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Notes of Hydropower Engineering

Prepared by

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017 BATCH

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NAME: Sumit Paudel STD.: ______ SEC.: CVL 1 ROLL NO.: 017-022 SUB.: Hydropower engineering

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Different Investigations in different planning stages

	Topographical	Geotechnical/Geological	Seismological	Hydrological	Sedimentological	Construction Material
Reconnaissance	Topo map study Aerial photograph River L-section X-section at dam site	Geological map Aerial photos Sieve Analysis Rock Orientation	Tectonic map study Historical EQ > 4 Rector scale	Mean Monthly flows Discharge measurement in dry periods peak flood marks, GLOF	Not needed	Not needed
Prefeasibility	Establish BMs Topographical Survey X-section at u/s & d/s HFLs in section	Atterberg limits Sp. gravity test Crushing test LA Test	Fault characteristic Seismic Zoning Seismic Hazards Analysis	Discharge @ intake Long term data High flow, low flow Flood Frequency Analysis Rating Curves Pontentiality of GLOF (if)	Determine sediment load in river	Identify borrow area & quarry sites.
Feasibility	Survey of access roads Additional Survey Walkover survey of transmission routes.	Detailed geological & geomorphological map	Critical review of all previous data	PMP & PMF calculation Water quality Analysis	Sampling sediment in wet season Sediment tests Daily Sediment Load	Laboratory Analysis of the construction materials

GLOF → Glacial Lake Outburst Flood
PMP → probable Maximum Precipitation
PMF → Probable Maximum Flood.

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Hydropower Engineering — Sumit Paudel

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Course Objective:-

→ After completing this course, student will be able to:
(i) Describe the hydropower development & oppertunities in Nepal.
(ii) Take part in study selection, selection & planning of hydropower projects
(iii) Design headworks, water conveyance structures, spillways & energy dissipaters of hydropower plant &
(iv) Select & describe different electro-mechanical & hydromechanical components of hydropower plant.

Course Contents:-

→ Introduction to hydropower development
→ Planning & investigation of hydropower projects
→ Power & Energy potential study
→ Storage type of hydropower projects.
→ Run of River types of Hydropower projects
→ Water conveyance structures
→ Spillways & Energy Dissipaters
→ Hydro-mechanical & Electro-mechanical Equipments.
→ Powerhouse planning
→ Micro Hydropower plant.

Laboratories:-

→ Performance characteristics of a pelton/Francis/kaplan turbine
→ Working principle & characteristics of centrifugal / reciprocating pump.

Chapter 1,

Introduction to Hydropower Development

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Outline:

→ Sources of energy & Importance of Hydropower
→ Hydropower Development in Nepal: Historical Background, Present Development, Challenges & opportunities.
→ Hydropower potential in Nepal: Gross, technical & economical
→ Introduction to some large hydropower plants in the world
→ A brief introduction to government policy & major institutions related to hydropower development in Nepal. Hydropower development policy, Ministry of Energy, WECS, Electricity tariff fixation commission, DOED, NEA, IPPAN & NMHDA (private sector)

* Sources of Energy:

→ Sun is ultimate source of energy
→ Source of Energy are basically classified into two types

(i) Conventional Source of Energy

→ Source of energy that have been in use from the time immemorial are called conventional sources.
→ Eg: water, firewood, nuclear power, hydropower, natural gas etc
→ They are exhaustible except hydro energy
→ Cause pollution
→ Generation & use involves high expenditure
→ Expensive to store, maintain & transmit

(ii) Non-Conventional Source of Energy.

→ Those source of energy that have been identified in the recent past & are still in the process of identification are called non-conventional sources of energy.

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→ Eg: Solar energy, wind energy, tidal energy, geothermal energy, biogas etc.
→ They are inexhaustible & generally pollution free
→ Low expenditure
→ less expensive due to local use & easy maintenance.

* Renewable Sources of Energy:-

→ Sources of energy that are available in abundant & are replenished naturally over short period of time.
→ Major five renewable sources:- Solar, wind, water, biomass & geothermal

* Non-Renewable Sources of Energy:

→ The sources of energy that are available in limited quantity as it takes a very long time to be replenished naturally.
→ Eg: Coal, natural gas, petroleum etc.

* Introduction to Hydropower:

→ The ~~energy~~ power that can be generated in the form of mechanical power or electric power on account of energy head associated with flowing or still water.
→ Energy posses by water is generally expressed as head

$$E = \underbrace{P/\gamma}_{\text{pressure head}} + \underbrace{z}_{\text{potential head}} + \underbrace{v^2/2g}_{\text{kinetic head}}$$

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→ Velocity head is very small & in open channel flow, pressure is atmospheric. So, we can say that the major portion of energy possessed by water is due to elevation it has acquired.

→ We convert hydraulic energy into electrical energy because electrical energy is high grade energy & can be converted to many other forms of energy with high efficiency.

* Importance of Hydropower

→ Environmental friendly sustainable development
→ Important for maintaining base load in electricity transmission compared to thermal & wind load
→ Create employment oppertunities to people.
→ Important for socio-economic development
→ Can foster the nations economy
→ Hydropower production is flexible, can be operated in small scale to large scale.

* Advantages & Disadvantages of Hydropower:-

Advantages	Disadvantages
① Reliable & renewable / clean source of energy.	① High initial investment
② Dispatchable, can be made available as reqd by demand	② High payback period
③ Long life span	③ precipitation dependent
④ Low operation & maintenance cost.	④ Large scale hydropower causes high flood & resettling of people that increases project cost
⑤ Leaves water available for other uses.	⑤ Often requires foreign contractors & funding.

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* History of Hydropower Development:

→ In Nepal, the first hydropower project:- pharping (500kW) in 1 during primeminister Chandra Shamsher Rana's time.
→ Second hydropower plant → Sundarijal (900 kW) in 1936
→ Morang Hydropower in 1939 (677kW), destroy by landslide in 196
→ 2.4 MW panauti Hydropower plant in 1965
→ Sunkoshi hydropower (10 MW) in 1973 AD with aid of china
→ Kulekhani Hydropower (I & II), in 1982, only project offering seasonal water storage in Nepal.
→ Seti power station (1.5 MW) in 1985 (china aid)
→ 60 MW Marsyangdi power station in 1989
→ Biggest hydropower of Nepal, kaligandaki A (144MW) in 20
→ Upper Tamakoshi is the project of national pride (456M recently completed & already started its production.
→ Karnali-chisapani, 10800 MW is the biggest proposed hydropower of Nepal.
→ Budi-Gandaki Hydropower project, prefeasibility study was prepared in 1984, (1200 MW capacity with FSL 540m

* Top 6 largest hydropower stations of Nepal

Upper tamakoshi - 456MW - ~~Gorkha~~ Dolakha
kaligandaki A - 144MW - Mirmi, Syangja
Kulekhani (I & II) - (60+32 = 92 MW) - Makawanpur
Middle Marsyangdi - 70MW - Lamjung
Marsyangdi - 69MW - Tanahun
Khimti - 60MW - Dolakha & Ramechhap

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* Hydropower plants under construction

1) Tanahu – 140MW
2) Rasuwagadhi – 111MW
3) Rahughat – 32MW
4) Chamelia – 30MW
5) Sanjen – 42.5MW

* Identified Hydropower plants in Nepal

1) Karnali Chisapani – 10800MW
2) Pancheswor – 6480MW
3) Budi-Gandaki – 1200MW
4) West Seti – 750MW
5) Kaligandaki II – 660MW

* Classification of Hydropower projects:-

(a) Based on head
1) High Head > 300m (pelton turbine is used)
2) Medium Head: (50m – 300m) (Francis turbine is used)
3) Low Head < 50m (Kaplan / Francis turbine is used.

(b) Based on installed Capacity:-
1) Pico Hydropower (< 10 kW)
2) Micro Hydropower (10 kW – 100 kW)
3) Mini Hydropower (100 kW – 1 MW)
4) Small Hydropower (1 MW – 25 MW)
5) Medium Hydropower (25 MW – 100 MW)
6) Large hydropower (> 100 MW)

(C) Based on operation

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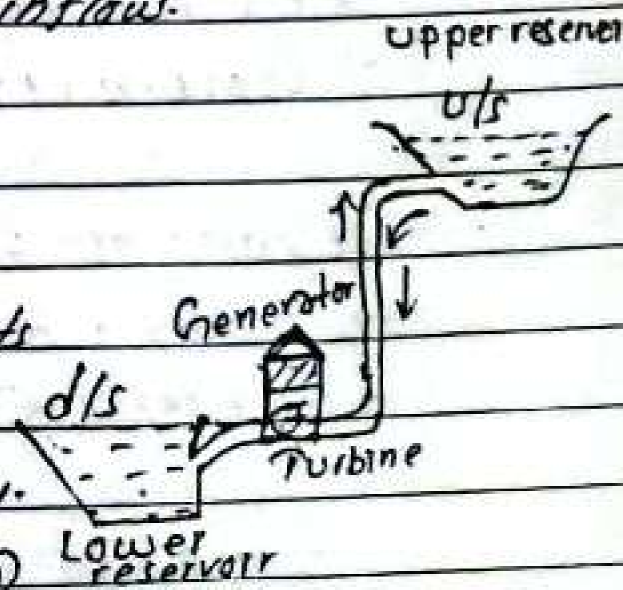
1) Isolated – not connected in grid (small hydro generally)
2) Grid Connected – (Connected to national grid)
3) Base load plant – (Constructed to fulfill min^m demand)
4) Peak load plant – Constructed for peak time.

(d) Based on storage Capacity:
1) ROR (Run off River) plants – donot regulate the hydrograph of the river by one or more seasons. Utilize minimum flow in a river having no appreciable pondage on its u/s.
Eg: Khimti, Bhotekoshi etc

2) PROR (Peaking Run off River plants)
ROR plants with pondage that can regulate weekly/daily hydrograph to run the plant under full capacity.
Eg: Marsyangdi, Kaligandaki A, Sunkoshi

3) Storage plants
→ Those plants which regulate the hydrograph of river by one or more seasons are termed as storage plants.
→ Include a dam/reservoir to store water & release when needed during dry seasons Eg: Kulekhani
→ reduces dependence on variability of inflow.

4) Pumped storage plant:
→ plants having water reservoir at u/s as well as d/s of powerhouse, which collects water in u/s reservoir by pumping water from d/s reservoir using cheaper energy.



→ Eg: Sunkoshi II (1110 MW) & Sunkoshi III (536 MW) (planned to develop)
Rupatal-Begnastal pumped storage project (100 – 300 MW)

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Principles of Hydropower Development:

→ Resettlement, submergence of valuable land etc. will lead to increase in cost. Hence, projects with less social & environmental impacts should be selected.
→ River reach with competent geo-technical conditions should be selected so that huge amount of cost of civil works can be saved.
→ The basin with lesser sediments load should be selected.
→ Hydropower projects should be developed from the upstream so that the same head-works components can be shared downstream.
→ Maximum head should be utilized because it is available 100% of time.
→ Sound Engineering practices should be applied during design & construction.
→ Projects should be developed in cascade so that structures are shared & capital cost is shared by mutiple stakeholders.

Challenges/Issues of Hydropower Development:-

1) Lack of plans & policies
→ No clear plan & policies to exploit the potential.
→ Rules & regulation donot seem supportive.
→ Goverment processes are time consuming in nature.

2) Economic Issues:-
→ Benefit is major concern.
→ Benefit Cost ratio > 1

3) Financial Issues. → It includes issue of currency, loan repatriation, legislation on tax, customs, revenues, issue of market, interest rate, rate of inflation, mode of payment etc.

4) Social Issue:-
→ Large number of people might have to be displaced
→ Affect pastureland & forest.
→ Indigenous communities, vulnerable social groups, public health, education transition, cultural heritages, safety etc should be kept in attention.

5) Environmental Issue:
→ It incorporates habitats, biodiversity, invasive species, water quality, erosion, reservoir sedimentation etc.

6) Technical Issue:
→ Includes infrastructure safety, asset reliability & project efficie[ncy]

7) Goverment Issue: For any hydropower project, institution is necessary for its system management & project control. It is necessary to decide whether the project is isolated, privatized, multi-proposed & also construction & maintenance authority should be clear. Lack of coordination betn govn. bod[ies]

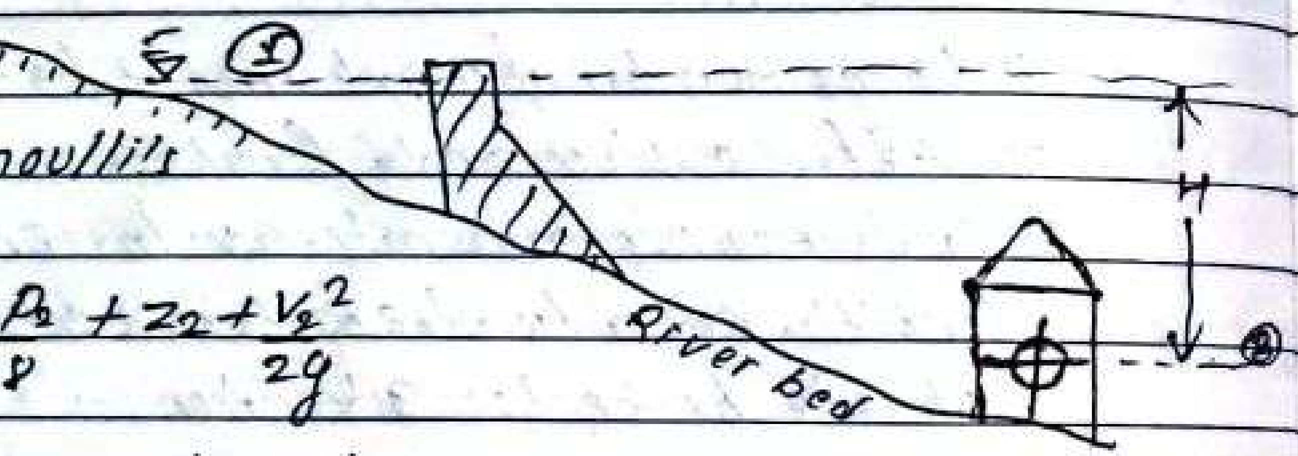
8) International Issues of use of water:
→ Change in international rules & regulations

9) Data availability: etc.

* Hydropower potential in Nepal:-

Estimation of hydropower potential:

Let, H = diff^n betn reservoir pond level & turbine axis level. (Gross head)



Applying Bernoulli's eqn at 1 & 2,

$$\frac{P_1}{\gamma} + z_1 + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + z_2 + \frac{V_2^2}{2g}$$

Since, pressure is atmospheric, $P_1 = P_2 = 0$

Assuming, $V_1 = 0$,

$$V_2^2/2g = z_1 - z_2 = H \quad \therefore V_2 = \sqrt{2gH}$$

Kinetic energy of water is,

$$K.E = \frac{1}{2} m V_2^2 = \frac{1}{2} m \times 2gH = mgH$$

So, power developed is, $P = \frac{\text{Energy}}{\text{time}} = \frac{mgH}{t} = \frac{\rho \times V \times gH}{t}$

$$P = \gamma Q H$$

Since, some loss is always associated with the flow & production, $P = \eta \gamma Q H$

where, η = overall efficiency

(a) Gross potential:- (83000 MW → Nepal)

→ Total power that can be theoretically generated.

→ A river basin is divided into several cascades. Based on head & hydrograph of a particular cascade, the power can be calculated.

$$P = \sum_{i=1}^{n} \frac{\eta \gamma Q H}{1000} \ (kW)$$

n = number of cascade.

(b) Technical potential:- (44,000 MW → Nepal)

→ Due to unfavorable geology, topography, climatic condition, accessibility etc. all the theoretical power cannot be obtained.

→ Power which can be technically produced is techinal potential.

(c) Economical potential (42,000 MW → Nepal)

→ A project is economically feasible if benefit cost ratio >=

→ A project which is technically feasible may not be economically feasible.

→ The power which can be economically produced is called economical power potential.

* Introduction to large hydropower plants in the world.

→ Hydropower constitutes 21% of the world's electricity generating capacity.

→ Theoretical potential of worldwide ~~potential~~ hydropower is 2800 GW.

→ Top largest power producing nations

China, Brazil, United state, Canada, India, Japan, Russia, Norway, Turkey, Italy

→ Top hydropower projects of world.

1) Three Gorges (22,500 MW) (Yichang, china)
2) Itaipu (14,000 MW) (Border betn Brazil & Paraguay)
3) Xilodu (13,860 MW) (China)
4) Bela Monte (11,233 MW) (Brazil) [Source: Wikipedia]
5) Guri (10,200 MW) (Venezuela)
6) Tucurui (8370 MW) (Brazil)
7) Grand Coulee (6809 MW) (Washington, US)

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Hydropower development policy, 2058 (2001)

Objectives:-

Hydropower shall be developed to achieve following objective.

1) To generate electricity at low cost by utilizing the water resources available in the country.
2) To extend reliable & qualitative electric service throughout Nepal at a reasonable price.
3) To tie-up electrification with the economic activities
4) To render support for the development of rural economy by extending the rural electrification.
5) To develop hydropower as an exportable commodity.

Policies:- (नीति)

→ Maxm use of hydropower potential of country
→ Develop storage projects as per requirement
→ Encourage concept of Build, Operate, Own & Transfer
→ Attract national & foreign investment in hydropower development.
→ Involve GON with private sector for possibility of irrigation dev.
→ Environmental protection, alternative to biomass & thermal energy
→ Resettled of displaced families
→ Mobilization of internal capital market for investment in power sector.
→ Operate small & micro hydropower at local level.
→ Rural electrification
→ Control unauthorized leakage of electricity.
→ Provide appropriate benefits at local level
→ Made provision to cover risk in hydropower project
→ Encourage for export of electricity.
→ Make electricity tariff fixation rational & transparent.
→ Utilize labour & skills of Nepal in hydropower project implementation.

→ Develop institution to produce skilled manpower & enhance the capability of the persons involved in this sector.
→ Study & research work related to hydropower development.

Provisions for Hydropower Development policy 2001:- (प्रावधान)

→ Environmental provision (EIA Report, 10% water in d/s)
→ Provision concerning Water Rights.
→ Provision for investment in generation, transmission & distbn
→ Provision of special investment for infrastructure development o
Rural electrification. (15 year free on electric energy consumption or that are, 1% of royalty obtain by GoN provide to local level)
→ Provision relating to Transfer of project
→ Provision relating to power purchase (PPA)
→ Provision relating to visa
→ Maxm Utilization of local Resources & Means.
→ Management of Investment Risks
→ Provision on Internal Electricity Market
→ Provision on Export of Electricity.
→ Provision on license (Study & Survey license, generation license, Transmission license & Distribution license.)
→ Provision relating to Fees (Royalty → upto & after 15 years - Annual capacity Royalty per kW & Energy Royalty per kWh)
→ Facilities relating to tax & customs
→ Institutional provision
→ Construction & operation of Hydropower projects by Government of Nepal.

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* Major Institution related to hydropower development in Nepal.

(a) Government bodies

1) Ministry of Energy (MoEn)
→ Make rules, regulation & acts related to power sector development including hydropower development & is responsible for its implementation.

2) Water & Energy Commission Secretariat (WECS)
→ WECS was established by government in 1975 AD with the objective of developing water & energy resource in an integrated & accelerated manner.

3) Electricity Tariff Fixation Commission (ETFC)
→ ETFC, established under the Electricity Act, 1992 is a tariff regulatory body set up to review and approve tariff fixing by NEA & other licensed entities.

4) Department of Electricity Development (DoED)
→ Established in 2050 B.S. The main aim of this institution is to develop & promote electricity sector & to improve financial effectiveness of this sector at national level by attracting private sector investment.

5) Nepal Electricity Authority (NEA)
→ NEA was established in 2042 B.S. The main objective of NEA is to generate, transmit & distribute adequate, reliable & affordable power by planning, constructing, operating & maintaining all generation, transmission & distribution facilities in Nepal's Power System both interconnected & isolated.

(b) Private Sectors:-

1) Independent power producers Association Nepal (IPPAN)
→ Butwal Power Company (BPC)
→ Himal power limited (HPL)
→ Bhotekoshi Power Company (BKPC)
→ Hydro Solutions
→ Sanima Hydropower
→ National Hydropower Company (NHPC)
→ - - - -

2) Nepal MicroHydro Development Association (NMHDA)

(c) International Institutions:-

(a) World bank
(b) Asian development bank (ADB)
(c) JICA (Japan International Cooperation Agency)
(d) US-AID (United states Agency for International Development

Chapter-2

Planning & Investigation of Hydropower Project

Outline:-

→ Hydropower project planning stages: Reconnaissance, Pre-feasibility & feasibility studies.

→ Hydrological data processing: mass curve & flow duration curve (Weibull Method), their characteristics & its uses in hydropower planning engineering.

→ Reservoir planning & regulations: classification, site selection, need of reservoir regulation, life of reservoirs.

→ Environmental study policy based on type & size (IEE/EIA)

→ climate change & ecology: river engineering, social costs, population displacement, change in lifestyle, global worry, clean energy alternatives.

※ Hydropower Project Planning Stages:- Reconnaissance, Pre-feasibility & feasibility Studies:-

→ Every project has its timebound & certain objectives. To achieve this target, proper planning is necessary.

→ Hydropower planning is the initial stage of work of Hydropower development.

→ The planning involves the sequential process which consists of identifying the problems & opportunities, inventory & forecasting conditions, formulating alternative plans & finally selecting the plan.

* Hydropower Development Cycle:-

project planning (3-4 yrs)	Project Implementation (4-6 yrs)	Project operation (40-60 yrs)

* Necessity of Hydropower project planning:-

→ HPP are capital intensive, gestation period is long

→ Involves large number of risks

→ Complicated in nature

→ Each HPP is unique.

→ Involves different parties (govmnt, developer, loan provider, invest…

※ Risks in Hydropower Project:

a) Hydrological Risk → Lack of hydrometeorological data, gauging stations are available more in lower belts - while HPP are being developed in upper reaches, insufficient production penalty to NEA, underestimate of flood may damage hydraulic structure etc.

b) Geological Risk:-

→ Construction of different underground structures, geological formation may not be suitable, alignment may change, project delay, increase in cost.

c) Financial Risk:- Increase in interest rate, increase in dollar rate etc.

d) Environmental/Social Risk:-

High demand & weak support from community, forest issues, high land acquisition cost, unnecessary demands from people of project area

e) Sedimentation Risk:- Nepali rivers are sediment laden. Thats why sedimentation has always been a headache for HPP developers.

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f) political/policy risk:- Change in policy due to unstable government, lack of good governance etc.

Different Stages of planning

* Reconnaissance:-

→ Major objective:-

- To identify the suitable project for the stated purpose & their alternatives.
- To record, screen & rank the project alternatives.

→ Major Steps:-

- Data collection
- Desk studies
- Field work & design
- Estimates & schedules
- Environmental & Social studies
- Economic Assessment
- Report

→ Screening & Ranking of project during Reconnaissance

SN	Item	Point	High	Low
1	Road length	10	>40KM (2pt)	<15KM (10pt)
2	Transmission length	10	>75KM (2pt)	<30KM (10pt)
3	Storage	10	>40% (10pt)	<5% (2pt)
4	Hydrological Risk	10	High (2pt)	low (10pt)
5	Geological Risk	10	High (2pt)	Low (10pt)
6	Environmental Risk	20	High (4pt)	Low (20pt)
7	Sedimentation Risk	10	> 4000 $t/km^2/yr$ (2pt)	< 1000 $t/km^2/yr$ (10pt)
8	Financial Risk	5	> 200MW (2pt)	< 75 MW (10pt)
9	BC Ratio	10	High (10pt)	low (2)
10	Others	5	High (5pt)	Low (2)

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* Pre-feasibility:-

→ Major objectives

- Establish the need & justification for the project
- Formulate the plan for developing the project
- Determine the technical, Economic & Environmental practicability of the project.
- Define the limitation of the project
- Make recommendation for further action.

→ Major steps (same as reconnaissance)

* Feasibility:-

→ This phase includes the detailed study of the project.

→ Major objectives.

- To ascertain the identified project for the implementation.
- To sought the measures for financing the project
- To carryout detail design of the project
- To direct project towards Construction.

→ Major steps (same as reconnaisse) (description will be diff.)

Types of investigations during planning

→ Topographical Survey

→ Geological & geotechnical investigation

→ Seismological Investigation

→ Hydro-meteorological Investigation

→ Sedimentological Investigation

→ Construction material Investigation.

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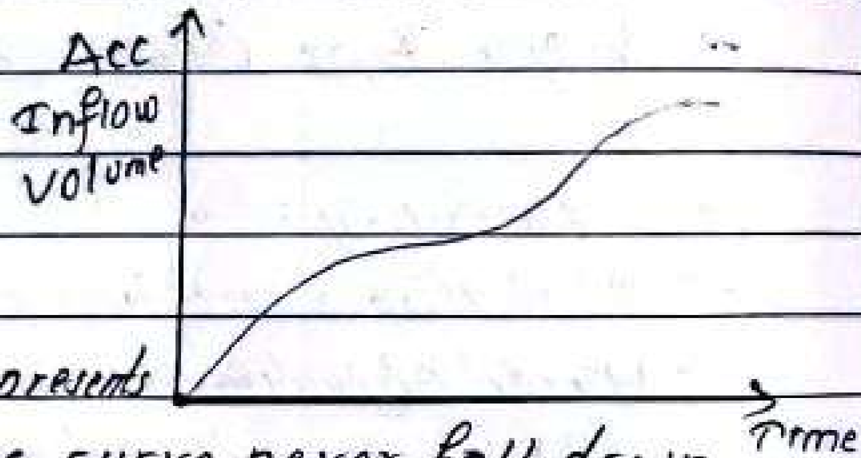
* Mass curve:-

→ ~~Also called ripple curve~~
→ plot of accumulated volume versus time
→ Used to determine storage capacity of reservoir from uniform demand flow.

* Mass Inflow Curve:-

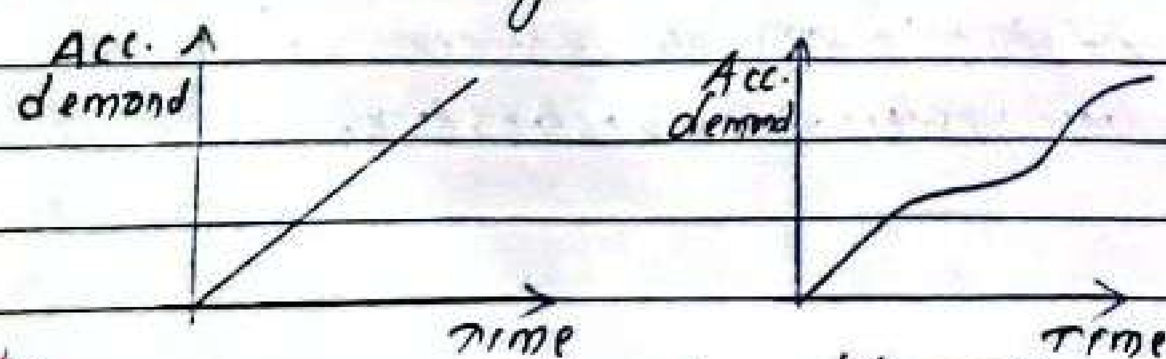
→ Also called ripple curve
→ plot of accumulated inflow volume versus time.
→ It is continuously rising curve
→ The slope at any point on curve represents the inflow rate.



→ If the curve is horizontal, it represents no inflow at that time. Mass curve never fall down
→ If the curve rises sharply, it indicates the high rate of inflow within that period.
→ Relatively convex rise indicate flood while concave depression indicate drought.

* Mass demand Curve:-

→ Mass demand curve is plot of accumulated demand volume versus time. For the constant demand mass curve is straight line while for the variable demand mass curve is increasing non-linear curve.



Constant demand   Variable demand

* Determination of storage Capacity using mass Curve

→ Draw the mass curve of inflow
→ Draw the mass demand curve in same graph.
→ Identify the peak & trough points in the curve.
→ From peak, draw tangent parallel to demand line.
→ From trough, draw tangent line parallel to demand line
→ Calculate the vertical ordinate between these two tangents of adjacent peaks & troughs. Let them be $y_1, y_2, ---- y_n$. The number of these ordinates will be depending upon the time interval provided.
→ The maximum of the ordinate gives the storage capacity of the reservoir that should be provided.

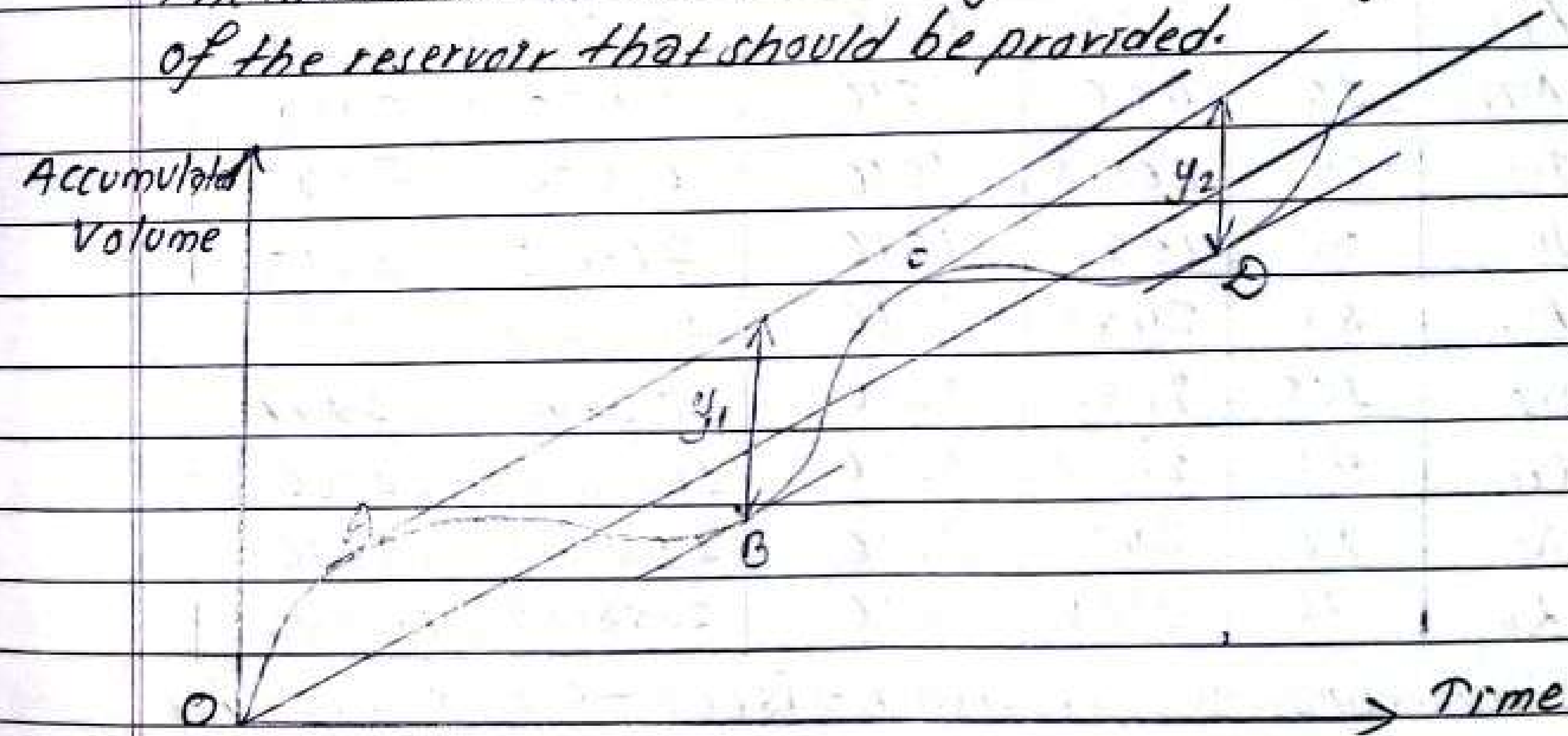
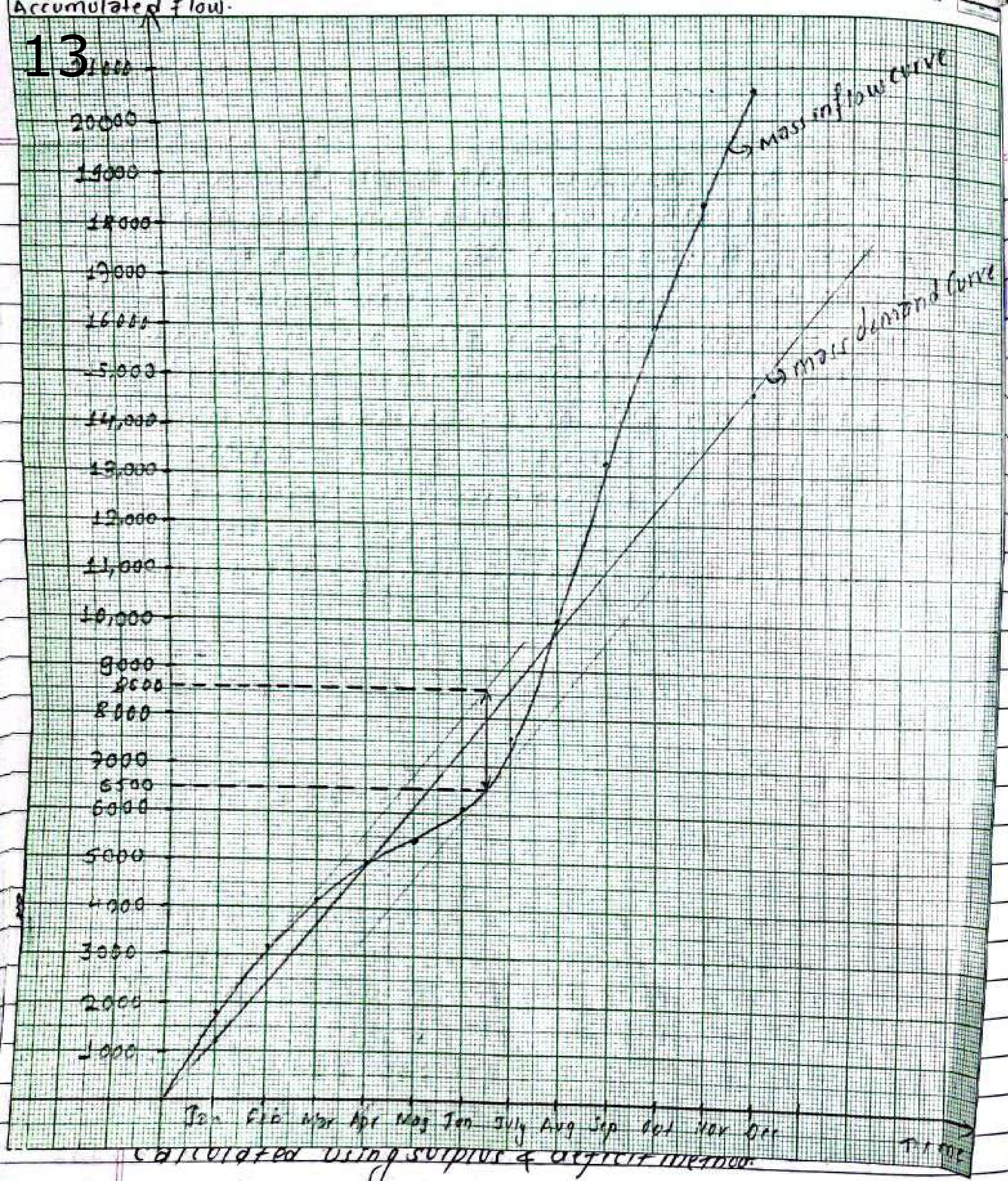


Fig: storage capacity determination

* Determination of storage Capacity for variable demand.

→ For the variable demand, similar process can be adopted but the process became complicated. Hence, the storage can be computed using surplus & deficit method as discussed in water supply Engineering.



calculated using surplus & deficit method.

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Month	Inflow (m3/sec)	No. of days	Inflow volume (cumec-day)	outflow volume (cumec-day)	Surplus (cumec-day)	Deficit (cumec-day)	cum. Deficit (cumec-day)
Jan	60	31	1860	1240	620		–
Feb	45	28	1260	1120	140		–
Mar	35	31	1085	1240		155	155
Apr	25	30	750	1200		450	605
May	15	31	465	1240		775	1380
Jun	22	30	660	1200		540	1920
Jul	50	31	1550	1240	310		–
Aug	80	31	2480	1240	1240		–
Sep	105	30	3150	1200	1950		–
Oct	90	31	2790	1240	1550		–
Nov	80	30	2400	1200	1200		–
Dec	70	31	2170	1240	930		–

Storage capacity of reservoir = 1920 cumec-day

$= 1920 \times 86400 \times 10^{-6} \, Mm^3$

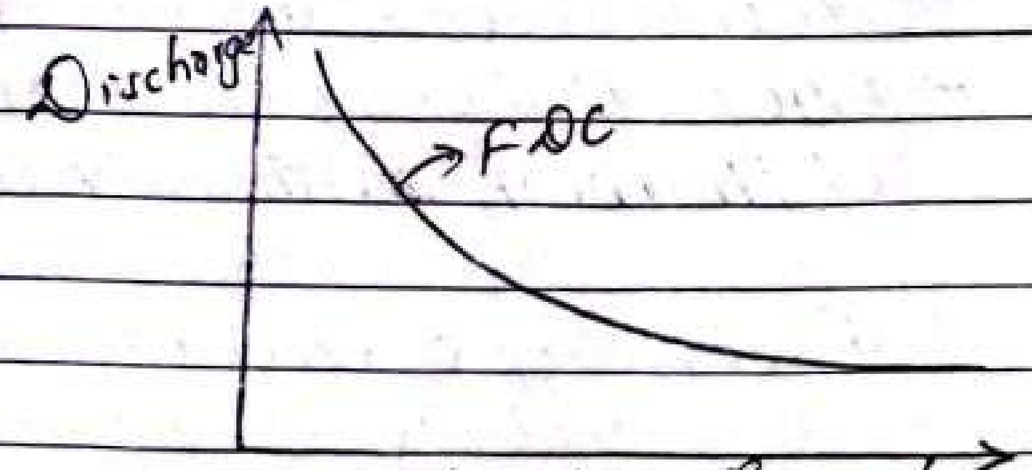
$= 165.88 \, Mm^3$

* Notes:-

Avg. Annual flow ($Q_{avg, annual}$) $= \dfrac{Q_J N_J + Q_F N_F + \cdots + Q_D N_D}{365}$

* Flow Duration Curve:-

→ It is the plot of discharge versus percentage of time exceedance of discharge.



Q. The average monthly flow in a river is given...

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Q (m³/sec)	60	45	35	25	15	22	50	80	105	90	80	70

For a constant demand of 40 cumecs, determine the storage capacity assuming number of days in each month be 30.4

→

Month	Inflow (m3/s)	Inflow volume (cumec-day)	Demand volume (Cumec-day)	Inflow Acc. vol. (Cumec day)	Acc. demand (cumec day)
Jan	60	1824	1216	1824	1216
Feb	45	1368	1216	3192	2432
Mar	35	1064	1216	4256	3648
Apr	25	760	1216	5016	4864
May	15	456	1216	5472	6080
Jun	22	668.8	1216	6140.8	7296
Jul	50	1520	1216	7660.8	8512
Aug	80	2432	1216	10092.8	9728
Sep	105	3192	1216	13284.8	10944
Oct	90	2736	1216	16020.8	12160
Nov	80	2432	1216	18452.8	13376
Dec	70	2128	1216	20580.8	14592

storage capacity of reservoir $= (8600 - 6500)$ cumec-day

$= 2100 \times 86400 \times 10^{-6}\ Mm^3$

$= 181.44\ Mm^3$

→ For above question, the storage capacity can also be calculated using surplus & deficit method.

Month	Inflow (m3/sec)	No. of days	Inflow volume (cumec-day)	outflow volume (cumec-day)	Surplus (cumec-day)	Deficit (cumec-day)	cum. Deficit (cumec-day)
Jan	60	31	1860	1240	620		–
Feb	45	28	1260	1120	140		–
Mar	35	31	1085	1240		155	155
Apr	25	30	750	1200		450	605
May	15	31	465	1240		775	1380
Jun	22	30	660	1200		540	1920
Jul	50	31	1550	1240	310		–
Aug	80	31	2480	1240	1240		–
Sep	105	30	3150	1200	1950		–
Oct	90	31	2790	1240	1550		–
Nov	80	30	2400	1200	1200		–
Dec	70	31	2170	1240	930		–

Storage capacity of reservoir $= 1920$ cumec-day

$= 1920 \times 86400 \times 10^{-6}\ Mm^3$

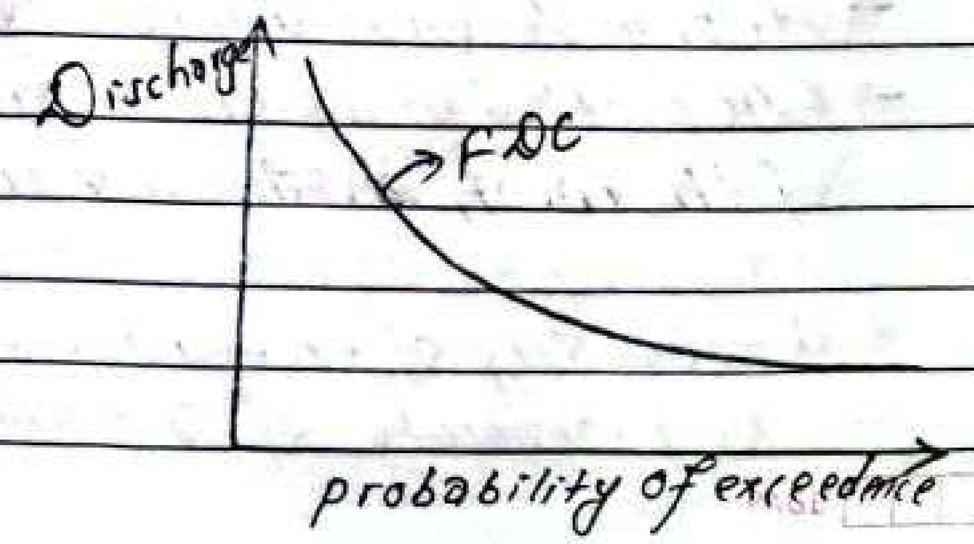
$= 165.88\ Mm^3$

* Notes:-

Avg. Annual flow ($Q_{avg,annual}$) $= \dfrac{Q_J N_J + Q_F N_F + \cdots + Q_D N_D}{365}$

\# Flow Duration Curve:-

→ It is the plot of discharge versus percentage of time exceedance of discharge.



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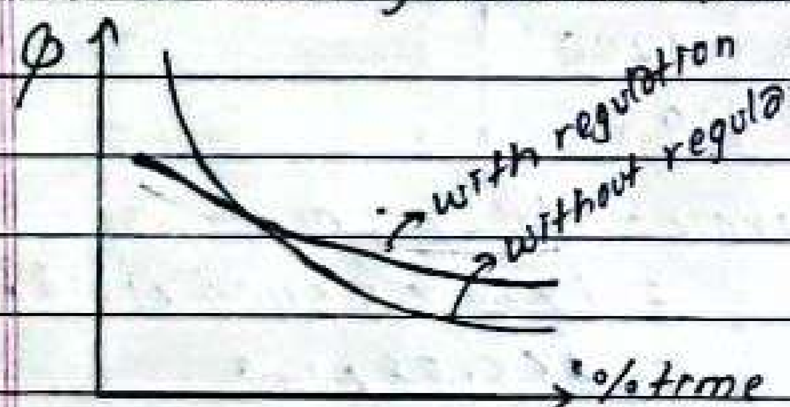
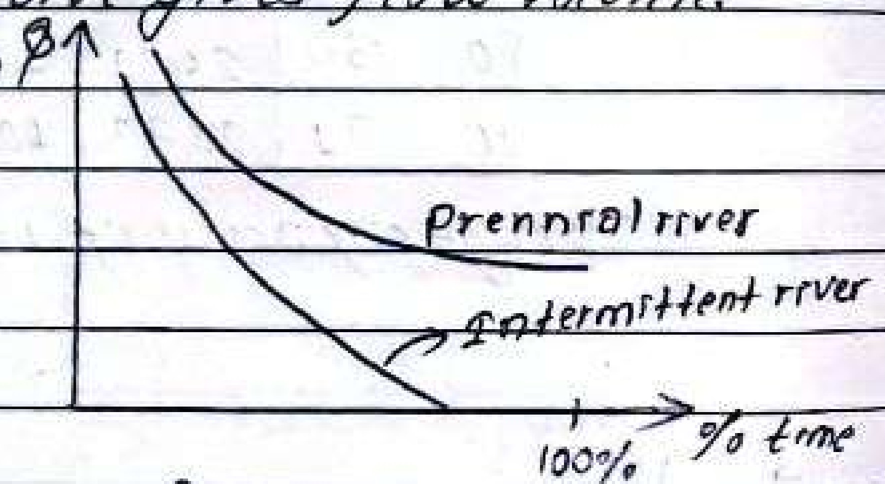
→ The slope of flow duration curve depends upon the interval of the data. The slope of flow duration Curve from daily flow data is steeper than that of average monthly data.
→ Flow duration curve is decreasing curve. With the increase in probability of exceedence, the flow decreases.
→ chronological sequence is disturbed in flow duration curve
→ With the increase in storage, the flow duration curve is flatter.
→ The flow curve of intermittent river ends before probability of exceedance reaches 100%.
→ Area under flow duration curve gives flow volume.



FDC with & without Storage.



FDC for prennial & intermittent river.

* Uses of FDC:-
→ Helps to evaluate the flow expected certain % of time.
→ Useful in planning & design of water resource projects
→ Helps in design of drainage systems & in the flood control studies.
→ FDC plotted in log graph provides the qualitative description of the runoff variability in the stream.

* Probability of exceedance:-
→ The probability of a particular parameter being equalled or exceeded during a certain period is known as probability of exceedance. It is calculated by using empirical formula.

(a) California formula, $P = \frac{m}{N}$

(b) Weibull's formula, $P = \frac{m}{N+1}$

where, m = Rank & N = Total data.

* Power Duration Curve:-
→ It is the plot of power versus percentage of time exceedance of discharge.
→ PDC can be plotted in similar manner once power is calculated using,

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

Here, η, γ & H are constant. So, $P = kQ$, $k = \eta \gamma H$
It means nature of PDC is exactly similar to FDC.

* Firm power/Primary power:-
→ Power corresponding to minm stream flow.
→ Such power can be increased by use of the pondage.

$$\text{Primary power} = \frac{\text{Primary Energy}}{\text{Time}}$$

* Secondary power/Surplus power
→ The power that is available in excess of firm power is called secondary power.

$$\text{secondary power} = \frac{\text{Secondary Energy}}{\text{Time}}$$

* Spill Energy:-

→ The amount of water that is spilled due to limitation of maximum usable flow is called spilled flow. And the energy that could have been produced from the spilled water is called spill energy.

$$\text{Spill power} = \frac{\text{Spill Energy}}{\text{Time}}$$

Since, we generally take the analysis period of one year, the time mentioned above represent the period of year.

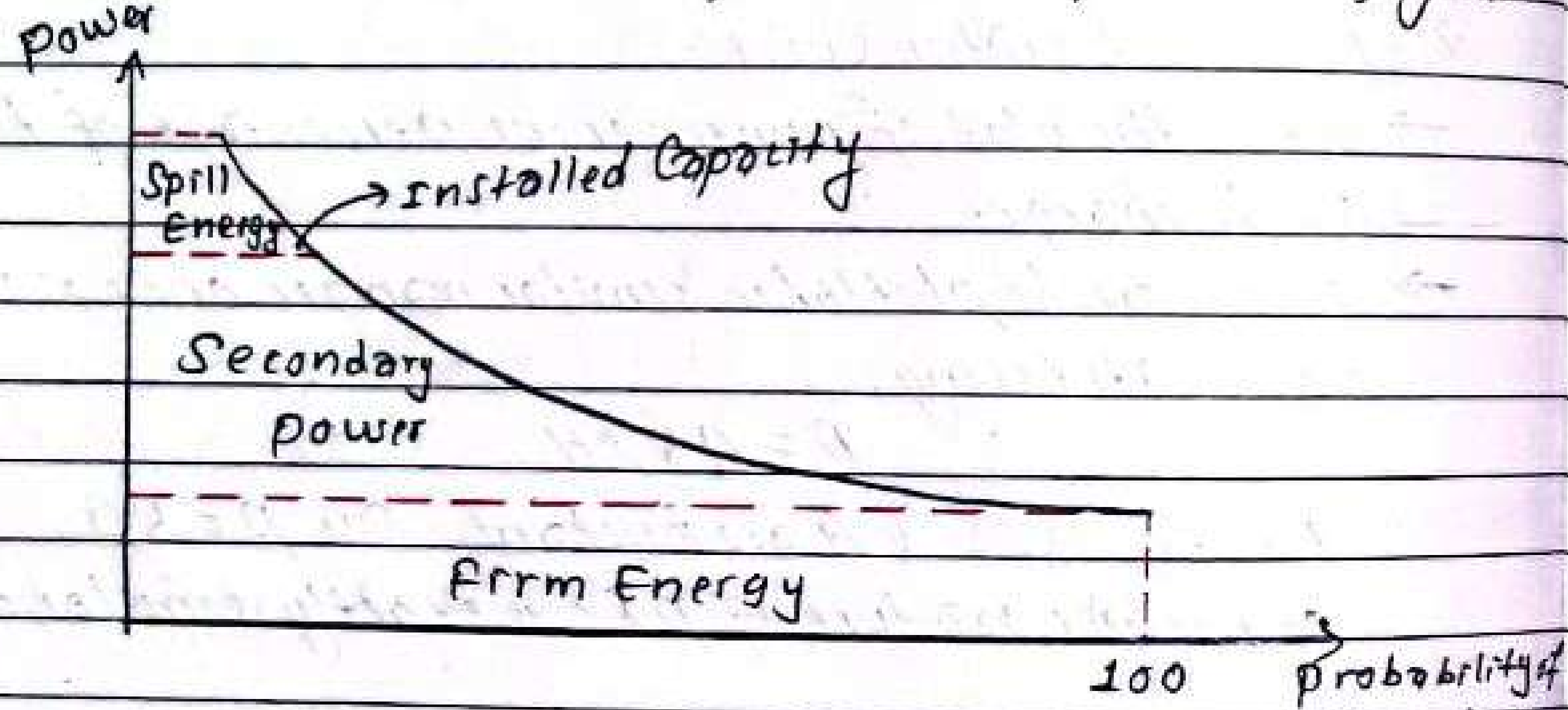


Fig:- power duration curve.

Q. The average monthly flows for a certain river is given below. The net head is 20m & overall efficiency is 90%

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Q (m^3/s)	117	150	203	115	80	118	82	79	58	45	57	152

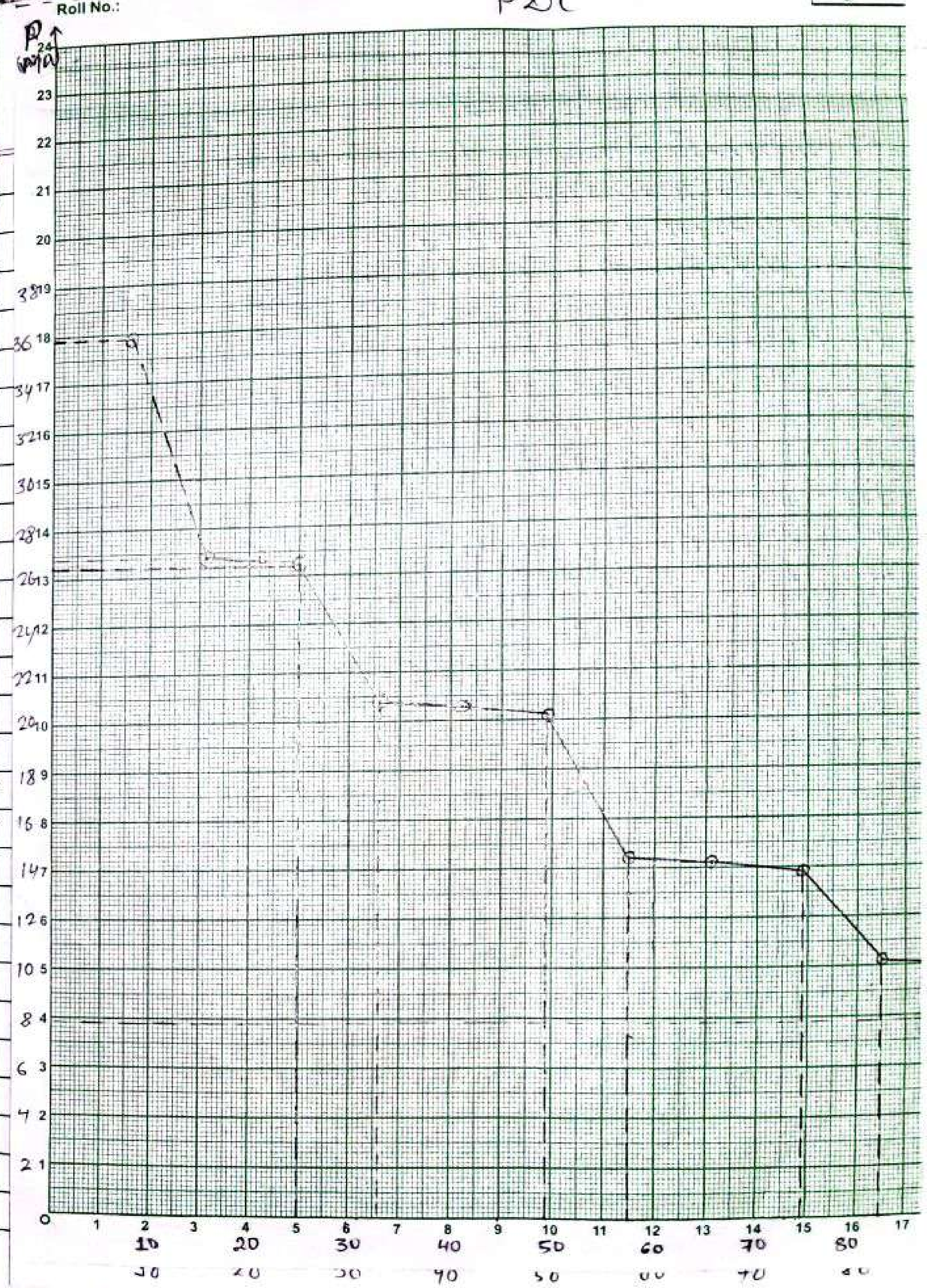
a) Plot FDC & PDC

b) Find Q_{es}

c) What is the probability of exceedance of 100 cumecs?

d) Find P_{es}

e) What is the probability of exceedance of 20 MW?



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* Spill Energy:-

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Since, we generally take the analysis period of one year, the time mentioned above represent the period of year.

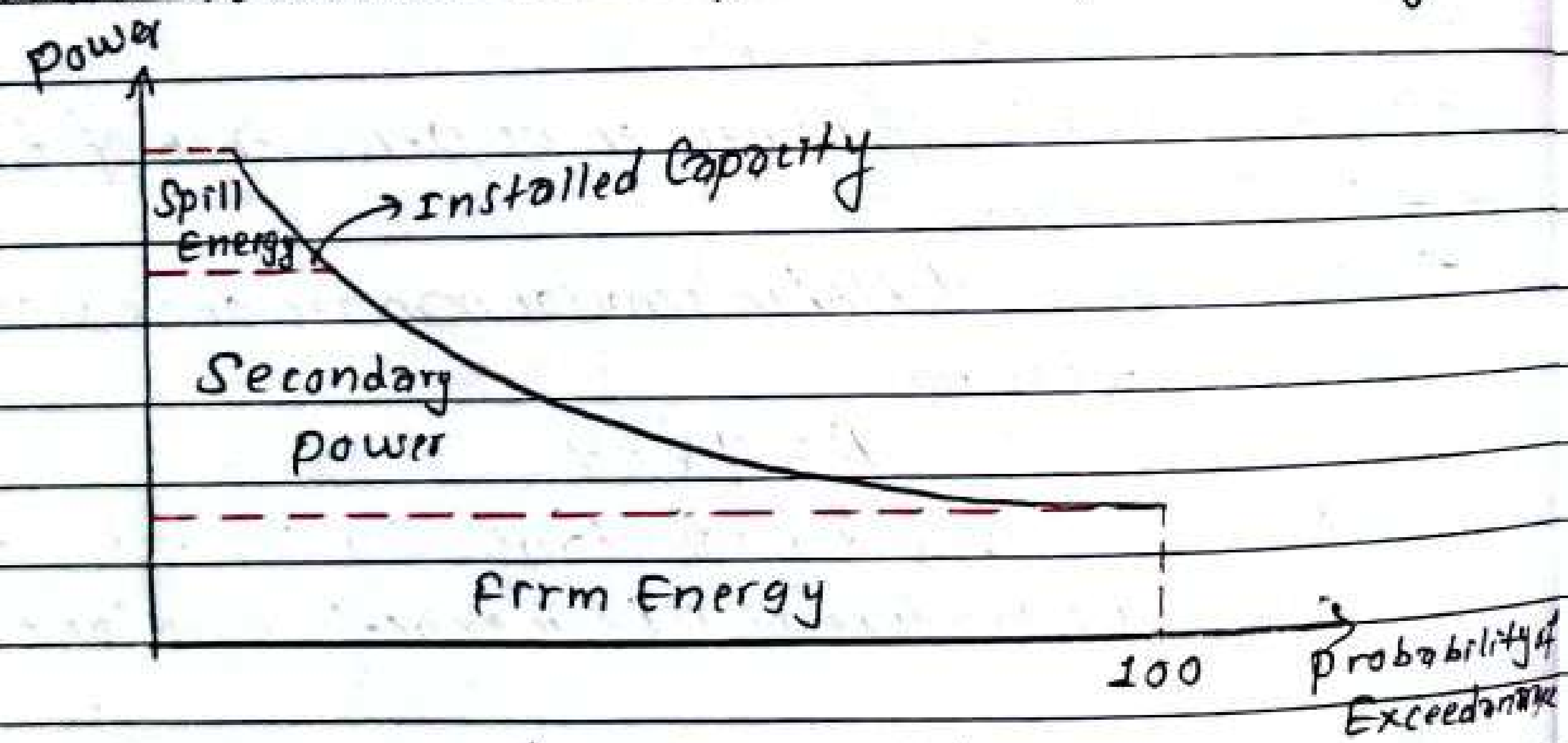


Fig:- power duration curve

Q) The average monthly flows for a certain river is given below. The net head is 20m & overall efficiency is 90%.

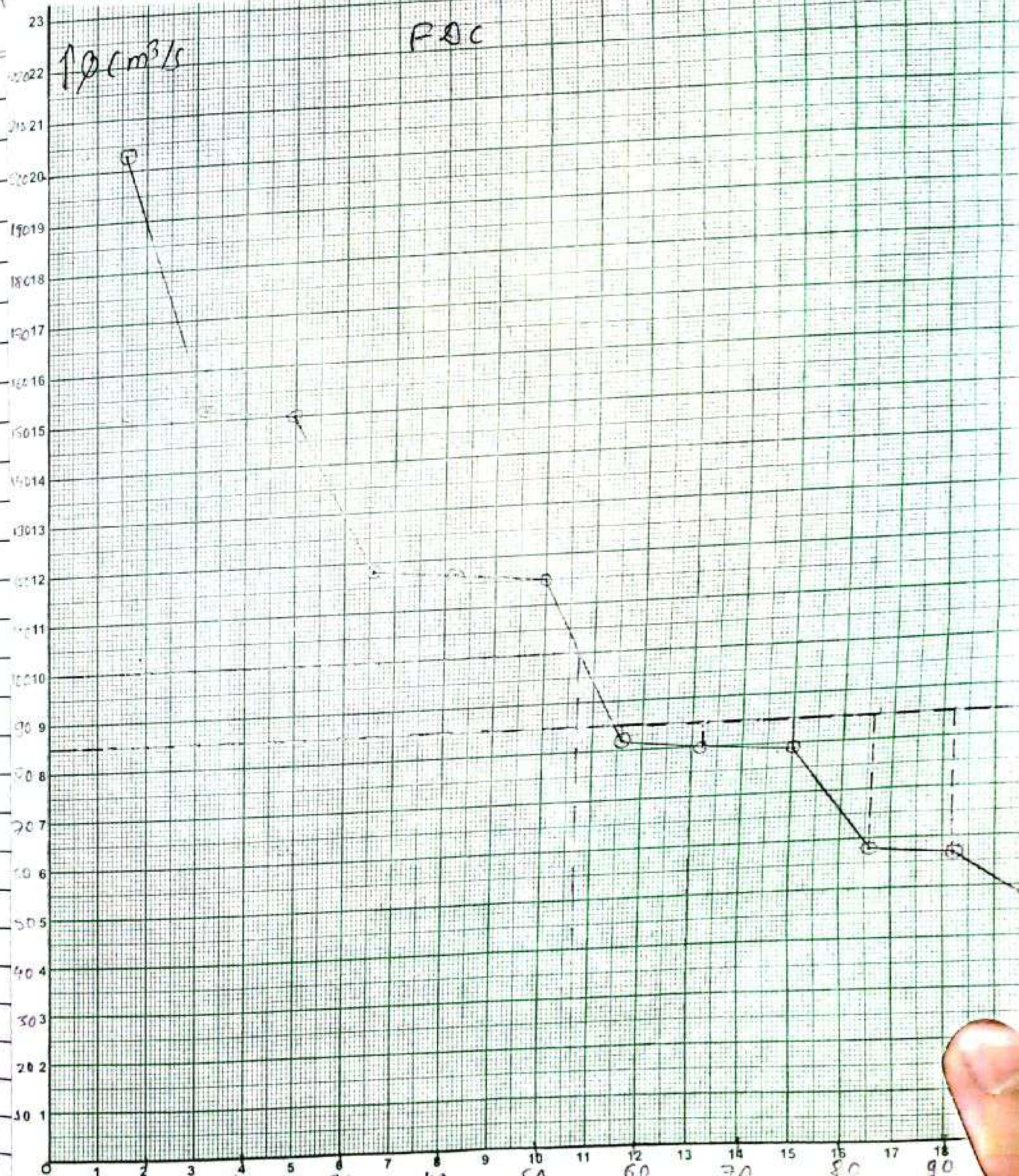
Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Q (m^3/s)	117	158	203	115	80	118	82	79	58	45	57	152

a) Plot FDC & PDC

b) Find Q_{CS}

c) What is the probability of exceedance of 100 cumecs?

d) Find P_{CC}



f) Calculate the firm power & secondary power if maximum usable flow is limited to 150 m³/s.

g) If it intended to develop a power at a firm rate of 15 MW, determine the storage capacity.

→

Q (m³/s)	Rank	% Time	Power (MW)	Usable power (MW)	Energy (MWhr) = power at	Spill Energy	~~Storage~~ Volume for 15MW (Mm³)
203	1	8.33	35.85	26.49	19329.96 (Recd)	6830.06	-
152	2	16.67	26.84	26.49	19329.96 (Trap)	3542.73	
150	3	25	26.49	26.49	19329.96	127.69	
118	4	33.33	20.84	20.84	17268.53	0	
117	5	41.67	20.66	20.66	15141.44	0	
115	6	50	20.31	20.31	14948.06	0	
82	7	58.33	14.48	14.48	12693.27	0	~~[illegible]~~ 19.83
80	8	66.67	14.13	14.13	10438.47	0	212.78
79	9	75	13.95	13.95	10245.10	0	208.84
58	10	83.33	10.24	10.24	8825.81	0	179.94
57	11	91.67	10.07	10.07	7410.18	0	151.04
45	12	100	7.95	7.95	6574.66	0	133.97
					$\Sigma = 161535.4$	10500.48	906.425

Here, $k = \eta \gamma H = 0.9 \times 9810 \times 20 = 176580$

$P = kQ = 176580 \times Q \times 10^{-6}$ MW

(a) FDC & PDC are plotted in graph

(b) For Q_{65},

82	58.33
?	65
80	66.67

$Q_{65} = 82 + \dfrac{80-82}{66.67-58.33}(65-58.33)$

$= 80.4$ m³/sec

(c) probability of exceedance of 100 cumecs.

115	50
100	?
82	58.33

$\%T = 50 + \dfrac{58.33-50}{82-115} \times (100-115)$

$= 53.78\%$

If Environmental flow is given, then find usable Q as $Q - Q_{env}$ & proceed further

d) For P_{60},

58.33	14.48
60	P_{60} = ?
66.67	14.13

$P_{60} = 14.41$ MW

e) Probability of exceedance of 20MW,

20.31	50
20	?
14.48	58.33

$\%\ time = 50 + \dfrac{58.33-50}{14.48-20.31} \times (20-20.31)$

$= 50.44\%$

f) Here, Maximum usable flow = 150 m³/sec.

Maximum usable power $= 176580 \times 150 \times 10^{-6} = 26.48$ MW

Firm Energy $= 7.95 \times \dfrac{100}{100} \times 365 \times 24 = \cancel{72.54}\ 69.64$ GWhr

Firm power = 7.95 MW

Total Energy = 161535.4 MWhr

Total power $= \dfrac{161535.4}{365 \times 24} = 18.44$ MW

Secondary power $= 18.44 - 7.95 = 10.49$ MW

→ If spill Energy is required,

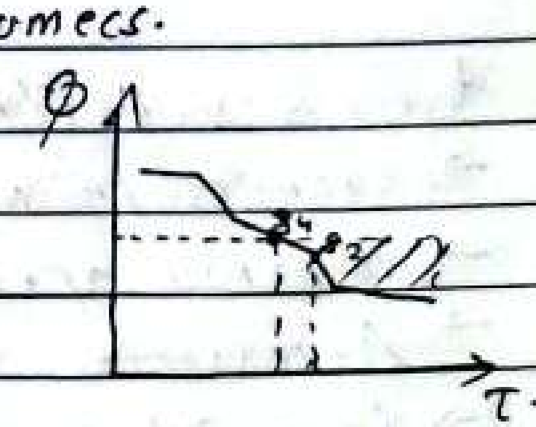
Spill Energy = 10500.48 MWhr

g) If it is intended to develop a power at a rate of 15MW,

$Q = \dfrac{P}{k} = \dfrac{15 \times 10^6}{176580} = 84.94$ cumecs.

115	50
84.94	t
82	58.33

$\Rightarrow t = 57.58\%$



Storage volm

Reqd $= 84.94 \times \dfrac{(100-57.58)}{100} \times 365 \times 86400 \times 10^{-6} - 906.425$

$= 230.29$ Mm³

Reservoir planning & Regulations:-

* Reservoir:-

→ When a barrier is constructed across the river, the pool of the water formed on upstream side of barrier is called reservoir.

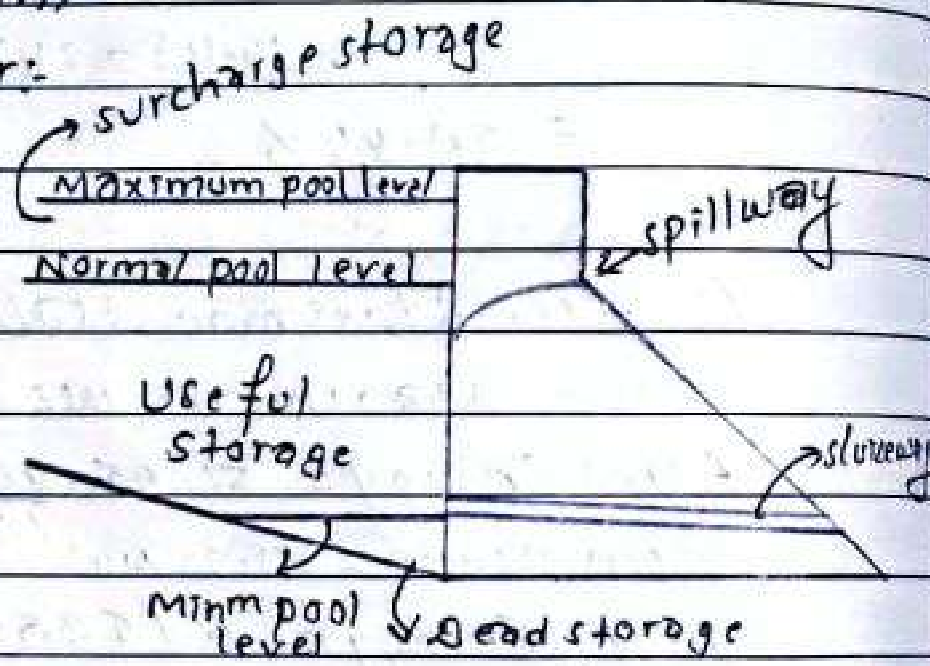
→ Barrier may be weir or dam.

→ Zones & levels of reservoir:-

(a) Normal pool level

(b) Max^m pool level

(c) Min^m pool level



* Classification of Reservoir

(a) On the basis of purpose

1) Flood control reservoir

(i) Retarding Type → It simply retards the flow without the control of gates or spillways.

(ii) Detention Type → Gates & spillways are used for flood control.

2) Storage Reservoir:-

Basic purpose is to store water when it is available in excess so that it can be used in time deficit.

* Site selection for Reservoir:-

→ located in maximum inflow & minimum percolation

→ Narrow opening of basin

→ Accessible

→ less submergance

→ free from objectionable minerals

→ Sufficient water depth

→ Construction materials locally available.

→ Heavy silt laden tributaries should be avoided.

* Regulation of Reservoir:

→ It is defined as the rational dist^b of river flow in time & space among different fields of water resource system.

→ Need of Reservoir Regulation

i) The hydroelectric plant will not operate with efficiency if it is operated below certain head.

ii) To prevent the excessive silting in reservoir, under-sluice should be regulated accordingly.

* Useful life of reservoir:-

→ It is impossible to completely stop the flow of sediment of water into reservoir. A dead storage is made available to accomodate the volume of sediment.

→ The useful life of reservoir is said to exist till the storage is reduced to 20% of designed capacity.

* Few Terms related to life of reservoir:-

(a) Trap Efficiency:

$$\text{Trap Efficiency} = \frac{\text{Total sediment deposited in reservoir}}{\text{Total sediment flowing in river}}$$

(b) Capacity Inflow Ratio:-

$$\text{Capacity inflow Ratio} = \frac{\text{Capacity of Reservoir}}{\text{Total inflow into the reservoir}}$$

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※ Estimation of Useful Life of Reservoir:-

a) Calculate annual volume of sediment carried by the river.

b) Divide total duration into suitable intervals.

c) For each interval calculate capacity inflow ratio at beginning & end of interval

80% reduction

20% 20% 20% 20%

d) Calculate corresponding trap efficiency

e) Adopt average value of trap efficiency

f) Calculate Annual volume of trapped sediment

g) Calculate no. of years required to fully fill up corresponding reservoir location.

h) Sum all the required years to get useful life.

Q. A reservoir has the following data. Estimate the probable useful life of reservoir.

Reservoir Capacity: 20 Mm^3

Average Annual Flood volume = 40 Mm^3

Annual Sediment:- 13.33×10^4 tonnes

Specific Gravity of the sediment = 2.2

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

→ Given,

Annual sediment = 13.33×10^4 tonnes

so,

$$\text{Annual sediment volume} = \frac{13.33 \times 10^4 \times 10^3}{2.2 \times 1000} = 0.06059\ Mm^3$$

	Calculation	Unit	First 20% fill up	Next 20% Fill up	Next 20% Fill up	Next 20% Fill up
a)	Capacity at beginning of interval	Mm^3	20	16	12	8
b)	Inflow at beginning of interval	Mm^3	40	40	40	40
c)	Capacity at end of interval	Mm^3	16	12	8	4
d)	Inflow at end of interval	Mm^3	40	40	40	40
e)	Capacity Inflow ratio at beginning	-	0.5	0.4	0.3	0.2
f)	Capacity inflow ratio at end of interval	-	0.4	0.3	0.2	0.1
g)	Trap Efficiency at beginning	%	96	95	94	92
h)	Trap Efficiency at end of interval	%	95	94	92	86
i)	Average trap efficiency	%	95.5	94.5	93	89
j)	Annual volm of trapped Sediment	Mm^3	0.0578	0.0572	0.0563	0.0532
k)	No. of years.	year	69.128	69.85	70.986	74.176

useful life of reservoir = 69.128 + 69.85 + 70.986 + 74.176

= 284.14 years.

※ Environmental Study policy based on type & size (IEE/EIA)

→ To check whether the proposed project has significant effect on the environment & whether such effects could be avoided or mitigated by any means or not, various studies are carried out at different stages of project planning which is known as Environmental Assessment (EA)

→ There are two types of environmental study policy

a) IEE (Initial Environmental Examination)

b) EIA (Environmental Impact Assessment)

MoEST ⇒ Ministry of Environment Science & Technology
MoE ⇒ Ministry of Energy

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* Difference betn IEE & EIA

IEE	EIA
① Generally conducted for small scale projects.	① Generally conducted for large scale projects.
② Scoping is not required.	② Scoping is required.
③ Environmental auditing not required.	③ Environmental auditing required.
④ Deals with simple & easily predictable impacts.	④ Deals with unknown impacts also.
⑤ It is to be approved by concerned authority within 21 days.	⑤ It is to be reviewed by the concerned body & approved by MoEST within 90 days.
⑥ Public input at different stages of report preparation.	⑥ Public inputs also during the approval process.

※ EIA/IEE Processing in Hydropower:

(a) Environmental Screening

→ This process determines the level of EA study & also check whether EIA or IEE is required or not.

→ Refer "Environment protection Rules" to see what projects require EIA & which one require IEE.

→ Examples of projects that required EIA,
- HPP with capacity >50 MW
- Transmission line with voltage >132 kV
- Any project that destroys > 5 ha of forest
- Any project in conservation area, wildlife reserve, national park & buffered zone declared by GoN.

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(b) Scoping:

→ Carried out to ascertain the major issues that are likely to arise due to project implementation.

→ In case of IEE, scoping is not required.

→ It involves the following tasks.
- Involvement of relevant authorities, interested parties & affected groups
- Identification of problems.
- Identification & selection of alternatives.
- Determination of TOR of EIA for further study.

(c) Terms of Reference (TOR)

→ Provides basic guidelines to conduct project specific EIA or IEE.

(d) EIA/IEE Report:

→ IEE report ⇒ review by MoEn ⇒ If everything is ok ⇒ grant approval for its implementation

→ EIA Report ⇒ Review by MoEn ⇒ Forwarded to MoEST ⇒ 30 days wait for public notice/issue ⇒ If everything is ok, MoEST shall grant approval.

* Climate change & Ecology:

※ River Engineering:

In himalayan area, discharge of river is low & velocity is also low but in plain area discharge is high & velocity is low, Sedimentation problem, meandering → lead to outflanking of hydraulic structure → Require river training structures.

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✲ Social Costs:
→ Due to large hydraulic structure, changes the flow regime of river.
→ Affect lives of upstream & downstream
→ Large settlement of people are displaced
→ May effect particular social group of community
→ Different religious places & conservation area may lie within project area.

✲ Population Displacement:
→ Affected by size of reservoir & topography of land
→ More displacement in flat area
→ For large reservoir, area of submergance will be more.
→ Destroys forest, wildlife habitat, agricultural land due to flooding.

✲ Change in lifestyle:
→ Health issue, drainage, road, irrigation canal, water supply issue, pollution, Limited Cultivation etc.

✲ Global Worry:
→ Due to stagnant water on reservoir, vegetation will decompose & release CO_2 & CH_4
→ Global warming due to deforestration

✲ Clean Energy Alternatives:
→ Solar Energy → Wind Energy → Geothermal Energy

Chapter-3, Power & Energy potential Study

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Contents:-
→ Gross, Net, operating & design head
→ plant & Installed Capacity
→ Energy flow diagram (related to FDC), firm & secondary power & energy
→ Economic consideration in HP system: Marginal cost benefit approach & introduction to optimization approach.
→ Estimation of power & Energy potential & its demand prediction methods
→ load curve, load factor, utilization & diversity factors
→ power demand variation: weekly, monthly & annual
→ Power grid: Introduction & Components of power grid System

✲ Some Terminologies:-
a) Gross Head:-
In general, Gross Head (H) is the difference of waterlevel in the headrace & waterlevel in tailrace.
→ For storage project, H = Waterlevel in reservoir − waterlevel in tailrace
→ For ROR project, H = Water level at point of diversion − RL when water is returned back to river
→ For Hydropower with pelton turbine,
H = Water level at intake − ~~Water~~ level of axis of turbine
→ For Hydropower with Francis Turbine
H = Water level at intake − Water level in tailrace.

b) Net Head:-
→ The head available at the turbine after deducting the head losses within the ~~[illegible]~~ system is called net head.

c) Operating Head: The difference of water surface elevation

$$F = P(1+i)^N = A\left[\frac{(1+i)^N - 1}{i}\right]$$ Sumit Paudel

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in the forebay & tailrace after making due allowance of approach & exit velocity heads.
Operating head = Total Energy level at forebay entrance − Total Energy level at Exit.

d) Rated Head: Head at which turbine operating at full gate opening will produce output specified in the nameplate.

e) Design Head:- Net head under which turbine reach peak efficiency at synchronous speed.

f) Plant Capacity: Capacity of plant with respect to available head, discharge & efficiency.

g) Installed Capacity:- plant capacity that is economically viable.

- Installed capacity = $\eta \gamma Q_d H$
  where, Q_d = design discharge.

Determination of Installed Capacity by Marginal Cost Benefit Approach:-

→ Let 1kW corresponds to optimum installed capacity at x% of time

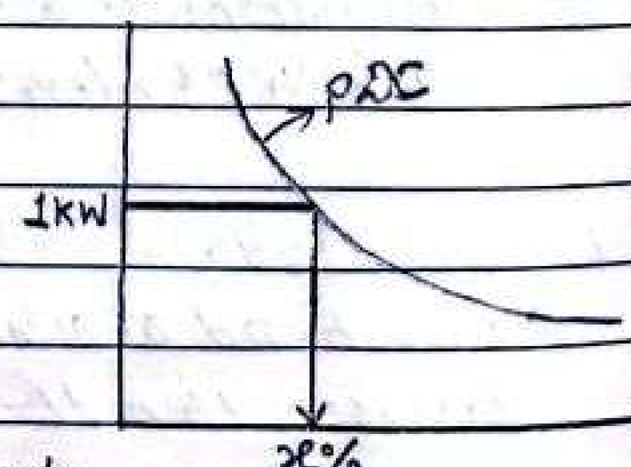

Fig: power duration Curve

Annual Energy = 1 kW * x% of 365×24hr
= 87.6 x kWHr

Marginal Benefit = Annual Energy x Rate
= 87.6x kWHr * Rate per kWhr ---- ①

Civil Cost is assumed as fixed cost (not included)
Electromechanical cost is assumed variable cost (VC).

$$\text{Annual Variable Cost} = VC\left[\frac{(1+i)^N * i}{(1+i)^N - 1}\right] \rightarrow \text{Capital Recovery Factor (CRF)}$$

Operation & Maintenance Cost = y% of VC
Marginal Cost = Annual Variable Cost + O&M ---- ⑪
Equating ① & ⑪, x = ---%.

Q) The average monthly discharge is given below. Determine Installed Capacity, design discharge & Secondary Energy.

Month	J	F	M	A	M	J	J	A	S	A	N	D
Q (m³/s)	40	35	30	25	20	38	75	95	70	55	40	55

Interest rate = 13%.
Energy price = US $ 40/MWH
Fixed cost:- US $ 2800/kW (Civil cost)
Variable cost: US $ 650/kW (Electromechanical)
O&M:- 2.5% of variable cost
Economic life of plant = 45 years, Net Head = 50m
Overall Efficiency of the system = 85%.

→ Solution,
Let 1 kW corresponds to optimum installed capacity at x% of time
Annual Energy = 87.6 * x kWHr

$$\text{Marginal Benefit} = 87.6 * x * \frac{40}{1000} = 3.504x \text{ ---- ①}$$

$$\text{Annual Variable Cost} = VC\left[\frac{(1+i)^N * i}{(1+i)^N - 1}\right]$$

$$= 650\left[\frac{(1+0.13)^{45} * 0.13}{(1+0.13)^{45} - 1}\right] = 84.84$$

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Operation & maintenance cost $= \frac{2.5}{100} \times 650 = 16.25$

$\therefore$ Marginal cost $= 84.84 + 16.25 = 101.09$ US\$ — (II)

Equating (I) & (II),

$$x = \frac{101.09}{3.504} = 28.85\%$$

Also, $K = \eta_0 \gamma H = \frac{85}{100} \times 9810 \times 50 = 416925$

Q in order	Rank	% time	P (MW)	Usable power	Total Energy (MWhr)
95	1	8.33	39.607	27.73	20234.8
75	2	16.67	31.269	27.73	20259
70	3	25	29.184	27.73	20234
		28.85	27.73	27.73	9352.2
55	4	—	—	—	—
55	5	41.67	22.93	22.93	28446
40	6	—	—	—	—
40	7	58.33	16.67	16.67	28896
38	8	66.67	15.84	15.84	11875
35	9	75	14.59	14.59	11102
30	10	83.33	12.507	12.507	9886.4
25	11	91.66	10.423	10.423	8366.1
20	12	100	8.338	8.338	6853.2

$\Sigma = 175507.8$

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For installed capacity,

25	29.184
28.85	?
41.67	22.93

$$\text{Installed Capacity} = 29.184 + \frac{22.93 - 29.184}{41.67 - 25} \times (28.85 - 25)$$

$$= 27.73 \text{ MW}$$

$$\text{Design discharge } (Q_d) = \frac{27.73 \times 10^6}{416925} = 66.53 \ m^3/s$$

$$\text{Primary/Firm Energy} = 8.338 \times \frac{100}{100} \times 365 \times 24 = 73040.88 \text{ MWhr}$$

Total Energy $= 175507.59$ MWhr

Secondary Energy $= 175507.59 - 73040.88$

$= 102466.71$ MWhr

$= 102.46$ Gwhr

* Determination of Installed Capacity of HPP by Optimization Approach:-

→ Power is a function of head & discharge. Increasing the percentile of available flow, design discharge decreases.

→ For low design discharge, the size of hydropower components 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. Thus, the discharge for different percentile time are calculated from flow duration curve & corresponding revenue & cost are calculated. The optimum capacity is fixed as installed capacity.

Estimation of power & Energy potential & its demand Prediction method

→ A demand forecasting is a projection of required electricity generation in the system for the electric load that the existing & potential consumers will like to be served in future.

→ The electricity demand in future depends on
i) Willingness & affordability to pay
ii) Frequency
iii) Time.

→ Prediction / forecasting
Short term - 4 to 5 years
Medium term - 8 to 10 years
long term - 20 years or more

→ Method of Estimation of Electrical Demand

(a) classwise consumption:-
Electricity is consumed at different rate by different people. Residential area will have different energy consumption than commercial area.

(b) Historical Trends:
The past recorded data can be mathematically interpreted & hence future demand can be forecasted using regression analysis.

(c) GDP:-
The exists a good correlation between power development & economic development GDP. The data of GDP & per capita energy consumption of country are used to find the relationship betn economic development & power consumption. After a relationship is sought, it can be used to find the energy demand for targeted economic development.

(d) Mathematical Formula:-
1) Incremental Formula,

$$P = P_0\left(1 + \frac{R}{100}\right)^N$$

where, P = Power demand after 'N' years
R = Incremental rate of power demand.

2) Country Specific formula, (not necessary ☺)

Load Curve:

→ It is a graphical representation of power consumption with respect to time.
→ A load curve may be daily, weekly, monthly or annual.
→ Area under load curve of particular duration gives the total energy consumption in that duration.
→ 1 unit = 1 kWHr

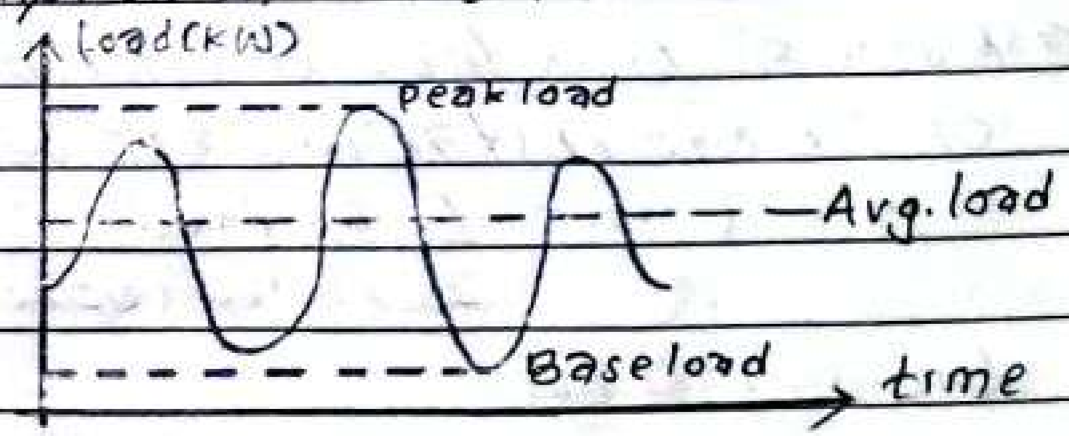

Fig: Load Curve

→ Base load = Minm power demand in given time duration
→ Peak load = Maxm power demand in given time duration.
→ Average load = Avg. power consumption over a defined time period

$$\text{Avg load} = \frac{\text{Area under curve}}{\text{Total duration of time}}$$

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✻ Load Factor:- (LF) ⇒ Ratio of average load to peak load.

→ It indicates the fluctuation in power demand.

$$\text{Load factor (LF)} = \frac{\text{Average load}}{\text{peak load}} = \frac{\text{Avg. load} * T}{\text{Peak load} * T}$$

$$LF = \frac{\text{Area under load curve}}{\text{Total area of rectangle on which curve lies}}$$

→ $LF \leq 1$.

✻ Plant Capacity factor:- (PCF) ≤ 1

→ Ratio of average energy produced by plant to maximum energy that can be produced.

$$PCF = \frac{\text{Average load}}{\text{Installed capacity}} \quad \text{------- (i)}$$

$$= \frac{\text{Average load} * T}{\text{Installed load} * T}$$

$$= \frac{\text{Energy produced}}{\text{Max}^m\ \text{energy that can be produced}}$$

✻ Utilization factor:- (UF) ⇒ (0.4 to 0.9)

→ Ratio of peak load developed during certain period of time to installed capacity of plant.

$$U.F = \frac{\text{Peak load}}{\text{Installed Capacity}} \quad \text{--- (ii)}$$

Dividing (i) by (ii)

$$\frac{PCF}{UF} = \frac{\text{Avg load}}{\text{Peak load}} = LF$$

$$\Rightarrow PCF = UF \times LF$$

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✻ Diversity factor (DF) ≥ 1

$$DF = \frac{\text{Sum of individual max}^m\ \text{demand}}{\text{Simultaneous max}^m\ \text{demand}}$$

Q. A power station with reserve capacity of 30MW has following load variation:

Time	0 − 6	6 − 10	10 − 12	12 − 16	16 − 20	20 − 24
Load (MW)	40	50	60	50	70	40

Draw a daily load curve & find

i) Maximum demand
ii) Units generated per day
iii) Average load
iv) Daily load factor
v) Plant Capacity factor
vi) Plant Utilization factor

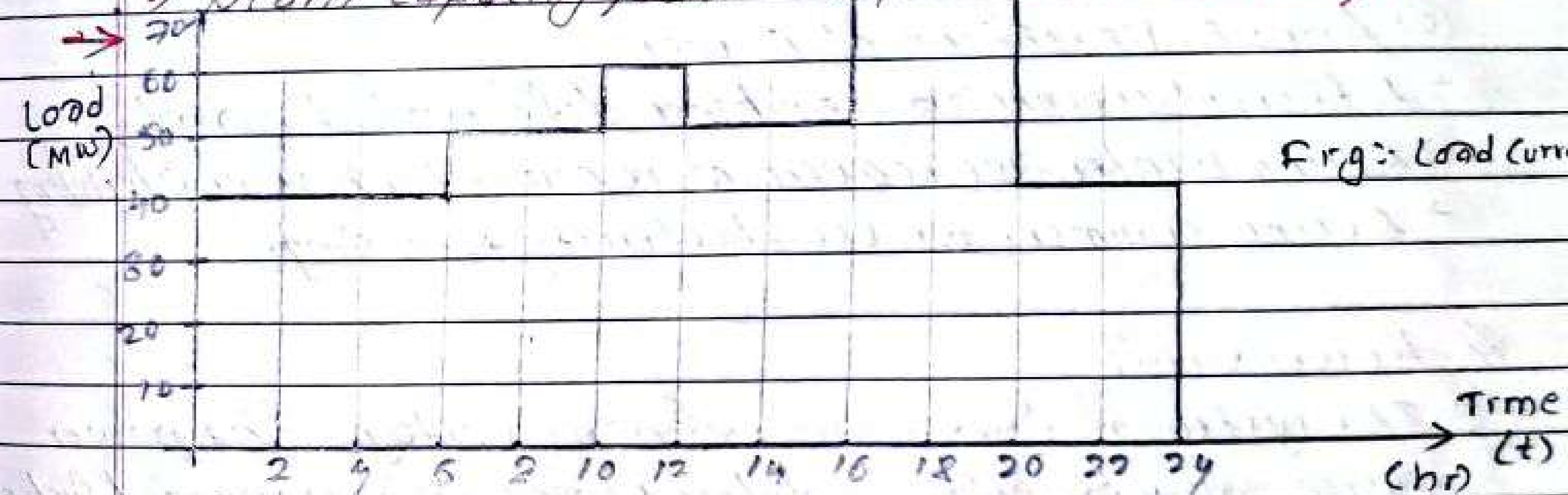

Fig:- Load Curve

1) Max^m demand = 70 MW

2) Units generated per day = $40 \times 24 + 10 \times 14 + 2 \times 10 + 4 \times 20 = 1200$ MWhr = 12000 Unit

3) Average load = $1200/24 = 50$ MW

4) Daily load factor = Avg load / peak load = $50/70 = 0.714$

5) Plant capacity factor = $\frac{\text{Avg load}}{\text{Installed Capacity}} = \frac{50}{70+30} = 0.5$

6) Utilization factor (UF) = $\frac{PCF}{LF} = \frac{0.5}{0.714} = 0.70$

(Installed Capacity = Max^m demand + Reserve Capacity.)

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Power Demand Variation:-

(a) Daily Variation of power:
→ Maxm demand in morning & evening
→ Less demand during office hours

(b) Weekly Variation of Power:-
→ Daily routine changes at weekend
→ If people stay at home during weekend, power demand is more. If people spend time on travels, power demand at home reduces.

(c) Monthly Variation of power:-
→ More demand during festivals in different months.

(d) Annual Variation of power:-
→ Different seasons in year have different demand.
→ During winter, we require more power for heating & lighting
→ During summer, we use electricity for cooling.

Power Grid:-

→ The system of transmission of high voltage. The power system may be either isolated or the interconnected System.
→ Isolated System is not connected to national grid.

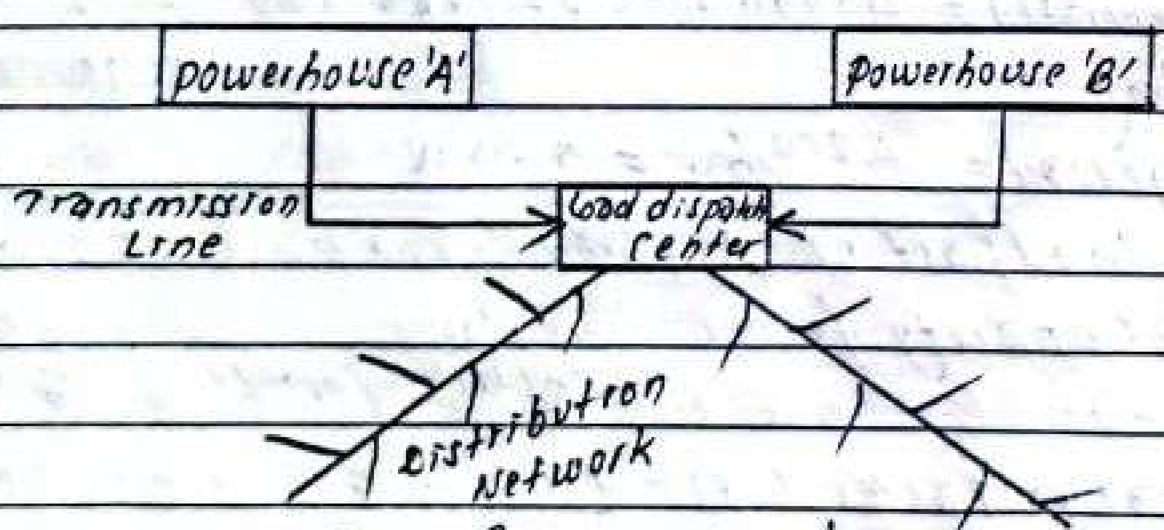

Fig: Systematic diagram of power grid.

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Components of Power Grid System:-

(a) Generation Station (Powerhouse, generator, transformer, switchgear)
→ Electricity is produced in powerhouse. Thus produced electricity is stepped up using transformers because transmission at high voltage increases transmission efficiency.

(b) Transmission lines:- They are designed to carry electricity or an electrical signal over large distances with minm losses & distortion.

(c) Load Dispatch Center (LDC) → It is a coordinating agency for state electricity boards for ensuring a mechanism for safe & secure grid operation.

(d) Substations: Consists of various equipment & is responsible for stepping up voltage levels for transmission or stepping down voltage levels for distribution purpose.

(e) Distribution Transformers:
Before the power is supplied to the consumers, it should be stepped down to the suitable voltage so that it can be safely used for designed purposes.

(f) Household / Consumers:
Consumers are the ultimate components of power System. They use electricity for various purposes. In Nepal, the electrical equipment of 220-240 volt are used at a frequency of 50 Hertz.

Chapter-4, Storage Type of Hydropower Projects

Contents:-

→ General layout of components in typical storage power plant
→ Dams & their appurtenant works.
- classification (function, material, head & mode of structural load transfer)
- Forces acting on a dam & their combination
- Site selection
- Principal variant of embankment & concrete dams
- Failure modes & their prevention measures
- Design of gravity (concrete) dam
- Design of earthen (embankment) dam

→ Intakes: Types, arrangement, location, functions (in ch-5)
→ Gates: Types & their location in dam (in chapter-7)
→ Reservoir sedimentation issue & sedimentation management

※ Components of Typical Storage power plant:-

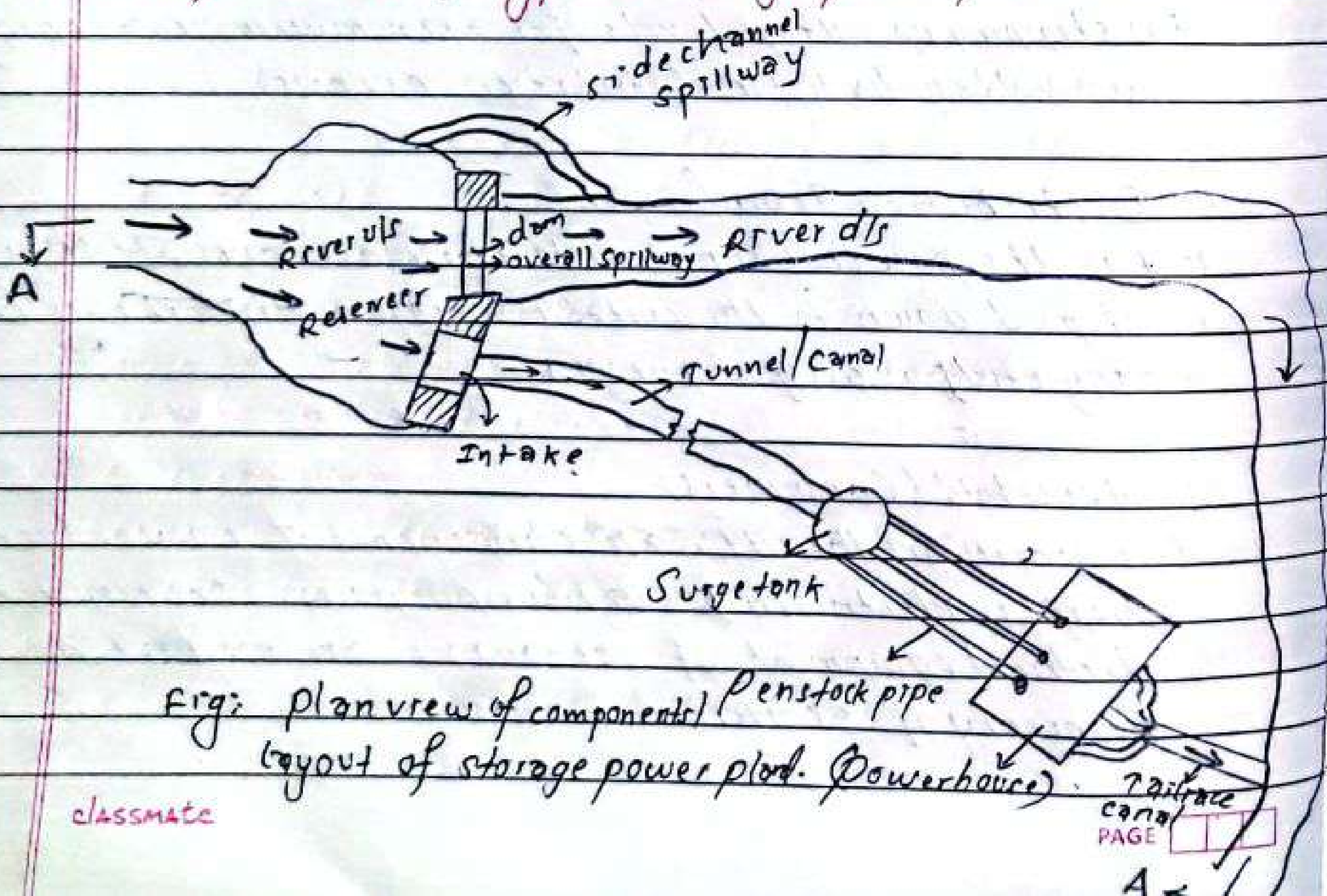

Fig: plan view of components/ layout of storage power plant.

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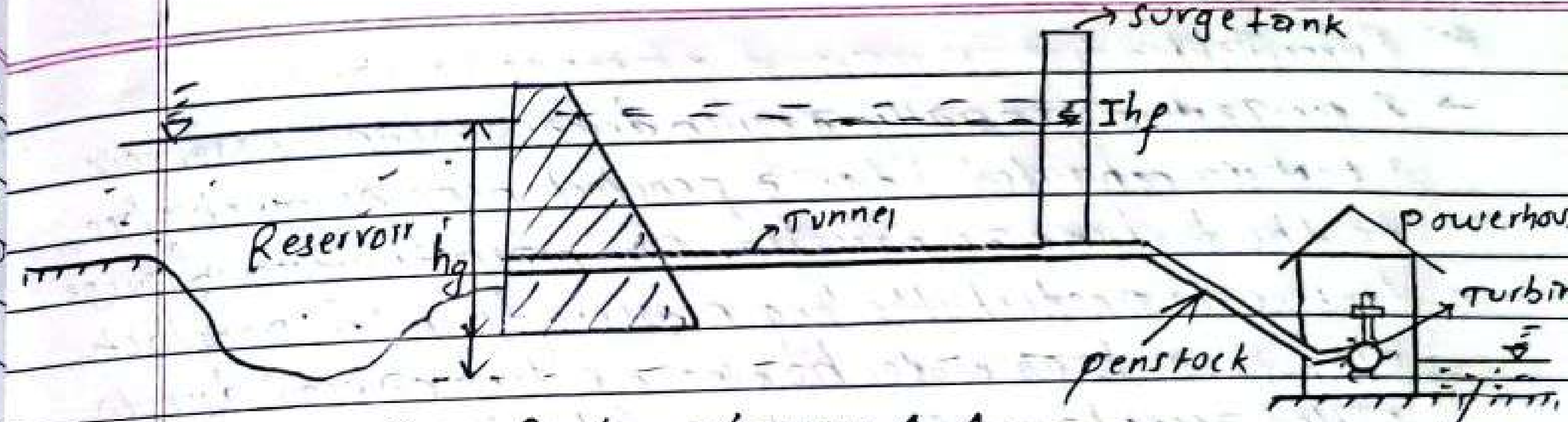

Fig: Sectional view A-A

(a) Dam:-
→ Barrier built across the river for accumulation of water on u/s side.

(b) Reservoir:-
→ The main purpose of reservoir is to store water during rainy season & supply the same water during the dry season & thus it helps in supplying water to the turbines according to the load.

(c) Intake:
→ Structure to divert water into the conduit leading to the power plant.
→ Trash rack screen is provided to prevent the entry of floating debris & coarse bed load into the water conveyance system. It is well equipped with control gate to regulate flow down the conveyance system.

(d) Headrace Conveyance:
→ Headrace conveyance is the system to carry water from intake to fore bay. The conveyance system may be consist of pipe or tunnel or open channel.

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(e) Surge Tank:-

→ Surge Tank is generally a cylindrical storage reservoir which is connected to a penstock pipe as much close to the turbine as possible. The main function of surge tank is to protect the long pressure tunnel & penstock pipe from high water hammer effect caused due to sudden acceptance & rejection of load.

(f) Fore-Bay

→ structure located at the beginning of penstock pipe

→ It supplies the required flow to the turbine during startup, accomodate the rejected flow during shut down & reduce water hammer effect.

→ Equipped with trash rack to prevent floating debris into the penstock. It also serves as secondary settling basin.

(g) Penstock

→ Pipe that conveys the flow from forebay or surge tank to the turbine.

→ Designed to carry water to the turbines with the least possible loss of head.

→ Anchor blocks are provided to stabilize force acting on the penstock pipe.

(h) Powerhouse:-

→ structure where all the equipment for providing electricity are suitably arranged.

(i) Tailrace:-

→ It is the structure to dispose the water after being used at powerhouse for the power generation.

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※ Classification of Dam:-

(a) Based on function:-

1) storage dam: Constructed to store water for drinking, irr, hydropower.
2) Diversion dam: For diversion of flow of river
3) Detention dam: To reduce the flood peak discharge.
4) Debris dam/check dam:- Constructed to reduce slope of channel or river due to accm of sediments.
5) Coffer dam: structure that retains water & soil that allows the enclosed area to be pumped out & excavated dry.

(b) Based on Head:-

1) Overflow dam: Weir is considered as overflow dam
2) Nonoverflow dam:-

(c) Based on material

1) Concrete dam
2) Rock fill dam
3) Masonry dam
4) Earthen dam
5) Steel dam
6) Timber dam
7) Rubber dam
8) Combined dam

(d) Based on structural load transfer:-

1) Gravity dam: balance all forces with its self weight.
2) Arch dam:- load is transferred to the abutments using arch action
3) Buttress dam:- The load acting on the slab is transferred to the foundation through buttress.

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* Forces acting on dam & their combination

(a) primary load:-

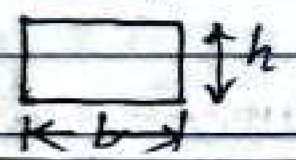
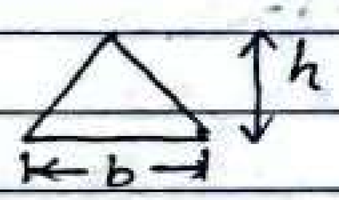
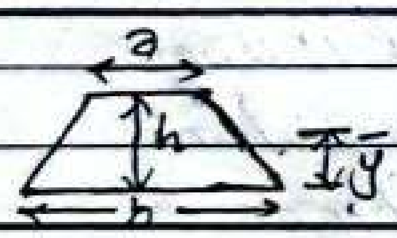
1) Self weight (Dead load)

$W = \gamma_c \times A$ for unit width of dam

where, γ_c = unit weight of material of dam
= 24 for pcc & 25 kN/m³ for RCC

→ If dam section is not regular in shape, it is broken down into small regular shapes & area of each is calculated.

→ Centroid & area of common shapes

Shape	Area (A)	centroid ($\bar{y}$)
	$A = bh$	$h/2$
	$A = \frac{1}{2} \times b \times h$	$h/3$ from base
	$A = \frac{1}{2}(a+b) \times h$	$\bar{y} = \frac{h}{3}\left(\frac{2a+b}{a+b}\right)$ from b.

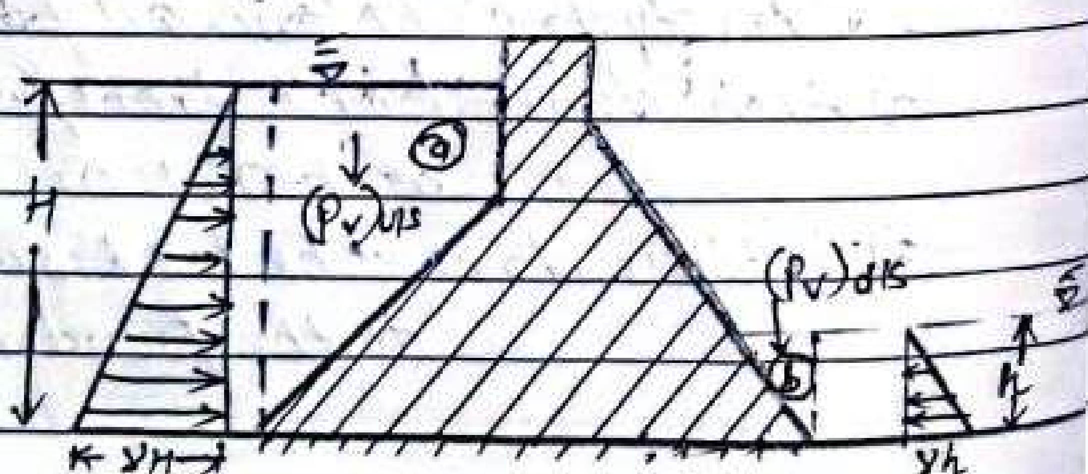
(2) Water Pressure:-

→ Area under pressure diagram gives forces.

$(P_H)_{u/s} = \frac{1}{2} \times H \times \gamma H = \frac{\gamma H^2}{2}$

acting at $H/3$ from bottom

$(P_H)_{d/s} = \frac{1}{2} \gamma h^2$ acting at $h/3$ from bottom



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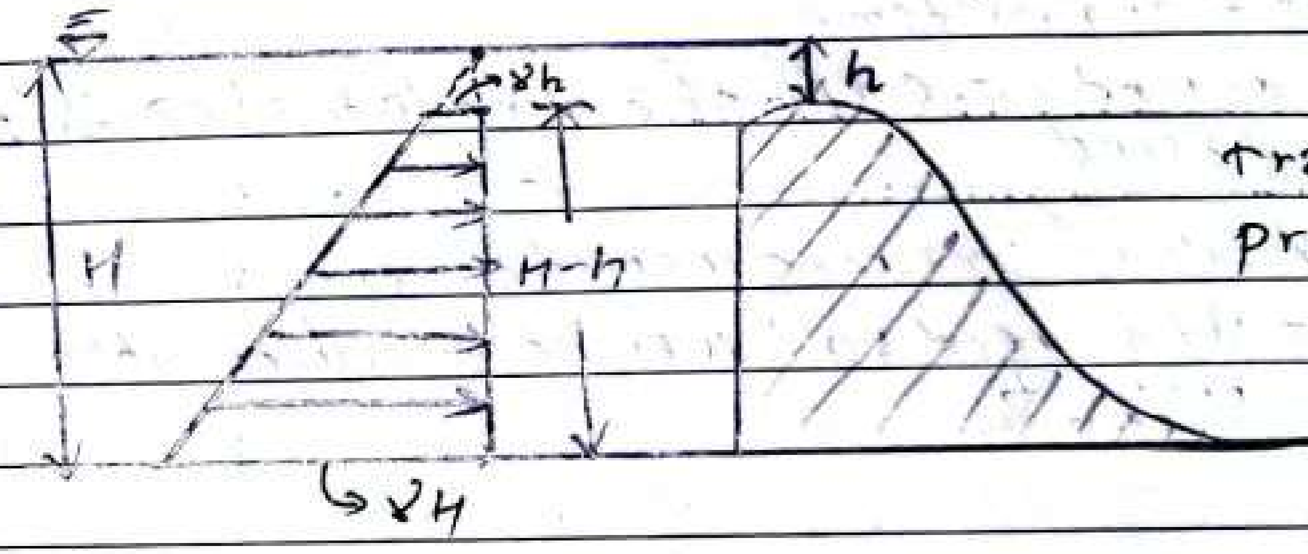
→ If the dam slope is vertical, it doesn't hold water in vertical direction. So, there will be no vertical component of water pressure.

$(P_v)_{u/s} = \gamma_w \times a$ & $(P_v)_{d/s} = \gamma_w b$

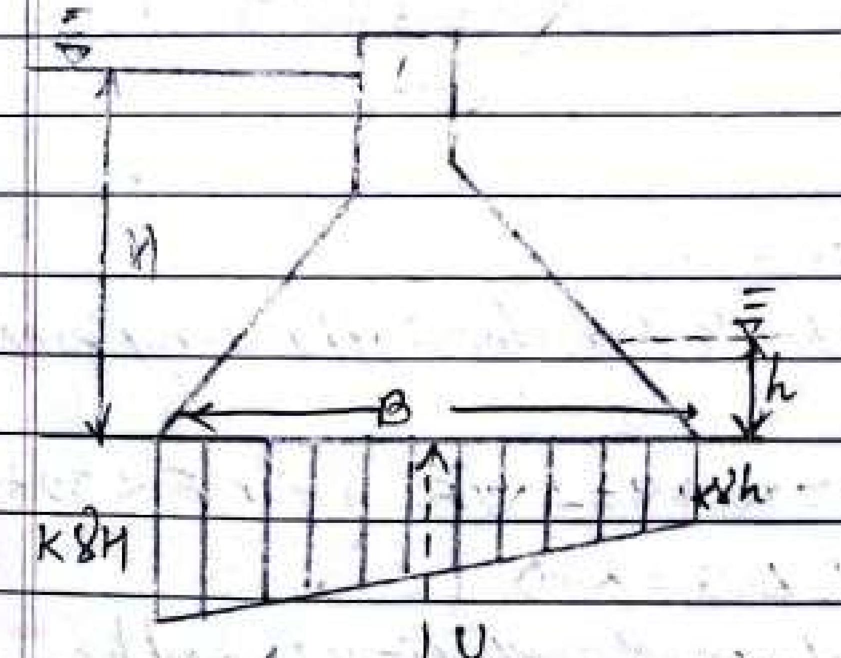
where, a = area occupied by water over dam u/s
b = area occupied by water over dam d/s

→ For overflow section,



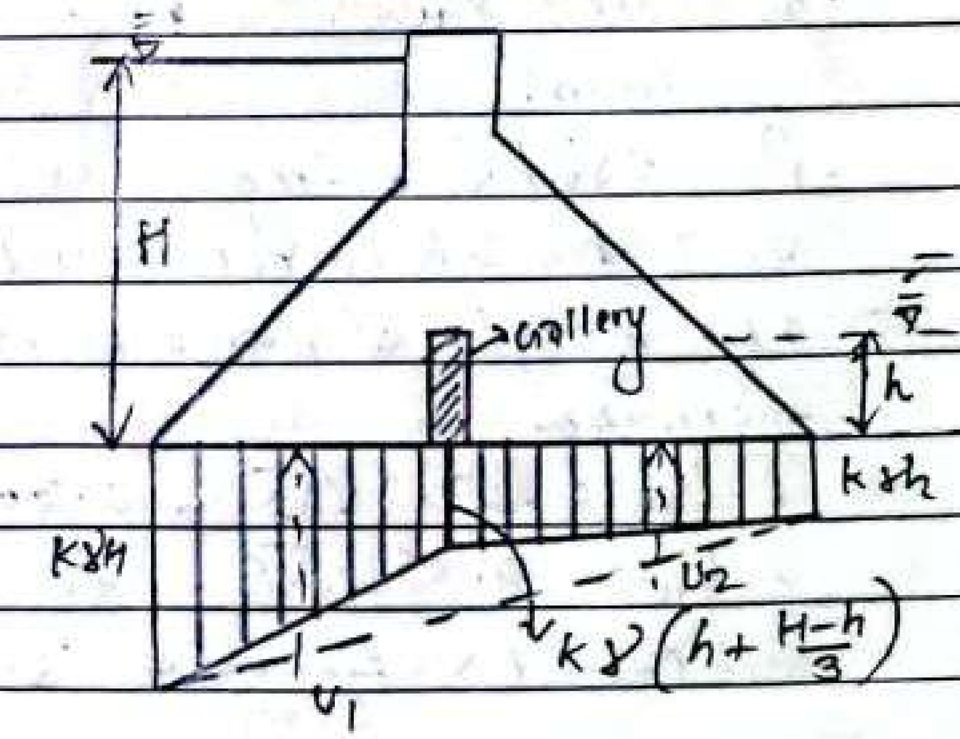
(3) Uplift Pressure:

→ The uplift pressure distribution is assumed trapezoidal.



$U = \frac{1}{2} \times B \times (k\gamma H + k\gamma h)$

acting at a centroid of press' diagram



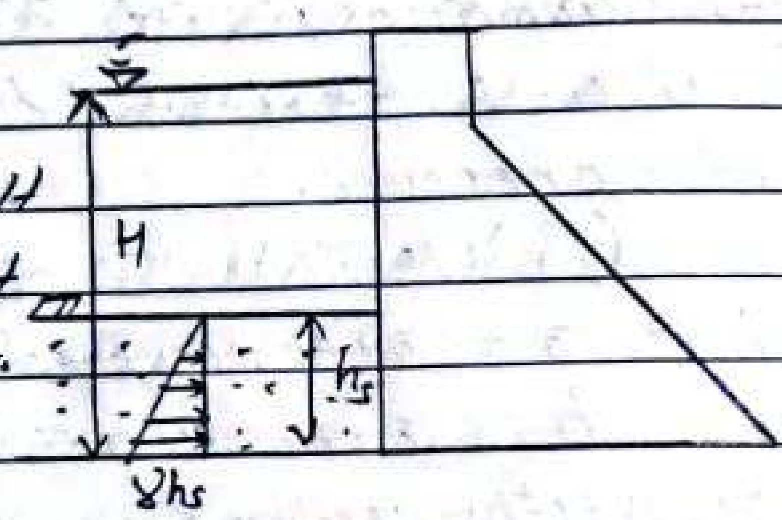
$U = U_1 + U_2$

Fig:- Uplift pressure with & without drainage gallery.

classmate k → Seepage Coeff.

(b) Secondary Load:-

(i) Silt load:-



The press^r dist^b due to silt accumulation upto height 'h_s' is assumed triangular.

$P_s = \frac{1}{2} k_a \gamma_{sub} \times h_s^2$ acting at $h_s/3$ from bottom.

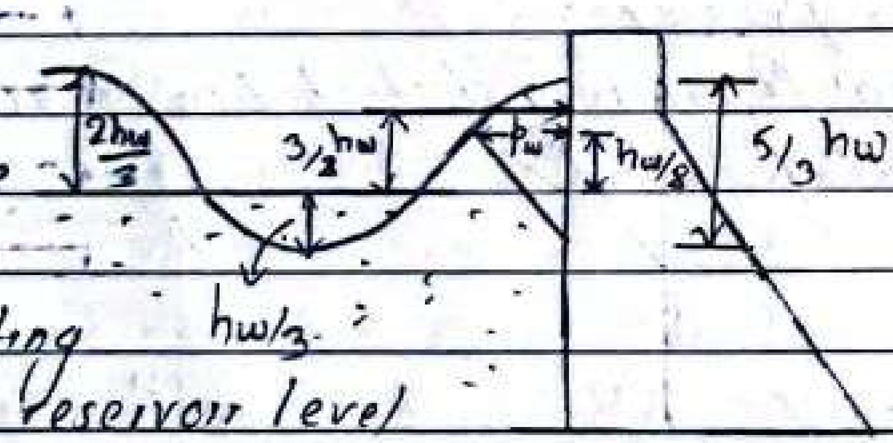
$k_a = \frac{1-\sin\phi}{1+\sin\phi}$ = coeff. of active lateral earth pressure.

ϕ = Angle of shearing resistance.

γ_{sub} = Submerged unit weight of sediment (= 30°)

$= \gamma_{sat} - \gamma_w$

(b) Wave load:



Let,

h_w = height of wave in 'm'

$P_w = 2\gamma_w h_w^2$ acting at $3/8 h_w$ above reservoir level

→ The height of wave is calculated by following empirical formula.

$h_w = 0.032\sqrt{VF} + 0.76 - 0.0271\sqrt[4]{F}$ (for F < 32 km)

$h_w = 0.032\sqrt{VF}$ (for F > 32 km)

where, F = Fetch (km) & V = wind velocity in km/hr

(c) Wind load:

→ Not considered in design

→ If considered, value → 981 N/m² to 1470 N/m² in exposed area

(d) Ice load:

→ The pressure due to ice of thickness < 0.4m is neglected.

→ pressure due to ice for thickness > 0.6m,

$P_{ice} = 145\ kN/m^2$

(c) Exceptional Load -

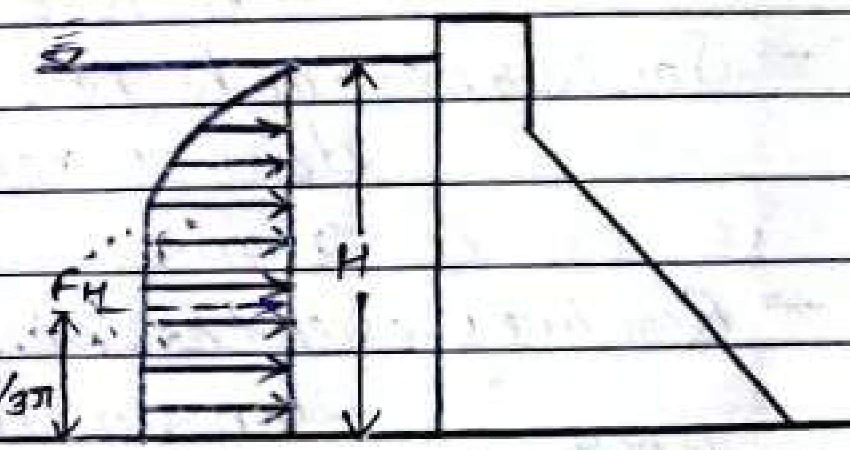
(a) Earthquake or Seismic load:

→ It is considered by multiplying weight of dam with acceleration coefficient & applying at C.G.

$F_{eqx} = W * \alpha_x$

$F_{eqy} = W * \alpha_y$

(b) Hydrodynamic Wave load:



→ $F_H = 0.555\, \alpha\, \gamma_w H^2$

acting at $4H/3\pi$ from bottom.

where,

α = Earthquake acc^n coefficient

γ_w = Unit weight of water

H = Depth of water in the reservoir.

Load Combination:

→ All the forces mentioned above donot occur simultaneously on a dam. Taking all the forces will give large section of the dam. So, a concrete dam is designed based on most rigorous adverse grouping or combination of loads which

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have a reasonable probability of simultaneous occurrence.

→ The design of gravity dam should be checked for two cases.

(a) when reservoir is full
- Standard load ~~condition~~ combination
- Extreme load combination

(b) When reservoir is empty.

Site selection for dams & selection of type of dam:-

*** Selection of type of dam**

→ Material of construction :- locally available
→ Geological condition :- Geology of area.
→ Topography :- in narrow valley → Construct gravity dam
in wide valley → Construct embankment dam
→ Spillways :- For large spillway discharge, overflow concrete dam is required. If separate spillway site is available, earth dam should be preferred.
→ Roadway over the dam → For roadway, earth or gravity dam must be constructed.
→ ~~Life~~ Length & height of dam : If L↑ but H↓ → Earth dam
L↓ but H↑ → Gravity dam
→ Life of dam : Life more, concrete dam is better
→ Generation of hydroelectrical power :-
→ According to climatic condition
→ According to seismic condition
→ According to diversion condition
→ According to environmental & ecological condition.

*** Selection of damsite :-**

→ Narrow valley to reduce length of dam
→ Major portion of dam should be preferably on high ground to reduce cost & drainage.
→ Good foundation at moderate depth should be available.
→ Bed & sides of river should be watertight & basin should be cup shaped as far as possible.
→ Locally available material of construction.
→ Good transport fascility to dam site.
→ Healthy climate condition at damsite.
→ For large storage capacity, select dam site at confluence of two rivers.
→ Submergence area of reservoir → minimum
→ Non-erodible u/s catchment area to reduce silting.
→ Availability of location for spillway.
→ Deep reservoir with small water surface to reduce evaporation loss.
→ Catchment are must be maximum etc.

Modes of failure of concrete dams

(a) Overturning

→ when resisting moment (+↺) is less than overturning moment (+↻) about toe, overturning of dam occurs.

→ $\text{FOS against overturning} = \dfrac{\text{Resisting moment } (+↺)}{\text{Overturning moment } (+↻)}$

→ FOS > 1.5

(b) Sliding :-

→ Critical section for sliding failure is base of dam.

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→ The net horizontal force tries to slide the whole dam & frictional force betn dam & foundation tries to resist the sliding.

$$\text{FOS against sliding} = \frac{\mu\Sigma V + Bq}{\Sigma H} > (4-5)$$

where, q = shear resistance betn surfaces due to joints, B = base width of dam.

Also, if shear resistance is not considered,

$$FOS = \frac{\mu \Sigma V}{\Sigma H} > 1$$

(c) Compression failure or Crushing failure:

→ Occurs when compressive stress in dam foundation or material is greater than permissible compressive stress in dam foundation or dam material.

(d) Tension failure:

→ when tension force occurs in dam, tension crack develop which enhance seepage & hence stability of dam is reduced. So, it is tried to avoid tension force at any section of dam.

→ For no tension to occur in the dam, the resultant should lie inside the one third of dam from the toe.

※ Stress Analysis in Concrete Gravity Dam:-

Let us consider the base section of dam. Let, H & V be the components of resultant forces acting along horizontal & vertical direction. 'e' is the eccentricity.

Assuming unit width, vertical stress $(\sigma_v) = \frac{\Sigma V}{B \times 1} = \frac{\Sigma V}{B}$

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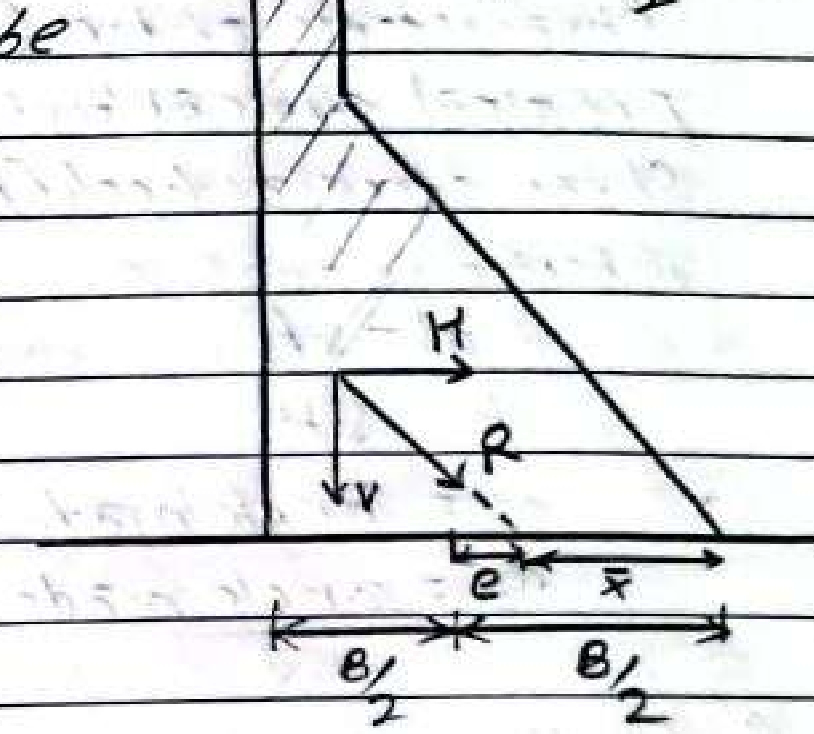
Due to eccentricity of resultant force, bending moment will be developed at the section.

$$(e = B/2 - \bar{x})$$



$M = \Sigma V \times e$

Then, bending stress will be,

$$\sigma_b = \frac{My}{I}$$

$$\sigma_b = \frac{(\Sigma V \times e)\ (\pm B/2)}{1 \times B^3/12}$$

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

Total stress $(\sigma) = \sigma_v + \sigma_b$

$$\sigma = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{\Sigma V}{B}\left(1 \pm \frac{e}{B/6}\right) \quad \text{---(1)}$$

→ When reservoir is full, maximum stress occurs at toe.

$$\sigma_{toe} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right), \quad \sigma_{heel} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right)$$

→ When the reservoir is empty, maximum stress occurs at heel.

$$\sigma_{toe} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right), \quad \sigma_{heel} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right)$$

Analyzing eqn (i), the stress becomes negative (tensile) as long as e > B/6. Thus, for no tension to develop in the dam eccentricity 'e' should be less than or equal to B/6.

→ At limiting case the position of resultant from the toe is equal to B/3. It means for no tensile force to occur in dam, the resultant should lie inside one third of its base from toe. This rule is called "Middle Third Rule".

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→ Principal stress at toe $(P_{max}) = \sigma_{toe}\sec^2\theta - p'\tan^2\theta$

Shear stress at toe $(P_t) = (\sigma_{toe} - p')\tan\theta$

principal stress at heel $(P_{max}) = \sigma_{heel}\sec^2\alpha - p\tan^2\alpha$

Shear stress at heel $(P_t) = (\sigma_{heel} - p)\tan\alpha$

where,

$p' = \gamma h$

$p = \gamma H$

α = angle made by ~~down~~ upstream slope with vertical

θ = angle made by downstream slope with vertical

Q For the following non-over flow section of the dam.

a) Calculate the forces acting on the dam

b) The point of application of all above forces from the toe.

c) Check the stability against

• sliding, • overturning • Development of tension • Crushing failure

→ Unit weight of material $(\gamma) = 25\ kN/m^3$

$(\gamma_{sat})_{sediment} = 18\ kN/m^3$, $\phi = 35°$ & $K = 0.8$

$\mu = 0.6$, Permissible stress in foundation $= 5000\ kN/m^2$

Permissible stress in dam material $= 10{,}000\ kN/m^2$

→ Solution

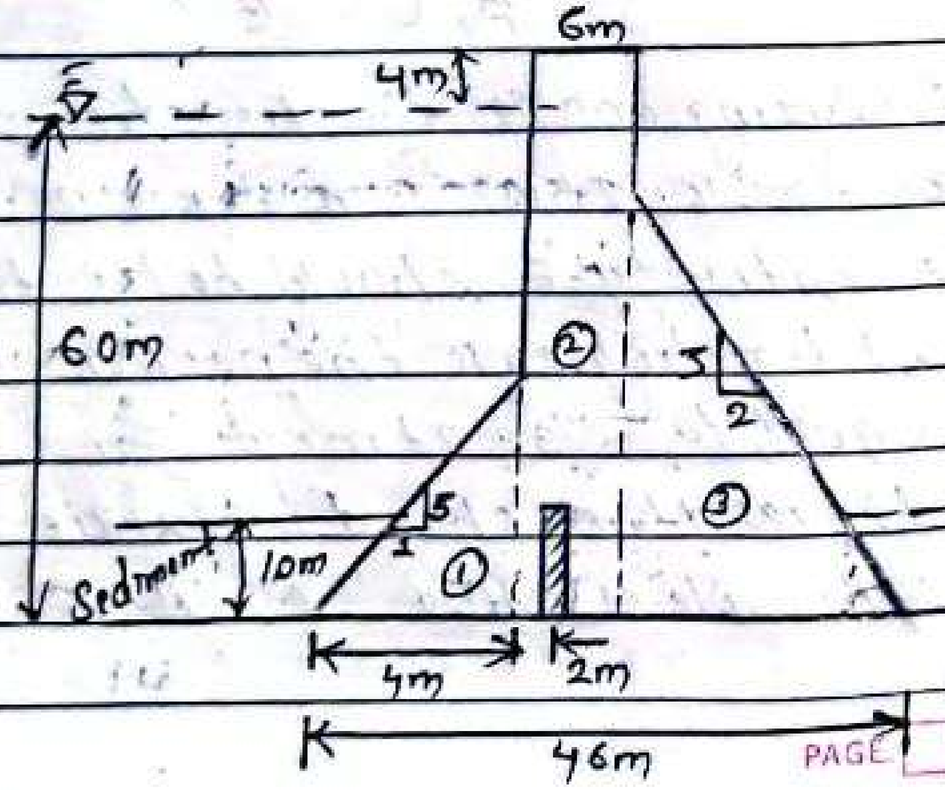
(a) Self weight

$w_1 = \gamma_c \times A_1$

$= 25 \times \frac{1}{2} \times 4 \times 20$

$= 1000\ kN$

All other calculations are done in table.



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Load	Description	Force calculation	Forces (kN) $H_z(+\rightarrow)$	$V_t(+\downarrow)$	Distance from toe	Moments (kNm) $+\circlearrowright$	$+\circlearrowleft$
self wt	W_1	$25 \times 0.5 \times 4 \times 20$		1000	43.33	43333.3	
	W_2	$25 \times 6 \times 64$		9600	39	374400	
	W_3	$25 \times 0.5 \times 36 \times 54$		24300	24	583200	
water press^r	$(P_H)_{u/s}$	$0.5 \times 9.81 \times 60^2$	17658		20		353160
	$(P_H)_{d/s}$	$0.5 \times 9.81 \times 6^2$	176.58		2	353.16	
	$(P_V)_{u/s}$	$9.81 \times [0.5(40+60) \times 4]$		1962	$\frac{b}{3}\left[\frac{a+2b}{a+b}\right] = 44.13$	86589.16	
	$(P_V)_{d/s}$	$9.81 \times 0.5 \times 6 \times 4$		117.72	1.33	156.96	
Uplift	U_1	$0.5(470.88 + 188.35) \times 6$		−1977.69	43.42		85888.51
	U_2	$0.5(188.35 + 47.088) \times 40$		−4708.8			113011.2
Sediment	P_s	$0.5 \times 0.2709 \times 8 \times 10^2$	110.97		3.33		369.9
		sum	17592.39	30293.22		1088033.1	552429.6

Due to gallery, uplift will decrease.

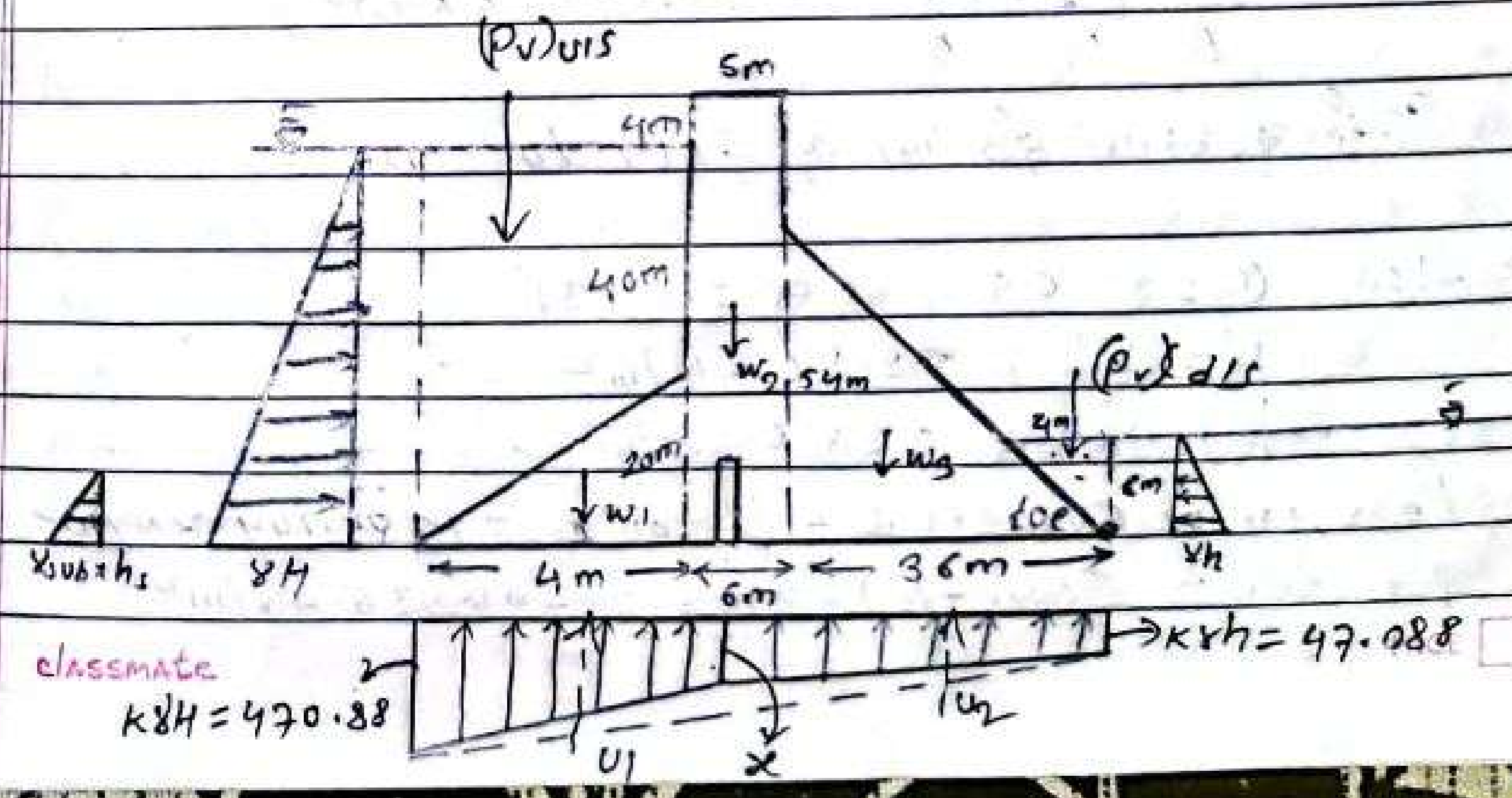
$x = K\gamma\left(h + \frac{H-h}{3}\right)$

$= 0.8 \times 9.81 \times \left(6 + \frac{60-6}{3}\right)$

$= 188.35$

$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.2709$ ($\phi = 35°$)

$\gamma_{sub} = \gamma_{sat} - \gamma_w = 8\ kN/m^3$



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From table,

$\Sigma H = 17592.39\ kN$

$\Sigma V = 30293.224\ kN$

$+\circlearrowright \Sigma M = 1088033.1\ kNm$

$+\circlearrowleft \Sigma M = 552429.6$

→ FOS against overturning $= \dfrac{1088033.1}{552429.6} = 1.97 > 1.5$ (safe)

→ FOS against sliding $= \dfrac{\mu \Sigma V}{\Sigma H} = 1.03 > 1$ (safe)

→ Development of tension,

$B = 46m$

$B/6 = 46/6 = 7.67m$

$\bar{x} = \dfrac{\Sigma M}{\Sigma V} = 17.67m$

$e = B/2 - \bar{x} = 5.32m$

Since $e < B/6$, so, no tension is developed in dam.

→ Crushing failure,

$\sigma_{toe} = \dfrac{\Sigma V}{B}\left(1 + \dfrac{6e}{B}\right) = 1115.52\ kN/m^2$

$\sigma_{heel} = \dfrac{\Sigma V}{B}\left(1 - \dfrac{6e}{B}\right) = 201.57\ kN/m^2$

$< (\sigma_{per})$ foundation

Safe against failure of foundation.

→ Also, $\theta = 33.69°$, $\alpha = 11.31°$

$p' = \gamma h = 58.86\ kN/m^2$

$p = \gamma H = 588.6\ kN/m^2$

$P_{max,toe} = \sigma_{toe} \sec^2\theta - p' \tan^2\theta = 1585.144\ kN/m^2$

$P_{t,toe} = (\sigma_{toe} - p')\tan\theta = 704.438\ kN/m^2$

$P_{max,heel} = \sigma_{heel} \sec^2\alpha - p \tan^2\alpha = 186.08\ kN/m^2$

$P_{t,heel} = (\sigma_{heel} - p)\tan\alpha = -77.406\ kN/m^2$

But,

(σ_{per}) material $= 10,000\ kN/m^2$

Since, principal & shear stress at toe & heel are less than (σ_{per}) material, dam is safe against compression failure of material.

Elementary profile & Practical profile of dam.

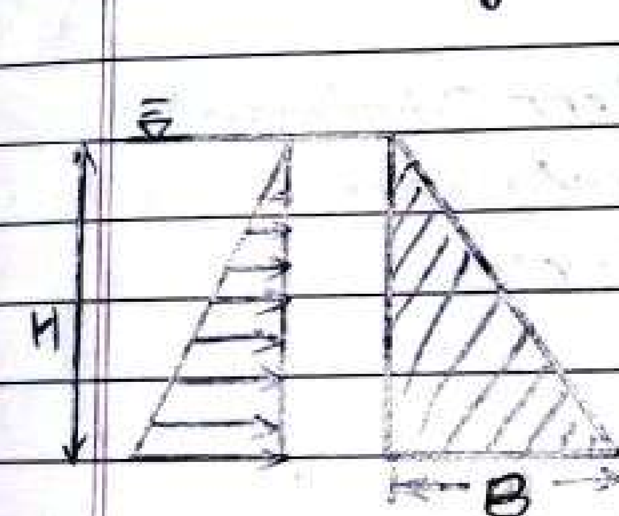


Fig: Elementary profile of dam

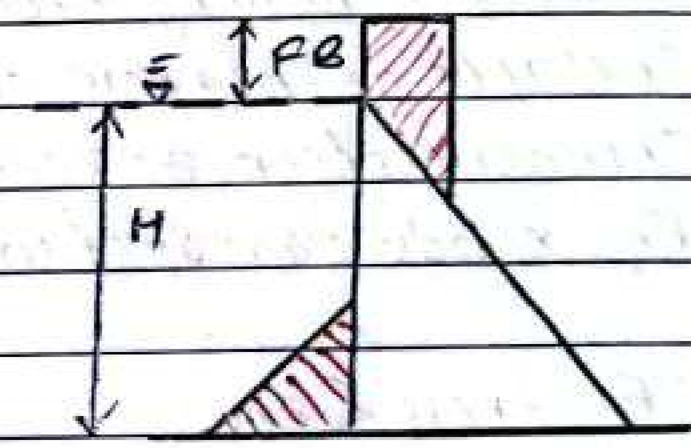


Fig: practical profile of dam.

→ Basewidth of elementary profile

(i) Stress basis, $B = \dfrac{H}{\sqrt{G-K}}$

(ii) Stability on sliding basis, $B = \dfrac{H}{\mu(G-k)}$

→ Limiting height of dam,

$$H = \frac{P_{max}}{\gamma_w (G-k+1)}$$

where, G = Specific gravity of dam material

k = Seepage coeff.

μ = Coeff. of friction betn dam & foundation

P_{max} = Maxm allowable stress on dam.

* Principal Variant of modern concrete dams

(a) Gravity dam:
→ Resist pressure & other forces due to its own weight
→ Usually made up of cement concrete & straight in plan.
→ Approximate triangular in cross section with its apex at top
→ Ex: kaligandaki A (43m), chameliya (54m), Middle Marsyangdi HEP (52m)

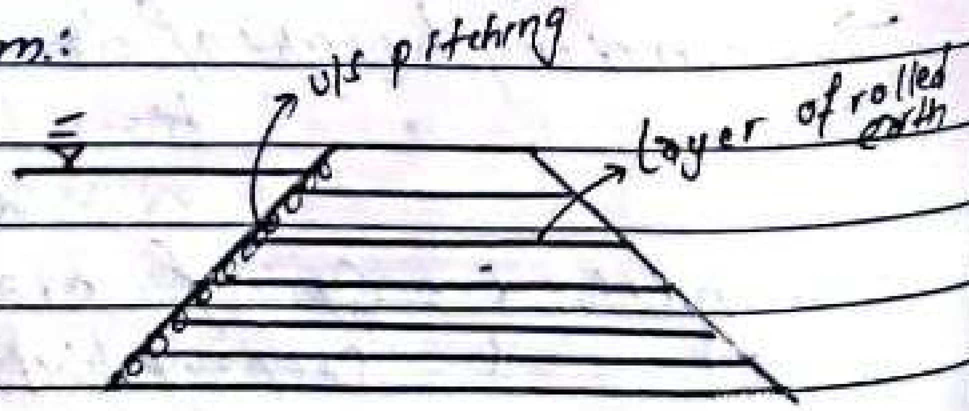
(b) Arch Dam:
→ It is curved in plan with its convexity towards U/s side.
→ Transfer load by arch action.
→ Suitable in narrow canyon with steep abutment
→ Thinner section as compared to gravity dam.
→ Eg: xiludu dam of china, Hoover dam etc.

(c) Buttress Dam:
→ It consists of slab with buttress
→ Transfer load from slab to foundation through buttress.
→ Prevents overturning & sliding failure of dam
→ Eg: Roseland dam (france), mount dell dam (USA)

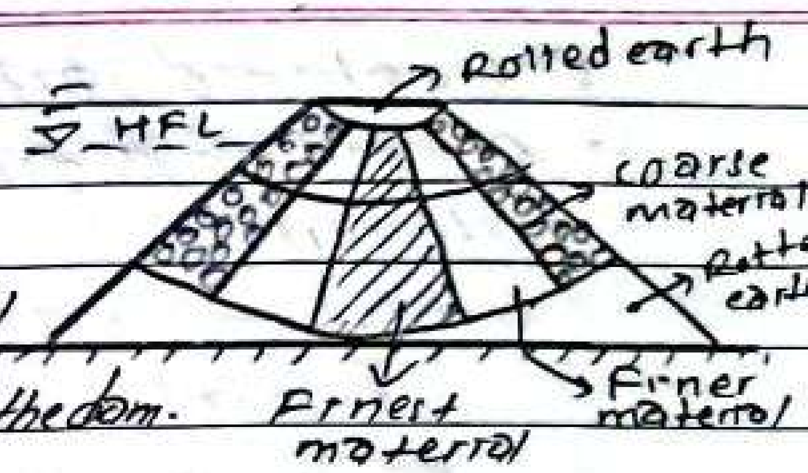
* Principal Variant of Earthen dam:

(a) Based on method of construction:

1) Rolled fill earth dam:
→ Earth dam constructed by compacting earth adequately using sheep foot roller.



2) Hydraulic fill earth dam:
→ Construction by hydraulic means
→ The mixture of excavated materials with water is pumped & discharge at the outer edges of the dam.
→ No compaction is required.



(b) Based on section of dam:

1) Homogeneous Earth Dam:
→ Dam made of single material
→ U/s slopes are kept flat to reduce path of seeping water.

2) Zoned Earth Dam
→ Made using more than one material
→ Central part of dam (core) is made from impervious material


Homogeneous earth dam


Zoned earth dam

(c) Based on material

1) Rock filled dam:
→ This dam is made predominantly from rock boulders.
→ Some impermeable layer is made on U/s face of dam.

2) Earth fill dam
→ Soil content > 50% of volume of material.
→ Consists of rock toe or horizontal drainage blankets.

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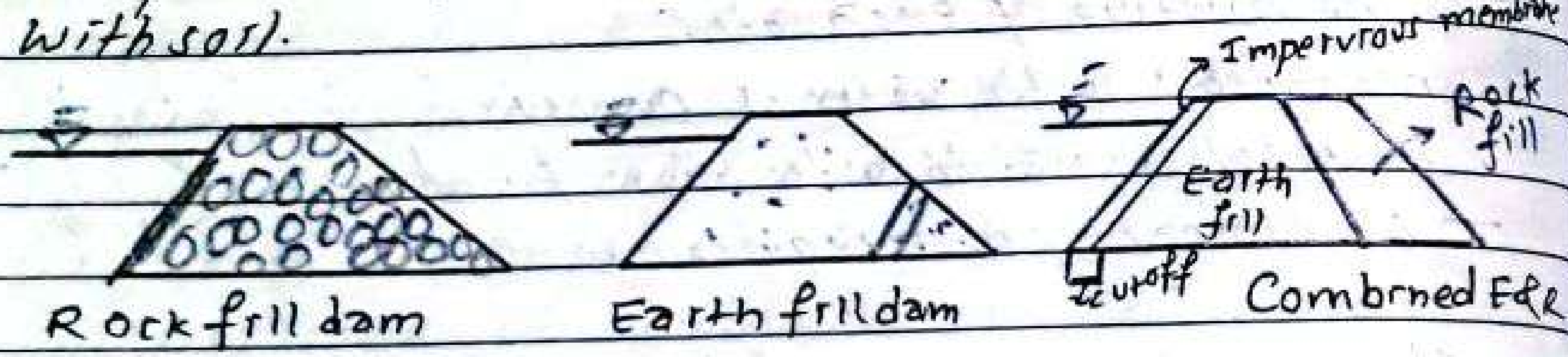
3) Combined earth fill & rock fill dam
→ D/s portion is filled with rock & u/s portion is filled with soil.



※ Modes of failure of embankment dam:-

(a) Structural failure
1) Failure due to pore pressure
→ In impervious compressive soil, drainage is extremely low which causes high ~~wat~~ pore water pressure resulting failure of dam.

2) Sudden drawdown ~~of~~ on upstream face.
→ when reservoir is suddenly emptied, the pressure due to water suddenly vanishes from water face.

3) Slope protection failures
→ Due to repetitive striking of water waves to the protection.

4) Downstream slope failure by sliding
5) Failure by foundation settlement
6) Failures due to holes by burrowing animals
7) Failure due to ~~[illegible]~~ earthquake
8) Piping of seepage erosion through foundation

(b) Hydraulic failure
1) Overtopping: Once the earthen dam is overtopped, there is max^m chance of destruction of whole dam. Overtopping is due to underestimate of flood during design.

2) Wave Erosion: If the u/s toe is not properly protected by riprap, the wave erosion will occur.

3) Toe Erosion: Erosion of toe due to water on d/s side of dam. It can be prevented by providing a thick riprap on d/s side

4) Gullying: Occurs due to heavy rainfall. Good system of drainage from d/s side helps to great deal in preventing this failure.

(c) Seepage failure
1) piping failure:
If the exit gradient of seepage flow is more than the critical gradient of the soil, the soil particles are susceptible to dislocation. This process is accelerated towards d/s direction. As a result, all the particles are removed & hollow pipe is formed. The failure of dam due to this piping action is called piping failure. It may occur in dam body or in foundation.

2) Sloughing: If the seepage line exists at the d/s face of the dam, the portion of the toe of dam below the exit point will always be saturated. It will cause the reduction

of the stability of slope & repeated wetting & sliding may lead to failure called sloughing.

※ Foundation Treatment, Grouting & its necessity:-

→ The foundation of gravity dam should be hard, strong, durable & impervious.

→ Imperviousness of foundation is very important as uplift pressure depends greatly upon seepage.

→ All the loose overlying soil from the site is removed & 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 dam over its foundation.

→ A layer of rich cement mortar should be laid on excavated rocky foundation before concreting.

→ All the faults, seams, cavernous rocks & crushed zones should be either corrected or removed from site.

→ In order to prevent seepage, trench may be excavated near the heel of dam & filled with cement concrete. This process is known as grouting.

※ Design of Embankment (Earthen) Dams

* Design consideration of embankment dam:-

1) Sufficient free board & spillway should be provided to prevent overtopping.
2) Seepage line should remains burried sufficiently on the d/s face when reservoir is full.
3) u/s face & d/s face should be protected against wave action.

4) The u/s & d/s slopes should not damage by pore pressure.
5) U/s & d/s slope should be flat to ensure safety of dam against shear failure of foundation.
6) In areas, frequently subjected to earthquakes, the dam should be designed considering earthquake resistant measures.

※ Pheratic Line:-

→ Seepage line or pheratic line is a line in the body of dam below which seepage takes place is called pheratic line.

→ This line separates the saturated & unsaturated soil mass.

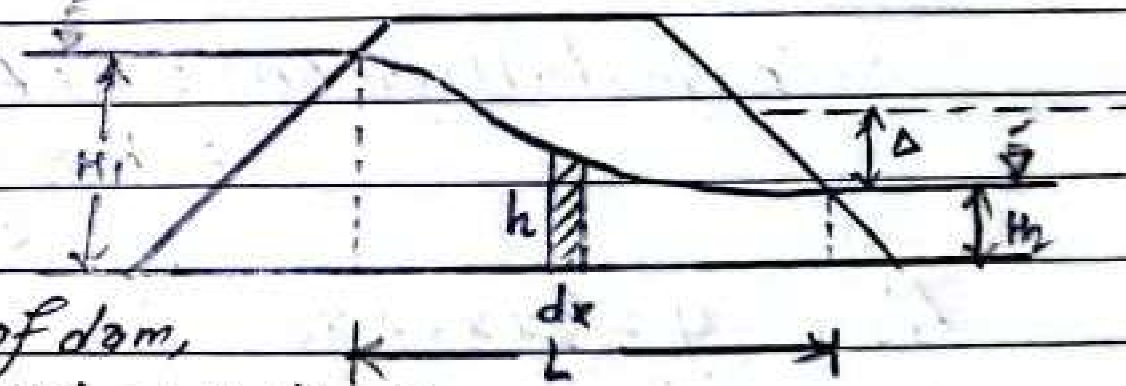
→ Above pheratic line, there is zone of capillary saturation.

* Seepage discharge through a dam with impervious foundation

From Darcy's law,

$Q = kiA$

$q = Q/B$



Considering unit width of dam,

Specific seepage discharge $(q) = \frac{k i \times (h \times 1)}{1}$

$$q = k \times \left(\frac{-dh}{dx}\right)(h \times 1)$$

$$q\,dx = -k h\,dh$$

$$q\int_{x_1}^{x_2} dx = -k\int_{H_1}^{H_2} h\,dH$$

$$\Rightarrow q(x_2 - x_1) = -\frac{k}{2} \times [H_2^2 - H_1^2]$$

$$\Rightarrow q \times L = -\frac{k}{2}(H_2^2 - H_1^2) \Rightarrow q = \frac{k}{2L}(H_1^2 - H_2^2)$$

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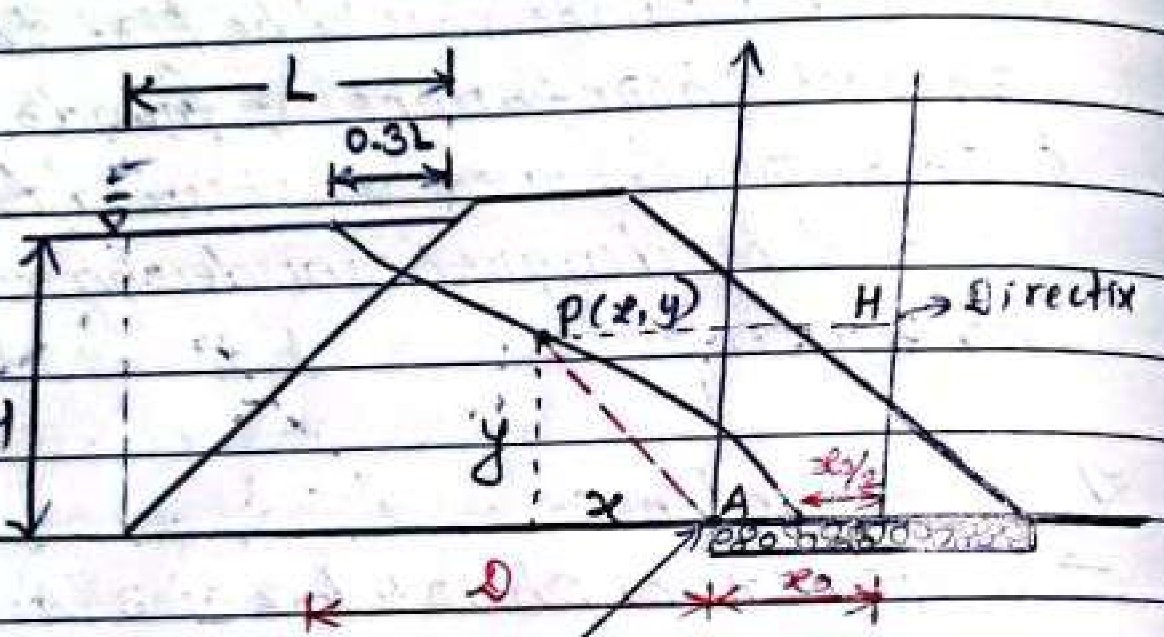
※ Seepage of pheratic line in an earth dam having a horizontal filter:

→ This is graphical method suggested by Casagrande.



For parabola

$PA = PH$

$\sqrt{x^2+y^2} = x + x_0$ ---(1)

$x^2+y^2 = x^2+2xx_0+x_0^2 \Rightarrow y^2 = 2xx_0 + x_0^2$

From boundary condition, when $x = D$, $y = H$

$x_0 = \sqrt{D^2+H^2} - D$ [from (1)]

Now,

Seepage discharge $(q) = kiA$

$= k \times \frac{dy}{dx} \times (y \times 1)$ ---(ii)

Also, $y = (2xx_0 + x_0^2)^{1/2}$

$\frac{dy}{dx} = \frac{2x_0}{2(2xx_0+x_0^2)^{1/2}} = \frac{x_0}{(2xx_0+x_0^2)^{1/2}}$

From (ii)

$q = k \times \frac{x_0}{(2xx_0+x_0^2)^{1/2}} \times (2xx_0+x_0)^{1/2}$

$q = kx_0$

where,

$x_0 = \sqrt{D^2+H^2} - D$.

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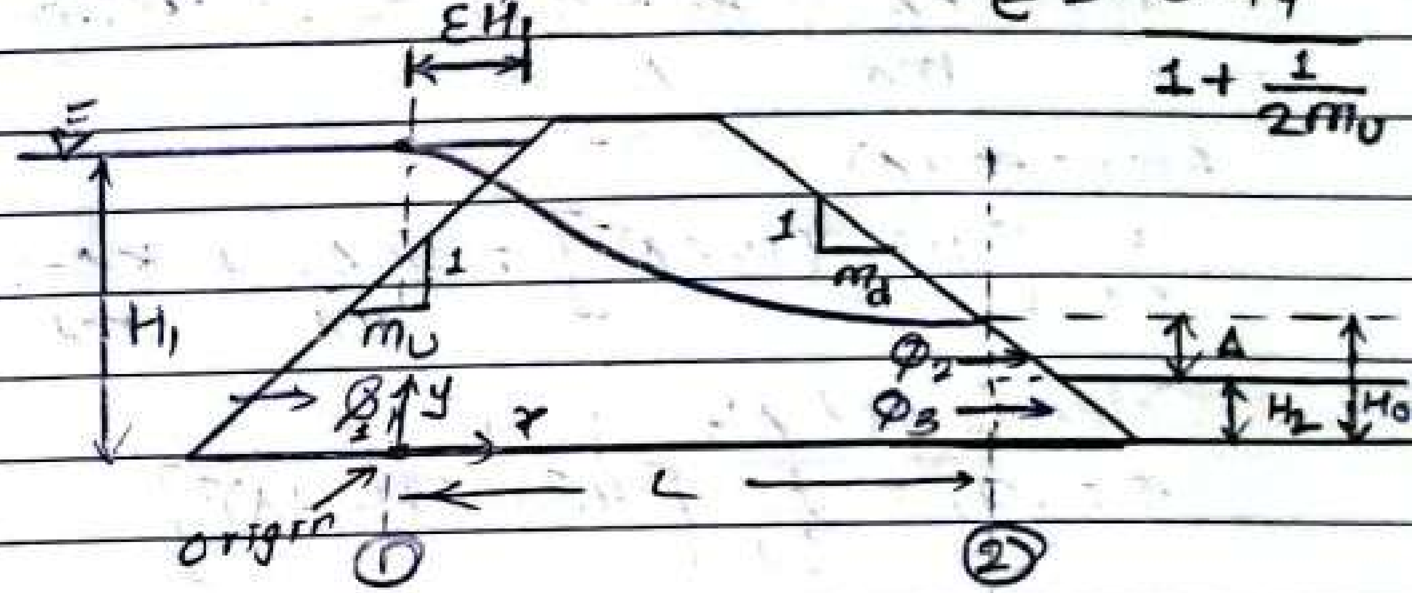
※ Seepage discharge through embankment dam without water in downstream.

$\epsilon = \frac{0.44}{1+\frac{1}{2m_u}}$

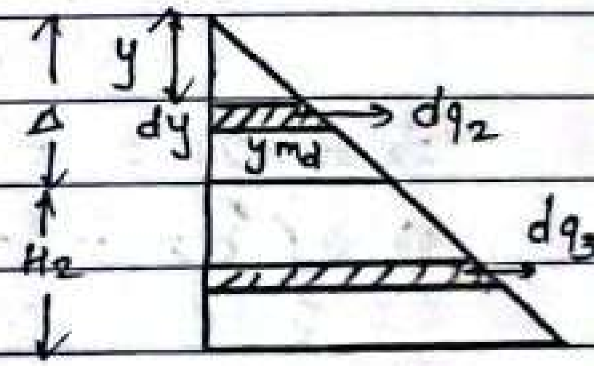
$q_1 = q_2 + q_3$ --- (A)



From fig,

$q_1 = \frac{k}{2L}(H_1^2 - H_0^2) = \frac{k}{2L}\left[H_1^2 - (H_2+\Delta)^2\right]$ ----(1)

From fig(2),



$dq_2 = kiA$

$dq_2 = k\left(\frac{y}{y m_d}\right) \times (dy \times 1)$

$\int dq_2 = \int_0^{\Delta} \frac{k\,dy}{m_d}$

$q_2 = \frac{k\Delta}{m_d}$ -----(ii)

Also,

$dq_3 = kiA = k \times \left[\frac{y-(y-\Delta)}{y m_d}\right](dy \times 1) = \frac{k\Delta}{m_d} \times \frac{dy}{y}$

$q_3 = \int dq_3 = \frac{k\Delta}{m_d}\int_{\Delta}^{H_0} \frac{dy}{y}$

$q_3 = \frac{k\Delta}{m_d}(\ln y)_{\Delta}^{H_0}$

$\Rightarrow q_3 = \frac{k\Delta}{m_d}\left[\ln H_0 - \ln \Delta\right]$.

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$$q_3 = \frac{k\Delta}{m_d}\ln\left(\frac{H_0}{\Delta}\right)$$

$$q_3 = \frac{k\Delta}{m_d}\left(\frac{H_2+\Delta}{\Delta}\right) \quad \text{--- (iii)}$$

From eqn (A)

$$\frac{k}{2L}\left[H_1^2-(H_2+\Delta)^2\right] = \frac{k\Delta}{m_d} + \frac{k\Delta}{m_d}\ln\left(\frac{H_2+\Delta}{\Delta}\right)$$

Solve to get 'Δ'.

$$q = q_1 = \frac{k}{2L}\left[H_1^2-(H_2+\Delta)^2\right]$$

For general eqn of pheratic line,

$$q = kiA$$

$$\Rightarrow q = k\left(-\frac{dy}{dx}\right)\times(y\times 1)$$

$$\Rightarrow q\,dx = -ky\,dy$$

$$\Rightarrow q\int_0^x dx = -k\int_{H_1}^{y} y\,dy$$

$$\Rightarrow qx = -\frac{k}{2}(y^2-H_1^2)$$

$$\Rightarrow y^2 = -\frac{2qx}{k} + H_1^2 \quad \text{--- (iv)}$$

→ If no-tail water, $H_2 = 0$

$$\frac{k}{2L}\left[H_1^2-\Delta^2\right] = \frac{k\Delta}{m_d} + \frac{k\Delta}{m_d}\ln\left(\frac{0+\Delta}{\Delta}\right)$$

$$\frac{k}{2L}(H_1^2-\Delta^2) = \frac{k\Delta}{m_d}$$

Solve above eqn & find Δ. Then find q & using eqn (iv) draw pheratic line.

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Q. A homogeneous earthen dam has the following data

permeability (k) = 5×10^{-6} m/s, Top width = 4.5m

Height of dam = 22m, Free board = 2.5m

U/s slope = 3:1, D/s slope = 2:1

Draw pheratic line & calculate seepage discharge if

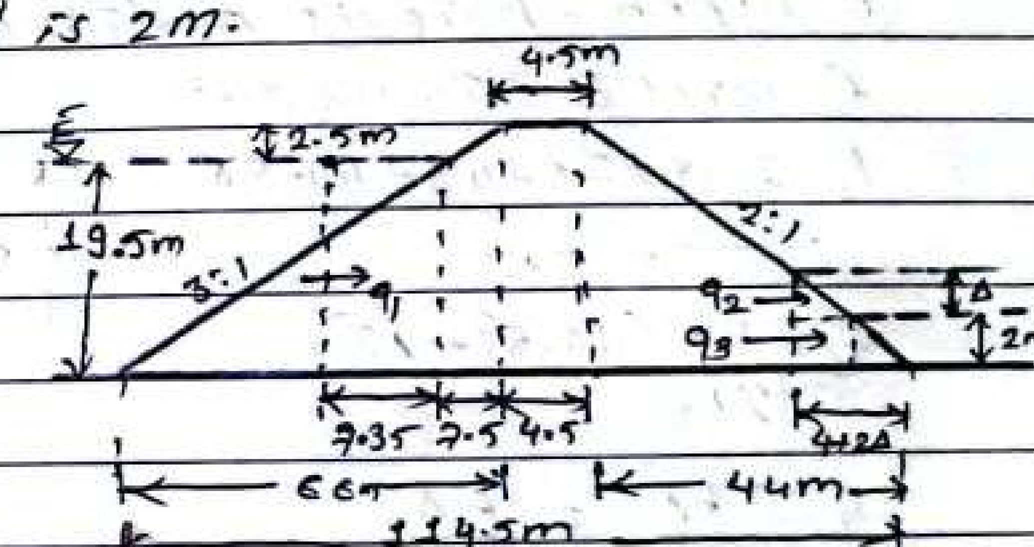
a) Tail water level is 2m

b) There is no tail water level

c) There is horizontal filter of length 25m provided inward from the downstream toe.

→

(a) When tail water level is 2m.



$$\varepsilon = \frac{0.44}{1+\frac{1}{2m_u}} = \frac{0.44}{1+\frac{1}{2\times3}}$$

$$\varepsilon = 0.377$$

$$\varepsilon H_1 = 0.377\times19.5 = 7.35\text{m}$$

Here,

$$q_1 = \frac{k\times[19.5^2-(2+\Delta)^2]}{2(59.354-2\Delta)} \quad \text{--- (i)}$$

$$L = 4.5+44-(4+2\Delta)+7.35+7.5 = 59.354-2\Delta$$

$$q_2 = \frac{k\Delta}{m_d} = \frac{k\Delta}{2} \quad \text{--- (ii)}$$

$$q_3 = \frac{k\Delta}{m_d}\ln\left(\frac{H_2+\Delta}{\Delta}\right) = \frac{k\Delta}{2}\ln\left(\frac{2+\Delta}{\Delta}\right)$$

Now, $q_1 = q_2 + q_3$

$$\frac{k[19.5^2-(2+\Delta)^2]}{2(59.354-2\Delta)} = \frac{k\Delta}{2} + \frac{k\Delta}{2}\ln\left(\frac{2+\Delta}{\Delta}\right)$$

$$\Delta = 5.027\text{m}$$

$$q = q_1 = \frac{5\times10^{-6}}{2(52.354-2\times5.027)}\left[19.5^2-(2+5.027)^2\right] = 1.677\times10^{-5}\ \text{m}^3/\text{s}$$

General eqn, $y^2 = \frac{-2qx}{k} + H_1^2$

$$y^2 = \frac{-2 \times 1.677 \times 10^{-5} \times x}{5 \times 10^{-6}} + 19.5^2$$

$$y = \sqrt{-6.71x + 380.25}$$

x varres from 0 to $59.35 - 2 \times 5.027 = 49.299$ m.

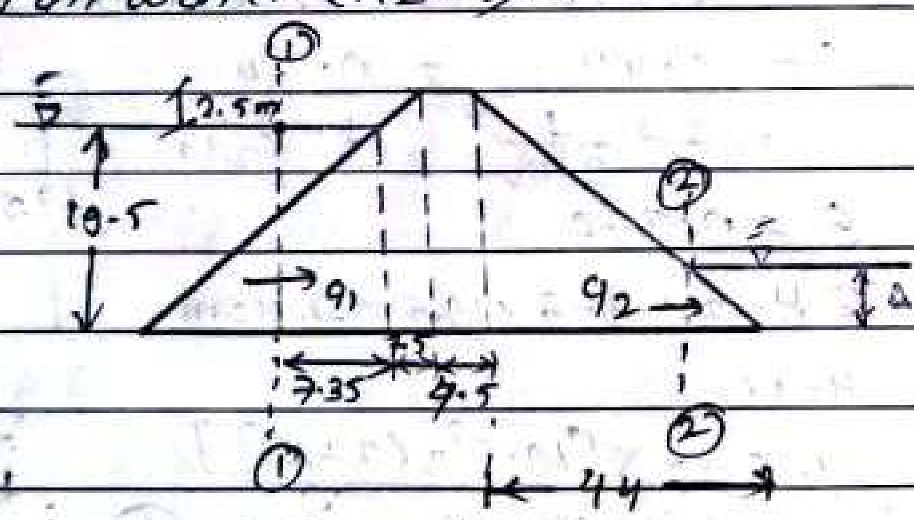
x	0	5	10	15	20	25	30	35	40	45	49.29
y	19.5	18.02	17.7	16.23	15.7	14.6	13.4	12.1	10.6	8.9	7.13

(b) When there is no tail water ($H_2 = 0$)

Distace betn ①-① & ②-②

$L = 7.35 + 7.5 + 4.5 + 44 - 2\Delta$

$= 63.35 - 2\Delta$



$$q_1 = \frac{k}{2L_0}(19.5^2 - \Delta^2)$$

$$q_2 = \frac{k\Delta}{m_d} = \frac{k\Delta}{2}$$

Now; $q_1 = q_2 \Rightarrow \frac{k}{2(63.35 - 2\Delta)} \times (19.5^2 - \Delta^2) = \frac{k\Delta}{2}$

$\Rightarrow \Delta = 6.71$ m

Seepage discharge $(q) = q_1 = q_2 = \frac{k\Delta}{2} = \frac{5 \times 10^{-6} \times 6.71}{2}$

$q = 1.677 \times 10^{-5}$ m³/s/m

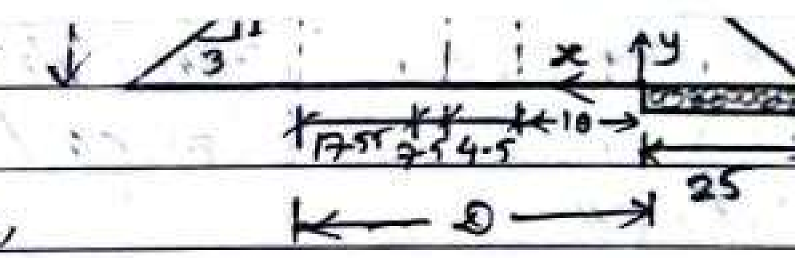
General eqn,

$$y^2 = \frac{-2qx}{k} + H_1^2$$

$$y = \sqrt{-6.708x + 380.25}$$

$= \sqrt{48.55^2 + 19.5^2} - 48.55$

$= 3.77$ m



General eqn of parabola,

$$y = \sqrt{2xx_0 + x_0^2}$$

$$y = \sqrt{7.54x + 14.21}$$

x varres from 0 to 48.55m

Also, seepage discharge $(q) = kx_0$

$= 5 \times 10^{-6} \times 3.77$

$= 1.885 \times 10^{-5}$ m³/s/m

x	0	5	10	15	20	25	30	35	40	45	48.55
y	3.77	7.2	9.46	11.28	12.84	14.23	15.5	16.67	17.77	18.8	19.5

→ Pheratic lines of all above three cases are shown in graph.

※ Seepage Control measures in Earthen Dam:-

(a) Seepage Control through dam

→ Rock toe: Lowers the pheratic line such that it lies well within dam section.

→ Horizontal drainage filter: provided on base of dam, starting from d/s end & extending backwards in the dam.

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General eq^n, $y^2 = \frac{-2qx}{k} + H_1^2$

$$y^2 = \frac{-2 \times 1.677 \times 10^{-5} \times x}{5 \times 10^{-6}} + 19.5^2$$

$$y = \sqrt{-6.71x + 380.25}$$

x varies from 0 to $59.35 - 2 \times 5.027 = 49.299$ m.

x	0	5	10	15	20	25	30	35	40	45	49.29
y	19.5	18.62	17.7	16.73	15.7	14.6	13.4	12.1	10.6	8.9	7.13

(b) When there is no tail water ($H_2 = 0$)

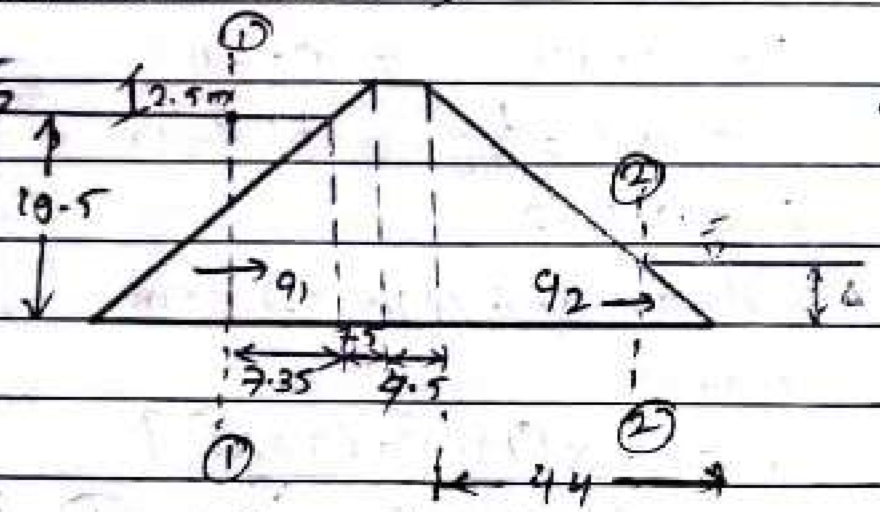
Distance betn ①-① & ②-②

$L_0 = 7.35 + 7.5 + 4.5 + 44 - 2\Delta$

$= 63.35 - 2\Delta$

$q_1 = \frac{k}{2L_0}(19.5^2 - \Delta^2)$

$q_2 = \frac{k\Delta}{m_d} = \frac{k\Delta}{2}$



Now; $q_1 = q_2 \Rightarrow \frac{k}{2(63.35 - 2\Delta)} \times (19.5^2 - \Delta^2) = \frac{k\Delta}{2}$

$\Rightarrow \Delta = 6.71$ m

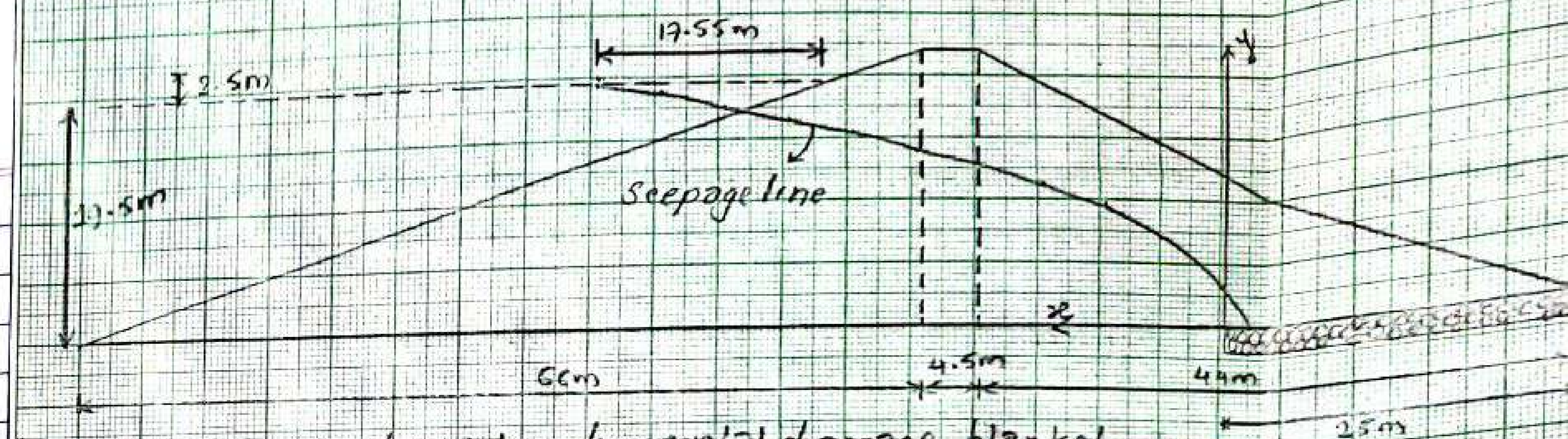
Seepage discharge $(q) = q_1 = q_2 = \frac{k\Delta}{2} = \frac{5 \times 10^{-6} \times 6.71}{2}$

$$q = 1.677 \times 10^{-5} \ m^3/s/m$$

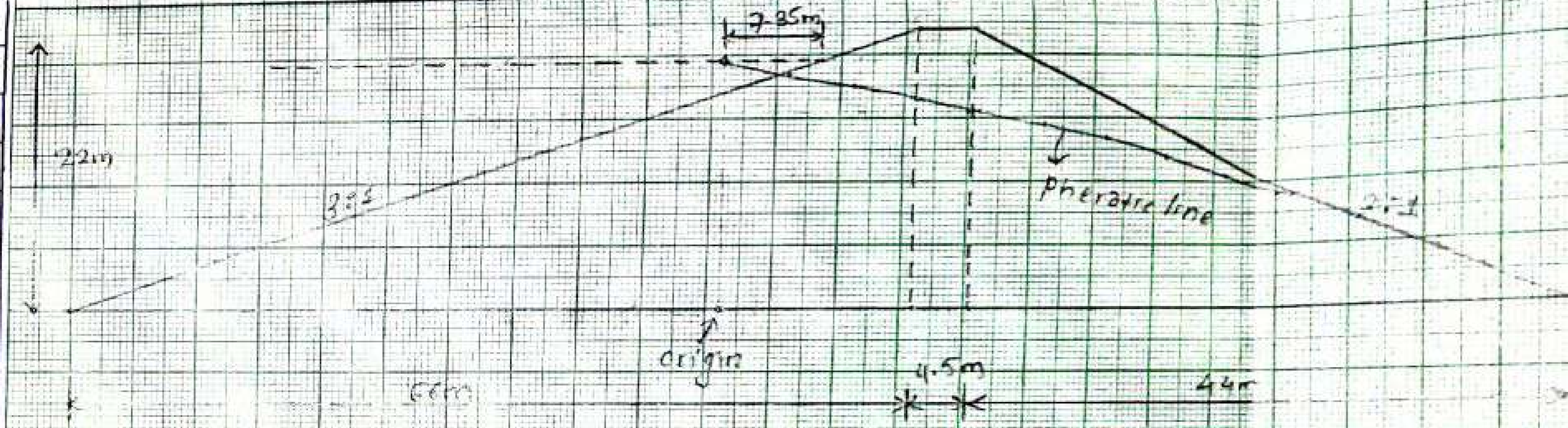
General eq^n,

$$y^2 = \frac{-2qx}{k} + H_1^2$$

$$y = \sqrt{-6.708x + 380.25}$$



(c) Seepage line when horizontal drainage blanket is provided on d/s of dam



(a) Phreatic line when tail water level is 2m

Scale:
2 small box = 1m

→ Pheratic lines of all above three cases are shown in graph.

Seepage Control measures in Earthen Dam:-

(a) Seepage Control through dam

→ Rock toe: lowers the pheratic line such that it lies well within dam section.

→ Horizontal drainage filter: provided on base of dam, starting from d/s end & extending backwards in the dam.

General eqn, $y^2 = \frac{-2qx}{k} + H_1^2$

$$y^2 = \frac{-2\times 1.677\times 10^{-5}\times x}{5\times 10^{-6}} + 19.5^2$$

$$y = \sqrt{-6.71x + 380.25}$$

x varres from 0 to $59.35 - 2\times 5.027 = 49.299$ m.

x	0	5	10	15	20	25	30	35	40	45	49.29
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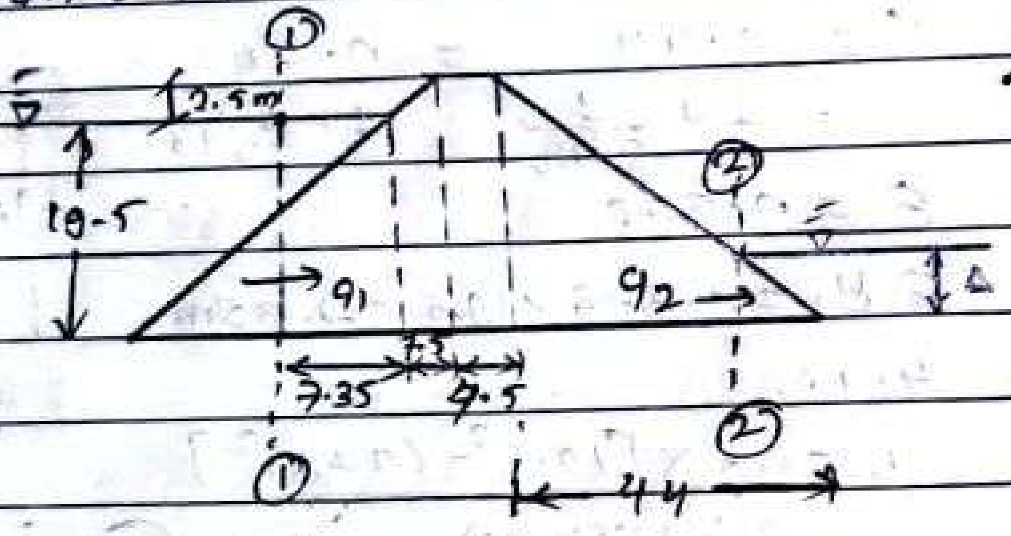
(b) When there is no tail water ($H_2 = 0$)



Distace betn ①-① & ②-②

$L = 7.35 + 7.5 + 4.5 + 44 - 2\Delta$

$= 63.35 - 2\Delta$

$q_1 = \frac{k}{2L_0}(19.5^2 - \Delta^2)$

$q_2 = \frac{k\Delta}{m_d} = \frac{k\Delta}{2}$

Now; $q_1 = q_2 \Rightarrow \frac{k}{2(63.35 - 2\Delta)} \times (19.5^2 - \Delta^2) = \frac{k\Delta}{2}$

$\Rightarrow \Delta = 6.71$ m

Seepage discharge (q) $= q_1 = q_2 = \frac{k\Delta}{2} = \frac{5\times 10^{-6}\times 6.71}{2}$

$q = 1.677\times 10^{-5}$ m^3/s/m

General eqn,

$$y^2 = \frac{-2qx}{k} + H_1^2$$

$$y = \sqrt{-6.708x + 380.25}$$

x	0	5	10	15	20	25	30	35	40	45	49.93
y	19.5	18.62	17.7	16.72	15.68	14.57	13.4	12.06	10.58	8.85	6.67

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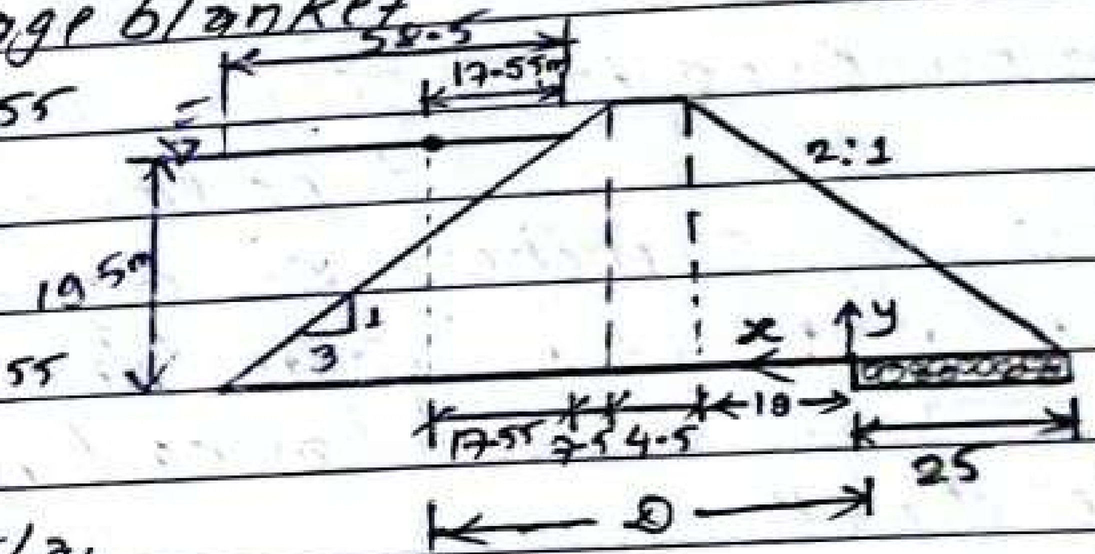
(c) Horizontal drainage blanket



$D = 19 + 4.5 + 7.5 + 17.55$

$= 48.55$ m

$x_0 = \sqrt{D^2 + H^2} - D$

$= \sqrt{48.55^2 + 19.5^2} - 48.55$

$= 3.77$ m

General eqn of parabola,

$y = \sqrt{2x x_0 + x_0^2}$

$y = \sqrt{7.54x + 14.21}$

x varres from 0 to 48.55 m

Also, seepage discharge (q) $= k x_0$

$= 5\times 10^{-6}\times 3.77$

$= 1.885\times 10^{-5}$ m^3/s/m

x	0	5	10	15	20	25	30	35	40	45	48.55
y	3.77	7.2	9.46	11.28	12.84	14.23	15.5	16.67	17.77	18.8	19.5

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※ Seepage Control measures in Earthen Dam:-

(a) Seepage Control through dam

→ Rock toe: Lowers the pheratic line such that it lies well within dam section.

→ Horizontal drainage filter: provided on base of dam, starting from d/s end & extending backwards in the dam.

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→ Chimney drains:- chimney drains can effectively intercept seepage from the dam regardless of the stratification.

(b) Seepage Control through foundation:-

→ Impervious cutoff: wall of relatively impervious material to prevent seepage through the foundation.

→ Drainage Trenches: Drainage trenches are provided along the axis of dam. They have a porous drain having longitudinal slope.

→ Relief well

→ Upstream impervious blanket

※ Reservoir Sedimentation Issues & Management

→ The sediments eroded from the catchment after transportation through river reach on approaching reservoir finally get collected due to the reduction of velocity which is known as reservoir sedimentation. It is biggest challenge in reservoir operation & its useful life.

→ The extent of silt load depends on

- Nature of soil of watershed.
- Topography of the watershed
- Vegetation cover of the watershed
- Intensity of rainfall
- Soil conservation & watershed management method adopted.

→ Effect of sediment on reservoir function:

- Loss of storage & service
- Sediment deposition at outlet gate
- Aggradation of the reservoir
- Degradation downstream of the dam.

→ Sedimentation Management

- Proper selection of reservoir site
- Control of sediment inflow by constructing small check dams upstream
- Vegetation cover & afforestration
- Soil conservation methods
- Constructing dams in stages
- Removal of deposited sediment by hydraulic or mechanical means.
- Removal by sluices during flood time.

Sumit Paudel

Chapter-5, Run of River types of Hydropower Projects

Outline:

→ General layout of components in a typical power plant: dam body, weir, spillway, undersluices & intakes with examples.

→ Different types of intakes, importance, location & types, design concept of intake structure, head loss calculation in intake structure (trash rack losses)

→ performance standards of headworks

→ Sediment handling measures

→ Flushing & Settling basin:

✗ Layout of Components of ROR Project:-

(a) Diversion weir:- Structure that is placed across the river to divert a part of flow to the conveyance system through the intake.

(b) Intake:- It is an opening to draw design flow from the river. It is provided with control gate to regulate the flow.

(c) Gravel Trap:- Gravel trap is a structure constructed close to the intake in order to prevent gravel from getting into the approach canal.

(d) Settling basin:- It settles the suspended particles contained in the water. The trapped suspended sediment is subsequently discharged back into the river through a flushing arrangement.

(e) Headrace Conveyance System: Canal/pipe/tunnel can be the conveyance system in order to carry the water to the forebay or surgetank with minimum energy loss.

(f) Forebay or Surge tank: These structures are provided to release the water hammer pressure & also to store additional volume of water for the startup of the plant.

(g) Penstock:- It is the pipe for the pressurized flow of water to turbines.

(h) Powerhouse: It is the location where all the machines are installed for power production

(i) Tailrace:- The water used in powerhouse is taken back to the river using tailrace.

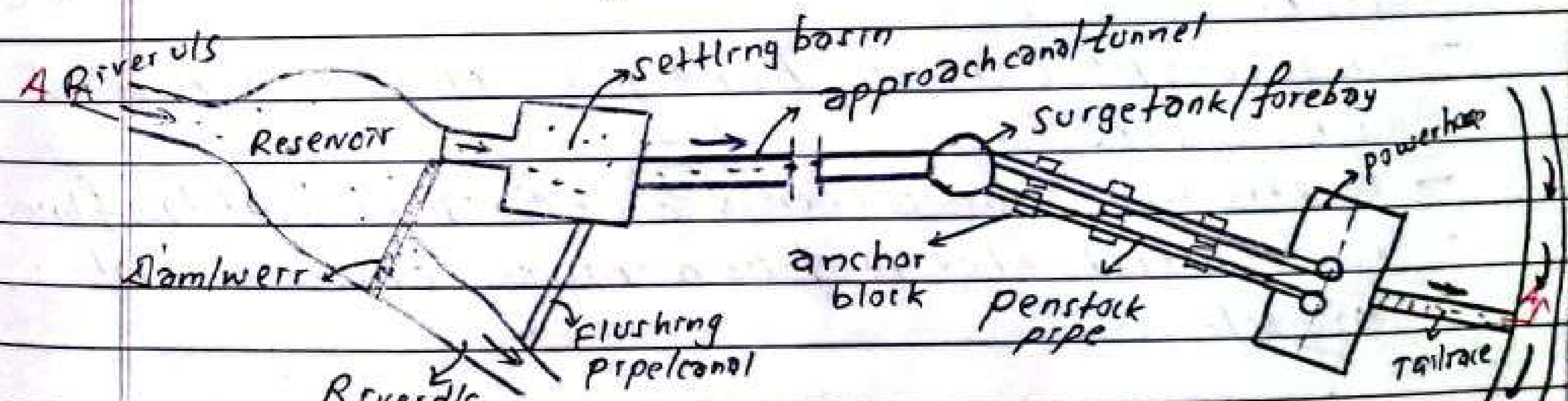

Fig: Layout of ROR HPP (plan)

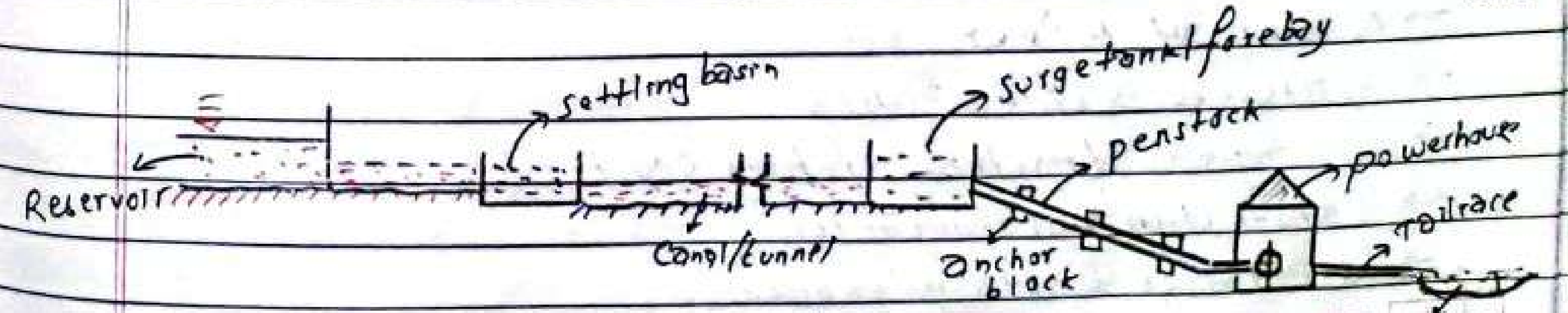

Fig: Section A-A

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※ Intake:-

→ Intake is the structure to obtain the required quantity of water from the river or reservoir for different engineering purpose such as irrigation, power generation, water supply etc.

* Functions of Intake:-

→ To control flow of water into conveyance system.
→ To prevent entry of trash, debris etc.
→ Enable smooth, easy & turbulence free entry of water
→ To minimize sediment entry from water.

※ Location of Intake:

→ Located such that largest possible portion of bed load remains in the river
→ In meandering portion of river, favorable site is deflecting bank.
→ In straight portion, bent flow can be introduced using spurs.
→ Located such that water is available even at low flow.
→ Located such that it has minimum environmental impact.

※ Selection of type of intake:-

→ Nature of River
→ Nature & Scale of HPP
→ Sediment, trash & debris content
→ Construction consideration
→ Operation & Maintenance

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※ Types of Intake

(1) Runoff River Intakes

(a) Side Intake:-

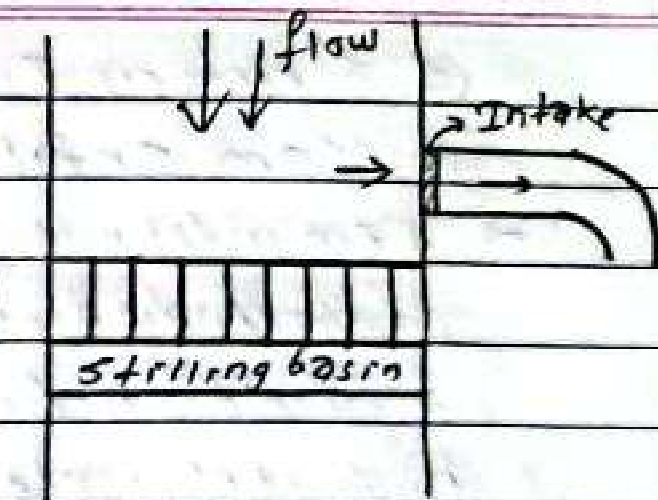


Fig: Plan of side intake

→ Longitudinal axis of intake shall generally be aligned perp^n to axis of river.
→ Most common type of intake in ROR HPP.
→ Suitable for mild slope of river.
Eg: Lower Marsyangdi, Sunkoshi etc.

(b) Frontal Intake

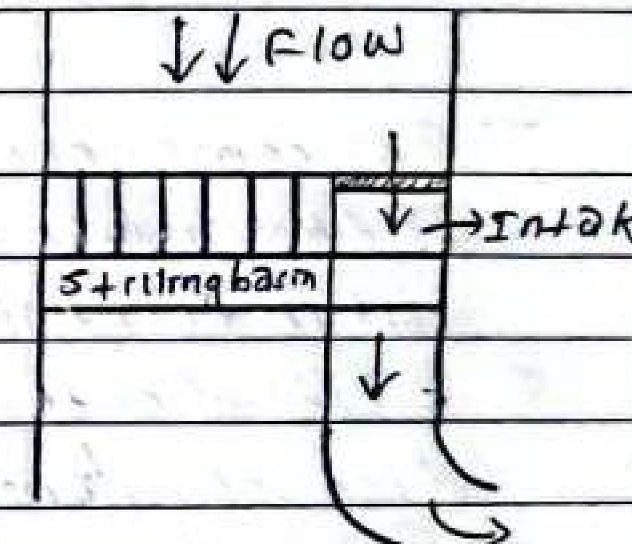
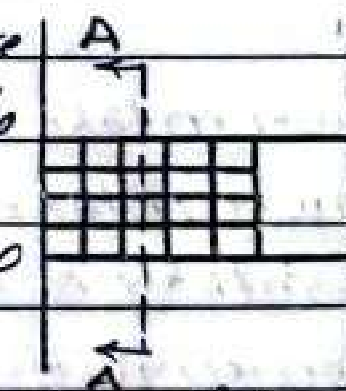


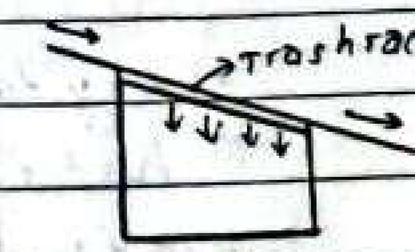
Fig:- Frontal Intake

→ Longitudinal axis of intake shall generally be aligned parallel to axis of river.
→ Suitable for relatively clean water carrying rivers & wider rivers.
→ Trash & debris are attracted towards intake
Eg: Seti hydro intake.

(c) Bottom Rack Intake / Drop Intake



Plan of drop intake



Section A-A

→ Consists of trench shaped intake gallery constructed on river bed to entrap river flow. A sediment trap trench may be provided u/s of intake gallery.
→ Trash rack is provided over intake.
→ Suitable for very steep rivers carrying boulders.
→ Simple & inexpensive
→ Disadvantage: Abrasion of rack, choking of rack & collecting channel.

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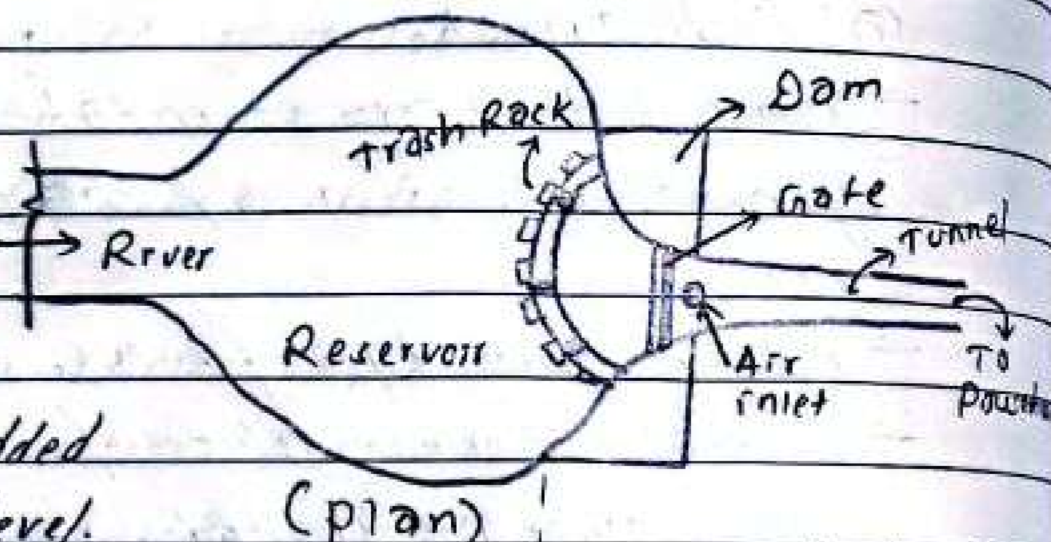
(2) Reservoir Intake:

(a) Dam Intake:-

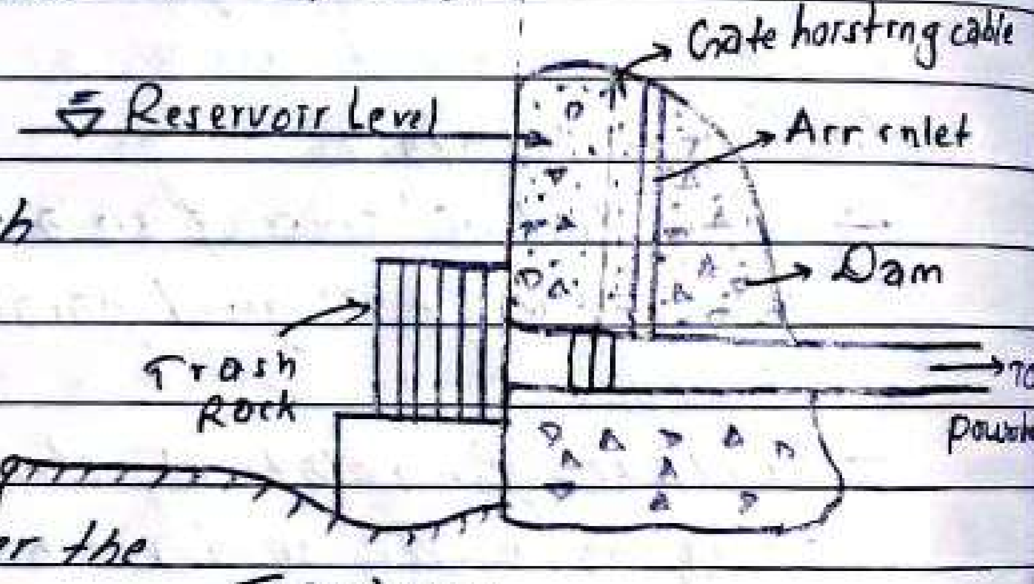
→ Dam intake is provided in the body of dam & is used in high head hydroelectric plant.

→ The penstock pipe is embedded in the dam at required level.



(plan)

→ The component parts of this intake are a trash rack in front of dam, a control gate operated from the dam by hoisting cable, an air inlet just after the gate & a penstock pipe.



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(b) Tower Intake:-

→ Generally used in large projects.

→ Types

1) Dry tower intake
2) Wet tower intake

→ wet tower intake is preferred as it can function well even in the fluctuation of reservoir level due to wave.

→ portals are available for entry of water.

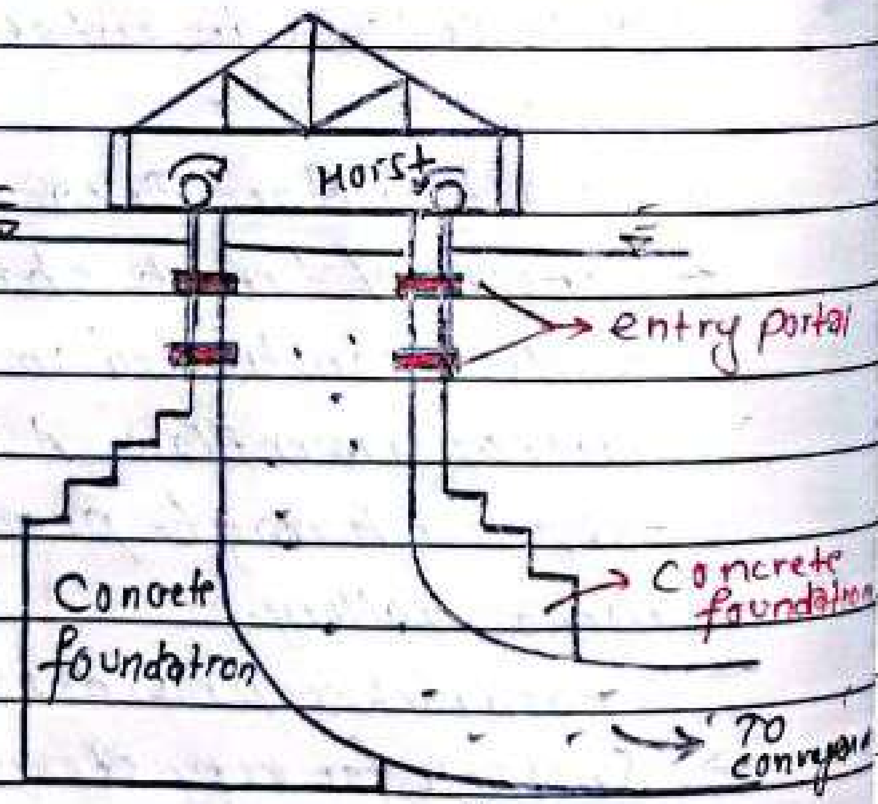

Fig: Section of tower intake

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(c) Submerged Intake:

→ Usually used in small power plants. It is used in reservoir or river which donot have higher sediment concentration.

→ It is economical & doesnot obstruct navigation.

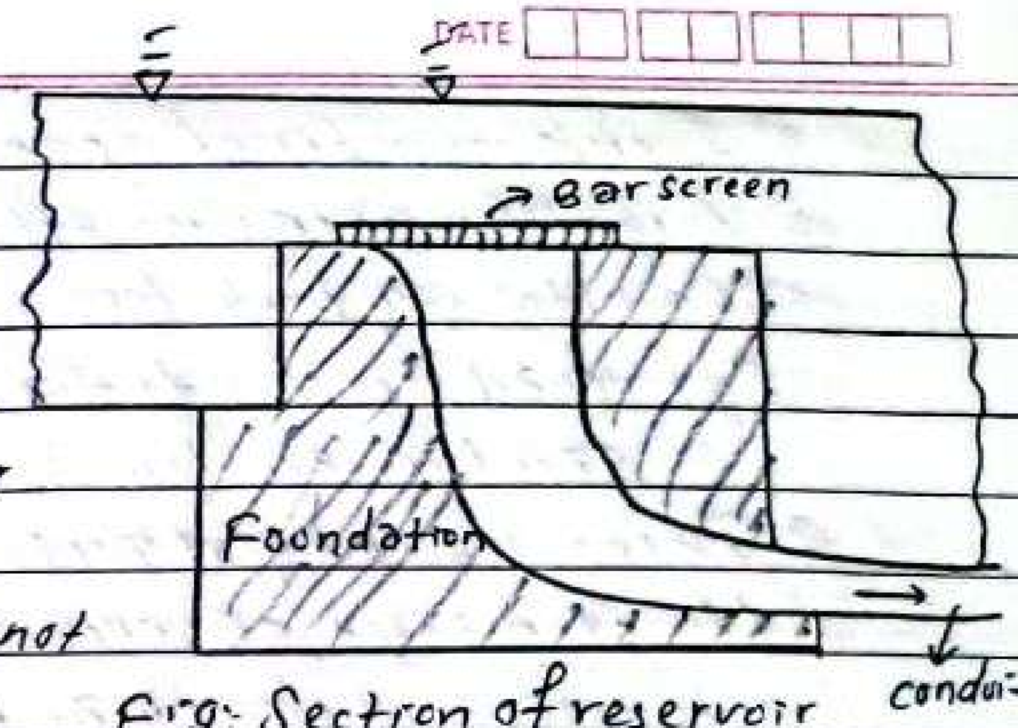

Fig: Section of reservoir intake.

(d) Shaft Intake

→ It is a vertical shaft driven into the river bed which carries water through underground conveyance system to the powerhouse for power generation.

→ It consists of trash rack at the entry of water, gate chamber, hoist chamber, & intake gate arrangement & penstock.

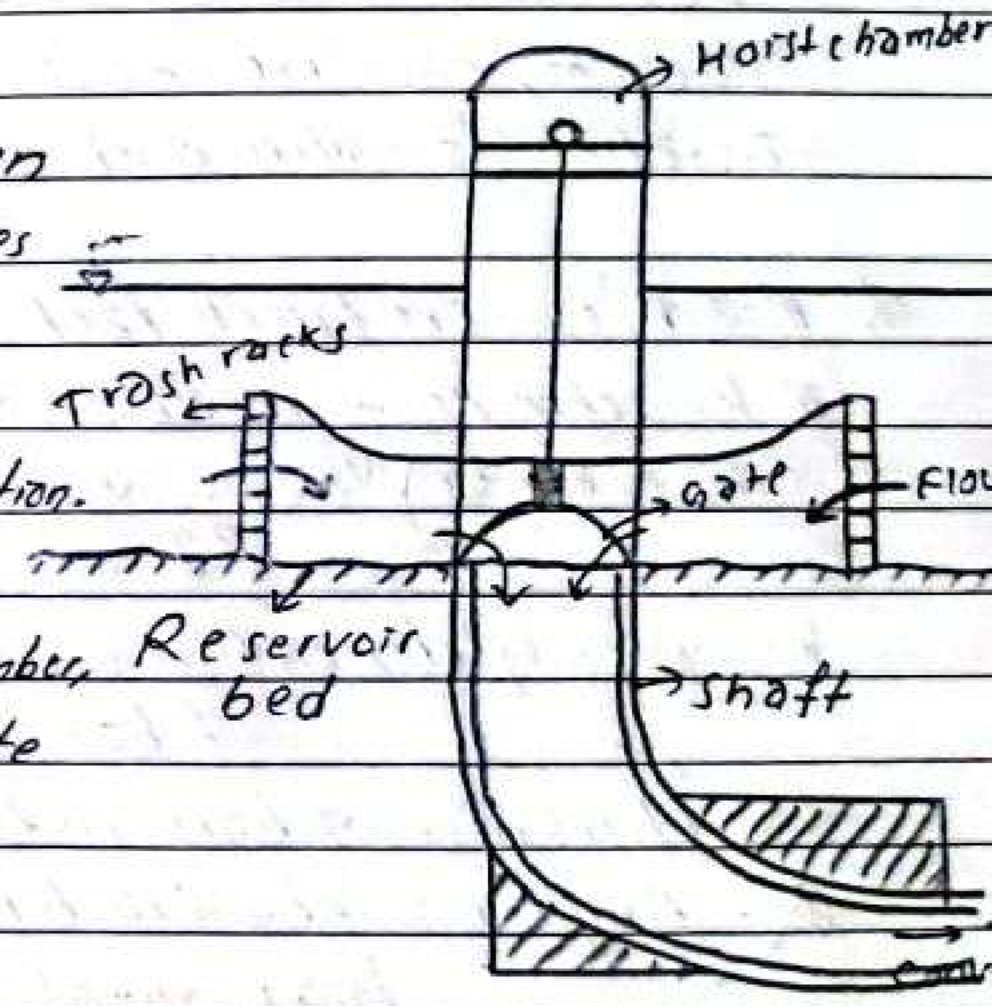

Fig: Section of shaft intake

Design Concept of Intake:-

1) Decide suitable type of intake based on site & hydraulic condition
2) Satisfy the velocity conditions
   - Approach velocity ⇒ 1 m/s
   - Trash rack ⇒ (0.6-0.75) m/s
   - Intake gate ⇒ (1-2) m/s.

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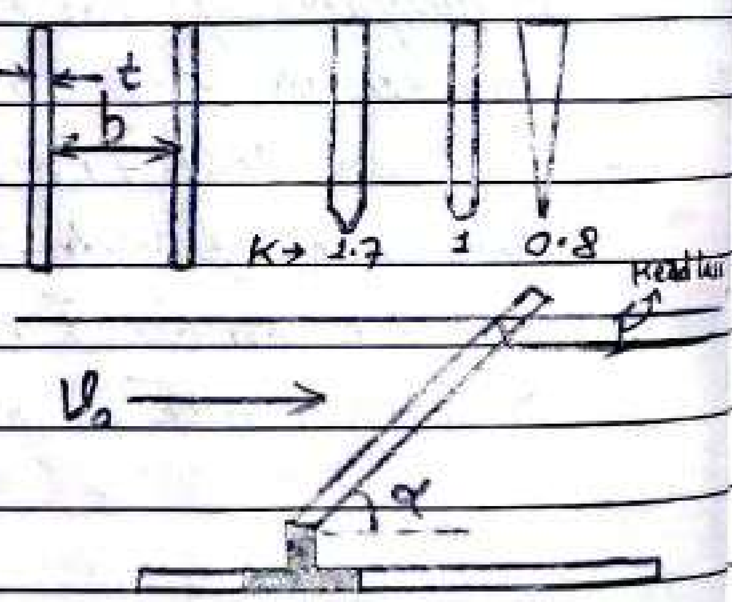
3) Decide number of opening of intake
4) Account for contraction effect due to abutment or piers.
5) Calculate Hydraulic loss
Eg: Head loss due to trashrack, entrance loss, transition loss, gate loss, exit loss.
6) Ensure no entry of air: The system can trap air due to the formation of vortex due to hydraulic jump condition, submergence, velocity at intake etc.
7) Ensure the release of trapped air to avoid formation of vacuum, cavities inside the system by providing inlet aeration facilities such as air vent.

※ Head loss in trash rack:-

a) kirschmer's formula

$$H_L = k\left(\frac{t}{b}\right)^{4/3} \times \frac{V^2}{2g} \times \sin\alpha$$



k = Loss coeff depends on cross section of bar

t = thickness of bars in trash rack

α = Angle made by trash rack with horizontal

V = Velocity of flow through trashrack

b = Spacing betn bars.

b) General Formula:

$$H_L = k_t * \frac{V^2}{2g}$$

k_t = loss coeff = $1.45 - 0.45R - R^2$

R = Ratio of net area to gross area of trash rack.

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※ Himalayan Intake:-

→ Himalayan Intake is a special type of intake that has proper system for management of both floating debris & bed load.

→ Used in ROR HPP intake in steep Himalayan Rivers.

→ The purpose is to maintain a reservoir volume for daily peaking by providing means of flushing of sediments from reservoir.

※ Sediment Handling Measures:-

→ Sediment transport is inherent phenomenon. Himalayan rivers are more prone to erosion due to
1) Young geology
2) Fragile geology
3) Intense Rainfall
4) Steep catchment.

→ There are two types of sediment load: bed load & suspended load. Bed load is excluded at the intake but the suspended load enter into the intake.

※ Effect of Sediment on Hydropower:-

→ Wear & Tear of Hydro-Mechanical Equipment
i) Turbine blades ii) Guide vanes, rings
iii) Steel pipes, valves, lining & other structures.
The wear & tear depends on size, shape & hardness of sediment.

→ Chocking of valves, flushing channel, Monitoring devices like sediment sampler, turbidometer, currentmeter etc.

→ Loss of Efficiency.

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* Settling phenomenon depends on
→ Particle characteristics
→ Temp' of water
→ Concentration of sediment

* What size of particles to remove?
1) Stole (1993) ⇒ $d > (0.15 - 0.3)$ mm
2) Mosonyi ⇒ Removal according to head
- Medium head, $d > (0.2 - 0.5)$ mm
- High head, $d > (0.1 - 0.2$ mm$)$
- Very high head, $d > (0.01 - 0.05$ mm$)$

* Parameters of Settling Basin Design:-
(a) Settling velocity (w)
→ For turbulent flow ($d > 1$mm), $w = \sqrt{3.33 g d \Delta}$, $\Delta = \frac{\gamma_s - \gamma_w}{\gamma_w}$

→ For transition flow (0.1 mm $< d < 1$ mm), $w = \frac{4 \Delta g d}{3 C_D}$

$$C_D = 24/R_e + 3/\sqrt{R_e} + 0.34$$

→ For laminar flow ($d < 0.1$ mm), $w = \frac{\gamma d^2 (G-1)}{18 \nu}$

$$C_D = 24/R_e$$

where, d = diameter of particle in mm
C_D = Coefficient of drag
R = Reynold's number
ν = kinematic viscosity of water
G = Relative density of sediment

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(b) Concentration (c)
→ Concentration is the amount of sediment present in given volume of water. It is expressed in ppm or mg/l
→ With increase in 'c', 'w' decreases due to interference of particles.

(c) Turbulence:
→ Due to turbulence, the trap efficiency of the settling basin is reduced due to reduction of settling velocity.

* Design Method of Settling Basin
(A) Particle Approach:-
Assumption:-
→ Steady flow with no turbulence.
→ Particle once settled down donot come back into suspension.
→ Depth of basin (H) is usually assumed.

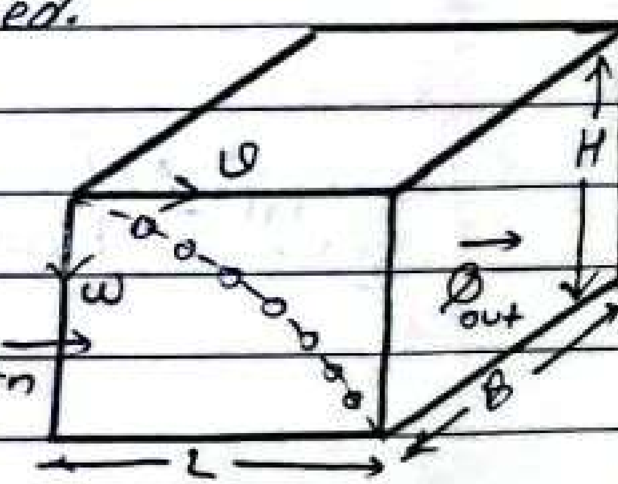

Fig:- particle Approach

Let, L, B, H be length, width & height of basin. 'V' be the horizontal flow velocity & 'w' be settling velocity.
Travel time = $\frac{L}{V}$ & settling time = $\frac{H}{w}$

Now, $L/V = H/w \Rightarrow L = VH/w$ --(1)
Also, $Q = AV = B \times H \times V \Rightarrow V = Q/BH$
From (1)
$$L = \frac{Q}{BH} \times \frac{H}{w} \Rightarrow w = \frac{Q}{LB} = \frac{Q}{A_s}$$
where, A_s = surface area of settling basin

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→ Horizontal flow velocity can be computed as,

(a) According to camp,

$v = 0.36\sqrt{d_{mm}}$ for $d > 1mm$

$v = 0.44\sqrt{d_{mm}}$ for $0.1mm \le d \le 1mm$

$v = 0.51\sqrt{d_{mm}}$ for $d < 0.1mm$

If turbulence is considered,

$W_{adopt} = w - w'$ where, $w' = \alpha v$ & $\alpha = \frac{0.132}{\sqrt{H}}$

$L = VH/W_{adopt}$

Also, breadth of basin $(B) = Q/VH$

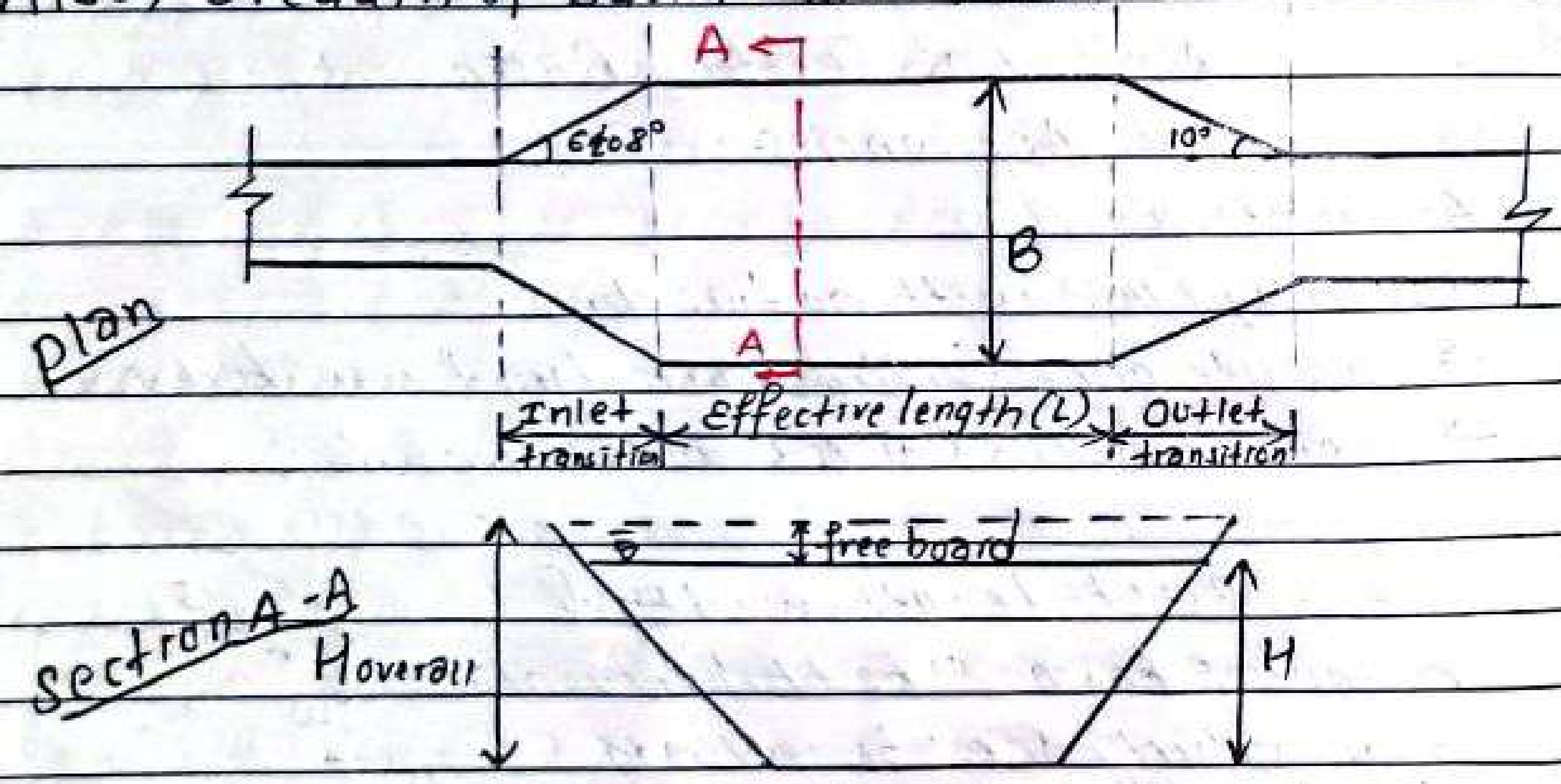

Fig: plan & section of settling basin

(B) Concentration Approach:-

→ In this method, basin is designed to achieve desired trapping efficiency.

(a) Camp's method:-

→ In this method, trap efficiency is function of dimensionless parameter w/u^* & wA_s/Q.

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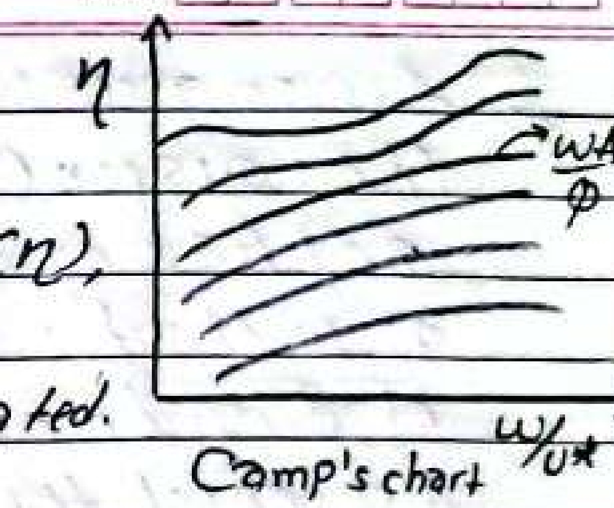
U^* = Shear velocity = $\sqrt{\tau_0/\rho}$



Camp's chart

For value of w/u^* & trap efficiency (η), wA_s/Q can be obtained.

Then, from wA_s/Q, A_s can be calculated.

→ Assume, L/B ratio = 4 to 10

Volume ⇒ $L \times B \times H = Q \times T$

$H = \frac{Q \times T}{L \times B} = \frac{Q \times T}{A_s}$, where T = detention time

(b) Vetter's Method:-

For given value of w & Q, A_s can be calculated directly using the formulae.

$$\eta = 1 - e^{-wA_s/Q}$$

(c) Hazen's method:-

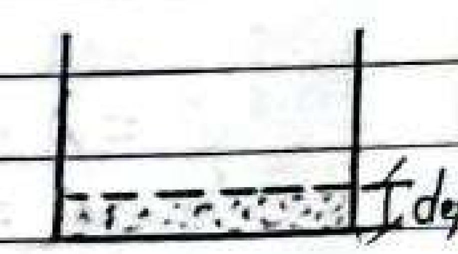
$$\eta = 1 - \left[1 + \frac{mwA_s}{Q}\right]^{-1/m}$$

Where, m = performance coeff

m = 0, best performance

m = 1, poor performance.

※ Estimation of Sediment volume:-



weight of sediment = $Q * T * C$

where,

Q = discharge, T = detention time

C = Concentration

Volume of Sediment = $\frac{\text{weight}}{\text{density} * \text{packing factor}(0.5)}$

Depth of sediment = $\frac{V}{A_s}$

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Q Design a settling basin to remove sediment particles larger than 0.15mm ($w = 1.7$ cm/s) from the water carrying mainly sand having design discharge of 35 m^3/s

a) without considering turbulence

b) Considering turbulence.

Also, check the length of basin using velikanov's formula for W=88% (For W=88%, $\lambda = 0.8$).

Given that depth cannot exceed 8m.

→ Given,

particle size (d_{mm}) = 0.15mm

Settling velocity (w) = 1.7cm/s = 1.7×10^{-2} m/s

Design discharge (Q) = 35 m^3/s

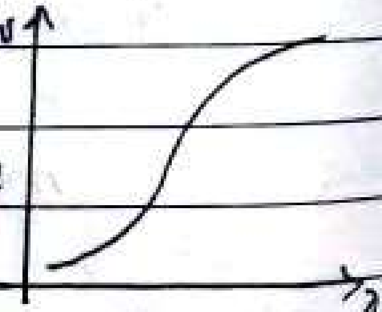
Velikanov's formula:-

It is used to check the length of the basin calculated from the analytical method,

$$L = \frac{\lambda^2 v^2 (\sqrt{H} - 0.2)^2}{7.51 w^2}$$

where, λ depends upon removal ratio 'W'.

W is ratio of settled sediment to total load entering.



(a) Without considering turbulence

Assume, flow depth in settling basin (H) = 7m

length of basin (L) = $\frac{vH}{w}$

$v = 0.44\sqrt{d_{mm}} = 0.44\sqrt{0.15} = 0.17$ m/s

$Q = A \times v = B \times H \times v$

$B = \frac{35}{7 \times 0.17} = 29.41$m

Here, $L/B = 70/29.41 = 2.38 < 4$

so, Let make two compartments.

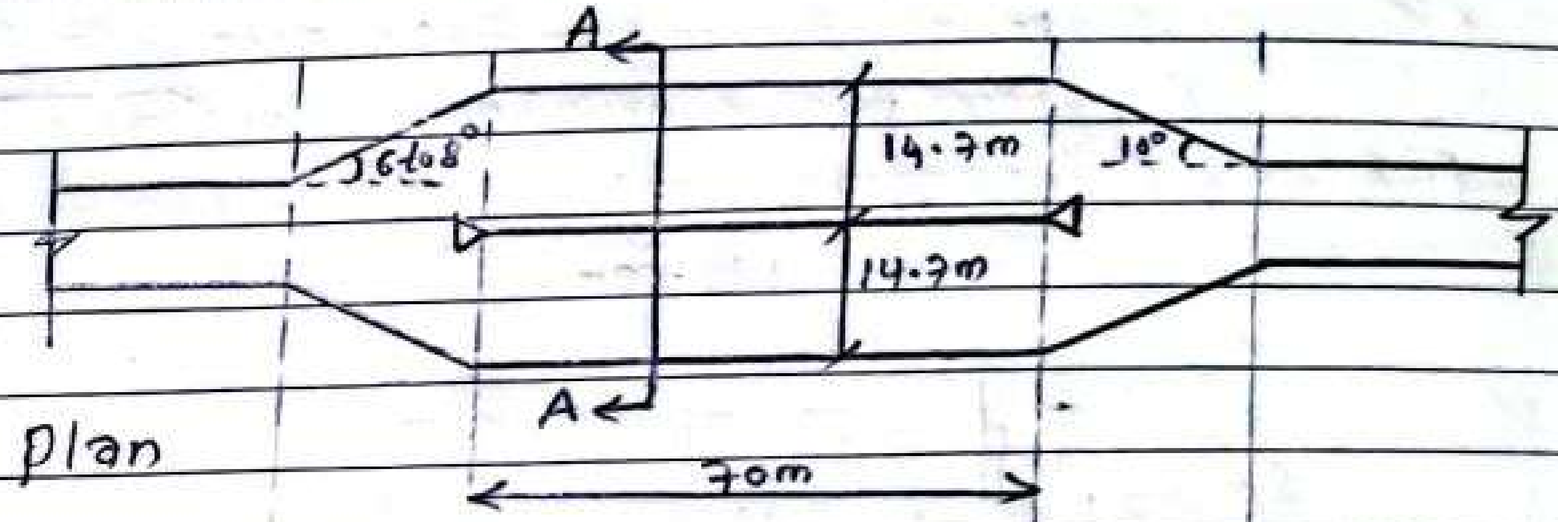
For each compartment,

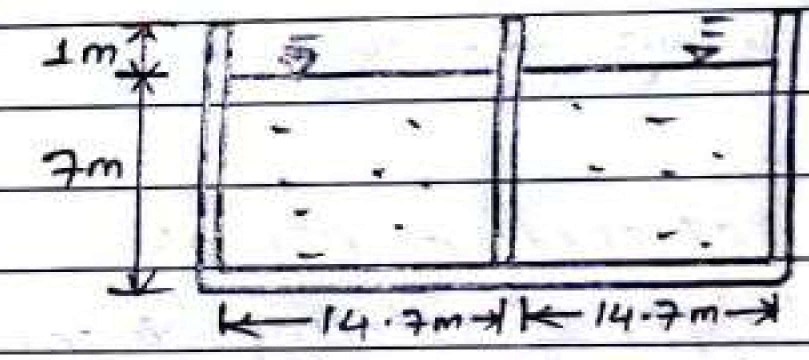
$L = 70$m & $B = \frac{29.41}{2} = 14.705$m

$L/B = 4.6 > 4$ (ok).

Assume 1m free board, $H = 7 + 1 = 8$m

$L_{eff} * B * H_{overall} = 70 \times 29.41 \times 8$m





Section A-A

(b) With considering Turbulence:

$W_{adopt} = W - W'$

$W' = \alpha v = 0.0498 \times 0.17 = 8.48 \times 10^{-3}$ m/s

$\left[\alpha = \frac{0.132}{\sqrt{7}} = 0.0498\right]$

$W_{adopt} = 1.7 \times 10^{-2} - 8.48 \times 10^{-3} = 8.52 \times 10^{-3}$ m/s.

$L = \frac{vH}{W_{adopt}} = \frac{0.17 \times 7}{8.518 \times 10^{-3}} = 139.69$ m

Again, $B = \frac{Q}{vH} = 29.41$m

Here, $L/B = \frac{139.69}{29.41} = 4.74 > 4$ (ok)

But, taking minm of two compartments.

Reqd dimension = 139.69 * 29.41 * 8m

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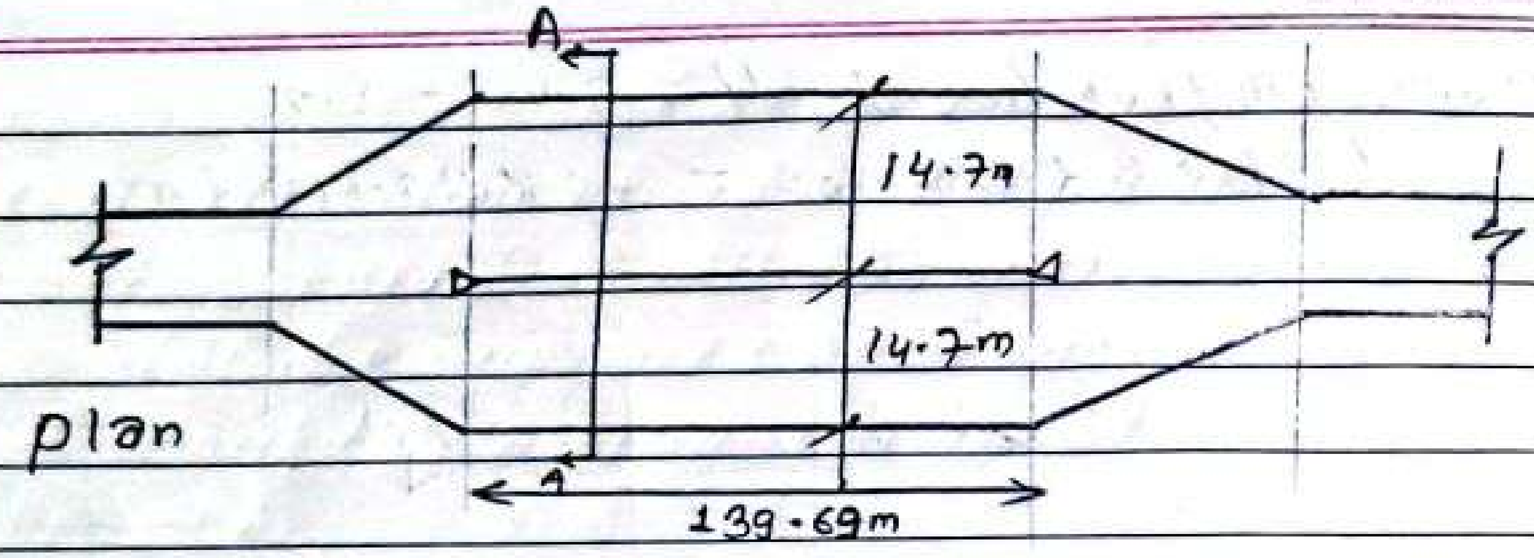
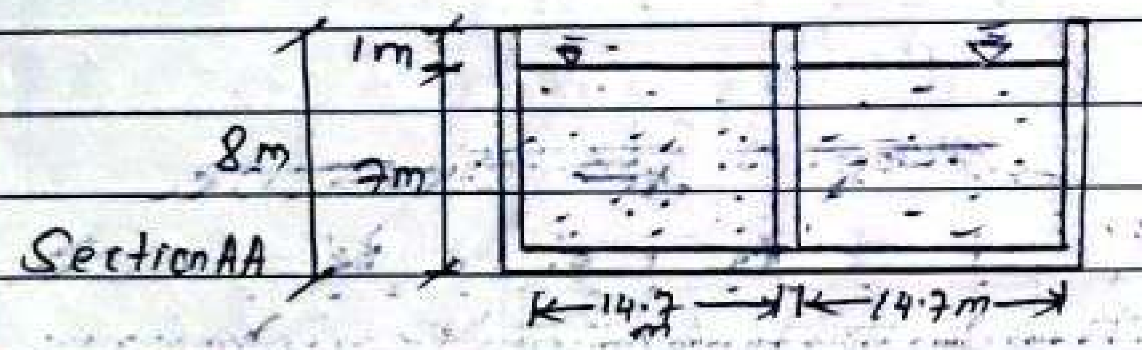


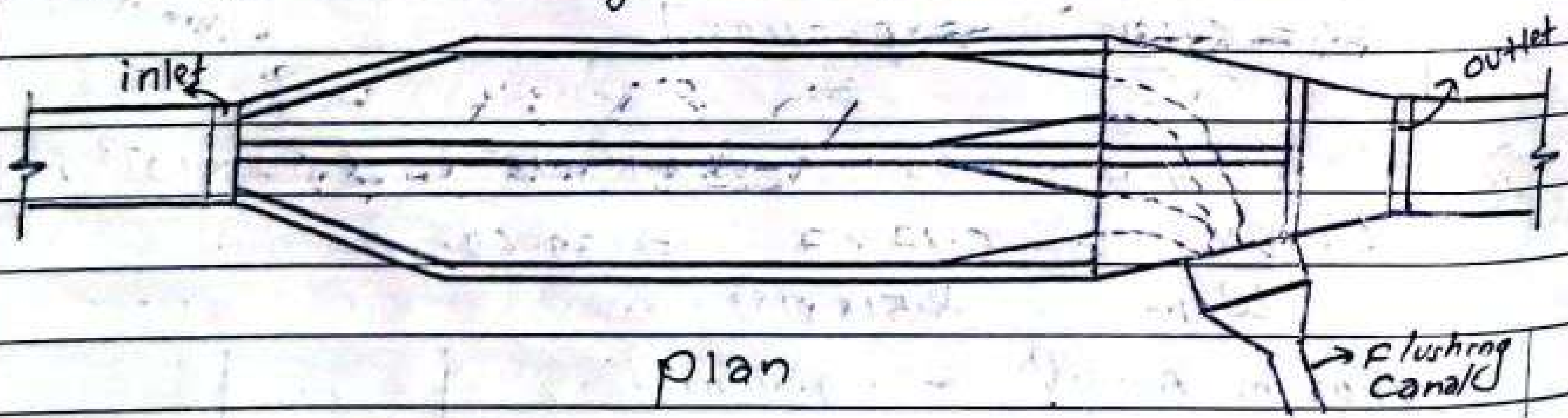


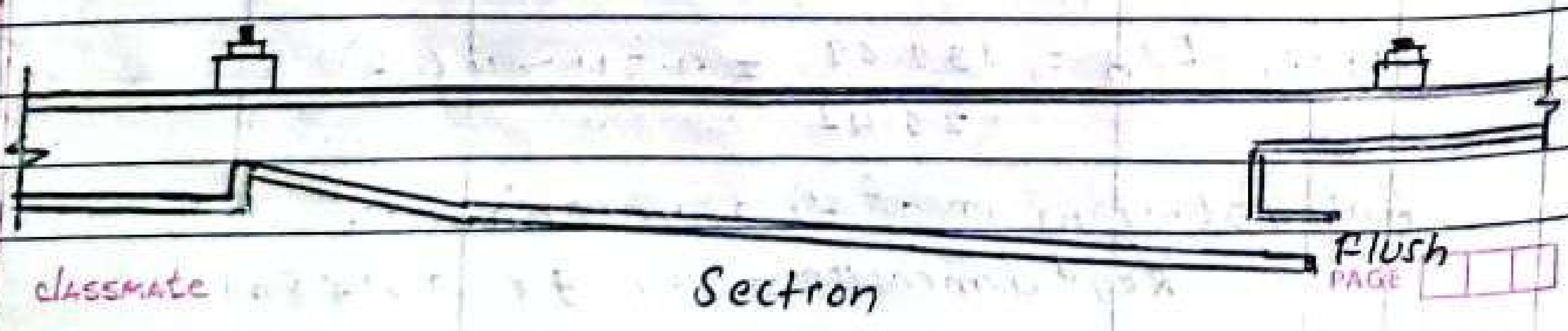
* Different types of settling basin:-

(a) Conventional type of settling basin.

→ This basin is dewatered when it is taken out of operation. Sediments may be removed manually or with mechanical equipment after the basin is dewatered.

→ Generally, two settling chamber are constructed.





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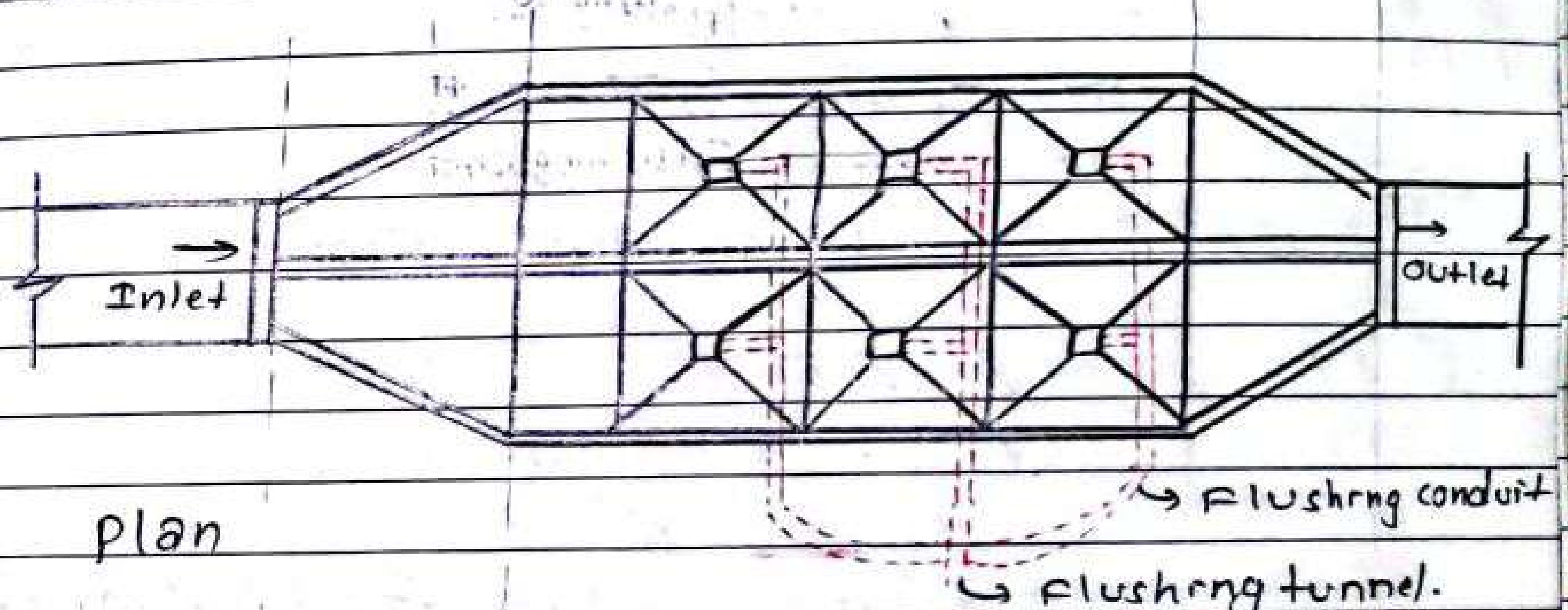
(b) Hooper Type:-

→ Water level & 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 or intermittent flushing or by use of some kind of suction or dredging device.



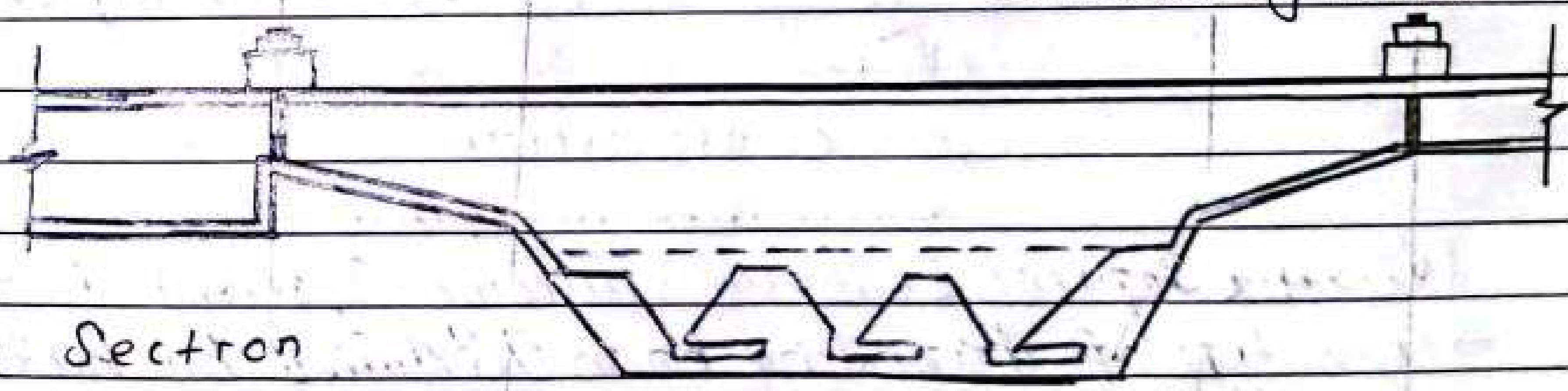
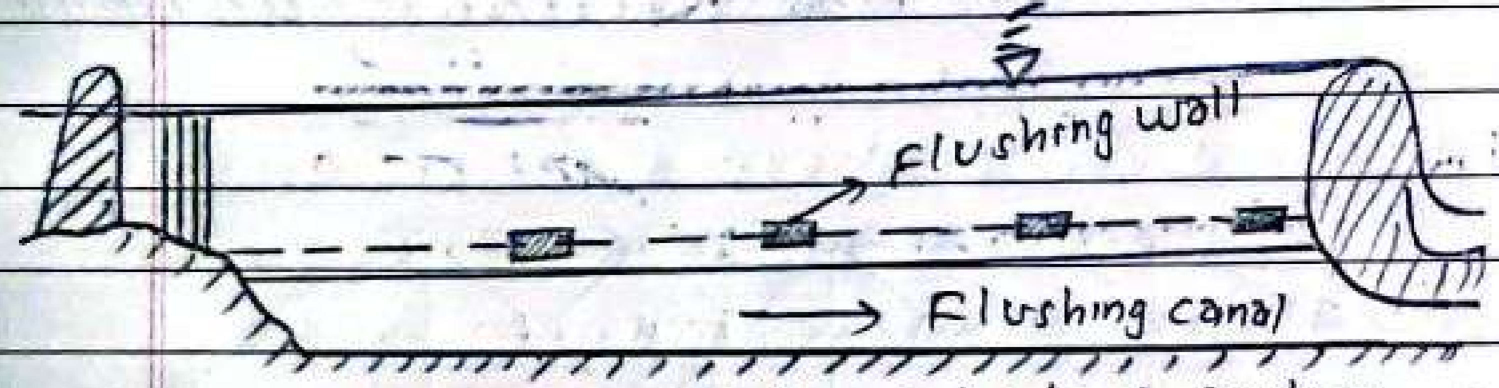


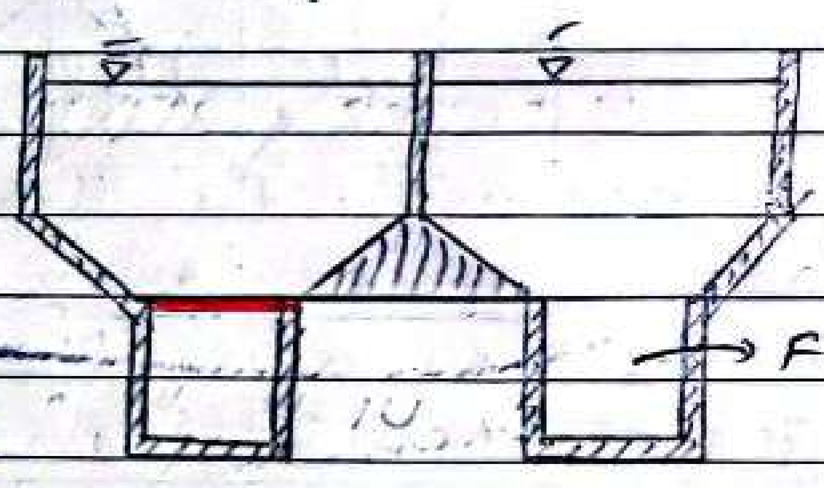
Fig: Hooper type settling basin.

(c) Beiri Type:-

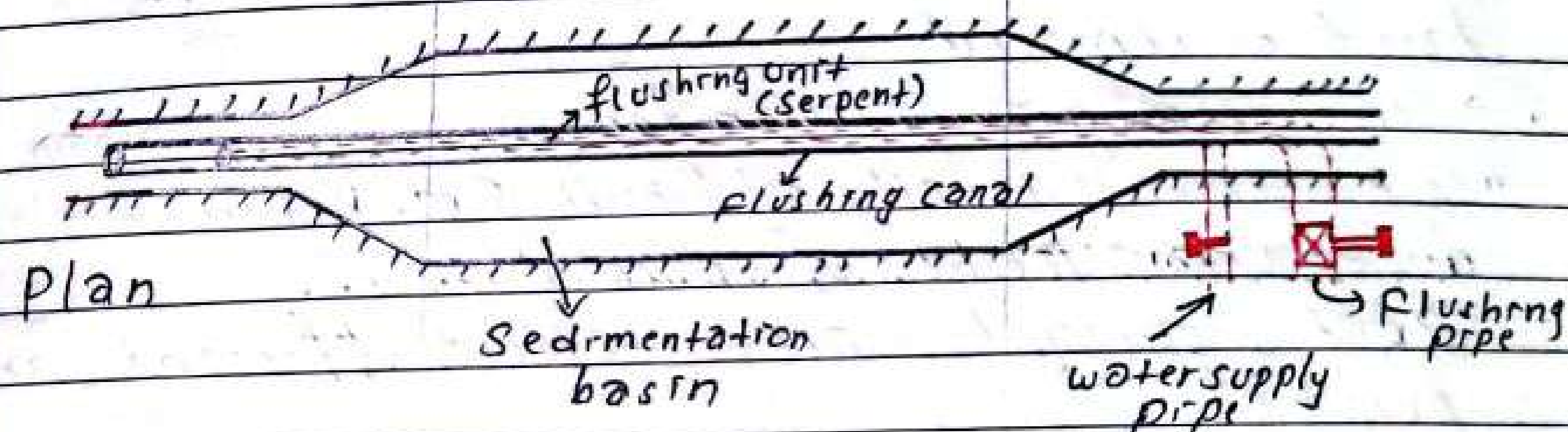
→ In Beiri type, the shutter mechanism in the bottom of basin is made of two plates with series of opening. One plate is fixed while other can be moved horizontally. Flushing is done when opening in both plate fall together.

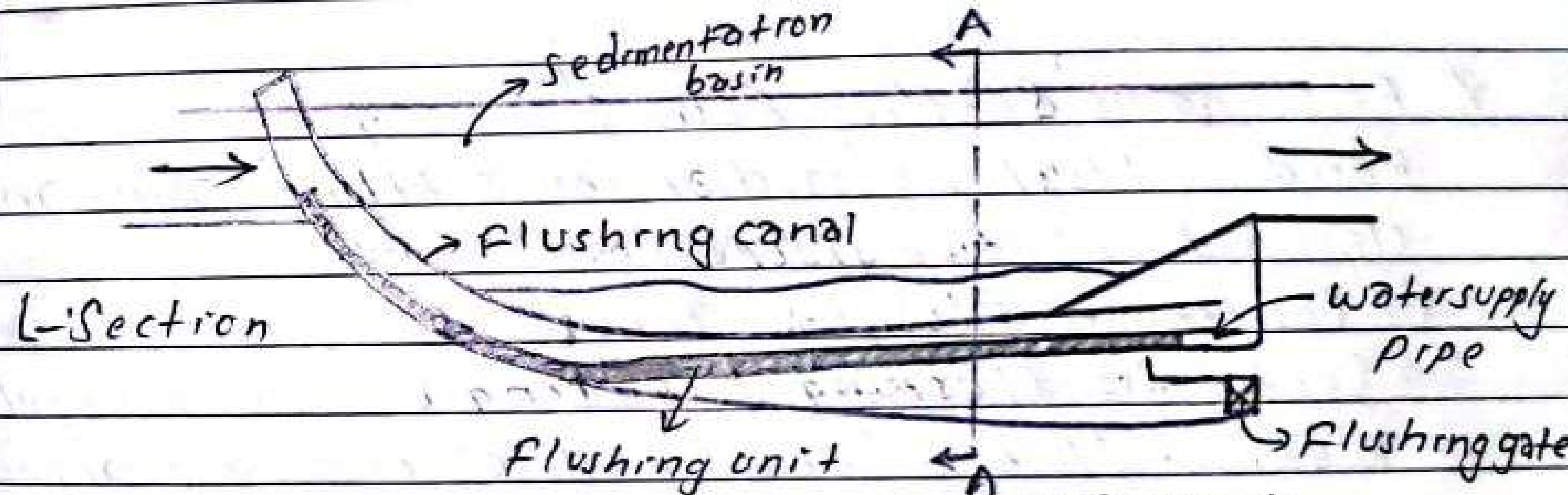
→ Sensor can be placed at the bottom of the plate so as to sense the volume of sediment deposited in the chamber.
→ Once the sediment collected increases the desired level of deposition, one plate is moved over the other to match the opening & sediment is flushed.

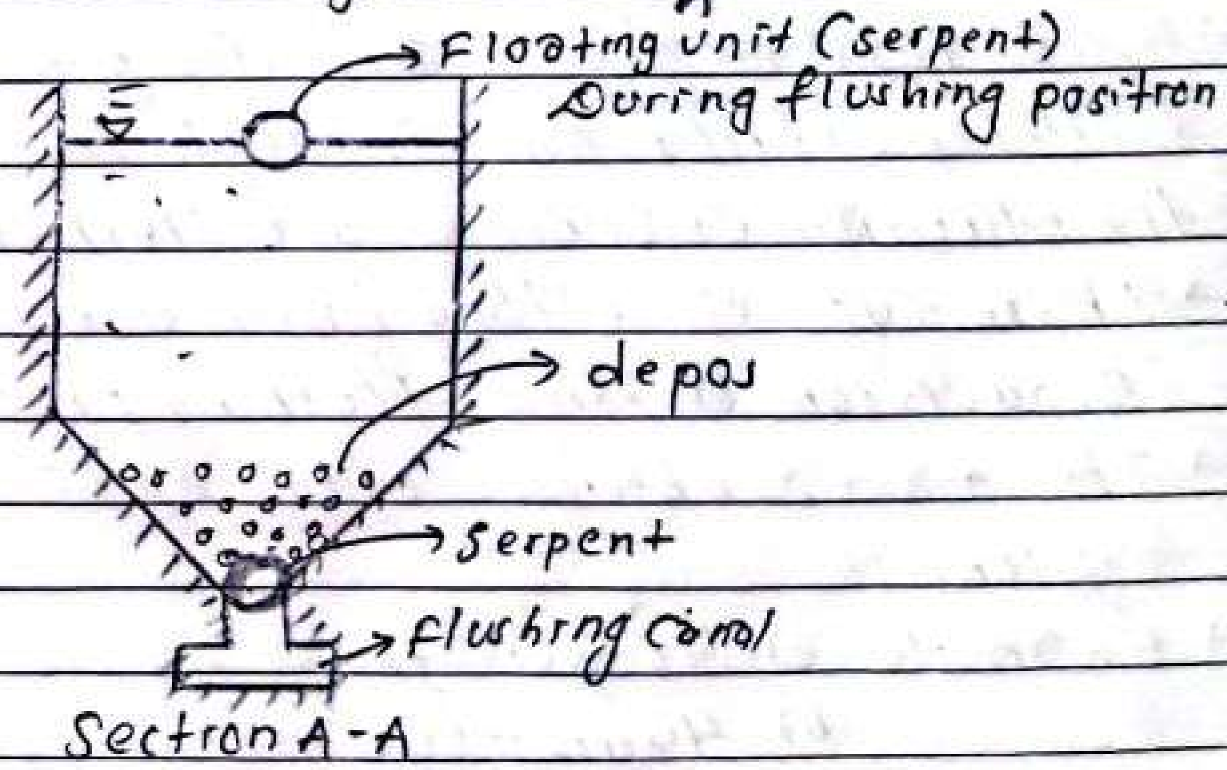




(d) S4 (Seprent Sediment Sluicing System)
→ New type of desanding basin developed by Hakoon stole
→ This type of settling basin is constructed in Andhikhola, Jhimruk khola & khimti khola HPP in Nepal.
→ One or more flushing channels are located at bottom of settling basin.
→ The flexible pipe (called seprent) can float or sink in water depending upon the fluid filled inside it. When the serpent is filled with heavy fluid, it settles & flushing canal is closed & sediment is deposited over it. When the deposition reaches a limit, the fluid in pipe is removed & light gas is installed which facilitates floating of serpent & hence sediments are flushed through flushing channel.







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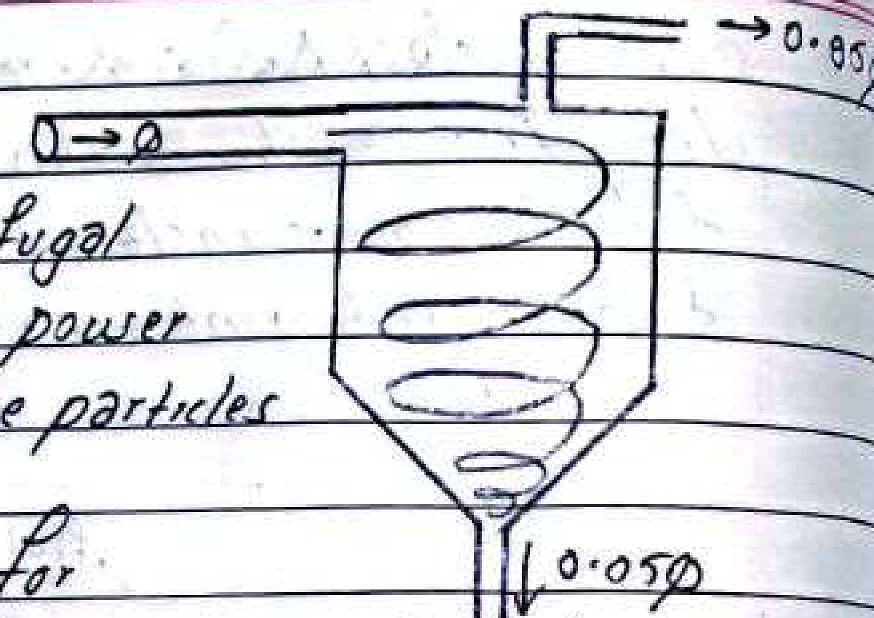
(e) Hydrocyclone:-



→ Particles in a hydro cyclone are trapped due to centrifugal action. When the head of the power plant is significant, very fine particles have to be removed.
→ Conventional gravity basin for such purpose is usually not feasible due to unavailability of space & cost. Hydrocyclones are suitable & efficient for such purpose.

Fig: Hydrocyclone

※ Flushing of Settling Basin:-
Based on flushing mechanism, settling basin can be classified into two groups.

(a) Continuous Flushing Type:-
→ Continuous flushing desanding basin uses surplush water for flushing (about 10% of plant discharge)
→ These type of basin are designed with hoppers. The settled particles pass through the bottom of hoppers to the collecting channel & are flushed continuously.
→ These type of settling basin doesnot interfere power production during flushing process.
→ The main problem is clogging of sediment extracting system.
→ Example a) Hopper Type
b) Hydro-Cyclone

(b) Discontinuous flushing type:-
→ In this type of settling basin, sediments are not flushed continuously.
→ Much simpler in design & less susceptible to clogging.
→ Power generation should be stopped during flushing for single basin.
→ They are further classified into two classes:

1) Periodic flushing type:
→ Powerplant is shut down during flushing.
→ Flushing is generally done by conventional gravity flushing method
→ Mechanical & Manual removal of sediment.
→ Eg: Conventional type settling basin.

2) Intermittent flushing type:
→ In this type of settling basin, flushing is not continuous but ~~of~~ powerplant is not shut down during flushing.
→ Flushing is done at regular intervals & the power generation is done continuously.
→ Ex: Bieri type & S4 type.

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Chapter 6 :- Water Conveyance Structures:

Outline:-

→ Introduction to powercanal, its suitability in hydropower proj
→ Hydraulic tunnels
→ Forebay & Surgetanks
→ Penstock & Pressure Shaft
→ Headloss calculation in conveyance System.

※ Power Canal:-

→ A power canal refers to a canal used for hydraulic power generation, rather than for transport~~ation~~. The flow of water in canal is open channel flow.
→ Suitability of canal in hydropower System:
- Construction of canal can be good medium of conveyance where construction of tunnel is expensive.
- Construction of canal is ~~suitable~~ easier than tunnel.
- Hydropower can be merged with other purposes if the conveyance system is canal.

※ Hydraulic Tunnel:-

→ The tunnel is an underground passage made without removing the overbudren.
→ The tunnel used for conveyance of flow is called hydraulic tunnel.
→ Types of tunnel

(a) Pressure Tunnel:-
The tunnel in which flow takes place with pressure is called pressure tunnel. The headrace tunnel is pressure tunnel.

(b) Non-pressure Tunnel:-
The tunnel in which open channel flow takes place is called non-pressure tunnel. Generally, spillway tunnel, diversion tunnel, tailrace tunnel are non-pressure type.

* Advantages of Tunnel:-

→ Reduces land acquisition, resettlement issue, forest clearance etc
→ Environmental Effect will be minimum.
→ Natural landscope of hilly area is not disturbed.
→ Possible shortest & direct way leads to economy.
→ Optimum space consumption.
→ Easy for interbasin transfer.
→ Time saving: Shortest route to connect two points.

* Disadvantages of tunnel:-

→ High construction cost
→ Construction period is long
→ High construction risk
→ Expensive investigation.
→ Additional Cost for lighting & ventilation.

※ Size & Shape of Tunnel:-

(a) Size of Tunnel:-
→ The minm diameter of tunnel is fixed with consideration of transportation, excavation & hauling during tunneling and should be greater than 2m for circular section in case of other shapes it should be greater than 1.9m in width & 2.1m in height.

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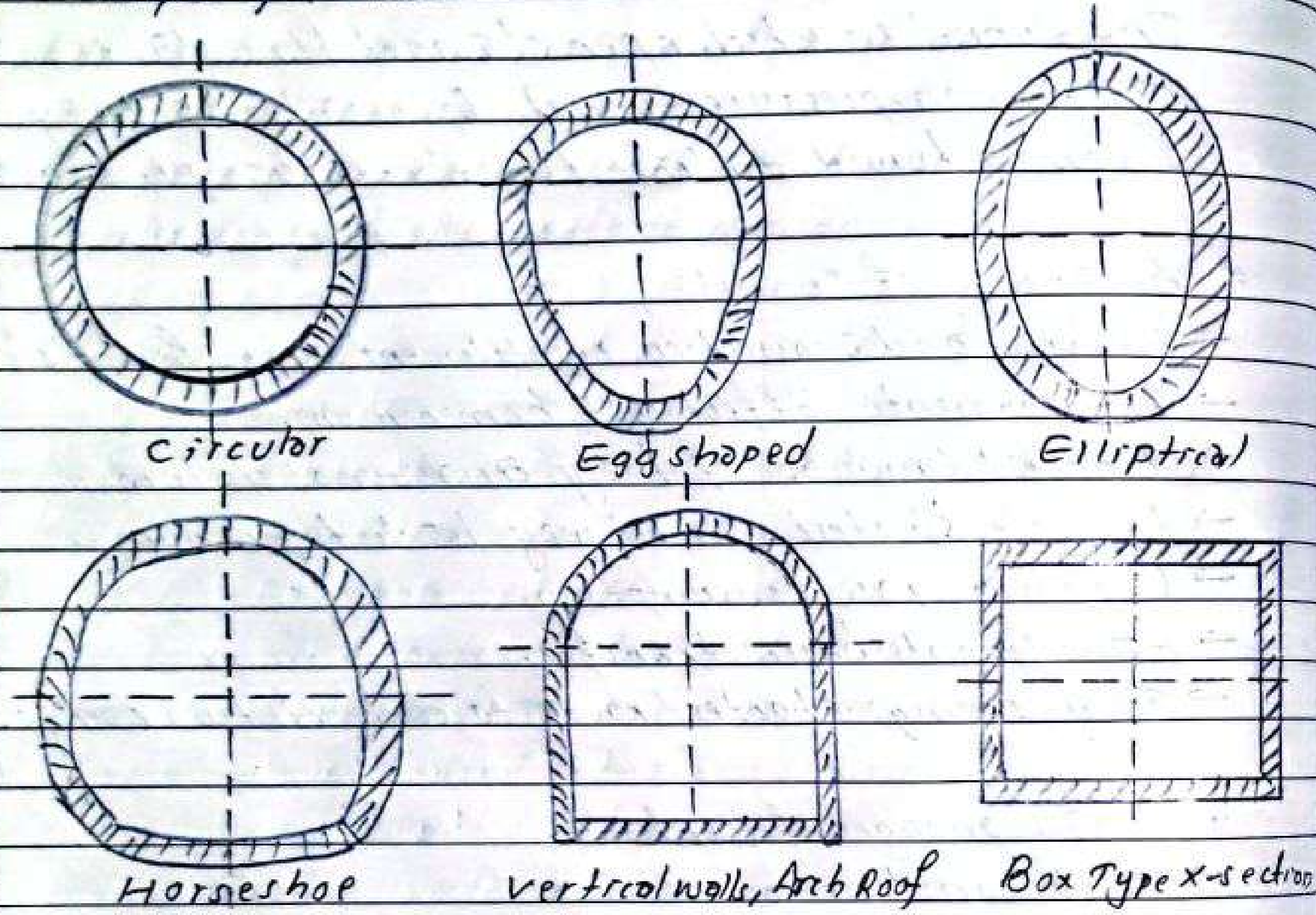
(b) Shape of Tunnel:-



1) Circular Section:-

Most suitable from structural consideration but difficult for excavation.

$$A = \frac{\pi d^2}{4} \quad \& \quad p = \pi d$$

2) D-shaped section

→ This section is suitable for tunnels located in good quality rocks. The main advantages of this section is the added width of the invert which gives more working space in the tunnel during driving.

$$A = \frac{\pi d^2}{4} + d \times \frac{d}{2} \quad \& \quad P = d + 2 \times \frac{d}{2} + \frac{\pi d}{2}$$

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3) Horse Shoe Section:-

These sections are compromise betn circular & D shaped sections. These sections are structurally strong to withstand external rock & water pressure. They are most suitable where a moderately good rock is available, advantages of flatter invert are required for construction purposes & tunnel has to resist internal pressure.

$$A = 0.8293 D^2$$
$$P = 3.267 D$$

4) Egg shaped Section:-

→ when the tunnel is stratified, soft & very closely laminated & where rock fall are caused due to high external pressure & tensile stress, egg shaped section may be considered.

$$A = 0.864 D^2$$
$$P = 3.313 D^2$$

Note: Generally, pressure tunnel is made of circular section

* **Stress in Tunnel:-**

→ The various forces acting in tunnel are
1) Lateral active earth pressure
2) Reaction pressure due to elastic deformation of the beneath medium.
3) Overburden Pressure
4) Hydrostatic Pressure
5) Earthquake pressure.

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Hardness coefficient of rock:-

→ The hardness of rock ~~present~~ depends upon the hardness of mineral present in it.

→ The best practical methods of estimating a mineral's hardness is Mohs' scale of hardness.

Talc	→	1
Gypsum	→	2
Calcite	→	3
Fluorite	→	4
Apatite	→	5
Feldspar	→	6
Quartz	→	7
Topaz	→	8
Corundum	→	9
Diamond	→	10

Increasing Hardness ↓

Hydraulic Design of Tunnel

(a) Non-pressurized Tunnel:-

→ Design of non-pressurized tunnel is similar to design of canal.

$$Q = \frac{1}{n} \times A \times R^{2/3} \times S^{1/2} \text{ (Manning's Eqn)}$$

(b) Pressurized Tunnel:

→ The design of pressurized Tunnel is computed as pipe flow.

$$Q = A * V$$

$$h_f = \frac{fLV^2}{2gd}$$

$V \rightarrow$ 2–2.5 m/s → unlined tunnel
4–5 m/s → Concrete lined
< 9 m/s → Maxm

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Tunneling Methods:-

(a) Cut & Cover Method:-

→ In this method a trench is cut in the soil & it is covered by some support which can be capable of bearing load on it. The cutting can be done by two methods. 1) Bottom-up Method 2) Top-down method

→ The tunnel is designed as rigid frame box structure.

→ Most of the underground metro rail stations are constructed using cut & cover method.

(b) Drill & Blast Method:-

→ Most common method of tunnel construction in Nepal.

→ The tunnel construction is done by drilling, blasting, mucking & hauling.

→ Steps

- Holes are drilled in tunnel face
- Holes are blown clean & then hand packed with explosives detonators
- Explosives are detonated.
- Tunnel is ventilated
- Rock is scaled
- Muck is removed.
- Support is installed.

(c) Tunnel Boring Method.

→ This method is based on modern technology.

→ In this method, TBM's are used which automatically work makes the entire tunneling process easier.

→ Tunnel Boring Machine (TBM) are available in different types suitable for different ground conditions.

→ A special, pressurized compartment is provided for TBM to work in below water table conditions.

→ The workers should not enter that compartment except for repair works.

→ Muck is usually removed by a number of buckets on the cutting head & dropped onto a conveyor belt system.

(d) Shaft Method:

→ In this method, tunnel is constructed at greater depth from ground surface. The shaft is built up to the depth where tunnel is required.

→ Shaft is permanent structure which is like well with concrete walls.

→ At required depth tunnels are excavated using TBM.

→ Shafts are provided at both inlet & outlet of the tunnels. Intermediate shafts are also provided if tunnel is too long.

→ After the construction process, these shafts can also be used for ventilation purpose as well as emergency exits.

e) Heading & Benching Method

→ In this method, workers dig a smaller Tunnel known as heading. Once, the top heading has advanced some distance into the rock, workers begin excavating immediately below the floor of top heading. this is a bench.

f) Pipe Jacketing Method:

→ Used to construct tunnels under existing structures like roadways, railways etc.

→ In this method, specially made pipes are driven into underground using hydraulic Jacks.

→ Max^{m} size of 3.2m diameter is allowed for tunnels.

\# Supports in Tunnel:

(a) Steel Ribs:

→ Steel ribs give rigid support to the tunnel. They are made from I-beam or H beam bent to confirm to the requirement of a particular tunnel cross section.

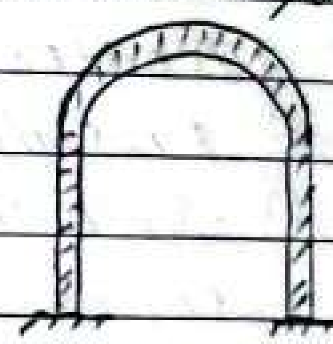

Fig: Steel Ribs

(b) Rock Bolts:-

→ Flexible method, commonly used in tunnel

→ Rock bolts are frequently used as initial support at the tunnel face to obtain safe working conditions to the crew & they also form part of final rock support.

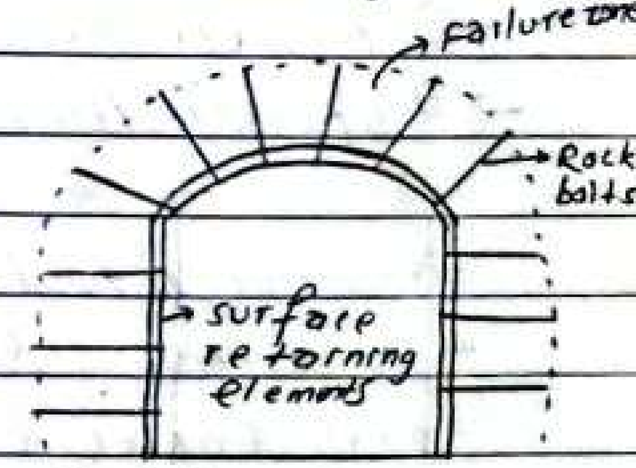

Fig: Rock Bolt support

(c) Timber Support:-

→ To provide propping or temporary support & to provide protective covering of the ribs.

(d) Wire Mesh:-

→ Heavy steel wire mesh & steel mats are used to contain the fragmented rock in the tunnel. They are pinned in the

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place with short, grouted steel pins or with rock bolts.

(e) Grouting:-
→ Grouting is a mixture of cement & water forced into rocks around the tunnel periphery to add strength to fractured rocks & intercept water flows.

※ Lining of Tunnel:-
→ After excavation of tunnel, lining is done to increase hydraulic capacity of the tunnel to reduce resistance, to increase strength & reduce losses from tunnel.
→ Advantages
1) Provide strength & stability
2) Hydraulic resistance of lined tunnel is much less than unlined tunnel.
3) Higher permissible velocity
4) Minimize the danger of accidental rock falls in the tunnel cavity
5) Reduce seepage loss through tunnel
→ Types of tunnel lining :

(a) Shotcrete Lining:
→ Shotcrete is the generic name for cement, sand & fine aggregate concrete which is applied pneumatically & compacted dynamically under high velocity.
→ Shotcrete is used for support of underground excavation
→ It may be dry shotcrete, or wet shotcrete.

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(b) Plain Concrete Lining:-
→ The tunnels are lined with plain concrete when tunnel walls are to support outside pressure due to rock but low pressure inside.

(c) Reinforced Concrete Lining:-
→ When internal pressure is high, PCC lining is inadequate & tension cracks are developed in lining. In such case, the tunnel can be reinforced to take up the tensile stress

(d) Steel lining:-
→ Steel lining is provided to provide more tensile strength to the tunnel.

※ Forebay:-
→ Structure located at the beginning of penstock shaft that supplies the required flow to the turbine during startup, accomodate the rejected flow & reduce water hammer effect

* Functions of forebay:-
→ Allow transition from open channel flow to pipe flow
→ Regulate flow into penstock
→ Release water hammer pressure
→ Serve as secondary settling basin
→ Provide water storage for use during peak hour.

* Components of forebay:-
→ Entrance bay or basin, spillway, trashrack, flushing sluice, valve/gate & penstock.

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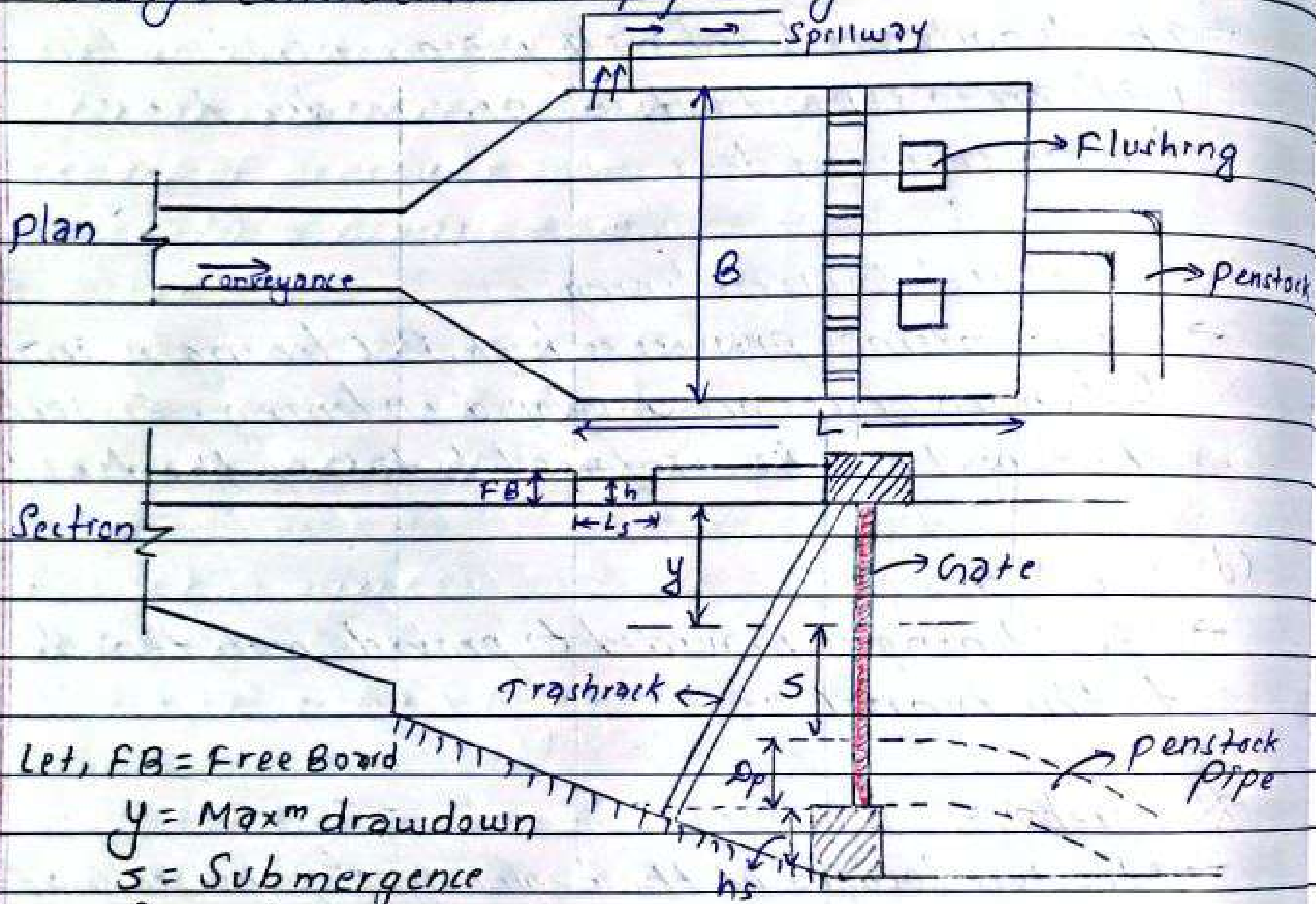
* Design Consideration of forebay:-



Let, FB = Free Board

y = Max^m drawdown

S = Submergence

D_p = dia. of penstock

h_s = Settling depth

1) The velocity of flow in forebay is assumed to be in the range of 0.1 − 0.2 m/s

2) To avoid air entrainment, submergence (S) is provided selecting max^m values out of 3 formulas

$$S > 1.5 D_p$$

$$S > 1.5 \frac{V_p^2}{2g}$$

$$S > 0.54\, V_p \sqrt{D_p}$$

where, V_p = velocity of flow through penstock.

3) Maximum drawdown is given by, $y = V_p \sqrt{\frac{A_p \times L_p}{A_{FB} \times g}}$

where, A_p = area of penstock pipe

L_p = length of penstock pipe

A_{FB} = plan area of forebay = $L \times B$

4) Effective depth of forebay,

H = dia of penstock (D_p) + Submergence (S) + Max^m drawdown (y)

Overall depth (H_o) = Effective depth (H) + Settling depth (0.5 − 1m) + Freeboard (0.5 − 1m)

5) Length of Spillway:

$$L_s = \frac{Q}{C_D h^{3/2}}$$

where, C_D = coeff. of discharge (1.6 to 2.2)

= 1.7 for sharp crest

h = head on crest

6) Storage volume, $V = Q * T$ $T \rightarrow (2-3)$ min

※ Design Steps

1) Calculate submergence (S)
2) Calculate max^m drawdown (y)
3) Assume H as effective depth, $H = D_p + S + y$
4) Express A_{FB} in terms of H, $Q \times T = A_{FB} \times H \Rightarrow A_{FB} = Q \times T/H$
5) Solve for H
6) Assume mean velocity of flow
7) Calculate width (B), $B = Q/VH$

   If 'B' is changed, Check for velocity (0.1 − 0.2 m/s)
8) Calculate length, $L = (Q \times T)/(B_{adopt} * H)$
9) Calculate length of spillway, check if $L_s < L$
10) Calculate $H_o = H + h_s + FB$
11) Draw plan & section

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Q. Design a fore-bay structure for the following data.
Design discharge = $20\ m^3/s$
Storage requirement = 4 min
Length of penstock = 200m
Diameter of penstock = 2m

→ Given,
$Q = 20\ m^3/sec$ $T = 4\ min$
$L_p = 200m$ & $D_p = 2m$

Velocity of flow through penstock $(V_p) = \frac{Q}{A_p} = \frac{20}{\pi \times 2^2/4} = 6.36\ m/s$

For submergence (S)
i) $1.5 D_p = 1.5 \times 2 = 3m$
ii) $1.5 \frac{V_p^2}{2g} = 1.5 \times \frac{6.36^2}{2 \times 9.81} = 3.098m$
iii) $0.54 V_p \sqrt{D_p} = 0.54 \times 6.36 \times \sqrt{2} = 4.85m$
$\therefore S = 4.85m$

Max^m drawdown $(y) = V_p \sqrt{\frac{A_p L_p}{A_{FB} \times g}}$
$= 6.36 \times \sqrt{\frac{\pi \times 2^2}{4} \times \frac{200}{A_{FB} \times 9.81}}$
$= \frac{50.899}{\sqrt{A_{FB}}}$

Effective depth $(H) = D_p + S + y$
$= 2 + 4.85 + \frac{50.899}{\sqrt{A_{FB}}}$
$= 6.85 + \frac{50.899}{\sqrt{A_{FB}}}$ --- (1)

Also, $Q \times T = A_{FB} \times H$

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$\Rightarrow 20 \times 4 \times 60 = A_{FB} \times \left(6.85 + \frac{50.899}{\sqrt{A_{FB}}}\right)$
$A_{FB} = 529.71\ m^2$

Then,
$H = 6.85 + \frac{50.899}{\sqrt{529.71}} = 9.061m$

Assume mean velocity of flow $(V) = 0.15\ m/s$
$\therefore B = \frac{Q}{VH} = \frac{20}{0.15 \times 9.061} = 14.71m$

Adopt, $B = 15m$,
$V = \frac{Q}{BH} = \frac{20}{15 \times 9.061} = 0.147\ m/s$ (0.1 to 0.2 m/s) ok

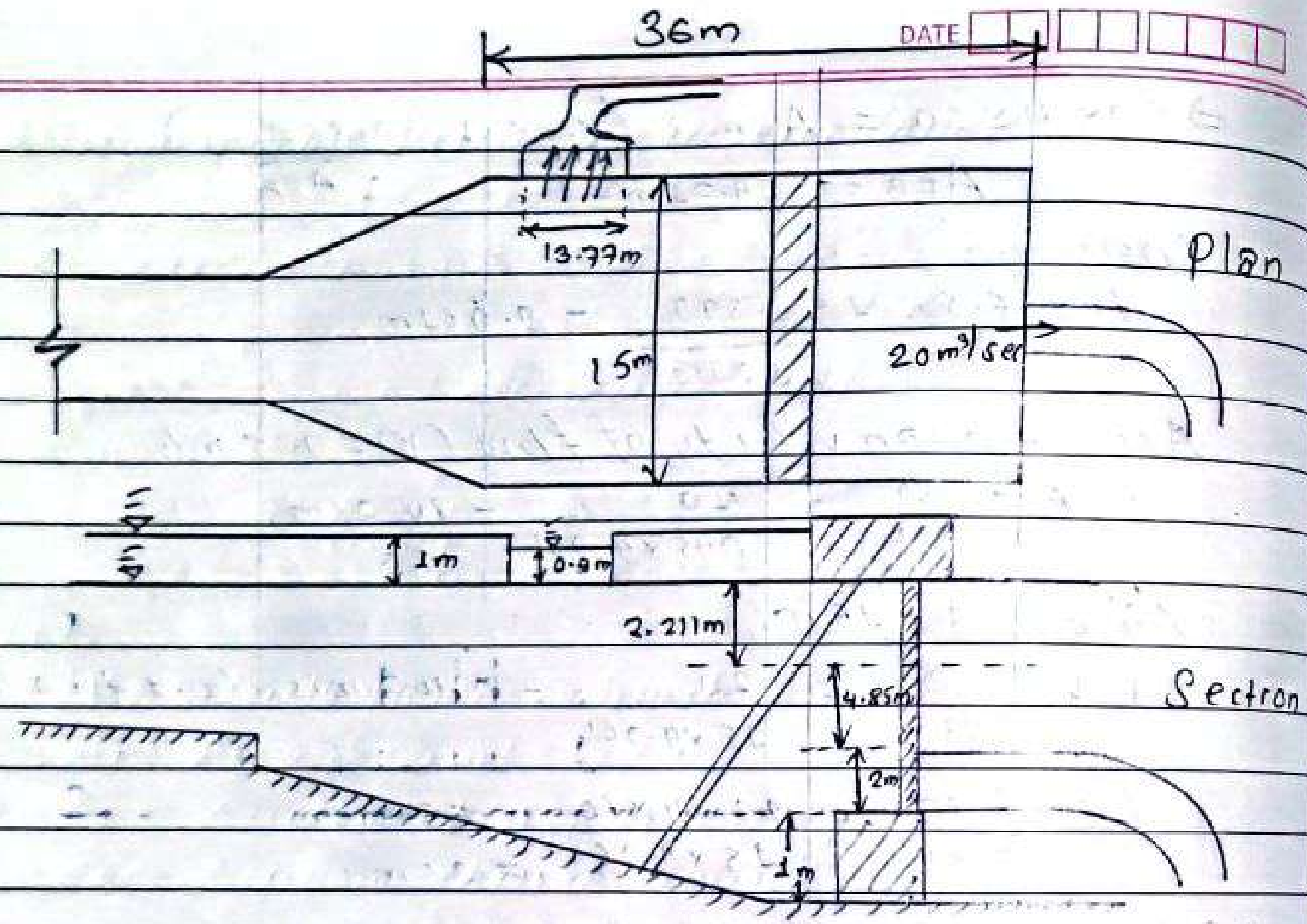
$L = \frac{Q \times T}{B \times H} = \frac{20 \times 4 \times 60}{15 \times 9.061} = 35.316m \approx 36m$

→ Length of spillway,
Let freeboard (FB) = 1m
Flowdepth in spillway (h) = 0.9m
$C_D = 1.7$ for sharp crest
$L_s = \frac{Q}{C_D \times h^{3/2}} = \frac{20}{1.7 \times 0.9^{3/2}}$
$= 13.77m < L$ (ok)

Overall depth (H_o)
$H_o = H + h_s + FB$
$= 9.061 + 1 + 1$
$= 11.061m$

Note: If no. of penstock pipers provided, V_p chang[es]
$V_p = \frac{Q_D}{A_p}$
$V_p = \frac{Q_{design}/n}{A_p}$
where, n = no. of penstock

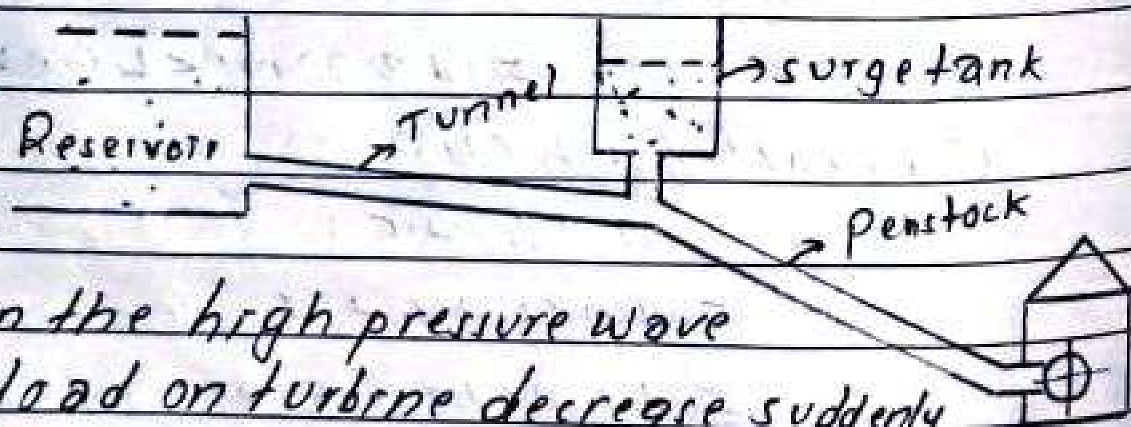
$\therefore$ Size of Forebay,
$L_{eff} \times B \times H_o = 36m \times 15m \times 11.061m$.



※ Surge Tank:-

→ Surge tank is the open topped storage structure which is connected to the penstock at a suitable location.

→ The selection of surge tank or forebay is governed by the topography.



* Functions:-

→ To intercept & damen the high pressure wave

→ To store water when load on turbine decrease suddenly

→ Temporarily supply water when the load on turbine is suddenly increased.

→ To provide protection to the penstock against detrimental effects of water hammer.

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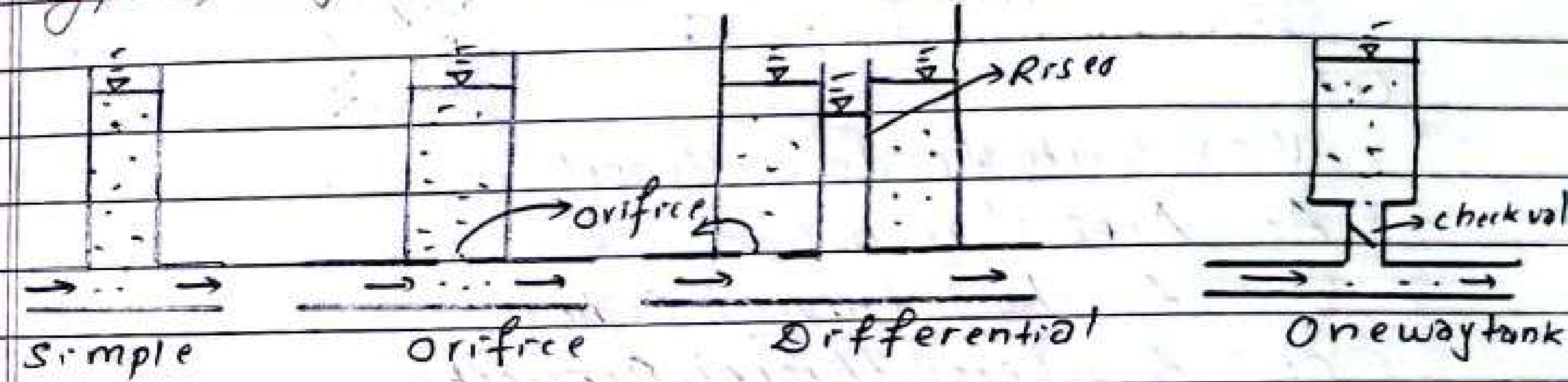
* Location of Surgetank:-

→ A surge tank is always located as close as possible to the powerhouse in order to reduce the length of penstock to a minimum & preferably on high ground to reduce height to tower.

※ Types of Surge Tank:



(a) Simple Surge Tank:-

→ Uniform crosssection & opens to atmosphere, acting as reservo

→ directly connected to penstock & has unrestricted opening into it

→ Usually large in size & hydraulic action is sluggish

(b) Restricted Orifice Type:-

→ Restricted orifice in betⁿ pipeline & surgetank

→ Allowed rapid pressure changes in pipeline due to head loss at orifice.

(c) Differential Surge tank:-

→ Consists of internal narrow riser shaft with orifice entry to the larger outer shaft at bottom. The head building functio is achieved through riser shaft & storage function is achieved through outer shaft.

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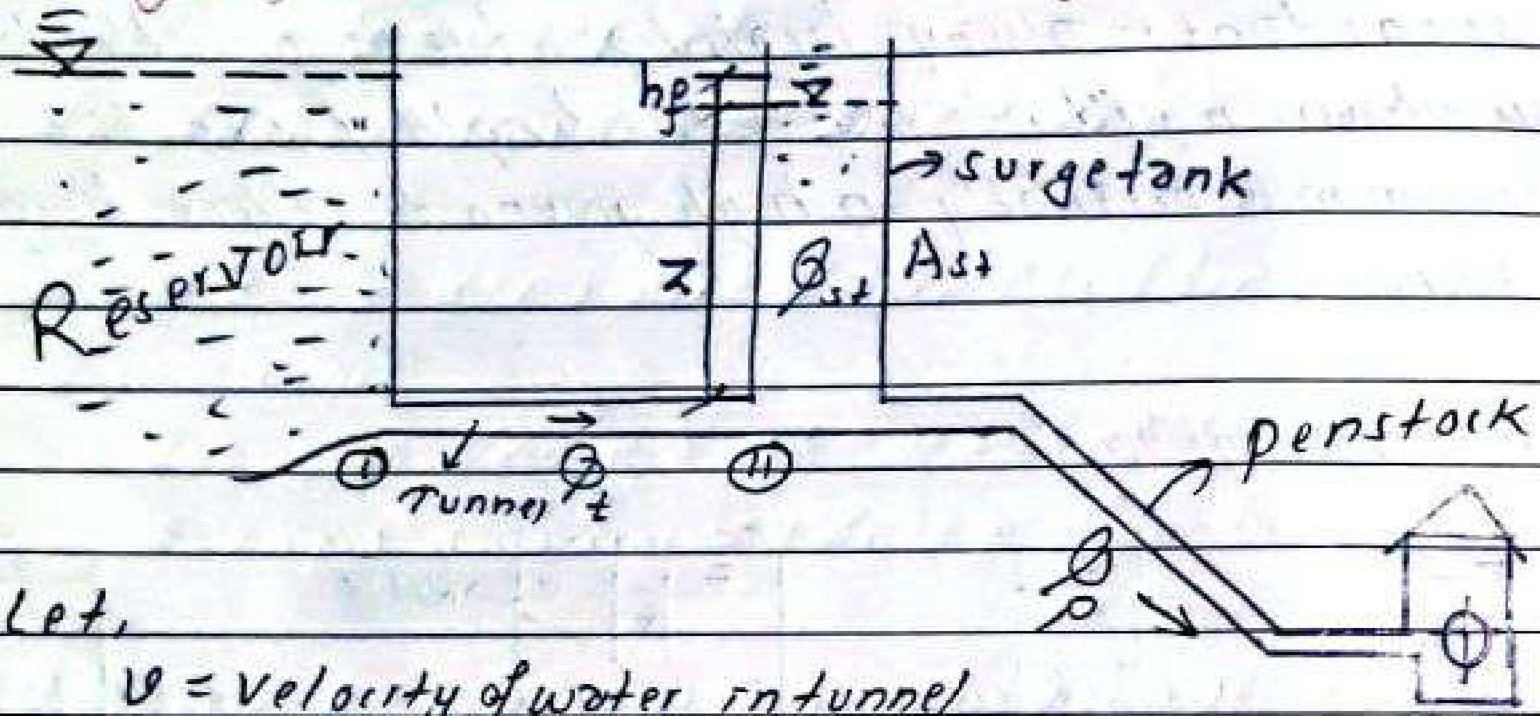
* Design of mass oscillation in Surge Tank:-



Let,

v = Velocity of water in tunnel

A_T = Area of Tunnel

Q_T = Discharge through Tunnel

Q_p = Discharge through penstock

Using momentum eqn betn (I) & (II)

$$P_{II} * A_T + \rho g z A_T - P_I \times A_T = m\frac{dv}{dt}$$

Assuming $P_I = P_{II}$ (Neglecting friction)

$$\rho g z A_T = m\frac{dv}{dt}$$

$$\rho g z A_T = \rho \times A_T \times L \times \frac{dv}{dt}$$

$$\frac{dv}{dt} = -\left(\frac{g}{L}\right) \times z$$

-ve sign indicates retardation.

Using continuity eqn at (II)

$$Q_T = Q_{ST} + Q_p$$

$$\frac{dQ_T}{dt} = \frac{dQ_{ST}}{dt} + \frac{dQ_p}{dt}$$

For sudden closure of valve, $Q_p = 0$,

$$\frac{dQ_T}{dt} = \frac{dQ_{ST}}{dt}$$

$$A_T \times \frac{dv}{dt} = A_{ST} \times \frac{dv_{ST}}{dt}$$

We have, $v_{st} = \frac{dz}{dt}$, so, $\frac{dv_{ST}}{dt} = \frac{d^2z}{dt^2}$

Then, the continuity eqn becomes,

$$\frac{d^2z}{dt^2} + \frac{A_T L_T}{A_{ST} g} \times Z = 0$$

Solving above differential eqn, we get,

$$Z = \frac{Q_T}{A_{ST}} \times \sqrt{\frac{A_{ST} L_T}{A_T g}} \sin\left(\sqrt{\frac{A_t g}{A_{ST} l_T}} * t\right)$$

The maximum upsurge is,

$$Z_{max} = \frac{Q_T}{A_{ST}} * \sqrt{\frac{A_{ST} \times L_T}{A_T * g}} \quad \text{when } \sin\sqrt{\frac{A_t g}{A_{st} L_T}} * t = 1 = \sin\frac{\pi}{2}$$

$$\therefore t = \frac{\pi}{2}\sqrt{\frac{A_{ST} L_T}{A_T g}}$$

Time period of oscillation is given as,

$$T = 4t = 2\pi\sqrt{\frac{A_{ST} L_T}{A_T g}}$$

* Approximate Solution with friction by JAEGER

(a) For 100% load rejection (sudden closure)

(i) Upsurge

$$Z^*_{max} = 1 - \frac{2}{3} K_o^* + \frac{1}{9}(K_o^*)^2$$

$$K_o^* = \frac{h_f}{Z_{max}}$$

h_f = Head loss between reservoir & surge tank $= \frac{fLv^2}{2gD}$

$$(Z_{damped})_{upserge} = Z_{max} * (Z_{max})^*$$

(ii) Downsurge,

$$Z^{*}_{min} = -1 + 2K_0^{*}$$

$$(Z_{damped})_{downsurge} = Z_{min} * (Z^{*}_{min}) \quad \text{where } Z_{min} = Z_{max}$$

(b) For 100% load Acceptance (sudden Open)

Downsurge,

$$Z^{*}_{min} = -1 - 0.125K_0^{*}$$

$$(Z_{damped})_{downsurge} = Z_{min} * Z^{*}_{min} \quad \text{Where } Z_{min} = Z_{max}$$

→ Thoma Formula for minimum area of surge tank

$$A_{ST} \geq \frac{A_T \times L_T \times v^2}{2g h_f \times (H_g - h_f)}$$

Where,

A_T = Area of Tunnel

L_T = Length of Tunnel

v = Velocity of flow in tunnel

h_f = Head loss betn surge tank & reservoir

H_g = Gross head

→ Factor of safety regarding area of surge tank,

$$FOS = \frac{\text{Provided area of Surge tank}}{\text{Min}^m \text{ reqd area of surge tank.}}$$

→ Height of surgetank is given by,

$$H = (Z_{damped})_{upsurge} - (Z_{damped})_{downsurge} + h_f + FB\,(2m)$$

Q. A hydroelectric power canal is fed through a concrete lined tunnel of 4.9m diameter operating under a gross head of 195m. The ~~dis~~ discharge through the tunnel is 28 cumecs. A surge tank of 295m² area has been provided at the end of tunnel. Head loss due to friction is 2% of gross head. Assuming friction factor of tunnel as 0.015, find (assuming sudden closure)

a) Total Length of the tunnel

b) Max^m upsurge

c) Max^m downsurge

d) FOS of cross section of surge tank

e) Height of surge tank (free board 2m)

f) Time period of Oscillation

→ Given,

diameter of tunnel (d) = 4.9m

Gross Head (H_g) = 195m

Discharge through tunnel (Q_T) = 28 m³/s

Area of Surge Tank (A_{ST}) = 295 m²

Head loss due to friction (h_f) = 0.02 × 195 = 3.9m

frictional factor = 0.015

Sudden closure of valve.

Now,

$$A_T = \frac{\pi \times 4.9^2}{4} = 18.85\, m^2$$

$$v = \frac{Q_T}{A_T} = \frac{28}{18.85} = 1.485\, m/s$$

$$h_f = \frac{f L_T v^2}{2g D_T} \Rightarrow 3.9 = \frac{0.015 \times L_T \times 1.485^2}{2 \times 9.81 \times 4.9}$$

$$\Rightarrow L_T = 11.334\, KM$$

$$Z_{max} \text{ or } Z_{min} = \frac{Q_T}{A_{ST}} \sqrt{\frac{A_{ST}}{A_T} \times \frac{L_T}{g}}$$

$$= \frac{28}{295} \sqrt{\frac{295}{18.85} \times \frac{11.334 \times 1000}{9.81}}$$

$$= 12.76m$$

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$$K_0^* = \frac{h_f}{Z_{max}} = \frac{3.9}{12.76} = 0.3055$$

$$Z_{max}^* = 1 - \frac{2}{3} \times 0.3055 + \frac{1}{9} \times 0.3055^2 = 0.8066$$

$$Z_{min}^* = -1 + 2 \times 0.3055 = -0.389$$

Then,

$$(Z_{damped})_{upsurge} = 12.76 \times 0.8066 = 10.3\,m$$

$$(Z_{damped})_{downsurge} = 12.76 \times (-0.389) = -4.96\,m$$

→ Factor of safety of cross section of Surge Tank,

$$FOS = \frac{295}{A_{ST,min}}$$

$$A_{ST,min} = \frac{A_T \times L_T \times V^2}{2g\,h_f \times (H_g - h_f)}$$

$$= \frac{18.85 \times 11334 \times 1.485^2}{2 \times 9.81 \times 3.9\,(195 - 3.9)}$$

$$= 32.219\,m^2$$

Then, $FOS = \frac{295}{32.219} = 9.15$

→ Height of Surge Tank $= \cancel{}\ (Z_{damped})_{upsurge} + (Z_{damped})_{down} + FB$

$$= 10.3 + 4.96 + 2$$

$$= 17.26\,m$$

→ Time period of oscillation,

$$T = 2\pi \sqrt{\frac{A_{st}\,L_T}{A_T \times g}}$$

$$= 2\pi \sqrt{\frac{295 \times 11.334 \times 1000}{18.85 \times 9.81}}$$

$$= 844.87\ sec.$$

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✱ Penstock:-

→ It is a pipe that conveys water from forebay or surgetank to powerhouse. Since, large head conversion takes place within small distance, penstock is usually made with strong material like steel.

→ Flow velocity is high in penstock pipe.

→ Penstock maybe located underground or on surface depending upon topography. For exposed penstock, concrete blocks called anchor blocks are made to hold the penstock.

✱ Water Hammer Pressure:-

→ The dynamic rise or fall of water pressure due to sudden obstruction of flow is called water hammer pressure.

→ Causes of waterhammer

1) Valve Operation
2) Power Failures
3) startup or shutdown of pumps
4) Fluctuation in powers
5) Mechanical failure of control devices

→ Effect of water hammer pressure

1) High pressure fluctuation in pipeline
2) Rupture of pipe or valve if beyond safety limit
3) Higher pressure requirements for pipedesign.

✱ Water Hammer Pressure Computation:-

→ Let 't' be the time of closure of valve. To study the pressure change with time at a point in the pipeline the valve closure time can be classified as,

1) Instantaneous closure ($t = 0$)

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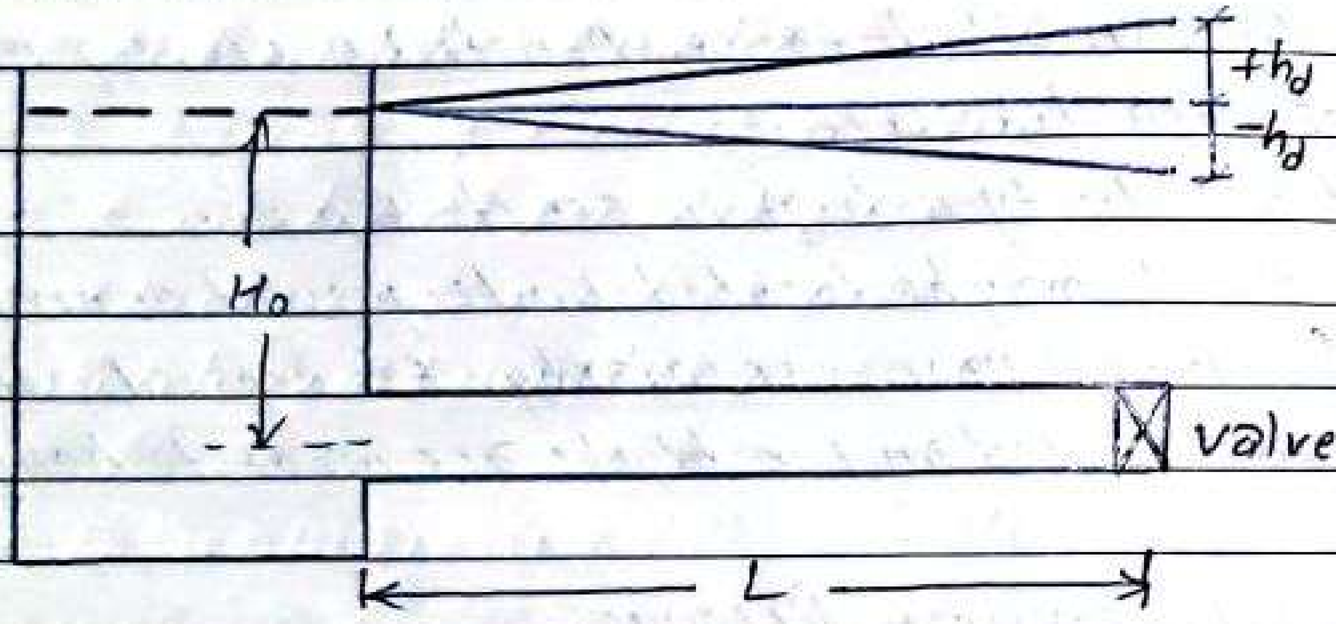
2) Rapid closure ($t \le 2L/c$)
3) Gradual Closure ($t > 2L/c$)



There are two theories for water hammer pressure computation.

(a) Rigid Water Column Theory:-

→ This theory is applicable to gradual closure of valve. Let H_0 be the static head in the reservoir & h_d be the dynamic head created due to water hammer. Force on the valve due to change in moment

$$F_1 = \frac{m(v-0)}{t} = \frac{\gamma A L v}{g t} \quad \text{---(i)}$$

Increase in pressure due to water hammer,

$$F_2 = \gamma h_d A \quad \text{---(ii)}$$

Equating eqn (i) & (ii),

$$\frac{\gamma A L v}{g t} = \gamma h_d A$$

$$\Rightarrow \text{Dynamic head } (h_d) = \frac{L v}{g t}$$

Water hammer pressure, $p_H = \gamma h_d = \frac{\gamma L v}{g t} = \frac{\rho L v}{t}$

According to Dandekar & Sharma,

$$\frac{(h_d)_{max}}{H_0} = \frac{k_1}{2} \pm \sqrt{k_1 + \frac{k_1^2}{4}} \quad \text{where, } k_1 = \left(\frac{L v}{g H_0 t}\right)^2$$

For small value of k_1, $\frac{(h_d)_{max}}{H_0} = \frac{k_1}{2} \pm \sqrt{k_1}$

Hence, water hammer pressure, $p_H = \gamma (h_d)_{max}$.

Total pressure is given by,

$$p = p_s + p_D$$
$$= \gamma H_0 + \gamma (h_d)_{max}$$

(b) Elastic Water Column Theory (EWCT)

→ This theory is applicable to rapid & instantaneous closure of valve.

From previous theory, $p_H = \frac{\rho L v}{t}$

For instantaneous & rapid closure, $t \to 0$, $p_H \to \infty$

But this is not true because the moving water in a pipeline of length 'L' cannot be brought to rest in time lesser than it takes sound wave to travel in water from the valve along the pipe upto reservoir.

The minimum time in which the water can be brought to rest is,

$$t_{min} = \frac{L}{C} \Rightarrow C = \frac{L}{t_{min}}$$

So, water hammer pressure, $p_H = \frac{\rho L v}{t_{min}} = \frac{\rho L v}{L/C} = \rho v C$

$$\Rightarrow p_H = \gamma \times \left(\frac{v C}{g}\right)$$

So, $h_d = vC/g$

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where, c = wave celerity = $\sqrt{\dfrac{(k/\rho)}{1+Dk/tE}}$

k = Bulk modulus of water

E = Elasticity of penstock material

D = Diameter of penstock pipe

t = Thickness of penstock pipe.

For absolutely rigid penstock pipe, $c = \sqrt{k/\rho}$

Thus, total water pressure $(P) = \gamma H_0 + \gamma\left(\dfrac{Vc}{g}\right)$

* Thickness of penstock pipe:-

→ The stress developed in penstock pipe are longitudinal stress & hoop stress.

Longitudinal stress $(\sigma_L) = PD/4t$

Hoop stress $(\sigma_c) = PD/2t$

→ The hoop stress is greater than longitudinal stress, hoop stress is used for design of penstock pipe. so, thickness of penstock is given by,

$t = \dfrac{PD}{2\sigma_a}$, σ_a = Allowable stress

Considering joint efficiency, $t = \dfrac{PD}{2\sigma_a \eta}$

→ According to ASME code (American Society of Mechanical Engineering)

$$t = \left(\frac{PR}{\sigma_a\eta - 0.6P} + \underbrace{0.15}_{\text{corrosion allowance}}\right) \text{cm}$$

where, P = total pressure in kg/cm²

R = Radius of penstock pipe in cm

σ_a = Allowable stress in kg/cm²

η = Joint efficiency.

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Q) A steel penstock 500m long with internal diameter 0.8m to supply water at 4.2 m³/s for a head of 120m. Find the wall thickness of penstock pipe for 40s closure time. Assume wave velocity 1100 m/s. Allowable stress of pipe material is 1300 kg/cm² & Joint efficiency is 90%

→ Given,

Length of penstock pipe (L) = 500 m

Internal dia of penstock pipe = 0.8m

discharge (Q) = 4.2 m³/s

Head (H_0) = 120m

Closure time of valve (t) = 40s

Wave velocity (c) = 1100 m/s

Allowable stress in pipe material (σ_a) = 1300 kg/cm²

Joint Efficiency (η) = 90%

Here,

$\dfrac{2L}{c} = \dfrac{2\times 500}{1100} = 0.91$ sec

$t > 2L/c$, valve is closed gradually.

→ Total pressure $(P) = P_s + P_d$

$= \gamma H_0 + \gamma (h_d)_{max}$

$(h_d)_{max} = H_0\left[\dfrac{k_1}{2} \pm \sqrt{k_1 + \dfrac{k_1^2}{4}}\right]$ where, $k_1 = \left(\dfrac{LV}{H_0 g t}\right)^2$

$k_1 = \left(\dfrac{500 \times 8.35}{120 \times 9.81 \times 40}\right)^2$ $\quad V = \dfrac{Q}{A} = 8.35$ m/s

$= 0.00786$

$(h_d)_{max} = 120\left[\dfrac{0.00786}{2} + \sqrt{0.00786 + \dfrac{0.00786^2}{4}}\right]$

$= 11.12$m

Now, $P = \gamma H_0 + \gamma h_{d,max}$

$= 9.81 * 120 + 9.81 \times 11.12$

$= 1286.28\ kN/m^2$

$= \frac{1286.28 \times 10^3}{9.81 \times 10^4}\ kg/cm^2$

$= 13.112\ kg/cm^2$

The thickness of penstock pipe is,

$$t = \left(\frac{PR}{\sigma_0 \eta - 0.6P} + 0.15\right) cm$$

$$= \left(\frac{13.112 \times 40}{1300 \times 0.9 - 0.6 \times 13.112} + 0.15\right)$$

$= 0.601$ cm

* Economic diameter of penstock pipe:-

→ With the increase in diameter of penstock, the head loss decreases as given by Darcy's formula ($h_f = 8fLQ^2/\pi^2 g D^5$) & hence revenue loss decreases. At the same time, with the increase in diameter of the penstock, the cost of pipe increases. Thus, the diameter of the penstock pipe that optimizes these two costs is called economic diameter of penstock.

→ Determination of economic diameter of penstock

(a) Empirical Formula:-

1) Sarkaria Formula:-

$$D_e = 0.62 \frac{P^{0.35}}{H^{0.65}}$$

Where, P = power in Hp, 1 Hp = 746 watts

H = Head in 'm'.

2) USBR Method,

$$V_{eco} = 0.125\sqrt{2gH}$$

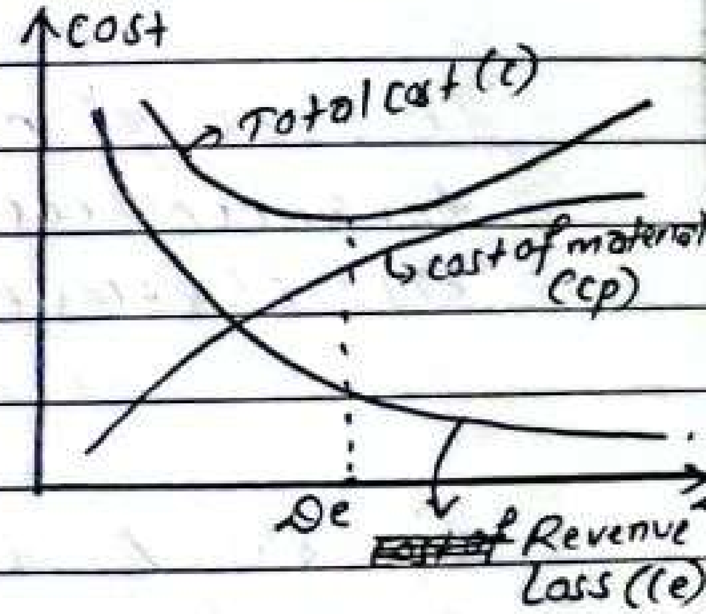
$$D_e = \sqrt{\frac{4Q}{\pi V_{eco}}}$$

3) Design Guidelines by JNN:-

$$D = \sqrt[7]{\frac{5.2 Q_d^3}{H_d}}$$ where, H_d = Dynamic Head (m)

(b) Analytical Method:-

→ In this method, the annual cost of the pipe & annual revenue loss are analytically calculated in terms of diameter of penstock. The total cost is differentiated with respect to diameter & equated to zero to get the economic diameter of penstock pipe.



Head loss through penstock pipe

$$h_f = \frac{fLv^2}{2gD} = \frac{8fLQ^2}{\pi^2 g D^5}$$

Corresponding power loss,

$$P = \eta \gamma Q h_f = \eta \gamma Q \times \frac{8fLQ^2}{\pi^2 g D^5}$$

$$\Rightarrow P = 9.81 \eta \frac{8fLQ^3}{\pi^2 g D^5}\ (kW)$$

Annual energy loss, $E = P * t = 9.81 \eta \times \frac{8fLQ^3}{\pi^2 g D^5} \times 365 \times 24$ (kwhr)

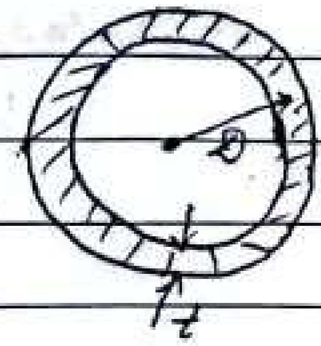
Let Re be the energy rate in Rs per unit,

Revenue loss (Ce) = Energy loss × Rate

$$= 9.81\eta \times \frac{8fLQ^3}{\pi^2 g D^5} \times 365 \times 24 \times Re$$

$$= Rs.\ \frac{A}{D^5}, \text{ where, } A = 9.81\eta \frac{8fLQ^3}{\pi^2 g} \times 365 \times 24 \times R$$

Thickness of pipe $(t) = \frac{pD}{2\sigma}$



Volume of penstock pipe $= \pi D t \times L$

$= \pi D \times \frac{pD}{2\sigma} \times L$

Weight of penstock pipe $= \pi D \times \frac{pD}{2\sigma} \times L \times \rho_s$ (kg)

Let R_p be the cost of pipe per kg, then

Cost of pipe (C_p) = weight * R_p

$C_p = \pi D \times \frac{pD}{2\sigma} \times L \times \rho_s \times R_p$

$C_p = BD^2$, where $B = \frac{\pi p L}{2\sigma} \times \rho_s \times R_p$

This cost of pipe is not annual cost. Thus, to obtain the annual cost of pipe, it must be multiplied by capital recovery factor

$$A = P\left\{\frac{(1+i)^N * i}{(1+i)^N - 1}\right\} \rightarrow CRF$$

Equivalent Annual Cost $= BD^2 \times CRF = ED^2$

Total Annual cost $(C) = \frac{A}{D^5} + ED^2$

For minimum cost, $\frac{dc}{dD} = 0$

$\Rightarrow -\frac{5A}{D^6} + 2ED = 0$

$\Rightarrow -5A + 2ED^7 = 0$

$\Rightarrow 2ED^7 = 5A$

$\Rightarrow D = \left(\frac{5A}{2E}\right)^{1/7}$. This gives the required economic dia. of penstock pipe.

Q. Discharge through a penstock pipe is 10 cumecs & head is 25m. The cost of the pipe is given by $40hD^2$ per meter where 'h' denotes head & D denotes diameter of penstock. Annual fixed charge are 10% of pipeline cost. The head loss in friction is $0.03Q^2/14D^5$ meter per m length of penstock pipe. The turbine efficiency is 90% & annual energy price is Rs 80/kW. Calculate economic diameter of penstock pipe.

→ Given,

$Q = 10\ m^3/s$

$h = 25m$

cost of pipe $= 40hD^2/m$

Annual fixed charge = 10% of cost of pipe

$h_f = \frac{0.03Q^2}{14D^5}$ per m length

$\eta = 0.9$

Annual energy price = 80/kW

Now,

$h_f = \frac{0.03 \times 10^2 \times 1}{14 \times D^5} = \frac{0.2142}{D^5}$

Corresponding power loss $= \eta \gamma Q h_f$

$= 0.9 \times 9810 \times 10 \times \frac{0.2142}{D^5} \times 10^{-3}$ (kW)

$= \frac{18.911}{D^5}$

Annual Revenue loss $= 18.911/D^5 * 80$

$= \frac{1512.937}{D^5}$

Cost of pipe $= 40hD^2 = 40 \times 25 \times D^2$

$= 1000D^2$

Equivalent annual cost per m' = 10% of cost of pipe
$= 100D^2$

Total cost (C) $= 100D^2 + \frac{1512.935}{D^5}$

For minm cost,

$$\frac{dC}{dD} = 0$$

$$\Rightarrow 200D - \frac{5 \times 1512.935}{D^6} = 0$$

$$\Rightarrow 200D^7 = 5 \times 1512.935$$

$$\Rightarrow D = \left(\frac{5 \times 1512.935}{200}\right)^{1/7} = 1.68\,m$$

∴ Economic diameter of penstock (D) = 1.68 m.

Head Loss Calculation in Conveyance System

* Major Head loss

→ For open channel flow, $h_f = \frac{V^2 \times n^2 \times L}{R^{4/3}}$

Where, V = Velocity of water
n = manning's constant = (0.012 − 0.018) for concrete lined tunnel
L = Length of conveyance System
R = Hydraulic Radius

→ For pipe flow,

$$h_f = \frac{fLV^2}{2gD}$$

Where, f = Darcy's friction factor
D = Diameter of tunnel/pipe
L = Length of pipe/tunnel

* Minor Head loss:-

(a) Trash rack loss

$$h_t = k_t \times \frac{V^2}{2g}$$

k_t = loss coeff $= 1.45 - 0.45R - R^2$
R = Ratio of net area to gross area

(b) Entrance loss, $h_e = k_e \times \frac{V^2}{2g}$

Where, k_e = loss coefficient for entrance.

(c) Transition loss,
Expansion loss, $h_{te} = k_{te}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$

Contraction loss, $h_{tc} = k_{tc}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$

(d) Bend loss,

$$h_b = k_b \times \frac{V^2}{2g}$$

(e) Gate loss,

$$h_g = k_g \times \frac{V^2}{2g}$$

k_g = gate loss coeff = 0.1

(f) Exit loss,

$$h_e = k_e \times V^2/2g$$

Where, k_e = loss coefficient for exit (1)

Chapter-7, Spillways & Energy Dissipaters

DATE

Outline:-
→ Spillway:- defn, purpose, types, design specifics (ogee shape) cavitation & preventive measures
→ Method of energy dissipation below the dam structure:-
→ Design of stilling basin

Spillway:-
→ Spillway is a structure constructed at a dam site for effective disposing of surplus water from u/s to d/s.
→ If the flood overtops the earthen dam, eventually the dam fails. so, spillway is essential in every dam storage project.
→ Due to high velocities of spilling water, some form of energy dissipation is usually provided at the base of spillway.

Purpose/ Functions of Spillway:-
→ dispose excess water without causing any damage to dam.
→ works as safety measure against overtopping & consequent damage & failure of dam.

Location of Spillway:
→ Spillway is generally located in the dam body in certain section. This section of dam is called overflow section of the dam.
→ Sometimes, the spillway may also be isolated from dam body specially when dam section is narrow & separate saddle is available. (Saddle Spillway)

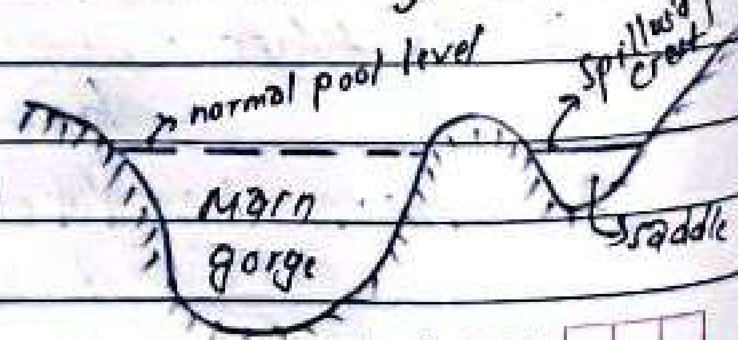


Fig:- Saddle Spillway

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Classification of Spillway:-
(a) Based on control Mechanism
1) Controlled Spillway:-
→ In controlled spillway, mechanical structures or gates are provided to regulate the rate of flow.

2) Uncontrolled Spillway:
→ Does not have gate to control flow.
→ When water rises above crest of spillway, it begins to be released from the reservoir.

(b) Based on purpose
1) Main Spillway:-
→ Designed for safe passage of design maximum flood.

2) Auxiliary Spillway:-
→ Supplementary spillway to main spillway
→ Favorable to provide auxiliary spillway when there is natural saddle existing.

3) Emergency Spillway:
→ When incoming discharge exceeds the maximum design flood or when gate of spillway malfunctions during flood, to avoid serious accident during such condition, emergency spillway is provided.

(c) Based on Prominent feature
(a) Freefall Spillway

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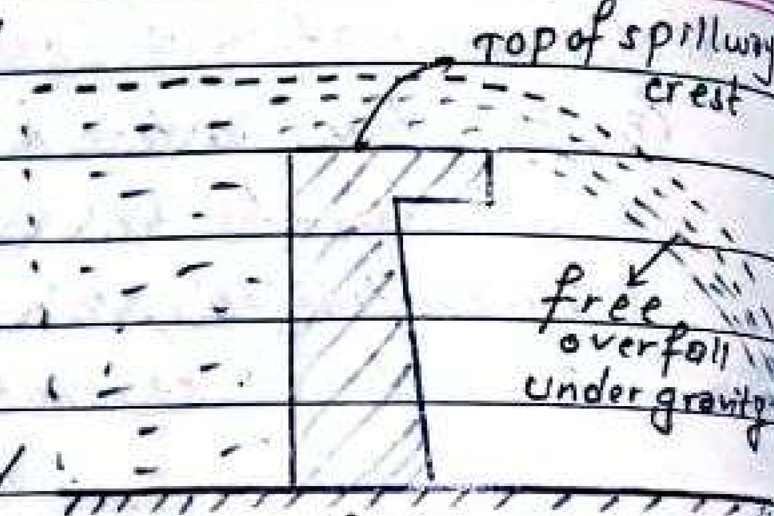
→ Flow drops freely from the crest

→ This type of structure is not adoptable for high drops on unstable foundations because of large impact of forces on d/s.

→ So, a pool of water can be created downstream in order to dissipate energy.



Free fall spillway

(b) Ogee (Overflow Spillways)

→ Modified of vertical drop spillway which has a control weir that is ogee (S) shaped in profile.

→ Generally used in high solid gravity dam.

→ Such spillways can be used on valleys where width of river is sufficiently more to provide sufficient crest length & river bed can be protected from scouring at moderate cost.

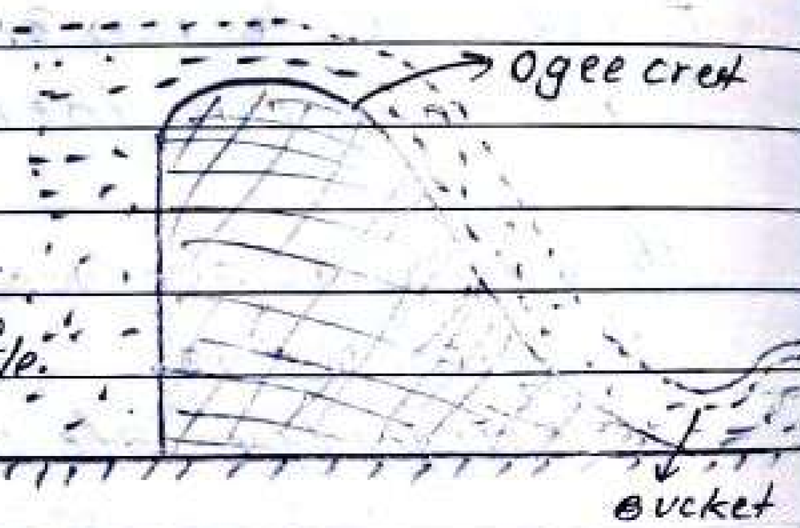
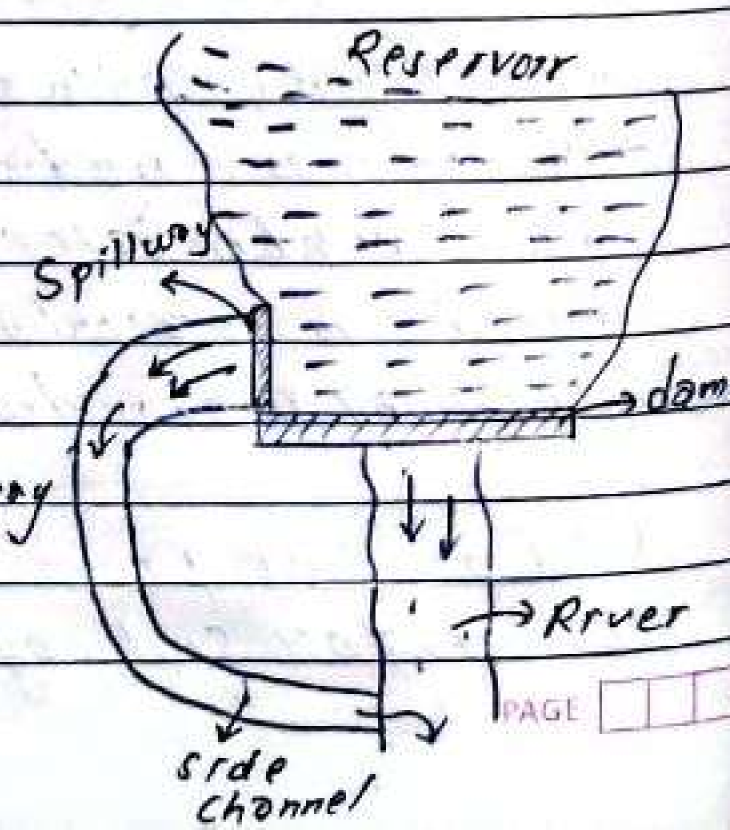


Fig: Ogee Spillway

(c) Side channel Spillway

→ Control weir is approximately parallel to spillway discharge channel

→ When dam is not rigid & if there is no place for construction of spillway on face of dam, side channel spillway is best.



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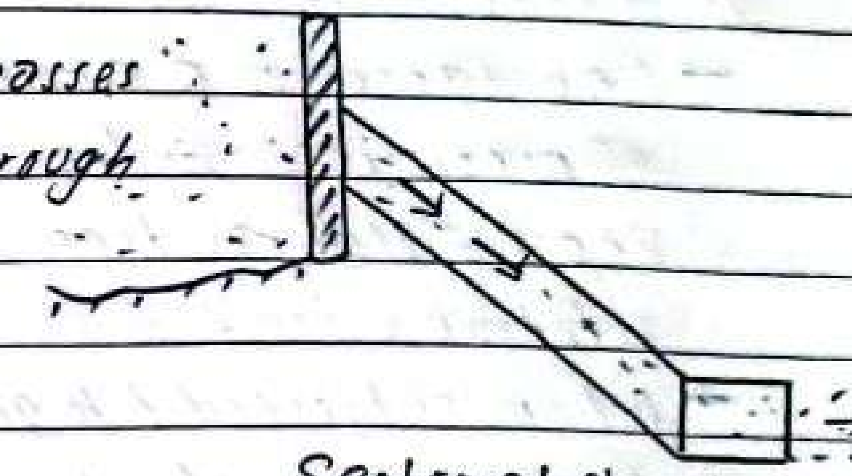
d) Chute Spillway:-

→ A chute or trough spillway passes the surplus water normally through a steep channel placed in dam body or saddle away from the dam.



Sectional elevation of Chute spillway

e) Siphon Spillway:-

→ If the available space is limited & 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.

→ Air vent pipe is provided at crown to prevent stopping of flow through siphon.

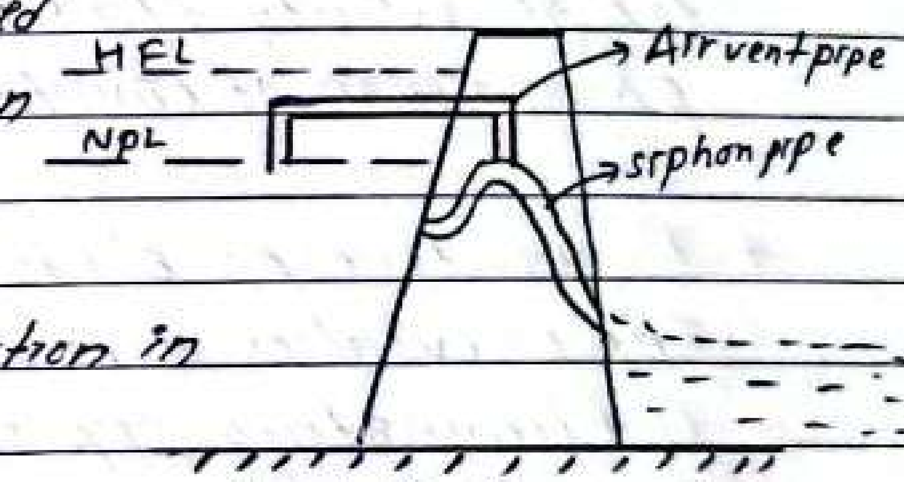


Fig: Siphon spillway

f) Morning Glory / Drop Inlet / Shaft Spillways:-

→ In this spillway, water drops through a vertical shaft & flows downstream river channel through a horizontal conduit. Top level of vertical shaft is kept at normal water level. Excess water enters through the flared vertical shaft & discharges d/s.

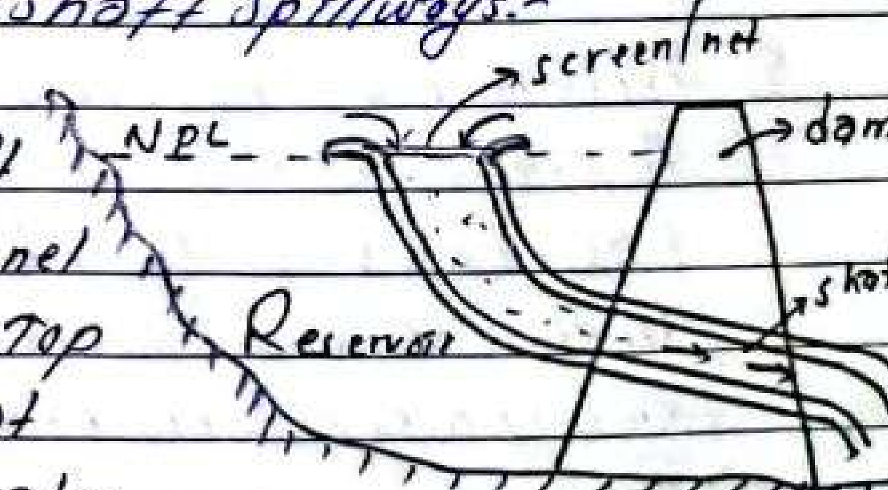


Fig: Morning Glory Spillway

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Cavitation:-

→ Cavitation is a phenomenon in which rapid changes of pressure in a liquid leads to the formation of small vapor filled cavities, in places where the pressure is relatively low.

→ When subjected to high pressure, these cavities collapse & can generate shock wave that is strong.

→ Spillways for medium & high head dams may be exposed to high velocity flows & the associated destructive phenomenon of cavitation.

* Preventive measures of cavitation in Spillway

The prevention of cavitation in spillway can be done by

i) Eliminating opportunities for flow separation

ii) Limiting flow velocities to non cavitating levels

iii) Maintaining sufficiently high operating pressures.

iv) Construction of smooth flow surface etc.

Method of Energy Dissipation:-

→ As stated earlier, water falling through the spillway has high velocity. This velocity may cause serious damage in downstream. Hence, the high energy of water should be dissipated to reduce the damage to the structure in downstream.

→ The energy dissipation takes place by two methods

1) Hydraulic Jump formation

2) Hydraulic diffusion.

Types of Energy Dissipators :-

(a) Hydraulic Jump Type Stilling Basins:-

→ This structure dissipates energy through formation of controlled hydraulic jump within the confines of the basin over the entire range of flow conditions under which basin is expected to operate. The design of this basin shall depend upon type of jump expected.

$$F_r = \frac{V}{\sqrt{gy}}$$

where, V = velocity of flow

y = Depth perpn to stilling basin

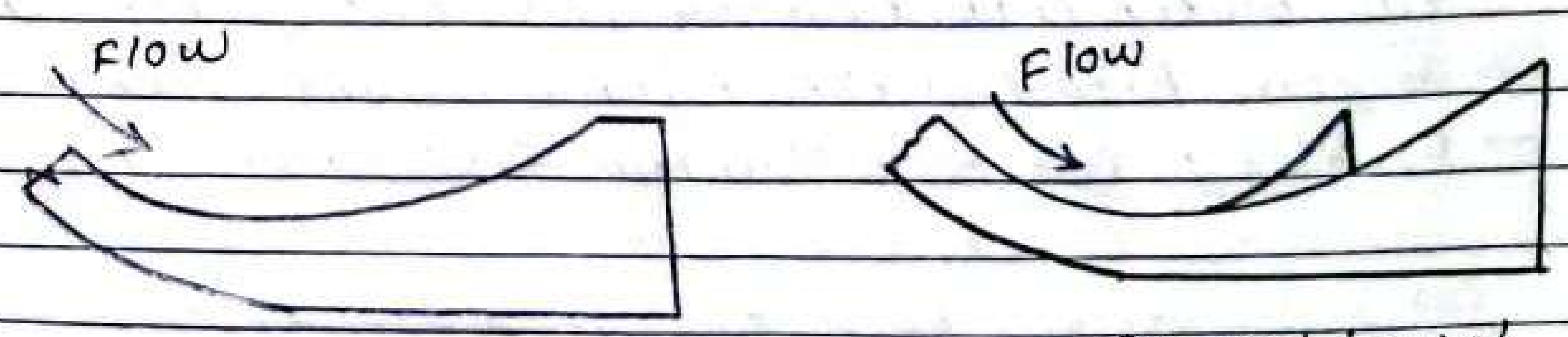
→ The stilling basin may be horizontal apron type or sloping apron type. The stilling basin shall also consists of chute blocks, baffle blocks or end sills to make it economical.

(b) Roller Buckets:-

→ Used where tailwater depth is much in excess of sequent depth.

→ Energy dissipation shall be achieved through two complementary elliptical rollers - a surface roller formed over the bucket & a ground roller formed d/s of the bucket.

→ It may be solid roller type or slotted roller type.



Solid bucket

Slotted bucket

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(c) Deflector or flip buckets:-

→ Used where tailwater depth is insufficient for the formation of hydraulic jump.

→ It consists of upturned solid bucket that throws away the incoming flow to a considerable distance d/s of the overflow section as a freely discharging upturned jet which falls into the river channel directly, thus avoiding excessive scour immediately d/s.

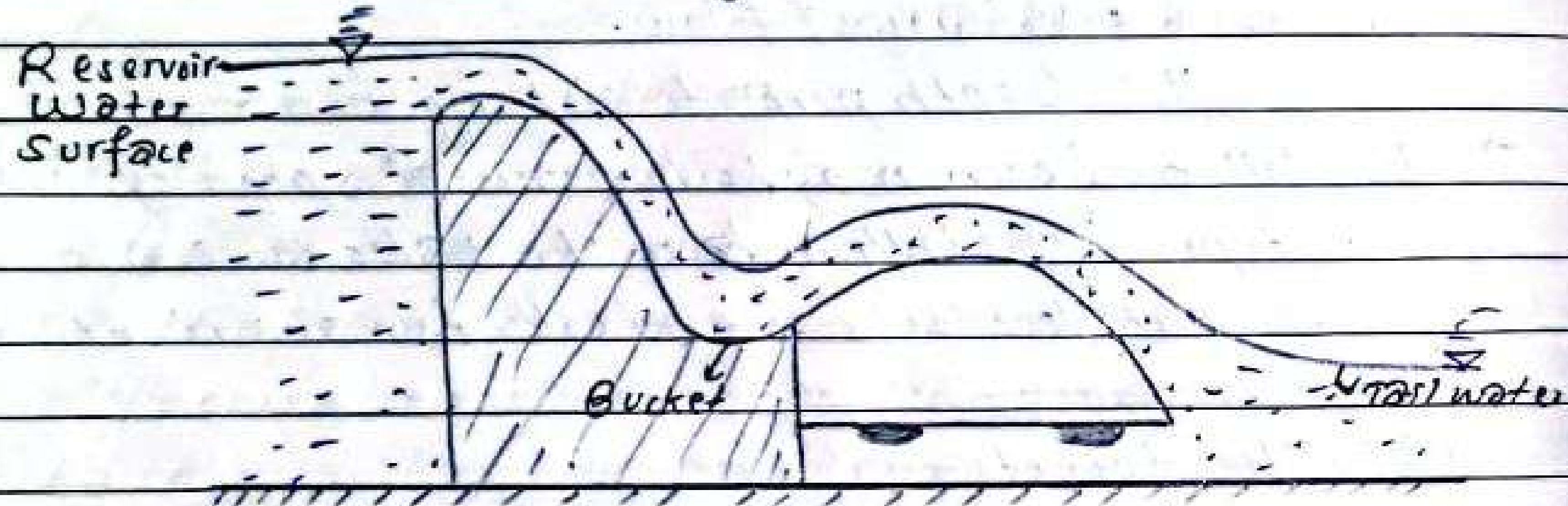


Fig: Flip bucket type energy dissipator

* Role of Tail Water Depth:-

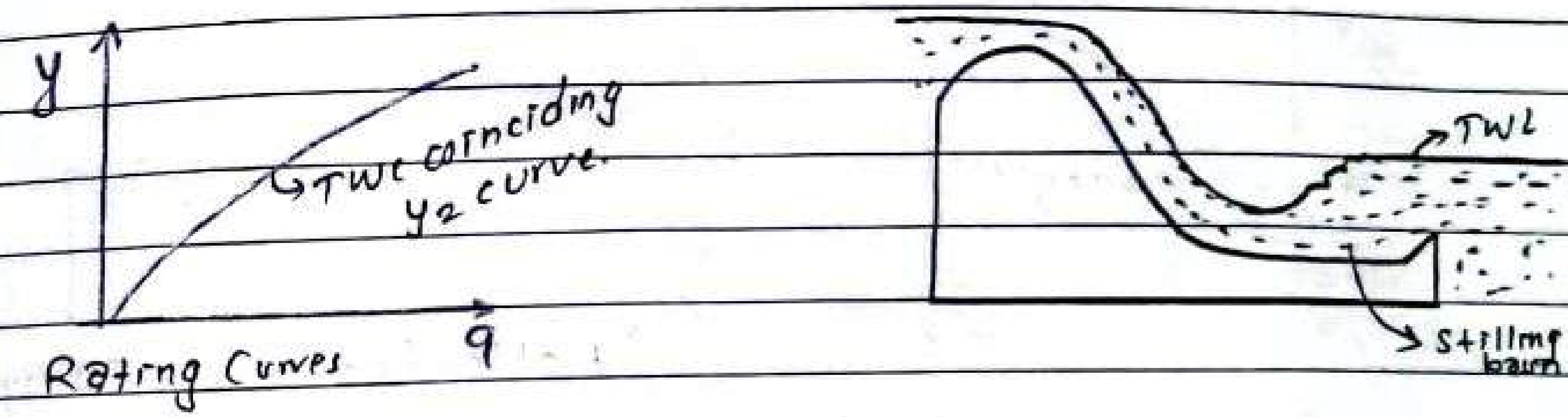
→ For a particular site, the type of energy dissipator & its arrangement shall be decided based on relationship between the heights of hydraulic jumps versus tailwater depth. The graph betn specific discharge & tailwater is called tail water curve (TWC) & that betn specific discharge & jump height is called jump height curve or y_2 curve.

→ For this purpose, following five cases occur.

(a) Jump Height equal to Tail Water Depth:-

→ Tailwater curve coincides with ~~tail~~ y_2 curve.

→ Hydraulic jump will form at the toe of spillway at all flow rates. In this situation, a horizontal apron stilling basin shall be used for energy dissipation.



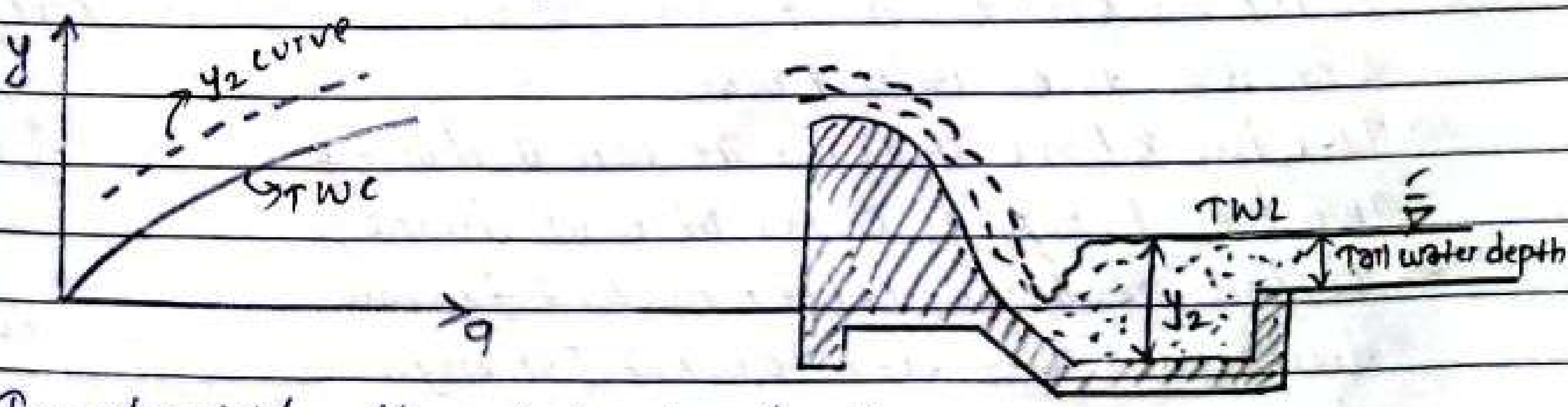
(b) Jump Height always above tailwater depth:-

→ Tail water curve (TWC) lies below y_2 curve

→ Tailwater depth will be insufficient for the formation of hydraulic jump.

→ In this case energy dissipation shall be achieved by

- lowering apron level of stilling basin
- Providing horizontal apron with baffles & sills at riverbed
- Using deflector bucket
- Providing subsidiary dam downstream.



(c) Jump height less than tail water depth

→ Tail water curve lies above y_2 curve.

→ In this case, the jump formed will be drowned by tailwater.

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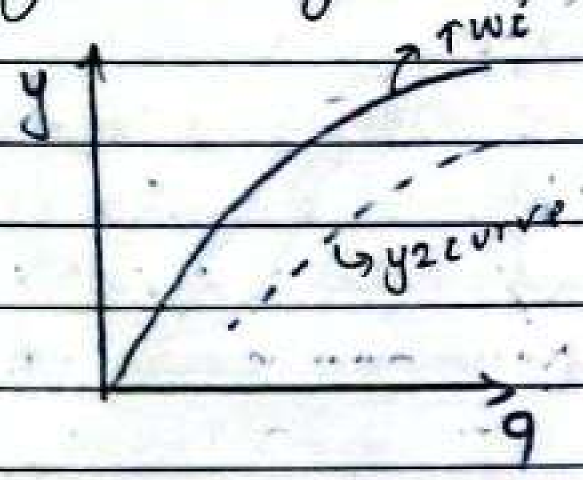
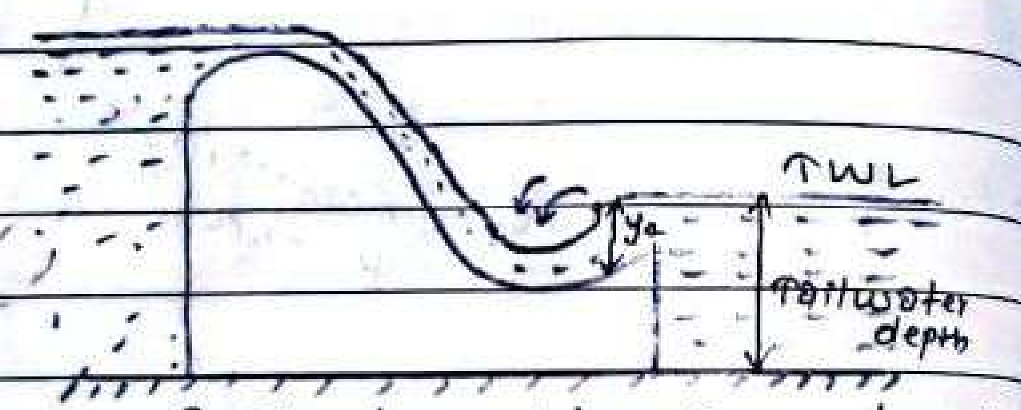
This problem can be overcome by constructing sloping apron over the river bed level or by providing roller bucket type energy dissipator.





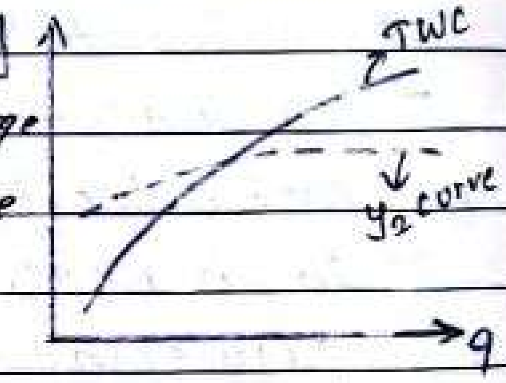
Roller bucket type energy dissipator

(d) Jump Height more than tailwater depth at low discharges & less at high discharges

→ TWC lies below y_2 curve for low discharge & y_2 curve lies below TWC for high discharge

→ In this case, one of the following alternatives may be used.

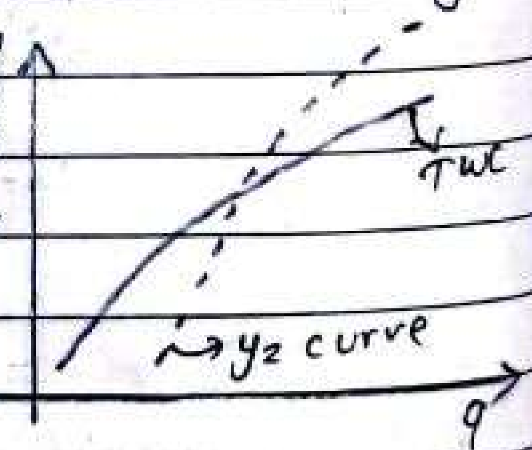
- Horizontal Apron Stilling Basin with low secondary dam.
- Stilling basin with baffle piers.



(e) Jump Height below tailwater depth at low discharge & Above at higher discharge:-

→ TWC lies above y_2 curve at low discharge & TWC lies below y_2 curve at high discharge

→ In this situation, roller bucket for low discharges & deflector bucket for high discharges may be used provided sound rock conditions exist in the river channel.



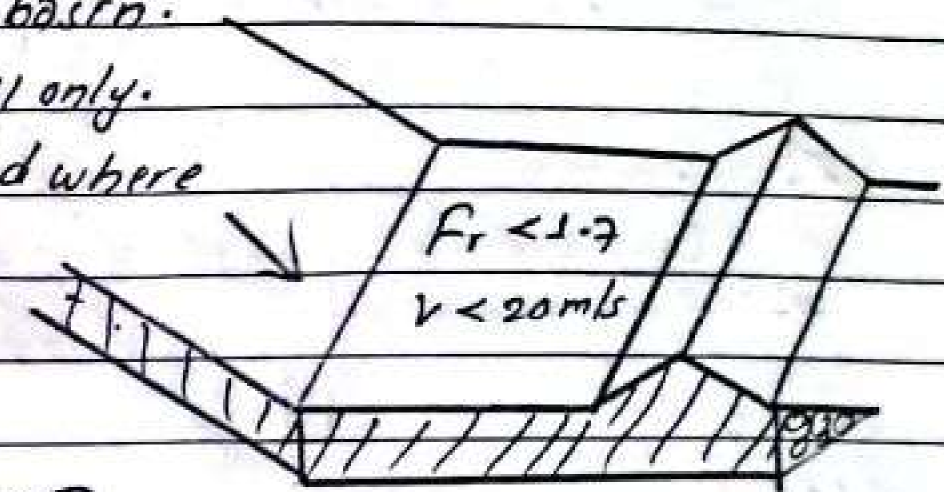
Design of Stilling basin:-

* Standard Stilling basins

(a) USBR Stilling Basin Type 'I':

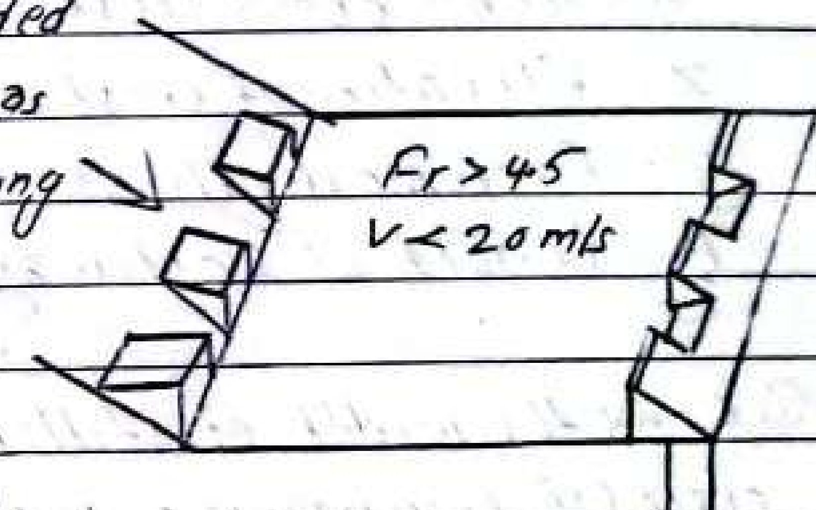
→ Simplest form of stilling basin. It consists of apron & end sill only.

→ This stilling basin is used where incoming froude number is less than 1.7.



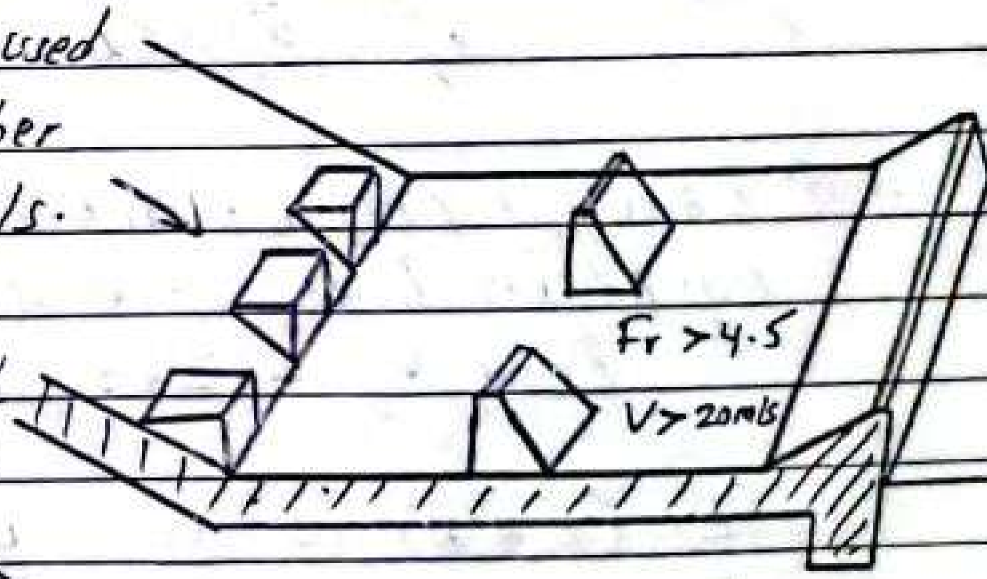
(b) USBR stilling basin type II

→ This type of basin is recommended for use on large structures such as dams & spillways where incoming Froude Number > 4.5



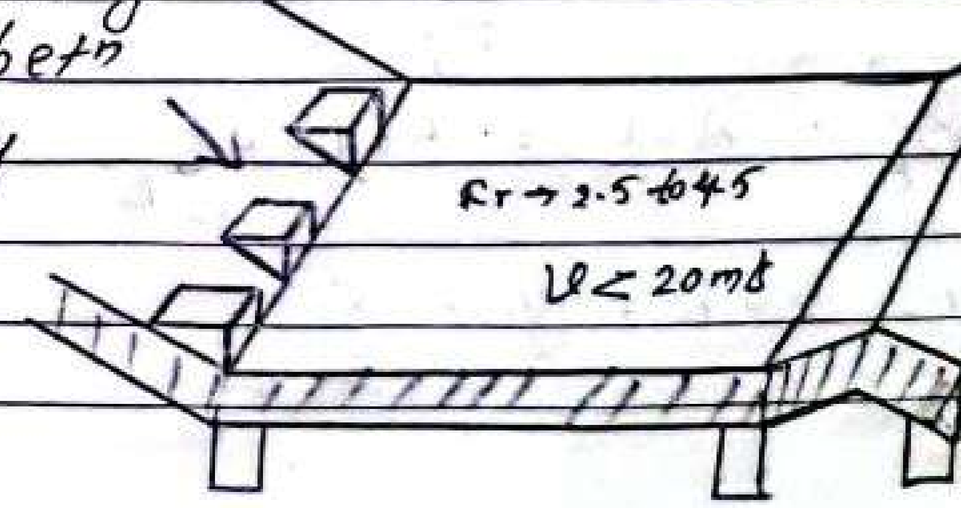
(c) USBR Stilling basin type III

→ This type of stilling basin is used when incoming froud number is greater than 4.5 & V > 20m/s.

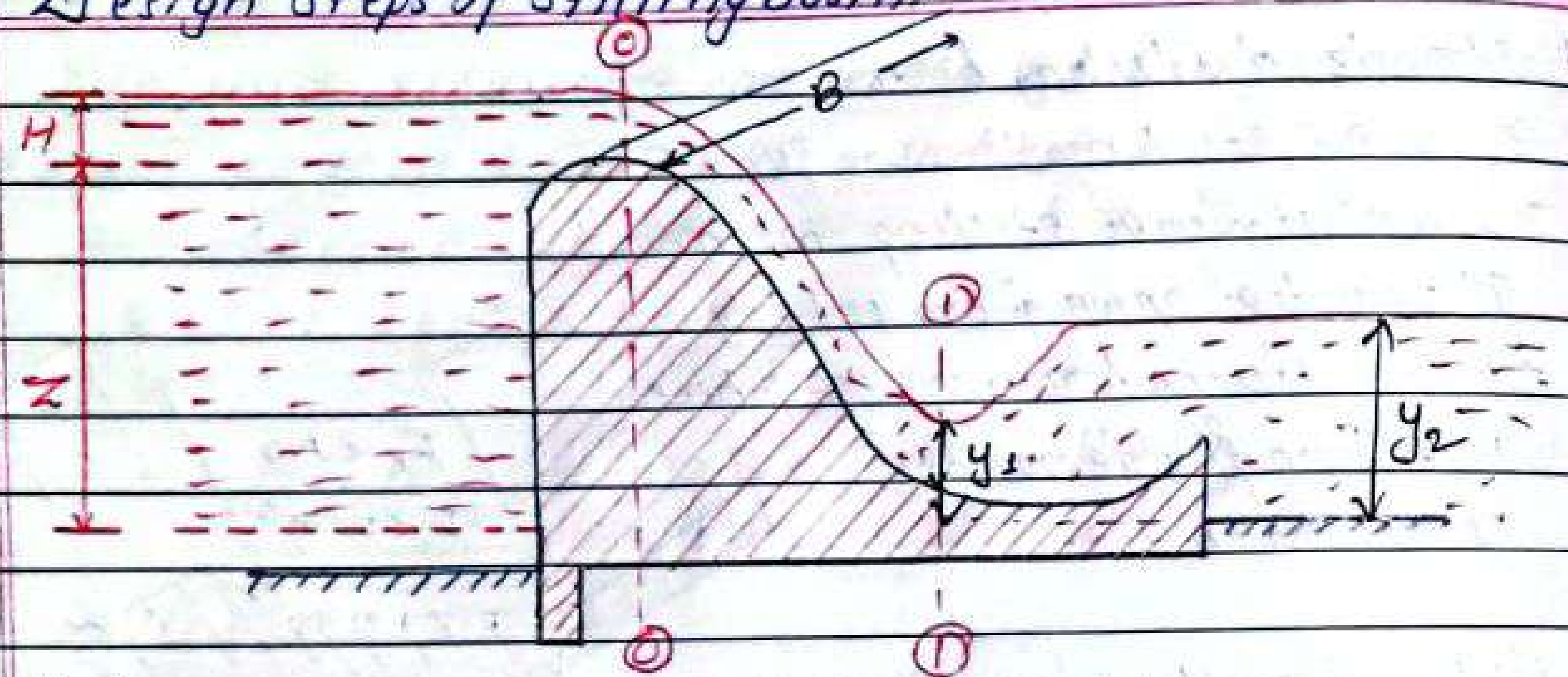


(d) USBR stilling basin type 'IV'

→ This type of stilling basin is used when incoming froud number lies betn 2.5 to 4.5 & velocity less than 20m/s



Design steps of stilling basin:



Let,

H = Head above crest

Z = Elevation of crest above downstream bed.

y_1 = Prejump depth , y_2 = Sequent depth

B = Length of spillway

① Find the width of stilling basin (B) if head over the crest (H) is known or vice-versa.

$$Q = \frac{2}{3} \times C_D \times \sqrt{2g} \times B \times H^{3/2}$$

② Calculate prejump depth (y_1) by applying Bernoulli's equation betn 0-0 & 1-1,

$$\frac{P_0}{\gamma} + \frac{V_0^2}{2g} + Z_0 = \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1$$

$$\Rightarrow (Z+H) = \frac{V_1^2}{2g} + y_1$$

$$q = \frac{Q}{B} = \frac{A \times V}{B} = \frac{B \times y \times V}{B} \Rightarrow V = \frac{q}{y}$$

Then, $Z + H = y_1 + \dfrac{q_1^2}{2g y_1^2}$

③ Calculate incoming Froude's number, $F_{r1} = \dfrac{V_1}{\sqrt{g y_1}}$

④ Calculate sequent depth, $y_2 = \dfrac{y_1}{2}\left(\sqrt{1+8F_{r1}^2} - 1\right)$

⑤ Calculate normal depth (y_0), $Q = \frac{1}{n} \times A \times R^{2/3} \times S^{1/2}$
(Assume rectangular channel)

⑥ Calculate critical depth, $y_c = \left(\frac{q^2}{g}\right)^{1/3}$

⑦ If $y_0 > y_c$, hydraulic jump is formed.

⑧ Select suitable type of stilling basin based on froud number & velocity.

Q. Design a stilling basin using fallowing data.
Height of crest above the d/s bed = 30m
Design discharge = 50 m³/s
Width of canal = 4m
d/s bed slope = 1/500 , Manning's n = 0.016 & C_D = 0.7

→ Given,
Height of crest above d/s bed = 30m
Design discharge (Q) = 50 m³/s
Width of canal (B) = 4m
d/s bed slope (S) = 1/500
Manning's (n) = 0.016
Coeff. of discharge (C_D) = 0.7

→ Head above crest,

$$Q = \frac{2}{3} \times C_D \times \sqrt{2g} \times B \times H^{3/2}$$

$$\Rightarrow 50 = \frac{2}{3} \times 0.7 \times \sqrt{2 \times 9.81} \times 4 \times H^{3/2}$$

$$\Rightarrow H = 3.32m$$

Applying Bernoulli's eqn betn 0 & 1,

$$\frac{P_0}{\gamma} + z_0 + \frac{V_0^2}{2g} = \frac{P_1}{\gamma} + z_1 + \frac{V_1^2}{2g}$$

$$\Rightarrow (30 + 3.32) = y_1 + \frac{V_1^2}{2g} \qquad V_1 = q/y_1$$

$$\Rightarrow 33.32 = y_1 + \left(\frac{12.5}{y_1}\right)^2 \times \frac{1}{2g} \qquad q = Q/B = 50/4 = 12.5\ m^3/s/m$$

$$\Rightarrow y_1 = 0.492 m$$

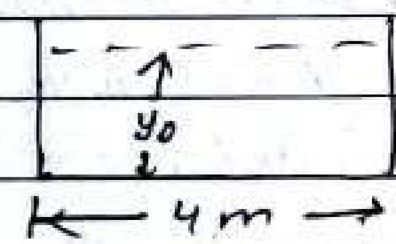
→ Also, $Fr_1 = \frac{V_1}{\sqrt{g y_1}} = \frac{25.4}{\sqrt{9.81 \times 0.492}} = 11.55$

Now,

$$y_2 = \frac{y_1}{2}\left(\sqrt{1 + 8 Fr_1^2} - 1\right)$$

$$= \frac{0.492}{2}\left(\sqrt{1 + 8 \times 11.55^2} - 1\right)$$

$$= 7.79 m$$



→ Normal depth (y_0)

Assuming rectangular section,

$$Q = \frac{1}{n} \times A \times R^{2/3} \times S^{1/2}$$

$$\Rightarrow 50 = \frac{1}{0.016} \times (4 \times y_0) \times \left(\frac{4 \times y_0}{4 + 2y_0}\right)^{2/3} \times \left(\frac{1}{500}\right)^{1/2}$$

$$\Rightarrow y_0 = 3.74 m$$

→ Critical depth $(y_c) = (q^2/g)^{1/3}$

$$= \left(\frac{12.5^2}{9.81}\right)^{1/3}$$

$$= 2.516 m$$

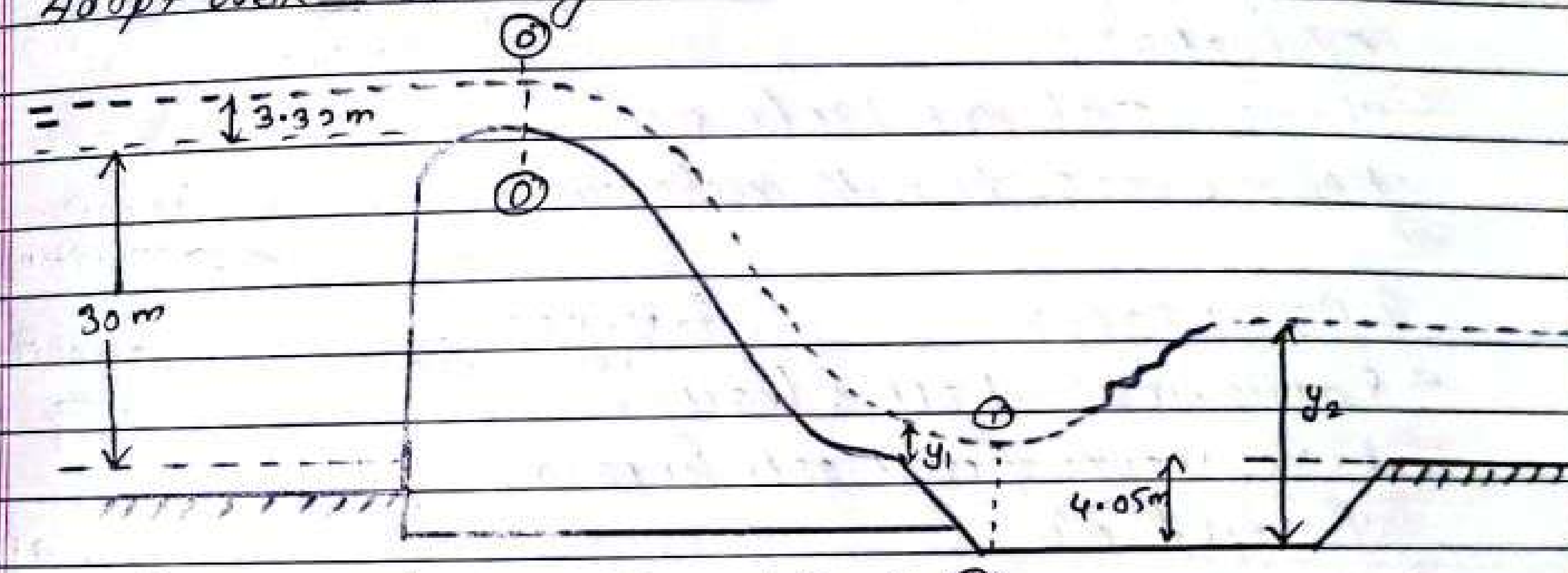
since, $y_0 > y_c$, hydraulic jump is formed.

→ Floor of basin depressed by 4.05m ($7.79 - 3.74 = 4.05m$)

Also,

$Fr = 11.55 > 4.5$ & $V = 25.4 m/s > 20 m/s$

Adopt USBR III stilling basin.



Depressed Floor Type Stilling Basin

※ Gates:-

→ In controlled spillways, different types of gates are provided on the crest of spillway specially on concrete dam.

→ The gates provide additional storage on the dam during dry season.

→ During flood time, these gates are lifted or opened to allow full original spillway capacity as designed.

→ Vertical & radial gates are commonly used in HPP.

→ The common types of spillway gates are

a) Vertical lift gate
b) Radial Gate
c) Flap Gate
d) Flash Board Gate
e) Stop logs gate
f) Needle Gate

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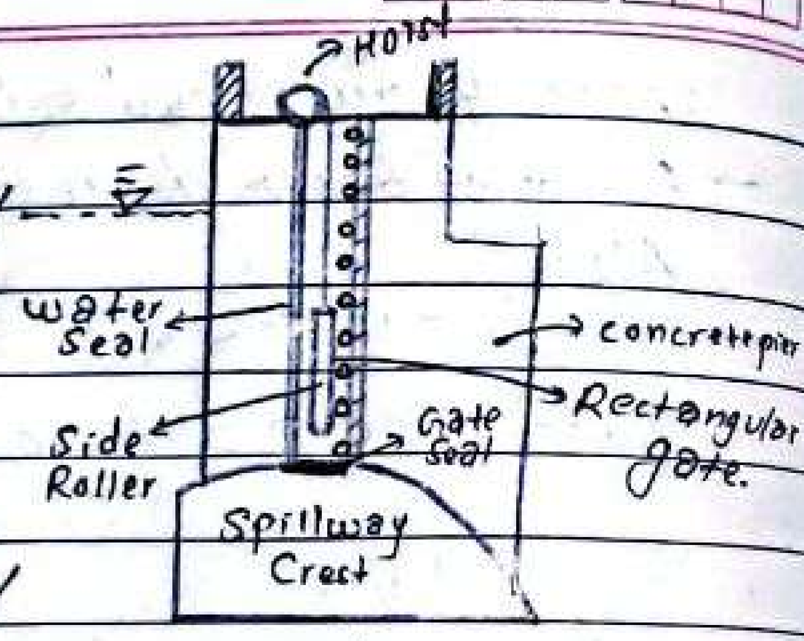
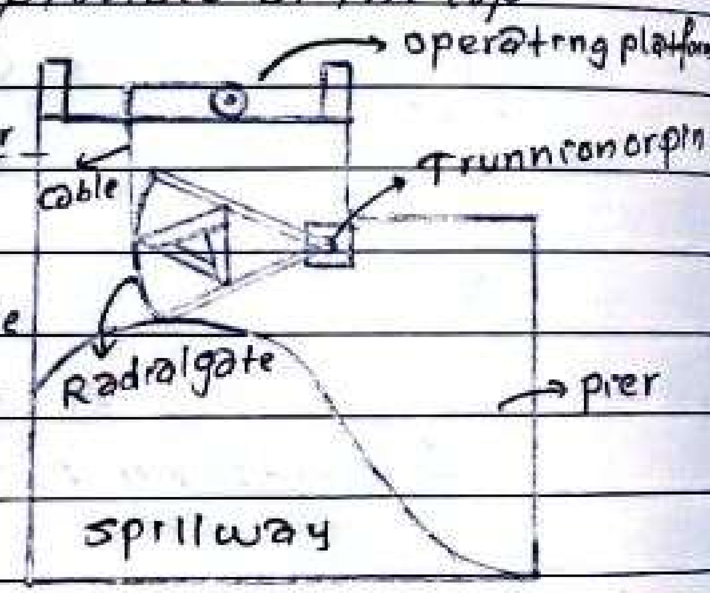
(a) Vertical lift gate:-



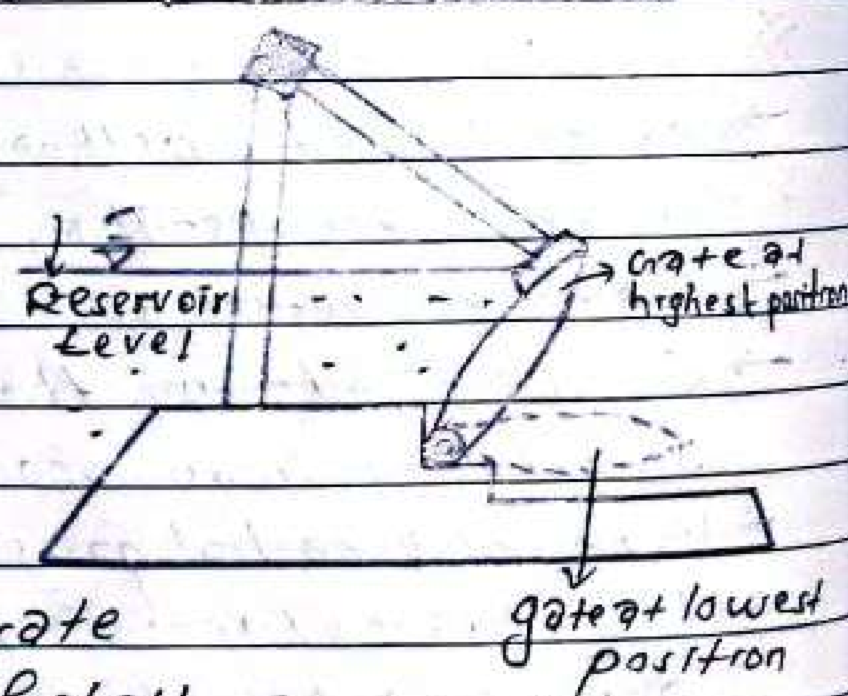
- → Ease of fabrication, considerably shortened erection time
- → Shorter supporting piers for spillways compared with those of radial gates.
- → The gates can be raised or lowered between groove guides with mechanism provided at the top.

(b) Radial Gates:-



- → Simplest, most reliable & least expensive type of crest gate for passage of large floods.
- → The gate rotates about trunnion or pins by hoisting mechanism in operating platform.

(c) Flap Gates:-



- → This type of gate is a leaf hinged at bearings along its lower edge.
- → The leaf may be flat or curved.
- → The flap gates normally operate at partially open conditions & shall be designed for hydrodynamic effects of the overflowing sheet of water.

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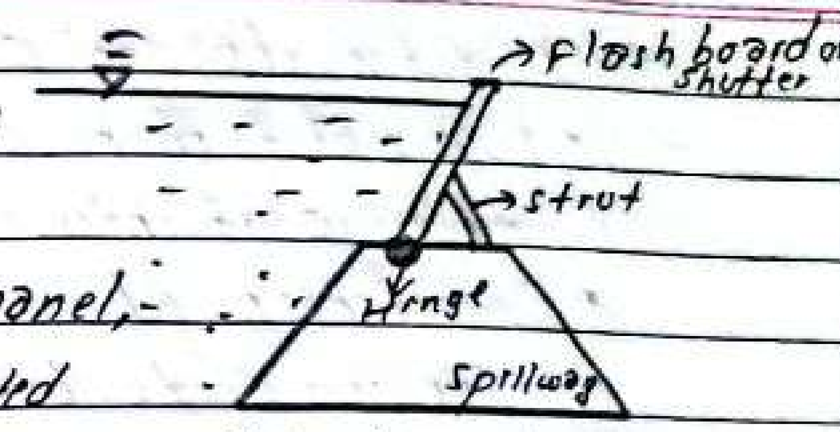
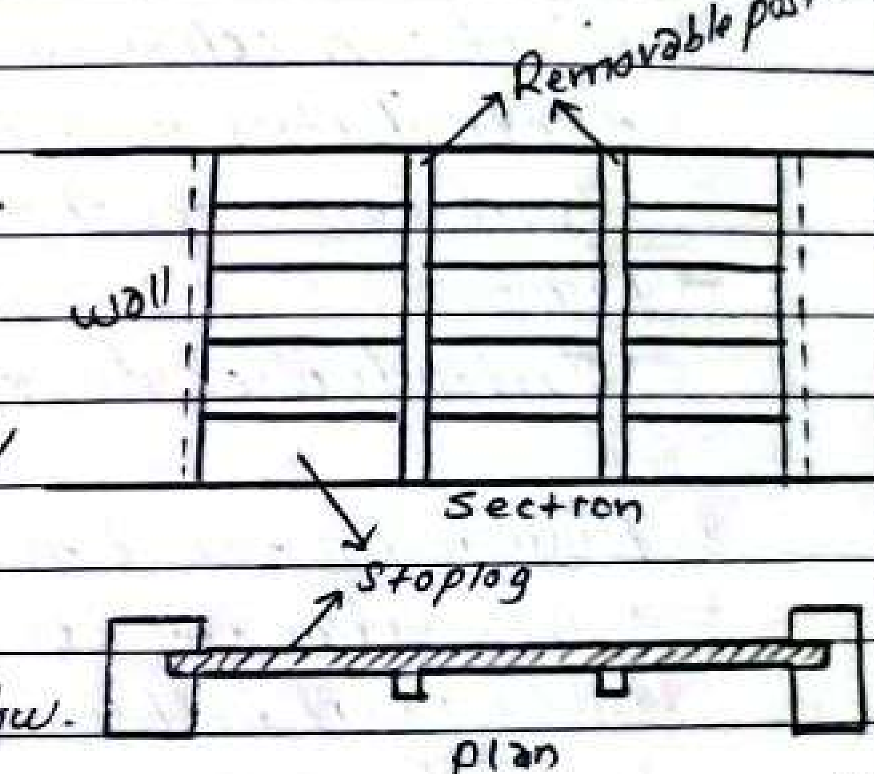
(d) Flash Board:-



Flash Board or Dropping Shutter

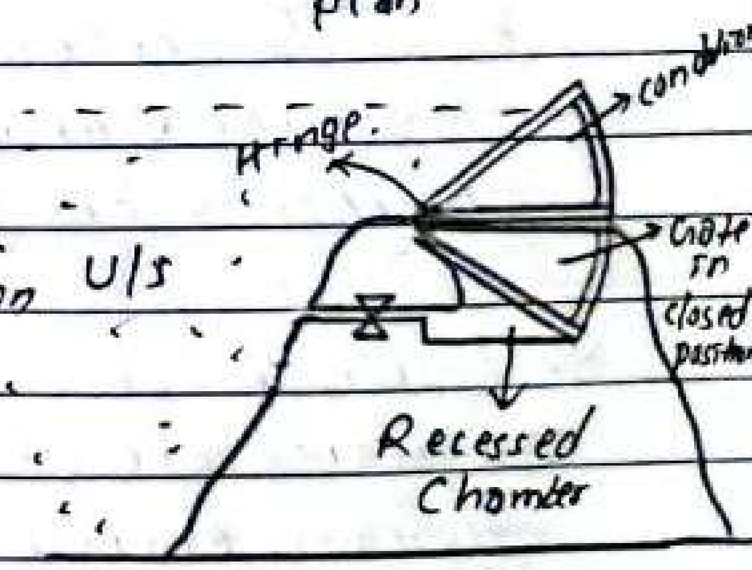
- → These gates are used only in small spillways of minor importance. It consists of a wooden board or panel, hinged at bottom, and is supported against water pressure by struts.
- → Flash board is installed in Jhimruk hydroelectric project in pyuthan.

(e) Stop logs gate:-



- → Stop log consists of wooden beams or planks placed one upon other spacing betn pier with grooves. The logs can be removed by hands.
- → plan & sectional elevation of stop logs is shown in figure below.

(f) Drum Gate:-



Drum Gate

- → This gate is used for long span. It consists of a circular sector in cross section formed by skin plates attached to internal bearings. It is hinged at centre of curvature such that entire section may be raised or lowered.
- → When water is allowed to enter the recession chamber, buoyant force helps in raising the gates. To lower the gate, water from the recessed chamber is allowed to drain away.

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Chapter 8 :- Hydromechanical & Electromechanical Equipments

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Outline:-

→ Hydromechanical Installation in powerhouse
- Types of turbines : pelton, Francis, kaplan turbines & their performance characteristics, introduction to bulb turbine
- Specific speed, synchronous speed & runway speed
- Selection of turbines
- Design of Francis & pelton turbines
- Scroll case, draft tube & tailrace canal & their importance

→ Electro-mechanical Installation
- Introduction to generator & their types
- Working principles of governors in pelton & Francis turbine

→ Pumps
- Introduction to centrifugal & reciprocating pumps.

* Hydromechanical Equipment :-

→ Hydromechanical equipment are defined as those equipment that convert either hydraulic energy into mechanical energy or mechanical energy into hydraulic energy.

→ Hydraulic energy to Mechanical Energy :- Turbine
Mechanical Energy to Hydraulic Energy :- Pump

※ Turbines:-

→ Turbines are the hydromechanical equipment that convert hydraulic energy into mechanical energy. This mechanical energy is used in running an electric generator, which is directly coupled to the shaft of the turbine. Electric generator converts mechanical energy into electrical energy.

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※ Classification of Hydraulic Turbines:

* Based on nature of energy head possessed by water at inlet:

(a) Impulse Turbine or Velocity Turbine:-

→ Turbine in which water entering the runner posses kinetic energy only.

→ Pressure is atmospheric at inlet & outlet of turbine

→ Eg:- pelton turbine, ~~turgo~~ turgo turbine

(b) Reaction Turbine:-

→ Turbine in which water entering the runner posses pressure as well as kinetic energy

→ This type of turbine is always enclosed by a air tight casing and runner & casing is completely full of water.

→ Eg: kaplan turbine, francis turbine, Thomson turbine

* Based on direction of flow:-

(a) Tangential flow turbine:

→ Flow in turbine is in tangential direction of rotation of runner.

→ Eg:- pelton Turbine.

(b) Radial Flow Turbine:-

→ Water moves towards the axis of rotation of the runner or away from it.

→ When flow is towards axis, it is called inward flow turbine (Francis Turbine) & when the flow is away from axis of rotation, the turbine is called outward flow turbine (Fourneyron turbine)

(c) Axial Flow Turbine:

→ Water flows parallel to the axis of rotation.

→ Kaplan Turbine

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(d) Mixed Flow Turbine:-
→ In this type of turbine, water enters radially inward at inlet & discharges water at outlet in a direction parallel to the axis of rotation of the runner.
→ Eg: Francis Turbine.

* Based on Head:-
(a) Low Head Turbine
→ 15m to 60m head
→ Kaplan Turbine (low head turbine)

(b) Medium Head Turbine
→ 60m to 250m head
→ Francis Turbine

(C) High Head Turbine
→ greater than 250m
→ Pelton Turbine

* Based on Specific Speed:-
(a) Low Specific Speed Turbine
→ Specific speed:- (8-30)
→ Pelton Turbine

(b) Medium Specific Speed Turbine
→ Specific speed:- (50-250)
→ Francis Turbine.

(c) High Specific Speed Turbine
→ Specific Speed:- (250-850)
→ Kaplan Turbine.

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* Specific Speed:- Specific speed is defined as the speed at which machine produces 1 horse power (1 Hp) under 1m head.

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

where, N = Sychronous speed in rpm
P = power in Hp
H = head of turbine (m)

* Synchronous Speed:-
→ Rotational Speed of turbine which is in synchronize with frequency of the energy system ~~should be~~ to which it is connected.
→ It means that the frequency of the energy system should be equal to frequency of each number of pole pair. Note that the number of pole pairs is always an integer.

$$N_p = \frac{f}{N/60} = \frac{60f}{N}$$

Where, f = frequency of generation (50 Hz for Nepal)
N_p = Number of pole pairs of the generator

* Runaway Speed:-
→ If the external load on the machine suddenly drops to zero & the governing mechanism fails at the same time, the turbine will tend to race up to the maximum possible speed which is known as runaway speed.

* Performance Characteristics of Turbines:-
→ The turbine has one designed condition, specific head, discharge efficiency & power but the condition donot remains same &

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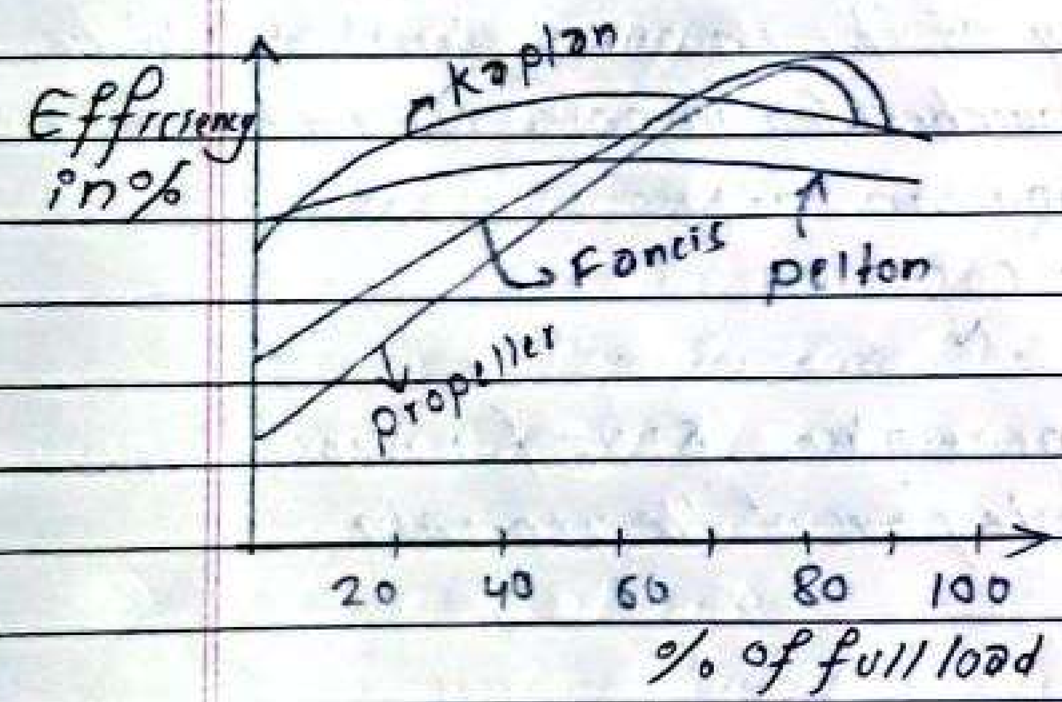
they have to operate in different conditions than designed specific conditions.

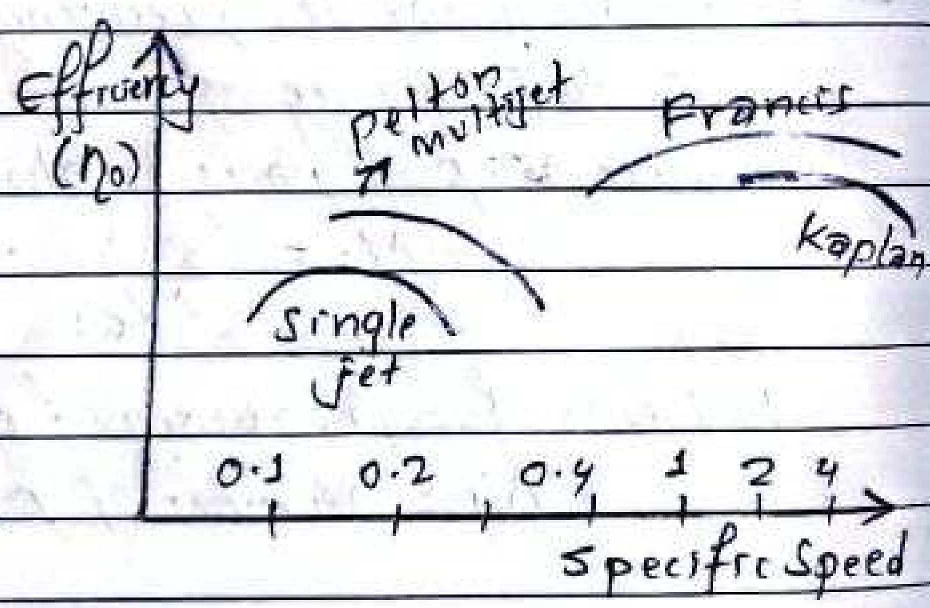
→ For the selection & design of turbines, it is essential to know their behavior at different conditions.

→ The plot of behaviour of turbines under different conditions are called performance characteristics of turbines.

→ Behaviour of turbines are indicated by the parameters like specific speed, efficiency while working conditions are determined by load conditions, percentage opening of gate & unit head.

→ Some characteristics curves are given below.





※ Bulb Turbines:-

→ The bulb turbine is a variation of propeller type turbine (similar to kaplan turbine). In the bulb turbine arrangement, the generator is encapsulated & sealed within a streamlined ~~through~~ watertight steel housing mounted in the center of the water passageway. The generator is driven by a variable pitch propeller located on the downstream end of the bulb.

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→ Unlike the kaplan turbine, water enters & exits this unit with very little change in direction. The compact nature of this design allows for more flexibility in powerhouse design. Bulb turbines can, however be somewhat more difficult to access for service, & they require special air circulation & cooling within the bulb.

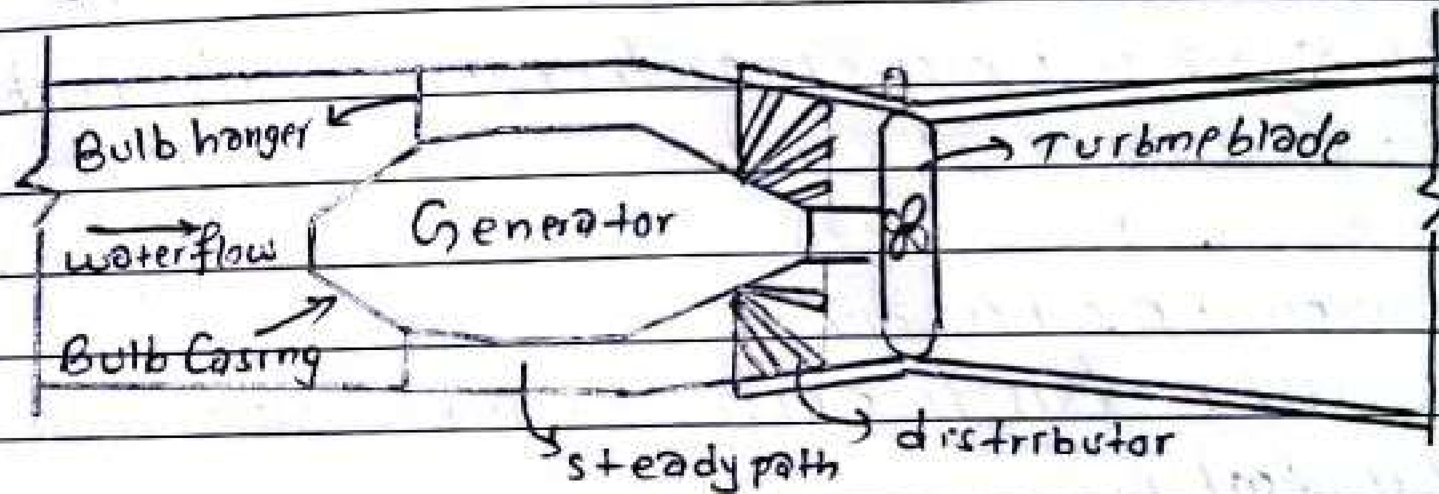

Fig: plan of bulb turbine.

※ Selection of Turbines:-

1) Availability of heads & its fluctuation

Very high head ($H > 350m$): pelton turbine

High Head (150m - 350m): pelton or Francis Turbine

Medium Head (60m - 150m): Francis Turbine

Low Head (< 60m): Kaplan & Francis Turbine

2) Efficiency:

The turbine that gives highest overall efficiency for various operating condition should be selected.

3) Specific Speed:

High specific speed is required where head is low & output is large else rotational speed will be low that will lead to high

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cost of turbo generator & powerhouse.

4) Rotational Speed:- Rotational speed depends upon specific speed but in practice it should have higher value.

5) Water Quality:
Quality of water is more important for impulse turbine than reaction turbine.

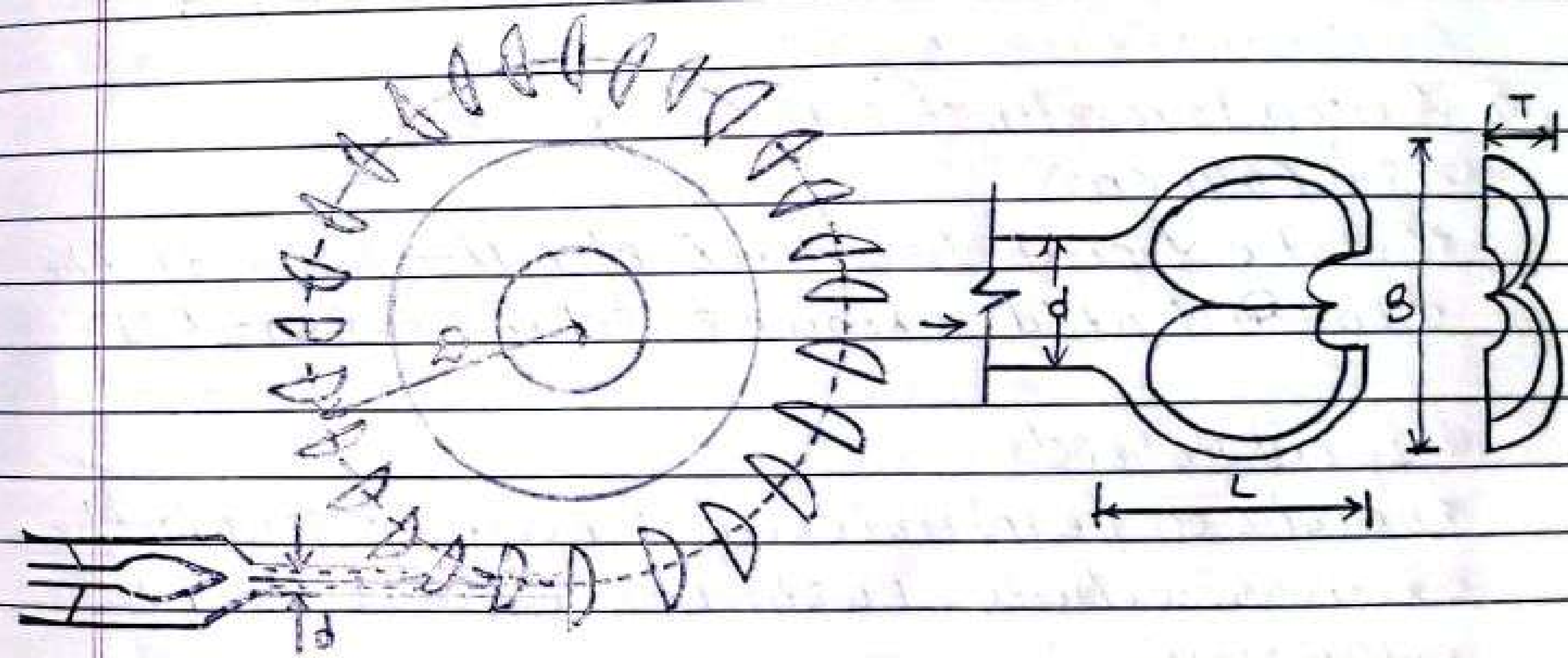
6) Conveyance or maintenance:
Impulse turbine has less cost of maintenance than that of reaction turbine.

7) Deposition of Turbine Shaft:-
Vertical shaft arrangement is better for large sized turbine, it is universally adapted. But in case of large size impulse turbine horizontal shaft arrangement is almost employed.

※ Pelton Turbine:-
→ Tangential flow impulse turbine
→ Used for high heads
→ The water strikes the bucket along the tangent of the runner.
→ The energy available at the inlet is only kinetic energy.
→ From the headrace tunnel, water is conveyed to the turbines installed in the powerhouse through penstock. The lower end of the penstock is joined with the nozzle in the turbine casing. Water is delivered by the nozzles at high velocity on the buckets. The buckets are mounted on the persphery of

a circular wheel called runner. The quantity of water coming out of the nozzle can be controlled by governing mechanism. The impact of the water on buckets causes runner to rotate. The buckets are so shaped that water enters tangentially in the middle & discharges backward & flows again tangentially in both directions.

* Components of pelton Turbine:-



(a) Nozzle:- The nozzle directs the flow in the buckets. It also governs the quantity of flow with the help of spear value controlled by the governer action. There can be upto six jets (nozzles) all symmetrically arranged & causing the rotation in the same sense.

(b) Runner with Buckets:- Runner consists of a circular disc on the periphery of which a number of buckets evenly spaced are fixed. The shape of the bucket is of double hemispherical cup. Each bucket is divided into two symmetrical parts by a dividing wall, which is known as splitter. The buckets are shaped in such a way that jet gets deflected through 100° to 170°.

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(c) Casing: The function of casing is to prevent the splashing of water & discharge water to tailrace. The casing in pelton turbine has no hydraulic action.

(d) Breaking Jet:- When the nozzle is completely closed by moving spear valve, the runner still tends to rotate due to inertia. To stop the runner in a short time, a small nozzle is provided which directs the jet of water on the back of vanes. This is called breaking jet.

※ Design Parameters of pelton Turbine:-

(a) Jet Ratio (m)

The ratio of pitch diameter (D) of pelton wheel to the diameter of jet (d) is known as jet ratio. ($m = D/d$)

(b) Speed Ratio (ϕ)

The ratio of peripheral speed of buckets or vanes to the theoretical velocity of water under effective head is called speed ratio.

$$\phi = \frac{\bar{V}}{\sqrt{2gH}} = \frac{\pi DN}{60\sqrt{2gH}} = \frac{ND}{84.6\sqrt{H}} \quad \left(\bar{V} = \frac{\pi DN}{60}\right)$$

※ Design steps of pelton Turbine:-

1) Calculate velocity at inlet, $V_1 = C_v\sqrt{2gH}$ $(C_v \to 0.98-0.99)$
2) Calculate tangential velocity, $\bar{V} = \phi\sqrt{2gH}$ $(\phi \to 0.43-0.48)$
3) Assume deflection of jet through bucket to be 165° if angle of deflection is not provided.
4) Assume number of jets, suppose single jet,

$Q = A * \text{velocity of jet}$

$Q = \frac{\pi \times d^2}{4} \times V_1$, d is calculated.

5) Assume jet ratio, $m = D/d$ (10 to 15) → D is calculated
6) From peripheral velocity, calculate synchronous speed (N)

$$\bar{V} = \frac{D}{2} \times \omega = \frac{D}{2} \times \frac{2\pi N}{60} \Rightarrow N = \frac{60\bar{V}}{\pi D}$$

7) Calculate number of pole pairs, $N_p = 60f/N$
Round off to get corrected N_p (integer)
8) Calculate corrected N, $N_{corrected} = 60f/N_{p,corrected}$
9) Calculate specific speed, $N_s = \frac{N\sqrt{P}}{H^{5/4}}$, $P = \eta\gamma QH$ (P in HP)
10) Calculate number of buckets, $N_b = 15 + m/2$
11) Spacing of bucket $= \pi D/N_b$
12) Bucket dimensions, $L = 2d$ to $3d$
$B = 3d$ to $4d$ & $T = 0.8d$ to $1.2d$.

Q. Design a pelton turbine for a hydropower plant which have a net head of 312.5m & discharge of 5m³/s. Take efficiency of turbine, $\eta = 85\%$. Assume any other suitable data if necessary.

→ Here,

Net head (H) = 312.5m

discharge (Q) = 5 m³/s

Efficiency (η) = 85%

Velocity of jet at inlet, $V_1 = C_v * \sqrt{2gH}$
$= 0.98 * \sqrt{2 \times 9.81 \times 312.5}$
$= 76.73$ m/s.

Tangential velocity $(\bar{V}) = \phi\sqrt{2gH}$

Taking $\phi = 0.45$,

$\bar{V} = 0.45\sqrt{2\times9.81\times312.5} = 35.236\,m/s$

Assume, no. of jet $(n) = 1$

$\therefore Q = n * Q_{jet}$

$\Rightarrow 5 = 1 * \frac{\pi\times d^2}{4} \times v,$

$\Rightarrow 5 = 1\times\pi\times d^2/_4 \times 76.73 \quad \Rightarrow d = 0.288\,m$

Assume, jet ratio $(m) = 10$,

$D/d = 10 \Rightarrow D = 2.88\,m$

Synchronous Speed $(N) = \frac{60\bar{V}}{\pi D} = \frac{60\times35.236}{\pi\times2.88}$

$N = 233.66\,rpm$

No. of pole pairs, $N_p = \frac{60f}{N} = \frac{60\times50}{233.66} = 12.839$

Corrected $N_p = 13$

$N_{corrected} = \frac{60f}{N_{p,corrected}} = \frac{60\times50}{13} = 230.769\,rpm$

Also, specific speed $(N_s) = \frac{N\sqrt{P}}{H^{5/4}}$

$P = \eta\gamma QH = \frac{0.85\times9810\times5\times312.5}{746} = 17465.02\,HP$

$N_s = \frac{230.769\times\sqrt{17465.02}}{312.5^{5/4}} = 23.21\,rpm$

Number of buckets, $N_b = 15 + m/_2 = 15 + {}^{10}/_2 = 20$

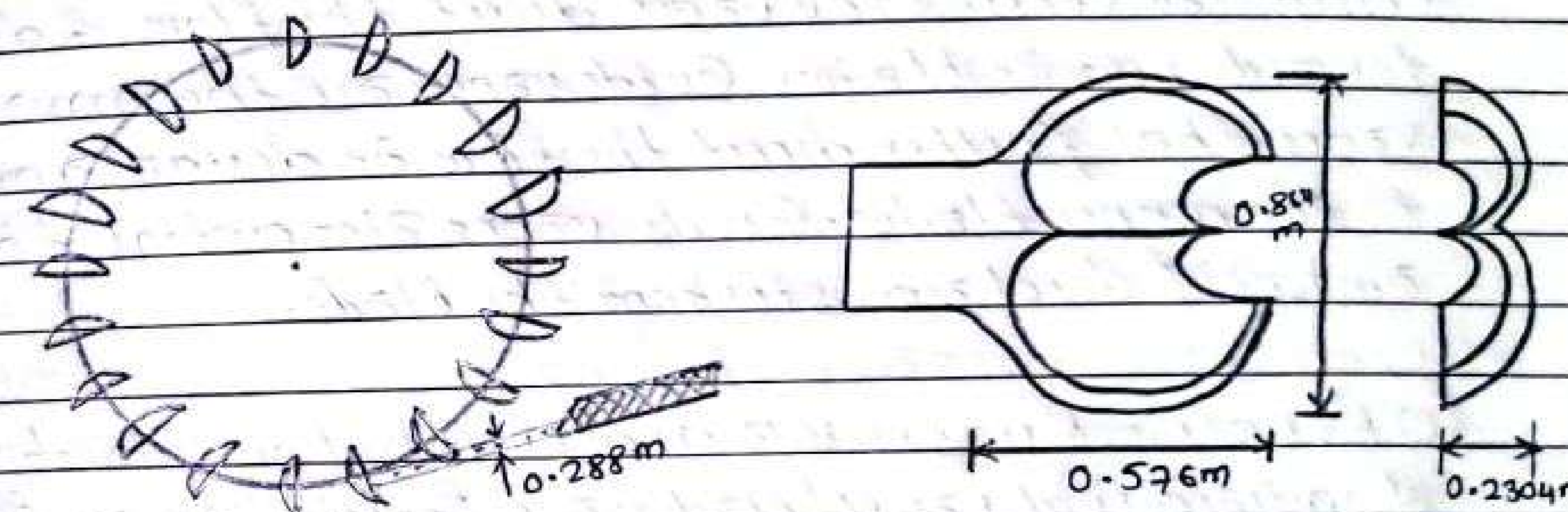
Spacing of bucket $= \frac{\pi D}{N_b} = \frac{\pi\times2.88}{20} = 0.452\,m$

Bucket dimensions,

Axial width $(B) = 3d = 3\times0.288 = 0.864\,m$

Radial length $(L) = 2d = 2\times0.288 = 0.576\,m$

Depth $(T) = 0.8\times0.288 = 0.2304\,m$



* Francis Turbine:-

→ Francis turbine is an inward mixed flow reaction turbine

→ In this turbine, water under pressure enter the runner from the guide vanes toward the centre in radial direction & discharge out of the runner axially.

→ The francis turbine operates under medium head.

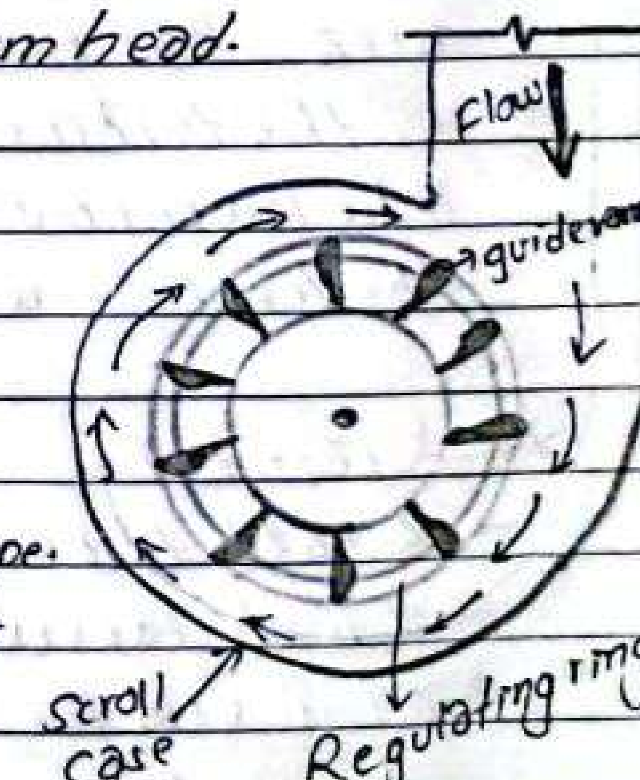


Fig: Francis Turbine

* Components of Francis Turbine:-

(a) Spiral Casing:-

→ Conveys water from the penstocks to the turbine.

→ The scroll case is made of spiral ~~case~~ shape. So that the water may enter the runner at constant velocity throughout the circumference of the runner.

→ Encloses the turbine & its components into air tight compartment.

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(b) Stay vanes & Guide Vanes:- Water entering into the scroll casing first encounters series of stationary vanes called stay vanes. Stay vanes divert the flow & directs towards runner blades. Guide vanes are the movable vanes that further direct the flow at design angle to the runner blades. Guide vanes also control the amount of discharge striking the blade.

(c) Runner:- Runner is a circular wheel on which series of radial curved vanes/blades are fixed. The surface of vane is made smooth. The radial curved vanes are so shaped that the water enters & leave the runner without shock.

(d) Draft Tube:- A tube or pipe of gradually increasing area that is used for discharging water from exit of the turbine to the tail race is called draft tube. The lower end of draft tube is ~~[illegible]~~ submerged below the level of water in the tail race.

Draft Tube:-

→ Draft tube is a pipe of gradually increasing area which ~~[illegible]~~ connects the outlet of the runner to tailrace.
→ It is used for discharging water from exit of turbine to the tailrace.
→ By using draft tube, net head on turbine is increased. The turbine develops more power and also the efficiency of turbine increases.

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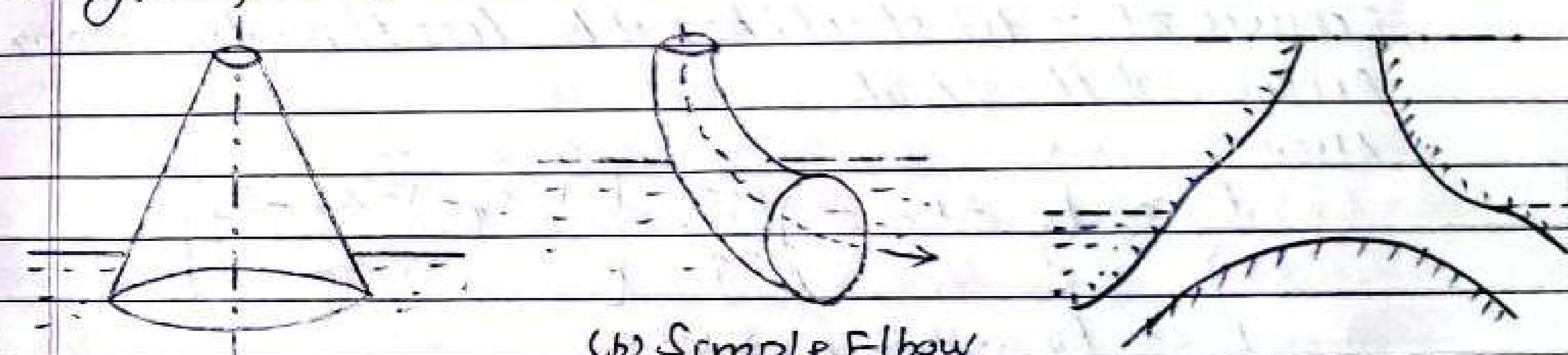
→ The major functions of draft tube are,
1) It permits a negative head to be established at the outlet of runner and thereby increase the net head of the turbine. The turbine may be placed above the tailrace without loss of net head & hence turbine may be inspected properly.
2) The draft tube helps to recover velocity head of water out of the runner. i.e the kinetic energy rejected at the outlet of the turbine is converted into useful pressure energy.

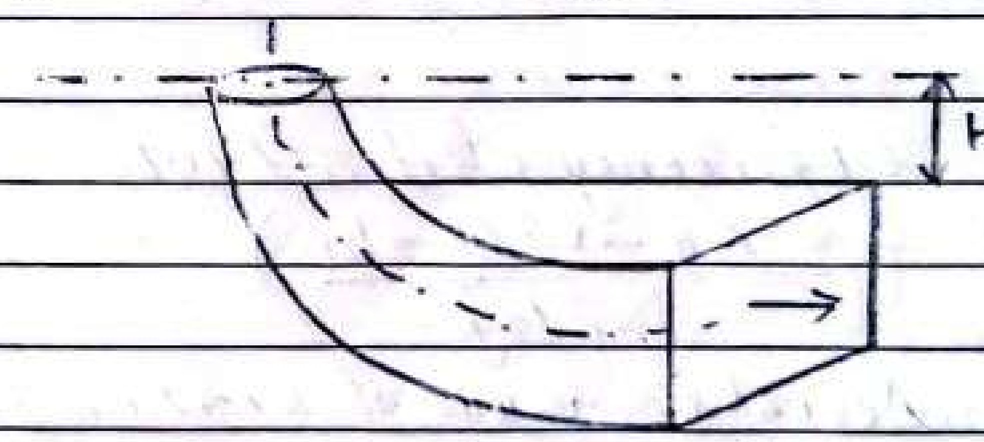
Types of Draft Tube:-



a) conical draft tube

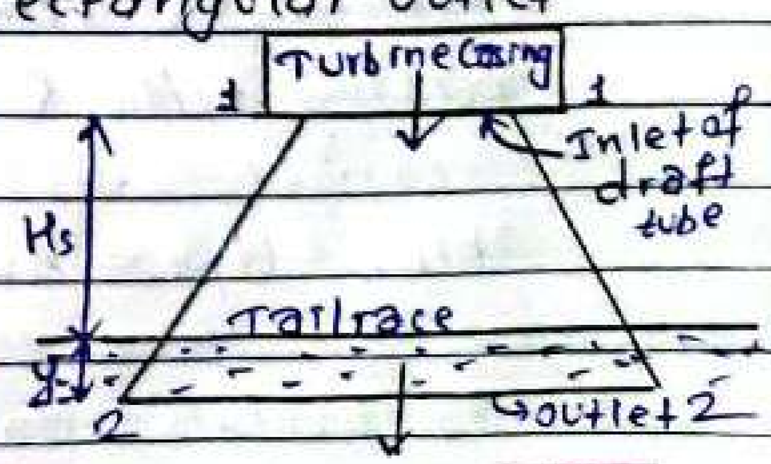
(b) Simple Elbow Tube

(c) Moody spreading Tube



(d) Draft tube with circular inlet & rectangular outlet

Draft Tube Theory:



Consider a conical draft tube as shown in fig.
let, H_s = Vertical Height of draft tube above tailwater level.
y = distance of outlet of draft tube from tail race.

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Applying Bernoulli's equation betn section 1-1 & 2-2

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

$$\Rightarrow \frac{p_1}{\gamma}+\frac{v_1^2}{2g}+(H_s+y)=\frac{p_a}{\gamma}+y+0+\frac{v_2^2}{2g}+h_f$$

$$\Rightarrow \frac{p_1}{\gamma}=\frac{p_a}{\gamma}-H_s-\left(\frac{v_1^2}{2g}-\frac{v_2^2}{2g}-h_f\right)$$

→ Hence above equation shows that the pressure at outlet of runner is less than atmospheric pressure. However, to prevent the cavitation phenomenon, the pressure at outlet should not be less than the vapour pressure of the fluid.

Also,

$$\frac{p_1}{\gamma}=\frac{p_a}{\gamma}-H_s-\frac{v_1^2}{2g}\left[\frac{v_1^2/2g-v_2^2/2g-h_f}{v_1^2/2g}\right]$$

$$\frac{p_1}{\gamma}=\frac{p_a}{\gamma}-H_s-\eta_d\times\frac{v_1^2}{2g}$$

where,

$$\eta_d = \text{Efficiency of draft tube}$$

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

$$=\frac{\text{Actual conversion of kinetic head into pressure head}}{\text{Available kinetic energy at inlet of draft tube.}}$$

Let, $\frac{p_a}{\gamma}=H_a$ & $\frac{p_1}{\gamma}=H_{p1}$

Then, $H_{p1}=H_a-H_s-\eta_d\times\frac{v_1^2}{2g}$

To avoid Cavitation, $H_{p1}=H_v+k'H$

Also, $v_1=C_v\sqrt{2gH} \Rightarrow \frac{v_1^2}{2g}=C_v^2H=xH$

Then, $H_v+k'H=H_a-H_s-\eta_d xH$

$$k'H+\eta_d xH=H_a-H_s-H_v$$

$$H(k'+\eta_d x)=H_a-H_s-H_v$$

$$\sigma H=H_a-H_v-H_s$$

Where, $\sigma=k'+\eta_d x$ called as Thoma Cavitation Number

$$\sigma=\frac{H_a-H_v-H_s}{H}=\frac{H_b-H_s}{H}$$

Where, $H_b=H_a-H_v=$ Barometric pressure Head (10.3m)

Then,

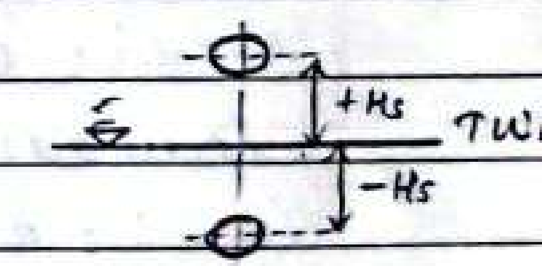
$$H_s=H_b-\sigma H \quad \text{----} (1)$$

When H_s is positive, the position of turbine will be above TWL & if H_s is negative, the position of turbine is below TWL.

→ 'σ' can also be related to specific speed (N_s),

For francis turbine,

$$\sigma=0.032\left(\frac{N_s}{100}\right)^2$$



※ Cavitation:-

We know that the velocity of flow at the outlet of turbine is high & pressure is low. If the pressure at the outlet of turbine is less than the vapour pressure of water, then the cavitation occurs. Cavitation is a phenomenon in which rapid changes of pressure in a liquid lead to formation of small vapor filled cavities, in places where the pressure is relatively low. When subjected to higher pressure, these cavities collapse & can generate shock wave that is strong very close to the bubble but rapidly weakens as it propagates away from the bubble. Cavitation

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causes wear & tear of turbine blades thus reducing the efficiency.

※ Tailrace:-

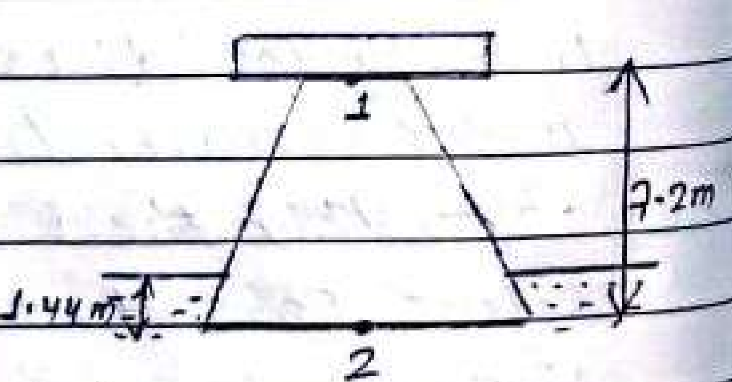
The tailrace, containing tailwater is a channel that carries water away from a hydroelectric plant or water wheel. The water in this channel has already been used to rotate turbine blades or the water wheel itself. This water has served its purposes & leaves the ~~water~~ power generation unit or water wheel area.

Q. A conical draft tube having inlet & outlet diameter 1.2m and 1.8m, discharge water at outlet with a velocity of 3m/s. Total length of draft tube is 7.2m & 1.44m of the length is immersed in water. If atmospheric pressure is 10.3 m of water & loss of head due to friction is 0.2 times of velocity head of outlet. Determine

a) Pressure head at inlet

b) Efficiency of draft tube

→ Given,



Inlet diameter $(D_1) = 1.2m$

Outlet diameter $(D_2) = 1.8m$

Velocity at outlet $(V_2) = 3$ m/s

Total length of draft tube $= 7.2m$

Length of draft tube immersed in water $= 1.44m$

$$P_{atm}/\gamma = 10.3m$$

$$h_f = 0.2 \times \frac{V_2^2}{2g}$$

Applying continuity eqn betn ① & ②

$$A_1 V_1 = A_2 V_2$$

$$\Rightarrow \frac{\pi}{4} \times 1.2^2 \times V_1 = \frac{\pi}{4} \times 1.8^2 \times 3 \quad \Rightarrow V_1 = 6.753 \text{ m/s}$$

Applying Bernoulli's eqn betn ① & ②,

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 + h_f$$

$$\Rightarrow \frac{P_1}{\gamma} + \frac{6.753^2}{2\times 9.81} + 7.2 = (10.3 + 1.44) + \frac{3^2}{2\times 9.81} + 0 + \frac{0.2\times 3^2}{2\times 9.81}$$

$$\Rightarrow \frac{P_1}{\gamma} = 2.766m$$

$$\text{Efficiency of draft tube } (\eta) = \frac{V_1^2/2g - V_2^2/2g - h_f}{V_1^2/2g}$$

$$= \frac{V_1^2/2g - V_2^2/2g - 0.2\times V_2^2/2g}{V_1^2/2g}$$

$$= \frac{V_1^2 - 1.2V_2^2}{V_1^2} = 0.7631$$

$$= 76.31\%$$

※ Design Considerations of Francis Turbine:-

1) The speed ratio is given by, $\phi = \frac{\overline{V}}{\sqrt{2gH}}$

Where, $\overline{V}$ = peripheral velocity of flow

$\phi \rightarrow 0.6$ to 0.9

The value of 'ϕ' can also be calculated using formula

$$\phi = 0.0197 N_s^{2/3} + 0.0275$$

2) Specific speed of francis turbine can be calculated as,

$$N_s = \frac{2400}{\sqrt{H}} \quad \text{(Empirical Formula)}$$

3) Diameter of turbine is given by, $D = \frac{84.6\phi\sqrt{H}}{N}$

4) Thomas Cavitation Number, $\sigma_c = 0.032\left(N_s/100\right)^2$

5) Setting of Francis Turbine is the vertical distance of turbine axis from the tailwater level. Note that negative setting means the axis is kept below tailwater level.

$$(H_s)_{max} = H_a - H_v - \sigma_c H$$
$$= H_b - \sigma_c H \qquad [H_b = H_a - H_v]$$

where, H_v = Vapour pressure head
H_a = Atmospheric pressure head
H_b = Barometric pressure Head (10.1m of water)

Q. A proposed hydropower having net head of 150m & design discharge of 25 cumec is going to use Francis Turbine. Taking turbine efficiency of 0.81, calculate specific speed, turbine diameter & setting of turbine.

→ Given,
$H = 150m$
$Q = 25\,m^3/s$
$\eta = 0.81$

Approximate specific speed $(N_s) = \frac{2400}{\sqrt{H}} = \frac{2400}{\sqrt{150}} = 195.95$ rpm

Now, $N_s = \frac{N\sqrt{P}}{H^{5/4}} \Rightarrow N = \frac{N_s \times H^{5/4}}{\sqrt{P}}$

$P = \eta \gamma Q H = \frac{0.81 \times 9810 \times 25 \times 150}{746} = 39943.53$ HP

$N = \frac{N_s \times H^{5/4}}{\sqrt{P}} = \frac{195.95 \times 150^{5/4}}{\sqrt{39943.53}} = 514.7$ rpm

No. of magnetic pole pairs, $N_p = \frac{60f}{N} = \frac{60 \times 50}{514.678} = 5.828$

Corrected $N_p = 6$,
Corrected $N = \frac{60f}{N_{p,corrected}} = \frac{60 \times 50}{6} = 500$ rpm

→ Actual/Corrected $N_s = \frac{N\sqrt{P}}{H^{5/4}} = \frac{500 \times \sqrt{39943.53}}{150^{5/4}} = 190.36$ rpm

Now, peripheral velocity of wheel ⇒ $\bar{V} = \phi\sqrt{2gH}$
$\phi = 0.0197 N_s^{2/3} + 0.0275$
$= 0.0197 \times 190.36^{2/3} + 0.0275$
$= 0.679 \quad (0.6 \text{ to } 0.9)$

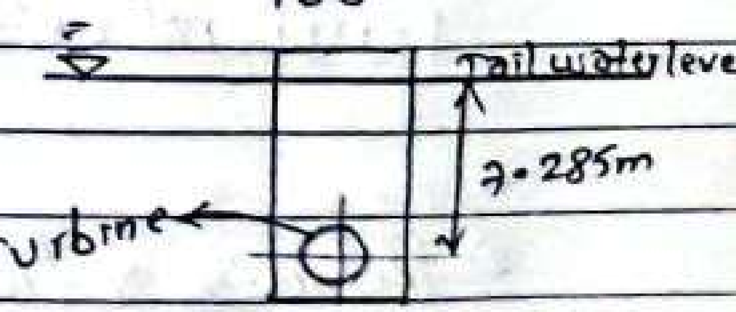
Then, $\bar{V} = 0.679 \times \sqrt{2 \times 9.81 \times 150} = 36.85$ m/s.

Now,
$\bar{V} = \frac{\pi D N}{60} \Rightarrow D = \frac{60\bar{V}}{\pi N} = \frac{60 \times 36.85}{\pi \times 500} = 1.407m$

→ Turbine Setting,
$H_s = H_b - \sigma_c H$

Thoma Cavitation Number $\sigma_c = 0.032\left(N_s/100\right)^2$
$= 0.032\left(\frac{190.36}{100}\right)^2 = 0.1159$

Then,
$H_s = 10.1 - 0.1159 \times 150$
$= -7.285m$



※ Propeller and kaplan Turbine:-
→ Axial flow reaction turbine
→ General arrangement for the propeller & kaplan turbine is much same as francis turbine.
→ Water strikes the vane only after moving few distance axially.

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→ The lower end of the shaft is made larger which is known as hub or boss.

→ In propeller turbine, the vanes are fixed to the hub & they are not adjustable.

→ In kaplan turbine, the vanes on the hub are adjustable. This turbine is suitable where a large quantity of water at low head is available.

→ The main parts are,

1) Scroll Casing
2) Guide vane/Stay vane
3) Shaft + Boss/hub
4) Runner Blade
5) Draft tube

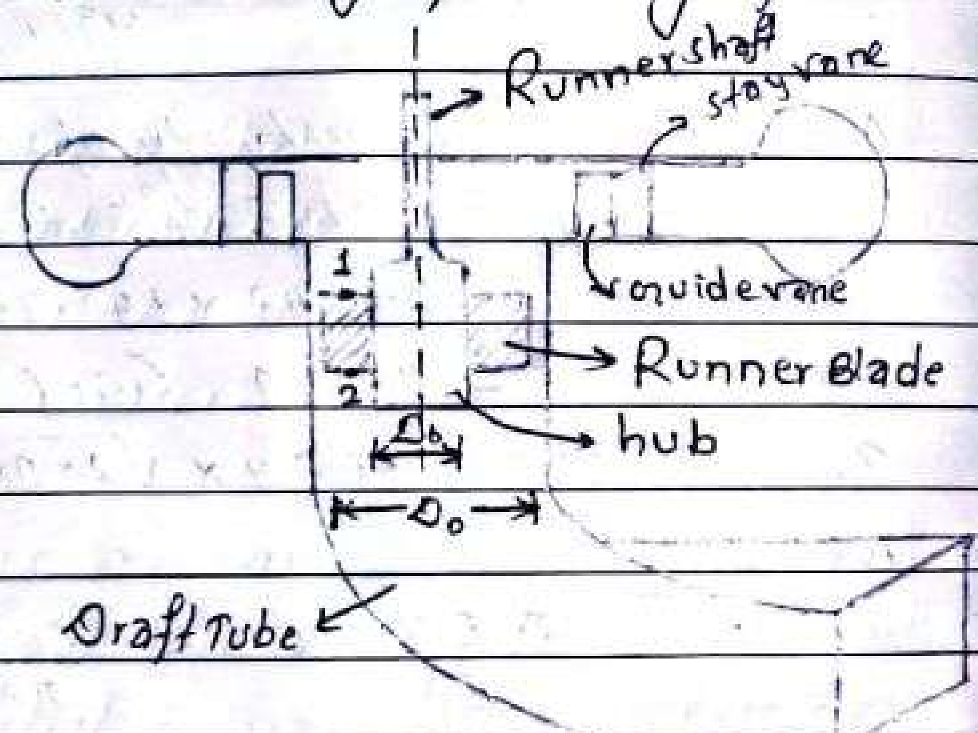

Fig: Propeller & kaplan Turbine

※ Design Steps:-

1) Speed ratio (ϕ) = $\dfrac{\text{peripheral velocity at inlet}}{\text{Available theoretical velocity}} = \dfrac{u_1}{\sqrt{2gH}}$

Peripheral velocity at inlet & outlet are equal,

$u_1 = u_2 = \dfrac{\pi D_o N}{60}$, D_o = outer dia. of runner

2) Velocity of flow at inlet & outlet $(V_{f1} = V_{f2} = V_m)$

3) Area of flow at inlet = Area of flow at outlet = $\dfrac{\pi}{4}(D_o^2 - D_b^2)$

(D_b = Diameter of boss)

4) Discharge through runner, $Q = V_{f1} * \pi/4 \, (D_o^2 - D_b^2)$

5) Diameter of turbine, $D = \dfrac{84.6 \, \phi \sqrt{H}}{N}$

$\phi = 0.0242 \, N_s^{2/3}$ → propeller turbine

$\phi = 0.0252 \, N_s^{2/3}$ → kaplan turbine

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Q. A kaplan turbine runner is to be designed to develop 7357.5kW power. The net available head is 5.5m. Assume the speed ratio is 2.09 & flow ~~velocity~~ ratio is 0.68 & overall efficiency is 60%. The diameter of boss is 1/3rd of diameter of runner. Find the diameter of runner, its speed & specific speed.

→ Given,

power (P) = 7357.5kW

Net available head (H) = 5.5m

Speed Ratio (ϕ) = 2.09

Flow Ratio = 0.68

Overall efficiency (η) = 0.6

$D_b = D_o/3$

we have, $\phi = \dfrac{U_1}{\sqrt{2gH}}$

$U_1 = \phi\sqrt{2gH} = 2.09\sqrt{2\times 9.81\times 5.5} = 21.71 \text{ m/s}$

Also,

Flow ratio $= \dfrac{V_{f1}}{\sqrt{2gH}}$ $\Rightarrow V_{f1} = 0.68\sqrt{2\times 9.81\times 5.5} = 7.06 \text{ m/s}$

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

$7357.5\times 10^3 = 0.6 \times 9810 \times Q \times 5.5$ $\Rightarrow Q = 227.27 \text{ m}^3/\text{s}$

$Q = V_{f1} \times \pi/4 \times (D_o^2 - D_b^2)$

$\Rightarrow 227.27 = 7.06 \times \dfrac{\pi}{4} \times [D_o^2 - (D_o/3)^2]$ $\Rightarrow D_o = 6.788\text{m}$

Also, dia. of hub/boss = 6.788/3 = 2.262m

Now, $U_1 = \dfrac{\pi D_o N}{60}$ $\Rightarrow N = \dfrac{60 U_1}{\pi D_o} = \dfrac{60 \times 21.71}{\pi \times 6.788} = 61.08 \text{ rpm}$

$N_s = \dfrac{N\sqrt{P}}{H^{5/4}} = \dfrac{61.08\sqrt{(7357.5\times 1000/746)}}{5.5^{5/4}} = 220.18 \text{ rpm}$

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* Electromechanical Equipment:-

→ Electromechanical equipments are defined as those equipments that convert either electrical energy into mechanical or mechanical energy into electrical energy.

→ Mechanical Energy to Electrical energy:- Generator
Electrical Energy to Mechanical Energy: Motor

※ Introduction to Generator & their types:-

→ A generator is a device that converts mechanical energy into electrical energy.

→ There are two types of generator

a) Synchronous Generator:-

→ These types of generators are equipped with DC excitation system associated with voltage regulator to provide voltage & phase angle control before the generators are connected to the grid.

→ Synchronous generators can run isolated from the grid & produce power since excitation is not grid dependent.

→ These are more expensive than asynchronous generator.

b) Asynchronous Generator:-

→ These generators draw their excitation current from the grid, absorbing reactive energy by their own magnetism.

→ They cannot generate when disconnected from the grid because they are incapable of providing their own excitation current.

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※ Working Principle of governers in pelton & Francis Turbine:-

→ The governing of a turbine is defined as the operation by which the speed of turbine is kept constant under all working conditions. It is done automatically by means of governor, which regulates the flow through turbine, according to changing load condition.

→ Governing of a turbine is necessary as turbine is directly coupled to an electric generator, which is required to run at constant speed under load fluctuation. The frequency of power generation by a generator of a constant number of pair of poles under varying load condition should be same.

→ When the load on the generator decreases, the speed of the generator increases beyond the normal speed. If the turbine or generator is to run at constant speed, the rate of flow to the turbine should be decreased till the speed becomes normal.

→ The governor of a pelton turbine decreases or increases the outlet area of the nozzle by moving the spear valve. In case of francis turbine, the governor decreases or increases the wicket gate.

※ Pumps:-

→ Pumps are hydromechanical equipment that convert hydraulic energy into mechanical energy.

→ Types of pumps

a) Centrifugal pump

b) Reciprocating pump

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※ Centrifugal pump:-

→ If the hydraulic energy is converted into mechanical energy by means of centrifugal force acting on the fluid, the pump is known as centrifugal pump.

→ The centrifugal pump works on the principle of forced vortex flow, which means that when an external torque rotates a certain mass of liquid, the rise in pressure head of the rotating liquid takes place. The rise in pressure head at any point is proportional to the square of tangential velocity of the fluid at that point.

→ The main parts of centrifugal pump are,

- Impeller
- Casing
- Suction pipe with foot valve & Strainer
- Delivery pipe

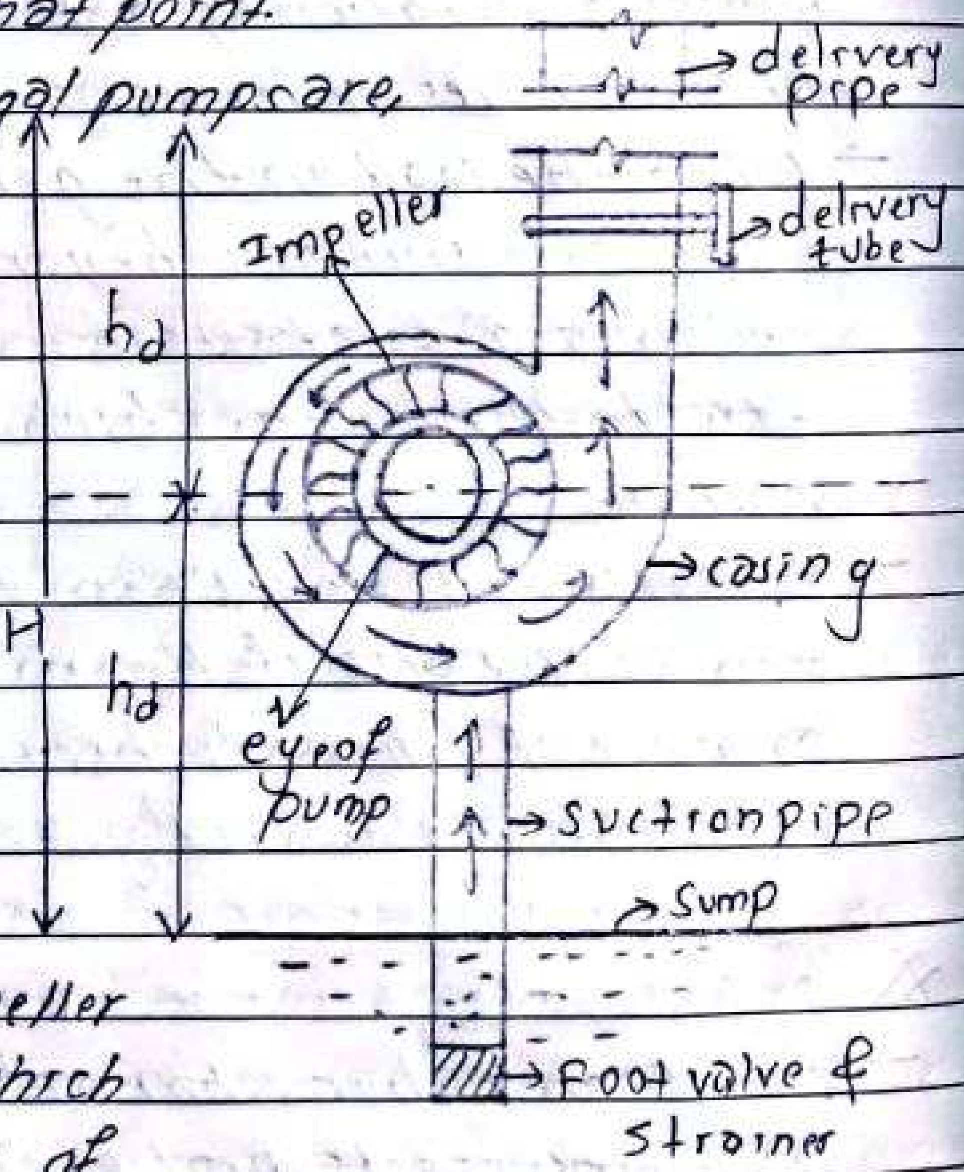

Fig: Main parts of centrifugal pump.

* Working Principle:-

The impeller is the key component of a centrifugal pump. It consists of series of curved vanes. The impeller is mounted on a shaft, which is connected to the shaft of electric motor. A casing is an air tight passage surrounding the impeller & is designed in such a way that kinetic energy at outlet of impeller is converted into pressure energy before the water leaves the casing & enters the delivery pipe. Casing may be volute or vortex casing.

→ The fluid enters the pump at the impeller eye. The velocity of the fluid increases by the centrifugal force created due to the rotation of the impeller. Therefore, the fluid is radially moved out towards the impeller periphery. The fluid is directed to an expanding volute casing or diffuser & thus its velocity energy is converted to pressure head. The increase in the fluid pressure head at any point is proportional to the square of tangential velocity of rotating fluid.

→ The process of filling the pump with liquid is called priming. If the pump casing filled with vapors or gases, the pump impeller becomes gas bound and incapable of pumping. To ensure that a centrifugal pump remains primed & does not become gas bound, most centrifugal pumps are located below the level of source from which the pump is to take its suction.

※ Reciprocating Pump:-

→ If the mechanical energy is converted into hydraulic energy by sucking the liquid into a cylinder in which a piston is reciprocating which exerts the thrust on the liquid & increases its hydraulic energy, the pump is known as reciprocating pump.

→ Reciprocating pumps are more suitable for low volumes of flow at high pressures.

→ Major components

a) A cylinder with piston rod, connecting rod & a crank
b) Suction pipe
c) Delivery pipe
d) Suction valve
e) Delivery Valve

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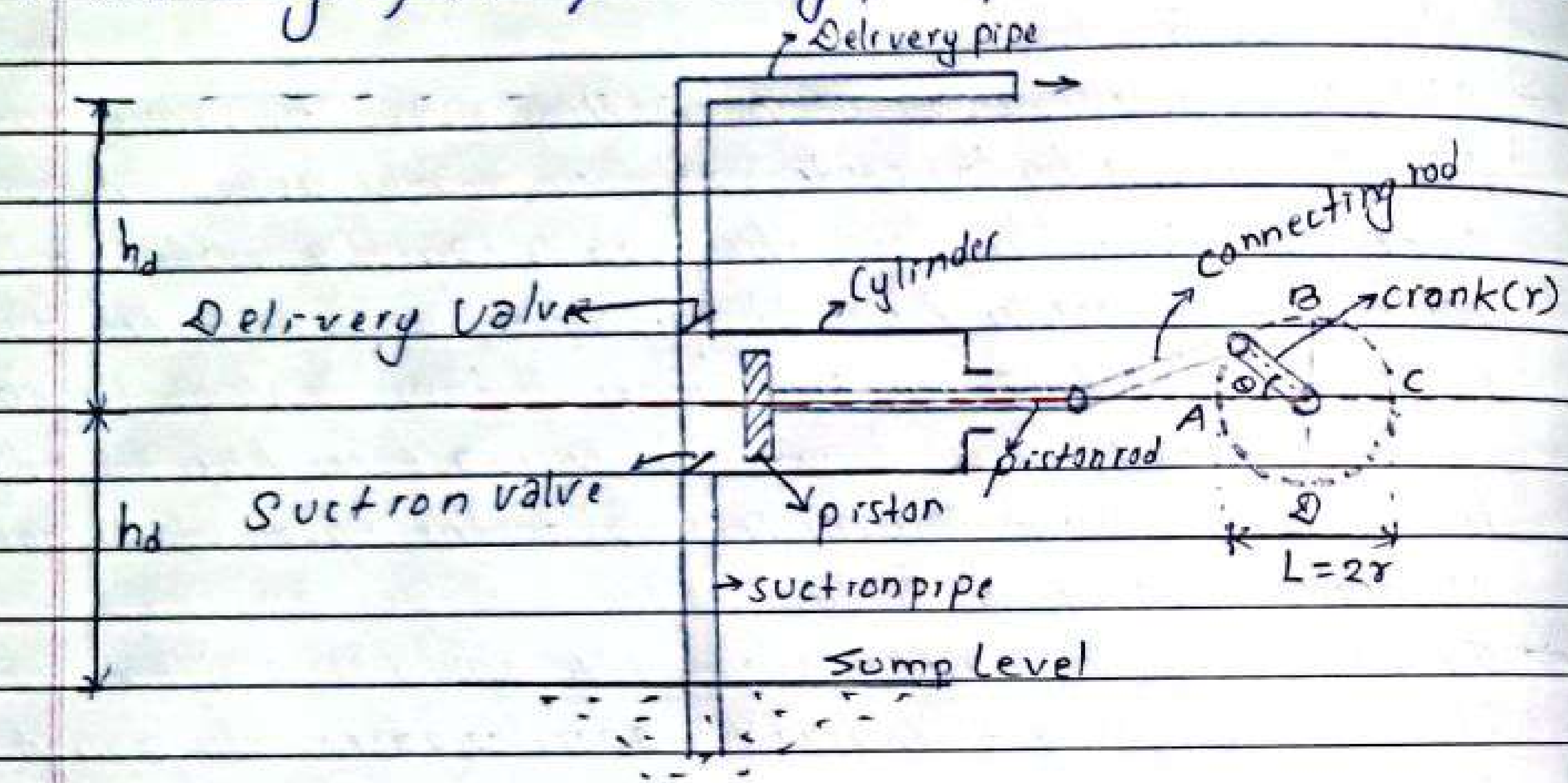
* Working of Reciprocating Pump:-



→ Figure shows a single acting reciprocating pump. The movement of the piston is obtained by connecting the piston rod to crank by means of a connecting rod. The crank is rotated by means of an electric motor. The suction & delivery valve allows the water to flow in one direction only. Suction valve allows water from suction pipe to the cylinder which delivery valve allows water from cylinder to delivery pipe.

→ When crank starts rotating, the piston moves to & from in the cylinder. The movement of the piston towards right creates a partial vacuum in the cylinder. But on the surface of liquid in the sump atmospheric pressure is acting, which is more than the pressure inside the cylinder. Thus the liquid is forced in the suction pipe from the sump. This liquid opens the suction valve & enters the cylinder. In the next round, the piston moves towards left & increases the pressure of the liquid inside the cylinder more than atmospheric pressure. Hence suction valve closes & delivery valve opens. The liquid is forced into the delivery pipe & is raised to a required height.

Chapter- 9. Powerhouse Planning

Outline:-

→ Classification, general arrangement & layout plan of powerhouse.

→ General dimension calculation of powerhouse.

* Introduction:-

→ The structural complex where all the equipments for producing & providing electricity are suitable arranged is called powerhouse.

→ The components of powerhouse are arranged in such a way that higher functional efficiency & aesthetic beauty is obtained.

* Classification of Powerhouse:-

(a) Surface Powerhouse:-

→ Such powerhouse is constructed above the ground so that it has less space restrictions.

→ But, the foundation analysis of surface powerhouse should be carefully examined. If solid bed rock is not available in surface, special foundation treatment is essential.

(b) Underground Powerhouse:

→ When enough is not available for surface powerhouse, underground powerhouse is adopted.

→ In some places where the cost of land is too expensive, underground powerhouse can be more feasible option.

→ The water conveyance length & penstock length can be shortened by providing underground powerhouse.

→ In some condition with good quality rock, the underground powerhouse may be economical.

* Powerhouse Structures:

→ The vertical section of powerhouse is divided into different parts based on the axis of turbine.

→ For horizontal axis turbines, two floors are provided called substructure & superstructure. In this arrangement, turbine & generator are placed at same floor.

→ A powerhouse with vertical axis turbine can commonly be classified into three main divisions based on the vertical setting of powerhouse.

(a) Substructure:

→ Structure that is situated below the axis of turbine.

→ It is located below ground level & includes draft tube, tailwater channel & galleries as per design

→ It transmits load to the foundation strata.

(b) Intermediate Structure:

→ It extends from turbine axis to top of generator

→ It includes casing, governor, generator & its appurtenances.

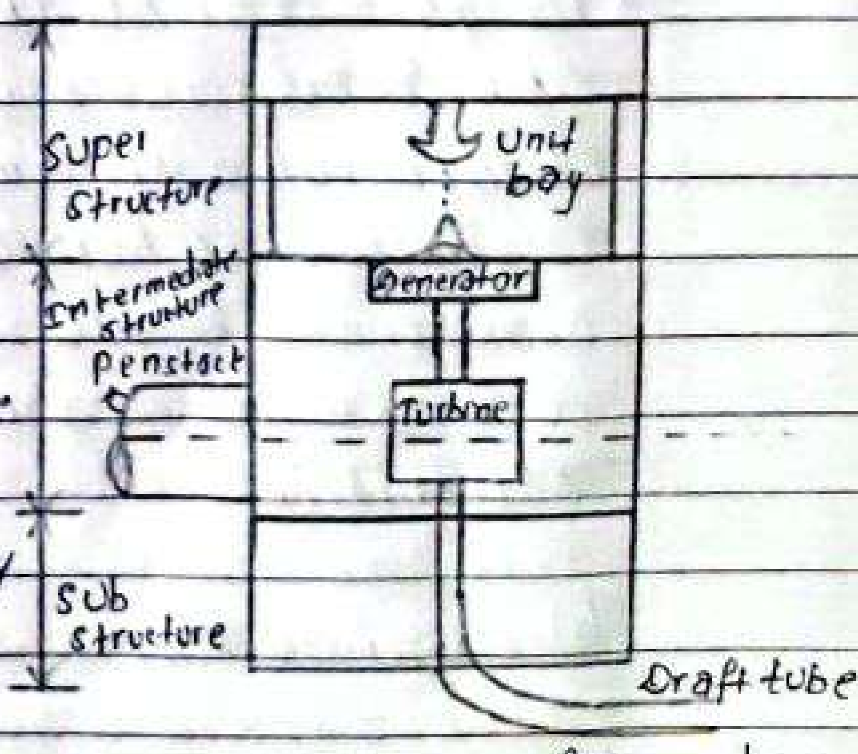
→ The turbine floor is generally provided immediately above the turbine axis & it can be used to have access to turbine runner.

(c) Super structure:

(A vertical section of powerhouse is shown in figure)

→ Superstructure extends from generator floor to roof of powerhouse

→ It consists of generator & governor control rooms, exciters & auxiliary equipments needed for ventilation & cooling. It also consists of walls & roof with a main travelling gantry crane at the roof level.

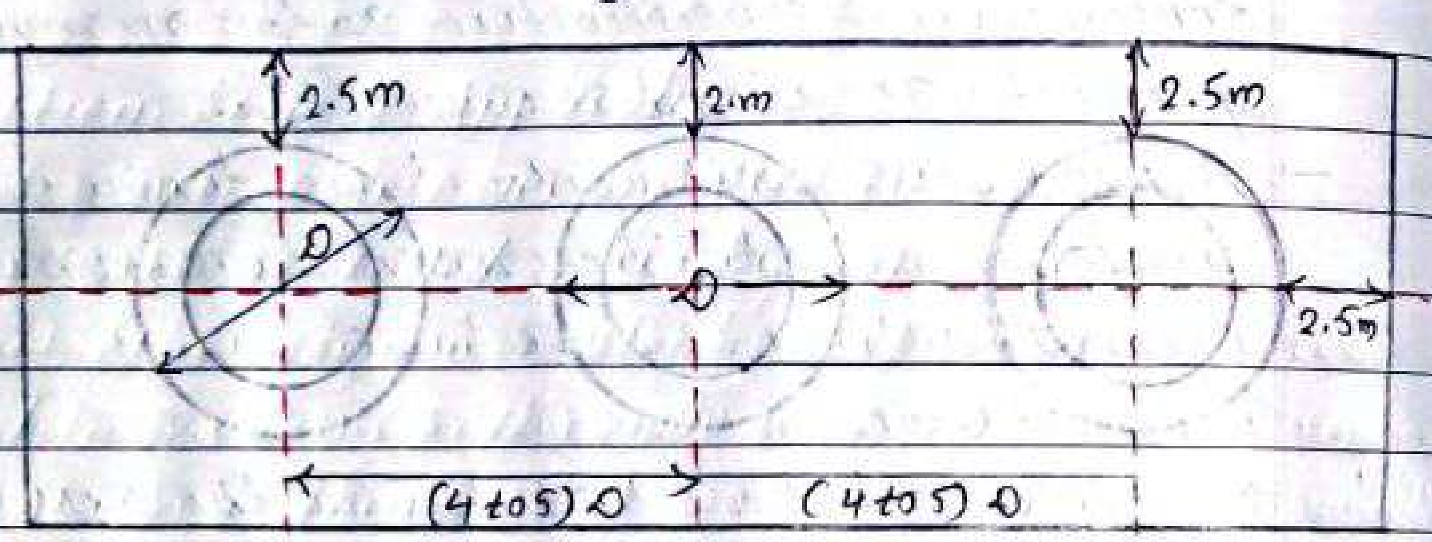


Vertical Section of powerhouse

* General dimension calculation of powerhouse:

The basic constituents (bay) of superstructe of powerhouse are,

(a) Machine Hall (Unit Bay)



→ Length: Length of machine hall depends on number of units, distance betn the units, size of machines & clearance. The spacing betn two turbine units is kept (4 to 5) D where D is the diameter of turbine.

→ Width:- Width of machine hall is determined by size & clearance space from the walls. The width of machine hall can be presumed to be (5D+2.5)m. The generator is not placed at the centre of width, it is shifted to one side so as to provide adequate operating space.

→ Height: The height of machine hall is fixed up by the head room requirements of crane operation. In general 2 to 2.5m head requirement is for crane operation. The hall must have a height which will enables the cranes to lift the rotor of the generator & runner of the turbine without any other machines sets forming ~~of~~ obstruction.

(b) Loading Bay:

→ It is a space where heavy vehicles can be loaded & unloaded, the dismantled parts of the machines can be placed & where assembling of the equipments can ~~be done~~. ~~The~~ be done. The loading bay should be of sufficient to receive the large parts like the rotor & runner. The loading bay floor will be having a width at least equal to the centre distance of machines.

(c) Control bay:-

→ Control bay is the main room where equipments like runner, gate valves, generator etc. are controlled. It may be adjacent to the machine hall as it sents instruction to the operation bay from where the operation control is achieved.

Chapter 10, Micro Hydropower plant

Outline:-

→ Introduction, scope & applications
→ Introduction to policy of MHP development in Nepal
→ Advantages & relevance of MHP in Nepal
→ General layout of basic components of MHP.

* Introduction, Scope & Applications:-

→ The hydropower plants having installed capacity of 10 kW to 100 kW are called micro hydropower plants.

→ The first Microhydropower plant of 5 kW capacity was installed in Godavari, kathmandu with the swiss assistance in 1962 AD. Since then around 3300 MHPs have been installed in the country in hilly & mountains locations. These mini/micro/pico hydropower plants are generating close to 30 MW of installed capacity to provide electricity for about 3,50,000 households approximately.

→ Grinding, hulling, water pumping are also done with electricity generated by MHPs & also powered to operate computers, photo studios, poultry farms & some small industries.

→ As radio, televisions & internet are used in the villages, electricity has been a meaning to provide opportunity to get latest & useful informations to the users. they get news & views of national & international importance & useful tips for their occupational betterment as well. Students can study in the evenings in bright lights. Thus, MHP has been proved to be instrumental in remote villages where access of electricity through grid connection is synonymous to a dream. The efforts made by organizations like ITDG Nepal, IUCN, ICIMOD, ADB, different educational

institutes etc. & dedicated professionals of MH sector are to be appreciated to bring the encouraging results in this sector.

→ Gradually through various workshops, technicians have obtained knowledge & skill to manufacture & all required technology for MHP. However, it took around 30 years to be a discipline of industries. The technicians & enterpreneurs have been able to carry jobs for survey, design, manufacturing of turbines & equipment, installatio commissioning as classified jobs as per their professional expertise in the later phase.

* Introduction to policy of MHP development in Nepal:-

→ Alternate energy Promotion Centre (AEPC), Under Ministry of Energy, water Resources & Irrigation is the government institution established with objective of developing & promoting renewable/alternative energy technologies in Nepal.

→ The various policies of government regarding microhydro development in Nepal are;

a) Hydropower Development policy 2001
b) Renewable Energy Subsidy policy 2000/2006
c) Subsidy Delivery Mechanism 2000/2006
d) Rural energy policy 2006.

→ The highlights of different hydropower development policies are,

1) To generate electricity at low cost
2) To extend reliable & qualitative electric service.
3) To tie up electrification with economic activities
4) To render support to the development of rural economy.

5) Operating small & mini hydropower projects at local level.
6) Make electric service available to as many people as possible.
7) Participation of local bodies.
8) Discourage dependence on excessive subsidy.
9) Encourage financing institutions to invest.

Advantages & Relevance of MHP in Nepal:-

→ Nepal is rich in water resources but due to lack of advance technology & skilled manpower, we have not been able to make its optimum use. Development of large scale hydropower requires large investment. We have not been able to invest ourself and on the other side attract international investors. So, meanwhile Microhydropower can be good source of energy to us.

→ The major importance of microhydropower plants can be highlighted in following points

1) It relies on a renewable, non polluting indigenous resource that can displace petroleum based fuels that are frequently imported at considerable expenses & effort.
2) A micro hydro can be source of income to the skilled & unskilled manpower within the vicinity.
3) As a component of water development scheme, it can be integrated with irrigation & water supply projects to maximize benefits while sharing the cost among several sectors.
4) It is well proven technology; generally well beyond the research and development stage in addition ~~to~~ hydropower resources have already been harnessed for years by rural entrepreneurs & farmers in our country.
5) Microhydropower utilizes the local resources for ~~engineering~~ generating the energy.

6) Microhydropower schemes permit local villager involvement in the full range of activities from initiation & implementation to operate & maintenance. When villagers contribute labor & local materials, the cost incurred are lower & when villagers are committed to a properly planned & executed project, the possibility of its long term success increases significantly.
7) Large hydropower are connected to national grid through large transmission lines. In remote areas, where it is very difficult to transmit national grid due to topography & uneconomical to transmit electricity to distant places microhydro can be economic to produce & supply electricity to the isolated villages even without being connected to the national grid.

General layout of basic components of MHP:-