

PENSTOCKS— These are the pipes that supply water from the head pond or the Forebay to the turbines. These are Pressure Conduit. The design principle for penstocks are the same as for pressure vessels and tanks. The water hammer which are likely to occur suddenly due to governor control and turbine gate operation, have to be considered while designing.

When the distance b/w the Forebay and the powerhouse is short, a separate penstock is preferred for each turbine, while for moderate heads and long distance, a single penstock is used to feed two or more turbines through a special bifurcated pipe.

Classification—

Classified on the basis of —

- a) **Material of Fabrication** — Various material employed are Steel, RCC, AC, wood staves, banded steel and cast steel.
→ Choice of materials depend upon the functions that penstocks have to serve.
 Joints — By welding & riveting.
- b) **Method of Support** — According to this we can distinguish pipes—
 - i) **Buried or Embedded Pipes** — Penstock may be either buried or embedded under the ground or exposed above the ground surface and supported on piers. These pipes are supported on the soil in a trench at a depth of 1-1.5m and after placement, the trench is backfilled.
 - ii) **Exposed Pipes** — constructed above the ground surface and supported by piers. They are ease in inspection for faults and maintenance. Their stability ensured with anchorages.
 - iii) **Partly Buried System** — intermediate b/w the buried and exposed system.
- c) **Rigidity of connections and Support** —
 - i) **Rigid Pipes** (without any connection)
 - ii) **Semi-Rigid Pipes** (with expansion joints)
 - iii) **Flexible or loosely-coupled Pipes** (expansion joints)

Design Criteria—

1. **For Non-embedded Penstocks** — According to American Society of Mechanical Engineers (ASME) code a penstock may be designed under the following conditions—
 - i) **Normal Conditions** — This includes the max. static head + pressure rise due to normal operation. The recommended FOS is 3.0. based on specified min. ultimate tensile strength.
 - ii) **Intermittent Conditions** — This includes conditions during filling and draining the penstock under normal operation. FOS is 2.25.

iii) Emergency condition — This include the governor cushioning stroke to be inoperative and Part gate closure in $24a$ seconds at the max. rate, where L is the conduit length in m, and a is pressure wave velocity in m/s. recommended FS is 1.5

iv) Exceptional conditions — This include malfunctioning of control equipment in the most adverse manner and shall not be used as the basis of design. Precautions must be taken to minimize the probability of Occurrence and effects of the exceptional conditions.

2. For Embedded Penstocks —

The allowable design stresses are calculated depending upon the min. yield stresses and tensile stresses of the corresponding steels. FOS is based on the ultimate tensile strength of the penstocks steels. For commonly used penstock steel, the FOS is taken to be 3.0.

→ For internal pressure, the min. plate thickness is computed using design head for normal conditions

The steel liner is designed to resist a minimum pressure of 5.25 kg/cm^2 with a radial gap.

→ Allowable stress = 0.5 to 0.65 times the yield point.

Wall Thickness of the Penstock —

$$t = \frac{pR}{S\eta - (0.6p)} + 0.15$$

where

t = thickness in cm

p = pressure in kgf/cm^2

R = internal radius in cm

S = design stress in kg/cm^2

η = joint efficiency Factor

0.15 cm is added as allowance for corrosion.

Number of Penstocks — A hydro-power scheme has the following alternative choices of number of Penstocks —

- i) To provide a single penstock for the complete powerhouse.
- ii) To provide as many penstocks as the number of Turbines.
- iii) To provide multiple penstocks but each penstocks supplying at least two turbines.

The Consideration which lead to the choice of number of Penstocks are —

- i) Economy
- ii) Operational safeguard
- iii) Transportation Facilities.

Economical Diameter of Penstock -

c2

→ larger the diameter for a given discharge, smaller will be the head losses and greater will be the net head available to the turbine, resulting in greater power generation.

To choose an optimum size which results in maximum economy -
A size that would give us least annual cost.

Two popular formulae have been proposed by USBR and G Sarkaria for Economical Diameter are as follows -

a) USBR Formula

$$V = 0.125 \sqrt{2gH}$$

Where V = optimum Velocity in m/s
 H = max. Working head in m.

applies to the middle range of heads.

b) Sarkaria Formula -

$$D = 0.62 \frac{P^{0.35}}{H^{0.65}}$$

Where

D = Penstock dia. in m

P = Power transmitted by the pipe in hp

H = max. net head at the end of penstock in m.

expression -

total annual cost, which is mathematical tractable and which on differentiating and equating to zero, yield a condition corresponding to the optimum diameter.

Let C_1 be the annual cost of the energy lost in friction.

C_2 , , , corresponding to any given dia D

The optimum condition then is

$$\frac{d(C_1 + C_2)}{dD} = 0$$

If $C_1 = K_1 D^m$ and $C_2 = K_2 D^n$

(Where K_1 and K_2 are constants)

$$\text{then } \frac{d}{dD} = (K_1 D^m + K_2 D^n) = 0$$

$$\text{or } K_1 m D^{m-1} + K_2 n D^{n-1} = 0$$

$$\frac{K_1 m}{K_2 n} = - \frac{D^{n-1}}{D^{m-1}} = - D^{n-m}$$

$$D = \left(\frac{-K_1 m}{-K_2 n} \right)^{\frac{1}{n-m}}$$

Since the sign of m is always $-ve$, the expression for D will be a $+ve$ value.

The values of k_1, k_2, m , and n can be determined easily for any project, thus yielding the desired value of the diameter D .

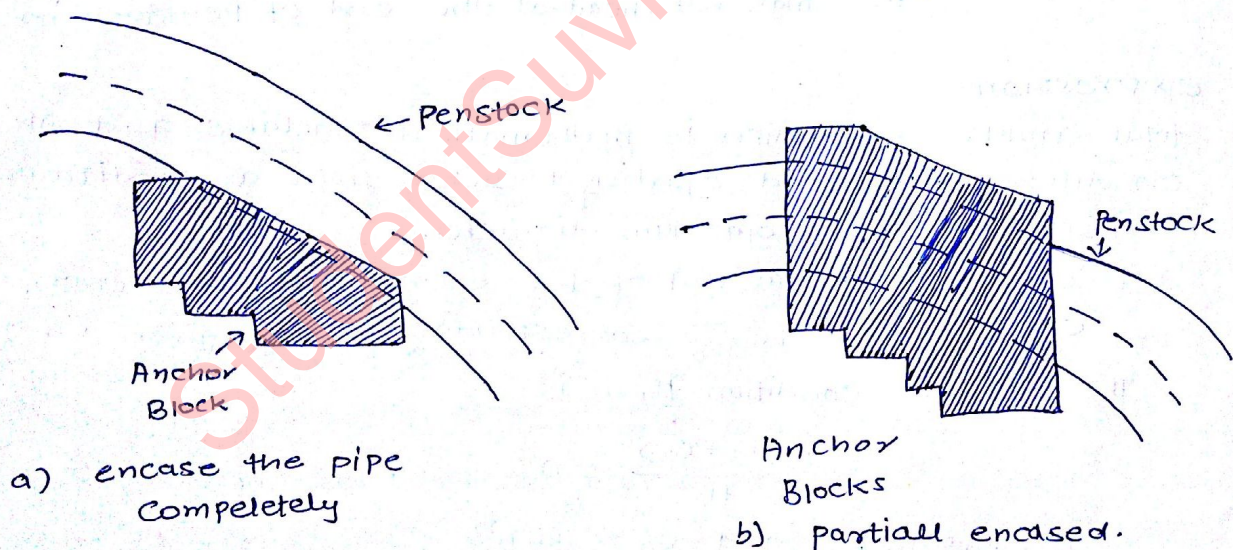
The most economical diameter D , on substituting the values of m and n works out to:

$$D = \left(\frac{5}{2} \frac{k_1}{k_2} \right)^{1/7}$$

ANCHOR BLOCKS — These are massive concrete blocks

encasing the penstock pipe at some intervals in order to anchor down the pipe to the ground securely. Such blocks are necessary at all horizontal and vertical bends of the pipes. Besides the bends, anchor blocks are customarily provided on straight reaches, at intervals of 150 m or so. These blocks help in preventing displacement of the penstocks due to steady or transient forces, including expansion or contraction forces and water hammer pressures. They provide the necessary degree of stability to the pipe assembly by transmitting the penstock load to the ground.

These may encase the pipe completely or partially upto the middle of it. However Complete coverage is better.



Method of Design —

a) **French Method** — In this, the pipe b/w the anchors is treated as a structural member and the anchor blocks are treated as the fixed-end abutments.

b) **Swiss Method** — In this method, the penstock pipe is not treated as a structural member transmitting loads with the beam or Arch Action. All the loads on the Anchor are only those directly coming on the Anchor.

→ More popular method, involving greater cost.
allow use of Flexible Joints.

Stability Conditions-

The basic stability conditions of an Anchor blocks are-

- There must not be any sliding at any section, overturning or rotation, any tension in any part of the concrete and any crushing or failure due to compression in the block.

Forces acting at the points of intersection of the central line of the pipe U/S and d/s of the the anchor blocks are resolved into horizontal and vertical components.

$$\Sigma H = \text{Sum total forces in horizontal direction}$$

$$\Sigma V = \text{Sum total forces in vertical direction}$$

Conditions of Design Stability- The different conditions under which the stability of the anchor block is checked are as follow:

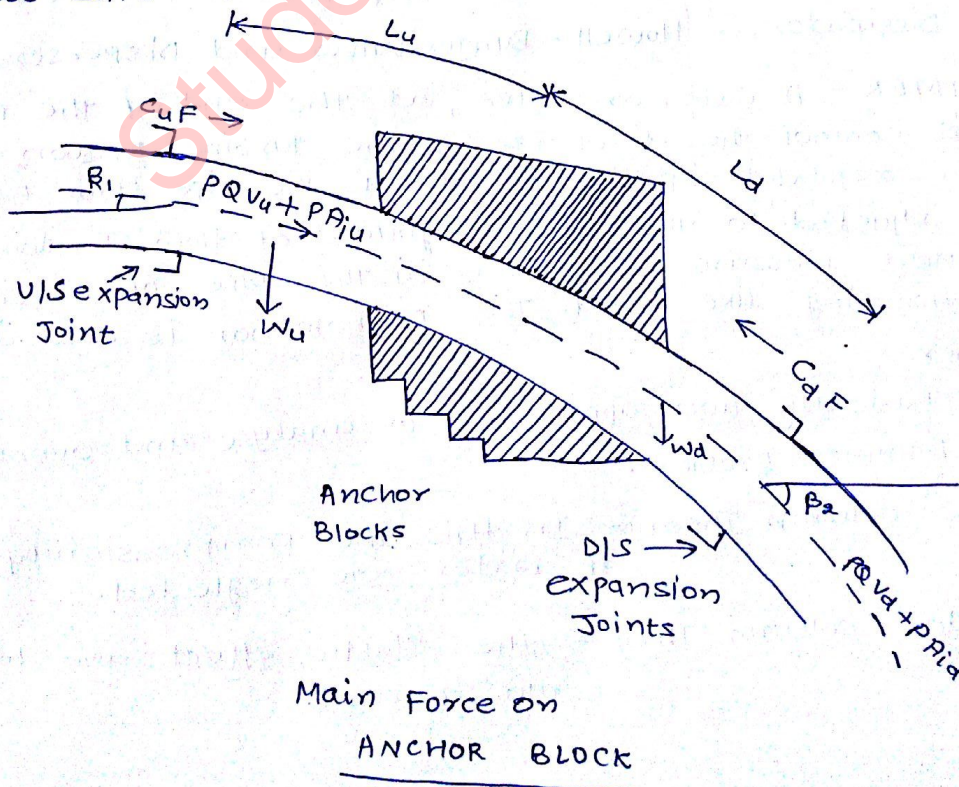
- Pipe flowing at design discharge in expanding and contracting condition.
- Pipe flowing - Transient loads such as due water hammer.
- Pipe empty - both for expanding and contracting conditions.

→ Anchor Blocks for different bends (convex, concave) have different dimensions due to different magnitudes of resultant force, and hence need separate Analysis.

Forces on Anchor Blocks-

All the forces are assumed to act along the axis of the pipeline. When there is a bend, the U/S axis line denotes the direction of forces transmitted from upstream end of the anchor and the downstream axis line denotes the direction of forces transmitted from downstream side of the anchor.

Forces directed towards the Anchor are +ve and vice-versa.



L_u = length from centre of anchor to upstream

L_d = " " " " downstream

W_u, W_d = dead weights

$C_u F, C_d F$ = Friction of the expansion joints on U/S and D/S sides of the anchor.

P = Pressure at any point

A_u, A_d = Area of the exposed end of the pipe at the expansion joint, U/S and D/S of the Anchor

P_{A_u}, P_{A_d} = hydrostatic forces of the anchor.

A_{i_u}, A_{i_d} = inside areas of pipes at U/S and D/S

VALVES — It may be necessary in pressure conduits to

- i) Regulate the Flow
- ii) Completely stop the Flow
- iii) affect energy dissipation under special circumstances.

Valves may be used in Penstocks and in Scouring sluices.

Valves can be placed in

- i) At the beginning of a long Penstock
- ii) At the end of Penstock just before the Penstock joints the scroll casing of the Turbine.

Types of Valves —

- a) Regulating Valves — Needle, Tube and Hollow jet Valves
- b) Open — Shut Valves — Gate Valves (Wedge Gate, Ring follower Gate), Fishtail, Butterfly, Spherical Valves
- c) Energy Dissipators — Howell — Bunger Valves and Dispersers.

WATER HAMMER — A Gate, or Valve, at the end of the penstock pipe control the discharge of the turbine. As soon as this governor-regulated opening is altered, the pipe flow has to be ~~altered~~ adjusted to the new magnitude of flow. In doing so, There are rapid pressure oscillations in the pipe, often accompanied by hammering like sound. This phenomenon is called Water Hammer.

There are basically two approaches to analyse and overcome the water hammer problem.

Rigid Water Column Theory — In this the compressibility effects of water are neglected.

Elastic water column Thry — the elastic effect are taken into account.

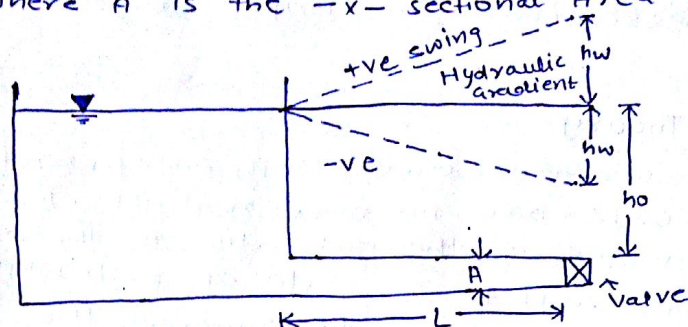
Rigid Water Column Theory -

Consider the simple case of a uniform diameter pipe and length of L , taking off from a reservoir where static head is h_0 . If we neglect the head losses in the pipe, the velocity of flow V_0 is given by

$$V_0 = \sqrt{2gh_0}$$

and $Q_0 = A\sqrt{2gh_0}$

Where A is the -x- sectional Area of the pipe



Swing of Hydraulic gradient

Now, as the Valve at the end is closed, the water in the pipe retards increasing the pressure inside the pipe. This increased pressure swing the normal hydraulic gradient to a position as shown in Figure, by dotted line. Since the pressure at the free reservoir surface is atmospheric and hence constant, the +ve swing results in the backflow from the pipe, into the reservoir. As the water flows back into the reservoir, it creates partial vacuum inside the pipe and the pressure in the pipe swings in the -ve direction. This induces the reservoir water to flow again into the pipe. But the valve being partially closed, much of this water is again retarded giving rise to a +ve swing of pressure once again. Thus a Valve closure brings about pressure oscillations. The max. additional water hammer head h_w over normal head h_0 can be worked out with the help of Newton's second law. Considering the Volume of water in the pipe is retarded, we can write:

$$F = P_w A = \frac{WAL}{g} \cdot \frac{dV}{dt}$$

Here, P_w = water hammer pressure = Wh_w

$$h_w = \frac{L}{g} \left(\frac{dV}{dt} \right)$$

The velocity past the gate at any instant is

$$V = \sqrt{2g(h_0 + h_w)}$$

If T is the total time of closure of the gates, then for complete closure, assuming uniform gate movement, it can be shown that the max. water hammer head h_{wm} is given by

$$\frac{h_{wm}}{h_0} = \frac{k_1}{2} \pm \sqrt{k_1 + \frac{k_1^2}{4}}$$

where $k_1 = \left(\frac{LV_0}{gh_0 T} \right)^2$

For small value of K_1 ,

$$\frac{h_{wm}}{h_0} = \frac{K_1}{2} \pm \sqrt{K_1}$$

Limitations-

- i) It ignores the elastic effect due to sudden pressure change.
- ii) Frictional effects were ignored in the above derivation.
- iii) The derivation is essentially for simple conduits.

Elastic Water Column Theory-

The water hammer pressure are usually high enough and the elasticity effects of the water are no more negligible. As a matter of fact, the pressure developed at the gate due to the closure brings the water mass next to it to a state of rest, while at the same time, the high pressure tends to compress the water mass and tends to extend the pipe.

For sudden closures, as the valve closure is completed before the reflected wave returns at the gate end, it is possible to imagine a total conservation of energy of the water in the pipe. Thus all the K.E. can be presumed to be converted into strain energy of water and strain energy of the pipe. Based on this assumption, we can derive an expression for water hammer pressure as follows:

Given that

Pipe Dia = D

Pipe wall thickness = t

Pipe Length = L

Pipe elasticity modulus = E

Bulk modulus of water = K

Initial steady velocity = V_0

Mass density of water = $w/g = \rho$

Water hammer pressure = p_w

Max. excess head due to water hammer = h_{wm}

$$\text{Hoop stress in the pipe wall, } f = \frac{p_w D}{2t}$$

$$\text{Volume of water} = \frac{\pi D^2}{4} L$$

$$\text{Volume of pipe wall under stress} = \pi D t L$$

$$\text{Total KE before closure} = \text{S.E. of water after closure} + \text{S.E. of pipe}$$

$$\text{Thus } \left(\frac{\pi D^2}{4} L \right) \frac{w V_0^2}{2g} = \frac{p_w^2}{2K} \left(\frac{\pi D^2}{4} L \right) + \frac{f^2}{2E} (\pi D t L)$$

On substituting the value of f and simplifying, we get

$$p_w = w h_{wm} = \sqrt{\frac{w V_0^2}{g \left(\frac{1}{K} + \frac{D}{tE} \right)}}$$

If we denote $V_c = \sqrt{\frac{km}{\rho}}$

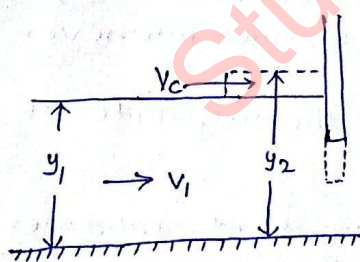
Where $\frac{1}{K_m} = \frac{1}{k} + \frac{D}{tE}$

then $h_{wm} = \frac{V_0 V_c}{g}$

V_c = compression wave velocity.

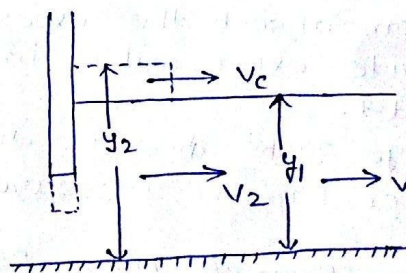
Resonance in Penstocks - If the penstock piping system is subjected to a periodic excitation with natural periods of the system, severe oscillations of the pressure head develop. The system is then said to be in resonance and is in danger of damage, due to the resulting auto-oscillations. Periodic excitations in case of penstock pipes is usually due to the rhythmic closing and opening of the penstock Valve at the d/s end of the pipe.

SURGES IN POWER CHANNELS - In hydro-power conveyance systems, sometimes long power canals are used to feed the powerhouse. In such cases the sudden regulation of discharge at the turbine produces a water wave or surge which then moves u/s. Similarly, if the water from the powerhouse is carried through a long tailrace canal, the turbine regulation produces a surge which moves in the d/s direction. Such surges in channels are the counterpart of water hammer in pipes. Surges can be +ve or -ve depending upon whether the depth of water after surges is greater or less than the normal. D/s gates produce surges which move against the flow while the gate at inlet produces a surge which moves in the same direction as the flow.



a) power canal (+ve)

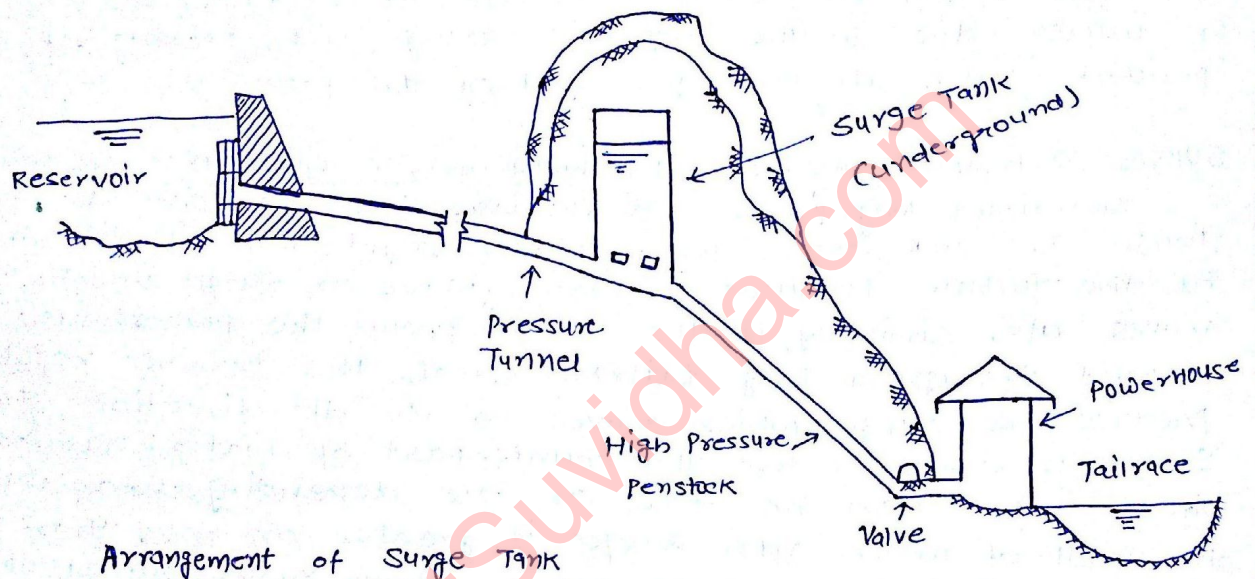
A +ve surge results from turbine gate closure, a -ve surge result from increase in turbine gate opening.



b) Tailrace Canal (+ve)

A +ve Surge results due to the turbine gate opening, A -ve surge results due to turbine gate closure.

SURGE TANKS— It is a structure which forms an essential part of the conveyance pressure conduit system whenever such systems are long. It is a sizeable water receptacle interposed b/w the powerhouse and the high pressure penstocks on one side and low pressure tunnels and the reservoir on the other side. These are usually associated with high head development schemes where water is taken to the powerhouse through tunnels which are designed to be only for low to moderate heads and can't withstand the water hammer heads. Surge Tanks are needed to block these high pressure waves travelling into tunnels.



Necessity of Surge Tanks—

- i) To protect the low pressure conduit system from high internal pressures.
- ii) It can absorb the excess discharge from the reservoir, to provide extra water in emergency to turbines whenever needed.
- iii) Surge Tanks do not directly influence the magnitude of water hammer pressure.

Note: The Run-off River Plants and medium head schemes require no surge tank.

- Also canal diversion schemes also not require this.
- When powerhouse is located within a short distance of headwork surge tanks are not required.

Types of Surge Tanks -

Classified in different ways -

- i) By material of construction, concrete or steel
- ii) By location w.r.t. ground, underground or overground
- iii) Location in the hydraulic system
- iv) By hydraulic functioning and -x- shape.

Classification based on their shape which also determine their hydraulic characteristics, is

- a) Simple cylindrical Surge Tank
- b) Restricted orifice "
- c) Simple surge Tank of simple design
- d) Differential Surge Tank
- e) Reverse Flow throttle Surge Tank

Hydraulic design of Simple surge Tanks -

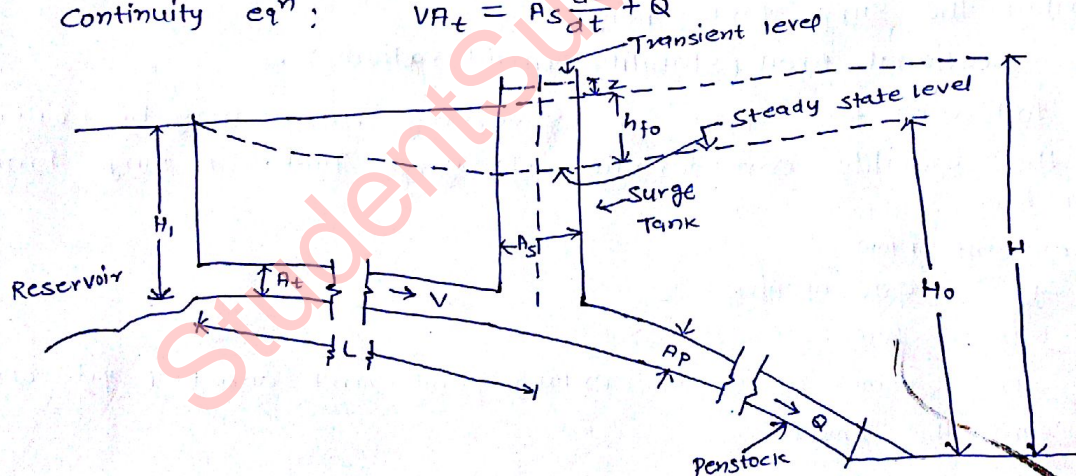
It consist of determine of

- a) Height.
- b) cross-sectional Area.

→ height can be fixed, once the max. upsurge and downsurge are known, while -x- Area should be atleast so much as to ensure a stable behaviour of the Tank.

- a) Height - elementary analysis of the system can be carried out with the help of momentum and continuity equations -

Continuity eqⁿ: $VA_t = A_s \frac{dz}{dt} + Q$



Momentum eqⁿ: $\frac{L}{g} \frac{dv}{dt} + z \pm hf = 0$

Where

Q = rate of flow to the powerhouse at any time t

$\frac{dz}{dt}$ = rate of water rise

A_s, A_t = -x- Area of Surge Tank and headrace Tunnel

v = velocity in the tunnel

d = diameter

L = length

f = frictional Coefficient

$$h_f = \text{frictional head loss} = \frac{fLV^2}{2gd}$$

For the simplest case of full closure and negligible friction,

$$Z = V_0 \sqrt{\frac{L}{g} \frac{A_t}{A_s}} \sin 2\pi \frac{t}{T}$$

The max. upsurge or downsurge is thus given by

$$Z_{\max} = V_0 \sqrt{\frac{L}{g} \frac{A_t}{A_s}}$$

approx. formula for the calculation of upsurge in cases where friction is taken into account.

$$\frac{Z}{Z_{\max}} = 1 - \frac{2}{3} P_0 + \frac{1}{9} P_0^2$$

Where

Z = max. upsurge with friction

Z_{\max} = max. upsurge with negligible friction

$$P_0 = \frac{h_{f_0}}{Z_{\max}}$$

where h_{f_0} = frictional head loss in the steady state condition.

NOTE - The total height of the Surge Tank should be such that both the upsurge and downsurge should be contained within the surge tank height.

b) Cross-Sectional Area (Stability Considerations) -

The following four types of governing rules may be possible in the stability consideration of Surge Tank. A Surge Tank should have

- constant Flow
- " Gate opening
- " Power
- " power up to a certain point and constant gate opening beyond this limit.

In Actual practice, the -x- Area of the surge tank is

$$= F_s \cdot A_{smin}$$

where $F_s = FOS = 1.5$ (around)