7.33	-19.65	
June	13	2	8.33	-6.33	-25.98	
July	14	5	8.33	-3.33	-29.31	
Aug	15	17	8.33	8.67		8.67
Sept	16	28	8.33	19.67		28.34
Oct	17	20	8.33	11.67		40.01
Nov	18	15	8.33	6.67		46.68

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Month	I	Runoff volume	Demand (V_i)	(V_i - D_i)	Cumulative deficit	Cumulative surplus
Dec	19	12	8.33	3.67		50.35
Jan	20	7	8.33	-1.33	-1.33	
Feb	27	5	8.33	-3.33	-4.66	
Mar	22	4	8.33	-4.33	-8.99	
April	23	3	8.33	-5.33	-14.32	
May	24	2	8.33	-6.33	-20.65	

Table shows that the maximum cumulative deficit is 29.31 million m^3. Therefore the reservoir should have a capacity of 29.31 million m^3. This answer is differing slightly from what was obtained in the previous example using mass curve procedure. This is because of the curve drawn through the points of mass curve.

If the storage capacity of the reservoir is fixed as 29.31 million m^3 and if it is full at the beginning of the record, it will be depleted by 7.66 units by the end of July, full again by end of august, overflowing during September to December, empty by end of July, full again sometime October and so on.

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Solved Examples 5-4

The average inflows of a river corresponding to a driest year is shown below

Months	J	F	M	A	M	J	J	A	S	O	N	D
Inflow (m^3/sec)	110	90	70	50	30	25	65	220	300	190	115	110

Find (i) the minimum storage capacity of power generation at inflow discharge rate 85 m^3/sec. (ii) If the reservoir capacity is fixed at 270 cumec month what uniform rate of withdrawal is possible? Assume one month = 30.4 days. Solve by mass curve and analytical method.

Solution:

The flood hydrograph given in the problem may be written as months against volume and cumulative inflow or volume for plotting the mass curve. The discharge of every month in m^3/sec or cumec is multiplied by 1 month to convert the cumec into volume cumec month.

1 cumec month = $1 \times 30.4 \times 24 \times 60 \times 60 = 2{,}626{,}560\ m^3$

For example, discharge of January 110 m^3/sec (cumec) = 110 cumec month = $(110 \times 2626{,}560)\ m^3$

Again if 85 m^3/sec is drawn for power production in 12 months (January to December), then total volume drawn

= 85 cumes × 12 months = 1020 cumec month.

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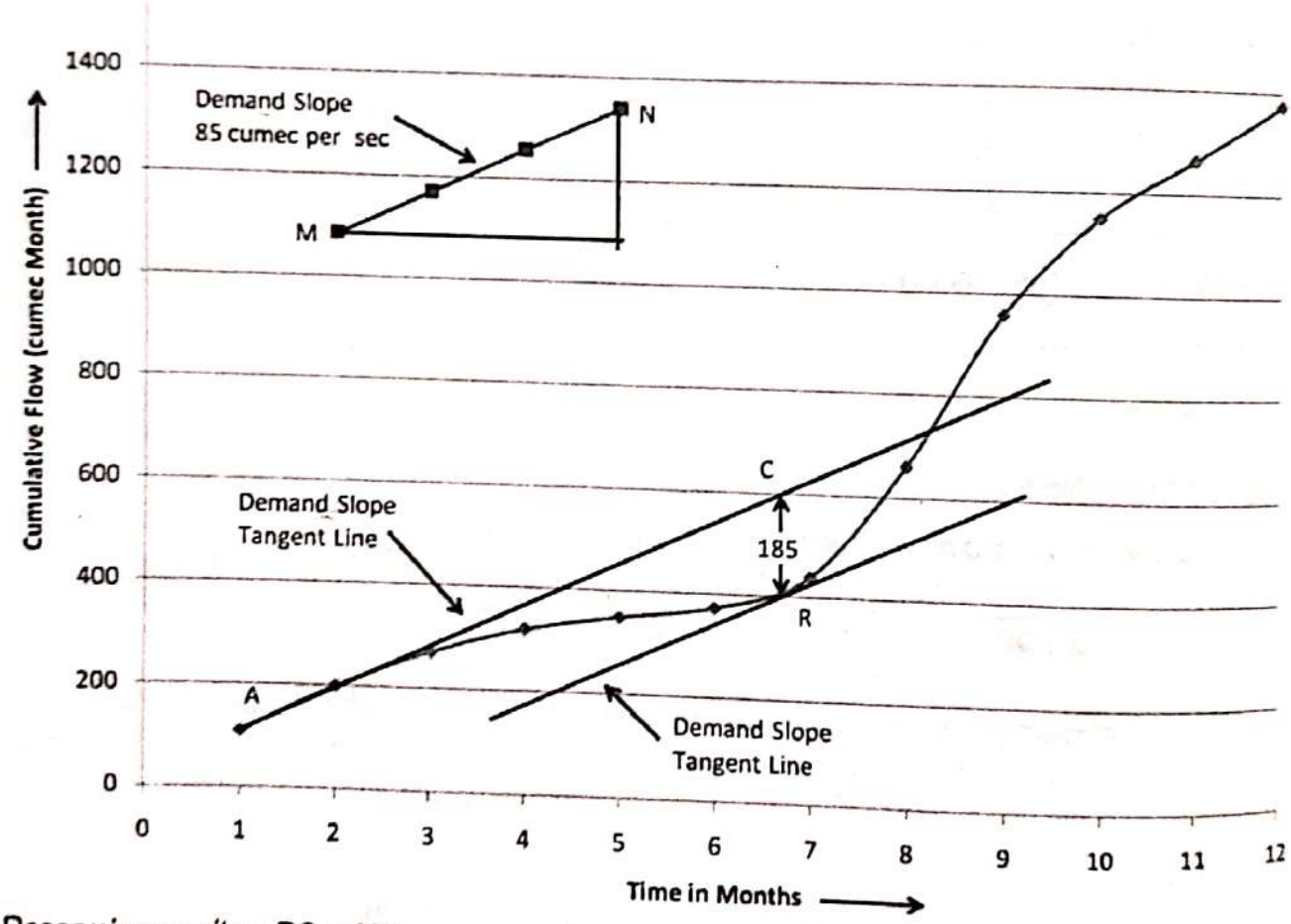
Months	J	F	M	A	M	J	J	A	S	O	N	D
Flow in cumec months	110	90	70	50	30	25	65	220	300	190	115	110
Cumulative flow in cumec month	110	200	270	320	350	375	440	660	960	1150	1265	1375

Plot the mass curve: Months against cumulative inflow is plotted to get mass curve of inflow. Uniform demand line or demand curve (UDL) corresponding to 85 m^3/sec is plotted as a line MN. The slope of the line MN gives the uniform demand.

Draw two tangents at A and R (point on convex and concave depressions, respectively), parallel to the demand line MN. The vertical distance between these two tangents i.e., RC = 185 cumec month is the required reservoir capacity.



Reservoir capacity = RC = 185 cumec month

= (185 x 30.4 x24 x 60 x 60) m^3

= 48,59,13.60 m^3 = 485.9136 x 10^6 m^3

= 485.9136 Million m^3

= 48591.36 x 10^4 m^3

= 48591.36 ha m

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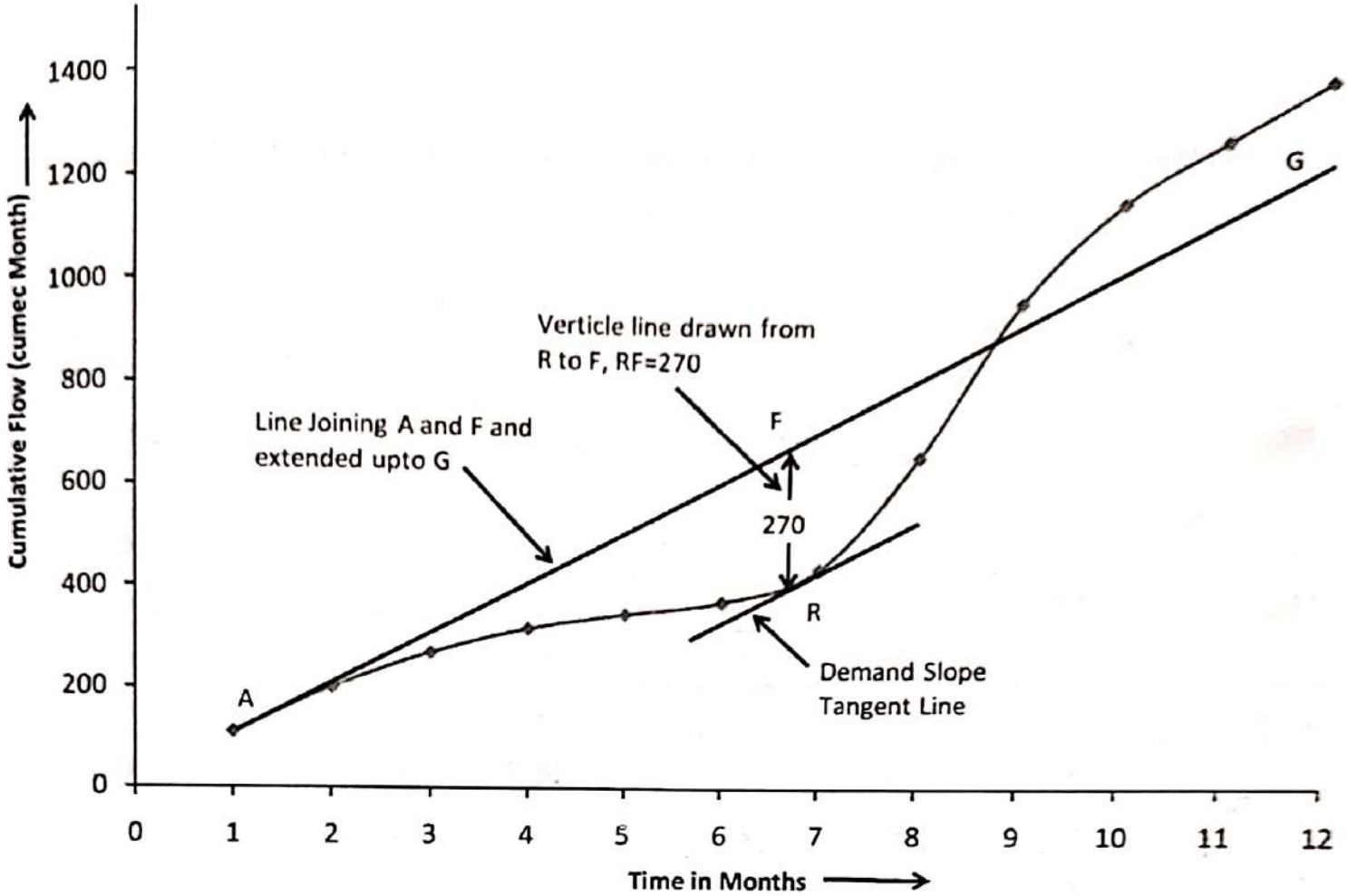

When reservoir capacity is fixed at 270 cumec month, from lowest point R, draw a vertical line RF = 270 cumec month to a scale.

Join A, F, and extend to point G slope of the line AFG is the uniform rate of withdrawal. (Note, the extended line AFG cut the mass inflow curve again so the reservoir gets full again at that point of intersection). From figure, the point G is intersected at 1200, so the cumulative demand of 1200 is for 12 months so demand rate is

$$Q = \frac{1200}{12} = 100 \ m^3 / \sec$$

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5.5 Zones of storage in reservoir

The storage in the reservoir are divided into some zones based on functions. Figure 5.6 shows the various zones of storage and levels of reservoir. The various zones are:

a. Surcharge storage
b. Useful or live storage
c. Dead storage
d. Valley storage
e. Bank storage

The various pool levels of the reservoir are:

a. Maximum pool level
b. Normal pool level
c. Minimum pool level

Maximum pool level: Maximum pool level is the level up to which water can be stored with the help of crest gates. During floods, water starts flowing over the spillways. But discharge over spillways increases with further rise in reservoir level. The maximum level attained by water during worst design flood is known as the maximum pool level.

Normal pool level: Normal pool level is the elevation of reservoir level which can rise under normal operating conditions. This level corresponds either to the crest level of the spillway or top level of the spillway gates. This level also known as conservation storage.

Minimum pool level: minimum level is the lowest water level which is to be maintained for minimum outflow or which has to be maintained in the reservoir under normal operating conditions is called minimum pool level. This elevation is guided by the lowest outlet in the dam or by minimum head required for efficient functioning of the turbines.

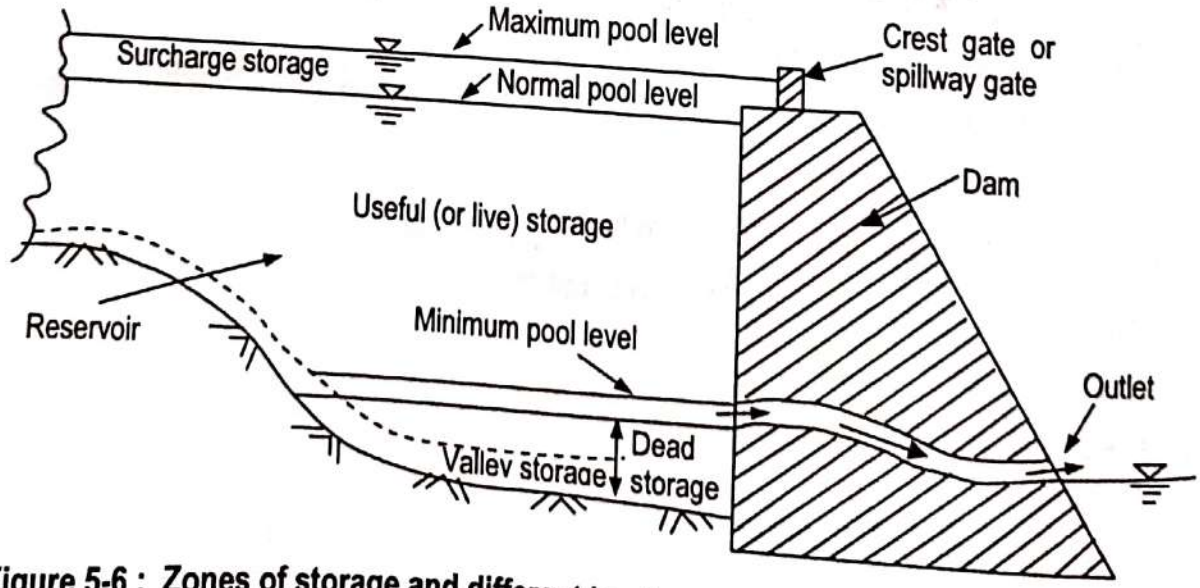

Figure 5-6 : Zones of storage and different levels of reservoir

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The storage between maximum and normal pool levels is called **surcharge storage**. It is actually uncontrolled storage during flood time and if needed it may be disposed downstream by overfall spillway when the crest gate is open. During heavy flood, a part of surcharge storage may be drained out over the gate. This storage cannot be stored for later use.

The storage between normal pool level and minimum pool level is called **useful storage**. This is mainly used for power production and for other uses if the project is multipurpose. In a multipurpose project, useful storage may be a combination of conservation storage and flood mitigation storage.

The storage below the minimum pool level is called dead storage. This is below the outlet and hence, this storage cannot be used for any purpose. Therefore, it is called dead storage.

The amount of water stored in the river before the dam is constructed is the valley storage. When dam construction is complete, reservoir is formed, storage goes on increasing. The net increase in storage is equal to storage capacity of the reservoir minus the natural valley storage.

When reservoir is full, a part of storage enters the soil below as seepage. The amount of seepage is dependent on soil condition. When reservoir level falls, this seepage from the permeable soil below comes out and contributes to the storage. This part of storage is called as **bank storage**.

5.6 Reservoir sedimentation

Reservoir sedimentation is a difficult problem economical solution of which has not yet been developed except by providing a bit greater dead storage to accommodate the sediment deposit during the life of the reservoir. River water normally carries certain amount of silt eroded from the catchment during heavy rain or due to the high velocity of runoff over the ground. The extent of erosion, i.e. silt or sediment load it carries depends on:

a. Nature of soil of watershed
b. Topography of the watershed
c. Vegetation cover of the watershed
d. Intensity of rainfall, i.e., higher intensity produces higher run-off or high velocity which erodes and carries silt along with water
e. Soil conservation and watershed management methods adopted in the catchment

Sheet erosion takes place on soft soil of the catchment resulting more sediment in river flow. Conversely hard soil is not eroded easily resulting less sediment. In steep topography, velocity of run-off is more and hence more sediment due to erosion. If the vegetation cover is more, both velocity and erosion are checked producing less sediment. Deforestation of the catchment and improper soil conservation of watershed add more sediment to the reservoir.

In any case, river water carries always a certain amount of sediment load. The sediment transported may be bed load or suspended load. When river water with sediment-laden water approaches the reservoir, the velocity decreases due to greater area. Due to this reduction of velocity, coarser particles settle in head reaches of the river and while suspended and finer

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particles take sufficient time to settle just upstream of the dam shown in Figure 5.7. A part of finer particles may pass through sluice ways to turbines or spillways.

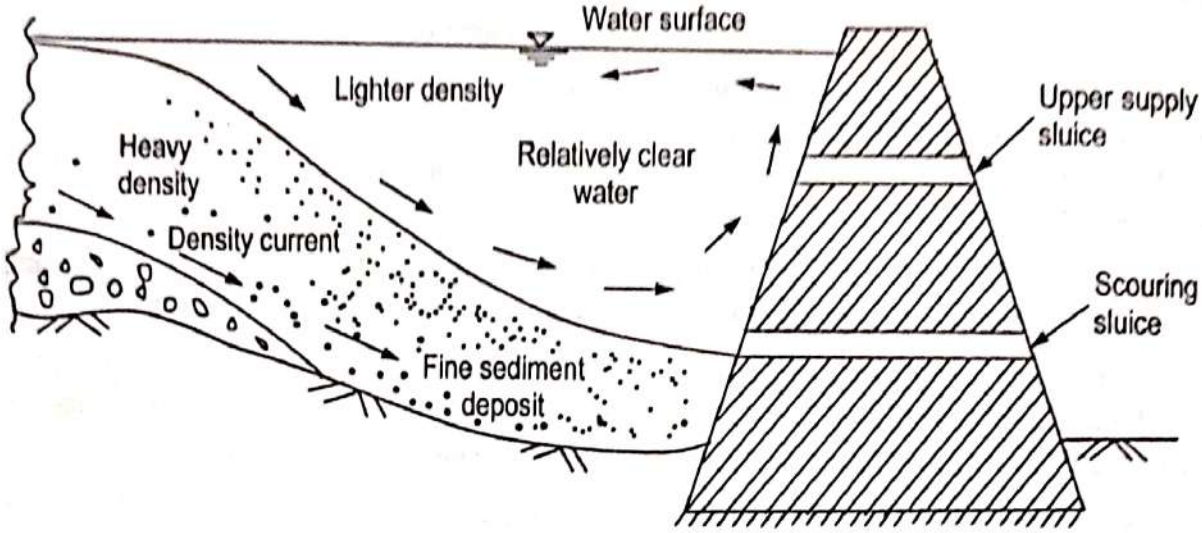

Figure 5-7 : Sediment deposit in reservoir and density current

5.7 Density currents (stratified flows) in reservoir

Density currents or stratified flows may be-defined as gravity flow of fluid under fluid of lighter density or over fluid of heavier density. Water stored in the reservoir is clear with lighter density, whereas inflow to the reservoir is muddy and the fluid upstream of reservoir have two different densities. The heavy turbid water flows along the channel bottom (shown in Figure 5.7) towards the dam under gravity slowly in the form of density currents or stratified flows. This is called reservoir stratification. Thus, in reservoir, it is the current or velocity of heavy density liquid with low velocity below a liquid of lighter density. The rate of silting or sedimentation may be reduced considerably if the density current is vented by proper location and operation of outlets called scouring sluices.

5.8 Control of sediment in reservoir

The following are some points to be considered to control sediment flow in the reservoir.

Proper selection of reservoir site

Reservoir should be located upstream of a tributary which carries large sediment load.

Control of sediment inflow

Series of small low dams in sandy river should be constructed to arrest the sediment upstream of reservoir.

Vegetation cover

The vegetation cover upstream of reservoir site and on the catchment area arrests a large sediment amount. Hence, proper care for growth of vegetation is necessary.

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Soil conservation methods

Soil conservation methods like cover and strip cropping, contour farming, crop rotations, control over grazing lands, terracing, benching on steep hill slopes, small embankments prevent erosion and arrest sedimentation.

Removal of deposited sediment

Sediments deposited may be removed be excavation, draining and flushing by sluices after disturbing the sediment by some mechanical and hydraulic method.

Removal by sluices during flood time

Sediment inflow with flood water is more. If sluices are provided at various levels of the dam. sediments in suspension may be removed by opening the sluices during flood time directly, thereby not allowing sufficient time to settle these sediments at reservoir bed.

Construction of dam in stages

Initially dam is designed for lower depth. When a part of it gets silted height of the dam is raised to prolong the life of reservoir.

Afforestation and control of deforestation

New plantations are planned in the catchment to extend forest cover to reduce erosion. Deforestation in the catchment should be controlled by the government so that soil in the forest will not be exposed to rainfall.

Construction of check dams

Construction of check dams in series in upstream tributaries of the river having huge sediment content. Also special treatment for landslide and debris flow area by constructing series of check dams in channel.

5.9 Effect of sediment on reservoir function

Loss of storage and service

It affects the useful life of reservoir and directly affects the services dependent upon the water storage.

Sediment deposition at outlet gate

Deposition of sediment is an important consideration in location of scouring sluices. It is difficult to locate the position without prior model study of the dam, reservoir in similar catchment.

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Aggradation of reservoir

Deposition of sediment upstream of reservoir is a common problem. These deposited sediments form an inflow section in the shape of a delta which reduces the inflow capacity to reservoir creating upstream of the river.

Degradation below reservoir

Outflow from the scouring sluices, overfall spillway has sufficient tractive or sediment transporting force. This tractive force causes initiation of motion of sediment downstream of the dam. Thus, degradation of the channel below the dam takes place.

5.10 Trap efficiency of reservoir

Trap efficiency is a measure of sedimentation of reservoir. It is defined as the percentage of ratio of sediment retained in reservoir to the sediment transported by the stream. It indicates the percentage of sediment deposited in the reservoir even inspire of taking precautions and measures to control its deposition.

$$E = \frac{\textit{Total sediment deposited in the reservoir}}{\textit{Total sediment flowing in the river}}$$

It is found by experimental analysis that this trap efficiency is 95 per cent to 100 per cent. It depends on the following factors

Capacity-inflow ratio

It is the ratio of reservoir capacity to the total inflow.

Silt gradation

Larger the silt gradation in size, higher is the trap efficiency.

Method of reservoir operation

Correct location of scouring sluices may decrease trap efficiency.

Shape of reservoir

When shape of the reservoir is diverging type towards the dam, deposition of sediment is more.

Age of reservoir

With increase in age, catchment becomes stable, sediment transportation decreases and hence, trap efficiency is less.

5.11 Capacity inflow ratio against trap efficiency

Capacity-inflow ratio is the ratio of capacity of reservoir to the total inflow of water into the reservoir.

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i.e., Capacity-inflow ratio = $\frac{Capacity\ of\ reservoir}{Total\ inflow\ into\ reservoir}$

This ratio is an important parameter of reservoir as the trap efficiency (η) is a function of capacity-inflow ratio.

If the reservoir is small and inflow is more, most of the inflow passes downstream as the detention time is small, allowing no time for sediment to settle. i.e., trap efficiency is very small. On the other hand, large reservoir allows more time for sediment to settle making trap efficiency large. Brunt in (1953), presented graphically a relationship between these two, i.e., trap efficiency and capacity-inflow ratio analyzing data from some existing reservoir. This graph is called trap efficiency vs capacity inflow curve (Figure 5-8).

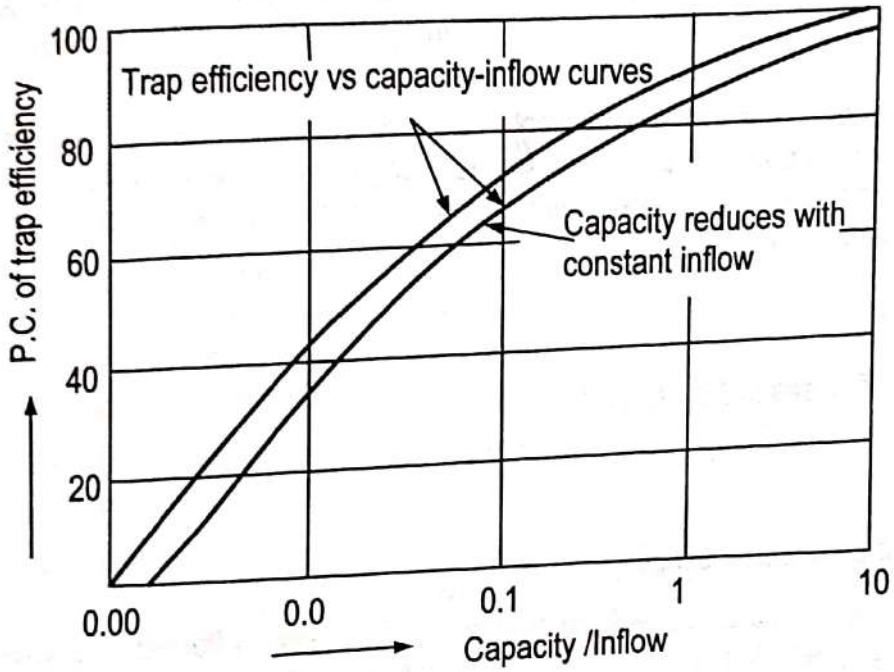

Figure 5-8 : Trap efficiency Vs capacity-inflow curve of reservoir

The figure shows if capacity is reduced and inflow remains constant, trap efficiency decreases, i.e., if inflow becomes more than capacity, sediment deposition becomes less, hence, silting rate in the reservoir shall be more in the beginning and as the capacity reduces with time silting rare decreases. Therefore, complete reservoir silting may take longer period.

5.12 Useful life of reservoir and its estimation

The dead storage in a reservoir is provided to accommodate the sediment. But when the sediment inflow is more, dead storage within few years gets fully silted. Even sediment begins to encroach the useful storage. This encroachment of sediment depends on (i) capacity-inflow ratio, (ii) sediment size and content in the flow, (iii) reservoir operation, (iv) characteristics of the valley, etc. The useful life of reservoir is said to exist till the useful storage is reduced to 20 per cent of the designed capacity. Reservoir sediment at the beginning is higher. When the whole catchment

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becomes stable, inflow of sediment decreases with years due to fall in trap efficiency, consolidation and shrinkage of sediment and formation of delta.

The estimation of useful life of reservoir is made by:

a. Working out the capacity-inflow ratio in percentage and finding annual loss of reservoir capacity.
b. Finding the correction factor for settlement and consolidation of deposited sediments which is about 15 per cent of the total over a period of 200 yrs.
c. Working out trap efficiency ratio for different reservoir capacity, which may be obtained from graphs or tables.
d. Ascertaining the total load for a given period, capacity curves are determined at intervals of 25, 50, 75, 100 years, etc. then check up the quantity of water in different zones of reservoir.
e. A plot may be developed to show the available capacity in different zones and total capacity against time period. From this plot, it becomes easy to evaluate the failure period of the project to meet the minimum basic demand as originally fixed and this period is taken as useful life of reservoir.

5.13 Reservoir losses

There are mainly three types of losses in the reservoir

- Evaporation losses
- Absorption losses
- Reservoir leakage or percolation losses

Evaporation losses: These are the major losses from a reservoir. This loss is affected by exposed surface of the reservoir, wind speed, temperature, relative humidity etc. this loss is expressed in mm of water depth and varies from place to place depending upon the local condition such as temperature, relative humidity and wind etc.

Absorption losses: This loss of water is considerable in the beginning but falls to small values after sometime when pores get saturated. These losses depend upon the soil forming the reservoir.

Reservoir leakage or percolation losses: Reservoirs being very large, their banks are permeable. But the permeability of the soil generally very low but the bank made of badly fractured rocks or having continuous stream of pores strata may causes serious leakage. So site should be properly investigated and such things should be rectified.

5.14 Reservoir regulation

It is defined as the rational distribution of a river flow in time and space among different field of water resource system such as irrigation, Hydropower, Recreation , Navigation, water supply etc

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Multipurpose reservoir need to be operated and regulated efficiently with a high degree of intelligence, institution, and experience, in order to ensure that they are neither left partially empty by the end of rainy season, nor they are found full at the time of arrival of a series of peak floods. Leafing to heavy releases, causing flood in the downstream valley.

Reservoir planning

Maximum water level (MWL): Maximum level to which the surface will rise when the design flood passes over the spillway

Minimum water level (MiWL): the lowest level up to which the water is withdrawn from the reservoir under ordinary condition. The storage below the minimum pool level is not useful and is called dead storage.

The volume of water between normal water level and minimum water level is useful volume and known as useful storage or live storage.

Water is utilized by two groups (i) consumer groups, such as water supply, irrigation and (ii) Non consumer groups, such as hydropower, navigation.

Rule curve or Guide curve: guiding tables and curves which are developed in advance, and kept ready for use for the efficient regulation of reservoir waters, with time, are called rule curve or guide curve. Such guiding curves are normally required only for flood season, because for the rest year, the reservoir will only discharge water for hydropower, irrigation and other purpose.

Multipurpose reservoir (irrigation, hydropower, flood control) is operated considering following reservoir regulation principles:

- The water level in the reservoir will normally fluctuate between the normal pool level and minimum pool level. The storage between these two limits of water level, will be maximum conservation of water which is sufficient for satisfying the irrigation need.
- As water level in the reservoir shall not be allowed to go below the minimum pool level, the firm commitments for power will continue to generate upto this level of water.
- Normal floods will be absorbed in the capacity laying below the normal pool level. Severe flood shall be absorbed by the capacity lying between normal pool level and maximum pool level. Spillway starts discharging water, as soon as the water level starts rising above the normal pool level.
- Because of heavy outflow for irrigation purpose and hydropower generation during dry season, the reservoirs are almost in completely depleted state. Thus just before the monsoon season, almost full capacity is available in empty form, not only for conservation of water but also to control the flood that may occur during rains.
- During monsoon, the irrigation demand is almost nil and water will be released only for hydropower generation. The reservoir level will thus continue to rise steadily. The rise in water level is allowed to reach the normal pool level by the end of monsoon season.

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5.15 Dam height optimization

For fixing the dam height, area elevation curve and storage elevation curve can be developed from a contour plan which helps in fixing the maximum operating level of the reservoir and thus fixing the height of the dam.

Area elevation curve

A contour plan of the proposed area of reservoir is prepared. Considering from the bottom, let A_1, A_2, A_3.....A_n be the areas of successive contours. These areas can easily determine with the help of planimeter or using computer application such as Autocad, GIS. The value of area A1 is minimum at the base of the basin. As elevation of the contours increases, the area under them also increases. The area elevation curve is shown in figure below.

Storage elevation curve

The volume of storage corresponding to different contours can be calculated either by using prismoidal formula or trapezoidal formula. We have seen that area of the contours goes on increasing as we proceed from bottom of the reservoir towards top. Consequently volume of storage also goes on increasing as we proceed from bottom of the reservoir towards top. The volume of storage for different contours may be calculated and curve between storage and elevation can be plotted as shown in figure 5-9 below.

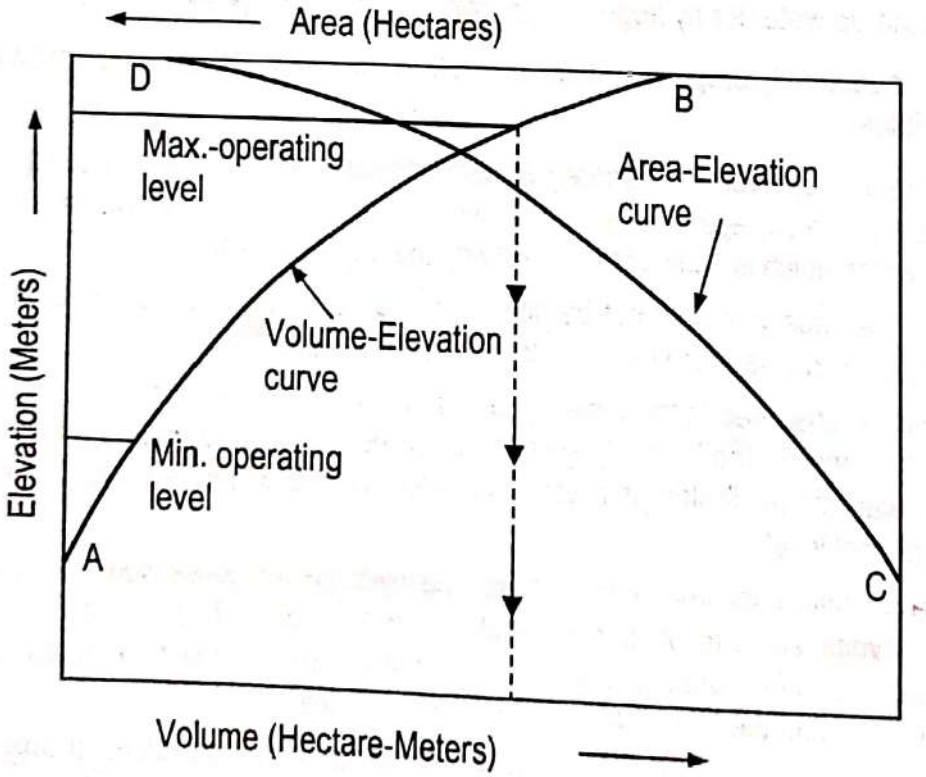

Figure 5-9 : Area elevation and volume elevation curve

The optimum or economic dam height is that height, corresponding to which the dam per unit of storage is minimum. This requires estimation of construction cost for several heights of dam and also corresponding storages in reservoir. For each dam height, the reservoir storage is known.

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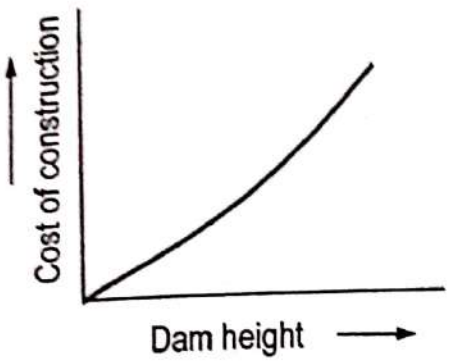
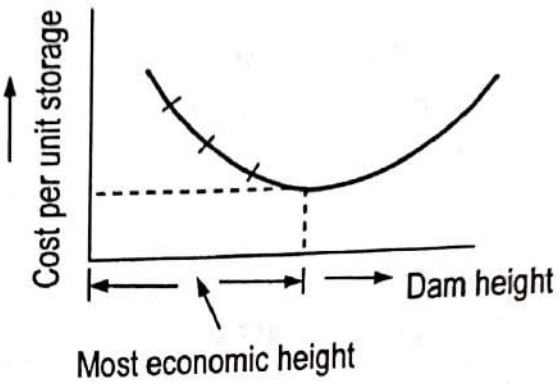
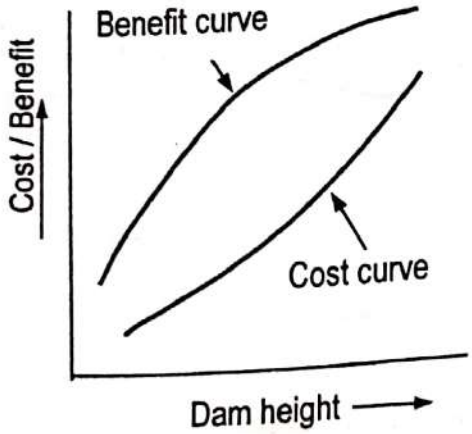
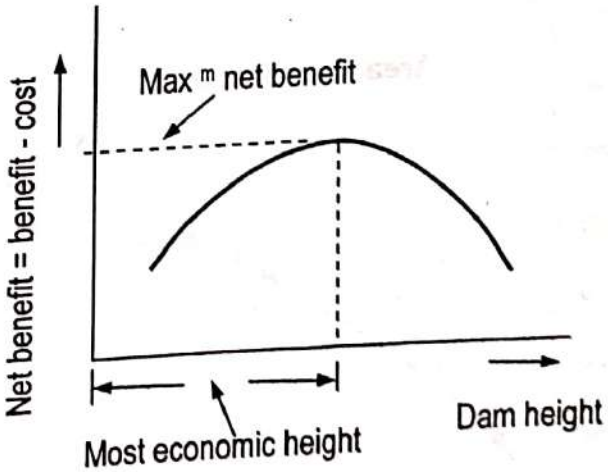
Draw the curve between dam heights and their corresponding costs of construction as shown in figure. Now, the construction cost per unit cost per unit storage for all the dam heights can be calculated and plotted as shown in figure below. The lowest point on this curve gives the height if dam for which the cost per unit of storage is minimum and hence adopted as the most economical.





Or the cost and benefit from the reservoir is calculated separately for different height of the dam. For cost part, increasing dam height, the length of the dam, cross section, inundation area increases. The increase in inundation area result in loss of land, forest, different cultural sites, build up area, infrastructure increases. Hence increasing height the cost of dam in totality increases. But increasing the dam height, the annual storage volume of water increases, more regulation of the river occurred, more flood control benefit achieved, other (irrigation, recreation, fisheries) benefit can be obtained. Hence increasing the dam height the benefit also increased. The net benefit (benefit – cost) from reservoir can be calculated for different height of dam can be calculated as shown in figure. The height of dam having maximum net benefit is called optimum dam height.





In a project, full supply level of the reservoir is 375m, tail water level of downstream is 220 m, from GIS analysis of the project, the area in square kilometer and volume elevation in 10^6 cubic meter is as shown below.

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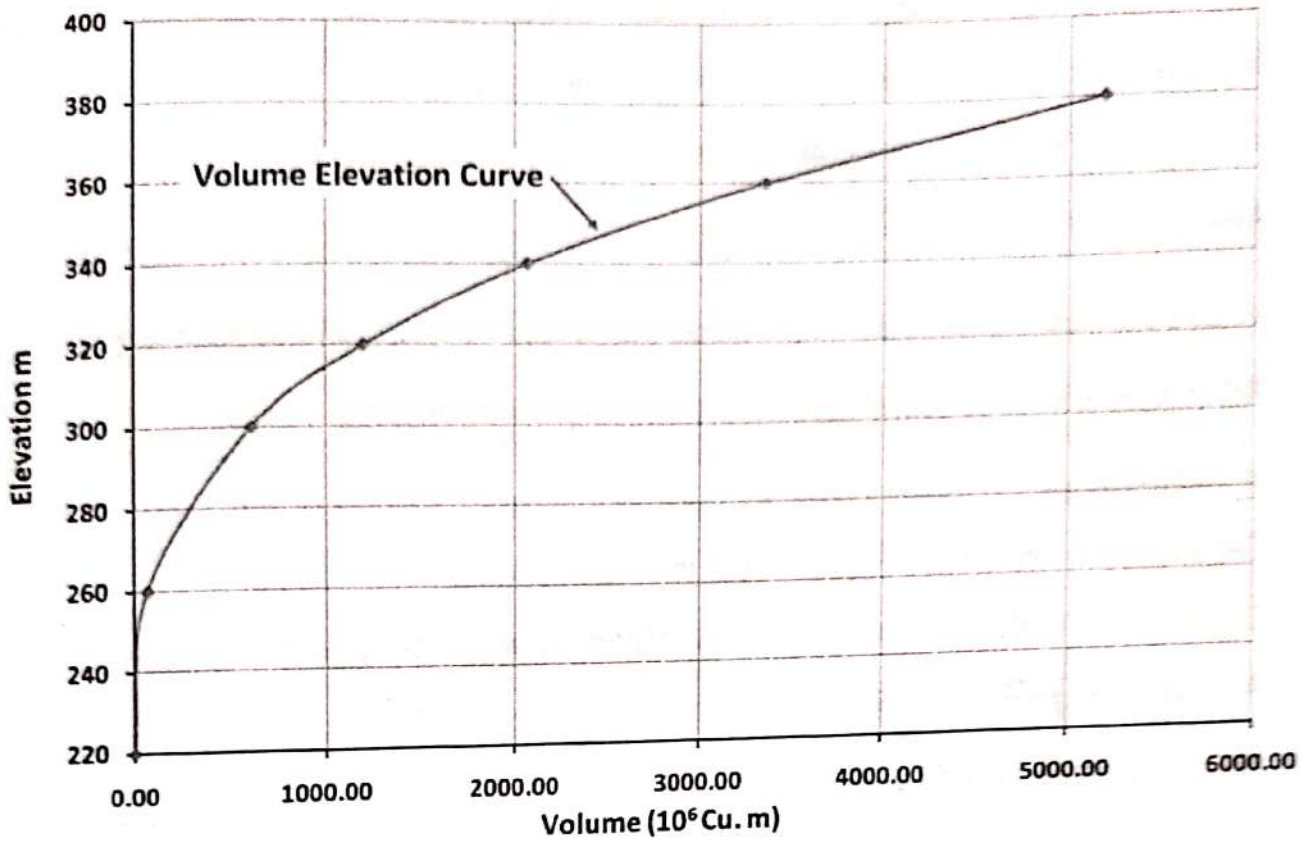

Solved Examples 5-6 :

A reservoir has the following available data. Find the probable useful life of the reservoir.
Reservoir capacity = 20 Mm^3 (Million cubic metre)
Average annual flood volume = 40 Mm^3
Annual sediment = 13.3333 x 10^4 tonnes
Specific gravity of sediment = 2.2
Life of reservoir terminates when 80 per cent of initial capacity is filled up.
Capacity-inflow ratio Vs trap efficiency in percentage hereinafter is given:

Capacity- inflow ratio	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.1
Trap efficiency (%)	86	92	94	95	96	96.5	97	97	97	97.5

Assume 20 per cent of the capacity is filled up in first interval.

Solution:

Annual Sediment =13.3333x10^4ton = (13.3333 X 10^7 X 9.81) N

Volume or annual sediment = $\frac{9.81X13.33.3333X10^7}{2.2X9.81X1000} = \frac{1}{16.5} X\ 10^6 m^3 = \frac{1}{16.5} X\ Mm^3$

20 per cent of reservoir capacity = 20 x$\frac{20}{100}$ = 4 Mm^3

Initial capacity-inflow ratio = $\frac{20Mm^3}{40Mm^3}$=0.5

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Corresponding trap efficiency given = 96 per cent = 0.96

Capacity-inflow ratio at the end of interval = $\frac{20-4}{40}$ = 0.4

Corresponding trap efficiency = 0.95

Average trap efficiency, = $\frac{0.96+0.95}{2} = 0.955$

Volume sediment deposited annually till 20 per cent is filled = $(\frac{1}{165} X 0.955)$ percent Mm^3

No. of years to fill 20 percent of capacity or 4 Mm^3 = $\frac{4}{\frac{1}{16.5} X 0.955} = 69.11\ years$

In the 2nd interval

Capacity-inflow ratio (initial) = $\frac{16}{40}$=0.4 and at the end = $\frac{12}{40} = 0.3$

Corresponding trap efficiencies are 0.95 and 0.94 respectively as per given data.

Average = $\frac{0.95+0.94}{2} = 0.945$

No. of years reqd. to be filled = $\frac{4}{\frac{1}{16.5} X 0.945}$

In the 3rd interval

Capacity ratios are 0.3 and $\frac{8}{40}$ = 0.2

Corresponding trap efficiencies are 0.94 and 0.92 respectively.

:. Average trap efficiency = $\frac{0.92+0.86}{2} = 0.89$

:. No. of years reqd. to be filled = $\frac{4}{\frac{1}{16.5} X 0.89}$ =74.15 years

Total probable life of reservoir (till filling of 80 per cent)

= (69.11 + 69.M + 70.96 + 74.15) years = 284.06 years

Solved Examples 5-7 :

In a reservoir type hydropower project of installed capacity of 250MW with dam height 180m (effective dam height = head) and overall efficiency 87%, has a reservoir volume of 3500 million cubic meters for its regulation period. Find the time taken for its regulation period if the regulated flow discharge continuously used for power generation.

Solution:

Regulated flow discharge, $Q = \frac{P}{9.81 x\ \eta x H} = \frac{250x1000}{9.81 x\ 0.87 x 180} = 162.73 m^3/sec$

Scanned with CamScanner

3. Power and Energy Potential study (6 hours)

3.1 Processing of hydrological data, Use of extreme and long term hydrological data, mass and elevation volume curves, flow duration curves

3.2 Gross and net head and estimation

3.3 Reservoirs and their regulation, need for flow regulation, Source of sediment, sediment yield in Rivers, sediment handing in reservoirs, life of the reservoirs

3.4 Methods of fixing installed capacity of a hydropower plant

3.5 Estimation of Power and energy potential

3.6 Mean and peak load, load curve, load factor, utilization and diversity factors

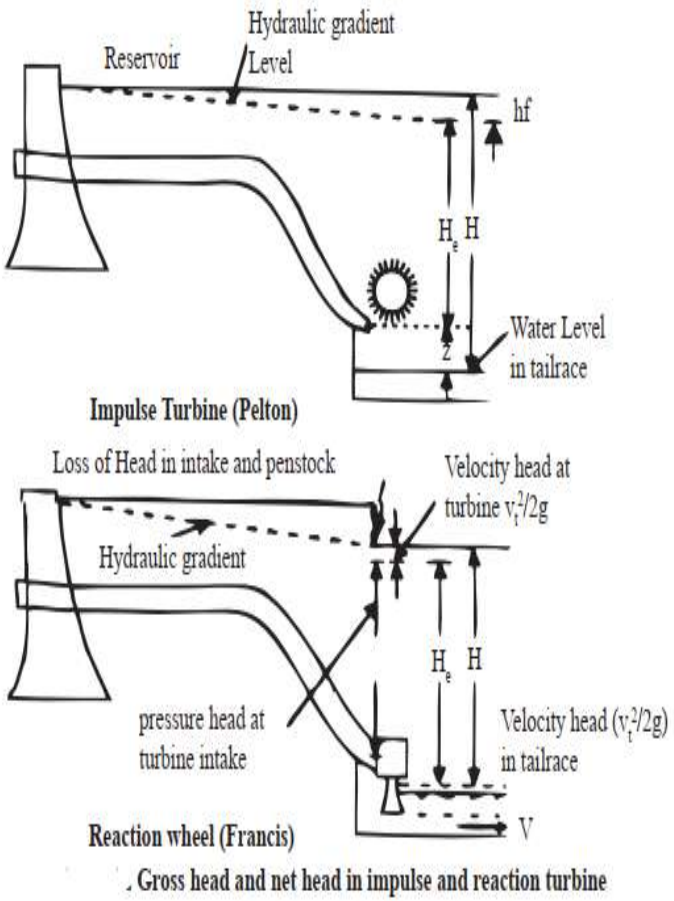
3.2 Gross and net head and estimation

- **Gross head and net head**
- **Gross head (Hg) :** Gross head is the difference in
- elevation between the water levels of the fore bay

and tail race.

Hydraulic losses: Losses due to friction, the entrance, expansion and contraction and exit losses.

Net head (HN): Net head is the elevation difference between the gross head and all hydraulic losses except those chargeable to the turbine. Net head is the head available for doing work on the turbine. Spiral case and draft tube losses chargeable to the turbine.



. Gross head and net head in impulse and reaction turbine

For impulse turbine,

- Net head = Gross head – sum of all head losses

For reactive turbine,

- Net head = Gross head –sum of all the head losses

+ suction pressure head

- **Maximum head (Hmax):** It is the *gross head* computed from the difference in elevation between fore bay level and tailrace level without spillway discharge.
- **Minimum head (Hmin):** It is the *net head* computed from the difference in elevation between minimum fore bay level and tailrace level with all turbines operating at full gate.

3.2 Gross and net head and estimation…..

Design head (Hd):

- It is the net head under which the turbine reaches its peak efficiency at synchronous speed.
- Usually it is the head difference between normal water level and minimum tail water level.

Power (P):

The electrical power generated from the hydro plant is the function of the discharge and head.

Power (P) =$\eta \times \gamma \times Q \times H$

Where, η = overall efficiency of the plant

γ = specific weight of the water, the value of γ = 9810 N/m3

H = head of the project in meters

Q = Available discharge in m3/s

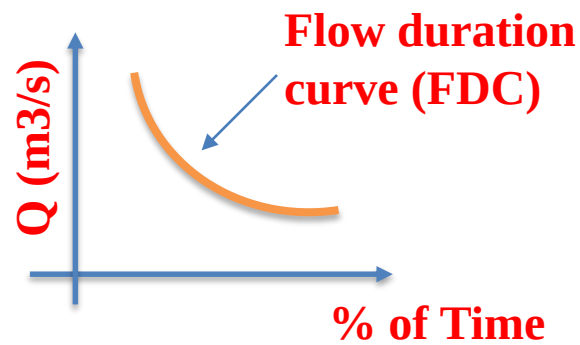
P= power generated in watts

3.4 Methods of fixing installed capacity of a hydropower plant

Flow duration curve (FDC) :

- *A graphical representation of runoff is the flow duration curve.* It has discharge plotted on the Y – axis and the percentage of time duration for which that magnitude (or more) is available on x – axis.
- Flow duration curve are used in assessing the dependability of the discharge. Basically these are used in assessing dependable power in runoff river plant with or without pondage.

Steps of flow duration curve



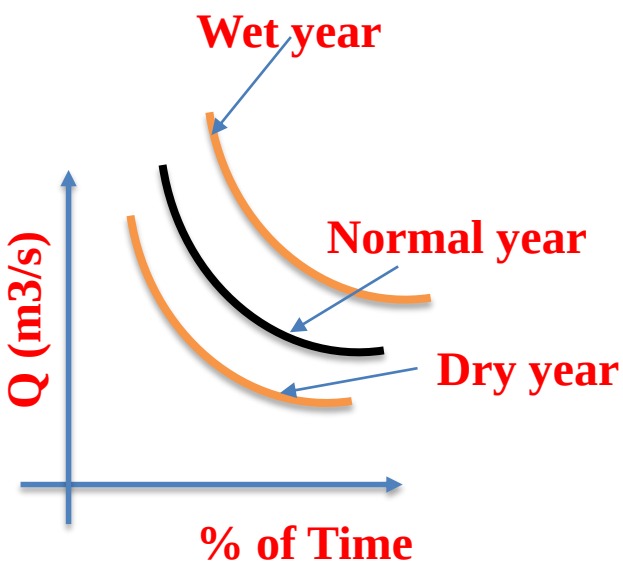
1. Calculate total number of data N
2. Rank 1, 2, 3, ……..N, represented by n
3. Compute frequency as f = N/n
4. Compute the probability exceedence as p = 1/f = n/N
5. Place discharge in descending order
6. Draw probability of exceedence vs discharge curve to get the flow duration curve.

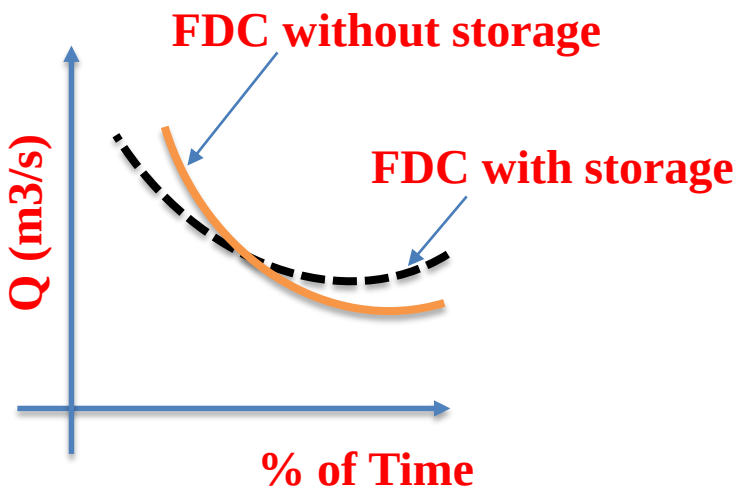
Uses of FDC:

- Highly useful in the planning and design of water resources projects (Determine the potential for firm power).
- If the sediment rating curve is available for the given stream, the flow duration can be converted into cumulative sediment transport curve by multiplying each flow rate by its rate of sediment transport. The area under this curve represents the *total amount of sediment transported.*
- A flow duration curve plotted to a 10g – log pater provides a qualitative description of the runoff variability in the stream.
- Use in the design of drainage systems and flood control.

3.4 Methods of fixing installed capacity of a hydropower plant

Flow duration curve (FDC) …





3.5 Estimation of Power and energy potential

Firm (Primary) power:

- It is maximum continuous power obtainable from a plant under the most adverse hydraulic conditions.
- It can be defined as the power which is always available to customer at any hour of the day and is therefore totally dependable power.
- The firm power corresponds to the minimum stream flow and is available for all times for run-of-river plant.

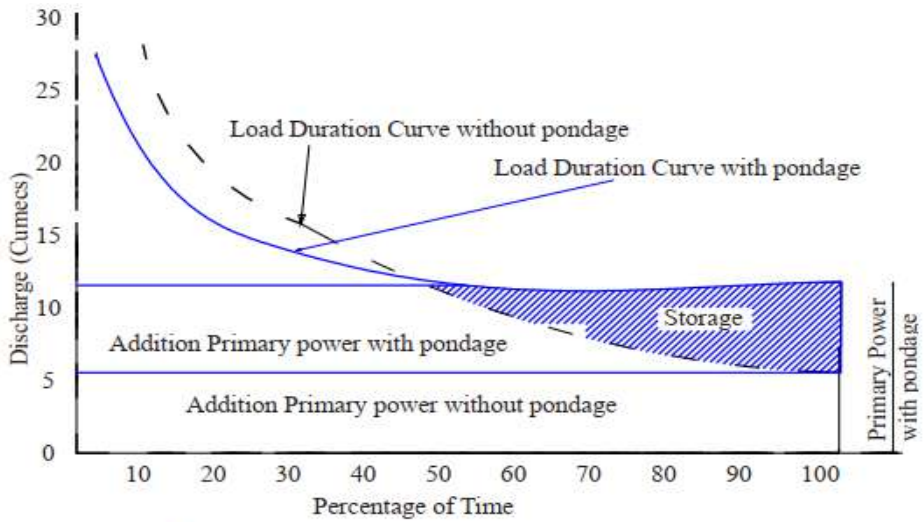


Fig. Primary and Secondary Power With and Without Pondage

- However, firm power can be increased if storage reservoir is provided.
- The firm power corresponds to the continuous 24 hrs flow available (100% of time).
- The firm capacity relies on the minimum stream flow during the peak load period as well as available storage, shape and size of linked load curve and the interconnection of other existing plants.

Secondary power:

- It is also known as surplus power which is the power other than the primary power.
- It is comparatively less valuable as it is available intermittently at unpredictable time.
- The plant can deliver the secondary power only for a part of the year (or for some percentage of time).
- In a run- of -river plant, the secondary power is available when the discharge in the river is greater than the minimum discharge.
- The secondary power is useful in an interconnected system of power plants.
- *At off peak hours,* the secondary power may be called upon to relieve the interconnected stations thus affecting economy.

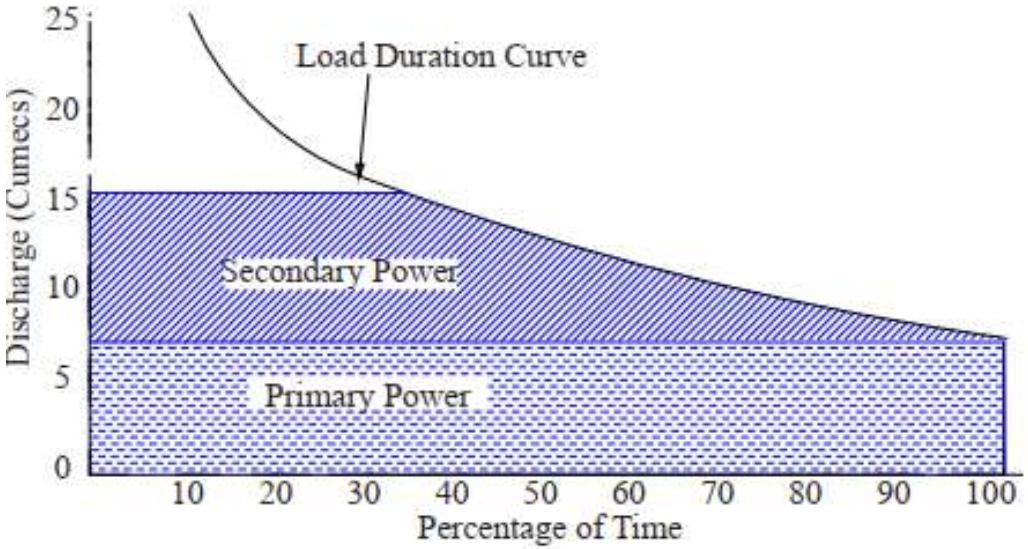


Fig. Primary and secondary power

3.6 Mean and peak load, load curve, load factor, utilization and diversity factors

Load Curves:

- The load on the power station is rarely constant; it changes from time to time.
- The daily variation in loads on the power station from time to time (hourly or half hourly) can be plotted on a graph taking time on x-axis and load on y–axis.
- The curve achieved in such a way is called ***daily load curve***.
- The *monthly load* curve can be drawn by calculating the average value of power at a particular time of a day with the help of daily load curve.
- Likewise the annual load curve can be obtained on the basis of monthly load curves of a particular year.
- Thus load curve is a graph of load consumption with respect to time and directly gives an indication of the power used at any time.

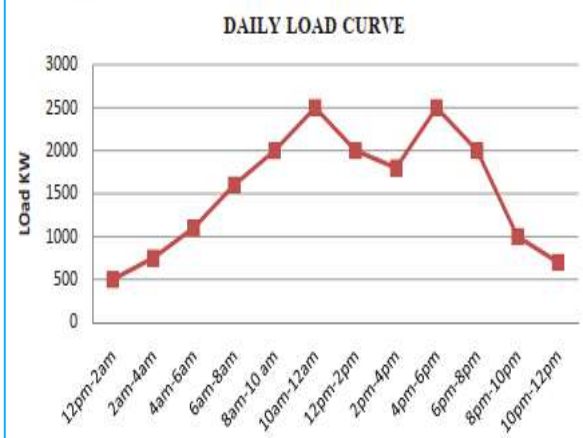


Fig . Daily load curve

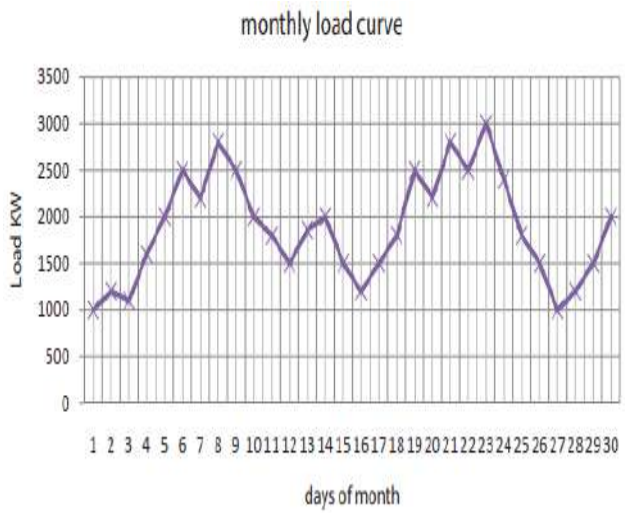


Fig monthly load curve

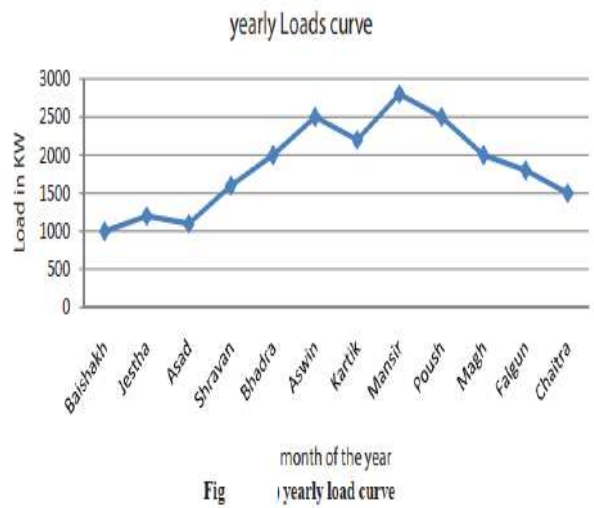


Fig yearly load curve

3.6 Mean and peak load, load curve, load factor, utilization and diversity factors….

The load curves deliver the following information

i. The deviation of the load during different hours of the day

ii. The area below the curve signifies the total consumption of units

iii. The peak of the curve stands for the maximum demand on the station

iv. The average load on power stations can be derived from area under load curve divided by time.

v. The load factor is the ratio of the area under the load curve to the area of the rectangle in which it is contained

Peak load: The highest demand in a day is called peak load. Peak demand fluctuations may occur daily, monthly and yearly. The peak of daily load curve denotes the highest demand on the station on the concerned day.

Base load: For daily load curve, minimum load within a day is called **base load** for that particular day.

- Similarly for monthly load curve, the minimum load in particular month is called *base load*.

Average load: Average load is defined as the average consumption over a defined time period.

- It is obtained from the ratio of area under the load curve and time.

Connected load: The sum of the continuous rating of all electrical equipment connected to the supply system is known as *connected load.*

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

- Generally peak load is defined as that part of the load carried at intensity greater than 4/3 times the mean load intensity.

3.6 Mean and peak load, load curve, load factor, utilization and diversity factors….

Demand factor:

It is defined as the ratio of actual maximum demand on the system to the total rated load connected to the system and it is always less than unity.

Mathematically, Demand factor = Maximum demand / Connected load

Average load or demand:

- It is the average of load occurring at the different occasions on the power station.
- It can also be expressed as the ratio of energy transmitted in a certain period to the number of hours in that period.
- We may derive daily, monthly or annual average load as per the duration of time period such as a day, a month or a year respectively.
- Daily average load = KWh supplied in a day / 24
- Monthly average load = KWh supplied in a day / 24 * 30
- Yearly average load = KWh supplied in a day / 24 * 365

Load Factor:

- From the load curve, it is possible to determine the average load for the duration of plotting and also the maximum load used at any time.
- The load factor can be defined as the ratio of average load to the maximum demand during a certain period of time. Since *average load is always less than the maximum demand*, hence load factor is always less than unity

Load factor = Average load / Maximum demand

Load factor = Energy Consumed in 24 hours / (maximum demand * 24 hours)

3.6 Mean and peak load, load curve, load factor, utilization and diversity factors….

Capacity factor

- It is the ratio of the average load to the rated capacity of the power plant.
- Every plant requires a reserve capacity to take care of the future expansion and increase in load.
- Therefore total installed capacity of the plant is usually greater than that actually required (maximum demand)

Capacity (Plant) factor = Average Demand / Plant Capacity

Also, Capacity of Plant factor = Actual Energy produced by the plant /Maximum Energy production capacity of the plant

Diversity factor

- The sum of individual maximum demand all the consumers connected to a power station is always greater than the maximum demand on the installation.
- The ratio of sum of the individual maximum demands of all the consumers supplied by power station to the simultaneous maximum demand of it is called the diversity factor which is *always greater than Unity.*

Diversity factor = Sum of individual maximum demand / Maximum demand of power station

Utilization factor

- It is measure of the utility of the power plant capacity and is the ration of maximum demand to the installed capacity of the power plant.
- In other words, it is the ratio of quantity of water actually used for power production to that available in the river.
- It varies from 0.4 to 0.9 for a hydal plant depending upon the plant capacity, load factor and storage.

Utilization factor = Peak load / installed capacity

1. The load on hydropower plant varies from a minimum of 15MW to a maximum of 45MW.Two turbo generators of capacities 26MW each have been installed. Calculate a) Total installed capacity of the plant b) plant factor c) Maximum demand d) Load factor and e) utilization factor.

Solution

- Installed capacity of the plant = 26x2 = 52MW
- $Average\ load = \frac{15+45}{2}$

= 30 MW

- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{30}{45}$

= 0.6667 = 66.67%

- Maximum demand = 45MW
- $Plant\ factor = \frac{\text{Average output of the plant}}{\text{Plant capacity}}$

$$= \frac{30}{52} = 0.577 = 57.7\%$$

- $Utilization\ factor = \frac{\text{Peak load}}{\text{Installed capacity}}$

$$= \frac{45}{52} = 0.865 = 86.5\%$$

2. When a run of river plant operates as a peak load station with a weekly load factor of 25% all its capacity is firm capacity. What will be the minimum flow in the river so that the station may serve as the base load station? It is given that, Rated installed capacity of generator=15,000KW, Operating head =20m, Plant efficiency=85%.Estimate the daily load factor of the plant, if the stream flow is 18cumecs.

Solution

From question given as following data

Rated installed capacity of the generator =15000KW, Operating head (H) =20m, efficiency (Ƞ) =85%, weekly load factor = 25%

- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = 25\% = 0.25$
- Average load = 0.25x15000 = 3750Kw…….. (I)
- Average power (P) = ƞxϒxQxH

= 0.85x9.81xQx20=166.77Q…… (ii)

Equating equation (I) and (ii)

- 3750=166.77Q
- **<u>Q = 22.486m^3/s</u>**
- When Q=18 cumecs =18m^3/s
- Power (P) = 0.85x9.81x18x20 =3001.86Kw
- $Daily\ Load\ factor = \frac{\text{Average load}}{\text{Peak load}}$

$= \frac{3001.86}{15000} = 0.2 = 20\%$

3. A run off river hydroelectric power station is proposed across a river at a site, where a net head of 25m is available on the turbine. The river carries a sustained minimum flow of $30m^3/s$. The power station is to be provided with poundage to supply daily peak load of demand with a load factor of 71% in dry weather season. Assuming the plant efficiency of 56.4%, determine the maximum generating capacity of the generator to be installed at the power station.

Solution; Head (H) = 25m, $Q_{min}=30m^3/s$, L.F. =71%=0.71, Ŋ=56.4%=0.564

- Average load = ŋxϒxQxH = 0.564x9.81x30x25=4149.63KW
- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{4149.63}{\text{Peak load}}$
- i.e. Peak load $= \frac{4149.63}{0.71} = \mathbf{5844.55}\boldsymbol{KW}$

4. If a run of river hydroelectric plant is used as peak load plant operating only 8 hours a day, determine (I) firm capacity of the plant without pondage or storage (ii) with pondage if the river has minimum flow $12m^3/s$ under a head of 12m with plant efficiency 85%.

Solution; Head (H) = 12m, $Q_{min}=12m^3/s$, Ŋ=85%=0.85

<u>Case (I) without pondage</u>

- I.e. Firm Power (P) = ŋxϒxQxH =0.85x9.81x12x12=**<u>1200.744KW</u>**
- **<u>Case (II) With Pondage (Assuming no losses)</u>**
- The plant is used as a peak load plant operating only 8 hours a day.
- I.e. Total volume stored during (24-8=16 hours) =12(m^3/s) x16x60x60 = **<u>691200m^3</u>**
- Rate of flow available from pondage = $\frac{691200}{8x60x60} = \mathbf{24m^3/s}$
- Total rate of flow to generate power = 24+12 = $36m^3/s$
- I.e. Firm power with pondage =36x0.85x9.81x12= **<u>3602.232KW</u>**

5. A run of river plant is installed on a river having a minimum flow of 15m3/s. If the plant is used as a peak load plant operating only for 6 hours daily (i.e.6 hours peaking plant).Compute the firm capacity of the plant (a) Without pondage b) with pondage but allowing 10% of water to be lost in evaporation and other losses. Head at the plant is 16m and the plant efficiency may be assumed as 80%.

Solution

Head (H) = 16m, Q =15m3/s, η=80%=0.80, Peak hour = 6 hour, losses =10%

Case (I)

Firm capacity without pondage = ηxϒxQxH = 0.8x9.81x15x16=1883.52KW

Case (II) with pondage

- Total Volume of water stored during 18 hours (24 – 6) = 15x18x60x60 = 972000m3
- Loss of water due to evaporation = 10% of 972000 = 97200m3
- Net amount of water at reservoir = 972000 – 97200 = 874800m3
- Rate of water available from pondage = **874800/6x60x60** = 40.5 m3/s
- Total rate of water available to generate power(QT) = 15 + 40.5 = 55.5m3/s
- Firm power due to pondage = 0.8x9.81x16x55.5 = 6969.024KW

6. A power station has to supply the following loads on an average day. Calculate the daily load factor, plant capacity factor and utilization factor if it has reserve capacity of 1200KW.

Time	11pm to 5am	5am to 6am	6am to 7 am	7am to 9am	9am to 12am	12am to 1Pm	1pm to 5pm	5pm to 7pm	7pm to 9pm	9pm to 11pm
Load(KW)	400	550	1100	2000	2500	1600	2500	2200	2500	1000

Solution:

Time	Hour	Load(KW)	Energy (KWh)
11pm to 5 am	6	400	6x400=2400
5am to 6 am	1	550	1x550=550
6 am to 7 am	1	1100	1x1100=1100
7 am to 9 am	2	2000	2x2000=4000
9am to 12 am	3	2500	3x2500=7500
12 am to 1 pm	1	1600	1x1600=1600
1pm to 5 pm	4	2500	4x2500=10000
5 pm to 7 pm	2	2200	2x2200=4400
7 pm to 9 pm	2	2500	2x2500=5000
9pm to 11 pm	2	1000	2x1000=2000
			Total Energy **=38550KW**

- *Maximum load = 2500KW*
- *Plant capacity*

= maximum load + Reserve capacity

= 2500+ 1200 = ***3700KW***

- $Average\ load = \frac{38550}{24}$

$= \mathbf{1606.25KW}$

- $Load\ factor = \frac{Average\ load}{Peak\ load}$

$= \frac{16606.25}{2500}$

$= 0.6425 = \mathbf{64.25\%}$

- $Plant\ factor = \frac{Average\ output\ of\ the\ plant}{Plant\ capacity}$

$= \frac{1606.250}{3700} = 0.4341 = \mathbf{43.41\%}$

- $Utilization\ factor = \frac{Peak\ load}{Installed\ capacity}$

$= \frac{2500}{3700} = 0.6757 = \mathbf{67.57\%}$

7. From the following data of the flow at a given area for an average year. Compute and draw the power duration curve. Assume an average available net head of 10m and combined efficiency of 89%.Determine primary and secondary energy available during a year if the plant capacity is fixed at power corresponding to the flow available for 25% of time.

Flow (m^3/sec)	900	600	500	450	400	350	340	300	280	200	140	100
% time	1	10	20	30	40	50	60	70	80	90	93	100

Solution: From question given as following data, Head (H) = 10m, Ŋ=89%=0.89, flow available for % of time = 25%

- Calculation of design flow, $\frac{(Q-500)}{(25-20)} = \frac{(450-500)}{(30-20)}$ or Q = 475m^3/s
- i.e. installed capacity (P) = ŋxϒxQxH = 0.89x9.81x475x10 = 41471.775KW = 41.472MW
- Detail calculation

Flow(m^3/s)	% of time	Power(MW)	Actual power(MW)
900	1	78.578	**41.472**
600	10	52.385	**41.472**
500	20	43.655	**41.472**
475	25	41.472	**41.472**
450	30	39.289	39.289
400	40	34.924	34.924
350	50	30.558	30.558
340	60	29.685	29.685
300	70	26.193	26.193
280	80	24.447	24.447
200	90	17.462	17.462
140	93	12.223	12.223
100	100	8.731	8.731

7.

Flow(m^3/s)	% of time	Power(MW)	Actual power(MW)	Energy (GWh)
900	1	78.578	41.472	3.633
600	10	52.385	41.472	32.697
500	20	43.655	41.472	36.329
475	25	41.472	41.472	18.165
450	30	39.289	39.289	17.687
400	40	34.924	34.924	32.505
350	50	30.558	30.558	28.681
340	60	29.685	29.685	26.387
300	70	26.193	26.193	24.475
280	80	24.447	24.447	22.180
200	90	17.462	17.462	18.356
140	93	12.223	12.223	3.901
100	100	8.731	8.731	6.425
				Total Energy =271.421

Sample calculation of Energy,

$$E_N = \left[\frac{T_N - T_{N-1}}{100}\right] * \left[\frac{P_{N(Actual)} + P_{N-1(Actual)}}{2}\right] * \left[\frac{365*24}{1000}\right]$$

$$E_1 = \frac{1-0}{100} * \frac{41.472 + 41.472}{2} * \frac{365*24}{1000} = 3.633 \text{ GWh}$$

$$E_2 = \frac{10-1}{100} * \frac{41.472 + 41.472}{2} * \frac{365*24}{1000} = 32.697 \text{ GWh}$$

$$E_3 = \frac{20-10}{100} * \frac{41.472 + 41.472}{2} * \frac{365*24}{1000} = 36.329\text{GWh}$$

$$E_4 = \frac{25-20}{100} * \frac{41.472 + 41.472}{2} * \frac{365*24}{1000} = 18.165 \text{ GWh}$$

$$E_5 = \frac{30-25}{100} * \frac{39.289 + 41.472}{2} * \frac{365*24}{1000} = 17.687\text{GWh}$$

$$E_6 = \frac{40-30}{100} * \frac{34.924 + 39.289}{2} * \frac{365*24}{1000} = 32.505\text{GWh}$$

$$E_7 = \frac{50-40}{100} * \frac{30.558 + 34.924}{2} * \frac{365*24}{1000} = 28.681\text{GWh}$$

$$E_8 = \frac{60-50}{100} * \frac{29.685 + 30.558}{2} * \frac{365*24}{1000} = 26.387\text{GWh}$$

$$E_9 = \frac{70-60}{100} * \frac{26.193 + 29.685}{2} * \frac{365*24}{1000} = 24.475\text{GWh}$$

$$E_{10} = \frac{80-70}{100} * \frac{24.447 + 26.193}{2} * \frac{365*24}{1000} = 22.180\text{GWh}$$

$$E_{11} = \frac{90-80}{100} * \frac{17.462 + 24.447}{2} * \frac{365*24}{1000} = 18.356\text{GWh}$$

$$E_{12} = \frac{93-90}{100} * \frac{12.223 + 17.462}{2} * \frac{365*24}{1000} = 3.901 \text{ GWh}$$

$$E_{13} = \frac{100-93}{100} * \frac{8.731 + 12.223}{2} * \frac{365*24}{1000} = 6.425\text{GWh}$$

7.

Flow(m^3/s)	% of time	Power(MW)	Actual power(MW)	Energy (GWh)
900	1	78.578	**41.472**	3.633
600	10	52.385	**41.472**	32.697
500	20	43.655	**41.472**	36.329
475	25	41.472	**41.472**	18.165
450	30	39.289	39.289	17.687
400	40	34.924	34.924	32.505
350	50	30.558	30.558	28.681
340	60	29.685	29.685	26.387
300	70	26.193	26.193	24.475
280	80	24.447	24.447	22.180
200	90	17.462	17.462	18.356
140	93	12.223	12.223	3.901
100	100	8.731	8.731	6.425
				Total Energy =271.421

- Firm Energy (Primary Energy)= $\frac{100-0}{100} * \frac{8.731}{1000} * 365 * 24 = \mathbf{76.484\ GWh}$

$$\boldsymbol{OR}\ \ Firm\ Energy = \frac{100-0}{100} * \frac{8.731+8.731}{2} * \frac{365*24}{1000} = \mathbf{76.484GWh}$$

- Secondary Energy = Total Energy – Primary Energy =271.421 – 76.484 = **<u>194.937 GWh</u>**

8. Mean monthly flow of a river is as follows

If head is 200m and design discharge is 200m3/s, find primary and secondary energy. Take overall efficiency of project 88%.

Months	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sep	Oct	Nov	Dec
Flow (m^3/sec)	150	130	110	90	120	280	360	470	270	230	190	170

Solution: From question given as following data; Head (H) = 200m, Ƞ=88%=0.88, design flow = 200m^3/s

Month	Flow (m^3/s)	Flow in Descending order(m^3/s)	Rank	% of time	Power (MW)	Actual power (MW)	Energy (GWh)
Jan.	150	470	1	8.333	811.4832	345.312	252.08
Feb.	130	360	2	16.667	621.5616	345.312	252.08
Mar.	110	280	3	25	483.4368	345.312	252.08
Apr.	90	270	4	33.333	466.1712	345.312	252.08
May	120	230	5	41.667	397.1088	345.312	252.08
Jun	280	**200**		**47.918**	**345.312**	345.312	189.10
July	360	190	6	50	328.0464	328.0464	61.404
Aug.	470	170	7	58.333	293.5152	293.5152	226.87
Sep.	270	150	8	66.667	258.984	258.984	201.66
Oct.	230	130	9	75	224.4528	224.4528	176.45
Nov.	190	120	10	83.333	207.1872	207.1872	157.55
Dec.	170	110	11	91.667	189.9216	189.9216	144.95
		90	12	100	155.3904	155.3904	126.04
						Total Energy	= 2544.411

- i.e. Total Energy = **2544.441 GWh**
- Primary Energy = **1361.22GWh**
- Secondary Energy = **1183.191 GWh**

7. HW

Mean monthly flow of a river in a typical year as follows. The gross available head is 229m and average head loss in transition is 9m.The efficiencies of turbines, generators and gearboxes are 95%, 92% and 90% respectively.8m^3/s of water is required to be released for downstream users and to maintain ecosystem. Determine (I) best installed capacity (ii) firm energy.

Month	Jan	Feb	Mar.	Apr.	May	Jun	July	Aug.	Sep.	Oct.	Nov.	Dec.
Flow (m^3/s)	92	88	114	210	595	1012	2016	2544	1456	833	195	126

3.4 Methods of fixing installed capacity of a hydropower plant

Plant Capacity:

It is the capacity of the plant with respect to available head, discharge and efficiency.

Installed capacity:

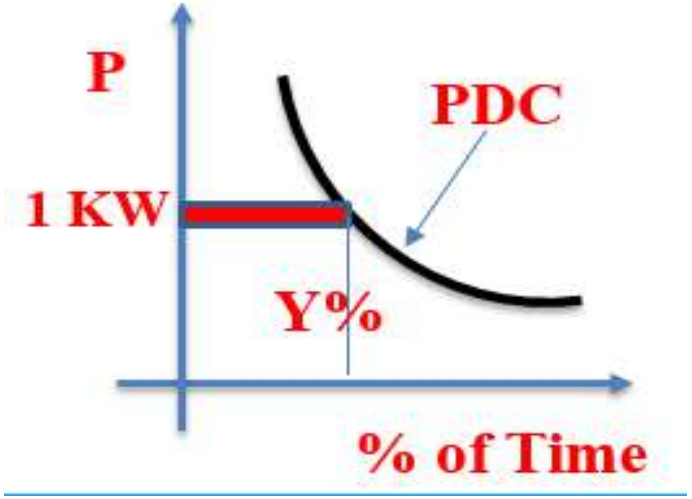
The plant capacity, which is economically viable, is known as installed capacity. Ii is also defined as the maximum power, which can be developed by all generators of the plant at the plant at the normal head and with full flow.

Most cost and benefit approach:

The installed capacity or economical plant capacity is determined by equating the marginal cost and benefits for that project, while determining the fixed cost of the project is assumed to be constant.

Let us analyze marginal benefit and cost for 1KW installation. Assuming Y% of the duration corresponds to the best installed capacity for 1KW power generation then,

- Energy price = U$ E/MWH
- Variable cost = U$ Vcost/KW (the variable cost is electromechanical installation cost. The civil cost is not included in the variable cost because it is fixed cost)
- O&M = certain % of variable cost
- Economic life of Plant = N yrs.



3.4 Methods of fixing installed capacity of a hydropower plant

Most cost and benefit approach…..

Now analyzing by the marginal benefit and marginal cost approach

- Total energy generated $=\frac{(Y\%x365x24)}{1000}$ MWH = 8.76 Y% MWH (Annual)= $\frac{(Yx365x24)}{1000x100}$ =0.0876 Y MWH
- Marginal benefit = Total Energy generated * Energy rate = U$ 0.0876YE ……..(I)
- The electromechanical equipment is installed at the time of construction so converted it into annual cost and annual operation and maintenance cost is added in it. *The civil component cost in hydropower is fixed cost so it is* *<u>not included in marginal cost</u>*.
- AC = $V_{cost}\left[\frac{(1+i)^N * i}{(1+i)^N - 1}\right]$
- Operation and maintenance cost is generally considered as certain percentage of variable cost.
- O and M cost = % $of\ V_{cost}$
- Marginal cost = Annual Cost + O and M Cost
- $= \boldsymbol{V_{cost}\left[\frac{(1+i)^N * i}{(1+i)^N - 1}\right] + \%\ of\ V_{cost}} \ldots\ldots\ldots\ldots\ldots\ldots (\boldsymbol{II})$

Now equating marginal cost and benefit, the value of 'Y' can be obtained.

- The power corresponding to Y% of tie in power duration curve is the best installed capacity of the project.

Chapter 3 : Power and Energy of Potential Study

Q.N. Long term average monthly flow of a large tropical river is given follows.
Other data are given as, the effective head of the plant is 100m, Efficiency (hydraulic, turbine, generator and transformer) = 96%, 93%, 98%, 99% respectively. Interest rate = 10%, Environmental release 10% of minimum monthly flow, Energy price = U$ 40/MWH, Fixed cost = 10%, Variable cost = U$ 600/KW, Annual O&M = 3% of variable cost, Economic life of Plant=40 yrs. Determine (I) best installed capacity (ii) firm energy.

Month	Jan	Feb	Mar.	Apr.	May	Jun	July	Aug.	Sep.	Oct.	Nov.	Dec.
Flow (m^3/s)	280	250	210	84	84	210	868	1064	2380	2492	2156	659

Sol:
Overall Efficiency (η)= 96%x93%x98%x99% = 0.96x0.93x0.98x0.99 = 0.866

Month	Flow (m^3/s)	Flow in Descending order(m^3/s)	D/S released Flow(m^3/s)	Remaining Discharge (m^3/s)	Rank	% of time	$Power = \frac{\eta x \Upsilon x Q x H}{1000}$ (MW)
Jan.	280	2492	8.4	2483.6	1	8.333	2109.932
Feb.	250	2380	8.4	2371.6	2	16.667	2014.783
Mar.	210	2156	8.4	2147.6	3	25	1824.485
Apr.	84	1064	8.4	1055.6	4	33.333	896.781
May	84	868	8.4	859.6	5	41.667	730.269
Jun	210	659	8.4	650.6	6	50	552.715
July	868	280	8.4	271.6	7	58.333	230.737
Aug.	1064	250	8.4	241.6	8	66.667	205.250
Sep.	2380	210	8.4	201.6	9	75	171.268
Oct.	2492	210	8.4	201.6	10	83.333	171.268
Nov.	2156	84	8.4	75.6	11	91.667	64.226
Dec.	659	84	8.4	75.6	12	100	64.226

Chapter 3 : Power and Energy of Potential Study

Q.N. Long term average monthly flow of a large tropical river is given follows.
Other data are given as, the effective head of the plant is 100m, Efficiency (hydraulic, turbine, generator and transformer) = 96%, 93%, 98%, 99% respectively. Interest rate = 10%, Environmental release 10% of minimum monthly flow, Energy price = U$ 40/MWH, Fixed cost = 10%, Variable cost = U$ 600/KW, Annual O&M = 3% of variable cost, Economic life of Plant=40 yrs. Determine (I) best installed capacity (ii) firm energy….

- Let us analyze marginal benefit and cost for 1KW installation. Assuming Y% of the duration corresponds to the best installed capacity for 1KW power generation then,
- Total energy generated $=\frac{(Y\%x365x24)}{1000}$ MWH = 8.76 Y% MWH (Annual)
- $=\frac{(Yx365x24)}{1000x100}$ =0.0876 Y MWH
- Marginal benefit = 0.0876x U$ 40 = **U$ 3.504Y**……(I)
- Total cost = Variable cost + O and M cost
- $= V_{cost}\left[\frac{(1+i)^N * i}{(1+i)^N - 1}\right] + 3\ \%\ of\ V_{cost}$
- $= U\$600\left[\frac{(1+0.1)^{40} * 0.1}{(1+0.1)^{40} - 1}\right] + \frac{3x600}{100} = \boldsymbol{U\$\ 79.356}\ \ldots\ldots\ldots\ldots\ldots\ldots (II)$
- Equating equation (I) and (II), we get
- U$ 3.504Y = U$ 79.356or, **Y = 22.647%**
- Hence the advantageous duration is 22.647%.
- Installed capacity calculated by interpolation, $\frac{(P-2014.783)}{(22.647-16.667)} = \frac{(1824.485-2014.783)}{(25-16.667)}$
- **P= 1878.219MW**, which is best installed capacity of the power plant.

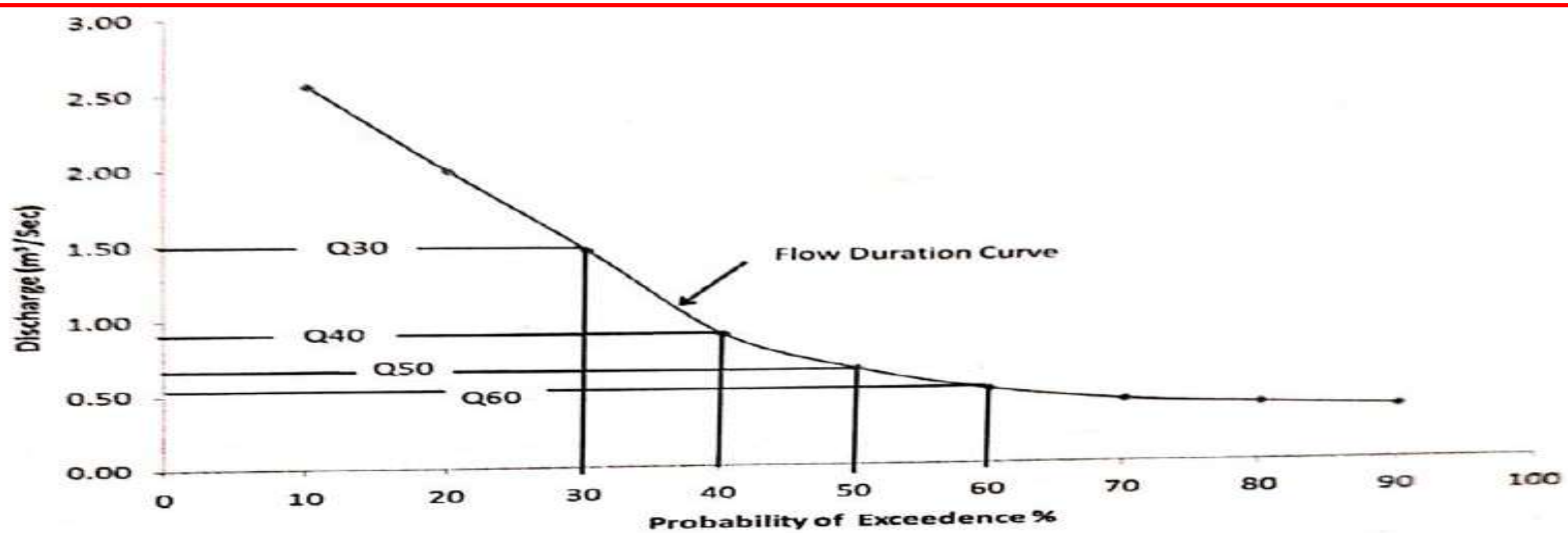
% of time	Power =
8.333	2109.932
16.667	2014.783
25	1824.485
33.333	896.781
41.667	730.269
50	552.715
58.333	230.737
66.667	205.250
75	171.268
83.333	171.268
91.667	64.226
100	64.226

Installed capacity optimization approach

We know that the power generated from a plant is function of head and discharge and the formula is giver by, $P = \frac{\eta \gamma Q H}{100}$ KW . Where, η is overall efficiency in %, γ is specific weight of water whose value is 9.81 kN/ m³, Q is discharge in m³/s and H is head in meter. The power can be calculated based on the different percentile available flow as $P_{30} = \frac{\eta \gamma Q_{30} H}{100}$ KW , $P_{40} = \frac{\eta \gamma Q_{40} H}{100}$ KW and so on.

From flow duration curve, for run of river plant, increasing the percentile of available flow, the design discharge reduces. Decreasing the design discharge, the size of the hydropower components such as intake structure, settling basin, conveyance system, penstock etc decreases. Hence, the project cost reduces. But in decreasing the design discharge, the project capacity as well as energy decreases, thereby the annual revenue is decreased. Hence the revenue and cost is traded to get optimum benefit.

For this generally, the flow with 30% available to the 60% available with certain interval is calculated from flow duration curve as shown in figure above and power and revenue is calculated. And cost for each option is calculated then optimization study is done. The optimum capacity is fixed as installed capacity.



In Nepal for projects upto 25MW the installed capacity is fixed based on Q_{40} basis because NEA conduct power purchase agreement (PPA) in this pattern but for projects greater than 25MW, the installed capacity is fixed based on the optimization study.

3.3 Reservoirs and their regulation, need for flow regulation, Source of sediment, sediment yield in Rivers, sediment handing in reservoirs, life of the reservoirs

5.1 Introduction

When a barrier is constructed across some river in the form of a dam, water gets stored on the upstream side of the barrier, forming a pool of water, called a dam reservoir or an impounding reservoir or a river reservoir. Any collected water in form of pool or lake, may be termed as reservoir.

Dams and reservoirs are the most important elements of any multipurpose river basin development or development of large storage hydropower projects. Dams and reservoirs are complementary to each other. Reservoir can be developed only by constructing dam and conversely when a dam is constructed reservoir is bound to develop. Dam and reservoir planning such as selection of site for dam and reservoir, fixation of reservoir capacity, safe yield of storage and other connection works has very important role for development of multipurpose project. The plan and sectional views are shown in figure 5-1 below.

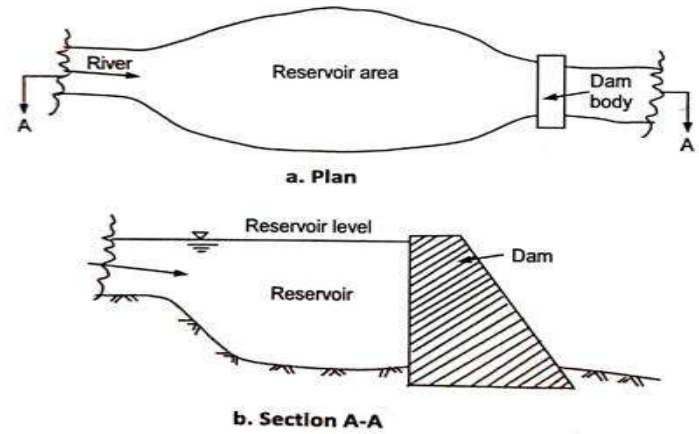


Figure 5-1 : Plan and section of reservoir upstream of dam

Classification of Reservoir

It has two main purposes

a) Flood control storage b) Storage or conservation storage

The aim of flood control storage is to store some of the flood water of river when discharge rate at a stage likely to causes damage downstream of the valley and release this food storage slowly at a safe rate when flood subsides.

Conservation storage to store water for different purpose like

a. Hydropower
b. Irrigation
c. Public and industrial water supply
d. Navigation
e. Soil conservation and land reclamation
f. Fishery and wild life
g. Recreation
h. Growing useful aquatic life
i. Debris control
j. Sediment control of river etc

Thus a reservoir is called **single purpose reservoir** if it serves only one purpose and is called **multipurpose** if it is used for almost all the purposes mentioned above or more than one purpose.

A reservoir is called a **distribution reservoir** without a dam in which the water from river is pumped into some concrete tank constructed at suitable places. Water stored in those reservoirs is mainly used for supplying to the consumers though distribution networks.

The flood control storage discussed above may be retarding type or detention type. In retarding type, it simply retards the flood water without control like gates or spillway, whereas in detention type it detains major portion flood water with the help of gates, etc. and releases the extra water when flood subsides.

3.3 Reservoirs and their regulation, need for flow regulation,…….

5.3 Location of site for reservoir

In selection the site for a reservoir the following points are to be considered:

a. It is located in the area of minimum percolation and maximum runoff (Sufficient inflow).
b. Leakage in the selected area should be minimum to minimize the grouting works.
c. It should not be located on highly permeable rocks like shale's, slates, gneisses, granite, etc.
d. Suitable dam site with water tight rock base should be available in the located area.
e. To reduce the length of the dam, narrow opening of the basin is essential.
f. Site should be easily accessible by road and if required to construct them, cost of construction should be minimum
g. Topography of the location should have adequate capacity without submerging excessive land and other properties.
h. Reservoir location should be free from objectionable minerals and salts.
i. Site should exclude water from tributaries solution of minerals and salts.
j. Located area should provide sufficient water depth with smaller water area. Higher depth provides lower submerged area/unit capacity and decreases the possibility of weed growth. Smaller surface causes less evaporation losses.
k. Construction materials for the dam and other allied works should be locally available.
l. Suitable area near the location should be available for construction of staff quarters, labor colonies, godowns, stack yards, etc.
m. The site should be selected after investigating, so that cost of construction should be economic.
n. Heavily silt laden tributaries should not lead their discharge to the reservoir.
o. Less submergence of habitat area, fertile land etc

5.4 Determination of storage capacity

The following two methods are most reliable for determination of storage capacity reservoir:

5.4.1 By contour map

A contour map of entire area as shown in figure 5-2 is required to estimate the storage capacity. If the reservoir area is large enough, a ground survey requires a considerable time and expenditures. In such big reservoir, aerial survey is used. A contour map of a reservoir with areas A_1, A_2, A_3……A_n between two contours is shown in figure 5-2.

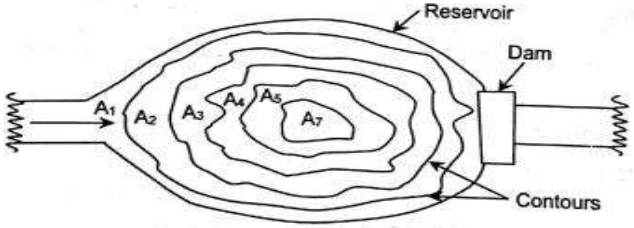


Figure 5-2 : Contours of reservoir

A contour interval of 0.5 m is taken for moderate reservoir and 1.5 m for a high dam. Thus, a very accurate triangulation survey is required with required contour interval to a scale of 1/1000 and preferable 1/500. From the contour map of reservoir area, water spread area of the reservoir at any elevation is directly determined by measuring the area at the contour by planimeter.

Elevation against water spread area may be plotted. Capacity may be measured by taking contour at equal intervals and totaling by the trapezoidal rule or by Simpson's rule or by cone formula.

$$V = \frac{h}{2}(A_1 + A_2) \quad \text{Trapezoidal rule}$$

$$V = \frac{h}{6}(A_1 + A_2 + 4A_m) \quad \text{Simpson's rule}$$

where A_m is the area of a contour line between midway A_1 and A_2

$$V = \frac{h}{3}\left[A_1 + A_2 + \left(\frac{A_1 + A_2}{2}\right)\right] \quad \text{Cone formula}$$

V is the volume between two adjacent contours at an interval of having areas A_1 and A_2. With the above formulae volumes between the successive contours A_1 and A_2, A_2 and A_3 ..... may be determined. Adding all these volumes calculated between successive contours, total capacity or volume of the reservoir can be determined.

3.3 Reservoirs and their regulation, need for flow regulation,…….

5.4.2 Determination by mass curve of inflow and demand

Mass inflow curve (mass curve)

It is called Ripple mass curve after its originator W Ripple in 1983. Mass curve of inflow is a plot of accumulated flow volume against time, accumulated volume as ordinate and time as abscissa. The mass curve is used to determine

a. Reservoir capacity from uniform demand of flow
b. Safe yield from reservoir from reservoir capacity

A mass inflow curve is prepared from the inflow hydrograph of a river for a large number of consecutive years. From this flood hydrograph, ordinates of the mass curve (accumulated inflow volume) at any time can be obtained when accumulated inflow volumes (in cumec month, or ha-m or 10^6 m^3) are plotted against time, mass curve is obtained.

The discharge ordinates represent the annual average discharge (i.e. average discharge of the full year, obtained from the total, annual volume of stream flow by dividing it by (365 x 24 x 3600)

Mass curve is continuously rising curve as it shows the accumulated volume upto that time. If there is no flow during a certain period, the mass curve can be horizontal but it can never fall. The slope of the curvature at any point indicates the usable flow for power production. The slope of the curvature at any point indicates the rate of inflow at that particular time.

For many water resources investigations it is important to know the total volumes of water that have to be dealt with over long periods of time. For such purposes the most convenient means of presenting the runoff data is the mass curve. When it is desired to use water at uniform or nearly uniform rate greater than the minimum discharge in the stream, it is necessary to provide storage where water is impounded during periods of high flow for use during periods of low flows. Mass curve serves as a very useful tool to determine the required storage capacity for any uniform rate of demand.

A graph of the cumulative value of any hydrologic quantity, generally as the ordinate, plotted against the time as abscissa is known as a mass curve. When cumulative volume of runoff is taken as the hydrologic quantity, the mass curve may be called of low mass curve or simply a mass curve. It is the integration of the hydrograph and therefore represents the area under the hydrograph from one time to another. Mathematically, the mass curve is expressed by

$$V(t) = \int_0^t Q(t).dt$$

Where, V (t) = cumulative volume of flow up to time t from the start of the record, that is, the ordinate of the mass curve at any time t.

Q (t) = the discharge as a function of time, i.e., the ordinate of the hydrograph, and

t = a dummy variable.

The difference in the ordinates of any two points on the curve is the summation of the flow during the intervening period of time.

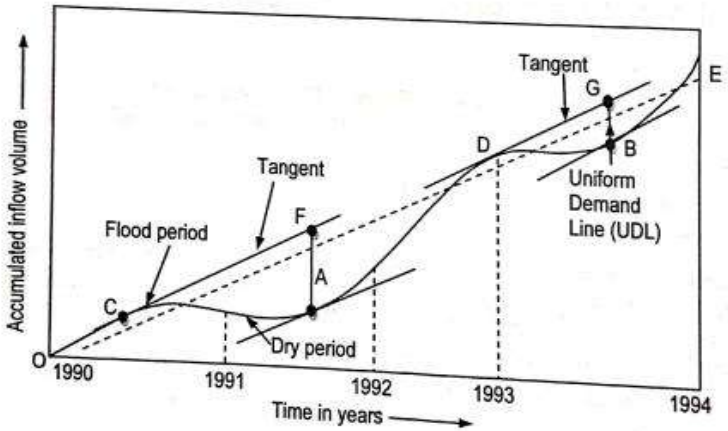


Figure 5-3 : Mass curve of inflow

Characteristics of mass curve

- The mass curve is a continuously rising curve as it shows the accumulated inflow volume with time.
- The slope of the curve at any point indicates the rate of inflow at that particular time.
- If the curve becomes horizontal at some point, it indicates that rate of flow at that particular time is zero. But the mass curve never fall down.
- If the curve rises sharply, it indicates the high rate of inflow within that period.

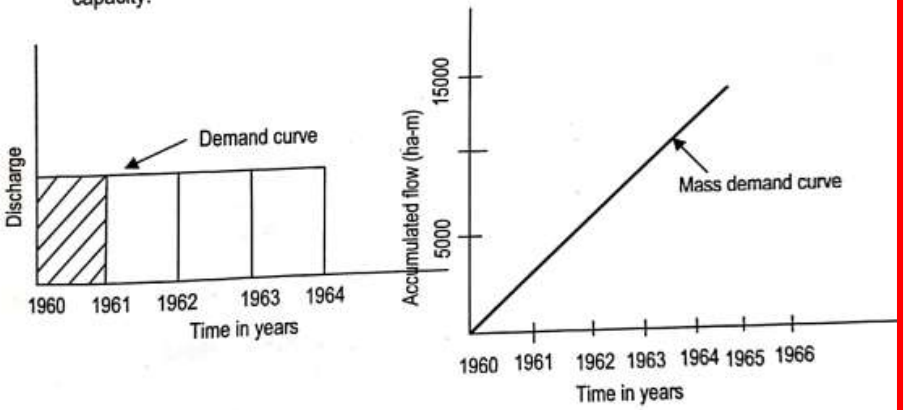
3.3 Reservoirs and their regulation, need for flow regulation,…….

- Relatively convex rise and concave depression indicate flood and dry period respectively. The points A and B in mass curve (figure 5-3) indicate dry periods. Whereas points C and D indicate flood or high flow.

Mass demand curve

A demand curve is a plot between the demand rate as ordinate and the time as abscissa. If the demand is uniform, the demand curve is horizontal line, and the corresponding mass demand curve is straight line, having a slope equal to the demand rate. On the other hand, if the demand is variable, the mass demand curve is a rising curve with different slope at different time interval.

- The dotted line OE indicates uniform demand line (UDL), slope of which gives the uniform rate of demand.
- If OE is the uniform demand line, tangents at highest point C and D are drawn parallel to OE.
- The maximum vertical intercept between the mass curve (Here AF) gives the reservoir capacity.



Reservoir capacity by graphical method, using mass curve of inflow and demand

Using mass curve, the storage required for uniform flow can be calculated by the following steps

1. Draw the mass curve of inflow (mass curve)
2. Draw the demand line at the demand rate slope (mass demand curve)
3. From peak, draw tangent parallel to demand line
4. Maximum vertical ordinate between the mass curve and tangent gives the storage capacity of the reservoir that should be provided.
5. If the demand curve is not uniform it shows a non uniform demand
6. Determine the vertical intercepts AF, BG, etc which indicates the volume by which the inflow volume short of demand.

Determination of yield of a reservoir

Tangents are drawn from peaks such that the maximum vertical distance of any tangent from the mass curve does not exceed the given reservoir capacity. The slope of these lines gives the safe yield of the reservoir for that period.

The tangent when extended must intersect the mass inflow curve; otherwise the reservoir will not be full again. The flatest slope indicates the firm dependable yield from the reservoir.

5.4.3 Analytical method (Sequent peak algorithm)

In the mass curve analysis, the reservoir is assumed to be full at the beginning of the dry period and d the storage required to pass the dry period is estimated. If the mass curve contains only one ridge point and if there is no well defined subsequent trough point, it may be necessary to repeat the given data for one more cycle to arrive at the desired storage requirement. Also, the demand rate is invariably non-uniform due to seasonal variations in the demand. The sequent peak algorithm is a simple and straightforward analytical procedure leading to faster determination of the storage requirement. Unlike in the simple mass curve, it can easily take care of variable demands. Evaporation and seepage losses from the reservoir which are not considered so far can be easily taken care of by adding them to the demands when reasonable estimates of them are available.

a. Calculate the net inflow into the reservoir (V_i-D_i) for all the time periods i =1,2,.....,2N, where N is the number of periods of the data series and V_i for $i = N+1$, 1, to 2N are same as V_i for i=1 to N. That is, the given data is repeated once again to have a series of length 2N. V_i and D_i denote the volume of runoff and volume of Demand in the ith period.

b. Calculate the cumulative net inflow volumes, namely $\sum_{i=1}^{t}(v_i - D_i)$ *for* t = 1,2....., 2N.

c. Locate the first peak (local maximum) P_1, in the cumulative net inflows.

d. Locate the sequent peak P_2 which is the next peak of greater magnitude than the first.

e. Between this pair of peaks find the lowest trough (local minimum) T_1 and calculate (P_1 - T_1).

f. Starting with P_2, find the next sequent peak P_3, that has magnitude greater than P_2.

g. Find the lowest trough T_2, between P_2, and P_3, and calculate (P_2- T_2).

h. Starting with P_3, find P_4 and T_3; calculate (P_3 - T_3).

i. Repeat the procedure for all sequent peaks available in the 2N period and compute (P_j- T_j) for all j values.

j. The required reservoir storage is given by S = Maximum of all (P_j - T_j) values. It may be noted that the sequent peak algorithm is nothing but the numerical adaptation of the graphical procedure of the residual mass curve using two cycles of the given data series.

Q. N. The monthly runoff volume into a reservoir in Mm3 for a 12 month period form January to December are 22.8, 18.3, 13.7, 11.4, 9.1, 13.9, 91.4, 102.8, 68.4, 41.1, 32.1 and 27.4 respectively. Determine the storage capacity of the reservoir to meet a demand which is equal the average flow in the river.

Solution: The average flow in the river is given by

$$\frac{22.8+18.3+13.7+11.4+9.1+13.9+91.4+102.8+68.4+41.1+32.1+27.4}{12} = \frac{452.4}{12} = \textbf{37.7 Mm3 per month}.$$

Determination of storage by the sequent peak algorithm

Month	Runoff (Vi) Mm3	Demand (Di) Mm3	(Vi –Di)	Cumulative (Vi –Di)	Remarks
Jan	22.8	37.7	-14.9	-14.9	
Feb	18.3	37.7	-19.4	-34.3	
Mar	13.7	37.7	-24	-58.3	
Apr	11.4	37.7	-26.3	-84.6	
May	9.1	37.7	-28.6	-113.2	
Jun	13.9	37.7	-23.8	**-137**	(Trough) T1
July	91.4	37.7	53.7	-83.3	
Aug	102.8	37.7	65.1	-18.2	
Sep	68.4	37.7	30.7	12.5	
Oct	41.1	37.7	3.4	**15.9**	(Peak) P1
Nov	32.1	37.7	-5.6	10.3	
Dec	27.4	37.7	-10.3	0	

∴The required storage capacity = P1 – T1 = 15.9 – (– 137) = **152.9 Mm3**

Q.N. The average inflows of a river corresponding to a driest year is shown below.

Month	Jan	Feb	Mar	Apr	May	Jun	July	Aug.	Sep.	Oct.	Nov.	Dec.
Flow (m^3/s)	110	90	70	50	30	25	65	220	300	190	115	110

Find (i) the minimum storage capacity of power generation at inflow discharge rate 85 m3/sec.
(ii) If the reservoir capacity is fixed at 270 cumec month what uniform rate of withdrawal is possible? Assume one month = 30.4 days. Solve by analytical method.

Solution:

- Let the uniform withdrawal rate is below 90 in first trial when reservoir capacity is 270 cumec month then.
- Let X = actual rate of withdrawal (X < 90),
- Then three will be deficit in March, April, May, June and July month. The deficit will be (X – 70), (X – 50), (X – 30), (X – 25) and (X- 65) respectively.
- Then, cumulative deficit be (X-70), (X – 70) + (X – 50), (X – 70)+ (X – 50)+ (X – 30), (X – 70) + (X – 50) + (X – 30) + (X – 25), (X – 70) + (X – 50) + (X – 30) + (X – 25) + (X- 65).
- The maximum cumulative deficit will be in the month of July which is equal to the reservoir capacity, so
- (X – 70) + (X – 50) + (X – 30) + (X – 25) + (X- 65) =270
- X = 102 m3/sec > 90 so assumption is wrong.
- So assume withdrawal rate is below 110 m3/sec.

<u>Calculation of Reservoir Capacity</u>

Month	Flow (m^3/s)	Flow through turbine	Deficit in storage	Cumulative deficit	For Reservoir Capacity 270, Let uniform withdrawal rate X < 100	
					Deficit in Storage	Cumulative Deficit
Jan.	110	85	-	0		-
Feb.	90	85	-	0	(X – 90)	(X – 90)
Mar.	70	85	15	15	(X – 70)	(2X – 160)
Apr.	50	85	35	50	(X – 50)	(3X – 210)
May	30	85	55	105	(X – 30)	(4X – 240)
Jun	25	85	60	165	(X – 25)	(5X – 265)
July	65	85	20	**185**	(X – 65)	**(6X – 330)**
Aug.	220	85				
Sep.	300	85				
Oct.	190	85				
Nov.	115	85				
Dec.	110	85				

Case (I)

The minimum storage capacity of power generation at inflow discharge rate 85 m3/sec is 185 Cumec month

= 185 *(30.4*24*60*60) m3

= 485813.60 M3

Case (II) Uniform rate of withdrawal

Then maximum cumulative deficit is

(X – 90) +(X – 70) + (X – 50) + (X – 30) + (X – 25) + (X- 65) = 270

6X = 600

X = 100 M3/sec, which is *uniform withdrawal* rate when reservoir capacity is 270 cumec month.

THANK YOU
STAY HOME
STAY SAFE

UNIT: 3 Power and Energy Potential Study

Q.N. 1

- Installed capacity of the plant =26x2=52MW
- $Average\ load = \frac{15+45}{2} = 30MW$
- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{30}{45} = 0.6667 = 66.67\%$
- Maximum demand =45MW
- $Plant\ factor = \frac{\text{Average output of the plant}}{\text{Plant capacity}} = \frac{30}{52} = 0.577 = 57.7\%$
- $Utilization\ factor = \frac{\text{Peak load}}{\text{Installed capacity}} = \frac{45}{52} = 0.865 = 86.5\%$

Q.N.2

From question given as following data

Rated installed capacity of the generator =15000KW, Operating head (H) =20m, efficiency (ɳ) =85%, weekly load factor=25%

- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = 25\% = 0.25$

  Average load =0.25x15000=3750Kw…….. (I)

  Average power (P) = ɳxϒxQxH=0.85x9.81xQx20=166.77Q…… (ii)

  Equating equation (I) and (ii)

  3750=166.77Q

  Q = 22.486m³/s
- When Q=18 cumecs =18m³/s

  Power (P) =0.85x9.81x18x20 =3001.86Kw
- $Daily\ Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{3001.86}{15000} = 0.2 = 20\%$

Q.N.3 From question given as following data

Density of liquid (***ρ***) =1200kg/m³, discharge (Q) =1100 LPs = 1100x10⁻³=1.1 m³/s, Head (H) = 60m, efficiency (ɳ) = 70% =0.7

- Power of the pump lifting = ɳxϒxQxH =0.7x(1200x9.81)x1.1x60=543866.4 Watt =**543.8664KW**

Q.N.4 From question given as following data

Head (H) = 25m, Q_{min}=30m³/s, L.F. =71%=0.71, ɳ=56.4%=0.564

- Average load = ɳxϒxQxH = 0.564x9.81x30x25=4149.63KW
- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{4149.63}{\text{Peak load}}$
- i.e. Peak load $= \frac{4149.63}{0.71} = \mathbf{5844.55KW}$

Q.N.5

- $Capacity(Plant)\ factor = \frac{\text{Average output of the plant}}{\text{Plant capacity}} = \frac{40 \times 10^6 \text{KWh}}{110\text{x}10^3\text{x}30\text{x}24} = 0.5051 = 50.51\%$
- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{40 \times 10^6}{90\text{x}10^3\text{x}30\text{x}24} = 0.6173 = 61.73\%$

Q.N.6 Same as Q.N.2

Q.N.7 From question given as following data

Reserve capacity = 1200KW

Time	Hour	Load(KW)	Energy (KWh)
11pm to 5 am	6	400	6x400=2400
5am to 6 am	1	550	1x550=550
6 am to 7 am	1	1100	1x1100=1100
7 am to 9 am	2	2000	2x2000=4000
9am to 12 am	3	2500	3x2500=7500
12 am to 1 pm	1	1600	1x1600=1600

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1pm to 5 pm	4	2500	4x2500=10000
5 pm to 7 pm	2	2200	2x2200=4400
7 pm to 9 pm	2	2500	2x2500=5000
9pm to 11 pm	2	1000	2x1000=2000
			Total Energy =38550KW

- Maximum load = 2500KW
- Plant capacity = maximum load + Reserve capacity = 2500+ 1200 = **3700KW**
- $Average\ load = \frac{38550}{24} = \mathbf{1606.25KW}$
- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{16606.25}{2500} = 0.6425 = \mathbf{64.25\%}$
- $Plant\ factor = \frac{\text{Average output of the plant}}{\text{Plant capacity}} = \frac{1606.250}{3700} = 0.4341 = \mathbf{43.41\%}$
- $Utilization\ factor = \frac{\text{Peak load}}{\text{Installed capacity}} = \frac{2500}{3700} = 0.6757 = \mathbf{67.57\%}$

Q.N.8 From question given as following data

Head (H) = 60m, Q_{min}=25m^3/s, L.F. =80%=0.8, η=75%=0.75

- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{\eta x \Upsilon x Q x H}{\text{Peak load}}$
- $i.e.\ Peak\ load = \frac{0.75x9.81x25x60}{0.8} = \mathbf{13795.313KW}$

Case (I) Assuming there is no reservoir

I.e. maximum generating capacity =13795.313KW

Case (II)

- Average power= ηxϒxQxH=0.75x9.81x25x60=**11036.25KW**
- Excess power required to be developed during 4 hrs =13795.313 - 11036.25=**2759.063KW**
- Excess discharge required for developing this excess power is (P)= ηxϒxQxH

$$i.e.\ Q = \frac{2759.063}{0.75x9.81x60} = 6.25\ M^3/s$$

- Required pondage per day =6.25x4x60x60 = 90000m^3

Q.N.9 From question given as following data

Head (H) = 12m, Q_{min}=12m^3/s, η=85%=0.85

Case (I) without pondage

I.e. Firm Power (P) = ηxϒxQxH =0.85x9.81x12x12=**1200.744KW**

Case (II) With Pondage (Assuming no losses)

- The plant is used as a peak load plant operating only 8 hours a day.
  I.e. Total volume stored during (24-8=16 hours) =12(m^3/s) x16x60x60 = **691200m^3**
- Rate of flow available from pondage $= \frac{691200}{8x60x60} = \mathbf{24m^3/s}$
- Total rate of flow to generate power =24+12=36m^3/s
  I.e. Firm power with pondage =36x0.85x9.81x12=**3602.232KW**

Q.N.10 From question given as following data

Head (H) = 16m, Q =15m^3/s, η=80%=0.80, Peak hour = 6 hour, losses =10%

Case (I)

- Firm capacity without pondage = ηxϒxQxH = 0.8x9.81x15x16=**1883.52KW**

Case (II) with pondage

- Total Volume of water stored during 18 hours(24 – 6) = 15x18x60x60 = **972000m^3**
- Loss of water due to evaporation = 10% of 972000 = **97200m^3**
- Net amount of water at reservoir = 972000 – 97200 = **874800m^3**
- Rate of water available from pondage= $\frac{874800}{6x60x60} = \mathbf{40.5\ m^3/s}$
- Total rate of water available to generate power(Q_T) = 15 + 40.5 = **55.5m^3/s**
- Firm power due to pondage = 0.8x9.81x16x55.5 = **6969.024KW**

Q.N.11 Same as Q.N.10

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Q.N. 12 From question given as following data

Head (H) = 70m, Q =30m^3/s, η=85%=0.85, Peak hour = 6 hour, losses =5%

Case (I) Power developed without pondage by 30m^3/s discharge is given by (P) = ηxϒxQxH

=0.85x9.81x30x70 = **17510.85KW**

Case (II) with pondage

From question state that, one unit is continuously operated during off peak hour i.e. (30/2) = 15m^3/s water is used.

So power **(P_1)** = ηxϒxQxH = 0.85x9.81x15x70=**8755.425KW**

- Total volume of water stored during 18 hours when only one unit is operating =(15m^3/s)x18x60x60 = **972000m^3**
- Loss of water = 5% of 972000 = **48600m^3**
- Net amount of pondage per day = 972000 – 48600 = **923400m^3**
- Rate of water available from pondage for 6 hours only = $\frac{923400}{6x60x60} = \mathbf{42.75m^3/s}$
- Total flow available from to generate power = 15 + 42.75 = 57.75 m^3/s
- Firm power due to pondage**(P_2)** = 0.85x9.81x57.75x70 = **33708.386KW**
- Total Power of the plant is **(P)** = P1+ P2= 8755. 425+33708.386 = **42463.811KW**

Q.N.13 From question given as following data

Head (H) = 10m, η=89%=0.89, flow available for % of time = 25%

- Calculation of design flow, $\frac{(Q-500)}{(25-20)} = \frac{(450-500)}{(30-20)}$ or Q = 474m^3/s
- i.e. installed capacity (P) = ηxϒxQxH = 0.89x9.81x475x10 = 41471.775KW = 41.472MW
- Detail calculation shown in table

Flow(m^3/s)	% of time	Power(MW)	Actual power(MW)	Energy (GWh)
900	1	78.578	**41.472**	3.633
600	10	52.385	**41.472**	32.697
500	20	43.655	**41.472**	36.329
475	**25**	**41.472**	**41.472**	**18.165**
450	30	39.289	39.289	17.687
400	40	34.924	34.924	32.505
350	50	30.558	30.558	28.681
340	60	29.685	29.685	26.387
300	70	26.193	26.193	24.475
280	80	24.447	24.447	22.180
200	90	17.462	17.462	18.356
140	93	12.223	12.223	3.901
100	100	8.731	8.731	6.425
				Total Energy =271.421

Sample calculation of Energy,

$$E_N = \left[\frac{T_N - T_{N-1}}{100}\right] * \left[\frac{P_{N(Actual)} + P_{N-1(Actual)}}{2}\right] * \left[\frac{365 * 24}{1000}\right]$$

$$E_1 = \frac{1-0}{100} * \frac{41.472 + 41.472}{2} * \frac{365 * 24}{1000} = 3.633\ GWh$$

$$E_2 = \frac{10-1}{100} * \frac{41.472 + 41.472}{2} * \frac{365 * 24}{1000} = 32.697\ GWh$$

$$E_3 = \frac{20-10}{100} * \frac{41.472 + 41.472}{2} * \frac{365 * 24}{1000} = 36.329 GWh$$

$$E_4 = \frac{25-20}{100} * \frac{41.472 + 41.472}{2} * \frac{365 * 24}{1000} = 18.165\ GWh$$

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$$E_5 = \frac{30-25}{100} * \frac{39.289 + 41.472}{2} * \frac{365*24}{1000} = 17.687GWh$$
$$E_6 = \frac{40-30}{100} * \frac{34.924 + 39.289}{2} * \frac{365*24}{1000} = 32.505GWh$$
$$E_7 = \frac{50-40}{100} * \frac{30.558 + 34.924}{2} * \frac{365*24}{1000} = 28.681GWh$$
$$E_8 = \frac{60-50}{100} * \frac{29.685 + 30.558}{2} * \frac{365*24}{1000} = 26.387GWh$$
$$E_9 = \frac{70-60}{100} * \frac{26.193 + 29.685}{2} * \frac{365*24}{1000} = 24.475GWh$$
$$E_{10} = \frac{80-70}{100} * \frac{24.447 + 26.193}{2} * \frac{365*24}{1000} = 22.180GWh$$
$$E_{11} = \frac{90-80}{100} * \frac{17.462 + 24.447}{2} * \frac{365*24}{1000} = 18.356GWh$$
$$E_{12} = \frac{93-90}{100} * \frac{12.223 + 17.462}{2} * \frac{365*24}{1000} = 3.901\ GWh$$
$$E_{13} = \frac{100-93}{100} * \frac{8.731 + 12.223}{2} * \frac{365*24}{1000} = 6.425GWh$$

- Firm Energy (Primary Energy)= $\frac{100-0}{100} * \frac{8.731}{1000} * 365 * 24 = \mathbf{76.484\ GWh}$

  $\boldsymbol{OR}\ \ Firm\ Energy = \frac{100-0}{100} * \frac{8.731+8.731}{2} * \frac{365*24}{1000} = \mathbf{76.484GWh}$
- Secondary Energy = Total Energy – Primary Energy =271.421 – 76.484 = **194.937 GWh**

Q.N.14 From question given as following data

Head (H) = 100m, η=90%=0.9, design flow = 900m^3/s

Month	Flow (m^3/s)	Flow in Descending order(m^3/s)	Rank	% of time	Power (MW)	Actual power(MW)	Energy (GWh)
Jan.	100	2510	1	8.333	2216.079	794.61	580.1117052
Feb.	120	2000	2	16.667	1765.8	794.61	580.1117052
Mar.	140	2000	3	25	1765.8	794.61	580.0420974
Apr.	300	1800	4	33.333	1589.22	794.61	580.0420974
May	320	900	5	41.667	794.61	794.61	580.1117052
Jun	1800	320	6	50	282.528	282.528	393.1396438
July	2000	300	7	58.333	264.87	264.87	199.792278
Aug.	2510	300	8	66.667	264.87	264.87	193.3705684
Sep.	2000	140	9	75	123.606	123.606	141.7880683
Oct.	900	120	10	83.333	105.948	105.948	83.78385851
Nov.	100	100	11	91.667	88.29	88.29	70.90254175
Dec.	300	100	12	100	88.29	88.29	64.44912193
							Total Energy = 4047.645391

- I.e. Total Energy = **4047.645391GWh**
- Primary Energy= **773.4204GWh**
- Secondary Energy= **3274.224991GWh**

Q.N.15 From question given as following data

Head (H) = 100m, η=86%=0.86, design flow = 1000m^3/s

Month	Flow (m^3/s)	Flow in Descending order(m^3/s)	Rank	% of time	Power (MW)	Actual power(MW)	Energy (GWh)
Jan.	150	1700	1	8.333	1434.222	**843.66**	**615.8718**

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Feb.	200	1300	2	16.667	1096.758	**843.66**	**615.8718**
Mar.	300	1200	3	25	1012.392	**843.66**	**615.8718**
Apr.	400	1100	4	33.333	928.026	**843.66**	**615.8718**
May	450	1050	5	41.667	885.843	**843.66**	**615.8718**
Jun	900	**1000**		**44.445**	**843.66**	**843.66**	205.2906
July	1200	900	6	50	759.294	759.294	390.05214
Aug.	1100	600	7	58.333	506.196	506.196	461.90385
Sep.	1050	450	8	66.667	379.647	379.647	323.332695
Oct.	1300	400	9	75	337.464	337.464	261.745515
Nov.	1700	300	10	83.333	253.098	253.098	215.55513
Dec.	600	200	11	91.667	168.732	168.732	153.96795
		150	12	100	126.549	126.549	107.777565
							Total Energy =**5198.984445**

- I.e. Total Energy = **5198.984445GWh**
- Primary Energy= **1108.56924 GWh**
- Secondary Energy= **4090.415205GWh**

Q.N.16 From question given as following data

Head (H) = 200m, η=88%=0.88, design flow = 200m^3/s

Month	Flow (m^3/s)	Flow in Descending order(m^3/s)	Rank	% of time	Power (MW)	Actual power(MW)	Energy (GWh)
Jan.	150	470	1	8.333	811.4832	**345.312**	**252.07776**
Feb.	130	360	2	16.667	621.5616	**345.312**	**252.07776**
Mar.	110	280	3	25	483.4368	**345.312**	**252.07776**
Apr.	90	270	4	33.333	466.1712	**345.312**	**252.07776**
May	120	230	5	41.667	397.1088	**345.312**	**252.07776**
Jun	280	**200**		**47.918**	**345.312**	**345.312**	**189.0986524**
July	360	190	6	50	328.0464	328.0464	61.40462987
Aug.	470	170	7	58.333	293.5152	293.5152	226.869984
Sep.	270	150	8	66.667	258.984	258.984	201.662208
Oct.	230	130	9	75	224.4528	224.4528	176.454432
Nov.	190	120	10	83.333	207.1872	207.1872	157.5486
Dec.	170	110	11	91.667	189.9216	189.9216	144.944712
		90	12	100	155.3904	155.3904	126.03888
							Total Energy =**2544.411**

- i.e. Total Energy = **2544.441 GWh**
- Primary Energy= **1361.22GWh**
- Secondary Energy= **1183.191 GWh**

Q.N.17 From question given as following data

Head (H) = 18.3m, η=70%=0.7, design power = 8515 KW

- **Calculation of number of days**
- Odd number = (no. of days = 31 days)
- Even number = (no of days = 30 days except Feb = 28 days)

1	2	3	4	5	6	7

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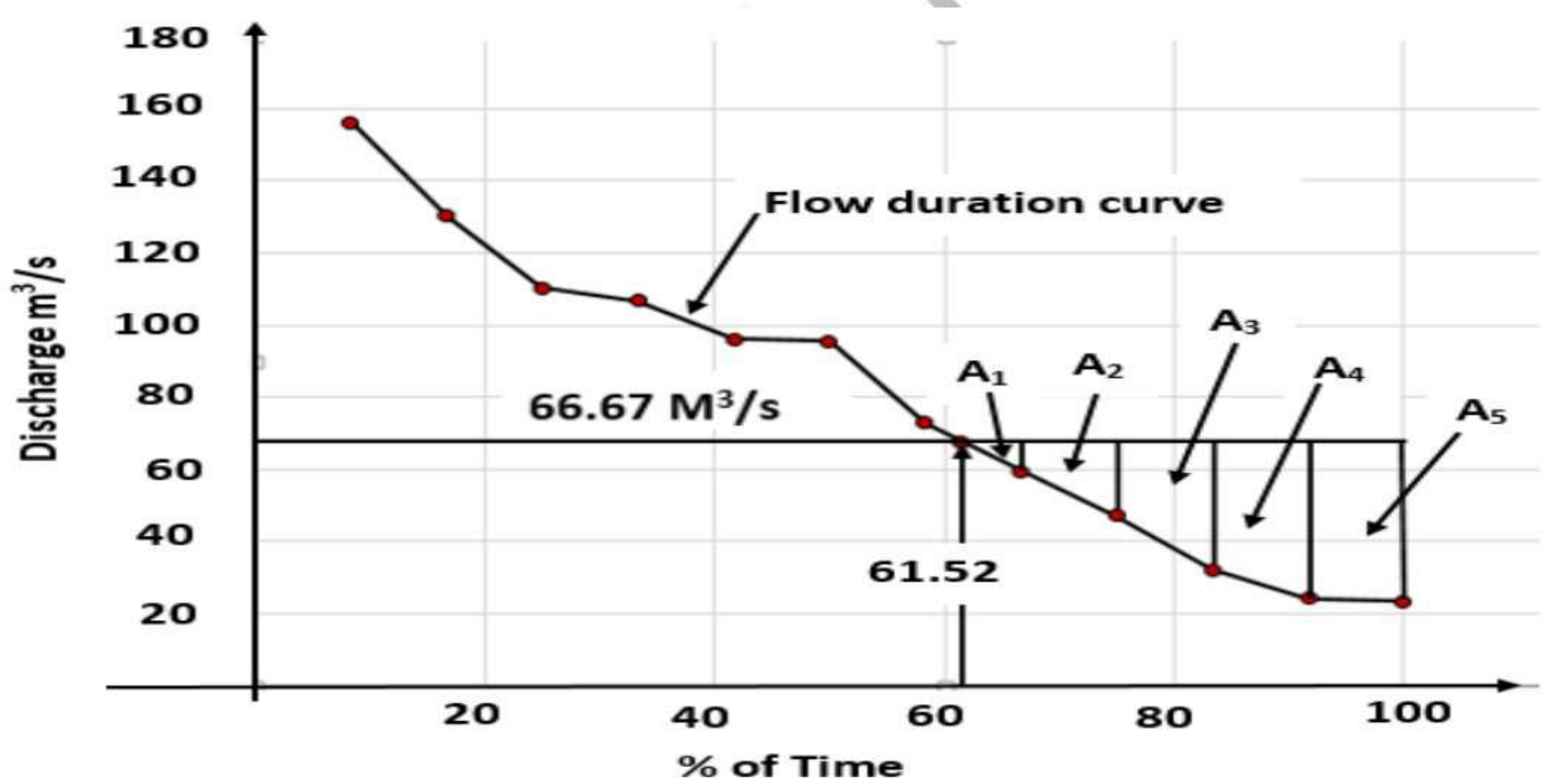
Jan	Feb	Mar	Apr	May	Jun	July
Aug	Sep	Oct	Nov.	Dec.		

- Calculation of design discharge $Q = \frac{\text{power}}{\eta x Y x H} = \frac{8515}{0.7x9.81x18.3} = 67.76\ M^3/s$

Month	Flow ($10^6 m^3$)	Days	Q(m^3/s)	Flow in Descending order(m^3/s)	Rank	% of time	Power (MW)
Jan.	349	31	130.30	156.25	1	8.333	19.64
Feb.	378	28	156.25	130.3	2	16.667	16.37
Mar.	286	31	106.78	110.34	3	25	13.87
Apr.	154	30	59.41	106.78	4	33.333	13.42
May	126	31	47.04	95.95	5	41.667	12.06
Jun	63	30	24.31	95.58	6	50	12.01
July	86	31	32.11	72.92	7	58.333	9.16
Aug.	63	31	23.52	**67.76**		**61.52**	**8.52**
Sep.	189	30	72.92	59.41	8	66.667	7.47
Oct.	257	31	95.95	47.04	9	75	5.91
Nov.	286	30	110.34	32.11	10	83.333	4.04
Dec.	256	31	95.58	24.31	11	91.667	3.05
				23.52	12	100	2.96

- Calculation of % of time, $\frac{(X-58.33)}{(67.76-72.92)} = \frac{(66.67-58.33)}{(59.41-72.92)}$ $i.e\ X = \mathbf{61.52\%}$



- **Calculation of storage capacity**

Design discharge	Actual Discharge	Deficit Discharge	% of time	Area	$Volume = \frac{\text{Area} * 365 * 24 * 60 * 60}{100}$
67.76	67.76	0	61.52	$A_o = 0$	0
67.76	59.41	8.35	66.667	$A_1 = 21.5$	6780240
67.76	47.04	20.72	75	$A_2 = 121.08$	38183788.8

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67.76	32.11	35.65	83.333	A_3 = 234.78	74040220.8
67.76	24.31	43.45	91.667	A_4 = 329.45	103895352
67.76	23.52	44.24	100	A_5 = 365.23	115178932.8
					Total = 338078534.4

I.e. Total storage volume = **338078534.4 M^3**

Q.N.18

- $Capacity(Plant)\ factor = \frac{\text{Average output of the plant}}{\text{Plant capacity}} = \frac{150 \text{ x } 10^6 \text{KWh}}{30\text{x}10^3\text{x}365\text{x}24} = 0.571 = 57.1\%$
- $Load\ factor = \frac{\text{Average load}}{\text{Peak load}} = \frac{150 \text{ x } 10^6}{25000\text{x}365\text{x}24} = 0.685 = 68.5\%$

Q.N.19

Let us analyze marginal benefit and cost for 1KW installation. Assuming Y% of the duration corresponds to the best installed capacity for 1KW power generation then,

- Total energy generated $=\frac{(Y\%\text{x}365\text{x}24)}{1000}$ MWH = 8.76 Y% MWH (Annual)= $\frac{(Y\text{x}365\text{x}24)}{1000\text{x}100}$ =0.0876 Y MWH
- Marginal benefit = 0.0876x U$ 30 = U$ 2.628Y……..(I)
- Total cost = Variable cost + O and M cost = $V_{cost}\left[\frac{(1+i)^N * i}{(1+i)^N - 1}\right] + 2\ \%\ of\ V_{cost}$

$$=U\$600\left[\frac{(1+0.1)^{40} * 0.1}{(1+0.1)^{40} - 1}\right] + \frac{2\text{x}600}{100} = U\$\ 73.36 \ldots\ldots\ldots\ldots\ldots\ldots (II)$$

Equating equation (I) and (II), we get

U$2.628Y = U$ 73.36 or, **Y = 27.915%** Hence the advantageous duration is 27.915%.

- Installed capacity calculated by interpolation, $\frac{(P-537)}{(27.915-25)} = \frac{(305-537)}{(33.33-25)}$

  P= 455.814 MW, Which is best installed capacity of the power plant.

Q.N.20 From question given as following data

Head (H) = 50m, η=85%=0.85

Month	Flow (m^3/s)	Flow in Descending order(m^3/s)	Rank	% of time	$Power = \frac{\eta\text{xYxQxH}}{1000}$ (MW)
Jan.	40	95	1	8.333	39.607875
Feb.	35	75	2	16.667	31.269375
Mar.	30	70	3	25	29.18475
Apr.	25	55	4	33.333	22.930875
May	20	50	5	41.667	20.84625
Jun	38	45	6	50	18.761625
July	75	40	7	58.333	16.677
Aug.	95	38	8	66.667	15.84315
Sep.	70	35	9	75	14.592375
Oct.	55	30	10	83.333	12.50775
Nov.	50	25	11	91.667	10.423125
Dec.	45	20	12	100	8.3385

Let us analyze marginal benefit and cost for 1KW installation. Assuming Y% of the duration corresponds to the best installed capacity for 1KW power generation then,

- Total energy generated $=\frac{(Y\%\text{x}365\text{x}24)}{1000}$ MWH = 8.76 Y% MWH (Annual)= $\frac{(Y\text{x}365\text{x}24)}{1000\text{x}100}$ =0.0876 Y MWH
- Marginal benefit = 0.0876x U$ 40 = U$ 3.504Y……..(I)
- Total cost = Variable cost + O and M cost = $V_{cost}\left[\frac{(1+i)^N * i}{(1+i)^N - 1}\right] + 2.5\ \%\ of\ V_{cost}$

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$$=U\$650\left[\frac{(1+0.13)^{45}*0.13}{(1+0.13)^{45}-1}\right]+\frac{2.5x650}{100}=U\$\ 101.097\ \ldots\ldots\ldots\ldots\ldots\ldots(II)$$

Equating equation (I) and (II), we get

U$ 3.504Y = U$ 101.097 or, **Y = 28.852%** Hence the advantageous duration is 28.852%.

- Installed capacity calculated by interpolation, $\frac{(P-29.18475)}{(28.852-25)}=\frac{(22.930875-29.18475)}{(33.33-25)}$

  P= 26.2928 MW, Which is best installed capacity of the power plant.

Q.N.21 From question given as following data

Head (H) = 100m, η=85%=0.85

Month	Flow (m^3/s)	Flow in Descending order(m^3/s)	Rank	% of time	$Power=\frac{\eta xYxQxH}{1000}$ (MW)
Jan.	146	2061	1	8.333	1718.56485
Feb.	125	1850	2	16.667	1542.6225
Mar.	134	1802	3	25	1502.5977
Apr.	268	1140	4	33.333	950.589
May	417	762	5	41.667	635.3937
Jun	1140	439	6	50	366.06015
July	1850	417	7	58.333	347.71545
Aug.	2061	288	8	66.667	240.1488
Sep.	1802	268	9	75	223.4718
Oct.	762	146	10	83.333	121.7421
Nov.	439	134	11	91.667	111.7359
Dec.	288	125	12	100	104.23125

Let us analyze marginal benefit and cost for 1KW installation. Assuming Y% of the duration corresponds to the best installed capacity for 1KW power generation then,

- Total energy generated $=\frac{(Y\%x365x24)}{1000}$ MWH = 8.76 Y% MWH (Annual)= $\frac{(Yx365x24)}{1000x100}$ =0.0876 Y MWH
- Marginal benefit = 0.0876x U$ 40 = U$ 3.504Y……(I)
- Total cost = Variable cost + O and M cost = $V_{cost}\left[\frac{(1+i)^N*i}{(1+i)^N-1}\right]+2\ \%\ of\ V_{cost}$

$$=U\$500\left[\frac{(1+0.1)^{40}*0.1}{(1+0.1)^{45}-1}\right]+\frac{2x500}{100}=U\$\ 61.1297\ \ldots\ldots\ldots\ldots\ldots\ldots(II)$$

Equating equation (I) and (II), we get

U$ 3.504Y = U$ 61.1297 or, **Y = 17.446%** Hence the advantageous duration is 17.446%.

- Installed capacity calculated by interpolation, $\frac{(P-1542.6225)}{(17.446-16.667)}=\frac{(1502.5977-1542.6225)}{(25-16.667)}$

  P= 1538.8808 MW, Which is best installed capacity of the power plant.

Q.N.22 From question given as following data

Gross Head (H) = 229m, Head loss = 9m, η=95%x92%x90%=0.95x0.92x0.90 = 0.7866

I.e. Net head (h) = 229 – 9 = 220m

Month	Flow (m^3/s)	D/S released Flow(m^3/s)	Remaining Discharge (m^3/s)	Flow in Descending order(m^3/s)	Rank	% of time	$Power=\frac{\eta xYxQxh}{1000}$ (MW)
Jan.	40	8	84	2536	1	8.333	4305.215
Feb.	35	8	80	2008	2	16.667	3408.861
Mar.	30	8	106	1448	3	25	2458.183
Apr.	25	8	202	1004	4	33.333	1704.4307
May	20	8	587	825	5	41.667	1400.553
Jun	38	8	1004	587	6	50	996.515

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July	75	8	2008	202	7	58.333	342.923
Aug.	95	8	2536	187	8	66.667	317.459
Sep.	70	8	1448	118	9	75	200.3215
Oct.	55	8	825	106	10	83.333	179.949
Nov.	50	8	187	84	11	91.667	142.602
Dec.	45	8	118	80	12	100	135.811

Let us analyze marginal benefit and cost for 1KW installation. Assuming Y% of the duration corresponds to the best installed capacity for 1KW power generation then,

- Total energy generated $=\frac{(Y\%x365x24)}{1000}$ MWH = 8.76 Y% MWH (Annual)= $\frac{(Yx365x24)}{1000x100}$ =0.0876 Y MWH
- Marginal benefit = 0.0876x U$ 48 = U$ 4.2048Y……..(I)
- Total cost = Variable cost + O and M cost = $V_{cost}\left[\frac{(1+i)^N*i}{(1+i)^N-1}\right] + 2.5\ \%\ of\ V_{cost}$

$$=U\$500\left[\frac{(1+0.12)^{40}*0.12}{(1+0.12)^{40}-1}\right] + \frac{2.5x500}{100} = U\$\ 73.152 \ldots\ldots\ldots\ldots\ldots\ldots (II)$$

Equating equation (I) and (II), we get

U$ 4.2048Y = U$ 73.152 or, **Y = 17.397%** Hence the advantageous duration is 17.397%.

- Installed capacity calculated by interpolation, $\frac{(P-3408.861)}{(17.397-16.667)} = \frac{(2458.183-3408.861)}{(25-16.667)}$

P= 3325.578 MW, Which is best installed capacity of the power plant.

Q.N.23 From question given as following data

Head (H) = 100m, η=96%x93%x98%x99%=0.96x0.93x0.98x0.99= 0.866

Month	Flow (m^3/s)	Flow in Descending order(m^3/s)	D/S released Flow(m^3/s)	Remaining Discharge (m^3/s)	Rank	% of time	$Power = \frac{\eta xYxQxH}{1000}$ (MW)
Jan.	280	2492	8.4	2483.6	1	8.333	2109.932
Feb.	250	2380	8.4	2371.6	2	16.667	2014.783
Mar.	210	2156	8.4	2147.6	3	25	1824.485
Apr.	84	1064	8.4	1055.6	4	33.333	896.781
May	84	868	8.4	859.6	5	41.667	730.269
Jun	210	659	8.4	650.6	6	50	552.715
July	868	280	8.4	271.6	7	58.333	230.737
Aug.	1064	250	8.4	241.6	8	66.667	205.250
Sep.	2380	210	8.4	201.6	9	75	171.268
Oct.	2492	210	8.4	201.6	10	83.333	171.268
Nov.	2156	84	8.4	75.6	11	91.667	64.226
Dec.	659	84	8.4	75.6	12	100	64.226

Let us analyze marginal benefit and cost for 1KW installation. Assuming Y% of the duration corresponds to the best installed capacity for 1KW power generation then,

- Total energy generated $=\frac{(Y\%x365x24)}{1000}$ MWH = 8.76 Y% MWH (Annual)= $\frac{(Yx365x24)}{1000x100}$ =0.0876 Y MWH
- Marginal benefit = 0.0876x U$ 48 = U$ 3.504Y……..(I)
- Total cost = Variable cost + O and M cost = $V_{cost}\left[\frac{(1+i)^N*i}{(1+i)^N-1}\right] + 3\ \%\ of\ V_{cost}$

$$=U\$600\left[\frac{(1+0.1)^{40}*0.1}{(1+0.1)^{40}-1}\right] + \frac{3x600}{100} = U\$\ 79.356 \ldots\ldots\ldots\ldots\ldots\ldots (II)$$

Equating equation (I) and (II), we get

U$ 3.504Y = U$ 79.356or, **Y = 22.647%** Hence the advantageous duration is 22.647%.

- Installed capacity calculated by interpolation, $\frac{(P-2014.783)}{(22.647-16.667)} = \frac{(1824.485-2014.783)}{(25-16.667)}$

P= 1878.219MW, which is best installed capacity of the power plant.

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Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

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4. Head works of Storage Plants

4.1 General Arrangement of components in a typical storage power plant: Spillways, bottom outlets or Under sluices, intakes with examples.

4.2 Dam Engineering

- 4.2.1 Classification based on materials, function and head
- 4.2.2 Principal variants of concrete and embankment dams
- 4.2.3 Dam site evaluation and selection of type of dam
- 4.2.4 Loads on dams and their combinations
- 4.2.5 Failure modes of concrete and embankment dams and their remedies
- 4.2.6 Gravity (concrete) dam analysis, stability (overturning, sliding), stress and material failure
- 4.2.7 Seepage Control and foundation treatment in Dams : Types of grouting and drainage and their necessity,
- 4.2.8 Embankment Dam Analysis-phreatic line and seepage analysis.

4.3 Intakes

General arrangement of Intakes for storage plants, Location, Hydraulics of intake

4.4 Spillways and Energy Dissipaters

- 4.4.1 Purpose of spillways, general arrangement, types, and hydraulics (sizing) of spillways, Cavitation in spillways, preventive measures
- 4.4.2 Methods of dissipating energy below a dam, stilling basin, ski-jump and flip buckets, their suitability, hydraulics of stilling basin, the role of tail-water in energy dissipation

4.5 Types of gates and their location.

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4.1 General Arrangement of components in a typical storage power plant: Spillways, bottom outlets or Under sluices, intakes with examples:

Storage Hydro Plant

- Storage based hydro plant is essential to increase potential of power of the country and also to fulfill the demand of power in dry season by using surplus storage water during wet season.
- The economic power generation depends upon the type of large and high dam and terrain of the site for tapping the water of the summer season.
- Storage hydro plant serves as a peak load plant.
- There is only one large storage power plant {Kulekhani (60 MW)} in Nepal yet.
- If can't fulfill the demand of power at peak time so that load shedding problems may rise every year due to the increase of demand of power.
- Proposed storage type hydropower will be Karnali hydropower plant (10,800 MW) at Chisapani which will be the biggest hydropower plant in Nepal after construction of the project.
- The large water storage projects are essentially better than medium and small storage projects because of the analysis of the parameters such as evaporation losses of land area submergence, the volume of the water, geological and topographical considerations, environmental effects and power potential development.
- To address growing energy needs and to reduce power cutoff problems of the country, it is necessary to develop high power potential projects like Chisapani, Kaligandaki II, Seti (west) and Budhi Gandaki as soon as possible through tapping of the water by storage power projects.

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4.1 General Arrangement of components in a typical storage power plant: Spillways, bottom outlets or Under sluices, intakes with examples:

Component of the storage power plant:

The component of storage hydropower development project may be site specific. It is different in different site and also depends upon the type of the storage project. The typical components of the storage power development are;

i) Dam and reservoir

ii) Spillway

iii) Intake with trash rack

iv) Headrace tunnel

v) Surge shaft

vi) Penstock

vii) Power house

viii) Tailrace tunnel/ channel

4.2.1 Classification of dams

Dam may be classified in various ways. They can be classified based on their function, shape, material of construction and hydraulic and structural design.

A) Classification based on function:

Dam can be classified based on their function as

a) Storage dam:

- Storage dam are mainly for storage water and using it for various uses such as hydropower, irrigation, and water supply.
- A storage dam may be constructed of wide variety of materials such as stone, concrete, earth and rock fills etc.

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A) Classification based on function:…

b) Diversion dam:

- Diversion dam are constructed to raise the water level and to divert the river flow in another direction.
- A diversion dam is therefore, of smaller height and no reservoir is formed to store water.
- Weir and barrage are the best examples of the diversion dams.

c) Detention dams: They are constructed primarily to store flood waters and release it gradually at a constant, uniform rate after receded of flood.

d) Coffer dam: It is a temporary nature dam to make the dry space for the construction of the project.

- It is constructed upstream of main dam to divert water into a diversion channel during the construction of the dam. *Actually coffer dam is not a dam.*
- It is a temporary dam constructed to facilitate construction of actual dam or other structure on the river.

e) Debris dam: A debris dam is constructed to retain debris such as sand, gravels and drift wood flowing in the river with water.

- It is constructed for erosion control e.g. check dam (sabo-dam).
- Detention dams are constructed across tributaries carrying large silt and sediment, in such case the detention dam is known as Debris dam.

B) Based on construction material used

a) Concrete dam: Concrete dam are made either from plain cement concrete or reinforced cement concrete.

- Concrete dams are demanding with respect to foundation conditions, requiring sound and stable rock.
- The *concrete dam may be classified* according to the structural load transfer as gravity dam, arch dam and buttress dam.

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B) Based on construction material used

i) Gravity dam:

- It is one in which the retained water, uplift, earthquake and wave thrust is resisted by gravity action.
- Therefore a concrete gravity dam is entirely dependent upon its own weigh for stability.
- It may be constructed either of masonry or of concrete. However, large heights and important gravity dams are now constructed of concrete only

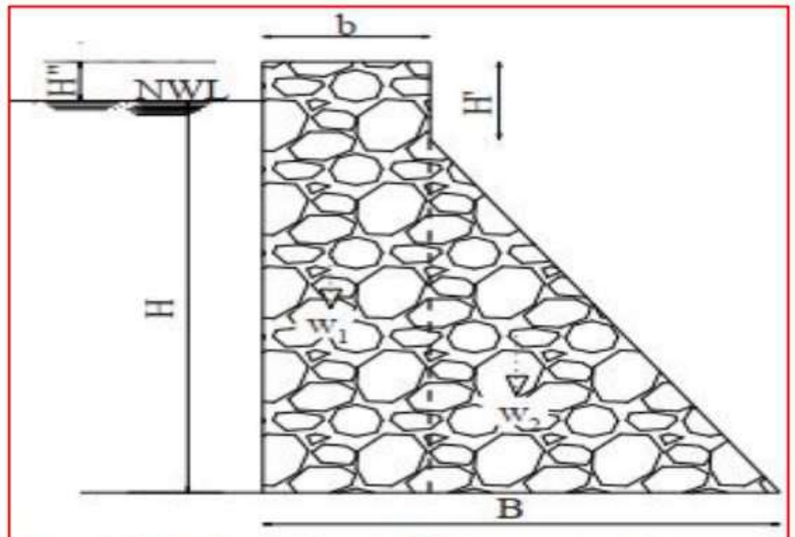

Fig. 17.2 Section of Concrete Dam

Advantages of gravity dams

- Gravity dams are quite strong, stable and durable.
- It is well adapted for use as an overflow spillway section.
- It is most suitable across wide valleys and gorges having steep slopes.
- It does not fail suddenly.
- It can be constructed during all types of climatic conditions.
- The maintenance cost is very low.
- The sedimentation in the reservoir on upstream of dam can be reduced by operation of deep bypass sluices.
- It can be constructed to very great heights, provided good rock foundations are available.

Disadvantages of gravity dam

- Initial cost is more high than earth dams.
- Require more skilled labor.
- Usually take longer time in Construction.
- Suitable only in sound rock foundation.
- Subsequent raising is not possible

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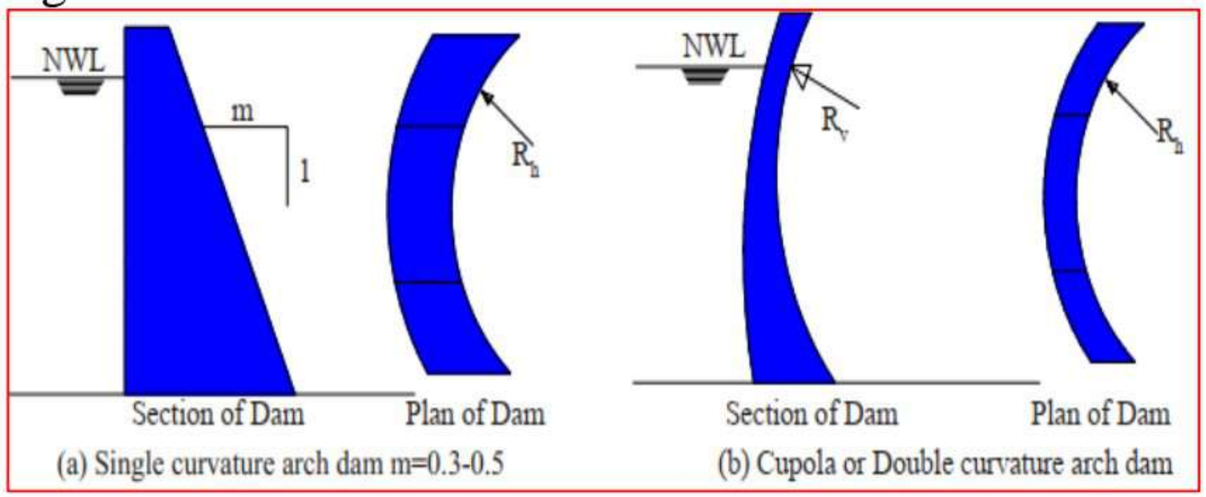
B) Based on construction material used….

ii) Arch dam:

- It is one where the thrust is resisted by the arch action.
- Structurally, it functions primarily as a horizontal arch or single curvature, transmitting the major portion of the water load to the abutment or valley sides rather than to the floor of the valley.
- It is more efficient than gravity and buttress dam, greatly reducing the volume of concrete required.
- It may be double curvature arch dam. It introduces complex curvatures in the vertical as well as the horizontal plane.
- The double curvature arch dam is an extremely strong and efficient structure and can be built in a narrow valley with competent abutments.



(a) Single curvature arch dam m=0.3-0.5 (b) Cupola or Double curvature arch dam

Advantage of arch dams

- Arch dams are suited to narrow , v-shaped valley.
- Uplift pressure is not an important factor in the design.
- Require less concrete.
- Can be constructed on relatively less strong foundation because a small part of load is transferred to base.

Disadvantages of Arch Dam:

- The construction speed is generally low.
- This dam construction requires skilled labor.
- It needs classy formwork.
- Also, requires a special design.
- It is not suitable in places where strong abutments are not available.

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B) Based on construction material used….

iii) Buttress dam:

- Resist the water thrust with the help of buttress which support the water through an inclined structural member such as a slab.
- In structural concept the buttress dam consists of a continuous upstream face supported at regular intervals by downstream buttresses.
- Buttress dam may be considered for conceptual purposes as a modified form of the gravity dam.

Advantages of buttress dams

- Require less concrete than gravity dams.
- Uplift pressure is generally not a major factor in the design of buttress dams.
- Less time for construction.
- Easily subsequently by extending buttresses and deck slabs.
- It can be designed to accommodate moderate movements of foundations without serious damages.
- Power house can be housed between buttresses.

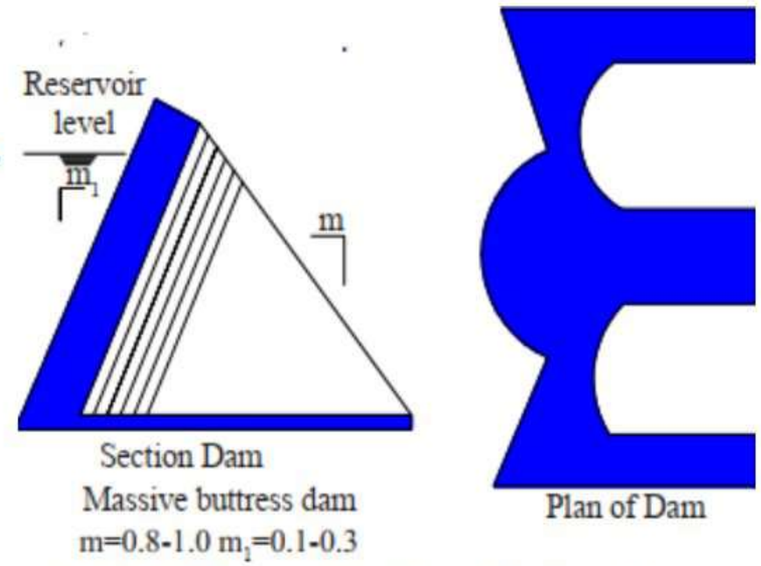

Fig 17.4 Plan and Section of Buttress Dam

b) Earthen dam:

Earthen dams are built mainly from soil. Early time, earthen dams for the storage of water for irrigation purpose have been constructed of low height, as these were designed by empirical methods and based on only experience. With development of new construction technology, very high earthen dams were built in nearly 19th century. Some of earthen and rockfill dams are Nurek dam, Tajikistan (300m), mica dam, Canada (244m), Tehri Dam, India (261m) and Orville dam, USA (235m).

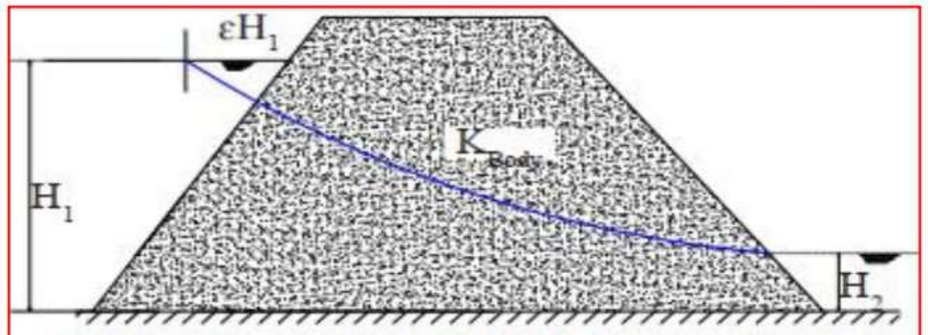

Fig 17.5 Typical Earth Fill Homogenous Dam

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B) Based on construction material used….

c) Rock fill dam:

- A rock fill dam, like an earthen dam, is composed of fragment material.
- The size of material is large. These dams are constructed with mainly from rock-fill or pieces of rocks.
- The rock-fill dam has an impervious membrane to check the seepage of water through it. Recently, composite earth and rock fill dams have also been adopted.

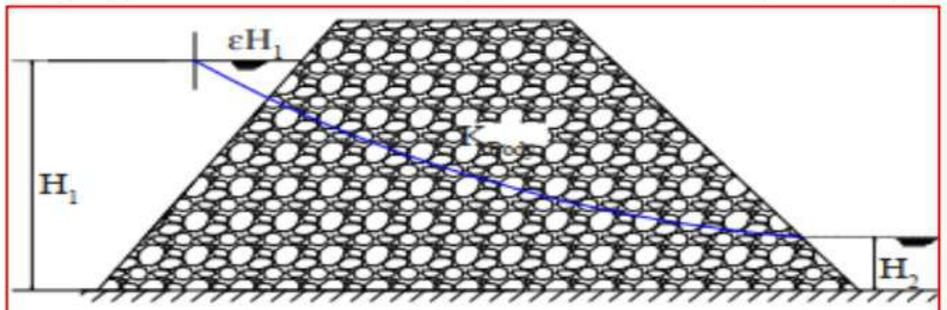

Fig 17.6 Typical Rock Fill Homogenous Dam

d) Wooden dam:

- The wooden dam is constructed of framework of wooden struts and beams with wooden plank facing to resist water pressure.
- A wooden dam is an ideal temporary dam. They are suitable to place where wood is available in plenty.

Advantages of wooden dam

- Low initial cost.
- Suitable for many type of foundation.
- Suitable for temporary dams.
- Early and quicker construction.

Disadvantages of the wooden dam

- High maintenance cost.
- Short lifetime.
- Suitable for small heights.
- Greater seepage loss through the body of the dam.

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B) Based on construction material used….

e) Steel dam:

Steel dams are constructed with a framework of steel with a thin plate as deck slab on the upstream side.

Advantages of steel dams

- Steel dam can be constructed with great speed with the modern methods of fabrication.
- It can easily withstand stress due to unequal settlement.
- It may be cheaper than gravity dam.
- It can be fabricated with more confidence.
- It can be easily repaired.
- Not affected by frost action.

Disadvantages

- Require regular maintenance.
- Unable to withstand stresses due to shocks of spilling water and vibrations.
- Require strong anchorages in foundation.
- Short life period.

f) Masonry dam:

- The masonry dams are constructed with stone fragments in cement mortar.
- They are mostly gravity dam.
- The masonry dam, being man oriented rather than machine oriented , could be considered *under following two conditions.*

- Where large scale unemployment problem is existing
- Where concreting may be difficult due to the non availability of either cement or the concreting plant

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C) Based on hydraulic consideration

- Overflow dam
- Non- over flow dam

a) Overflow dam:

- Water is allowed to flow over the dam.
- Concrete gravity dam may or may not be overflow dams.
- Usually for concrete gravity dam, part of the length of dam is designed for overflow while the rest is designed for non overflow.

b) Non-overflow dam:

- Water is not allowed to flow over the dam.
- The entire embankment dams are non - overflowed.

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4.2.3 Selection of type of dam: Various considerations in the selection of type of dam are topography, geography and nature of foundations, spillway location, seismicity of the area, economic life of dams. Following are the governing criteria on selection of the type of dam.

SN	Character of site	Condition of Site	Type of Dam
1	**Availability of construction materials**	• Ingredients of concrete i.e. sand gravel and cement- locally available.	Concrete gravity dam
		• Locally available of the previous and impervious soils and stone	Earth/rock-fill dam
		• Ingredients of concrete involve long distance carriage	Buttress dam
		• Available stone in required size and specific gravity.	Stone masonry dam
2	**Foundation character**	• Impervious solid rock such as schist and gneiss	Any type
		• Gravel and coarse sands	Arch- rock fill dam
		• Sand and silt clay	Earthen dam
		• Very fine and uniform sands	Earthen dam
		• Clay, plastic clay	Earthen dam with very flat slopes
		• Silt and fine sand	Low gravity dam or earthen dam
		• Heavy overburden on rock layer	Earthen or buttress dam

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4.2.3 Selection of type of dam

S N	Character of site	Condition of site	Type of Dam
3	**Character of gorge**	• V- shaped narrow gorge	Arch dam
		• U shaped narrow gorge	Overflow concrete dam
		• Low –wide gorge	Earth/rock fill dam
4	**Spillway requirement**	Spillway location separate from main dam	Earthen dam
		Adequate width for dam only	Gravity dam with overflow spillway
		Large spillway requirements	Overflow concrete dam
5	**Character of the region**	Seismic zone	Earth dam and concrete gravity dam
		Excessive uplift pressure exerted	Arch dam
6	**Climatic Condition**	Rainy zone	Concrete dam
		Frequently change in climatic condition	Earthen dam
7	**Design period of dam & Environmental condition**	High deign period	Earthen dam
		Moderate period	Concrete dam

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4.2.3 Site selection for dam

The following are the requisites of good sites for various types of dams.

i) Location: Below river confluence to get the more water.

ii) Type of gorges : Have narrow gorge to reduce the length of the dam.

iii) Sediment tributaries : Without tributaries with high sediment.

iv) Availability of the construction materials: Materials required for a particular type of dam should be available nearby, without requiring much of transportation.

v) Foundation condition: Suitable foundations should be available at the site selected for a particular type of dam.

vi) Geological condition: The geological conditions of the catchment area should be such that minimum percolation losses with *maximum runoff*.

vii) Development of the backward area: The dam site should be selected to develop the backward area.

viii) Landslide: The surroundings near the site should free from landslide zones.

ix) Land and property submergence: The cost of the submerged land and property due to the construction of reservoir dam should be within the acceptable limit.

x) Relocation of the settlement: The reservoir site should be located where the existing settlement can be relocated with minimum cost if required.

xi) Accessibility: Dam site should be accessible which is connected by a road for transportation of cement, labour, machinery parts and other required equipment.

xii) Spillway suitability: Good site for the location of a separate spillway is essential especially in case of earth or rock fill dam.

- However in case of gravity dam, spillway may be located at its middle. Hence site should be selected where; the spillway can be located in suitable location of dam.

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4.2.2 Dam and reservoir

- A reservoir is created with impounding of runoff from the catchment upstream by the construction of dam.
- It is a hydraulic structure built across a river or a stream to store part of runoff from the catchment upstream of the dam.
- Stream flow changes within very wide limits from season to season largely conforming to the pattern of rainfall and snow melting in snow fed rivers.
- It also fluctuates from year to year.
- The rate at which water is required to meet demands of hydropower, irrigation, water supply, flood control rarely conform to the pattern of water availability. Hence the need to conserve water during wet periods for use during dry periods.
- Construction of dam across the river is an important component for hydro-power projects.
- The primary purpose of a dam is to provide safe retention and storage of water.
- Dams are individually unique structures.
- Irrespective of size and numerous types they demonstrate great complexity in their load response and in their interaction relationship with site hydrology and geology.
- A dam may be classified in to **two groups** on the basic of utilization of construction materials.

i) Embankment dam:

- It is built with the natural construction materials such as earth and/or rock.
- The slopes of upstream and downstream face of dam may be either similar or moderate angle depending upon height of dam and purposes and it has high construction volume relative to height of dam.

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4.2.2 Dam and reservoir…

ii) Concrete dam:

- It is constructed by means of mass concrete.
- Upstream face may be generally near vertical slope and downstream face has been steep slope relative to upstream slopes.
- According to world register statistics by the international commission on large dams (ICOLD 1988a), 80-90% of the all large dam are embankment dam, 11.3% are concrete gravity and Arch and Buttress dam are 5.8%.
- The dam built at Sadd-el-kafara in Egypt around 2600BC is the earliest embankment dam.
- It was constructed with an earth-fill center flanked by rock shoulders and with rubble masonry face protection with height of 14m.
- The first significant masonry dam is constructed in Turkey at 10m high Kesis Golu.
- The 12m high and 18m long Baume arch dam in France was completed by the Romans in the second century AD.
- The highest dam was the earth rock embankment dam called Nurek dam in Tajikistan which is over 300 m high and was completed 1980 AD.
- A 235m high embankment dam is in progress in Tajikistan which is also the next highest embankment dam in the world.

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Function of dam

The dam fulfills two fundamental functions

- As a reservoir , which has a capacity to store water
- Built up potential head for the river water.

4.2.6 Gravity (Concrete) Dams Analysis

Gravity dams are constructed either in stone masonry or in concrete. Now a day stone masonry dam has been almost replaced by concrete dam because of the masonry dams being man oriented rather than machine oriented. Concrete dam can be classified into three types based on the structural load transfer. They are gravity dam, arch dam and buttress dam.

4.2.4 Load in gravity dam design

For analyzing the structural stability of dam, following force has to be taken into consideration.

- Self weight of dam (W)
- Hydrostatic load (P)
- Reaction of the foundation (R)
- Uplift pressure due to subsoil flow (U)
- Wind and wave forces (Pw)
- Earthquake forces
- Silt pressure due to deposited silt in the reservoir (Ps)
- Pressure due to ice formation at water surface (Pi)

Relatively important loads are as follows.

a) Primary loads

b) Secondary loads

c) Exceptional loads

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4.2.4 Load in gravity dam design…

a) Primary loads:- primary loads are recognized as universally applicable and of prime importance to all type dams. The primary loads are obtained by Water pressure forces, corresponding seepage loads, and self weight of the particular dam.

b) Secondary loads:- Secondary loads are generally discretionary and of less important in magnitude and it may become major importance only to certain types of dams. Secondary loads occur due to sediment load, thermal load, ice load and hydrostatic wave load etc.

c) Exceptional loads:- Exceptional loads are so considered according to the limited applicability or having a low probability of occurrence. These loads occur due to tectonic effect or inertia loads and seismic loads.

a) Primary Load

i) Hydrostatic pressures force:-

The horizontal component of hydrostatic force acts on the upstream and downstream faces of the dam. The upstream water pressure force is the overturning force acting on the dam. The tail water pressure force acts as for stability of dam.

- Upstream horizontal component of hydrostatic pressure force $(P_H) = \frac{1}{2} * \gamma_w * H^2$
- Tail water horizontal component of hydrostatic pressure force $(P_H{}') = \frac{1}{2} * \gamma_w * H'^2$

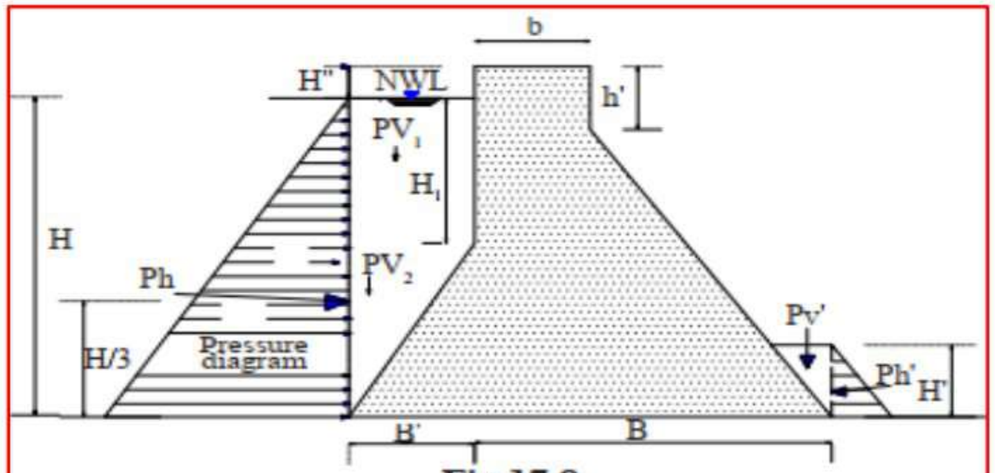

Fig 17.8

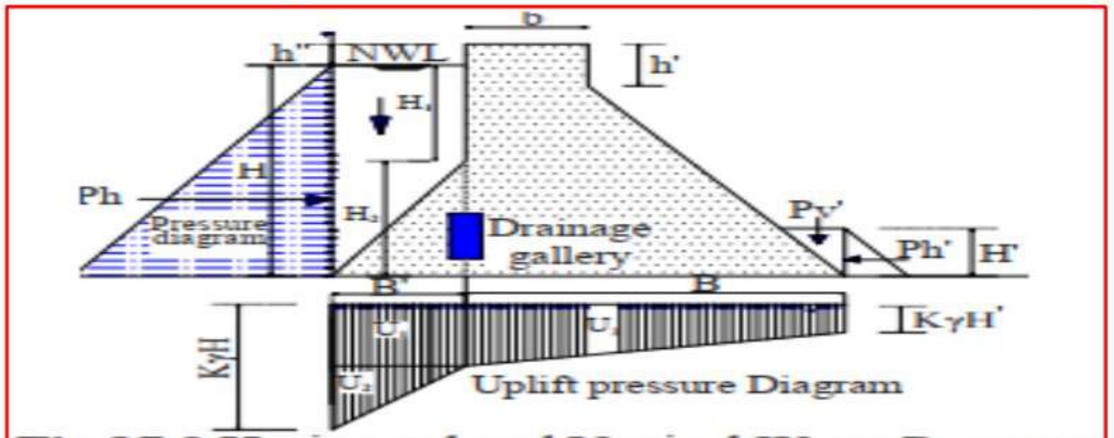

Fig 17.9 Horizontal and Vertical Water Pressure Force

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a) Primary Load

i) Hydrostatic pressures force:-

- The resultant horizontal component of hydrostatic pressure force acts H/3 from the base of the gravity dam.
- The resultant vertical component of force (P_{V1}) must be considered if the upstream face is inclined.

P_{V1} = $\gamma_w \times$ area of the water supported by the flare, and acts through the centroid of area.

- The pressure of permanent tail water,

$P_V{}'$ = $\gamma_w \times$ area of the water supported by the downstream flare, and acts through the centroid of area.

ii) Self weight load:- Structure self weight per unit length is computed in terms of its resultant (W) force of the structure of dam. It is measured to act through the centroid of the cross sectional area A_P of the dam profile. For convenience, the cross section of the dam is divided into simple geometric shapes, the areas and corresponding centroid of these shapes can be easily determined for computation of the weights.

The total weight of the dam (W) = $\gamma_C\ (A_1 + A_2 + A_3) = \gamma_C * A_P$

Where, γ_C = unit weight of concrete. It is usually taken as 24 KN/m3

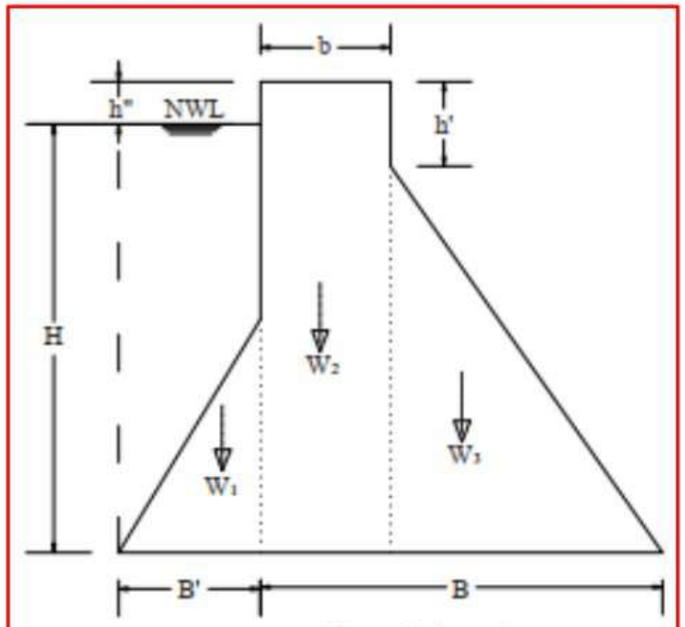

Fig : 17.10 Self Weight of Dam

iii) Seepage and uplift pressure:- Uplift pressure is the second major external force acting upward due to seeping water. It occurs due to internal pressure in pores, formation of cracks within the body of the dam, at the contact surface between the dam and its foundation and within the foundation. Uplift force (Pu) is represented by the resultant effective vertical components interstitial water pressure. Uplift force is equal to the product of unit weight of water and area of uplift diagram.

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iii) Seepage and uplift pressure…..

There are three different possibilities,

a) Full uplift acting but without tail water

b) Full uplift acting with tail water

c) Uplift acting with drainage gallery

The uplift pressure at any point depends upon the depth of water at that point. The pressure intensity at the base of the upstream base is equal to the hydrostatic pressure corresponding to the full reservoir level. The uplift pressure force on the base of the given dam without drainage gallery with tail water U=U1+U2 =Kγ

$U = U_1 + U_2 = K_\gamma * \left[\{H' * (B + B')\} + \frac{1}{2}\{(H - H') * (B + B')\}\right]$ acting at the centroid of the pressure diagram. Where, K=uplift coefficient, value varies from 0 to 1, depending upon the type of foundation of the dam.

- The pressure intensity at the drainage gallery is usually given by the empirical Formula

$(u) = K * \gamma * \left[H' + \frac{1}{3}(H - H')\right]$

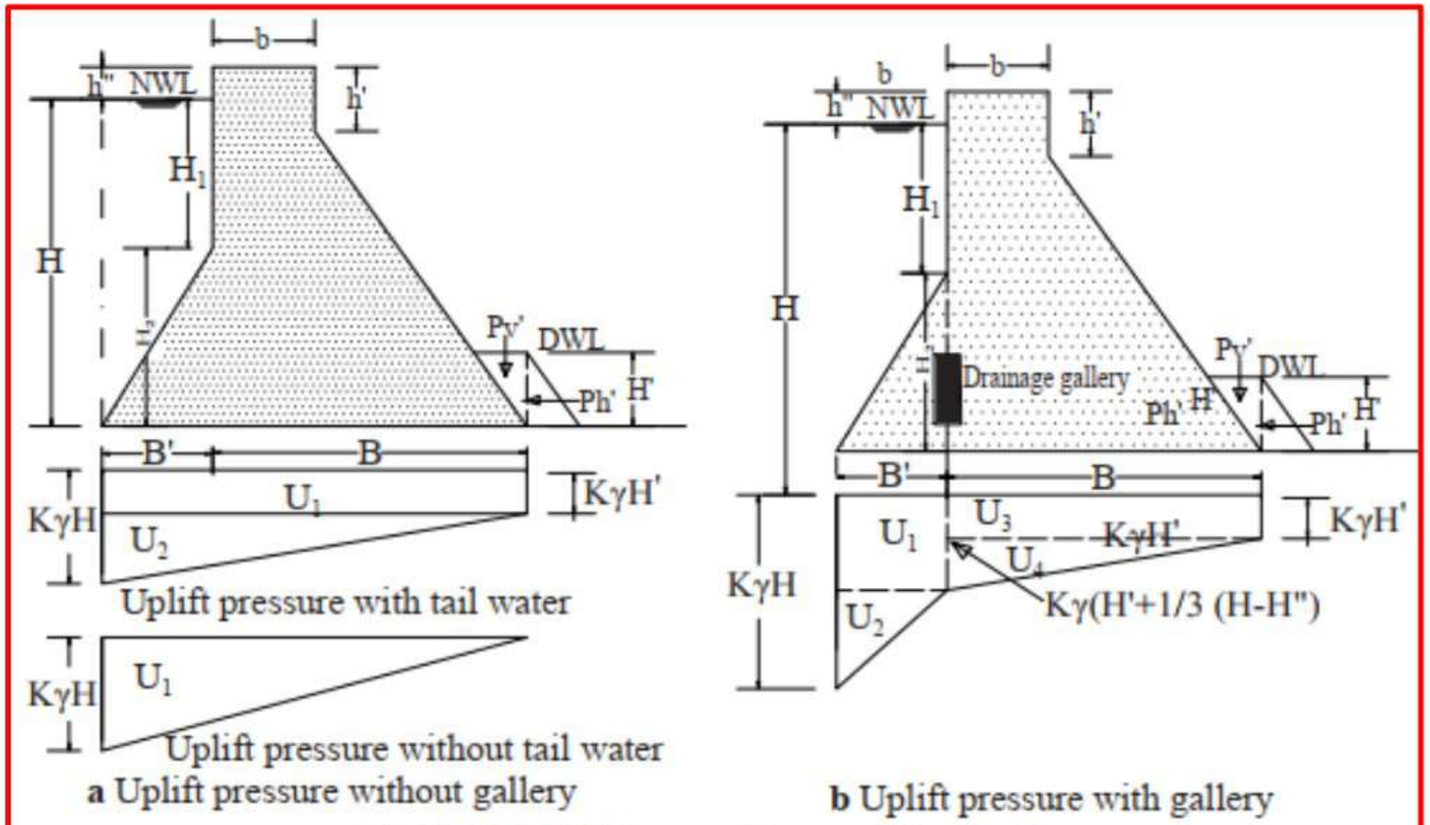

Fig 17.11 Uplift Pressure Force on Gravity Dam

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b) Secondary load

(i) Silt pressure force:

- Silts have a minor effect on the stability of the gravity dam. It is deposited in the reservoir on the upstream of the dam.
- Dam is thus subjected to silt pressure in addition to the water pressure. The deposited silt upstream of dam is treated as saturated cohesion-less soil developing full uplift.
- The angle of internal friction is materially altered on adaptation of submergence.
- The silt pressure can be evaluated by Rankin's theory.
- In the case of the vertical upstream face, the horizontal force due to Silt pressure $(\mathbf{Ph_{silt}}) = \frac{1}{2} * K_a * \gamma_S{}' * H_2^2$ acting at a height $\frac{H_2}{3}$ above the base of the Dam.

Where K_a = active lateral pressure coefficient = $\frac{1-\sin\varnothing_s}{1+\sin\varnothing_s}$

γ_s' = submersed unit weight of silt.

H_2 =Depth of the silt above the bed and

$\varnothing_s$= angle of shearing resistance of the sediment. ($value of \varnothing_s = 30^0$)

$\gamma_s' = \gamma_s - \gamma_w$ where γ_s = sediment saturated unit weight, value of γ_s=18-20KN/m^3

Vertical forces acting at the dam body of unit length due to the silt (Pv_{silt}) = weight of silt × Ka

But in actual, silt deposition is not isotropic in nature, so the analysis is done with the assumption of the equivalent liquid of the sediment and water mixture. According to IS; 6512-1984 recommendation, in case of the vertical calculation, $\gamma_{equivalent}$ is taken to be

1925 kgf/m^3 and for the horizontal load calculation $\gamma_{equivalent}$ liquid is taken 1360 kgf/m^3

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Secondary load:…

Hydro dynamic wave load (P_{hw}): The transient hydrodynamic force produced by wave action against the face of the dam (P_{wave}). It is considered only in exceptional cases and also having relatively of small magnitudes. An empirical relation for wave load may be made by adjusting the static reservoir level used in determines P_{hw}. The p_{wave} at the reservoir surface is provided by $P_{wave} = 2\times \gamma_w \times h_w^2$ KN

Where h_w is the significant wave height i.e. the mean height of the highest third of wave.

Wave height can be found in the function of the fetch distance and wind velocity. This could be calculated through the empirical relationship

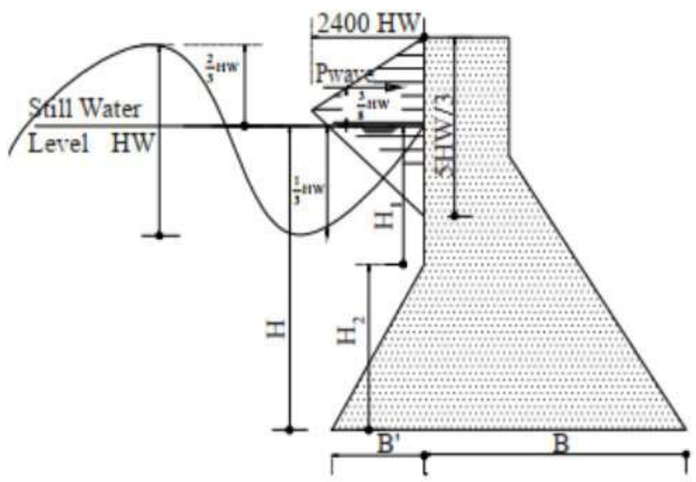

Fig 17.13 Wave Force on Typical Gravity Dam

For Fetch distance $\leq 32km.$

$h_w = 0.0322\sqrt{FV} + 0{,}763 - 0.271F^{1/4}$

For f $\geq 32Km$

$h_w = 0.0322\sqrt{FV}$

Where v = velocity of wind in km/hr

F= fetch distance in Km

The pressure diagram is shown in figure. The total wave P_w is given by the area of triangle or $P_w = 2400 x h_w x \frac{1}{2}\times\frac{5h_w}{3} = 2000\, h_w^2$ kg/m ($P_w = 2x\gamma_w x h_w^2$ KN/m) acting at $\frac{3h_w}{8}$ above the still water level assuming pressure distribution is triangle and height equal to $\frac{3h_w}{8}$.

17.6.2.3 Exceptional load

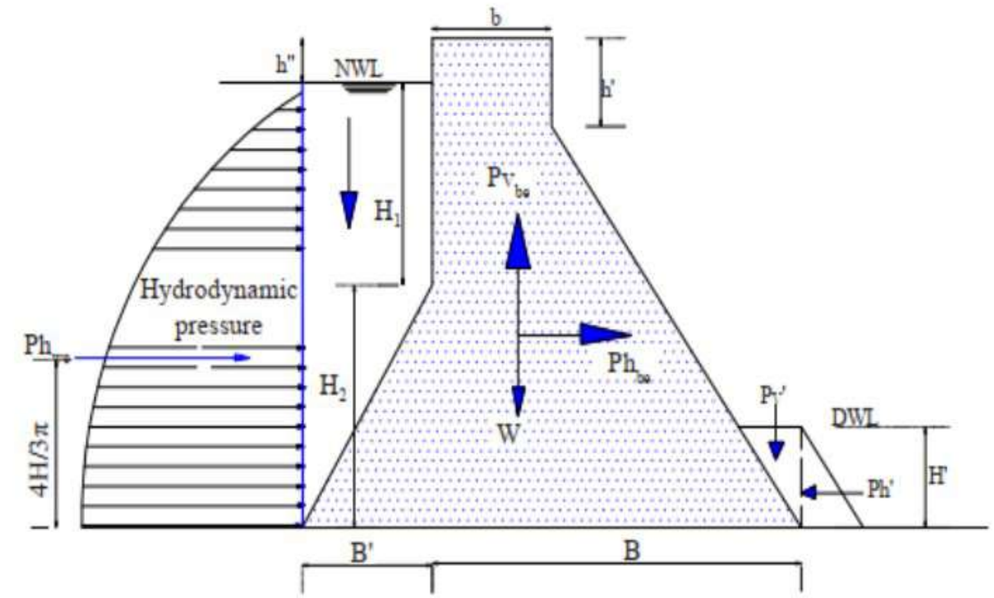

Fig 17.14 Earthquake Pressure Force

Earthquake pressure on the water body (Hydrodynamic pressure): Horizontal acceleration acting upward towards the reservoir causes a temporary increase in the water pressure. Due to hydrodynamic pressure, the dam and its foundation accelerate towards the reservoir and the water resists the movement owning to its inertia, and hence the water pressure increased. The additional water pressure exerted due to Earthquake is known as the hydrodynamic pressure.

a) **Von Karman's method:** According to Von Karman's suggestion, the hydrodynamic pressure variation is assumed to be parabolic and the total pressure force due to earthquake is given by $P_e = 0.555\alpha_h\gamma_w H^2$

Where, α_h=horizontal acceleration coefficient and H is the depth of water above the base. The point of application of these forces is at a height of $\frac{4H}{3\pi}$ above the base.

b) **Zanger's method:** In 1952 Zanger offered formulae to determine the hydrodynamic pressure exerted on vertical and inclined face by horizontal acceleration based on electrical analogy. The hydrodynamic pressure variation is supposed to be elliptical as well as parabolic.

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4.2.5 Modes of failure of gravity dams:

A gravity dam can fail due to the following reasons;

1) Overturning of the dam
2) Sliding of the dam
3) Crushing of the dam or foundation
4) Development of tension in the dam

1) Overturning of the dam

- Factor of safety with respect to overturning $(F.S.)_O$ can be defined as the ratio of the sum of all stabilizing (positive) moment (ΣM+ve) to the sum of all overturning moment (ΣM-ve) thus $(F.S.)_O = \frac{\sum M_{+ve}}{\sum M_{-ve}} = \frac{\sum M_R}{\sum M_O}$
- Value of $(F.S.)_O$ in excess of 1.25 is generally be regarded acceptable. But $(F.S.)_O \geq 1.5$ is desirable.
- Normally before overturning failure, the dam would fail due to other reasons such as tension and shear.
- If resultant is maintained within the body of the dam, there will be no overturning.
- The moment of all the forces are generally taken about the toe of the dam.

2) Sliding of the dam

- In this mode of failure, the dam fails in sliding.
- The dam as a whole slides over its foundation or one part of the dam may slide over the part of the dam itself, lying below it.
- Sliding stability is a function of loading pattern and of the resistance to translational displacement which can be mobilized on the plane.
- It is expressed in terms of a factor of safety of sliding or stability factor against sliding (Fs).
- Sliding factor of safety can be expressed in following **<u>two definitions:</u>**
- i) Sliding factor (F_{SS}) ii) Shear friction factor (F_{SF})

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2) Sliding of the dam….

i) Sliding factor (F_{SS}):

- It is expressed as a function of the resistance to simple sliding over the considered plane.
- It is assumed that resistance is purely frictional and no shear strength or cohesion can be mobilized.
- Sliding factor can then be defined as the ratio of the sum of all horizontal load components (ΣH) to the sum of all vertical loads (ΣV), on the horizontal plane.
- Sliding friction (sliding factor) = $\tan Ø = \frac{\Sigma H}{\Sigma V}$ and
- The factor of safety against sliding is defined as the ratio of actual coefficient of static friction (μ) on the horizontal joint to the sliding friction.
- Here it is assumed that concrete dam is just placed over the rock foundation i.e. no bondage between the concrete and rock foundation.
- Factor of safety against sliding (F.S.) = $\frac{\mu * \Sigma V}{\Sigma H}$
- The coefficient of friction (μ) varies from 0.65 to 0.75.
- The factor of safety against sliding should be greater than 1.

ii) Shear friction factor (F_{SF}):

- In actual, there is bondage between the concrete surface and rock foundation, thus the driving horizontal forces will be resisted by the shear stress of the material.

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2) Sliding of the dam….

ii) Shear friction factor (F_{SF}):

- Hence F_{SF} can be defined as the ratio of the total resistance including shear strength which can be mobilized on a plane to the total horizontal load.
- Shear friction factor $= \frac{\mu * \sum V + (B * q)}{\sum H}$

Where, $\sum V$= Total vertical forces equal to $\sum(W - U)$

μ = Coefficient of internal friction of the material.

q = Allowable shear stress of the concrete and rock material which varies from 140t/m2 for poor rocks to 500t/m2 for good rocks

$\sum H$ = Total horizontal forces

- From safety point of view, the value od shear friction factor (F_{SF}) should be lie between 4 and 5.

3) Crushing stability

- The sum of net active vertical forces must be balanced by the reaction from the mass of foundation materials.
- If the active vertical force on the dam is greater than the reaction beyond the foundation materials, there is a failure by compression called crushing failure.
- To check against crushing failure, the maximum compressive stress anywhere in the dam must not exceed the safe compressive stress (σ_C).
- The maximum compressive stress is given by the major principal stress. Maximum principal stress $\leq \sigma_C$.

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3) Crushing stability….

3.1 Middle third rule: Let take the section A-B on the dam body. Length of the dam along the transverse direction of the fig is taken to be unity. If R is the resultant force of all forces acting on the dam body above the section A-B. Let P & V are horizontal and vertical component of resultant forces.

- $\boldsymbol{\sigma_n}$ = normal stress along that surface = $\frac{total\ resultant\ force\ along\ that\ surface}{area\ of\ the\ section}$
- $\boldsymbol{\sigma_n} = \frac{V}{B*1} = \frac{V}{B}$
- The stress will act uniformly on the considered surface. Due to the eccentricity Of the resultant force, bending moment (M) will be develop at the considered Section;

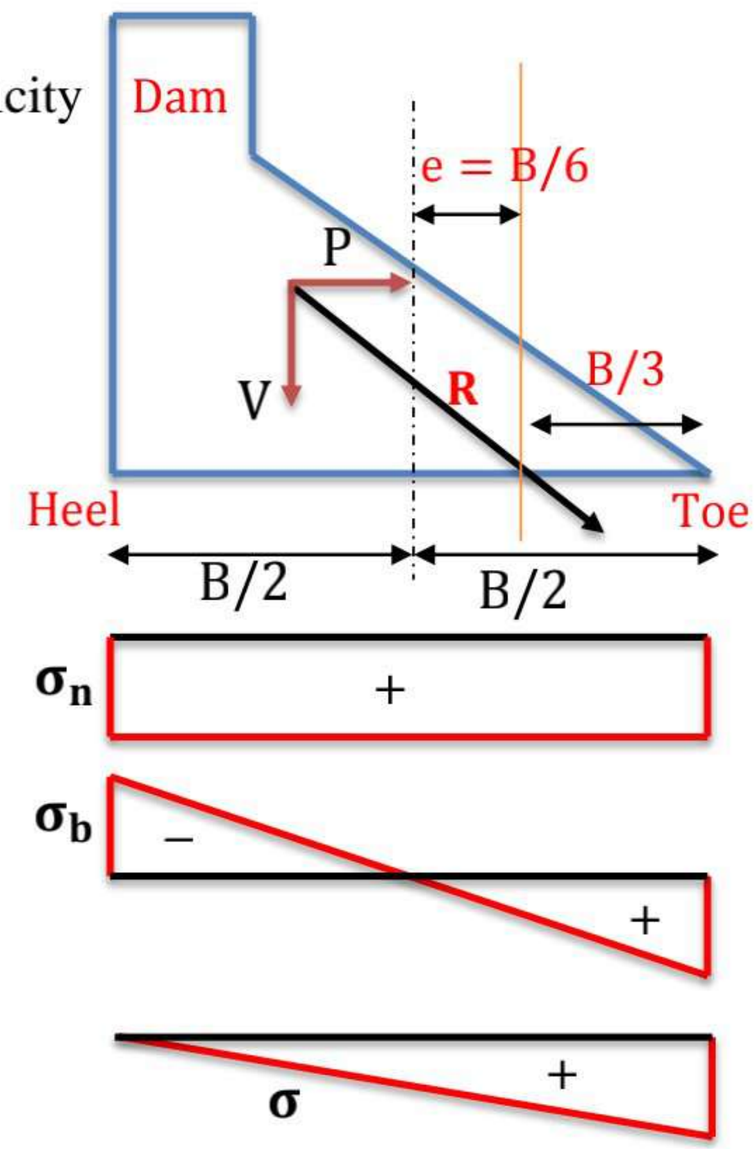
M = V*e

- Bending stress developed at any point 'y' along AB is , $\boldsymbol{\sigma_b} = \frac{M}{I} * y$

Here, I – moment of inertia of the section w. r. to axis passing through C.G. Of the figure and parallel to the 'y' direction (rectangle) $\frac{1*B^3}{12}$

- $\boldsymbol{\sigma_b} = \frac{V*e}{\frac{1*B^3}{12}} * y$
- The BM will be maximum at the points A and B, y = ±B/2
- $\boldsymbol{\sigma_b} = \frac{V*e}{\frac{1*B^3}{12}} * \left(\pm\frac{B}{2}\right) = \pm\frac{6V}{B^2} * e$
- Now, resultant Stress, $\boldsymbol{\sigma} = \boldsymbol{\sigma_n} \pm \boldsymbol{\sigma_b} = \frac{V}{B} \pm \frac{6V}{B^2} * e$

$= \frac{V}{B}\left(1 \pm \frac{6e}{B}\right)$



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3.1 Middle third rule:…

- Now, resultant Stress, $\boldsymbol{\sigma} = \frac{V}{B}\left(1 \pm \frac{6e}{B}\right)$
- The positive sign is assumed to calculate normal stress at the toe as the bending stress is compressive there but negative sign is assumed to calculate normal stress at the heel.
- Thus the normal stress at the toe $(\boldsymbol{\sigma_t}) = \frac{V}{B}\left(1 + \frac{6e}{B}\right)$
- The normal stress at the heel $(\boldsymbol{\sigma_h}) = \frac{V}{B}\left(1 - \frac{6e}{B}\right)$
- By superimposition of the normal and bending stress diagrams shown in figure. Following possibilities of stresses can occur are shown in figure .
- Case a) for $\boldsymbol{\sigma_n} > \boldsymbol{\sigma_b}$. The resultant normal stress due to both stresses would results compressive stress throughout the base of dam.
- Case b) For $\boldsymbol{\sigma_n} = \boldsymbol{\sigma_b}$. The resultant normal stress due to both stresses would results compressive stress at the toe and zero stress at the heel of dam
- Case c) For $\boldsymbol{\sigma_n} < \boldsymbol{\sigma_b}$The resultant normal stress due to both stresses would results compressive stress at the toe but tension occurs at the heel of dam.
- For no tension to occur in the dam, the normal stress should be always equal or greater than the bending stress ie $\boldsymbol{\sigma_n} \geq \boldsymbol{\sigma_b}$
- At tension stress is zero (limiting case) $\frac{V}{B}\left(1 \pm \frac{6e}{B}\right) = 0$, $e = \frac{B}{6}$. This rule is called middle third rule.
- It is evident that if $e > \frac{B}{6}$, the normal stress at the heel will be –ve or tension. No tension should be permitted anywhere in the dam under any circumstance for moderately high dams. The eccentricity should be less than B/3 for no tension to develop.

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Stability Analysis Methods:

There are the various methods for carrying out the stability analysis of gravity dams. These are gravity method, trial load twist method, slab analogy method, lattice analogy method, and experimental method. Out of which, the analytical techniques used in graphical method is used mostly.

Gravity method (Analytical method)

Stability analysis by this method is done as per following steps

1. Consider unit length of the dam. And (B is the base width of the dam.)

2. Find out the algebraic sum of all the vertical forces ($\sum V$), acting on the dam. The forces that come in this category are weight of the dam, weight of water acting on inclined faces of the dam, uplift pressure etc.

3. Determine the algebraic sum of all the horizontal forces ($\sum H$) acting on the dam.

4. Determine the overturning moment $(\sum M_O)$ and the resisting or stabilizing moment $(\sum M_r)$about the toe of the dam. Find out the algebraic sum of moments($\sum M$) as follows, $\sum \mathbf{M} = (\sum \mathbf{M_r}) - (\sum \mathbf{M_O})$

5. Determine the position of resultant of force (R), from the toe (X) as follows, $X = \frac{\sum M}{\sum V}$

6. Determine the eccentricity (e) of resultant (R) form the Centre of the base (b), $e = \frac{B}{2} - X$

7. Find out the normal stress at toe and heel, by the equation $(\boldsymbol{\sigma}) = \frac{\sum V}{B}\left(1 \pm \frac{6e}{B}\right)$

8. Determine the principle and shear stresses at toe and heel.

9. Find out factor of safety against overturning as follows, $(\mathbf{F.S.})_O = \frac{\sum \mathbf{M_R}}{\sum \mathbf{M_O}}$

10. Find out the factor of safety against sliding by following expression

i) Factor of safety against sliding $(F.S.) = \frac{\mu * \sum V}{\sum H}$

ii) Shear friction factor $= \frac{\mu * \sum V + (B * q)}{\sum H}$

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Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

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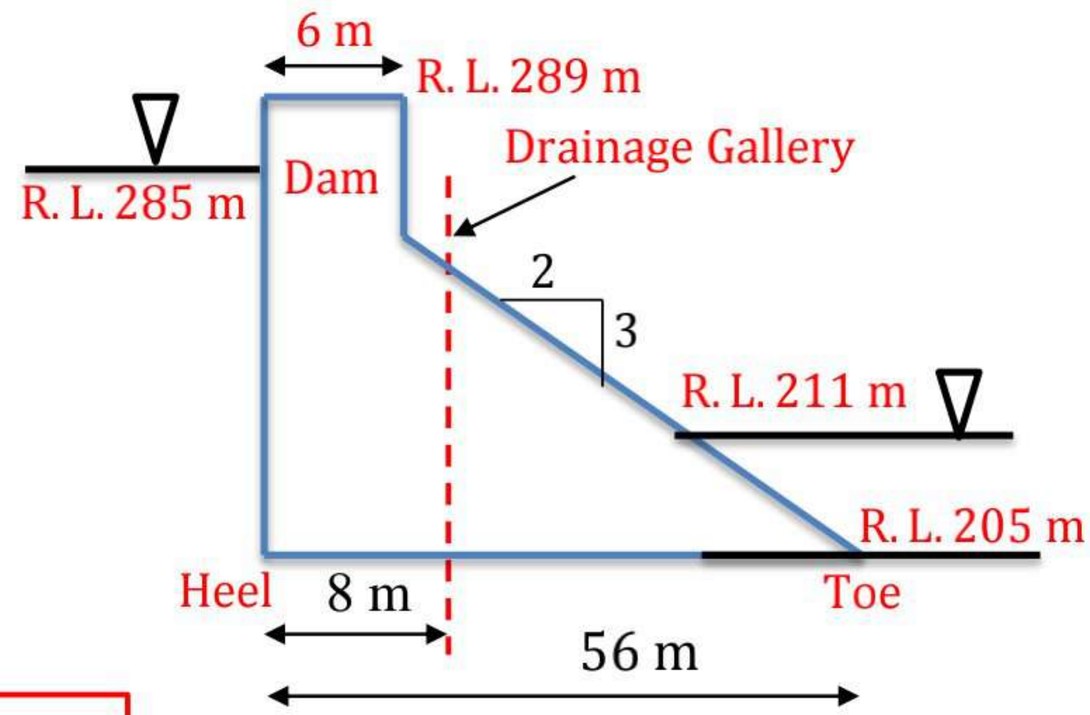
1. For a concrete gravity dam as shown in figure below, calculate the following;

(I) Maximum vertical stress at the heel and toe of the dam

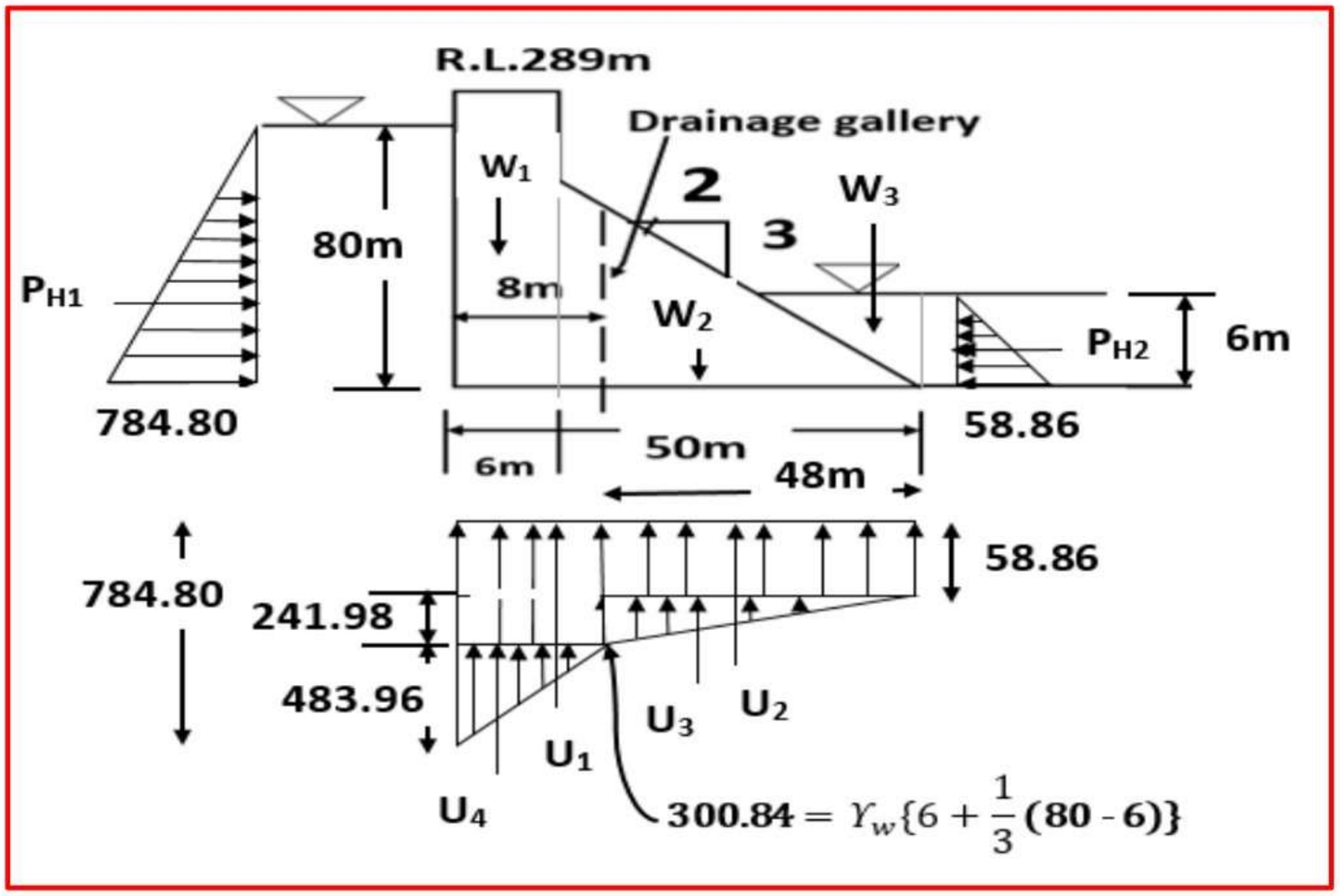
(ii) Major principal stress at the toe of the dam

(iii) Intensity of shear stress on a horizontal plane near to toe

(iv) Check the stability of the dam, Take $\mu = 0.75$, $\gamma_{Concrete} = 23.5KN/m3$, $\tan \emptyset = 0.75$, $\gamma_{Water} = 9.81KN/m3$, permissible compressive stress = 2500KN/m2.



Solution: Consider 1m length of dam



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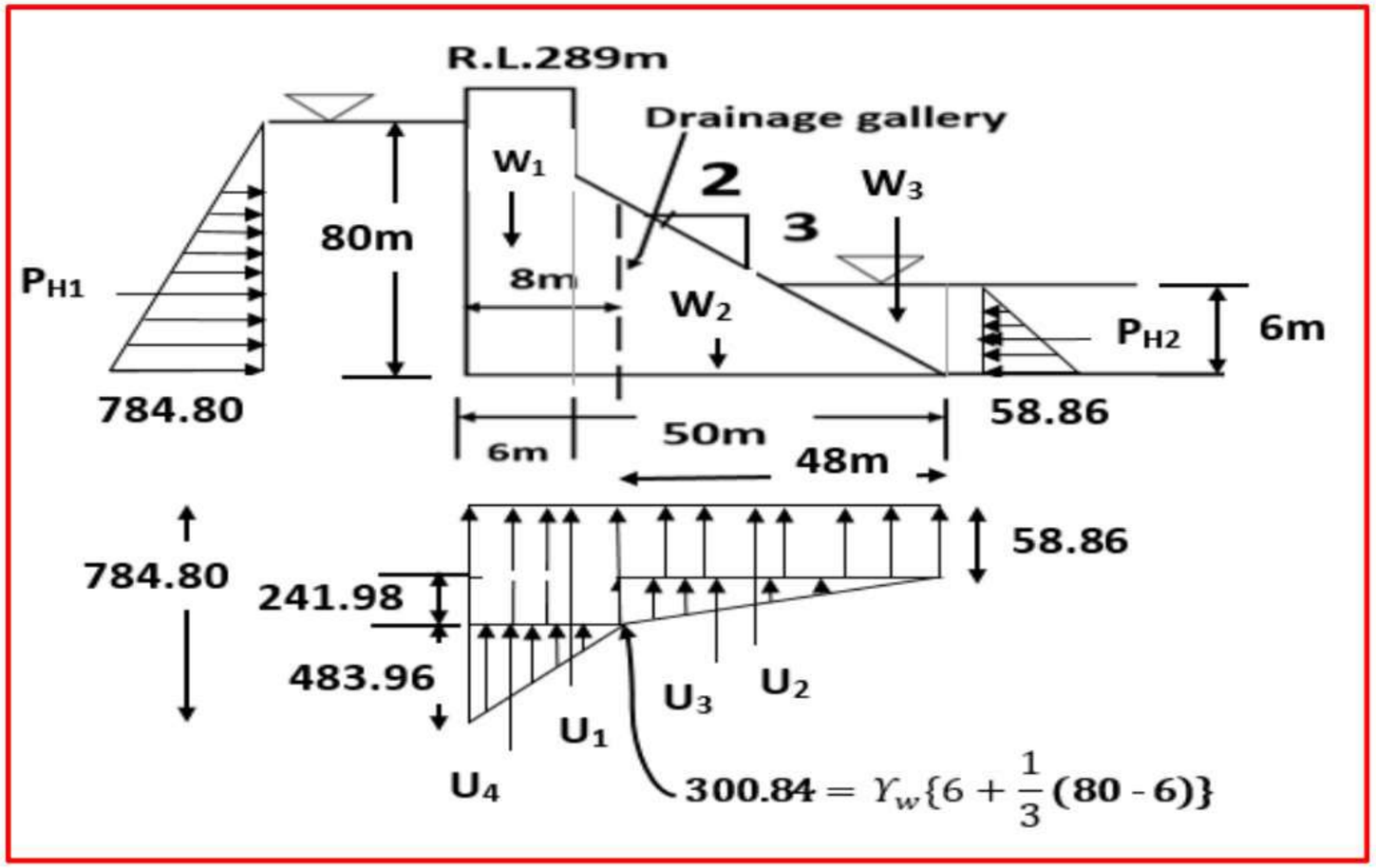
Calculation in Table.



Name of force	Designation	Magnitude of force↓(+) ↑(-), ← − → +	Lever arm(m)	Moment about toe (KN-m) Clockwise(-),Anticlockwise(+)
a) Vertical forces	W_1	(6x84x23.5)= 11844	53	+627732
1)Weight of dam	W_2	$(\frac{1}{2} * 75 * 50 * 23.5) = 44062.5$	33.33	+1468603.13
	W_3	$(\frac{1}{2} * 6 * 4 * 9.81) = 117.72$	1.33	+156.57
		$\sum V_1 = 56024.22$		

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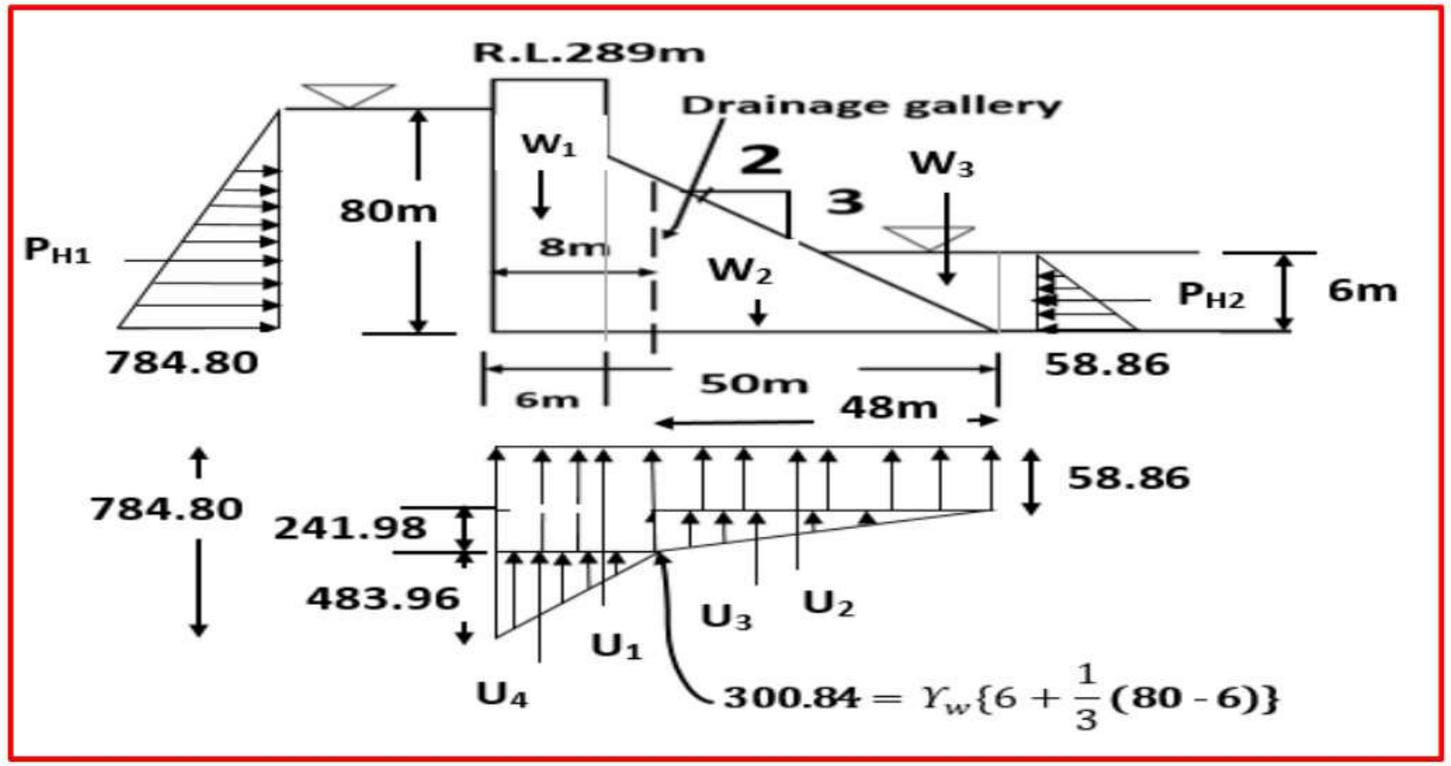
Calculation in Table...



Name of force	Designation	Magnitude of force↓(+) ↑(-), ← − → +	Lever arm(m)	Moment about toe (KN-m) Clockwise(-),Anticlockwise(+)
2)Uplift pressure	U_1	$(300.84 * 8) = -2406.72$	52	- 125149.44
	U_2	$(58.86 * 48) = -2825.28$	24	-67806.72
	U_3	$(0.5 * 48 * 241.98) = -5807.52$	32	-185840.64
	U_4	$(0.5 * 8 * 483.96) = -1935.84$	53.33	-103238.35
		$\sum \mathbf{U} = \mathbf{-12975.36}$		
b)Horizontal force	P_{H1}	$(0.5 * 784.8 * 80) = 31392$	26.67	- 837224.64
	P_{H2}	$(0.5 * 6 * 58.86) = -176.58$	2	+ 353.16
		$\sum \mathbf{H} = \mathbf{31215.42}$		

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Calculations

1) $\sum M_R$(anticlockwise) = 2096844.86 KN-m, $\sum M_O$(clockwise) = 1319259.79 KN-m

$\sum M = \sum M_R - \sum M_O$ = 777585.07 KN-m

2) Total vertical forces acting on the dam = $\sum V$

- $= \sum V1 - \sum U$ = 56024.22-12975.36
- = 43048.86 KN

3) The total horizontal forces acting on the dam

= $\sum H$ = 31215.42 KN

4) $X = \frac{\sum M}{\sum V} = \frac{777585.07 \text{ KN-m}}{43048.86 \text{ KN}}$ = **18.06 m**

5) Eccentricity (e) = $\frac{B}{2} - X = \frac{56}{2} - 18.06 = 9.94$m

6) From middle third rule, $\frac{B}{6} = \frac{56}{6} = 9.33$ m

7) $e > \frac{B}{6}$ i.e. ***Unsafe against tension***

8) Overturning factor (Fo) = $\frac{\sum M_R}{\sum M_O} = \frac{2096844.86}{1319259.79}$

=1.59 > 1.5 ***Hence stable***

9) Sliding factor = $\frac{\mu * \sum V}{\sum H} = \frac{(0.7*43048.86)}{31215.42}$

= 0.965 < 1 ***Hence Unsafe***

a) Normal stress (Vertical stresses) at heel and toe

$(\sigma) = \frac{\sum V}{B}\left(1 + \frac{6e}{B}\right) = \frac{43048.86}{56}\left(1 + \frac{6*9.94}{56}\right)$

σ = 768.729 (1 ± 1.065)

σ_{heel} = 768.729 (1 ± 1.065) = – 49.967 KN/m2

σ_{toe} = 768.729 (1 ± 1.065) = 1587.425 KN/m2

b) Major principal stresses at toe

Here, $\theta = \tan^{-1}\left(\frac{2}{3}\right) = 33.69$

$\sigma_{max,toe} = \sigma_{toe} * Sec^2\theta - \sigma' * \tan^2\theta$

$\sigma_{max,toe} = \sigma_{toe} * Sec^2\theta - \gamma_w * H_2 * \tan^2\theta$

$\sigma_{max,toe}$ = [1587.425*{1 + $\tan^2$(33.69)] − {9.81 ∗ 6*$\tan^2$33.69}

= 2266.78 KN/m2 < permissible stress, hence OK.

c) Intensity of shear stress on a horizontal plane near toe

$\sigma_{t,toe} = (\sigma_{toe} - \sigma')$ Tan θ

= {1587.425− (9.81 ∗ 6)}Tan 33.69

= 1019.1 KN/m2

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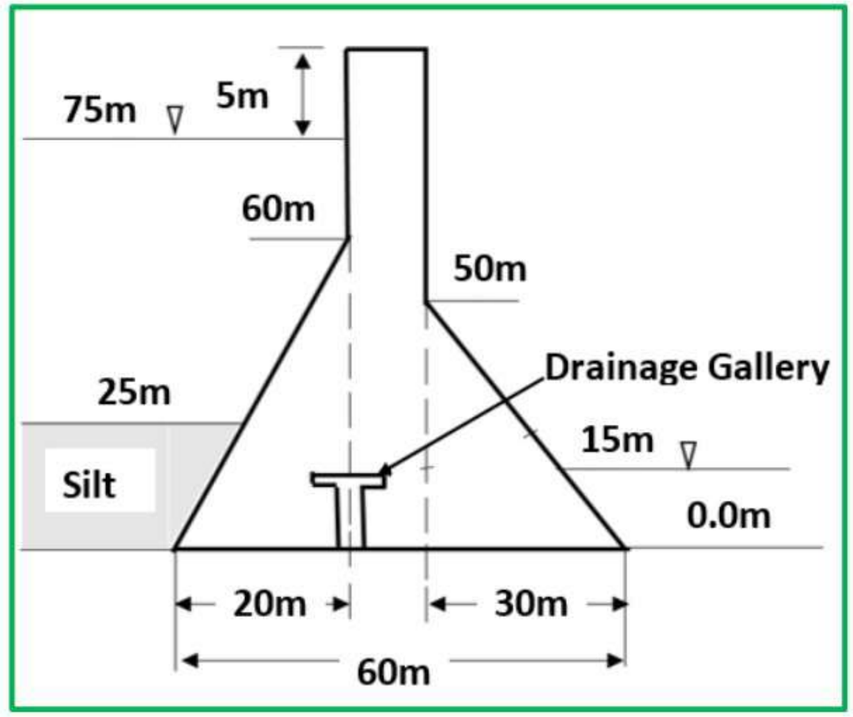
2. A concrete dam as shown in figure below. Check the stability against

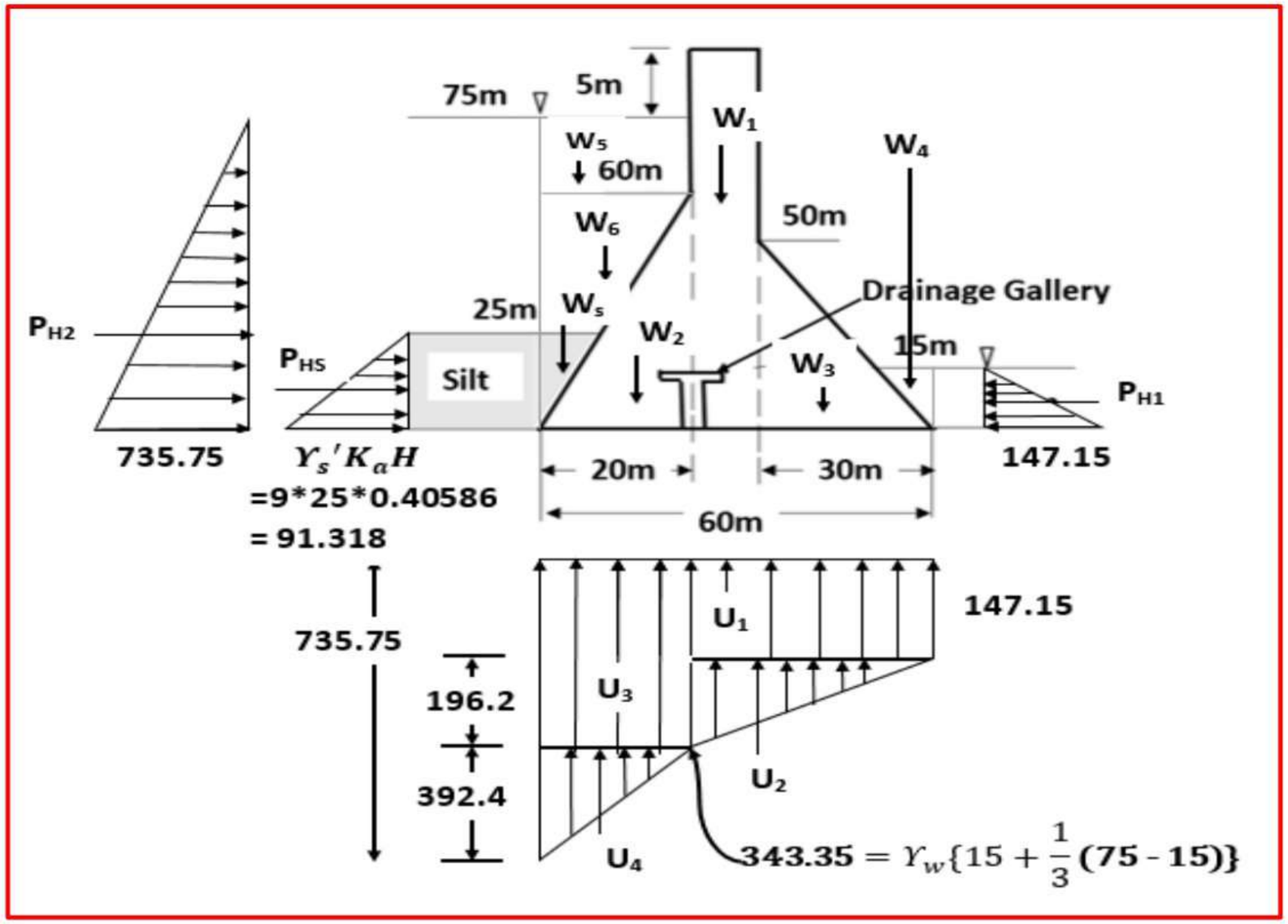
a) Overturning b) sliding c) bearing d) tension

Take γ_{Silt} (submerged) = 9 KN/m3, Friction coefficient = 0.62, Angle of friction of sediment particles = 25°, Specific Gravity of dam concrete (G) = 2.4, Allowable Compressive Stress in concrete = 4000KN/m2

Solution:

- $\Upsilon_c = 9.81 \times 2.4 = 23.522 KN/m^3$
- $K_a = \frac{1-\sin Ø}{1+\sin Ø} = \frac{1-\sin 25^0}{1+\sin 25^0} = \mathbf{0.40586}$





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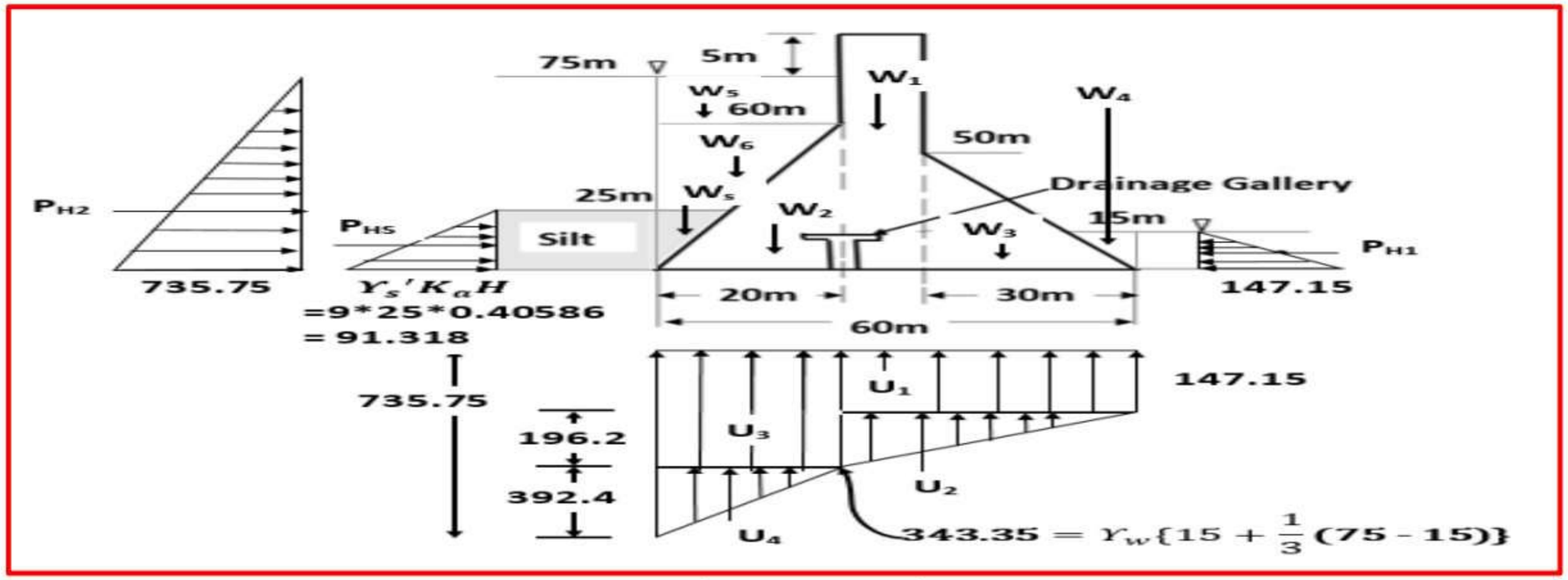
Calculation in Table:



Name of force	Designation	Magnitude of force↓(+) ↑(-), ← − → +	Lever arm(m)	Moment about toe (KN-m) Clockwise(-),Anticlockwise(+)
a) Vertical forces				
1)Weight of dam	W_1	(10x80x23.544)= 18835.2	35	+659232
	W_2	$(0.5 * 20 * 60 * 23.544) = 14126.4$	46.67	+659279.088
	W_3	$(0.5 * 30 * 50 * 23.544) = 17658$	20	+353160
2) Weight of water supported on U/S face	W_4	$(0.5 * 9 * 15 * 9.81) = 662.175$	3	+1986.525
	W_5	$(15 * 20 * 9.81) = 2943$	50	+147150
	W_6	$(0.5 * 20 * 60 * 9.81) = 5886$	53.33	+313900.38
3) Weight of silt supported on U/S face	W_{silt}	$(0.5 * 8.33 * 25 * 9) = 937.125$	57.223	+53625.104
		$\sum V_1 = 61047.9$		

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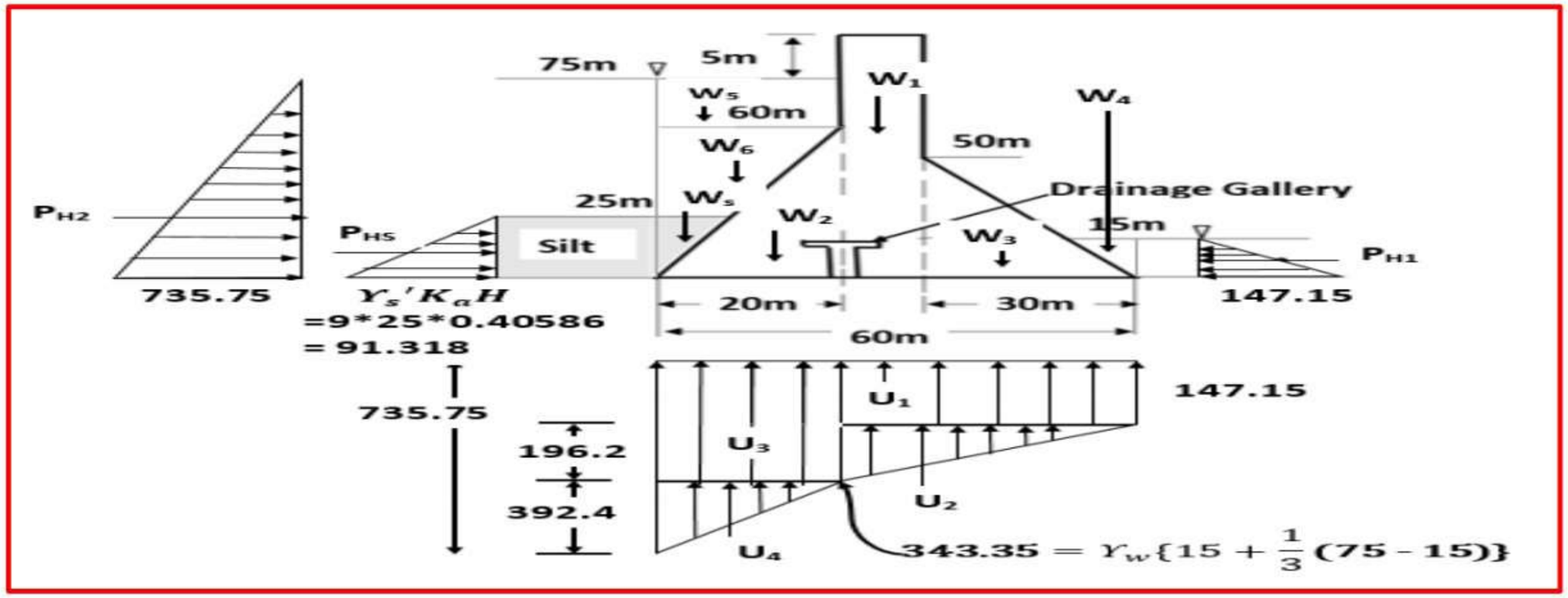
Calculation in Table:



Name of force	Designation	Magnitude of force↓(+) ↑(-), ← − → +	Lever arm(m)	Moment about toe (KN-m) Clockwise(-),Anticlockwise(+)
4)Uplift pressure	U_1	$(40 * 147.15) = -5886$	20	-117720
	U_2	$(0.5 * 40 * 196.2) = -3924$	26.67	-104653.08
	U_3	$(343.35 * 20) = -6867$	50	-343350
	U_4	$(0.5 * 392.4 * 20) = -3924$	53.33	-209266.92
		$\sum \mathbf{U} = \mathbf{-20601}$		
b)Horizontal force	P_{H1}	$(0.5 * 147.15 * 15) = -1103.625$	5	$+5518.125$
	P_{H2}	$(0.5 * 735.75 * 75) = 27590.625$	25	-689765.625
	P_{HS}	$(0.5 * 91.318 * 25) = 1141.475$	8.33	-9508.487
		$\sum \mathbf{H} = \mathbf{27628.475}$		

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Here, $\sum M_R$(anticlockwise) = 2193851.222 KN-m, $\sum M_O$(clockwise) = 1474544.112 KN-m

1) $\sum M = \sum M_R - \sum M_O$= 719307.11 KN-m

2) Total vertical forces acting on the dam = $\sum V$

- $= \sum V1 - \sum U$ = 61047.9-20601=40446.9KN

3) $X = \frac{\sum M}{\sum V} = \frac{719307.11}{40446.9}$ =17.78m

4) Eccentricity (e) = $\frac{B}{2} - X = \frac{60}{2} - 17.78$=12.22m

5) From middle third rule, $\frac{B}{6} = \frac{60}{6}$ =10m

6) $e > \frac{B}{6}$ i.e. Unsafe against tension

7) Overturning factor (Fo) = $\frac{\sum M_R}{\sum M_O} = \frac{2193851.222}{1474544.112}$

=1.49 ≅1.5 Hence Stable

8) Sliding factor = $\frac{\mu * \sum V}{\sum H} = \frac{(0.62*40446.9)}{27628.475}$

=0.907 < 1 Hence Unsafe

9) $(\sigma) = \frac{\sum V}{B}\left(1 \pm \frac{6e}{B}\right) = (\sigma) = \frac{40446.9}{60}\left(1 \pm \frac{6*12.22}{60}\right)$

σ = 474.12(1±1.222)

$\sigma_{U/S}$ = 674.12 (1-1.222) = – 149.65, Hence tensile stress occurs at Heel

$\sigma_{D/S}$ = 674.12 (1+1.222) = 1497.68 < 4000, Hence safe against compression or bearing

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Elementary profile of Gravity dam

- If a gravity dam is subjected to horizontal pressure due to water along with its own weight, the elementary profile of the dam would be a right angled triangle.
- The thickness of the dam at the level of water surface will be zero as there is no hydrostatic pressure to be resisted.
- The thickness at base would be maximum as there will be maximum hydrostatic pressure to be resisted by the dam.
- As the variation in hydrostatic pressure from water level to the bottom, is linear, variation in elementary profile from top to the dam to its bottom would also be linear.
- The triangular elementary profile also provides the maximum possible stabilizing force against overturning without causing tension in the base when reservoir is empty. This is because in this case weight of dam W acts as B/3 form the upstream vertical face.
- It is right angled triangle with its apex at the water surface.
- Elementary profile of the dam is also sometimes known as theoretical profile of the dam.
- Let, B = Base width of the dam
- H = Height of the elementary profile
- G = Specific gravity of the dam material
- γ = Unit weight or density of water
- K = Uplift pressure intensity coefficient (Values varies from 0 – 1)
- U = Uplift pressure
- P = Horizontal water pressure
- W = weight of the dam as a whole

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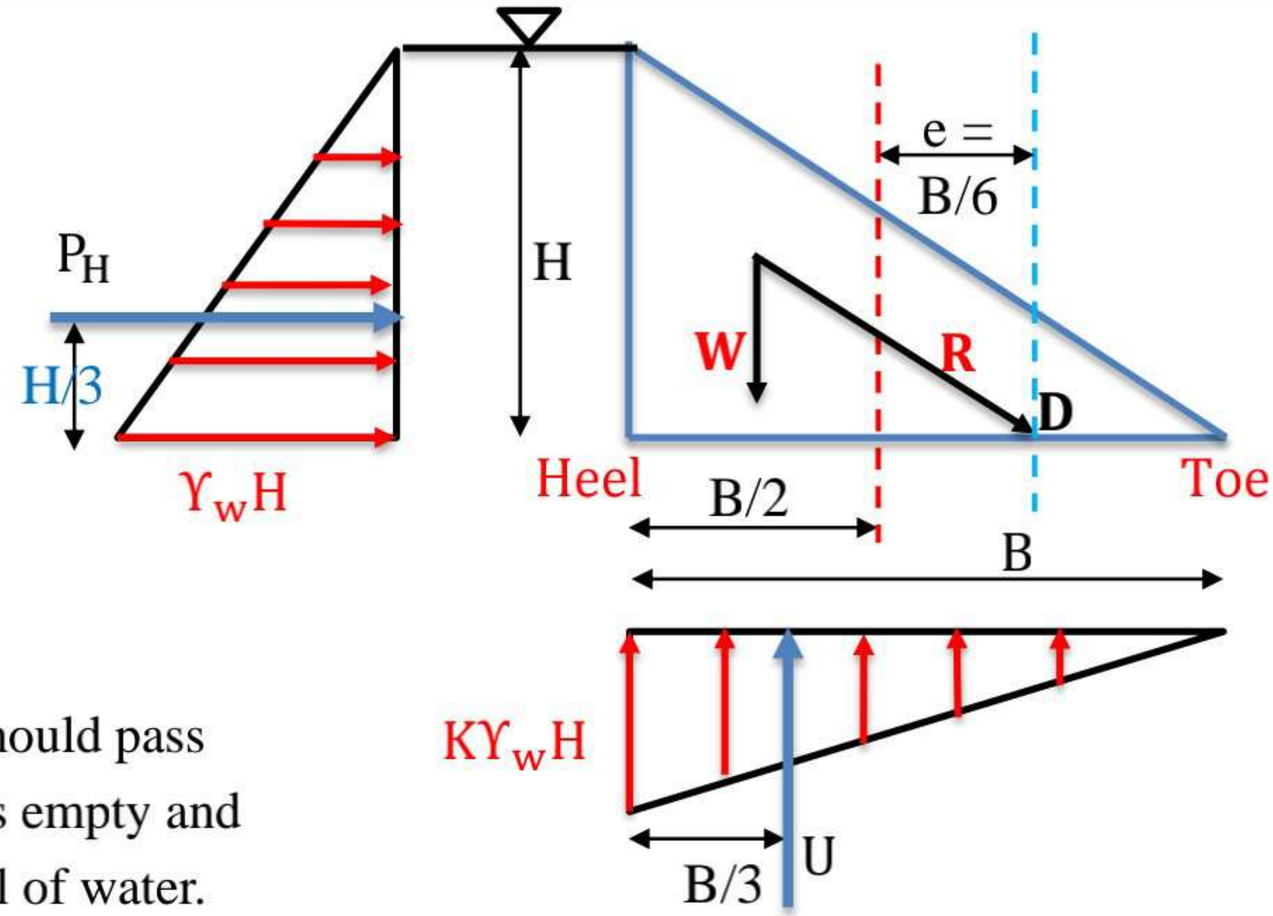
Elementary profile of Gravity dam

- Weight of dam (W) $= \frac{1}{2} * G * \Upsilon_w * H * B$
- Uplift Pressure(U) $= \frac{1}{2} K * \Upsilon_w * H * B$
- Hydrostatic Pressure (P_H) $= \frac{1}{2} * \Upsilon_w * H^2$



The base width of elementary profile is to be Determined tor following conditions

a) Stress basis b) Stability of sliding basis

a) Stress basis:

We know that for no tension to develop the resultant should pass Through the inner middle third point, when reservoir is empty and Through outer middle third point when reservoir in full of water.

Take moment of all forces acting on the profile about the middle third point and equate it to zero.

$(\circlearrowleft +) \sum M_D = 0$

$$- P_H * \frac{H}{3} - U * \frac{B}{3} + W * \frac{B}{3} = 0$$

$$-\left(\frac{1}{2} * \Upsilon_w * H^2 * \frac{H}{3}\right) - \left(\frac{1}{2} K * \Upsilon_w * H * B * \frac{B}{3}\right) + \left(\frac{1}{2} * G * \Upsilon_w * H * B * \frac{B}{3}\right) = 0$$

$$\left(\frac{1}{2} * \Upsilon_w * H^2 * \frac{H}{3}\right) = \left(\frac{1}{2} * G * \Upsilon_w * H * B * \frac{B}{3}\right) - \left(\frac{1}{2} K * \Upsilon_w * H * B * \frac{B}{3}\right)$$

$$H^2 = GB^2 - KB^2$$

$$B = \frac{H}{\sqrt{G - K}}$$

If uplift pressure is not considered, K = 0 and hence $B = \frac{H}{\sqrt{G}}$

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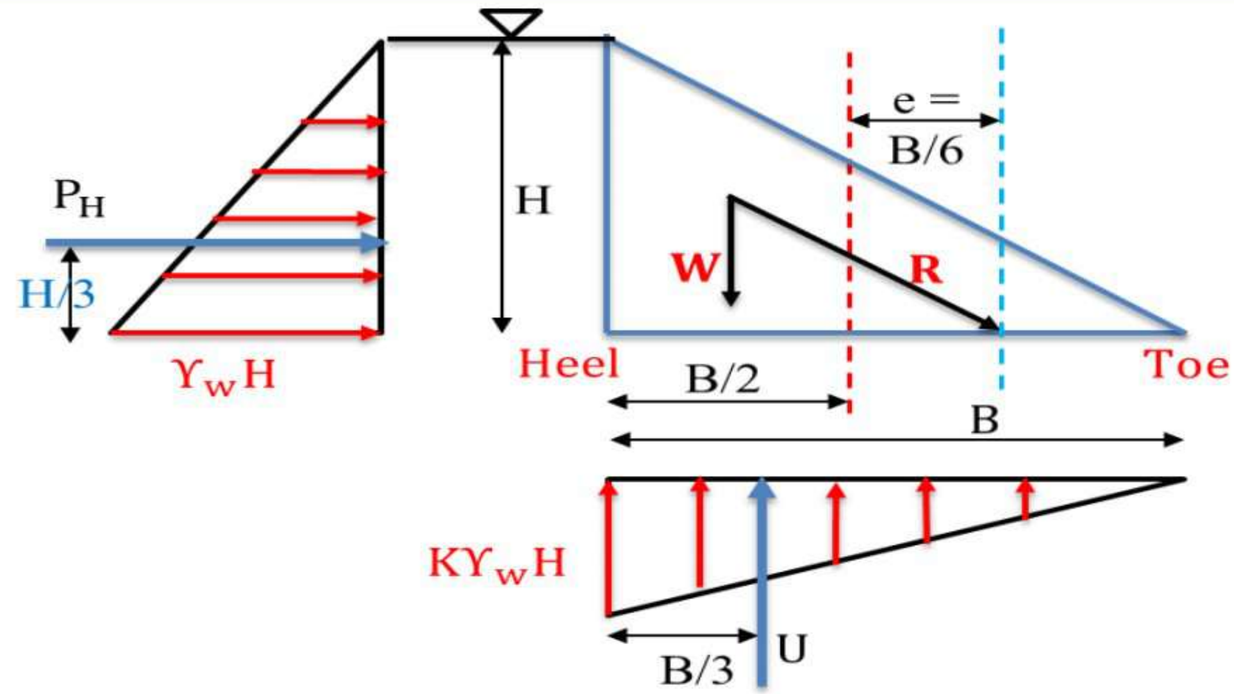
Elementary profile of Gravity dam



b) Stability of sliding basis

In order to prevent sliding of the dam, the horizontal force causing sliding should be either equal or less then the frictional resistance opposing the sliding of the dam. Critical condition occurs when horizontal force is equal to the frictional resistance.

i.e. $P_H = \mu(W - U)$

$$\frac{1}{2} * \Upsilon_w * H^2 = \mu\left(\frac{1}{2} * G * \Upsilon_w * H * B - \frac{1}{2} K * \Upsilon_w * H * B\right)$$

$$H = \mu(G * B - K * B)$$

$$\mathbf{B} = \frac{\mathbf{H}}{\boldsymbol{\mu}(\mathbf{G} - \boldsymbol{K})}$$

- If uplift pressure is neglected, $B = \frac{H}{\mu G}$
- The value of B should be adopted greater then the values obtained on stress basis and sliding basis.

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Q. N. Design the elementary profile of a gravity dam made of stone masonry given a following data. RL of base of dam = 193m, RL of HFL of dam = 223m, Specific gravity of masonry = 2.4, safe compressive stress in masonry is 1200KN/m2, tan Ø = 0.7 and seepage coefficient = 1.

Solution: From question given as following data

- RL of base of dam = 193m, RL of HFL of dam = 223m, Specific gravity of masonry (G) = 2.4, safe compressive stress in masonry (σ) = 1200KN/m^2, tan Ø = ($\boldsymbol{\mu}$) = 0.7 and seepage coefficient (K) = 1
- Height of water = 223m – 193m = 30m
- Let width of elementary triangular profile = B
- We know, $B = \frac{H}{\sqrt{G-K}} = \frac{30}{\sqrt{2.4-1}} = \mathbf{25.35}$m
- Considering friction coefficient, $(B) = \frac{H}{\mu(G-K)} = \frac{30}{0.7(2.4-1)} = \mathbf{30.61}$m
- I.e. Take B = 30.61m (Take maximum value of two cases)
- Limiting height $= \frac{\sigma}{\Upsilon_w(G-K+1)} = \frac{1200}{9.81(2.4-1+1)} = 50.97 >$ Height of dam
- **<u>Take(B) = 25.35m</u> (Stress basis)**
- $\propto = \tan^{-1}\left(\frac{25.35}{30}\right) = \mathbf{40.20}$
- Weight of dam$(W) = \frac{1}{2} * G * \Upsilon_w * H * B = \frac{1}{2} * 2.4 * 9.81 * 30 * 25.35 = \mathbf{8952.61KN}$
- Uplift Pressure$(U) = \frac{1}{2} K * \Upsilon_w * H * B = \frac{1}{2} * 1 * 9.81 * 30 * 25.35 = \mathbf{3730.25KN}$
- Net Vertical force$\sum V = 8952.61 - 3730.25 = \mathbf{5222.36KN}$
- Hydrostatic Pressure$(P_H) = \frac{1}{2} * \Upsilon_w * H * H = \frac{1}{2} * 9.81 * 30 * 30 = \mathbf{4414.5KN}$
- Net horizontal force$(\sum H) = \mathbf{4414.5KN}$

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- Net moment about toe$(\sum M) = \left(\sum V * \frac{2B}{3}\right) - \left(\sum H * \frac{H}{3}\right) = \left(5222.36 * \frac{2*25.35}{3}\right) - \left(4414.5 * \frac{30}{3}\right) =$ $\mathbf{44112.884KN - m}$
- Distance of the resultant from toe$(\overline{X}) = \frac{\sum M}{\sum V} = \frac{441112.884}{5222.36} = \mathbf{8.45m}$
- Eccentricity (e) $= \frac{B}{2} - \overline{X} = \frac{25.35}{2} - 8.45 = \mathbf{4.23}$m
- Normal stress at toe $= \frac{\sum V}{B}\left(1 + \frac{6e}{B}\right) = \frac{5222.36}{25.35}\left(1 + \frac{6*4.23}{25.35}\right) = \mathbf{412.26KN/m^2}$
- Shear stress at toe$(\tau) = (\sigma_{toe} * \tan\alpha) = (412.26 * \tan 40.20) = \mathbf{348.39\ KN/m^2}$
- Principal stress at toe$(\sigma_\tau) = \sigma_{toe}(1 + \tan^2\alpha) = 412.26\left((1 + (\tan 40.20)^2)\right) = 706.67N/mm^2$
- Factor of safety against sliding $= \frac{\mu\sum V}{\sum H} = \frac{0.7*5222.36}{4414.5} = \mathbf{0.828} < 1$ Hence unsafe
- **<u>Take(B) = 30.61m</u> Stability of sliding basis**
- Weight of dam(W) $= \frac{1}{2} * G * \Upsilon_w * H * B = \frac{1}{2} * 2.4 * 9.81 * 30 * 30.61 = \mathbf{10810.23KN}$
- Uplift Pressure(U) $= \frac{1}{2}K * \Upsilon_w * H * B = \frac{1}{2} * 1 * 9.81 * 30 * 30.61 = \mathbf{4504.26KN}$
- Net Vertical force$\sum V = 10810.23 - 4504.26 = \mathbf{6305.97KN}$
- Hydrostatic Pressure$(P_H) = \frac{1}{2} * \Upsilon_w * H * H = \frac{1}{2} * 9.81 * 30 * 30 = \mathbf{4414.5KN}$
- Net horizontal force$(\sum H) = \mathbf{4414.5KN}$
- Net moment about toe$(\sum M) = \left(\sum V * \frac{2B}{3}\right) - \left(\sum H * \frac{H}{3}\right) = \left(6305.97 * \frac{2*30.61}{3}\right) - \left(4414.5 * \frac{30}{3}\right) =$ $\mathbf{84538.83KN - m}$
- Distance of the resultant from toe $(\overline{X}) = \frac{\sum M}{\sum V} = \frac{84538.83}{6305.97} = \mathbf{13.41m}$

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- Eccentricity (e) $= \frac{B}{2} - \overline{X}$
- $= \frac{30.61}{2} - 13.41 = \mathbf{1.895m}$
- Normal stress at toe $= \frac{\sum V}{B}\left(1 + \frac{6e}{B}\right)$
- $= \frac{6305.97}{30.61}\left(1 + \frac{6*1.895}{30.61}\right)$
- $= \mathbf{282.53KN/m^2}$
- Shear stress at toe$(\tau) = (\sigma_{toe} * \tan\alpha)$
- $= (282.53 * \frac{30.61}{30}) = \mathbf{288.27N/mm^2}$
- Principal stress at toe$(\sigma_\tau) = \sigma_{toe}(1 + \tan^2\alpha)$
- $= 282.53\left(1 + \left(\frac{30.61}{30}\right)^2\right) = 576.67\mathrm{N/mm^2}$
- Factor of safety against sliding $= \frac{\mu\sum V}{\sum H}$
- $= \frac{0.7*6305.97}{4414.5} = 1 \cong 1$ Hence safe

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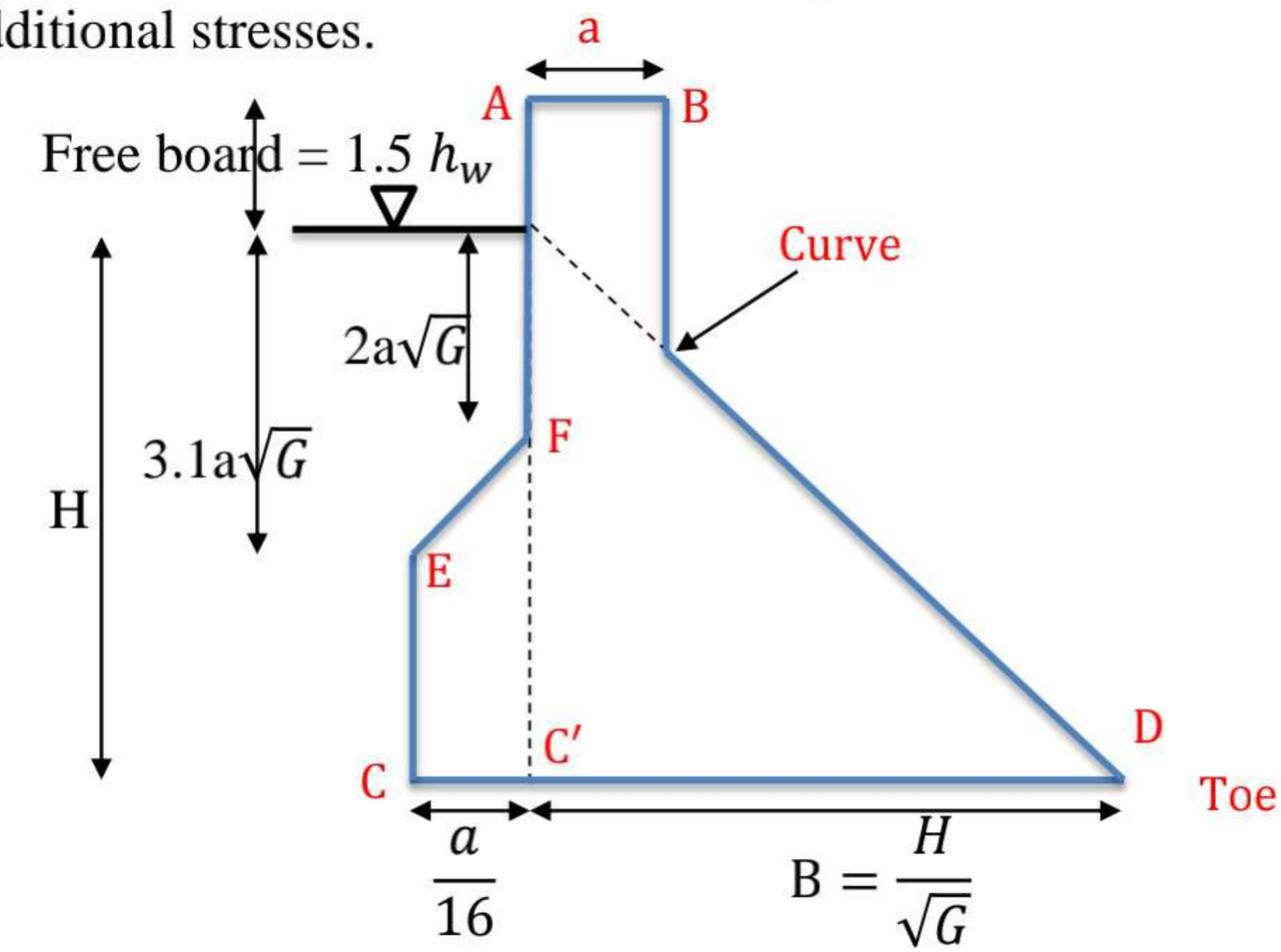
Practical profile of Gravity dam

The elementary profile cannot be adopted as such. Some modification have to be incorporated, to make it adaptable in practice. Modification in elementary profile are necessitated due to following reasons.

1. Some free board is essential whereas elementary profile does not provide nay free board. The amount of free board usually provides is 1.5 h_w where h_w is the height of waves in meters between through and crest.

2. Road way is generally provided at the top of the dam. This necessities quite thick top of the dam, whereas elementary profile does not provide any thickness at the top. Minimum top width of the dam should be one – seventh the height of the dam. (a > H/7)

3. Additional load due to extra height provided for free board and also due to the thick top of the dam, induces some additional stress in the dam section. Some extra dam section has to be provided at the base of the dam along the U/S face to counteract such additional stresses.



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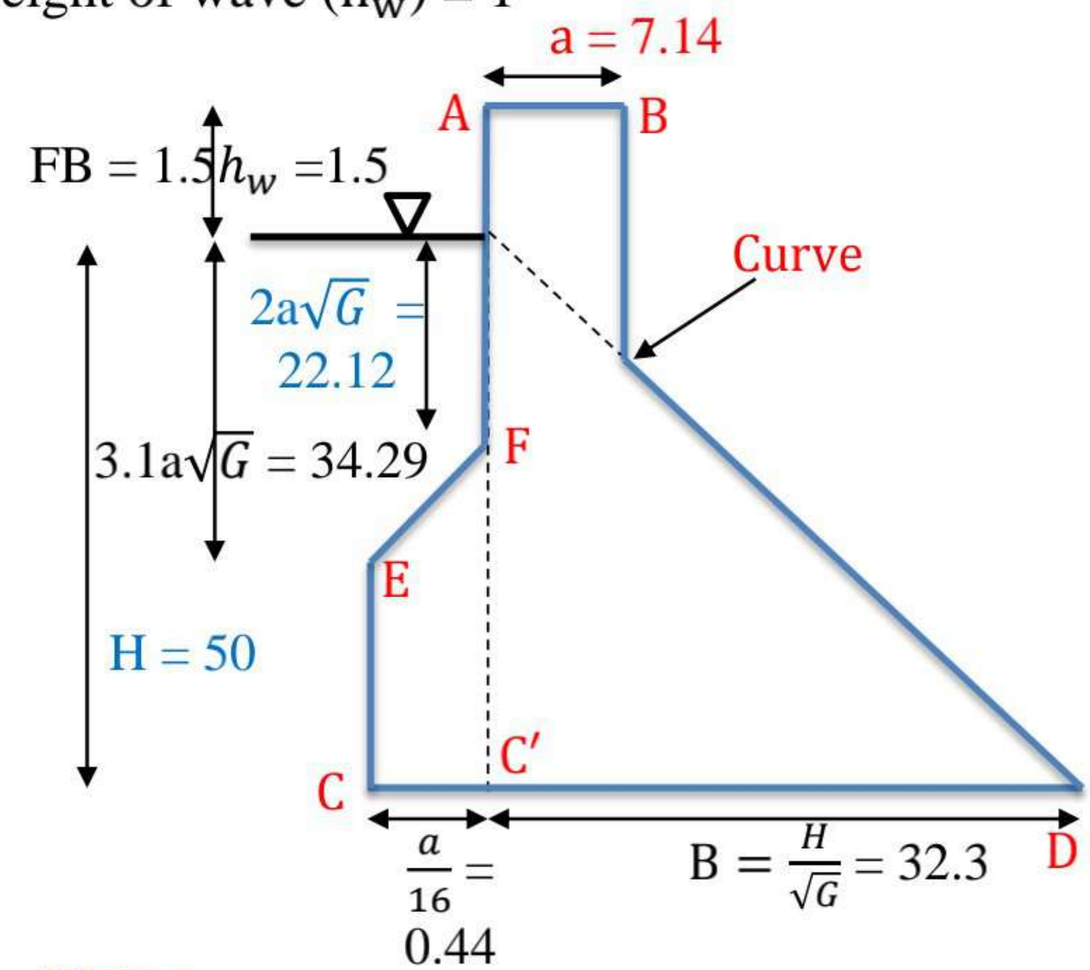
Q.N. Design a practical profile of a gravity dam of concrete from the following data

RL of the base of the dam = 1350m, RL of HFL = 1400m, Specific gravity of concrete = 2.4

Safe compressive stress of concrete = 3000 KN/m2 and Height of wave (h_w) = 1

Solution:



- Free Board (FB) = 1.5*h_w = 1.5 m

∴ RL of the top = RL of HFL + FB = 1400 + 1.5 = 1401.5 m

∴ Height of the dam = 1401.5 – 1350 = 51.5 m

- Limiting height = $\frac{f}{\gamma(G-K+1)} = \frac{f}{\gamma(G+1)}$

$= \frac{3000}{9.81\,(2.4+1)} = 89.94 > 51.5$ m

Hench, it is low gravity dam.

- Depth of water (H) = 1400 – 1350 = 50m
- Top width (a) = H/7 = 50/7 = 7.14 m
- Base width of the elementary profile (B) = $\frac{H}{\sqrt{G}} = \frac{50}{\sqrt{2.4}}$ = 32.3 m
- Upstream offset = $\frac{a}{16} = \frac{7.14}{16}$ = 0.44 m
- Total base width = 0.44 + 32.3 = 32.74 m
- Distance from water surface up to which u/s slope is vertical = $2a\sqrt{G} = 2*7.14\sqrt{2.4}$ = 22.12 m
- Distance from water surface up to which slope is given upstream = $3.2a\sqrt{G} = 3.1*7.14\sqrt{2.4}$ = 34.29 m

The design profile is shown in figure.

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Foundation treatment of Gravity dam

- The foundation of the gravity dam should be hard, strong, durable and impervious.
- Imperviousness of foundation is very important as uplift pressure depends greatly upon the seepage.
- Uplift pressure is increased when seepage is more.
- Hence to render the foundation waterproof or impervious, it has to be suitably treated.
- All the loose overlying soil from the site is removed and solid rocky foundation is reached.
- The rocky foundation should also be excavated for some depth so that the proposed dam fits in the rock.
- This aspect will prevent the sliding of the dam over its foundation.
- A 3 cm thick layer of rich cement mortar should be laid on the excavated rocky foundation before concreting is done over it.
- All the faults, seams, cavernous rocks, crushed zone etc., should be either made good or removed from the foundation site.
- In order to prevent seepage through foundation, trenches may be excavated near heel of the dam and filled with cement concrete.
- Holes are drilled covering whole of the foundation and filled with cement grout.
- If foundation is to be treated for target depths, the holes are also drilled for large depth.
- The holes are filled with cement grout in stages.
- Any other fault noticed in the foundation is rectified.

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Advantages / disadvantages of Concrete Dam:

Advantages:

- Maintenance cost of gravity dams is very small.
- Spillways can be installed in the dam itself and no separate site for them is required.
- They can be constructed for very large heights.
- Ice and other outer effects do not affect the stability of the dam.
- Water is not lost by seepage.
- Outlets sluices may be installed in the dam.
- At valleys where side slopes are very steep, only this dam is found as the most suitable choice.
- This dam gives pre – warning before failure. If timely measures are taken, the dam may even be made safe.

Disadvantages:

- They are very costly.
- They require very skilled labour for construction.
- They have to be continuously cured during construction.
- Large calculation works involved in their design.
- They required solid hard foundation.
- The height of the dam cannot be increased later.

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Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

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4.2.8 Embankment Dam Analysis-phreatic line and seepage analysis.

- The embankment dam is the dam constructed using the natural materials either excavated or acquired nearby the site.
- Without any binding agent, the natural fill materials are dumped and compacted by using high capacity mechanical plant.
- Embankment dam construction now a day being an almost continuous and highly mechanized process is thus, plant intensive instead of labor intensive.
- Embankment dam can be classified broadly as being earthen or rock fill dam.
- The classification of embankment variants is not absolute because many dams are utilizing fill materials of both types within approximately designed internal zones.
- The following are the main reasons due to which these dams have been and still continue to be the most commonly used types

i) The basic material used in the construction, is the local soil. This aspect makes these dams economical.

ii) The science of soil mechanics has recently developed to such a stage that dams constructed from earth by adopting the modern theories of soil mechanics, can be as equally relied upon those of gravity dams.

iii) No specific type of foundation is required. They can be constructed on any type of foundation by making little bit changes for different foundation conditions.

iv) No skill of very high standard is required.

v) No material have to be imported form outside as locally available soil is the main material to their construction.

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Classification of Earthen dams:

Earthen dams may be classified in two ways

1. Classification based upon the method of construction
2. Classification based upon the section of the dam

1. Classification based upon the method of construction

Under this classification earthen dams can be

i) Rolled fill earth dams

ii) Hydraulic fill earth dams

i) Rolled fill earth dams

- In this technique, compaction of the filled materials such as soil, gravel, boulder etc., is done by the rolling equipment (sheep foot pneumatic plain roller and vibrator) with the minimum thickness of the layer 15 cm to 45 cm.
- Compaction is done at the optimum moisture content of the given soil established through the laboratory test and filed control.
- Proper slope of the dam is maintained during construction.
- Construction materials are transported through the dumper or conveyer and spreading is done through bulldozer or the scraper for the required thickness of the layer.

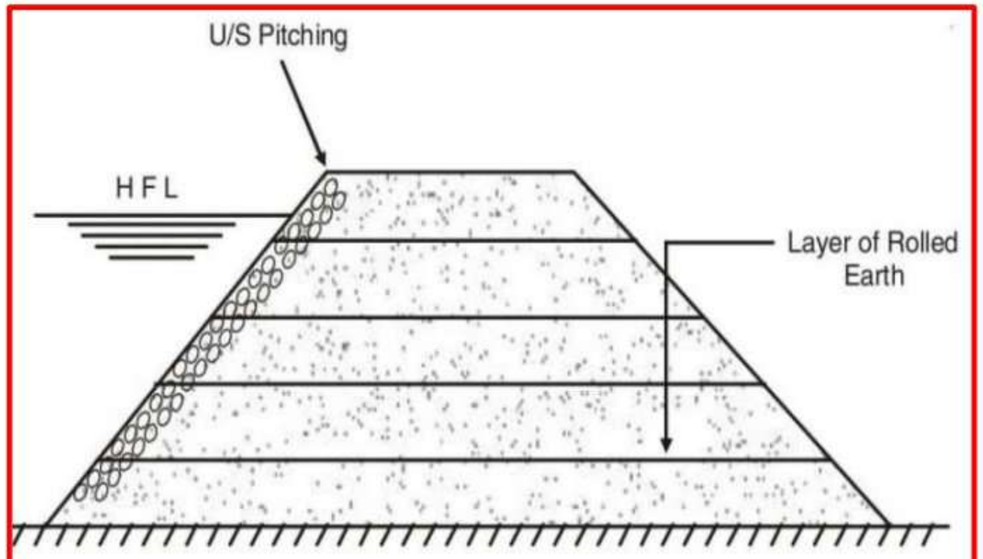

Fig. . *Rolled fill earth dam.*

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Classification of Earthen dams:…

ii) Hydraulic fill earth dams

- In this method construction, excavation, transportation and placing of the earth is done by hydraulic methods.
- Outer edges or the embankment are maintained slightly higher than the middle part of the dam.
- Water is used during the laying of the earthen dam construction.
- Slurry or mud of the embankment materials is prepared at the borrow pit (with 1:7 Soil: water) to 1:10 and they are transported to the site through the pipes or flumes.
- They are spread in the layers through the perforated pipes located at the outside edge of dam.
- The coarse material is deposited near the pipe exit and fine materials (impervious) are carried to the Centre of the dam body.
- This will cause the zoned section of the dam with impervious core at the Centre.
- No rolling and compaction is needed in this technique.
- The main disadvantage of this method is that there is excessive pore water pressure inside the dam which decreases shear strength of the soil.
- This method is now almost obsolete.
- In Semi hydraulic filled earthen dam, outside core is constructed with the dry method and inside core is advanced through the hydraulic fill technique.

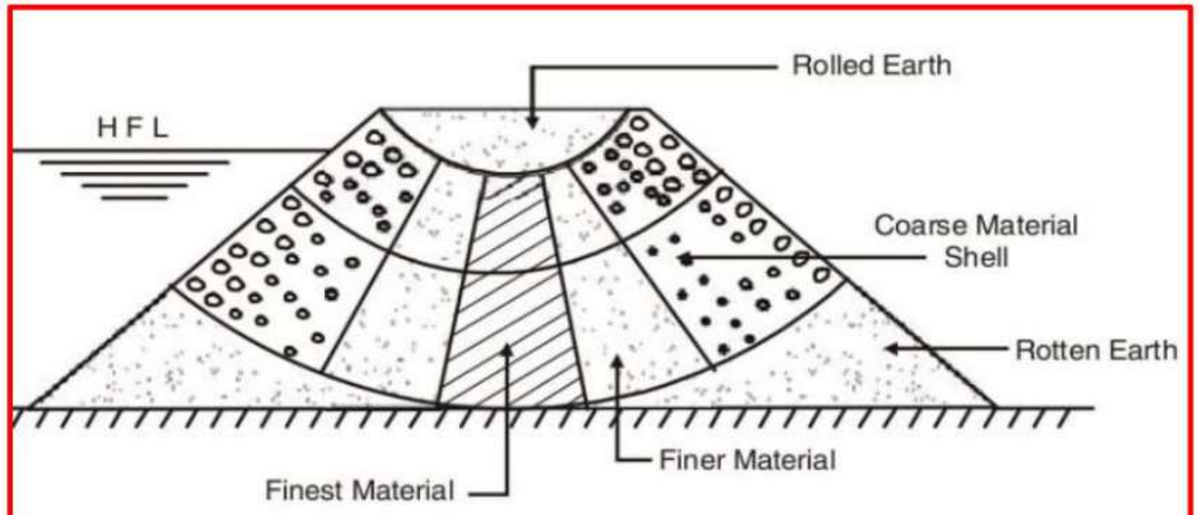

Fig. *Hydraulic fill dam.*

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Classification of Earthen dams:

2. Classification based upon the section of the dam

According to this classification the dams can be of the following three types

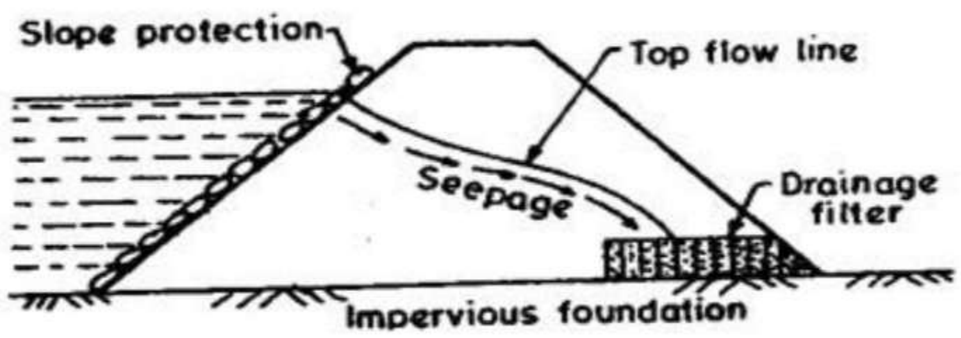
i) Homogeneous earth dams

ii) Zoned earth dams

iii) Diaphragm type earth dams

i) Homogeneous earth dams

- This dam is made form a single material.
- It is suitable for low height only.
- Purely homogeneous section can be modified a little by constructing rock toe at the down stream lower end of the dam and also by putting horizontal filter drains.
- Both these measures control the seepage and thus enable to construct much steeper slopes of the dam.
- These measures also keep the phreatic line of seeping water, well within the body of the dam.
- The homogeneous dams are made from impervious or semi-impervious soils.
- This is essential to put a barrier against the flow of seeping water.
- Upstream slope of the dam is generally kept flat so as to reduce the path of seeping water and to counter act the effect of sudden draw downs.



Homogeneous embankment provided with a drainage filter.

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2. Classification based upon the section of the dam…

ii) Zoned earth dams

- This dam is made by using more than one material. In this case, the central part of the dam, which is *known as core* is made from impervious material.
- Considerably more pervious material is used on both the sides of core.
- The dam also consists of a rock toe, a system of horizontal drains, and sometimes even system of inclined filters to carry out proper drainage of seeping water from the dam.
- If, at certain place, a variety of soils is available the dam should always be of zoned type. Impervious material should be used for core, while pervious soils at flanks of core.
- U/S pervious soil provides stability against rapid drawdown, while that on D/S side acts as a drain to control the seepage line.
- Central core checks the seepage.

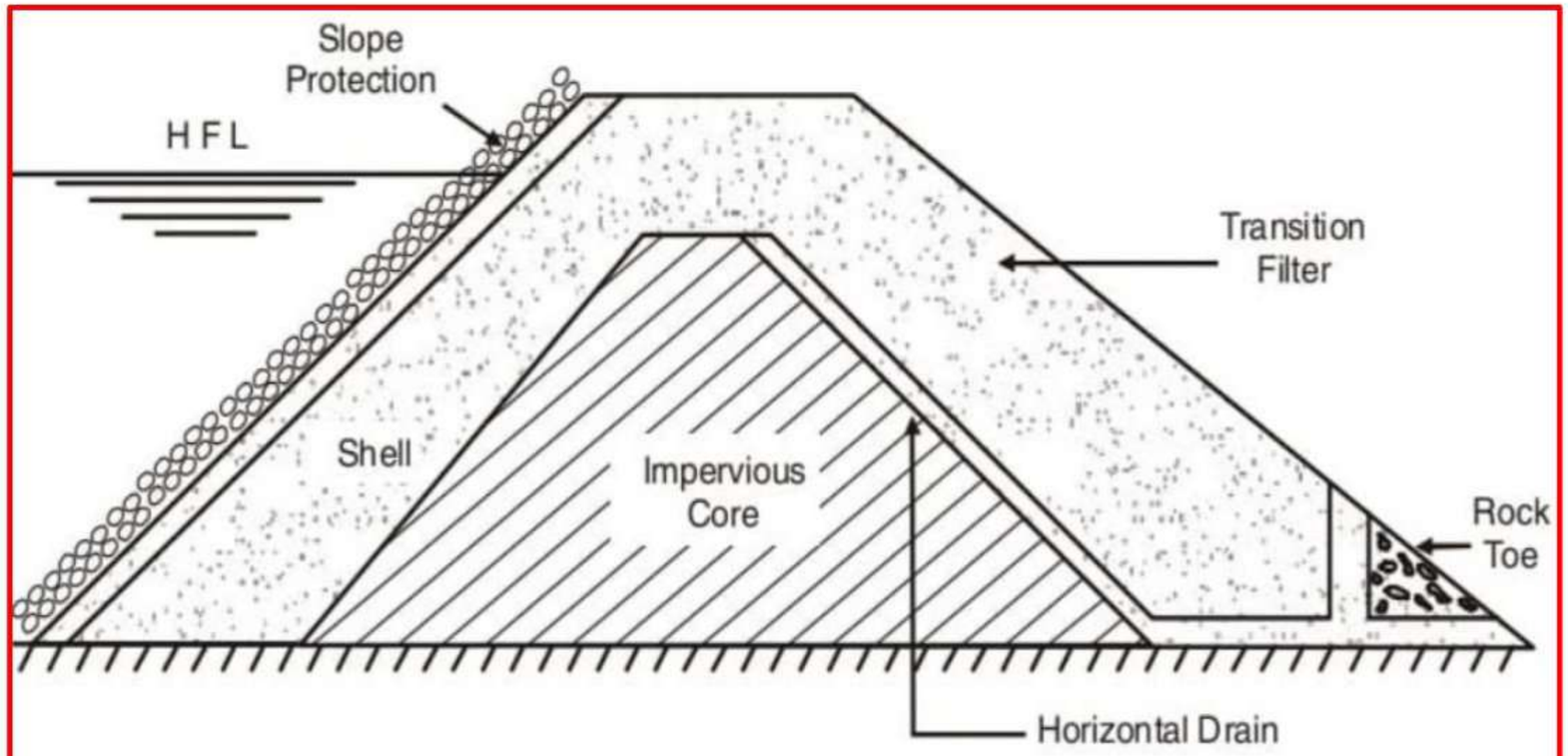

Fig. *Zoned earth dam.*

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2. Classification based upon the section of the dam….

iii) Diaphragm type earth dams

- This is a sort of combination of both the previously, mentioned earth dams.
- The bulk of the embankment is made from pervious soil, but a thin diaphragm of impermeable material is provided at the central part of the dam to check the seepage.
- The impervious diaphragm may be made of impervious clayey soil, cement concrete, masonry or of any other impervious material.
- It can be located somewhere in the central part of the dam or it may be put on the upstream face of the dam also.
- The difference in zoned and diaphragm type earth dams is due to thickness of impervious core or diaphragm only.
- If thickness of impervious core is **less than 10 m** or less than the height of the dam above that level the dam is known as **diaphragm type**, otherwise it would be zoned earth dam.

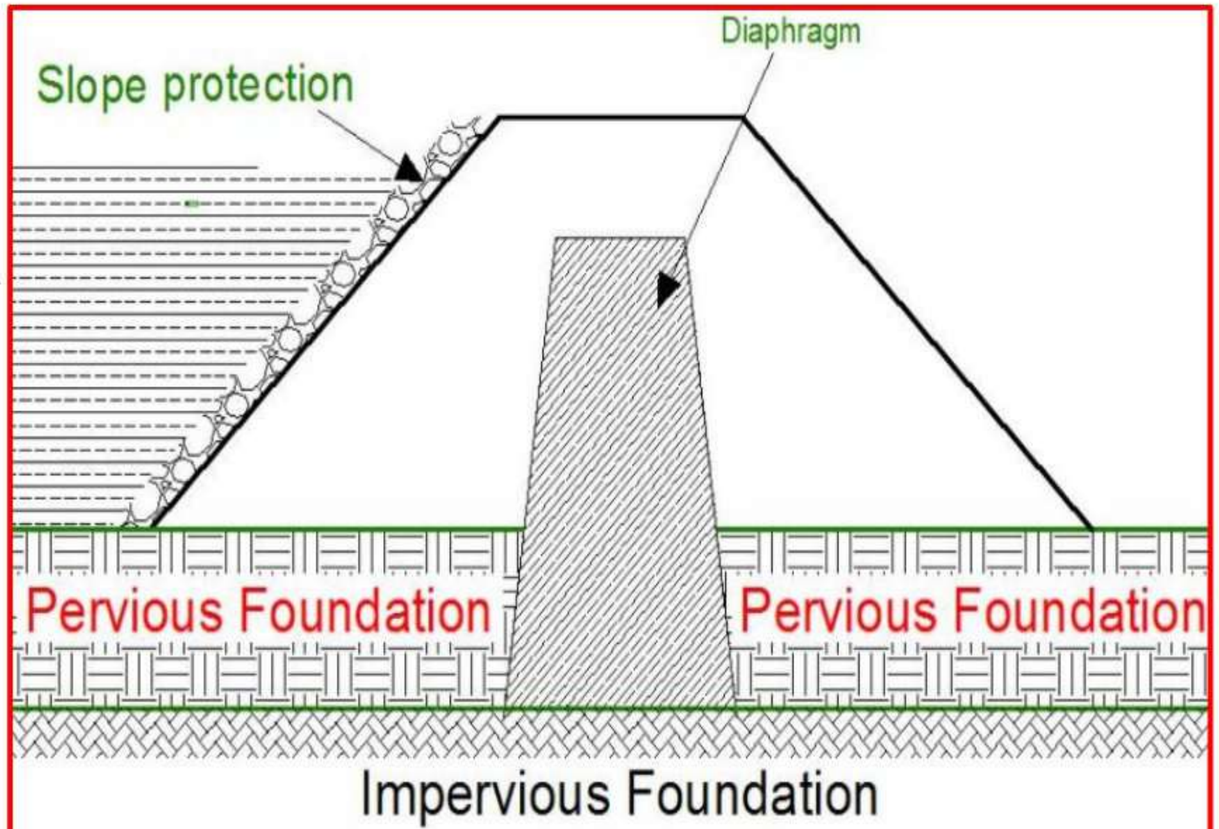

Fig .: Diaphragm type embankment

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 # Causes of failure of earth dams: Different causes of failure of earthen dams are as follows:

1. Hydraulic Failures 2. Seepage Failures 3. Structural Failures

1. Hydraulic Failures

About 40% of failures occur in embankment dam due to the hydraulic failure. Reasons for the failure are

a) Overtopping failure:

- Overtopping failure occurs due to the flow taking place above the crest level due to the various causes i.e. underestimation of flood discharge, inadequate spillway design, insufficient free board, settlement of embankments etc.
- Overtopping leading to washout; less cohesive silts, sands etc., at greatest short-term risk.
- Almost 30-35% of serious incidents and failures has occurred due to the internal erosion and overtopping mechanisms.

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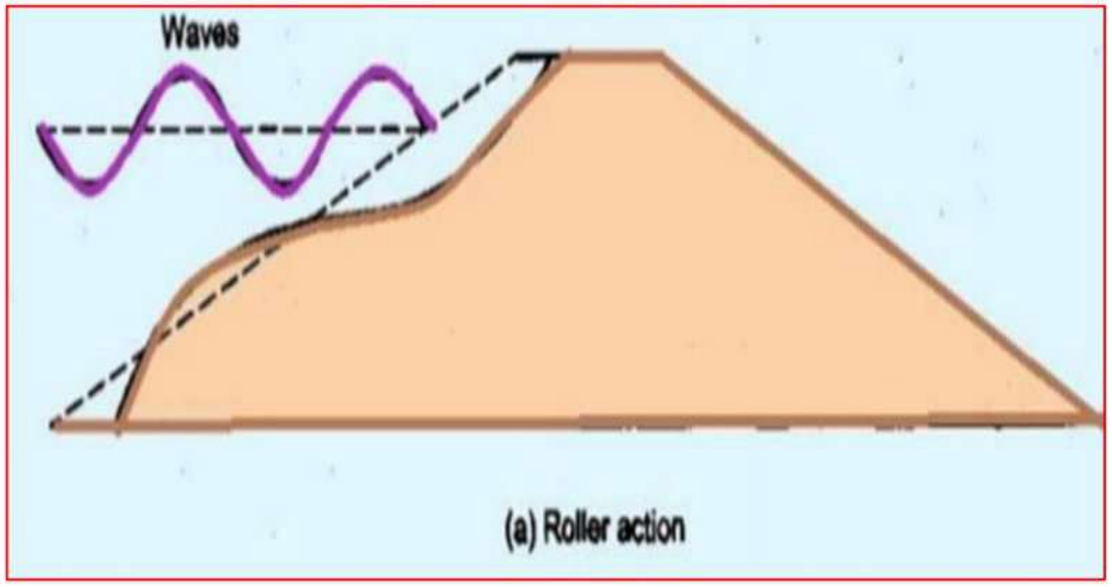
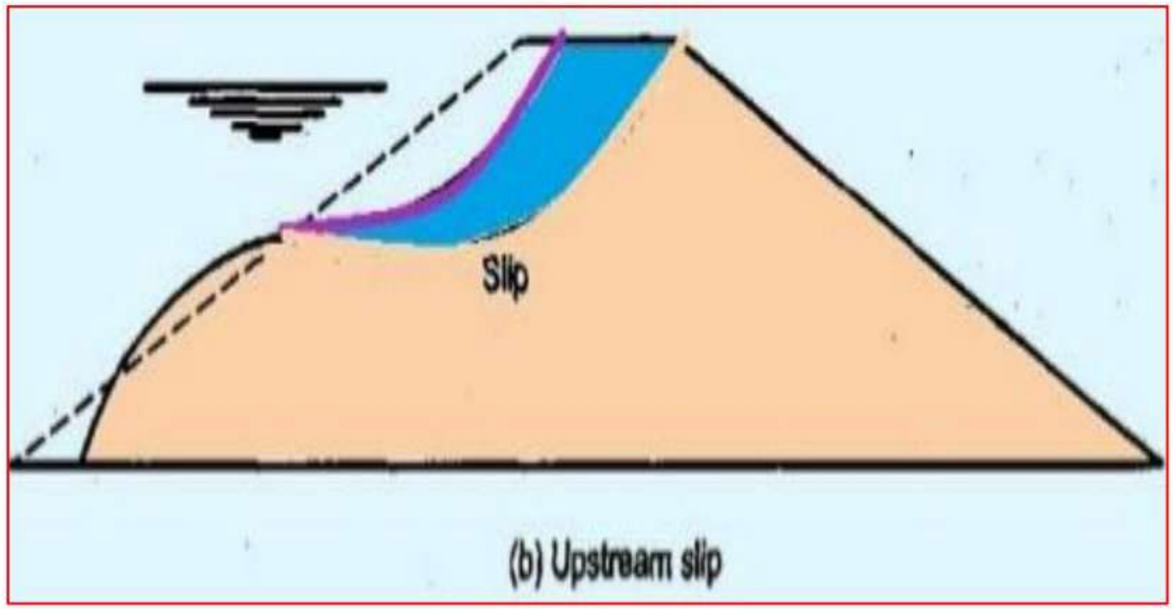
1. Hydraulic Failures

b) Wave erosion:

- The wave action led by wind and tide causes erodes of dam materials at the *upstream face*.
- If the dam materials cannot uphold wave velocity, it will lead wearing up of the dam material or overturning of the protective materials over the upstream side of embankment dam.



(a) Roller action



(b) Upstream slip

c) Toe erosion:

- Toe erosion may occur due to the flood discharge by badly designed spillway channel.
- Downstream slope erosion may be influenced by the downstream water.

d) Surface erosion of the downstream slope:

- Due to heavy rainfall intensity, downstream slope may be eroded and may form several gullies along its slope direction.
- Ultimately serious damage to the dam may occur with long duration of rainfall.

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 # Causes of failure of earth dams…

2. Seepage Failures

Almost 30% of failure will occur due to the seepage failure which can be examined by past experiences. Seepage failure may be categorized as follows:

a) Piping failure:

- If the exit gradient of the seepage flow is greater than the critical gradient of the soil, soil particles are easily influenced for the displacement. ($\mathbf{i_{exit} > i_{critical}}$)
- Piping is nothing but process of progressive erosion of concentrated leaks.
- The piping trouble can be in the body of the dam or in the foundation of the dam.
- The water seeping though the body of the dam causes following detrimental effects.

i) Seeping water develops erosive force which dislodges soil particles from soil structure and cause rearrangement of void between larger grains.

ii) The seeping flow, along with associated differential pore pressure can lift soil particles, causing boiling.

iii) Internal erosion of the soil starts from the point of exit and slowly progresses backwards.

iv) As concentration of leak goes on increasing with more and more movement backwards a conduit or pipe is formed through the soil. Formation of this conduit is known as **piping**.

v) The internal pressures in the soil water reduce internal friction and thus weaken the soil mass.

- The piping trouble can easily occur if soil around the concrete pipe is pervious and is not properly compacted.
- Piping may also take place if there is poor bond or compaction between the dam fill and the foundation or abutment.

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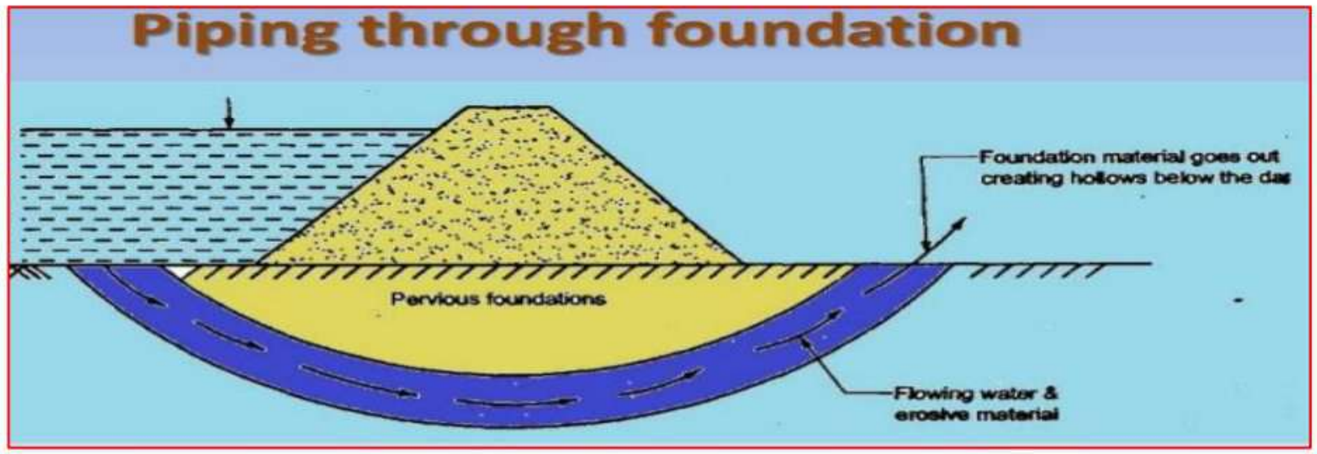
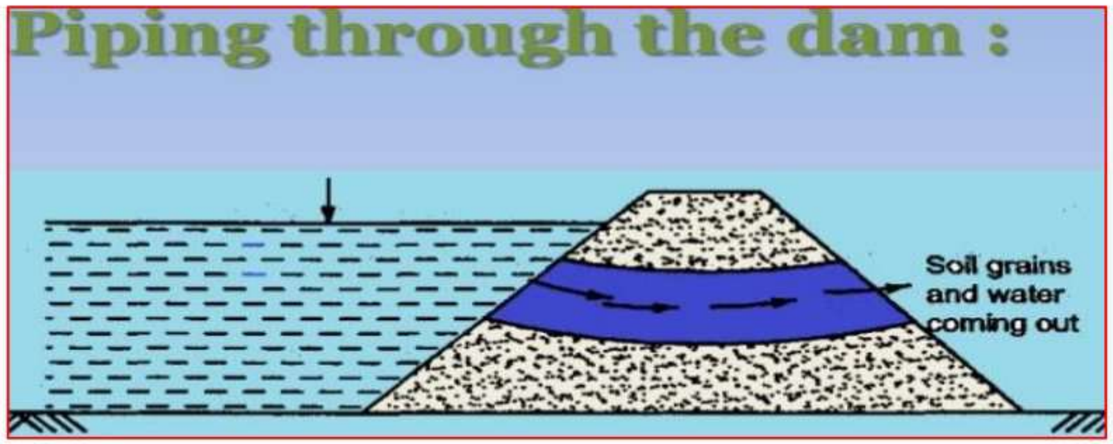
Causes of failure of earth dams…

a) Piping failure:

- As the surface particles are moved, process will be further accelerated towards the downstream direction because seepage gradient will increase constantly.
- Internal erosion and piping are formed with migration of fine particles from cores or/and from foundation particles of the dam body due to the formation of internal cavities.





b) Sloughing.

- The phenomenon of sloughing is very closely related to the process of piping.
- When reservoir is full, the Toe of the dam is generally saturated.
- The Toe may erode producing a small slide.
- This slide makes down stream face more steep which again becomes saturated by seepage and slides again.
- This again causes still more steepness of D/S face.
- This process of seepage and sliding continues till the dam section is rendered too thin to withstand any water pressure and thus cause failure of the dam.
- This process of repeated saturation of the Toe and then its sliding is know sloughing which may also be named as **ravelling.**

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 # Causes of failure of earth dams…

3. Structural Failures

- Causes of structural failure of the earthen dams are the following.

(a) Failure due to pore pressure:

- The drainage capacity of impervious compressible soils is very slow.
- When earth dams are made from such soils, excessive pore pressures are developed in the soil, during and immediately after construction of the dam.
- The amount of pore pressure is dependent upon the permeability of the soil.
- Lesser the permeability greater is the pore pressure.
- If the permeability of the soil being used in the construction of the dam is very low, it is possible that there may be no substantial drop in pore pressure in the central part of the dam, by the time the construction of the dam finishes.
- The amount of pore pressure, equal to weight of the soil has been noticed above the point where pressure is being measured.
- The pore pressure thus may cause failure of the slope and may even impair the stability of the dam.
- Hence construction stage is more critical when soil being used for construction is almost impervious.
- It has been further noticed that pore pressures cause two types of construction slides which differ in speed and magnitude of the movement.
- The first type of slide is uniform and slow and continues for a period of about two weeks. Failure of North Ridge Dam in Canada had been attributed to this cause.
- The second type of slide is sudden and rapid.

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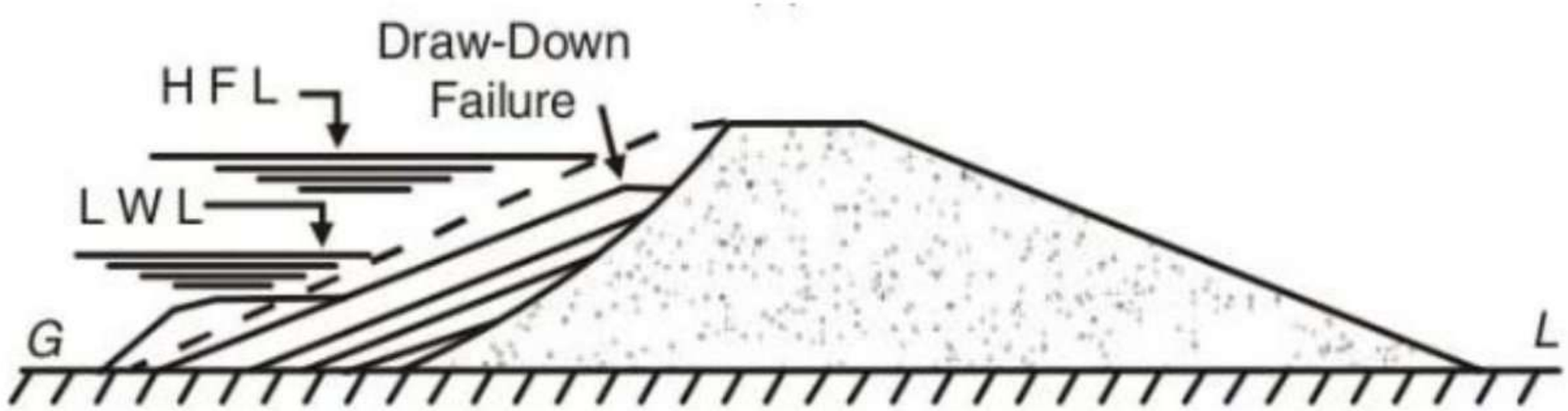
3. Structural Failures….

(a) Failure due to pore pressure:…

- The most of this sliding is completed within few minutes.
- The failure of Marshal Creek Dam in USA is the example of such a failure.
- It is stated that this movement took 15 to 20 minutes only.

b) Sudden drawdown on the upstream face:

- When reservoir is emptied suddenly, the pressure due to water suddenly vanishes from the water face.
- But this action being sudden, the saturated soil up to which water was filled before emptying, does not get time to release water so as to develop equilibrium conditions.
- This phenomenon is know sudden drawdown.
- Due to drawdown the hydrostatic pressure due to reservoir water is removed, without leaving any counteraction against the pressure due to water held in the soil.
- This unbalanced outward pressure due to water held in the soil of upstream face causes upstream face to slide. Such a slide of upstream face normally does not cause failure of the dam, but soil thus slided may block the outlets of conduits and cause difficulties.
- This aspect does not give birth to the process of further sloughing and sliding and hence chances of failure.



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3. Structural Failures….

c) Down stream slope slide:

- The chances of down stream slope sliding are maximum when reservoir is full and rate of percolation is at its maximum rate.
- The earthen dam can easily fail by this reason.
- The slides that may occur on D/S side of the dam may be deep slides or shallow slides.
- Shallow slides occur in sandy soils only and they never exceed 1 to 2 in in depth in direction normal to the slope.
- Deep slides take place in clayey dam and clayey foundations.
- When a deep slide has taken place there is no relief to pore pressure and the unstable vertical slide scrap, left standing often sloughs or slides.
- This process of slides keeps on repeating until dam reaches the point of breach.
- One strong wave of water will now cause breach and failure of the dam.

d) Foundation slide:

- The dam can slide as a whole if it is founded on fine silt or soft soil.
- Presence of soft weak clayey seem or that of silt and sand both can cause failure of foundation.
- Expansion of soil on saturation may cause lifting of the slopes and thus may cause failure of foundation.

e) Failure by spreading:

- When dam is constructed on stratified deposits containing layers of soft clay, the failure by spreading of the fill may take place.

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3. Structural Failures….

f) Slope protections failure:

- The slopes of the dam are generally protected by pitching or rip rap.
- Pitching or rip rap is a protective layer of large boulders.
- This layer is generally laid over a layer of gravel or filter blanket.
- Due to repeated striking of water waves to the riprap at level of the reservoir, the rip rap may be dislocated.
- This exposes the embankment towage erosion.
- The failure of the rip rap may ultimately lead to failure of the dam.

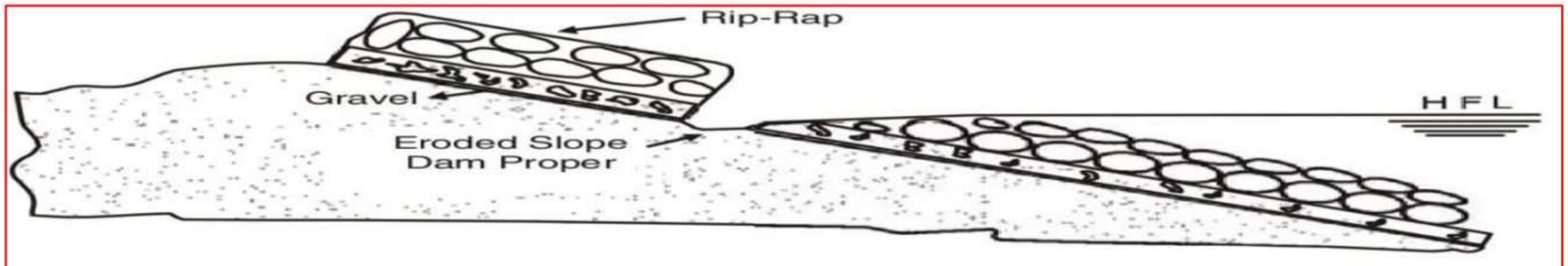

Fig. *Failure of rip-rap.*

g) Failure due to holes caused by burrowing animals:

- The burrowing animals may dig holes through the dam or through foundation and may cause failure of the dam by piping.
- Normally such animals do not dig in moist soils i.e. below the seepage line.
- However, if water level in the dam is very low for a number of years, the burrowing animals may honey comb the dry upper part of the dam.
- As soon as water level rises, the water may escape through the honey combed holes and may cause failure of the dam.

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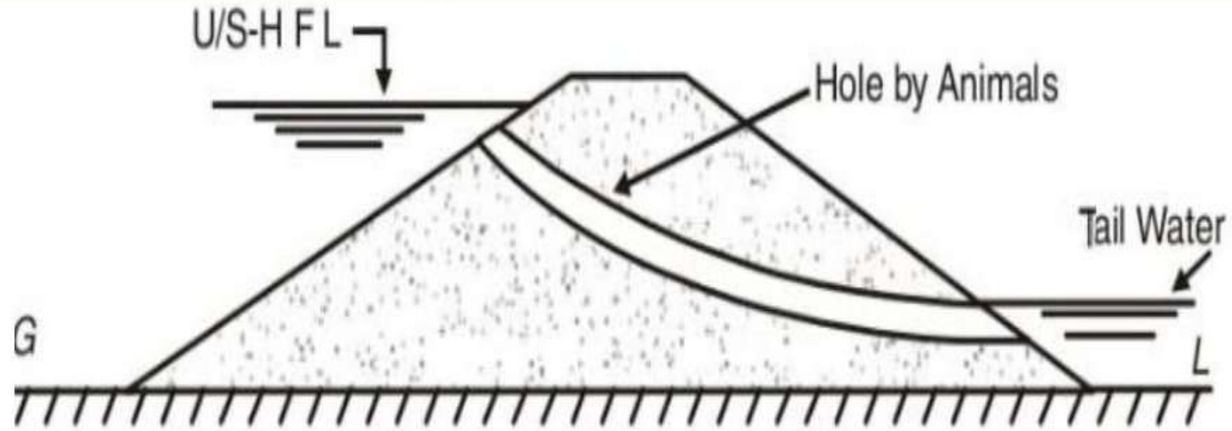
3. Structural Failures….



Failure due to holes caused by burrowing animals:

h) Failure due to earthquake:

Following may be causes of failure of earthen dam due to the effect of earthquakes.

- The core may develop cracks and may lead to leakage and then to ultimate failure.
- Dam itself or its foundation may settle down due to shaking action. This may lead to over topping of the dam.
- Sliding of hill tops into the reservoir and causing rise in reservoir level. This phenomenon may again cause over topping of the dam.
- Horizontal component of the acceleration force due to earthquake may cause shear slide of the appreciable portion of the slope of the dam and may cause dam failure.

i) Failure due to soluble material in dam: If some material which is soluble in water, is present in the dam or in its foundation, it may cause failure of the dam.

j) Piping of seepage erosion through foundation:

- Such materials are washed away with time and may cause settlement of the dam and thus failure.
- Such materials if get deposited in the filters, designed for drainage purpose, may cause clogging of the filters and this may ultimately lead to dam failure, as drainage of the dam will not be proper in that case.

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ESSENTAIL REQUIREMENTS FOR THE SAFE DESIGN OF THE EARTH DAM

If an earth dam has to be safe and stable during construction and in its subsequent operation, it must be designed considering following points.

1. The earth dam should never be allowed to be over topped during heavy flood. This object can be achieved by providing *spillways of adequate capacity*.
2. Sufficient **Free board** should be provided.
3. Seepage line should remains buried sufficiently on the D/S face when reservoir is full. This will prevent occurrence of sloughing of the D/S face.
4. The dam portion laying D/S of the impervious core should be properly drained.
5. The U/S and D/S slopes should be such that they are not damaged by pore pressure during and immediately after construction.

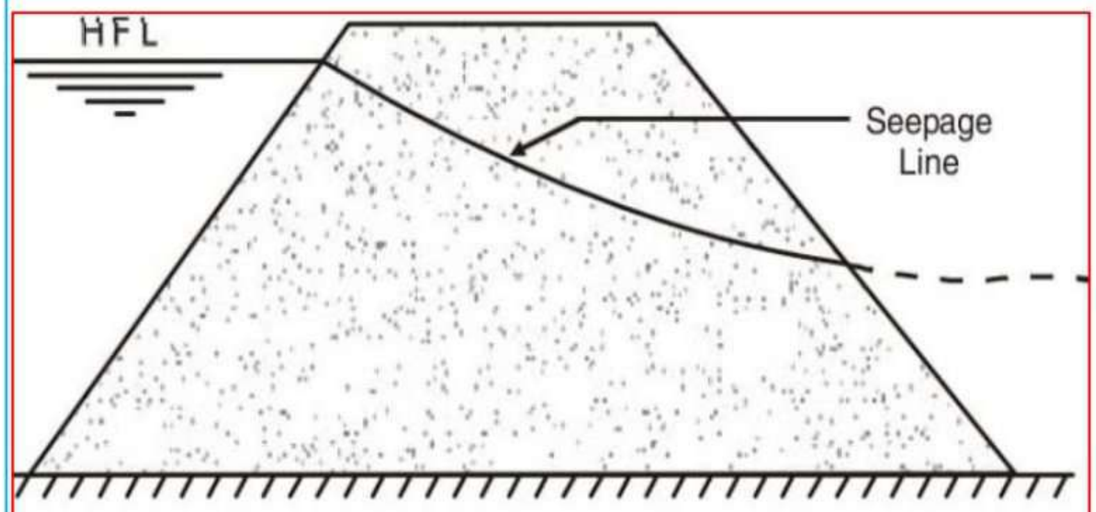

Fig. *Seepage line in the dam.*

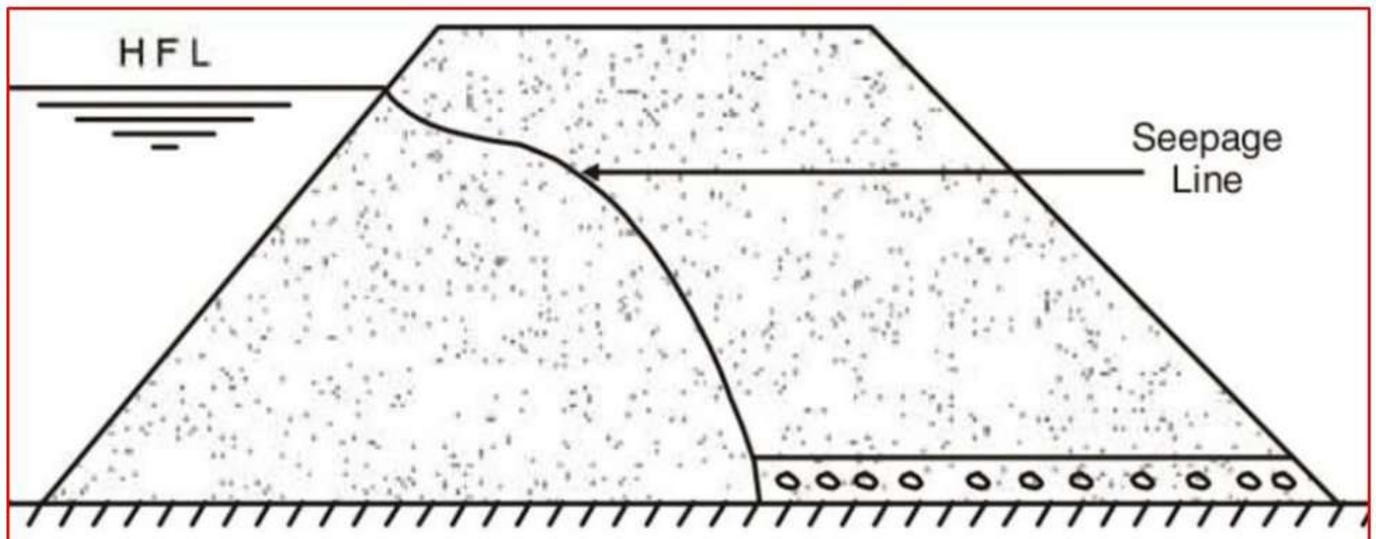

Fig. *Effect or filter on seepage line.*

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ESSENTAIL REQUIREMENTS FOR THE SAFE DESIGN OF THE EARTH DAM…

7. The U/S face of the dam should remain stable during rapid draw downs.

8. U/S face should be protected against wave erosion and D/S face against the erosion due to heavy down pours and strong winds. For this rip rap, revetment or pitching is down on U/S face and counter booms and turfs are developed on D/S face.

9. U/S and D/S slopes of the dam should be flat. This will ensure safety of the dam against shear failure of the foundation.

10. In order to prevent internal erosion of the soil the seepage flow through the dam and foundation should be suitably controlled.

11. Free passage of water through holes made by burrowing animals or by piping should be allowed.

12. In areas, frequently subjected to earthquakes, the dam should be designed considering earthquake resistant measures.

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DESIGN OF THE EARTH DAM…

Design of the section of an earth dam evolves fixing the sizes of the following elements.

1. Height of the dam
2. Top width of the dam
3. Fixation of free board
4. Upstream and downstream slopes
5. Central core of the dam
6. Drainage from downstream side of the dam

1. Height of the dam:

The height of the dam is governed by the High flood level (HFL) and free board i.e. Height of the dam (H) = High Flood level + Free board(FB)

2. Top width of the dam:

- $B = \left(\frac{H}{5} + 3\right)$ for very low dams (H << 30 m)
- $B = \left(0.55\sqrt{H} + \frac{H}{5}\right)$ for (H < 30 m)
- $B = 1.65(H + 1.5)^{1/3}$ for (H > 30 m)
- $B = 1.67\sqrt{H}$ is commonly used formula to calculate the width of the dam.
- Width of the top of the dam, should not be less than 3m for any height of the dam.

3. Fixation of free board:

The vertical height lying between the top of the dam and maximum reservoir level is known as the free board.

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DESIGN OF THE EARTH DAM…

3. Fixation of free board…

- Amount of free board depends upon the wave height of the reservoir.
- The free board should be at least 1.5 times the wave height.
- The wave height (h_w) can be found by
- $h_w = 0.032\sqrt{F.V.} + 0.763 - 0.271\ F^{0.25}$ when F < 32 km
- And $h_w = 0.032\sqrt{F.V.}$ when F >32 km

Where F = Fetch in kilometers

V = velocity of wind in km/hr

h_w = wave height in meters measured between trough and crest of the wave

4. Upstream and downstream slopes

The U/S and D/S slopes of the earth dam depend upon following factors

i. Type of soil used in the construction of the dam.

ii. Foundation condition

iii. Height of the dam

- Flatter slope should be provided on U/S side so as to remain stable under the action of sudden drawdown condition.
- If soils are quite permeable steep slopes can withstand the action of sudden drawdown as pore pressure gets dissipated almost immediately.
- Terzaghi gave following slopes for fixing a tentative section of the earth dam.

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DESIGN OF THE EARTH DAM…

4. Upstream and downstream slopes….

Slopes as recommended by Terzaghi

Types of material	U/S slope (H:V)	D/S slope (H:V)
1. Well graded homogeneous soil	2.5 :1	2:1
2. Homogeneous course silt	3:1	2.5:1
3. Homogeneous silty clay or clay		
a) Height less than 15 m	2.5 :1	2:1
b) Height more than 15 m	3:1	2.5:1
4. Sand or sand and gravel with clay core	3:1	2.5:1
5. Sand or sand gravel with R.C.C. core wall	2.5 :1	2:1

The dimensions fixed by Strange for earth dam are as follows:

Height of Dam	Minimum F. B.	Minimum Top Width	U/S slope (H:V)	D/S slope (H:V)
Up to 4.5m	1.5 m	1.8 m	2 : 1	1.5 : 1
4.5 to 7.5 m	1.5 to 1.8 m	1.85 m	2.5 : 1	1.75 : 1
7.5 to 15m	1.85 m	2.5 m	3 : 1	2 : 1
15 to 22.5 m	2.1 m	3 m	3 : 1	2 : 1

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DESIGN OF THE EARTH DAM…

5. Central core of the dam:

- The core provides impermeable barrier within the body of the dam.
- Form stability point of view thinner core is preferable.

The *thickness of the impervious* core depends upon the following factors

i) Rate of seepage through the core.

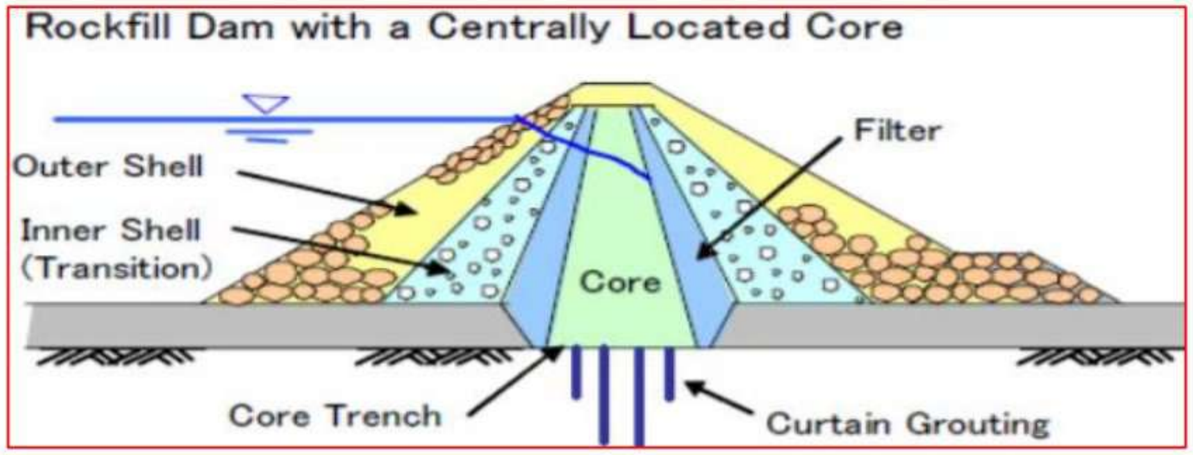
ii) Type of material available for the construction of the core.

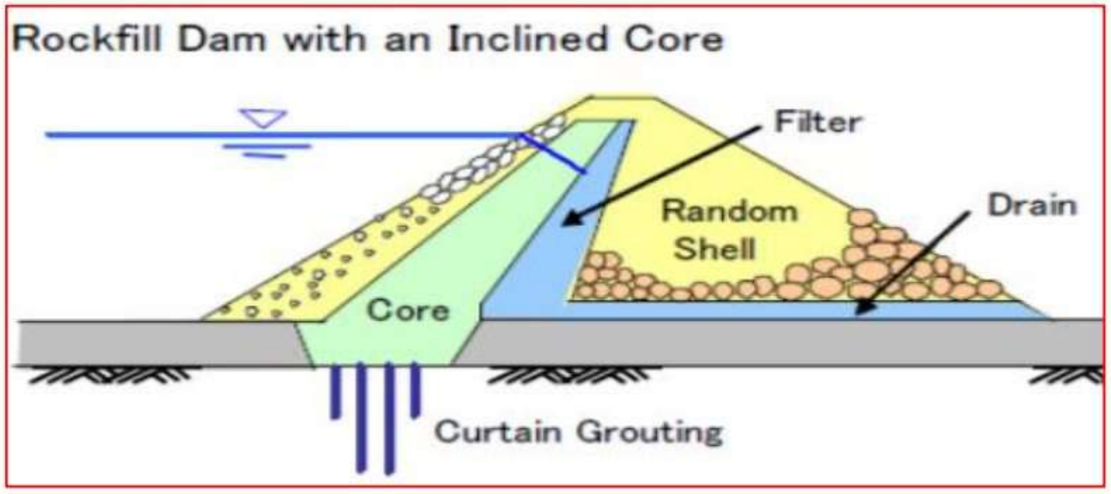
iii) Minimum width required that will permit proper construction.

iv) Design of proposed filter that is to be provide on the D/S side of the core.

- Following guidelines are recommended for design of core (for Small Dams)

1. The core may be located either *centrally or inclined upstream*.
2. The minimum top width *should be kept 3 m*
3. The top level of the core should be fixed at 0.5 m above MWL.
4. The side slopes may be kept 0.5:1 and 1:1.
5. Thickness of core at any section shall not be lesser than 30% (preferably not less than 50 percent) of maximum head of water acting at that section.





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5. Central core of the dam…

Advantages

- The pressure at the contact surface of foundation and the dam is maximum. This reduces the possibilities of leakage along the contact.
- Height of the vertical core being less than the inclined length of the inclined core, thicker vertical core is constructed with the same quantity of core soil. This reduces the *possibilities of piping failure* of the core.

Disadvantages

- D/S part of the dam can be constructed first and inclined core may be laid later.
- Grouting of foundation and construction of embankment can be done simultaneously.

6. Drainage from downstream side of the dam

- Drainage in earth dams is primarily provided to bring the phreatic line well within the downstream face.
- A proper drainage system also helps in avoiding heaving and piping by arresting the soil particles, which may otherwise move by seepage discharge.
- The stability of downstream face is increased by reducing the pore water pressure in the downstream portion of the dam.
- Design of the drainage system is *mainly governed by the height of the dam*, availability of pervious materials and permeability of foundation.

Type of the drainage in earthen dam:

Following are the drainage arrangement in earthen dams

- Horizontal drainage blanket
- Chimney drain
- Toe drain

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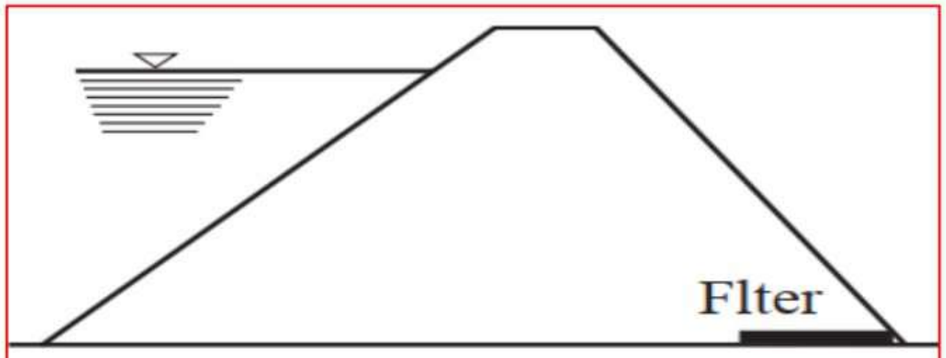
DESIGN OF THE EARTH DAM…

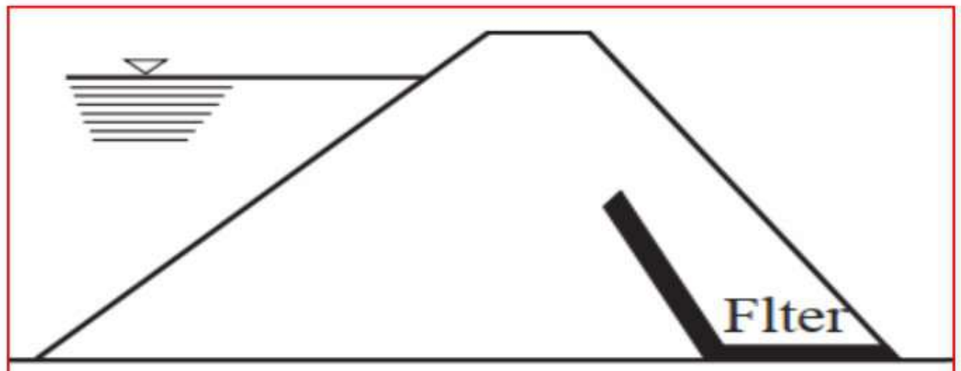
6. Drainage from downstream side of the dam…

a) Horizontal drainage blanket:

- The horizontal blanket is provided over that portion of the foundation downstream from the impervious zone of the dam where high upward seepage force exists.
- The blanket must be pervious while being sufficiently permeable to freely discharge seepage (permeability criterion) and its design should fulfill the filter criterion (piping criterion) in order to prevent migration of the protected soil.
- The length of the drainage can be determined by analysis of flow net, permeability of the foundation



a) Horizontal Drainage blanket



b) Chimney drains

b) Chimney drains:

- It is difficult to make horizontal pervious drain in the earthen dam, thus to overcome this difficulty, the vertical or inclined drains are provided in many homogenous dams to intercept seepage water before it reaches the downstream slope.

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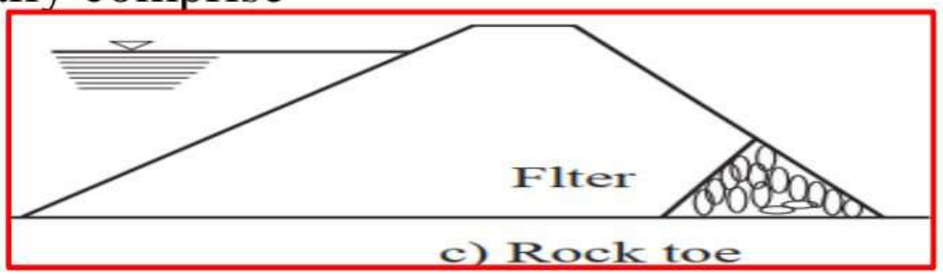
DESIGN OF THE EARTH DAM…

6. Drainage from downstream side of the dam…

c) Rock toe:

- The rock toe consists of size usually varying from 15 cm to 20 cm filter is required to be provided between the homogenous fill and the rock toe. The filter usually comprise
- 30 cm thick layer of fine sand 15% size=0.09mm
- 45 cm thick layer of coarse sand 15% size=0.81mm
- 60 cm thick layer of gravel 15% size=7.3mm
- The height of the rock toe depends on the minimum allowable cover on the phreatic line and downstream water level.



c) Rock toe

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FILTER DESIGN:

- Filter drains should be design in such a way that all the seeping water through the dam is effectively drained off. The filter consists of more than one layer.
- The filter layer which comes in contact with the seeping water *first is of fine material.*
- Subsequent layers of filter are made of sand of increased coarseness.
- The *last layer* from which water comes out of the filter is made of *gravel.*
- The filters of filter drains are also know reverse or invert filter.
- The soil of earth dam and foundation surrounding the filter is know the **base material**.

According to earth manual of US Bureau of Reclamation Washington 1960 following four main requirements should be satisfied.

- Filter material should be fine and poorly grad so that the voids in the filter are small and thus prevent base material from entering the filter.
- The filter material should be coarse and pervious in relation to base material. This aspect facilitates *rapid removal of seeping water* without building up any seepage force within the filter.
- The openings or perforation in the pipe drain should be adequate to admit all the seeping water safely.
- The thickness of filter material should be sufficient to provide a good distribution of all particle sizes, throughout the filter.
- The thickness should also be adequate to provide safety against piping.

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FILTER DESIGN….

According to Terzaghi, the filter should fulfill the following two requirements

1. The D15 size of filter material must not be **more** than 4 to 5 times D_{85} size of the base material. This prevents the foundation material from carrying through the pores of the filter material.

2. The D15 size of filter material should be **at least** 4 to 5 times D_{15} size of the base material. This keeps seepage forces within the filter to permissible limit.

The two criteria can be expressed as follows also;

$$\frac{D_{15}\text{ of filter}}{D_{85}\text{of base material}} < \mathbf{4\ to\ 5} < \frac{D_{15}\text{ of filter}}{D_{15}\text{of base material}}$$

- The requirements cited above should be satisfied between any two adjacent layers of the filter.
- Terzaghi's criteria has been modified as per U. S. Bureau of Reclamation Design of Small Dams'. The modifications are as follows

a. $\frac{D_{15}\text{ of filter}}{D_{15}\text{of base material}}$ = 5 to 40, provided that the filter does not contain more than 5 % of material finer than 0.075mm.

b. $\frac{D_{15}\text{ of filter}}{D_{85}\text{of base material}}$ = 5 or less,

c. $\frac{D_{85}\text{ of filter}}{\text{Maximum opening of performance of the pipe}}$ = 2 or more

d. The grain size curve of the filter material should be about parallel to the curve of the base material.

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SEEPAGE CONTROL MEASURES:

The safety of earth dam depends almost entirely on seepage control through dam and its foundation. Hence measures of seepage control are very important for the success of an earthen dam.

A. Seepage control through the dam.

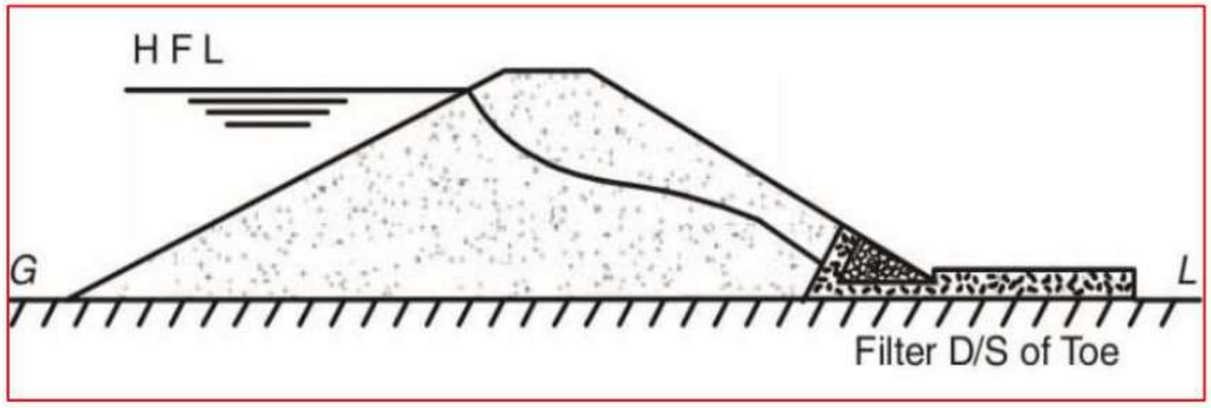
B. Seepage control through the foundation.

- Various measures of seepage control under each category have been discussed in brief.

A. Seepage Control Through the Dam

1. Rock Toe:

- Rock toe keeps seepage line well within the dam section.
- It also helps a great deal for the drainage purposes.
- The *height of the rock toe* is kept about *one fourth of the height* of the dam.
- Rock toe should be designed like filter.



2. Horizontal Drainage Filter:

- It is provided at the base of the dam, starting from *down stream end of the dam* and extending backwards into the dam.
- Backward extension of the filter depends upon so many factors.
- But this extension may at the most be up to *Centre line of the dam.*
- This filter controls seepage line and does not allow it to get exposed on D/S face of the dam.

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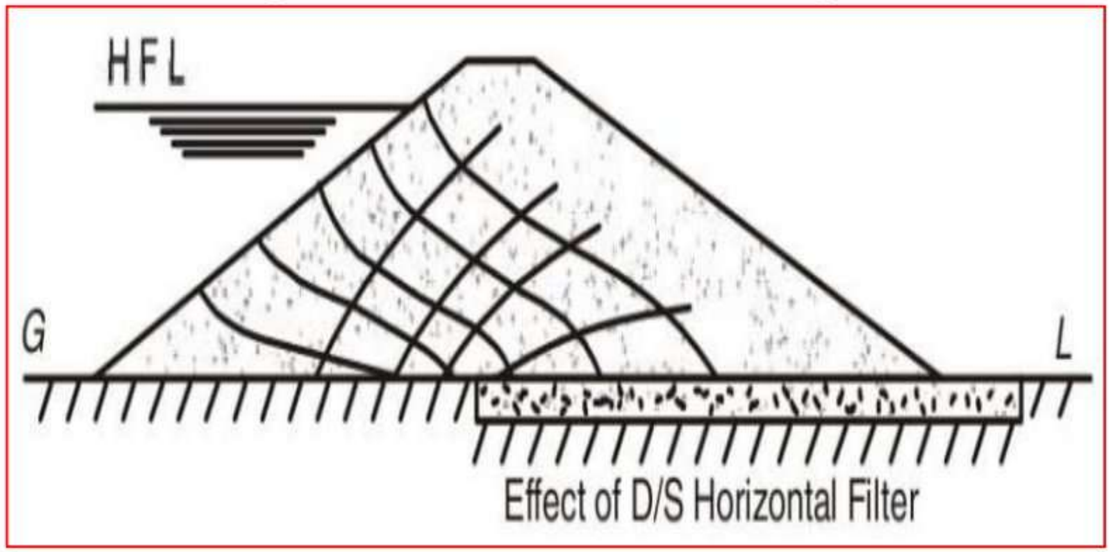
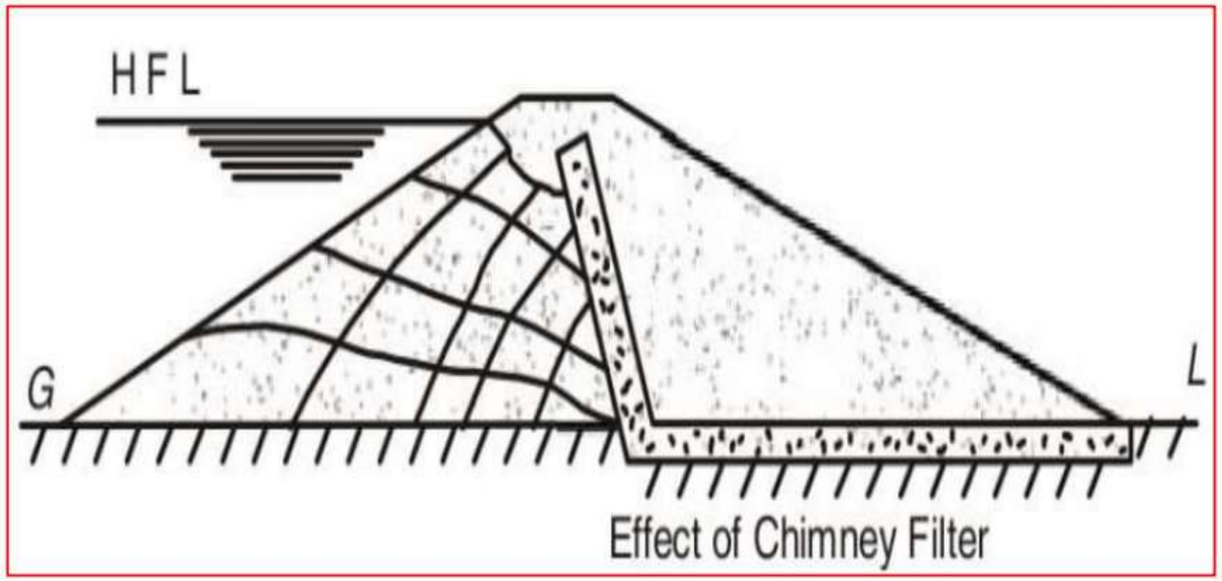
SEEPAGE CONTROL MEASURES…

2. Horizontal Drainage Filter….

- It also accelerates the process of consolidation.
- It also causes drainage of foundation.
- If seepage pressure at the D/S end of the dam is *still excessive*, the horizontal filter drain may be continued even *beyond the D/S toe* of the dam.
- Sometimes rock toe and horizontal filter drains are distributed with and entire D/S portion of the dam may be made from coarse-grained soil.



Effect of D/S Horizontal Filter



Effect of Chimney Filter

3. Chimney Drains:

- Under conditions of large stratification, the permeability in the horizontal direction is more than in the vertical direction. This causes a *greater speed of horizontal seepage* than vertical seepage.
- Chimney drain or filter, if correctly built, intercepts all the seepage from the dam regardless of the stratification in the dam.
- Chimney drains also render earth dam earthquake resistant.

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SEEPAGE CONTROL MEASURES

B. Seepage control through the foundation:

1. Impervious Cut-off:

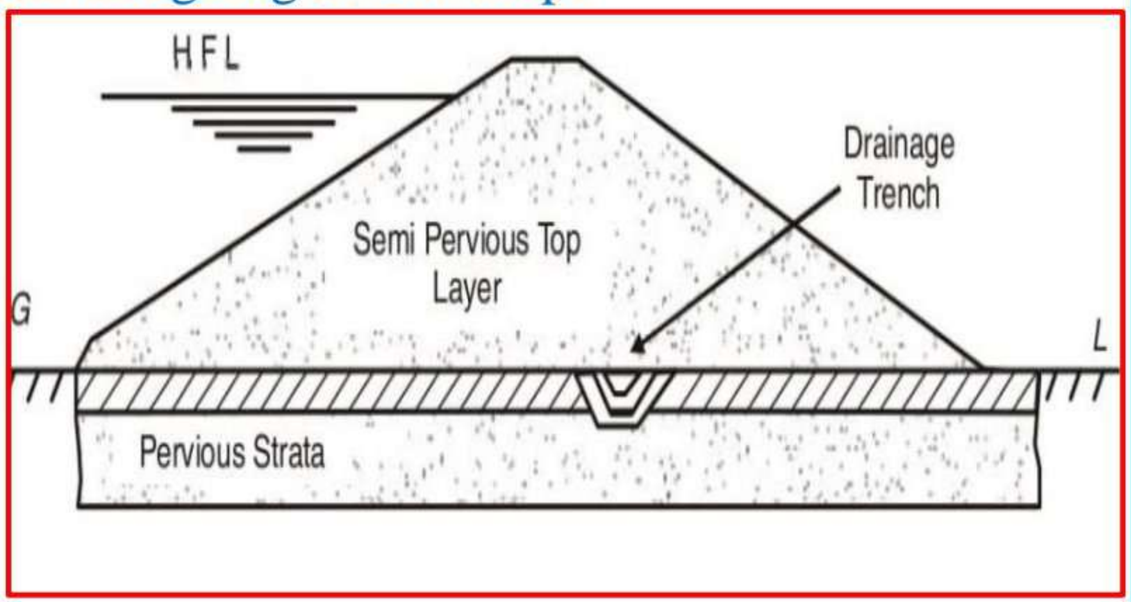
- The cut-off is a wall of relatively impervious material.
- It is used to prevent seepage through the foundation.
- It is always led down into the foundation from the ground surface.
- If possible, cut-off should extend up to the impervious strata lying below the ground level.
- Partial cut-offs do not prove much effective in preventing seepage.
- A 90% depth of cut-off reduces about 25% seepage.

2. D/S Seepage Berms:

- Additional berms may be built on the D/S side of the dam in continuation of the D/S end.
- Such a berm is useful in controlling seepage where D/S top strata is relatively thin and uniform or even top strata is absent.
- Such berms help in checking uplift of soil and also check sloughing of D/S slope of the dam.

3. Drainage Trenches:

- This measure is adopted when top stratum is pervious and thin.
- They are provides along the axis of the dam.
- They have a porous drain *having longitudinal slope*.
- Porous drains remain enclosed in gravel filters.



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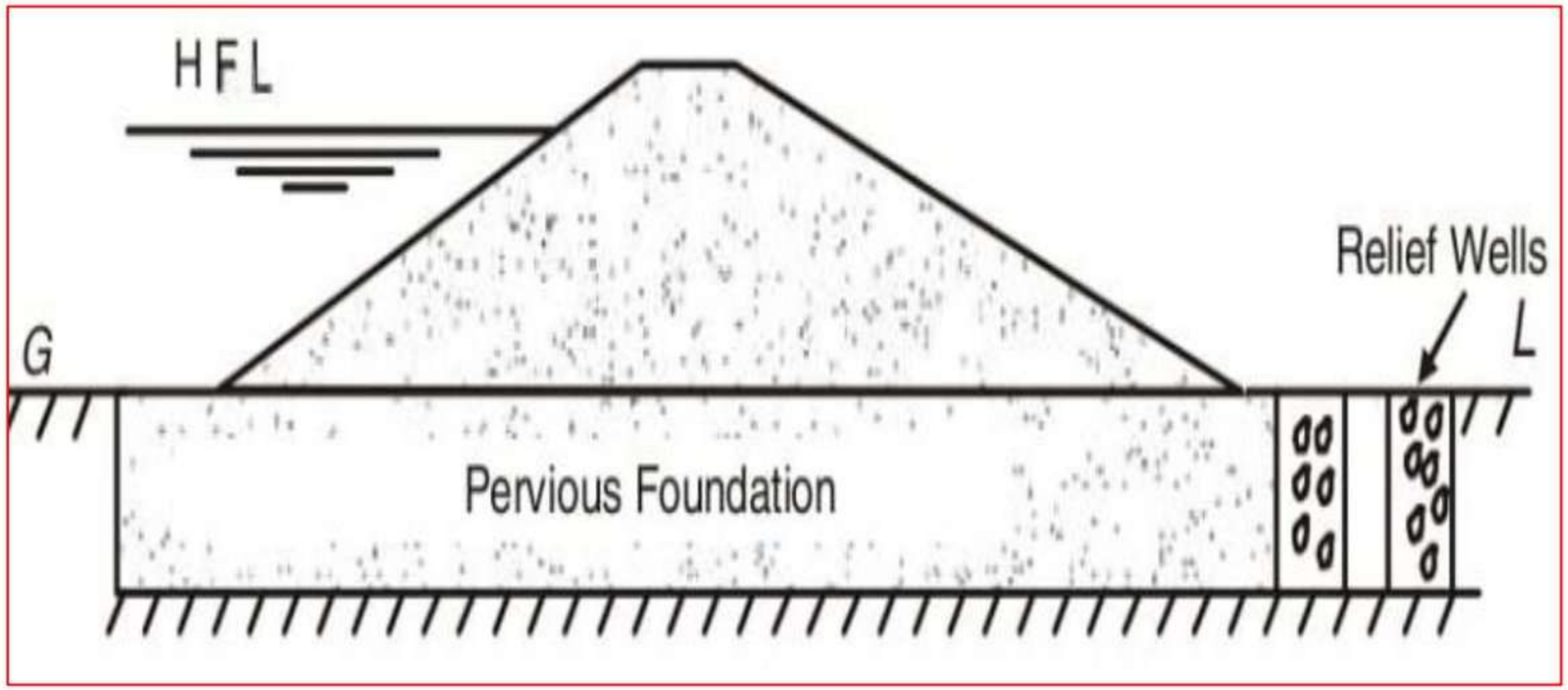
B. Seepage control through the foundation….

4. Relief Well:

- It is such a well which if not constructed would cause formation of *sand boils and possibly sub-surface piping.* They reduce the sub surface uplift pressure D/S of the dam.
- They intercept the seepage through the foundation and control the outlet for seepage.
- Relief wells become necessary when impervious layer overlies a pervious layer and the thickness of overlying impervious layer is less than the depth of water impounded.
- Relief wells consist of 10 cm to 15 cm diameter holes filled with filter material.



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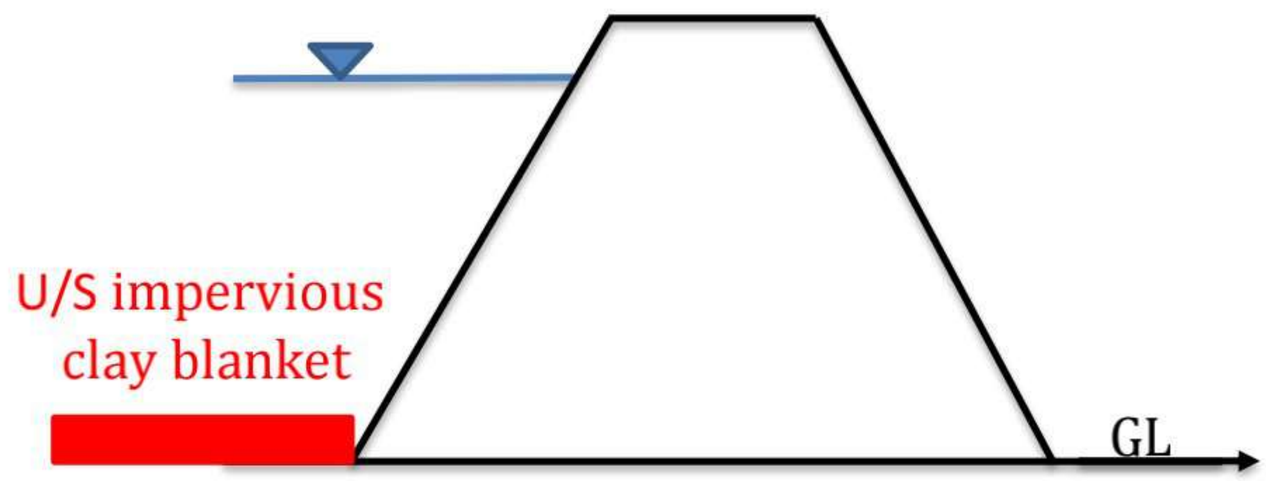
B. Seepage control through the foundation….

5. Upstream Impervious Blanket:

- Such a blanket when constructed over a pervious foundation reduces the quantity of seepage on D/S side.
- It also causes reduction in uplift pressure throughout the D/S side.
- The provision of U/S blanket is found economical and more effective when the depth of previous overburden is large and the provision of the cut-off wall is uneconomical.
- Blankets are particularly effective when there are cracks and fissures in the foundation beneath the dam structure. In such cases they seal such openings and reduce the seepage considerably.
- The blanket should be composed of such material which is *at least 100 times less pervious* than the foundation material.
- Thickness of the blanket varies usually between 1 m and 3m.
- It is such a well which if not constructed would cause formation of sand boils and possibly sub-surface piping.



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Seepage Analysis:

Seepage is defined as the flow of a fluid, usually water through a soil under a hydraulic gradient. In the stability of slopes and dam design, the seepage force is very important factor.

Two dimensional flow – Laplace's equation

The analysis of two dimensional flows through a saturated soil mass involves determination of quantity of seepage and distribution of seepage pressure and can be done with method based on Laplace equation.

The derivation of Laplace equation is based on the **following assumptions**

- The soil mass is fully saturated and is incompressible. Water is also incompressible.
- Quantity of water entering any element of soil mass is equal to the quantity of water leaving the element during a given time.
- Darcy's law for flow through soil medium is valid.
- The hydraulic boundary conditions at entry and exit are known.

Consider an element of size Δx, Δz and of unit thickness perpendicular to plane figure.

The corresponding velocity components at exit will be

$V_x + \frac{\partial V_x}{\partial x}\Delta x$ and $V_y + \frac{\partial V_z}{\partial z}\Delta z$

According to assumption that the quantity of water Entering an element is equal to the quantity of water leaving the element in any given time, we have

$$V_x\,(\Delta z * 1) + V_z\,(\Delta x * 1) = \{(V_x + \frac{\partial V_x}{\partial x}\Delta x) * (\Delta z * 1)) + \{(V_z + \frac{\partial V_z}{\partial z}\Delta z) * (\Delta x * 1)\}$$

$$V_x \Delta z + V_z\,\Delta x = V_x\,\Delta z + \frac{\partial V_x}{\partial x}\Delta x * \Delta z + V_z\,\Delta x + \frac{\partial V_z}{\partial z}\Delta z * \Delta x$$

$$\frac{\partial V_x}{\partial x}\Delta x * \Delta z + \frac{\partial V_z}{\partial z}\Delta z * \Delta x = 0$$

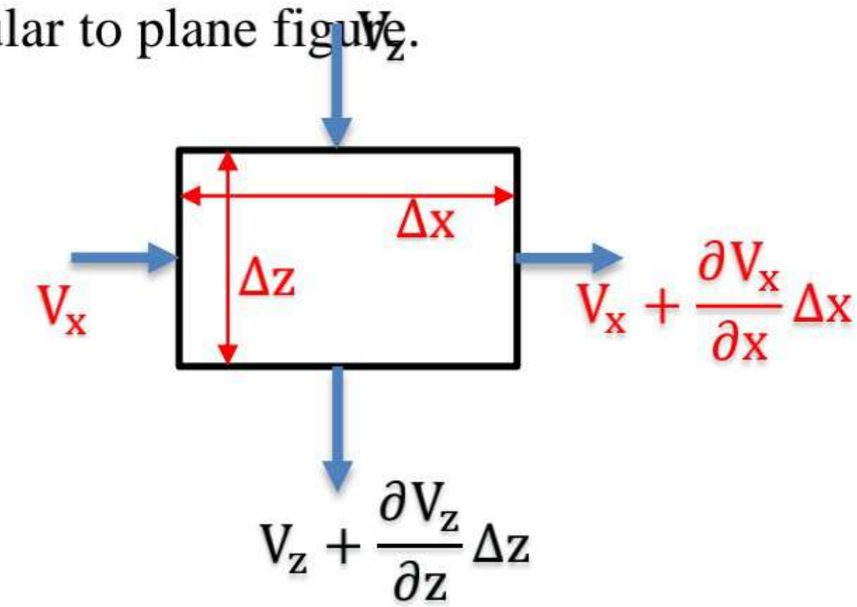

Figure: Two dimensional flow

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$\frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z} = 0 \ldots (1)$, This is continuity equation for two dimensional flow.

According Darcy's Law

$V_x = K_x * i_x = K_x * \frac{\partial h}{\partial x}$

and $V_z = K_z * i_z = K_z * \frac{\partial h}{\partial z}$

Where K_x and K_z are coefficient of permeability in 'X' and 'Z' direction respectively. By substitution in equation (1). We get,

$$K_x * \frac{\partial^2 h}{\partial x^2} + K_z * \frac{\partial^2 h}{\partial z^2} = 0$$

For an isotropic soil medium $K_x = K_z = K$

$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0 \ldots..(2)$, This is Laplace equation governing the steady state two dimensional flow of incompressible fluid through an incompressible porous material.

<u>Potential Drop:</u>

The loss of head between the two successive equipotential lines is called the potential drop. For convenience usually the equipotential lines are so constructed that the drop between any two successive equipotential lines is constant. If N_d is the numbers of drops and total head causing the flow is h_1 then,

$$\Delta h = \frac{h_1}{N_d}$$

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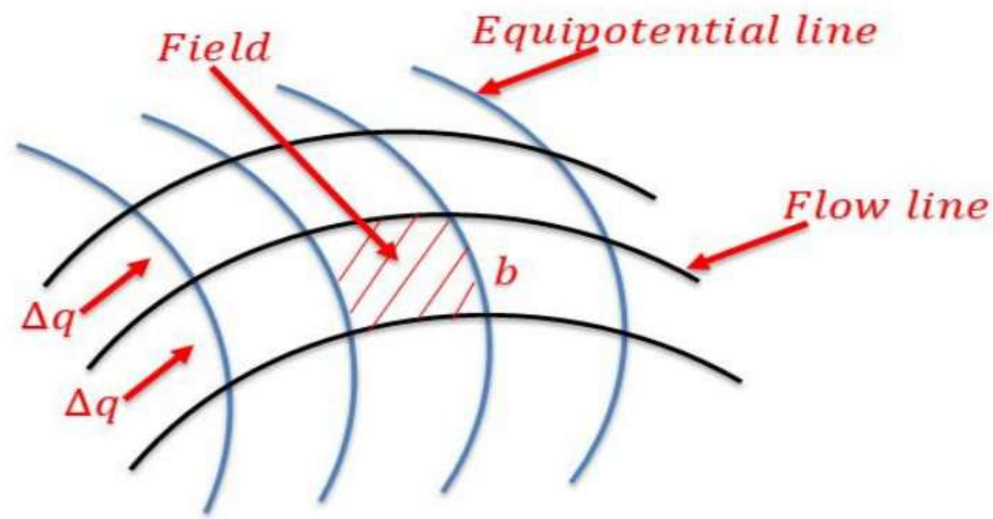
Flow Net:

- The general solution of the Laplace equation yields two sets of curves orthogonal to each other. One set of curve is known as the flow lines and other set as equipotential lines.
- The net formed by intersecting the possible flow lines and equipotential lines known as flow net. Flow net is a graphical representation of seepage flow through soil mass.
- A flow line represents the path traced by an individual water particle.
- An equipotential line is contour or line joining points of **equal potential or head**.
- The flow lines and equipotential lines cut each other at right angles i.e. they are mutually orthogonal as shown in figure.
- The space between any two adjacent flow lines is

Called **flow channel**.

- The space enclosed between two

adjacent flow lines and two successive equipotential lines is called a **field**.



Properties of Flow lines:

Following properties of flow net are considered before preceding for construction and application of flow nets

- Flow lines and equipotential lines cut each other at right angle, i.e. they are mutually orthogonal.
- Each field is an approximate square and in a well constructed flow net one should be able to draw a circle in a field touching all the four sides.
- In a homogeneous soil, every transition in the shape of the two types of curves will be smooth, being either elliptical or parabolic in shape.
- The rate of flow through each flow channel is same.
- The same potential drop occurs between two successive equipotential lines.

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Construction of Flow Net:

The flow net can be constructed by any one of the following methods,

i) Graphical method
ii) Mathematical or Analytical method
iii) Electrical flow method
iv) Capillary flow method
v) Sand model

Out of various methods listed above, the most convenient and least expensive procedure of obtaining flow nets is the graphical method where the flow net is constructed by trial and error.

Graphical method:

This method is simple and quickest. But the method needs a lot of practice. Before constructing a flow net using the graphical procedure the following points are to be kept in mind

- The boundary condition should be established.
- Each flow lines must cur the equipotential lines at right angles to each other.
- The space enclosed by adjacent flow lines and equipotential lines must be curvilinear squares.

The following steps are suggested for drawing a flow net

1) Draw smooth curves representing flow lines satisfying the requirements mentioned earlier.
2) Draw equipotential lines such that they intersect the flow lines at right angles. The equipotential are so drawn that the fields are approximately curvilinear squares.
3) Inspect defect if any and rectify them.
4) When all the fields are curvilinear squares the net obtained will be satisfactory for all practical purposes.

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Application of Flow Net:

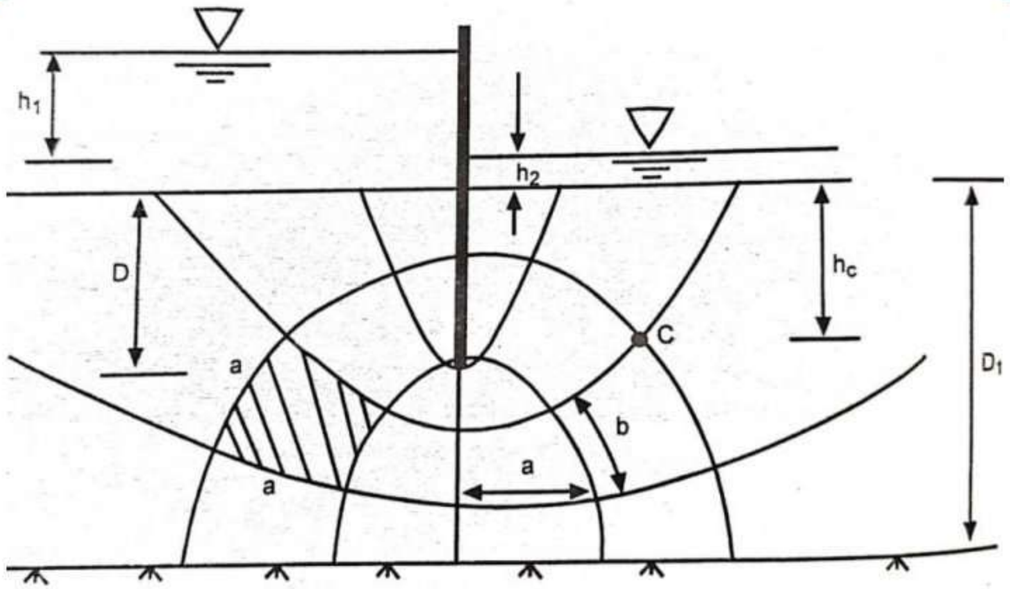
The flow net can be utilized for computing the following,

i) Determination of seepage discharge

ii) Determination of hydrostatic pressure

iii) Determination of seepage force

iv) Determination of exit gradient



Flow of water underneath of a sheet pile

i) Determination of Seepage Discharge:

Let us consider a field indicated by a shaded area As shown in figure made for sheet pile as mentioned Before. Suppose the length of field in the direction of flow is 'a' right angles to it is 'b'. The hydraulic gradient across the field is, $i = \frac{\Delta h}{a}$

The discharge velocity is given by

$$V = K*I = K * \frac{\Delta h}{a} = \frac{K}{a} * \frac{h_1}{N_d}$$

Where N_d = No. of potential drops

Let ΔQ be the quantity of water flowing the field. The flow per unit widths is given by,

$$\Delta Q = V * A = \frac{K}{a} * \frac{h_1}{N_d} *(b*1)$$

$$= \frac{b}{a} *K * \frac{h_1}{N_d}$$

If we make a square field, then b = a and the discharge becomes

$$\Delta Q = k * \frac{h_1}{N_d}$$

If N_f be the number of flow channels in the flow net then the total discharges is given by

$$Q = (N_f * \Delta Q) = N_f * k * \frac{h_1}{N_d} = k * h_1 * \frac{N_f}{N_d}$$ .....(1) this equation gives the seepage discharge if a flow net diagram is available.

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1) A concrete dam is constructed across a river over permeable strata of the soil of limited thickness. The water head on U/S side is 16 m and 2 m on the D/S side. The flow net constructed under the dam gives Nf = 7 and Nd = 21. Calculate the seepage loss through the subsoil if the average value of coefficient of permeability is 6*10^-3 cm/sec horizontally and 3*10^-4 cm/sec vertically. Calculate the exit gradient if the average length of the last field is 0.9m.

Solution:

No. of flow channel (Nf) = 7

No. of equivalent drops (Nd) = 21

Water level on U/S side (h1) = 16 m

Water level on D/S side (h2) = 2 m

Total hydraulic head causing flow (h) = 16 – 2 = 14 m

(i) Equivalent coefficient of permeability $k_e = \sqrt{k_x * k_z}$

$= \sqrt{(6 * 10^{-3} * 3 * 10^{-4})}$

$= 1.34 * 10^{-3}$ cm /s = $1.34 * 10^{-5}$ m /s

Again,

(ii) $q = k * h * \frac{N_f}{N_d}$

$= 1.34 * 10^{-5} * 14 * \frac{7}{21}$

= $6.25 * 10^{-5}$ m^3 /s /meter length

(iii) Head loss per potential drop = $\Delta h = \frac{h}{N_d} = \frac{14}{21} = 0.67$m

(iv) Exit Graident (i) = $\frac{\Delta h}{L_e}$

$= \frac{0.67}{0.9}$

$= 0.74$

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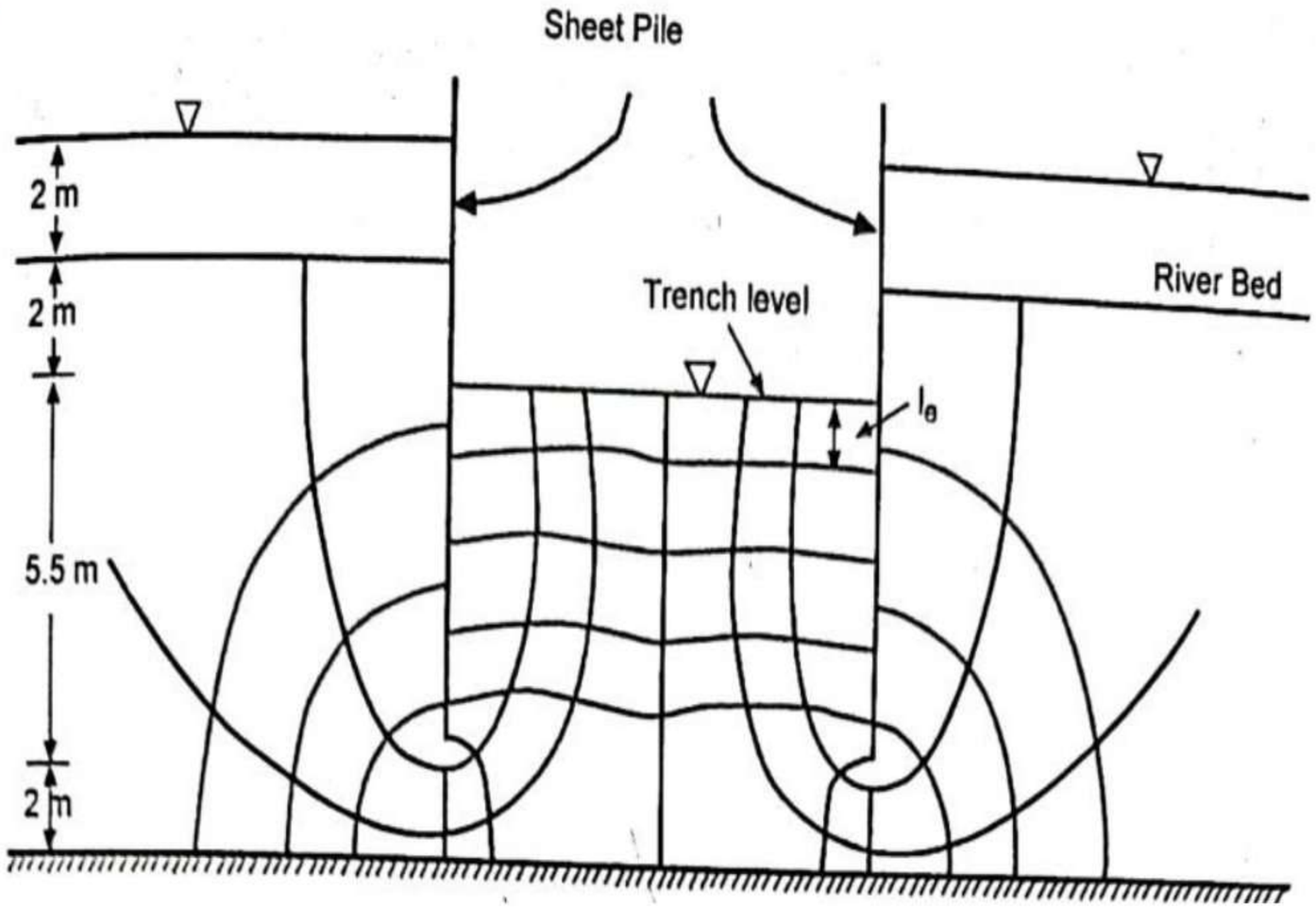
2) If in figure the water level between the sheet piles is kept at trench level at a rate of pumping 0.2 m^3/hr per meter length of sheet pile, compute the coefficient of permeability of soil. Calculate the exit gradient if the average length of the last field is 0.9m.



Solution:

No. of flow channel (Nf) = 6

No. of equivalent drops (Nd) = 10

Total head of flow (h) = 4 m

Now,

(i) $q = k * h * \frac{N_f}{N_d}$

$K = \frac{q}{h} * \frac{N_d}{N_f}$

$= \frac{0.2}{(60*60)*4} * \frac{10}{6}$

$= \mathbf{2.31 * 10^{-5}}$ **m /s**

(ii) Head loss per potential drop

$\Delta h = \frac{h}{N_d}$

$= \frac{4}{10}$

= 0.4 m

(iii) Exit Gradient (i) $= \frac{\Delta h}{L_e}$

$= \frac{0.4}{0.9}$

= 0.44

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4) A soil stratum with permeability, K = $5*10^{-7}$ cm/sec. overlies an impervious stratum. The impermeability stratum lies at a depth of 18 m below the ground surface (surface of soil stratum). A sheet pile wall penetrates 8 m into the permeable soil stratum. Water stands to a height of 9 m on upstream side and 1.5 m on downstream side, above the surface of soil stratum sketch the flow net and determine,

i) Quantity of seepage
ii) The seepage pressure at a point 'P' located at a 8m below surface of soil stratum and 4 m away from the sheet pile wall on its upstream side.
iii) The maximum exit gradient , if average length of the field is 2.8 m.
iv) The pore pressure at point 'P'

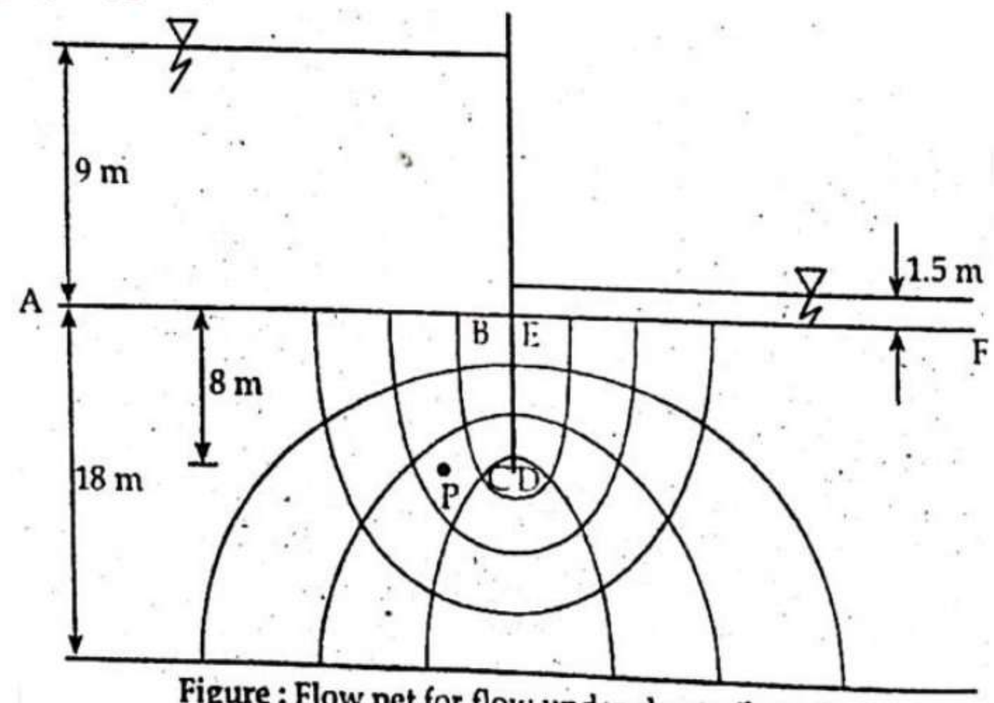

Figure : Flow net for flow under sheet pile wall

Solution:

No. of flow channel (Nf) = 4

No. of equivalent drops (Nd) = 8

Total head of flow (h) = 9 – 1.5 = 7.5 m

Now, $q = k*h*\frac{N_f}{N_d} = 5*10^{-9}*7.5*\frac{4}{8}$

$= 18.75*10^{-9}$ m^3 /s /meter length

(ii) Head loss per potential drop, $\Delta h = \frac{h}{N_d} = \frac{7.5}{8} = 0.9375$ m

(iii) No. of potential drops up to point 'P', **n = 2.5**

- Total head at point 'P', $h_s = H - (n*\Delta h)$

= 7.5 – (2.5*0.9375) = 5.156 m

- Seepage pressure at point 'P' $P_s = \gamma_w * h_s$= 9.81*5.156 = 50.58 KN / m^2
- Hydrostatic head at 'P', $h_w = h_s - z$ = 5.156 – (– 9.5) = 14.656 m
- Pore pressure at 'P', $u = \gamma_w * h_w$ = 9.81*14.656 = 143.776 KN / m^2
- Exit Gradient (i) = $\frac{\Delta h}{L_e} = \frac{0.9375}{2.8} = 0.335$

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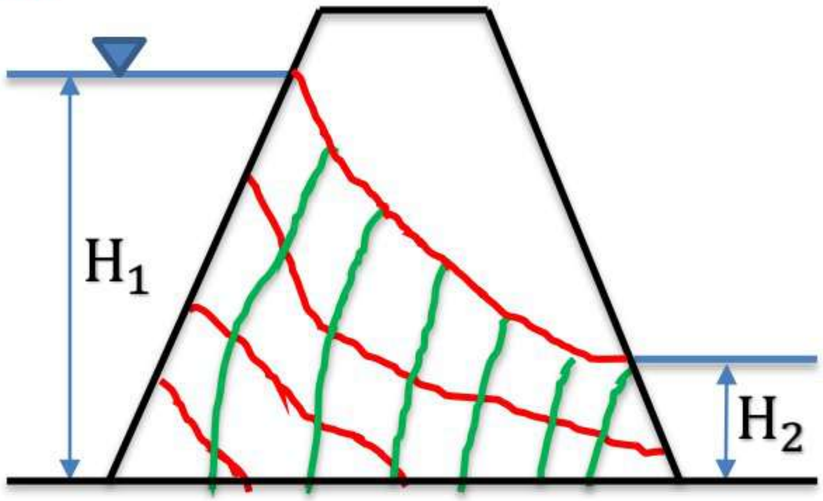
Seepage Discharge through a dam with impervious foundation:



Case I : Phreatic line emerging with the downstream water:

Here $\in = \frac{0.41}{1+\frac{1}{2m_u}}$

From Darcy's law, the seepage discharge is Q = KiA

Where, K = coefficient of permeability, i = hydraulic gradient,

Specific seepage discharge, q = Q/B

Considering unit width of dam, $q = \frac{KiA}{B}$

$= \frac{Ki(h*1)}{1} = Kih$

$q = kh(-\frac{dh}{ds})$, this negative sign indicated the head is decreasing with respect to the distance.

$q*ds = -Kh*dh$

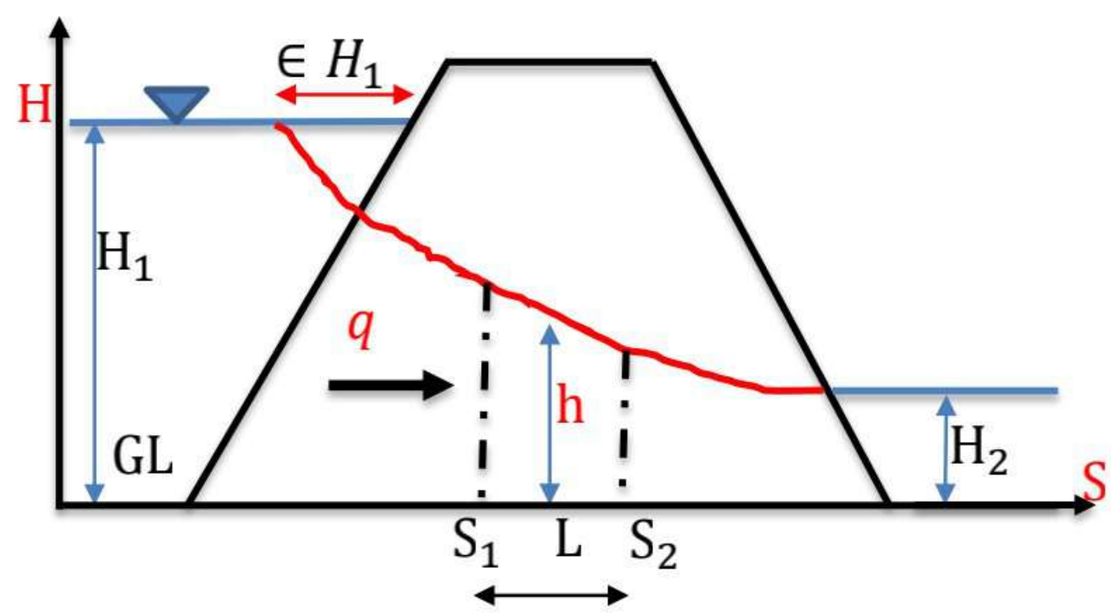
Integrating

$q\int_{S1}^{S2} ds = -K\int_{H1}^{H2} hdh$

$q[s]_{S1}^{S2} = -K\left[\frac{h^2}{2}\right]_{H1}^{H2}$

$q(S_2 - S_1) = -K\left(\frac{H_2^2 - H_1^2}{2}\right)$

$q = \frac{K}{2L}(H_1^2 - H_2^2)$ {where $(S_2 - S_1) = L$}



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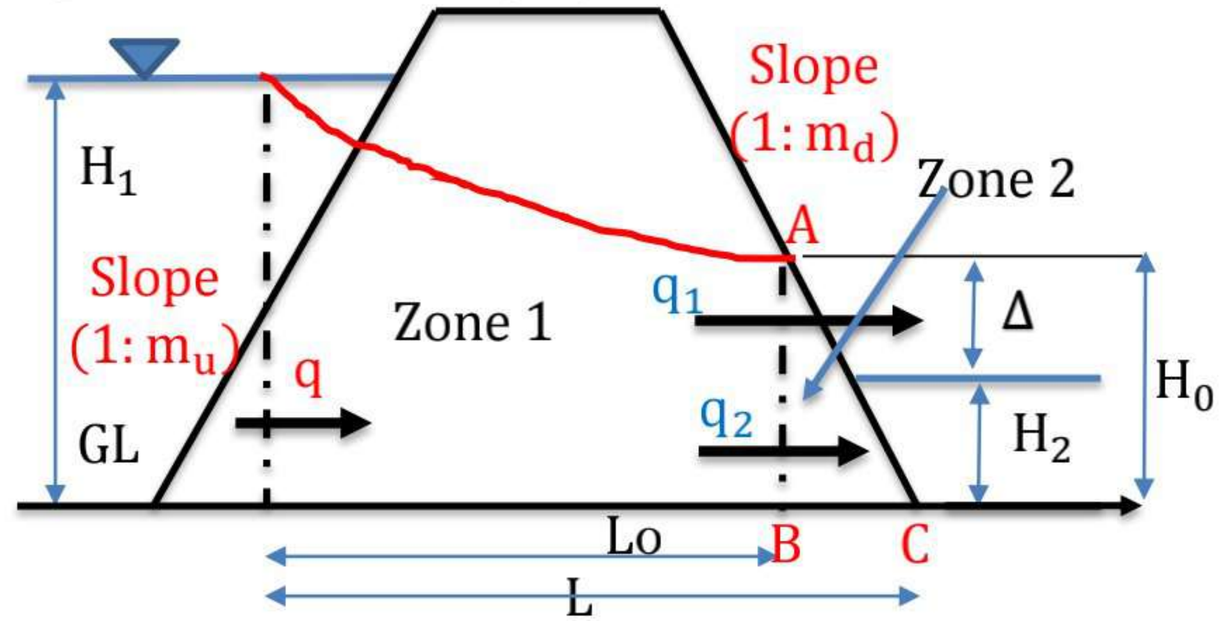
Case II: Phreatic line and seepage discharge with phreatic line emerging at the D/S face of the dam above the downstream water:



Specific seepage discharge through zone 1 =
Total specific seepage discharge through zone 2
The specific discharge through zone 1
$= \frac{K}{2L0}(H_1^2 - H_0^2)$
The specific discharge through zone 2 = $q_1 + q_2$

Now, to calculate q_1 and q_2 the zone 2 consist of two part one up to tail water level and on above tail water level but below phreatic line. The detail breakdown is shown in figure below.

Now q1 can be obtained by integrating of dq1 and q2 by integration of dq2.

$dq_1 = kiA = K\frac{y}{ym_d}(dy * 1) = \frac{K}{m_d}(dy)$

Integrating

$q_1 = \int_0^{\Delta} \frac{K}{m_d} dy = \frac{K\Delta}{m_d}$

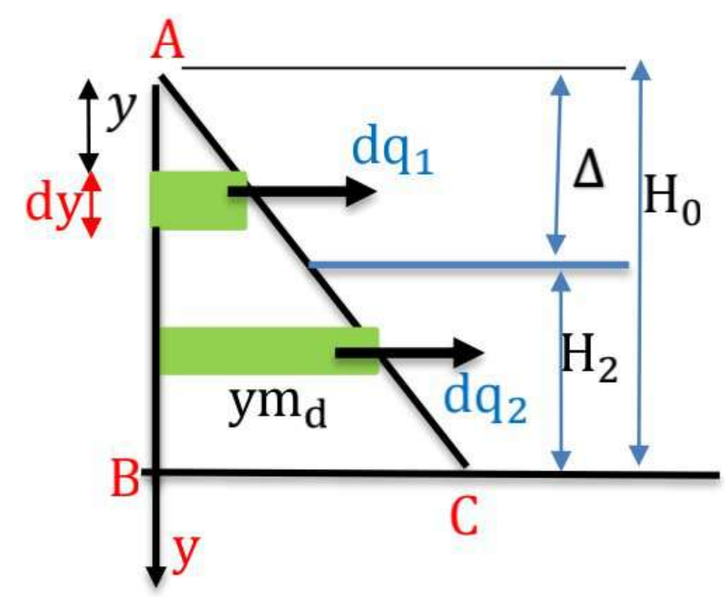
Also, $dq_2 = kiA = K\frac{\Delta}{ym_d}(dy * 1) = \frac{K\Delta}{ym_d}(dy)$

Integrating

$q_2 = \int_{\Delta}^{Ho} \frac{K\Delta}{ym_d} dy = \frac{K\Delta}{m_d}\int_{\Delta}^{Ho} \frac{1}{y} dy$

$= \frac{K\Delta}{m_d}[\ln y]_{\Delta}^{Ho} = \frac{K\Delta}{m_d}(\ln Ho - \ln \Delta)$

$q_2 = \frac{K\Delta}{m_d} \ln\left(\frac{Ho}{\Delta}\right)$



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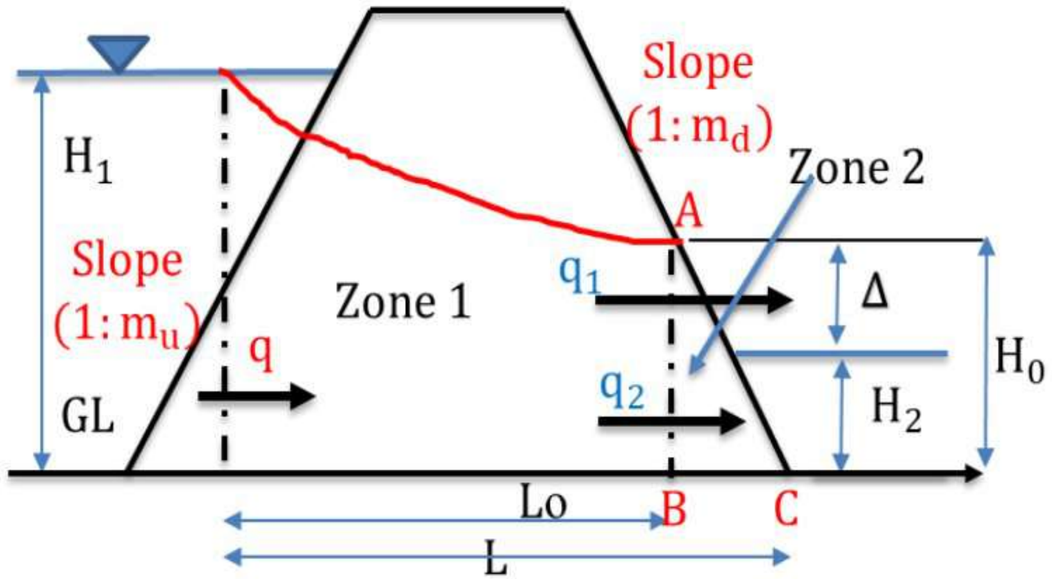
Phreatic line and seepage discharge with phreatic line emerging at the D/S face of the dam



Specific seepage discharge through zone 1 =
Total specific seepage discharge through zone 2
The specific discharge through zone 1

$$= \frac{K}{2L0}(H_1^2 - H_0^2)$$

The specific discharge through zone 2 = $q_1 + q_2$

$$= \frac{K\Delta}{m_d} + \frac{K\Delta}{m_d}\ln\left(\frac{Ho}{\Delta}\right)$$

We know $q = q_1 + q_2$

$$\frac{K}{2L0}(H_1^2 - H_0^2) = \frac{K\Delta}{m_d} + \frac{K\Delta}{m_d}\ln\left(\frac{Ho}{\Delta}\right)$$

$$\frac{K}{2L0}\left\{H_1^2 - (H_2 + \Delta)^2\right\} = \frac{K\Delta}{m_d}\left\{1 + \ln\left(\frac{H_2 + \Delta}{\Delta}\right)\right\}\ldots\ldots(1)$$

Hence from this equation find the value of Δ.

 # Phreatic line and seepage discharge with phreatic line emerging at the D/S face of the dam (No downstream water)

If there is no tail water, $H_2 = 0$

$$\frac{K}{2L0}\{H_1^2 - (\Delta)^2\} = \frac{K\Delta}{m_d}$$

$$\{H_1^2 - (\Delta)^2\} = \frac{2L0\Delta}{m_d}$$

$$\Delta^2 + \frac{2\Delta(L - \Delta m_d)}{m_d} - H_1^2 = 0$$

$$\Delta^2 + \frac{2\Delta L}{m_d} - 2\Delta^2 - H_1^2 = 0$$

$$\Delta^2 - \frac{2\Delta L}{m_d} + H_1^2 = 0$$

Solving quadratic equation,

$$\Delta = \frac{\frac{2L}{m_d} \pm \sqrt{\frac{4L^2}{m_d^2} - 4H_1^2}}{2}$$

$$\Delta = \frac{L}{m_d} \pm \sqrt{\frac{L^2}{m_d^2} - H_1^2}$$

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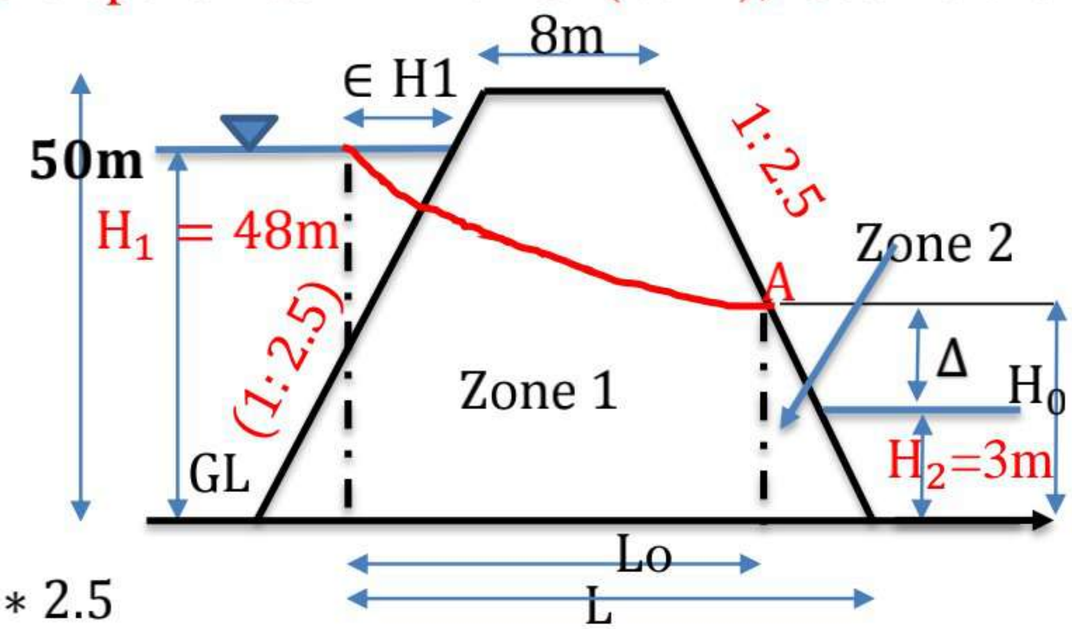
Q. N. Find the specific seepage discharge of the flow through the homogenous earthen embankment dam with the following data; Height of the dam = 50m, Upstream water level = 48m, Downstream water level = 3m, width of the dam at the top = 8m, U/S & D/S slope of dam = 1: 2.5 (V: H), Coefficient of permeability of the soil = 2.5 cm/s.



Solution:

- We know, $\epsilon = \frac{0.44}{1+\frac{1}{2m_u}} = \frac{0.44}{1+\frac{1}{2*2.5}} = 0.367$
- Considering seepage discharge for the dam with phreatic line emerging at the downstream face of the dam above the downstream water level.
- $\therefore L_o = \epsilon H_1 + (2.5 * 2) + 8 + (2.5 * 50) - (\Delta + 3) * 2.5$
- $= (0.367 * 48) + (2.5 * 2) + 8 + (2.5 * 50) - (\Delta * 2.5) - (3 * 2.5)$
- $L_o = 148.116 - 2.5\Delta$
- From continuity equation, Total specific seepage discharge from zone I =Total specific discharge through zone II
- Total specific seepage discharge from zone (I) $= K\left\{\frac{H_1^2 - H_o^2}{2L_o}\right\} = K\left\{\frac{H_1^2 - (H_2+\Delta)^2}{2L_o}\right\}$
- $= K\left\{\frac{H_1^2 - (H_2+\Delta)^2}{2(L - \Delta m_u)}\right\} \ldots\ldots\ldots (I)$
- Total specific discharge through zone II $= q_1 + q_2$
- $q_1 = \frac{K\Delta}{m_d}$

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- $q_2 = \frac{K\Delta}{m_d} * \ln\left(\frac{H_O}{\Delta}\right)$
- i.e. from continuity equation
- $q_1 + q_2 = K\left\{\frac{H_1^2-(H_2+\Delta)^2}{2(L-\Delta m_u)}\right\}$
- $\frac{K\Delta}{m_d}\left\{1 + \ln\left(\frac{H_O}{\Delta}\right)\right\} = K\left\{\frac{H_1^2-(H_2+\Delta)^2}{2L_o}\right\}$
- $\frac{\Delta}{2.5}\left\{1 + \ln\left(\frac{3+\Delta}{\Delta}\right)\right\} = \left\{\frac{48^2-(3+\Delta)^2}{2(148.116-2.5\Delta)}\right\}$
- $\left\{1 + \ln\left(\frac{3+\Delta}{\Delta}\right)\right\} = \left\{\frac{2304-9-6\Delta-\Delta^2}{2(148.116-2.5\Delta)} * \frac{2.5}{\Delta}\right\}$
- $\left\{\ln\left(\frac{3+\Delta}{\Delta}\right)\right\} = \left\{\frac{2868.75-7.5\Delta-1.25\Delta^2-(148.116\Delta-2.5\Delta^2)}{(148.116\Delta-2.5\Delta^2)}\right\}$
- $\left\{\ln\left(\frac{3+\Delta}{\Delta}\right)\right\} = \left\{\frac{2868.75-155.616\Delta+1.25\Delta^2}{(148.116\Delta-2.5\Delta^2)}\right\}$
- Calculation by trial and error method, $\Delta = 19.82\text{m}$
- $\therefore$ Specific discharge(q) $= \frac{K(H_1^2-(H_2+\Delta)^2}{2L_O}$
- $= \frac{\frac{2.5}{100}*(48^2-(3+19.82)^2}{2\{148.116-(2.5*19.82)\}}$
- $= \frac{0.226\text{m}^3}{\text{sec per meter}}$
- $= \mathbf{226 litre/sec\ per\ meter}$

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Seepage through an Earth Dam on an impervious Base

Let us consider the case of a homogeneous earth dam on an impervious foundation and having a horizontal filter at the downstream end as shown in the figure. The horizontal filter starts at point 'A' . The impermeable boundary AE is flows line which forms the lower boundary of flow net. The upstream face EF is an equivalent line. The discharge face AG is the equipotential line of zero potential. Thus, three hydraulic boundary conditions are known.

The fourth boundary of flow net is the top flow line FG which is also known as **phreatic line or seepage** line. *Seepage lie or phreatic line is a line in the body of dam below which there are positive hydrostatic pressures*. On the line **itself**, the hydrostatic pressure is zero.

Graphical procedure for obtaining phreatic line

- The graphical procedure for obtaining phreatic line consists of the following steps and is illustrated in the figure. Here phreatic line is assumed as base parabola.
- Let EF be the wetted portion of upstream face and EI be the projection of AB on the water line.
- Let EI = L, A point 'C' is marked on FI such that CF = 0.3 L. Then 'C' will be the first point of the base parabola.

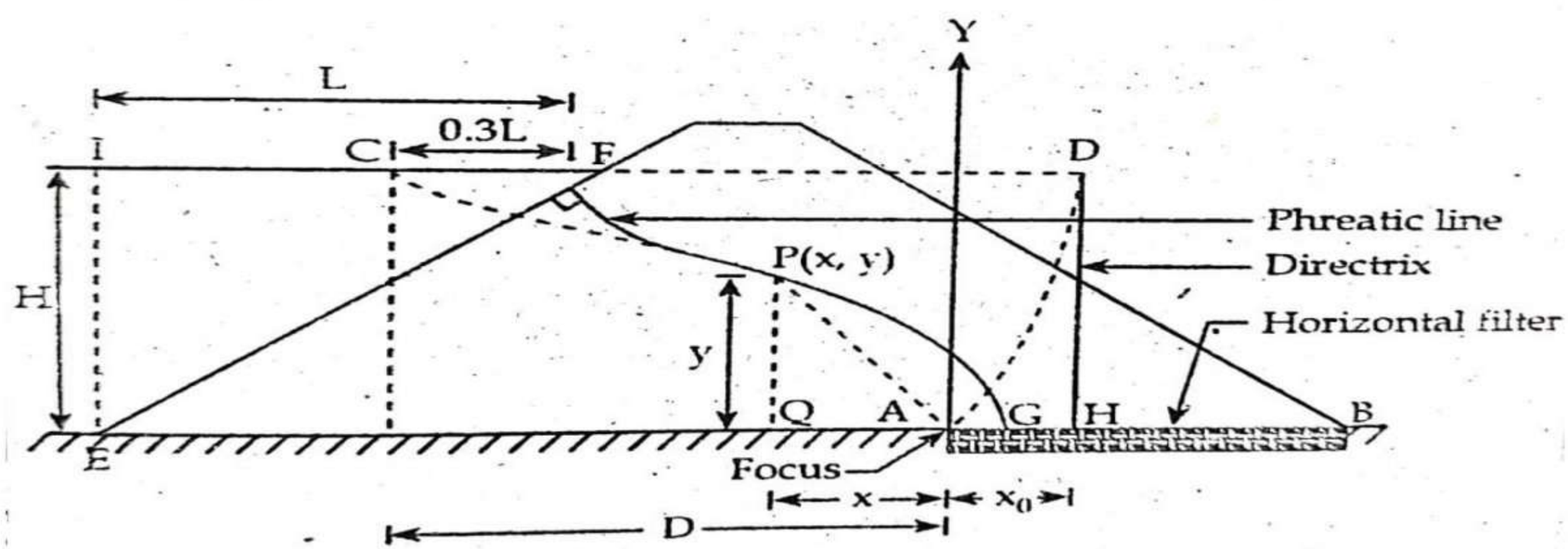

Figure. Graphical procedure for obtaining phreatic line

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Graphical procedure for obtaining phreatic line

- The beginning point 'A' of the filter will be the focus of the base parabola. With 'C' as center and CA as radius, an arc is drawn to cut the water line produced at 'D'. Through 'D', vertical line DH is drawn. DH will be the directrix of the base parabola.
- The midpoint 'G' of AH will be the last point of parabola in the body of dam.
- Intermediate points on the base parabola are obtained from the property that any point on the parabola is equidistant from the focus and the directrix.
- A smooth curve is drawn joining C, G and intermediate points representing base parabola. The phreatic line is obtained by correcting this base parabola to meet upstream face and filter, both at right angles at 'F' and 'G'.

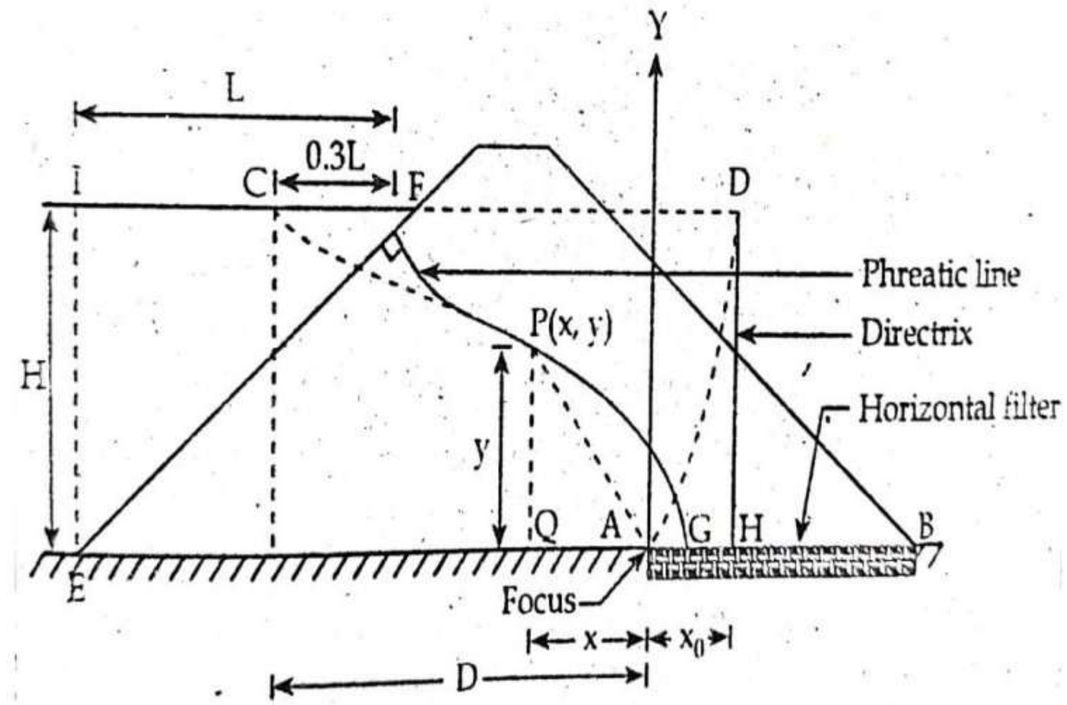

Figure. Graphical procedure for obtaining phreatic line

Equation of top flow line (Phreatic line) and Seepage discharge:

In order to develop an expression for top flow line consider any point 'p' having (x, y) as its coordinates, with respect to the focus point 'A' as the origin as shown in the figure above.

From property of base parabola, any point on the parabola is equidistant form the focus and directrix

AP = QH

But, AP = $\sqrt{X^2 + y^2}$ = QH

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Equation of top flow line (Phreatic line) and Seepage discharge:

If focal distance AH is taken as x_o then,

$QH = QA + AH = x + x_o$

Or, $\sqrt{X^2 + y^2} = x + x_o$

Squaring both sides, we get

$X^2 + y^2 = (x + x_o)^2$

Or, $X^2 + y^2 = X^2 + 2xx_o + x_o^2$

Or, $y^2 = 2xx_o + x_o^2$

This is the equation of base parabola or top flow line.

Also, $y = \sqrt{2xx_o + x_o^2}$

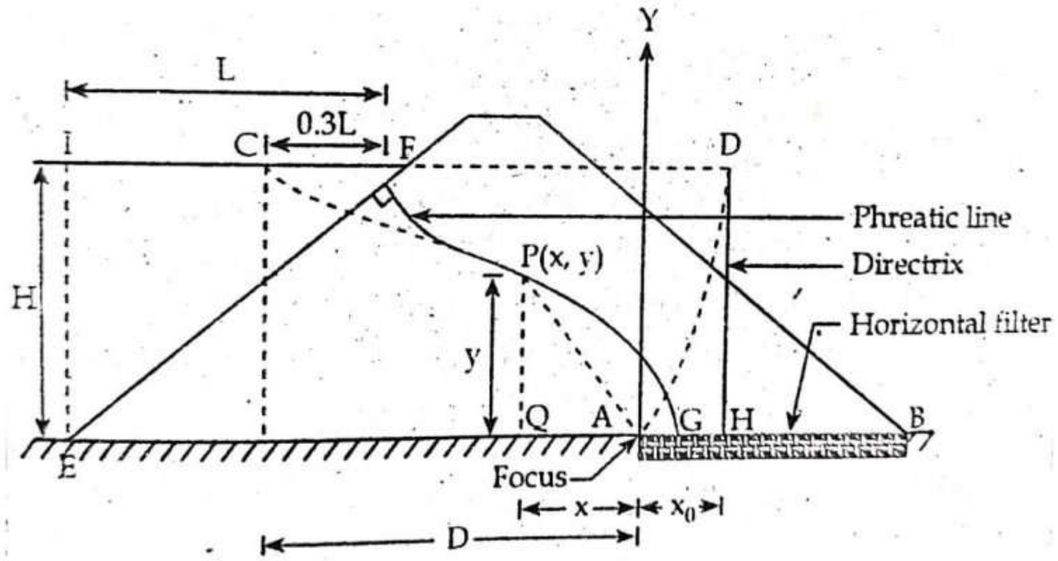

Figure. Graphical procedure for obtaining phreatic line

For discharge (q), seeping through the body of the dam

When horizontal filter is placed, we consider the section PQ.

Form Darcy's law, we have

$q = KiA = k * \frac{dy}{dx} * (y*1)$

$\frac{dy}{dx} = \frac{d}{dx}(\sqrt{2xx_o + x_o^2}) = \frac{1}{2} * \frac{2x_o}{\sqrt{2xx_o + x_o^2}} = \frac{x_o}{\sqrt{2xx_o + x_o^2}}$

Now, $q = k * \frac{x_o}{\sqrt{2xx_o + x_o^2}} * \sqrt{2xx_o + x_o^2}$

$\mathbf{q = K*x_o}$

The focal distance x_o can be determined by considering coordinate of point 'C' as follows,

$x_o = \sqrt{X^2 + y^2} - x$

At point 'C' x = D and y = H (depth of water)

$\therefore x_o = \sqrt{D^2 + H^2} - D$

$\therefore \mathbf{q = K*x_o = k*(\sqrt{D^2 + H^2} - D)}$

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If the discharge face is not horizontal, a further correction at the exit may have to be made to theoretical parabola, depending *upon the conditions at the toe*. The figure below shows the top flow line details at exit.

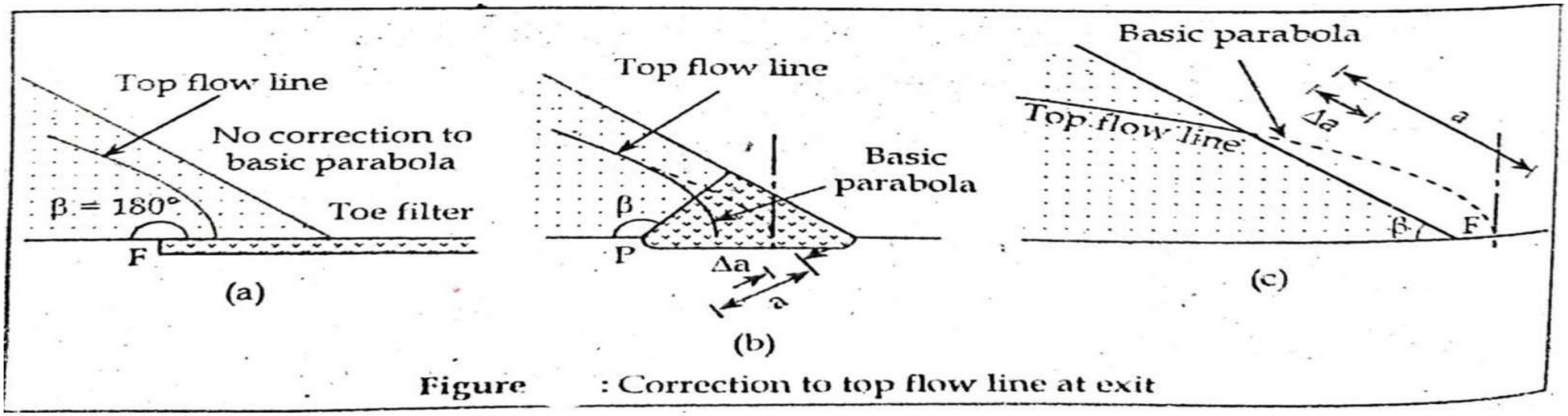

Figure : Correction to top flow line at exit

When the exit surface is horizontal i.e. β = 180°, no correction is needed to the base parabola.

β	30°	60°	90°	120°	150°	180°
$\frac{\Delta a}{a}$	0.36	0.32	0.26	0.18	0.1	0

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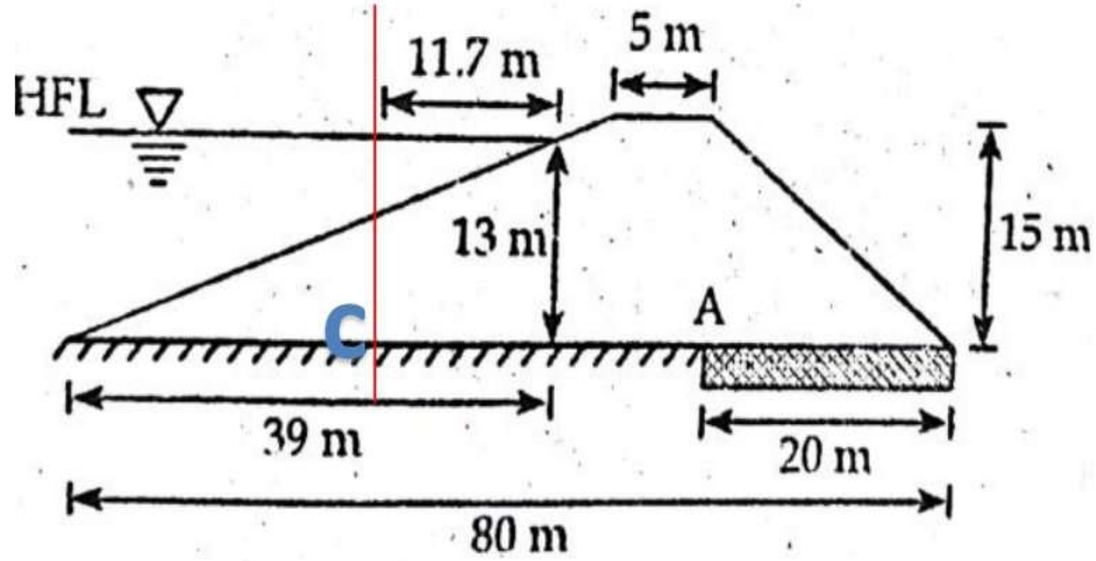
An earth dam of homogeneous section having horizontal filter is shown in the figure, the coefficient of permeability of the soil used in the dam is $5*10^{-3}$ cm/sec. Determine the equation of basic parabola and estimate the quantity of seepage.



Solution

Considering starting point of filter as origin, the equation of parabola is given by,

$x_o = \sqrt{X^2 + y^2} - x$

Where x_o = distance of direction from the origin and (x, y) are the coordinates of any point on the parabola.

The parabola cuts the reservoir water surface at point 'C' such that.

AC = 80 – 20 – 39 + 11.7 = 32.7 m

Depth at entry point = 13 m

When X = 32.7 m , y = 13 m

$x_o = \sqrt{(32.7)^2 + (13)^2} - 32.7 = 2.49$ m

$y = \sqrt{2xx_o + x_o^2} = \sqrt{(2 * x * 2.49) + (2.49)^2}$

$\mathbf{y = \sqrt{4.98x + 6.2}}$

This is the equation of basic parabola to construct the flow net,

X (m)	0	10	20	30	32.7
Y (m)	2.49	7.483	10.286	12.474	13

Seepage flow per unit length of dam

$\mathbf{q = kx_o} = 5*10^{-3}*2.49 * 100 =$ **1.245 cm³/sec / m**

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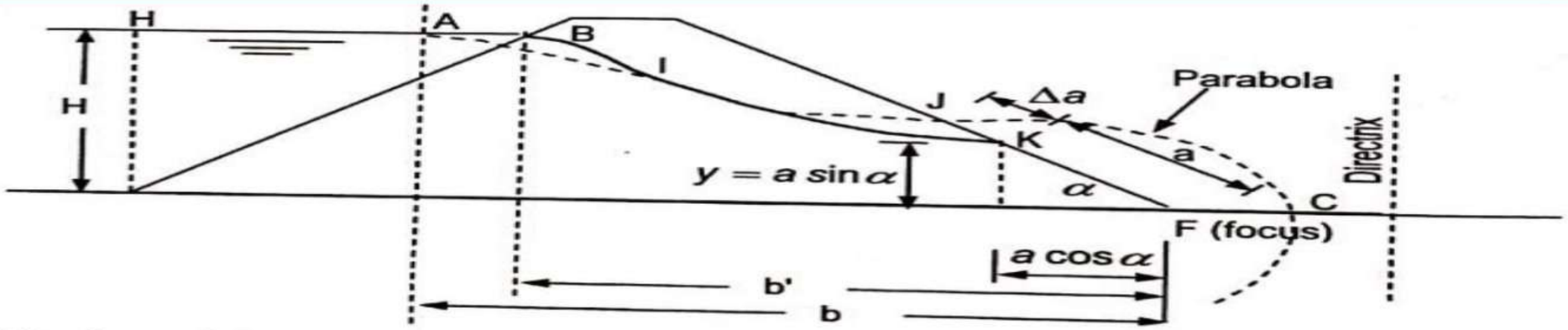
Phreatic line for homogeneous dam without horizontal filter:



The focus (F) of the parabola will be the lowest point of the downstream slope as shown in figure. The base parabola BIJC will cut the downstream slope at I and extend beyond the dam toe up to the point C (i.e. vertex of parabola)

Case (a) When $\alpha < 30$

The equation is derived on the assumption that the hydraulic gradient is equal to the slope of phreatic line. This assumption is nearly true so long as the downstream slope is sufficiently flat () $\alpha < 30°$

(Schaffenak & Von Iterson)

$$\therefore q = K.i.A = K\left(\frac{dy}{dx}\right)(y.1)$$

$$\therefore q = K.\tan\alpha .a.\sin\alpha \qquad \frac{dy}{dx} = \tan\alpha, y = a\sin\alpha$$

$$\therefore k\frac{dy}{dx}.y = K\,a\sin\alpha\ \tan\alpha$$

$$or,\ \frac{dy}{dx}.y = a\sin\alpha\ \tan\alpha$$

$$or, dy.y = a\sin\alpha\ \tan\alpha\ dx$$

Integrating both sides between limits

$x = a\cos\alpha$ to $x = b'$

$y = a\sin\alpha$ to $y = H$

$$\int_{a\sin\alpha}^{y=H} y\,dy = a\sin\alpha\ \tan\alpha \int_{a\cos\alpha}^{x=b'} dx$$

$$\frac{H^2 - a^2\sin^2\alpha}{2} = a\sin\alpha\ \tan\alpha\,(b' - a\cos\alpha)$$

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Phreatic line for homogeneous dam without horizontal filter:

$$\frac{H^2}{2}-\frac{a^2}{2}\sin\alpha = ab'\sin\alpha\tan\alpha - a^2\sin\alpha\,.\cos\alpha.\,\tan\alpha$$

$$\text{Or, } \frac{H^2}{2}-\frac{a^2}{2}\sin^2\alpha = ab'\sin\alpha\tan\alpha - a^2\sin^2\alpha$$

$$\text{Or, } \frac{H^2}{2} = ab'.\sin\alpha\,.\tan\alpha - \frac{a^2}{2}\sin^2\alpha$$

$$\text{Or, } \frac{a^2}{2}.\sin^2\alpha - ab'\sin\alpha\,\tan\alpha + \frac{H^2}{2} = 0$$

$$\text{Or, } a^2\sin^2\alpha - 2ab'\sin\alpha\,\tan\alpha + H^2 = 0$$

$$\text{Or, } a^2 - \frac{2ab'}{\cos\alpha} + \frac{H^2}{Sin^2\alpha} = 0$$

$$a = \frac{\frac{2b'}{\cos\alpha} \pm \sqrt{\left(\frac{2b'}{\cos\propto}\right)^2 - \frac{4H^2}{\sin^2\alpha}}}{2}$$

Taking negative sign

$$a = \frac{b'}{\cos\alpha} - \sqrt{\frac{b'^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}}$$

Case b: when α lies between 30° & 60°.

Cassegrade has suggested the use of $\sin\alpha$ in place of $\tan\alpha$

$\therefore$ The hydraulic gradient (i) is given by $\frac{dy}{ds}$ (*i.e* $\sin\alpha$) of the seepage line

$\therefore q = K.i.A.$

$= K.\frac{dy}{ds}.y.1 = K\sin\alpha\ \ a\ \ \sin\alpha$

$\therefore K\ \frac{dy}{ds}.y = K\,a\sin^2\alpha$

Integrating between two limits

$y = a\sin\alpha$ to H

$S = a$ to S_o

Where, S_0 is total length of parabola from point A to F

$$\int_{a\sin\alpha}^{y=H} y\,dy = a\sin^2\alpha \int_{a}^{x=s_0} dx$$

$$\therefore \frac{H^2 - a^2\sin^2\alpha}{2} = a\sin^2\alpha\ (S_0 - a)$$

$$\text{Or, } \therefore \frac{H^2}{2} - \frac{a^2\sin^2\alpha}{2} = a\sin^2\alpha\ .S_0 - a^2\sin^2\alpha$$

$$a^2 - 2S_0\ \ a + \frac{H^2}{\sin^2\alpha} = 0$$

$$\therefore a = \frac{2S_0 \pm \sqrt{4S_0^{\,2} - 4\frac{H^2}{\sin^2\alpha}}}{2}$$

Ignoring -ve sign

$$\therefore a = S_0 - \sqrt{S_0^{\,2} - \frac{H^2}{\sin^2\alpha}}$$

The total length of parabola S_0 can be approximately taken to be equal to $\sqrt{b^2+H^2}$

$$\therefore a = \sqrt{b^2+H^2} - \sqrt{b^2+H^2-\frac{H^2}{\sin^2\alpha}}$$

$$\therefore a = \sqrt{b^2+H^2} - \sqrt{b^2 - H^2\cot^2\alpha}$$

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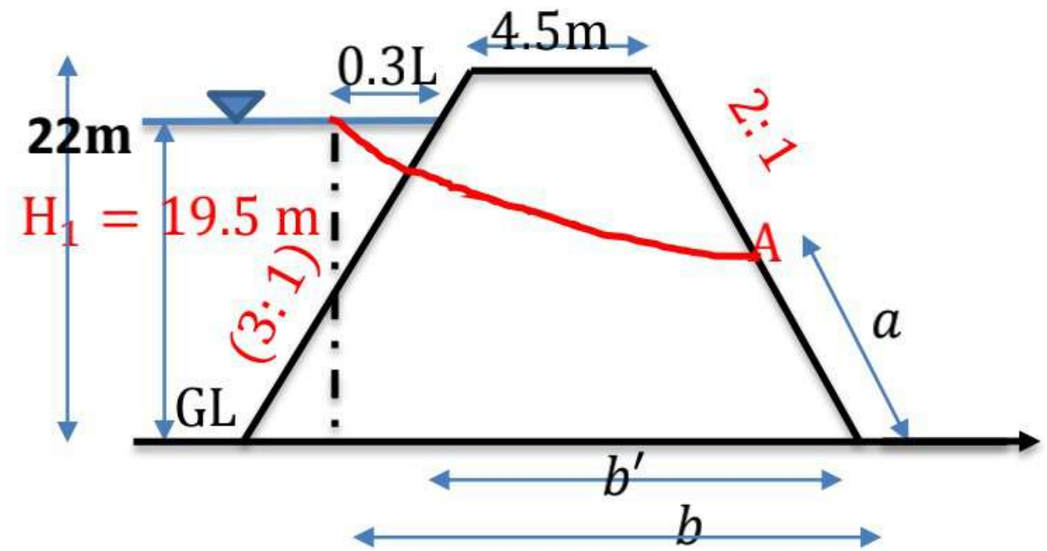
6) A earth dam made of homogeneous material has the following data; coefficient of permeability of dam material = $2*10^{-4}$ cm/ sec, level of top of dam = 200, level of deepest river bed = 178m, HFL of reservoir = 197.5m, width of top of dam = 4.5m, upstream slope = 3:1, downstream slope = 2. Determine the phreatic line of this dam section and discharge passing through the dam.

Solution



Total height of dam = 200 – 178 = 22m

Horizontal length of upstream sloped portion = 66m

EB = L = 3*19.5 = 58.5m

Horizontal distance from point B to F =

b' = (22*2) + 4.5 + (2.5*3) = 56 m

Distance AB = 0.3*BE = 0.3*(3*19.5) = 17.5

Horizontal distance from A to F = b' + 0.3L

= 56 + 17.5 = 73.5 m

Taking the focus F at eh downstream toe as the origin, the equation of the base parabola is given by $\sqrt{X^2 + y^2} = x + x_o$ where = distance of the point (x, y) from the directrix called focal distance.

Select point A in such that AB = 0.3 EB = 0.3*58.5 = 17.5m

When X = 73.5 m , y =19.5 m

$x_o = \sqrt{(73.5)^2 + (19.5)^2} - 73.5 = 2.54$ m

$y = \sqrt{2xx_o + x_o{}^2} = \sqrt{(2 * x * 2.54) + (2.54)^2}$

$\mathbf{y = \sqrt{5.08x + 6.451}}$

X (m)	0	10	20	30	40	50	60	70	73.5
Y (m)	2.54	7.57	10.39	12.60	14.48	16.14	17.64	19.03	19.5

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Solution…

Here $\alpha = \tan^{-1}\left(\frac{22}{44}\right) = 26.56 < 30$

H = 22m and b' = 56 m

Now,

$$a = \frac{b\prime}{\text{Cos}\,\alpha} - \sqrt{\frac{b\prime^2}{Cos^2\alpha} - \frac{H^2}{Sin^2\alpha}} = \frac{56}{\text{Cos}\,26.56} - \sqrt{\frac{(56)^2}{Cos^2(26.56)} - \frac{(22)^2}{Sin^2(26.56)}} = 23.89$$

Now this parabola has to be corrected at entry and exit as expanded. At entry the phreatic line is started from the point B in such was that it becomes right angle to upstream face. At exit knowing a, the point K is plotted and phreatic line BIK is completed.

Hence the seepage discharge through dam can be computed by

$\mathbf{q = kx_o} = 5*10^{-6}*2.54$

$= \mathbf{12.7*}\ 10^{-6}\mathbf{m^3/sec\ /\ m}$

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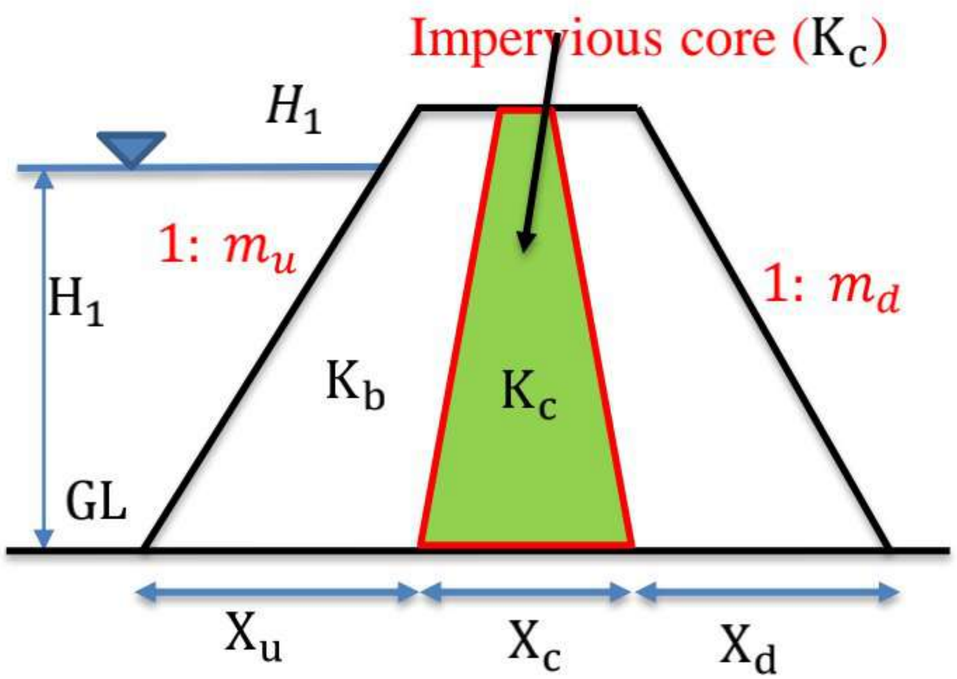
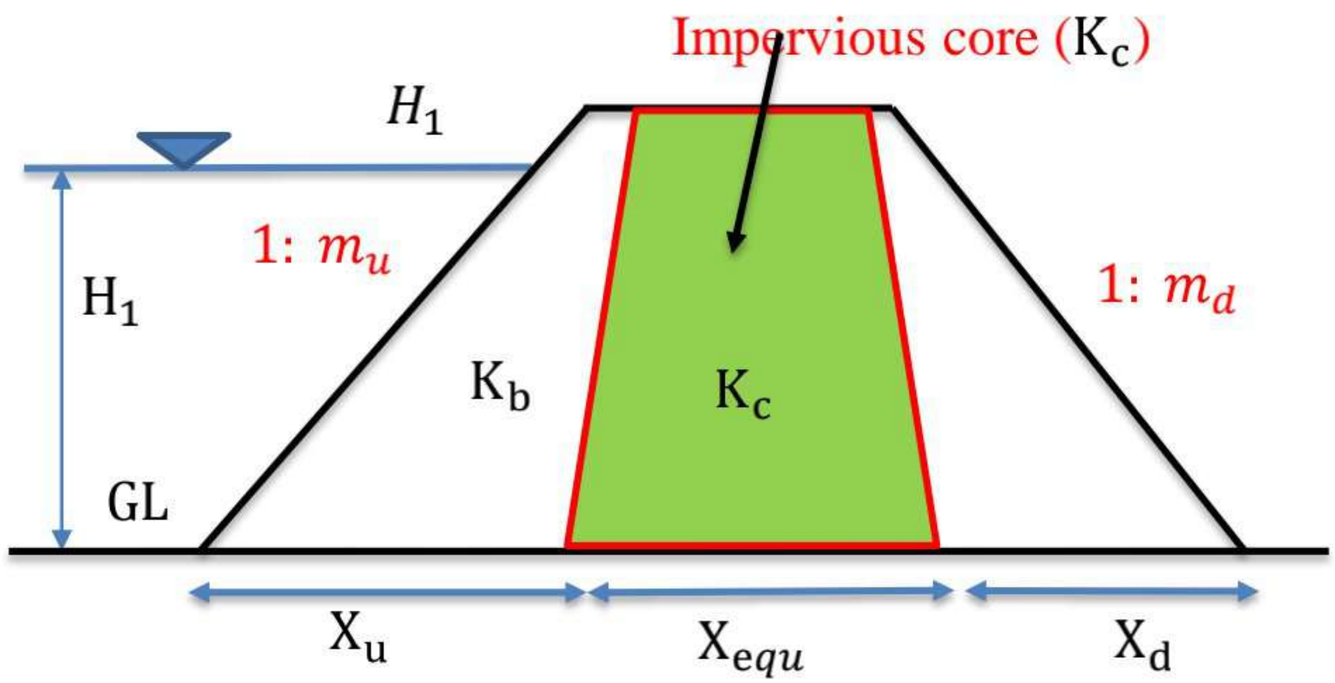
Seepage through dam with impervious core:

- Let X_c be the thickness of impervious core placed inside the dam body.
- Let X_u and X_d be the dimension of pervious dam material at upstream and downstream side.
- $1: m_u$ and $1: m_d$ be the upstream and downstream side slope of the dam face.
- For this convert the thickness of impervious core to the equivalent width of same pervious dam material using the relation

$$X_{equivalent} = X_c * \frac{K_b}{K_c}$$

- Now draw the transformed dam section with Homogeneous dam material of permeability K_c and analysis Should be done as per analysis of the pervious section.





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Q. N. Find the specific seepage discharge through homogeneous earthen embankment dam with 2m thick central impervious core. Take Height of the dam = 50m, U/S water level = 48m, D/S water level = 3m, width of the dam at the top = 10m, U/S & D/S slope of the dam = 1:3 (V: H), coefficient of permeability of the soil = 3cm/hr, coefficient of permeability of impervious core = 1 x 10^{-8} m/s.

Solution:

Coefficient of permeability of the soil (K_{soil}) = 3cm/hr $= \frac{3}{100*60*60} = 8.33 * 10^{-6}$m/sec

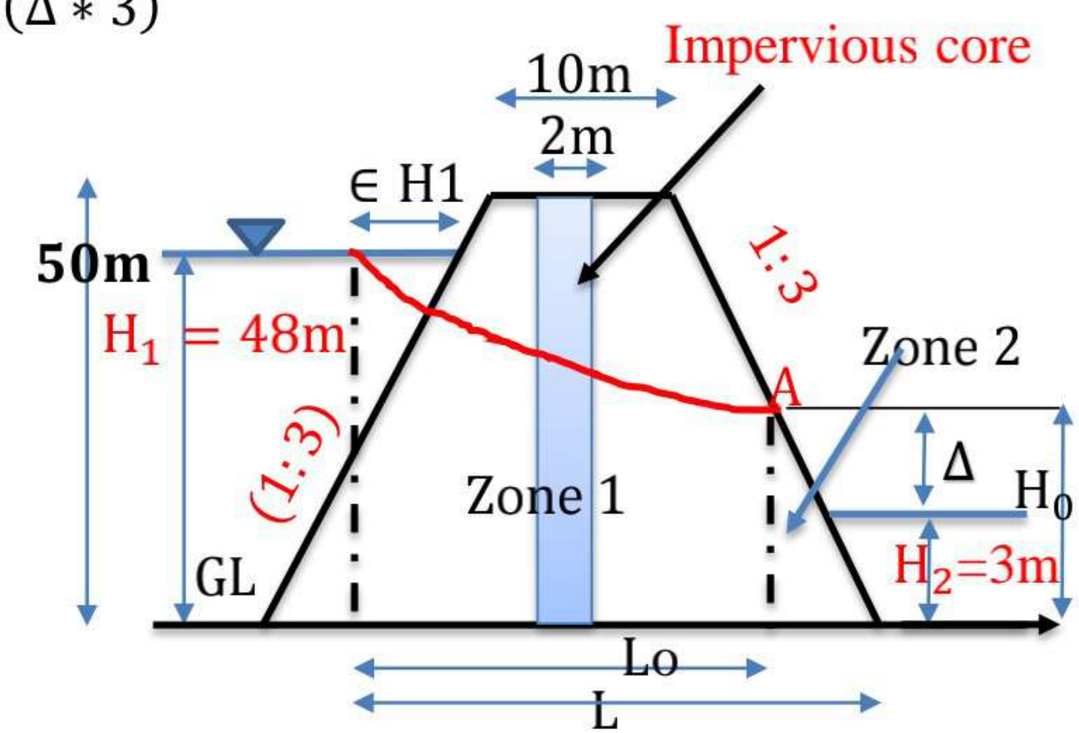
$\therefore \delta_{equivalent} = \delta_c \left(\frac{K_{Soil}}{K_{Core}}\right) = 2 * \left(\frac{8.33*10^{-6}}{1*10^{-8}}\right) = 1666$m

$\epsilon = \frac{0.44}{1+\frac{1}{2m_u}} = \frac{0.44}{1+\frac{1}{2*3}} = 0.377$

- $\therefore L_o = \epsilon H_1 + 3 * (50 - 48) + (10 - 2) + \delta_{equivalent} + (50 * 3) - (\Delta + 3) * 3$
- $= (0.377 * 48) + (3 * 2) + 8 + (1666) + 150 - 9 - (\Delta * 3)$
- $= 1839.1 - 3\Delta$
- Total specific seepage discharge from zone I

=Total specific discharge through zone II

- $K\left\{\frac{H_1^2-(H_2+\Delta)^2}{2L_o}\right\} = \frac{K\Delta}{m_d}\left\{1 + \ln\left(\frac{H_O}{\Delta}\right)\right\}$
- $\left\{\frac{48^2-(3+\Delta)^2}{2(1839.1-3\Delta)}\right\} = \frac{\Delta}{3}\left\{1 + \ln\left(\frac{3+\Delta}{\Delta}\right)\right\}$
- Solving by trial and error method, $\Delta = \mathbf{0.703m}$
- $\therefore$ Specific discharge (q) $= \frac{K(H_1^2-(H_2+\Delta)^2}{2L_O}$
- $= \frac{8.33*10^{-6}*(48^2-(3+0.703)^2}{2\{1839.1-(3*0.703)\}} = 5.19 * 10^{-6} m^3/\text{sec per meter} = 5.19 * 10^{-3}$litre/ sec per m



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Seepage through dam with pervious Foundation:

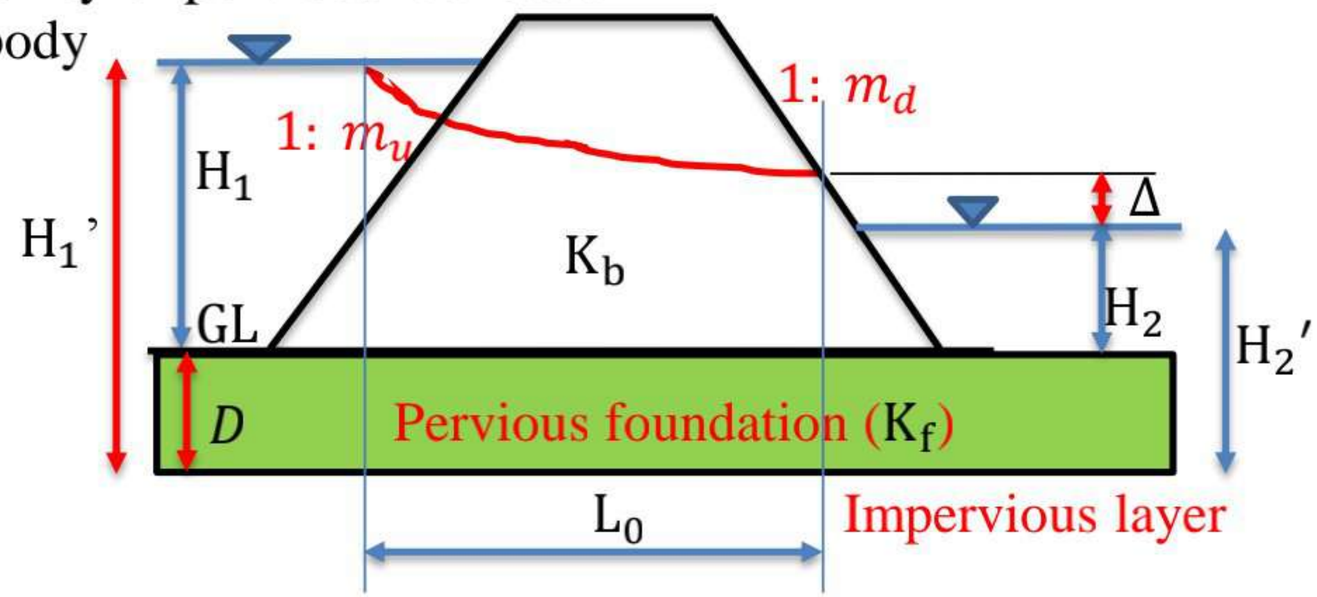
- Let $K_{foundation}$ be the coefficient of permeability of pervious foundation
- K_{body} be the coefficient of permeability of body

Case I : If $K_{foundation} = K_{body}$

$H_1' = H_1 + D$

$H_2' = H_2 + D$

Where D = Depth of foundation



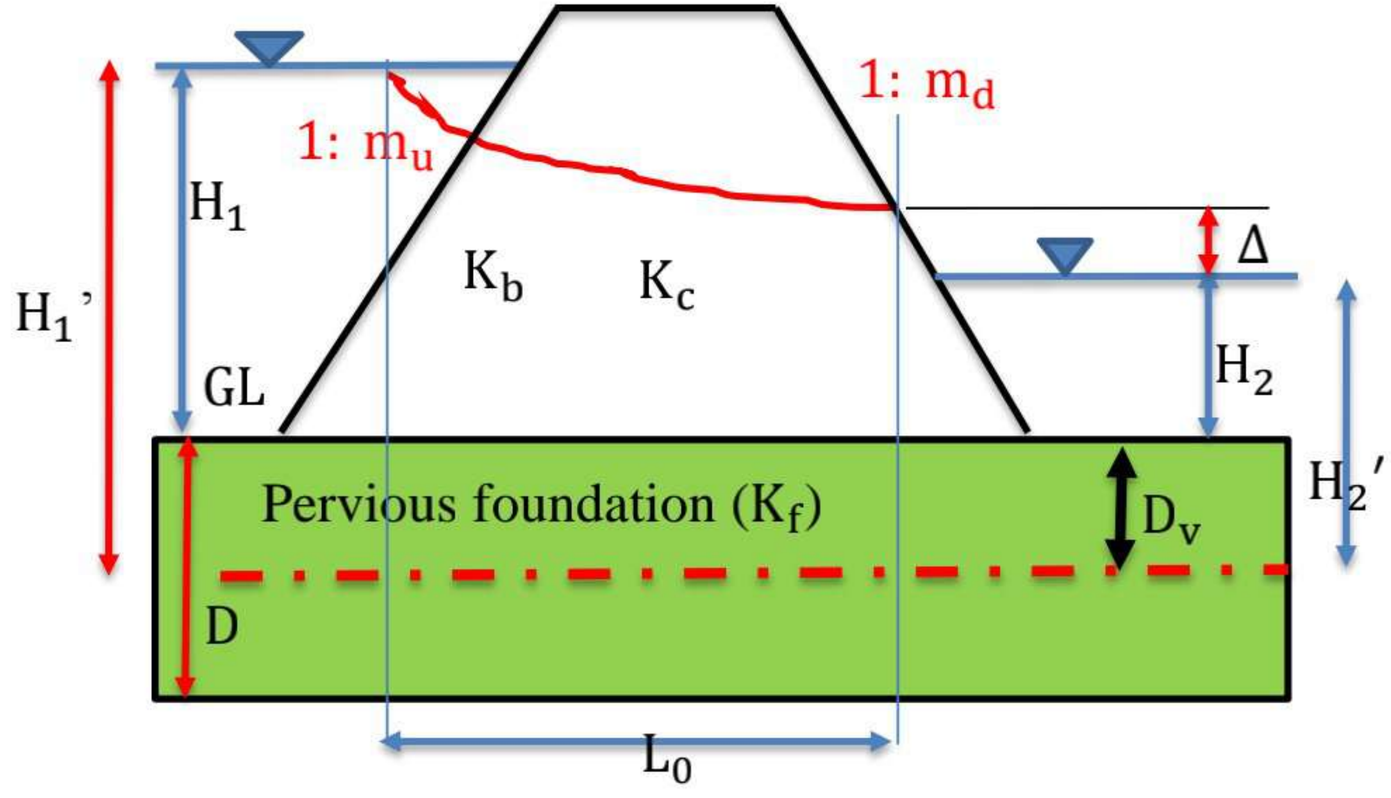
Case II : If $K_{body} > K_{foundation}$

Virtual Depth $(D_v) = D * \frac{K_{foundation}}{K_{body}}$

$H_1' = H_1 + D_v$

$H_2' = H_2 + D_v$

Where D = Depth of foundation



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Seepage through dam with pervious Foundation:

- Let $K_{foundation}$ be the coefficient of permeability of pervious foundation
- K_{body} be the coefficient of permeability of body

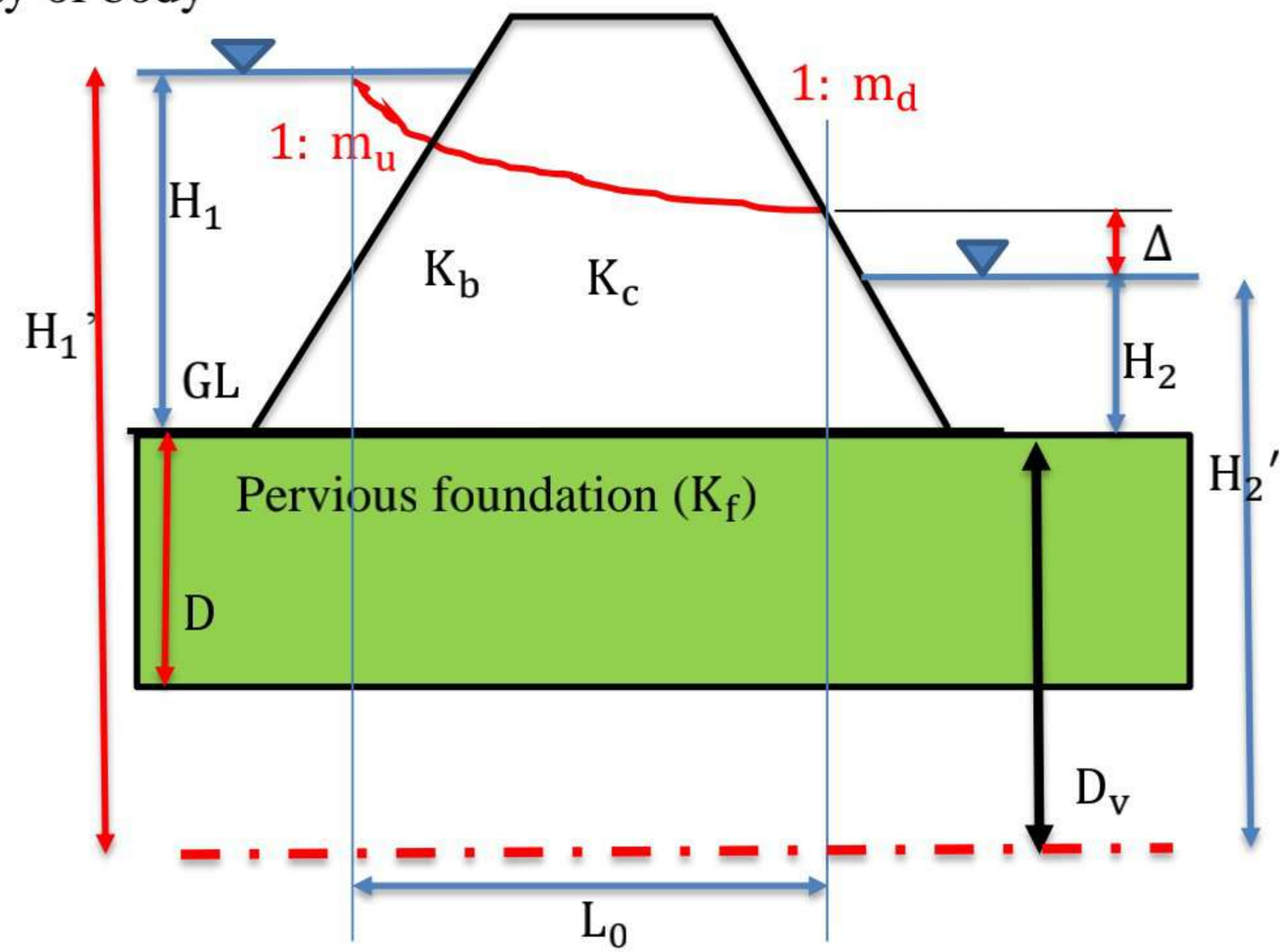
Case III : If $K_{foundation} > K_{body}$

Virtual Depth $(D_v) = D * \frac{K_{foundation}}{K_{body}}$

$H_1' = H_1 + D_v$

$H_2' = H_2 + D_v$

Where D = Depth of foundation



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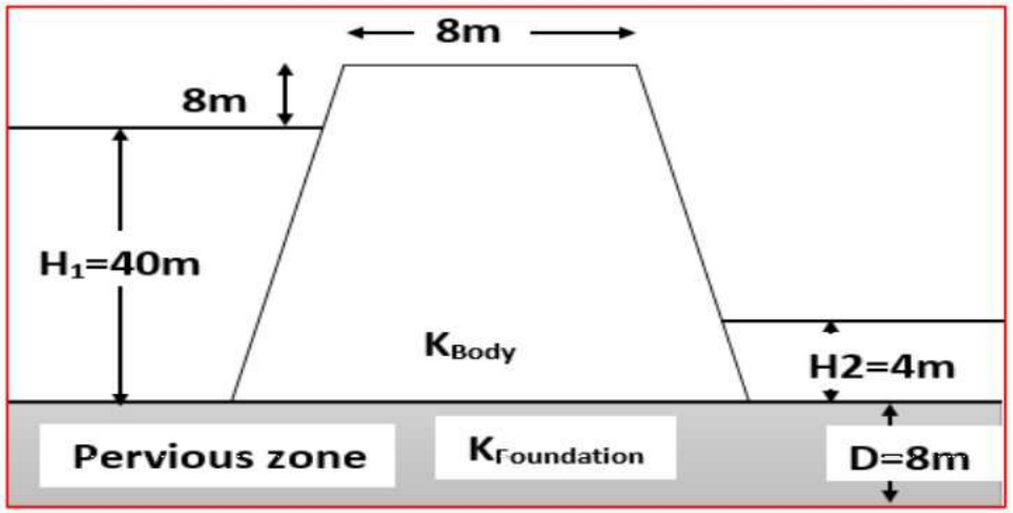
Q. N. Find the specific seepage discharge through homogeneous earthen embankment dam below which 8m depth pervious foundation is provided. Take Height of the dam = 48m, U/S water level = 40m, D/S water level = 4m, width of the dam at the top = 8m, U/S & D/S slope of the dam = 1:2.5 (V: H), coefficient of permeability of the soil = 10^{-4} cm/sec, coefficient of permeability of pervious foundation = $1x10^{-8}$ m/s.

Solution:

- Height of the dam (H) = 48m, U/S water level (H_1) = 40m, D/S water level (H_2) = 4m, width of the dam at the top = 8m, U/S & D/S slope of dam = 1: 2.5(V: H), permeability coefficient of the body material (K_{body}) =10^{-4} cm/s = 10^{-6} m/s, permeability coefficient of foundation ($K_{foundation}$)= 10^{-8}m/s
- We know, $\epsilon = \frac{0.44}{1+\frac{1}{2m_u}} = \frac{0.44}{1+\frac{1}{2*2.5}} = 0.367$



- Here $(K_{body}) > (K_{foundation})$
- i. e. $D_{Virtual} = D\left(\frac{K_{foundation}}{K_{body}}\right)$
- $D_V = 8 * \left(\frac{10^{-8}}{10^{-6}}\right) = 0.08m$
- $\therefore$ U/S and D/S depth should be changed,
- where $H_1' = 40 + 0.08 = 40.08m$
- $H_2' = 4 + 0.08 = 4.08m$
- Considering seepage discharge for the dam with phreatic line emerging at the downstream face of the dam above the downstream water level.
- $\therefore L_o = \epsilon H_1' + (8 * 2.5) + 8 + (2.5 * 48.08) - (\Delta + 4.08) * 2.5$
- $= (0.367 * 40.08) + 20 + 8 + 120.2 - 10.2 - 2.5\Delta = 152.709 - 2.5\Delta$
- $L_o = 152.709 - 2.5\Delta$

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i.e. from continuity equation

- $\frac{K_b \Delta}{m_d}\left\{1 + \ln\left(\frac{H'_0}{\Delta}\right)\right\} = K_b\left\{\frac{H_1'^2 - (H'_2 + \Delta)^2}{2L_o}\right\}$
- $\frac{\Delta}{2.5}\left\{1 + \ln\left(\frac{4.08+\Delta}{\Delta}\right)\right\} = \left\{\frac{40.08^2 - (4.08+\Delta)^2}{2(152.907 - 2.5\Delta)}\right\}$
- Calculation by trial and error method
- $\underline{\Delta = \mathbf{10.34m}}$
- $\therefore$ Specific discharge$(q) = K_b\left\{\frac{H_1'^2 - (H'_2 + \Delta)^2}{2L_o}\right\}$
- $= \frac{10^{-6} * \{(40.08)^2 - (4.08 + 10.34)^2\}}{2\{152.709 - (2.5 * 10.34)\}}$
- $= 5.511 * \frac{10^{-6} m^3}{\text{sec per m}}$
- $= 5.511 * 10^{-3}$litre/ sec per m

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Prevention of Erosion – Protection Filters

- In hydraulics structure, built on pervious material, sometimes failure occurs by the formation of pipe shaped channels beneath its foundation.
- The failure occurs when water flowing through the foundation has a very high hydraulic gradient.
- The prevailing hydraulic gradient increases the soil permeability and as a result the fines particles are washed away with the formation of pipe shaped tunnel starting from the downstream side of the structure.
- The possibility of piping occurring during seepage is more in regions when the flow is concentrated and seepage lines emerge vertically upward.
- If the seepage forces are large enough to erode the soil particles, piping can initiate and may progressively lead to a structural collapse. Soils most susceptible to piping by erosion are sands and silts.
- The failure due to piping can be of two types

i) Backward erosion piping

ii) Heave piping

Backward Erosion Piping

In this type of piping, fines in the soil mass are washed away near the discharge face of the structure at the downstream side and a scour hole is formed. This results in the increase in exit gradient, increase in permeability and increase in seepage velocity.

As a result progressive washing away of fines takes place towards the reservoir. The scour hole formed initially is extended towards the reservoir at the upstream side forming an underground pipe – shaped channel. When the channel reaches the reservoir at the upstream side of the structure, failure takes place. This is illustrated in figure. This phenomena causes huge damage to infrastructure developed towards the downstream side resulting loss of investment, loss of property and even loss of lives.

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Backward Erosion Piping

- The phenomenon of backward erosion of soil takes place by seeping water through the soil mass.
- The piping begins when the prevailing hydraulic gradient known as exit gradient exceeds the critical hydraulic gradient ($i_{exit} > i_{critical}$).

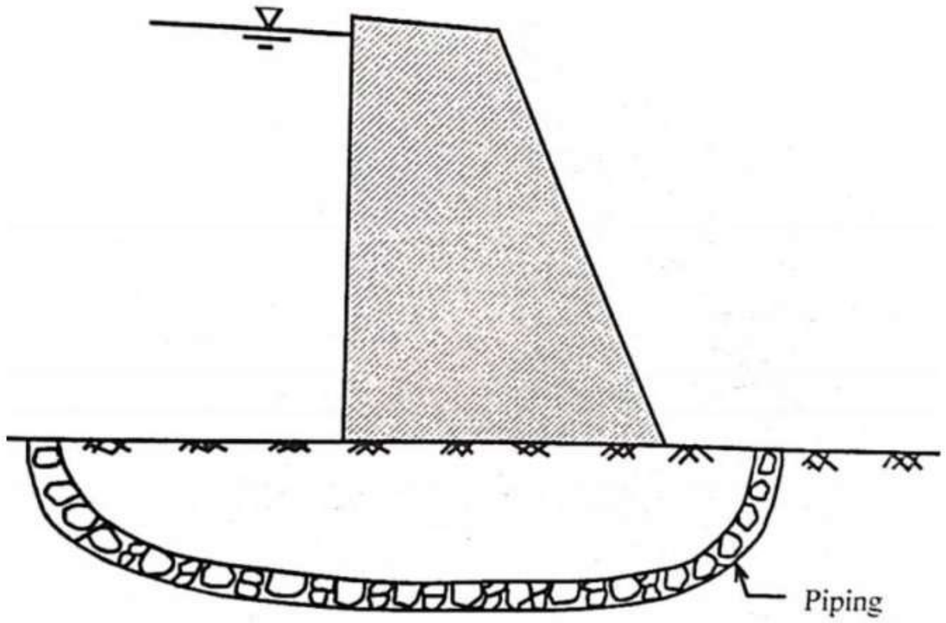
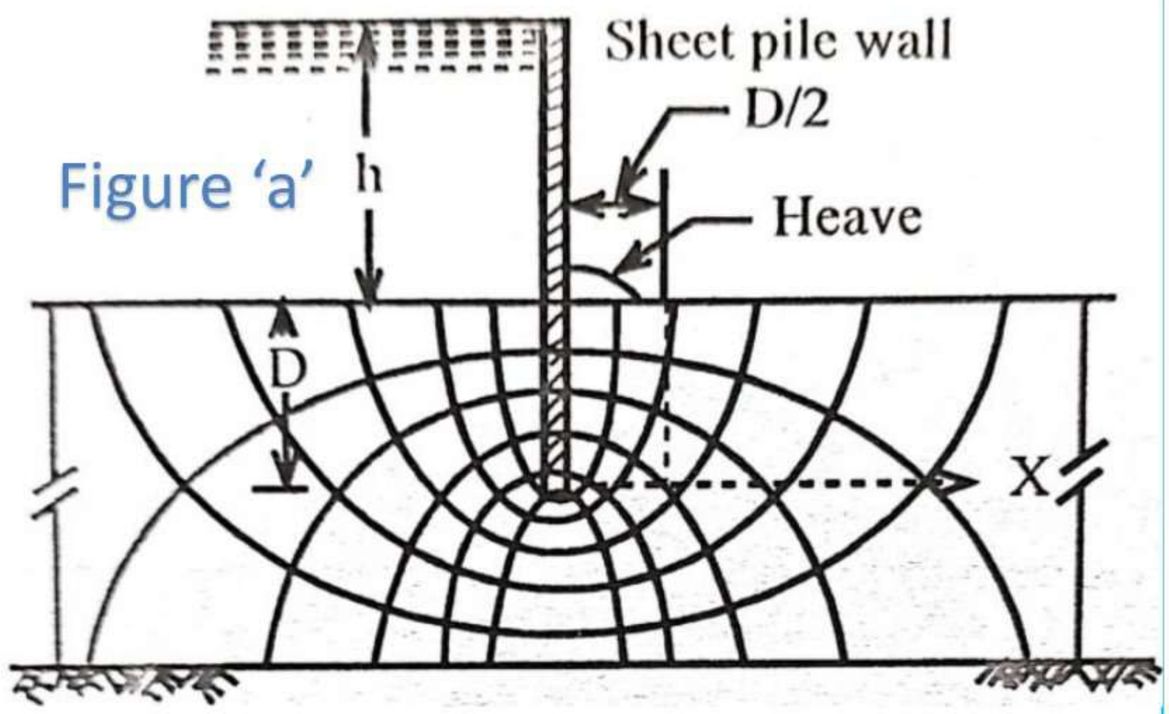


Fig *Backward erosion piping*

Heave Piping

- In heave piping, a large mass of soil at the exit point (downstream side) of a hydraulic structure heaves up due to seepage pressure.
- Piping failure cause by heave takes place when uplift forces due to seepage exceed the submerged unit weight of the soil.
- The mechanics of failure due to seepage was first developed by Terzaghi (1992).
- The principle of this method may be explained with respect to seepage flow below a sheet pile cutoff wall as shown in figure 'a'.



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Heave Piping

- The uplift pressure is obtained as per the procedure explained in application of flow Net.
- The ordinate of curve C in figure 'b' represent the uplift pressure at any point on the line ox.
- It is seen in the figure that the uplift pressure is greatest near the cutoff wall and gradually decreases as the distance form the wall increases.
- When the upward force on the portion of the line 'ox' near the cut off wall becomes equal to the downward force exerted by submerged soil, the surface of the soil rises as shown in figure 'b'.
- Once the soil is **heaved up**, volume of soil increases and consequently the permeability of the soil also increases.
- As a result, flow of water gets accelerated, soil appears to boil and finally failure takes place.
- This type of failure is known as Heave Piping.

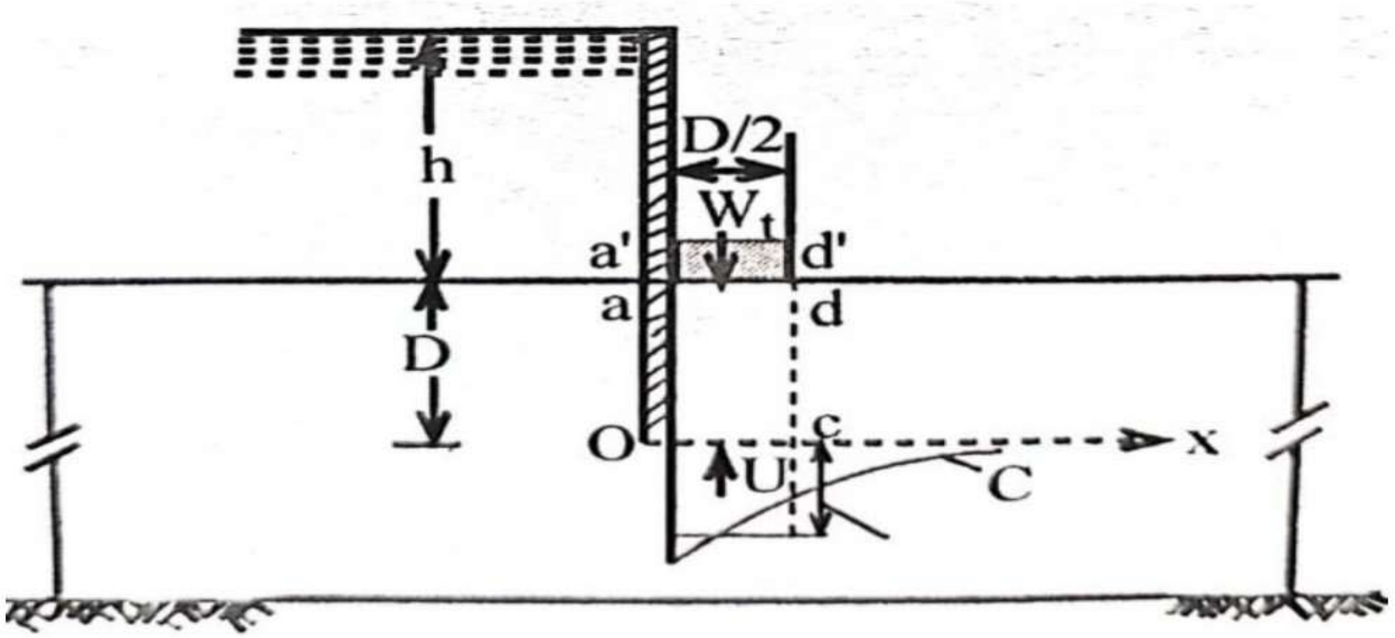

Figure b: *Heave piping phenomena*

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Factor of Safety Against Heave

- The modal tests conducted by Terzaghi have confirmed that the heave piping occurs within a distance about D/2 form the wall and the critical section 'ox' passes through the base of wall. D is the depth of penetration of the sheet pile cutoff wall.
- In figure 'b' a prism of soil **aocd** having depth 'D' and width D/2 is shown. This prism of soil is subjected to uplift. The average uplift pressure of water on this prism is $\gamma_w * h_a$. The total uplift force U, per unit length of the wall is given by

$$U = \frac{1}{2} * \gamma_w * h_a * D$$

The submerged unit weight of the prism is

$$W_{sub} = \frac{1}{2} * \gamma_{sub} * D^2$$

Where h_a= Average pressure head at the base of the prism abcd

γ_{sub}= submerged unit weight of soil

The **factor of safety** is given by

$$F = \frac{W_{sub}}{U} = \frac{\frac{1}{2}*\gamma_{sub}*D^2}{\frac{1}{2}*\gamma_w*h_a*D} = \frac{\gamma_{sub}*D}{\gamma_w*h_a} \quad \ldots..(1)$$

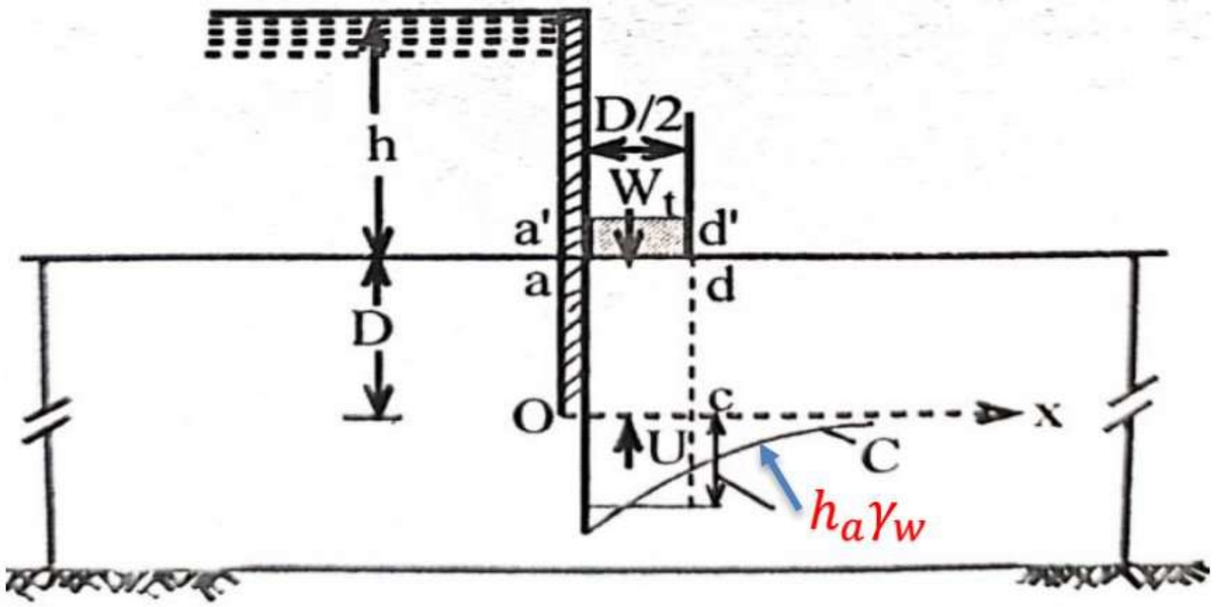

Figure b: *Heave piping phenomena*

- It is seen that in equation (1) that the factor of safety increases if depth of penetration of sheet pile increased or head due to seepage is decreased.
- It is not always possible to increase D to increase the factor of safety.
- The factor of safety can be increased by placing a filter **aa'd'd** over the prism **aocd**. If the weight of such filter is Wt, the new factor of safety can be written as

$$F' = \frac{W_{sub}+W_t}{U} \quad \ldots\ldots(2)$$

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Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

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INTRODUCTION

- A structure whose main purpose is to supply water to the off taking canal is know as head works.
- When the off taking canal is to supply water for irrigation purpose, its headworks is call irrigation head works.
- Irrigation headworks may be divvied into two categories.
- 1. Diversion headworks or simply head works.
- 2. Storage headwork.
- **1. Diversion head works.**
- The purpose of this head work is to divert river water into the off taking canal.
- It is located across the river near the point of the take-off of the canal.

A diversion headwork performs the following functions:

- It regulates the flow in the off-taking canal.
- Silt entrance into the canal is controlled.
- It raises the level of water in the river.
- This causes diversion of the river water into the off taking canal, under gravity flow.
- By raising the level of water in the river with the help of diversion work.
- Consequently raising the off-take level of the canal, more area may be brought into the command of the propose canal system.

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- **A diversion headwork performs the following functions:….**
- The level of water in the rivers remains fluctuating due to changing flow.
- Diversion works reduce such fluctuations in river water level.
- Although the purpose of diversion head work is not to store water but still some water definitely gets store.
- This stored water can be use to augment the short supplies of the river.
- This type of augmentation is possible for few days only, because stored water is not much in amount.
- **2. Storage head works.**
- Storage works the works which store surplus water when available in the river.
- This store water is use to supplement the available flow when flow in the river falls short of the demand.
- Storage head work comprises the construction of dams across the river.
- Various aspects of storage of water have already been discussed in chapter on Reservoir Planning Construction Aspects.

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- **Intake:**

Intake structure is a hydraulic device constructed at head of an irrigation, tunnel conduit or power canal to obtain the required quantity of water from the river or reservoir for the different engineering purposes.

Purpose of intake

The main purposes of intake structure are follows:

- To allow access for and control water from the source and probably to measure the flow rate.
- To control the sediment entry into canal.
- To smooth, an easy and turbulence free entry of water.
- To prevent clogging of entrance with floating debris.

Following are major appurtenances of intake structure

- The raised inlet sill to prevent entry of the bed load of river
- The skimmer wall at the inlet to trap floating ice and debris.
- The trash rack to trap sub-surface trash, equipped with either manual or automatic power driven rack cleaning devices
- Settling basin followed by a entrance sill diverting the sediment layer towards the flushing canal
- Flushing sluice to flush the deposited silt
- The intake gates to control the flow rate into canal
- The scouring sluices in the diversion weir to flush bed load upstream of the inlet sill.

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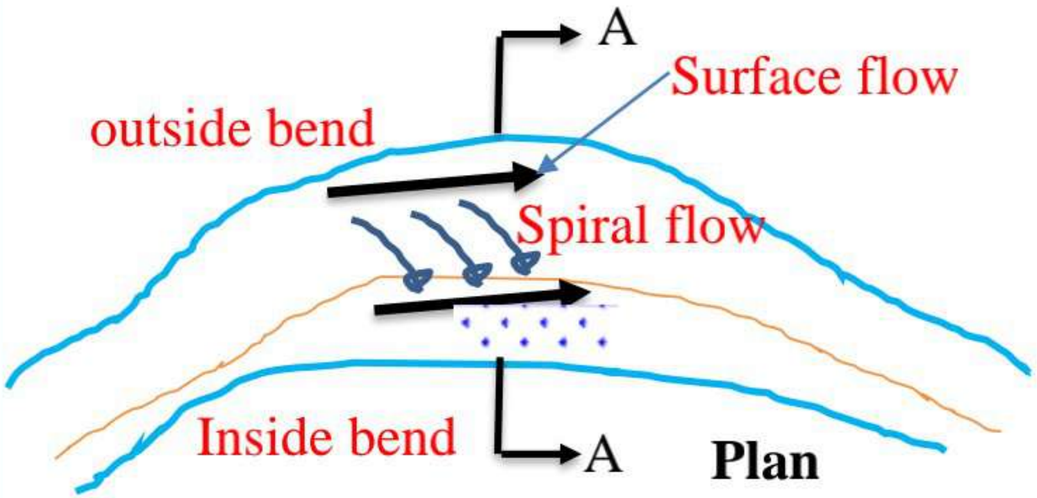
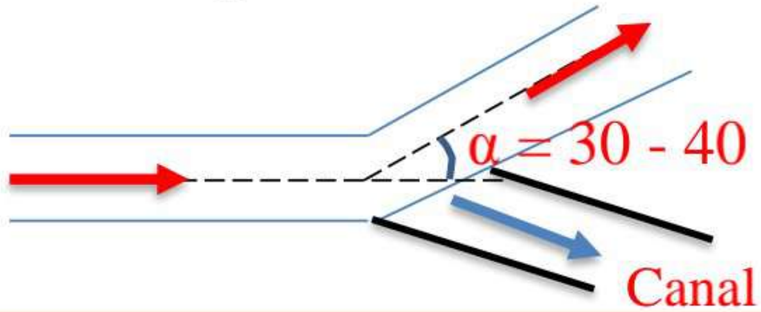
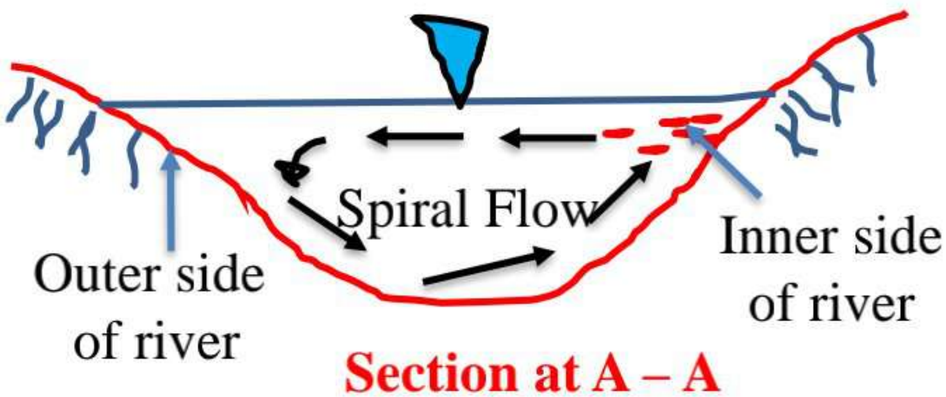
Location and types of intake:

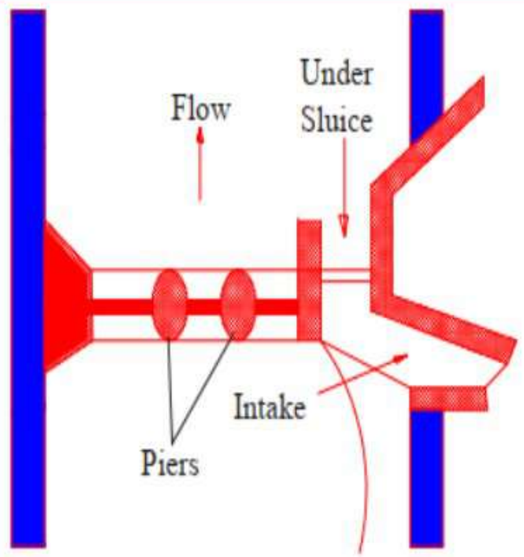
1) Location: River reaching upstream of intake should be well established with stable banks.

- Intake should be placed on the outside of the bend but not inside of the bend. A spiral flow forms which transports the bed load to inner side of the river. i.e. bed load is diverted from the outer banks.
- The stream should not have large gradient at upstream of intake.
- The location of pond level and their variations, navigation hazards, and location of the diversion structure must be considered.
- The structure is aligned to produce a suitable curvature of flow into the intake, and a diversion angle of around 30 °- 45° is usually recommended to produce curvature of flow.
- The entry to tunnels and penstocks taking from the reservoir has to be placed lower than dead storage level.









Plan View of Side Intake

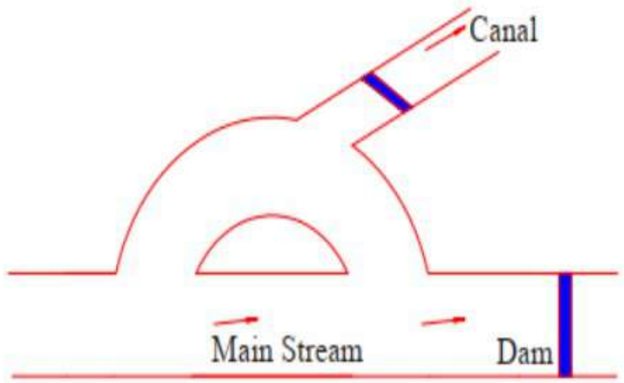


Fig 20.1.2 Plan View of Diversion Canal Intake

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2) Types of Intake

- Type of intake structure depends upon type of power plant as well as its layout.
- According to the location, intake can be classified into following two types.

A) Surface intake

i) Side intake

ii) Frontal intake

iii) Drop intake

iv) Himalayan intake

B) Sub surface intake

i) Power or fore bay intake

ii) Tower intake

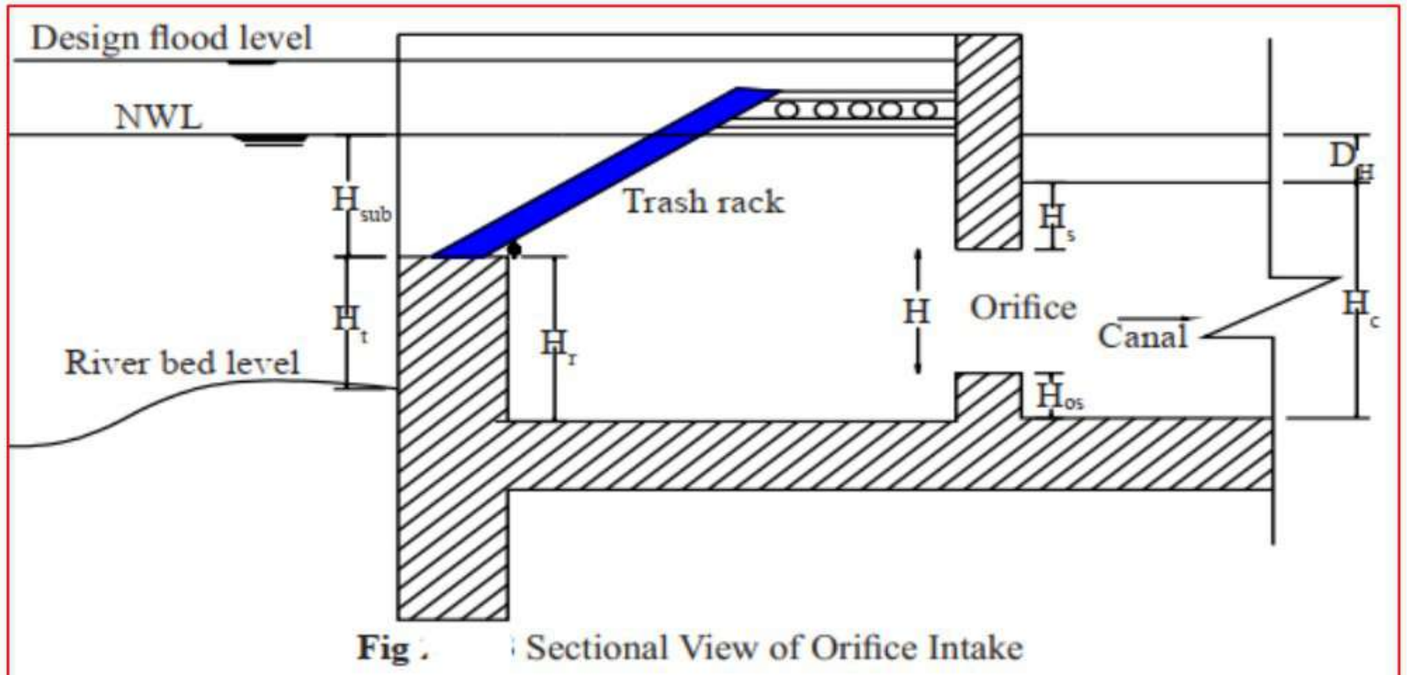

Fig . Sectional View of Orifice Intake

A. Surface Intake Types

i) Side intake: It is the most common type of intake at ROR hydropower plants. The basic principle of hydropower projects is to abstract water from river through an intake structure located on the riverside. Side intakes are used on all type of rivers having mild slope to steep slope.

The main advantages of side intake concepts are as follows:

- Easy access for operation of gates and stop logs and handling for trash racks during floods
- Have best foundation conditions for intake at bank of river.
- Constructed on dry land on the bank of river.
- Relatively easy and safe to operator.
- Easy access for general maintenance

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ii) Frontal intake:

- Frontal intakes are constructed in front of river at dam body of river.
- It is normally used at low head plants where the intake and the powerhouse is an integral part of the dam.
- Under sluice is constructed where sediment load transported by the river contains large particles.

The **main advantages** of frontal intakes are as follows:

- The length of waterway from head water to tail-water is minimum.
- No unnecessary bends, curves in waterway which causes head loss.
- Easy to obtain uniform flow distribution in upstream and downstream of intakes as of the turbines.
- Easy to arrange series of identical generation units with identical waterway.

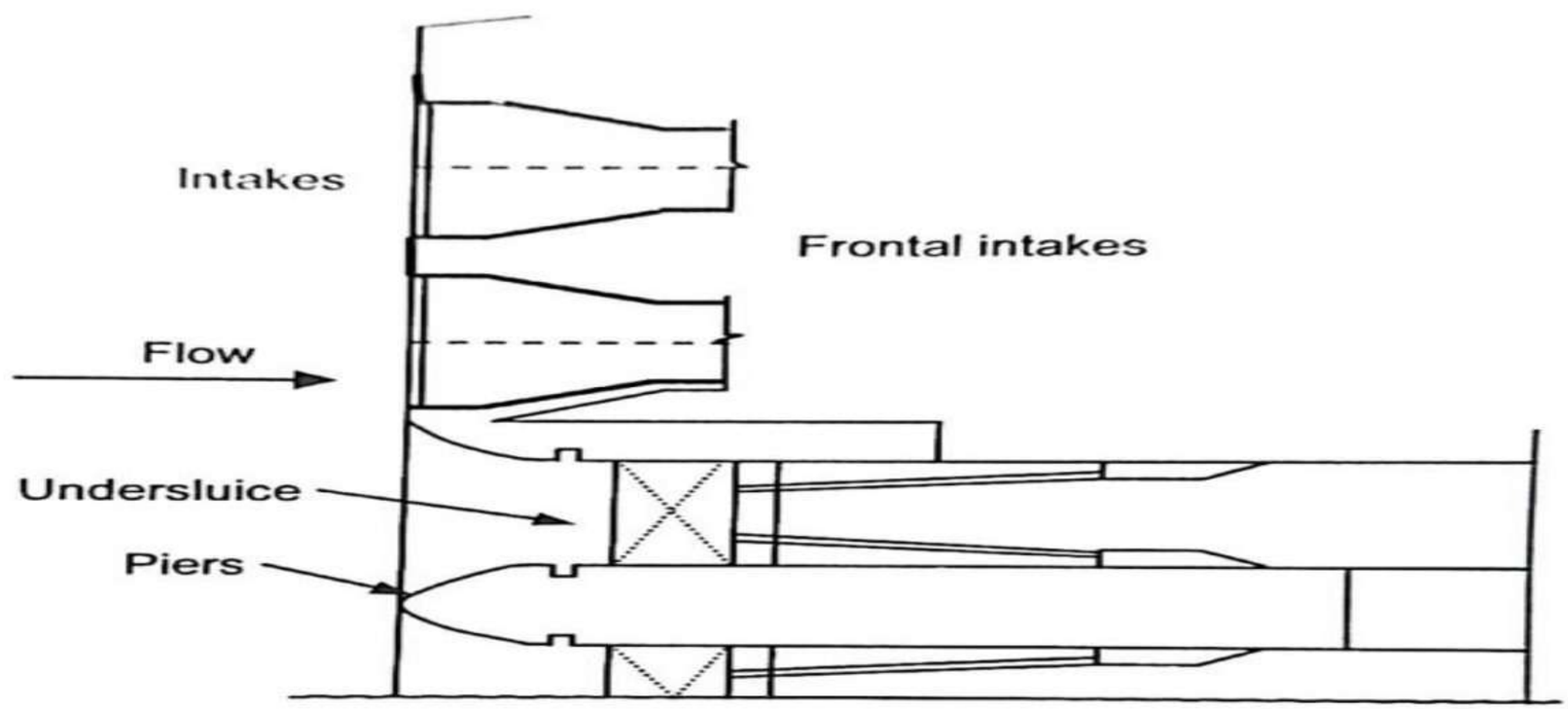

Figure : Typical arrangement for frontal intake

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iii) Drop intake: Drop intakes are located in small streams and they add water either to reservoir feeder tunnel or directly to a power tunnel. The trash rack is at same level as initial river bed and it is furnished with a flat apron upstream as well as downstream. Collector canal below trash rack is shaped like a trench in river bed so it is also called trench intake.

Advantages:

- It has minimal impact on flow pattern of river.
- It is favorable for small and steep river plants.

Disadvantages:

- The trash rack and intake chamber remains almost inaccessible throughout monsoon season
- Frequent damage of trash rack
- Unfavorable during low flow season because almost all water and sediment takes place during this season.
- Difficult access for handling the trash racks during floods and general maintenance.

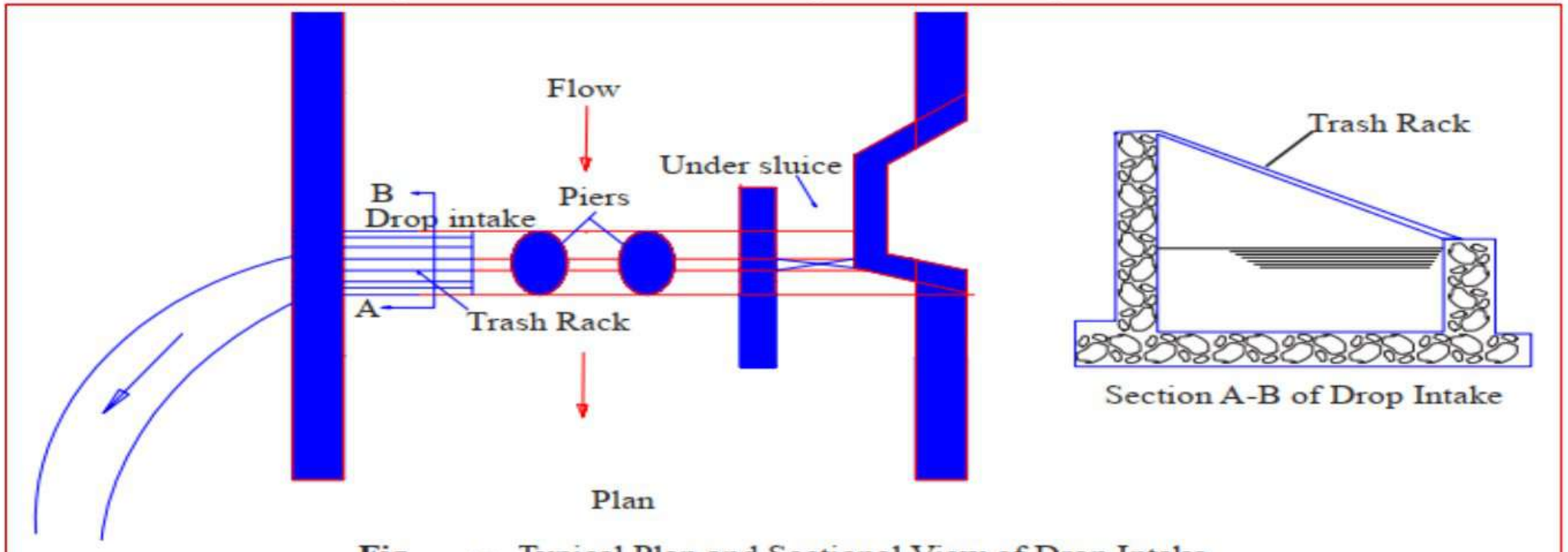

Fig · · Typical Plan and Sectional View of Drop Intake

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iv) Himalayan intake:

Himalayan intake is provided for ROR project with daily peaking requirement in relatively Steep River. The objective of new intake design is to include a daily peaking reservoir and to meet special demands for intake in the Himalayan region.

- It is feasible in range from 5 cumec to 50 cumec depending on local conditions.
- There are two intakes with coarse trash racks to intake chamber. One located at top of chamber and other at bottom of chamber just upstream of sluice gate.
- If crest is long, settling basin is located inside dam structure itself.
- If a hazard flood occurs, diversion dam has a free over flow crest which facilities to bypass flood.
- The lower intake is used during low flow season when pond is operated for peaking purposes.

The objectives of the Himalayan intake are to:

i. Meet daily peaking during low flow season
ii. Debris control during monsoon season
iii. Sediment bypass during monsoon season
iv. Sediment control at intake
v. Bypass of possible floods hazard

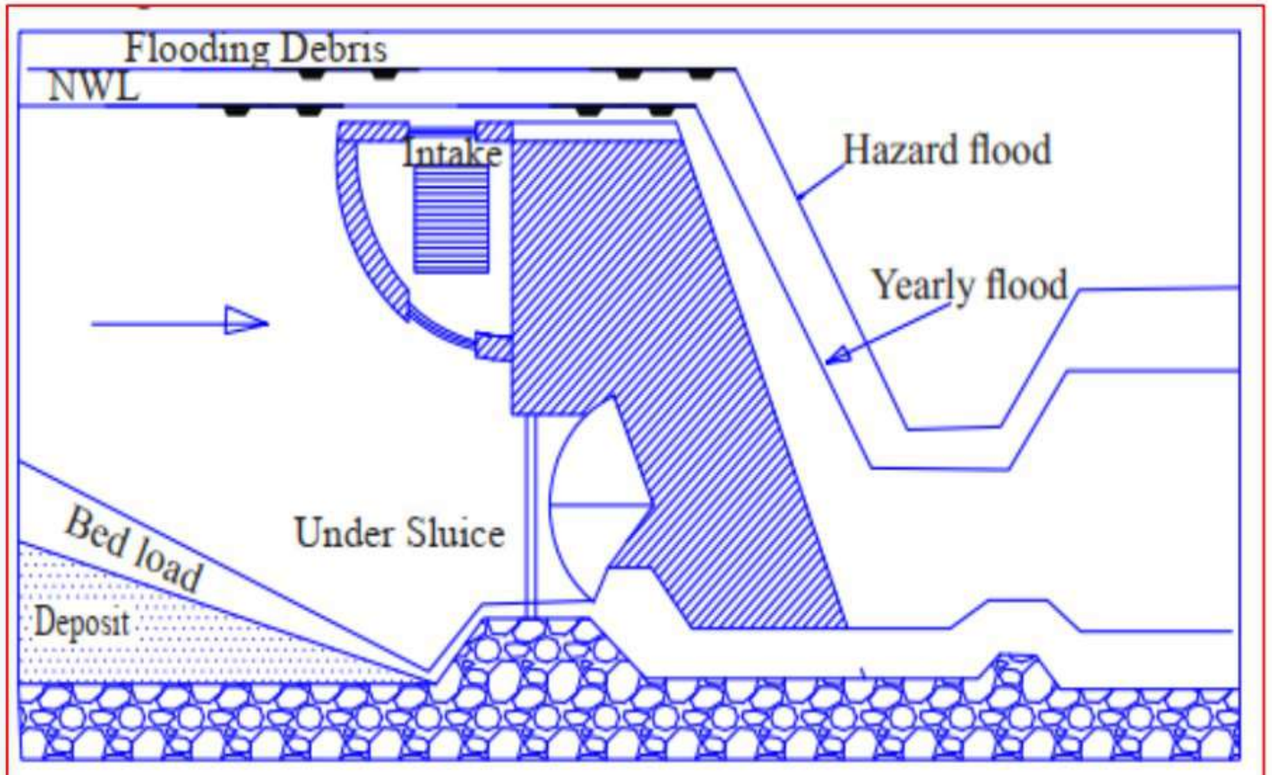

Fig . Sectional Views of Himalayan Intake

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2) Types of Intake….

B) Sub surface intake

i) Power or fore bay intake

- The intake is generally provided in the body of a dam for the power house located at the toe of dam.
- Such type of intake is called power or dam intake. The dam intake controls water-flow into the penstocks.
- It is constructed in semi circular shape and provided trash around its opening to control sediment and floating bodies.

ii) Tower intake

- It is provided upstream of dam where other type of intake is inconvenient to provided in it.
- It is constructed near the abutment or a side hills. The tower intake is connected with the dam by providing bridge near the dam.
- The flow into the tower intake occurs through the ports provided at different levels and flow in the channel or penstock is controlled by gates.
- Tower intake is suitable in case of discharge is large or in at fluctuation of water level.

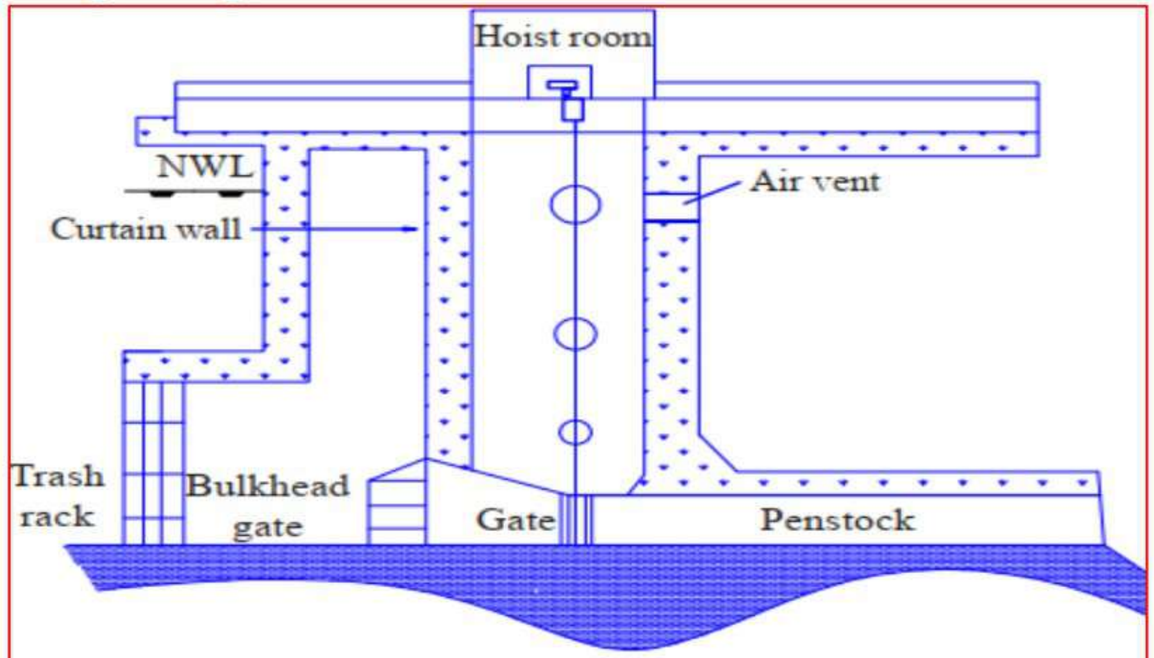

Fig . Tower Intake

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 # Classification according to the hydraulic features

i) Run of river intake ii) Canal intake

iii) Dam intake iv) Tower intake v) Shaft intakes

i) Run of river intake: Run -of-river intake is provided for run of river plants and form an integral part of the power house. In silty rivers special arrangement for the desilting are usually constructed upstream of the intake.

ii) Canal intake: It is similar to run-of- river intake but it admits water into the canal not directly through the penstock

iii) Dam intake: Generally suitable for the powerhouse located at toe of dam.

- It is suitable when topography supports foundation of dam.
- In case of valley dam plants, this type of intake is suited in body of dam.

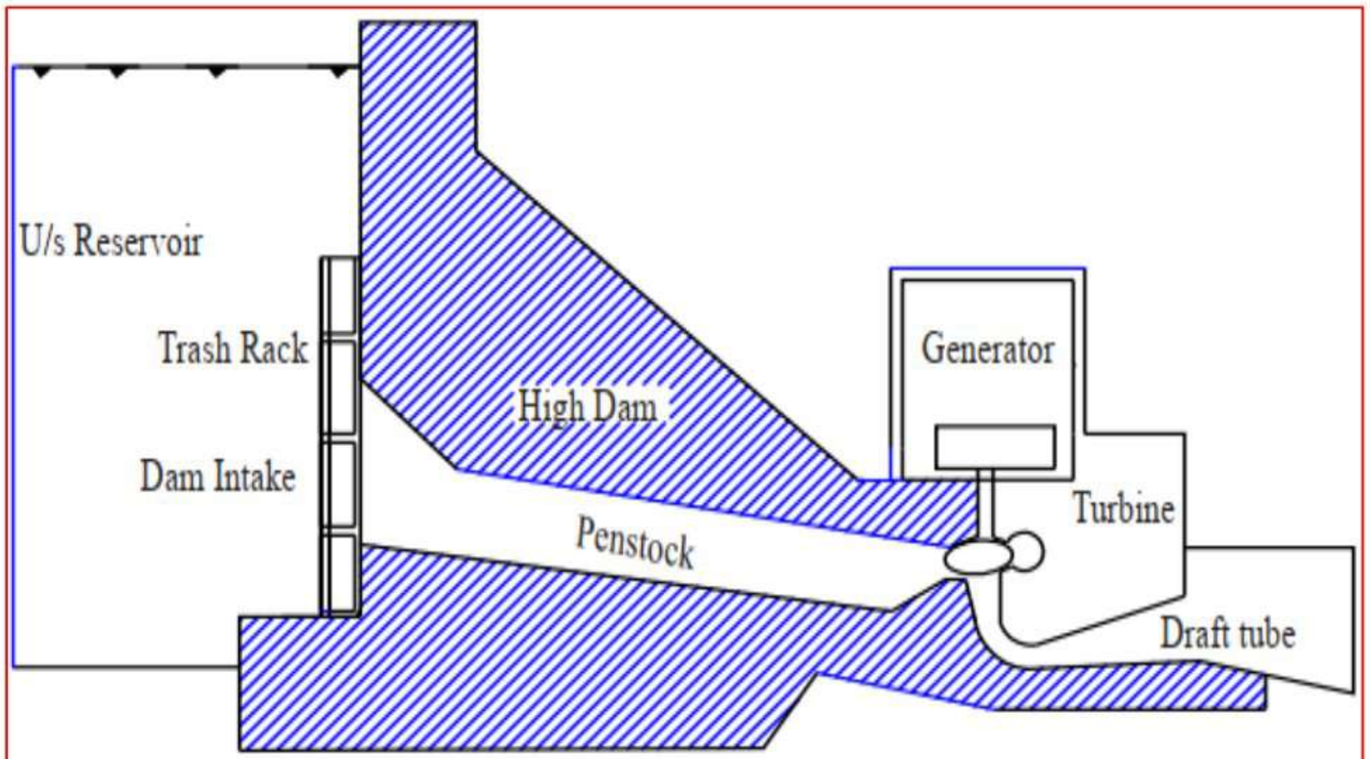

Fig Typical Power or Dam Intake

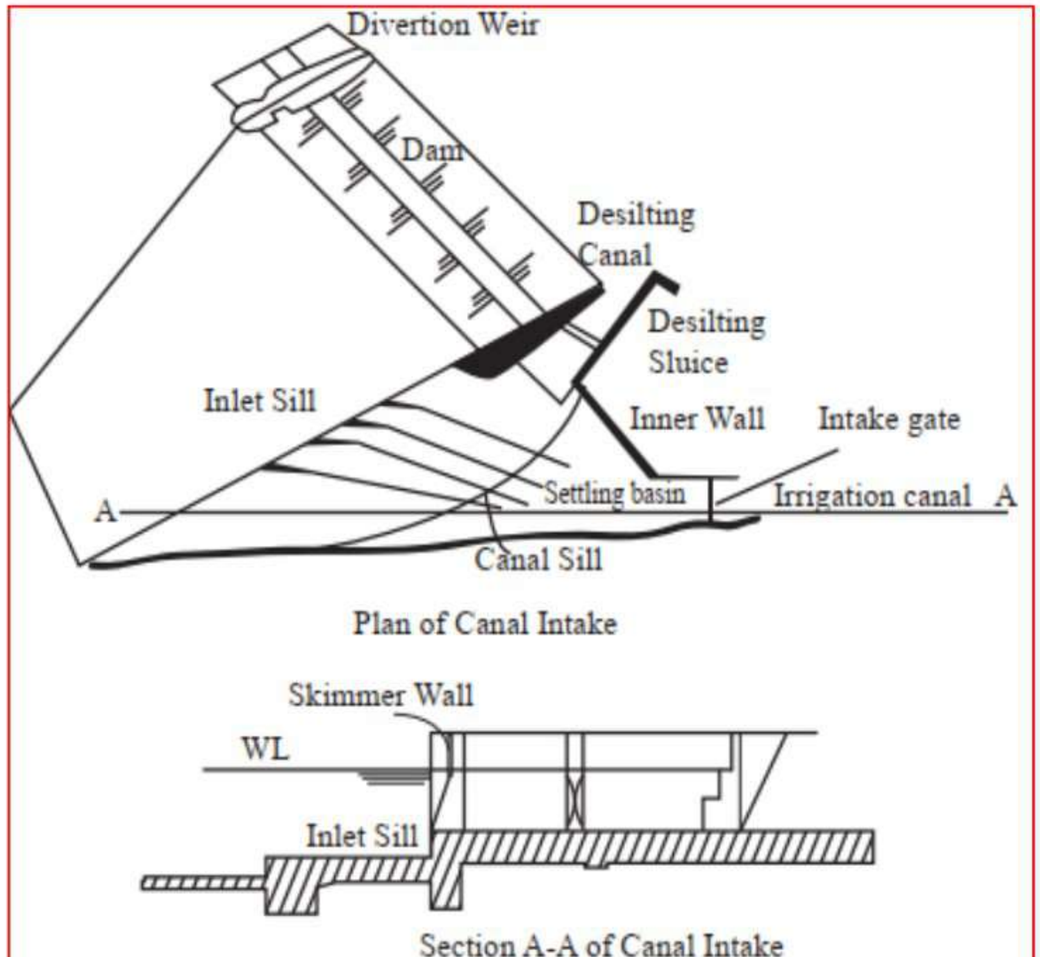

Fig . Typical Canal Intake

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Classification according to the hydraulic features

iv) Tower intake

- Tower intakes are normally on hill sides or near the abutment but not far off from dam, when it is suitable where inconvenient to provide other type of intake directly on the u/s face of dam.
- They are also constructed when there is a large discharge or wide fluctuation of water level. it maybe two types

a. Wet Tower Intake **b. Dry Tower Intake**

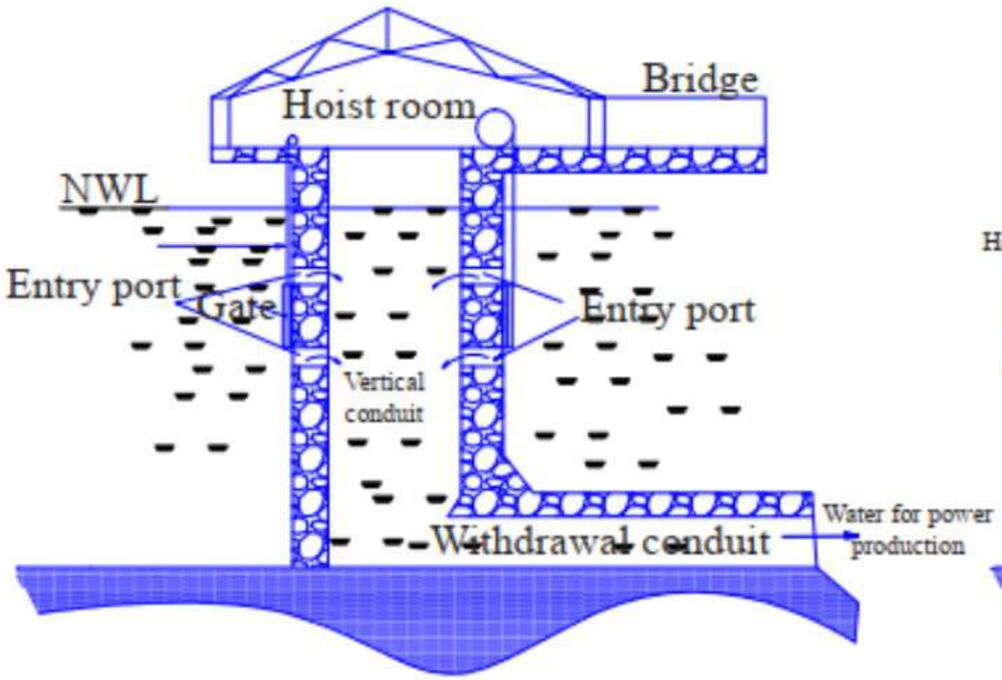
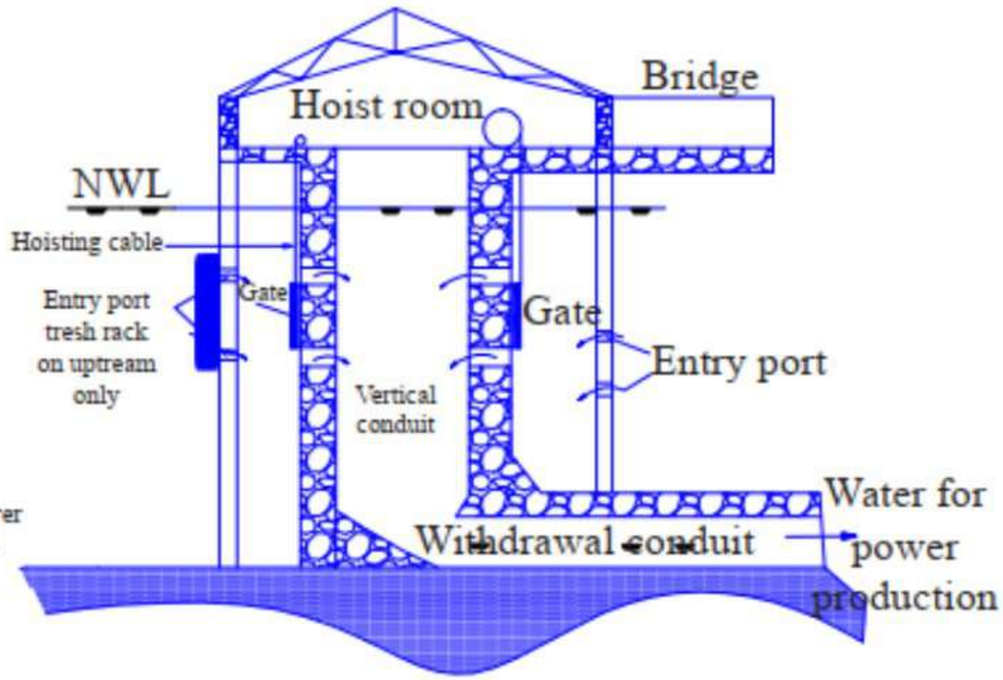


Fig **(a)** Wet Tower Intake



(b) Dry Tower Intake

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 # Classification according to the hydraulic features…

v) Shaft intakes

It is a vertical or near vertical shaft driven in the bed which carries water to penstock for powerhouse for power production. This shaft intake consists of following parts

- Trash rack at the entry of water to the shaft.
- Vertical or near vertical shaft followed by an elbow which connects the shaft and the penstock.
- Intake gate arrangement.
- Gate chamber
- Hoist chamber
- Sometimes an access tunnel from top of the dam with intake to have entry of water under submerged condition.

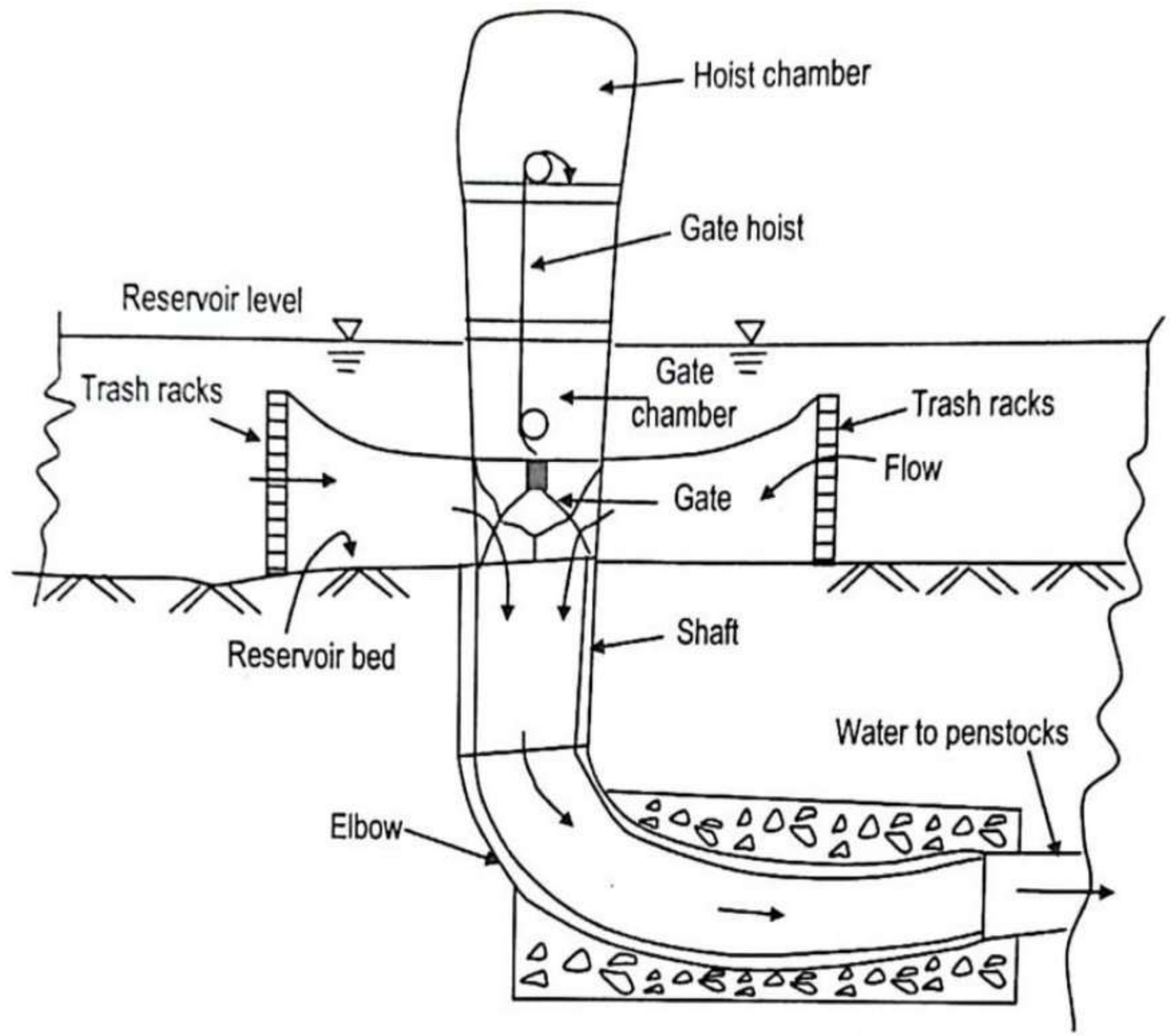

Figure : Shaft intake

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Selection of type of intake:

The most suitable type of intake for a particular site shall be selected considering the following factors

1. Nature of river.
2. Nature and scale of hydropower development
3. Sediment, trash and debris content
4. Construction consideration
5. Operation and maintenance considerations

1. Nature of river:

- Side intake may be used on all type of rivers, ranging from mild sloping silt and sand – bed rivers to steep boulder bed – river or step – pool type of rivers.
- The use of **drop intakes** shall generally be limited to small hilly rivers which witness flash floods under heavy rainfall, high velocities of flow capable of transporting large quantities of sediments, flood of sufficient duration exceeding the mean discharge.

2. Nature and scale of hydropower development

- *Side intake* may be used for any type of run – of – river hydropower project.
- *Frontal intakes* may be preferred for low head plants where minimization of head losses commonly associated with other intakes is essential for optimal generation from the plant.

3. Sediment, trash and debris content

- *Side intakes* shall generally be preferred over other intakes for Nepali rivers which carry large amount of sediments, trash and debris during monsoon.
- A *frontal intake* located next to a free overflow section may be used in rivers with floating debris and bed load.

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Selection of type of intake….

- *Drop intakes* shall generally be avoided in rivers with high sediment content because the sediment content in the abstracted water will be high as this water is drawn from the bottom of the water column where the sediment concentration is highest.

4. Construction consideration

- *Side intake* most convenient for construction as it is usually constructed on dry land on the bank of river
- This advantage may also hold for frontal intakes located at a certain distance form the diversion structure, however, this arrangement may not be suitable for restricting sediment entry.

5. Operation and maintenance considerations:

Considering the large amount of trash and debris carried by Nepali rivers during floods, side intakes shall generally be preferred over other type of intakes for the following reasons

- Ease of trash handling, gate and stop log operation and general maintenance
- Low maintenance cost due to reduces likelihood of trash rack damage.

Drop intakes shall not be adopted in rivers in which the mean flow remains relatively high throughout the wet season. Where such conditions exist, the intake may remain inaccessible for repair or cleanup for long periods in the case of clogging or damage of the trash rack, gravel flushing arrangements or the intake gate.

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Basic design consideration (Guidelines) for surface intake:

The necessary considerations to be taken in design of intake are as follows:

i) Hydraulic consideration

- Minimum head loss
- Minimize flow contraction
- Minimum vortex formation

ii) Structural /operation consideration:

- Structural consideration Concrete structure- (weir, intake, energy dissipater, divide wall) and metallic structure (trash, rack, gate) structurally stable for various loading consideration
- Operational consideration : Smooth operation of gate and trash racks without failure, sediment exclusion.

iii) Environmental consideration:

- Fish ladder in head works-to enhance fish migration
- Minimum flow to downstream

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Design consideration of side intake

A. The consideration for the design of the side intake are as follows

1. A square or rectangular orifice type of intake should be chosen. The intake orifice should be fully submergence (minimum of 30 to 50 am submergence) during the dry season so as to provide the full design flow. The side intake should be at

- Straight river U/S and D/S of the intake.
- Alternatively, on the outer side of the bend to minimize sediment problems and maximize the assured supply of desired water.
- Relatively permanent river course.
- By the side of rock outcrops in both banks of the river or large boulders for stability and strength.

2. In rivers that do not have significant floods or where the site conditions do not permit placing of an orifice (e.g. high flood walls required), the extension of the headrace canal is another option.

3. Fixing its invert level

a. The invert level of the intake shall be fixed considering the sediment content in the river flow and pervious design and construction experience.

b. This invert shall be minimum 0.5 to 2 m above the under sluice crest level, according to site condition, to prevent entry of bed sediments into the intake opening due to turbulence is sluice bay flow.

4. Capacity(intake opening): According to the flushing requirement and tentative losses the intake has to be oversized than the design flow by about 10% to 20 %. In other words the design discharge for intake will be taken as 10 to 20 % in addition of the turbine discharge.

5. Entrance velocity: The entrance velocity through the intake opening should be less than 0.6 to 0.8 m/sec but for small system the velocity can be up to 1 m/sec.

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A. The consideration for the design of the side intake are as follows….

6. Intake losses:

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

a) Entrance losses: The entrance losses comprise;

i) Loss due to change in direction of flow is given by (h_b)

$$= \left(\frac{V^2}{2g}\right) - C\left(\frac{V_a^2}{2g}\right) = \left(\frac{V_a^2}{2g}\right)\sin\alpha$$

Where V= velocity in the canal.

V_a = velocity of flow in the main stream (approach velocity)

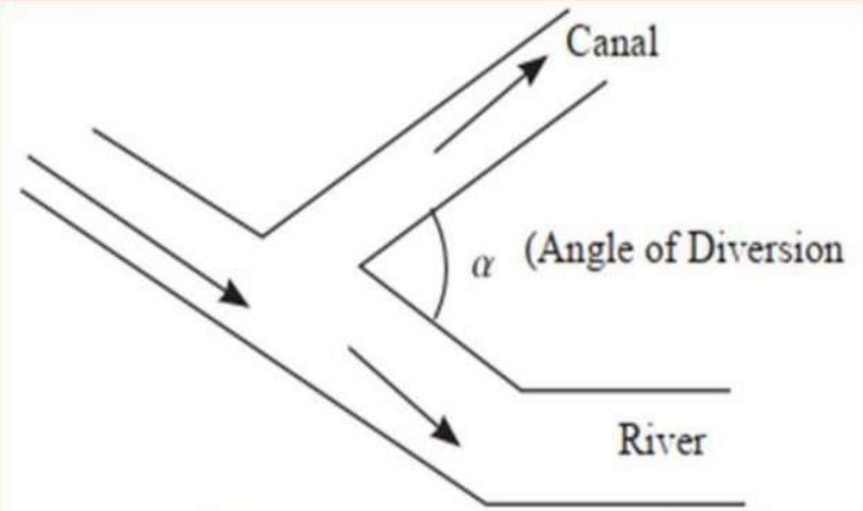

Fig : Angle of Diversion of Canal

C = a constant depending on the angle of diversion and ranging between 0.8 to 0.4; Coefficient C has higher values for intakes with lower off take angles and lower values for 90° diversions.

ii) Loss due to sudden contraction of area at the diversion. The loss is equivalent $h_{L1} = K\left(\frac{V^2}{2g}\right)$

Where K varies between 0.03 for round mouthed entry to 1.3 for sharp entry.

b) Head gate losses: Gate contract flow, the coefficient Cd in following formula

$$Q = C_d A\sqrt{2gh}$$

Where Q = flow in conduit, A = area of gate opening, h = difference in head acting on the gate, C_d = coefficient of discharge (Value of Cd= 0.62-0.83)

7. A course trash rack should be provided to prevent big boulders and floating logs from entering into the headrace canal.

8.A gate/stop log should be provided to regulate flow (adjust/close) during operation and maintenance.

9. To optimize the D/S canal and other structures, a spillway should be provided close to the intake.

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B. Orifice design for side intake:

- A side intake normally includes an orifice downstream of the trash rack at the riverbank, through which water is initially drawn into the headrace.
- An orifice is an opening in the intake form which the river water is conveyed towards the headrace.
- The orifice allows the design flow to pass through it under normal conditions (i.e. low flow) but restricts higher flows during floods. The discharge through an orifice for submerged condition is
- Discharge through orifice (Q) = $C_d A\sqrt{2g\,(h_r - h_c)}$
- Velocity through orifice (V) = $C_d\sqrt{2g\,(h_r - h_c)}$
- Where h_r = water level in the river next to the orifice relative to a datum, h_c = water level in the headrace canal measured form the same datum as h_r, C_d= coefficient of discharge

C. Trash racks for side intake:

- The trash rack for side intakes can be manufactured form flat steel, angles, tees or round bars welded together at fixed interval.
- The trash rack at the intake is also known as "coarse trash rack".
- For side intakes, the function of the trash rack is to stop boulders, cobbles, floating logs and branches from entering the headrace.
- Coarse trash racks for side intakes are not designed to exclude gravel and sediment. This is the job of the gravel trap and the settling basin.
- Depending on the length and width of the opening, nature of the sediment load and the required flow, a clear spacing of 50 mm to 200 mm can be used.
- The trash rack is designed so the approach velocity (V_a) remains between 0.60 m/s and 1.5 m/s (a low velocity is uneconomic, where as high velocity tends to attract bed load and debris, and results in increased head loss).

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C. Trash racks for side intake…

- The total surface of the screen will be given by the equation
- $A_S = \frac{1}{K_1} * \left(\frac{t+a}{a}\right) * \frac{Q_d}{V_a} * \frac{1}{\sin \emptyset}$
- Where, Q_d = Rated flow, V_a = approach velocity, t = bar width, a = space between bars, K_1 = coefficient related to the particle clogging of the screen (no automatic raker 0.20 – 3.30, automatic raker with hourly programmer 0.40 – 0.60, automatic raker plus differential pressure sensor 0.80 – 0.85), Ø = angle of the screen with the horizontal {Take $\frac{0.3+0.8}{2}$ = 0.55 for Nepal}
- The head loss through trash rack may be computed using the following formula (IS:11388 – 1995):

i) Kirschmer's formula $h_{Lr} = K_r * \left(\frac{t}{a}\right)^{\frac{4}{3}} * \left(\frac{V_b^2}{2g}\right) \sin \emptyset$

Where;

K_r = a factor depending on cross section of bars

h_{Lr} = rack losses

T = thickness of rack bars

B = spacing between bars

V_b = velocity of flow in front of bars = approach Velocity = V_a

Ø = angle of bars with horizontal

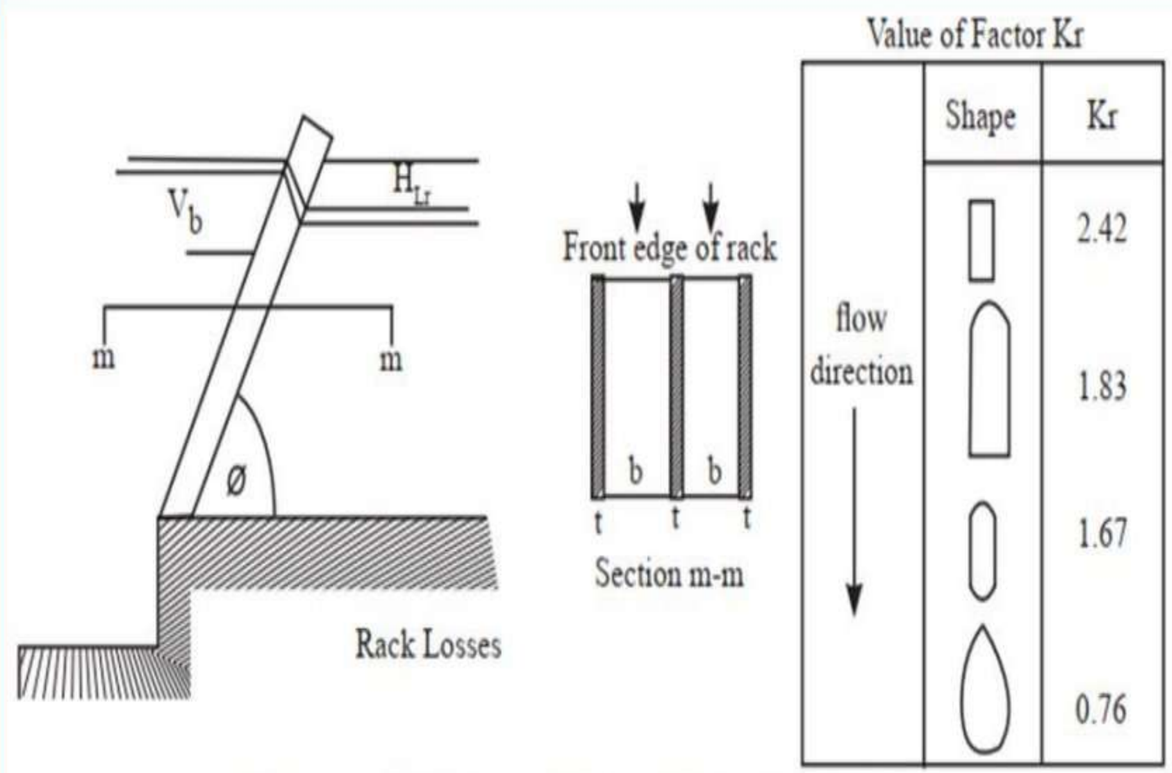

Fig : .. . Inclination and Shape of the trashrack in Intake

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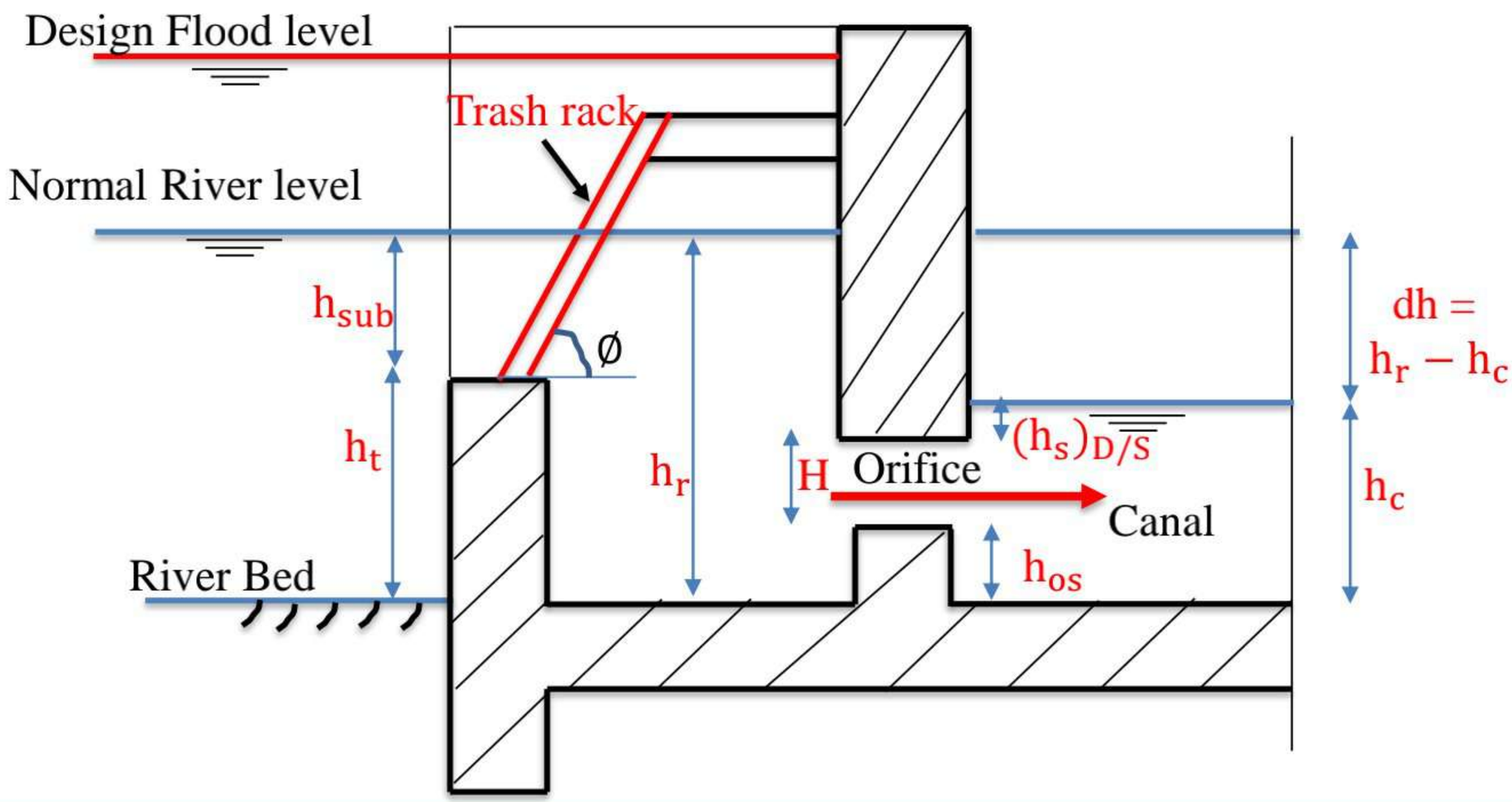
C. Trash racks for side intake…

- In view of large amount of trash rack in Nepali rivers during floods, 25 to 50 % of the area of racks may be considered to be obstructed by trash.
- Trash racks can be cleaned by hand up to 4 meter depth.
- A horizontal platform above high – water level should be provided to facilitate the operation.
- On unattended plants operated by remote control, mechanical raker are used.
- The mechanical raker can be designed to be operated either on a time basis or on a head differential basis.
- The latter uses a sensor to detect the drop in head across the trash rack.



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Trash rack Design Step for side intake:

1. Calculation of head loss

a) Head loss due to friction in trash rack $(h_{Lr}) = K_r * \left(\frac{t}{a}\right)^{\frac{4}{3}} * \left(\frac{V_b^2}{2g}\right) \sin \emptyset$

b) Head Loss due to change in direction of flow is given by $(h_b) = \left(\frac{V^2}{2g}\right) - C\left(\frac{V_a^2}{2g}\right) = \left(\frac{V_a^2}{2g}\right) \sin \alpha$

$\therefore$ Total head loss $(h_L) = (h_{Lr}) + (h_b)$

2. Calculation of head loss coefficient (K)

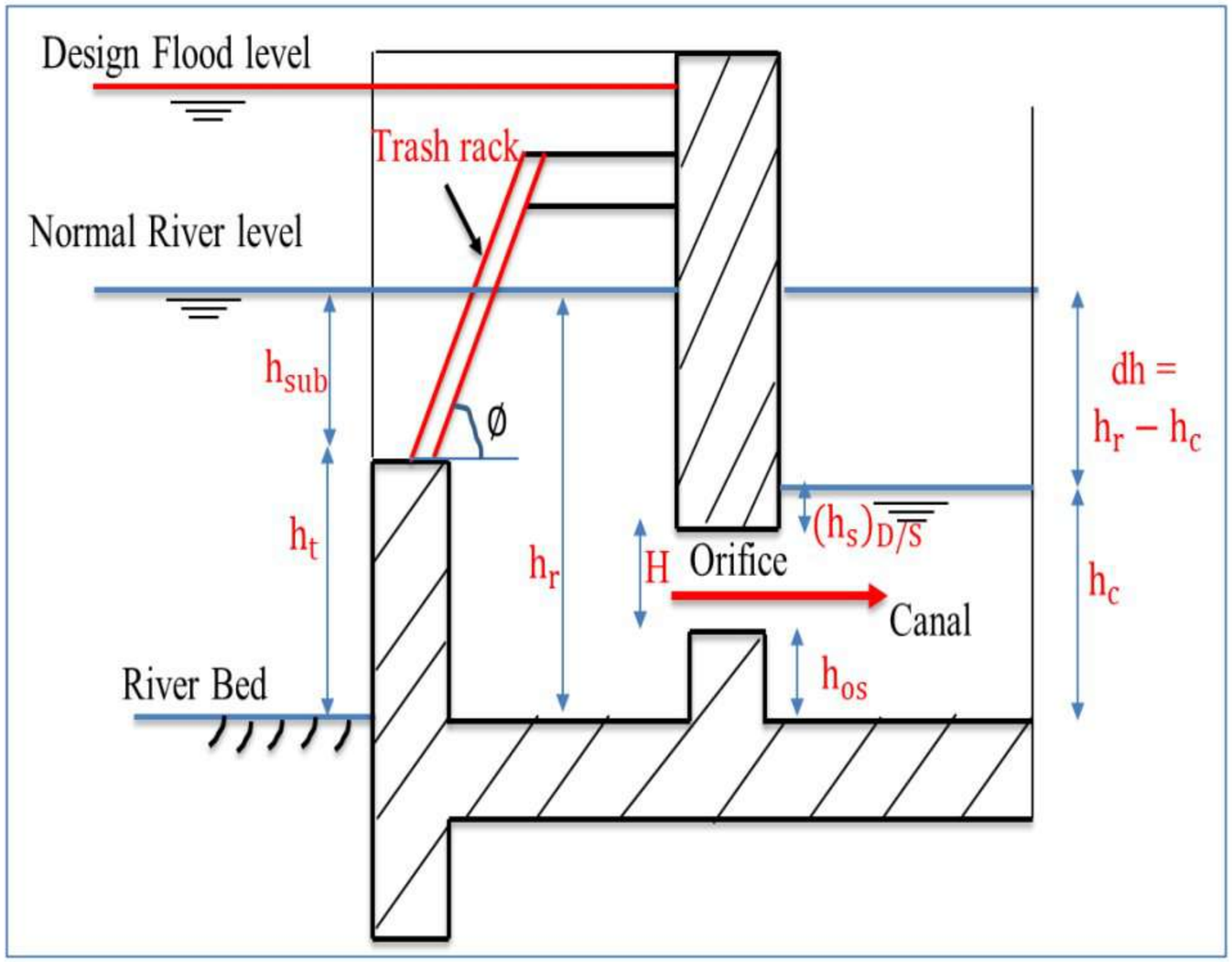
Head loss coefficient $(K) = \frac{(h_{Lr} + h_b) * 2g}{V_a^2}$

3. Calculation of total area of the submerged part of the screen (A_S)

$A_S = \frac{1}{K_1} * \left(\frac{t+a}{a}\right) * \frac{Q_d}{V_a} * \frac{1}{\sin \emptyset}$

4. Calculation of trash rack width (B)

- $B = \frac{A_S \sin\emptyset}{h_{submerged}}$
- $h_{submerged} = h_r - h_t$
- $h_r = h_c + dh$
- $h_c = (h_s)_{D/S} + H + h_{os}$
- Driving head $(dh) = \frac{\left(\frac{V_o}{c}\right)^2}{2g}$
- C = Velocity coefficient of orifice, V_o = velocity through orifice



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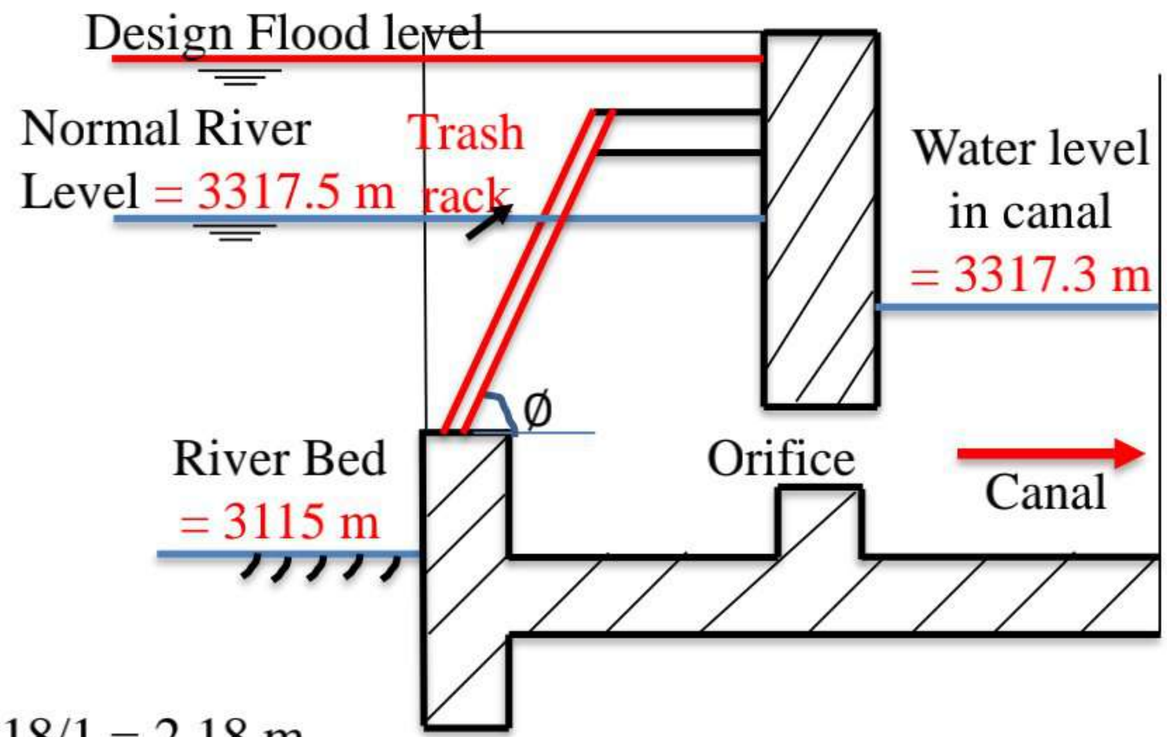
Q. N. Design and draw section of a side intake with coarse trash racks for a project in which river bed level is 3115 m amsl weir crest level is fixed to 3317.5 m amsl. The highest flood level in 100 years return period is 3320.83m amsl and flood level in 20 years return period is 3319.55 m amsl. The canal water level is fixed as 3317.3 amsl. The turbine discharge of a period is 1.45 m3/sec. Assume other suitable data fro design. (TU Exam)

Solution: From question given as following data, $(Q_T) = 1.45\ m^3/s$, Design discharge for intake $(Q_D) = 1.2 * 1.45 = 1.74\ m^3/s$,



- River bed level = 3115 m
- Weir crest level $(h_r) = 3317.5$ m
- Canal water level $(h_c) = 3317.3$ m
- Assume intake (top) sill level 50 cm below the normal water level and assume entrance Velocity through intake is 0.8 m/sec.
- Cross – sectional area of intake

$$(A) = \frac{Q_D}{V_0} = \frac{1.74}{0.8} = 2.18\ m2$$

Assume depth of intake is 1 m, then length of intake is 2.18/1 = 2.18 m.

- Hence adopt 1m width by 1.1 m depth 2 number of intake opening.

New check discharge carrying capacity of intake in given NWL condition and flood condition, by using formula, Discharge through orifice $(Q) = C_d A\sqrt{2g\,(h_r - h_c)}$

Assume $C_d = 0.6$ for roughly finished masonry orifice, A = (1*1.1)*2 = 2.2 m2 and $(h_r - h_c) = 3317.5 - 3317.3 = 0.2$ m

$\therefore (Q) = C_d A\sqrt{2g\,(h_r - h_c)} = 0.6 * 2.2\sqrt{2 * 9.81\ * 0.2} = 2.615 > 1.74\ m^3/s$, Hence OK.

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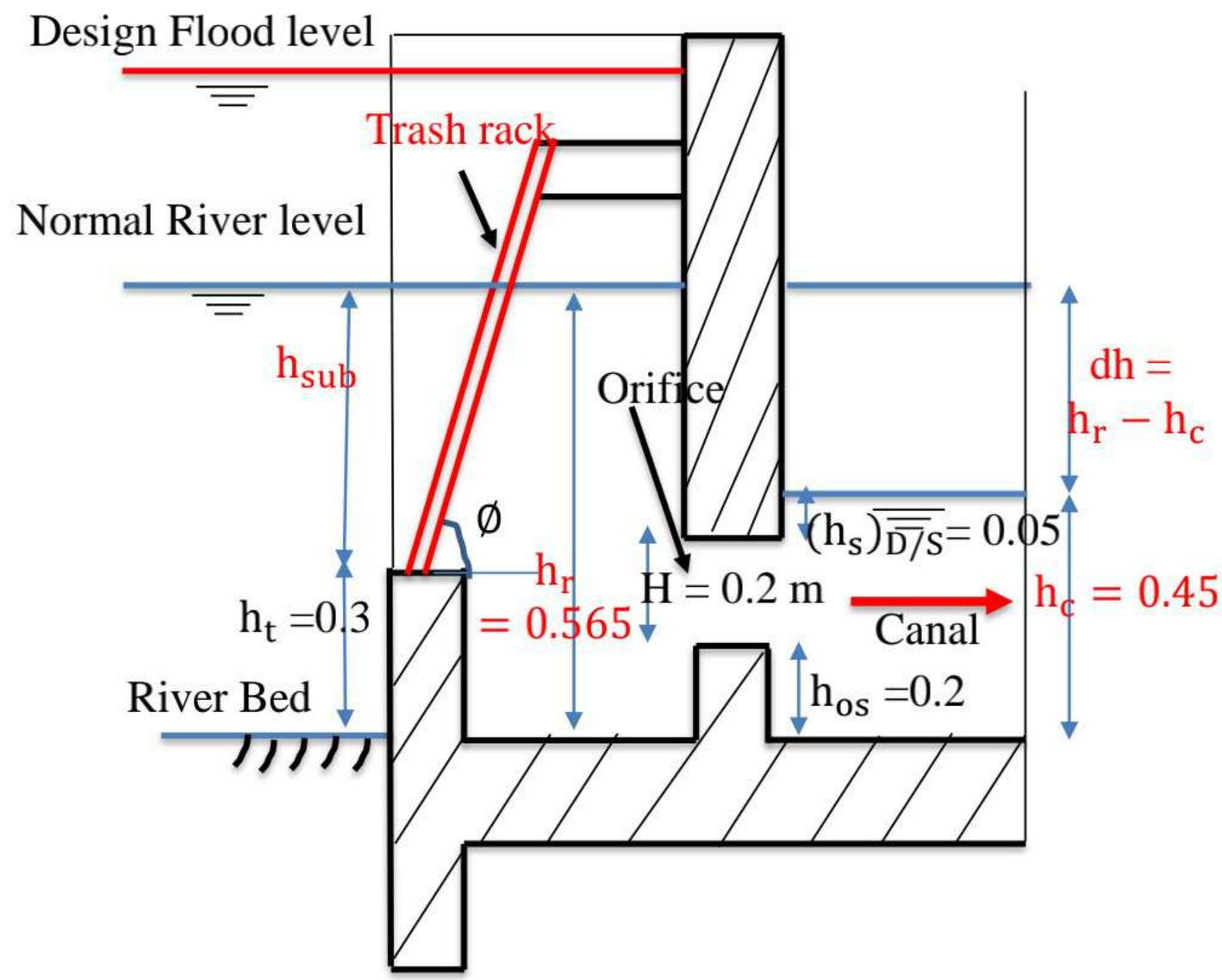
Q. N. Design a trash rack for side intake of design discharge $0.077m^3/s$ for a Micro Hydropower project. other data are mentioned below; Trash rack coefficient (K_t) =2.4, Bar thickness = 4mm, clear spacing of bar = 25mm Approach velocity = 0.50m/s, angle of inclination of bar from horizontal = 60^0, flow deviation =20^0 ,Height of the trash rack bottom from river bed = 0.3m, velocity coefficient of orifice = 0.8, velocity through orifice = 1.2m/s, orifice height =0.2m, height of orifice sill from bottom of the canal=0.2m, downstream submergence depth=0.05m.

Solution: From question given as following data, (Q) = $0.077m^3/s$, (K_t) = 2.4, (t) = 4mm, (a) = 25mm, (Va) = 0.5m/s, (Ø) = 60^0, (α) = 20^0, (h_t) = 0.3m, (C) = 0.8, (V_O) = 1.2m/s, (H) = 0.2m, (h_{os}) = 0.2m, $(h_s)_{D/S}$ = 0.05m

- **Calculation of head loss**
- (i)$h_{friction} = K_t * \left(\frac{t}{a}\right)^{\frac{4}{3}} * \left(\frac{V_a^2}{2g}\right) \sin Ø$
- $= 2.4 * \left(\frac{4}{25}\right)^{\frac{4}{3}} * \left(\frac{0.5^2}{2*9.81}\right) \sin 60°$
- $= \mathbf{0.0023m}$
- (ii)$h_{bend} = \left(\frac{V_a^2}{2g}\right) \sin \alpha$
- $= \left(\frac{0.5^2}{2*9.81}\right) \sin 20°$
- $= \mathbf{0.0044m}$
- Total head loss (h_L)
- = 0.0023 + 0.0044
- **= 0.0067m**
- ∴ Head loss coefficient(K) $= \frac{h_L*2g}{V_a^2}$
- $= \frac{0.0067*2*9.81}{0.5^2} = \mathbf{0.526}$



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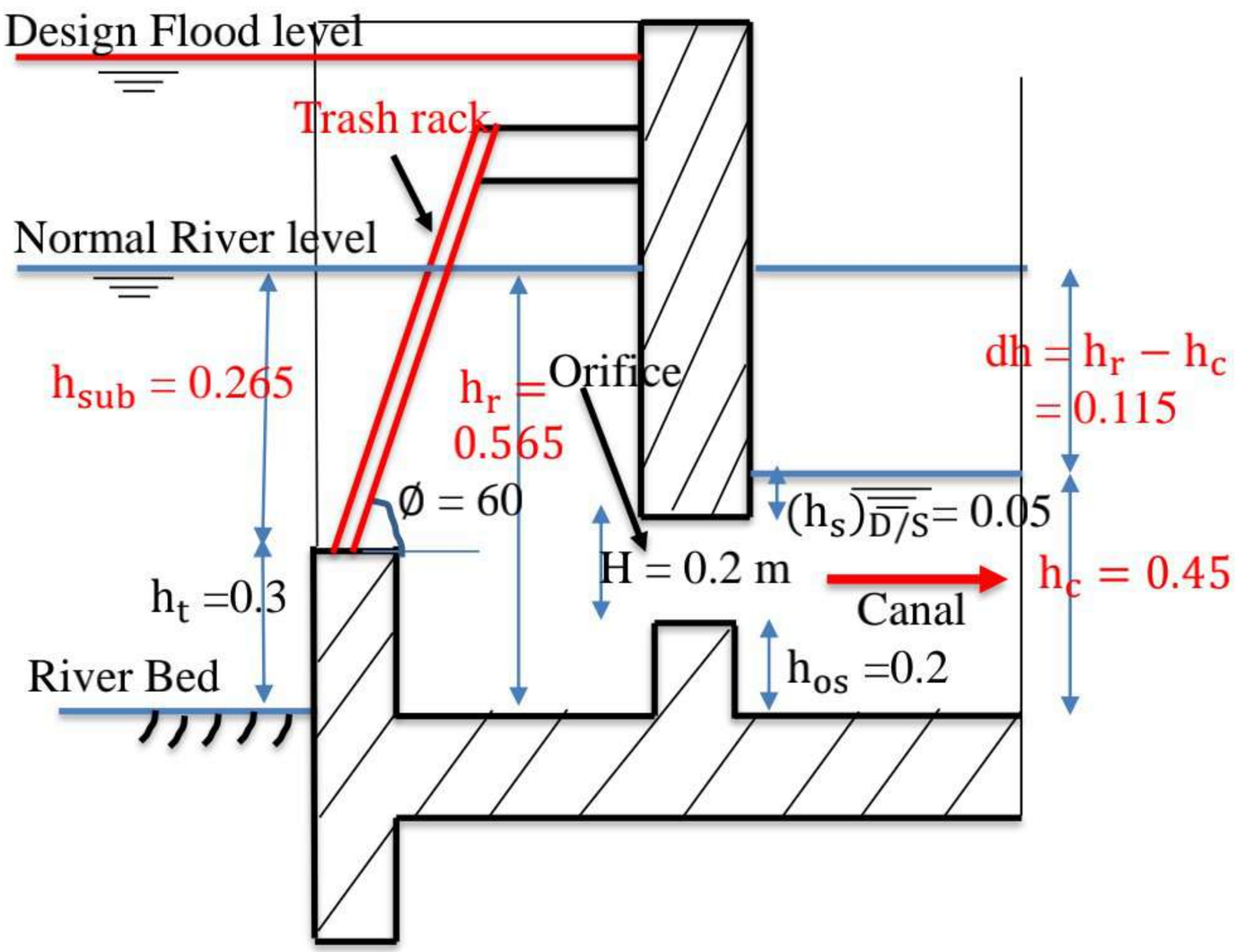
- **Calculation of total area of the submerged part of the screen(A_S)**
- $A_S = \frac{1}{K_1} * \left(\frac{t+a}{a}\right) * \frac{Q_d}{V_a} * \frac{1}{\sin \emptyset}$ Here K_1 for hydropower in Nepal is equal to 0.55
- $\therefore A_S = \frac{1}{0.55} * \left(\frac{4+25}{25}\right) * \frac{0.077}{0.5} * \frac{1}{\sin 60°} = \mathbf{0.375m^2}$
- **Calculation of width of trash rack (B);**
- We Know,
- Width $(B) = \frac{A_S * \sin \emptyset}{h_{submerged}}$ …… (i)
- where $(h_{submerged}) = h_r - h_t$
- and $h_r = h_c + dh$
- Also, $h_c = (h_{os}) + H + (h_s)_{D/S}$
- $= 0.2 + 0.2 + 0.05 = \mathbf{0.45m}$
- $(dh) = \frac{\left(\frac{V_o}{c}\right)^2}{2g} = \frac{\left(\frac{1.2}{0.8}\right)^2}{2*9.81} = \mathbf{0.115m}$
- $\therefore h_r = 0.45 + 0.115 = \mathbf{0.565m}$
- $\therefore (h_{submerged}) = 0.565 - 0.3$
- $= \mathbf{0.265m}$
- Now equation (i)becomes,
- Width $(B) = \frac{0.375 * \sin 60°}{0.265}$
- $= 1.225$ m
- $\mathbf{i.e. B = 1.225\ m}$



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B. Orifice design Step for side intake:

Given Parameter for Orifice

Velocity coefficient (C), velocity through orifice (V_o), Design discharge (Q_d), Manning's coefficient of roughness (n), Downstream submergence depth $(h_s)_{D/S}$, Orifice Height (H), height of orifice from canal bed (h_{os})

Given Parameter for Canal and Spillway

Spillway crest height above NWL (H_S), Spillway discharge coefficient (C_d), Free board in canal (h_{FB}), Slope of canal immediately D/S of orifice (S)

Given Parameter River

Crest Length (L), Flood discharge (Q_F)

To Find

- Depth of canal $(h_C) = \{(h_s)_{D/S} + H + h_{os}\}$
- Driving head $(dh) = \frac{\left(\frac{V_o}{C}\right)^2}{2g}$
- Head at river $(h_r) = h_C + dh$
- Height of weir $(h_w) = h_r + 0.1$

To Find (Orifice)

- Orifice area (A) $= \frac{Q_d}{V_o}$ and Orifice Width $(B_o) = \frac{A}{H}$

To find Circular references (Flood)

- Critical depth crest $(y_{cf}) = \left\{\frac{\left(\frac{Q_f}{L}\right)^2}{g}\right\}^{1/3}$
- Flood head at river $(h_{rf}) = h_w + y_{cf}$

To find Canal

- Head differences $(dh_f) = h_{rf} - h_{cf}$
- Discharge at intake due to flood $(Q_{If}) = AC\sqrt{2g(dh_f)}$
- Depth of water at canal due to flood $(h_{Cf}) =$
- $\left\{\frac{(Q_{If} * n * \sqrt{S})^3}{2}\right\}^{1/8}$
- Velocity through orifice $(V_{of}) = C\sqrt{2g(dh_f)}$

For Spillway

- Spillway overtopping height(h_{oh}) = 50 % {Free board – spillway crest above NWL (H_S)}
- Spillway length $(L_{S1}) = \frac{Q_{If}}{C_d * (h_{oh})^{1.5}}$
- Spillway length $(L_{S2}) = \left\{\frac{Q_{If} - Q_d}{C_d * (h_{oh})^{1.5}}\right\} * 2$
- ∴ Design length of spillway = Maximum value of (L_{S1}) and (L_{S2})

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Q. N. Determine the dimension of orifice, spillway design length, depth of water at canal due to flood, velocity through orifice due to flood and spillway overtopping height of the given data; hydropower of discharge 0.077 m^3/s. Velocity coefficient of orifice (C) = 0.8, velocity through orifice (Vo) = 1.2 m/s, manning's coefficient (n) = 0.02, D/S submergence depth $(h_s)_{D/S}$ = 0.05 m, orifice height (H) = 0.2 m, height of orifice from canal bed (h_{os}) = 0.2m,

1. At Canal and spillway: Spillway height above NWL = 0.05m, Spillway discharge coefficient (C_d) = 1.6, freeboard in Canal (h_{Fb}) = 0.3 m, Slope of canal immediately D/S of orifice (S) = 1865

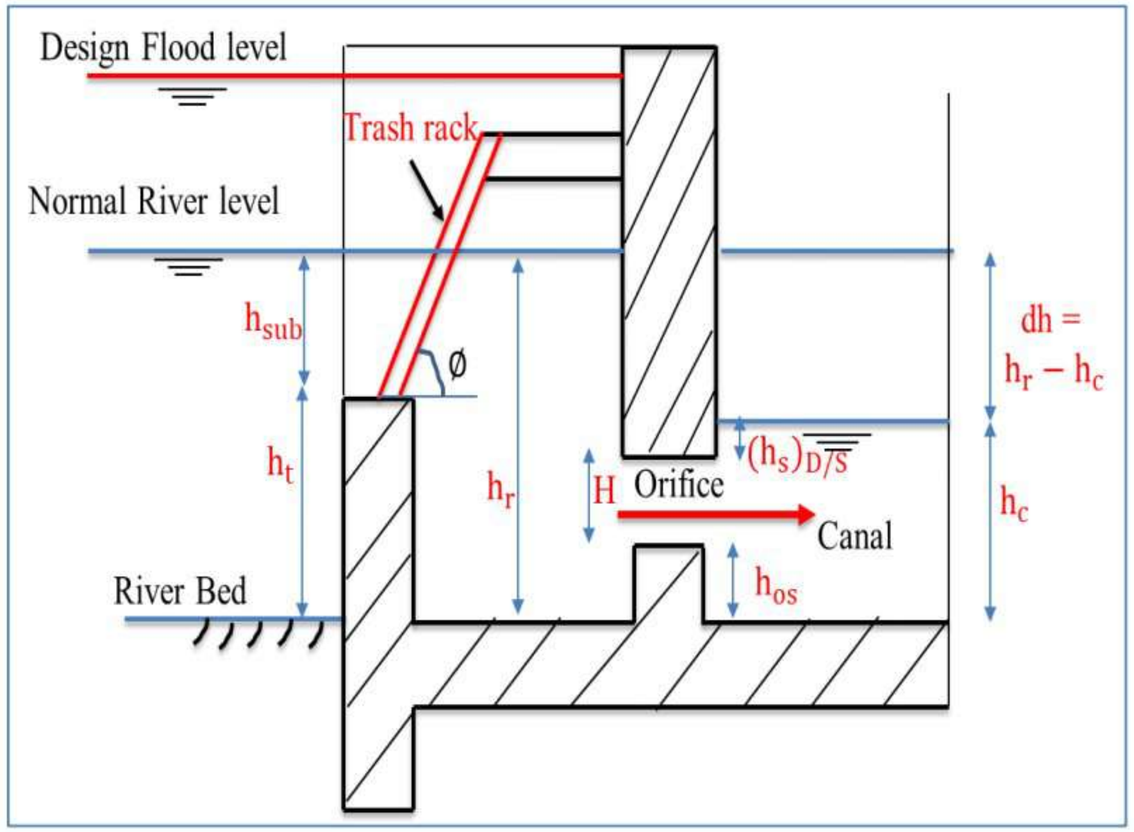
2. At River: Crest length (L) = 5m, Flood discharge (Q_F) = 10 m^3/s.

Solution: Given data

For Orifice: (V_o) = 1.2 m/s, n = 0.02, C = 0.8, (Q_d) = 0.077 m^3/s $(h_s)_{D/S}$ = 0.05m, H = 0.2m, (h_{os}) = 0.2m

Depth of canal (h_c) = $(h_{os}) + H + (h_s)_{D/S}$ = 0.05 + 0.2 + 0.2 = **0.45 m**

Driving head (dh) $= \frac{\left(\frac{V_o}{c}\right)^2}{2g} = \frac{\left(\frac{1.2}{0.8}\right)^2}{2*9.81} = \mathbf{0.115m}$



- ∴ head at river (h_r) = $0.45 + 0.115 = 0.565$m
- ∴ height of weir (h_w) = $(h_r) + 0.1 = 0.565 + 0.1$
- = 0.665 m
- Area of Orifice $(A) = \frac{Q_d}{V_o} = 0.077/1.2 = \mathbf{0.0642\ m^2}$
- Orifice width (B) $= \frac{A}{H} = \frac{0.0642}{0.2} = \mathbf{0.321\ m}$
- **At Canal and Spillway**
- Spillway height above NWL (H_s) = 0.05 m, S = 1865
- (C_d) = 1.6, (h_{Fb}) = 0.3 m,

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- **At River:**
- (L) = 5m, (Q_F) = 10 m^3/s.
- Critical depth at crest due to Flood $(y_{cf}) = \left\{\frac{\left(\frac{Q_f}{L}\right)^2}{g}\right\}^{1/3} = \left\{\frac{\left(\frac{10}{5}\right)^2}{9.81}\right\}^{1/3} = \mathbf{0.742\ m}$
- Flood head at river (h_{rf}) = $h_w + y_{cf}$ = 0.665 + 0.742 = **1.407 m**

Calculation of (h_{cf}), dh_f and Q_{IF} in tabular form by iterative method:

Trial Value of (h_{cf})	(dh_f) = $h_{rf} - h_{cf}$	(Q_{If}) = **AC** $\sqrt{2g(dh_f)}$	(h_{Cf}) = $\left\{\frac{(Q_{If} * n * \sqrt{S})^3}{2}\right\}^{1/8}$
h_{cf} = 0.6	0.807	0.204	0.4785
h_{cf} = 0.4785	0.9285	0.219	0.491
h_{cf} = 0.491	0.916	0.2177	0.490
h_{cf} = 0.49	0.917	0.218	0.490

∴ depth of water at canal de to flood (h_{cf}) = 0.490 m

(dh_f) = 0.917 m and Q_{If} = 0.218 m^3/s

Velocity through orifice $(\mathbf{V_{of}}) = \mathbf{C}\sqrt{\mathbf{2g(dh_f)}}$

$= 0.8\sqrt{2 * 9.81 * 0.917}$ = **3.393 m/s**

OR (V_{of}) = Q_{If} /A = 0.218 / 0.0642 = **3.393 m/s**

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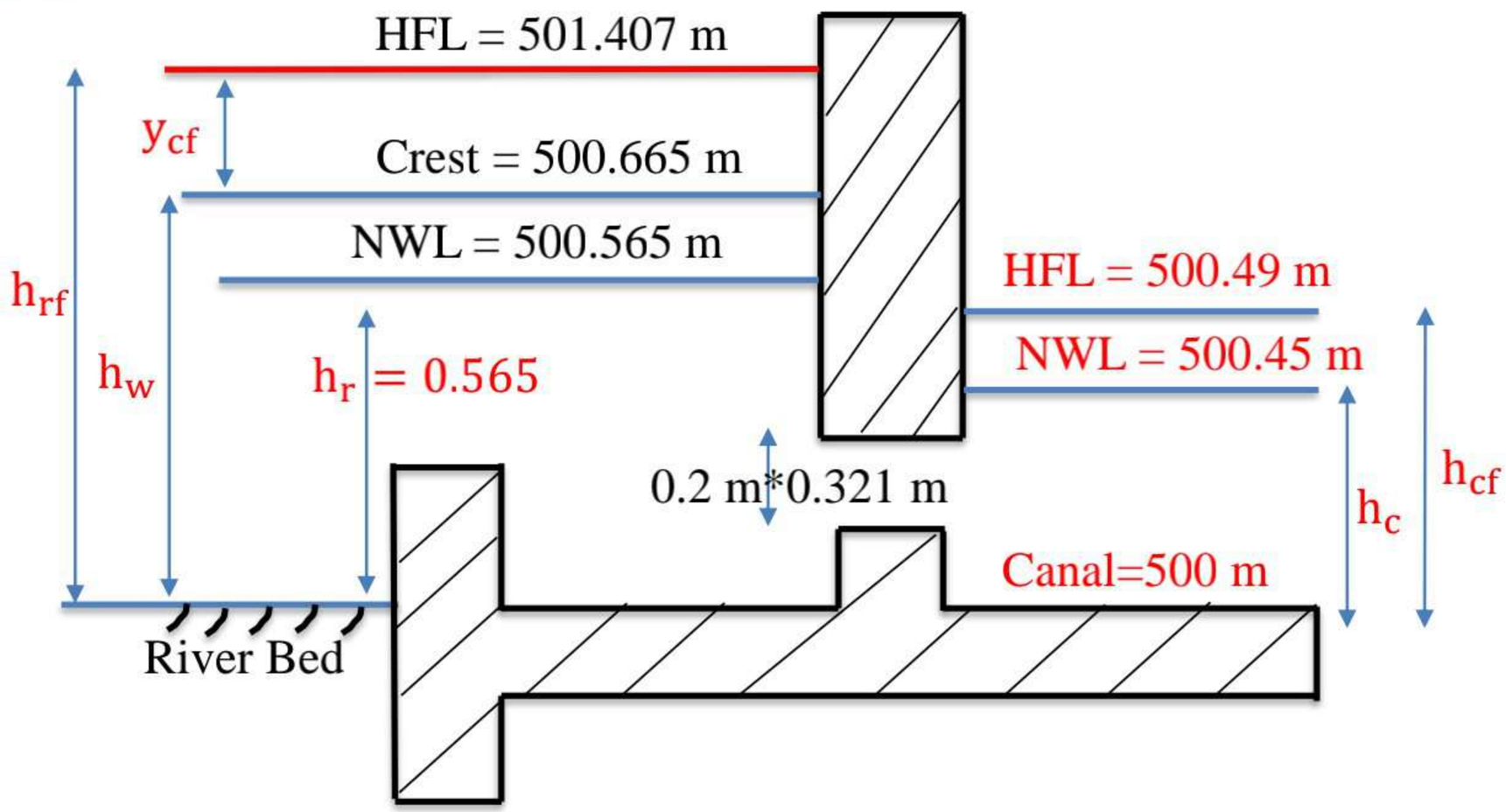
At Spillway

Spillway overtopping height(h_{of}) = 50% {Free board – spillway crest above NWL (H_S)}
= 0.5(0.3 – 0.05) = 0.125m

- Spillway length (L_{S1}) = $\frac{Q_{If}}{C_d*(h_{oh})^{1.5}} = \frac{0.218}{1.6*(0.125)^{1.5}}$ = 3.083 m
- Spillway length (L_{S2}) = $\left\{\frac{Q_{If}-Q_d}{C_d*(h_{oh})^{1.5}}\right\}*2 = \left\{\frac{0.218-0.077}{1.6*(0.125)^{1.5}}\right\}*2$ = 3.988 m
- ∴ Design length of spillway = Maximum value of (L_{S1}) and (L_{S2}) = **3.988 m**
- **Wall Geometry**
- Let canal bed level = 500 m



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Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

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Spillway:

- Spillway is a structure, use to pass surplus flood water from reservoir to the downstream side of the dam.
- It is sometimes also know surplussing work, as its main job is to handle the surplus water.
- It is a sort of safety valve for a dam.
- During floods, the reservoir level goes on increasing, as more and more water enters the reservoir.
- If spillways had not been there, the level in the reservoir will go on rising even beyond the maximum reservoir level. This may lead to over topping of the dams.
- Additional stresses may be induced in the dam and this may cause failure of the dam.
- More rise in water level beyond the maximum flood level will cause submergence of vast additional areas and may cause suffering to the people living on the U/S side.
- Spillways act as safety valve for the safety of the dam and also for the adjoining areas.
- When water level in the reservoir reaches some specified level, the surplus water automatically starts escaping through the spillways.
- As more and more flood water enters the reservoir, there is more and more rise in water level and consequently more and more water starts escaping over the spillways.
- The spillways should be of sufficient capacity so as to be capable to effectively handle the largest floods.
- Spillways are also design according to the safe capacity of the D/S channel, otherwise D/S areas may get flooded.

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Essential requirements of the spillway

- Designed such that only the safe discharge is passed to downstream in case of maximum flood flow.
- Hydraulically (less flow separation, smooth flow at entrance) and structurally (stable and strong for different possible loading condition) efficiently designed.
- Sufficient erosion and cavitation resistance capacity of surface to resist high flow velocity.
- Provision of appropriate dissipation of high kinetic energy possessed by flow at D/S; otherwise there will be severing bed erosion.

Location and selection of the spillway:

- A spillway can be locate either within the body of the dam or at one end of it or entirely away from it, independently in a saddle.
- In determining the best combination of storage capacity and spillway capacity to accommodate the selected inflow design flood, all pertinent factors of hydrology, hydraulics, design, cost and damage should be considered.
- In this connection and when applicable, *consideration should be given to such factors as*

i. The characteristics of the flood hydrograph.

ii. The damages that would result if such a flood occurred without the dam

iii. The damages that would occurs if the dam or spillway were breached

iv. The relative costs of increasing the capacity of spillways

v. The effect of various dam and spillway combinations on the probable damages U/S and D/S of the dam (as indicated by reservoir backwater curves and tail water curves)

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Classification of the Spillway

- **Classification according to control of spilling discharge**

Spillways are ordinarily according to their most prominent feature, either as it pertains to the control, to the discharge channel, or to some other feature.

1. Controlled spillways
2. Uncontrolled spillways

- If the flow of water over a spillway is controlled by installing gates over a spillway crest, then the spillway is known as **controlled** spillway. But in **uncontrolled** spillway, spillways are left just by constructing their crest at normal pool level.
- As water over such a spillway, depending upon the reservoir level and the corresponding head over the spillway, such **uncontrolled** spillways are guided only by the available water head, and hence, are called uncontrolled spillway.
- **Classification according to function**

According to the function, spillway may be classified as

1. Main spillway

Main spillway is designed for the safe passage of maximum flood.

2. Auxiliary spillway

- Auxiliary spillway is the supplementary spillway to the main spillway.
- In this case, the main spillway is designed for the capacity less than the passage of the designed flood for the shake of economy of the construction.
- If there is flood that exceeding the capacity of the main spillway, water level will rise and leads towards the overtopping.

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- **Classification according to function**

2. Auxiliary spillway…

- In this case the auxiliary spillway will come in function. It is favorable to provide the auxiliary spillway when there is natural saddle existing.
- The total discharge form the main and auxiliary spillway is equal to the maximum safe discharge to downstream in case of occurrence of design maximum flood discharge without overtopping of the dam.

3. Emergency spillway

- The emergency situations arise when incoming discharge exceeds the maximum flood discharge or when the gate of the spillway failures in case of flood.
- In such case the water level will exceed beyond the maximum water level and there will be danger of overtopping.
- In such case, emergency spillway provided will come into action.
- The crest level of emergency spillway is higher than the normal water level (NWL).
- Emergency spillway is the fuse plug type and it comes in action as soon as the water level reaches to some alert water level.

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Classification according to prominent features

Commonly referred to types are

1. Free overfall (straight drop) spillway
2. Ogee (Overflow) spillway.
3. Chute (open channel or trough) spillway
4. Side channel spillway.
5. Drop inlet (shaft or morning glory) spillways
6. Conduit and tunnel spillways
7. Culvert spillways
8. Syphon spillway
9. Saddle spillway

1. Free overfall (straight drop) spillway

- A free overfall, or straight drop, spillway is one in which the flow drops freely form the crest. This type is suited to a thin arch or to a crest that has a nearly vertical downstream face.
- Flows may be free discharging, as is the case with a sharp – crested weir control, or they may be supported along a narrow section of the crest.
- Occasionally, the crest extended in the form of an overhanging lip to direct small discharges away form the face of the overfall section. In free overfall spillways the underside of the nappe is ventilated sufficiently to prevent a pulsating, fluctuating jet.
- This type of structure is not adaptable for high drops on unstable foundations because of the large impact forces that must be absorbed by the apron at the point of impingement of the jet.

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Classification according to prominent features….

1. Free overfall (straight drop) spillway…

- The impact caused by high drops might crack or displace the structure, with danger failure by piping or undermining.
- Ordinarily, the use of this structure for hydraulic drops from head pool to tail water of 6m or more should not be considered.

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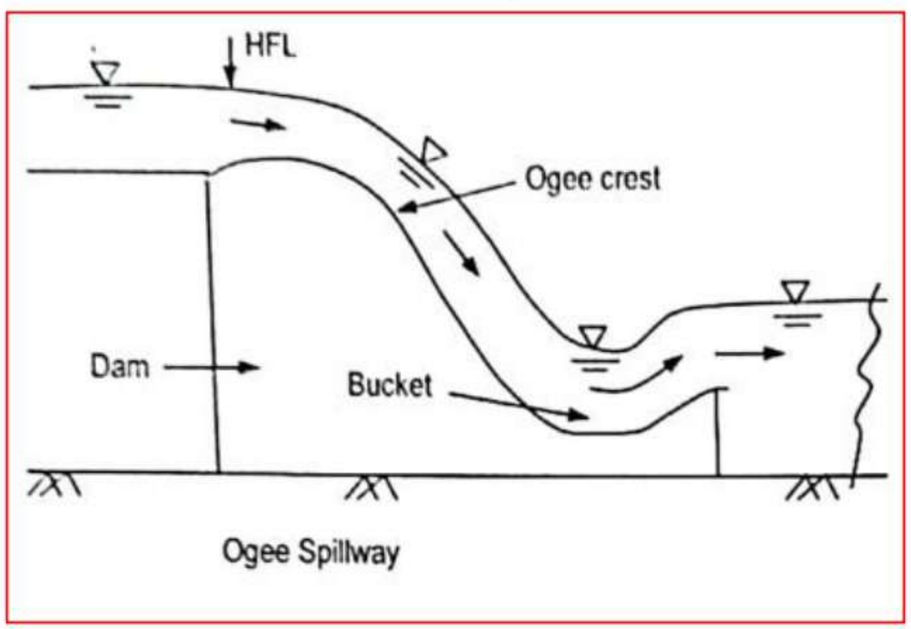
Classification according to prominent features……

2. Ogee (Overflow) spillway.

- Ogee spillway is modified type of vertical drop spillway which has a control weir that is ogee – shaped (S shaped) in profile.
- The upper curve of the ogee spillway ordinary conforms closely to the profile of the lower nappe of a ventilated sheet falling from a sharp – crested weir.
- Flow over the crest adheres to the face of the profile by preventing access of air to the underside of the sheet.
- For discharges at designed head, the flow glides over the crest with no interference from the boundary surface and attains near – maximum discharge efficiency.
- Ogee spillway is generally used in high solid gravity dam.
- This spillway can also be used in arch and buttress dam.
- Such spillway can easily be used on valleys where width of the river is sufficiently more to provide sufficient crest length, and river bed can be protected form scouring at moderate cost.



Ogee Spillway

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Classification according to prominent features…..

2. Ogee (Overflow) spillway……

- Maximum flood discharge can safely be disposed of D/S to the stream.
- In most of the high dams with wide crest, width provides this type of spillway.
- If the discharge over the spillway is more than the flood nappe may separate from the body profiles creating cavitation which is undesirable for the dam.
- The discharge may be calculated by the equation
- $Q = CL\,H^{3/2}$
- Where, C = coefficient of discharge, L = length of the spillway crest and H is the static head.
- C value is normally ranges between 1.66 for board crested weir to 2.2 for a weir designed with the optimum profile.
- The discharge coefficient is influenced by a number of factors, such as the depth of approach, relation of the actual crest shape to the ideal nappe shape, upstream face slope, downstream apron interference, and downstream submergence.

Shape for uncontrolled ogee crest

- Discharge (Design flood)
- $Q = C_0 L\,H_0^{3/2}$
- Approach velocity $(V_t) = \frac{Q}{L*(P+h_0)}$
- Velocity head $(h_t) = \frac{Q^2}{L^2*2g*(P+h_0)^2}$
- $= \frac{V_t^2}{2g}$

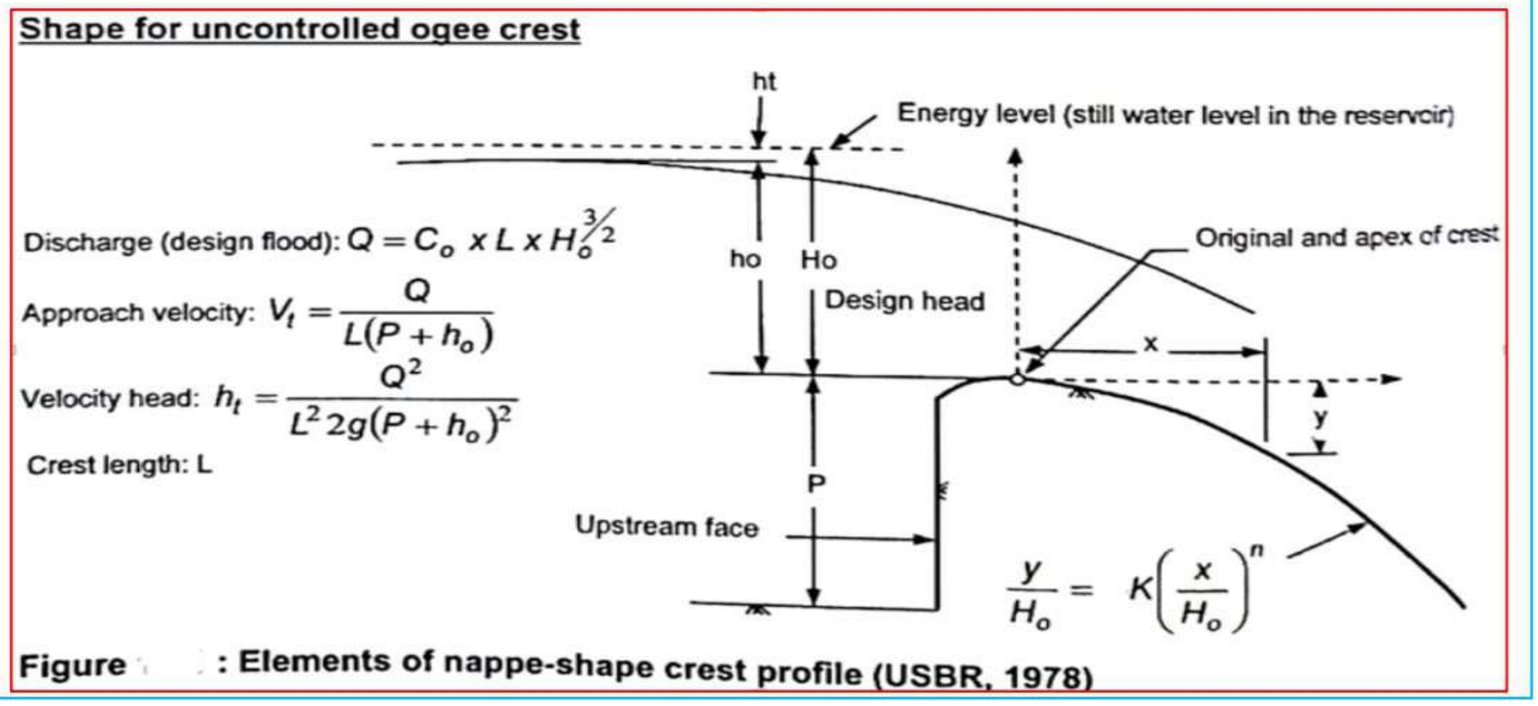

Figure : Elements of nappe-shape crest profile (USBR, 1978)

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Classification according to prominent features…..

2. Ogee (Overflow) spillway…

- The portion of the crest upstream form the crest apex shall be defined as a single curve and a tangent or as a compound circular curve. Likewise, its downstream portion shall be defined by the equation (USBR, 1978)
- $\frac{y}{H_d} = K\left(\frac{x}{H_d}\right)^n$ {For vertical upstream face; K = 0.5 and n = 1.85; ∴ $X^{1.85} = 2YH_d^{0.85}$ }
- Where, H_d is the designed head on the crest during design flood, K is a constant whose value depends on the velocity of approach, and n is a constant whose value depends on the upstream inclination and on the velocity of approach.

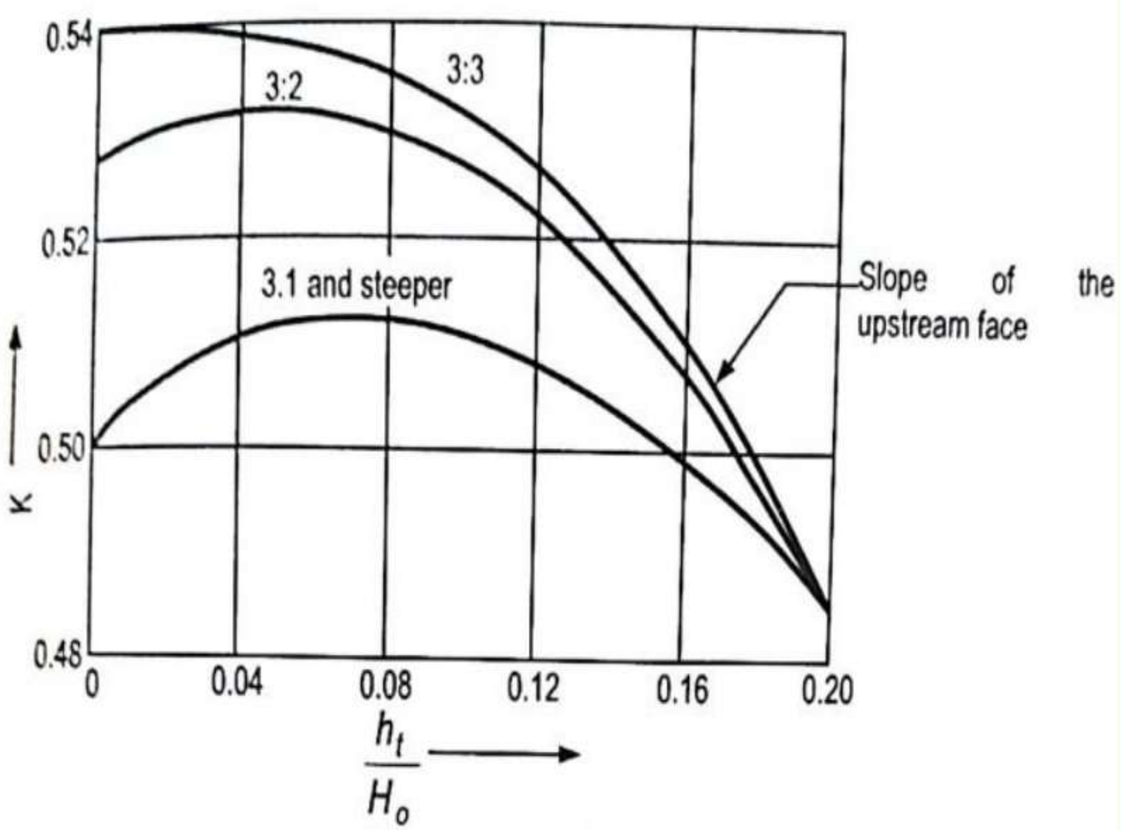

Figure : Values of *K* for definition of crest profile (USBR, 1978)

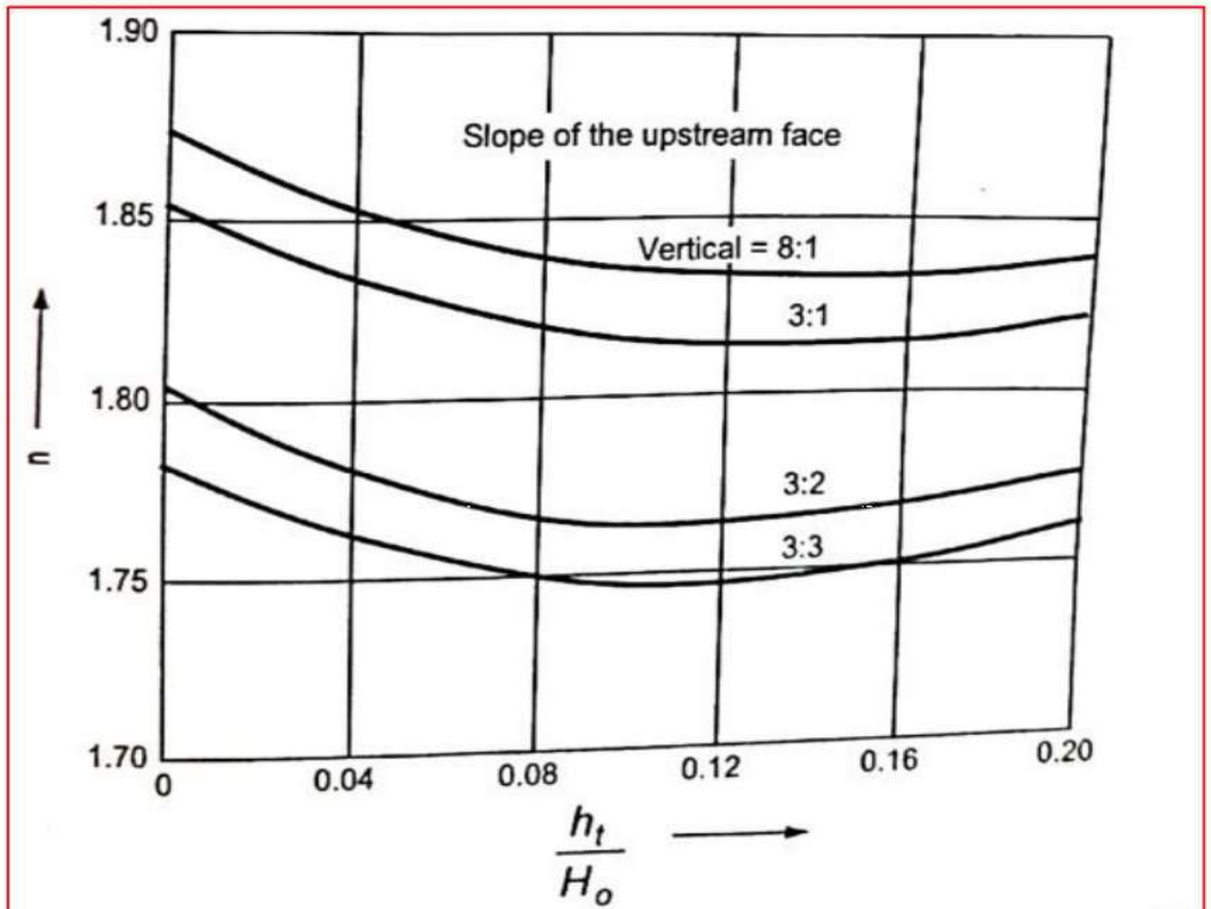

Figure : Values of *n* for definition of crest profile (USBR, 1978)

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Classification according to prominent features…..

2. Ogee (Overflow) spillway…

- **Effective length of crest**
- The value of effective length, L shall take into account the side contractions of the overflow due to crest piers and abutments. Accordingly, L shall be calculated using the relation (USBR, 1978)
- $L_e = L_0 - 2(NK_P + K_a)H_e$
- Where L_0 is the net length of the weir crest, N is the number of piers, K_P is the pier contraction coefficient, K_a is the abutment contraction coefficient and is the total head on the crest, including velocity of approach, head in m.
- Coefficient of K_P and K_a shall be taken form table shown in below where r is the radius of the abutment.

Pier shape	K_P
Square – nosed pier with corners round on a radius equal to about 0.1 of the pier thickness	0.02
Round – nosed piers and 90° cut water nosed piers	0.01
Pointed – nosed piers	0

Abutment shape	K_a
Square abutment with headwall at 90° to the direction of flow	0.2
Round abutments with headwall at 90° to the direction of flow, when $0.5H_o \geq r \geq 1.5H_o$	0.1
Round abutments where $r > 0.5H_o$ and headwall is placed not more then 45° to the direction of flow	0

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Head of water over the crest of ogee spillway is 3m and coefficient of discharge is 2.5. weir is 100m long and height of the crest above the base of the approach channel is 10m. Width of approach channel is equal to the length of the weir. Find out the discharge passing over the spillway.

Solution: From question given as; C = 2.5, H_0 = 3m, P = 10, L = 100m

- If effect of approach velocity and end contractions has been neglected.
- $Q = CL\,H_0^{3/2}$
- $= 2.5*100*(3)^{3/2} = \mathbf{1299.04\ m^3/sec}$
- But Approach velocity

$(V_a) = \frac{Q}{L*(P + h_0)}$

$= \frac{1299.04}{100*(10+3)}$

$= 0.9993 \cong \mathbf{1\ m/sec}$

- Velocity head $(H_a) = \frac{V_a^2}{2g}$

$= \frac{1^2}{2*9.81} = 0.05$ m

- ∴ Effective Height $(H_e) = H_0 + H_a$

$= 3 + 0.05 = \mathbf{3.05\ m}$

- ∴ Modified discharge (Q) $= CL\,H_e^{3/2}$

$= 2.5*100*(3.05)^{3/2}$

$\mathbf{= 1331.65\ m^3/sec}$

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Design a suitable section for the overflow spillway portion of concrete gravity dam having downstream face sloping at a slope of 0.7H : 1V. The design discharge for the spillway is 8000 m^3/sec. The height of the spillway crest is kept at RL 204m. The average river bed level at the site is 100m. The spillway length consists of 6 spans having a clear width of 10m each. Thickness of each pier may be taken as 2.5m. Assume coefficient of discharge = 2.2.

Solution: From question given as; Q = 8000 m^3/sec, D/S face slope 0.7H : 1V, number of span = 6, number of pier (N) = 6 – 1 = 5, L_0 = 6*10 = 60 m, P = 204 – 100 = 104 m

- Effective length of spillway $(L_e) = L_0 - 2(NK_P + K_a)H_e$
- Assuming 90° cut water nose piers and rounded abutment $K_p = 0.01$, $K_a = 0.1$,
- $L_e = 60 - 2\{5 * 0.01) + 0.1\}H_e$
- $= \mathbf{60 - 0.3H_e}$
- **We know,** Discharge $(Q) = CL_e\ H_e^{3/2}$

$8000 = 2.2\{60 - 0.3H_e\} * (H_e)^{3/2}$

$8000 = \{132 - 0.66H_e\} * (H_e)^{3/2}$

By trial and error, H_e = 16.331 m

Hence, $L_e = 60 - 0.3(16.331)$

= 55.10m

- Also Approach velocity $(V_a) = \frac{Q}{L*(P + h_0)}$

$= \frac{8000}{\{60+(5*2.5)\}*(104 + 16.331)}$

= 0.917 m/sec

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- Velocity head $(H_a) = \frac{V_a^{\,2}}{2g}$

$= \frac{(0.917)^2}{2*9.81} = 0.043$ m

- ∴ Design head including velocity head $(H_d) = H_e + H_a$

$= 16.331 + 0.043 =$ **16.374 m**

- **We know** downstream portion shall be defined by the equation (USBR, 1978)
- $\frac{y}{H_d} = K\left(\frac{x}{H_d}\right)^n$ {For vertical upstream face; K = 0.5 and n = 1.85; ∴ $X^{1.85} = 2YH_d^{0.85}$ }
- $\therefore X^{1.85} = 2YH_d^{0.85}$
- $Y = \frac{X^{1.85}}{2H_d^{0.85}} = \frac{X^{1.85}}{2(16.374)^{0.85}} = \frac{X^{1.85}}{21.53}$....(1)
- To determine the tangent point
- Taking downstream slope of dam as 0.7 H : 1V
- $\frac{dy}{dx} = \frac{1}{0.7}$

Differentiating the above equation (1) with respect to X,

- $\frac{dy}{dx} = \frac{1.85\,X^{0.85}}{21.53}$
- Now, $\frac{1.85\,X^{0.85}}{21.53} = \frac{1}{0.7}$

∴ X = 27.302m, Y = 21.082m

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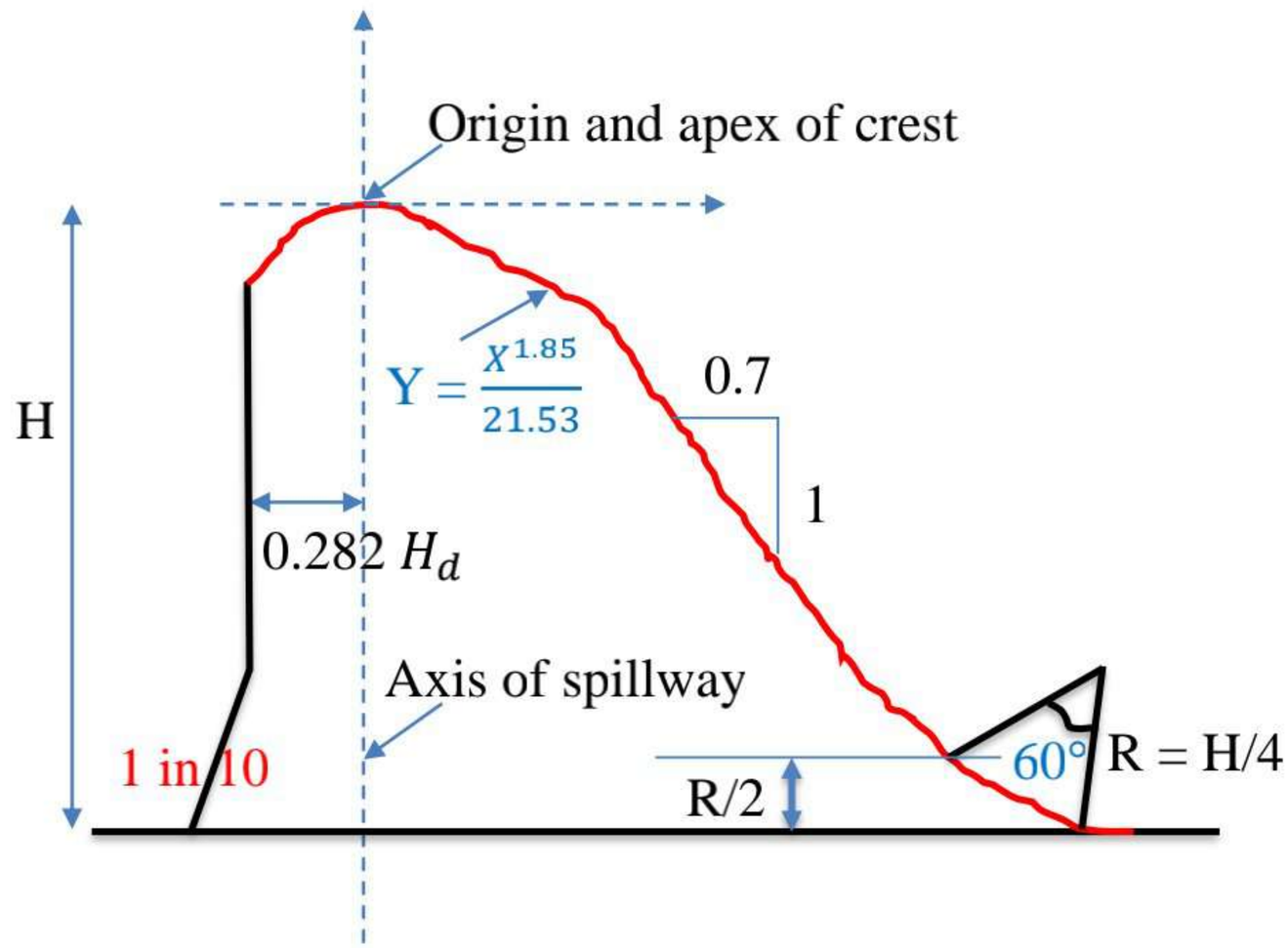
∴ X = 27.302m, Y = 21.082m

$Y = \frac{X^{1.85}}{21.53}$

X	**Y =** $\frac{X^{1.85}}{21.53}$
0	0
2	0.1674
4	0.6036
6	1.278
8	2.176
10	3.288
12	4.607
14	6.127
16	7.845
18	9.755
20	11.853
22	14.139
24	16.609
26	19.26
27.302	21.082



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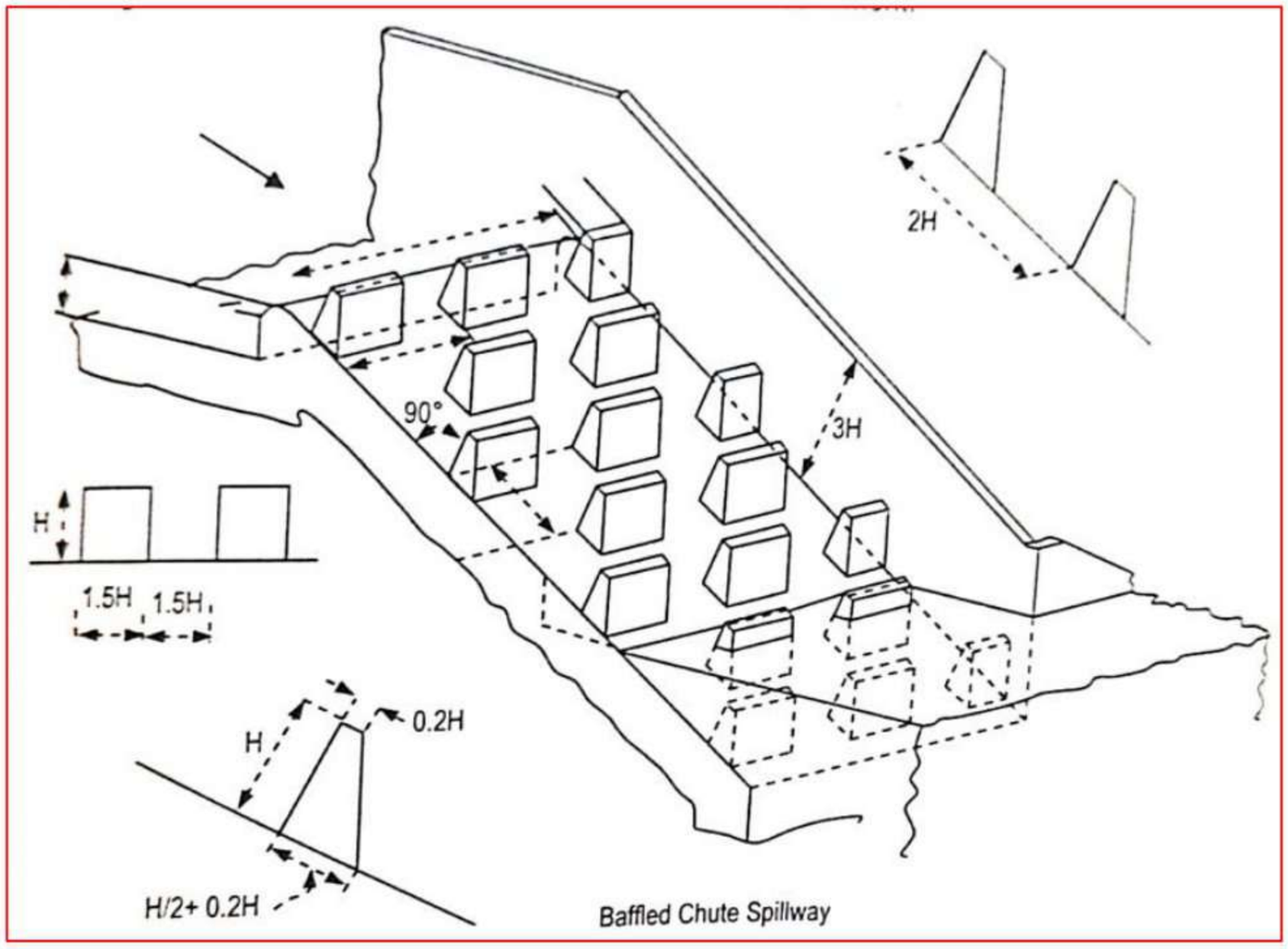
Classification according to prominent features…..

3. Chute (open channel or trough) spillway:

- A spillway whose discharge is conveyed from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle, might be called a chute, open channel or trough spillway.
- A chute spillway passes the surplus water normally through a steep channel placed in dam body or saddle away form the dam.
- A spillway having a chute – type discharge channel, through controlled by an overflow crest, a gated orifice, a side channel crest, or some other control device, may still be called a **chute spillway**.



Baffled Chute Spillway

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Classification according to prominent features…..

3. Chute (open channel or trough) spillway..

- Factors influencing the selection of chute spillways are the simplicity of their design and construction, their adaptability to almost any foundation condition.
- Chute spillway ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel.
- The simplest form of chute spillway has a straight center line and uniform width.
- Flows upstream form the crest are generally at subcritical velocity, with critical velocity occurring when the water passes over the control.
- Flows in the chute are ordinarily maintained at supercritical stage, either at constant or accelerating rates, until the terminal structure is reached.
- For good hydraulic performance, abrupt vertical changes or sharp convex or concave vertical curves in the chute profile should be avoided.

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Classification according to prominent features…..

- **4. Side channel Spillway:**
- Side channel spillway are mainly constructed when it is not possible to use a direct overfall spillway, e.g. at earth and rock fill dams.
- Suitable in a narrow valley where required crest length of overflow spillway is not available.
- Convenient where it is advantageous to leave central portion of stream for power house in a less wide stream.
- Are applicable to narrow gorges.
- Advantageous in the case of embankment dams.

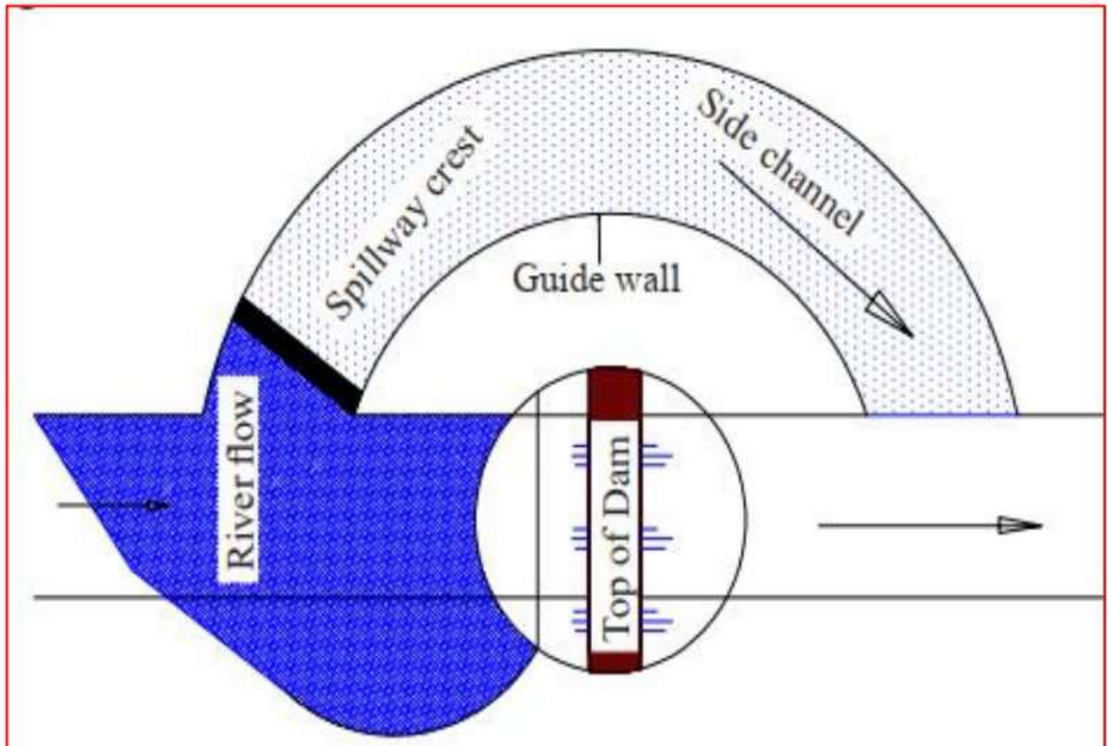
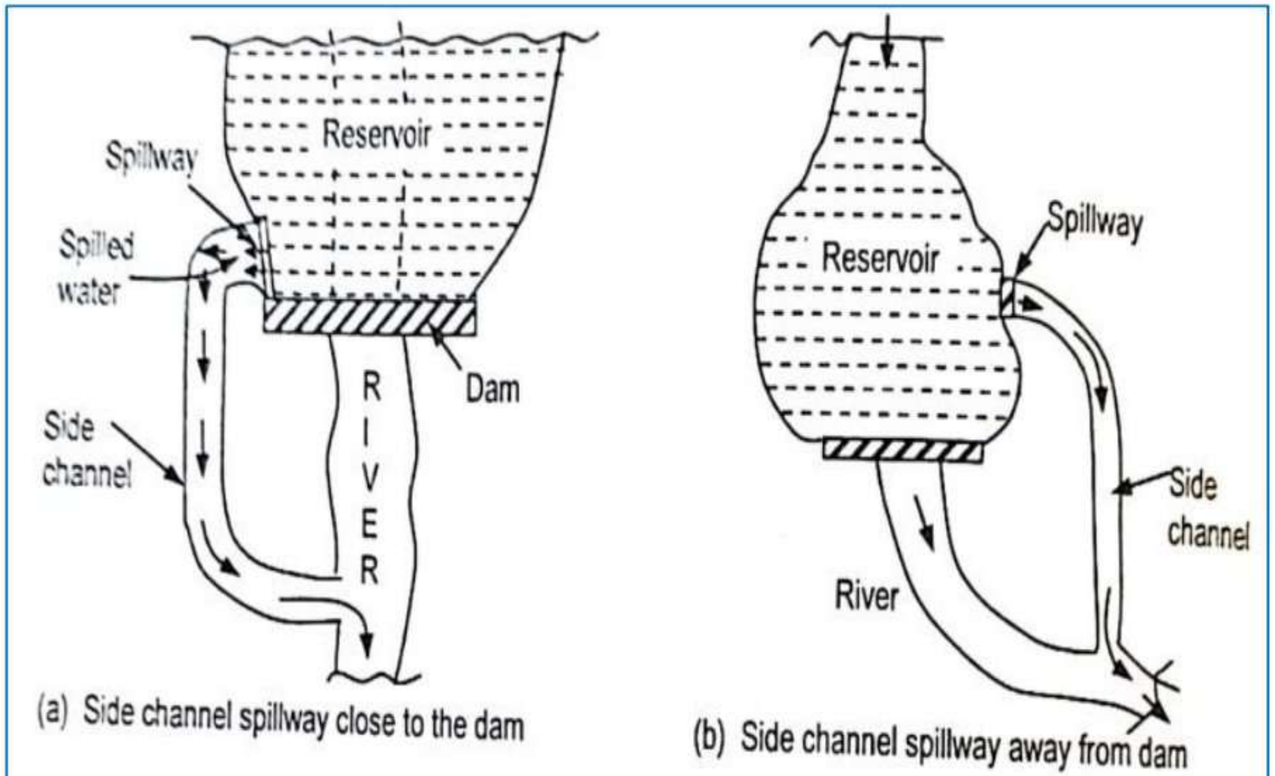


Fig Side Channel Spillway



Side Channel Spillway

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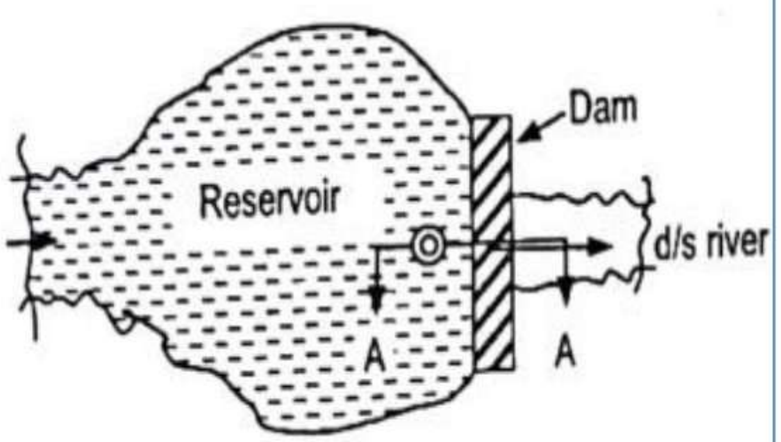
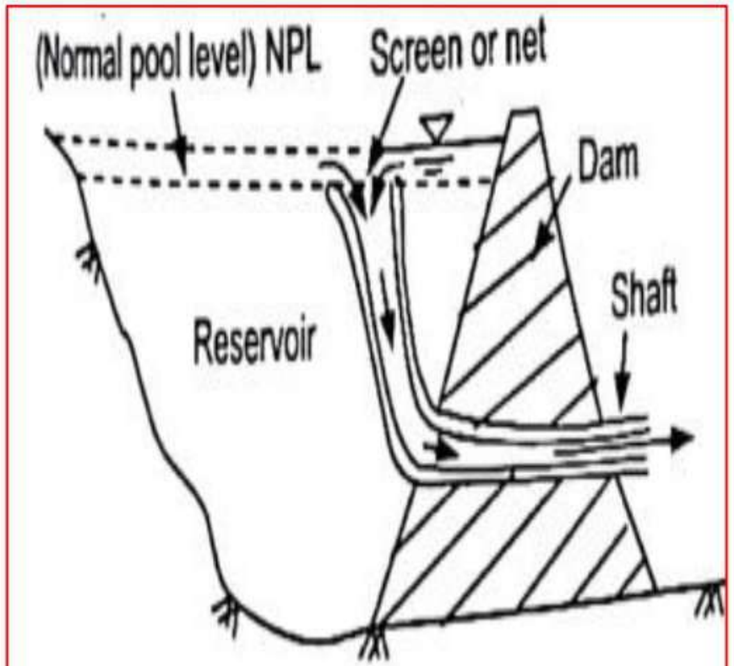
Classification according to prominent features…..

5. Drop inlet (shaft or morning glory) spillways:

- In this spillway water enters over a horizontal lip, drops through a vertical or sloping shaft and then flows to the D/S river channel through a horizontal or nearly horizontal conduit or tunnel.
- The structure is considered to comprise three elements; an overflow control weir, a vertical transition and a closed discharge channel.
- If the inlet is funnel – shaped, this type of structure is often called a "morning glory" or "glory hole" spillway.
- Top level of the vertical shaft is kept at Normal Pool Level (NPL).
- Excess water enters through the flared vertical shaft and discharge downstream in the river through the horizontal shaft. Sometimes the horizontal shaft passes through the body of the dam and sometimes by the side of the dam.



(a) Plan of shaft spillway through the body of dam



(b) Sectional View on AA

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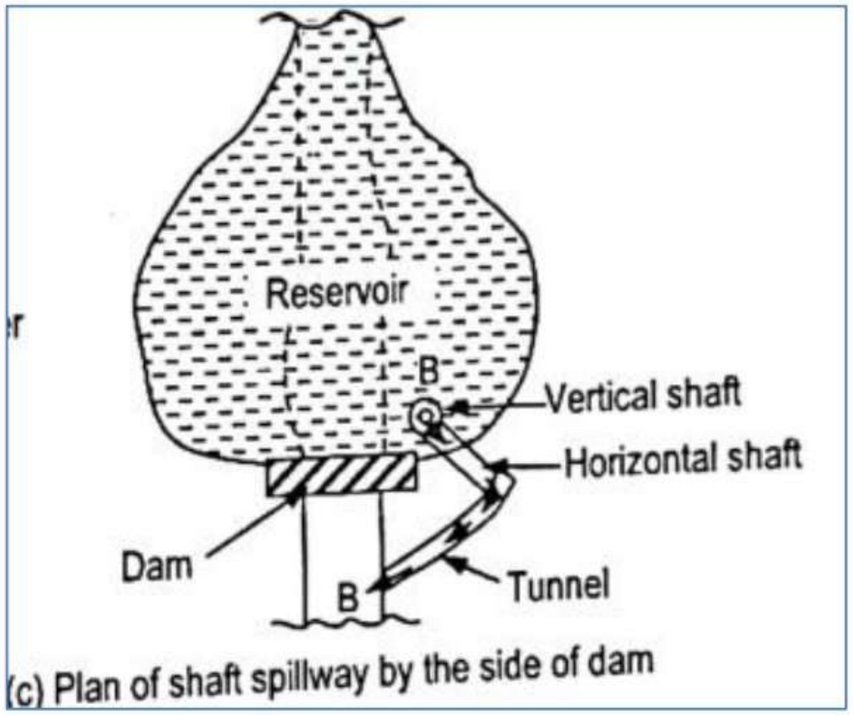
Classification according to prominent features…..

5. Drop inlet (shaft or morning glory) spillways:

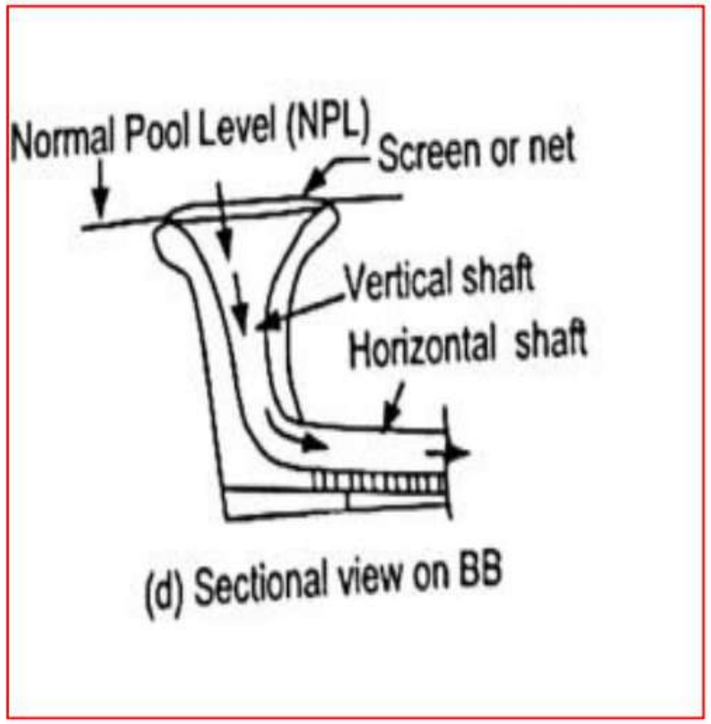
- The shaft spillway is suitable for :

i. In narrow gorges where other types of spillway cannot be constructed due to inadequate space.

ii. In earthen dams where channel side or chute channel spillway is unsuitable for space



(c) Plan of shaft spillway by the side of dam



(d) Sectional view on BB

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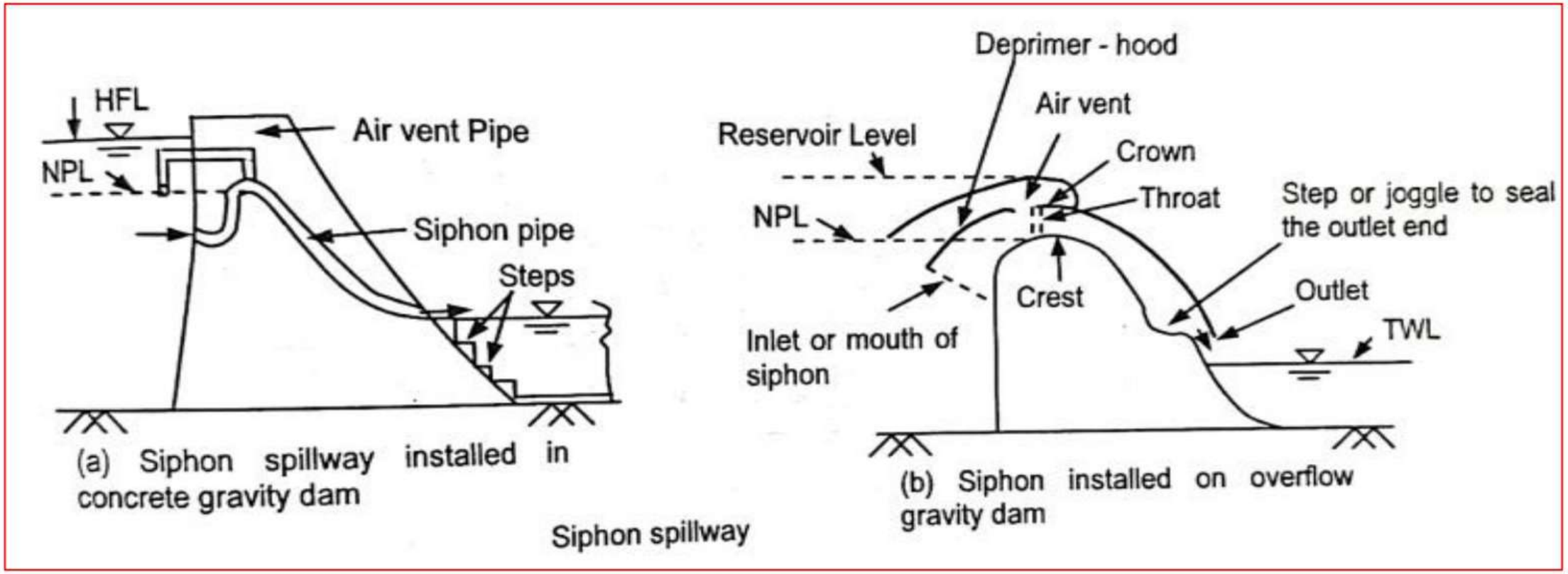
Classification according to prominent features…..

6. Siphon Spillway

- If the available space is limited and discharge is not large, siphon spillway is often superior to other forms.
- It is based on the siphonic action in the shape of an inverted pipe.
- The spillway must be vented properly to maintain the water level within normal water level.
- Without air vent provision, once the siphonic action started, the U/S water is draw down to the level of entrance before flow cases (rather than normal water level).
- Advantage of the siphon spillway

i. Occupies less space and regulates reservoir level within narrow limits.

ii. Useful in canal fore bay where surges created by change in turbines loads are to be equalized.



(a) Siphon spillway installed in concrete gravity dam

(b) Siphon installed on overflow gravity dam

Siphon spillway

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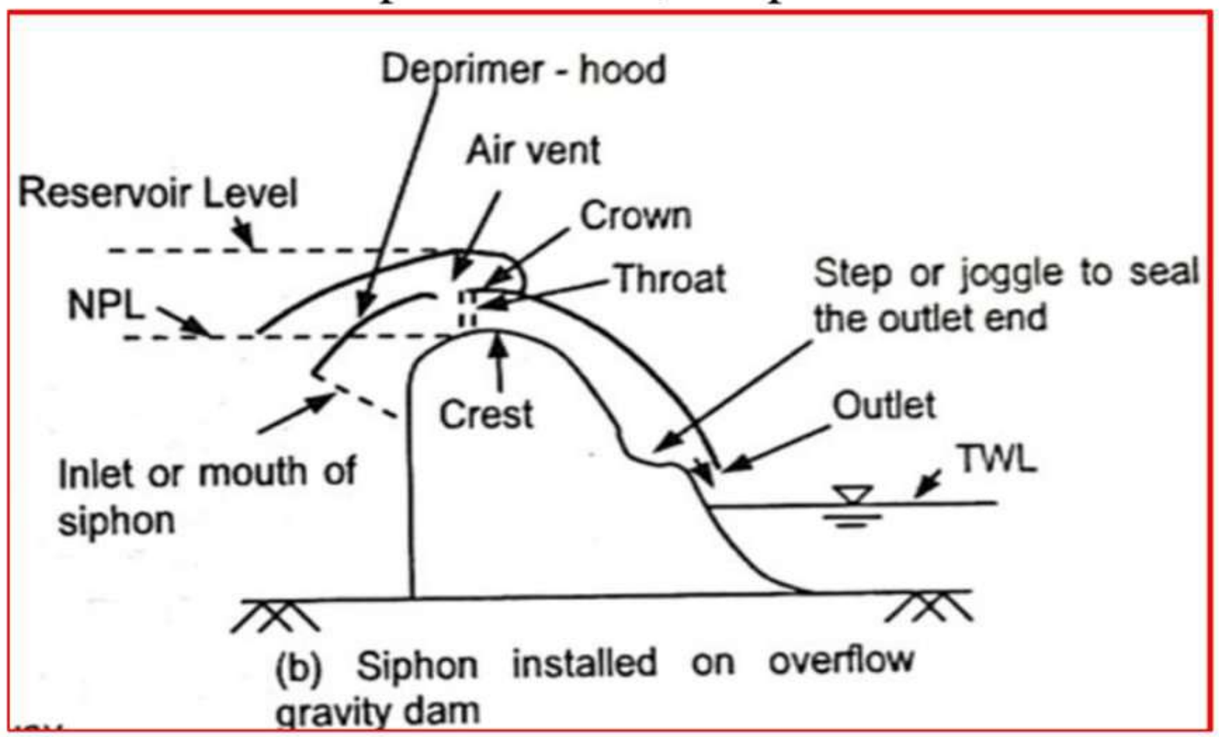
6. Siphon Spillway…..

Working principle of siphon spillway:

1. As soon as the water level reaches above NWL, it begins to spill from siphon spillway.
2. Water from both ends locks the duct of siphon when water rises just above deprimer hood.
3. Baby priming is developed due to the increase of rate of priming of siphon spillway.
4. There is gradual increase in vacuum (sucking) pressure inside duct as a result of air entrapped by the flowing water from crest of the spillway.
5. When the vacuum pressure increases, it sucks more water from reservoir and finally duct is filled with running water.
6. After development of complete vacuum (negative) pressure inside duct, siphon gets into full operation.
7. Period of time between the moment water just begins to spill from the crest to the moment siphon operates in full flow is termed as the priming.
8. With the decrease in water level reducing just to the inlet of the deprimer hood, air passes to the throat section, vacuum pressure thus released and air pressure developed inside the duct results into the state of siphon being stopped. In the lack of deprimer, there is loss of water as water level decreases just above inlet pipe.



(b) Siphon installed on overflow gravity dam

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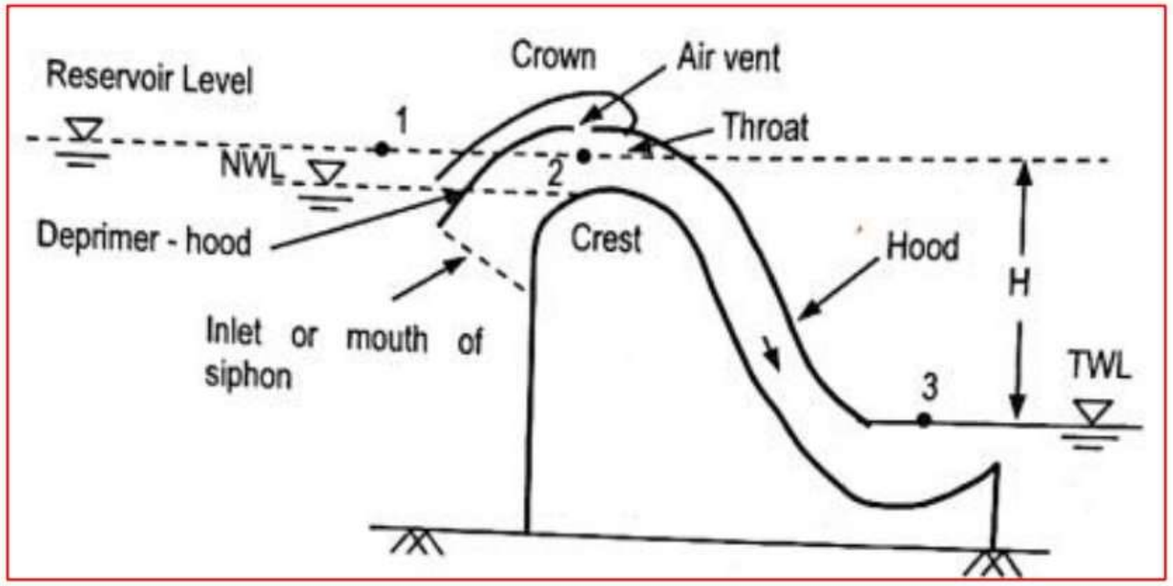
6. Siphon Spillway…..

Working principle of siphon spillway:

- When the water level of the reservoir decrease and reach just to inlet of the depriming hood, air will be passed to the throat section and negative pressure developed will be released and siphon will stop running. If the deprimer is not provided then water level will decrease just to the upper level of the inlet pipe which will cause the loss of water.
- For calculation of pressure at the Centre of the throat section, applying Bernoulli's equation between the point 1 and 3



$$\frac{V_1^2}{2g} + H + \frac{P_{atm}}{\gamma} = \frac{V_3^2}{2g} + H_L + \frac{P_{atm}}{\gamma}$$

Where H_L head loss between the point 1 and 3.

$$V_3 = \sqrt{2g(H - H_L)}$$

- Applying Bernoulli's equation between 2 and 3

$$\frac{V_2^2}{2g} + H + \frac{P_2}{\gamma} = \frac{V_3^2}{2g} + h_L + \frac{P_{atm}}{\gamma}$$

$$\frac{P_2}{\gamma} = \left(\frac{V_3^2}{2g} - \frac{V_2^2}{2g}\right) + \frac{P_{atm}}{\gamma} - (H - h_L)$$

It the area of the flow at throat and exit are equal then $V_3 = V_2$

$\mathbf{\frac{P_2}{\gamma} = \frac{P_{atm}}{\gamma} - (H - h_L)}$**….(1)** This equation will shows us that the pressure at the throat will be decreased by the value H if frictional loss (h_L) is neglected.

- There is mathematically possibility of the decrease of the pressure at the throat section up to zero creating the large suction pressure. Thus maximum value of H = 10.3m.

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6. Siphon Spillway…..

- But when the pressure is negative, then the dissolved air in the water will be starting to escape and starts to accumulate at the summit of the pipe and will cause the stoppage of the flow.
- From the experimental research pressure equal to 2.43m of water head could be achieved at the highest point of the siphon without the fear of the releasing of dissolved air.
- So, the maximum of the head difference that is allowed is

$$H = \frac{P_{atm}}{\gamma} - \frac{P_v}{\gamma} + h_L = 10.3 - 2.43 + h_L = 7.92 + h_L$$

- So for the trouble free flow of the siphon, it is necessary to maintain the head loss than the value (7.92 + h_L) meter.

<u>The discharge over the siphon spillway</u>

$$Q = C_d A\sqrt{2gH}$$

$C_d \approx 0.65$

- Where, L is length of the throat and H is height of the throat.
- Here, the size of the throat and exit are equal.
- Maximum velocity at the throat

$$V_{max} = \sqrt{2g(7.92 + h_L)}$$

$V_{max} = 14.47$ m/s

$$\mathbf{Q_{max} = 14.47\ C_d A}$$

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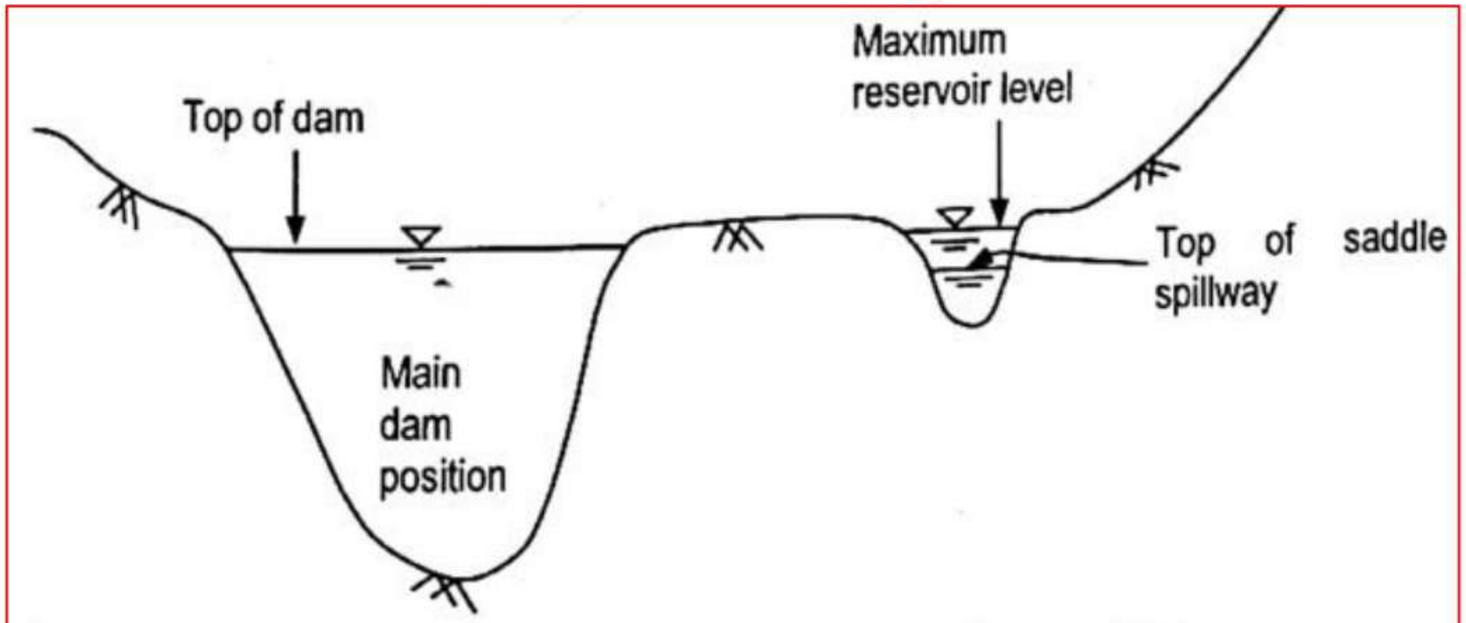
7. Saddle spillway

- When a natural depression or saddle in the profile of the basin away form the dam site exists, specially in earth dam, this spillway is provided in the natural depression. This type of spillway is called saddle spillway.
- It is essential that the level of depression should be at fill reservoir condition, i.e. level.
- For ideal condition, there should be a good foundation at the site of spillway.



Profile of basin along periphery showing dam and saddle spillway position.

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Design of Spillway:

Following factors are considered for design of spillway;

i) Safety considerations consistent with economy:

- Many dam failures are caused by the improper design and inadequate capacity.
- It must be hydraulically and structurally safe.
- Water sliding from the spillway creates high scouring velocity therefore the structure must be constructed resistant to erosion withstanding it from reservoir surface to tail water.
- Usually stilling basin is needed for dissipation of energy at the toe of the dam.

ii) Hydrological and site conditions:

The spillway design depend on inflow discharge, its frequency, capacity of reservoir, height of crest, geological and other site condition and topographical features.

iii) Operating condition and purpose of dam.

iv) Condition of downstream of dam

v) Type of dam

vi) Nature and amount of sediment carried by the river

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Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

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Hydraulic Jump:

- Hydraulic jump is a local phenomenon where by the water surface jumps and abruptly rises from a level below critical depth line to level above normal depth line representing the change of supercritical flow into a subcritical flow and releasing immense energy in the process.
- It is classified as rapidly varied flow (RVF).
- The flow in the hydraulic jump is accompanied by the formation of extremely turbulent rollers and hence there is a considerable dissipation of energy.
- The turbulent eddies break up into smaller ones as they move downstream.
- $\frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1 + 8F_1^2}\right)$

Energy dissipation:

- When flood discharge passes over the spillway crest, it has high potential energy which gets converted into kinetic energy as slides along it.
- It would cause erosion at the downstream toe of the dam.
- There is provided any structures to protect erosion below a spillway is called **energy dissipater**.
- Any process for reducing kinetic energy at the toe of the dam is called **energy dissipation**.
- Energy dissipating devices can be generally categorized into two types.

a) Basin using a hydraulic jump

b) Basin using bucket types

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Energy dissipation:…..

- The tail water depth and characteristic of the hydraulic jumps govern the choice of energy dissipating device at a particular spillway.
- Some of the most common methods of energy dissipaters below spillways are as follows:

i) Hydraulic jump type stilling basin

ii) Roller bucket type energy dissipater

iii) Ski-jump (Deflector) bucket energy dissipater

i) Hydraulic jump type stilling basin

- Hydraulic jump type stilling basins shall be used to dissipate the energy of discharge passing the overflow section before the discharge is returned to the downstream river channel.
- This shall be attained through formation of a controlled hydraulic jump within the confines of the basin over the entire range of flow conditions under which basin is expected to operate.
- Possibility of Hydraulic jump is formed only if the flowing down the face of spillway slope is **supercritical** before jump (Froude Number $F_{r1} = \frac{V_1}{\sqrt{gy_1}} > 1$) and enters over horizontal apron of basin with **subcritical slope** after jump (Froude Number $F_{r2} = \frac{V_2}{\sqrt{gy_2}} < 1$) & the tail water depth must be equal to or less than post- jump depth as given by equation $\left\{\frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1 + 8F_1^2}\right)\right\}$.
- Due to formation of hydraulic jump in stilling basin, major part of initial flow energy is dissipated.

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Energy dissipation:…..

i) Hydraulic jump type stilling basin…..

- If tail water depth is greater than y2 then jump is submerged and hence there occurs little loss of energy.
- On other hand, if tail water depth is less than y2 then jump gets swept off downstream.
- In such condition, supercritical flow continues for some distance on downstream. It creates problem of river bed erosion.
- Hence success of stilling basin for formation of hydraulic jump at different conditions of depth, discharge and Froude number before jump is facilitated by providing some structures like chute blocks, friction blocks or baffle pier notched end sill etc.
- The hydraulic jump type stilling basin is usually used for spillway as an energy dissipater.

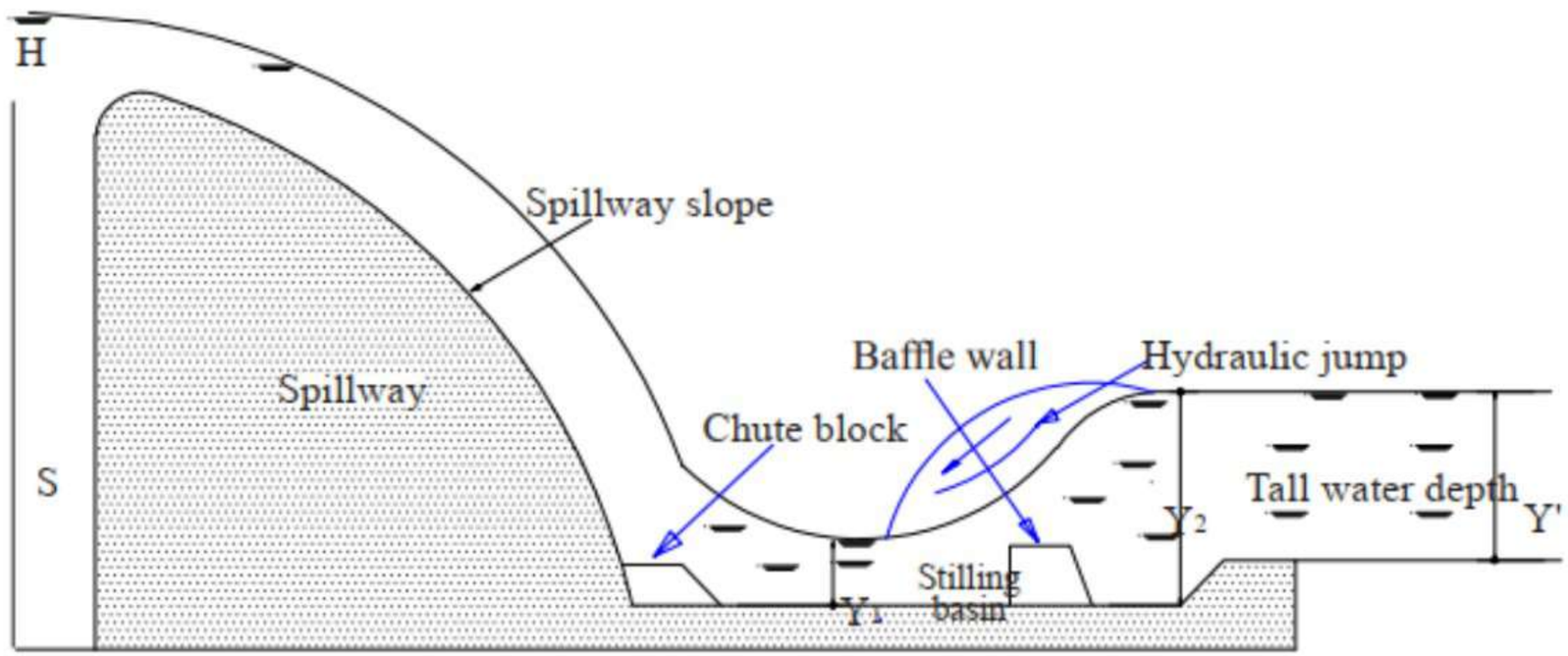

Fig Hydraulic Jump in Stilling Basin

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Energy dissipation:…..

ii) Roller bucket type energy dissipater:

- In the case of the tail water condition not favorable for hydraulic jump type basin, roller bucket type energy dissipater is provided.
- High velocity water sliding down over the spillway slope is drawn toward the tail water.
- This creates a surface roller as well as bottom roller action and eddy turbulence, along with the energy dissipation.
- Bucket type energy dissipater has a relatively short structure as compared to hydraulic jump type stilling basin.
- Roller action becomes successful when tail water depth is slightly greater than that needed by hydraulic jump type basin.
- Roller buckets for energy dissipation may be provided in either of the following forms

a. Solid roller bucket b. Slotted roller bucket

- Both roller buckets shall have upturned buckets for the formation of the rollers. The slotted roller bucket shall have teeth in its bucket to laterally spread the jet passing through it.

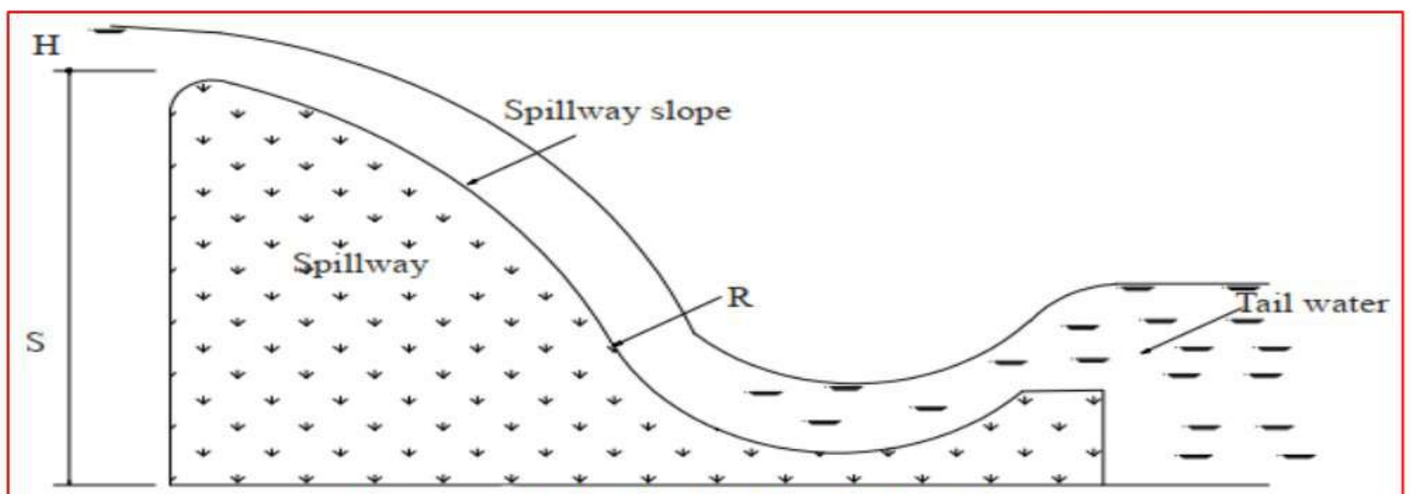

Fig Roller Bucket Type Stilling Basin

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Energy dissipation:…..

iii) Ski-jump (Deflector)bucket type Dissipater

- It is constructed very similar to roller buckets.
- The hydraulic action however is entirely different.
- The jet of water from spillway flows over bucket and springs up clearly in air and following a trajectory, hits river bed at some distance away from toe of the dam.
- This type of spillway is appropriate where there is good quality foundation rock that can withstand erosive action of plunging jet; tail water is low so that a clear ski jump formation can occur; and usually roller bucket type ski jump type stilling basin is constructed below the arch dams.

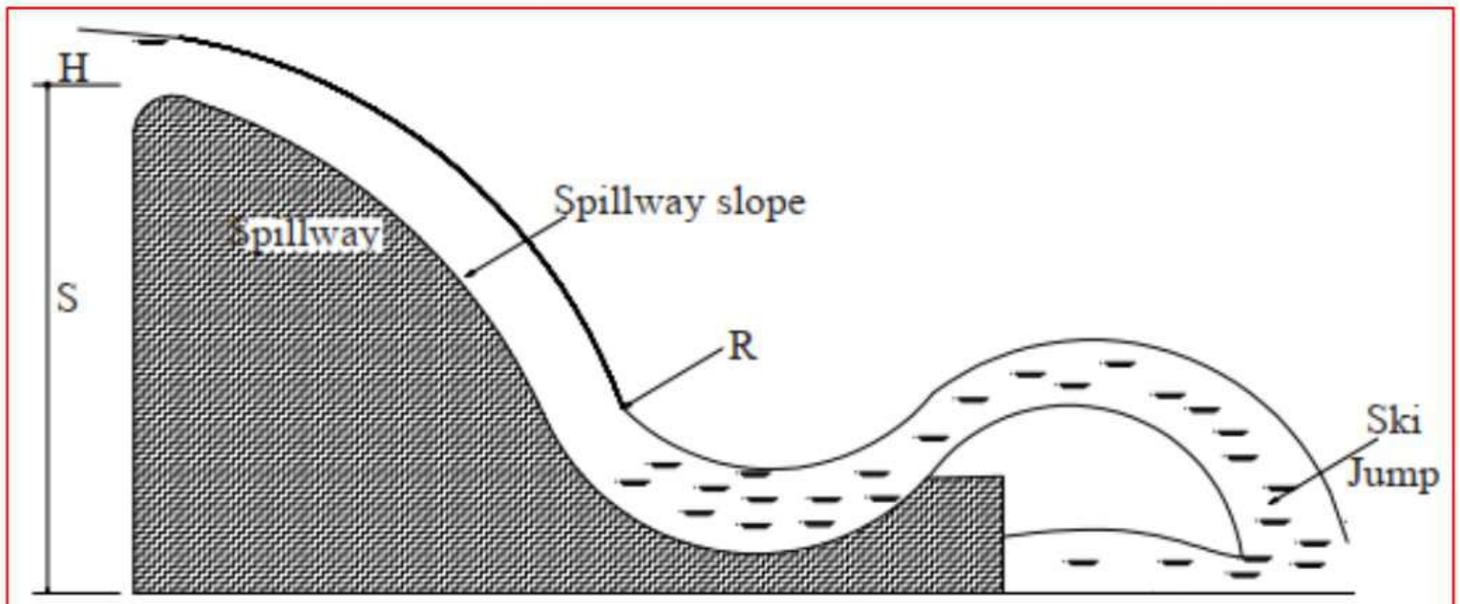

Fig Ski Jump Bucket Type Stilling Basin

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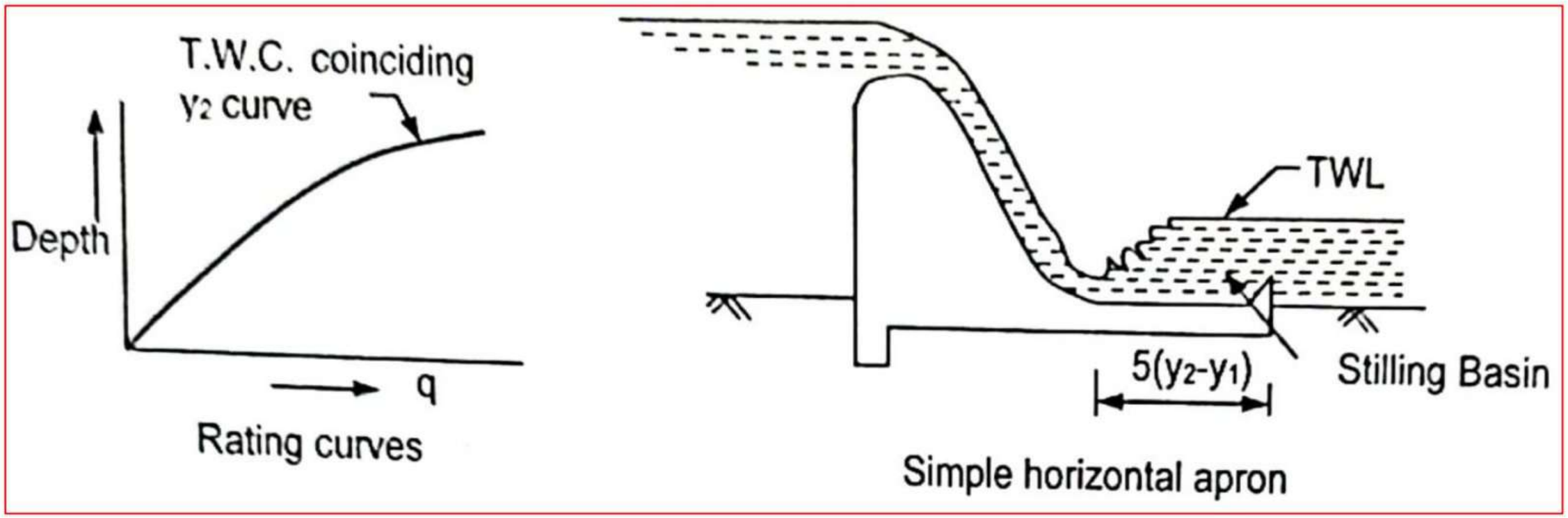
Tail water Conditions

- For a particular site, the type of energy dissipater and its arrangement shall be decided based on the relationship between the height of the hydraulic jump and tail water depth.
- For this purpose, the following four cases shall be considered

Case I : Jump height equal or nearly equal to tail water depth

- When the tail water rating curve approximately follows the hydraulic jump curve or is only slightly above or below it for all discharges, the horizontal apron stilling basin, with or without baffle blocks and end sills, shall be used for energy dissipation.
- The hydraulic jump will be formed at the toe of the spillway at all discharges.
- In such case, a simple concrete pron of length $5(y_2 - y_1)$ is generally sufficient to provide in the region of hydraulic jump.



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Tail water Conditions…..

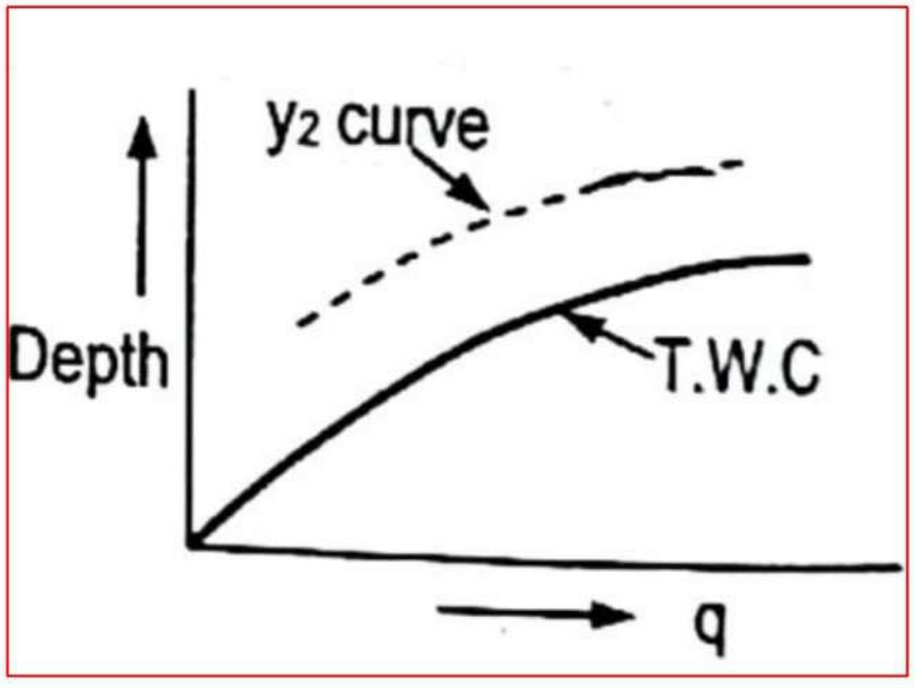
Case II : Jump height always above tail water depth

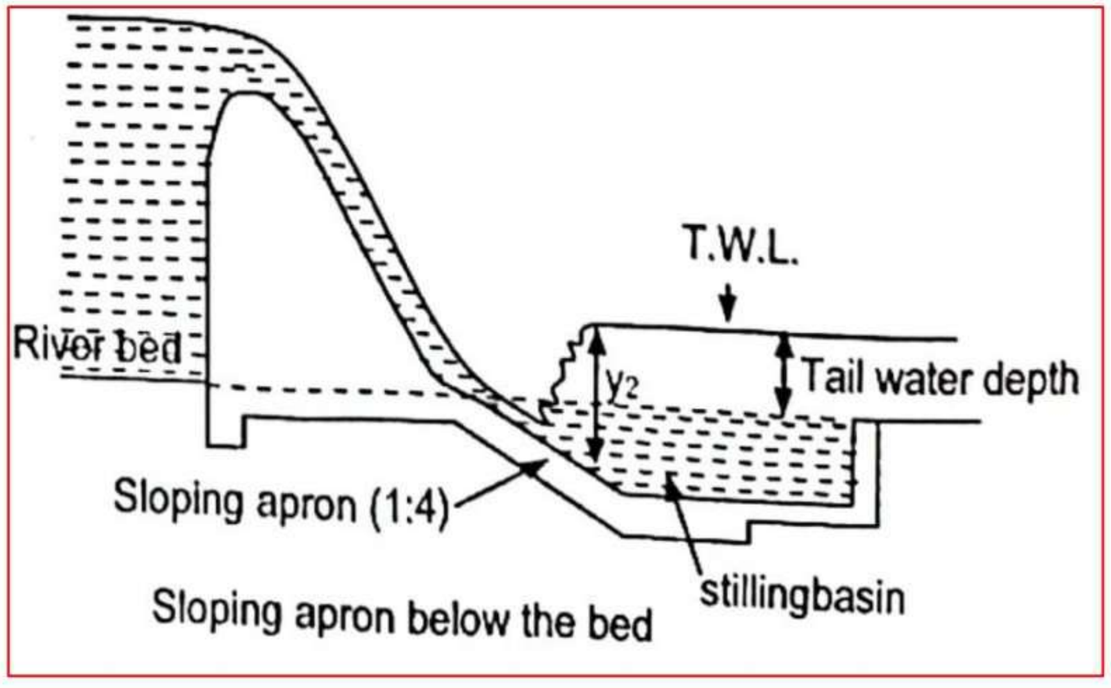
- As the tail water depth shall be insufficient for the formation of a hydraulic jump at the toe of the diversion structure, energy dissipation in this case shall be achieved in one of the following ways

a) lowering the apron level of the stilling basin to make the tail water depth in it equal to the jump height for all discharges, leading to the following alternatives

- Horizontal apron stilling basin with apron depressed below the river bed level.
- Sloping apron stilling basin with depressed floor rising up towards the downstream end.
- Sloping apron stilling basin with depressed floor sloping away from the toe of the overflow section.

This sloping apron and horizontal cistern of length $5(y_2 - y_1)$ shall be entirely in cutting and may be expensive, through otherwise satisfactory.





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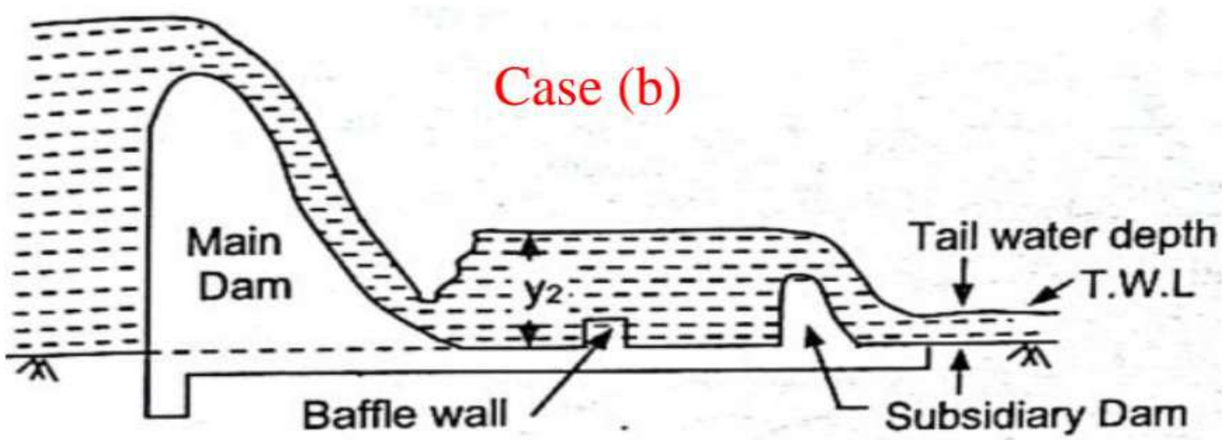
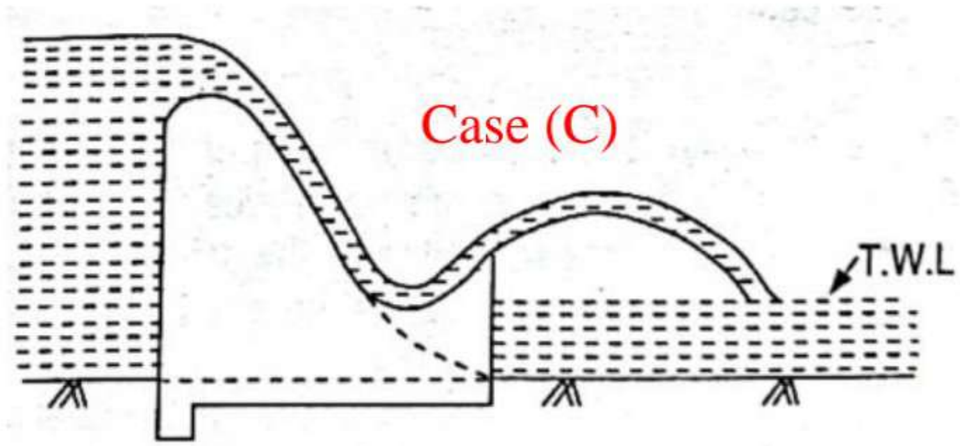
Case II : Jump height always above tail water depth

- b) Horizontal apron stilling basin with a low subsidiary dam downstream:
- Construction of a subsidiary dam below the main dam increase the tail water depth and cause the jump to form at the toe of the main dam.
- If the tail water deficiency 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 for low spillways or weirs.
- They should be sufficiently strong to withstand impacts from ice and floating debris.
- The location, shape and size and spacing of these baffles can be best determined by model studied.
- c) Deflector (Ski – jump) bucket type:
- If the tail water lies very low, the water may shoot up out of the above bucket, and fall harmlessly into the river at some distance downstream of the river.
- This type of energy dissipater requires sound and rocky river bed, because part of the energy dissipation takes place by impact, although some of energy dissipated in air by diffusion and aeration.



Stilling basin with subsidiary dam



Ski jump bucket

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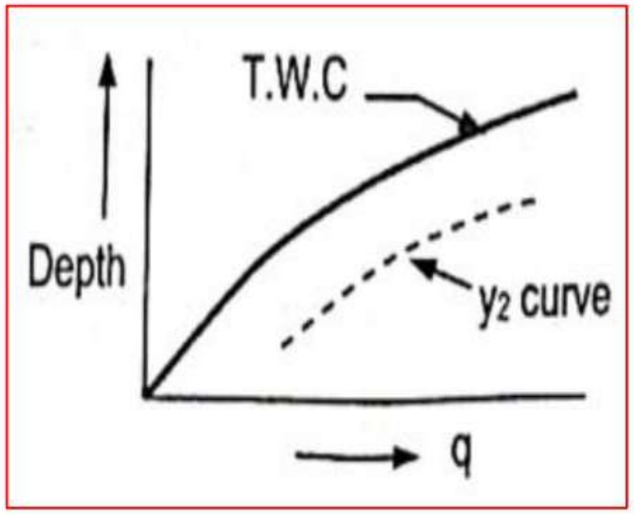
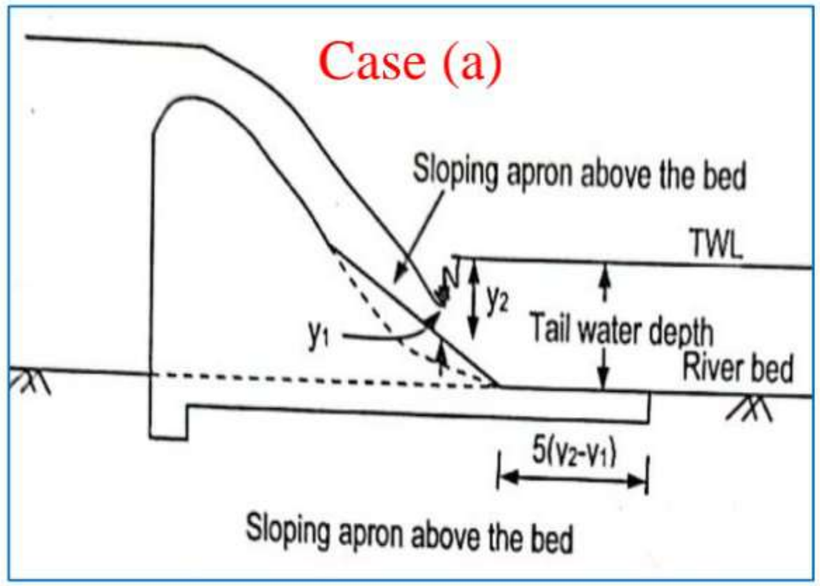
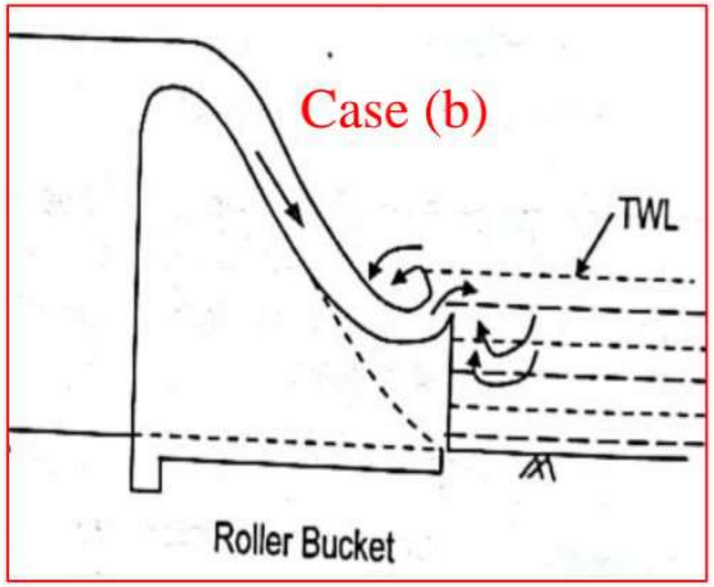
Case III : Jump height less than tail water depth

- As an imperfect jump would only form under high velocities, one of the following energy dissipaters may be used in this case
- **a) Sloping apron stilling basin**
- The sloping apron above the river bed is constructed so that the jump is formed in the sloping apron where *depth equal to* y_2 (lesser than the 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 make the proper slope.
- **b) Roller buckets:**
- Roller buckets shall be preferred if the tail water depth is 10 to 20% greater than the sequent depth.
- This type of energy dissipater consists of an apron, which is unturned sharply at the ends.
- Tow main rollers are formed which dissipate the energy due to internal turbulences.
- The roller which is formed downstream of the bucket, tends to move the scoured bed materials towards the dam, thus preventing serious scour at the toe of the dam.







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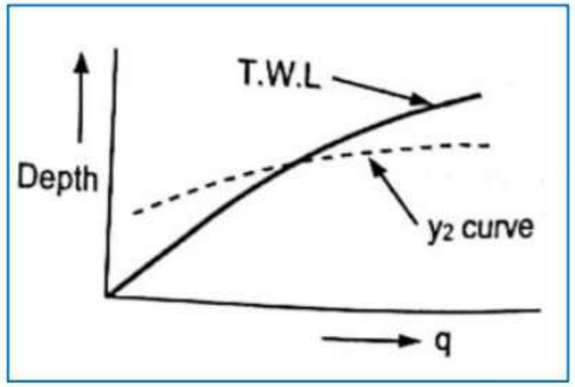
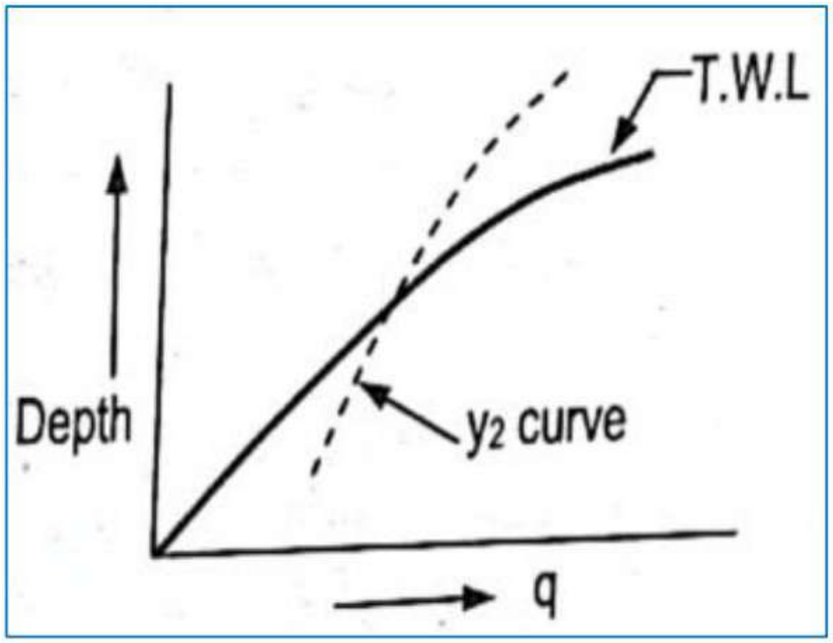
Case IV : Jump height more than tail water depth at low discharge and less at higher discharge

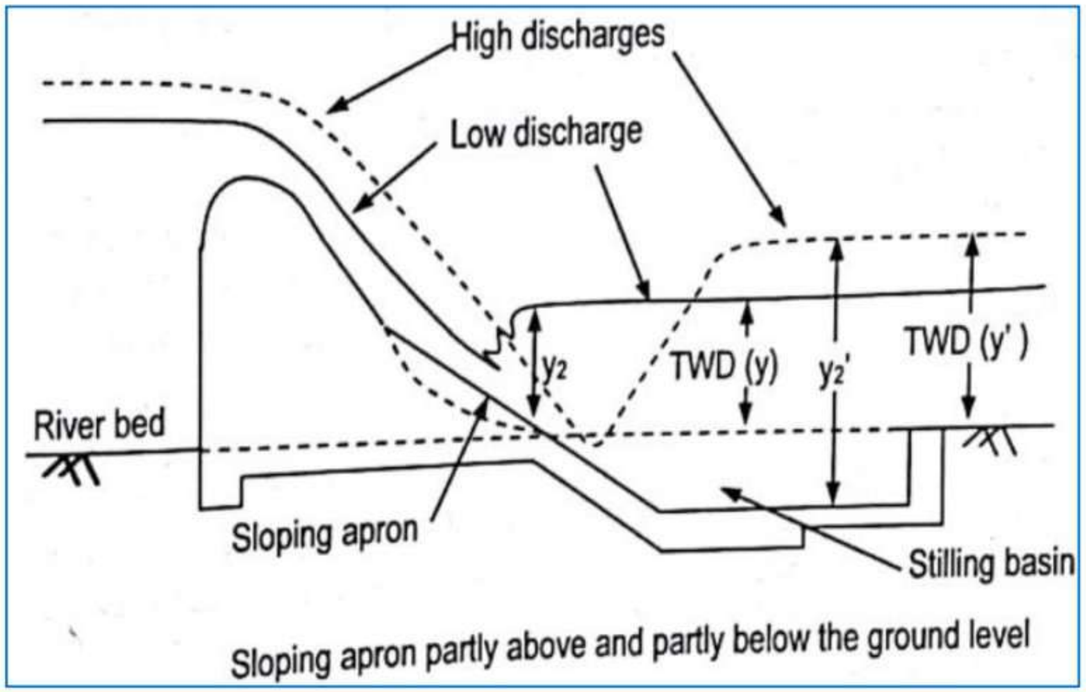
- In this case, enough tail water depth shall be artificially created to form a jump on the apron at low discharges. For this purpose, one of the following alternatives may be used
- a) Horizontal apron stilling basin with a low secondary dam
- b) Stilling basin with baffle piers or some form of dentated sill.



Case V : Jump height below tail water depth at low discharge and above at higher discharge:

- Under these conditions, sufficient tail water depth may be created to form a jump at high flows suing the one of the following methods;
- a) Horizontal apron stilling basin with a low secondary dam
- b) Sloping apron stilling basin
- Alternatively, a bucket type energy dissipater that acts as a roller bucket for low discharges and as a deflector bucket at higher discharges may be used provided sound rock conditions exist in the river channel.





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Design of Stilling Basin

1. Find width of basin (B) if head over the crest (H) is known or find 'H' if width (B) is known from the following formula, Discharge$(Q) = \frac{2}{3} C_d B H^{3/2} \sqrt{2g}$, Where C_d= discharge coefficient

2. Find the value of by $V_1$by Applying Bernoulli's equation at section 0 – 0 and 1 – 1

$$\frac{P_0}{\gamma} + \frac{V_0^2}{2g} + Z_0 = \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1$$

3. find the value of pre – jump depth (y_1)

$$y_1 = \frac{Q}{V_1 * B}$$

4. Find the value of Froude number, $F_{r1} = \frac{V_1}{\sqrt{g y_1}}$

5. Find the value of Post jump depth (y_2)

$$\frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1 + 8F_1^2}\right)$$

6. Calculate normal depth of flow (Y_n)

Discharge$(Q) = \frac{1}{n} R^{2/3} S^{1/2} * A$

$$= \frac{1}{n} * \left(\frac{BY_n}{B + 2Y_n}\right)^{\frac{2}{3}} * (S)^{\frac{1}{2}} * BY_n$$

7. Find the value of critical depth $(Y_C) = \left(\frac{q^2}{g}\right)^{\frac{1}{3}}$

8.If $Y_n > Y_C$, Subcritical flow (jump formed) ⇒ Select Stilling Basin

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U.S.B.R. Stilling basin II ($F_r \geq 4.5$) and ($V_1 \geq 15$ m/sec)

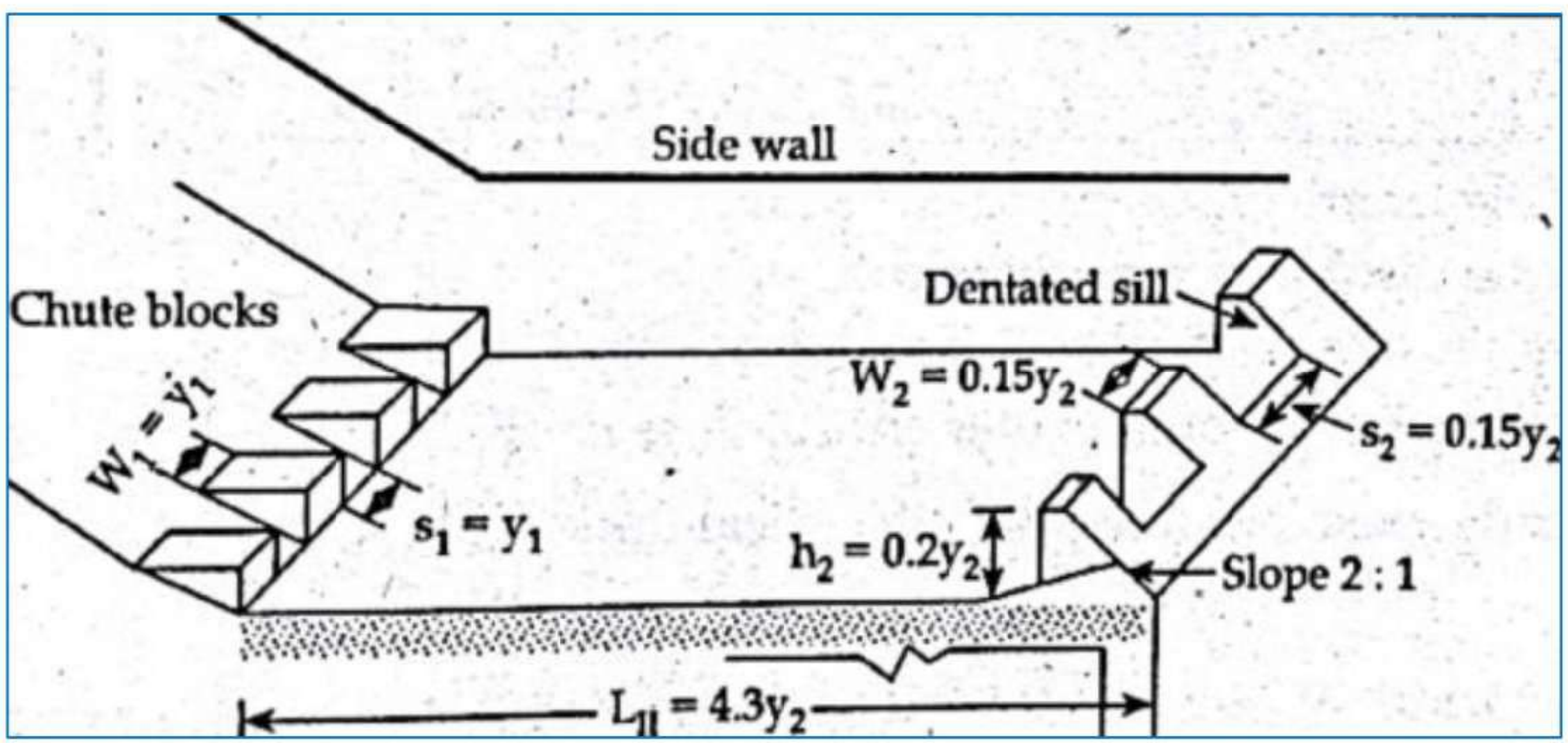
USBR basin type II is recommended for use on large structures such as dams spillways, large canal structures etc., when incoming Froude number is greater than 4.5 and velocity greater than 15 m/sec.

The dimensions of chute block $h_1 = y_1$
Width of the chute block $w_1 = y_1$
Spacing of the chute block $S_1 = y_1$
Spacing of first chute block = $y_1/2$

Height of dented sill $h_2 = 0.2y_2$
Width of the dented sill $w_2 = 0.15\ y_2$
Spacing of the dented sill $S_2 = 0.15y_2$
Top width of dented sill = $0.02y_2$

The length of the basin (L) is related to the Froude number (F_{r1}), which is shown in table

F_{r1}	4	6	8	10 or more
Length of basin	$3.6y_2$	$4y_2$	$4.2y_2$	$4.3y_2$



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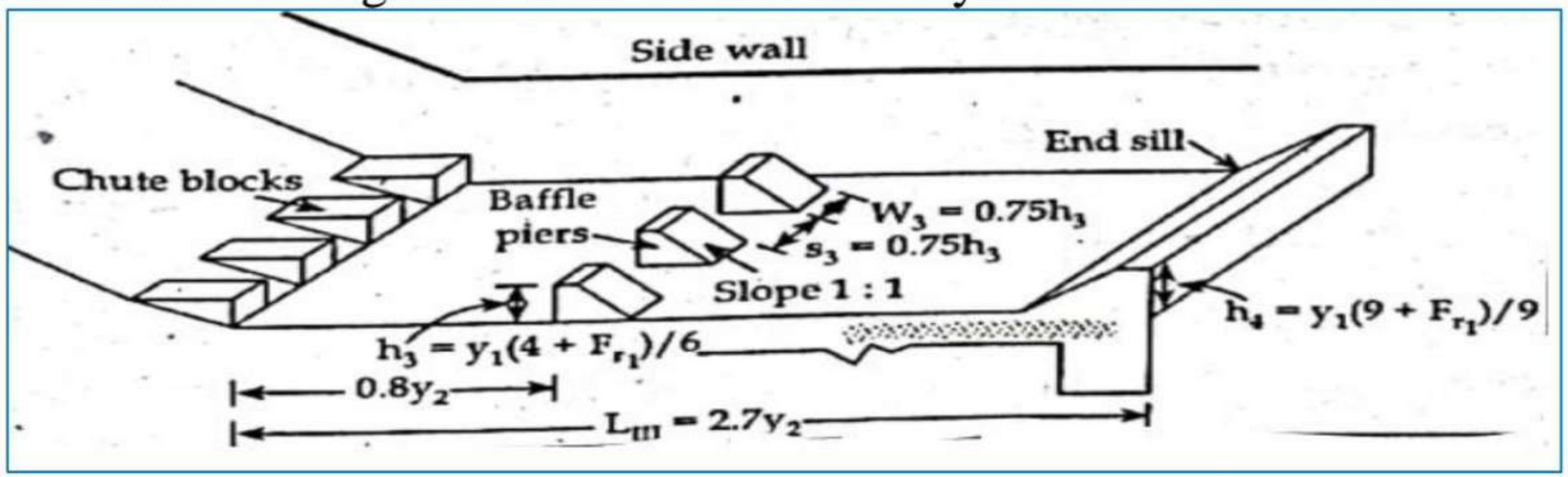
U.S.B.R. Stilling basin III ($F_r \geq 4.5$) and ($V_1 < 15$ m/sec)

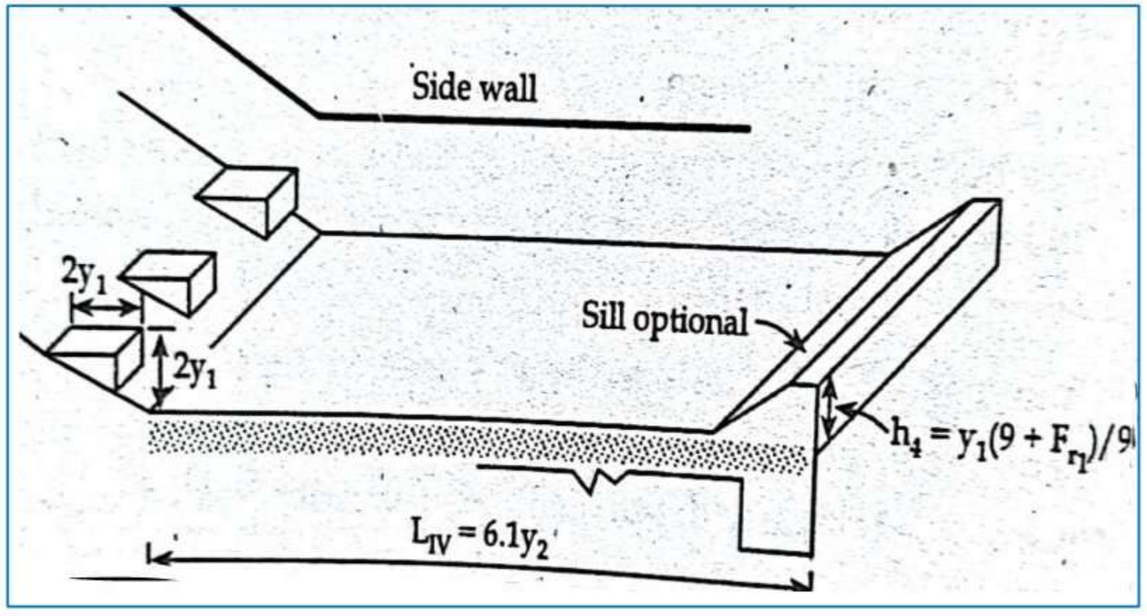
USBR basin type III is recommended for use on large structures such as dams spillways, large canal structures etc., when incoming Froude number is greater than 4.5 and velocity less than 15 m/sec.



U.S.B.R. Stilling basin IV (2.5 $< F_r <$ 4.5)

USBR type IV of stilling basin is used for Froude number between 2.5 to 4.5, which generally occurs in canal weirs, canal falls, diversion dams etc. the basin is applicable only to rectangular canal sections. Since oscillating waves are generated in this range of Froude number, they tried to be controlled at source by providing large chute blocks. The length of the basin is generally kept equal to $5(y_2 - y_1)$.

- Height of the chute block $= 2y_1$
- Length of the chute block = minimum $2y_1$
- Maximum tooth width of the chute block w $= y_1$
- Spacing of first chute block = 2.5 w
- Top surface of the chute block is kept 5° downward slope with horizontal



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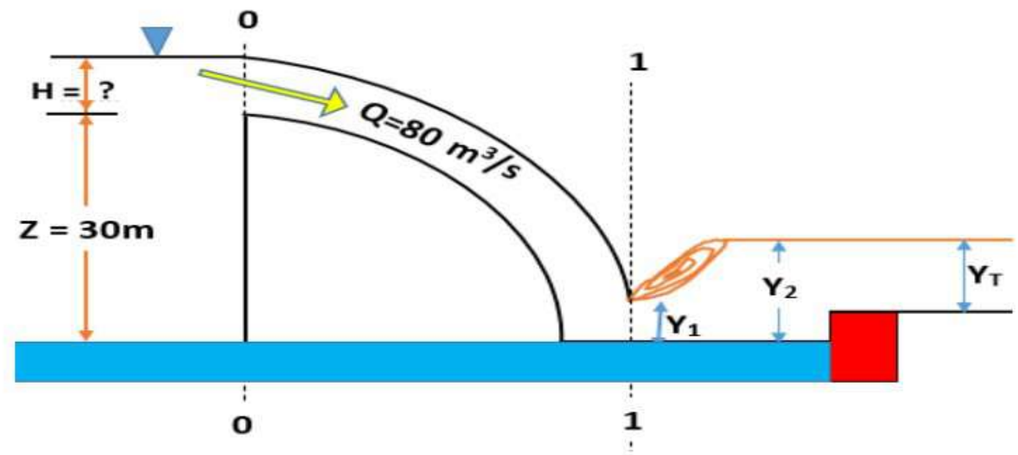
Q. N. Check whether the stilling basin is required or not for the following data. If yes, design the stilling basin for the spillway. Height of the crest above downstream bed level = 30m, Discharge = 80 m3/s, width of channel = 10m, Coefficient of discharge (Cd) = 0.7, Downstream bed slope = 1:500 and Manning's coefficient (n) = 0.016.

Solution: From question given as, Discharge (Q) = 80m^3/sec spillway crest height above the downstream bed = 30m, slope = 1:500 and n = 0.016, Coefficient of discharge (C_d) = 0.7, width of the canal = 10m



- We have Discharge(Q) $= \frac{2}{3} C_d B H^{3/2} \sqrt{2g}$
- Or, $80 = \frac{2}{3} * 0.7 * 10 * (H)^{3/2} \sqrt{2 * 9.81}$
- $3.87 = (H)^{3/2}$
- $H = (3.87)^{2/3} = \mathbf{2.465m}$
- Specific Energy at 0 – 0 = Specific Energy at 1 – 1
- $\therefore 30 + 2.465 = Y_1 + \frac{V_1^2}{2g} = Y_1 + \frac{Q^2}{2gB^2Y_1^2}$
- $\Rightarrow 32.465 = Y_1 + \frac{80^2}{2*9.81*10^2*Y_1^2}$
- OR $32.465 = Y_1 + \frac{3.262}{Y_1^2}$
- $\Rightarrow Y_1^3 - 32.465Y_1^2 + 3.262 = 0 \cdots\cdots (1)$
- Solving Eq^n(1), We get $Y_1 = 32.462$m, 0.319m, −0.315m i. e. possible value is 0.319m
- $V_1 = \frac{Q}{B*Y_1} = \frac{80}{10*0.319}$
- $= \mathbf{25.078\ m/s}$

OR: Applying Bernoulli's equation at section 0 – 0 and 1 – 1

- $\frac{P_0}{\gamma} + \frac{V_0^2}{2g} + Z_0 = \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1$
- i.e. P_0 and $P_1 = P_{atm}$
- Velocity at reservoir (V_0) $= \frac{Q}{H*B} = \frac{80}{2.465*10} = 3.245$ m/s
- $\frac{3.245^2}{2*9.81} + 30 + 2.465 = \frac{V_1^2}{2*9.81} + 0$
- **V_1 = 25.445 m/sec**
- $y_1 = \frac{Q}{V_1*B} = \frac{80}{25.446*10} = \mathbf{0.314\ m}$

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- Calculation value of post jump $(Y_2) = \frac{Y_1}{2}\left\{\sqrt{1+8F_1^2}-1\right\}$
- Here $F_1 = \frac{V_1}{\sqrt{gY_1}} = \frac{25.078}{\sqrt{9.81*0.319}}$
- $= \mathbf{14.176}$
- i.e. $(Y_2) = \frac{0.319}{2}\left\{\sqrt{1+8*(14.176)^2}-1\right\}$
- $= \mathbf{6.238m}$
- Also Discharge $(Q) = \frac{1}{n}R^{2/3}S^{1/2} * A$
- $\Rightarrow 80 = \frac{1}{0.016} * \left(\frac{10Y_O}{10+2Y_O}\right)^{\frac{2}{3}} * \left(\frac{1}{500}\right)^{1/2} * (10 * Y_O)$
- $2.862 = Y_O * \left(\frac{10Y_O}{10+2Y_O}\right)^{\frac{2}{3}}$
- Solving by trial and error method $\mathbf{Y_O = 2.171m}$
- $\therefore$ Critical depth $(Y_C) = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} = \left(\frac{Q^2}{g*B^2}\right)^{\frac{1}{3}} = \left(\frac{80^2}{9.81*10^2}\right)^{\frac{1}{3}}$
- $= \mathbf{1.869m}$
- Here $Y_O > Y_C$, i.e. Hydraulic jump formed i.e. **Stilling basin is required.**
- Here $F_1 = 14.176 > 4.5$ & $V_1 = 25.078$ m/s > 15 m/s, Choose **USBR Type II** Stilling Basin
- Length (L) = $4.3Y_2$ = 4.3*6.238 = 26.823m

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10. A concrete dam has been designed to pass a flood of $3550 m^3/s$ in the D/S of the river. The profile of the dam permits a head of 5.25m over the crest. If the discharge coefficient is 0.95 and approach velocity = 0.8m/s, Determine the crest length to safety discharge the above flow. Also if S = 0.001, n = 0.025 and dam height = 25m. Design the hydraulic jump stilling basin.

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Spillway gates

The gate is a hydraulic structure constructed by steel or wooden across the water passage or conduits to control the flow and to raise water level in fully closed condition. However, under full opened condition, there is no obstacle to flow due to gate. The number of the gates depends on the size of gate and the design of hydrostatic pressure acting on the gate. It is designed by considering hydrostatic as well a hydrodynamic forces such as wave forces, ice formation and seismic pressure force on water contact with gates. Now a day, usually welded or riveted steel frame consisting skin steel plate on upstream side with vertical and horizontal girder are used to hold water against it. The operational requirement for gates should be water tightness, rapidity of operation, minimum hoist capacity and convenience in installation and maintenance.

Performance criteria of gates

The gates usually require fulfilling the following performance criteria:

i) The gates should be desirable watertight, the maximum permissible leakage may be less than 5 liters/min/m of the crest length for medium head conduit gate.

ii) The gate should be simple and economical.

iii) The whole parts related accessories should be smooth, rigid and can be quick operation system as requirements.

iv) It should have reasonable opening capacity to prevent vibration and cavitation in case of partially opening.

v) It should be smooth outlet to entry water with maximum discharge in case of full opening of gate.

vi) It should have minimum hoist capacity at specified speed and easy in installation and maintenance.

vii) It should be corrosion free accessories

viii) Whenever necessary, model studies may be carried out for high head regulating gates.

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Types of gates

Gate can be classified into following types

a) Based on use as construction materials

i) steel gates ii) Aluminum alloy gates

iii) Reinforced concrete gates iv) Wooden gates v) Rubber gates

b) based on the position in the dam

i) Crest gates ii) Submerged (high head) gates

c) Based on the functions

i) Service gates ii) Maintenance gates iii) Emergency gates

d) Based on the pressure transmission

i) pressure transmission to the gate sill e.g. sector (drum) gates, flap gates,

ii) Pressure distribution to pier or abutments e.g. Vertical gates, lift gates, radial gates and roller gates.

iii) Pressure transmission to the gate sill and pier e.g. Flap gates and floating (Pontoon) gates.

iv) Pressure transmission to the whole structure.

e) Based on type of motion of the gates opening

i) Translatory gates ii) Rotary gates iii) Floating gates

iv) Gates moving along or across the flow

**The basic feature of the structural design of crest gates is the method used for transferring the pressure acting on gates. The common types of spillway gates are as follows

1. Vertical gates 2. Radial gates 3. Stop log gate

4. Drum gate 5. Flap gates 6. Rolling gates

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Types of gates….

1. Vertical (Plain) gates:

- Vertical gates were commonly used in earlier time.
- However, now days, it obsoletes by the development of radial gates. Vertical lift gate is a rectangular shape constructed by steel frame with steel plate in suitable size of girders and beam.
- Due to the water pressure force against the gates and lifting mechanism, develops high frictional force between gate and pier or abutment guide grooves.
- It can be minimized by providing cylindrical rollers them. It may be constructed roller type and slide type gate.
- The wheeled type roller gate more popular than slide type gate because of requires lower hoisting capacity due to gravity.
- Rubber seal are provided around the circumference of gate plate for the purpose of water tight.
- The slide type gate is not suitable for operation in rivers carrying large quantity of silt and debris.

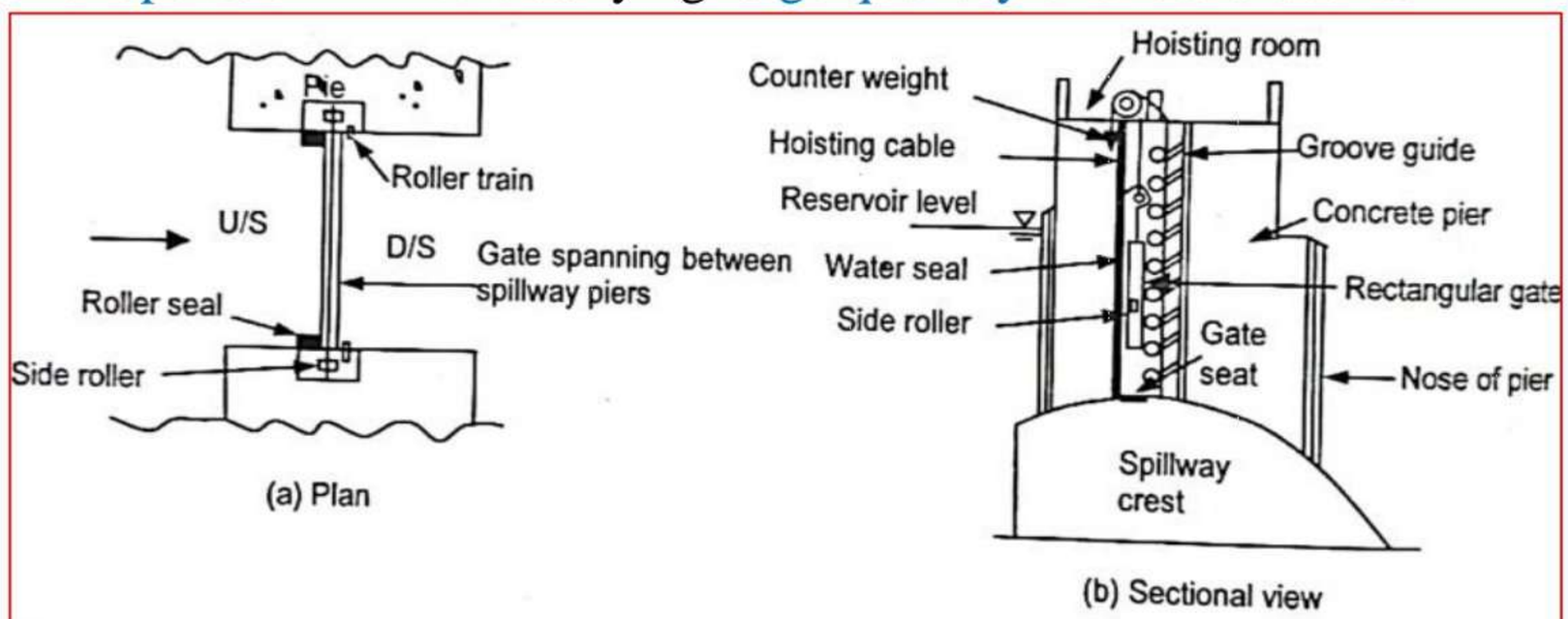

Figure : Vertical lift gate or vertical sliding gate

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Types of gates….

2. Radial gates:

- A radial gate is a curve shaped (Convex side) gate are mostly used as spillway gates and in outlet gates in a dam but sometimes it also may be used concave side of curve to resist water pressure.
- It is most common and economical type of spillway gates in certain range of size.
- The radial gate is operated by rotating around its hinge; hence the function of lever arms acts for reduction in hoist capacity and contributes to the economy.
- The main problem of the conventional radial gate is not suited for passage of floating bodies without full gate open.
- The radial gates are usually constructed as Figure Photo of the Radial Gate portals with cross bars and radial or inclined or horizontal or straight arms, but could also be cantilevered over the arms.
- End beams are supported by radial arms.

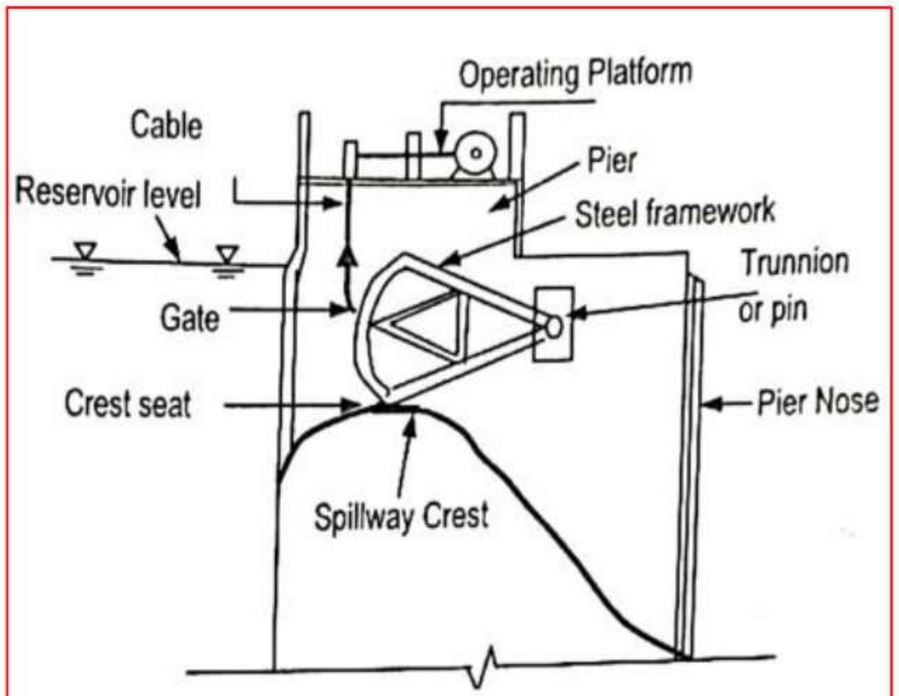
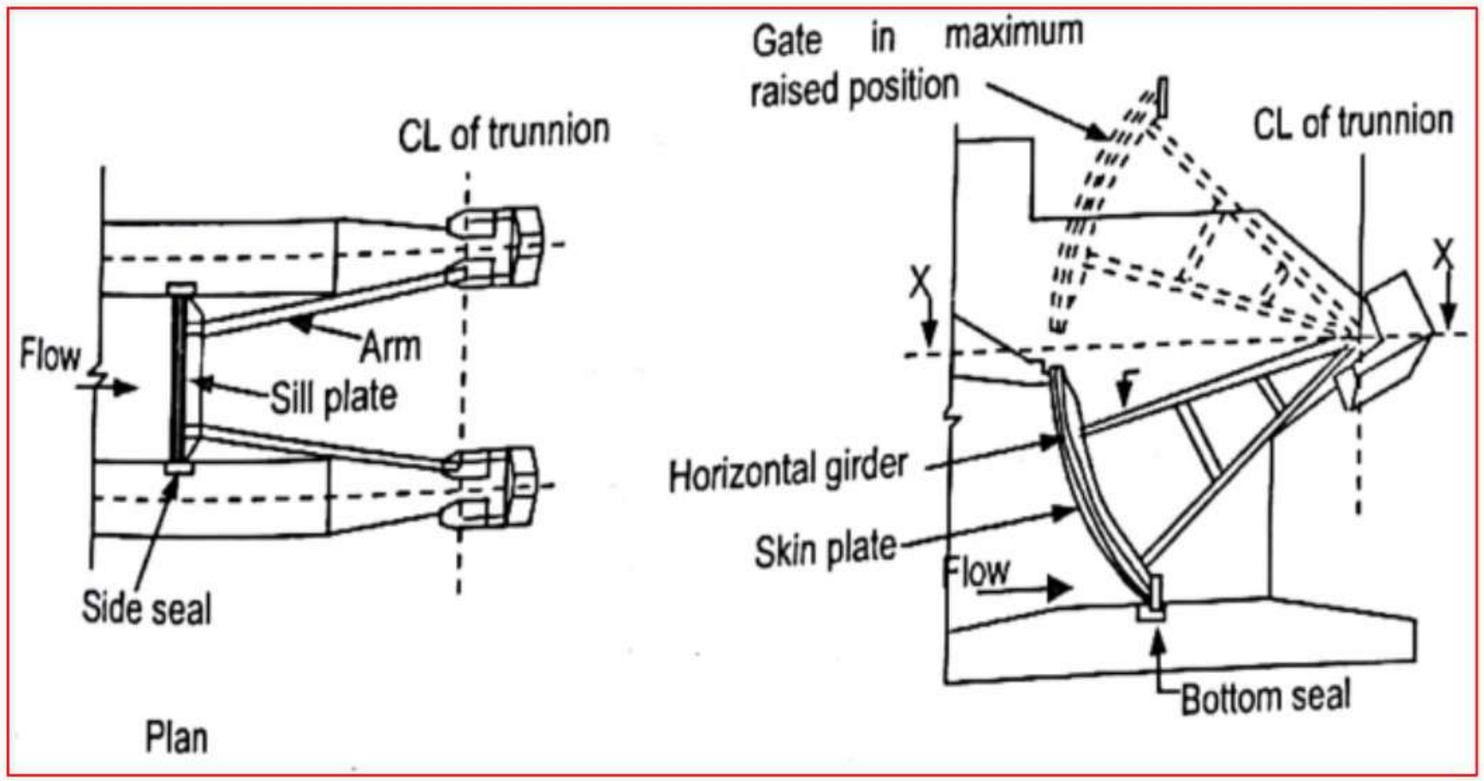


Fig: Taintor Gate



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Types of gates….

2. Radial gates:

- The arms transformed water pressure loads to pivoted anchorage girder.
- Each gate is pivoted on two support hinges at generally downstream for low heads and upstream for shorter piers thus it can be rotated easily about fixed horizontal axis.
- It is usually hoisted by cables fixed to each end to prevent from twisting and jamming and also can be adopted itself easily to automatic control.

3. Stop log gate:

- Stop log gate consists of wooden planks placed one upon other spacing between piers with grooves.
- The removal and placing of planks are done manually.
- It is more suitable in the low head dams.
- It requires to change planks at short duration because it can be easily affected by high current of flow.

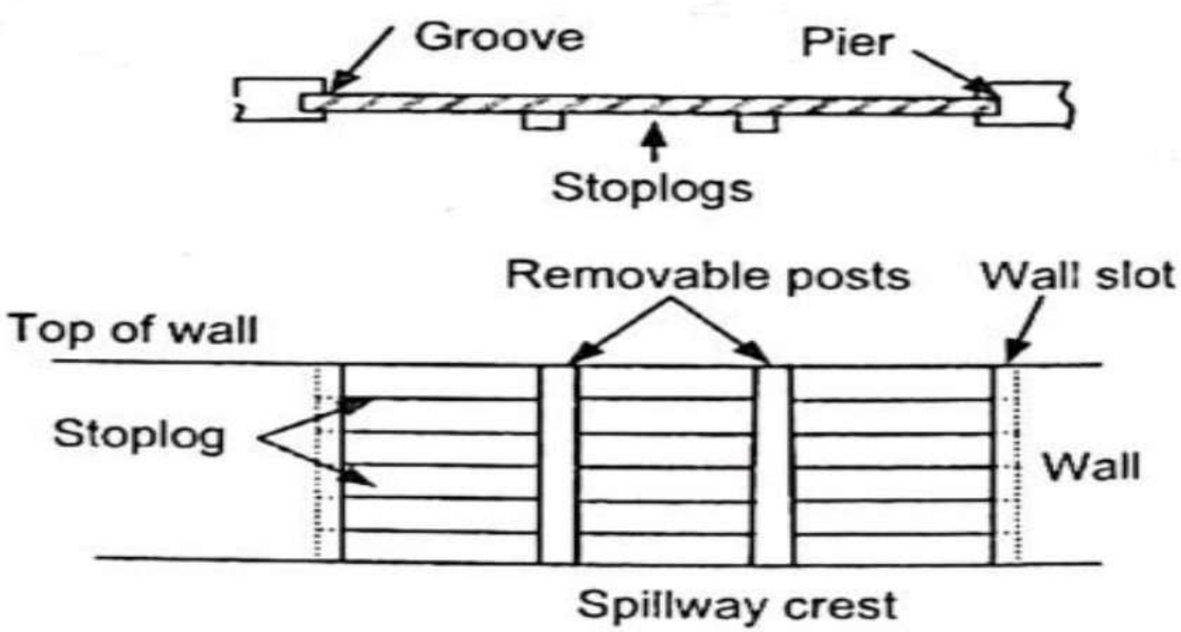

Fig: Plan and section of Stoplog gate

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Types of gates….

4. Drum gate:

- It is also called sector gate.
- The design features of the drum gates are enclosed on all the three faces and forms water tight vessel.
- The nearly similar shaped of quarter size of circle arc skin plate is formed on the upstream side associated by internal bearings.
- The gate pivoted about the hinge usually positioned at the upper edge of the gates.
- It is operated by buoyant forces and lifted by control equipment housed inside the piers.
- The drum gates are suited for long span and medium height of dam and not suitable for low head dams.
- The main advantages of the drum gates are ease of automation, fast movement and accuracy of regulation.
- Especially it is more suited for easy passage to the ice, debris and floating materials on downstream of the spillway.
- However, these gates are complex in installation and operations as well as very costly than other gates like radial gates and require careful maintenance.

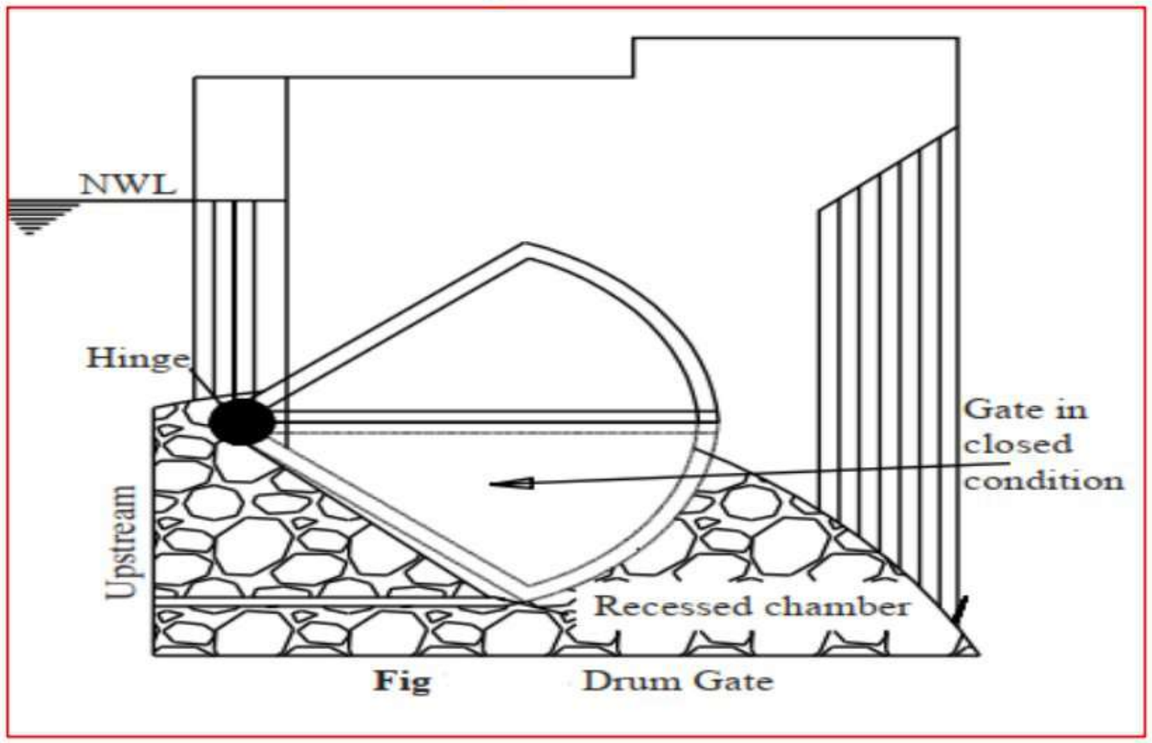

Fig Drum Gate

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Types of gates….

5. Flap gates:

- Flap gates are the bottom hinged gates which are assembled in simple manner and most commonly used type of regulating gates.
- Normally, these types of gate are used up to height 6m and span up to about 30m.
- For larger span, several flaps are connected to each other but each flap is attached with own hydraulic hoist.
- Flap gates provide high level performance for regulation, easily flushing of debris, floating materials and environmentally more acceptable than other type of gates.
- The main disadvantage of the flap gate is to require protection against freezing and sensitive to aeration demand and vibration.

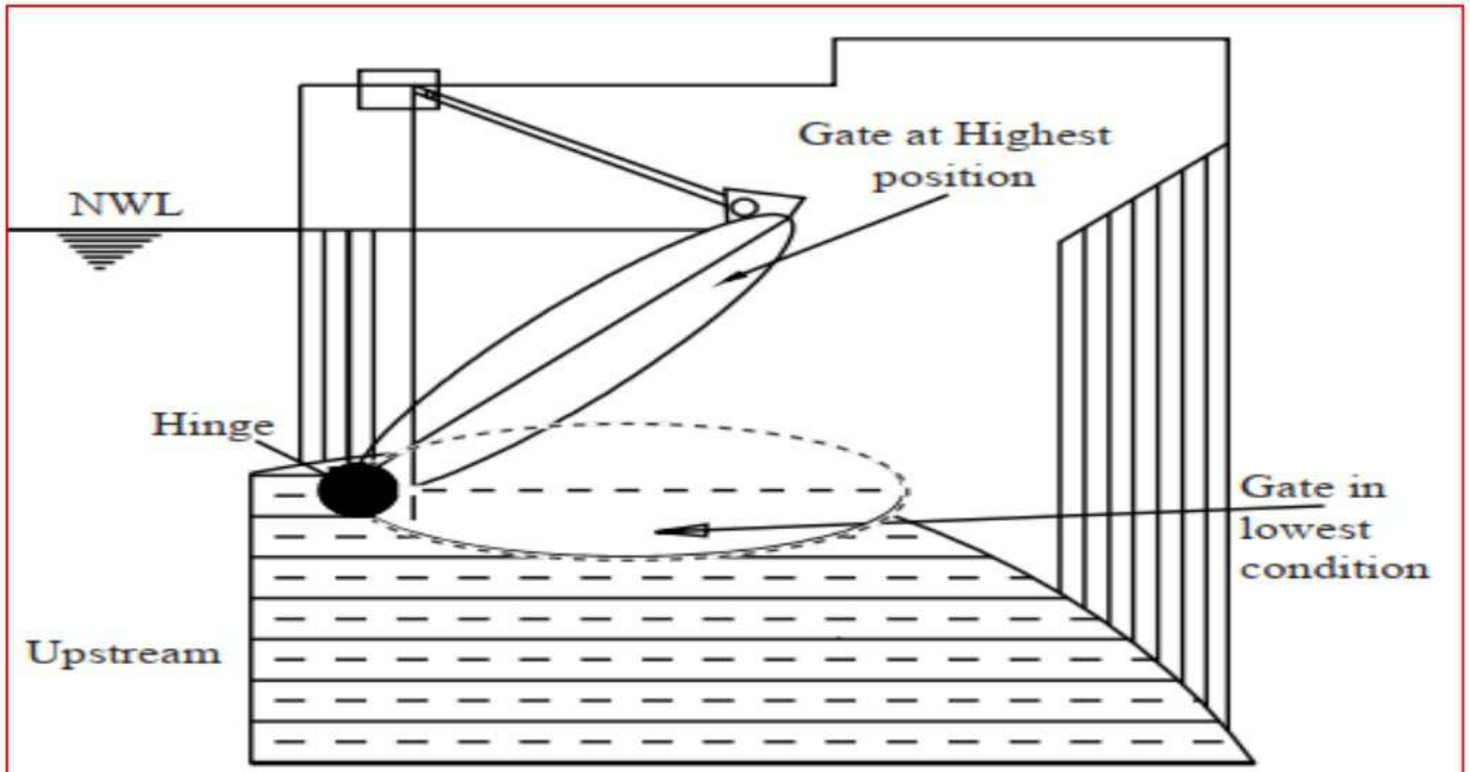

Fig . . Flap Gate

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Types of gates….

6. Rolling gate:

- Rolling gate consist of steel cylinder as large in diameter and height of opening and spanning between piers.
- A heavy annular rim with gear teeth on its periphery encircle the end of the cylinder.
- Each pier has an inclined rack which engages the gear teeth.
- From the hoist room with the help of a cable, the gate can be rolled up on the inclined rocks.
- A cylindrical segment attached to the lower portion of the gate makes the contact with spillway.

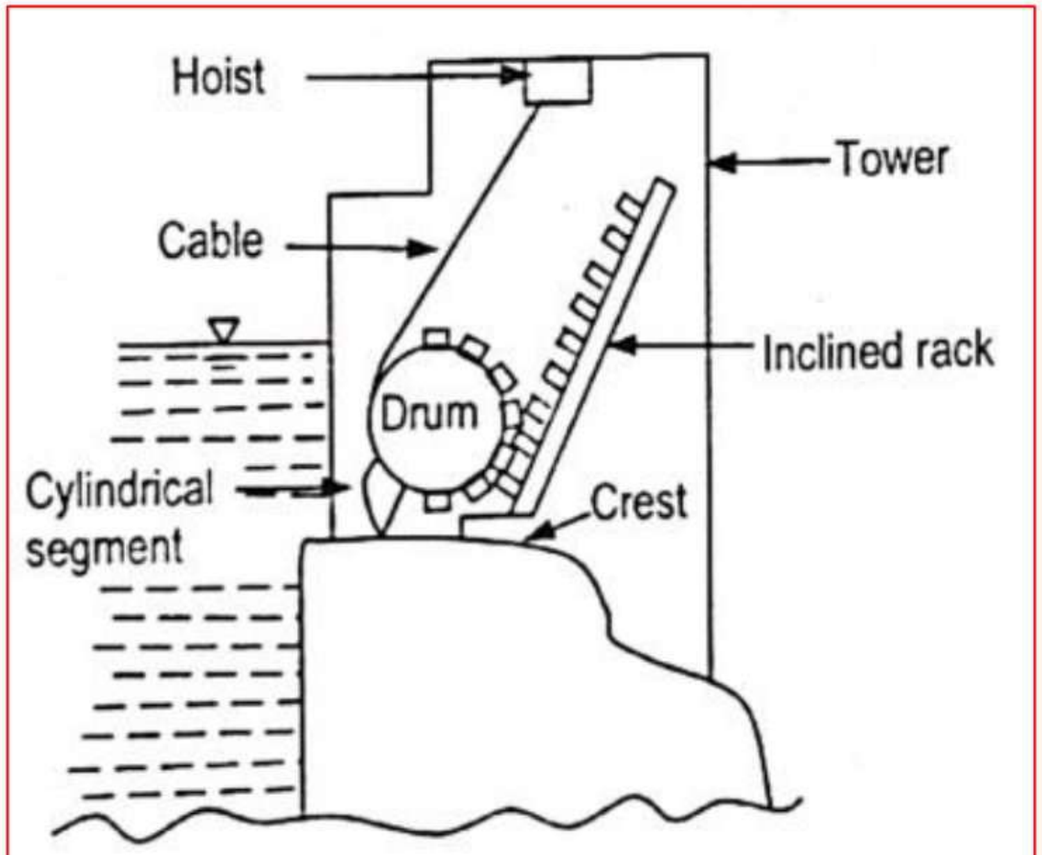

Fig: Rolling gate

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- **Headworks of Run-of- River (RoR) Plants [10 hours]**

5.1 General Arrangement of components of a typical RoR plant: Spillways, undersluices, intakes with examples

5.2 General requirements of a functional RoR headworks

5.3 Intakes of RoR headworks: Location, Non pressure and pressurized intakes, General arrangement of intake, Control of bed load and floating debris in RoR intakes

5.4 Sediment Handling measures: Methods of bed load and suspended load handling in RoR headworks, Design of settling basin (Particle and concentration approach), Estimation of sediment volume in Settling basin, Flushing of deposited sediment, estimation of frequency of flushing

5.1 General Arrangement of components of a typical RoR plant: Spillways, undersluices, intakes with examples:

The main components of run of river headworks with side intake are weir, intake, undersluices, divide wall, flood wall, gravel trap, approach canal and settling basin.

Intake : [Referred Unit 4]

Spillway: [Referred Unit 4]

Intake : [Referred Unit 4]

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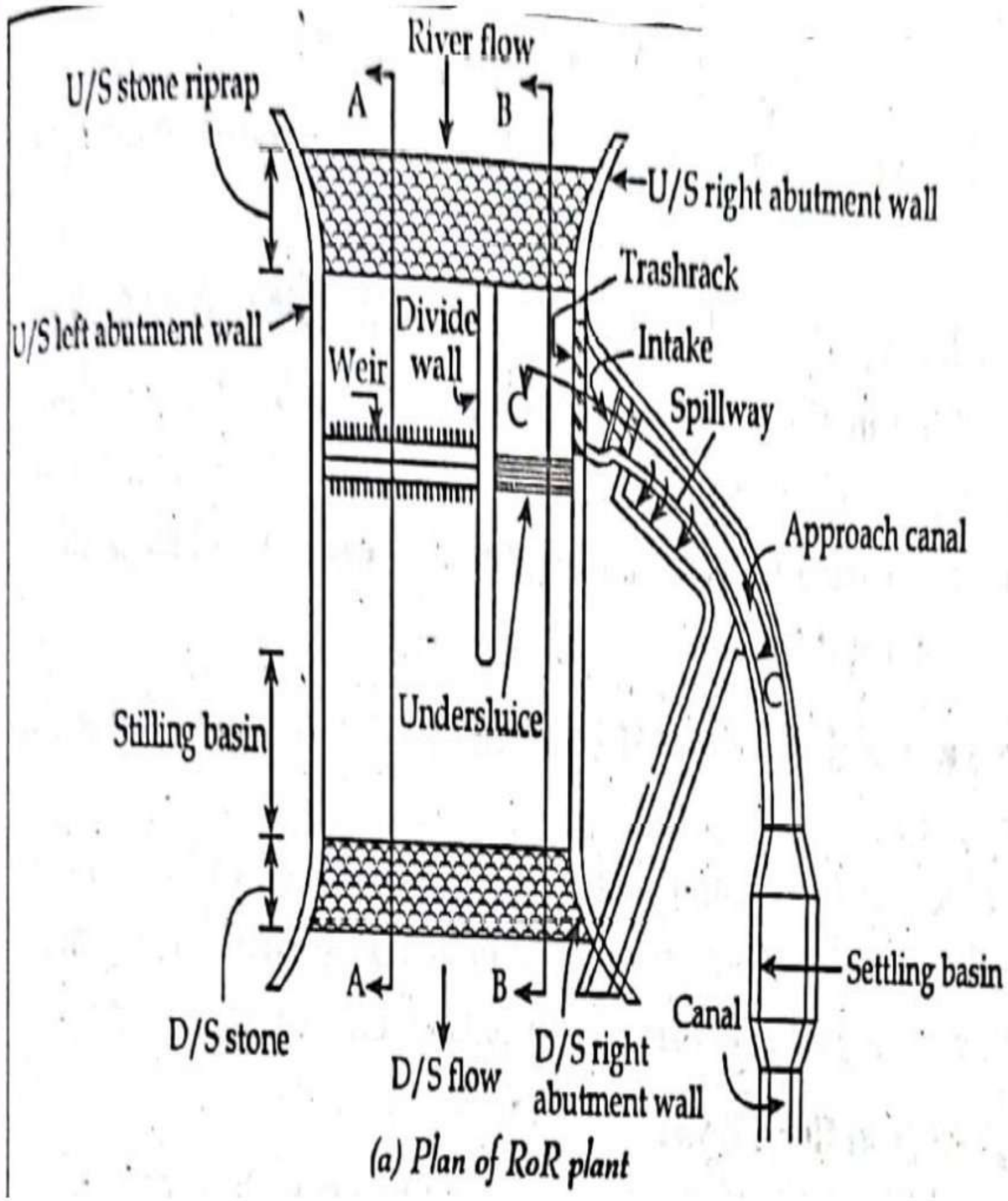
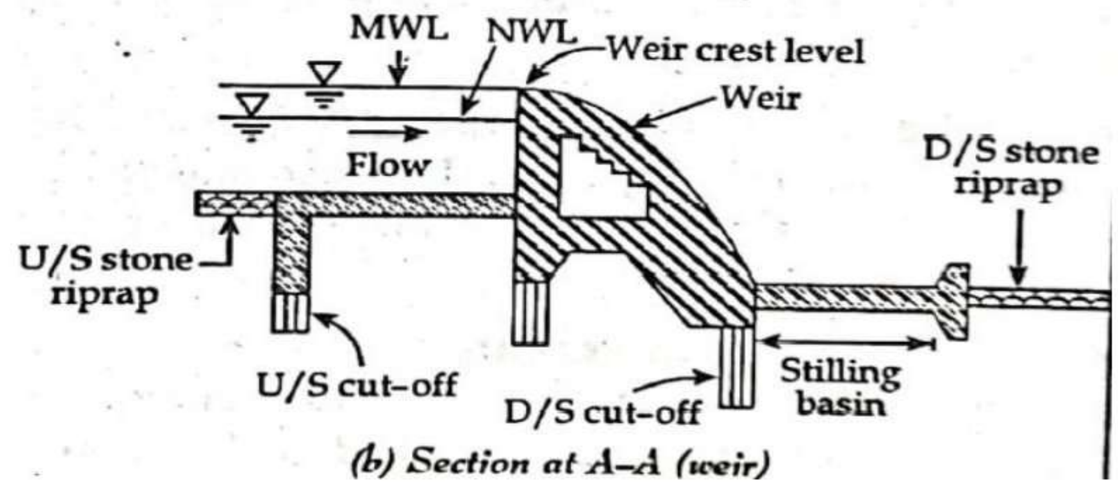
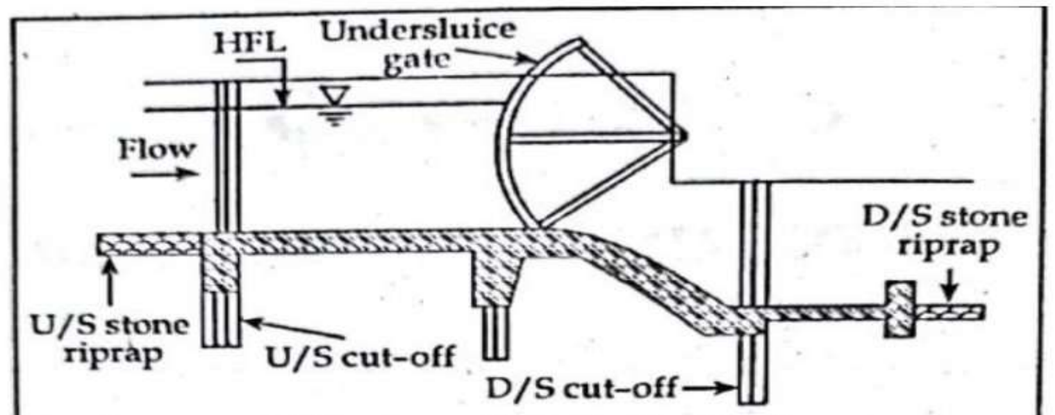
5.1 General Arrangement of components of a typical RoR plant: Spillways, undersluices, intakes with examples:



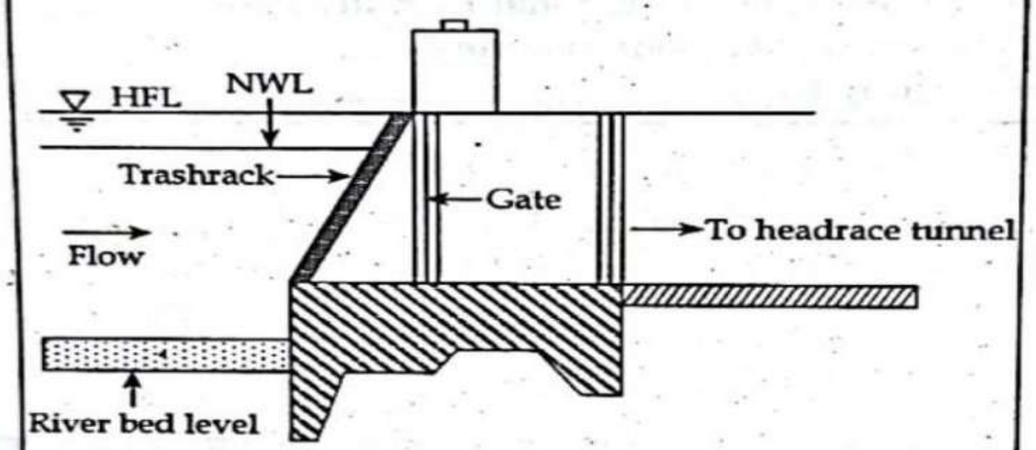
(a) Plan of RoR plant



(b) Section at A–A (weir)



(c) Section at B–B (undersluice)



Section at C–C (intake)

Figure: General arrangement of typical RoR plante

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Undersluices:

- The undersluice shall be provided to flush out the sediments deposited in front of the intake and thus control the bed levels in its approach area. Hydraulic design of the undersluice shall consist of determining its location, length, profile and the opening size.
- The undersluice shall be located close to the intake.
- It shall be provided in continuation with the overflow section, separated from the latter by a divide wall.
- The length of the undersluice shall be fixed based on the following considerations;

i. It shall be capable of passing at least twice the diverted discharge to ensure efficient flushing capacity.

ii. It shall be capable of passing about 10 to 20 percent of the maximum flood discharge at high floods.

iii. It shall be capable of passing fair weather freshets and low monsoon floods for obviating overtopping and /or operation of crest gates

- Based on the flow to be passed, the undersluice may consist of one or more bays. However, for small rivers flows, a single sluice bay may be preferred over two or more bays with smaller opening.
- The bay opening of the undersluice shall be sized to pass the largest possible boulders brought along by the river.
- Generally, the minimum opening shall be twice the boulder size expected in the river stretch in the vicinity of the headworks.
- The undersluice shall be designed as a broad – crested weir with downstream submergence. Its crest level shall be maintained close to the general ground level, about 2m below the intake invert level.
- The design shall be performed in accordance with the design of a flumed section.
- The capacity of the undersluice shall be sufficient under submergence conditions.
- Normally, energy dissipation downstream of the undersluice shall be achieved through a stilling basin with a downstream cutoff to check undermining.

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General requirement of a functional ROR Headworks

- The headworks should be planned and designed to ensure safe and regular power generation from the hydropower plant under normal conditions. Headworks should be fulfill the following requirements
- i) safe passage of flood flows
- ii) withdrawal of desired quantity of water from the river for power generation
- iii) safe passage of heavy sediments load
- iv) safe passage of floating debris and ice
- v) bed control at intake
- vi) flushing of settled sediments
- vii) minimum hydraulic losses
- viii) exclusion of suspended sediments and air

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Determination of Settling velocity (Fall velocity):

- The settling velocity or fall velocity of a sediment particle is defined as the rate at which the sediment settles in still fluid.
- It is diagnostic of grain size, but is also sensitive to the shape (roughness and sphericity) and density of the grain as well as to the viscosity and density of the fluid.
- It integrates all of these into a key transport parameter.
- The sediment will be descending with retardation due to varying resultant force, until the frictional resistance is fully developed.
- After the frictional resistance force is fully developed and then the sediment will be falling with constant velocity due to constant frictional resistance is called **fall velocity.**
- The force due to self weight of the particle is
- $F_{\text{drag resistance}} = (\rho_s - \rho)*g*$ volume of the particles
- $= (\rho_s - \rho)*g*\frac{1}{6}\pi d^3$ …(1)
- The force due to the motion of the particle in horizontal plane inside the fluid is given by
- $F_{drag} = C_D * A * \rho * \frac{{V_s}^2}{2}$ …(2)
- The particle will only move when equation (1) and (2) are equal.
- $(\rho_s - \rho)*g*\frac{1}{6}\pi d^3 = C_D * A * \rho * \frac{{V_s}^2}{2}$
- ${V_s}^2 = \frac{4}{3} * \frac{g}{C_D} * \left(\frac{\rho_s}{\rho} - 1\right) d \ \{\because \frac{\rho_s}{\rho} = G\}$

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Determination of Settling velocity (Fall velocity):…

- **Settling velocity** $(V_s) = \sqrt{\frac{4}{3} * \frac{g}{C_D} * (G - 1) * d}$ **…..(3)**
- Where, G = specific Gravity of the particle, ρ_s = specific weight of the particle, ρ = unit weight of water, C_D= coefficient of drag, d = diameter of particle
- **<u>For spherical particles moving in the water,</u>**
- Reynolds's number R_e <1,
- $C_D = \frac{24}{R_e}$ (Linear Relationship)
- For R_e between 1 and 1000,
- $C_D = \frac{24}{R_e} + \frac{1}{\sqrt{R_e}} + 0.34$
- For R_e >1000, C_D = 0.4 (Constant value)
- **<u>For fully turbulent flow,</u>**
- C_D = 0.4 and Settling velocity $(V_s) = \sqrt{3.33 * (G - 1) * d}$
- **<u>For laminar flow,</u>**
- Drag force due to viscosity (Stoke's law)
- $F_D = 3\pi\mu V_S d$ …..(4)
- Now equating equation (1) and (4), we get
- $(\rho_s - \rho) * g * \frac{1}{6}\pi d^3 = 3\pi\mu V_S d$
- $\therefore \boldsymbol{V_S = \frac{g}{18\vartheta}(G - 1)d^2}$

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Determination of Settling velocity (Fall velocity):….

- Where ϑ = kinematic viscosity
- $= \frac{dynamic\ viscosicity\ (\mu)}{\rho}$
- ϑ depends on temperature
- **For temperature**
- $V_S = 418(G - 1)d^2 \frac{(3t+70)}{100}$

Temp (°C)	0	5	10	15	20	25
ϑ (centistokes)	1.79	1.52	1.31	1.15	1.01	0.9

- **Note:**
- 1 Stokes = 1 mm^2/sec
- $= 10^{-4} m^2/sec$
- 1 Centistokes = 0.01 cm^2/sec
- $= 10^{-6} m^2/sec$

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1. Design analytical method for 0.2mm diameter of particle, T=25^{0}C, G= 2.65.Calculate the settling velocity.

- From question given as following data, Diameter of particle =0.2mm, T =25^{0}C, G= 2.65.
- We know, Fall velocity$(V_f) = 418(G-1)d^2 * \left(\frac{3t+70}{100}\right)$
- $= 418(2.65-1) * (0.2)^2 * \left(\frac{(3*25)+70}{100}\right)$
- $(V_f) =$ 40 mm/sec
- $\therefore$ Reynold's number$(R_e) = \frac{\rho d V_f}{\mu} = \frac{d V_f}{\vartheta}$,
- When temperature = 25°C, $\vartheta = 0.9$ centistokes $= 0.9 * 0.01 \frac{cm^2}{sec}$
- $= 0.9 * 0.01 * (10)^2\ mm^2/sec = 0.9\ mm^2/sec$
- $\therefore$ Reynold's number$(R_e) = \frac{\rho d V_f}{\mu} = \frac{d V_f}{\vartheta} = \frac{0.2*40}{0.9} = 8.89$
- R_e lies between 1 to 1000 which indicates the flow in transition so,
- $V_f = \sqrt{\frac{4}{3} * \frac{g}{C_D} * (G-1) * d}$,
- Where $C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34 = \frac{24}{8.89} + \frac{3}{\sqrt{8.89}} + 0.34 = 4$
- $\therefore V_f = \sqrt{\frac{4}{3} * \frac{9.81}{4.05} * (2.65-1) * \frac{2}{1000}}$
- $= 0.03264$ m/sec = **32.64 mm/sec**

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- which is less than the initial taken value 40 mm/sec so repeat this process as shown in table

S.N.	$V_f = \sqrt{\frac{4}{3} * \frac{g}{C_D} * (G-1) * d}$	$R_e = \frac{dV_f}{\vartheta}$	$C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34$	V_f
1	32.64	7.25	4.76	31.10
2	31.10	6.68	5.09	29.13
3	29.13	6.47	5.23	28.73
4	28.73	6.38	5.28	28.57
5	28.57	6.35	5.30	28.51
6	28.51	6.33	5.31	28.48
7	28.48	6.33	5.32	28.47
8	28.47	6.32	5.32	28.47

- $\therefore$ Settling Velocity or Fall Velocity $(V_f) = \mathbf{28.47}$ mm/sec
- For actual shape sediment taking 65% of theoretical velocity, i.e. Actual fall velocity = 0.65*28.47 = **<u>18.50</u>** mm/sec. i. e. Settling Velocity or Fall Velocity $(V_f) = \omega = \mathbf{18.50}$ mm/sec

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5.4 Sediment Handling measures:

Methods of bed load and suspended load handling in RoR headworks, Design of settling basin (Particle and concentration approach), Estimation of sediment volume in Settling basin, Flushing of deposited sediment, estimation of frequency of flushing

Sediment Handling Measures

- Following methods are adopted to control the deposition of sediment in ROR projects
- **1) Catchment management (Treatment):**
- Vegetation screens may be developed by promoting the growth of the vegetation in catchment as well as the entrance to the head works.
- **2) Control Sediment deposition**
- Designing and operating should be done in such a manner that water having higher sediment content is discharged to the D/S through the under – sluice.
- **3) Constructions of sediment excluder or sediment ejector:**
- sediment excluders are constructed in the river pocket.
- Sediment ejectors are constructed in the canal.
- 4) Construction of sloping intake
- 5) construction of sediment tank and gravel trap
- 6) Frequent and timely flushing provision

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Design of settling basin (Particle and concentration approach):

- Settling basin is the structure to remove suspended sediments form the conveyance water for power plant.
- Settling basins shall be designed to ensure that the water entering the water conveyance system is free of sediments that can damage the penstock and turbines runners due to abrasion.
- This shall be achieved by reducing the turbulence level in the water flow to allow suspended sediment particles to settle out from the water body and deposit on the bottom of the basin.
- The main principle of the design of settling basin is to reduce the mean velocity of the flow, by increasing the cross sectional area (widening its width and lowering the floor).
- Settling basins can remove suspended inorganic particles ranging from sand (2 mm in diameter) to silt (0.02 mm in diameter), depending on basic design.
- The finer sediments require longer settling times, hence, longer water transit times through the basin.
- It is usually not practical to remove small silt and clay particles (smaller then 0.002 mm).
- Without chemical coagulation, the sedimentation time required is extremely long.

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Purpose of settling basin:

The entry of sediment is controlled by the proper intake arrangements, such as by providing intake at outer bank, undersluice arrangement, position of intake bottom at certain height above the bed of the river, trash rack etc. and so on.

- The main purpose of settling basin is to remove sediments from the intake from such flood events. More specifically its objectives are

1. To remove the fine grained suspended matter from the water drawn from intake to ensure that the water entering the water conveyance system is free of sediments that can damage the penstock and turbines due to abrasion.
2. To remove the suspended particles and minimize the wear and tear of the nozzle and runners of the turbines and other hydro – mechanical components.
3. To remove the unwanted sediment particles entering into the conveyance system so that the conveyance capacity shall not be reduced in the course of time of operation of the plant.
4. To remove sediments as it causes abrasion and erosion of civil structures, gates, valves.
5. The excess sediment in the water causes the operation and maintenance difficult.

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Design principle and criteria:

The basic principle of settling is that the greater the basin surface area and the lower the through velocity, the smaller the particles that can settle

1. The settling basin shall be designed to be functional, easily operable and economical both for construction and operation.

2. The settling basin shall be designed to remove as much of sediment load in the water as is economically and hydraulically possible.

- As removal of all suspended sediments is physically not possible, the design shall attempt to remove as much of the coarser fractions of the suspended load as possible so that the hydraulic transport capacity of the water conveyance system can be maintained and the sediment load to the turbines, valves, etc., is reduced to acceptable limits.

3. The settling basin shall be designed to ensure efficient flushing of settled sediments so that frequent flushing during floods, when the sediment content of rivers is at its peak, is not required.

- The settling basin shall be planned and designed such that power generation is not interrupted, or reduced, during flushing operations.

4. A settling basin must satisfy the **following design considerations**

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Design Consideration:…

a) Settling capacity:

- The size of the basin must be large enough to allow a large percentage of fine sediment to fall out of suspension and be deposited on the bottom.
- The geometry of the inlet and outlet transitions and any other curvatures must be such as to cause minimum turbulence, which might increase the trapping efficiency of the basin.

b) Storage capacity:

The basin should be able to store the settled particles for some time unless it is flush out.

c) Flushing Capacity:

The basin should be able to flush all these settled particles along with the incoming flow in the basin by operating flushing gates or valves.

The following factors should be considered in the design of the settling basin;

i. Characteristics of suspended sediments, particles of hard or soft rock or soil origin like quartzite, granite, basalt, gneiss etc. (harder the sediment minerals smaller the particle to be remover).

ii. Concentration of sediments with the river flow.

iii. Required removal percentage of suspended sediments.

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Components of settling basin

Settling basins shall consist of the following components

- An inlet transition including gated arrangement
- Settling chamber/s to settle the sediments in the incoming water
- An emergency spillway to spill the settling basing design discharge into the parent river, if required
- An outlet transition with regulator to control the flow of water from the settling chamber
- A flushing channel for flushing the settled sediment.

1. Inlet transition:

- The inlet transition for settling basins shall be designed to prevent turbulent flow at the entrance to the settling basin chamber.
- The approach canal to the settling basin shall have a straight alignment for a stretch equal to about 10 times the width of the canal upstream of its junction to maintain an even flow distribution at the start of the inlet transition.

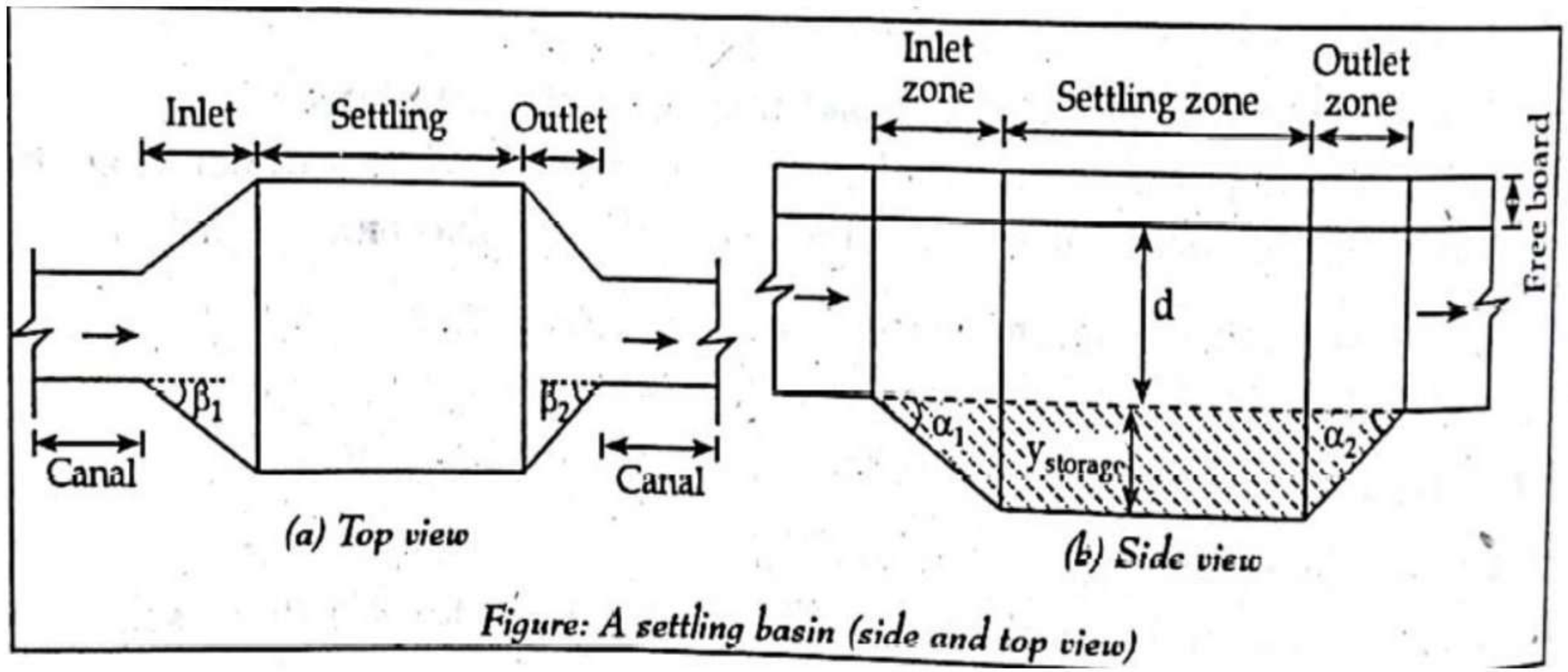

Figure: A settling basin (side and top view)

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Components of settling basin…

1. Inlet transition:

- The hydraulic design of the canal shall eliminate secondary currents in its flow caused by rotational flow. It shall also ensure flow velocities in the range of 1.1 to 1.3 m/s.
- To maintain gradual expansion of inlet channel, set the horizontal expansion ratio at about 1:5 (or α = 7° to 15°). The vertical expansion ratio can be set at about 1:2 (or α = 20° to 30°).
- If space constrains do not permit the long transition, the inlet transition may be shorten through guide walls in the transition.
- If space constraints do not permit a straight section of approach canal, even flow in the settling basin may be achieved using a tranquilizer or baffle blocks.

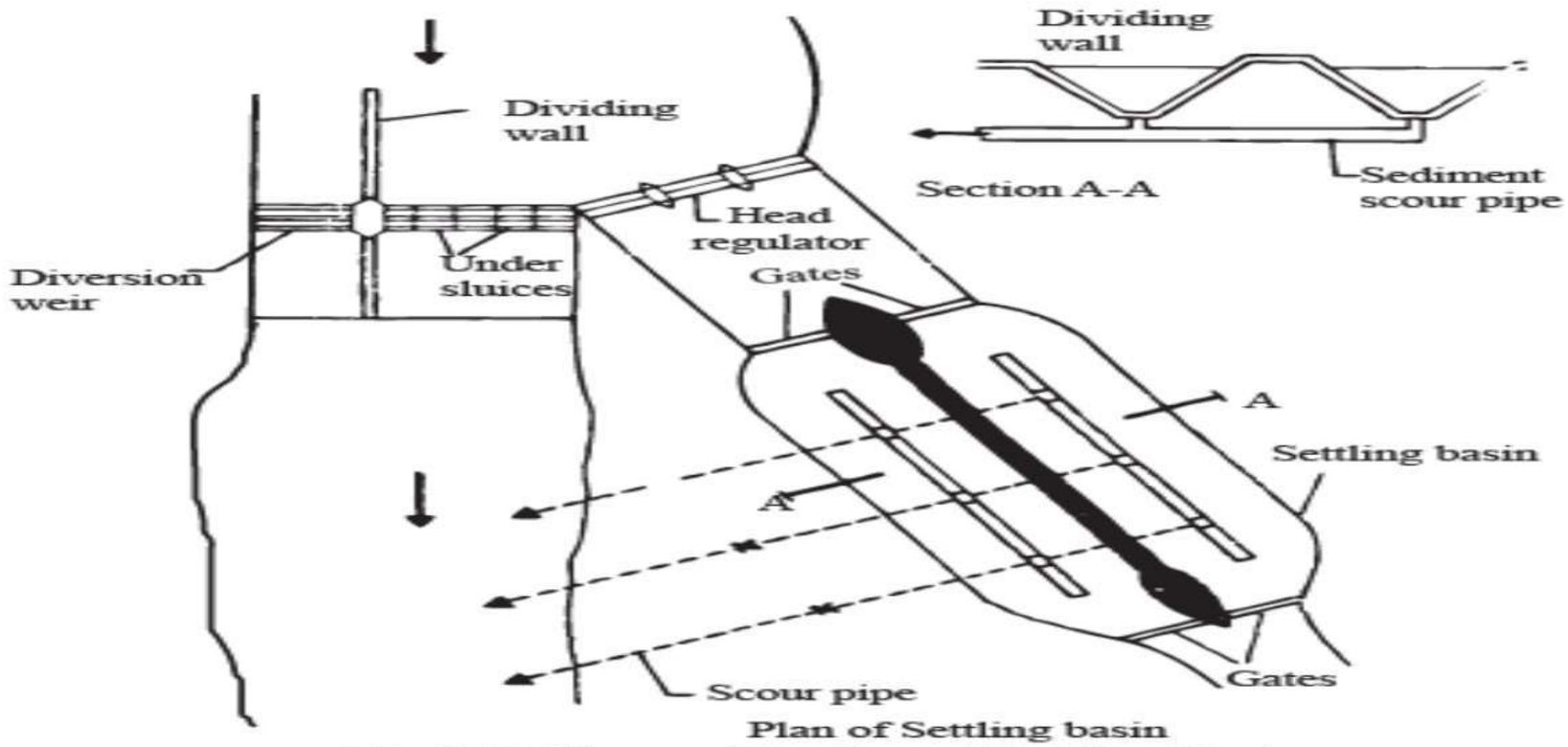

Fig 22.1 Plan and Section of Settling Basin

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Components of settling basin…..

2. Settling chamber:

The hydraulic design of the chamber should be carried out to secure

- An even flow distribution between parallel settling basins for various flows.
- An even flow distribution internally inside each basin for various flows.
- Efficient removal of deposits during flushing of the basin.
- Design parameters

➢ Mean depth of flow in the basin
➢ Mean longitudinal velocity
➢ Fall velocity
➢ Flushing discharge
➢ Sediment concentration of flow

3. Outlet zone

- The operation water level of the settling basin is controlled at the outlet zone.
- The outlet zone is designed for decanting the outflow from the settling basin over a small weir/ spillway or simple transition structure.
- The major parts included in the outlet zone are; transitions to headrace/ forebay inlet with or without control gate mechanism, sediment flushing gates/ valves arrangements.
- The outlet transition can be more abrupt than the inlet expansion.
- The horizontal and vertical expansion ratio can be set at about 1 : 2 and 1 : 1 respectively.

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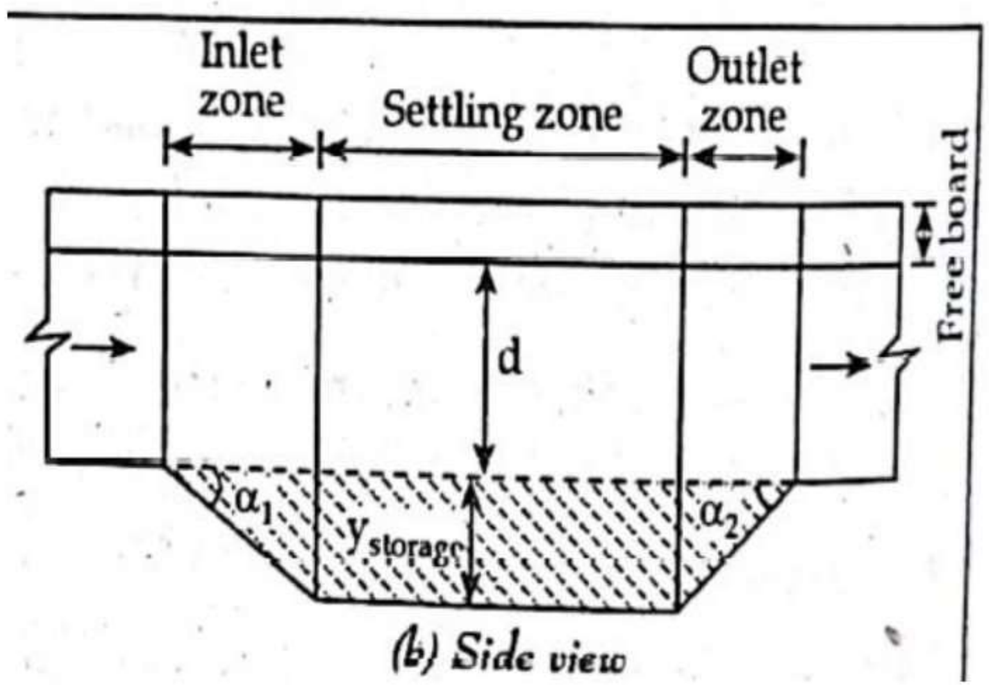
 # Components of settling basin…..

4. Sediment flushing system

- The flushing arrangement should be provided to flush the sediment deposited in the basin.
- It should ensure that when the flushing gate or valve is opened, the water level in the basin decreases and the sediment deposited in the bed starts flushing out.
- To facilitate the flushing of sediment, the bed slope at the bottom should be 1:20 to 1:50 in longitudinal direction.
- The bed can be provided with a flat or under sluicing manifold system with purging channel.
- The under sluicing of flushing system is designed as a manifold system to withdraw sediment laden water uniformly along the length of the purging channel.
- The design should be based on pressurized flow.
- The flow velocity should be more than 1.75 m/sec and not higher than 2.5 m/sec for concrete structure of flushing conduit.



(b) Side view

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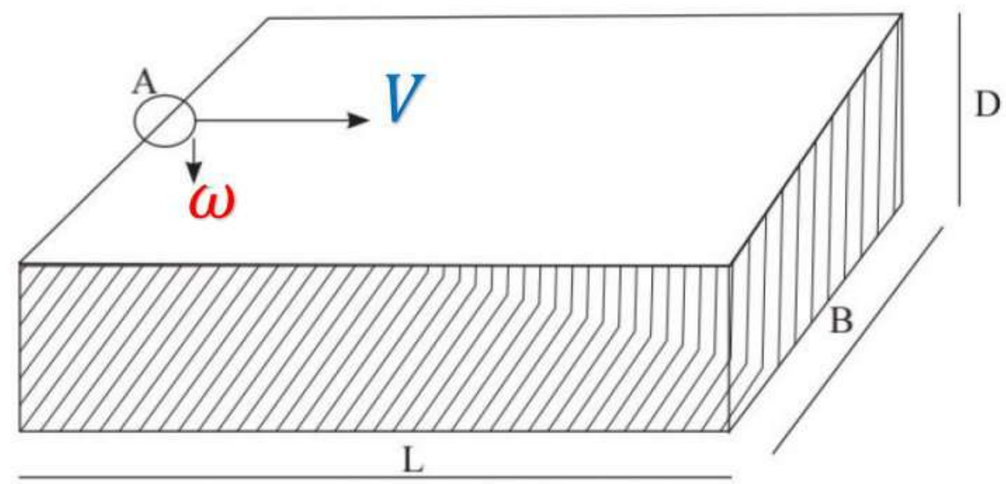
Design of Settling (Desanding) Basin

Ideal Settling basin (Particle Approach):

Hazan developed basic relation based on particle approach of an ideal basin.

Following assumptions are made for an ideal horizontal settling basin.

- The flow is steady and uniform.
- The flow is quiescent (no turbulence).
- Solid entering in deposition zones are not suspended.



Settling time (Ts) = D/ ω…… (1)

Detention time (T_D) = L/V …..(2)

Where ω = settling velocity (fall velocity)

V= horizontal velocity (flow velocity)

- For quiescent liquid; all particles with settling velocity (ω) are removed when detention time equal to settling time
- D/ ω $= L/V = \frac{LA}{Q} = \frac{LBD}{Q}$
- $\boldsymbol{\omega = \frac{Q}{LB} = \frac{Q}{A_s}}$
- The ratio $\frac{\omega A_s}{Q}$ can be regarded as a dimensionless indicator of the physical ability of a basin of surface area A_S to remove particles of fall velocity ω at supply discharge Q. The ratio $\frac{Q}{A_s}$ is termed as surface loading.

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Settling basin design

For design of settling basin, the following parameters need to be calculated by the relations.

1. Calculate surface area (A_S)

$$(A_S) = \frac{KQ}{\omega_t}$$

Where A_S = the required surface area of a settling basin

ω_t= target sediment particle size with a fall velocity

Q = the design flow rate

K = coefficient to account for turbulence (value of K is taken 1.2 to 1.5)

2. Calculate L and B using the relation L/B 4 to 10

A_S = L*B; and check for necessary length and breadth using other approaches.

3. Calculate the limiting flow velocity (V) in the basin

Calculate the limiting flow velocity (V_C in m/sec) in the basin based on the particle size to be settled is given by

$$V_C = a\sqrt{d}$$

Where, d = size of gravel to be removed in mm and

a = 0.36 for d > 1mm

a = 0.44 for 1mm > d > 0.1mm

a= 0.51 for 0.1mm >d

The horizontal velocity of flow must not be so great as to prevent, by turbulence, the settling particles under gravity. The maximum horizontal velocity of flow to allow the sand grain to fall out should not be greater then 0.3 m/sec.

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Settling basin design….

4. Calculate depth (D) of basin

The depth of the basin (D) is calculated by using the equation of the horizontal velocity i.e.

$$(V) = \frac{Q}{BD}$$

$$(D) = \frac{Q}{BV}$$

Generally, the depth is assumed based on the construction easiness criteria such as more theoretical depth may lead to difficult in construction as well as operation.

5. Check the calculated values of L, B, D

Knowing L, B, H check these parameters from other methods

Check width $B = 4.75\sqrt{Q}$

i) Length of basin considering turbulence effect

The fall velocity of the particle is reduced by the turbulence effect this results in larger basin length to settle the targeted particle in a desire limit.

The fall velocity $\omega_1 = \omega - \omega'$

$$\omega_1 = \omega - \alpha V$$

Where $\alpha = \frac{0.132}{\sqrt{D}}$

Now, Setting time $t_s = \frac{D}{\omega}$ and Travelling time $t_t = \frac{L}{V}$

For a particle to settle within basin, settling time (t_s) should be equal to travelling time (t_t) so

$$\frac{D}{\omega} = \frac{L}{V}$$

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Settling basin design….

5. Check the calculated values of L, B, D….

So length of basin (L) = $\frac{DV}{\omega}$

Now considering turbulence effect, (L) = $\frac{DV}{\omega - \omega'} = \frac{DV}{\omega - \frac{0.132V}{\sqrt{D}}}$

$$\mathbf{L} = \frac{VD^{1.5}}{\omega\sqrt{D} - 0.132V}$$

ii) Settling length for turbulent flow is computed from the settling velocity in stagnant water ω and the flow – through velocity is calculated using the ***M. A. Velikanov's formula***

$$L = \frac{\lambda^2 V^2 \left(\sqrt{D} - 0.2\right)^2}{7.51\,\omega^2} \text{ (m)}$$

Where, λ depends on the removal ratio, defined by the function

$\lambda = f(W)$

Where, W denotes the ratio of settled sediment to the total load entering with the flow and can be computed from, $W = 100 - 100\frac{C_p}{C}$ percent

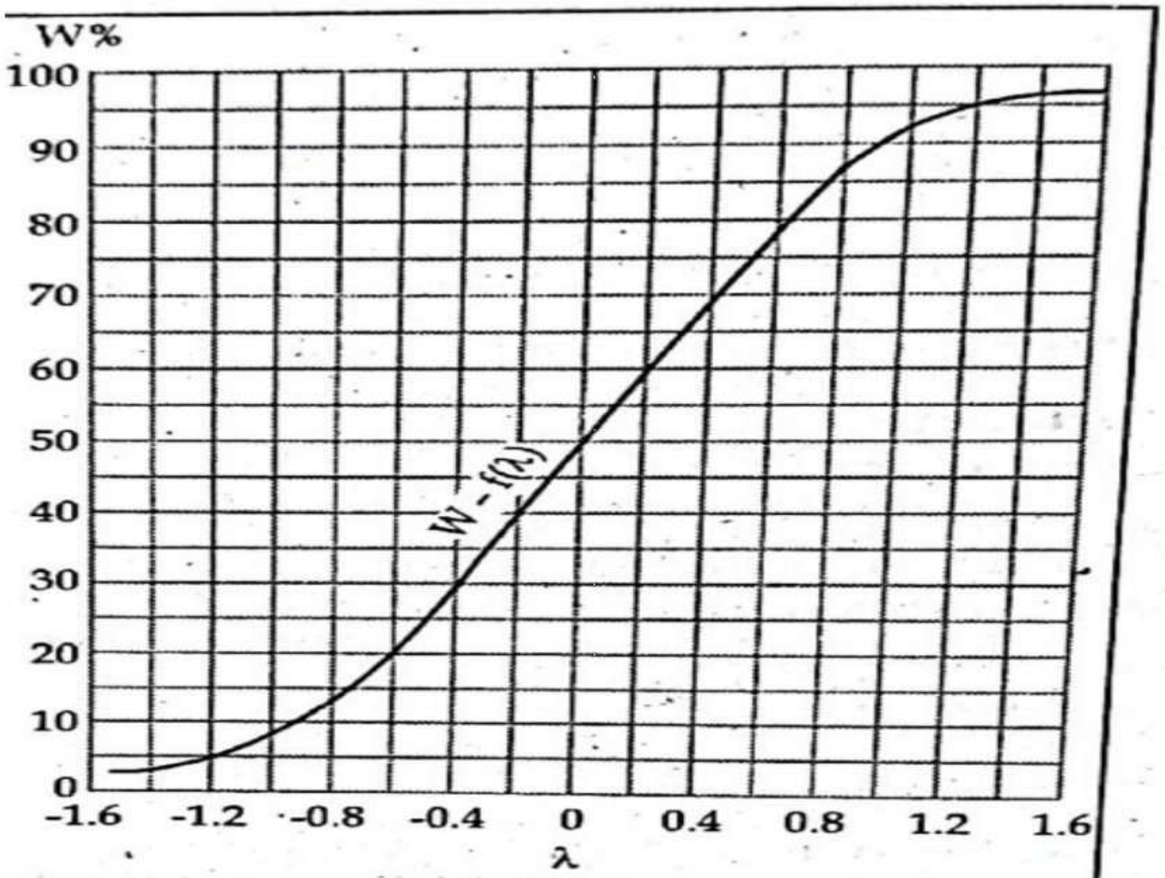

Figure: Velikanon's relationship W = f(λ) for designining settlng basins

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Settling basin design..

6. Check the efficiency of the particle settled

- Turbulence velocity fluctuations, especially those in the vertical, will result in mixing and a reduced effective average settling rate.
- Turbulence will tend to keep sediment suspended, and hence reduce the basin efficiency.
- Check the efficiency of the particle settled using Hazen's, Camp's and Vetter's other methods. If desired percentage of the particle settled is within the limit, the design is ok otherwise repeat the procedure changing the dimensions.
- **a) Hazen equation:**
- $\eta = 1 - \left(1 + m\frac{\omega}{V_0}\right)^{-1/m}$
- The parameter, m is used to quantify performance of the settling basin, where m = 0 represents best performance, and m = 1 represents very poor performance. m is therefore used to represent all of the non – linear effects, but in itself is not based on any physical characteristics.
- **b) Vetter equation:**
- Vetter's method for determining the sediment trapping efficiency, based on the sediment concentration approach and simplified version of Hazen's method, is expressed by the formula
- $\eta = 1 - e^{-\left(\frac{\omega A_S}{Q}\right)}$, Neither of these methods quantifies the degree of turbulence based on hydrodynamic criteria.

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Settling basin design…….

c) Camp (1996):

The sediment trapping efficiency, η, based on the sediment particle approach for design of the settling basin is found from Camp's diagram, considering the dimensionless parameters (ω /u*) and ($\omega A_S/Q$), where 'w' is the settling velocity of sediment particles and u* is the shear velocity that can be found theoretically using Manning's formula

$$u^* = \sqrt{gRS_e}$$

Where g is the acceleration due to gravity, R is the hydraulic mean depth of flow, n is the Manning's roughness coefficient, A is the cross – sectional area of flow of the basin and Se is the energy gradient by

$$S_e = \left(\frac{Q}{nAR^{2/3}}\right)^{1/2}$$

From the practical case, the shear velocity shall be determined form the following formula instead of from the manning's equation shown above.

$$u^* = \frac{0.042V_m}{AR^{1/6}}$$

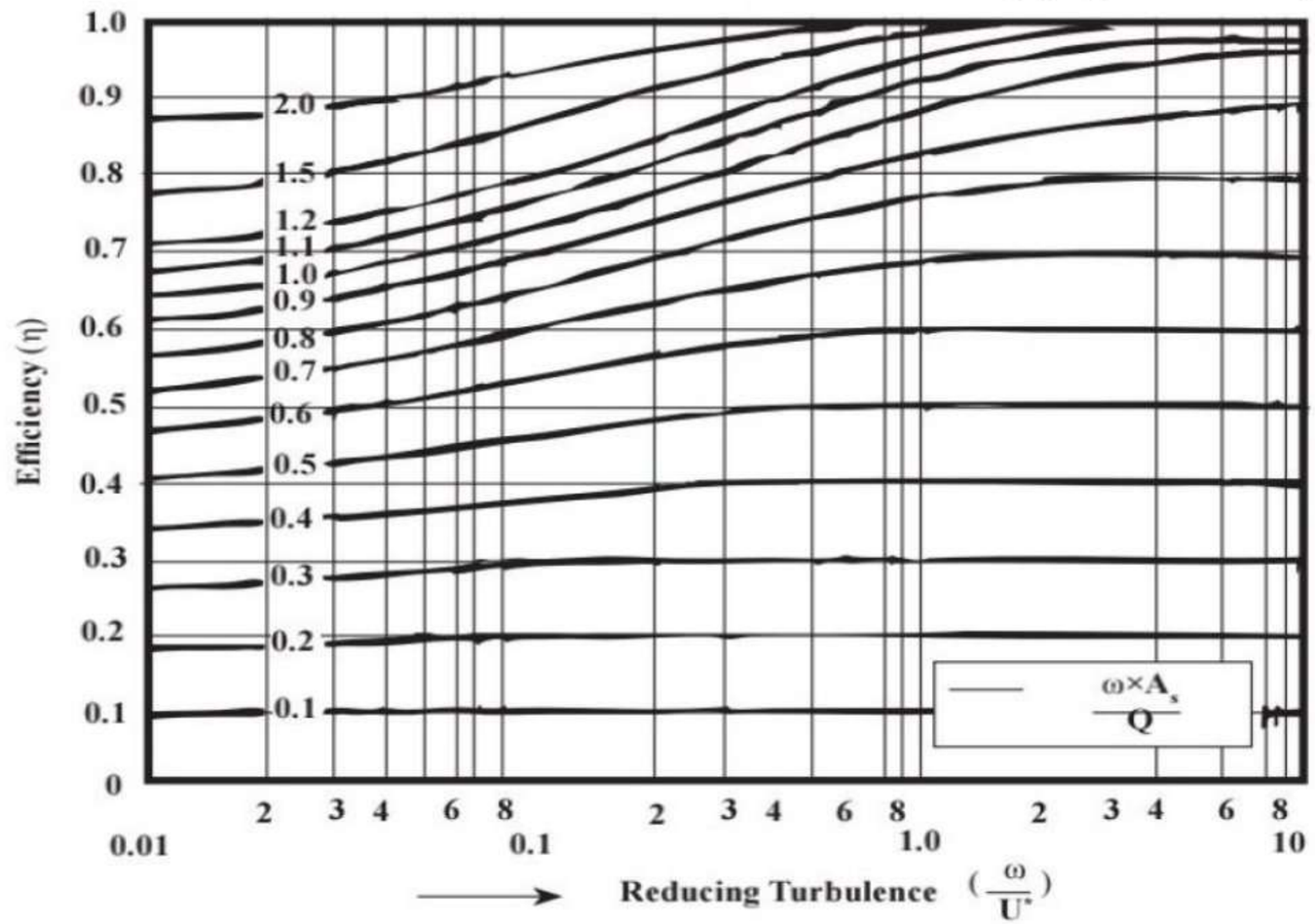

Fig Camps diagram for trap efficiency

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Settling basin design….

7. Compute the sediment depth in the basin:

Sediment depth is calculated for design discharge (Q) m3/sec, with sediment concentration C kg/m3 and detention time for sediment (T, generally 6 hours) is calculated using following method.

So, Theoretical value of sediment load (in Kg) = Q*T*C

Apply packing factor of sediment in submergence

$$\text{Volume of sediment} = \frac{\text{Sediment load}}{Density * Packing\ factor}$$

Packing factor of sediment submerged in water = 0.5 (50%)

We have, basin plan area = $\mathbf{A_S}$

So, depth of sediment $\mathbf{H_S}$ = volume of sediment / basin plan area

Since, the above calculation is considered for the case of rectangular base, we can recalculate the area for the base with the sloping shape with the angle of inclination of the slope = 40° when the area resembles with rectangular base.

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Types of Settling basin on flushing operations

Based on flushing operational consideration the settling basin can be divide into two types

1. Continuous flushing Settling Basin (Dufour, Bieri types)
2. Discontinuous flushing Settling Basin

1. Continuous flushing Settling Basin:

Settling basins with continuous flushing shall be designed to supply sediment – free water to the water conveyance system through simultaneous settling of the suspended sediments and flushing of deposited sediments. The flushing shall be achieved by continuously abstracting water from the bottom of the settling basin during operation.

i. Use surplus water (10 – 15% of plant design flow) during rainy season for flushing.

ii. More convenient from a power plant operation view point as flushing operations do not interfere with power production.

iii. Flushing can be done intermittently during flow season.

iv. To improve reliability and enhance operating flexibility, it can usually be constructed with a minimum of two basins.

v. Much more care is required for its operation to minimize the clogging of sediment extracting system.

- It has one or more chambers.
- More than one chamber may be desirable to enable dewatering of one basin during the dry season for inspection and maintenance of the flushing system, etc. without affecting the operation of the plant.

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Types of Settling basin on flushing operations….

1. Continuous flushing Settling Basin:….

- Alternately, single chamber settling basin with a bypass channel for this purpose may be provided.
- This settling basin shall be provided with a series of bottom galleries along the length of its chamber to pass settled sediments.
- The flow area of these galleries shall be increased along the length to accommodate the increasing flushing discharge.
- The channel shall be covered with perforated screens kept horizontally and flush with the chamber bed.
- A flushing gallery shall be provided at the end of the settling basin chamber.
- The crest of the flushing gallery shall be placed at a depressed level to create unsubmerged flow regime in it.

2. Discontinuous flushing Settling Basin

- Settling basin with periodic flushing shall be designed to operate in two distinct phases.
- In the first phase, the suspended sediments in the abstracted water shall be permitted to settle in the settling basin and water that is free form as much of the sediments as possible shall be conveyed to the water conveyance system.
- In the second phase, the deposited sediments shall be hydraulically removed form the settling basin through a flushing system using gravity flow of water at high velocities.
- During the flushing, water supply to the water conveyance system may be either be stopped or alternately channeled through another settling chamber or bypass system.

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Types of Settling basin on flushing operations…

2. Discontinuous flushing Settling Basin…..

i. Simple in design and are much less susceptible to blockage or clogging of sediment extracting system.

ii. Main operating inconvenience is that plant output must be cut back, for multi basin design or shut down entirely for single basin design.

iii. Flushing is only required when the settling basin is over loaded than the design levels of sediments.

iv. Releases much larger flushing flows than continuous type and do so suddenly. This can pose a significant hazard to downstream water users.

- It normally consist of two or more chambers. These chambers shall be separated from each other by longitudinal divide wall.
- These walls shall have a rectangular or trapezoidal cross – section and their top level shall be fixed providing a freeboard of 0.3 to 0.5m above the full supply level, considering the mode of plant operation.
- The flushing gallery shall be provided at the end of the settling basin chamber across its entire width. The bed of this gallery shall lie below the flushing crest.
- Deformation joints along the length of the settling basin may be provided in concrete and reinforced concrete structures in accordance with standard particle. These joints shall be sealed with water stops.

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Different Types of Settling basin on flushing operations

A. Conventional type of Settling Basin

- The basin is dewatered when it is taken out of operation.
- Sediments may be removed manually or with mechanical equipment after the basin is dewatered.
- Deposited sediments may also be removed by lowering water level inside the basin and generating swift flow in free surface gravity flow through the basin. i.e. conventional flushing system.
- The power plant is closed when flushing operation is continued.
- So two settling chamber is constructed for continuous operation of the plant.

B. Hopper type settling basin:

- Water level and water flow is maintained in the basin throughout the flushing period in order to facilitate continuous power generation.
- Removal of sediments while the basin is operational may be achieved with continuous flushing or intermittent flushing or by use of some kind of suction or dredging device.
- In order to remove the settled particles by flushing, it is necessary to generate a current close to the particles, able to erode and carry the particles away with the flushing flow without mixing them with the main water flow in the basin.

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Different Types of Settling basin on flushing operations….

C. Bieri type

- In Bieri type of settling basin, the shutter mechanism in the bottom of the basin is made of two plates with a series of opening.
- One plate is fixed while other can be moved horizontally.
- Flushing is facilitate when openings in each plate falls together.
- Sensor can be places in the bottom of the plate so as to sense the volume of sediment deposited in the chamber.
- One the sediment collected increases the desired level of deposition, one plate moved over another and sediment is flushed. And after flushing the plate is again moved and flushing operation stopped.
- This type of settling basin is exists in Middle Marsyandi hydropower project.

D. Serpent Sediment Sluicing System (4S):

- Serpentine system is a new type of Desanding basin flushing system which was developed by Hakoon Stole at the Norwegian institute of science and technology.
- This type of settling basin is constructed in Andhi khola, jhimruk khola and Khimti khola hydropower plants in Nepal.
- One or more flushing channels are located at the bottom of the settling basin.
- The flushing channel is connected to the settling basin by a longitudinal slot.
- A float unit (The serpent) covers the slope in downstream end.
- A channel is connected to a flushing it.

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Different Types of Settling basin on flushing operations

E. Split and Settle

- The split and settle concept is new approach to exclusion of sediment in water withdrawn from rivers with high sediments loads.
- The basic principles of design of split and settle type of settling basin is the concentration of sediment is not uniform over the cross section of a water conduit.
- The concentration is higher close to bottom than higher up in the water column in the low in an open channel or a pressurizes tunnel flow.
- The split and settle concept takes advantage of this difference in sediment concentration river depth.
- The flow in the tunnel upstream of the settling facilities (the approach flow) is split horizontally at the first tunnel cross.
- The bottom water contains relatively more sediments than the water higher up.

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1.) Compute the dimension of settling basin for a hydropower plant using the simple theory with the data given below

Design discharge = 5 m3/sec, particle diameter to be removed = 0.2mm, Depth of basin = 3m, Settling velocity = 2 cm/sec. if turbulence is considered, what will be the dimension of basin? [2064 Poush]

Solution:

- $Q = 5m^3/s$, d= 0.2mm, $\omega = 0.02m/s$, D = 3m
- Limiting velocity $(V_c) = a\sqrt{d} = 0.44\sqrt{0.2} = 0.197\ m/sec$
- **<u>i) without considering turbulence effect</u>**
- Length of settling basin$(L) = \frac{D*V}{\omega} = \frac{3*0.197}{0.02} = 29.55$ m
- Also, Width of basin (B) $= \frac{Q}{D*V} = \frac{5}{3*0.197} = 8.46$ m
- Size of settling basin without considering turbulence effect = **29.55 m* 8.46 m* 3 m**
- **<u>ii) With considering turbulence effect</u>**
- $\therefore$ Length $= \frac{D^{1.5}*V}{(D^{0.5}\omega)-0.132V} = \frac{3^{1.5}*(0.197)}{(3^{0.5}*0.02)-(0.132*0.197)} = 118.52$ m
- Check for Length (For 95% removal $\lambda = 1.2$)
- $L = \frac{\lambda^2 V^2(\sqrt{D}-0.2)^2}{7.51\ \omega^2} = \frac{1.2^2*0.197^2(\sqrt{3}-0.2)^2}{7.51\ (0.02)^2} = 43.66$ m < 118.52 m Hence OK
- Size of settling basin with considering turbulence effect = **118.52 m * 8.46 m* 3 m**

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2) Find out the dimension of a continuous flushing settling basin for a high head project in Himalayan River which utilizes a discharge of 60 m3/sec and head of 300 m the sediment particle larger than 0.15 mm have to be trapped efficiency 95 % in the basin. Consider the effect of the turbulence and check the length of basin using Velikanov's relation of the density of the silty water of 1.105 ton/m3. [2070 Chaitra]

Solution: Q = 60m^3/s, d = 0.15mm

For continuous flushing settling basin type 10 % of plant discharge is taken as flushing discharge

- = 1.1*60 = 66 m^3/s
- Discharge through one Basin (Q_T)
- = 66 /2 = 33 m^3/s

For density (ρ) = 1.105 ton/m3 and particle size 0.15 mm settling velocity (ω) = directly read from the graph is,

(ω) = 1.5 cm/s = 0.015 m/s

- For, high turbulence case the trapping
- efficiency of settling basin is given by,
- $\eta = 1 - e^{-\left(\frac{\omega A_S}{Q}\right)}$
- 0.95 = $1 - e^{-\left(\frac{0.015 * A_S}{33}\right)}$
- We get, A_s = 6590.611 ≈ 6600 m2

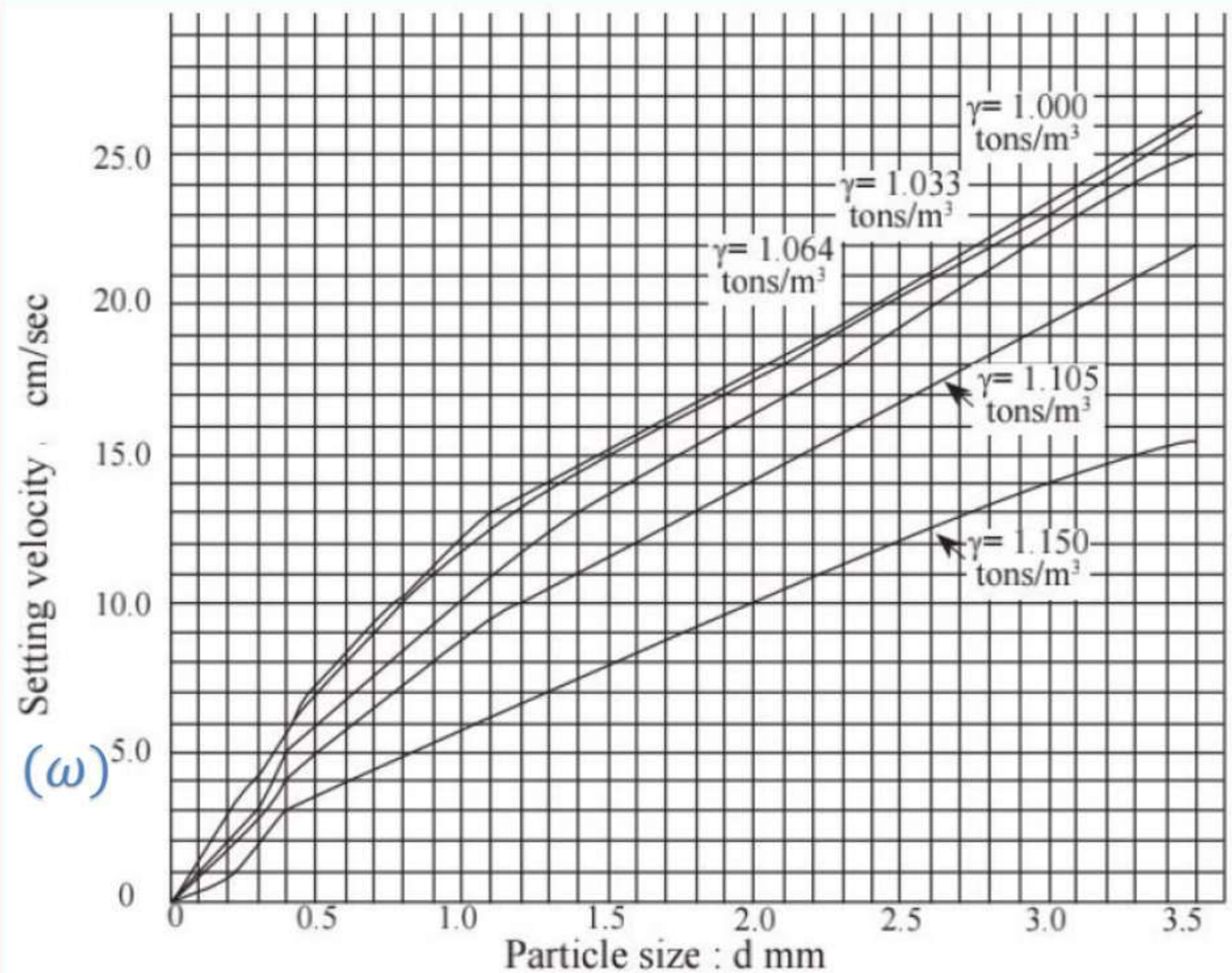

Fig Setting Velocity in Stagnent water plotted against the Density of silty water and particle diameter
(After L. Surdy)

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- Assume $\frac{L}{B} = 9 \therefore L = 9B$
- We know $A_S = L * B = 9B * B = 9B^2$

$6600 = 9B^2$

$B = 27.08$

Check $B = 4.75\sqrt{Q_T} = 4.75\sqrt{33} = \mathbf{27.286} > 27.08$m Hence OK

$\therefore L = 6600 / 27.08 = 243.722$ m $\approx$ **244 m**

Limiting velocity $(V_c) = a\sqrt{d} = 0.44\sqrt{0.15} = 0.17$ m/sec

- Depth of settling basin(D) $= \frac{Q_T}{V*B}$
- $= \frac{33}{0.17 * 27.08} =$ **7.168 m**
- <u>With considering turbulence effect</u>
- $\therefore$ Length (L) $= \frac{D^{1.5}*V}{(D^{0.5}\omega)-0.132V}$
- $= \frac{7.168^{1.5}*(0.17)}{(7.168^{0.5}*0.015)-(0.132*0.17)} =$ **184.1 m**
- Length of basin Velikanov's relation is, (For 95% removal $\lambda = 1.2$)
- $L = \frac{\lambda^2 V^2 (\sqrt{D} - 0.2)^2}{7.51\ \omega^2}$
- $= \frac{1.2^2 * 0.17^2 (\sqrt{7.168} - 0.2)^2}{7.51\ (0.015)^2} = 151.14$ m
- **Final dimension of settling basin = 244 m*27.08* 7.168 m**

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3) Design a settling basin using following data ; Discharge = 10 m3/sec, particle size to settle = 0.2mm, particle fall velocity (ω) = 0.02m/s, horizontal velocity (v) = 0.15m/s.

Use two basin for flushing purpose i.e. while one flushing the other basin can run, use factor of safety for basin area (K) = 1.3, sediment concentration (C) = 5kg/m3,time for settling (detention of sediment) = 6hrs. Assume 20% additional discharge for flushing, performance coefficient of Hazen (m) = 0.17, Packing factor (β) = 0.5, density of sediment (***ρ***) = 2600kg/m3.

Solution: Q=10m^3/s, d=0.2mm, ω=0.02m/s, V=0.15m/s, C=5kg/m^3, T=6hrs, β=0.5, ***ρ***=266kg/m^3

- Total discharge (Including flushing discharge) = 1.2*10 = 12m^3/s
- Discharge through one Basin (Q_T) = 12/2 = 6 m^3/s
- Surface area $(A_S) = \frac{K*Q_T}{\omega} = \frac{1.3*6}{0.02} = \mathbf{390}\ m^2$
- Assume $\frac{L}{B} = 8 \ \therefore L = 8B$
- We know $A_S = L * B = 8B * B = 8B^2$

$390 = 8B^2$

B = 6.98 m

Check the basin width by the following equation

Check $B = 4.75\sqrt{Q_T} = 4.75\sqrt{6} = \mathbf{11.64m} > 6.98m$ Hence OK

∴ Length of the basin (L) = 8*B = 8*6.98 = 55.84 m

- Limiting velocity $(V_c) = a\sqrt{d} = 0.44\sqrt{0.2} = 0.197\ m/sec$
- Here, Flo**w velocity (V) = 0.15 m/s**
- Take minimum value of limiting velocity and given flow velocity for safe design of depth of settling basin.

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Calculation of depth of settling basin

- Depth of settling basin(D) $= \frac{Q_T}{V*B} = \frac{6}{0.15*6.98} = 5.73$ m
- Size of settling basin without considering turbulence effect = 55.84 m* 6.98 m* 5.73m
- With considering turbulence effect
- $\therefore$ Length (L) $= \frac{D^{1.5}*V}{(D^{0.5}\omega)-0.132V} = \frac{5.73^{1.5}*(0.15)}{(5.73^{0.5}*0.02)-(0.132*0.15)} =$ **73.27 m**
- Check for Length (For 95% removal $\lambda = 1.2$)
- $L = \frac{\lambda^2 V^2 (\sqrt{D}-0.2)^2}{7.51\,\omega^2} = \frac{1.2^2*0.15^2(\sqrt{5.73}-0.2)^2}{7.51\,(0.02)^2} = 51.906$ m
- Size of settling basin without considering turbulence effect = 73.27 m * 6.98 m* 5.73m
- **Calculation of sediment depth of the basin**
- Sediment load $= Q_T * T * C = 6*(6*60*60)*5 = 648000$ Kg
- Sediment Volume $= \frac{\text{Sediment load}}{\text{Density}*\text{Packing factor}} = \frac{648000}{2600*0.5} = 498.461\ m^3$
- **Case a:** Assume base of the settling basin is **Rectangular:**
- **(i) Without turbulence effect**
- Height of the sediment(h_s) $= \frac{\text{Sediment Volume}}{\text{Base plane Area}} = \frac{498.461}{55.84*6.98} = \mathbf{1.28m}$
- **(ii) With turbulence effect:**
- Height of the sediment (h_s) $= \frac{\text{Sediment Volume}}{\text{Base plane Area}} = \frac{498.461}{73.27*6.98} = \mathbf{0.975\ m}$

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Case b: Assume base of the settling basin is in standard recommended shape and size.

(i)Without turbulence effect

Total Volume of sediment = length ∗ Area of sediment

$$498.461 = \left[55.84\left\{(1*1)+(1*x)+\left(\frac{1}{2}*x*1.192x*2\right)\right\}\right]$$

$$\frac{498.461}{55.84} - 1 = (x + 1.192x^2)$$

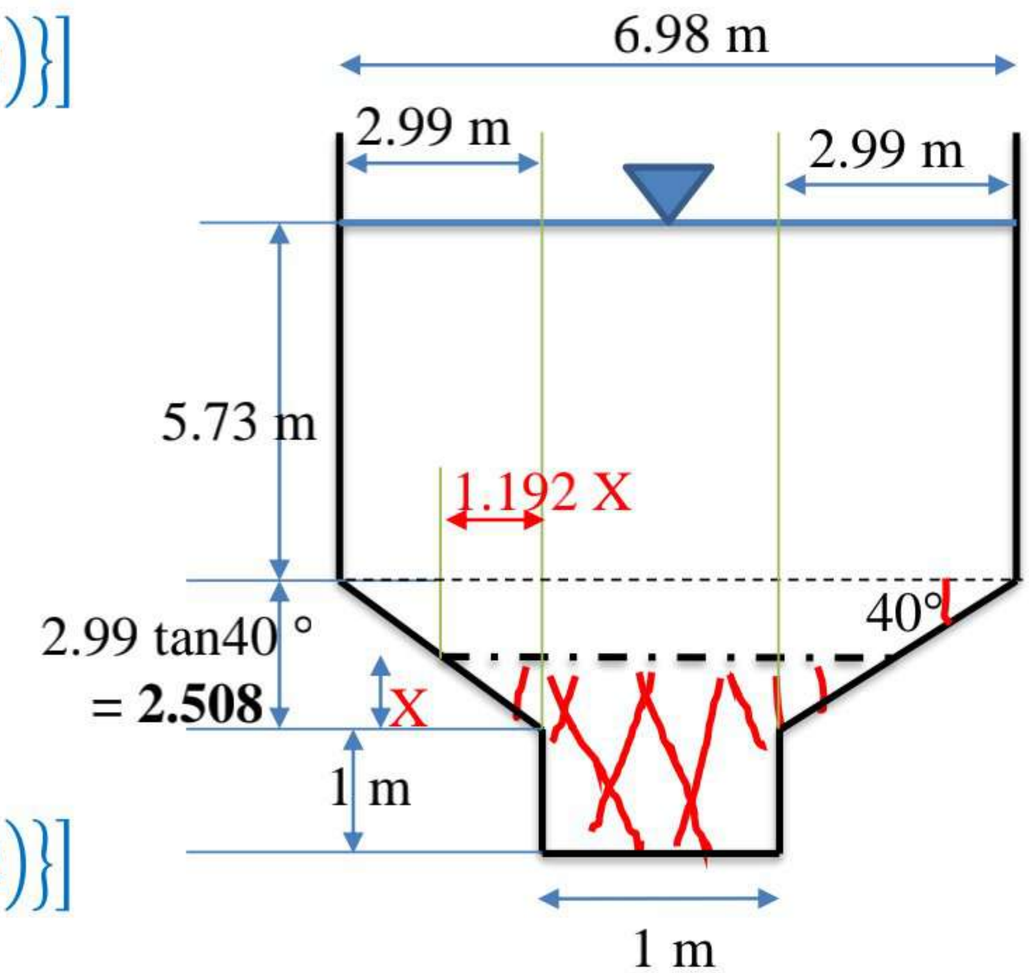
$$1.192x^2 + x - 7.926 = 0$$

Solving by trial and error method,

we get X= 2.193m, -3.031m

i.e. Depth of sediment = 2.193 + 1

= **3.193m**



(ii) With turbulence effect

Total Volume of sediment = length ∗ Area of sediment

$$498.461 = \left[73.27\left\{(1*1)+(1*x)+\left(\frac{1}{2}*x*1.192x*2\right)\right\}\right]$$

$$\frac{498.461}{73.27} - 1 = (x + 1.192x^2)$$

$$1.192x^2 + x - 5.803 = 0$$

Solving by trial and error method, we get X = 1.826 m, -2.665 m

i.e. Depth of sediment =1.826 + 1 = **2.826 m**

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Hydropower Engineering BCE IV/I

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- **Water Conveyance Structures [8 hours]**

6.1 Hydraulic Tunnels, Geometrical shapes, hydraulic design (velocities, sizing), tunneling method, supports in tunnels, lining of tunnels

6.2 Forebay and Surge Tanks: importance, general arrangement, condition of their application, hydraulic design

6.3 Penstocks and Pressure shaft: importance, conditions of their application, general arrangement, hydraulic transients (water hammer), Computation of hydrodynamic pressure, sizing of penstock /pressure shaft and estimation of thickness of steel in penstock/pressure shaft

6.1 Hydraulic Tunnels, Geometrical shapes, hydraulic design (velocities, sizing), tunneling method, supports in tunnels, lining of tunnels

Introduction:

- An underground passage made without removing overburden.
- Tunnels are the structure constructed for the conveyance of flow or for the transportation purpose or to store food.
- Hydro – technical tunnel could be classified as non pressure tunnel and pressure tunnel.
- The tunnel in which flow takes place the free surface exposed to the atmosphere or flow in the tunnel is open channel flow is called non pressure tunnel. For example spillway tunnel, diversion tunnel, tailrace tunnel etc.
- The tunnel in which flow takes place with pressure is called pressure tunnel. The headrace tunnel is pressure tunnel.
- Usually pressure tunnels are designed of the circular section and non – pressure tunnels are other shape too.
- Pressure tunnels are subjected to an internal water pressure which is frequently many times in excess of the external rock and groundwater pressure.
- The resultant tensile stresses can be resisted most economically by a circular cross – section.
- Pressure tunnels are therefore built with a circular or horseshoe – shaped cross – section.

Advantages and disadvantages of tunnel

Advantages

- Less environmental effect: as tunneling work reduces the land acquisition, resettlement issues, forest clearance etc., the environmental effect will be minimum.
- Natural landscape of the hilly area is not disturbed.
- Possible shortest and direct way so minimum losses and considerable economy in the project cost.
- Less cost at urban area with good rock quality: as urban area, the cost of land is expensive so construction of tunnel in underground will be less costly.
- Optimum space consumption
- Low maintenance cost
- Advantage to get more head
- Less seismic effect
- Easy for inter – basin transfer
- Time saving: shortest route to connect two points

Disadvantages:

- Normally high construction cost
- The construction period is normally long
- High construction risk
- Expensive investigations
- Additional cost for lightening and ventilation

Size and Shape of tunnel

Size of the tunnel

- The minimum diameter of the tunnel is fixed with consideration of the transportation, excavation and hauling during tunneling and should be greater than 2 m in case of circular section and in case of other shapes should be greater than 1.9 m in width and 2.1 m in height.
- Other sizes are fixed based on the functional requirements, for example, to carry large discharge, the size of tunnel is fixed based on the requirements.
- For transportation purpose, the size of the tunnel is fixed based on the types of road construction, one lane or multi lane road.

Geometric design of tunnel:

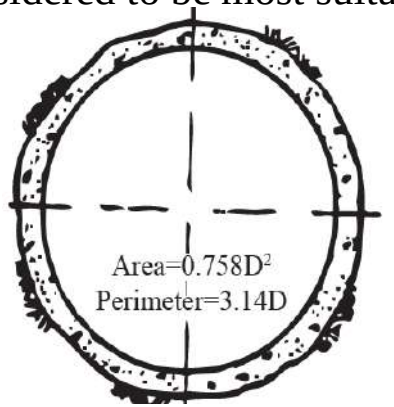
- After the finalization of tunnel alignment and geological investigation, tunnel geometry is selected. There is no hard and fast rule for the tunnel geometry selection.
- Therefore, the tunnel geometry entirely depends on the prevailing geological condition and judgment of the designer.
- No general recommendations can be made to fit in each and every individual case but a few important tunnel geometry widely used in the hydraulic design are as follows
- 1. Circular section
- 2. D – shape section
- 3. Horse – shoe section
- 4. Modified horse shoe section

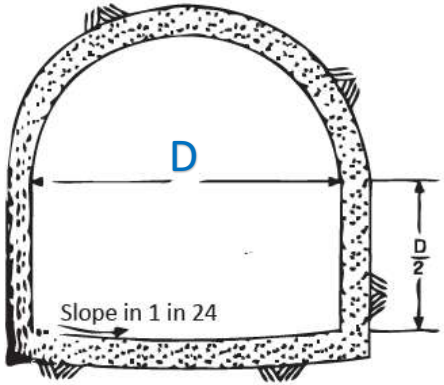
Size and Shape of tunnel

1. Circular section:

- Circular section is most suitable from structural considerations.
- However, it is difficult for excavation, particularly where the cross section area is small.
- In a case where the tunnel is subjected to high internal pressure but does not have good quality rock or adequate rock around it, circular section is considered to be most suitable.
- $A = \frac{\pi D^2}{4} = 0.785 D^2$
- $P = \pi D = 3.14\ D$

Circular Section





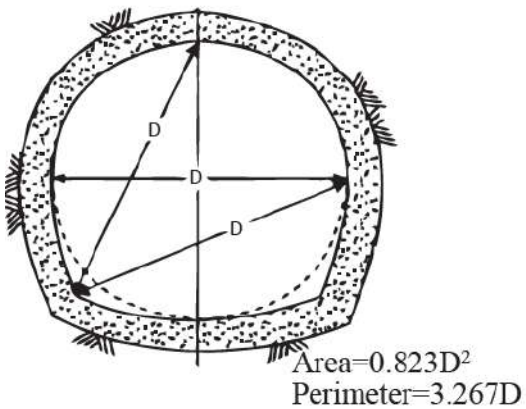
D - Shape Section

2. D – shape section:

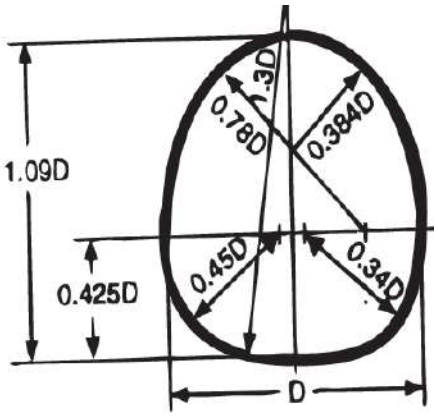
- D shaped sections are suitable for tunnel located in good quality rocks.
- The main advantages of this section over horse shoe section are the added width of the invert which gives more working floor space in the tunnel during driving and flatter invert which helps to eliminate the tendency of wet concrete to slump and draw away from the tunnel sides.
- The added invert width also permits the use of simultaneous lining of the tunnel which may not be possible for circular and horse – shoe tunnels of the same dimensions.
- $A = \frac{1}{2} * \frac{\pi D^2}{4} + D\frac{D}{2} = 0.892\ D^2$
- $P = D + 2\frac{D}{2} + \frac{\pi D}{2} = 3.57\ D$

3. Horse – shoe or Modified horse shoe section:

- These sections are a compromise between circular and D shaped sections.
- These sections are structurally strong to withstand external rock and water pressure.
- These sections are found to be most suitable, where a moderately good rock is available, advantages of a flatter invert are required for construction purposes and the tunnel has to resist internal pressures.
- These modified horse – shoe sections also afford easy change over the circular sections with minimum additional cost in reaches where rock quality is poor or rock cover is inadequate.



Horse shoe Section



Egg shaped Section

Modified Egg shaped section

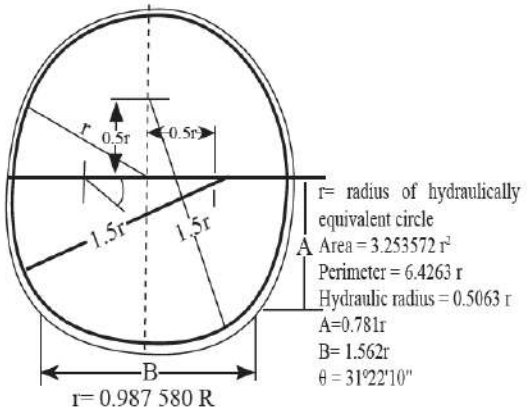


Fig . . Modified Egg Shaped Section

4. Egg shaped section:

- These types of section are provided to require for constructing tunnel in condition where available soft soil, laminated and stratified layer in crown of tunnel.
- These sections are structurally more efficient section.

Tunnel Alignments

- The position of the inlet portal, outlet portal, forebay, surge tank and surge chamber usually determines the location of the tunnel alignment despite the relation of orientation of the rock attitude with the tunnel axis.
- In general, the tunnel alignment connecting those locations will be shortest and the most economical condition.
- Under condition where the rock bedding/ joints dipping at 45° to 90 ° is almost parallel to the tunnel axis, the shortest tunnel route might face very unfavorable geological condition resulting much instability problems and more support requirement.
- In such situation selection of the tunnel route oblique to the shortest tunnel axis will be preferable as far as possible.
- *Following points should be considered during the tunnel alignment selection*

i. It should be shortest as far as possible – this should ensure minimum loss and shall be economically cheapest.

ii. It should be straight as far as possible – introduction of bends in the alignment shall involve losses at all such bends and the cost of tunneling would increases.

iii. It should be easily accessible – an easy access near the entrance and exit to the tunnel becomes essential for the construction facility.

iv. Careful selection of entry and the exit locations with minimum length and depth of approach cutting and no weathered, lose fractured layers slope towards portal.

Tunnel Alignments

However, it is not always possible to follow a straight alignment because of the following parameters affecting the design of hydraulic tunnels

- **Topography:** there may not be sufficient vertical and or lateral cover.
- **Geological section along the alignment:** it may show certain difficult strata through which the process of tunneling may be cumbersome and uneconomical.
- **Ground and / or rock water loads** along the tunnel alignment may be excessive thus increasing the overall cost of the tunnel.
- **Rock mechanics properties:** the in – site stresses, joint pattern shear strength, unconfirmed compressive strength, shear modulus of deformation etc.. may not be favorable along a particular alignment.
- **Creep or tectonic movement along the tunnel:** the tunnel may be passing through active faults which could cause collapse of the tunnel.

Load and stress in Tunnel

Following are the loads acting in tunnels

1. **Lateral active earth pressure**
2. Reaction pressure due to elastic deformation of the beneath medium – passive loading condition (if the whole earth mass is tended to move by the lining, then passive earth pressure should be countered – as of retaining wall – not possible condition in tunnel case)
3. **Over burden pressure**
4. Hydrostatic pressure
5. **Earthquake pressure**

- Depending upon the strength of the rock, the horizontal pressure on the wall of the tunnel is different. For high strength rock, vertical pressure exist with no lateral pressure and only the reactive stress due to the elastic deformation of the bed medium above which the tunnel lining is done. For the low strength rock, there is active earth pressure too exist.
- The vertical pressure depends upon the height of the arch (h_q) that could withstand the rock without support by lateral transfer of the load due to the internal frictional resistance. The greater the hardness of the rock less will be the height of the arch. Loose rock will need greater height of the arch.
- $\left(45° - \frac{\emptyset}{2}\right)$ - it is the angle at which the soil wedge will slide due to the active earth pressure.
- $tan^2\left(45° - \frac{\emptyset}{2}\right) = \frac{1}{tan^2\left(45° + \frac{\emptyset}{2}\right)} = Cot^2\left(45° + \frac{\emptyset}{2}\right)$
- The height of the arch, $(h_q) = \frac{b_q}{2f}$
- Where, f is hardness coefficient.

Load and stress in Tunnel

$b_q = b + 2H * \left(45° - \frac{\emptyset}{2}\right)$

- Where, $\emptyset$ = arch tan f is angle of internal friction.
- **Calculation of Rock pressure on the tunnel:**

1. If $f < 4$ and if depth of the tunnel from the ground surface (h_{qz}) is less than the height of the arch (h_q) , then vertical pressure could be defined as

$(g_{qzn}) = \gamma_{rock} * h_{qz}$

2. If $f < 4$ and if depth of the tunnel from the ground surface (h_{qz}) is more than the twice the height of the arch ($2h_q$) , then vertical pressure could be defined as

$(g_{qzn}) = \beta * \gamma_{rock} * h_{qz}$

Where, β could be taken equal to 0.7 if $b_q \leq 5.5$ and could be taken equal to 1 if if $b_q \geq 7.5$.

The value between the limit could be taken through the linear interpolation.

3. If $f < 4$, then the lateral active earth pressure could be defined as

$(g_{qzn}) = \gamma_{rock} * (h_q + 0.5H) * tan^2\left(45° - \frac{\emptyset}{2}\right)$

4. If $f \geq 4$, H > 6m then vertical pressure could be defined as

$(g_{qzn}) = \beta * \gamma_{rock} * h_{q1}$

Where h_{q1} = depth of fractured zone = K_a*b

Coefficient $K_a$considers the hardness of the rock (f) and degree of fracture (Counting number of crack formation per unit length)

If M_q = number of cracks in 1 meter length of rock.

Load and stress in Tunnel….

Hardness coefficient	Degree of Fracture (K_a)		
	Less fractured ($M_q \leq 5$)	Fractured $15 < M_q < 5$	Highly Fractured $5 < M_q < 15$
4	0.2	0.3	0.4
9 – May	0.1	0.2	0.3
10 and greater	0.05	0.1	0.15

5. If $f \geq 4$, $M_q < 5$ and for tunnel of height less than 6 m, lateral pressure could be ignored.

6. If $f \geq 4$, $5 \leq M_q < 5$ (Highly fractured) and for tunnel of height less than 6 m, lateral pressure could be calculate as follows

$(g_{qzn}) = 0.1 * \gamma_{rock} * H$

Hydraulic design of the Tunnel:

Basically, there are two flow conditions in the hydraulic tunnel design:

1. Free flow tunnel:

Hydraulic design of the free flow tunnel is executed as the hydraulic design of canal and Manning's frictional factors are used to compute the head loss in length.

2. Pressure flow tunnel:

- Hydraulic design of the pressure flow is computed as the pipe flow and the head loss is computed using Darcy/ Weisbach frictional factor.
- Discharge through a pressurized hydraulic tunnel is calculated using a continuity equation with control volume approach.

Discharge (Q) = Velocity (V) *Area (A)

- The optimization study with the permissible velocity in the given type of tunnel will determine the optimum size of tunnel for the given discharge.
- Optimization study comprises of annual charges composed of fixed cost, cost of operation and maintenance and value of energy losses.
- These can be expressed as a function of diameter and should be minimum for the economical diameter.
- The velocity of unlined tunnel can be 2 to 2.5 m/sec.
- For concrete lined tunnel, the velocity can range up to 4 to 5 m/sec.
- In steel lined tunnels the velocity can be up to 9 m/sec.

Head – loss in the Tunnel:

Head loss in tunnel is also categories as major head losses and minor head losses.

a) Major Head loss

1. Frictional loss in length:

Frictional loss (h_f) in length for tunnel is calculated either by using Manning's formula or Darcy – Weisbach formula as given below

i. Manning's formula

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

Where V = velocity of water in the tunnel in m/s

L = Length of the tunnel in m

R = Hydraulic Radius (Wetted Area / Wetted perimeter)

n = manning's roughness coefficient

ii. Darcy Weisbach formula:

$$h_f = \frac{fLV^2}{2gd}$$

f = Frictional factor

L = Length of the tunnel in m

D = Diameter of tunnel in m

V = Velocity of water in the tunnel in m/s

Head – loss in the Tunnel:…

b. Minor head loss:

The minor losses include trash – rack loss, entrance loss, transition loss, bend/junction losses, gate loss and exit loss.

1. Trash – rack loss:

Tunnel openings are provided with trash racks at the intake to prevent the entry of floating debris into the tunnel. Where minimum loss value is desired, it is usual to assume 50 % of the rack area as clogged. The trash rack loss shall be computed by using the following formula

$$h_t = K_t \frac{V^2}{2g}$$

Where,

K_t = loss coefficient for trash – rack calculated by the formula

$$K_t = 1.45 - 0.45 * \frac{a_n}{a_t} - \left(\frac{a_n}{a_t}\right)^2$$

a_n = net area through the trash rack bars

a_t = gross area of the opening

V = velocity

2. Entrance loss:

To minimize the head losses and to avoid zones where cavitation pressure may developed, the entrance to a pressure tunnel should be streamlined to provide gradual and smooth changes in flow. For best efficiency the shape of the entrance should stimulate that of a jet discharge into air and should guide and support the jet with minimum interference until it is contracted to the tunnel dimensions.

 # Head – loss in the Tunnel:…

2. Entrance loss:

Entrance loss shall be computed by using following formula

$$h_e = K_e \frac{V^2}{2g}$$

Where,

V = velocity of flow

K_e = loss coefficient for entrance {for circular bell mouth (0.04 – 0.1), for square bell mouth (0.07 – 0.20)}

3. Transition loss:

In hydraulic tunnel, transitions are often required at the intake, junctions with desanding chambers, gate galleries, surge shafts etc., and at outlets. All these transitions cause head loss in tunnels. To minimize the head loss and to avoid cavitation tendencies along the tunnel surface, the transitions should be gradual. The transition can either be for contraction or for expansion.

i. For expanding transition, the head loss is given by the following formula

$$h_e = K_e \left(\frac{V1^2}{2g} - \frac{V2^2}{2g}\right)$$

Where,

V_1 = average velocity in (m/s) at the beginning of transition

V_2 = average velocity in (m/s) at the end of transition

K_e = loss coefficient for expansion which is given by $= 3.50 \left(\tan\frac{\alpha}{2}\right)^{1.22}$

α = Angle of the tunnel wall surface with respect to Centre line

 # Head – loss in the Tunnel:…

ii. For contraction transition, the head loss is given by the following formula

$$h_c = K_c\left(\frac{V2^2}{2g} - \frac{V1^2}{2g}\right)$$

Where, V_1 = velocity in (m/s) in normal section

V_2 = velocity in contraction section in (m/s)

K_c = loss coefficient for contraction {for gradual contraction (0.1 – 0.5)}

4. Bend losses:

Bend and junction in hydraulic tunnel is unavoidable owing to their functional and constructional requirements. These bends and junctions also cause loss of head which must also be computed.

$$h_b = K_b\frac{V^2}{2g}$$

5. Gate losses:

If the entrance to the tunnel is designed properly the velocity of flow would be approximately 1 m/s. in such case no gate loss needs to be considered. However, there will be head loss due to the gate groove and the gate shaft which is given by the formula

$$h_g = K_g\frac{V^2}{2g}$$

K_g = loss coefficient for gate

6. Exit loss:

$$h_{ex} = K_{ex}\frac{V^2}{2g}$$

Tunnel Optimization:

- The factors considered while working out the economical diameter are velocity requirement, head loss in tunnel, interest of capital cost of tunnel, Annual operation and maintenance charge, whether lined or unlined, and cost of gates and their hoisting equipment.
- It is based on the increment of tunnel cost with respect to the tunnel diameter (sectional area) and the value of energy lost which is a function of the tunnel sectional area.
- A larger diameter for a given discharge leads to smaller head losses and hence greater will be the net head available to the turbine.
- Thus the power and energy production will be increased. On the other hand a greater size tunnel means less velocity and greater capital investment.
- Therefore, a size that will give the least capital cost over the lifetime of the plant is considered to be the optimum diameter/ sectional area.

The steps followed for the optimization of tunnel are;

1. Take different diameter (D1, D2, ….Dn) of tunnel for optimization process.
2. Calculate area of the tunnel (A1, A2, …..An) for different diameter of tunnel.
3. Calculate perimeter of the tunnel (P1, P2,….Pn) for different diameter of tunnel.
4. Find out hydraulic Radius (R1, R2,….Rn) of the tunnel for different diameter of tunnel.
5. Calculate equivalent diameter ($D_{e1}, D_{e2}, \dots \dots . . D_{en}$) of the tunnel for different diameter of tunnel.
6. Calculate velocity of the flow (V1, V2, ……Vn) for different diameter of tunnel from V = Q/A relation, where Q is the design discharge.

Tunnel Optimization:

7. Calculate head loss by using Manning's head loss and Darcy's head loss formula for every diameter of tunnel. Manning's formula is generally adopted, as tunnel is generally designed for open channel flow or low pressure condition.

Darcy Weisbach head loss formula:

$$h_f = \frac{fLV^2}{2gD_{equ}}$$

Manning's head loss formula

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

f = Frictional factor

L = Length of the tunnel in m

D_{equ} = Equivalent Diameter of tunnel in m

V = Velocity of water in the tunnel in m/s

n = manning's roughness coefficient

8. Calculate the power output of the headloss taking maximum of Darcy headloss and manning's headloss, generally manning's headloss is more

Power = 9.81*Q*H*η

9. Calculate the energy output in one year multiplying power output with (356*24)

10. Cost of energy is calculated multiplying with unit cost of energy. Unit cost of energy is taken according to the existing rate.

Tunnel Optimization:

11. Cost of the tunnel is calculated considering excavation of tunnel, shotcrete in tunnel surface, concrete lining etc. for every depth of tunnel.

Annual cost of the tunnel is calculated multiplying capital recovery factor with cost of the tunnel for internal rate of return i and life of project n.

Capital recovery factor = $\frac{i*(1+i)^n}{(1+i)^n-1}$

12. Sum of the cost is calculated adding cost of energy with annul cost of the tunnel for every depth of tunnel.

13. Then one chart is plotted between depth of tunnel and annual cost of the tunnel for every depth and another chart is plotted between depth of tunnel and cost of energy loss for every depth and also another graph is plotted between depth of tunnel and sum of the tunnel for every depth. Observing these ***three charts***, optimum depth of tunnel will be found out in which cost of the tunnel is minimum.

Method of Tunnel Construction:

There are mainly two method of tunnel construction, the first method is called Heading and benching method, is an older method. The second method of tunneling is called full face tunneling method, is comparatively new method.

In heading and benching method of tunneling, only part of the tunnel cross section is excavated first.

- This part is known as benching. This part is enlarged to the required size. The heading may be at the top, bottom or Centre of the tunnel to be excavated.

In the full face tunneling method, the entire tunnel cross section is excavated uniformly.

- In full face tunneling, generally temporary supports made of prefabricated steel construction are erected till a permanent lining is installed.
- The full face tunneling method of construction of tunnel became possible due to the development of jumbos which are trolley car in which are mounted a number of drills and drilling operation can be quickly completed.

Factors that affect the selection of tunnel construction method vary from geological to economical to sociological.

The different methods of tunnel construction include

1. Drill and blast method
2. Cut and cover method
3. Tunnel boring machine (TBM) method
4. New Austrian blasting Method (NATM)

Method of Tunnel Construction:….

1. Drill and blast method

- Drill and blasting (D &B) is the most common method of tunnel construction in Nepal.
- It originates in hard rock tunneling and utilize rock bolts and shotcrete applied immediately after blasting.
- This is often followed by a cast in – situ concrete lining works using formwork.
- The tunnel construction is done by drilling, blasting, Mucking and Hauling.
- The drilling, there are different type of cut such as angle cut and parallel cut.

The steps of construction by drill and blast method

i. Holes are drilled in the tunnel face
ii. Holes are blown clean and then hand packed with explosives and detonators.
iii. The explosives are detonated
iv. Tunnel is ventilated
v. Rock is scaled
vi. Muck is removed
vii. Support is installed

Method of Tunnel Construction:….

2. Cut and cover method

- Cut – and – cover is a method of tunnel construction where a trench is excavated and roofed over.
- Strong supporting beams are necessary to avoid the danger of the tunnel collapsing.
- The decision to use either bottom – up or top – down construction methods for a cut – and – cover tunnel depends two issues.
- **The first** is available right – of – way. The use bottom – up construction methods, the excavation is first constructed by laying back side slopes that requires significant right of way.
- Typically in urban applications right – of – way is limited requiring that top – down construction methods be used.
- **The second issue** is ground water table. If the groundwater table is above the base of the tunnel excavation and dewatering is anticipated to be difficult, some top – down methods construction can control inflow of water into the excavation.
- It is usually economical for tunnel depths up to 20 meters.
- The tunnel is designed as a rigid frame box structure.

Method of Tunnel Construction:

3. Tunnel boring machine (TBM) method

- TBM is used as an alternative to drilling and blasting (D&B) methods.
- TBMs are used to excavate tunnels with a circular cross sections through a variety of subterranean matter; hard rock, sand or almost anything in between.
- As the TBM moves forward, the round cutter heads cut into the tunnel face and splits off large chunks of rock. The cutter head carves a smooth round hole through the rock, the exact shape of tunnel.
- Conveyor belts carry the rock shavings through the TBM and out the back of the machine to a dumpster.
- A TBM generally is a device for excavating a tunnel in such a way that the material to be removed is disintegrated by the continuous rotation of a group of cutting tools thrust against the surface of the material at working face.
- A TBM works most efficiently in relatively homogeneous ground, (all soft or all hard), because it is very difficult to change cutters during the tunneling process.
- **For mixed ground,** where the tunnel passes from soft to hard ground, or vice – versa, New Austrian blasting Method (NATM) is usually a more economical tunnel method.
- A tunnel boring machine generally is a device for excavating a tunnel in such a way that the material to be removed is disintegrated by the continuous rotation of a group of cutting tools thrust against the surface of the material at the working face.
- The cutting tools are mounted in an arrangement suitable to excavate a tunnel of the required diameter.

Method of Tunnel Construction:…

4. New Austrian blasting Method (NATM)

- The tunnel is sequentially excavated and supported, and the excavated sequences can be varied.
- **In soft ground tunnel,** initial ground support in the form of shotcrete, usually with lattice girders and some from of ground reinforcement, is installed as excavation proceeds, followed by installation of a final lining at a later date.
- In cases where soft ground conditions do not favor an open face with a short length of uncompleted lining immediately next to it (flowing ground or ground with a short stand – up – time), a ground arch does not develop.
- Unless such unstable conditions can be modified by dewatering, spilling, grouting or other methods of ground improvement, closed – face shield tunneling, and not NATM should be considered.

6.1 Tunnel Lining:

After excavation of tunnels, tunnel lining is done to increase hydraulic capacity of tunnel, to reduce resistance, to increase strength and to reduce the losses from tunnel. *Tunnel lining is done when the following condition arises;*

- Internal water pressure are high (generally lining for head greater than 100m but for the tunneling in hard rock higher head can be remain unlined)
- In strata of low strength and where the rock is anisotropic.
- The hydraulic resistance is desired to be low. To increase the discharge carrying capacity and minimize losses.

Advantages of tunnel lining

i. Lining act as a structural unit and transmit only part of the load to the surrounding rock. Hence it provides strength and stability and facilitates tunneling in weaker strata. Further it can distribute the internal loads uniformly in the surrounding rock mass.

ii. The hydraulic resistance of lined tunnel is much less than that of unlined tunnel so it minimize loss, saves energy and revenue.

iii. The **velocity of water** in lined tunnel can be higher than that in the unlined tunnels. Hence small section may have better discharge carrying capacity.

iv. Lining can be done simultaneously as the excavation proceeds. This minimize the danger of accidental rock falls in the tunnel cavity.

v. The structural stability of tunnels is increased.

vi. The **seepage losses** through the tunnel are reduced significantly.

#Types Lining:

i. Shotcrete lining

ii. Plain concrete lining

iii. Reinforced concrete lining

iv. Grouting:

v. Steel lining

i. Shotcrete lining:

- It is one of the most common and well recognized methods for supporting rock tunnels and caverns.
- Shortcreting is a supporting method in which the dry or wet mixture of cement sand and aggregate of 10-20 mm size is pumped, conveyed through a hose and then pneumatically shot at high speed onto the prepared surface and gets compaction by impact of projection.
- The concept of Shortcreting with dry mix technique was visualized at the end of the first decade of the twentieth century, but its frequent and practical use on rock slopes, in tunnels and as an underground rock mass support in civil and mining engineering works.
- After 1970s, wet mix technique is introduced for supporting the excavated rock mass.
- Dry mix remained a dusty and slow operation when used in combination with welded mesh to impart tensile strength. Besides, the quality control was also lacking since the quantity of water being added to shotcrete mix depended on the experience and expertise of the nozzle man.
- After introduced wet shotcrete technique, steel fibers (steel mesh reinforcement) has been used instead of welded mesh.
- The shotcrete is sprayed on the rock surface of an excavated slope or underground opening.

i. Shotcrete lining…

The main functions of shotcrete are as follows;

- To provide an arch action in the crevices created as a result of blasting.
- To prevent weathering of rock surface due to exposure
- To provide immediate support to rock mass
- To help in maintaining the intrinsic shear parameters of rock mass

ii. Concrete lining:

- It is a cement concrete supporting method cast –in place at the tunnel in section with exceptionally poor rocks by using steel framework the compaction is achieved by vibration of the laid mass.
- Depending upon the mode of placement, the type of concrete may be plain or reinforcement and structural requirement etc. The length of lining is normally 4 to 5 m and the thickness of lining is designed with a minimum thickness of 30 cm.
- Minimum concrete qualities equivalent to M25 is used in lining.
- When swelling is the major stability problem, a cast –in situ concrete lining has the advantage over a Shortcreting lining in being subjected to a considerable lower swelling pressure.

iii. Grouting:

- Basically grouting in tunnel is carried out to improve the rock mass stability and to reduce ground water inflow in the tunnel.
- In major faults and weakness zones, grouting is much more likely to be successful when performed ahead of the tunnel face (pre-grouting) than behind the tunnel face (post grouting).

6.2 Forebay and Surge Tanks: importance, general arrangement, condition of their application, hydraulic design

- **Introduction of fore bay**
- A forebay (or head pond) is a temporary water storage, regulating reservoir provided at downstream end of canal just at upstream of penstocks to act as a small balancing reservoir.
- At the instance when turbine rejects the load, it acts as a storage reservoir whereas it supplies water as a sort of regulating reservoir when load increases.
- It is required in the case of ROR project at U/S of diversion works.
- In case of a storage plant, it is required only when power house is located away from the dam. If power house is located at toe of the dam, a separate forebay is not required.

 # Functions of forebay

- Stores water to act as regulating reservoir for penstocks.
- Act as a outlet (or a waste way) to dispose excess water.
- Prevent debris from entering penstocks.
- Store water temporarily when load on plant is low and provided extra water to meet sudden rise in demand.
- Act as a surge tank for low head plants.
- It release the surge pressure.
- It can also serve as a secondary / final settling basin and trap some particles that enter the headrace downstream of the settling basin.

Component / Part of the fore bay

Following are the component parts of a typical forebay:

1. Entrance bay or basin
2. Flushing for accumulated sediment (Flushing sluice)
3. Repairing gate
4. Screens
5. Emergency /repairing gate
6. Penstock inlet
7. Deviation canal
8. Air vent
9. Spillway
10. Valve chamber or gate chamber

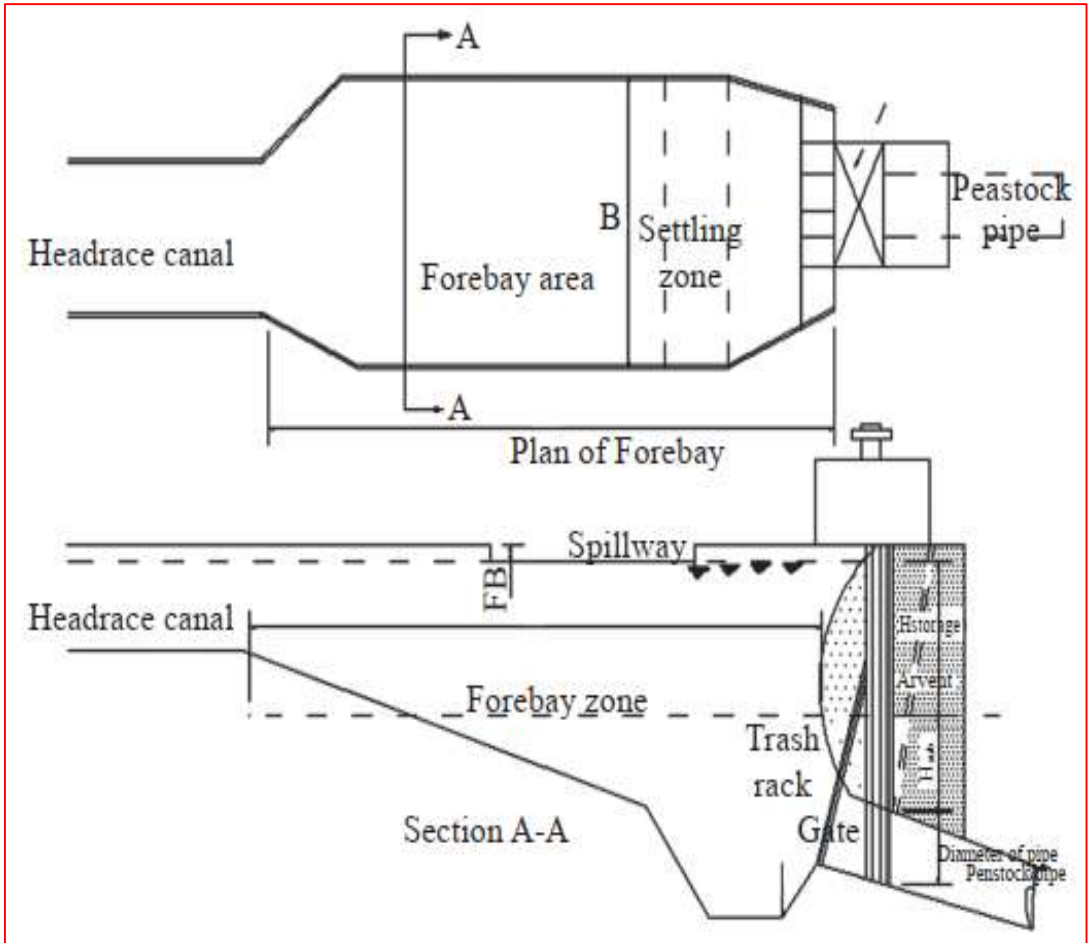


Fig Components of The Forebay

Design of Forebay Structure

1. Assume, mean velocity (ϑ') = 0.15 m/sec (0.1 – 0.2 m/s) & Mean depth (h) = 6m

2. Storage Capacity (V) = Q*t, Where t = detention period

3. Discharge (Q) = ϑ' * A = ϑ'* B* h

$\therefore \text{Width(B)} = \frac{Q}{\vartheta' * h}$

4. Then, Volume (V) = L*B*h

$\therefore L = \frac{V}{B * h}$

5. Estimation of drawdown level: The inlet section of penstock should remain fully submerged during sudden start up in order to avoid entrainment of air. $(y) = \vartheta_P \sqrt{\frac{L_P * A_P}{g * A_f}}$

Where, y = maximum drawdown / down surge

L_p = Length of penstock

A_p = Cross – sectional area of penstock $\left(\frac{\pi d^2}{4}\right)$

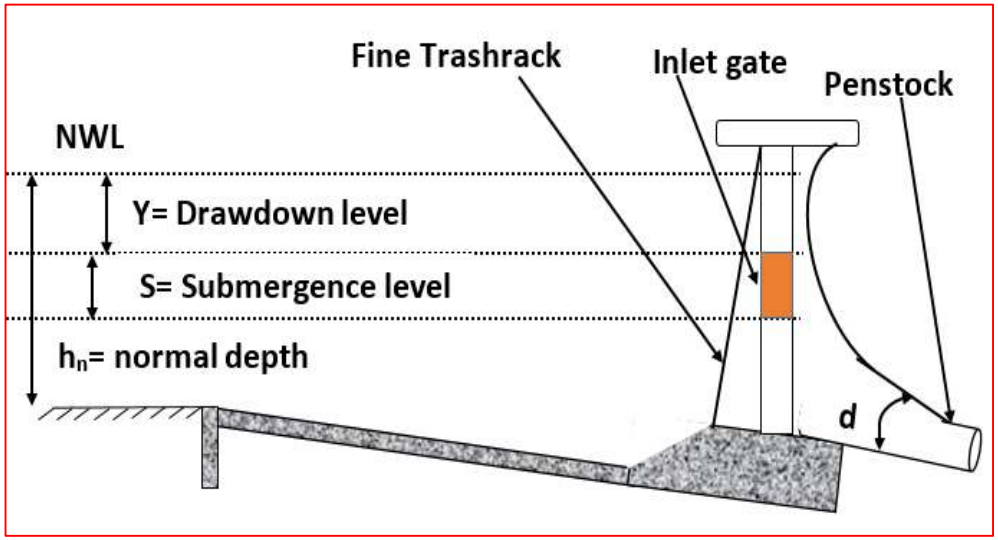
Velocity in penstock $(\vartheta_P) = \frac{Q}{A_p}$

A_f = Cross – sectional area of forebay = (L*B)

6. Minimum submergence at intake:

$\frac{S}{\vartheta_P \sqrt{d}} > 0.545$ (Symmetric inflow)

> 0.723 (Asymmetric inflow)



Design of Forebay Structure

6. Minimum submergence at intake:

Where, d = diameter of penstock

- Find $\left(\frac{S}{d}\right)_{calculated}$ ……(i)

7. Vortex free criteria:

$\frac{S}{d} > 0.5 + 0.4\ F_r$

But $F_r = \frac{\vartheta_P}{\sqrt{gd}} < 0.5$

- Find $\left(\frac{S}{d}\right)_{required}$ ……(ii)
- **If** $\left(\frac{S}{d}\right)_{Calculated} > \left(\frac{S}{d}\right)_{required}$ **then ok.**

Q.N. Design a forebay with following given data, Discharge (Q) = 15m^3/sec, Storage requirement = 4 minutes, length of penstock= 500m and Diameter of penstock=2m.

Solution:

1. Let us assume mean velocity$(\vartheta') = 0.15\text{m/s}$ & Mean depth (h) = 6m

2. Volume of storage (V) = Q*t

= 15*4*60 = 3600m^3

$$\text{Width(B)} = \frac{Q}{\vartheta' * h} = \frac{15}{0.15*6} = 16.67\text{m}$$

3. $$\text{Length(L)} = \frac{V}{B*h} = \frac{3600}{16.67*6} = 35.99\text{m} \cong 36\text{m}$$

4. Estimation of draw down Level $(y) = \vartheta_P \sqrt{\frac{L_P * A_P}{g * A_f}}$

$$\text{Here Area of penstock}(A_P) = \frac{\pi * 2^2}{4} = 3.14\text{m}^2$$

$$\text{Velocity through penstock } (\vartheta_P) = \frac{Q}{A_P} = \frac{15}{3.14} = 4.78 \text{ m/sec}$$

$$\text{Area of forebay}(A_{f)} = L * B = 36 * 16.67 = 600.12\text{m}^2$$

$$\therefore (y) = \vartheta_P \sqrt{\frac{L_P * A_P}{g * A_f}}$$

$$= 4.78 \sqrt{\frac{500*3.14}{9.81*600.12}}$$

$$= \mathbf{2.468m}$$

5. Submergence

$$\frac{S}{\vartheta_P\sqrt{d}} > 0.545$$

or $S > (0.545 * 4.78\sqrt{2})$

$\therefore \mathbf{S = 3.68m}$

6. Vortex free criteria

$$\frac{S}{d} > 0.5 + (0.4f_r)$$

Where $f_r = \frac{\vartheta_P}{\sqrt{gd}} = \frac{4.78}{\sqrt{9.81*2}} = 1.08$

$$\left(\frac{S}{d}\right)_{required} = 0.5 + (0.4 * f_r)$$

$= 0.5 + (0.4 * 0.5)$

$= \mathbf{0.70}$

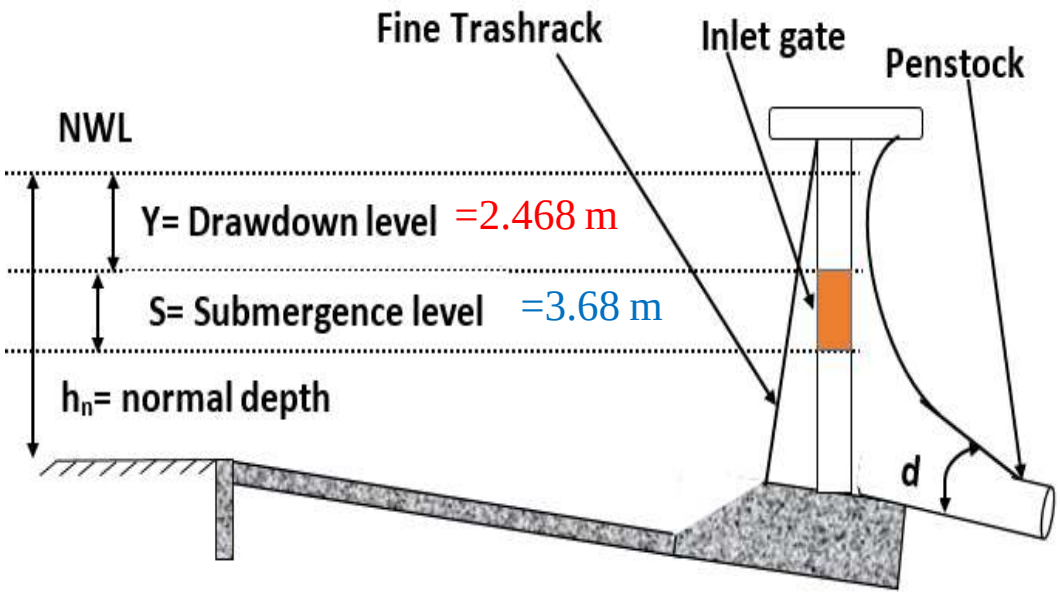
we know f_r should be < 0.5

if not take $f_r = 0.5$

7. Again $\left(\frac{S}{d}\right)_{Calculated} = \frac{3.68}{2} = \mathbf{1.84}$

Here, $\left(\frac{S}{d}\right)_{Calculated} > \left(\frac{S}{d}\right)_{required}$

Hence OK (There is Vortex Free criteria)



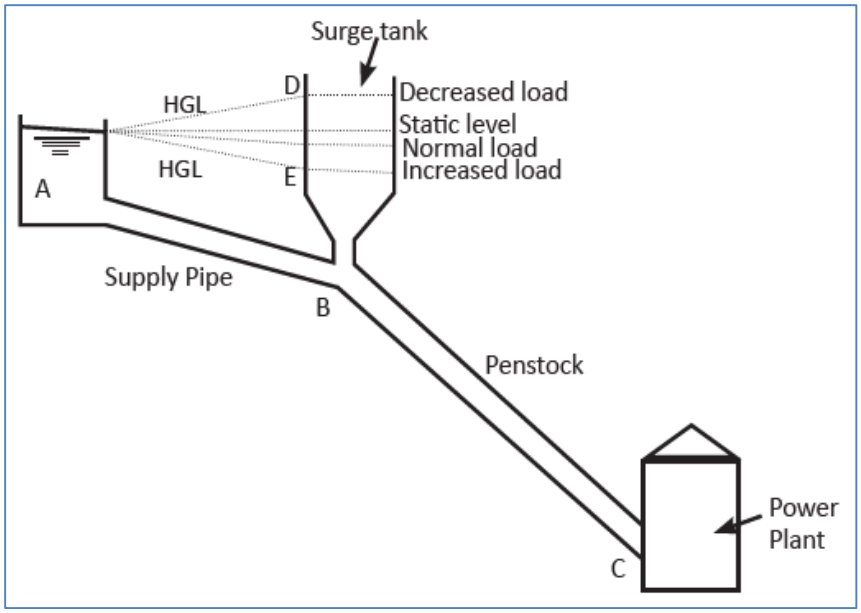
Surge Tank:

- Surge tanks are large cylindrical tanks provided near downstream end of long penstock to relieve excess pressure caused by sudden closure of valve.
- The surge tank is located between almost horizontally or slightly inclined to pressure conduit and steeply sloping penstock, respectively.
- It is designed either as a chamber excavated in the mountain or as a tower rising high above surrounding terrain.
- Rapid velocity fluctuations occur in pipelines during

i) sudden closure and operating of valves

ii) start and shut down of a turbine or

iii) closure of wicket gates during speed regulation of a turbine.



Surge Tank:

Pressure variation is setup due to the rapid fluctuation of velocity and these excessive pressures may lead to bursting of pipes.

- A provision (vertical open tank) are adopted in just upstream of penstock of hydroelectric plant for eliminating water hammer action is called surge tanks.
- During shut down (when load is being decreased), the water from the supply reservoir flows to the surge tank and is stored there and during starting up phase (when load is being increased), the additional requirement of water is met by taking water from the surge tank.

Function of surge tank

- Provide a supply of water when valve is suddenly opened
- Reduces water hammer pressure
- It improves the regulation characteristic of a hydraulic turbine.

Stores water at time of load rejection until penstock velocity has been decelerated to a new steady state valve.

The surge tank dimensions and location are based on the following considerations:

- The surge tank should be located as close to the power or pumping plant as possible;
- The surge tank should be of sufficient height to prevent overflow for all conditions of operation;
- The bottom of surge tank should be low enough that during its operation the tank is drained out and admit air into the turbine penstock or pumping discharge line; and
- The surge tank must have sufficient cross sectional area to ensure stability.

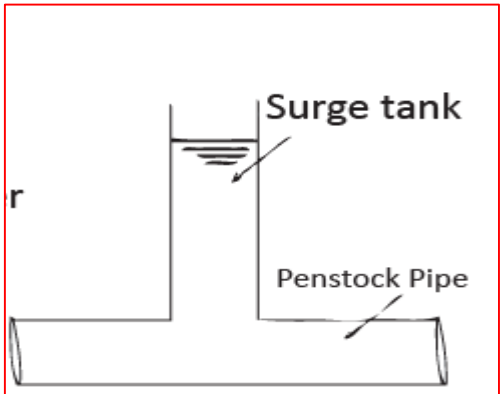
Types of surge tank

According to the hydraulic design, they are classified into following classes

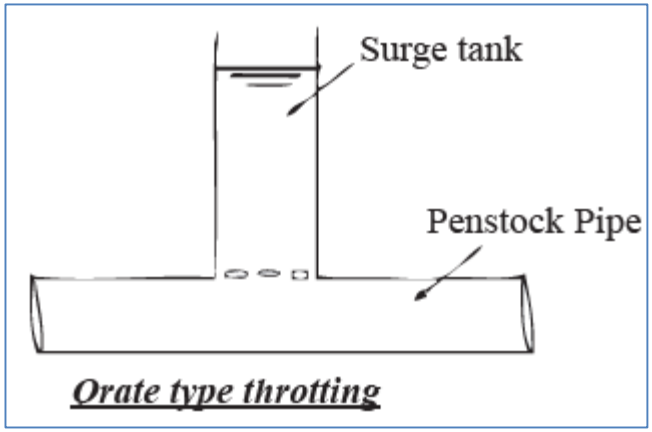
- Simple surge tanks.
- Restricted orifice type (throttled) surge tanks
- Differential surge tanks
- Downstream surge tanks on tailrace tunnels
- Surge tank with expansion chamber
- Multiple surge tanks

1) Simple surge tanks:

- It is unrestricted tank of constant cross sectional area in which maximum variation of water level is contained within the tank.
- This is very sluggish (moving or reaching more slowly than normal) in action and costly since it requires greatest volume.
- It is seldom adopted in preference to others.



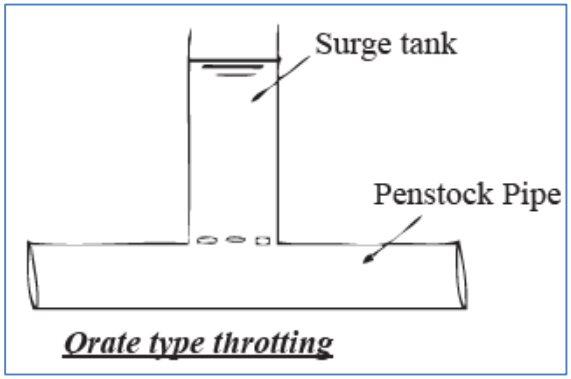
Simple cylindrical type



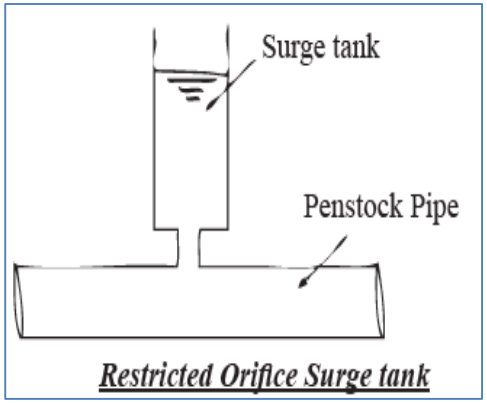
Orate type throtting

2) Restricted orifice or throttled surge tanks:

- It decreases amplitude of surge and effects more rapid reduction of mass oscillation and thus gives an economy up to 40% compared to simple tanks.
- Disadvantage is that it allows hammer pressures into conduit system.



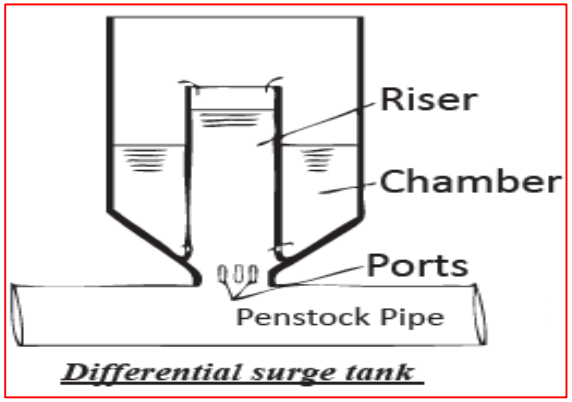
Orate type throtting



Restricted Orifice Surge tank

3) Differential surge tanks:

- Differential tank separates two functions of simple storing of additional water and effective accelerating and decelerating of water in main conduit by outside main tank and central river respectively.
- Thus this type a the combined advantage of a simple and restricted orifice type surge tanks shown in Figure.



Differential surge tank

Hydraulic Design of Surge Tank

Here, A_{St} = Area of Surge tank

- A_t = Area of tunnel
- L = length of tunnel
- Q_t = Discharge through tunnel at any time (t)
- z = variation in water level in surge tank

1) Maximum up – surge or down – surge is given by

- $$Z_{max} = \frac{Q_t}{A_{St}} * \sqrt{\frac{L\,A_{St}}{gA_t}}$$

2) Time for oscillation (T) = 4*t $= 4*\frac{\pi}{2}*\sqrt{\frac{L\,A_{St}}{gA_t}}$

- $$(T) = 2\,\pi * \sqrt{\frac{L\,A_{St}}{gA_t}}$$

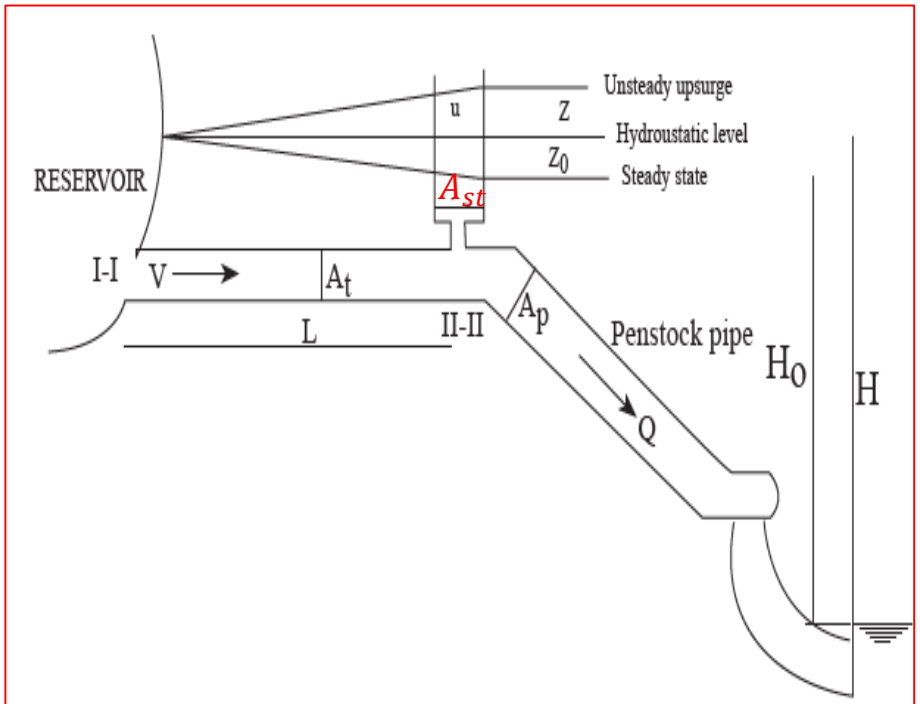


Fig Surge Analysis for Simple Surge Tank

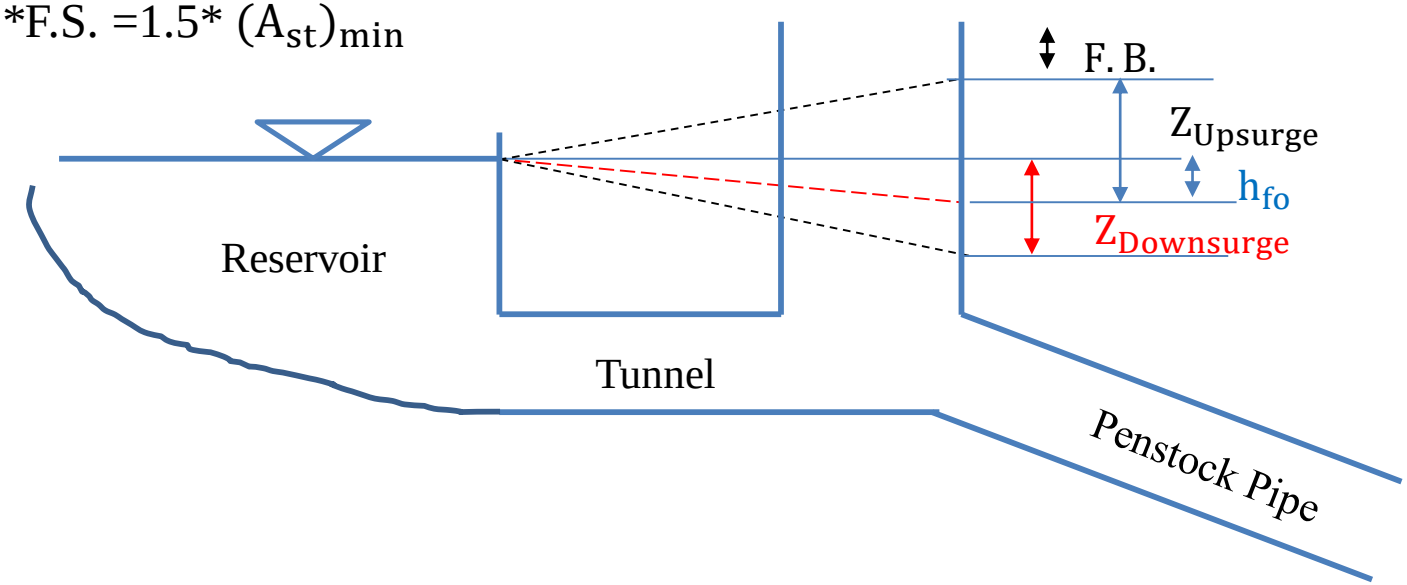
- Approximate formula for the calculation of upsurge with consideration of frictional effects:
- i) $\frac{Z_{Upsurge}}{Z_{max}} = 1 - \frac{2}{3}P_O + \frac{P_O^2}{9}$
- Where $P_O = \frac{h_{fo}}{Z_{max}}$

h_{fo} = frictional head loss in steady state condition

- ii) $\frac{Z_{Duwn\ surge}}{Z_{max}} = 2P_O - 1$

Hydraulic Design of Surge Tank….

- Thoma formula for calculation of Area of Surge tank
- $(A_{st})_{min} \geq \frac{A_t L V_0^2}{2gh_{fo}(H_g - h_{fo})}$
- Where, V_O = velocity of flow at the headrace tunnel
- H_g = Static head between the reservoir and power house (Gross head)
- In actual practice Area of surge tank
- $(A_{St}) = (A_{st})_{min}$*F.S. =1.5* $(A_{st})_{min}$



Height of surge tank (H) = $Z_{Upsurge}$ - h_{fo} + $Z_{Downsurge}$ + Free Board

1) In a hydropower project penstock of diameter 2.5m carries a flow of 6.5m3/sec to the turbine located in the power house a simple surge tank of diameter 5m is at a distance of 1500m from the reservoir, considering the instantaneous total closure if friction factor = 0.02, find (I) maximum height of surge tank required (II) total time period of oscillation wave.

Solution: here, Diameter of penstock (d) = 2.5m, Discharge through pipe (Q_T) = 6.5 m^3/sec, Surge Tank diameter (d_s) = 5m, Length of Tunnel (L) = 1500m

- Area of penstock $(A_T) = \frac{\pi * 2.5^2}{4}$
- $= 4.909 m^2$
- Area of Surge tank $(A_{ST}) = \frac{\pi * 5^2}{4}$
- $= 19.635 m^2$
- Velocity through Tunnel$(V) = \frac{Q}{A_T} = \frac{6.5}{4.909}$
- $= \frac{1.324 m}{s}$
- Maximum height at surge tank without considering frictional head loss $(Z_{max}) = \frac{Q_T}{A_{ST}} * \sqrt{\frac{L * A_{ST}}{g * A_T}}$
- $= \frac{6.5}{19.635} * \sqrt{\frac{1500 * 19.635}{9.81 * 4.909}}$
- $= 8.187 m$
- **Considering frictional losses**
- Frictional head loss $(h_{fo}) = \frac{FLV^2}{2gd} = \frac{0.02 * 1500 * (1.324)^2}{2 * 9.81 * 2.5}$
- $= \mathbf{1.0722 m}$

- $\therefore P_O = \frac{h_{fo}}{Z_{max}}$
- $= \frac{1.0722}{8.187} = 0.13$
- $\therefore \frac{Z_{upsurg}}{Z_{max}} = 1 - \frac{2P_O}{3} + \frac{P_O^2}{9}$
- $Z_{upsurg} = 8.187 * \left(1 - \frac{2*0.13}{3} + \frac{0.13^2}{9}\right)$
- $= 7.493\text{m}$
- $\therefore \frac{Z_{downsurg}}{Z_{max}} = 2P_O - 1$
- $Z_{downsurg} = 8.187 * \{(2 * 0.13) - 1)\}$
- $= \mathbf{-6.058m}$
- Time Period of Oscillation(T) $= 2\pi\sqrt{\frac{L*A_{ST}}{g*A_T}}$
- $= 2\pi\sqrt{\frac{1500*19.635}{9.81*4.909}}$
- $= \mathbf{155.385\ Sec}$
- Assume, Free Board = 2 m
- Height of surge tank (H) = $Z_{Upsurge}$ - h_{fo} + $Z_{Downsurge}$ + Free Board
- = 7.493 –1.0722 + 6.058 + 2
- = 14.479 m

2. A hydro electrical power channel is fed through a concrete lined tunnel of 4.9m diameter operating under gross head of 195m. The discharge through the tunnel is $28m^3$/sec. A surge tank of $295m^2$ has been provided at the end of the tunnel. Head lost due to friction under steady state flow condition is 2 % of gross head. Assuming friction factor of the tunnel to be 0.015.Find (I) Total length of the tunnel (II) Maximum upsurge in the tank (III) Maximum down surge in the tank (IV) Calculate factor of safety of the tank and ascertain whether it is adequate.

Solution: from question given as following data

- Discharge through tunnel (Q_T) = $28m^3$/sec, Friction value for headrace tunnel = 0.015, diameter of headrace lined tunnel (d) = 4.9m, Area of surge tank (A_S) = $295m^2$, Gross head (H) = 195m
- Velocity of flow through headrace tunnel$(V_O) = \frac{Q}{A_T}$
- $= \frac{28}{\pi * \frac{4.9^2}{4}} = 1.4848\ m/sec$
- From question given as, head loss due to friction= 2% of gross head
- =2%*195 = 3.9m
- Head loss due to friction in tunnel$(h_f) = \frac{F*L*V^2}{2*g*d}$
- Length of tunnel$(L) = \frac{h_f*2*g*d}{F*V^2} = \frac{3.9*2*9.81*4.9}{0.015*1.4848^2} = \mathbf{11337.897m}$
- Maximum height at surge tank $(Z_{max}) = \frac{Q_T}{A_{ST}} * \sqrt{\frac{L*A_{ST}}{g*A_T}}$
- $= \frac{28}{295} * \sqrt{\frac{11337.897*295}{9.81*\pi*\frac{4.9^2}{4}}}$
- $= \mathbf{12.763m}$

- Frictional head loss$(h_{fo}) = 3.9m$
- $\therefore P_O = \frac{h_{fo}}{Z_{max}}$
- $= \frac{3.9}{12.763}$
- $= 0.306$
- $\therefore \frac{Z_{upsurg}}{Z_{max}} = 1 - \frac{2P_O}{3} + \frac{P_o^2}{9}$
- $Z_{upsurg} = 12.763 * \left(1 - \frac{2*0.306}{3} + \frac{(0.306)^2}{9}\right)$
- $= \mathbf{10.292m}$
- $\therefore \frac{Z_{downsurg}}{Z_{max}} = 2P_O - 1$
- $Z_{downsurg} = 12.763 * \{(2 * 0.306) - 1)\}$
- $= \mathbf{-4.952\ m}$
- According to Thoma formula
- $A_{ST} \geq \frac{A_T * L * V_o^2}{2gh_f(H_g - h_f)}$
- $\text{i.e} A_{ST} = \frac{\pi * \frac{4.9^2}{4} * 11337.897 * 1.4848^2}{2 * 9.81 * 3.9(195 - 3.9)}$
- $= \mathbf{32.235m^2}$
- $\text{i.e. Factor of safety(F.S.)} = \frac{A_S}{A_{ST}}$
- $= \frac{295}{32.235}$
- $= \mathbf{9.152}$ so it is adequate value.

6.3 Penstocks and Pressure shaft: importance, conditions of their application, general arrangement, hydraulic transients (water hammer), Computation of hydrodynamic pressure, sizing of penstock /pressure shaft and estimation of thickness of steel in penstock/pressure shaft

Definition

- Penstock is pipes that supply water from forebay to turbines.
- These may be pressure conduits or shafts.
- Penstocks carry water to turbines with least possible loss of head consistent with overall economy of project.

Condition of penstock application

- In a power station situated at toe of a dam, penstocks are taken through the dam monoliths, with necessary transitions for smooth hydraulic flow and control devices at the intake structure.
- In underground power development these penstocks meet spiral case which supplies water to the machine. Where there is an elbow in penstock just ahead of turbine, a reducer is required. Minimum losses usually results from shaping the elbow to serve as a reducer.

Classification of penstock

Penstock are classified on the basis of

i) Material of fabrication :

- Steel penstocks
- Reinforced concrete
- Asbestos cement
- Wood
- Cast irons

ii) Supporting of penstocks pipes

- Buried penstock
- Penstock at saddle support
- Penstock at girder support
- Embedded penstock

iii) Rigidity of supports

- Rigid pipe line with anchors
- Flexible pipe lines
- Semi rigid pipes lines

iv) Location of penstocks

- Buried penstocks
- Exposed penstocks
- Partly buried and partly exposed penstock

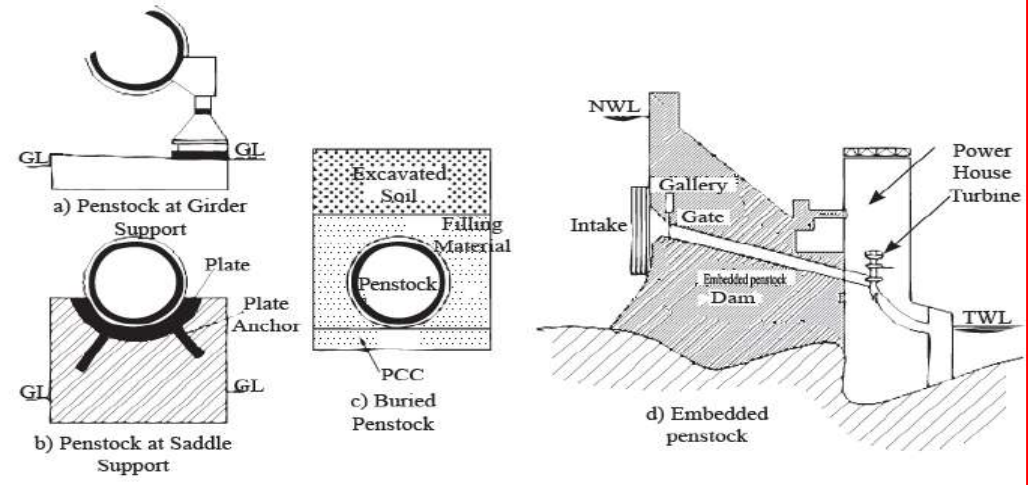


Fig Supporting Types of Penstock Pipe

Buried penstocks: Such penstocks are supported continuously on soil at bottom of a trench back filled with excavated material.

Advantages:

- Protection against effects of temperature.
- Protection against freezing of conveyed waster.
- Protection against earthquake shocks.
- Protection against animals.
- Expansion joint not needed.

Disadvantages:

- Less accessible for inspection.
- Possibility for damage by corrosion action.
- Difficult maintenance and repairs.

Exposed penstocks: These are installed above terrain surface and are supported on pier or saddles. The penstocks are not in contact with ground surface.

Advantages:

- Easy and regular inspection.
- Installation less expensive.
- Easy maintenance and repairs.

Disadvantages:

- Expansion joints necessary.
- Not safe against sliding.
- Not safe against animals.

Design of Penstock

- The empirical formulae are some over simplified thumb rule solutions based on the available experience.
- These may not fit to conditions different than those from which they are deduced. Following empirical formulae are popular for penstock design

a) USBR formula:

$V = 0.125\sqrt{2gH}$

b) According to Sarkaria Formula:

$D = 0.62 * \frac{P^{0.35}}{H^{0.65}}$

H= maximum net head at the end of the penstock in (m)

D= penstock diameter in (m)

P = hp transmitted by the pipe

c) Design Guideline by JNN

$D_e = \left(\frac{5.2Q_d^3}{H_d}\right)^{1/7}$

d) Thickness of the penstock : The ASME code gives the formula for thickness as follows, $t = \frac{pR}{(\sigma * \eta) - 0.6p} + 0.15$

Where, V= optimum velocity in m/s

t = wall thickness in (cm)

σ = allowable stress Kg/cm2

p = pressure in kg/cm2

R= internal radius in (cm)

η = joint efficiency factor and 0.15 cm is allowance for correction

Q. N. A penstock of discharging capacity of $5m^3/s$ is to be functioned in a hydropower station with a design head of 50m. Determine the economic diameter of the penstock. Take overall efficiency of the plant as 85%.

- **Solution:** Discharge (Q) = $5m^3/s$, Head (H) = 50m, overall efficiency (Ƞ) = 0.85

➢ **Using USBR (United States Bureau of Reclamation)**

$$V = 0.125\sqrt{2 * g * H}$$

$$= 0.125 * \sqrt{2 * 9.81 * 50} = 3.915m$$

- $\text{Area}(A) = \frac{Q}{V}$
- Or, $\frac{\pi * D^2}{4} = \frac{5}{3.915}$
- or, $D = \sqrt{\frac{5*4}{3.915*\pi}} = 1.275m$

➢ **Using Sarkaria formula,** $D = 0.62 * \frac{P^{0.35}}{H^{0.65}}$

Where Power $(P) = \eta * \Upsilon * Q * H = 0.85 * 9.81 * 5 * 50 = 2084.625KW = \frac{2084.625*1000}{746} = 2794.403HP$

- $\therefore D = 0.62 * \frac{(2794.403)^{0.35}}{(50)^{0.65}}$
- $= \mathbf{0.784m}$

➢ **Using JNN Guide line**

- $D = \sqrt[7]{\frac{5.2*Q^3}{H}}$
- $= \sqrt[7]{\frac{5.2*5^3}{50}} = \mathbf{1.44m}$

Q. N. A penstock having internal diameter 2.5m supply water from head of 150m, there is a possibility of the 25% increase in the pressure head due to the transient condition of the flow. If the efficiency of the joint and design stress is 85% and 1250 Kgf/cm^2 respectively. Find the thickness of penstock pipe.

Solution: From question given as following data

- Internal diameter (d) = 2.5m, head = 150m, increase in pressure = 25% of head
- η = 85%, design stress = 1250Kg/cm^2
- $\text{Internal Radius(R)} = \frac{2.5*100}{2} = 125\text{cm}$

$\text{Increase in pressure} = 25\% \text{ of H} = \frac{25*150}{100}$

$= 37.5\text{m}$

- $\therefore \text{Pressure head} \left(\frac{P}{\gamma}\right) = 150 + 37.5$
- 187.5m
- $\text{i.e. pressure(P)} = \frac{(9.81*1000*187.5)}{9.81}$
- $= 187500\ \text{Kg/m}^2$
- $= \frac{187500}{100^2}$
- $= 18.75\ \text{Kg/cm}^2$
- I.e. Pressure (P) = 18.75 Kg/cm^2
- $\text{Wall thickness(t)} = \frac{PR}{f\eta - 0.6P} + 0.15$
- $= \frac{18.75 * 125}{(1250*0.85)-(0.6*18.75)} + 0.15$
- $= \mathbf{2.379cm}$

Q. N. A steel penstock pipe is 500m long with internal diameter 0.8m to supply water at the rate 4.2m^3/s for the head of 120m. Find the wall thickness of penstock pipe for 40 sec gate closure time. Take wave velocity 1100m/s, allowable stress of pipe material 1300Kgf/cm^2 and joint efficiency 90%.

Solution: Time taken by pressure wave from reservoir to gate and back to the reservoir $(T) = \frac{2L}{C} = \frac{2*500}{1100} = 0.909$ Sec

- Here $T_C > T$ The closure is slow (gradual)closure.
- Hence $\frac{h_d}{h_O} = \frac{K_1}{2} \pm \sqrt{K_1}$ Where $K_1 = \left(\frac{LV_O}{gh_OT}\right)^2$
- Velocity through penstock $(V_O) = \frac{4Q}{\pi*d^2} = \frac{4*4.2}{\pi*0.8^2} = 8.356\text{m/s}$
- $\therefore K_1 = \left(\frac{LV_O}{gh_OT}\right)^2$
- $K_1 = \left(\frac{500*8.356}{9.81*120*40}\right)^2 = 7.872 * 10^{-3}$
- i. e. $\frac{h_d}{h_O} = \frac{K_1}{2} \pm \sqrt{K_1}$
- Or, $h_d = 120 * \left(\frac{7.872*10^{-3}}{2} + \sqrt{7.872 * 10^{-3}}\right)$
- $= \mathbf{11.12m}$
- Pressure (P) $= (11.12 + 120) * 9.81 = 1286.287\,\text{KN/m}^2 = \frac{1286.287*1000}{9.81*100^2} = 13.112\,\text{Kg/cm}^2$
- $\therefore$ Wall thickness $(t) = \frac{PR}{f\eta - 0.6P} + 0.15$
- $= \frac{13.112*40}{(1300*0.9)-(0.6*13.112)} + 0.15 = \mathbf{0.601\ cm}$

THANK YOU
STAY HOME
STAY SAFE

Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

Unit 7 :

- **Hydro-electric Machines [6 hours]**

7.1 Hydro-mechanical Equipment

7.1.1 Hydro-mechanical installation in powerhouse

7.1.2 Type of turbines, Pelton, Francis, Kaplan and Bulb turbines and their performance characteristics

7.1.3 Selection of turbines and their specific speed, Turbine setting

7.1.4 Preliminary design of Francis and Pelton turbines

7.1.5 Scroll case and draft tubes, their importance

7.2 Electro-mechanical installation

7.2.1 Generators and their types, Rating of generators

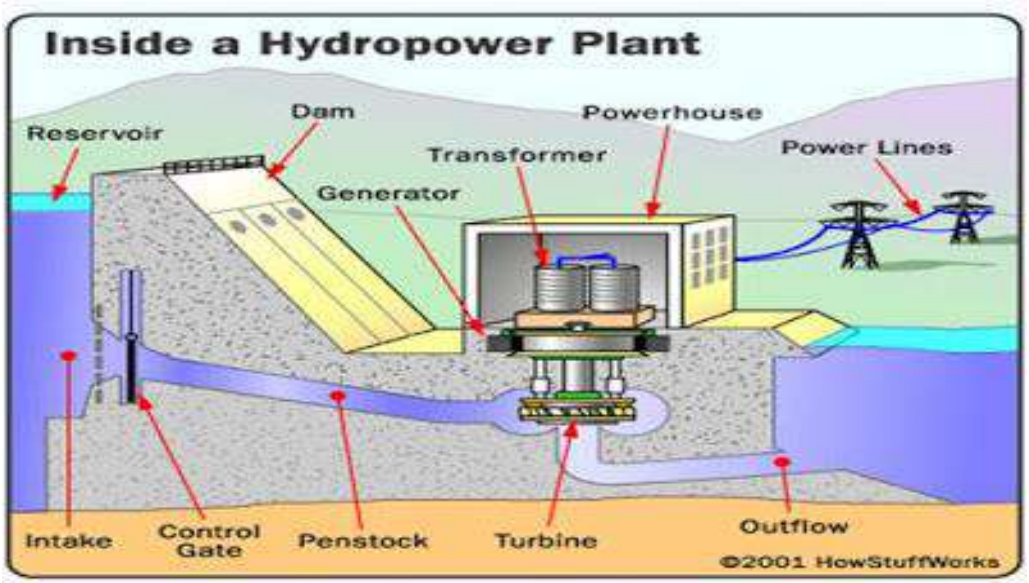
7.2.2 Purpose and working principle of Governs

7.3 Pumps

7.3.1 Introduction to Centrifugal and reciprocating pumps, their performance characteristics

Introduction:

- Hydraulic turbine may be considered as hydraulic motors or prime movers of a water power development which converts water energy (hydropower) into mechanical energy (shaft energy).
- The shaft power developed is used in running electricity generators directly coupled to the shaft of the turbines thus producing electrical power.
- In hydroelectric power plants, water turbines are used to convert the energy of running water into mechanical energy.
- That energy is afterwards utilized for driving the electrical generators to produce electrical energy.
- They are built in various sizes up to 1000000 hp with speed varying from 100 rpm to 1000 rpm depending on their size.
- The hydraulic turbine is a rotary machine actuated by the impulse and/or reaction of a water current attacking the rotor (runner) which consists of a series of buckets or vanes.



Classification of water turbine

They are classified in many ways on the basis of;

- Action of water on moving blades
- Head and quantity of water available
- Type of flow of water
- Name of originator
- Specific speed of the machine

Based on action of water on moving blades

Water turbines are two types

a) Impulse turbine

b) Reaction turbine

a) Impulse turbine:

- The water flowing through the penstock has always some pressure head besides a kinetic head.
- At the inlet to the turbine runner, all of the potential energy (head) of the water is converted into kinetic energy with the help of a contracting nozzle (flow rate controlled by spear type valve) provided at the delivery end of the penstock.
- After impinging on the curved bucket, the water discharge freely (at atmospheric pressure) into the tailrace channel.
- Such a design of the turbine is called pressure-less or Impulse turbines.
- The most commonly used impulse turbine is the **Pelton wheel.**

b) Reaction turbine:

- Major part of the available water energy is converted into kinetic energy at the entrance to the runner and a considerable remaining part is also utilized as pressure energy.
- Both kinetic and pressure energy are directly converted into mechanical energy by turbine runner before generating electrical energy.
- The runner casing (called the scroll case) has to be completely airtight and filled with water throughout the operation of the turbine.
- Such turbines are called reaction turbine.
- The water enters the scroll case and moves into the runner through a series of guide vanes called Wicket gates.
- After leaving the runner, the water enters a draft tube which delivers the flow to the tail race.
- There are two main types of reaction turbines, the Francis and propeller (also Kaplan or bulb) turbines.

➢ Pelton turbine is an impulse turbine, in which the water flowing over the turbine rotor blades remains constant.

➢ Francis, Kaplan and Propeller turbines are **reaction turbine**, in which, the rotor of the turbines operates while submersed in water, the turbine casing being full of water. As the water flows through the rotor blades their pressure change.

Differences between Impulse And Reaction Turbine:

Aspect	Impulse turbines	Reaction turbine
Conservation of fluid energy	All the available water energy is converted into kinetic energy in the forms a free jet through Nozzle.	Only a portion of the fluid energy is converted into kinetic energy before the fluid enters the turbine runner.
Change of pressure and energy	Atmospheric pressure throughout the action of water on the runner.	After entering the runner with excess pressure, available Water energy undergo changes into both the velocity and pressure while passing through runner.
Function of blades	Only the blades in front of the nozzle are in action.	All the blades are action at all the times.
Water movement	Water may be circulated partially or in whole of the wheel circumference.	Water is circulated all over the circumference of the wheel.
Flow velocity and pressure	The relative velocity of water gliding over the moving blades either remains constant or reduces slightly due to friction.	The relative velocity of water increases because there is continuous drop in pressure during flow through the blade passages.
Installation of units	Turbine is installed above tail race.	Turbine is kept entirely submerged in water below the tail race.
Specific speed	Low (10-40)	Moderate to high Francis (50-300) Kaplan(240-920)

 # Based on the head and quantity of available water

Based on the head, the turbines are classified into following four classes

i) Low head (head between 2m -15m)

ii) Medium head (head between 16m-70m)

iii) High head (head between 71 to 500m)

iv) Very high head (head 500m)

- **For low head** – Only propeller /Kaplan turbine are used.
- For medium head - Either Kaplan or Francis turbine are used.
- **For high head** - Either Francis or Pelton wheels are used.
- For very high head - Only Pelton turbines is used.

 # Based on the discharge,

- Pelton wheel turbines for low discharge turbine
- Kaplan turbines are high discharge turbine
- Francis is intermediate position

 # Based on the type of water flow

There are four types of water turbines used as prime movers in hydro-electric power stations. Namely they are as follow:

i) Axial flow turbines – having flow of water along the shaft axis

ii) Inward radial flow turbines-having flow of water along the radius

iii) Tangential or peripheral flow turbines - having flow of water along tangential direction

iv) Mixed flow (radial inlet and axial outlet) turbines

 # Based on the specific speed

- The specific speed of the turbines is defined as the speed of a geometrically similar turbine (i.e. a turbine identical in shape, dimension, blade angles and gate opening) that would develop one metric hp under a head of one meter.
- All geometrically similar turbines have the same specific speed when operating under the same conditions of head and flow.
- The specific speed Ns of a turbine is its most important characteristic and it is an important parameter for the design of the turbine as it includes all the three basic parameter i.e. speed, power and head of the turbine.
- Power available at the turbine shaft, $p = \eta QH$ ……(1)
- Since η are constant, p QH
- The flow velocity Vf, the absolute velocity V, tangential velocity u nd head H on the turbines are related as

$u \alpha V_f \alpha \sqrt{H}$…..(2)

- Peripheral velocity, u = DN …..(3)
- From equation 2 and 3
- $D \alpha \frac{u}{N} \alpha \frac{\sqrt{H}}{N}$ …..(4)
- Also, $Q = A*Vf \quad \alpha D^2*Vf$
- or $Q \alpha \frac{H}{N^2}*\sqrt{H}$
- Now, equation (1) becomes
- $P \alpha \frac{H}{N^2}*\sqrt{H} *H$
- $P \alpha \frac{H^{2.5}}{N^2}$

 # Based on the specific speed …..

- $P = K\frac{H^{2.5}}{N^2}$ …..(5)

Where K= proportionality constant

According to definition of specific speed, H= 1 m, P = 1 hp, then N= Ns

- $1 = K\frac{1^{2.5}}{N_s^2}$
- $N_s^2 = K$
- Substituting the value of K in equation (5) we get the specific speed.
- $P = N_s^2\frac{H^{2.5}}{N^2}$ Therefore specific speed for turbine is given as $(\mathbf{N_s}) = \frac{\mathbf{N\sqrt{P}}}{\mathbf{H^{1.25}}}$ **….(6)**
- Where Ns =specific rotational speed in metric units (rpm)
- N = actual rotational speed of turbine in rpm
- P = output in metric hp
- H =effective head in meters

 # Specific speed for different turbines

- For Francis turbine $(N_s) = \frac{2400}{H^{1/2}}$
- For Kaplan turbine $(N_s) = \frac{1475}{H^{1/3}}$
- For Propeller turbine $(N_s) = \frac{1080}{H^{1/4}}$

 # A classification of turbines according to the range of head and specific speed is given in tabular form:

Type of turbine	Specific speed in metric units	Head and other conditions
Kaplan or movable vane propeller	300-1000	Head less than 30m and for variable load condition
Fixed vane propeller	300-10000	Head less than 70m and fairly constant load operation
Francis	60-300	Head 30m-200m
Pelton	10-50	Head above 200m

 # Speed factor:

The speed factor 'Ø' is the ratio of the peripheral speed 'v' of the bucket or vanes at the nominal diameter 'D' to the theoretical velocity of water under the effective head 'H' acting on the turbine.

$$\emptyset = \frac{V}{\sqrt{2gH}}$$

$$= \frac{\pi DN}{60\sqrt{2gH}}$$

$$\emptyset = \frac{DN}{84.6\sqrt{H}}$$

By empirical relation of the speed factor is,

- $\emptyset = 0.0197\, N_s^{2/3} + 0.0275$ Francis turbine
- $\emptyset = 0.0242\, N_s^{2/3}$ Propeller turbine (here Ns is in Metric unit)

 # Synchronous speed

- If the turbine is directly connected to the generators, producing electricity the turbine speed 'N' must correspond to the nearest synchronous speed for a. c. machine. For turbine speed, N to be a synchronous, the following equation must be fulfilled.
- $N = \frac{120f}{N_P}$
- N = rotational speed
- f= electrical frequency in HZ (50 HZ)
- N_P = number of generator poles

Pelton (Impulsive) turbine:

- A Pelton turbine is the type of turbine that comprise of a rotor, at a periphery of which are mounted equally spaced double hemispherical of double elliptical buckets.
- Water flows from a high head through the penstock and transferred through the nozzle, which is fitted at the end of the penstock.
- All available water energy (potential energy) is converted into kinetic energy before the jet strikes buckets of runner.
- The pressure all over the runner remains constantly equal to atmospheric pressure, so that energy transfer occurs due to purely impulse action.
- Casing has no hydraulic function to execute besides preventing splashes and guiding water to the tail race.
- In Nepal, the pelton turbine is used in Khimti hydropower (60MW, H=650m), Kulekhani I (60 MW) and Kulekhani II (32 MW).
- It is suitable for high head plants.
- **Component parts of Pelton Turbine**
- Penstock
- Spear and Nozzle
- Runner with buckets
- Casing
- Governing Mechanism

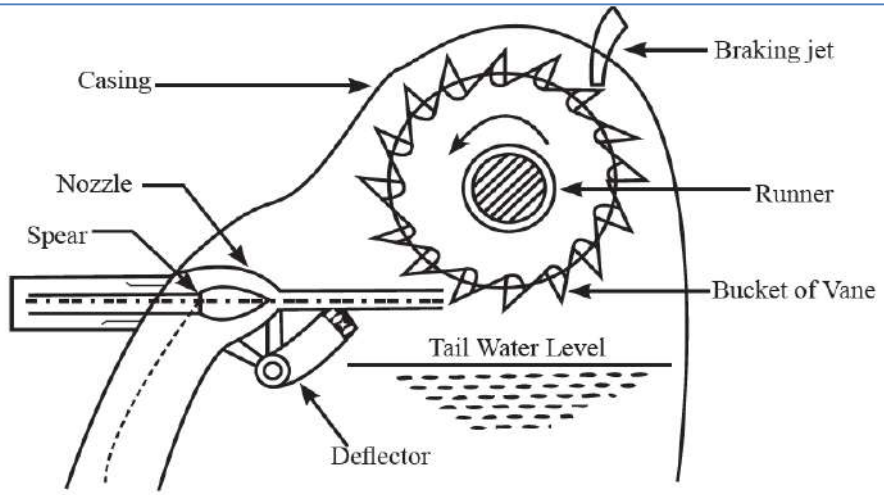


Fig : Runner with Buckets

 # Component parts of Pelton Turbine:…

i) Penstock

ii) Spear and Nozzle

- The penstock is fitted with an efficient nozzle with spear at the end of it that convert the whole of potential energy in to kinetic energy into jet.
- Water is regulated at all loading condition through the nozzle with the help of spear by moving backward or forward thereby increasing or decreasing the annular area of the nozzle flow passages.
- The movement of the spear is controlled either manually or by a hand wheel in case of very small units or automatically by a governing mechanism in case of almost all the bigger units.

iii) Runner with buckets

- The turbine runner is a circular disk having a number of cup - shaped buckets which are fitted equal distantly around periphery of the disk.
- The runner is mounted on a horizontal shaft supported in small thrust bearings, and buckets are cast integrally with the disk.
- Buckets may be manufactured of cast iron, bronze, cast steel or stainless steel depending upon the available heads.

iv) Casing

- A casing is provided around the runner to prevent splashing all directions out flow from runner and to guide the water to tailrace.
- It also acts as a safeguard against accident.

v) Governing Mechanism:

- Speed of the turbine runner should be maintained constant so that the electric generator coupled directly to the turbine shaft runs at constant speed under varying load conditions. It is maintained by using governor that automatically regulates the quantity of water flowing through the runner in accordance any variations of load.

Design of Pelton (Impulsive) turbine:

Design outcome should be

i) Diameter of nozzle (d) ii) Diameter of runner with buckets (D)

iii) Number of buckets(Nb) iv) Spacing of buckets

v) Width of the buckets (w) vi) Design speed(N) vii) Specific speed (Ns)

The following points need consideration while designing a pelton turbine.

1) Velocity of jet: The theoretical velocity of the jet is given by **(V') =** $\mathbf{C_V\sqrt{2gH}}$

Where C_V = coefficient of velocity = 0.98 or 0.99

2) The velocity of jet (tangential Velocity of the wheel) $\bar{V} = \emptyset\sqrt{2gH}$

Speed Ratio $(\emptyset) = \frac{\bar{V}}{\sqrt{2gH}}$

3) Diameter of wheel (D), Pitch diameter or mean diameter

$$\emptyset = \frac{DN}{84.6\sqrt{H}}$$

$$D = \frac{84.6\,\emptyset\sqrt{H}}{N}$$

4) Diameter of the jet (d):

We know (Q') $= \frac{P}{\eta\gamma H}$

Discharge through jet (Q') $= \pi\frac{d^2}{4} * V'$

$$\therefore \mathbf{d} = \sqrt{\frac{4Q'}{\pi V'}}$$

 # Design of Pelton (Impulsive) turbine….

5. Jet ration:

It is the ratio of pitch diameter (D) to the diameter of the jet (d).

∴ Jet ratio (m) = $\frac{D}{d}$ (12 in most case but takes as 10 – 15)

6) Width of the bucket (B) = 5d

7) Depth of the bucket = 1.2*d

8) Number of bucket o a runner (N_b) = 15 + $\frac{D}{2d}$

9) Number of the jet (n)

Total Discharge (Q) = n* $\pi \frac{d^2}{4}$*V'

∴ n = $\frac{\text{Discharge through wheel}}{\text{Discharge through jet}}$

Q. N. A Pelton turbine develops 6000Kw under a net head of 300m. The ratio of jet diameter to runner diameter is 1:10, overall efficiency of the turbine is 85% & rotational speed is 550 rpm. Find the number of jets, diameter of jet, diameter of wheel & quantity of water required. Take $C_V = 0.98$ & speed ratio = 0.46.

Solution: From question given as following data

- $P = 6000Kw$, $H = 300m$, $\eta = 85\% = 0.85$, $N = 550$ rpm, $C_V = 0.98$, $\emptyset = 0.46$, $\frac{d}{D} = \frac{1}{10}$
- We have, Power (P) = $\eta \Upsilon QH$
- $Q = \frac{P}{\eta \gamma H} = \frac{6000}{0.85*9.81*300} = \mathbf{2.399\ m^3/s}$
- Velocity of water through jet $(V') = C_V\sqrt{2gH} = 0.98\sqrt{2*9.81*300}$
- $= 75.19\ m/s$
- Also we know Speed ratio $(\emptyset) = \frac{\bar{V}}{V} = \frac{\pi DN}{60\sqrt{2gH}}$
- $D = \frac{\emptyset 60\sqrt{2gH}}{\pi N} = \frac{0.46*60*\sqrt{2*9.81*300}}{\pi*550}$
- $= 1.225m$
- From question given as $\frac{d}{D} = \frac{1}{10}$ $\therefore d = \frac{D}{10} = \frac{1.225}{10} = 0.1225m$
- Area of jet (a) $= \frac{\pi d^2}{4} = \frac{\pi*0.1225^2}{4} = 0.0118\ m^2$
- $\therefore$ Number of jet $(n) = \frac{Q}{a*V'} = \frac{2.399}{0.0118*75.19} = 2.703 \cong \mathbf{3}$

Francis turbine

- Fancies turbine is an inward flow reaction turbine which was designed and developed by the American engineer James B. Francis (1815-1892). It operates under medium heads and also required medium discharges. In the earlier stages, it is purely radial flow runner. However the modern Francis turbine is an inward mixed flow reaction turbine. When the water is supplied down to the turbine through the penstock, it is directed to a number of stationary orifices, called guide vanes or wicket gates which are fixed all around the circumference of runner. The runner of turbine contains 12-22 numbers of curved blades depending upon the specific speed.
- In this turbine, the pressure at inlet is more than pressure at the outlet and runner is full of water. The moment of the runner is affected by the change of both the potential and kinetic energies of the water. After water at outlet of the runner, water is discharged to the tail race through a closed tube of gradually enlarged section.
- Francis turbine is used in Kaligandaki (H= 115m, capacity =144 MW), Marsyandi (H= 95 m, capacity = 69MW) and Seti (H= 22m, Capacity 1.5 MW)

Component parts of the reaction turbine

- The main component parts of the Francis turbine are as follows:
- i) Penstock
- ii) Scroll casing
- iii) Guide vanes or wicket gate
- iv) Governing mechanism
- v) Runner and runner blades

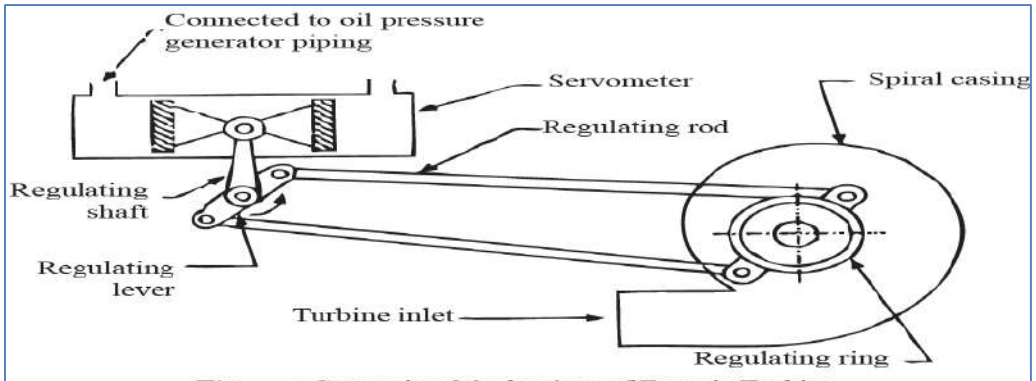


Fig Governing Mechanism of Francis Turbine

Design of Francis (Reaction) turbine:

Design steps

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

$$N_P = \frac{120f}{N}$$ where N_P=no of poles

iv) Calculate corrected synchronous speed $N = \frac{120f}{N_P}$

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

$\varnothing = 0.0197N_s^{2/3} + 0.0275$ for Francis turbine

vii) Setting of turbine $H_s = H_a - H_v - \sigma H$

Proposed hydropower developments have H_{net} = 195m, Discharge (Q) = 70m^3/sec, Turbine efficiency = 89.28%. Calculate specific speed and diameter of Francis turbine.

Solution: Q=70m^3/s, H=195m, η=89.28%=0.8928

- For Francis turbine , Specifec speed$(N_S) = \frac{2400}{\sqrt{H}} = \frac{2400}{\sqrt{195}}$
- $= 171.867$ rpm
- Power $(P) = \eta \Upsilon Q H = 0.8928 * 9.81 * 70 * 195 = 119551.7232$ Kwatt
- $= \frac{119551.7232}{0.746} = \mathbf{160257\ HP}$ {1HP = 746 Watt = 0.746 KW}
- Also Specific Speed $(N_S) = \frac{N\sqrt{P}}{H^{\frac{5}{4}}}$
- $\therefore N = \frac{N_S H^{\frac{5}{4}}}{\sqrt{P}} = \frac{171.867 * 195^{\frac{5}{4}}}{\sqrt{160257}} = \mathbf{312.843}$ rpm
- No of Pole $(N_P) = \frac{120f}{N} = \frac{120*50}{312.843} = 19.18 \cong \mathbf{20}$ {Assume Frequency = 50 Hz}
- Corrected Synchoronus Speed$(N) = \frac{120*50}{20} = \mathbf{300\ rpm}$
- Corrected Specific Speed$(N_S) = \frac{N\sqrt{P}}{H^{\frac{5}{4}}} = \frac{300\sqrt{160257}}{195^{1.25}} = \mathbf{164.81}$ rpm
- For Francis turbine $\emptyset = 0.0197 N_S^{2/3} + 0.0275 = 0.0197(164.81)^{2/3} + 0.0275 = 0.6197$
- $\therefore$ Diameter of penstock pipe$(D) = \frac{84.6\emptyset\sqrt{H}}{N} = \frac{84.6*0.6197\sqrt{195}}{300} = 2.44\text{m} \approx \mathbf{2.5m}$

Selection of turbine:

- Selection of turbine type and determination of plant capacity turbine require the detailed information on head and possible discharges for different conditions.

The following points should be taken under consideration for choosing appropriate type of turbines for hydropower plant.

- **i) Specific speed:** It is a fundamental index for the selection of correct type of turbine for a particular set of conditions. High specific speed for a given head is vital. If head is low and output is large, the rotational speed will be low which means cost of turbo-generator and powerhouse will be high.
- Furthermore, at **high head for a given speed** and power output, a low specific speed turbine like impulse turbine is essential for high efficiency.
- **ii) Rotational speed:** Rotational speed varies with the variation of specific speed. The rotational speed of an electrical generator directly coupled with the turbine depends on the frequency and number of pair of poles. The value of specific speed should be adopted with the synchronous speed of the generator and its frequency.
- **iii) Efficiency:** The turbine should be selected to give the highest overall efficiency for various operating conditions.
- **iv) Load operation:** Usually, the efficiency is less than that design parameters at part loads and overloads. For sake of economy, the turbine should be always run with maximum possible efficiency to acquire more revenue.
- **v) Cavitation:** Cavitation is an important consideration in the selection of turbine for given head and specific speed. The installation of the reaction turbine over the tailrace is effected by cavitation. The critical value of cavitation limit must be obtained to see that the turbine works in safe zones. Generally, high speed turbine is not used for high heads with consideration of cavitation.

 # Selection of turbine:…..

- **vi) Available head and its fluctuation:**
- a) For head greater than 350m (very high head), Pelton turbine is generally used and there is practically no choice except in special cases.
- b) For high heads (150m-350m), either Pelton or Francis may employs. For higher specific needs Francis turbine is more compact and economical than Pelton that for the same working conditions.
- c) For medium head (60m -150m), Francis turbine is usually employed.
- d) For low heads (below 60m), between 30 to 60 m both Kaplan and Francis may be employed.
- However Francis is more expansive but yields higher efficiency at part loads and over loads.
- Kaplan is generally used under 30 m.
- Propeller is commonly employed for heads up to 15 m. However they are adopted only no load conditions.
- **vii) Position of shaft:**

Experimentally shows that the horizontal arrangement of shaft is preferred for large sized impulse turbine but in case of large size reaction turbines, vertical shaft arrangement is adopted.

- **viii) Water quality:**

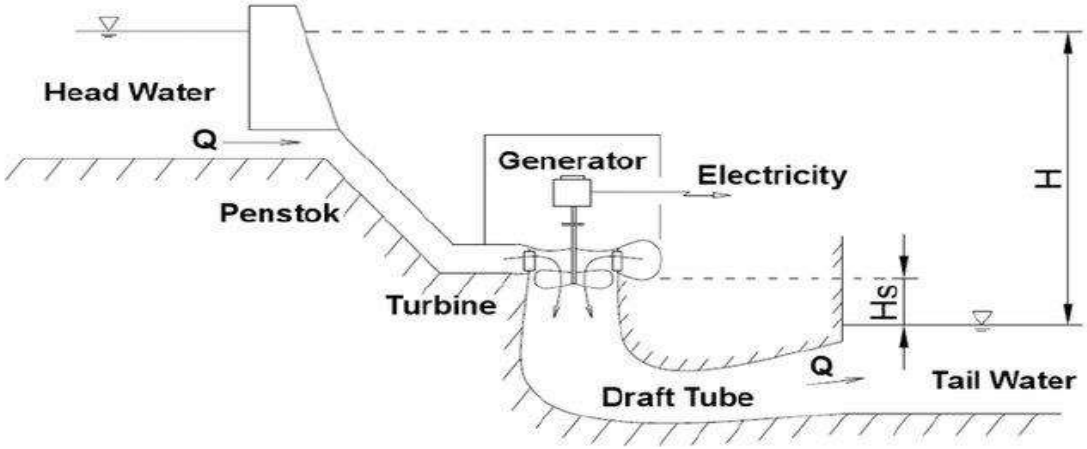
Impulse turbine is more effected by sand particles, chemical or other impurities due to wear than reaction turbine.

- **ix) Maintenance:**

Guide vanes nozzles and runner blades are in danger to damage by cavitation and necessitating repair. Francis can consent design, replacement of all runner blades without dismantling generator. Pelton turbines have the plus point of replacing spear and nozzle quickly.

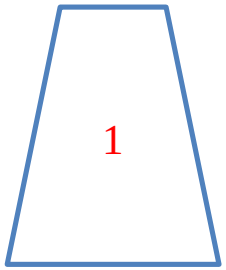
Draft Tube:

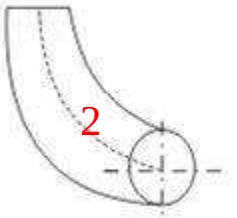
- The draft tube is conical discharging water from the runner to the tail race and having following purposes:
- To recover as much as possible of the velocity energy of the water leaving the runner, thus increasing the dynamic draft head.
- To utilize the vertical distance between the turbine exit and the tail water level called the static draft head.
- To increase efficiency of turbine.



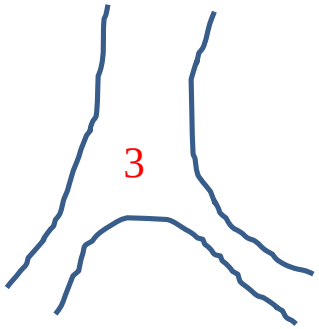
Types of Draft tube:

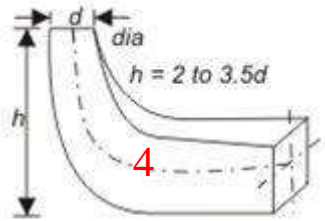
1) Conical draft tube
2) Simple elbow tube
3) Moody spreading tube
4) Draft tube with circular inlet and rectangular outlet





Simple elbow type

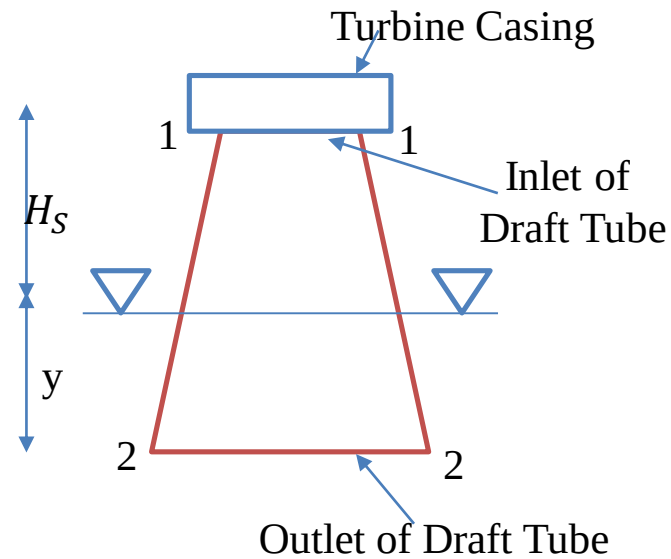




Elbow type with varying cross-section

Draft Tube:……

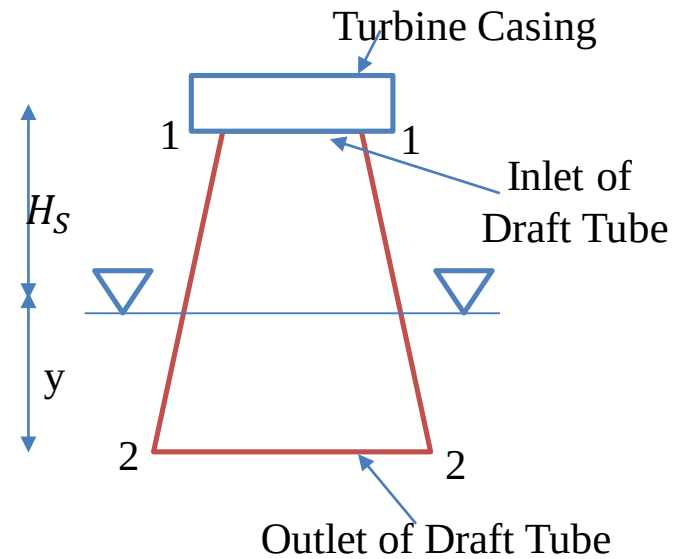
- Consider a conical draft tube section 1 is the exit from the runner and section '2' is the exit of draft tube. Let 'y' is the distance of bottom of draft tube from tail race and H_S be the suction heads.
- Applying Bernoulli's equation between point 1 and 2
- $\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 + h_f$



- $\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + H_S + y = (\frac{P_a}{\gamma} + y) + \frac{V_2^2}{2g} + h_f$
- $\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + H_S = \frac{P_a}{\gamma} + \frac{V_2^2}{2g} + h_f$
- $\mathbf{\frac{P_1}{\gamma} = \frac{P_a}{\gamma} - H_S - \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} - h_f\right)}$**……(1)**
- In this equation $\frac{P_1}{\gamma}$ is less than atmospheric pressure.
- **<u>Efficiency of Draft tube:</u>**
- $\eta = \frac{\text{Actual conversion of Kinetic head in to prssure head}}{\text{Kinetic head at the inelt of draft tube}}$
- Theoretical conversion of Kinetic head into pressure head in draft tube $\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$
- Actual conservation of kinetic head into pressure head $= \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} - h_f\right)$
- $\eta_d = \frac{\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} - h_f\right)}{\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)}$

Q.N. Determine the overall efficiency of Kaplan turbine developing 2850KW under a head of 5.2m. It is provided with a draft tube with its inlet diameter 3m set 1.8m above the tailrace level. A vacuum gauge connected to the draft tube indicates a reading of 5.2m of water. Assume draft tube efficiency as 75%.

Solution: P= 2850 KW, H= 5.2m, η= 75%=0.75, $\frac{P}{\gamma} = -5.2$ m of water



- $\text{Efficiency}(\eta) = \dfrac{\frac{V_1^2 - V_2^2}{2g} - h_f}{\frac{V_1^2}{2g}}$
- Applying Bernoulli's equation at inlet and outlet
- $\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 + h_f$
- $\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + (1.8 + y) = \frac{P_a}{\gamma} + y + \frac{V_2^2}{2g} + 0 + h_f$
- $\frac{P_1}{\gamma} + 1.8 = \frac{P_a}{\gamma} - \left(\frac{V_1^2 - V_2^2}{2g} - h_f\right)$
- $\frac{P_1}{\gamma} + 1.8 = \frac{P_a}{\gamma} - \left(\eta \frac{V_1^2}{2g}\right)$
- Assume $\frac{P_a}{\gamma} = 10.3$m of water
- $-5.2 + 1.8 = 10.3 - \left(0.75 * \frac{V_1^2}{2g}\right)$

- $V_1 = \sqrt{\frac{13.2*2*9.81}{0.75}}$
- $= 18.583\ m/s$
- Discharge $(Q) = A_1 * V_1$
- $= \frac{\pi*3^2}{4} * 18.583$
- $= 131.355\ m^3/s$
- Power $(P) = \eta \Upsilon Q H$
- $\eta = \frac{P}{\gamma Q H}$
- $= \frac{2850}{9.81*131.355*5.2}$
- $= 0.4253$
- $= \mathbf{42.53}\%$

 # Cavitation in turbine:

- The most problems for turbine maintenance are that of cavitation.
- Cavitation is a physical phenomenon created in fluid flow, whenever the prevailing pressure is near the vapour pressure i.e. vacuum pressure.
- As the pressure falls toward vapour pressure, the water starts vaporizing and at the same time, normally dissolved gases also liberated due to low pressures thus in the flowing water, minutes microscopic bubbles are formed containing vapour and gases.
- The bubbles can be formed cavities in the flow.
- The bubbles are generated continuously and in very large numbers.
- They may remains attached to the solids surfaces and form an attached cavity near the surfaces or they may be get transported with the flow towards regions of the higher pressures.
- The bubbles collapse resulting in a rapid change in flow causes due to the surface tension effect of the cavity bubbles.
- cavitation also produces noise and vibration of the machine. The end result is a drop in the efficiency of the machine.
- **The effect of cavitation is formed in**
- a) Hydraulic turbines
- b) Pumps
- c) Propeller of ships
- d) On the spillway of the dams
- e) On draft tubes and runner passages
- f) In Pelton wheels at the nozzles and spear valve etc.

The Cavitation can be avoided by the following measures

- i) Careful streamlined design of the flow passage of the runner as well as draft tube.
- ii) Average pressure at runner outlet is kept reasonably above the vapour pressure limit.
- iii) Used resistant to Cavitation metal e.g. stainless steel are used than plain steel or iron.
- iv) By periodic inspection of the turbine.
- v) By regularly spot welding in the eroded portions.

Setting height of turbine from tail water and with the cavitation consideration:

We have, Thoma cavitation parameter defined as, $\sigma_{Critical} = \frac{H_{atm} - H_V - H_s}{H}$

- Where, H_{atm} = atmospheric pressure at the installation site.
- H_{Vapour} = Vapour pressure H = head on the turbine
- H_s= height of setting of the turbine over the tail race level.
- **If $\sigma \geq \sigma_{Critical}$** for a given reaction thus for cavitation free running. Take $\sigma = \sigma_{Critical} = \frac{H_{atm} - H_V - H_s}{H}$
- If $\sigma < \sigma_{Critical}$ cavitation occurs.
- **Typical values of σ_C for reaction turbine given as below**

i) For Francis turbine $\sigma_C = 0.0432\left(\frac{N_S}{100}\right)^2$

ii) For Propeller turbine $(\sigma_C) = \left[0.28 + 0.0024\left(\frac{N_S}{100}\right)^3\right]$

iii) For Kaplan turbine $(\sigma_C) = 1.1\left[0.28 + 0.0024\left(\frac{N_S}{100}\right)^3\right]$

Where N_S = Specific speed (In SI Unit)

$(N_S)_{metric} = 1.157\ (N_S)_{SI\ Unit}$

A proposed hydropower development having net head of 150m and design discharge of 25m^3/s is going to use Francis turbine. Take efficiency is 81%. Find the specific speed, turbine diameter and elevation of turbine with respect to the water surface in tailrace. [2070 Chaitra]

Solution: Q=25m^3/s, H=150m, η=81%=0.81

- For Francis turbine , Specifec speed $(N_S) = \frac{2400}{\sqrt{H}} = \frac{2400}{\sqrt{150}} = 195.96$ rpm
- Power (P) = $\eta\Upsilon QH = 0.81 * 9.81 * 25 * 150 = 29797.875$ Kwatt $= \frac{29797.875}{0.746} = \mathbf{39943.53\ HP}$
- Also Specific Speed $(N_S) = \frac{N\sqrt{P}}{H^{\frac{5}{4}}}$
- $\therefore N = \frac{N_S H^{\frac{5}{4}}}{\sqrt{P}} = \frac{195.96*150^{\frac{5}{4}}}{\sqrt{39943.53}} = 514.704$ rpm
- No of Pole $(N_P) = \frac{120f}{N} = \frac{120*50}{514.704}$
- $= 11.66 \cong \mathbf{12}$ {Assume Frequency = 50 Hz}
- Corrected Synchoronus Speed(N) $= \frac{120*50}{12} = 500$ rpm
- Corrected Specific Speed$(N_S) = \frac{N\sqrt{P}}{H^{\frac{5}{4}}}$
- $= \frac{500\sqrt{39943.53}}{150^{1.25}} = 190.36$ rpm
- For Francis turbine $\emptyset = 0.0197 N_S^{2/3} + 0.0275 = 0.0197(190.36)^{2/3} + 0.0275 = \mathbf{0.679j}$

- $\therefore$ Diameter of penstock pipe(D) $= \frac{84.6\emptyset\sqrt{H}}{N}$
- $= \frac{84.6*0.679\sqrt{150}}{500}$
- $= 1.4m$
- For Setting of turbine, If Cavitation is not occur $\sigma \geq \sigma_C$
- For Francis turbine, $\sigma_C = 0.0432\left(\frac{N_S}{100}\right)^2$
- Where N_S in SI unit $= \frac{(N_S)_{Metric}}{1.157} = \frac{190.36}{1.157} = \mathbf{164.53\ rpm}$
- $\therefore \sigma_C = 0.0432\left(\frac{N_S}{100}\right)^2$
- $= 0.0432\left(\frac{164.53}{100}\right)^2$
- $= 0.117$
- $\sigma_{Critical} = \frac{H_{atm} - H_{Vapour} - H_s}{H}$
- $H_s = (H_{atm} - H_{Vapour}) - (\sigma_C * H) = 10 - (0.117 * 150)$
- $= \mathbf{-7.55m}$
- Hence the location of the turbine is 7.55m below the tail water level. (If H_S is positive, setting of turbine is above the tail water level.)

Q. N. In a hydropower plant, the Kaplan turbine has the following data,

Operating head = 28m, output power at this head = 153KW, Discharge = 600m^3/s, Speed = 68rpm, Runner tip-to – tip diameter (D) = 10m, Hub diameter (D_H) = 4.5m, Number of blades = 6. Calculate the speed ratio, flow ratio, the overall efficiency and the maximum suction draft head. Assume Ha-H_v = 10m.

Solution: Q=615m^3/s, H=150m, Head (H) = 28m, Power (P) = 153 MW, D = 10m, D_n = 4.5m, No. of blades = 6, Speed (N) = 68 rpm

- Power (P) = $\eta\Upsilon QH$
- $\eta = \frac{P}{\gamma QH} = \frac{153*10^3}{9.81*615*28}$
- $= \mathbf{92.83\%}$
- Discharge(Q) = $V_f * \pi\left(\frac{D^2-D_n^2}{4}\right)$
- or, $V_f = \frac{615}{\pi\left(\frac{10^2-4.5^2}{4}\right)}$
- $= \mathbf{9.819\ m/s}$
- Flow ratio = $\frac{V_f}{\sqrt{2gH}} = \frac{9.819}{\sqrt{2*9.81*28}}$
- $= \mathbf{0.42}$
- Tangential Velocity($\overline{V}$) = $\frac{\pi DN}{60} = \frac{\pi*10*60}{60}$
- $= \mathbf{35.605\ m/s}$

- Speed ratio$(\emptyset) = \frac{\bar{V}}{V} = \frac{35.605}{\sqrt{2*9.81*28}}$
- $= 1.519$
- **<u>For Setting of turbine</u>**
- For Kaplna Turbine$(\sigma_C) = 1.1\left[0.28 + 0.0024\left(\frac{N_S}{100}\right)^3\right]$
- Specific Speed $(N_S) = \frac{N\sqrt{P}}{H^{\frac{5}{4}}} = \frac{68\sqrt{\frac{153*10^3}{0.746}}}{28^{1.25}}$
- $= 478.120$ rpm(metric) $= 413.214$ rpm (SI)
- $(\sigma_C) = 1.1\left[0.28 + 0.0024\left(\frac{413.241}{100}\right)^3\right]$
- $= \mathbf{0.494}$
- **For No Cavitation $\boldsymbol{\sigma \geq \sigma_C}$**
- $\sigma_{Critical} = \frac{H_{atm} - H_{Vapour} - H_s}{H}$
- From question $H_a - H_V = 10$m
- $H_S = (H_a - H_V) - (\sigma_C * H)$
- $= 10 - (0.494 * 28) = \mathbf{-3.832m}$
- ∴ Location of turbine is 3.832m below the tail water level

Q.N. A Francis turbine works under a head of 25m and produces 11760 KW while running at 120 rpm. The turbine has been installed at a station where atmospheric pressure is 10m of water and vapor pressure is 0.20 m of water. Calculate the maximum height of the straight draft tube for a turbine. [2076 Ashwin]

- **Solution:**

Given P = 11760 Kw, H = 25 m, Turbine Speed (N) = 120 rpm, Atmospheric pressure (H_a) = 10m, vapor pressure (H_v) = 0.20 m

- **We know,** Specific Speed $(N_S) = \frac{N\sqrt{P}}{H^{\frac{5}{4}}}$
- $= \frac{120\sqrt{11760}}{25^{1.25}}$ = **232.788 rpm**
- $\sigma_C = 0.0432\left(\frac{N_S}{100}\right)^2$
- $= 0.0432\left(\frac{232.788}{100}\right)^2$ = **0.234m**

Also, $\sigma_{Critical} = \frac{H_{atm} - H_{Vapour} - H_s}{H}$

- $H_S = (H_a - H_V) - (\sigma_C * H)$
- $= (10 - 0.2) - (0.234 * 25)$
- **= 3.96 m**
- ∴ The maximum height of the straight draft tube is **3.96 m.**

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Hydropower Engineering BCE IV/I

Er. Tek Bahadur Katuwal

- **8. Power house [2 hours]**

  8.1 Power house types, general arrangement, dimension of powerhouse

 # Introduction:

- The powerhouse is a complex multi-storeyed structure housing essential power generating equipment such as turbine, generator, switchboard, low tension switching equipment, bus bar and office etc for generating hydroelectric power.
- It consists of separate or combined generator and turbine rooms, erection bay and service bay.
- It also may have substructure, intermediate structure and superstructure dictated by requirements of available space for different equipment to generate electrical power in site location.
- It may be surface, semi underground and underground dictated by availability of topography, site condition and economic considerations for construction of the power house structure.
- The transformers and high tension switching equipment are generally kept outsides but near the powerhouse.
- In design consideration of power house, it should be simple, efficient on functional basis of construction, operation and maintenance and harmonizing with surrounding with aesthetic point of view.
- It should be structurally stable for reasons of durability and economy.
- The shape and size of the powerhouse are fixed on the consideration of the arrangement of the mechanical, electrical and hydraulic requirements.

Power house structure

Powerhouse are divided into three parts in consideration of the structurally design such as sub structure, intermediate structure and super structure.

1) Substructure:

- Usually, substructure provides rooms including from foundation to the turbine level.
- It consist space for water passage as well as galleries for mechanical and electrical equipment and its services and access to water exit draft tube.
- The substructures are generally used in reaction turbine as a basement floor in it.

2) Intermediate structure:

- Usually, the intermediate structure is considered in between top of the draft tube and the generator floor.
- It is also called turbine floor.
- In this structure housing for turbine, generator, gate valves and governor servo motor system.

3) Super structure:

- Super structure is a part of the power house structure lying above the generator floor (turbine floor).
- It includes housing for control rooms, engineer's room, service bay, stores, space for overhead travelling crane, auxiliary equipment for ventilation and cooling etc.

 # Type of Power House: Basically there are three types of power house

- - Surface power house - Semi- underground structure - Underground structure

1) Surface power house:

- It is best suited when the power house is far away from flood plane. It is cost effective.
- The surface power pause generally here less space restrictions than other type of power house.
- In this type of powerhouse the foundation of the power station should be sound rock otherwise pile or mat fonder have to be used. The surface powerhouse are planed with aesthetic point of view.

2) Semi Underground Structure House: In semi-underground power house, mainly super structure are located over the surface and part of structure situated below the ground level.

- Mainly, draft tube, tail water, drainage galleries etc. are situated on the underground and some units such as generator may be located on the surface.

3) Underground Power House: Under certain topographic conditions the power plant is situated in narrow canyons with no convenient site for a conventional type of power house. *This may be located underground*.

- When a gorge or a valley forms the site of a hydro electric development an underground power station may be considered.
- This type of power house is more suited when there are frequent earth tremors, other physical surface hazards like rock slides snow avalanches etc.
- The basic requirement for the underground structure is the availability of good sound rock at the desired location and depth.
- It is more safer during war, time, earthquake condition and physical hazards of the topography.
- Any particular layout of underground power house depends largely on the positions of head and tail ponds, control values, turbine, generator, transformers, control room, access shaft and ventilation staff and it also depends on the topography geology and read to be required.

Dimensioning of the Power House:

- Power house is one of the major components of the hydropower project.
- It is used to house the electro-mechanical components mainly the site for switchgear, control room, engineer's room, reception rooms, operator accommodation, turbine, generation draft tube, and other miscellaneous equipment and machines are provided in it.
- Power house size mainly depends on the discharge, head, type of turbine and generation, number of units and the general arrangement in the power house.
- **Usually the power house contains:**

1) Machine Hall or Unit Bay

- It is a room in which the generating sets are usually arranged in a single line; the orientation at which will be determined according to the arrangement at the intake or penstock and of the Tailrace.
- The dimension of machine hall depends upon the dimensions of the unit scroll case and generator space required for auxiliaries.

2) Length of Unit Bay

- The length of machine bay depends upon the number of units, distance between them and size of the machines.
- Usually the length of one unite is fixed equal to the 4.5 D to 5D where D is the diameter of the turbine and 2.5 m is the extra clearance provided.
- The center to center distance between the units may be equal to (5 D + 2.5) m.
- However, for higher specific speed turbine the center to center distance may be reduced to (4 D + 2.5) m. knowing the total number of units the total length of the machine bay can computed.

Dimensioning of the Power House:…..

3) Width of the Machine Bay

- Width of the power house is usually taken as width of one unit and extra gangway width form the wall (approximately equal to 2.5 m) as the passage for the movement.
- Excessive width may cause the increase in the size of the girder of the crane, supporting beam, column and the roof covering provision.
- The width of the machine is equal to the (D + 2.5), where D is the diameter of the scroll case of the turbine.

4) Height:

- Height of power house is fixed by the dimensions of power turbine block and its super structure, Height of the lower turbine black from the foundation to the floor of the machine hall is to be determined by the thickness of foundation plate, dimension of the turbine.
- The height of the power house should be sufficient for the installation of turbine, generation and shaft gear mechanism.
- There should be sufficient space for removal and overhaul of any of the components without disturbing other components.
- The height of the powerhouse depends upon the height clearance from the ground to the lifting object through the crane, maximum size of the plant machinery (Turbine or rotor of the generator), depth of the girder and crane system etc.

 # Dimensioning of the Power House:…..

5) Erection or Loading Bay

- The erection bay or service bay is a space where the heavy vehicles can be loaded and unloaded place for the dismantled parts of the machine and space for the assembling of the equipment.
- The size of the erection bay should be sufficient to receive the large parts.
- The service bay floor should be having a width at least equal to the distance between the machine center lines.
- Generally the space of the service bay may provided equal to the 1 to 1.5 times of the other bays.

6) Control Bay:

- The control bay is the house main control and other equipment.
- This bay be situated the either upstream of the machine bay or on the extended portion of the draft tube or one side of the power house.
- Usually a service bay and workshop annexes are located on one end of the machine bay and control bays are located on either side of the erection bay.

7) Miscellanies equipment bay

- It is essential to equip a power house with a crane to lift and more equipment for installation and maintenance purposes.
- Travelling crane, spanning the width of the building and capable of travelling its entire length are normally used.
- The crane rail elevation depends on the maximum clearance required when the crane is in operation which determines the overall height of the superstructure.

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

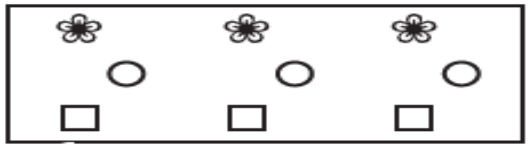

Fig. (i) Composite Arrangement

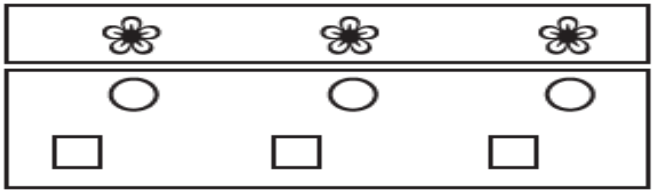

Fig. (ii) Composite Arrangement

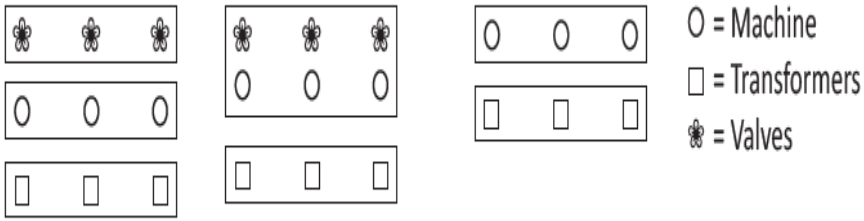

Fig (i) Parallel Hall Layout

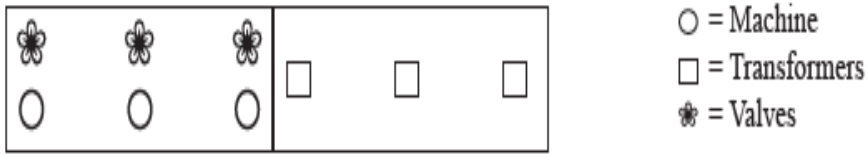

Fig.: (ii) Linear arrangement

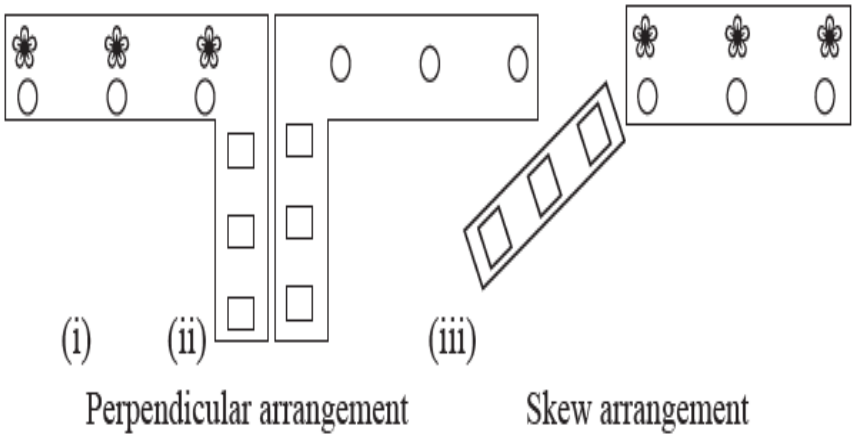

Fig Perpendicular and Skew Arrangement

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