History of Hydropower Development in Nepal

1. First hydropower plant was Pharping commissioned in 1911. Capacity is 500 KW
2. After that Sundarijal 640KW and Khoposi 2400 KW were constructed. Khoposi commissioned in 1965.
3. In 1989, total installed capacity of country reached 250 MW after construction of Kulekhani, Marsyangdi, Trishuli, Sunkoshi, etc. with assistance from India, China, Russia along with loan assistance from Worldbank, ADB, German and Japanese governments.
4. In 2001 (2058 BS) Hydropower Developent policy was formed following the previous policy in 1992. The Hydropower development policy in 2058 BS attracted private sector towards hydropower development.
5. Government of Nepal adopted policy to purchase power from hydropower developers.
6. The fund investment for hydropower development is also managed by banks and other institutions.
7. In 2049 BS, Electricity act was formulated which facilitated developers for hydropower developement.
8. Some hydropower projects developed by private sector are: Khimti (60MW), Bhote Khoshi (36MW), Indrawati (7.5MW), Sunkoshi small (2.6MW), Khudi (4MW), Kalarga gad, Naugad, Makarigad, etc.

9. As of now, almost 50% of hydropower development is contributed by Independent Power Producers (IPP'S).

10. Lower Marsyangdi - 69 MW in 1989. - NEA

11. Kaligandaki - A, 144 MW in 2022 - NEA

12. Middle Marsyangdi, 70 MW in 2008.

13. Kulekhani - I, 60 MW in 1982 is the single storage project.

14. Kulekhani - II as cascade of Kulekhani - I with capacity 32 MW in 1986

15. Kulekhani - III with capacity 14 MW in 2019.

16. Upper Trishuli - 3A, 60 MW in 2019.

17. Upper Tamakoshi - 456 MW in 2021, the largest hydropower plant in country.

18. Vidyut Utpadan Company Limited, VUCL in 2016, Rastriya Prasaran Grid Company Limited, RPGCL in 2015 and Hydroelectricity Development Company Limited, HIDCL in 2011 were formed.

19. As per annual Report of NEA 2021 - 2022, total installed capacity is 2081.78 MW hydropower is developed in country.

Present Status of Hydropower Development in Nepal:

As per annual Report of NEA, 2021-22

Type	Capacity (kW)
Total hydro - NEA - Grid connected	578,624
Total small hydro NEA - Isolated	4536
Total hydro - NEA	583,160
Total Hydropower NEA Subsidiary	478,100
Total Hydro IPP	10,20,580
Total hydro (NEA+ IPP	2081,788
Total Thermal (NEA)	53,410
Total solar (NEA)	33,140
Total installed capacity (NEA+IPP) Grid connected	2,185,382
Total installed capacity	2,189,918

* Total contribution in Energy for year 2021-22 by NEA = 47.32%.
* Total contribution in Energy by IPP = 38.74%.
* Total energy import from India = 13.94%.
* Total energy exported to India = 493 GWh
  GWh = Gigawatt Hour.

Hydropower Development Policy; 2058

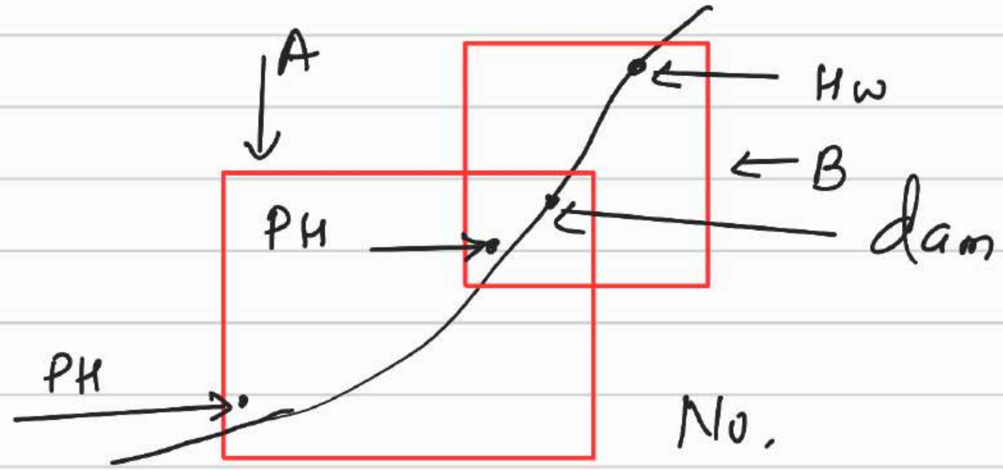
* Formulation: 15 October, 2001.

Objectives:

1. To utilize existing water resource of the country and to produce electricity at low cost.
2. To ensure dependable reliable and quality electricity at reasonable price.
3. To tie up electrification with rural activities.
4. To extend rural electrification to support rural economic development.
5. To develop hydropower as export commodity.

Some Provisions to fullfill objectives:

1. Environmental provisions:
   → 10% of minimum monthly flow as down-stream release.
2. Provisions for water rights.



3. Provisions of Investment in Production, Transmission and distribution.
4. Provisions for special Investment for Infrastructure Development of Rural Electrification

5. Projects should be transferred to GON in good running condition.
6. Provision for power purchase
→ power shall be purchased by Government.
7. Provisions related to Visa:
8. maximum utilization of local resources.
9. management of investment risks.
10. Provision on internal electricity market
11. Provision for export of Electricity.
12. Provision for license:
   a. Survey licence – 5 years.
   b. Generation license – 35 years for domestic consumption. and 30 years for export oriented.
   c. Transmission license – 25 years.
   d. Distribution license – 25 years.
   → No license required for hydropower upto 1mw.
13 Provision related to fees:
→ The developer shall pay Capacity Royalty and Energy Royaltee to government.
14. Facilities relating to Taxes and customs.
→ Custom duty – भन्सार महसुल – 1%.
→ VAT discount

15. Institutional Provisions:

a. Regulatory Body:–

To determine electricity tarrif, relaible electricity supply.

b. Study and Promotion Body:

* WECS, Department of Electricity Development

c. Electricity Management and Research Institute.

16. Construction and operation of Hydropower projects by GON –

→ Government may develop projects on it's own or by entering treaty with any friendly country.

Electricity Act, 2049: (विद्युत ऐन, २०४९)

* It regulates survey (design), generation (construction), transmission and distribution of electricity.

Section 3: → No survey, generation, transmission and distribution without license.

Section 4: * Application for licence.
* survey license within 30 days
* Generation license within 120 days.

Section 5: Terms of license:
* Survey license : maximum 5 years
* Generation, transmission & distribution license: maximum 50 years.

Section 8: License may be cancelled
* If any licensee violates terms of license.

Section 9 Agreement may be entered with licensee → licensee लिने संस्था / व्यक्ति
* GoN may enter agreement for purchase of electricity, capital investment, technical matters.

Section 10 : Ownership of GoN.
* Any project after expiry of license goes under GoN.

Section 11: Royalty to be Paid:
* Capacity Royalty = 100 Rs/kW upto 15 years
= 1000 Rs/kW after

15 years.

* Energy Royalty = 2% of energy revenue for 15 years.
  = 10% of energy revenue after 15 years

Section 12: Facilities related to Tax and other charge.

→ Custom: only 1% on machinory, tool plants to imported.

→ VAT discount for machinory, tool, plants.

Section 13: Facility for foreign exchange:

* foreign exchange facility at prevailing exchange rate for repatriation of investment or repayment of principal or interest of loan.

Section 14: Electricity Charges and other charges to be realized.

* Government of Nepal may fix tarrif as per Electricity Regulatory Commission act.

Section 19: Electricity service may be stopped.

→ for inspection, repair & maintenance

→ Calamities, disaster.

→ Who donot pay tarrif (महसुल)

Section 21: Sale of Generated Electricity

→ Government of Nepal may purchase or cause to purchase the electricity generated by any licensee./developer.

Section 23: * Fixing quality and standard of electricity by GON after publishing notice to Nepal Gazette.

Section 24: * No substantial Adverse effect to be made on environment.

Section 31: Security of Electricity structure:
* GON may make necessary arrangement for security of generation plant, transmission plant, substation at the cost of licensee.

Section 34: * Government of Nepal may generate generate and develop electricity.
→ No licensee required.

Section 38: Penalties:
* If any person/institution violates this act, penalty upto 5000 may be imposed.
* If any damage is made, the person has to compensate.
* If any developer/licensee violates terms of license, prescribed officer may impose fine upto 5 lacs.

Types of Hydropower Projects:

1. Run of River (RoR) Projects:

* Run of river projects are those projects which utilize available flow in the river by diversion of available water.
* They donot have provision for storage.
* A simple diversion weir/barrage is constructed to divert the river discharge into conveyance structure.
* When flow available in the river is less than design flow, their generation falls below design capacity. So, they donot generate at uniform rate throughout the year.
* Components of RoR projects are weir/barrge, undersluice, stilling basin, intake, gravel trap, settling basin, headrace (tunnel, canal, pipe), surge tank/ fore bay, penstock, powerhouse including all equipments, tailrace, switchyard, etc.

2. Peaking Run of River (PROR) Projects:

* It is similar to run of river project. However a reservoir is provided to store daily water so that it can be supplied to the turbines when during peak time.
* In the moths when discharge in rivers is less than design discharge, the reservoir stores water in off- peak hours and supplies during peak hour.
* It is constructed to manage daily peak load.

* In our system, demand is more during 5pm to 11:00 pm in evening and less during other times of day.

5pm to 11pm → peak hour.

11pm through day to 5pm off-peak hour.

* When discharge in river is more than design discharge, there is no need of storing water.
* PRoR projects are costlier than RoR projects therefore, their energy rate during peak hour in dry months is higher than RoR projects.

E.g: Consider a PRoR project with design discharge 40 m3/sec, net head 100m, overall efficiency = 85%. Then,

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

$$= \frac{0.85 * 9.81 * 40 * 100}{1000} \text{ MW}$$

$$= 33.35 \text{ MW}.$$

1) Consider a month of April, when discharge is 10 m3/sec.

* Power generation in April as RoR

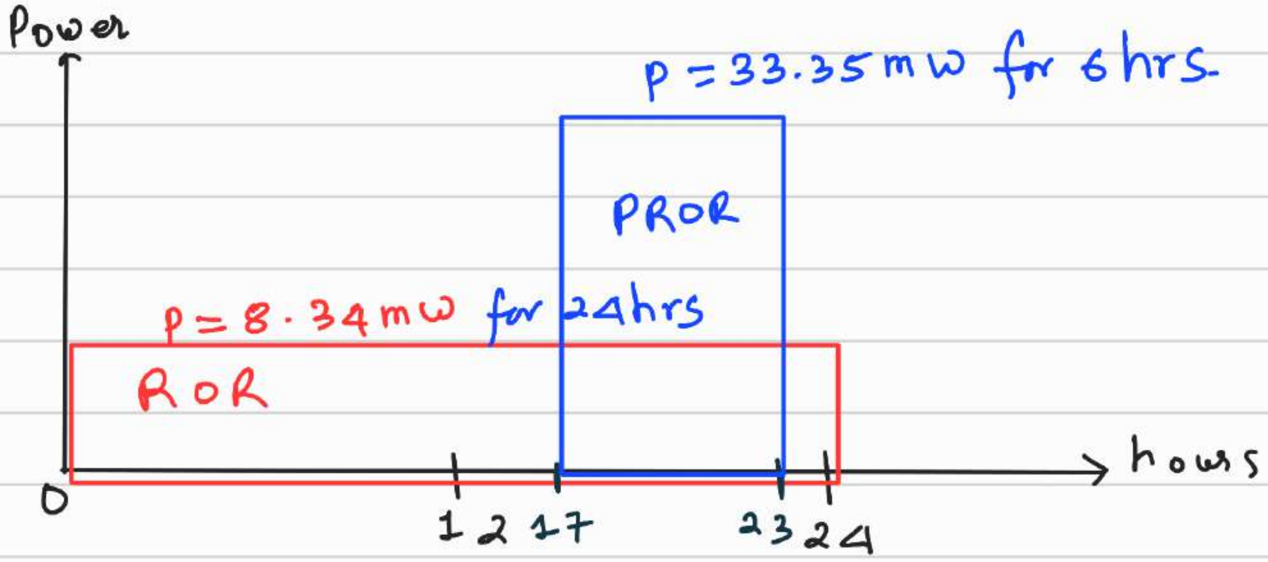
$$= \eta \gamma Q H = \frac{0.85 * 9.81 * 10 * 100}{1000}$$

$$= 8.34 \text{ MW}$$

* If the plant is to be operated as 6hr PRoR, then, discharge available,

$$Q = 10 * \frac{24}{6} = 40 \text{ m3/sec}$$

∴ If the plant is operated as PROR, it would generate 33.35 mw for 6 hrs.



Volume in 24hr = $10 * 24 * 60 * 60$

discharge if this vol. is used in 6hr

$$= \frac{vol}{time} = \frac{10 * 24 * 60 * 60}{6 * 60 * 60}$$

$$= 40 \, m^3/sec$$

(2) In the month of July, let discharge = $80 \, m^3/sec$

∴ Power generation, $P = \eta \gamma Q H$

$$= \frac{0.85 * 9.81 * 40 * 100}{1000}$$

$$= 33.35 \, mw$$

⇒ No need of storage.

* Components: dam, peaking pond, undersluice, intake, settling basin, tunnel/pipe, surge tank, penstock, powerhouse with all equipments, tailrace and switchyard.

3. Storage Project:

* A storage project stores the water during wet season and utilizes the water stored during dry season when discharge available in the river is low.

* Since, the discharge in rivers falls significantly during dry season, the RoR project constructed on the rivers would generate at low capacity. However demand remains same.

* So, to maintain seasonal gap between demand and generation, storage projects are constructed, which can generate at uniform capacity throught year.

* A storage project incurs large cost as well as more environmental impacts. However, the rate of energy from storage projects is higher than RoR and RoR projects.

* A storage project has other benefits as well like irrigation, flood control, fishery, etc.

* It maintains gap of seasonal variation in energy production.

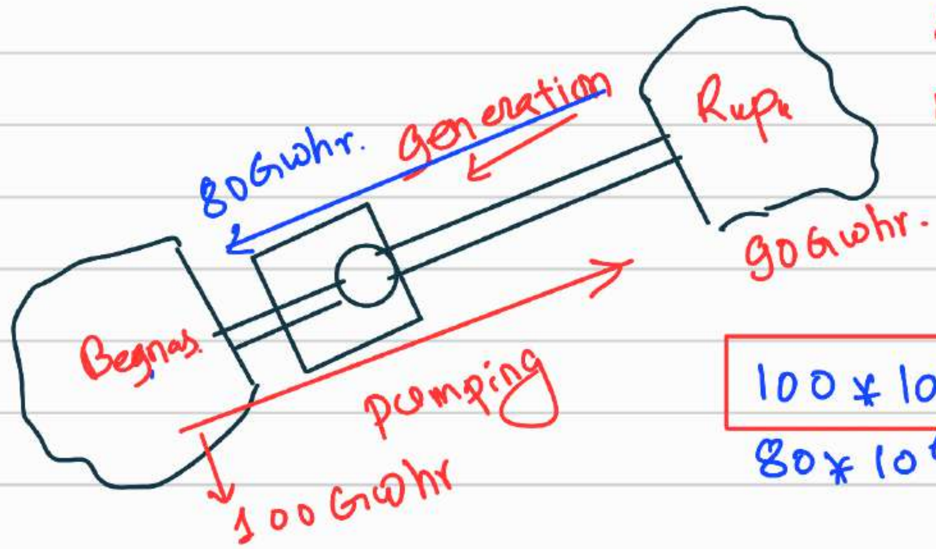
* It increases dry season energy.

* Components are: dam, spillway, intake, undersluice or bottom level outlets, intake, tunnel, surge tank, penstock, powerhouse, tailrace, switchyard.

4: Pumped Storage Projet:

* It consists of two reservoirs, one on upstream and one on downstream.
* During generation time, water flows from upper to lower reservoir.
* During pumping time water flows from lower to upper reservoir.
* This type of project is required to adress peak load demand. Generation is done when energy rate is high and pumping is done when cheap energy is available in the grid.

* Components are: upstream reservoir, intake, tunnel/penstock, surge tank, powerhouse, downstream reservoir, switchyard.



Energy rate: 12.4 Nrs.

min. energy rate = 4.8

$\boxed{100 * 10^6 * 4.8 = \text{Cost}}$

$80 * 10^6 * 12.4 = \text{Benefit}$

Layout of RoR Plants:

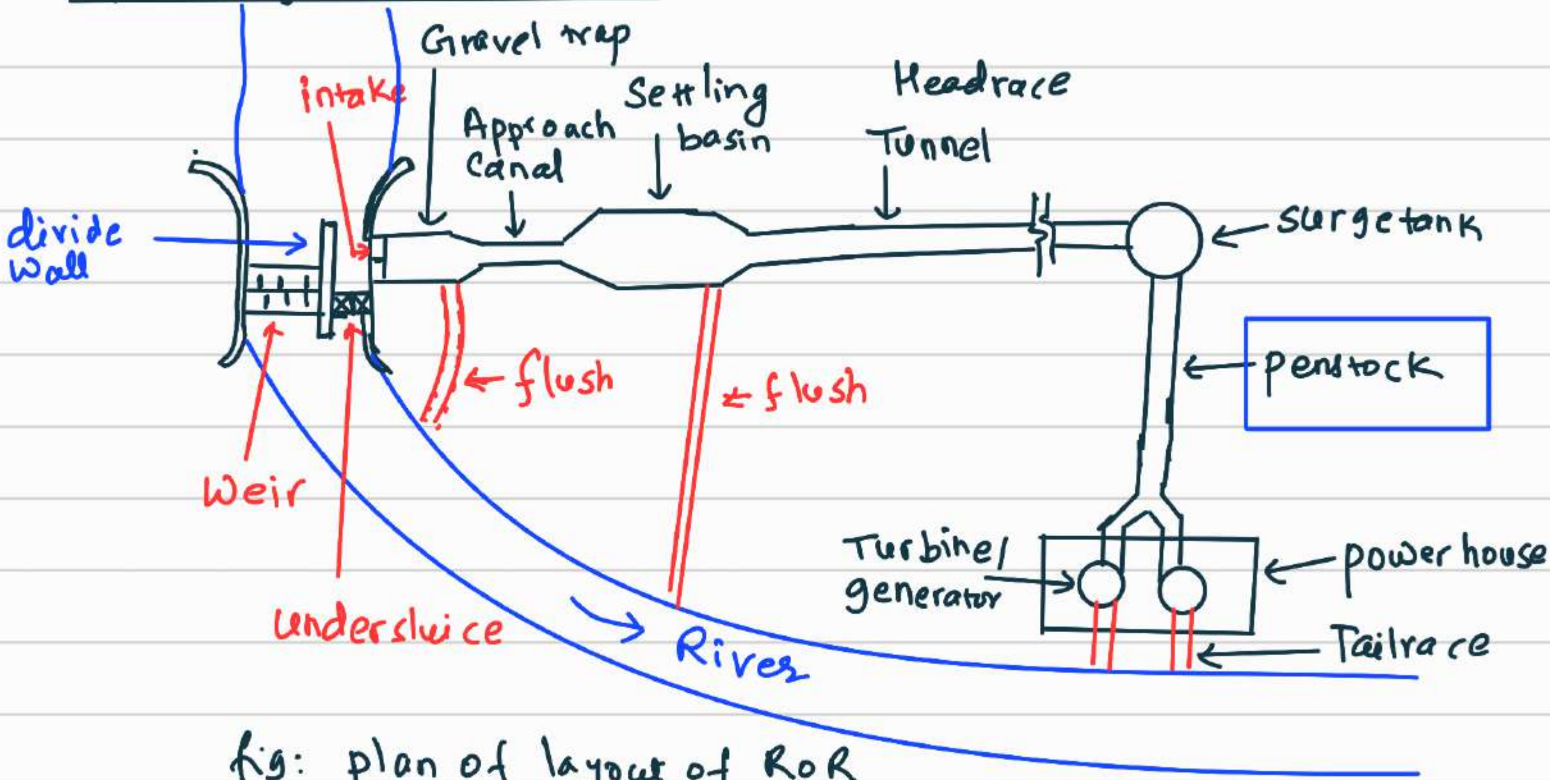


fig: plan of layout of RoR with tunnel option

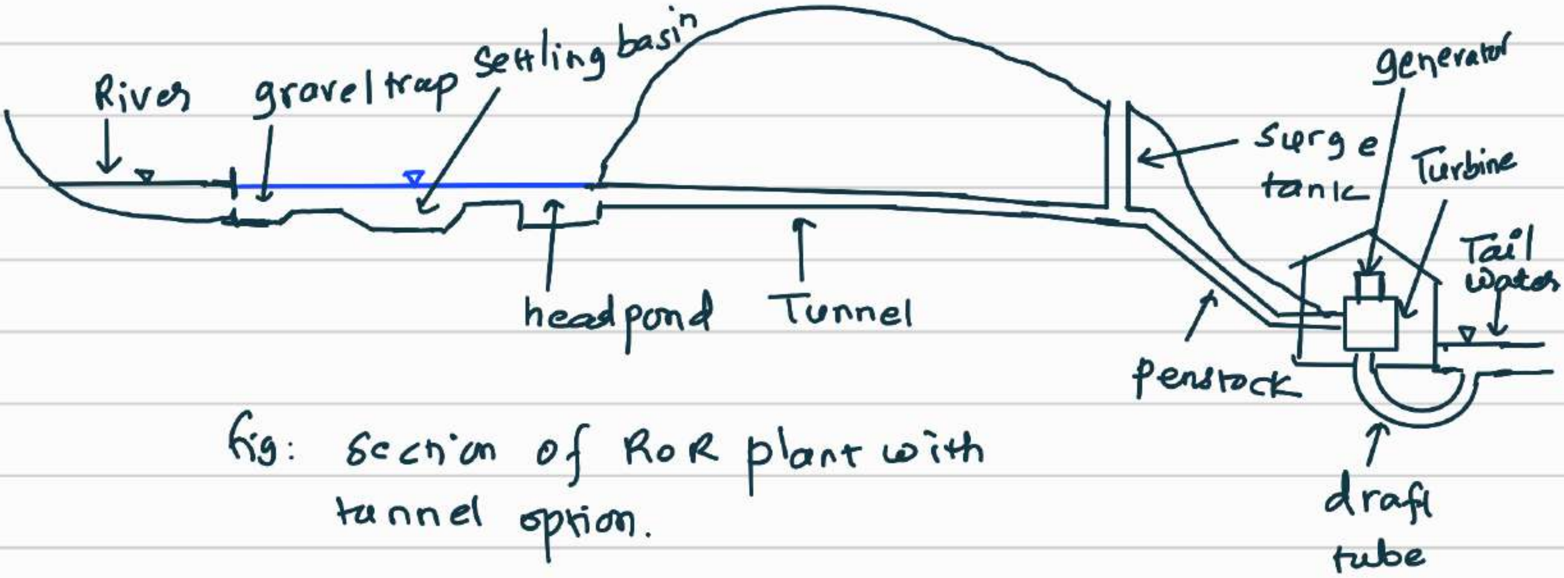


fig: section of RoR plant with tunnel option.

Note: RoR and PRoR have same layout. However, a PRoR project have peaking pond.

When canal option is Preferred:

* Canal option is economical option if the terrain has gentle slope and the terrain is geologically stable.
* But, Canal option includes land acquisition,

which may not be feasible if land cost is high.

When Pipe option is Preferred:

* Pipe option is suitable when discharge is low and the alignment is safe from landslides, rockfalls, etc.
* Also if no. of crossings are minimum this option is economical.
* If discharge is high, alignment has so many loops, and alignment is geologically unstable, this option is not preferred.

When Tunnel option is Preferred:

* When discharge is high, surface geology is weak and topography is such that canal and pipe length becomes large, no. of crossings are more, tunnel option is preferred.
* It is costly, takes longer construction period.
* land acquisition is minimum, head loss is reduced.

Layout of Pumped Storage Project:

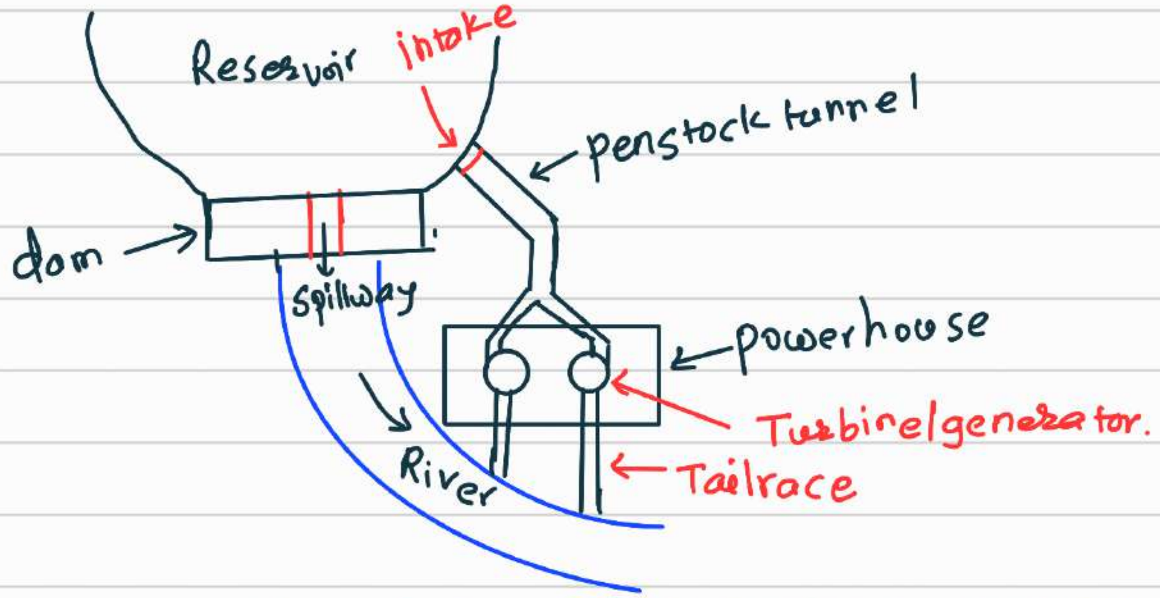
Power and Energy

Flow Duration Curve:

Layout of Storage Project:

a. layout with powerhouse at dam toe:

b. layout with powerhouse away from dam toe



plan of layout of storage project with powerhouse at dam toe

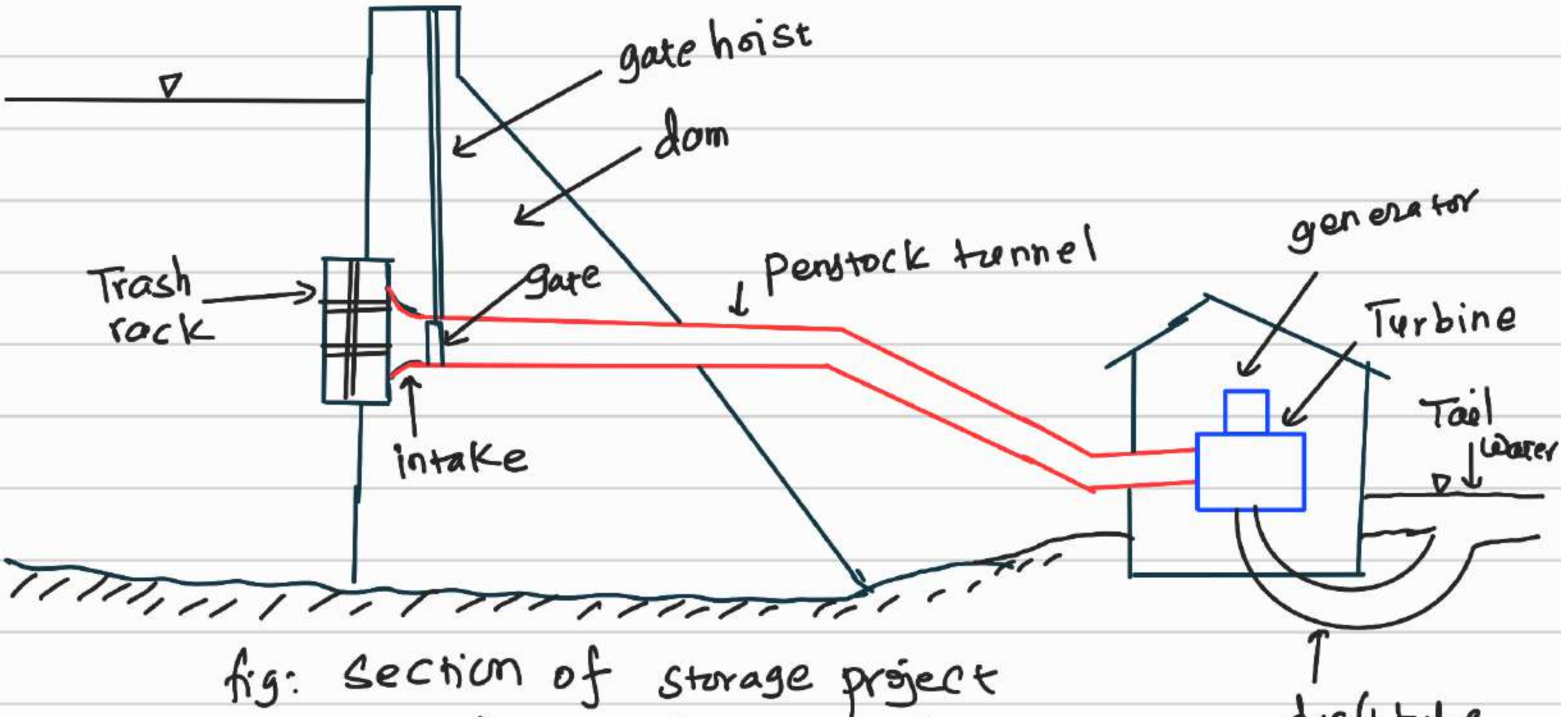
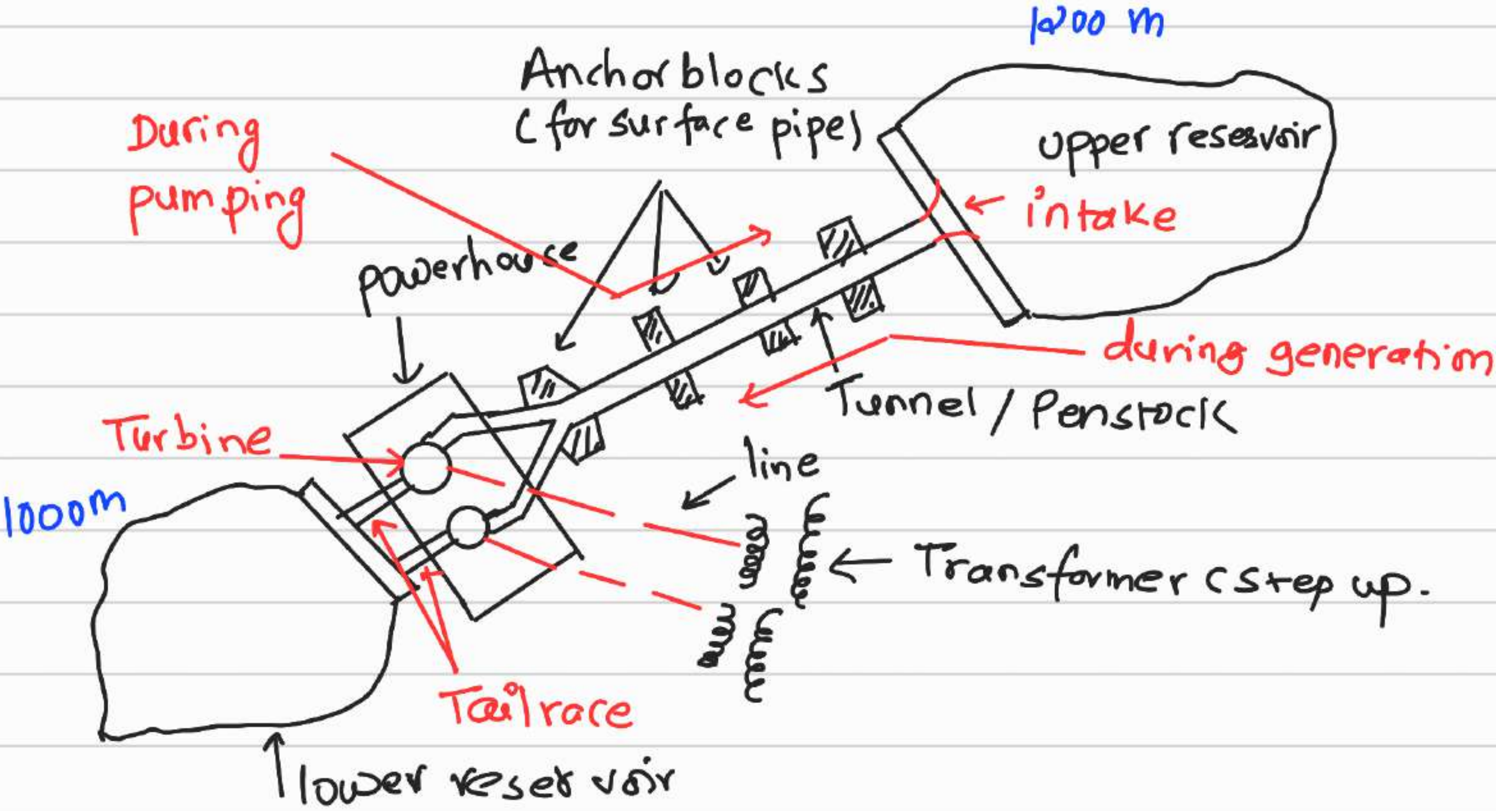


fig: section of storage project layout with powerhouse at dam toe.

Layout of Pumped Storage Projects:



Power and Energy

1. Flow Duration Curve:

* It is a curve obtained by plotting discharge in y- axis and corresponding probability of exceedance in x- axis
* It is a declining curve since, discharge decreases with increase in PoE (probability of exceedance).

Q: 10 m³/sec, 20 m³/sec, 30 m³/sec, 40 m³/sec

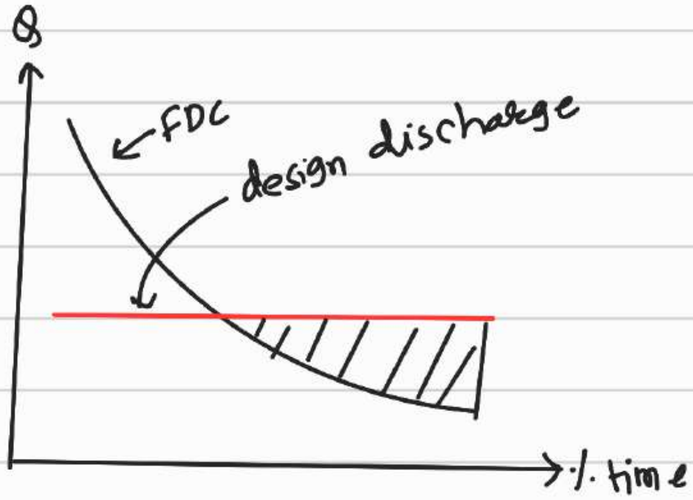
$\geq 10\,m^3/sec \rightarrow$ 4 out of 4 $\Rightarrow PoE = \frac{4}{4} * 100\% = 100\%$

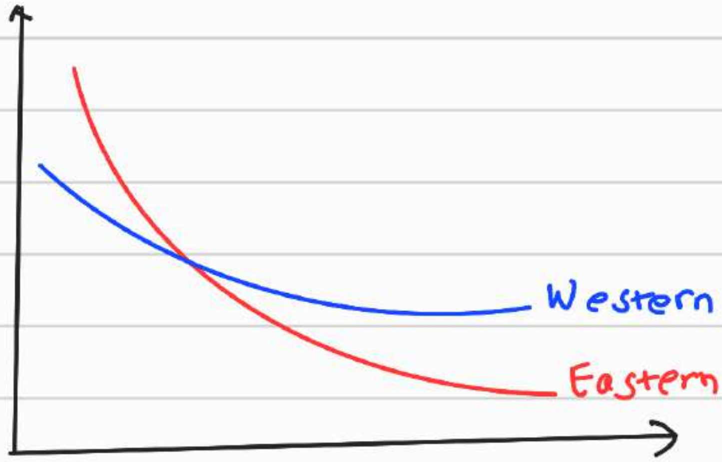
$\geq 20\,m^3/sec \rightarrow$ 3 out of 4;

$PoE = \frac{3}{4} * 100 = 75\%$

Uses of flow Duration Curve, FDC

1. To know variation pattern of discharge.
2. To asses primary and secondary power and energy.
3. To fix design discharge and installed capacity of project. Q_{25} to Q_{70}
4. To determine storage volume required.
5. To select number and type of turbine.
6. To assist the decision makers for system planning, policy formations.
7. To obtain sediment yield by integrating with sediment rating curve.





Design discharge

* The discharge at which the project is designed.
* The project can not utilize more discharge than design discharge.

Installed Capacity:

* The optimum or most economic capacity of a project by considering cost and benefit.
* It is the maximum capacity of a project.
* $P = \eta \gamma Q_{design} * H_{net}$

  η = efficiency

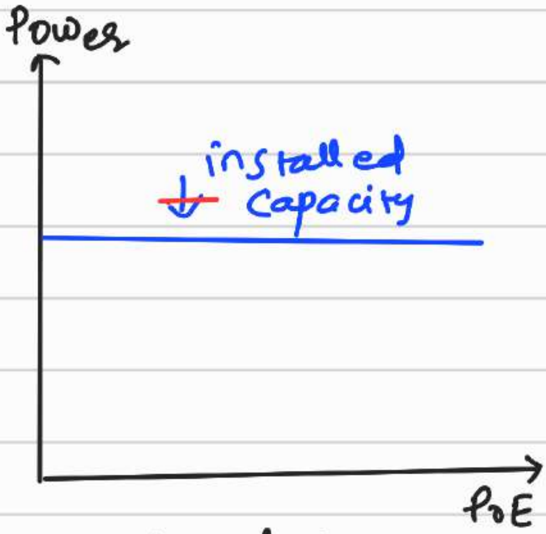
  Q_{design} = design discharge

  H_{net} = Net head

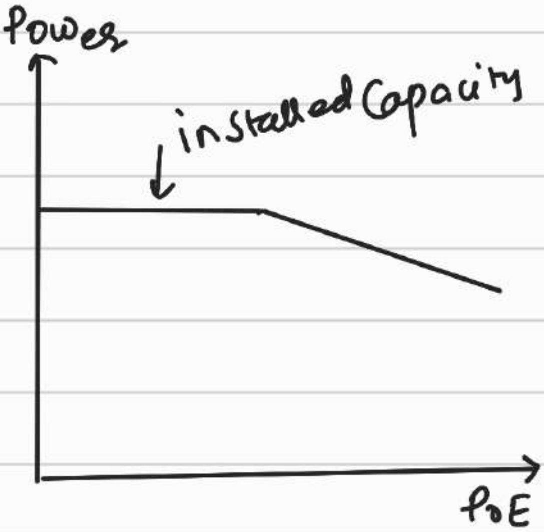
Power Duration Curve

* The plot of power generated vs probability of exceedance.
* Since, Power = $\eta \gamma Q H$, it also follows some nature as FDC.
* However, power duration curve of RoR is straight line (Horizontal) upto certain time and then declining curve. Whereas, power duration curve of a storage project may be a horizontal line or slightly declining curve.

fig: power duration curve
of RoR project

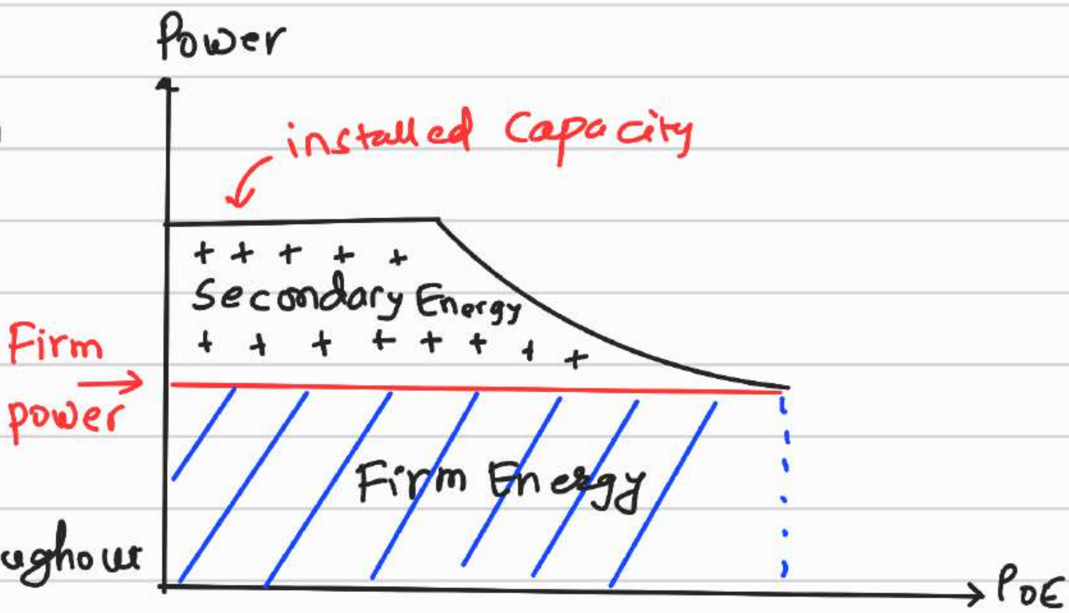


power duration
curve of storage
project



Firm Power, Firm Energy, Secondary Energy

* Firm power:
  minimum power
  generated from
  the projet
* Firm energy
  = energy
  generated
  if plant runs at
  firm power throughout
  the year.



Firm Energy = Firm power * 1 year

* Secondary Energy = Energy in excess of of firm energy = Total energy − firm energy
* Average annual energy = Annual energy generated in an average year.
  * Average year is a synthetic year in which flow is average of all year.
* Spill energy = The energy which is in excess of demand.

Seasonal Energy:

a. Dry season Energy: * Energy generated in dry season (Mansir 16 to Jestha 15 in our context). → December − May

b. Wet season Energy: * Energy generated in wet season.
  * e.g; energy generated in Jestha 16 to Monsir 15 in our context. (June − Nov)

Load Curve:

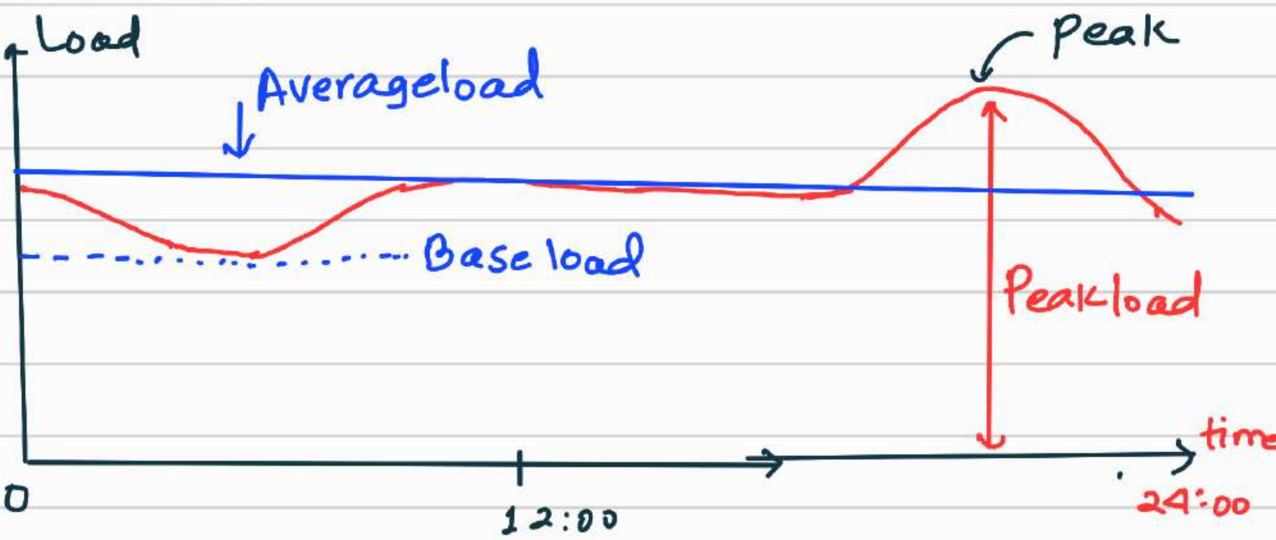


fig: daily load curve

* Load curve:− * It is the plot of load vs time.
  * load is the equipment / applionce connected to a system which demands electrical power.

* The load curve of Nepal is not a straight line due to change in demand throughout the day.

Peak load: * Maximum load on the system

Average load: * The value of load which consumes same energy as actual if it occurs uniformly in that period.

Variation of load:

1. Daily load variation:

* Variation of load within hours of a day
* Daily load curve of Nepal shows peak during evening hours.

2. Weekly Variation:

* Variation of load within days of a week.

3. Monthly load variation:

* Variation of load in months of a year.

4. Seasonal load variation:

* Variation of load within different seasons of a year.

Some terms Related with load:

1. Load factor = $\frac{\text{Average load}}{\text{peak load}}$

2. Plant / Capacity factor = $\frac{\text{average load}}{\text{installed capacity}}$

Kolchajor Storage Project:

Capacity = 63 MW

Energy = 305 GW-hr ← Annual

$$\text{Average load / Capacity} = \frac{\text{Energy}}{\text{time}} = \frac{305 * 1000}{365 * 24}$$

$$= 34.8 \text{ MW}$$

34.8 MW → constant → throughout year → 305 GW-hr

$$\text{plant factor} = \frac{34.8}{63} = 0.552$$

3. Reserve capacity = unutilized capacity
= Installed capacity − peak load

4. Utilization factor $= \dfrac{\text{Peak load}}{\text{Installed capacity}}$

Relation ⇒ plant / capacity factor = load factor * utilization factor

Q. A Hydropower plant is planned to be developed in river with following discharge. Taking net head of 100 m, overall efficiency of 85% and design discharge as Q_{40}, determine

a. Design discharge
b. Installed Capacity
c. Firm power
d. Firm Energy
e. Annual energy
f. Plant factor.

min. d/s release.

→ Assume minimum riparian release as 10% of minimum monthly flow.

month	Q	Desc. order	Rank	min d/s release	Flow for power	day	Power, MW	Energy, Gwhr	PoE $= \frac{rank}{12} * 100\%$
Jan	100	2500	1	10	90	31	75.0	55.8	$1/12 \times 100 = 8.33$
Feb	120	2100	2	10	110	28	91.7	61.6	$\frac{2}{12} * 100 = 16.67$
Mar	140	2000	3	10	130	31	108.4	80.65	$\frac{3}{12} * 100 = 25$
April	300	1800	4	10	290	30	241.8	174.1	33.33
May	320	900	5	10	310	31	258.5	192.3	41.67
June	1800	500	6	10	1082.82	30	902.9	650.1	50
July	2000	320	7	10	1082.82	31	902.9	671.8	58.33
Aug	2500	300	8	10	1082.82	31	902.9	671.8	66.67
Sept	2100	300	9	10	1082.82	30	902.9	650.1	75
Oct	900	140	10	10	890	31	742.1	552.1	83.33
Nov	500	120	11	10	490	30	408.6	294.2	91.67
Dec	300	100	12	10	290	31	241.8	179.9	100

* from above table, it means that,

$$Q_{33.33} = 1800 \ m^3/sec$$

$$Q_{41.67} = 900 \ m^3/sec$$

$\therefore$ Design discharge: $Q_{40} =$

6.67   1.67

33.33   40   41.67

1800   900

8.34

$$Q_{40} = \frac{6.67 * 900 + 1.67 * 1800}{6.67 + 1.67} = \frac{m_1 x_2 + m_2 x_1}{m_1 + m_2}$$

$$= 1082.8 \ m^3/sec.$$

a. Design discharge $= Q_{40} = 1082.8\ m^3/sec$

b. Installed capacity $= P = \eta \gamma Q H$; where; Q = design discharge

$$\text{or } P = \frac{0.85 * 9.81 * 1082.8 * 100}{1000}\ MW$$

$$= 902.89 \simeq 902.9\ MW$$

minimum downstream release = 10% of minimum monthly flow

$= 10\%$ of $100 = 10\ m^3/sec$

flow for power = $\boxed{\text{flow in month} - \text{min dis release subjected to maximum of design discharge} \Rightarrow \text{No more than design discharge}}$

$$\text{monthly power} = \frac{\eta * \gamma * Q * H}{1000}\ MW$$

where Q = monthly flow for power.

$$\text{monthly energy} = \frac{\text{monthly power (MW)} * \text{no of days} * 24}{1000}\ GW\text{-}hr$$

e.g; Energy of January $= \dfrac{75 * 31 * 24}{1000}\ GW\text{-}hr$

$= 55.8\ GW\text{-}hr$

c. Firm power = minimum power = 75 MW

d. Firm energy = firm power * 1 year

$= 75\ MW * 1$ year

$= \dfrac{75}{1000} * 365 * 24\ GW\text{-}hr$

$= 657\ GW\text{-}hr$

e. Annual energy = sum of energy of all months

$= 4234.5\ GW\text{-}hr$

secondary energy = Annual energy − Firm energy

$= 4234.5 - 657$

$= 3577.5$ Gw-hr

f. Plant factor/capacity factor

$$\text{plant factor} = \frac{\text{average load}}{\text{installed capacity}}$$

* $$\text{Average load} = \frac{\text{Annual energy}}{1\text{ year}}$$

$$= \frac{4234.5}{365 * 24} * 1000 \quad \text{MW}$$

$$= 483.4 \text{ MW}$$

* $$\text{plant factor} = \frac{483.4}{902.9} = 0.535$$

Reservoir Sedimentation:

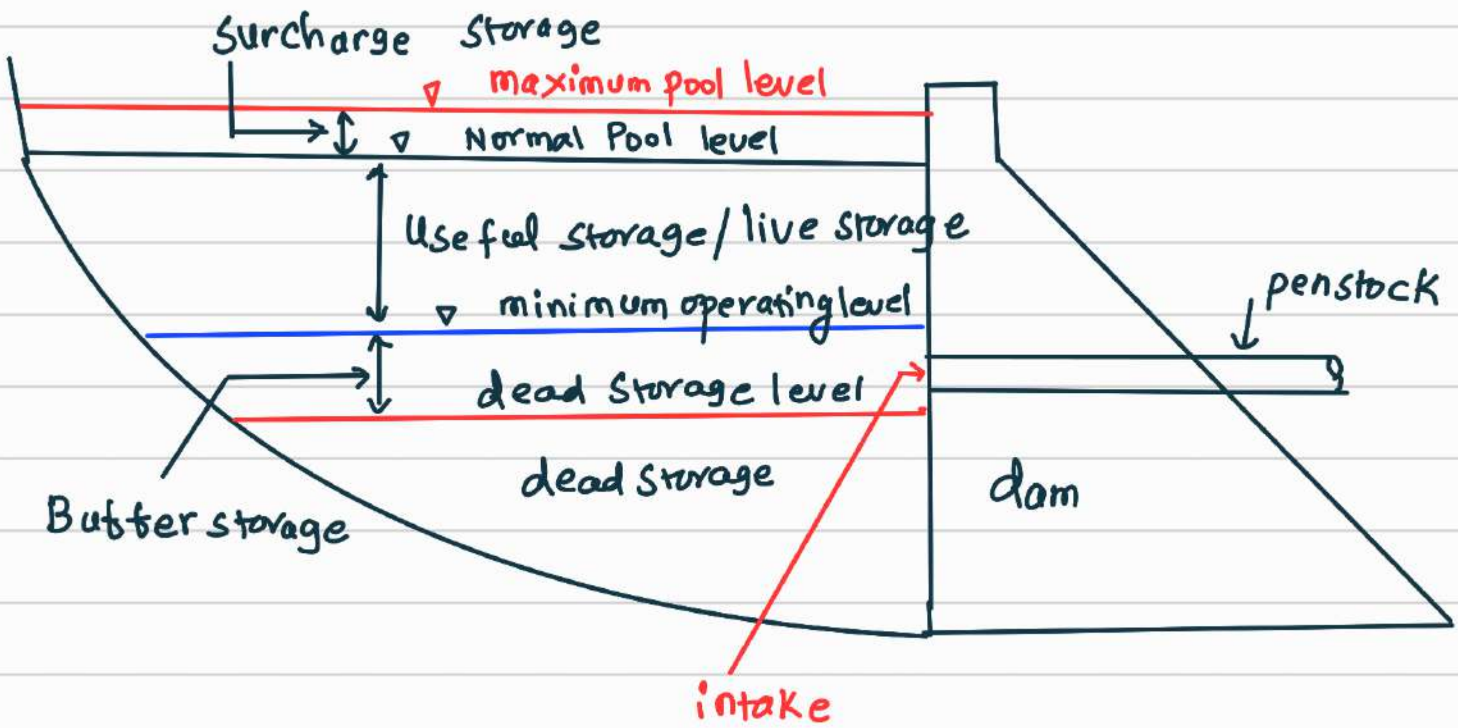


fig: Reservoir with different storage zones

* When sediments approach a reservoir, due to increase in area of flow in reservoir, velocity of water gets reduced.
* Due to decrease in velocity, sediments are settled in reservoir. large size particles are settled at upper reach of reservoir and fine particles travel downstream and some may even enter the intake and reach turbine.
* To accomodate deposition of sediments in reservoir, dead storage is allocated. Generally dead storage volume is equal to volume of sediments deposited upto 50 years in Nepal.
* However, sediments encroach the live storage since the first year and reduce storage capacity of reservoir.

Annual sediment yield = 42.5 mcm
Volume upto 50 yr = 22.5 * 50 = 2125 mcm

mcm = million cubic meter

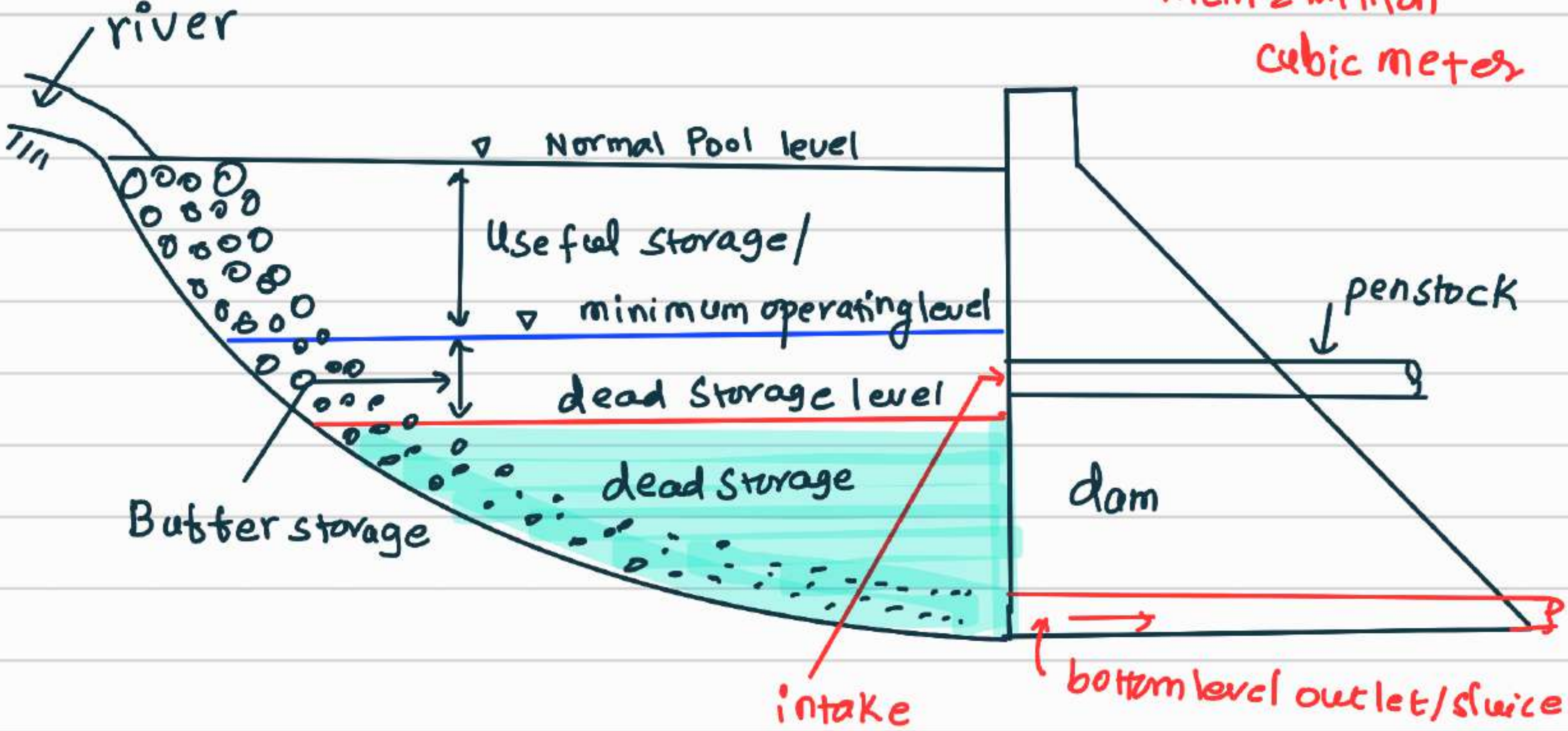


Fig: Reservoir sedimentation

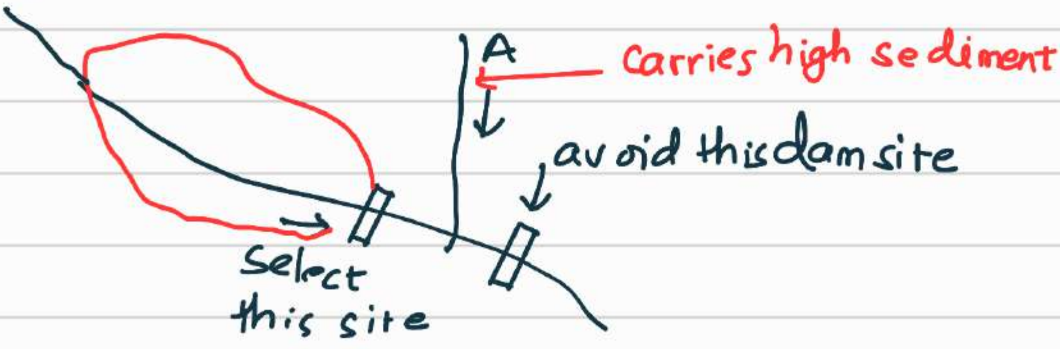
* When turbid flow during flood time enters the reservoir, it has high desity due to sediments but clear reservoir water has low density. So, the turbid water with high density flows through bottom of reservoir, which is called density current.
* If this density current is allowed to leave the reservoir through low level outlets, it is called density current venting.

Useful life of Reservoir: The time upto which reservoir fullfills it's design commitments.

Economical life of Reservoir: The time upto which the benefit from reservoir operation equals cost of running it.

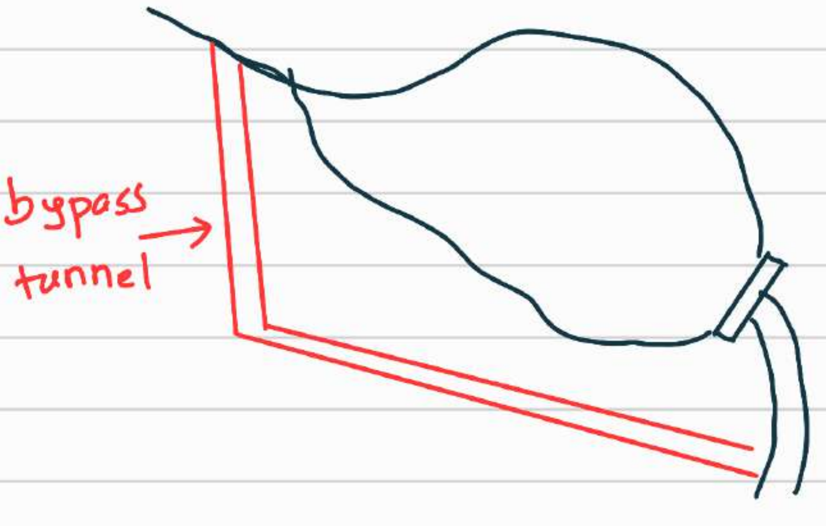
Control of Sedimentation in Reservoir:

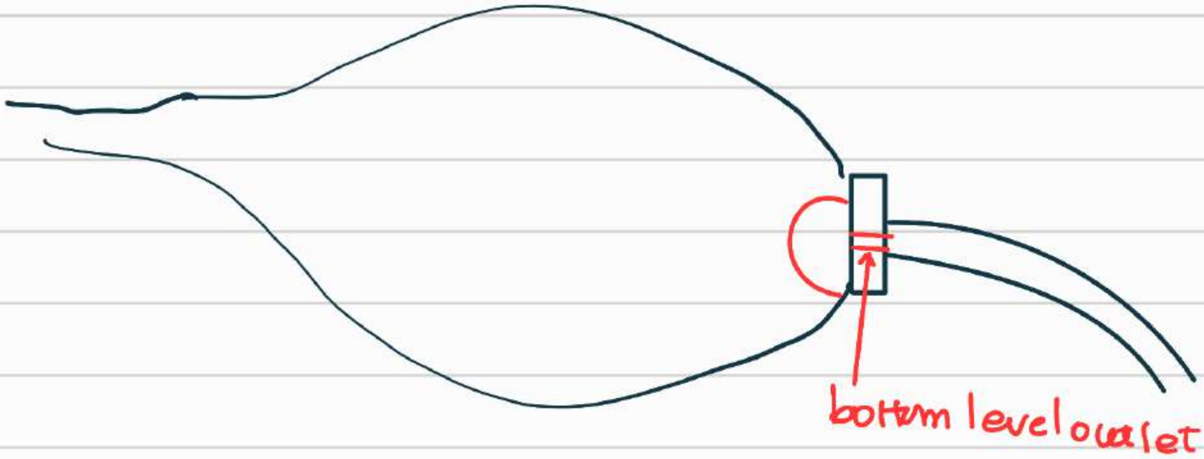
1. Exclude the tributaries having high sediments from entering reservoir. Select dam site accordingly

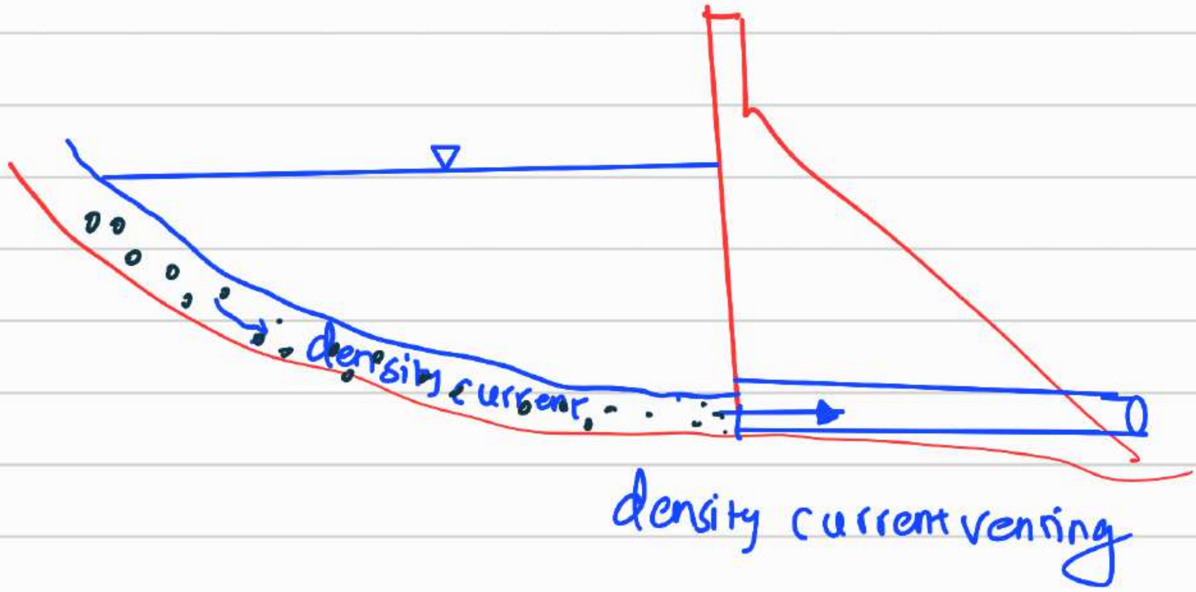


2. Construct check dams on rivers on upstream of reservoir.
3. Adopt watershed conservation practices like vegetation cover, terracing, gully control.
4. Provide bypass tunnels to remove high sediment flow before entering reservoir.
5. Adopt density current venting.
6. Construction of dam in stages.
7. Provide bottom level outlets
8. Removal of sediments mechanically / manually

a → avoid sediments - 1, 2, 3, 4
b → let the sediment pass - 5
c → remove sediments. - 7, 8



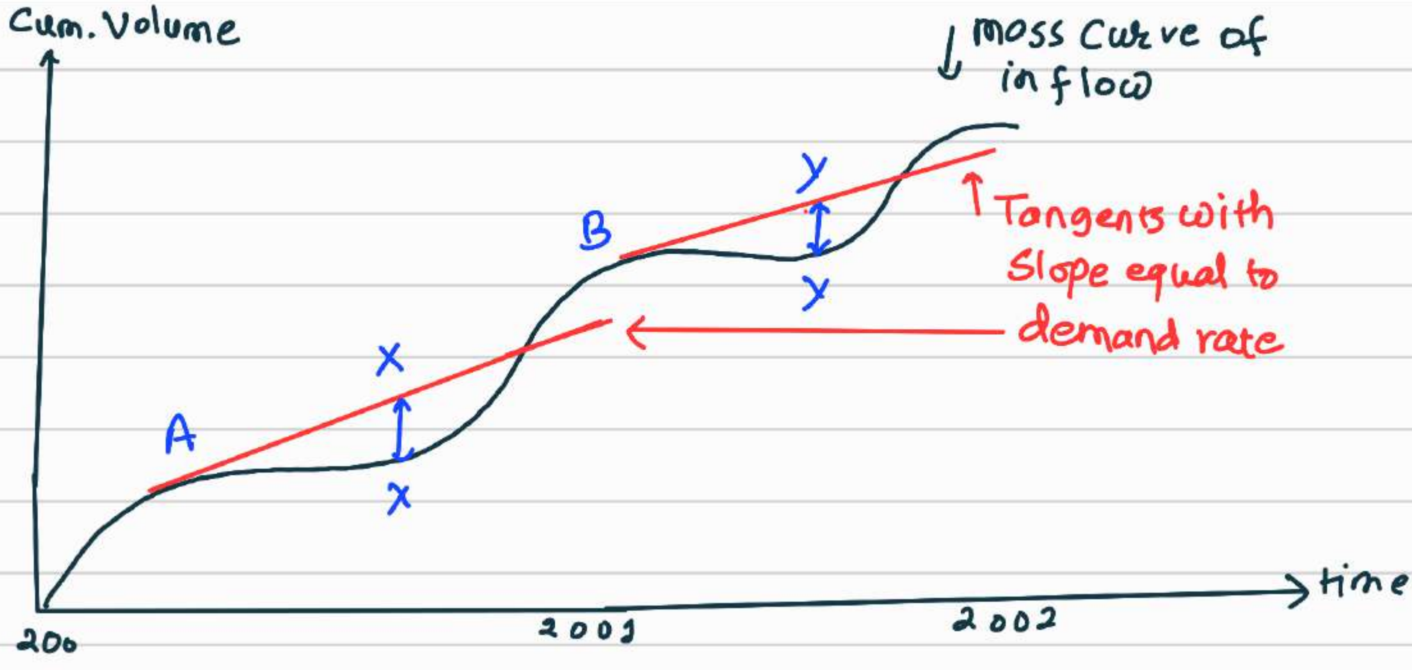




Estimation of Reservoir Capacity / Storage Capacity

* Reservoir capacity may be estimated by:

a. From mass curve of inflow and outflow

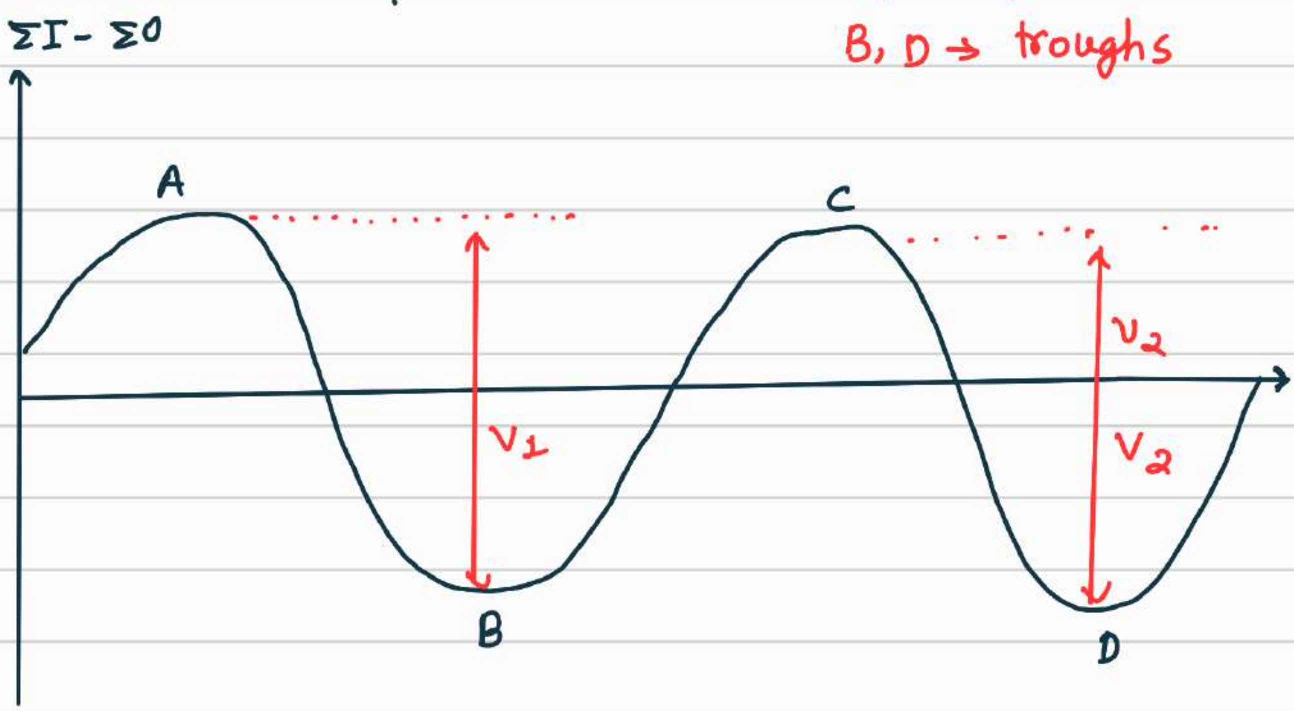


* mass curve of inflow is plotted. Preferably plot mass-curve of several year.
* plot mass-curve of demand. mass curve of demand may be straight line or curve based on demand rate. For simplicity constant demand rate may be assumed.
* If demand is constant, draw tangents at peak point with slope equal to demand rate.
* Required storage capacity is maximum of difference between tangents and inflow mass-curve, i.e, maximum of xx, yy, etc.

Note: The gap is deficit and storage capacity is maximum cumulative deficit.

b. Sequent Peak method:

A, C → peaks
B, D → troughs



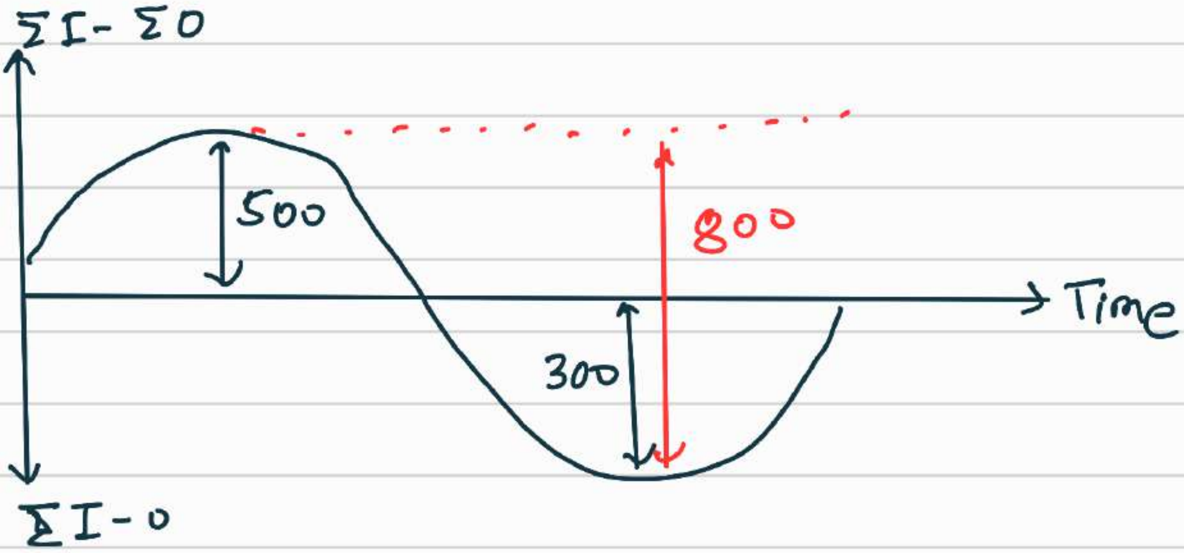
* $\Sigma I - \Sigma O$ is plotted with time
* +ve value ploted above x-axis, -ve value ploted below x-axis.
* The gap betn peaks and subsequent troughs noted.

* maximum gap (i.e; maximum of V_1, V_2, V_3) is Storage capacity.

Q. Calculate Reservoir Capacity with following data.

month	inflow mcm	Demand mcm	ΣInflow	ΣOutflow	ΣI − ΣO	
June	250	150	250	150	100	Remarks:
July	350	150	600	300	300	Peak is
Aug	400	200	1000	500	$\boxed{500}$	500 +ve
Sept	200	250	1200	750	450	and trough
Oct	150	350	1350	1100	25	is 300
Nov	150	400	1500	1500	0	negative.
Dec	100	250	1600	1750	−150	So, capacity
Jan	50	200	1650	1950	−300	= peak
Feb	150	150	1800	2100	$\boxed{-300}$	to trough
Mar	300	150	2100	2250	−150	= 800 mcm
Apr	400	100	2500	2350	150	
May	450	250	2950	2600	350	
June	150	350	3100	2950	150	
July	200	300	3300	3250	50	
Aug	450	100	3750	3350	400	

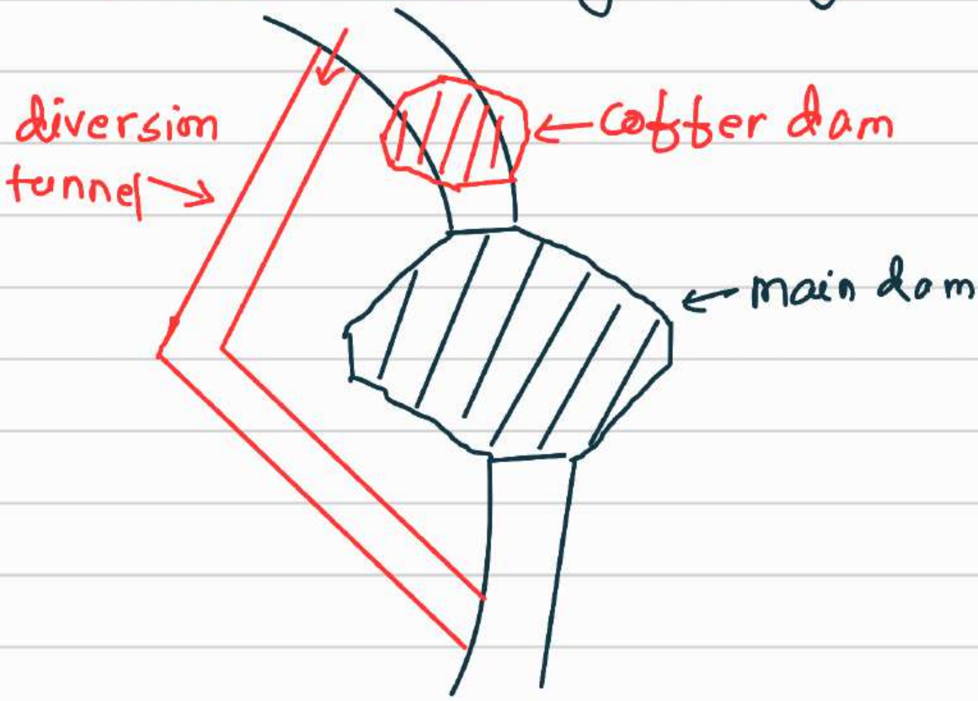
∴ Storage Capacity = 500 − (−300)
= 800 mcm



Chapter 3: Dams:

Dam: It is a barrier constructed across a river for storage of water in a reservoir behind it.

Purpose:
1. To store water
2. To create head for hydropower generation
3. Flood control
4. To make the working area dry.



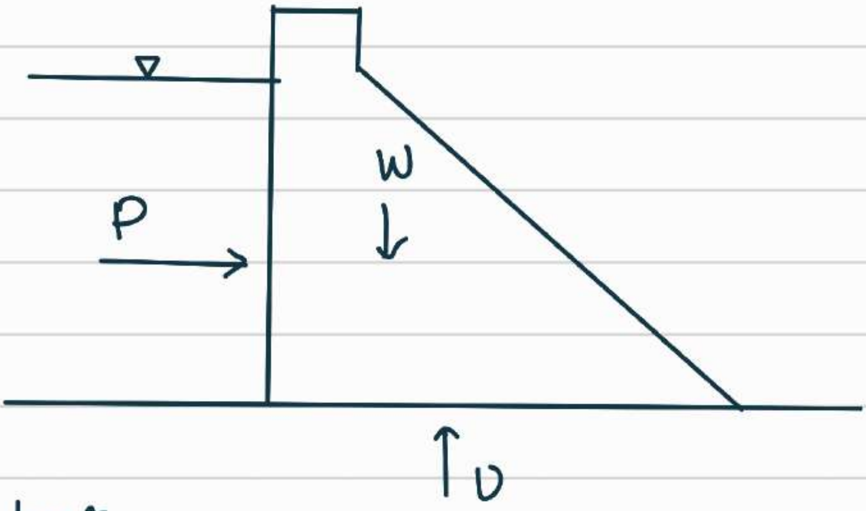
Dam classification:

1. Based on Function:
   - a. Storage dam: for storage of water
   - b. Detention dam: For flood control purpose
   - c. Coffer dam: To keep the working area dry.

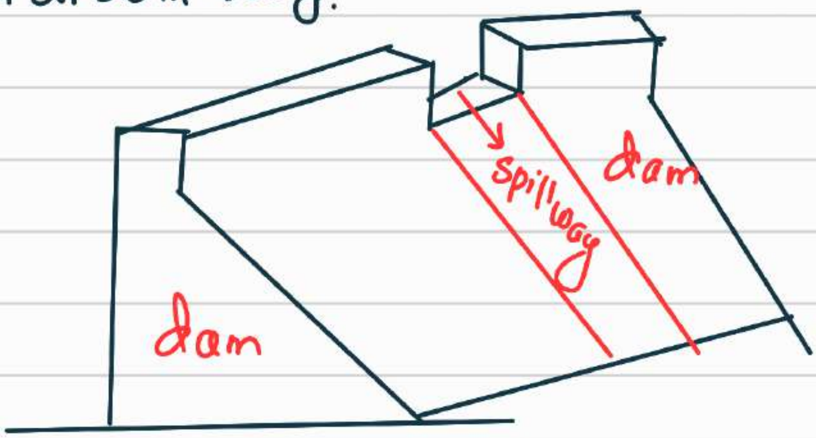
2. Based on Material:
   - a. Concrete dam: main material is concrete
   - b. Earthen dams: material is clay, sand, gravel, etc.
   - c. Rock fill dams: material is rock, clay, concrete
   - d. masonry dams: Built of stone masonry

3. Based on Load Transfer mechanism:

a. Gravity dam



* The dam attains stability due to self weight
* material: concrete
* Requires good foundation/ rocky foundation
* Initial cost is high but O&m cost is low.
* spillway is provided on dam body.



b. Arch Dam:

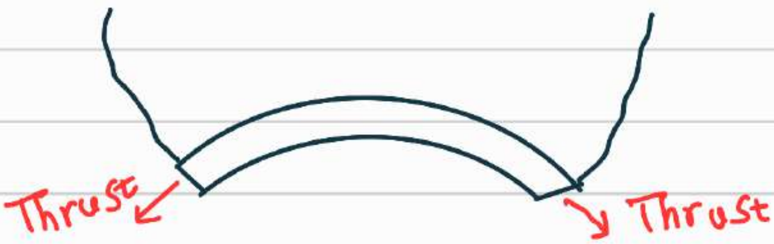
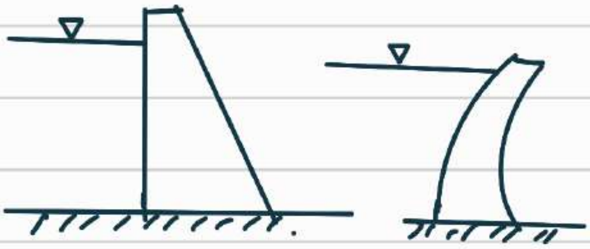


fig: plan of arch dam



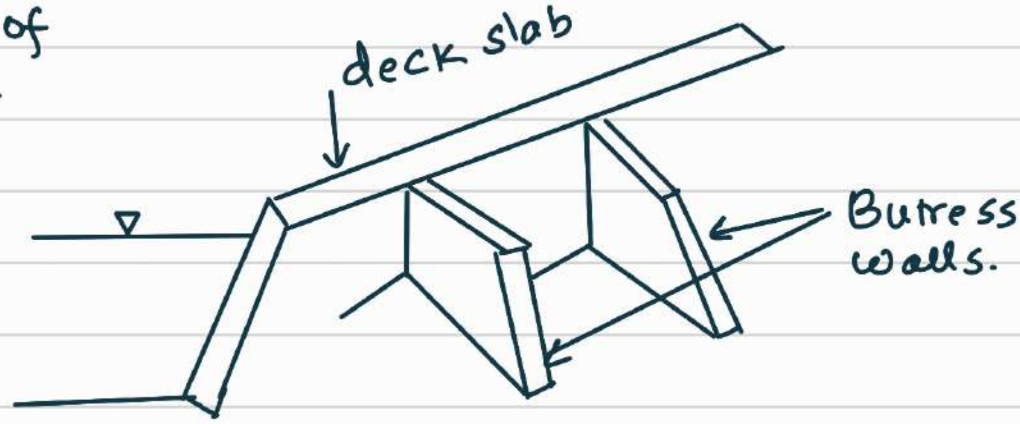
Section of arch dam

* curved in plan.
* Due to it's geometry, the water pressure or thrust on dam is transferred to abutment
* It requires strong rock i's foundation as well as abutment.

* Suitable for narrow gorges.
* Requires less concrete work as compared to grovity dam.

3. Butress Dam:

* It consists of main slab or deck slab and number of butress walls behind the deck slab.



* Water pressure on deck slab is transferred to butress walls.
* Can be constructed in dam sites with weak foundation.

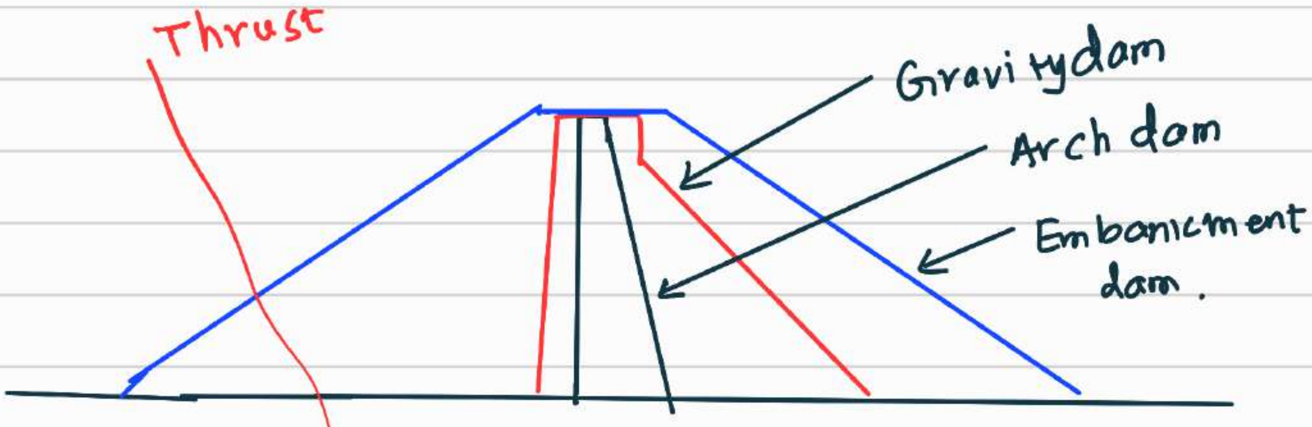
4. Based on Rigidity:
   a. Rigid dam: * Concrete doms, mosonry doms.
      * spillway can be constructed on dam body.
   b. Non-Rigid dam:-
      * earthen doms, rock fill dams.

Selection of dom type:

a. Construction material:
b. Foundation condition:

Embankment dom
- Rock fill
  - Clay core
  - Concrete face
- Earthen dam

C. Geological condition:



d. Topography including width:

flat river: embankment dam.

narrow valley: concrete dam

narrow gorge:- Arch dam

Foundation condition:

Embankment dam: any type of foundation.

Rocky foundation: Gravity dam

Rocky bed and rocky abutment: arch dam.

e. <u>spillway</u>:

* Good site/space available for spillway - embankment dam.

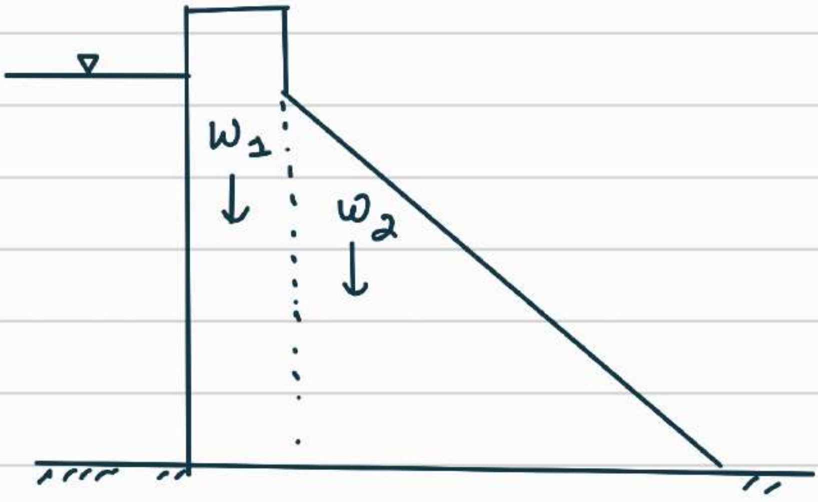
* Good site/space not available for spillway - Concrete dam.

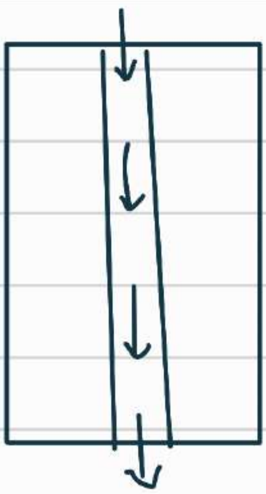
f. Siesmic condition:

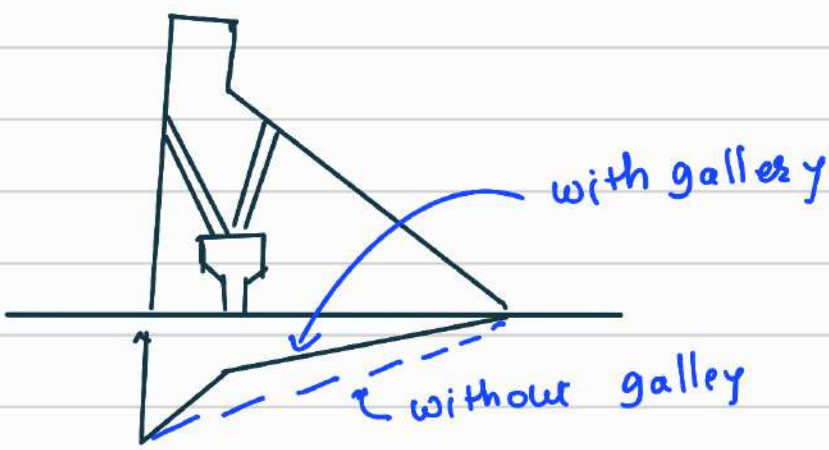
g. Overall cost:

Stability Analysis of Gravity Dam:

Forces acting on dam:







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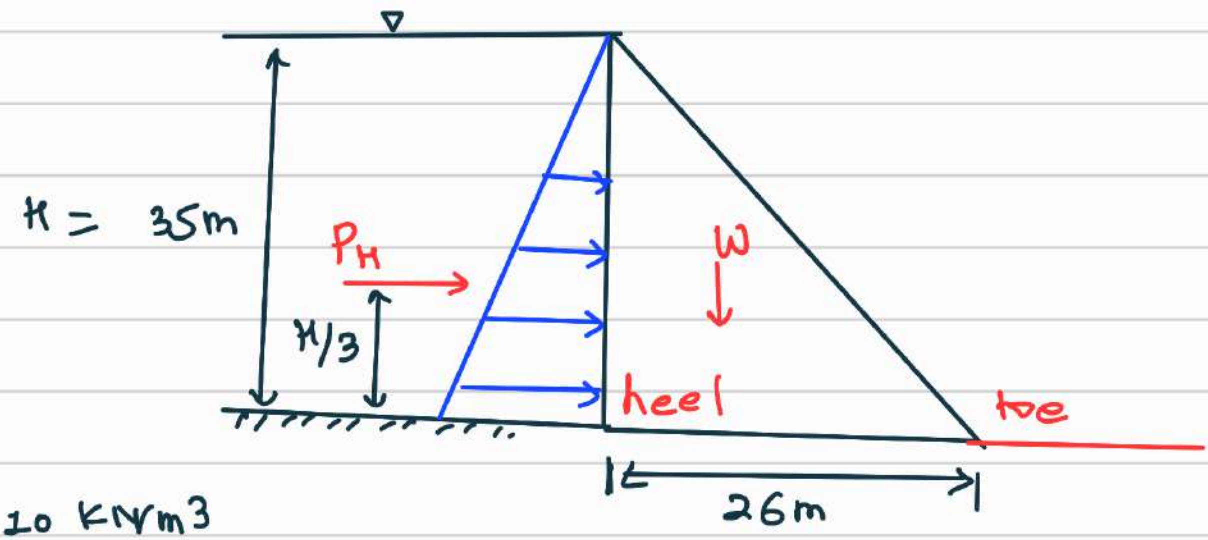
Q. Water stands on the upstream side of the gravity dam of triangular section upto the height of 35 m. The base width of the dam is 26 m. The uplift pressure intensity, K may be assumed to be 0.5. Show that

1. No tension exists anywhere along the base of dam.
2. The dam is safe against sliding
3. The maximum compressive stress in the body of dam is less than the allowable crushing stress of the material 11 kgf/cm².
4. The dam is safe against overturning.

Note: take the coeff. of friction betn base and foundation as 0.75 and unit wt. of material of the dam as 2400 kgf/m3

Soln:

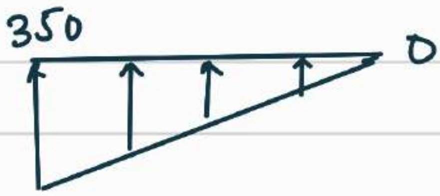


Take: $\gamma_w = 10\ kN/m^3$

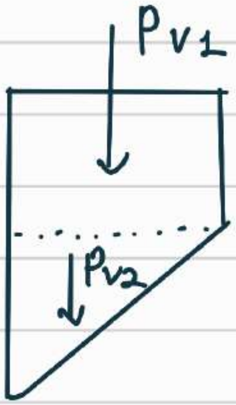
$\gamma_c = 2400\ kgf/m^3$

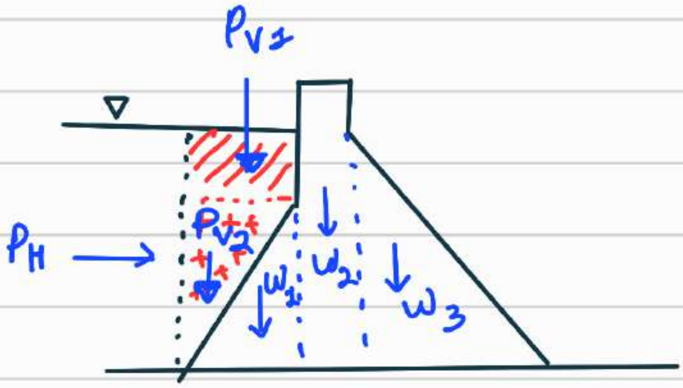
$= 2400 * 10\ N/m^3$

$= 24000\ N/m^3 = 24\ kN/m^3$

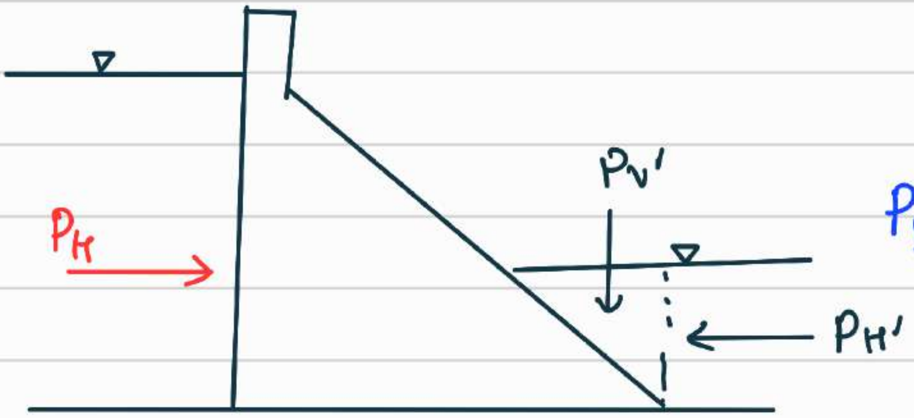


S.N.	Force Description	magnitude (KN)	lever arm	moment	
				Resisting	Overturning
1.	Hydrostatic force $P_H = \frac{1}{2} * \gamma_w * H^2$ $= \frac{1}{2} * 10 * 35^2$	6125 →	$35/3$ $= 11.67$		71478.75
2.	Self wt. of dam $= \gamma_c * \text{volume}$ $= \gamma_c * \text{Area} * \text{length}$ $= \gamma_c * \text{Area} * 1$ (length = 1m) $= 24 * \frac{1}{2} * 26 * 35$	10920	$\frac{2}{3} * 26$ $= 17.33$	189280	
3.	Uplift pressure $= \left[\frac{1}{2} * 350 * 26\right] * 0.5$ Pressure intensity factor 0.5 given	2275	$\frac{2}{3} * 26$ $= 17.33$		39425.75

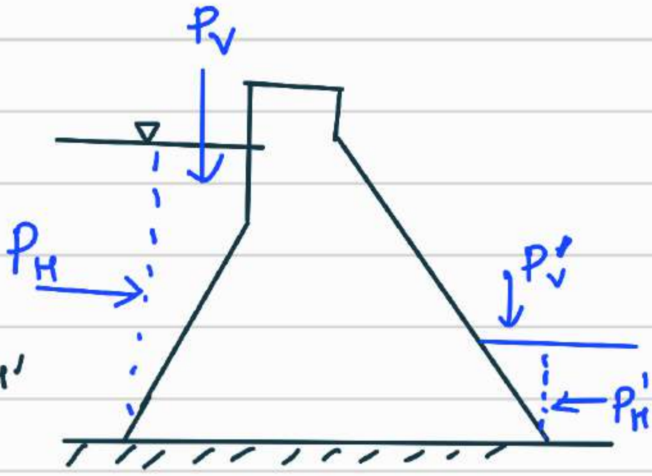




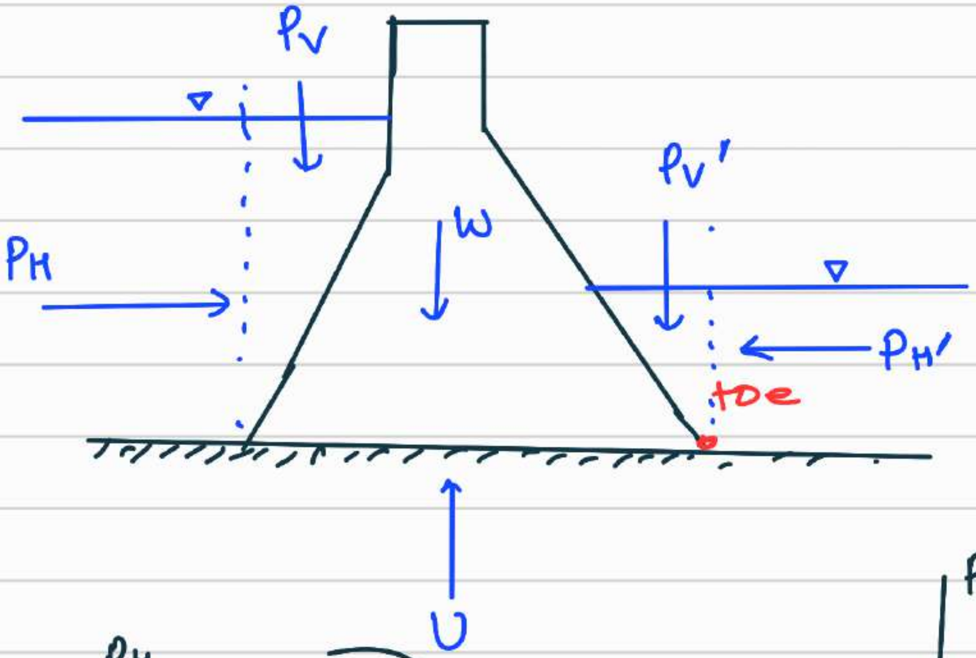
For uplift pressure: pressure intensity at heel
$= \gamma_w * 35 = 10 * 350 = 350 \text{ kN/m}^2$
pressure intensity at toe $= 0$ (No water)



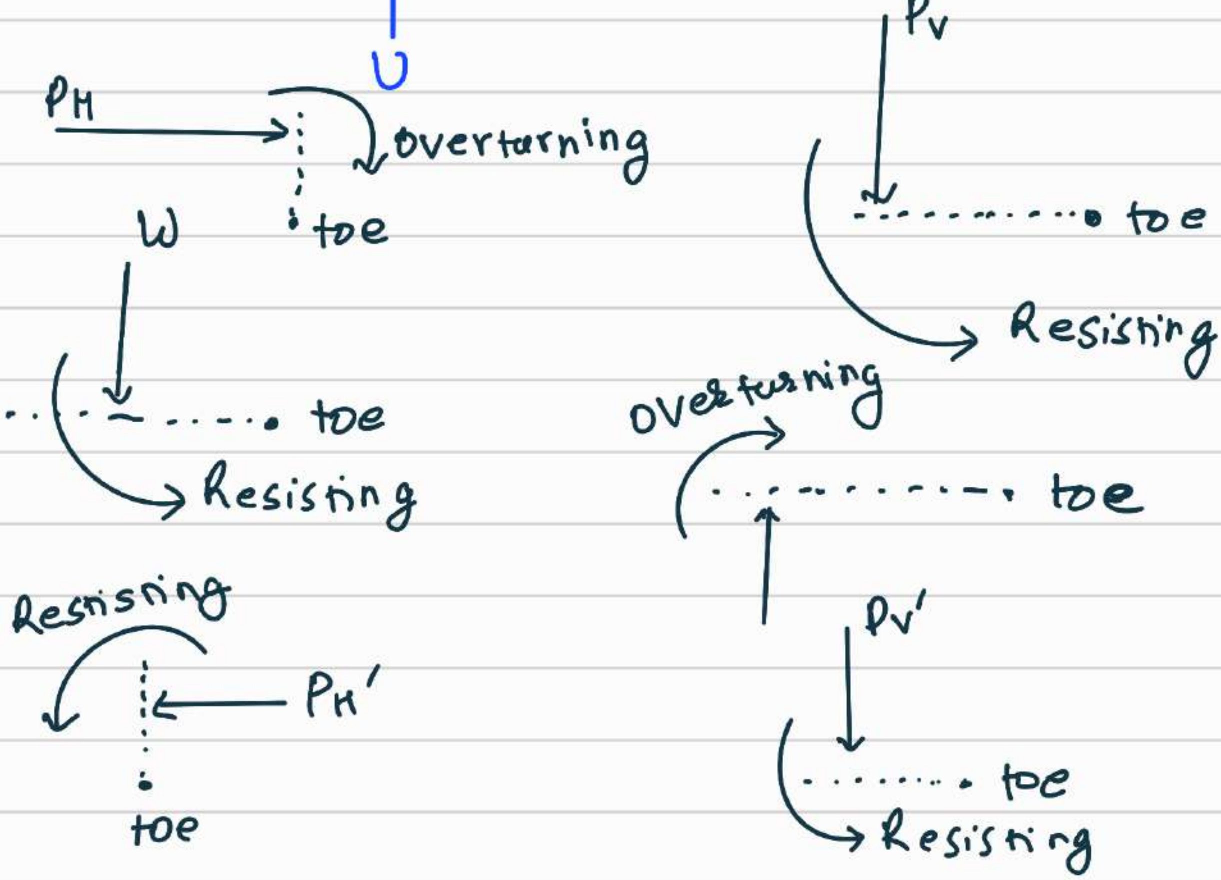
P_H
P_V'
P_H'



P_V
P_H
P_V'
P_H'



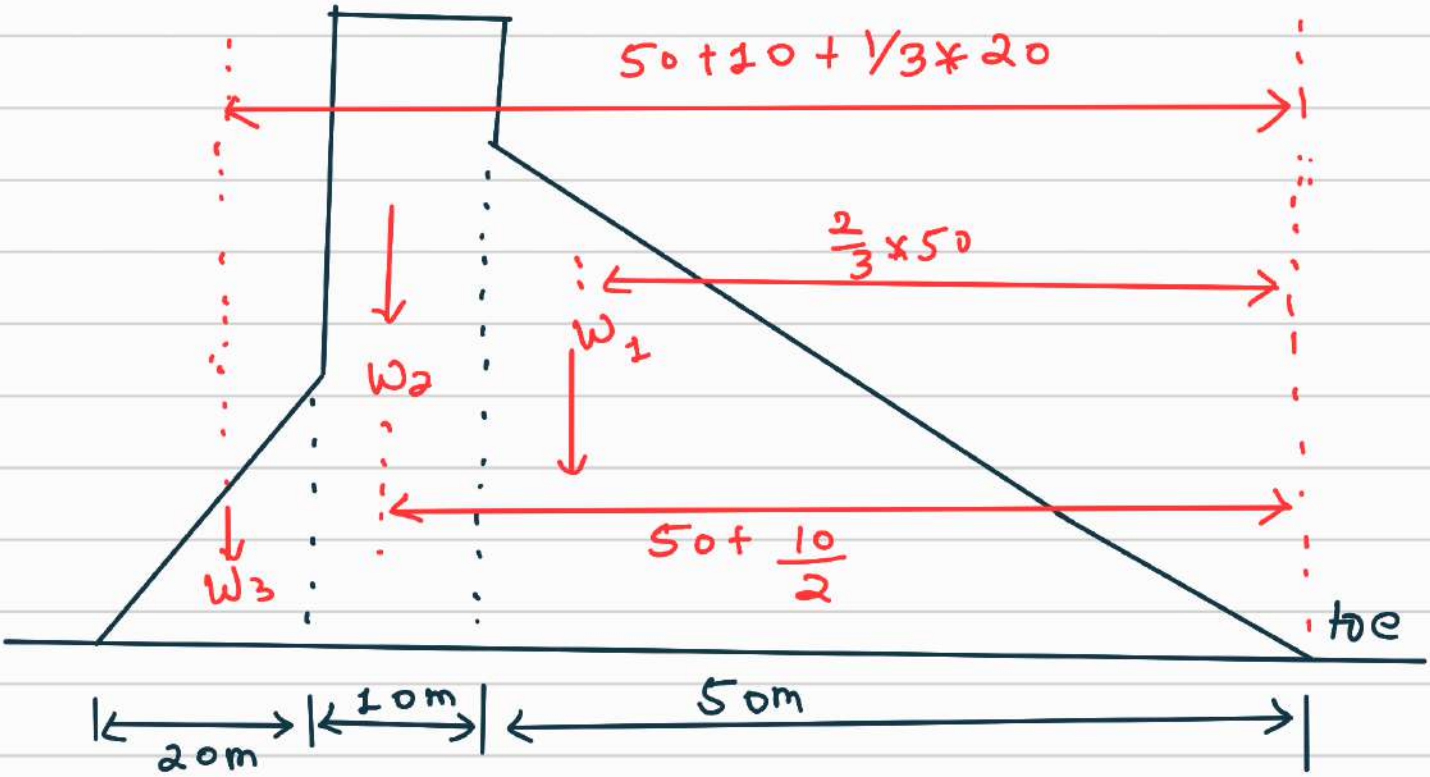
P_V
P_H
W
P_V'
P_H'
toe
U



P_H
overturning
toe
W
toe
Resisting
Resisting
P_H'
toe
P_V
toe
Resisting
Overturning
toe
P_V'
toe
Resisting

W, vertical water pressure (u/s & d/s), d/s horizontal water pressure → Resisting

upstream horizontal water pressure, uplift → overturning



listing of forces:

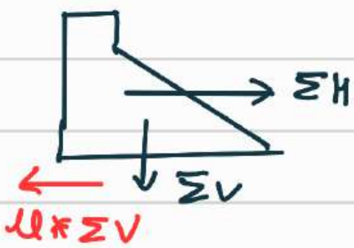
$\Sigma V = \quad W - U \quad = 10920 - 2275 = 8645$ KN ↓

$\Sigma H (\rightarrow) \; = P_H = 6125$ KN →

$\Sigma M_R =$ Resisting moments $= 189280.0$ KNm

$\Sigma M_O = \; 110904.5$ KNm

a. Safety in sliding



Factor of Safety $= \dfrac{\mu * \Sigma V}{\Sigma H}$

$$= \frac{0.75 * 8645}{6125} = 1.06 > 1 \Rightarrow \text{Safe.}$$

b. Safety in overturning:

Factor of safety $= \frac{\Sigma M_R}{\Sigma M_O} = \frac{189280.0}{110904.5} = 1.71$

$> 1.5 \Rightarrow$ Safe

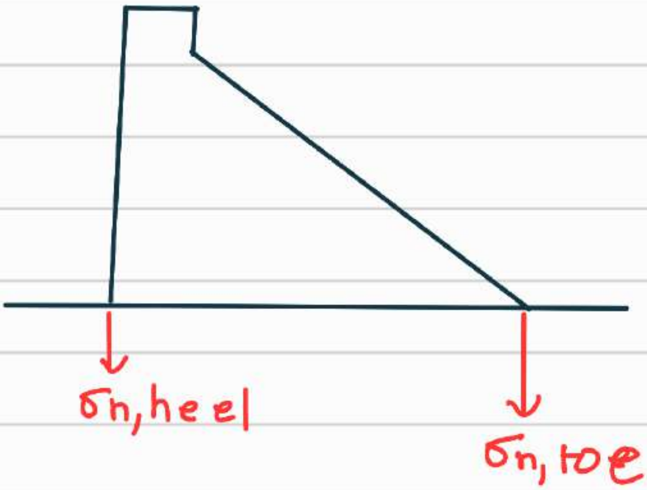
bonus

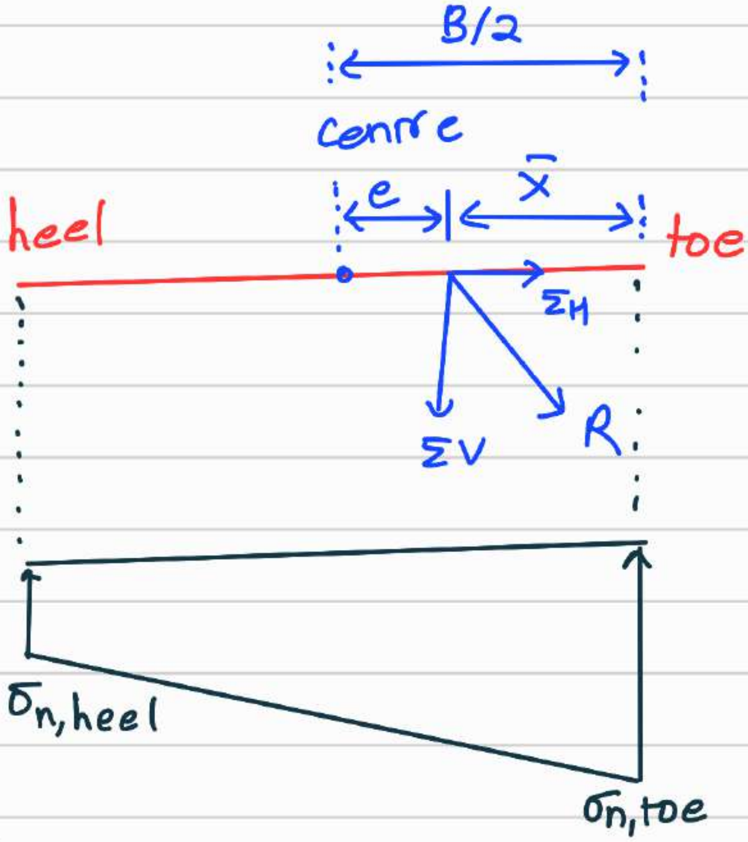
C. Safety in floating:

if vertical downward force > upward force ⇒ no floating.

⇒ Here, no floating.

d. Safety in tension:





* There will be tension in dam at heel if:

$$\boxed{e > B/6 \quad \text{or} \quad \sigma_{n,heel} = \text{normal compressive stress at heel} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right] < 0}$$

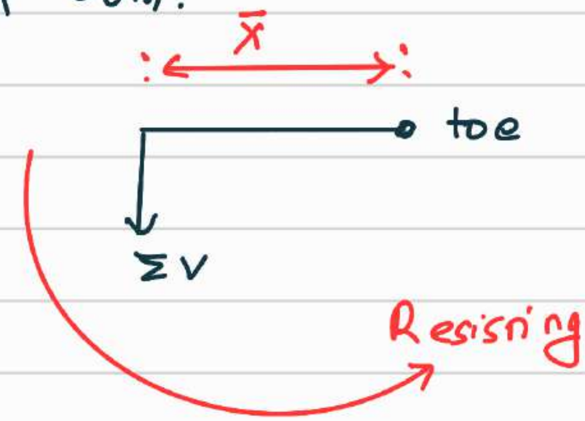
from fig: eccentricity, $e = B/2 - \bar{x}$

where; B = base width of dam.

Also: $M_{net} = \Sigma V * \bar{x}$

where M_{net} = Net resisting moment at toe

$= \Sigma M_R - \Sigma M_O$



$\therefore \bar{x} = \dfrac{M_{net}}{\Sigma V}$

$= \dfrac{189280 - 110904.5}{8645} = 9.06\,m$

$\therefore$ eccentricity, $e = B/2 - \bar{x}$

$= \dfrac{26}{2} - 9.06$

$= 13 - 9.06 = 3.94\,m$

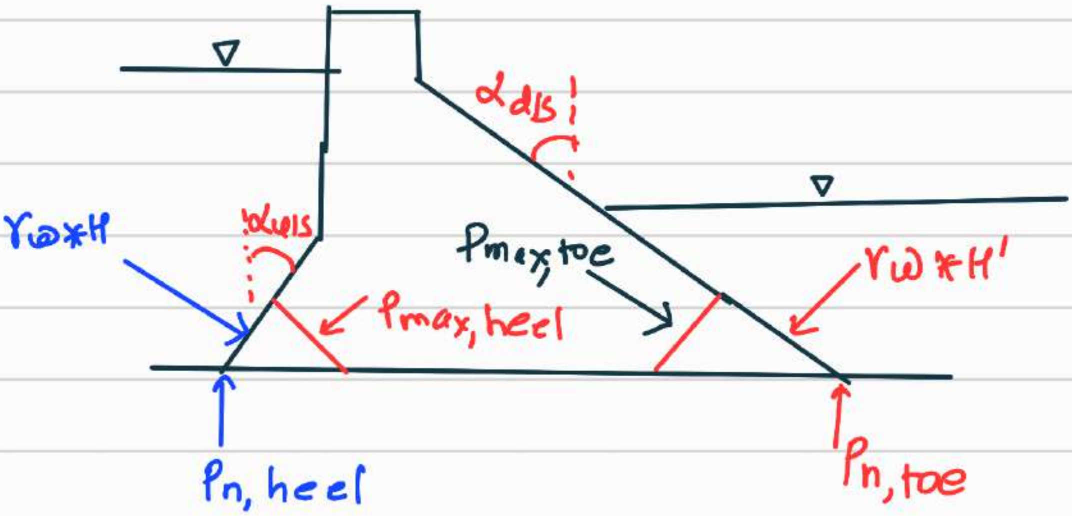
Now; $\dfrac{B}{6} = \dfrac{26}{6} = 4.33$

Here; $e < B/6 \Rightarrow$ No tension at heel

(For no tension; $e \leq B/6$, For tension; $e > B/6$)

e. Safety in crushing:

For dam to be safe in crushing,
principal stress < permissible crushing stress



Since; maximum compression occurs at toe, we check for principal stress at toe

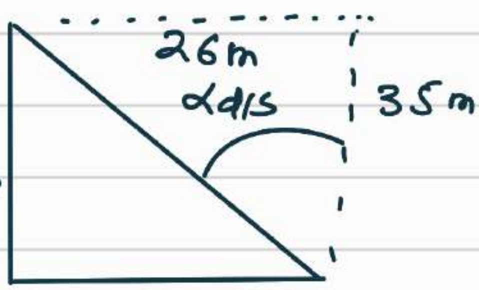
$p_{max,toe}$ = principal stress at toe

$$= \sigma_{n,toe} * \sec^2 \alpha_{d/s} - \gamma_\omega H' * \tan^2 \alpha_{d/s}$$

Since, no water at toe,

$H' = 0.$

So; $p_{max,toe} = \sigma_{n,toe} * \sec^2 \alpha_{d/s}$



$$\sigma_{n,toe} = \frac{\Sigma V}{B}\left[1 + \frac{6e}{B}\right]$$

$$= \frac{8645}{26}\left[1 + 6 * \frac{3.94}{26}\right] = 634.82 \text{ KN/m}^2$$

from fig; $\tan \alpha_{d/s} = \frac{26}{35}$

$$\tan^2 \alpha_{d/s} = \frac{26^2}{35^2} = 0.55$$

$$\Rightarrow \sec^2 \alpha_{d/s} = 1 + \tan^2 \alpha_{d/s}$$

$$= 1 + 0.55 = 1.55$$

$$\therefore p_{max,toe} = 634.82 * 1.55$$

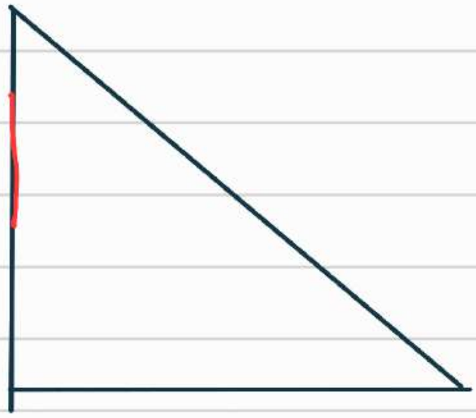
$$= 983.97 \text{ KN/m}^2$$

Principal stress at heel,

$P_{max, heel} = \sigma_{n,heel} * \sec^2 \alpha_{u/s} - \gamma_w * H * \tan^2 \alpha_{u/s}$

Here, $\alpha_{u/s} = 0$, $\tan \alpha_{u/s} = 0$;

$\sec \alpha_{u/s} = 1$



$\sigma_{n,heel} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right]$

$= \frac{8645}{26}\left[1 - 6 * \frac{3.94}{26}\right]$

$= 30.18 \; kN/m^2$

$\therefore P_{max, heel} = 30.18 * 1$

$= 30.18 \; kN/m^2$

Permissible crushing stress $= 11 \; kgf/cm^2$

$= \frac{11 * 10 \; N}{10^{-4} m^2} = 11 * 10^5 \; N/m^2$

$= 11 * 10^2 \; kN/m^2$

$= 1100 \; kN/m^2$

which is more than $983.45 \; kN/m^2$, so, safe in crushing.

Rockfill Dams:

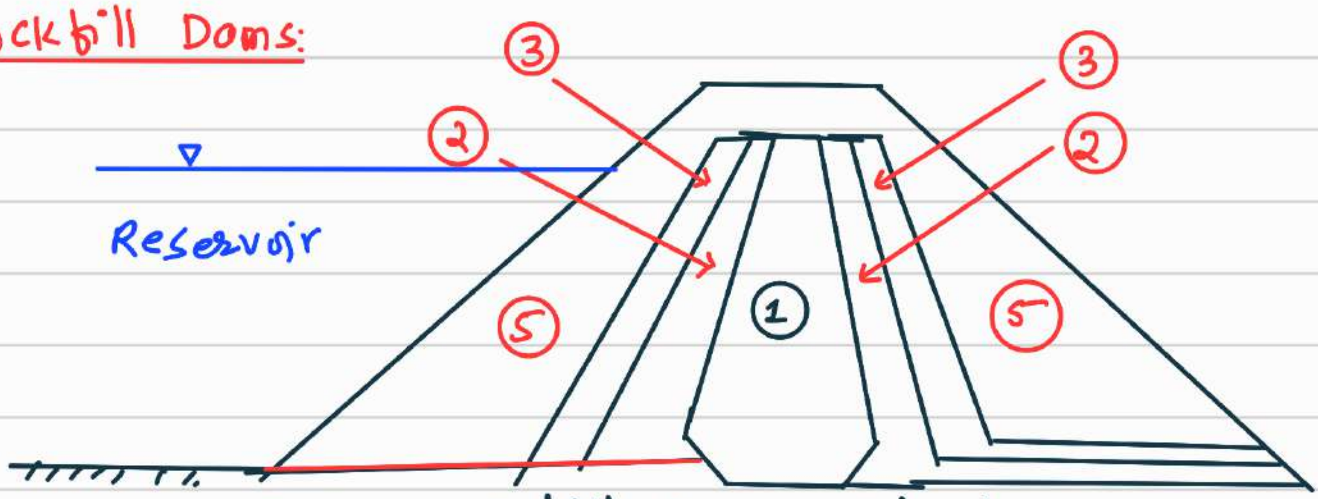


Fig: rockfill dam with clay core

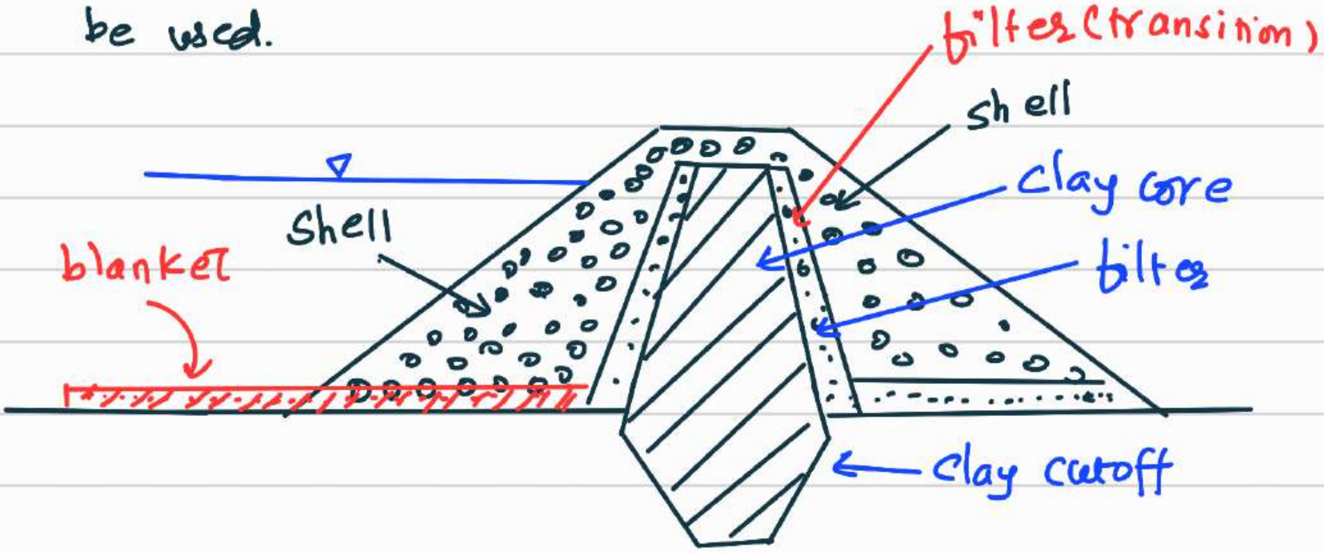
* The dam relies on rocks (pieces of rock) dumped in thin layers for it's stability.
* Impervious layer either of clay core or concrete facing is provided for seepage control.
* It lies between earthen dams and concrete dams.

1 = Impervious earth fill (clay core)
2 Filter zone
3 Well graded compacted rock
5 Best quality higher strength rock
4 Smaller sized rock: (only in CFRD)

* The side slopes of dam lie between 1.3H:1V to 2.0 H:1V.
* The foundation requirement is better than earthen dam and not as good as concrete dam.

Earthen dam

* They are flexible dams constructed from locally available materials such as clay, gravel, sand, boulders and sometimes concrete may be used.

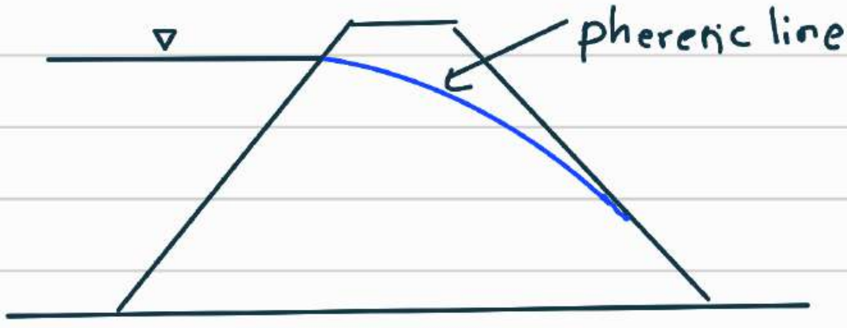


* Finer material like clay is used for seepage control and coarser material (gravel, boulder, sand) is used for structural stability, which is called shell.

Types of earthen dam: Based on Section

a. Homogenous dams:

* The dam section consists of same material throught.
* Not suitable for large dams.

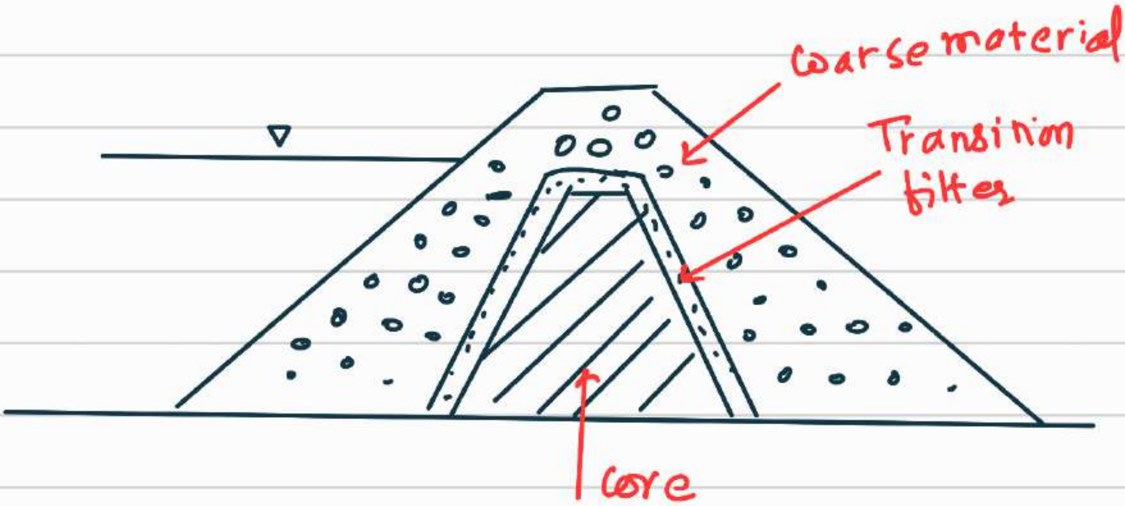


b. zoned type dam:

* consists of more than one kind of material.
* It consists of central impervious Clay

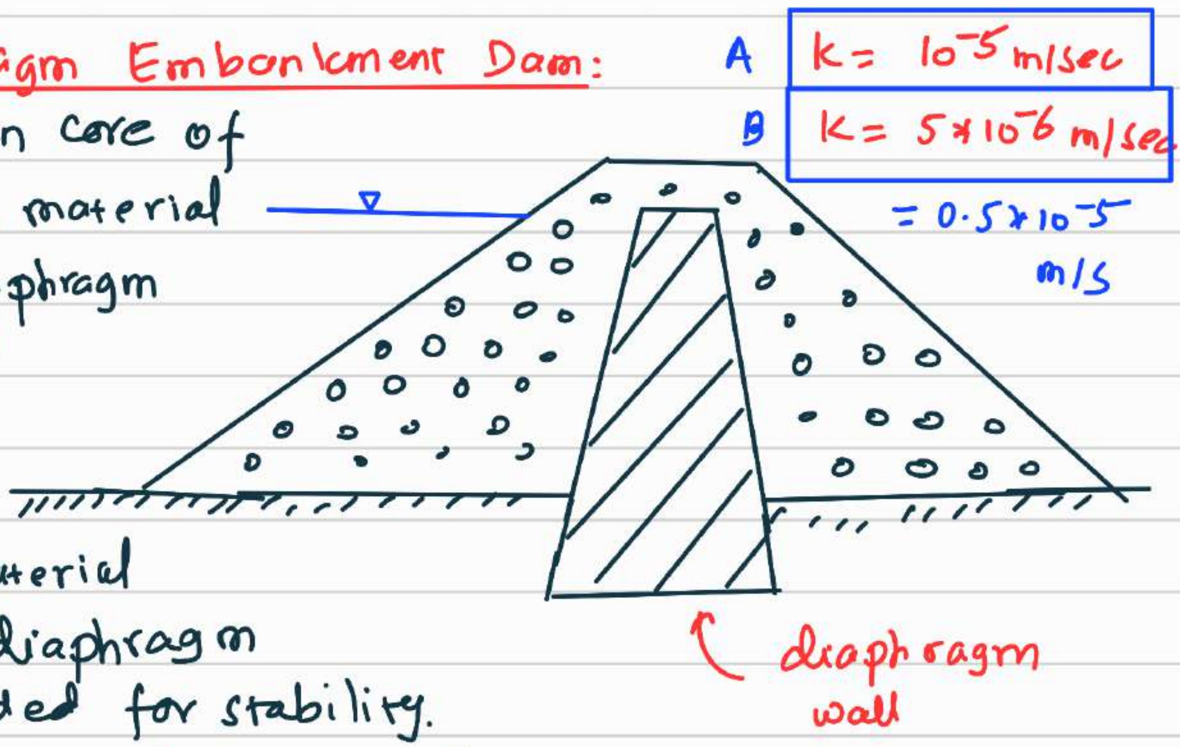
core surrounded by more pervious material.

* Core for seepage control, coarser permeable material for stability.



C. Diaphragm Embankment Dam:

* The thin core of impervious material called diaphragm is provided to check seepage.
* Coarser material outside diaphragm is provided for stability.
* Also called thin core dams.

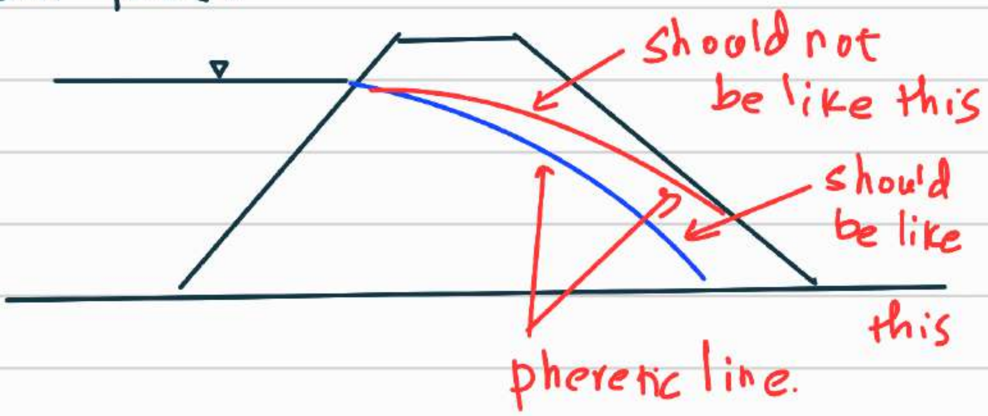


Basic Design Principles of Earthen Dams:

1. The dam should never be allowed to be overtopped during heavy flood. For this sufficient free board and spillway of adequate size is provided.

2. Free board = 1.5 * height of wave
$= 1.5 * h_w$

3. Seepage should be burried on base of dam and seepage line should not be exposed on downstream face.



4. Top width of dam shall be:

$B = H/5 + 3$ for $H <<< 30m$

$B = 0.55 * H^{1/2} + H/5$ for $H < 30m$

$B = 1.65 \ (H + 1.5)^{1/3}$ for $H > 30m$

5. U/s slope = 3H:1V to 2.5H:1V and d/s slope = 2.5H:1V to 2H:1V.

6. Upstream face shall remain stable during drawdowns.

7. Central impervious core shall have minimum width of 3m. maximum size is obtained by $(X - 1/2)H : 1V$ if $XH : 1V$ is side slope of dam.

8. Upstream face should be protected by riprap from waves and heavy winds.

9. The earthquake forces shall also be considered while fixing size.

Seepage through Embankment Dam:

Q. An earthen dam made of homogenous material has following data:

→ coefficient of permeability, $K = 5 \times 10^{-4}$ cm/sec

level of top of dam = 200m

level of deepest river bed = 178m

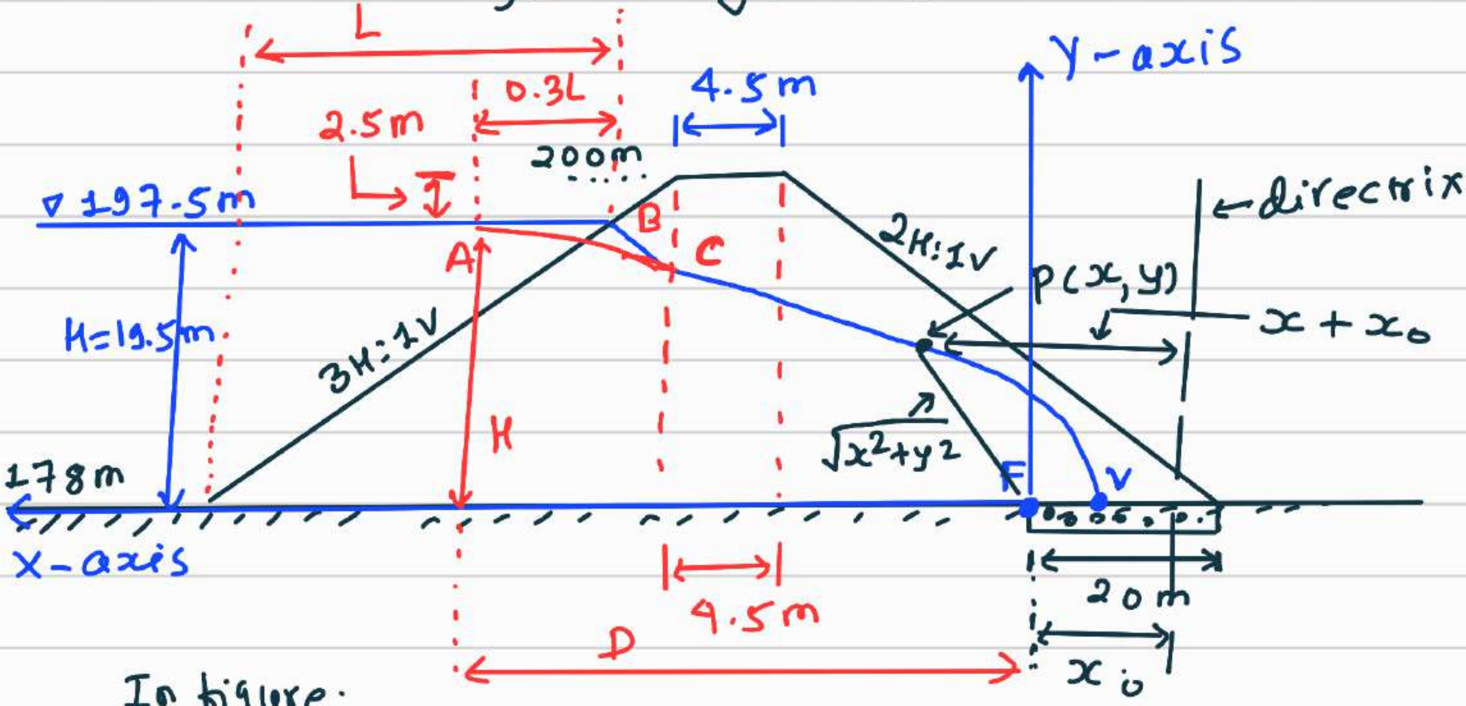
HFL of reservoir = 197.5m

width of top of dam = 4.5m

u/s slope = 3H : 1V

d/s slope = 2H : 1V

plot the phreatic line and seepage discharge through dam.



In figure:

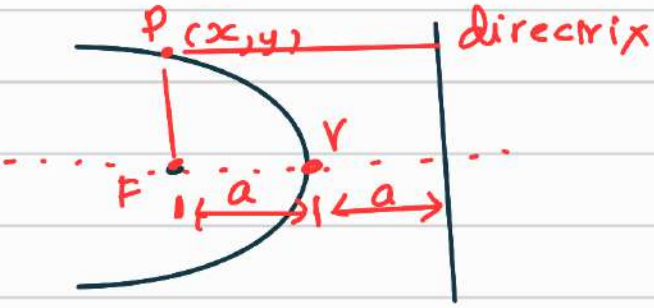
$$L = 19.5 \times 3 = 58.5\,m$$

$$AB = 0.3L = 0.3 \times 58.5 = 17.55\,m$$

Concept: Phreatic line is a parabola. The focus is at starting point of filter. The distance betn focus and directrix is x_0.

* Actual phreatic line is BCV.
* Extension of parabola is upto A.

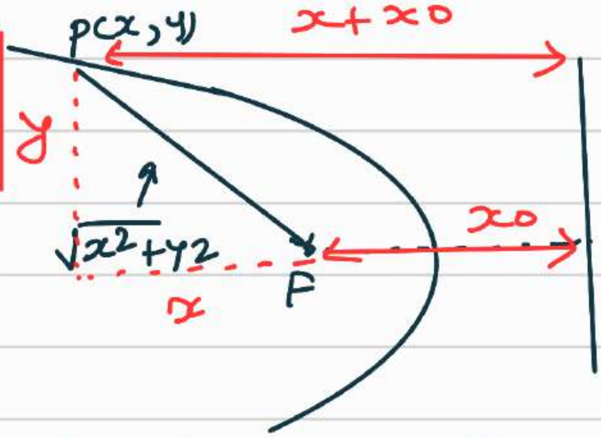
From property of parabola:
Distance from focus
= Distance from
directrix.



So, for any point P, distance from focus
$= \sqrt{x^2+y^2}$

distance from directrix $= x + x_0$

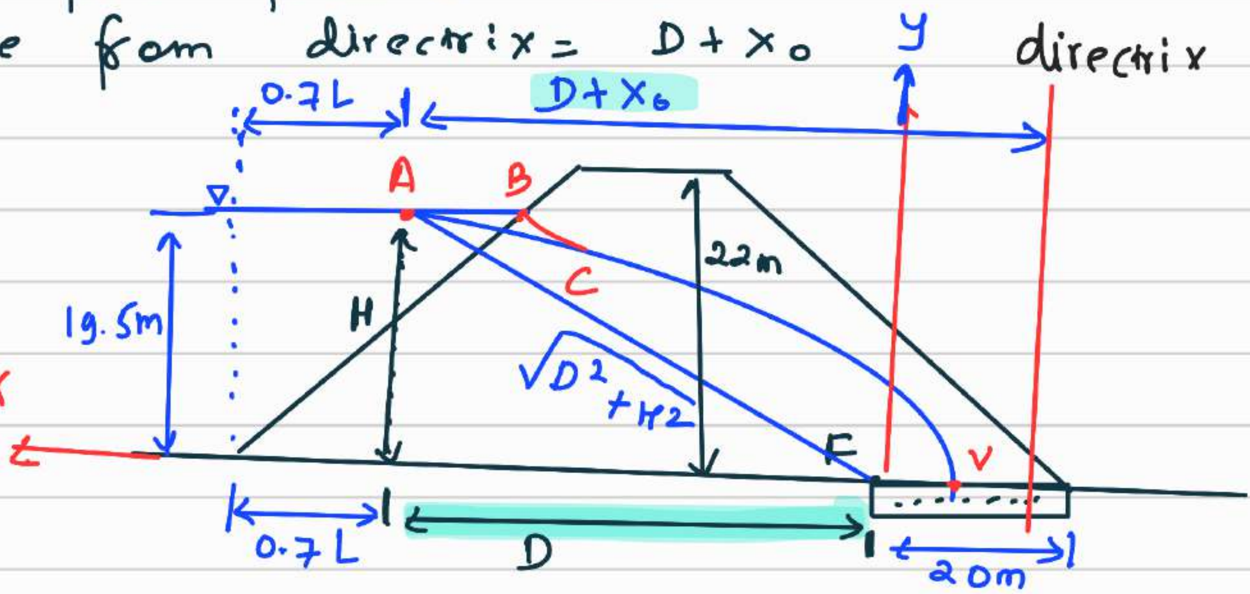
$$\boxed{\therefore \sqrt{x^2+y^2} = x + x_0}$$



Seepage discharge;
$q = k * x_0$

To calculate x_0, take point A on parabola:
Distance from focus $= \sqrt{D^2+H^2}$
Distance from directrix $= D + x_0$

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



from fig: $D = \text{total width} - 0.7L - 20$

Total width $= 22*3 + 4.5 + 22*2 = \underline{114.5}$

$\therefore D = \underline{114.5} - \underline{0.7 * 58.5} - \underline{20}$

$= \underline{53.55\,m}$

$\underline{H = 19.5\,m}$

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

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

or $x_0 = \sqrt{53.55^2 + 19.5^2} - 53.55$

$\boxed{\text{or } x_0 = 3.44m}$

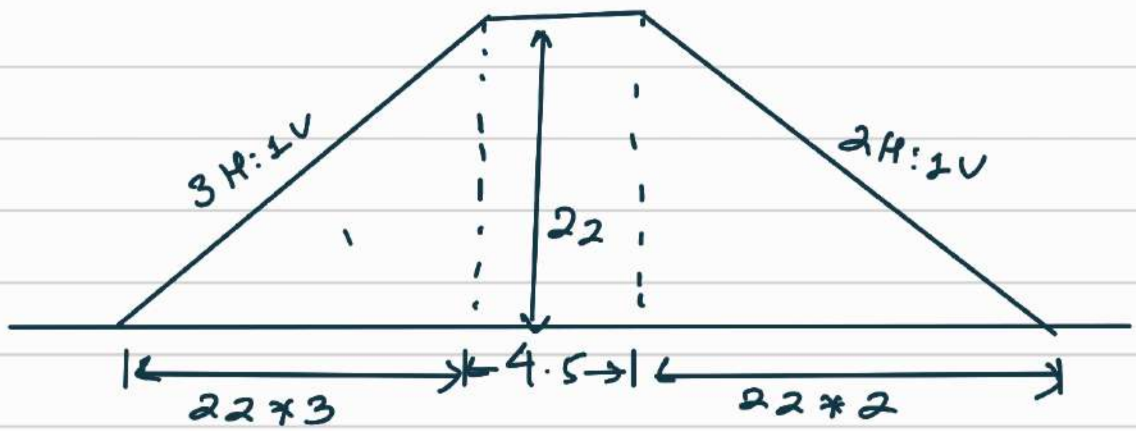
$\therefore$ seepage discharge; $\boxed{q = k * x_0}$

or $q = 5*10^{-4} cm/sec * 3.44m$

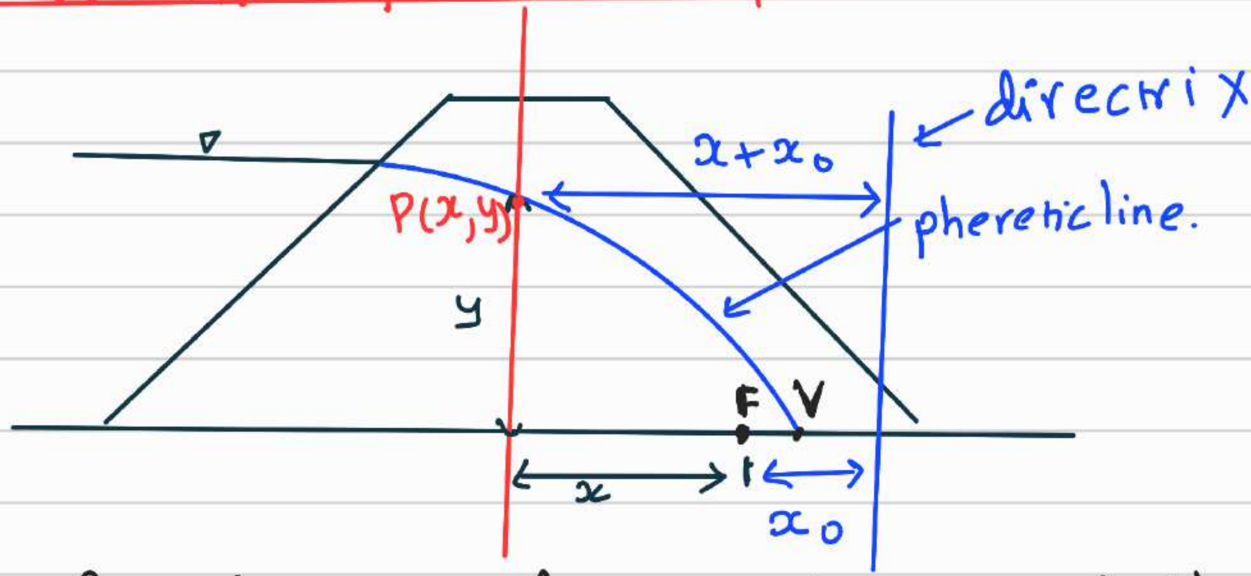
$= 5*10^{-6} m/sec * 3.44m$

$= 1.72*10^{-5} m^2/sec$

$\boxed{= 1.72*10^{-5} m^3/sec/m.}$



Derrivation of Above equation:

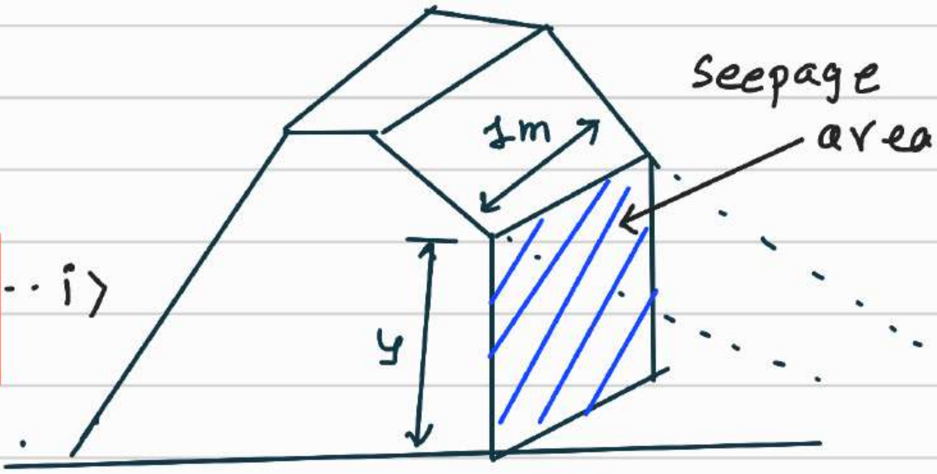


* Consider a dam as shown and the pheretic line as shown. The pheretic line is a parabola let F is focus of parabolo and V is the vortex.
* x_0 is the distance between focus and directrix.

$q = kiA$

$= k * \frac{dy}{dx} * y * 1$

$\boxed{\therefore q = K * y * \frac{dy}{dx} \quad \cdots\cdots \text{i)}}$



* q = discharge
* per unit length of dam
* K = coeff. of permeability
* i = hydraulic gradient $= \frac{dy}{dx}$
* A = Area $= y * 1$

From property of parabola,
distance from focus = distance from directrix

* Consider a point (x, y) on parabola:

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

$\Rightarrow x^2 + y^2 = (x + x_0)^2$

Taking derrivative with respect to x on both sides,

$\Rightarrow \frac{d}{dx}(x^2) + \frac{d}{dx}(y^2) = \frac{d}{dx}(x + x_0)^2$

$\Rightarrow 2x + 2y * \frac{dy}{dx} = 2(x + x_0) * \frac{d}{dx}(x + x_0)$

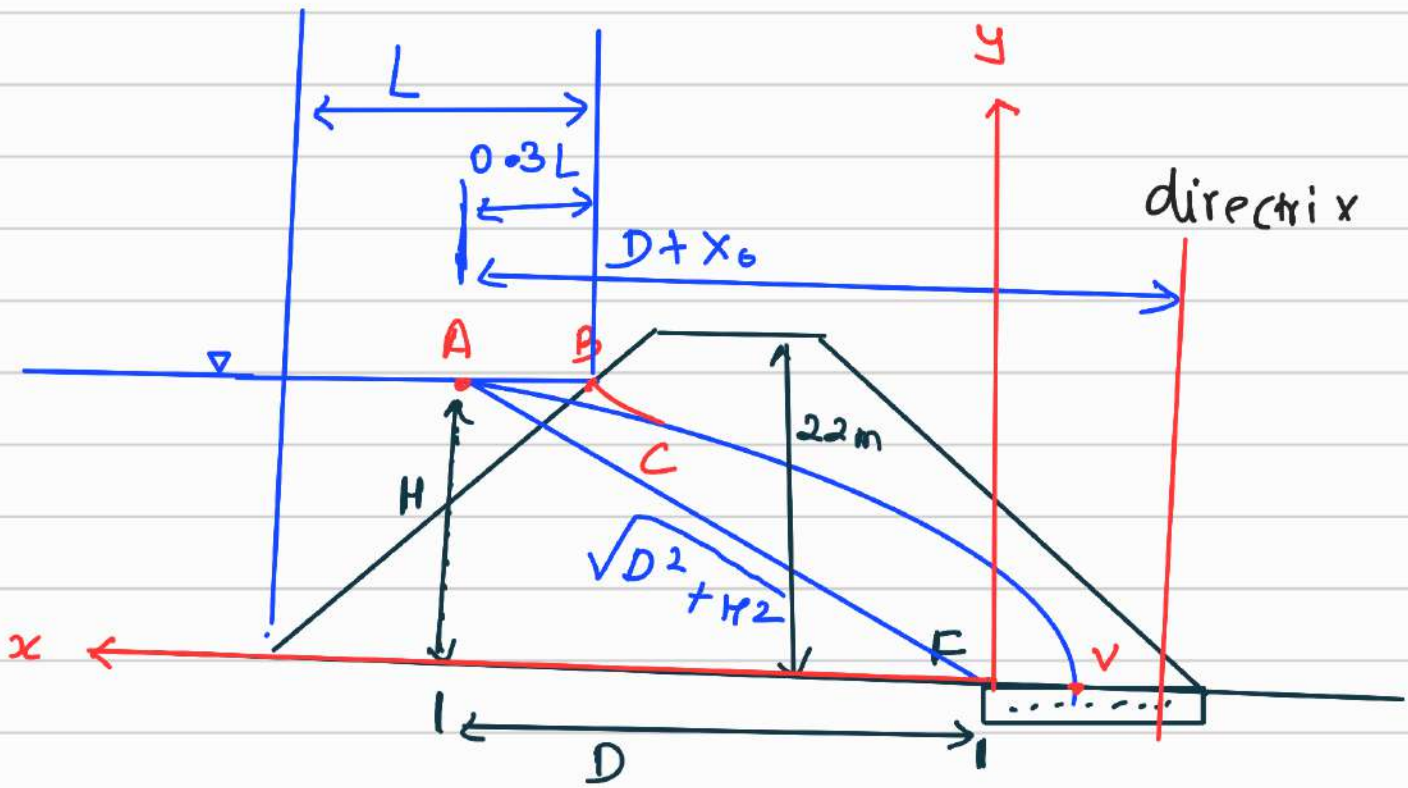
$\Rightarrow 2x + 2y\frac{dy}{dx} = (2x + 2x_0) * 1$

$\Rightarrow y\frac{dy}{dx} = x_0 \quad \cdots\cdots \text{ii)}$

$$\boxed{\text{so; } q = k * y * \frac{dy}{dx} = k * x_0}$$

To get value of x_0, take a point on parabola such that both x and y of that point are known.

* Cassagrande suggested that the pheretic line BCV is parabolic in portion CV but the portion BC is not a parabola. He further stated, the parabola would meet reservoir level at A, if it was extended beyond C.
* Note that AB = 0.3L, where; L = horizontal length of wetted portion of u/s face.



* Applying property of parabola at A,
distance from A = distance from directrix

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

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

* Therefore, discharge, $q = K * x_0$

or $q = K * [\sqrt{D2 + H2} - D]$

Headworks of Run of River Projects:

* The components constructed at the head of conveyance structure (canal, tunnel, pipe) to divert required amount of sediment free water are called headworks.
* Headworks of RoR projects are: weir/barrage, undersluice, intake, stilling basin, gravel trap, settling basin, etc.

Requirements of Good Headworks:

1. Withdrawl of required amount of water
2. minimum sediment entry
3. Flood bypass → $P = 4.75\sqrt{Q}$
4. minimum head loss $H_{net} = H_g - \text{Head loss}$
5. Prevent floating debris, ice, trashes from entering conveyance system.
6. Simple in construction.
7. minimum maintenance cost:

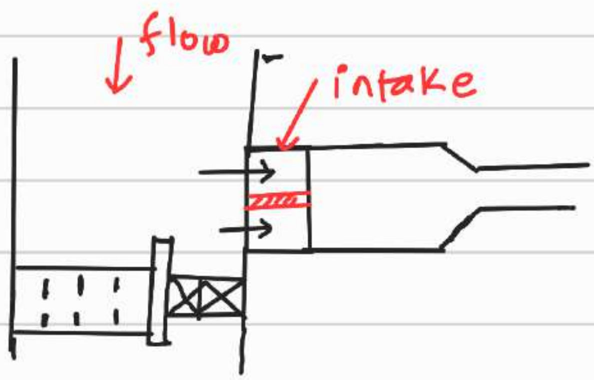
Intake:

* It is the hydraulic structure constructed to withdraw required amount of water.
* Requirements:
  (a) withdrawl of required quantity of water.
  (b) minimum sediment entry
  (c) simple in construction.
  (d) less maintenance cost.

Types:

a. Side intake:

* It is constructed at bank of river which



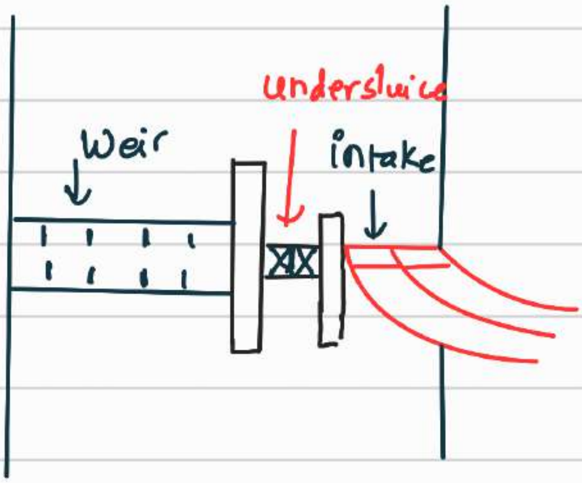
is perpendicular to river flow.

<u>Advantage:</u>
1. minimum sediment entry.
2. Easy in construction.
3. less maintenance cost

<u>Limitation:</u>
1. withdraws less water.
2. Requires large height of weir.

b. Frontal Intake:

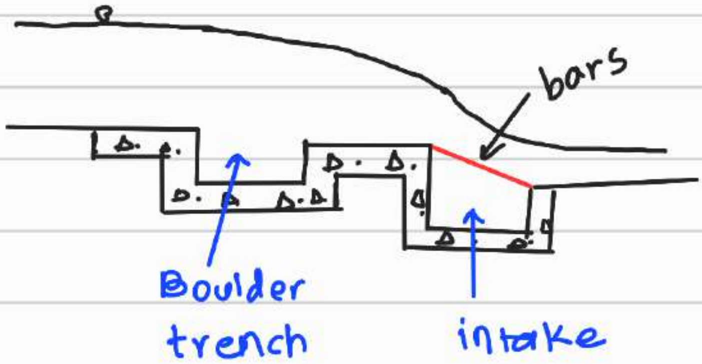
* Intake is constructed on river parallel to flow direction

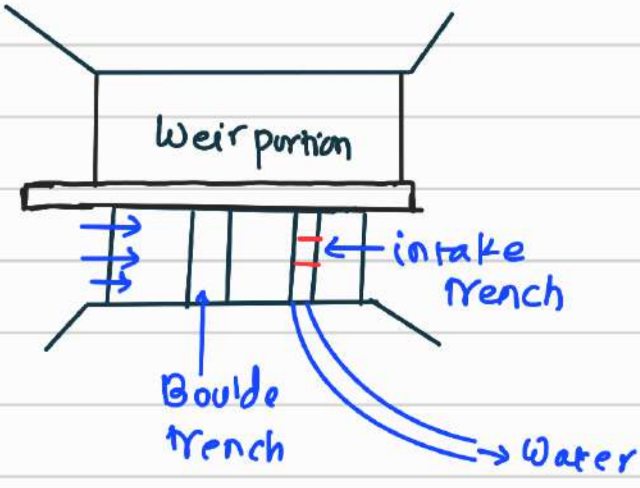


<u>Advantages:</u>
1. Withdraws more water.
2. Height of weir is less.

<u>Limitations:</u>
1. Withdraws more sediment.
2. Construction difficulty.
3. maintenance cost is more.

3. Bottom or Trench Intake:



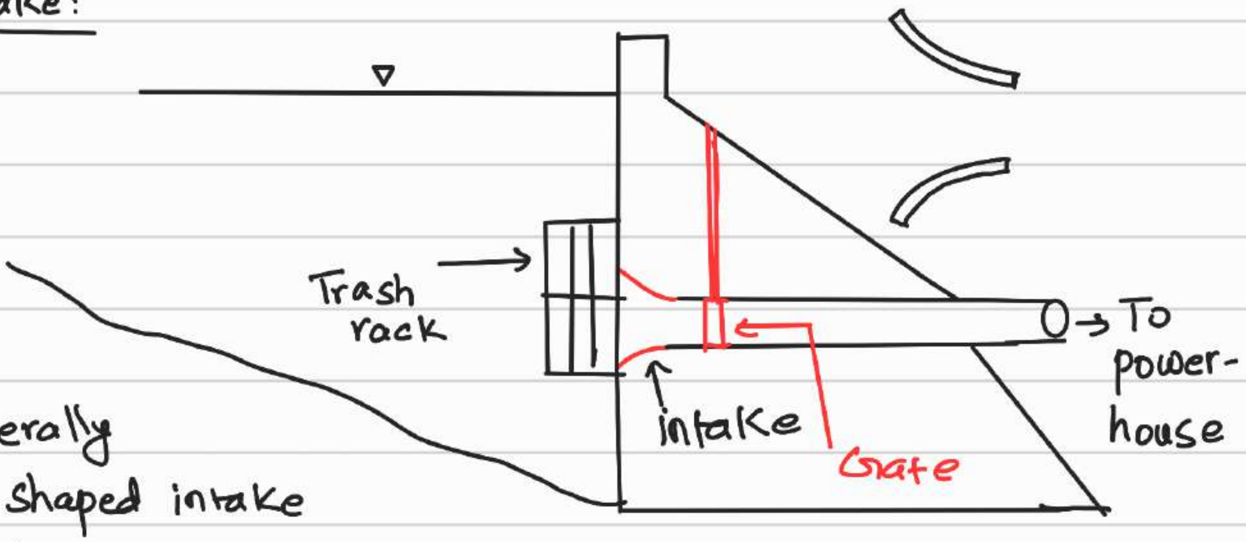


* This intake consists of a trench in which water enters from river bed. A boulder trench is provided on upstream of intake trench to trap boulders.

* It is suitable in steep rivers where water carries boulders mainly and other sediment load is less.

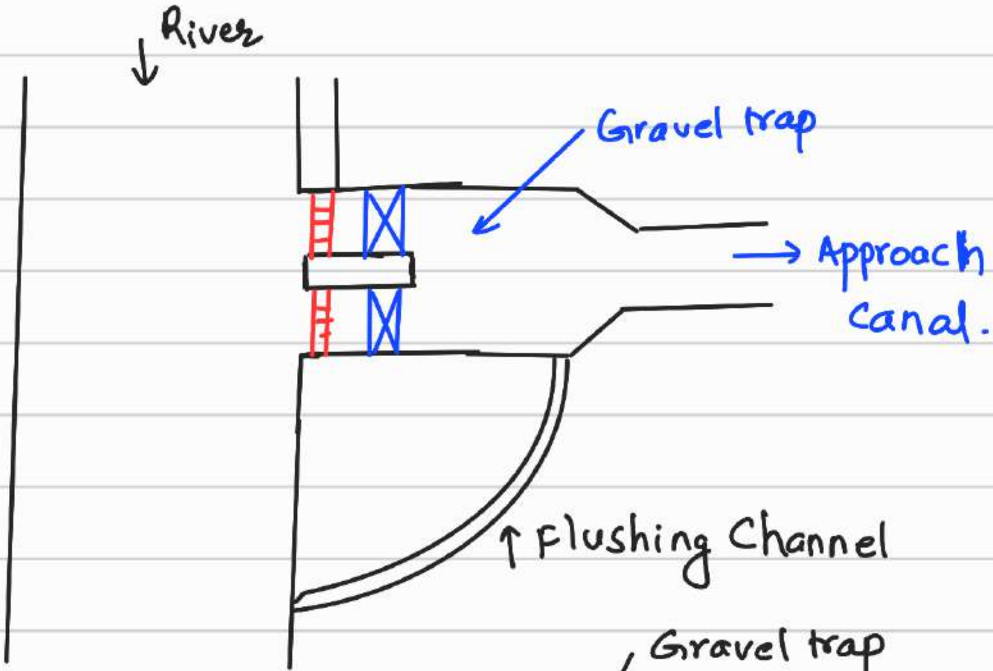
4. Dam intake:

* This type of intake is provided in storage projects.
* It is generally bellmouth shaped intake having trash rack at it's entry.
* Cost of hoisting equipment is more.



Gravel Trap:

* It is a hydraulic structure provided after intake to deposit and flush gravels of size ≥ 5mm.
* The flushing interval is generally 12 hours.



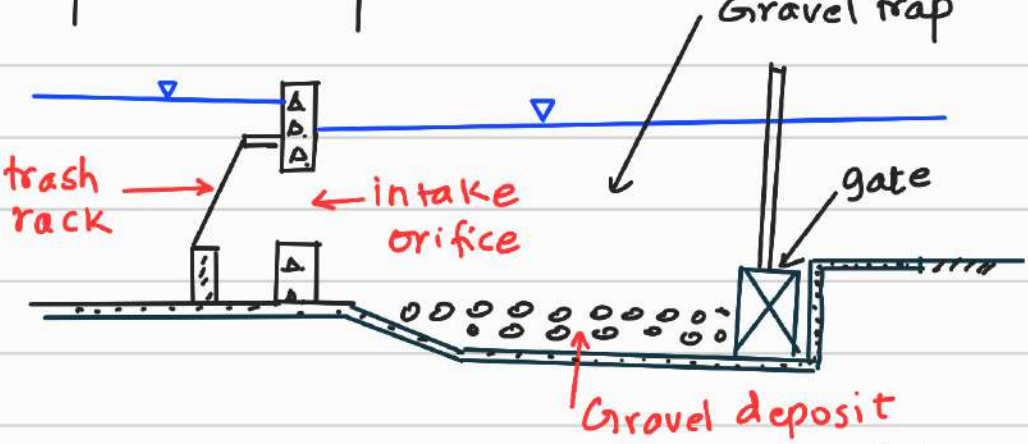
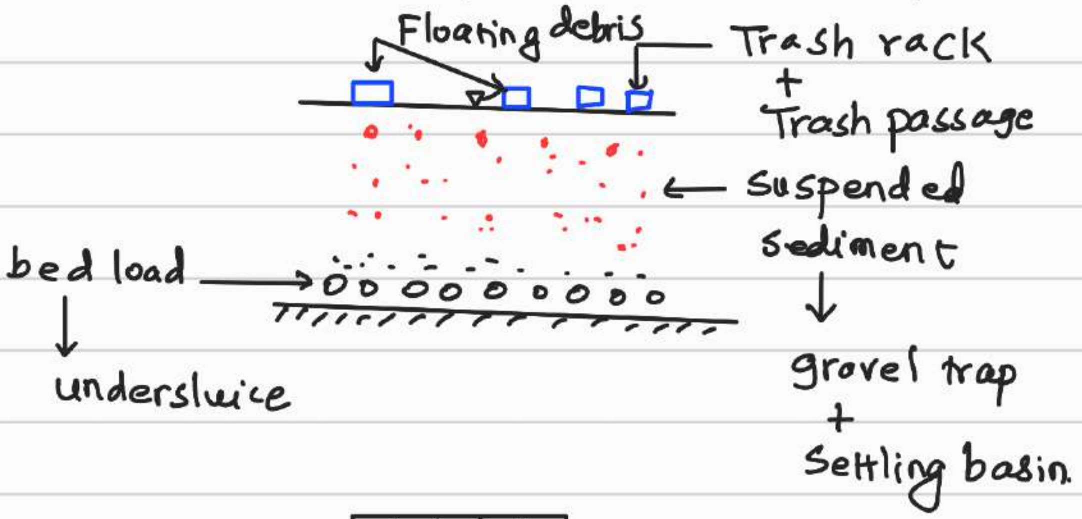
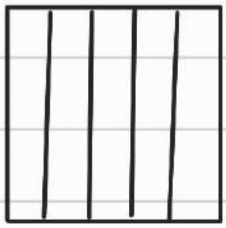


fig: L-section of gravel trap.

Settling Basin:

* Settling basin is a hydraulic structure provided to settle and remove suspended sediments from water.





* A settling basin creates quiscent condition or removes turbulence of water due to decreased velocity and causes suspended sediments to settle due to gravity

Advantages of Settling Basin:

1. It reduces sediment load on turbine.
2. It reduces silting tunnels, canals, pipes.
3. To reduce wear and tear of turbine runners, penstock pipes, valves, etc.
4. To reduce maintenance cost of turbine.
5. To reduce plant outage.

Target Particle size and settling velocity:

* The particle size for which the settling basin is designed is target particle size.
  * It depends on:

a. Head:- High head ⇒ Target particle size small

b. Hardness: more hardness ⇒ Target particle size small.

Settling velocity / fall velocity:

* master formula:

$$\omega = \sqrt{\frac{4}{3} * \frac{gd(G-1)}{C_d}}$$

ω = Settling velocity

d = diameter

G = specific gravity of sediment.

C_d = drag coefficient.

i) For $d < 0.1$ mm

$$\boxed{\omega = \frac{gd^2}{18\nu} * (G-1)}$$

⇒ $C_d = \frac{24}{Re}$ from stoke's law.

ii) For $d = 0.1$ mm to 1 mm

$C_d = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$ ...... i)

$Re = \frac{\omega * d}{\nu}$; Re = Reynold's no. ..... ii)

ν = kinematic viscosity.

$\omega = \sqrt{\frac{4}{3} * \frac{gd(G-1)}{C_d}}$ ...... iii)

By solving i), ii) and iii); we calculate ω.

iii) For $d > 1$ mm

* $C_d = 0.4$

$$\therefore \omega = \sqrt{\frac{4}{3} * \frac{g * d(G-1)}{0.4}} = \sqrt{3.33\, gd * (G-1)}$$

Design of Settling Basin:

Design a settling basin by considering turbulent approach and trap efficiency of 95%. Take design discharge = 7.5 m³/sec, particle size to settle = 0.2 mm, kinematic viscosity = 0.87 * 10-6 m²/sec (0.87 centistokes).

Soln:

① Calculation of settling velocity:

We have; $d = 0.2\,mm \rightarrow 0.1$ to $1\,mm$ range.

$\nu = 0.87 * 10^{-6}\ m^2/sec.$

We know;

$$\omega = \sqrt{\frac{4}{3} * \frac{gd * (G-1)}{C_d}}$$

$$\Rightarrow \omega^2 * C_d = \frac{4}{3} * g d (G-1) \quad \ldots\ldots i)$$

Also;

$$Re = \frac{\omega * d}{\nu}$$

$$\Rightarrow \omega = \frac{Re * \nu}{d} \quad \ldots\ldots ii)$$

$$C_d = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34 \quad \ldots\ldots iii)$$

∴ Eqn i) becomes:

$$\left[\frac{Re * \nu}{d}\right]^2 * \left[\frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34\right] = \frac{4}{3} * gd * (G-1)$$

$$\left[\frac{Re * 0.87 * 10^{-6}}{0.2 * 10^{-3}}\right]^2 * \left[\frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34\right]$$

$$= \frac{4}{3} * 9.81 * 0.2 * 10^{-3} * (2.65 - 1)$$

$Re = 6.75$

$$\therefore \omega = \frac{Re * \nu}{d} = \frac{0.87 * 10^{-6} * 6.75}{0.2 * 10^{-3}} = 0.029\ m/sec$$

* This velocity is spherical particles.

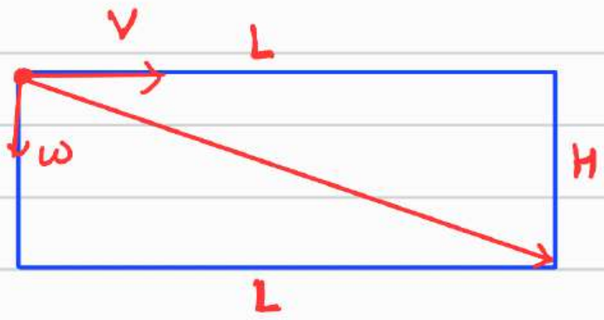
So, considering all particles are not spherical,

actual. $\omega = 0.65 * 0.029 = 0.019$ m/sec
$\simeq 0.02$ m/sec.

2. Since; efficiency; $\eta = 95\%$.
from Vetter's formula:
$$\eta = 1 - e^{-\omega A_s / Q}$$
A_s = surface area.
$$\Rightarrow 0.95 = 1 - e^{-0.02 * A_s / 7.5}$$
$$\Rightarrow A_s = 1123.34 \, m^2$$
$$\boxed{\text{where: } A_s = L * B}$$

3. Assume depth = 3.5 to 5 m
$\simeq$ 4 m.

then; length of basin is calculated by:
a. without considering turbulence:
Settling time = travel time



$$\Rightarrow \frac{H}{\omega} = \frac{L}{V}$$

v = Horizontal or flow velocity.
V should be less than or equal to critical velocity, Vc.
$V_c = a\sqrt{d_{mm}}$ m/sec
$a = 0.36$ for $d > 1$ mm
$a = 0.44$ for $d = 0.1$ to 1 mm
$a = 0.51$ for $d < 0.1$ mm
For 0.2mm particle, $a = 0.44$
$\therefore V_c$ = critical horizontal velocity $= 0.44 * \sqrt{0.2}$
$= 0.196$ m/sec

Take; $V = 0.196$ m/sec.

$$\therefore \frac{H}{\omega} = \frac{L}{V} \Rightarrow L = H * \frac{V}{\omega} = 4 * \frac{0.196}{0.02}$$

$$= 39.2m$$

b. By considering turbulence:

→ due to turbulence, ω reduces to ω'.

where; $\omega' = \omega - \alpha * V$

$$= \omega - \frac{0.132}{\sqrt{H}} * V$$

$$= 0.02 - \frac{0.132}{\sqrt{4}} * 0.196$$

$$= 0.007 \text{ m/sec}$$

∴ for length; travel time = settling time

$$\Rightarrow \frac{L}{V} = \frac{H}{\omega'}$$

$$\Rightarrow L = H * V/\omega' = \frac{4 * 0.196}{0.007} = 112m.$$

$\boxed{\therefore \text{Adopt length} = 112m.}$

4. Calculate width:

we have; surface area;

$A_s = 1123.34 \; m^2$

But; $A_s = L * B \Rightarrow B = \frac{A_s}{L} = \frac{1123.34}{112}$

$$= 10.03m.$$

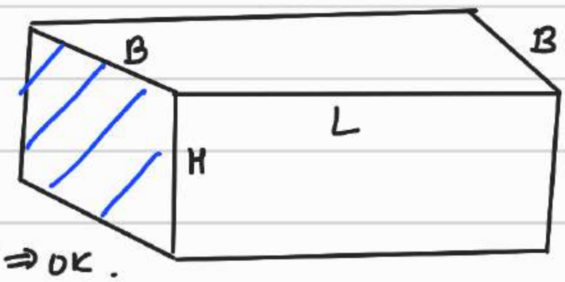
$$\simeq 10.1m.$$

Check for velocity:

$$V = \frac{Q}{A} = \frac{Q}{B * H} = \frac{7.5}{10.1 * 4}$$

$$= 0.185 \text{ m/sec}$$

< 0.196 m/sec ⇒ OK.



Final Parameters:

length $= 112\,m$

width, $B = 10.1\,m$

Height, $H = 4\,m$

Concentration Approach:

↓

get surface area of basin.

$A_s = K \times \frac{Q}{\omega}$

$$\boxed{\eta = 1 - e^{-\omega A_s / Q}}$$

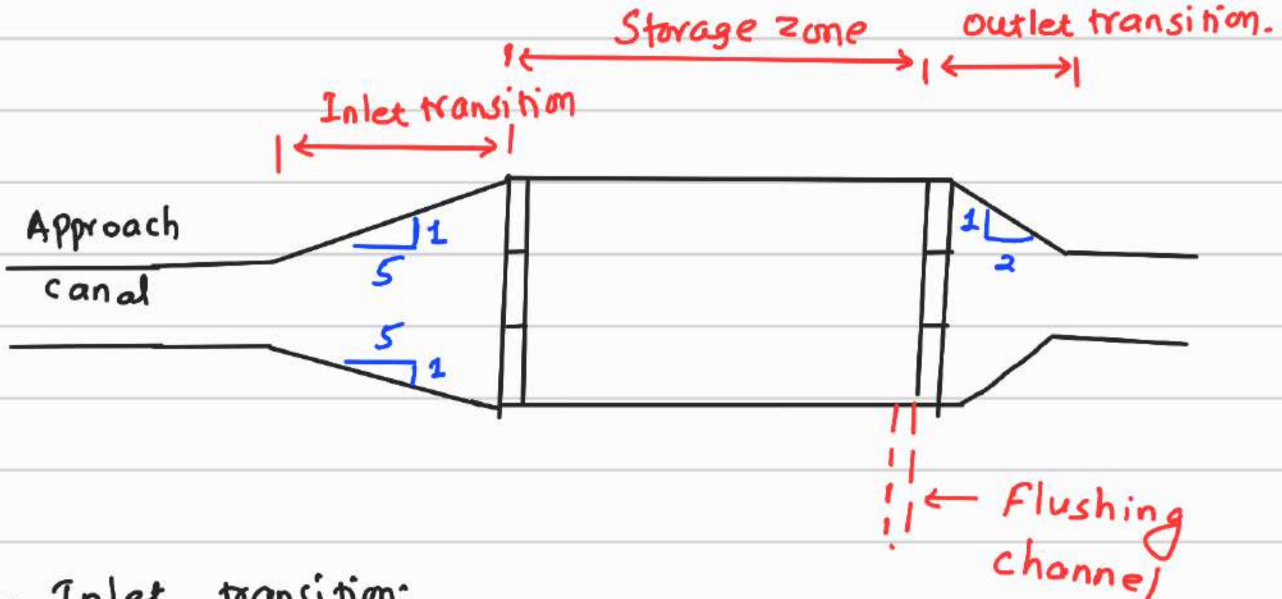
Particle Approach

travel time = settling time.

$\frac{L}{V} = \frac{H}{\omega} \Rightarrow L = \frac{HV}{\omega}$ → No turbulence

$L = \frac{HV}{\omega'}$ → with turbulence

Components of Settling Basin



1. Inlet transition:

* It is gradual transition from approach canal to full width of settling basin.
* Transition ratio may be 5:1 in length and

2:1 in depth.

* In order to avoid turbulence in the basin, inlet transition is made as gradual as 5:1 which also causes uniform distribution of sediments in the transition.
* If inlet transition angle is high flow separation may occur which causes turbulence in the basin and certain part of basin may be ineffective.

2. Storage Zone:

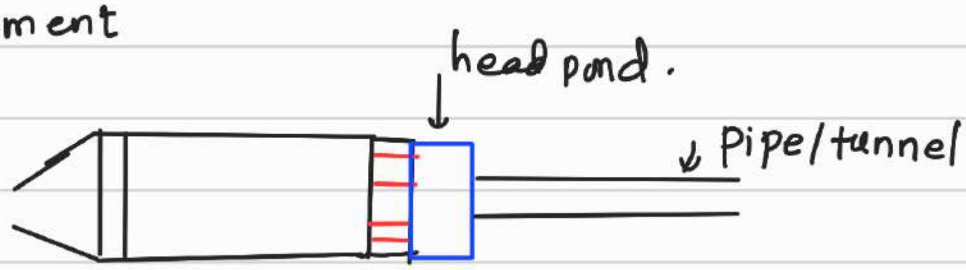
* This is main part of settling basin in which suspended sediments settle.
* This zone shall have enough area/volume for settling and accomodation of sediment.

3. Outlet transition/ Outlet zone:

* If site condition permits, outlet transition shall be provided.
* Outlet transition may be 2:1 in length and 1:1 in depth.
* This zone provides clear water to the conveyance system that follows settling basin.
* This zone may have outlet gate or may be connected with head pond for providing water to headrace pipe or tunnel.

4. Sediment Flushing System:

* Suitable arrangement including flushing channel, flushing gate or any other devices shall be provided for sediment flushing.

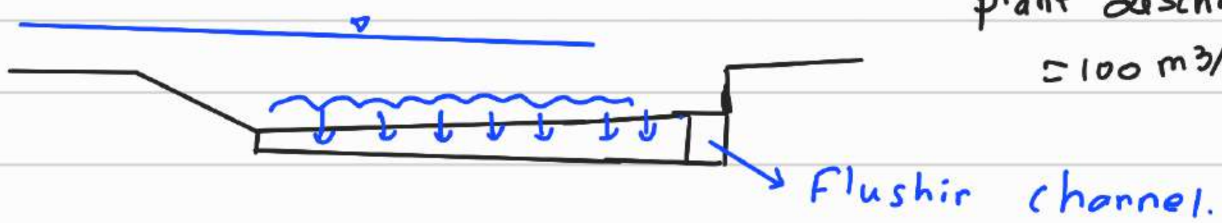


Types of Settling Basin Based on Flushing:

1. Continuous flushing settling basin.

* The settling and flushing are continuous.
* There is no interruption in power production.
* 10 to 15% additional flow is utilized for flushing of settling basin.
* The plant outage is low and plant factor is improved as there is no need to shutdown the plant for flushing.
* Some additional devices may be needed for suction or dredging of sediments.

plant discharge = 100 m^3/sec.



2. Intermittent Flushing Settling Basin:

* In these basins, some storage zone is allocated for sediments and the flushing is carried out at some regular intervals.
* The generation is stopped during flushing of the basin.
* The plant outage is increased and plant factor is reduced.
* The basin discharges sediments at high concentration, which is not good from environmental point of view / degrades water quality.

Tunnel:

* The underground conduit excavated without removing overburden is called tunnel
* Hydrotunnels:- Used for carrying water for hydropower generation.

Pressure Tunnels: * flow occurs under pressure.
* Full flow tunnels.

Non-Pressure Tunnel: * open channel flow.
* partial flow.

Advantages of Tunnel:

1. length of conveyance is short.
2. Head loss is reduced.
3. land acquisition is not required.
4. Safe from surface instabilities like landslide, rock fall, mass wasting, interruption, etc.
5. Economical for large discharge.
6. Easy for inter-basin transfer.
7. less environmental impacts.
8. less maintenance cost.
9. less siesmic effect.
10. lining cost can be reduced if good quality rock is available.
11. Reduced risk.

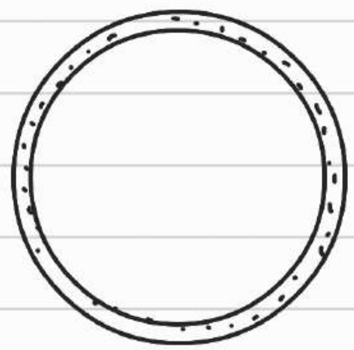
Disadvantages:

1. Not feasible for small discharge.
2. large cost
3. longer construction period.
4. uncertainities in sub-surface geology.
5. Extensive investigations.

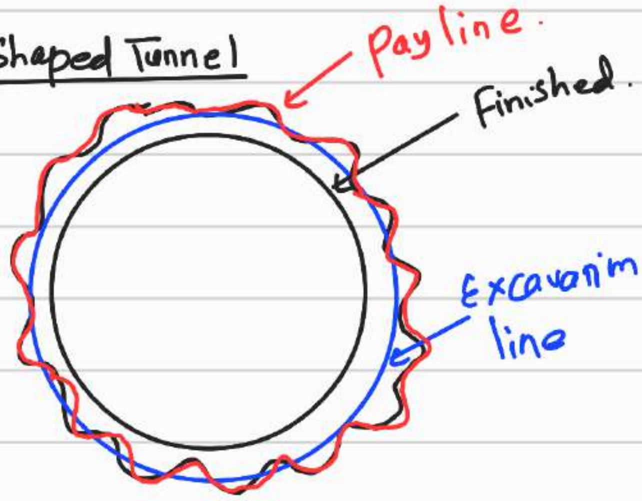
Geometric shape of Tunnel:

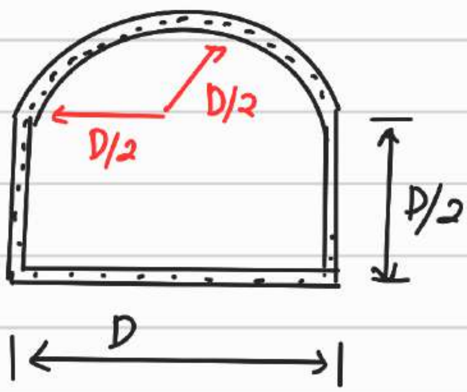
a. Circular section:

* Better from structural point of view.
* Difficulty in construction.
* Suitable if rocks are of poor quality.



b. D-Shaped Tunnel



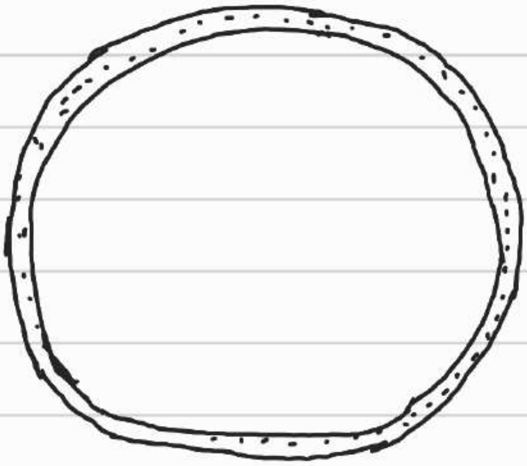


* Easy from construction point of view due to flatter invert.
* structurally weak.
* Suitable if rocks are of good quality.

C. Horse she section:

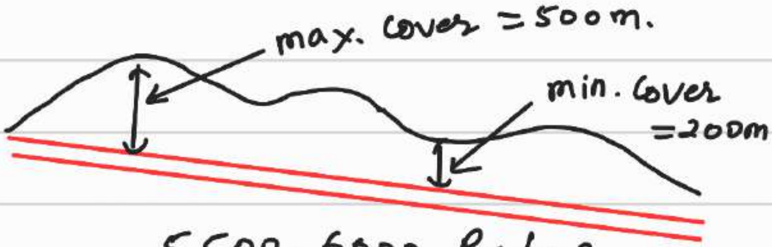
* It consists of flatter invert which gives working space and upper part is curved, which gives structural strength.

* If rock quality is poor or intermediate quality, this section is preferred.



Tunnel Supports:

* The tunnel supports shall be installed to resist both internal pressure of water and external pressure of overburden.



5500-6000 Rs/m3.

1 meter :- 4-5 lacs.

* Support requirement is more if rock is of poor quality.

a. Immediate support:

* Installed immediately after excavation.
* They may include shields, Jacks timber supports, coating of shortcrete, growtings, etc.

b. Initial Support:

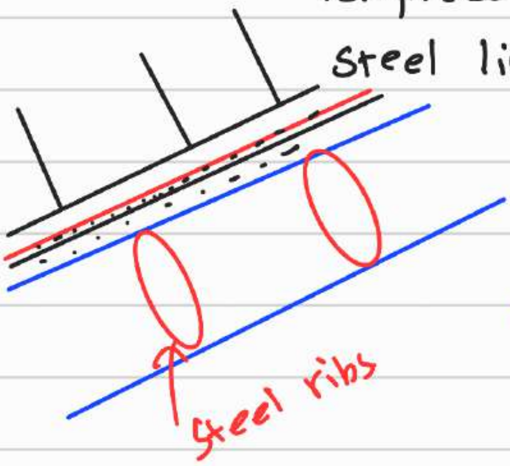
* Tensioned bolts, wire mesh, dowels shortcrete, steel ribs, etc.
* Before finalization of final support.

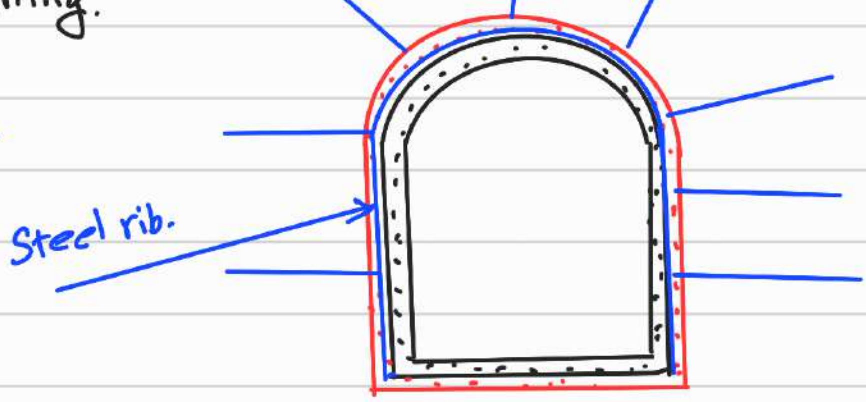
c. Ancillary Support:

* The supports which are different than planned supports due to change in geology, subsurface conditions.

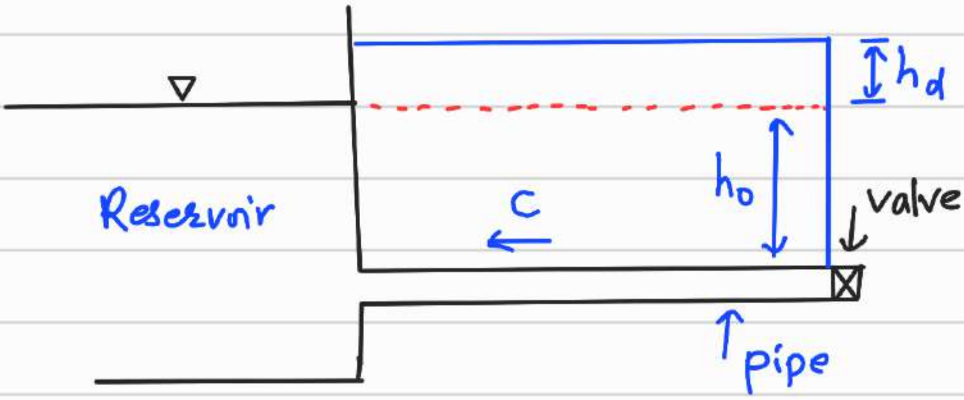
d. Final Support:

* The supports which are installed in the finished section.
* Final supports in addition to initial support may be reinforced cement lining, steel ribs, wire mesh, steel lining.





Water Hammer and Hydrodynamic Pressure Calculation:



* When a liquid column moving in a pipe is brough to rest by closing valve, its kinetic energy gets converted into pressure energy and a pressure wave travels in the pipe. This phenomenon is called water hammer.

* The pressure due to water hammer is called dynamic pressure.

* Water hammer pressure is positive as well as negative. For calculating thickness of pipe, we consider positive water hammer pressure.

Valve closure Time:

1. Critical closure time (T_c):
   * $T_c = \frac{2L}{c}$ = time taken by the wave to travel upto reservoir and return back to valve.

2. Gradual Closure:
   * Actual Closure time, $t > T_c$.

3. Rapid Closure:
   * Actual Closure time, $t < T_c$.

4. Instantaneous Closure:
   * closure time is zero.
   * Valve can be closed instantaneously, but liquid column can't be brought to rest earlier than time equal to L/c.
   * So, we take minimum closure time $= L/c$

Water Hammer Pressure Calculation:

a. Rigid water column theory:

* It considers water is rigid.
* Water hammer pressure = $\frac{\rho V L}{t} = P_d$

ρ = density of water, V = Velocity of flow in pipe
L = length of pipe.
t = time of closure.

$\Rightarrow$ Water hammer head, $h_d = \frac{P_d}{\gamma} = \frac{\rho V L}{t * \rho g}$

$$\Rightarrow \boxed{h_d = \frac{VL}{gt}}$$

b. Elastic water column theory:

* It considers elasticity of pipe as well as compressibility of water.
* This theory is better applied to instantaneous closure of valve.
* Water hammer pressure, $P_d = \rho V C$

where; V = Velocity in pipe
C = Celerity of wave.
ρ = density of water.

$$\boxed{\begin{array}{l} P_d = \frac{\rho V L}{t} \; ; \quad t_{min} = \frac{L}{C} \\ P_d = \frac{\rho V L}{L/C} = \rho V C \end{array}}$$

where; $C = \sqrt{\dfrac{K/\rho}{1 + \frac{DK}{tE}}}$

K = bulk modulus of water $= 2 * 10^9 \ N/m^2$

ρ = density of water.
D = diameter of pipe.
t = thickness of pipe.
E = Young's modulus of elasticity of pipe.

If pipe is considered rigid, $E \to \infty$.

So, $c = \sqrt{K/\rho}$

$$\boxed{\text{Water hammer head, } h_d = \frac{P_d}{\gamma} = \frac{\rho V c}{\rho g} = \frac{Vc}{g}}$$

Surge Tank:

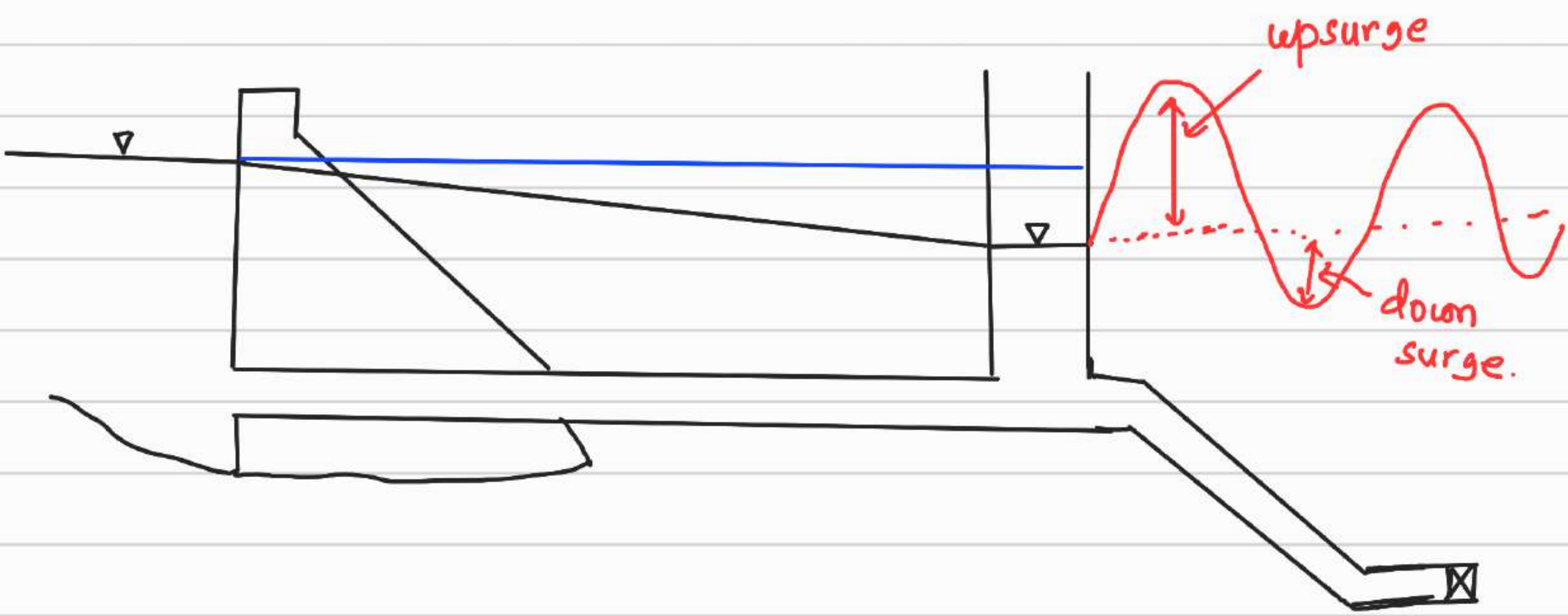


Fig: Surge action during load rejection (plant shutdown)

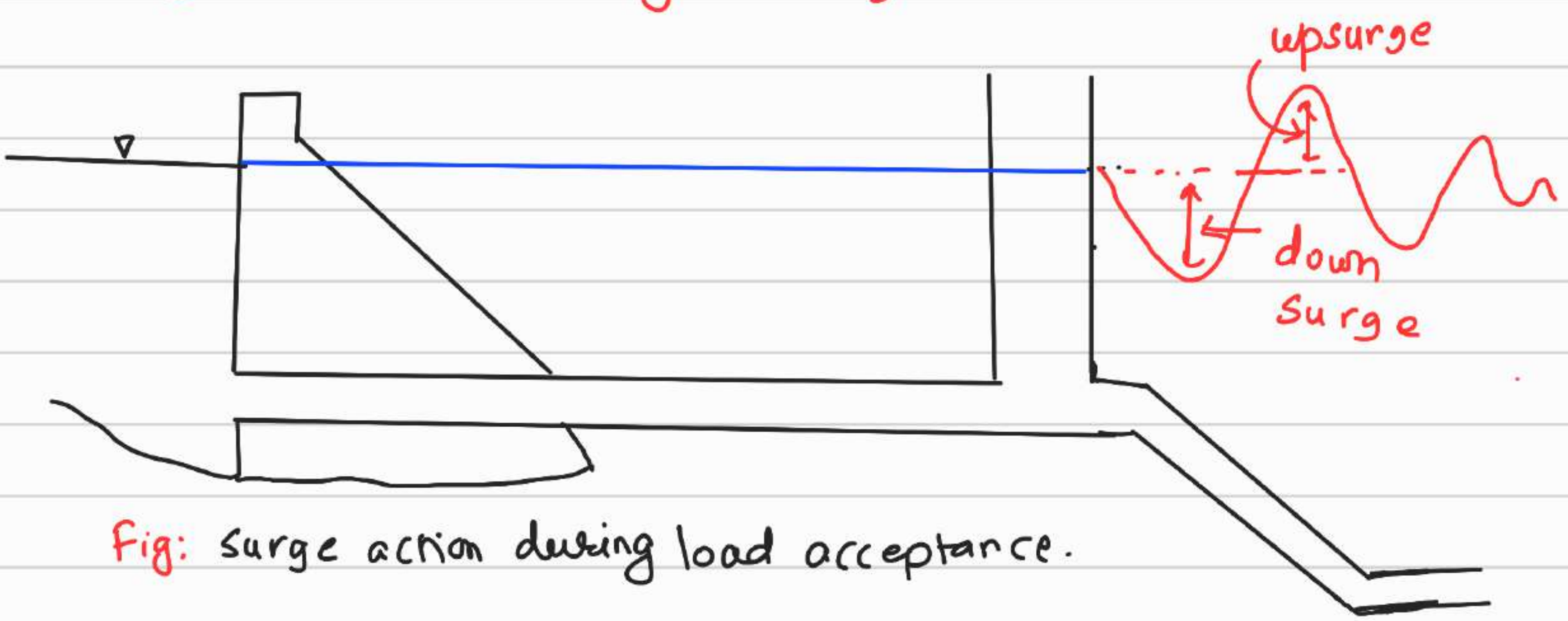


Fig: surge action during load acceptance.

* Surge tank is a cylindrical tank provided at the junction of headrace tunnel and penstock pipe so as to save the low pressure headrace tunnel from water hammer effect.

* A surge tank intercepts the water hammer pressure and due to water hammer pressure, mass oscillation occurs inside surge tank, which causes fluctuation of water level in the surge tank.

<u>Closure/opening of valve may be done due to</u>

① If plant is storage/PROR.

② If maintenance is required.

③ In case of problem in transmission line.

④ If plant has to shutdown due to no load on plant.

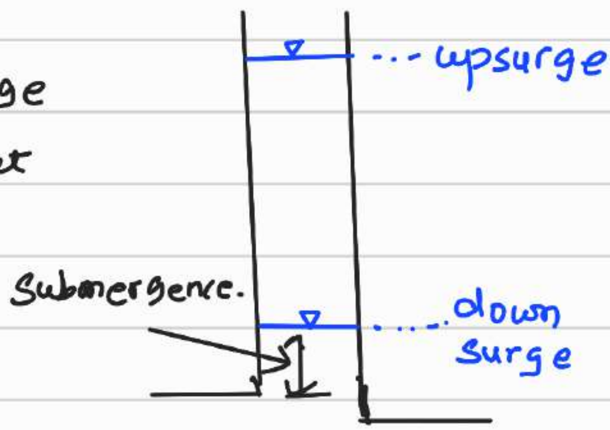
Design of Surge Tank:

Design criteria:

① * The height of surge tank should be enough so that there is no spillage during upsurge.

② The bed level of surge tank shall be such that some submergence is still maintained during down surge condition.

③ The sectional Area of surge tank shall be enough so that the damping of surge will be quickly achieved.



Q. Design a surge tank with following data.

Discharge = 60 m3/sec

tunnel diameter = 6.5m

length of tunnel = 7.8 km

f_{tunnel} = 0.028, gross head = 250m, Find area of surge tank, maximum upsurge and downsurge.

Soln: Velocity in tunnel, $V_T = \frac{Q}{A_T} = \frac{60}{\frac{\pi}{4} * 6.5^2}$

or V_T = 1.8 m/sec

Area of tunnel, $A_T = \frac{\pi}{4} * 6.5^2 = 33.18 m^2$

Head loss in tunnel, $h_f = \frac{f l V_T^2}{2 g D}$

$$= \frac{0.028 * 7800 * 1.8^2}{2 * 9.81 * 6.5} = 5.54 m$$

① minimum area of surge tank, $A_{ST, min}$

$$= \frac{A_T * L_T * V_T^2}{2 g h_f * (h_g - h_f)}$$ ← D. Thoma formula.

h_g ← gross head

$$= \frac{33.18 * 7800 * 1.8^2}{2 * 9.81 * 5.54 * (250 - 5.54)}$$

$$= 31.52 m^2$$

② Calculation of upsurge and downsurge:

Z_{max} = Theoritical maximum surge

$$= \frac{Q}{A_T} * \sqrt{\frac{L_T}{g} * \frac{A_T}{A_{ST}}} = \frac{Q}{A_{ST}} * \sqrt{\frac{L_T}{g} * \frac{A_{ST}}{A_T}}$$

$$= \frac{65}{33.18} * \sqrt{\frac{7800}{9.81} * \frac{33.18}{31.52}} = 56.64m$$

$$Z_{max, upsurge} = Z_{max}\left[1 - \frac{2}{3} P_0 + \frac{1}{9} P_0^2\right]$$

$$P_o = \frac{h_f}{Z_{max}} = \frac{5.54}{56.64} = 0.09$$

$$\therefore Z_{max,\,upsurge} = 56.64 * \left[1 - \frac{2}{3} * 0.09 + \frac{1}{9} * 0.09^2\right]$$

$$= 53.03\,m.$$

$$Z_{max,\,downsurge} = Z_{max}\,[-1 + 2P_o] \leftarrow \text{Zaeger formula}$$

$$= 56.64 * [-1 + 2 * 0.09]$$

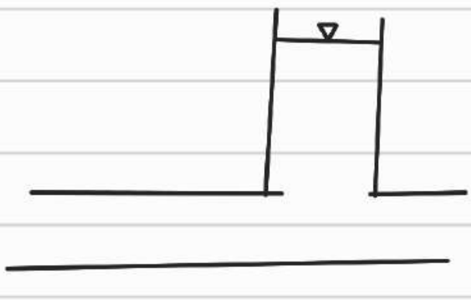
$$= -46.44\,m$$

$$\text{minimum submergence} = \left.\begin{matrix} 0.5 * V_p * \sqrt{D_p} \\ 1.5 * \dfrac{V_p^2}{2g} \end{matrix}\right\} \leftarrow \text{max.}$$

Types of Surge tank:

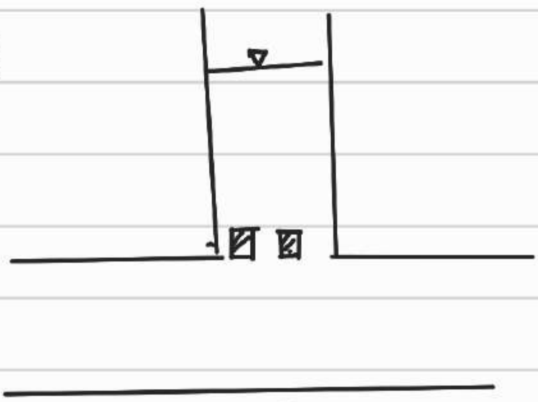
① Simple cylindrical surge tank

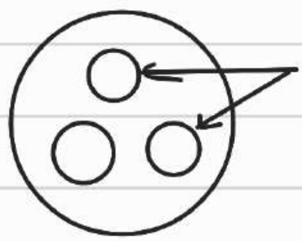
* Cylindrical in cross-section.
* only one opening provided at connection with tunnel.
* Hydraulic action is sluggish.



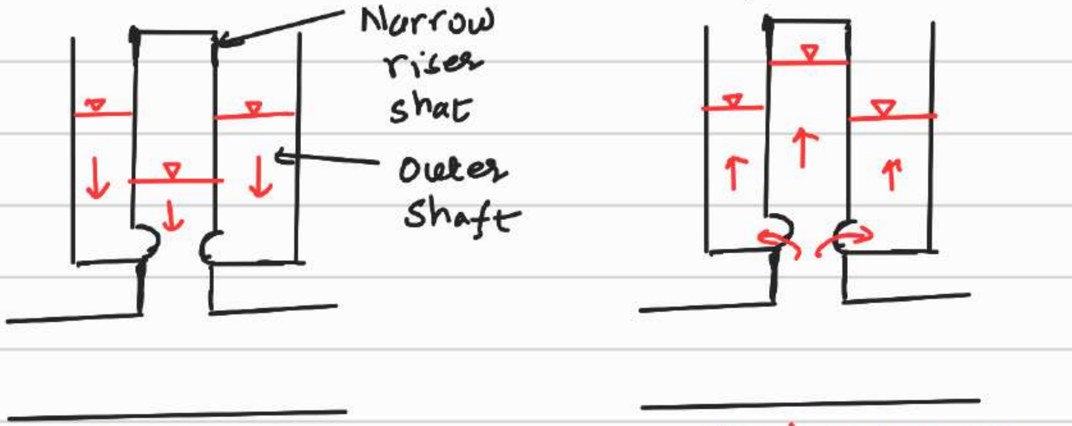
② Throttled surge tank / Restricted Orifice surge tank:

* It consists of restricted orifice at connection with tunnel/pipeline.
* Due to more head loss at restricted orifice, upsurge and downsurge can be minimized.





③ Differential Surge tank/ Johnson type Surgetank



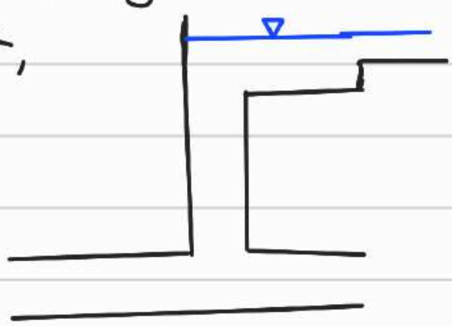
load acceptance

load rejection

* It consists of narrow riser shaft and large outer part.
  * Narrow riser shaft responsds to upsurge and downsurge quickly so that damping of surge is quick.
  * Outer large part provides required volume for sudden start of turbines.

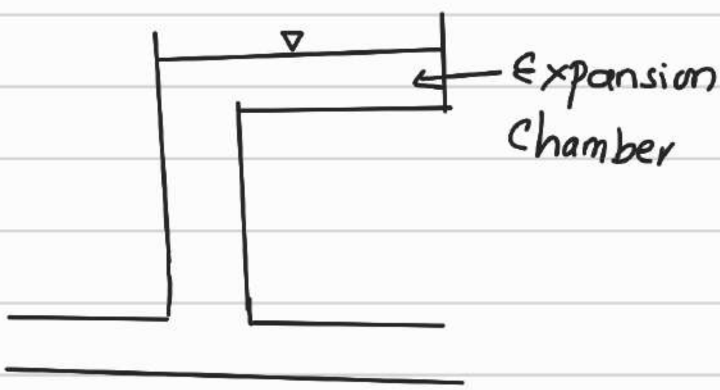
4. Surge tank with Spilling Chamber

* In this type surge tank, water is allowed to spill during upsurge.



5. Surge tank with Expansion Chamber:

* The surge tank with expansion chamber is used to provide required water during sudden start of turbine.

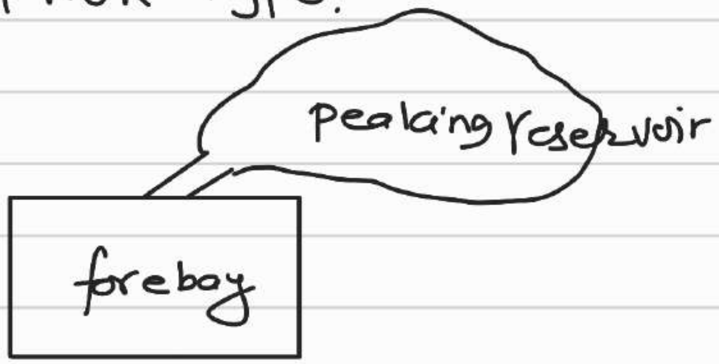


Forebay:

* It is a pond, which acts as an structure to relieve water hammer pressure and also works as a head pond for supplying water to penstock pipes.
* location: at junction of headrace canal/pipe and penstock pipe.

Functions:

1. To relieve water hammer pressure.
2. Also works as secondary settling basin.
3. It also works as transition from open channel flow in canal to pressure flow in penstock pipe.
4. It acts as a storage to provide water for sudden start of turbine.
5. It can be connected with peaking reservoir if the plant is PROR type.



Design of Forebay:

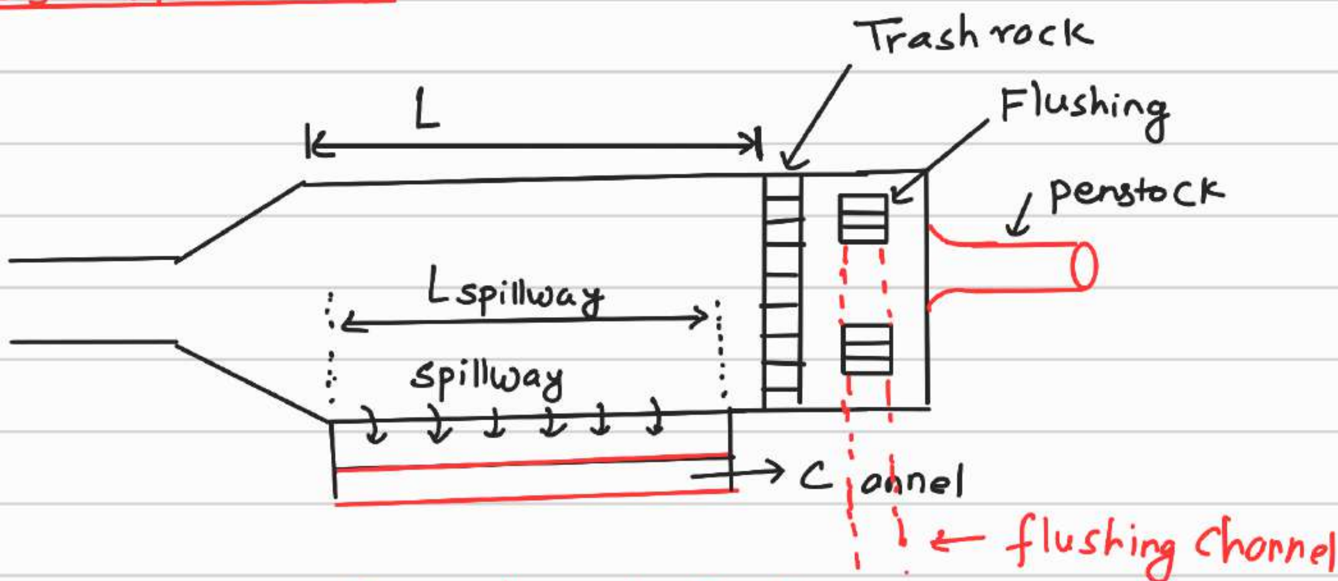
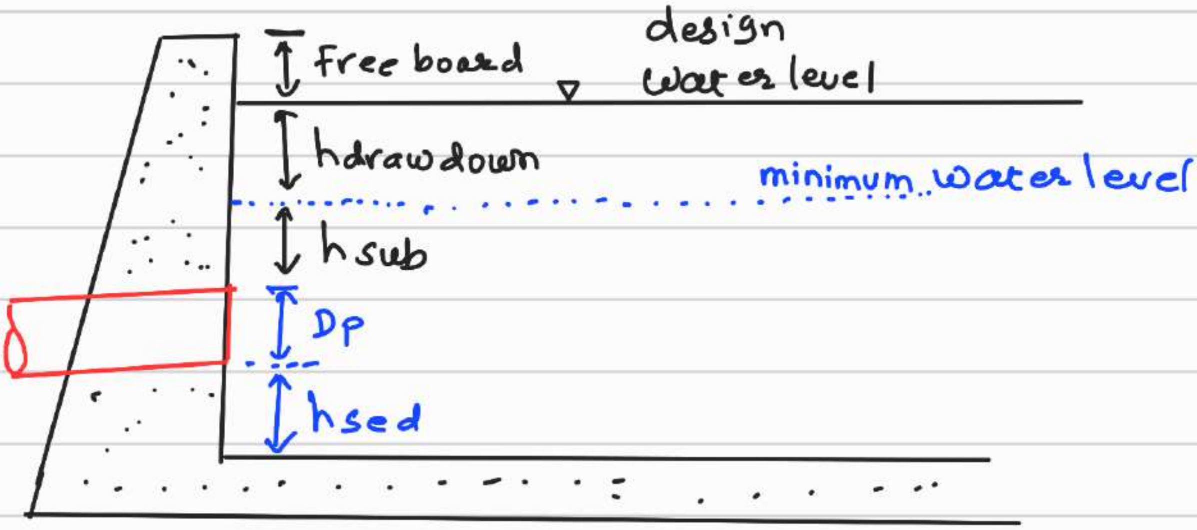


Fig: plan of forebay.



Design steps:

1. Forebay size

* The forebay size should be enough to provide required discharge to operate the plant for 2 to 3 minutes.

Volume of forebay; $= 2 * Q_{turbine} * t$

where; $Q_{turbine}$ = design discharge of turbine

t = detention time.

2. Velocity of flow:

* Velocity of flow = 0.2 to 0.8 m/sec

3. Submergence:

* Submergence; $h_{sub} = \max \begin{cases} 0.5 V_p \sqrt{D_p} \\ 1.5 * \frac{V_p^2}{2g} \end{cases}$

V_p = velocity of flow in penstock

D_p = diameter of penstock.

4. Trash rack:

* Velocity : around 1 m/sec
* slope : 3V : 1 H.

5. Downsurge

$$h_{downsurge} = v * \sqrt{\frac{L}{g} * \frac{A_p}{A_F}}$$

* L = length of penstock.
* v = velocity in fore bay
* A_p = sectional area of forebay
* A_F = surface area of forebay.

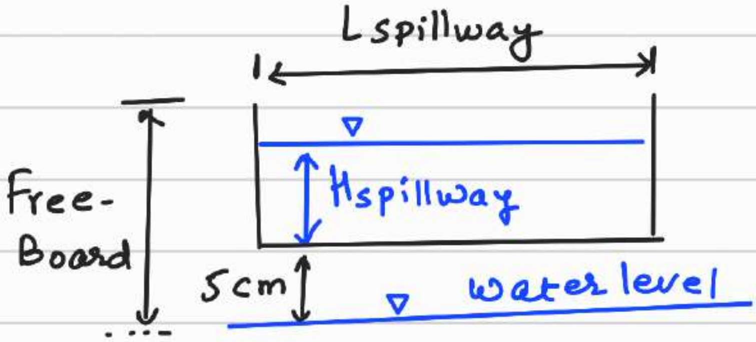
⇒ drawdown heigh should be more than down surge.

6. Spillway:

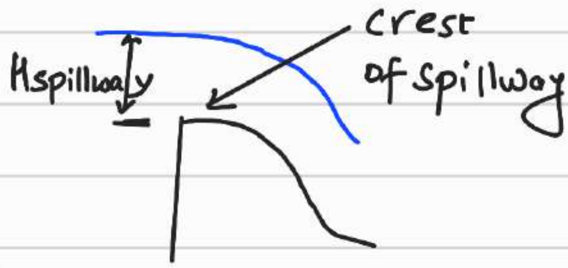
* $Q_{spillway} = C_d * L_{spillway} * H_{spillway}^{3/2}$

$L_{spillway}$ = length of spillway

$H_{spillway}$ = Head over spillway crest $= \dfrac{\text{free board}}{2}$



C_d = discharge coeff. of spillway



* length of spillway < total length of forebay.

Total height of forebay (water portion)
$= h_{sed} + D_p + h_{sub} + h_{drawdown}.$

h_{sed} — min 300 mm

D_p — Diameter of penstock

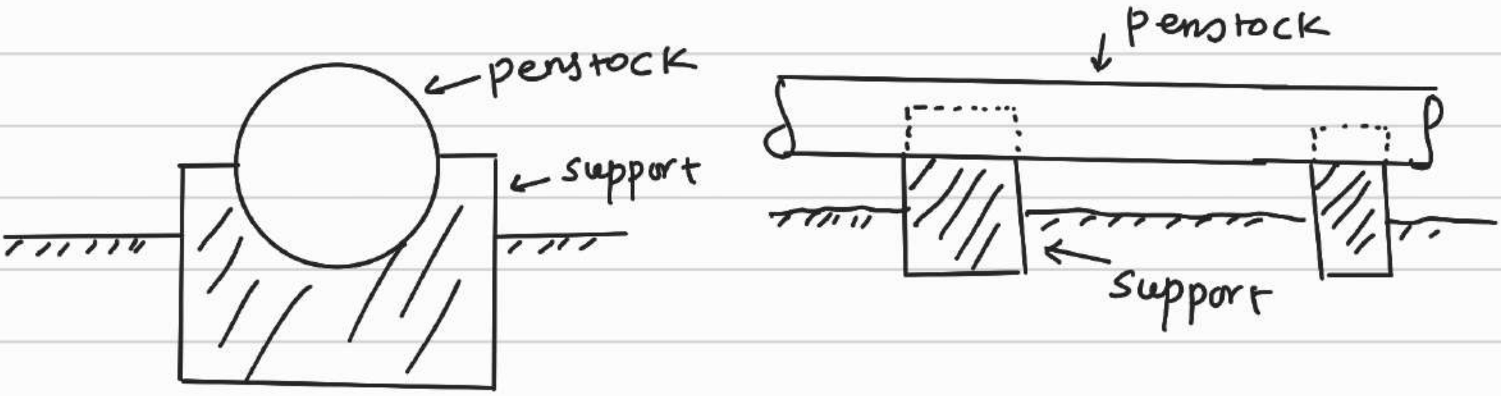
Total heigh of forebay including free board
$= h_{sed} + D_p + h_{sub} + h_{drawdown} + \text{free board}$

Penstock Pipe:

* It is a conduit which carries discharge from surge tank or fore bay to the turbines.
* Material: steel, HDPE, concrete

Types of Penstock Based on Installation

1. exposed penstock



* The penstock is laid above ground.

Advantages:

1. low cost
2. Easy for inspection and maintenance.

Disadvantages:

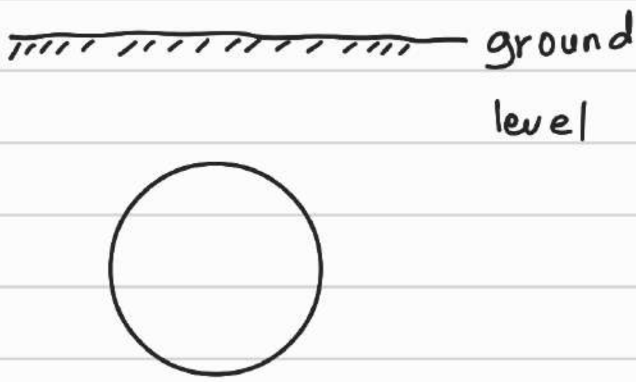
1. Unsafe due to surface instabilities.
2. Need of expansion joints.

2. **Burried Penstock:**

* Penstock is burried below Ground level

Advantages:

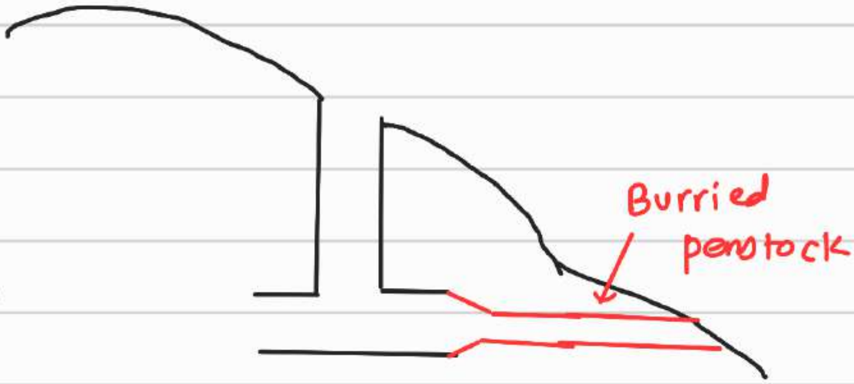
* Relatively safe from surface instabilities.
* No need to provide anchor blocks, support



piers.

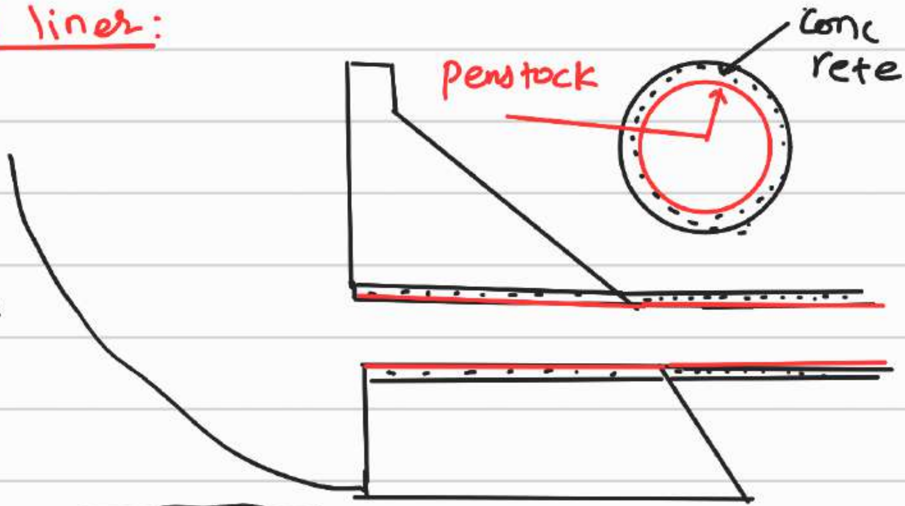
Disadvantages:

* Difficult for inspection and maintenance.
* Costly in rocky terrain.

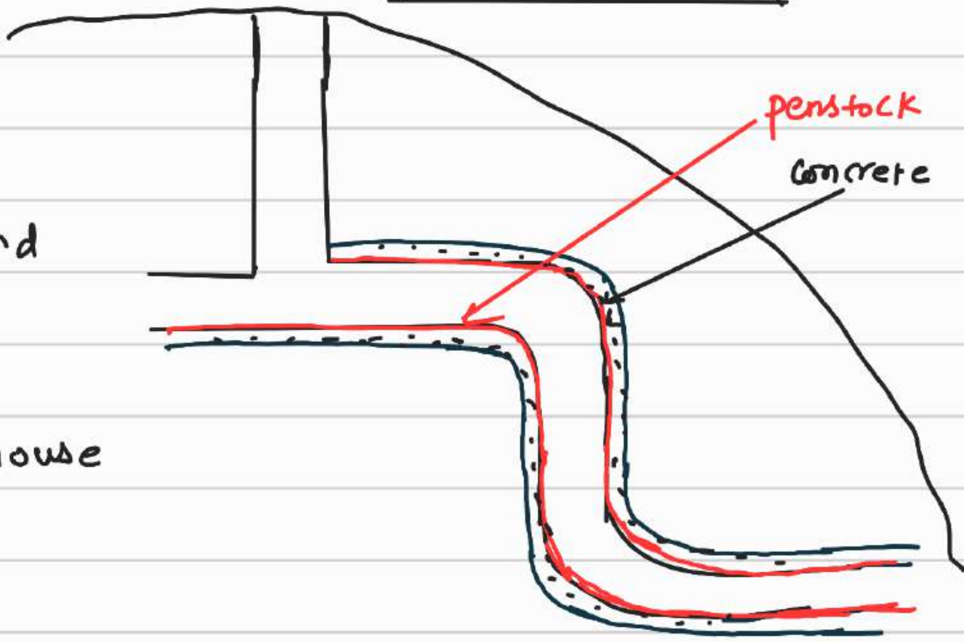


3. Penstock with tunnel liner:

* Such type of penstock is embeded into tunnel and outside the penstock, concrete lining is provided.



* This type of penstock is provided if the penstock is to be laid underground or in case of penstock in storage projects with power house at dam toe.



Design of Penstock:

a. Diameter of Penstock:

* Diameter of penstock shall be economical.

b. Thickness of penstock:

$$t = \frac{PD}{2\sigma_a * \eta_j} + \varepsilon$$

[Hoop stress, $\sigma = \frac{PD}{2t}$

$$t = \frac{PD}{2\sigma}$$

P = internal pressure = Static + dynamic/water hammer.

Water hammer pressure = 20 to 30% of static pressure.

σ_a = allowable hoop stress

η_j = efficiency of joint.

ε = corrosion allowance = 2mm

<u>ASME formula:</u>

$$t = \frac{PR}{\sigma_a * \eta_j - 0.6 * P} + \varepsilon$$

$$\left[t = \frac{PD}{2\sigma} = \frac{P*2R}{2\sigma} = \frac{PR}{\sigma} \right.$$

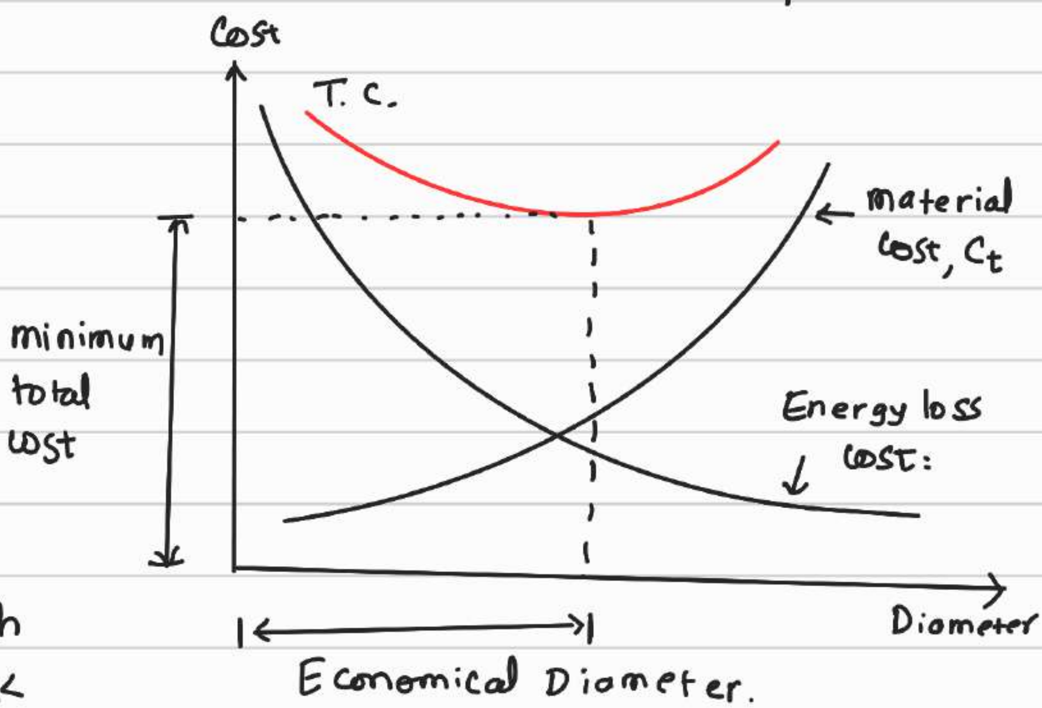
P = internal pressure → same as above

σ_a = allowable hoop stress

η_j = joint efficiency

ε = corrosion allowance = 1.5mm.

Q. How do you estimate economical Diameter of Penstock?



Economic Diameter:

* When diameter of penstock increases, the material cost also increases. However, the head loss decreases with increase in penstock diameter. So, power loss and energy loss decreases and energy loss cost also decreases.
* So, the diameter at which total cost, i.e; sum of material cost and energy loss cost is minimum

is the economical diameter.

Procedure:

1. consider different penstock diameters.
2. Calculate thickness of penstock:

$$t = \frac{Pd}{2\sigma}$$

3. Calculate weight of penstock per meter.

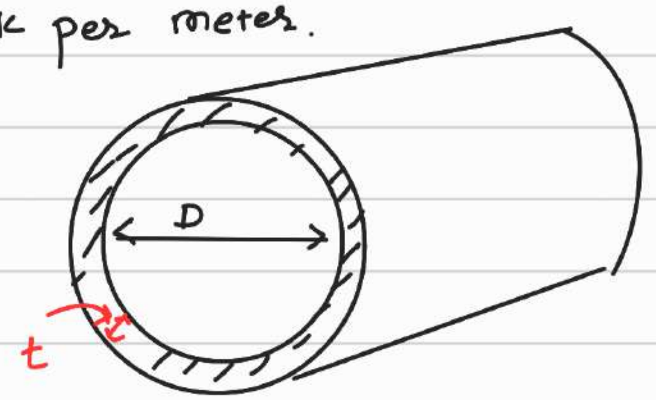
Volume = $\pi D * t *$ Length

$= \pi D * t$

mass of penstock

= volume * density

$= \pi D * t * \rho$



4. Calculate cost of material:

$C_t' =$ mass * rate per kg

* This cost is in present worth.

5. Calculate head loss; $h_f = \frac{8 f l Q^2}{\pi^2 g D^5}$

6. Calculate power loss; $P = \eta \gamma Q h_f$
7. Calculate energy loss; E = power loss * time.
8. Calculate cost of energy loss;

$C_e =$ Energy loss * Energy price ← Annual worth.

9. Convert initial penstock cost into annual worth.

$$\left[\begin{aligned} F &= \frac{A * [(1+i)^n - 1]}{i} \\ P &= \frac{A * [(1+i)^n - 1]}{i * (1+i)^n} \end{aligned}\right]$$

$P \rightarrow C_t'$, $A \rightarrow C_t$

$$C_t = \frac{C_t' * i * (1+i)^n}{(1+i)^n - 1}$$

$$\Rightarrow A = \frac{P * i * (1+i)^n}{(1+i)^n - 1}$$

10. Calculate total cost, T.C. for each diameter.

T.C. $= C_t + C_e$

11. The diameter for

which total cost, T.C. is minimum is economic diameter.

Q. The static pressure in a penstock is 17 kg/cm². The diameter of pipe is 1.2 m and allowable hoop stress is 1326 kg/cm². Calculate the thickness of pipe. Assume joint efficiency 85%.

Soln: Static pressure = 17 kg/cm2

$$P_o = \frac{17 * 10\,N}{10^{-4}\,m^2} = 17 * 10^5\ N/m^2$$

dynamic pressure: Assume 20 to 30%.

Take; P_d = 20% of P_o

$= 0.2 * 17 * 10^5\ N/m^2$

$\therefore P = P_o + P_d = 17 * 10^5 (1 + 0.2)$

$= 20.4 * 10^5\ N/m^2$

Allowable hoop stress, $\sigma_a = 1326\ kg/cm^2$

$$= \frac{1326 * 10\,N}{10^{-4}\,m^2}$$

$= 1326 * 10^5\ N/m^2$

$\therefore$ thickness; $t = \dfrac{PD}{2\sigma_a * \eta_j}$

$$= \frac{20.4 * 10^5 * 1.2}{2 * 1326 * 10^5 * 0.85}\ m$$

$= 10.86\ mm$

Adding corrosion allowance of 2mm;

$t = 10.86 + 2 = 12.86\ mm.$

ASME formula: $t = \dfrac{PR}{\sigma_a \eta_j - 0.6 * P}$; R = Radius

$$= \frac{20.4 * 10^5 * 0.6}{1326 * 10^5 * 0.85 - 0.6 * 20.4 * 10^5}$$

$$= 10.97 mm$$

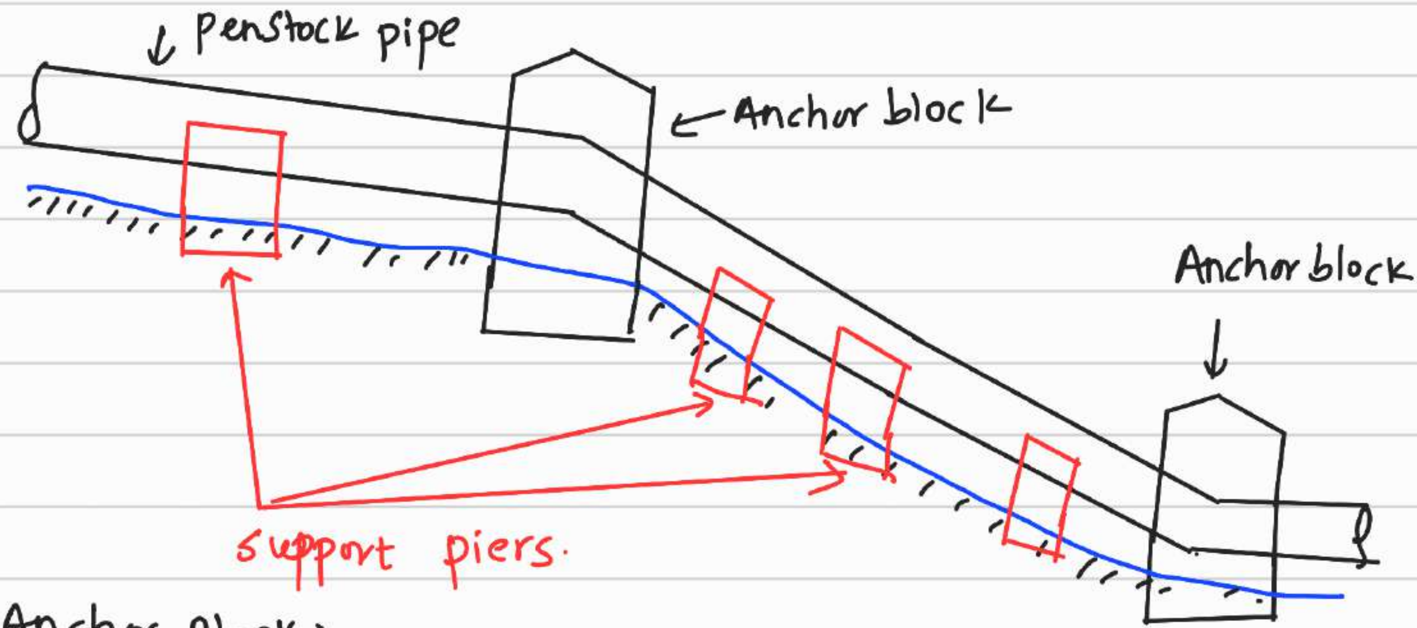
Adding corrosion allowance of 1.5 mm,

$t = 10.97 + 1.5$

$= 12.47 mm$

$\Rightarrow$ So; take thickness; $t = 12.86 mm$

Anchor Blocks and saddle supports:

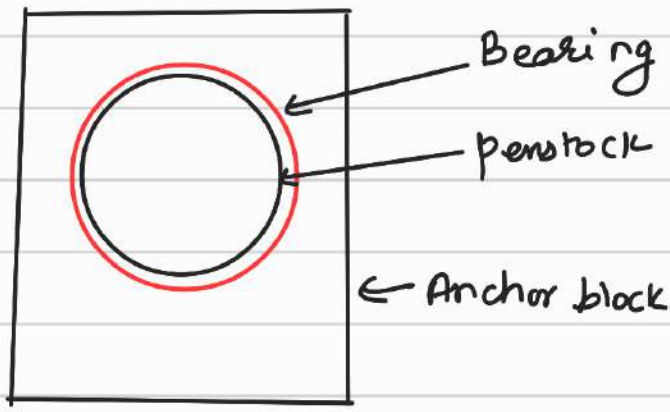


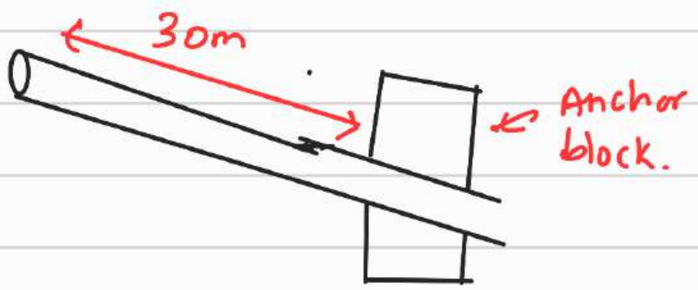
Anchor Block:

* It is a thrust block which is provided to restrain the pipe against movement in any direction as well as to resist load of pipe, water and other forces.

Anchor Block is needed:

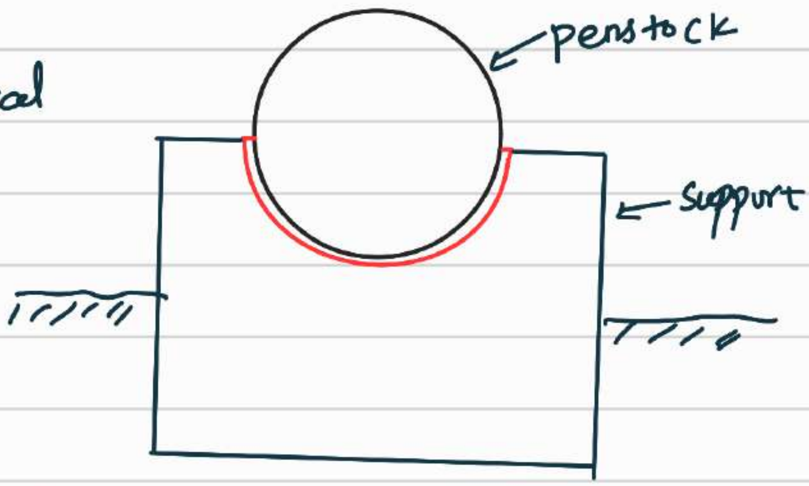
1. At horizontal and vertical bends.
2. Upstream of turbine housing; start of penstock.
3. At straight sections where length exceeds 30 m.





Support Piers:

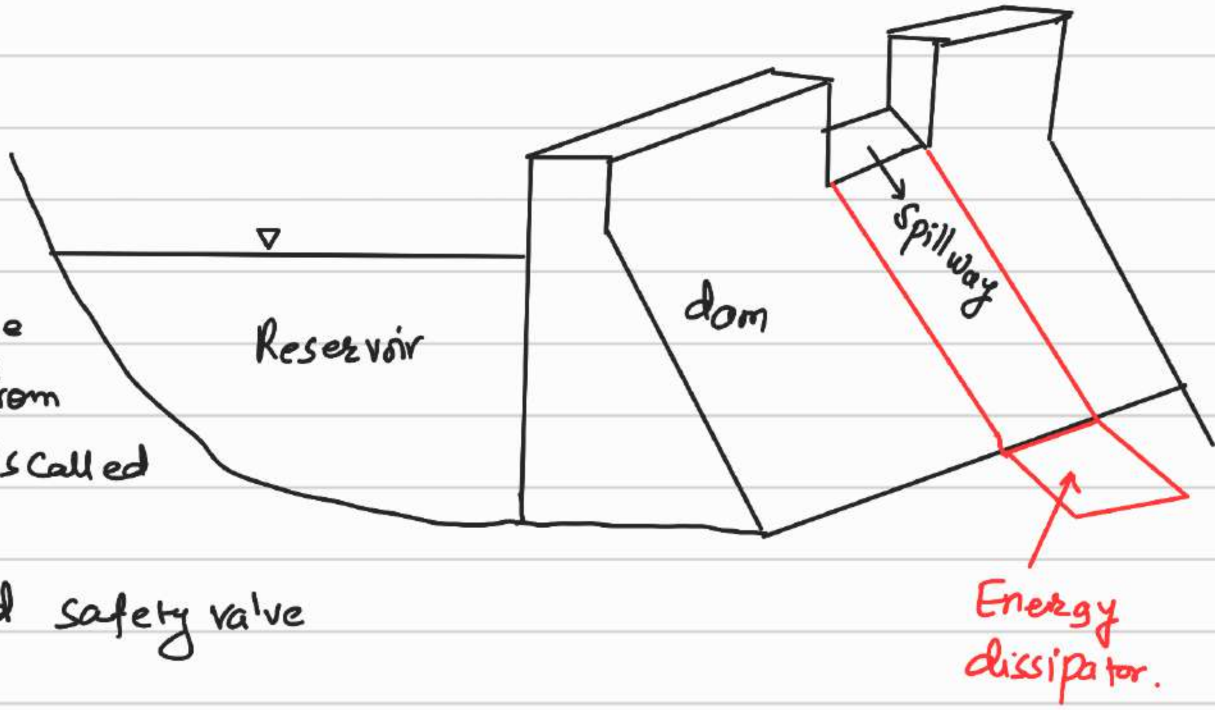
* These are the structural elements which are provided to support exposed penstock pipe and avoid sagging of the pipe.



* Support piers are provided at a spacing of not more than 5m.

Spillway:

* The structure provided to remove/discharge excess water from the reservoir is called spillway.
* Also called safety valve of dam.



Purpose:

1. To remove excess water
2. To avoid innundation on upstream.
   ↳ डुबान
3. To save the dam from failure due to overtopping / due to large hydrostatic forces.
   (overtopping ↳ overflow)
4. To dissipate the energy of water and save downstream from erosion.

Types of Spillway:

1. Based on Function:

a. main spillway: → The spillway which operates during all flooding conditions.

→ To pass design flood.

$Q = 10{,}000\ m^3/sec$

$Q_{main} = 8000\ m^3/sec.$

$Q_{auxilliary} = 2000\ m^3/sec.$

b. Auxilliary Spillway:

* This spillway is provided in addition to main spillway which removes flood discharge in excess of main spillway discharge.

c. Emergency Spillway:

* When water level in reservoir exceeds certain limit, this spillway comes in operation to remove excess flood.

2. Based on Control:

a. Controlled spillway:

* This is gated spillway.

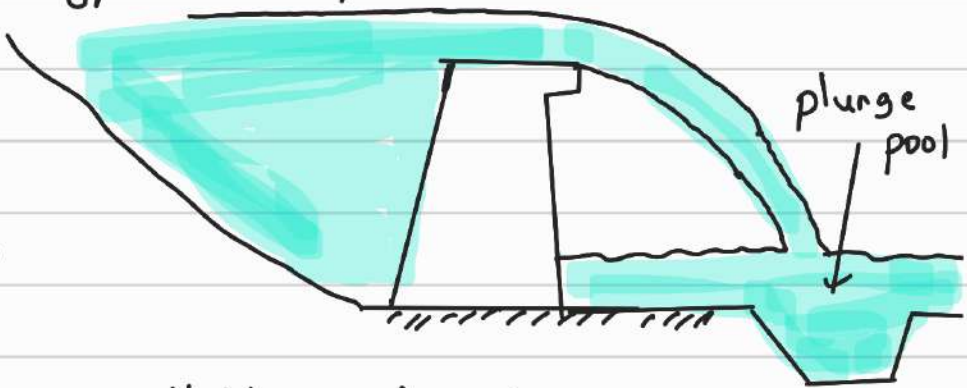
b. Uncontrolled spillway:

* Ungated spillway.
* Crest level of spillway is at Full Reservoir Level.

3. Based on Prominent Features:

a. Straight drop Spillway/Vertical Drop spillway

* Water drops vertically in such spillways.
* Generally suitable for low height dams.

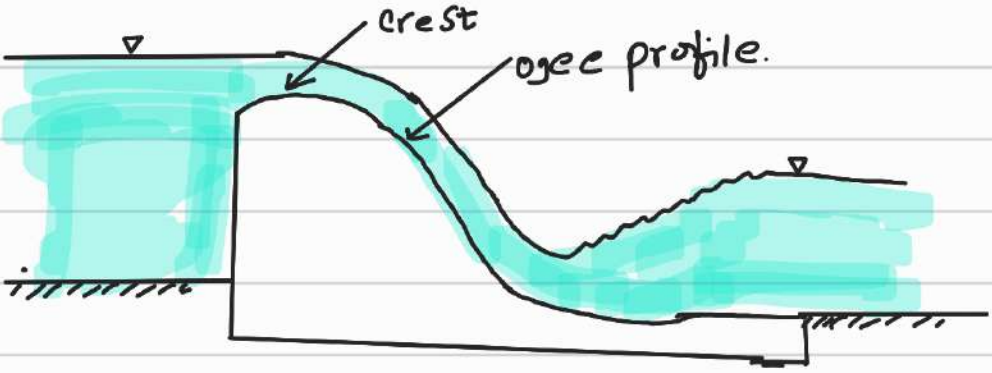


* If good rock is not available at downstream, plunge pool shall be provided.

b. Ogee Spillway:

* The profile of spillway is ogee shaped.

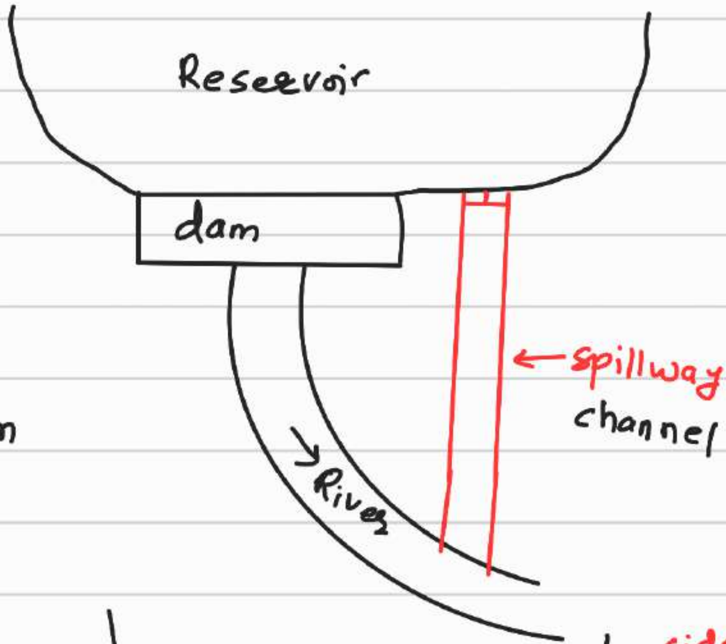
* The ogee profile is provided so that the nappe of water glides over it and remains in contact with spillway profile.



* Suitable for concrete gravity dams.

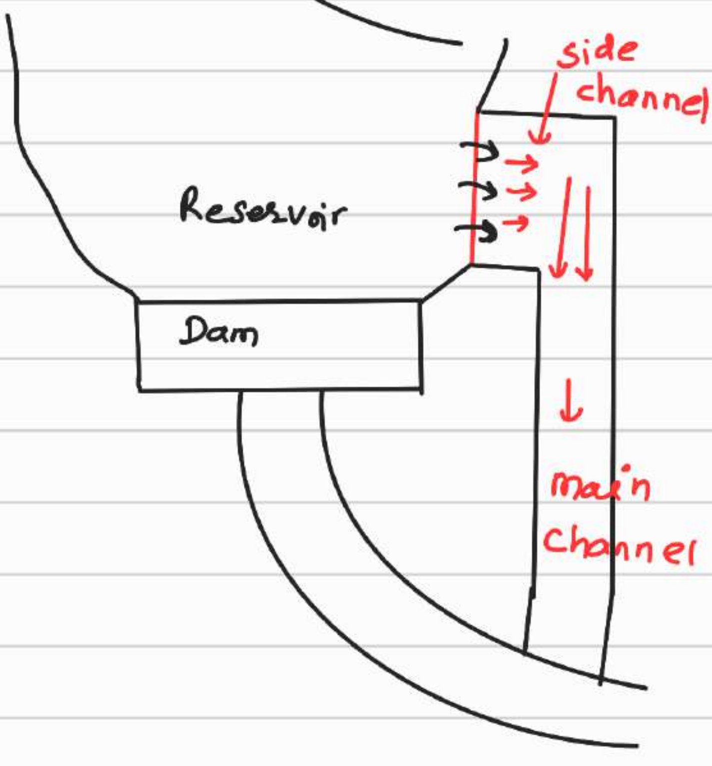
3. chute spillway:

* This type of spillway has straight profile.
* Generally preferred in embankment dams to reduce cost of excavation

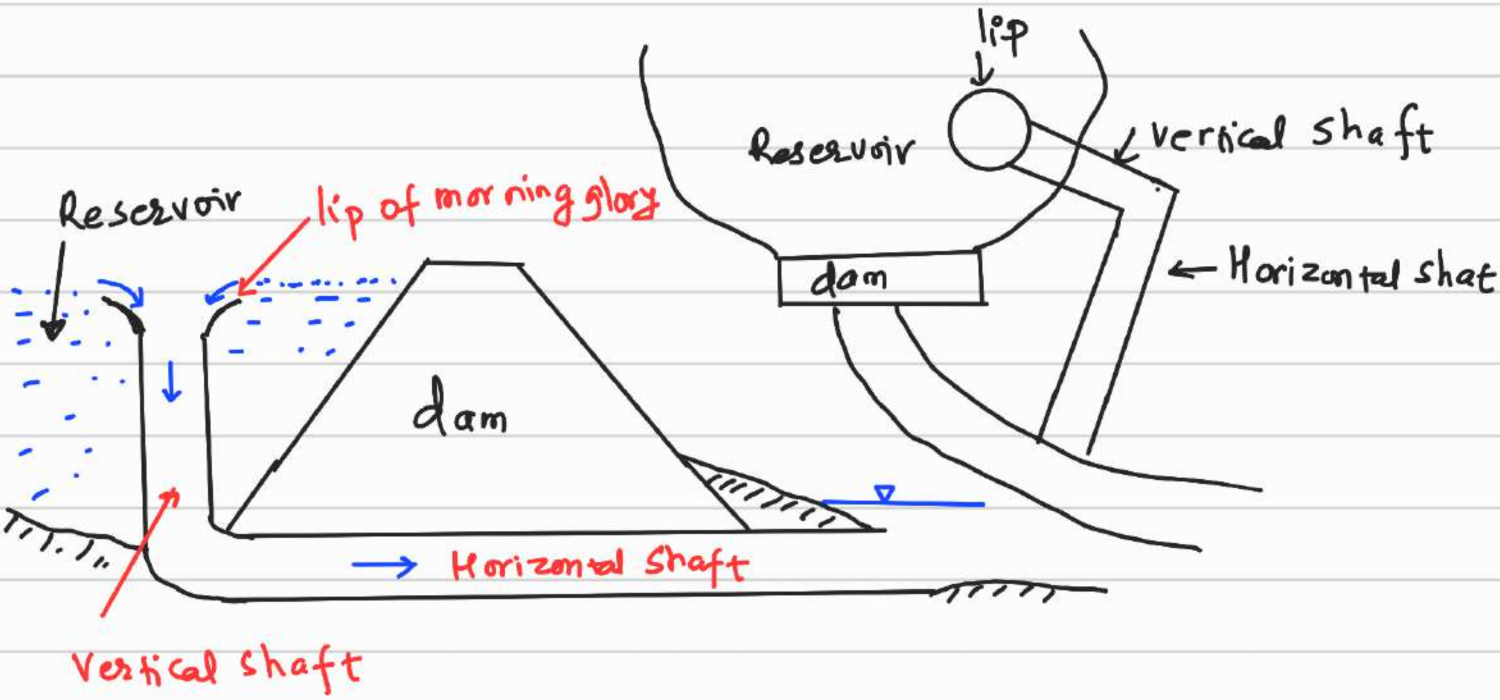


4. side channel spillway:

* The water spills on the side channel from spillway crest, from where it turns 90° and then flows in the main channel.



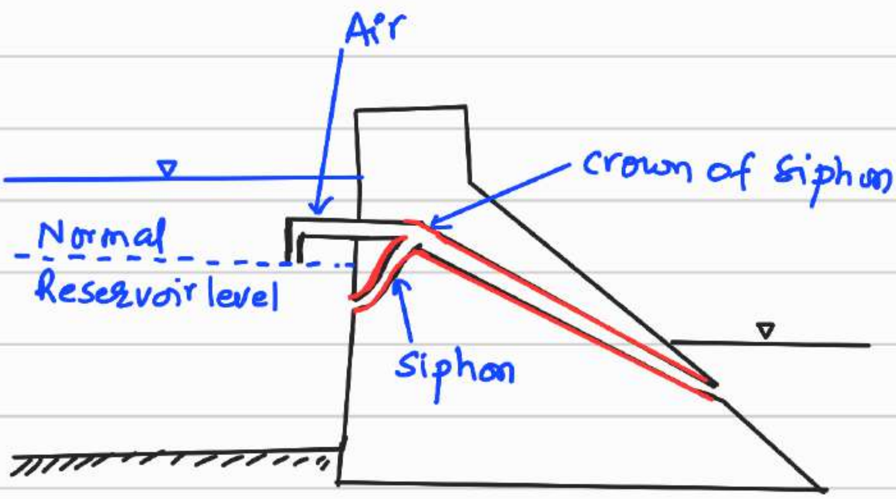
5. shaft spillway:



* In shaft spillway/morning glory spillway, water spills from lips and drops into vertical shaft, which then moves into horizontal shaft and water is finally discharged to downstream.
* Suitable when space is not available for construction of other spillways.

5. Siphon Spillway:

* If discharge to be removed is small and space is limited, this type of spillway is suitable.



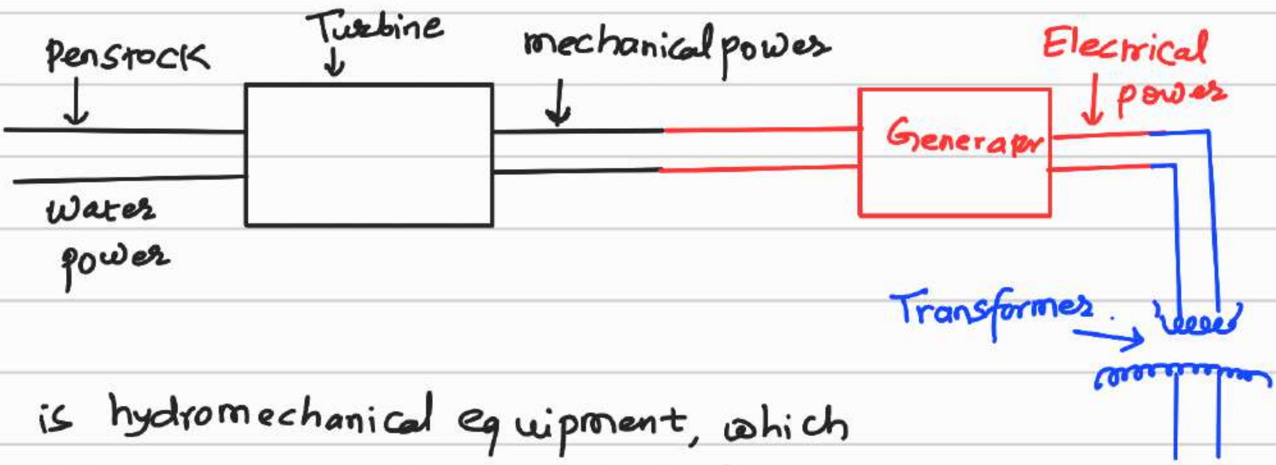
* When water level goes above normal reservoir level, the siphon comes into action and flood is discharged.

Hydromechanical and Electromechanical Equipment in Power House:

* In the powerhouse, following hydromechanical and electro-mechanical equipment exist.

1. Turbine
2. Valves
3. Pumps
4. Generator
5. Govenor
6. Protection systems.

Turbine:



* Turbine is hydromechanical equipment, which converts water power into mechanical power.

[Working principle of pelton, francis, Kaplan Turbine
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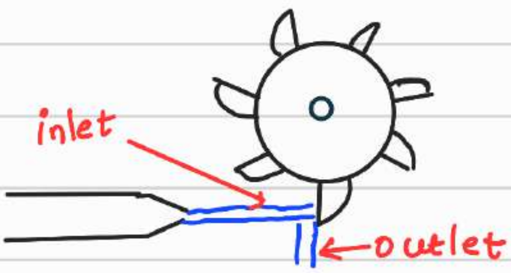
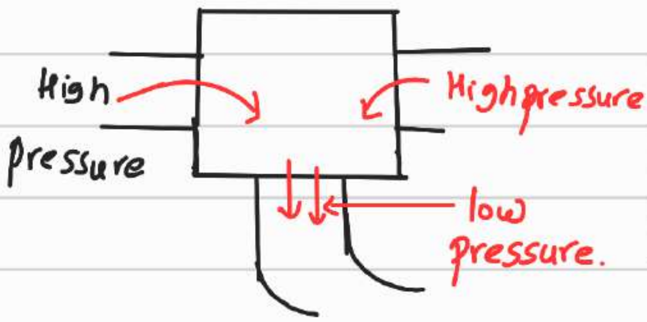
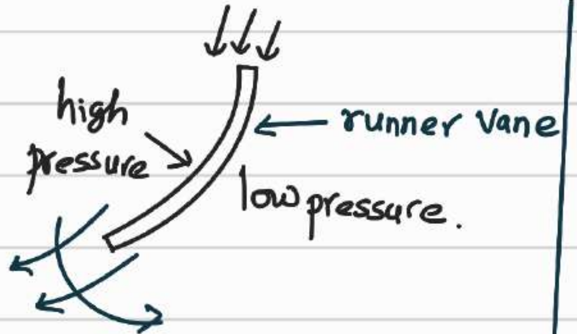
Types of Turbine.

a. Based on type of Energy at inlet of turbine

Impulse Turbine	Reaction Turbine
	



1. The water is at atmospheric pressure at the inlet and outlet of turbine	1. water enters turbine at high pressure and leaves at low pressure
2. It gains energy by changing kinetic energy of water	2. It gains energy by changing pressure of water.
3. Suitable for high head and low discharge.	3. Suitable for low to medium head and high discharge.
4. Installed above tailrace.	4. Generally installed below tailrace. Can be installed above or below tailrace.
5. Blades are in action only when in front of water,	5. Blades are in action all times.
6. less succeptible to erosion by sediment.	6. more succeptible to erosion by sediment.

b. Based on specific speed: (N_s)

* $N_s = \dfrac{N * P^{1/2}}{H^{5/4}}$; N = runner speed, R.P.M.
P = Power output
H = Head.

When $P = 1HP$ and $H = 1m$, $N_s = N$.

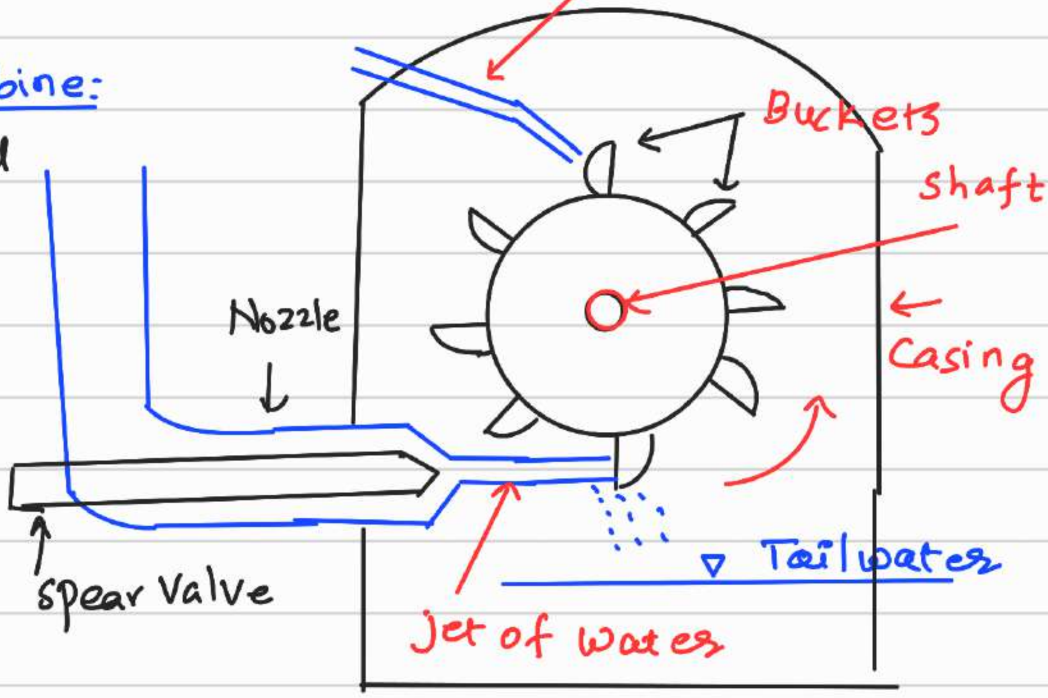
* So, specific speed is the speed of turbine at which it would generate 1HP power at 1m head.

Pelton: low specific speed
Francis: medium specific speed.

Kaplan: High specific speed.

1. Pelton Turbine:

* It is tangential flow impulse turbine which is suitable for high head and low discharge.

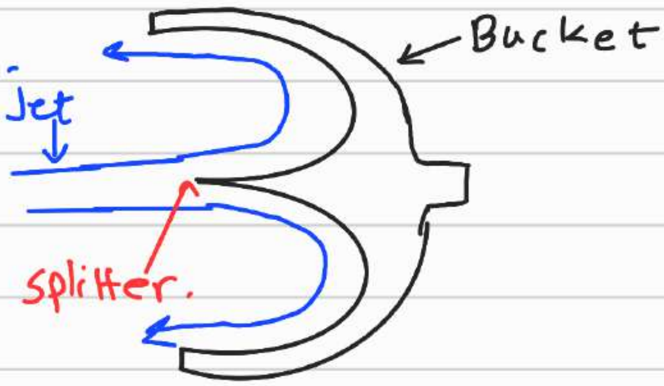


Parts of pelton Turbine

1. Runner:

* Runner of pelton turbine consists of a disc on which number of buckets are provided.

* Buckets are semi-spherical cups divided into two lobes by the spliter, which deflect water jet by almost 180°



2. Nozzle:

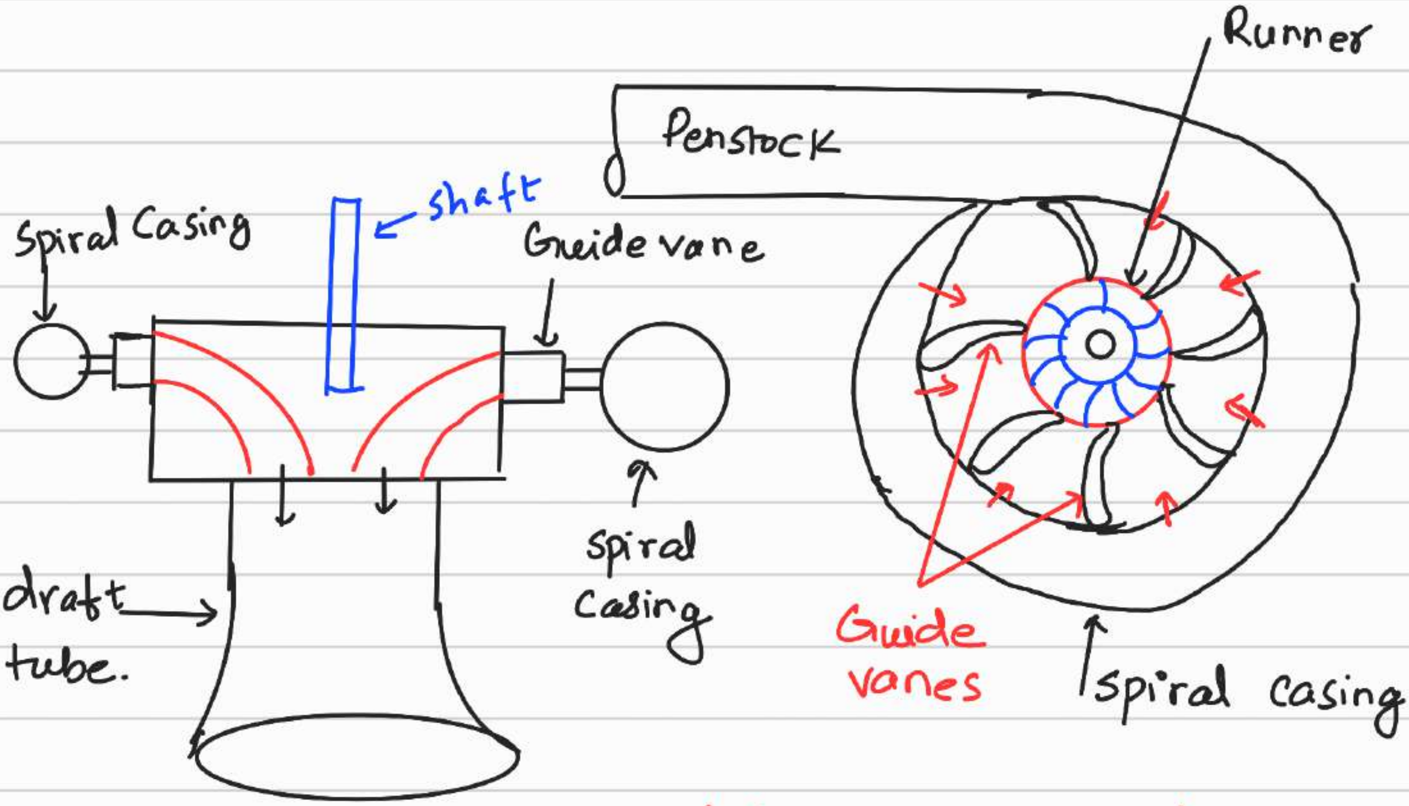
* It directs flow towards the buckets. Spear valve is also provided with nozzle, which controls flow through nozzle.

3. Casing:

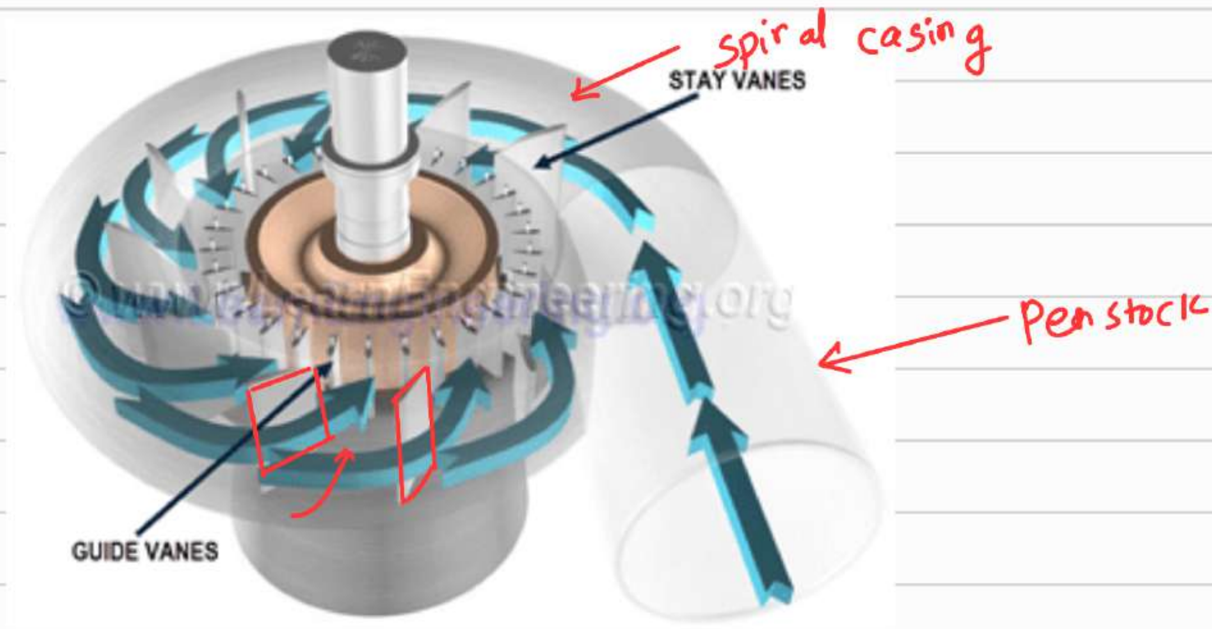
* To prevent splashing of water.

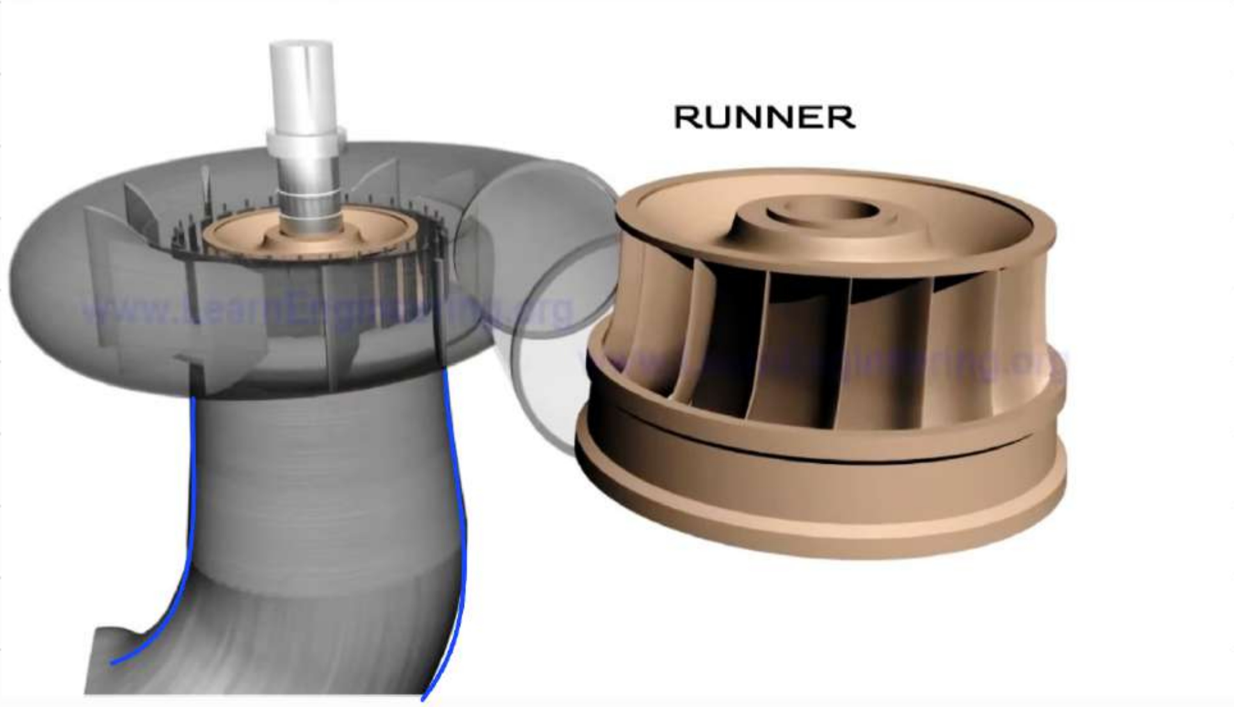
4. Braking jet * It is directed at the back of buckets to make the runner come at rest.

2. Francis Turbine:



* It is mixed flow (radial inflow and axial outflow) reaction turbine which is suitable for low to medium head and medium to high discharge.
* Components:





1. Spiral Casing:

* It is the part of turbine, which takes flow from penstock and supplies to runner via stay vanes.
* The area of spiral casing is decreasing.

2. Guide Mechanism / Guide Vane:

* They guide water from stay vane of spiral casing to runner.

3. Runner:

* It consists of numbers of curved vanes through which water enters radially and leaves axially.

4. Draft Tube:

* It conveys water from outlet of runner to tailrace.

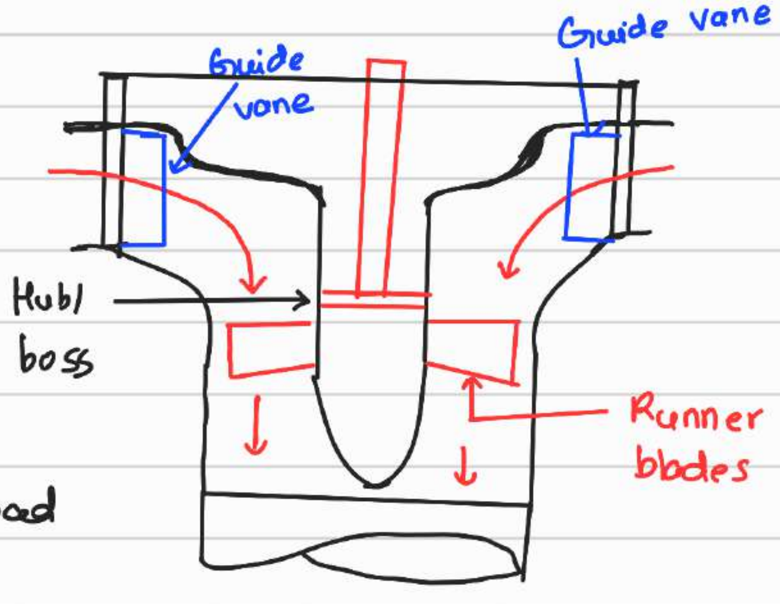
→ Water from penstock enters the spiral casing.

→ From spiral casing water leaves via stay vanes, which is guided towards runner by guide vanes.

→ Water enters runner radially and leaves axially.

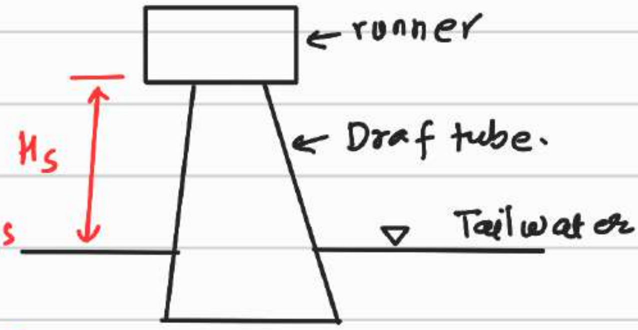
3. Kaplan Turbine:

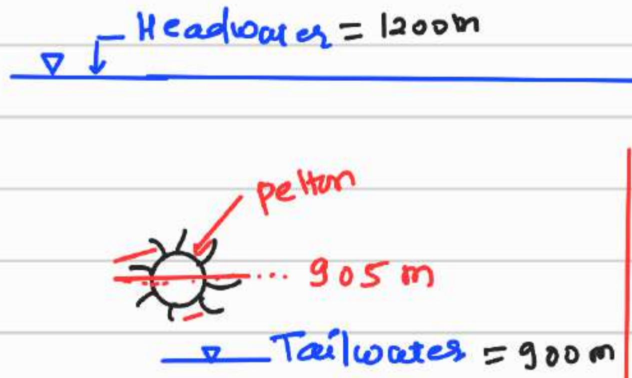
* It is axial flow reaction turbine, which is suitable for low head and high discharge
* It is adjustable blade propeller turbine, which has good efficiency in part load as well as over load.



1. Sprilar Casing: Same as francis turbine.
2. Guide vane mechanism:- It guides water to runner at optimum angle of attack.
3. Runner: * It consists of runner blades fitted on hub/ boss.
4. Draft tube:- To convey water from outlet of runner to tailrace.

Draft Tube:





Head of pelton = 1200 - 905
= 295m

Head of francis = 1200 - 900
= 300m.

* Draft tube is a device which conveys water from outlet of runner to tailrace.

Functions of Draft:

1. Conveys water from runner outlet to tailrace.

2. It recovers the rejected kinetic energy at outlet of runner into useful pressure energy.

Setting of Draft Tube:

* The pressure at entry of draft tube is negative. However, the pressure should not be such that cavitation occurs at the entry of draft tube.
* So, setting of draft tube is done to avoid cavitation.
* Setting Height, $H_s = H_a - H_v - \sigma_c * H$

  H_a = atmospheric pressure head

  H_v = vapour " "

  σ_c = Thomas cavitation number

  H = Net head.

  If H_s is +ve ⇒ turbine is installed above tailrace.

  If H_s is negative ⇒ " " " below " .

$\sigma_c = 0.625 * \left(\frac{N_s}{444}\right)^2$ for Francis Turbine

$\sigma_c = 0.28 + \frac{1}{7.5} * \left(\frac{N_s}{444}\right)^2$ for fixed blade propeller turbine

$\sigma_c = 1.1 * \left[0.28 + \frac{1}{7.5} * \left(\frac{N_s}{444}\right)^2\right]$ for Adjustable blade propeller or Kaplan turbine.

where; N_s = specific speed of turbine.

$$\boxed{N_s = \frac{N * P^{1/2}}{H^{5/4}} \; ; \; P \text{ is in HP.}}$$

Turbine Selection Criteria:

1. Head:

a. Very High head: Pelton

b. medium to high head = francis

c. low head = Kaplan and propeller

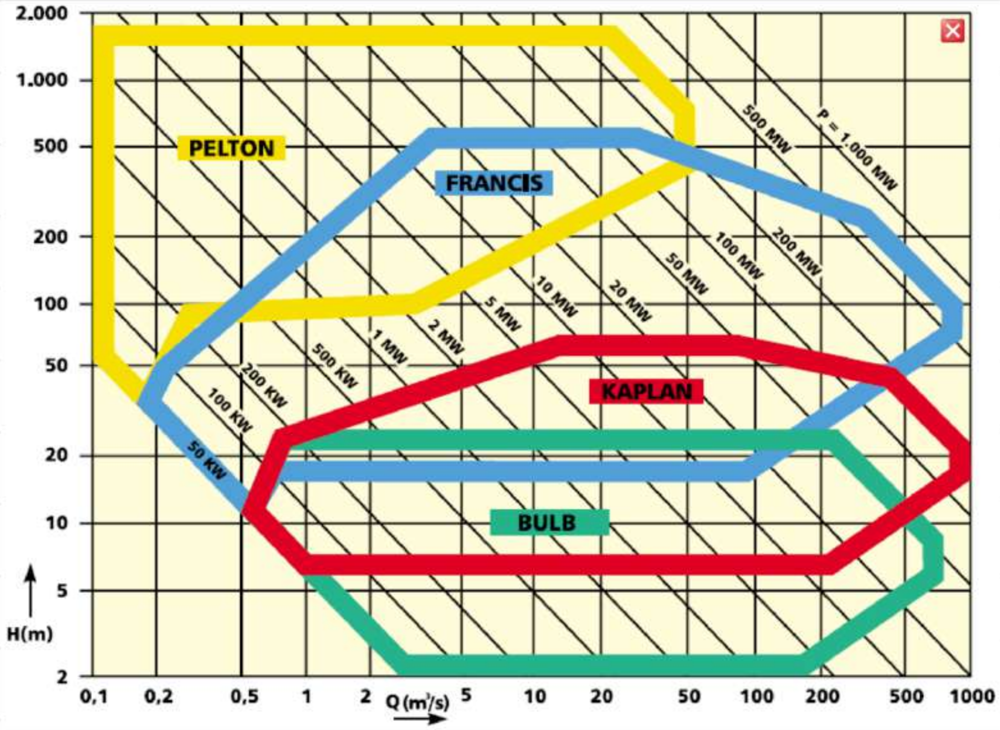


Fig: Turbine Selection Chart

2. Discharge:

a. High discharge: Kaplan

b. medium to high: francis

c. low discharge: pelton

3. Water Quality:

* High sediment concentration: pelton
* low sediment concentration: francis.

4. Discharge Variation:

* If discharge variation is high, kaplan turbine is more efficient, after that pelton, after that francis and least efficient is propeller.

5. Maintenance cost:

6. Specific Speed:

a. High: Kaplan

b. low: Pelton

c. Medium: francis.

Need and Working Principle of Governer:

* Governer is an electromechanical equipment which maintains the speed of runner of turbine, when the load on plant is increased or decreased.
* It also controls flow through the turbine.

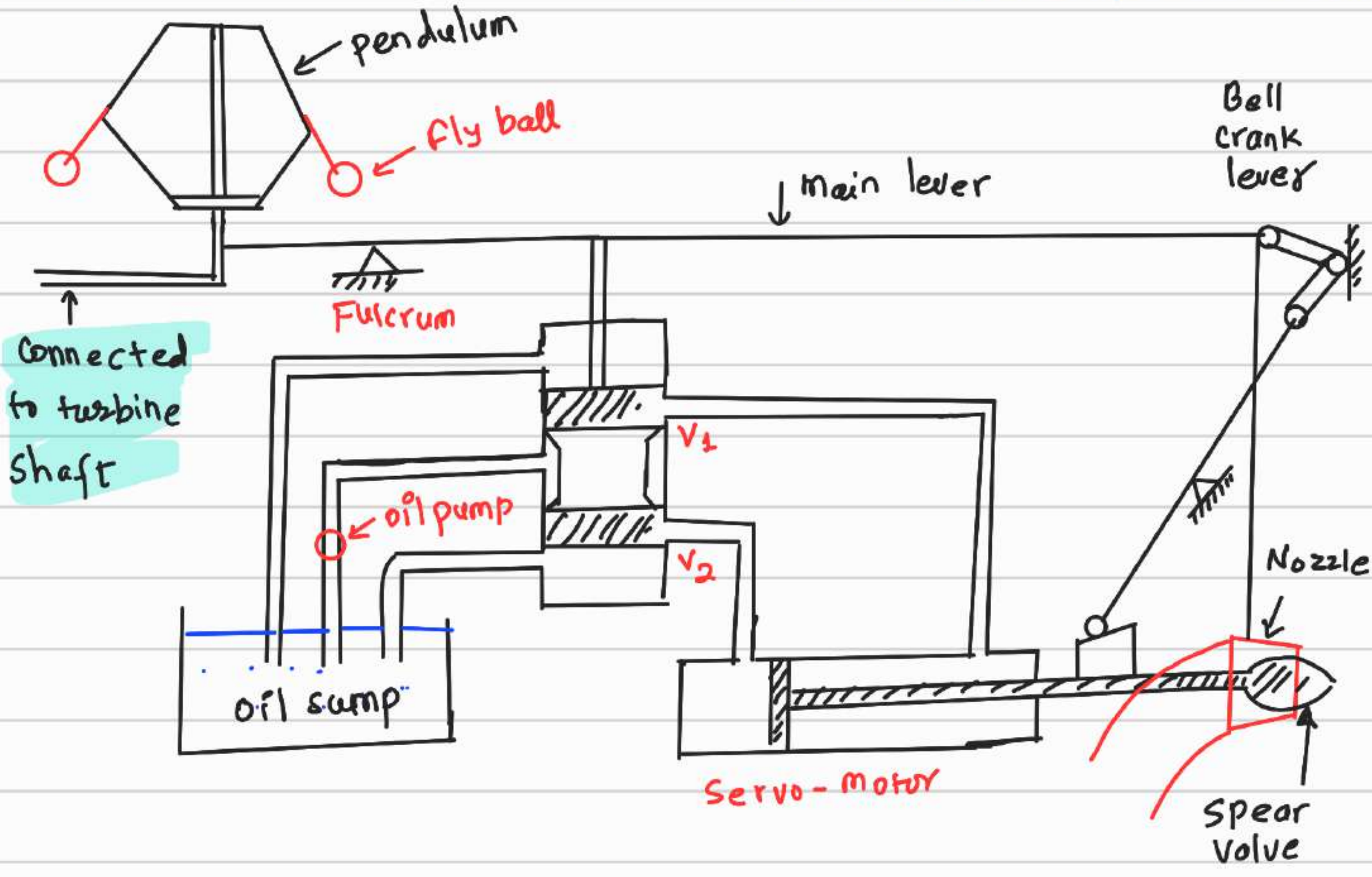
middle Marshyangdi - PROR - 2 units @ 35 MW each.

35MW ⟶ 20MW ⟶ rotation high. ⟶ Governer ⟶ Rotation normal. ⟶ decrease flow

20MW → 35MW → rotation low ⟶ Governer → Rotation normal. ⟶ increase flow.

Working Principle of Governer:

load increase → speed decrease



Power house:

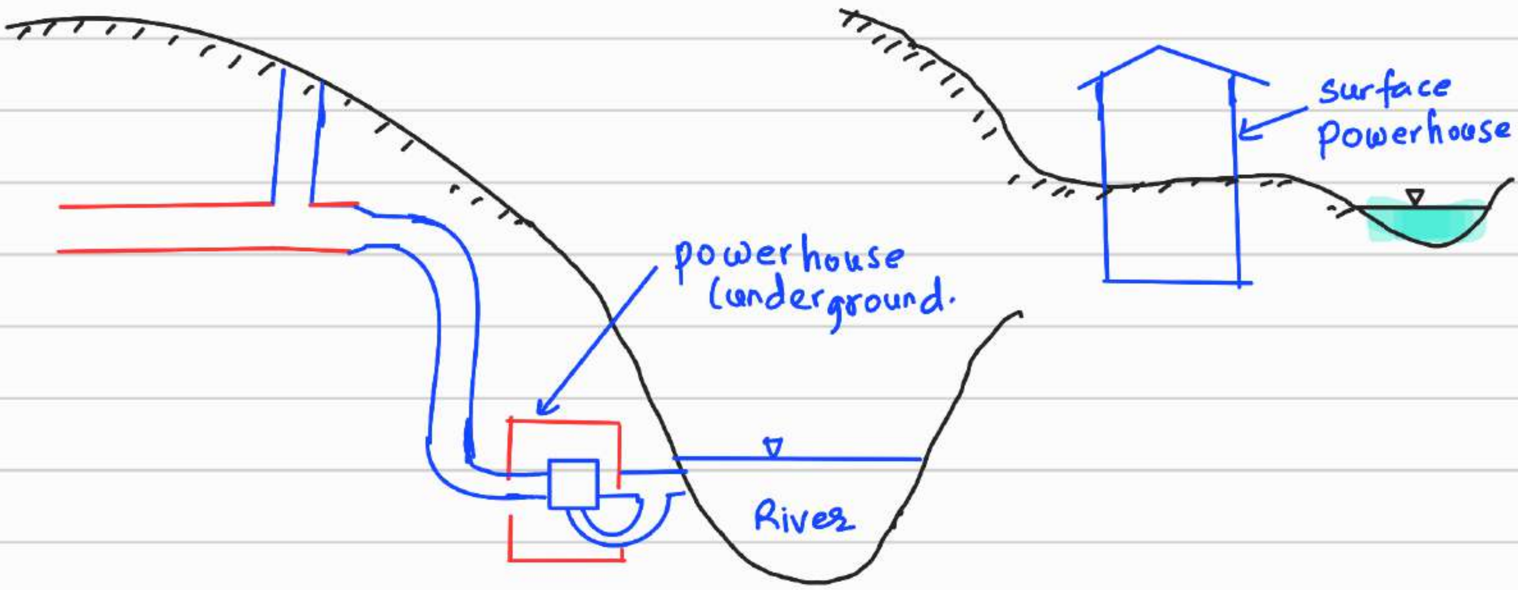
* Power house is a structural complex, where the equipments needed for power generation are arranged.

Surface Power house and Underground Power house:

a. Surface Powerhouse:

* A powerhouse constructed over ground surface.
* For surface powerhouse, sufficient space shall be available.
* If good rock is available at shallow depth, this powerhouse will be highly economical. If not, suitable foundation shall be provided.
* As compared to underground power house, surface powerhouse is economical.

b. Underground Powerhouse:



* If suitable site for surface powerhouse is not available, underground power-house is constructed.
* It is constructed below ground surface without removing overburden by constructing a cavern.

Advantages of Underground Power House:

1. Safer than surface power house against surface

instabilities like landslide, rock fall, etc.

2. Cost of land acquisition can be saved.
3. Anchor blocks and saddle supports for penstock can be avoided.
4. Flexibility in layout can be achieved.
5. If good rock is available, the cost can be reduced.

Note: * For construction of underground powerhouse, good rock shall be available.

* In underground power house, penstock and some/all part of tailrace is also underground.

Structure of Power House: