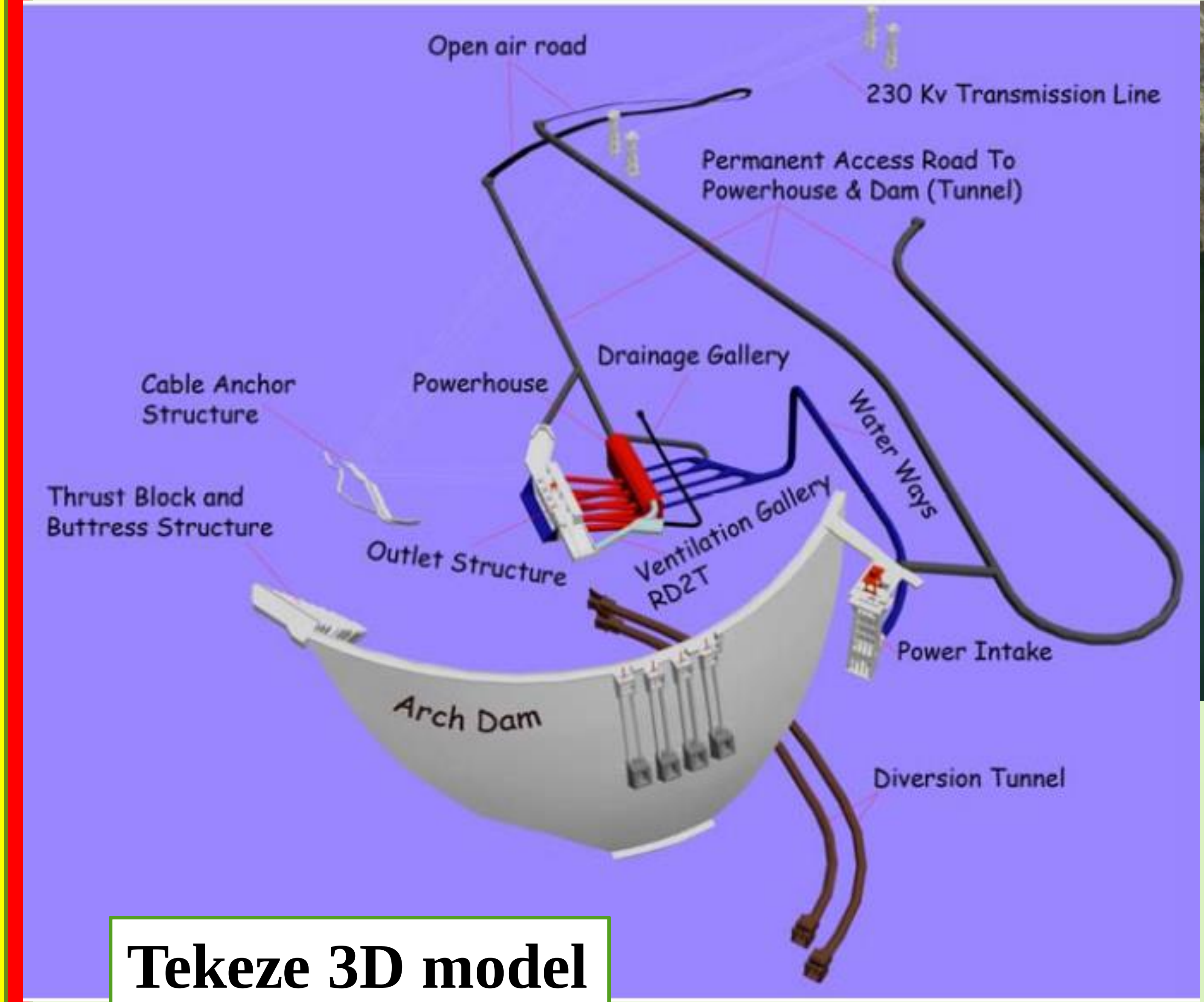
Chapter 6: Design of Hydropower Plants

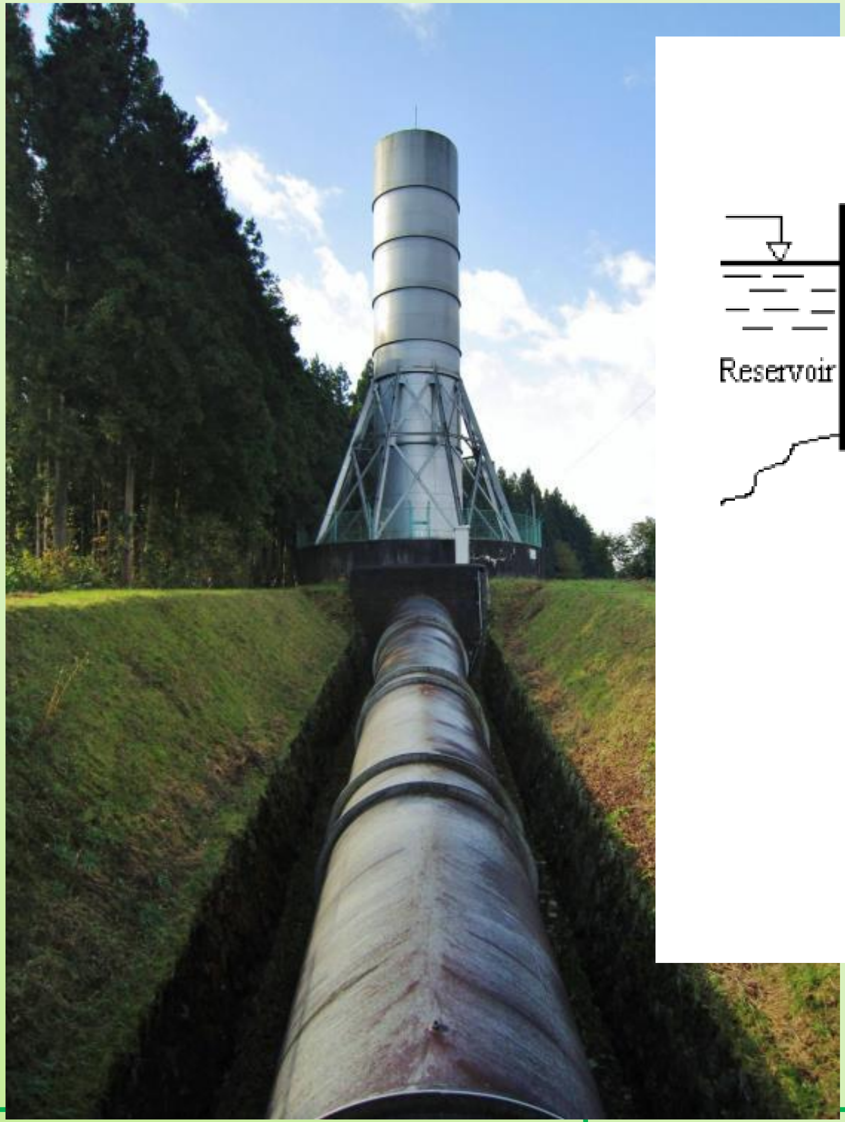


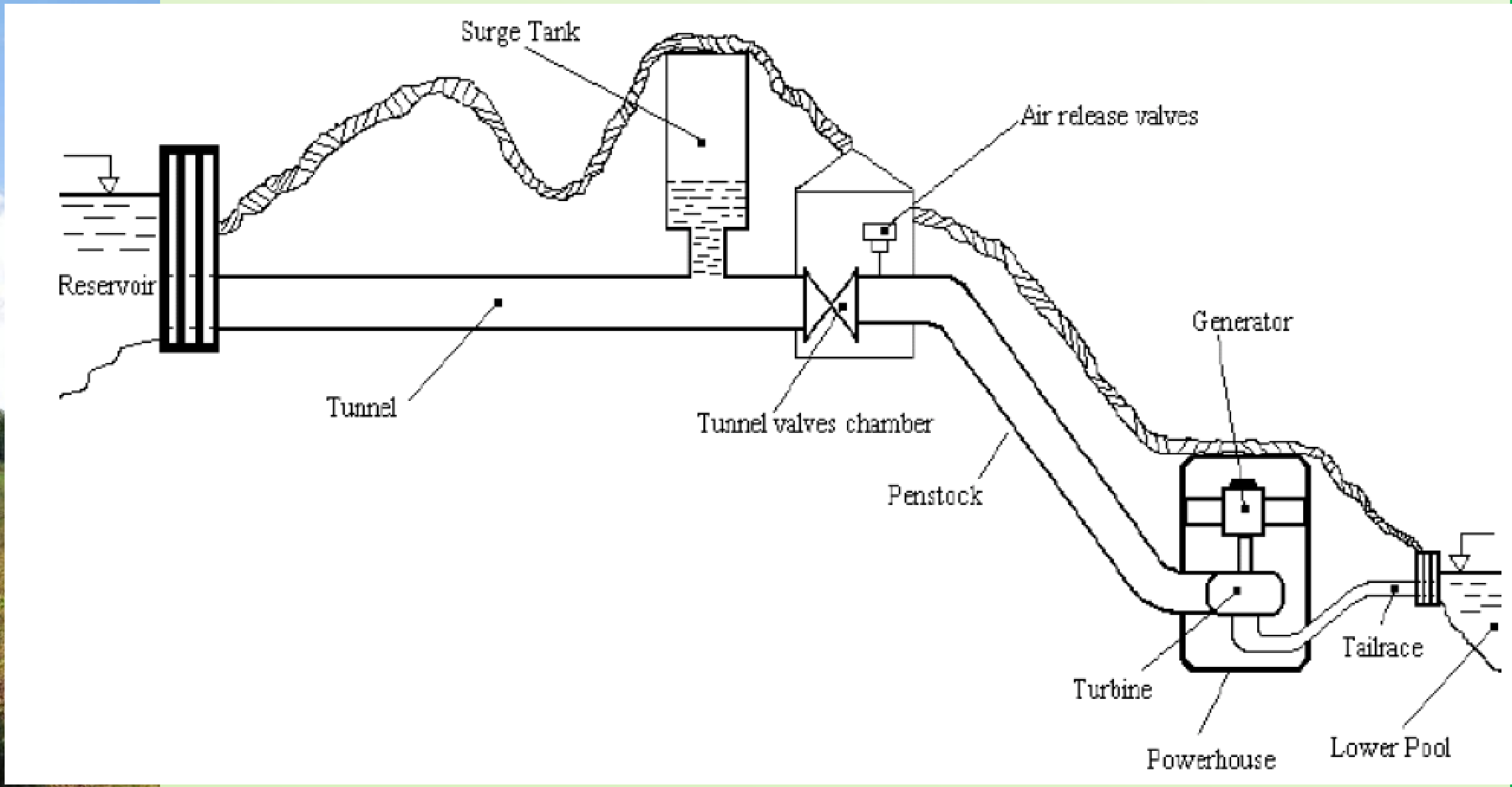
Tekeze 3D model

Arch Dam upstream view

Design of Civil Structures (cont.)

Water Hammer Analysis and Surge Tank Design





Introduction

Water Hammer

- Water hammer is a phenomenon of pressure change in closed pipes caused when flowing water in a pipeline is rapidly decelerated or accelerated
- The phenomenon is accompanied by a series of positive and negative pressure waves which travel back and forth in the pipe system until they are damped out by friction
- When a valve in a pipe or penstock carrying water is closed, the pressure head immediately upstream of the valve is increased and a pulse of high pressure is propagated upstream to the nearest open water surface
- On the downstream side of the valve a lowered pressure moves in a downstream direction to the nearest open water surface

Causes of rapid changes in flow (acceleration/deceleration):

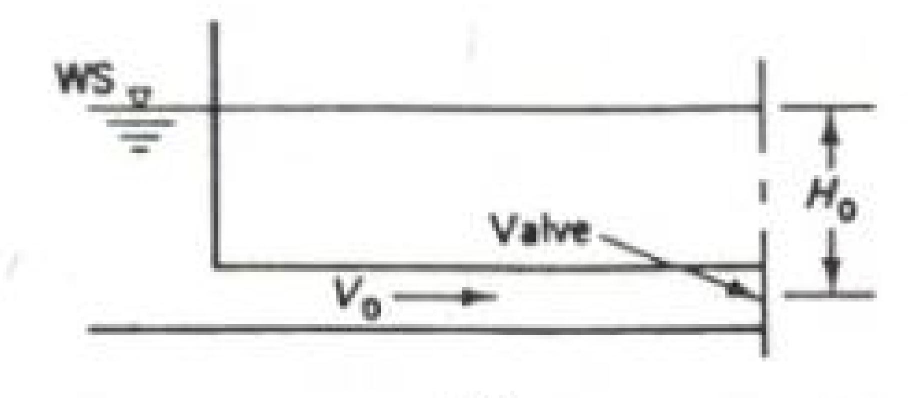
- Sudden opening or closing of control valves
- Starting or stopping of pumps
- Rejecting or accepting load by hydraulic turbine

There are two theories in analyzing water hammer:

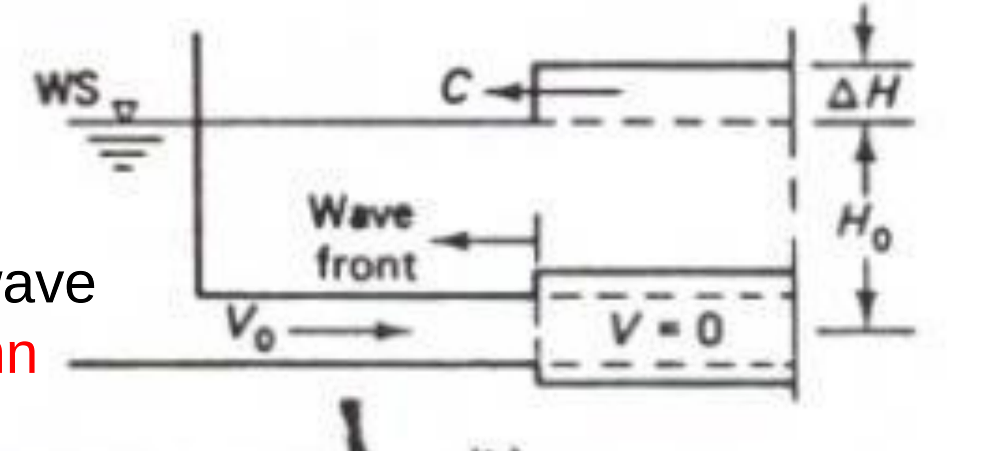
- Elastic water column (EWC) theory
- Rigid water column (RWC) theory

- When the time it takes a valve to close is long compared to the propagation time for a pressure wave to travel the length of the pipe, then the rigid column theory is appropriate;
- otherwise considering elastic water column theory is necessary.

Steady state prior to valve closure



(a)



(b)

Rapid valve closure followed by pressure increase, pipe walls expand , liquid compression ; and transient conditions propagate upstream

The maximum water hammer pressure

Conservation of linear momentum

- The total pressure at the valve *(Point B)* immediately after closure is

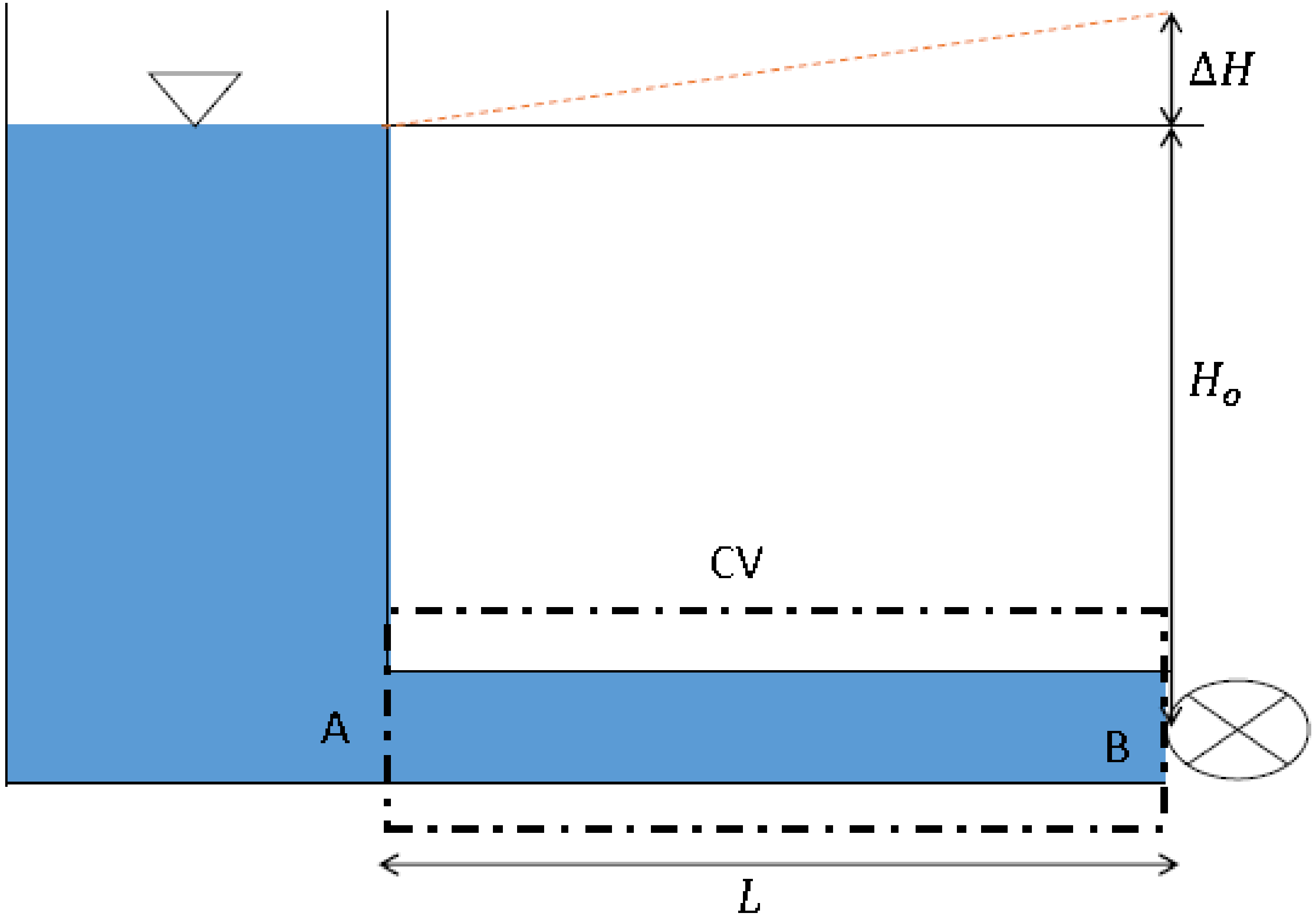
$P_B = \gamma(H_o + \Delta H)$ and

At *Point B is*

$$P_A = \gamma H_o$$

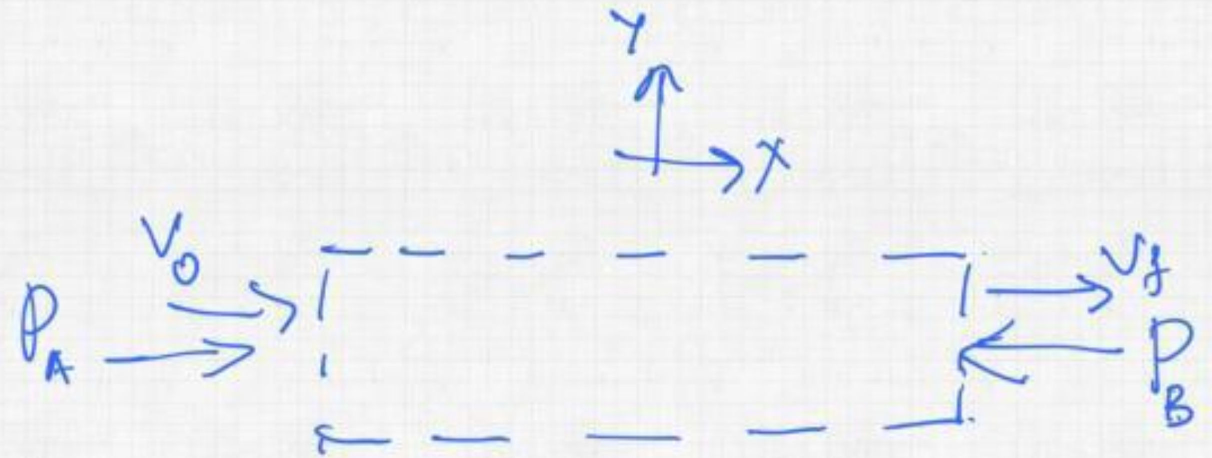
- The pressure pulse or wave moves at a velocity, c, which is essentially the velocity of sound in water.
- If the length of the pipe is L, the wave travels from valve to reservoir in time

$$t = L/c$$



The maximum water hammer pressure (cont.)

Conservation of linear momentum



$$\Sigma F_x = m \frac{dv}{dt}$$

$$P_A dA - P_B dA = m \frac{(v_f - v_0)}{t}$$

$$\gamma H_0 dA - \gamma dA (H_0 + \Delta H) = \rho dA L \frac{(0 - v_0)}{t}$$

The maximum water hammer pressure (cont.)

$$\cancel{\gamma H_0 dA} - \cancel{\gamma H_0 dA} - \gamma \Delta H \cancel{dA} = \rho \cancel{dA} L \left(\frac{-V_0}{t}\right)$$

$$\Rightarrow \Delta H = \frac{L}{g}\left(\frac{V_0}{t}\right)$$

$$\Delta H = \frac{\cancel{L}}{g}\left(\frac{V_0}{\cancel{L}/c}\right)$$

$$\Delta H = \frac{V_0 c}{g}$$

Elastic water column (EWC) theory

- This considers the effect of compressibility of the water column in the pipe and the dilation of the pipe under high pressure.
- Thus, for sudden valve closure, it is assumed that all the kinetic energy of the water is converted to the strain energy of the water (compression) and strain energy of the pipe (tensile)

Change in pipe volume due to added lateral stress

- When the increased pressure stretches the pipe, more space is available to store the accumulated net inflow of water.
- The pipe may stretch both circumferentially and longitudinally, so we must consider both contributions to the change in pipe volume.
- If a material is strained in one direction by an amount ε1 then a strain ε2 will occur in the perpendicular direction (provided the material is free to strain without developing a stress in that direction) according to $\varepsilon 2=\mu\varepsilon 1$, where μ is Poisson's ratio.
- If there is a restriction to free strain in either direction caused either by restraint or applied stress, the relation is more complicated.

Elastic water column (EWC) theory (cont.)

- Here σ_1 and ε_1 are the stress and strain, respectively, in the direction along the pipe axis,
- σ_2 and ε_2 are the values in the circumferential direction, and
- E is the modulus of elasticity of the pipe wall material.

Lateral strain: Hooke's Law $E = \dfrac{\sigma_2}{\varepsilon_2}$

$$\sigma_1 = \frac{\varepsilon_1 + \mu\varepsilon_2}{1-\mu^2}E \quad \text{or} \quad \varepsilon_1 = \frac{\sigma_1 - \mu\sigma_2}{E}$$

$$\sigma_2 = \frac{\varepsilon_2 + \mu\varepsilon_1}{1-\mu^2}E \quad \text{or} \quad \varepsilon_2 = \frac{\sigma_2 - \mu\sigma_1}{E}$$

$\varepsilon_2 = \frac{\Delta D}{D}$ implies $\Delta D = \varepsilon_2 D$ and the elongated diameter,

$$D' = D + \varepsilon_2 D$$

Pipe area increase: $\Delta A = \frac{\pi}{4}[(D + \varepsilon_2 D)^2 - D^2]$

$$\Delta A = \frac{\pi}{2}\varepsilon_2 D^2$$

- The change in volume caused by circumferential stretching is

$$\Delta V = \frac{\pi}{2}\varepsilon_2 D^2 L$$

- The change in volume caused by longitudinal stretching is

$$\Delta V = \frac{\pi}{4}D^2 \Delta L$$
$$= \frac{\pi}{4}D^2 \varepsilon_1 L$$

The total volume change due to pipe stretching as

$$\Delta V_{total} = \frac{\pi}{4}D^2 L(2\varepsilon_2 + \varepsilon_1)$$

$$\sigma_2 = \frac{\gamma \Delta H D}{2t} = \frac{\varepsilon_2 + \mu\varepsilon_1}{1-\mu^2}E$$

Elastic water column (EWC) theory (cont.)

While the relation between circumferential stress and pressure is valid for all types of restraint, the relation between longitudinal stress and strain varies with restraint type:

Case (a) pipe anchorage only at the upstream end;

Case (b) full pipe restraint from axial movement;

Case (c) longitudinal expansion joints along the pipeline.

- In a practical sense the actual pipe restraint situation probably will not conform precisely to any of these cases but lies somewhere in this range of possibilities.
- Because buried pipelines are relatively common and might be expected to be fully restrained axially by soil friction and anchor blocks, we will examine Case (b)

Wave Speed Solution for Case (b) Restraint

- For this restraint choice $\varepsilon_1 = 0$ and

$$\sigma_1 = \frac{\mu \varepsilon_2}{1-\mu^2} E \text{ or } \sigma_1 = \mu \sigma_2$$

And

$$\sigma_2 = \frac{\gamma \Delta H D}{2t} = \frac{\varepsilon_2}{1-\mu^2} E$$

Elastic water column (EWC) theory (cont.)

- *Substituting this equation into the total volume equation in place of ε_2*

$$\Delta V_{total} = \frac{\pi}{4} D^2 L(2\varepsilon_2 + \varepsilon_1)$$

$$= \frac{\pi}{4} D^2 L\left(\frac{\gamma \Delta H D}{t}\right)\frac{(1-\mu^2)}{E}$$

Reduction in the volume of water within the pipe is based on compressibility of water

$$K = \frac{\Delta P}{\Delta V/V} = \frac{\gamma \Delta H}{\Delta V/V}$$

$$\Delta V = \frac{V\gamma\Delta H}{K} = \frac{AL\gamma\Delta H}{K}$$

Here K is the bulk modulus of elasticity of water and P and V are the pressure and volume of water, respectively.

ΔV is the change in water volume in the control volume resulting from the pressure change ΔP .

Elastic water column (EWC) theory (cont.)

- Thus the overall volume change due to change in pipe volume and reduction in the volume of water within the pipe is given by:

$$\left[\frac{\pi}{4} D^2 L \left(\frac{\gamma \Delta H D}{t}\right) \frac{(1-\mu^2)}{E}\right] + \frac{A L \gamma \Delta H}{K}$$

$$V \gamma \Delta H \left[\frac{D}{t} \frac{(1-\mu^2)}{E} + \frac{1}{K}\right] = V_o A dt$$

But $dt = L/c$ and $\Delta H = V_o c/g$

$$\rho c^2 \left[\frac{D}{t} \frac{(1-\mu^2)}{E} + \frac{1}{K}\right] = 1$$

or, in the conventional form for wave speed,

$$c = \frac{\sqrt{K/\rho}}{\sqrt{1 + \frac{K}{E}\frac{D}{t}(1-\mu^2)}}$$

Wylie and Streeter (1993) show that the equation for wave speed can be conveniently expressed in the general form:

$$c = \frac{\sqrt{K/\rho}}{\sqrt{1 + \frac{K}{E}\frac{D}{t}(C)}}$$

where

for Case (a) restraint $C = (5/4 - \mu)$

for Case (b) restraint $C = (1 - \mu^2)$ and

for Case (c) restraint $C = 1$

Elastic water column (EWC) theory (cont.)

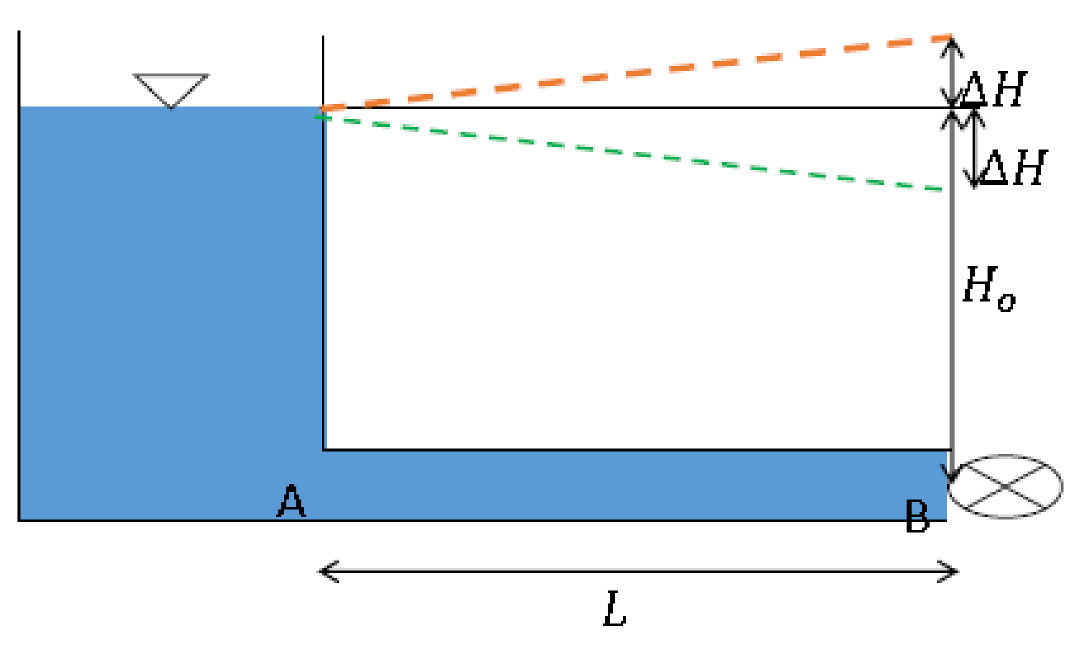
- Moduli of Elasticity and Poisson Ratios for Common Pipe Materials

MODULI OF ELASTICITY AND POISSON'S RATIOS

Material	Modulus of elasticity E		Shear modulus of elasticity G		Poisson's ratio
	ksi	GPa	ksi	GPa	
Aluminum alloys	10,000–11,400	70–79	3,800–4,300	26–30	0.33
2014-T6	10,600	73	4,000	28	0.33
6061-T6	10,000	70	3,800	26	0.33
7075-T6	10,400	72	3,900	27	0.33
Brass	14,000–16,000	96–110	5,200–6,000	36–41	0.34
Bronze	14,000–17,000	96–120	5,200–6,300	36–44	0.34
Cast iron	12,000–25,000	83–170	4,600–10,000	32–69	0.2–0.3
Concrete (compression)	2,500–4,500	17–31			0.1–0.2
Copper and copper alloys	16,000–18,000	110–120	5,800–6,800	40–47	0.33–0.36
Steel	28,000–30,000	190–210	10,800–11,800	75–80	0.27–0.30

Rigid water column (RWC) theory

- As the water flows to the reservoir it creates partial vacuum conditions and the pressure in the pipe swings in the –ve direction.
- This induces the reservoir water to flow in to the pipe.
- But the valve being partially closed, much of this water again retarded giving rise to a +ve swing pressure again.
- Thus a valve closure brings about pressure oscillations.
- On the other hand, the velocity past the gate at any instant is given by:



$$v = \sqrt{2g(H_o + \Delta H)}$$

Rigid water column (RWC) theory (cont.)

If the total time of closure of the gate is T, then for complete closure assuming uniform gate movement, it can be shown that

$$\frac{H_{wm}}{H_o} = \frac{K_1}{2} \pm \sqrt{K_1 + \frac{K_1^2}{4}}$$

where $K_1 = \left(\frac{L v_o}{g H_o T}\right)^2$

for small values of K_1 the above equation can be simplified as $\frac{H_{wm}}{H_o} = \frac{K_1}{2} \pm \sqrt{K_1}$

In the above equation the positive sign gives upward swing and negative sign gives the value of the downward swing.

Rigid water column (RWC) theory (cont.)

- The velocity of the pressure wave, c, in a pipe according to RWC

$$c = \left(\frac{K}{\rho_w}\right)^{0.5}$$

K= the modulus of elasticity of the water

ρ_w= the density of water

- c is about 1440 m/s for water under ordinary conditions
- The velocity of a pressure wave created by water hammer is less than 1440 m/s because of the elasticity of pipe
- The velocity of a pressure wave in a water pipe usually ranges from 600 to 1200 m/sec for normal pipe dimensions and materials.

Rigid water column (RWC) theory (cont.)

- RWC ignores elastic property due to sudden pressure rise
- The time taken by a pressure wave for a round trip from the valve to reservoir and back is called the critical time of closure and is given by

$$T_c = \frac{2L}{c}$$

- If the actual valve closure time T is greater than T_c, rigid water column theory gives fairly correct estimation of the excess head built up due to valve closure.
- Based on this, it can be defined that valve closure as gradual if $T > T_c$ and as rapid of $T \le T_c$.
- In the above derivation frictional effects which dampen the water hammer pressure is ignored.
- This simplifies the problem and also provides greater factor of safety.

Example

- Water flows at 2 m/sec from a reservoir into a 100 cm diameter steel pipe which is 2500 m long and has a wall thickness $t = 2.5\ cm$. Find the water hammer pressure developed by closure of a valve at the end of the line if the closure time is a) 1 sec , b) 8 sec.

$$K = 2 * 10^9 Pa, E = 2 * 10^{11} Pa\ and\ \rho_{water} = 1000 kg/m^3$$

Solution

$$c = \frac{\sqrt{K/\rho}}{\sqrt{1 + \frac{K}{E}\frac{D}{t}(C)}}$$

For fully strained in the axial case,
C=1-η^2

η=0, Hence C=1

$$c = \left(\frac{2\times10^9}{1000}\right)\times\left(\frac{1}{1+\frac{2\times10^9\times1}{2\times10^{11}\times0.025}}\right)^{0.5}$$

$$c = 1414\times0.714 = 1010\,m/\sec$$

$$t = \frac{2L}{c} = \frac{2500\times2}{1010} = 4.95\sec$$

a) If $t_c = 1$ sec < 4.95 sec,

$$t = \frac{2L}{c_p} = \frac{2500\times2}{1010} = 4.95\sec$$

a) If **T** = 1 sec < 4.95 sec,

$$p_h = \rho c.V = 1000\times1010\times2 = 2.02\times10^6\,kPa$$

b) If **T** = 8 sec > 4.95 sec,

$$p_h' \cong \frac{t}{t_c}p_h = \frac{4.95}{8}\times2.02\times10^6 = 1.25\times10^6\,kPa$$

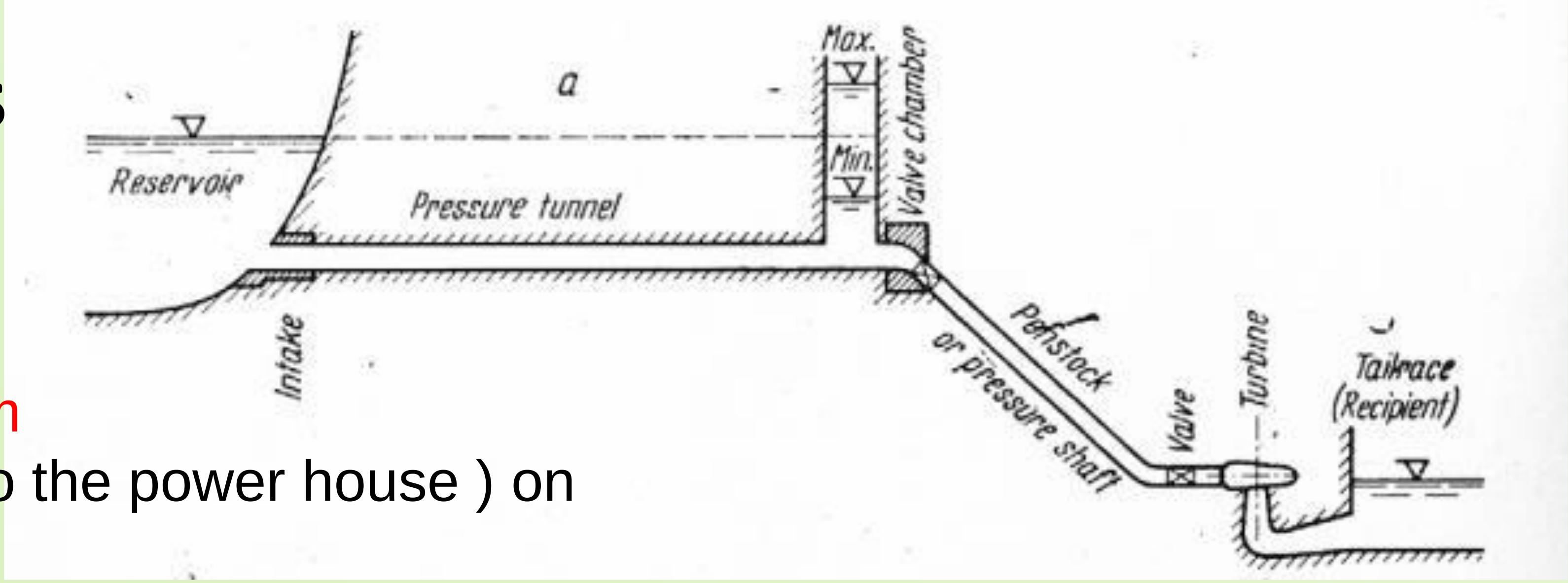
Surge Tanks

- The surge tank, also called the expansion chamber, is a structure which forms an essential part of the pressure conduit conveyance system whenever such system is long.
- Their primary purpose is protection of low pressure tunnel in medium and high –head plants against high water hammer pressure arising from sudden rejection or acceptance of load.
- The surge thank converts these high frequency, high pressure transients (water hammer) in to low frequency low pressure, mass oscillation.

Uses

- It shortens the distance between the turbine inlet and the nearest free water surface, and thereby greatly reduces the intensity of the water hammer waves.
- With a reduction of turbine load, the water level in the chamber rises until it exceeds the level in the main reservoir, thus retarding the main conduit flow and absorbing the surplus kinetic energy.
- In case of increase of turbine load, the surge tank acts as a **reservoir** which provides sufficient water to enable the turbine to pick up their new load safely and quickly and to keep them running at the increased load until the water level in the surge chamber has fallen below its original level.

•The surge tank is located between the almost horizontal or slightly inclined conduit and steeply sloping penstock and is designed as a chamber excavated in the mountain.



Types of Surge Tanks

Location relative to terrain

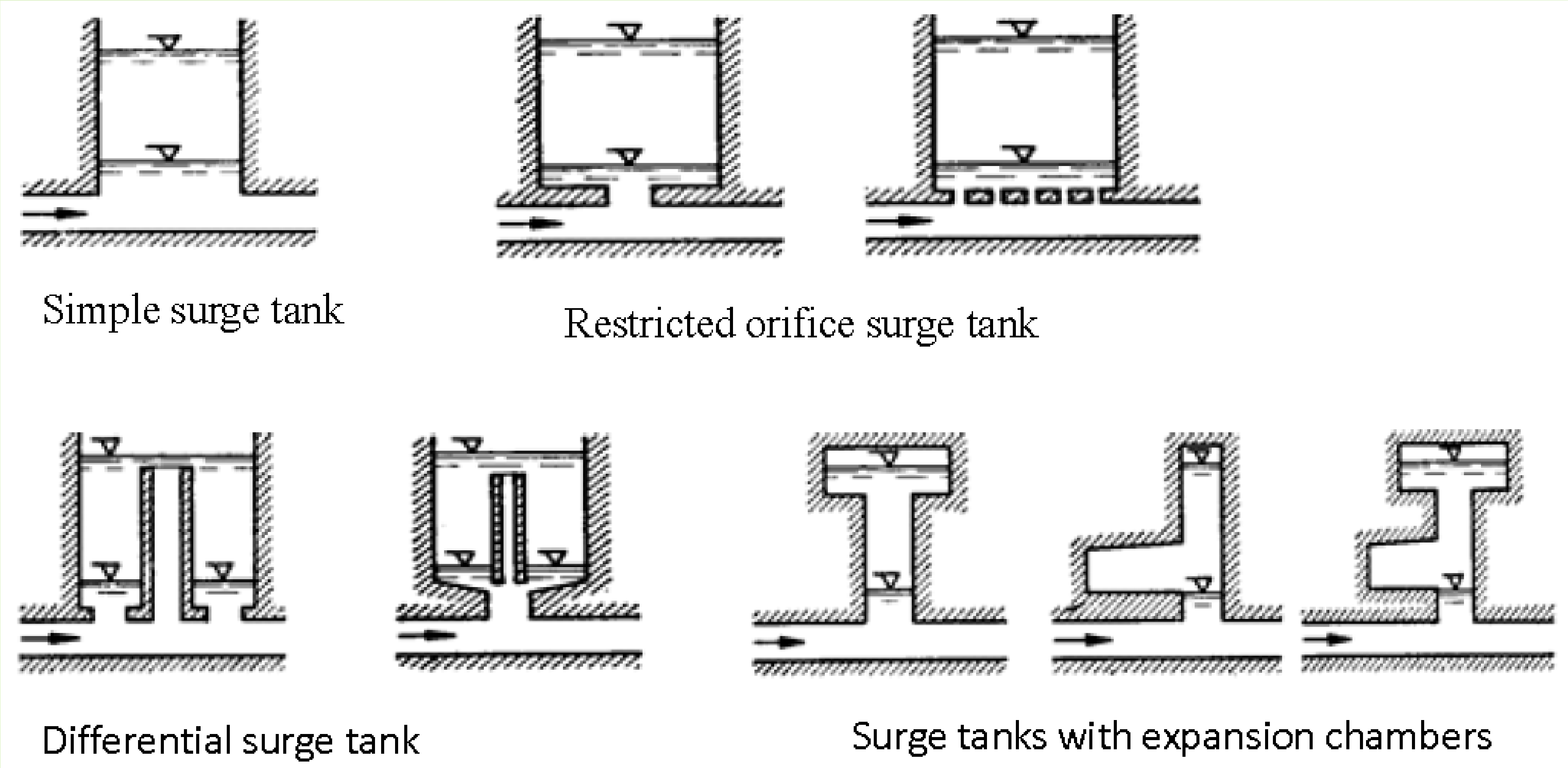
- underground surge tank
- over ground surge tank

Location in the hydraulic system

- Upstream surge tank (u/s to the power house) on the headrace tunnel
- Downstream surge tank on the tailrace tunnel

Hydraulic functioning & cross-sectional shape

- Simple surge tanks
- Restricted orifice (or throttled) surge tanks
- Differential surge tanks
- Surge tanks with expansion chambers and others


Simple surge tank
Restricted orifice surge tank
Differential surge tank
Surge tanks with expansion chambers

Design Consideration of Surge Tank

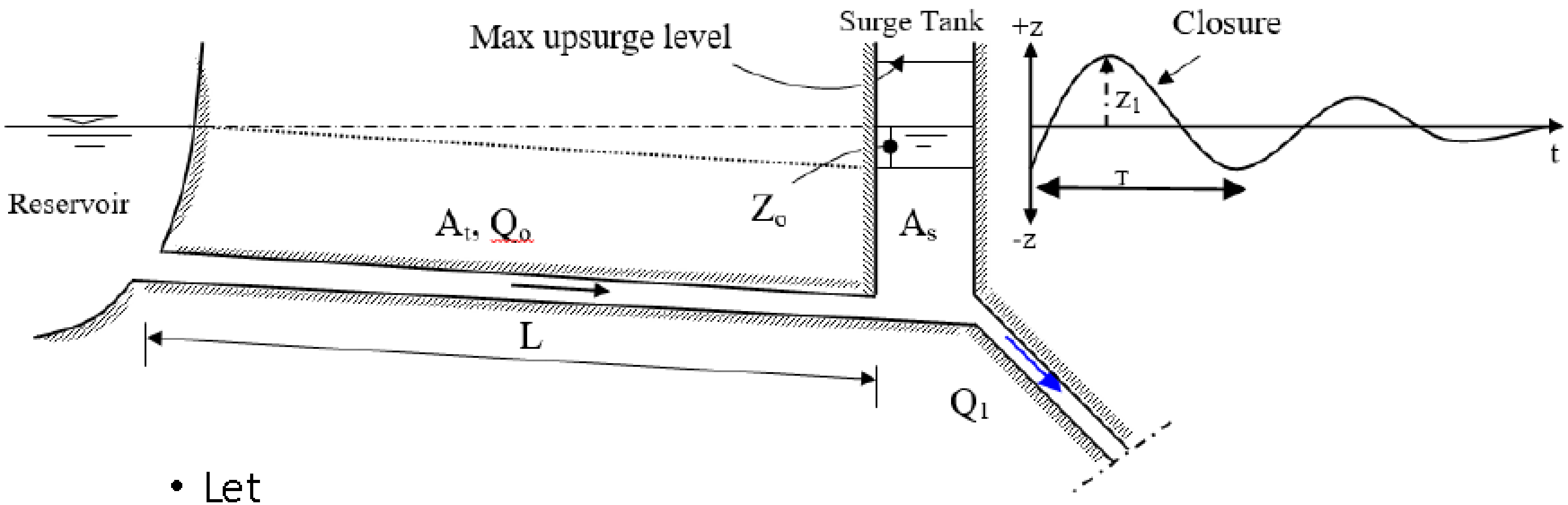
The surge chambers are designed to meet the following conditions:

- The surge chamber must be so located that pressure variations caused by water hammer are kept within acceptable limits.
- The chamber must be stable, i.e. the surges resulting from small partial load changes must be naturally damped and must not under any condition be continued or amplified.
- The chamber must be of such size and so proportioned that it will contain the maximum possible upsurge (unless a spillway is provided).
- The lowest down surge will not allow air to be drawn into the tunnel.
- The range of surges must not be great enough to cause undesirably heavy governor movements or difficulty in startup load.

It is usual to consider full-load rejection under two conditions:

- With the reservoir at its maximum level, in which case the maximum upsurge level will govern the top level of the chamber;
- With the reservoir at its lowest draw down, in which case the first down surge level may control the bottom level of the chamber if air drawing is to be avoided.

Hydraulic design of simple surge tank



Continuity equation

$$V A_t = A_s \frac{dz}{dt} + Q_1 \quad \Rightarrow \quad \frac{dz}{dt} = \frac{A_t V - Q_1}{A_s} \quad eq.1$$

Momentum equation

$$\frac{L}{g}\frac{dV}{dt} - z \pm z_o = 0$$

+ve and –ve sign in the equation is to used depending on the flow direction in the tunnel. For flow from reservoir +ve sign is used.

$$z_o = f\frac{LV^2}{2gD}$$

- Let

$$F_t = \frac{fL}{2gD}$$

Then during flow from penstock to surge

$$\frac{L}{g}\frac{dV}{dt} + z - F_t V^2 = 0$$

If surge tank throttle loss is considered

$$\frac{L}{g}\frac{dV}{dt} + z - F_t V^2 - F_s V_s^2 = 0$$

Undamped Oscillation

- From Eq. 1 if Q_1 is zero:
  - $dz/dt = v_t A_t / A_s \rightarrow d^2z/dt^2 = (A_t / A_s)dv_t/dt$ **(Eq. 3)**
- For simplest case of full closure and negligible friction the solution to eq. 2 has the following form:

$$z = z_{max}\sin\left(\frac{t}{T}\right);\ Where\ z_{max} = v_t\sqrt{\frac{A_t L}{A_s g}};T = \sqrt{\frac{A_t L}{A_s g}}$$

- Where z_{max} is the maximum up surge or down surge.

- However; Jaeger has recommended use of the following approximate formula for the calculation of up surge in cases where friction is taken in to account:

$$\frac{z_{up}}{z_{max}} = 1 - \frac{2}{3}k_o + \frac{k_o^2}{9}; \; k_o = \frac{F_t v_t^2}{z_{max}}; \; z_{max} = v_t\sqrt{\frac{A_t L}{A_s g}}$$

- The above formula is applicable for conditions where k_o is less than 0.7.
- Calame and Gaden have given the following approximate formula suitable for computation of the lowest water reached after the first up ward swing.

$$\frac{z_{down}}{z_{max}} = -1 + 2k_o$$

- Where z_{up} and z_{down} are maximum and minimum water level in the surge chamber

Cross-sectional Area (Stability Consideration)

- Characteristic oscillation in the surge tank damped by hydraulic friction in the conduits.
- The required cross-sectional area of a surge tank is determined based on stability considerations for the surge oscillations in the tank. Stability conditions of the surge system were established by Thoma.
- He stated that in order to prevent the development of unstable oscillations the cross-section of the surge tank should exceed a certain critical magnitude.

Cross-sectional Area (cont.)

According to Thoma, the limiting x-sectional all for small oscillation is given by:

$$A_{Sc} = \frac{V_0^2 \; A_t \; L_t}{2g \; P_o \; H_0} \quad (m^2)$$

where $P_o = h_f = Z_o$

$H_o = H - h_f$ = net head on turbine neglecting turbine loss.

Assuming $\beta = \dfrac{1}{m^2 \; R^{4/3}}$ $\quad m = 1/n$ $\quad$ Maning's n

R – tunnel hydraulic radius

$$A_{SC} = \frac{m^2 \; R^{4/3} \; A_t}{2g \; H_o} = \frac{m^2 \; D^{10/3}}{160 \; H_o}$$

$A_s \approx (1.5 \text{ to } 1.8)\, A_{sc}$ $\quad$ (stable tank)

for m=85; n=0.0118; $A_{sc} = 45D^{10/3}$

For large amplitude of oscillation, the Thoma formula was modified by Ch Jaeger as

$$A_S = \eta^* \frac{L \, A_t}{2g \; \beta H_o} = \eta^* \frac{m^2 \; R^{4/3} \; A_t}{2g \; H_0}$$

η^* = non constant factor of safety $= 1 + 1.0482 Z_{max}/H_o$

or $\quad A_S = \dfrac{\eta^* \; m^2 \; D^{4/3}}{160 \; H_o} = 170.482 \; \dfrac{Z_{max}}{H_0}$

$$Z_o = V_0 \sqrt{\frac{L}{g} \frac{A_t}{A_S}} \quad \text{(undamped friction loss)}$$

Forebays

- A forebay, also called a head pond, is a basin located at the end of a power canal just before the entrance to the penstock or pressure shaft.
- It acts as a transition section between the power canal and the penstock. It is formed simply by widening the power canal at the end.

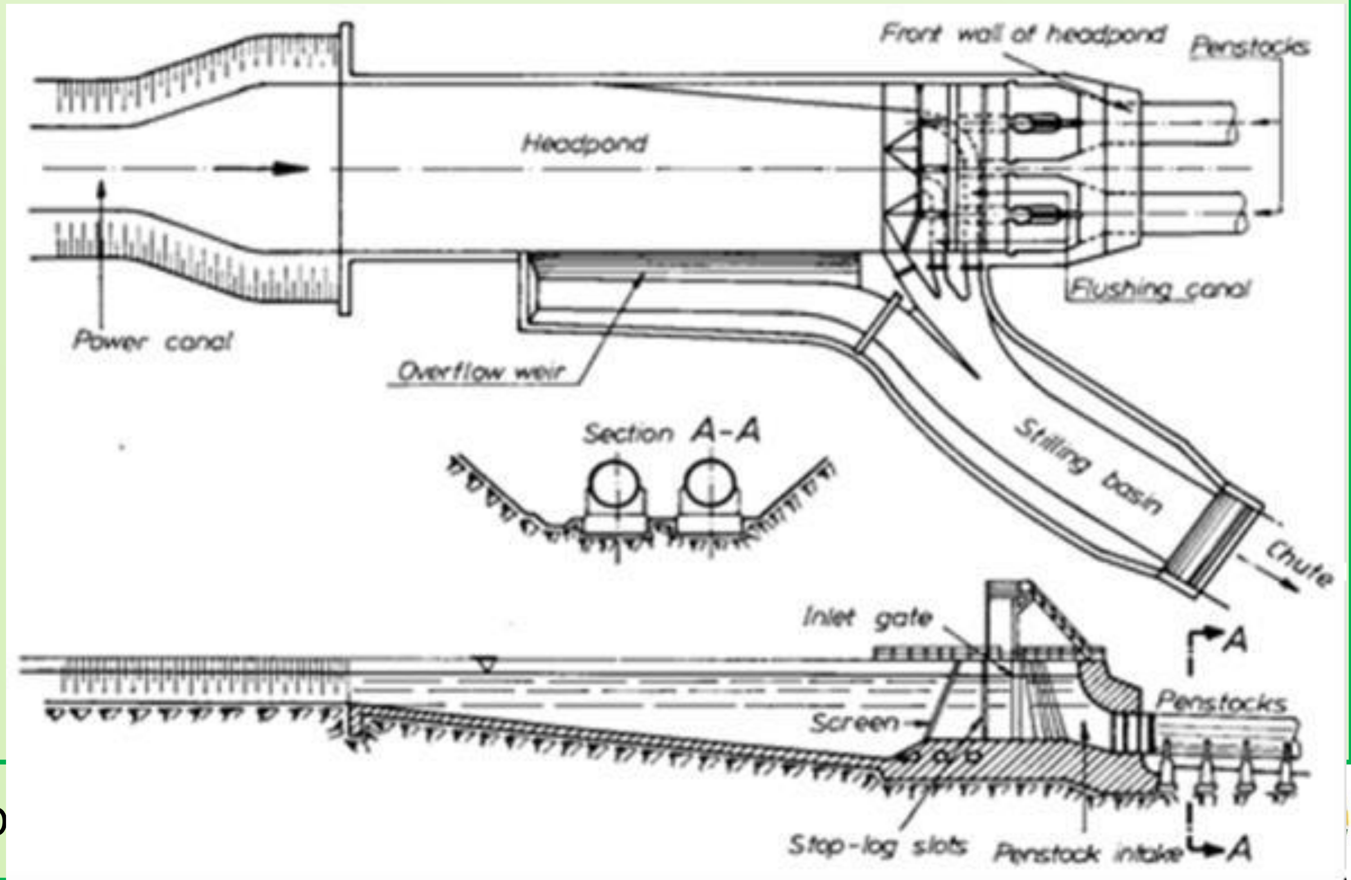
Functions of a Forebay

A forebay serves the following purposes:

- It can serve as a balancing reservoir.
- Water is temporarily stored in the forebay in the event of a rejection of load (turbine closure) and the stored water will be withdrawn from it when the load is increased (turbine opening).
- In the case of low-head power plants, the forebay may even provide daily pondage for the plant.
- It can serve as a final settling basin where any water borne debris which either passed through the intake or was swept in to the power canal can be removed before the water passes into the turbine.
- It can serve to distribute evenly the water conveyed by the power canal among the penstocks, where two or more penstocks are provided.

Components of a Forebay

- The basin: used to store water and sediment (if any)
- The spillway (sometimes of the siphon type), with the overflow weir: used to dispose excess water that might enter the forebay
- The bottom outlet which is generally flushing sluice gate for sediment and for de-watering the forebay and the power
- The penstock inlet: serves in controlling flow into the pressure conduit and in preventing floating debris from entering the conduit. It also provides smooth transition between the basin and the conduit.



Design guidelines for a forebay

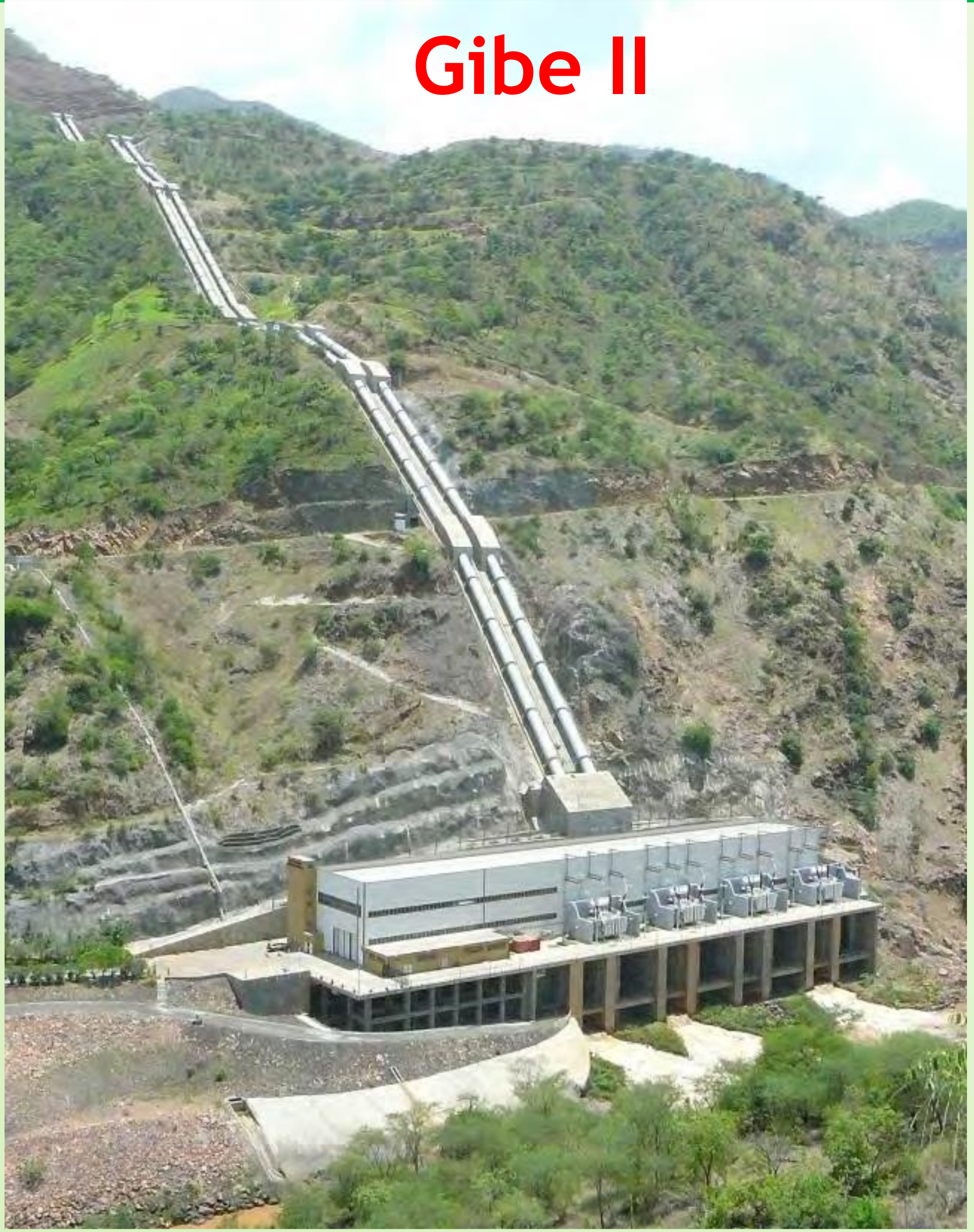
- The location of the forebay is primarily governed by topographic conditions and the geology of the site.
- The site of both the forebay and the powerhouse should be selected simultaneously with a view of ensuring the shortest possible penstocks/pressure shafts.
- The entire basin of the forebay may be either excavated in rock or constructed above the terrain, enclosed by embankments and retaining walls.
- The size of a forebay vary depending on the sediment content of the water conveyed in the power canal and whether it is to serve for storage.
- A gradual transition section should be provided between the power canal and the forebay basin. In the case of wide forebay, baffle piers are usually constructed at the basin inlet in order to ensure even distribution of flow to the basin.
- The bottom of the forebay basin should be provided with a proper slope to enable periodical flushing of the silt deposited.
- A bottom lining of the forebay basin is required in soils where large seepage is expected. The smoothed bottom of the basin is covered with plastic clay having thickness of 20 to 50 cm. The cover is compacted in several layers and is protected against disturbance due to soaking and wave action by a layer of gravel or crushed stone.

Design guidelines for a forebay *contd.*

- The spillway is usually an ogee type with stilling basin. If the discharge to be taken care of is great and if, at the same time, prevailing conditions do not permit the construction of a long overflow weir, water surface regulation within narrow limits can be attained by constructing a siphon type spillway.
- The spillway and the bottom outlet canal should be combined immediately at the foot of the basin. Water spilling over the spillway crest and through the bottom outlet can be either diverted into a suitable river bed (if any) in a nearby side valley or conveyed by a special chute.
- It is important to keep the entrance to the penstock fully submerged.
- The usual components of the intake such as trash racks, flow control devices (gates or valves), etc. must be provided at the penstock inlet.
- It is necessary to install an air vent behind the gate to prevent damage to the penstock if for some reason the penstock entrance is blocked or the gate is suddenly closed causing a low pressure inside the conduit. The air vent can also help remove air from inside the penstock during startup.

PENSTOCK

- The penstock is high pressure pipeline between forebay (surge tank or reservoirs) and the turbine.
- The design principle of penstocks are the same as that of pressure vessels & tanks but water hammer effect has to be considered.
- For short length, a separate penstock for each turbine is preferable. For a moderate heads & long distances a single penstock is used to find two or more turbines through a special branching pipe called Manifold.



Classification of penstock

Classification may based on:

- The material of construction
- Method of support
- Rigidity of connection and support
- Number of penstocks

Material of construction

- Factors for the choice of material are: head, topography & discharge.
- Materials used are steel, R.C., asbestos cement, PVC, wood stave pipes, banded steel, etc.

Factors which have to be considered when deciding which material to use for a particular project:

- Operating pressure
- Diameter and friction loss
- Weight and ease of installation
- Accessibility of site
- Cost of the penstock
- Design life
- Availability
- Weather conditions

Material of construction *contd.*

Steel penstocks: the most common type of installation, due to simplicity in fabrication, strength, and assurance that they will perform in a wide variety of circumstances. Normal practice is to use welded steel pipe sections.

Cast-in-place or precast reinforced concrete pipe: Very large diameters are somewhat impractical. Cast-in place concrete pipes are usually Limited to Heads of less than 35 m.

•According to Creagerand Justin (1950), these penstocks can be used up to 4m in diameter and under heads up to 185 m by using a welded steel shell embedded in the reinforced concrete

Fiberglass and polyvinyl chloride (PVC) plastic pipe : A penstock at the Niagara Mohawk plant uses a fiberglass pipe 3 m in diameter.

Wood stave pipes have been used in diameters ranging from 15 cm up to 6 m and utilized at heads up to 185 m with proper design.

Method of support

A penstock may be either buried or embodied underground (or inside dams) or exposed above ground surface & supported on piers

Advantages	**Disadvantages**
1. Continuity of support given by the soil provides better structural storability.	1- Difficulty in inspection
2. Pipe is protected from high temperature fluctuations	2- Possibility of sliding on step slopes
3. Conservation of natural land escape	3- Difficulty in maintenance
4. Protection from slides , storms & sabotage.	4- Expensive for loge diameter in rocky soils.

- Exposed penstocks: supported on piers or saddles

Advantages	Disadvantages
1- Ease in inspection of defects & maintenance	1- Direct exposure to weather effect
2- Economy in rocky terrain & large diameters.	2- Development of longitudinal stress due to support and anchorage, thus requiring expansion joints
3- Stability is insured with proper anchorage	

Rigidity of connection and support

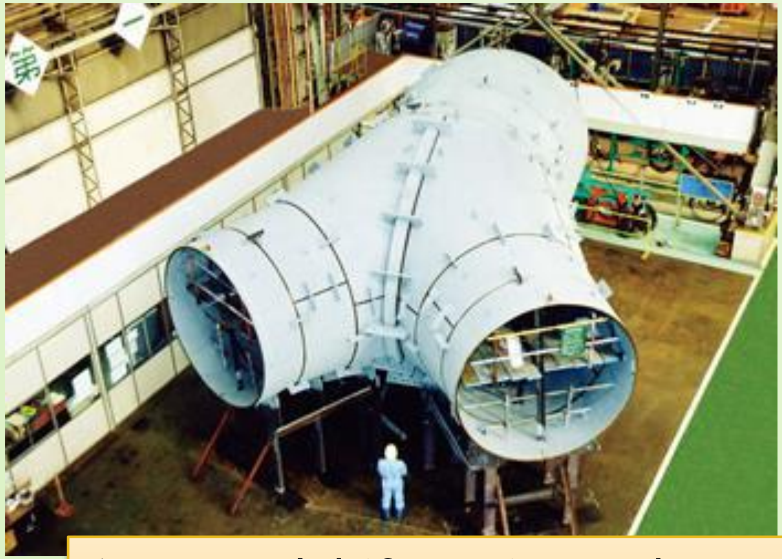
Rigid pipe support : Here every support is an anchorage so that any movement is checked. This type is suitable when the temperature variation is moderate.

Semi-rigid pipes: Here each member of the pipeline is fixed at one and leaving the possibility of movement over the other support.

Flexible support (Flexible or loose-coupled pipes): Here expansion joint are introduced between each adjacent section

Number of Penstocks

- The number of penstocks used at any particular installation can be single or multiple.
- The general trend at older power stations was to use as many penstocks between the forebay/surge tank and the powerhouse as the number of units installed.
- The recent trend is to use a single penstock, unless the size or thickness of the penstock involves manufacturing difficulties.



A penstock bifurcation under production at a fabrication plant Courtesy of Mitsubishi Heavy Industries, Ltd., Japan

The advantages of using a single penstock over the use of multiple penstocks are:

- The amount of material required to manufacture is less, making it economical.
- The cost of civil engineering components such as penstock supports and anchors is less.
- On the other hand, the use of a single penstock means reduced safety of operation and complete shutdown will become necessary in case of repair.
- In general, the use of multiple penstocks is preferably employed for low-head plants with short penstocks; whereas for high-head plants requiring long penstocks, provision of a single penstock with manifold at the end usually proves economical.

Permissible velocities

- 3 to 5 m/s (no abrasion property settled water) for properly settled water in exceptional cases up to 5m/s may be tolerated.

Safe penstock thickness

- The thickness of the pipe shell (s) for penstocks should be determined by:

$$t = \frac{\gamma H D}{2\eta\sigma}$$

where t = penstock shell thickness; γH= Static + water hammer pressure; D = pipe diameter; η = joint efficiency of welded or riveted joint; σ = allowable unit stress of hoop tension.

- The allowable equivalent unit stress for hoop tension will vary with the type of material used for the penstock.
- Minimum thickness of steel penstock (in inches), based on need for stiffness, corrosion protection, and strength requirements, is indicated by the U.S. Department of the Interior (1967) to be: $t_{min} = (D + 20)/400$

Size selection of penstocks

- Various experience curves and empirical equations have been developed for determining the economical size of penstocks.
- Some of these equations use very few parameters to make initial size determinations for reconnaissance or feasibility studies.
- Other more sophisticated equations use many variables to obtain more precise results which may be necessary for final design.
- Economical size varies with type of installation and materials, as well as whether it is used above ground or buried.

- Gordon and Penman (1979) give a very simple equation for determining steel penstock diameter for small hydropower installations: $D_p = 0.72Q^{0.25}$
- Sarkaria (1979) developed an empirical approach for determining steel penstock diameter by using data from large hydro projects with heads varying from 57 m to 313 m and power capacities ranging from 154 MW to 730 MW and reported that

$$D_p = 0.62p^{0.35}/h^{0.65}$$

where D = economical penstock diameter, m; p = rated turbine capacity, hp; h = maximum net head at the end of the penstock, m.

Size selection of penstocks (contd)

- USBR $v_{op} = 0.125\sqrt{2gh}$ where v_{op} is optimal capacity
- Donald's formula $D_p = 0.176\left(\frac{p}{h}\right)^{0.466}$
- Fahlbusch (1982) $D_p = 0.52h^{0.17}\left(\frac{p}{h}\right)^{0.43}$

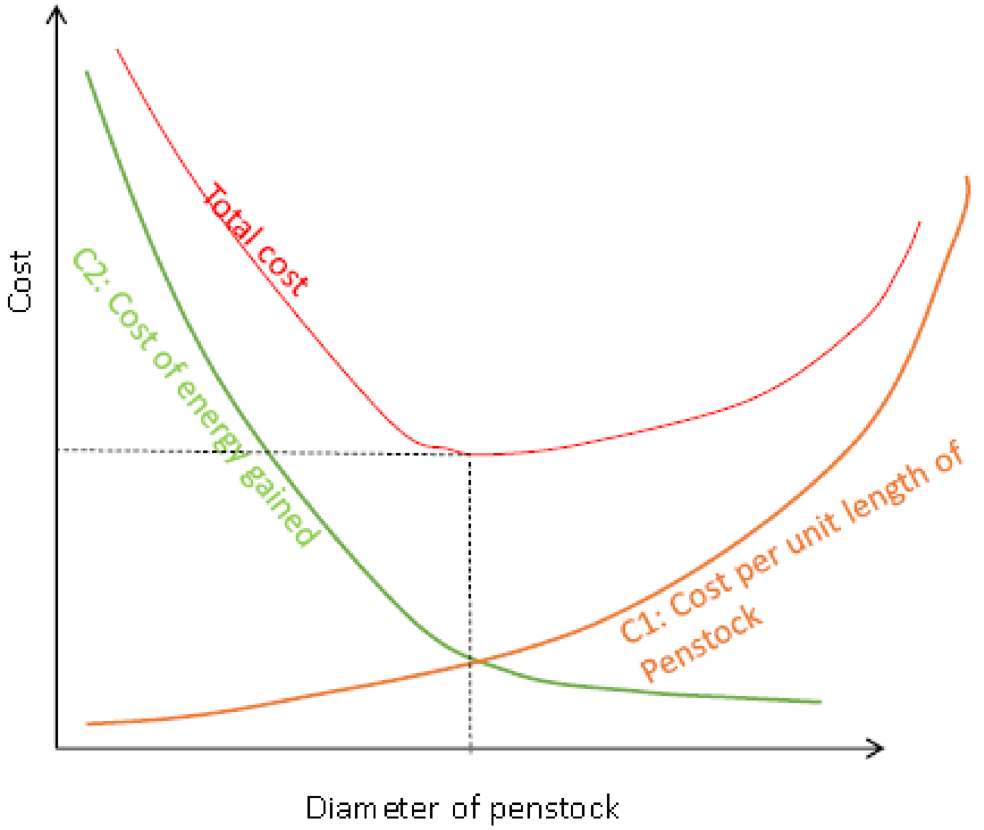
Graphical approach :

$D - f$ (capital cost, running cost)

Differentiating the total cost with respect to D and setting the result equal to zero will give us the economic diameter.

$$\frac{dC_1}{dD} > 0; \frac{dC_2}{dD} < 0$$

$$\frac{d}{dD}(C_1 + C_2) = \frac{dC_1}{dD} + \frac{dC_2}{dD} = 0$$



Size selection of penstocks using graphical approach (contd)

- Thickness of steel pipe: $t = \frac{\gamma H D}{2\sigma}$
- Weight of steel pipe: $G = g\rho_S \pi D t L$

Where

ρ_S-density of steel =7850kg/m3

ρ-density of water=1000kg/m3

H-head acting on the penstock

σ - allowable stress of steel

L - length of penstock

Substituting for t

$$G = g^2 \rho_S \pi D^2 \frac{\rho H}{2\sigma} L$$

$$G = 1187 * 10^3 \frac{D^2 H L}{\sigma} \; [kN]$$

Adding 20% for water hammer pressure surges

$$G = 1424 * 10^3 \frac{D^2 H_o L}{\sigma} \; [kN]$$

Size selection of penstocks using graphical approach (contd)

Let c_0 be cost of steel in USD per kN and α is annual operating charges on the penstock including depreciation, the annual costs of penstock:

- $C_1 = \alpha \cdot c_0 \cdot G$ *(USD per year)*

Substituting the previously defined expression for the weight of penstock G

$$C_1 = \alpha \cdot c_0 \cdot 1424 \cdot 10^3 \cdot \frac{h \cdot D^2 \cdot L}{\sigma}$$

Now that we have derived the expression for C_1 let us turn our attention to the expression for C_2.

Expression for C_2, cost of energy lost:

The cost of energy lost corresponds to the power calculated by considering head losses, where h_l stands for losses, is:

- $P_{lost} = 9.81 \text{ x } \eta \text{ x } Q \text{ x } h_l$ kW

Taking efficiency of say, 77% (this value can be adjusted for particular cases)

- $P_{lost} = 9.81 \text{ x } 0.77 \text{ x } Q \text{ x } h_l$

Major head loss in the penstock is given as:

$$h_l = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g} = f \cdot \frac{L}{D} \cdot \frac{Q^2 \cdot 16}{\pi^2 \cdot D^4 \cdot 2g} \approx \frac{f \cdot L \cdot Q^2}{12 \cdot D^5}$$

- Where *f is the friction coefficient for the pipe.*

For a given load factor of say λ given in decimal number, the total operation hours in a year would be = $8760 \text{ x } \lambda$.

Therefore, the annual energy generation would amount to:

- $E_{Lost} = P_{Lost} \cdot t = 7.55 \text{ x } Q \text{ x } h_l \text{ x } 8760 \text{ x } \lambda$

Size selection of penstocks using graphical approach (contd)

Taking the cost of energy per kWh at the existing tariff to be c_e

$$C_2 = \frac{7.55 \cdot Q^3 \cdot L \cdot f}{12 \cdot D^5} 8760 \cdot \lambda \cdot c_e = \left(0.63 \frac{Q^3 \cdot L \cdot f}{D^5}\right) 8760 \cdot \lambda \cdot c_e$$

Now applying the differentiation: $\frac{dC_1}{dD} + \frac{dC_2}{dD} = 0$

$$2848 \cdot 10^3 \cdot \frac{h \cdot L \cdot \alpha \cdot c_0}{\sigma} \cdot D + 0.63 \cdot (-5) \cdot \frac{f \cdot L \cdot Q^3}{D^6} 8760 \cdot \lambda \cdot c_e = 0$$

$$2848 \cdot 10^3 \cdot \frac{h \cdot L \cdot \alpha \cdot c_0}{\sigma} \cdot D = 27594 \cdot \frac{f \cdot L \cdot Q^3}{D^6} \lambda \cdot c_e$$

$$D = \sqrt[7]{\frac{f \cdot Q^3 \sigma}{103.2 \cdot h \cdot \alpha \cdot c_0} \lambda \cdot c_e} = \sqrt[7]{0.0097 \frac{f \cdot Q^3 \sigma}{h \cdot \alpha \cdot c_0} \lambda \cdot c_e}$$

D - optimum diameter of penstock (m); f - friction coefficient in pipe
Q - discharge in m^3/s; σ - allowable strength of steel (N/m^2)
α - annual operating charges on the penstock including depreciation
c_0 – unit cost of steel for finished penstock in USD per KN
λ - load factor; c_e – electric energy tariff in USD per kWh

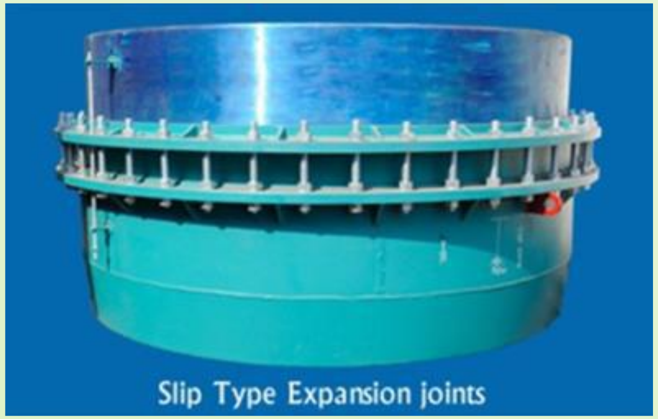
Penstock Joints

- Penstock pipes are generally supplied in standard lengths, and have to be joined together on site.
- There are many ways of doing this, and the following factors should be considered when choosing the best jointing system for a particular scheme.

Flanged Joints

•Flange jointed pipes are easy to install, but flanges can add to the cost of the pipe. Flange joints do not allow any flexibility.

•They are generally used to join steel pipes, and occasionally ductile iron pipes.



Spigot and Socket Joints

Spigot and Socket Joints

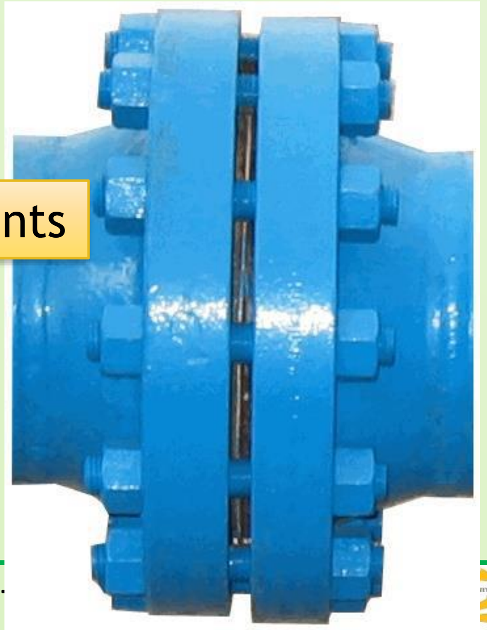
•Spigot and socket joints are generally used to join ductile iron, PVC, concrete, and asbestos cement pipes.

Welded Joints

•One advantage of welding on site is that changes in the direction of the pipe can be accommodated without preparation of a special bend section.

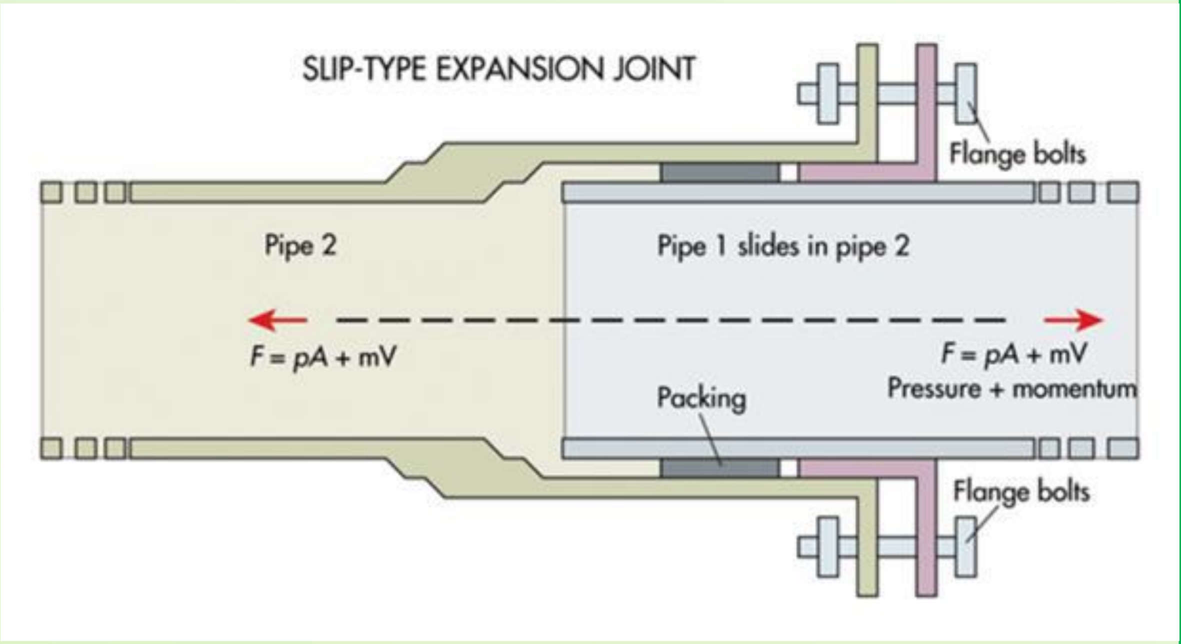
•It is relatively cheap method, but has the drawback of needing skilled site personnel.

Flanged Joints

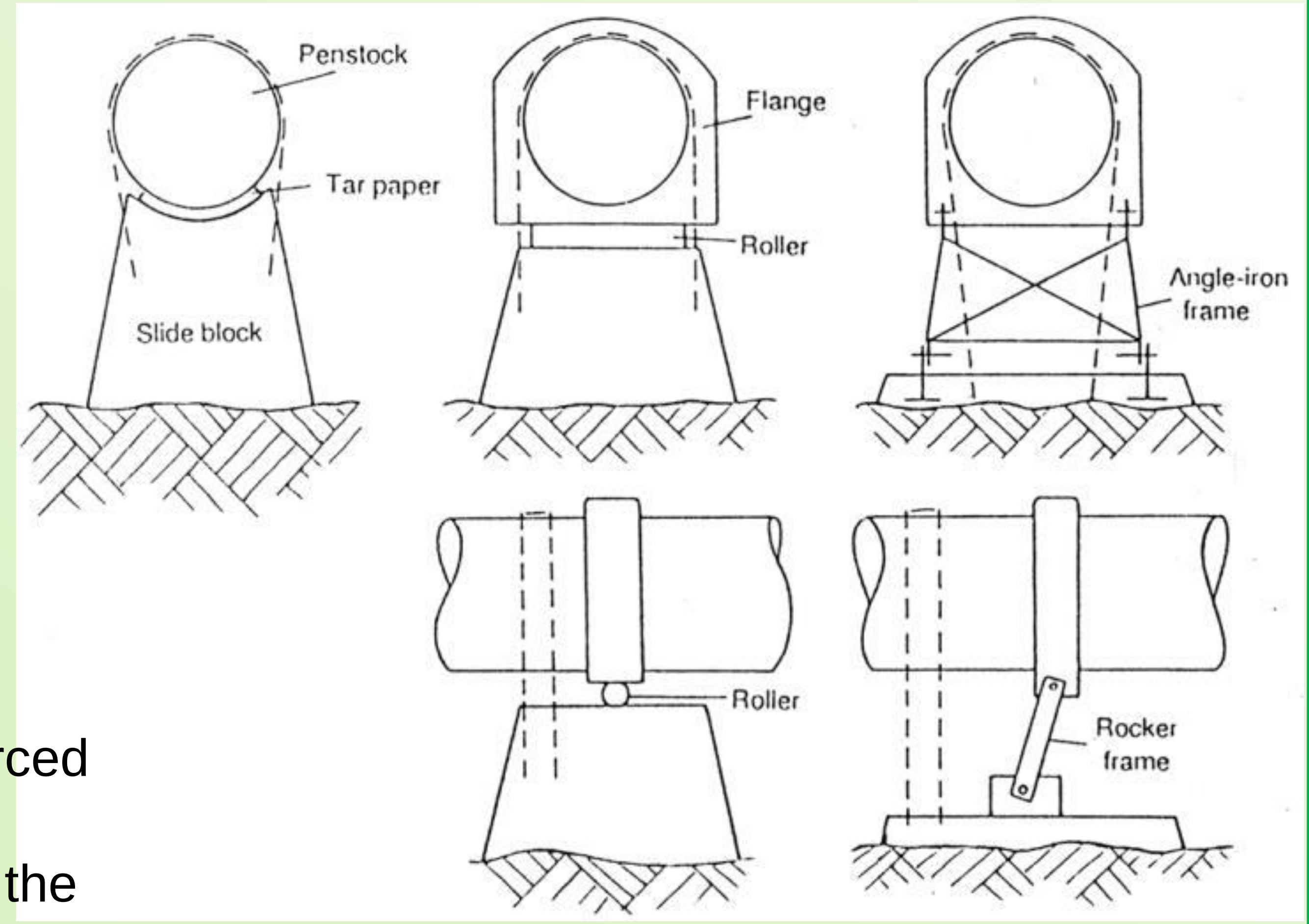


Expansion Joints

- A penstock, specially exposed ones, will change in length depending on temperature fluctuations.
- If it is fixed the thermal expansion forces are substantial. It is possible to relieve these forces by incorporating special joints called expansion joints, which allow the pipe to expand and contract freely.
- For short penstocks, provision of a single expansion joint may be sufficient, but for long penstocks with a multiple anchor blocks expansion joints should be placed below each anchor block.



Penstock Supports and Anchors



Slide Blocks

- A slide block, also called supporting pier, carries the weight of pipe and water, and restrains the pipe from upward and sideway movements, but allows it to move longitudinally.
- In most cases the spacing between slide blocks are assumed equal to the length of each pipe.

Anchor Blocks

- An anchor block consists of a mass of reinforced concrete keyed to the penstock so that the penstock cannot move in any way relative to the block.
- It is designed to withstand any load the penstock may exert on it.
- Anchors are often used at bends (horizontal and vertical) and before entrance to the powerhouse.
- They can also be used along long straight sections of penstock, each one next to expansion joint.

Fig. 4/93. Anchor detail showing hold-down straps for penstock, *Arnstein* power plant. (After *A. Schoklitsch*)

blocks at the *Cubatão* power station in Brazil. This was one of the reasons why engineers decided in favour of the underground arrangement for the proposed extension.

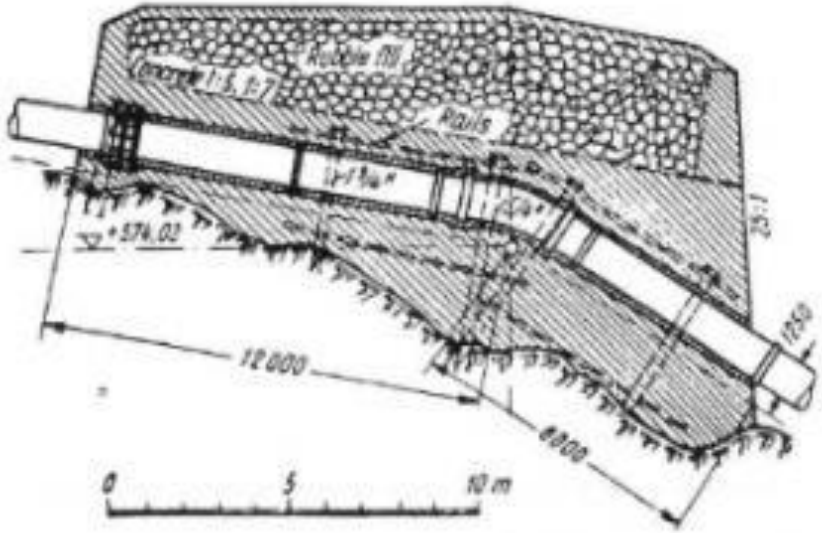


Fig. 5/93. Anchor block loaded by rock fill, *Rjukan* power plant, Norway. (After *A. Ludin*)

The major forces which act on anchor blocks are the following

- Weight of the pipe and enclosed water
- Hydrostatic force on a bend
- Friction forces on slide blocks located between the anchor and expansion joint
- Thermally induced stresses, when expansion joints are not incorporated
- The weight of the anchor block itself

Conditions of Stability for Supports and Anchors

The structure should be safe against sliding. For sliding not to occur:

$$\sum H < \mu \sum V$$

Where, $\sum H$ and $\sum V$, respectively, are the sum of all horizontal and vertical forces, and μ is the coefficient of friction between the structure and the ground often assumed as 0.5.

The structure should be safe against overturning. For this condition to be fulfilled, the resultant force should act within the middle third of the base.

$$e < \frac{L_{base}}{6}$$

Where, e is eccentricity of loading and L_{base} is length of the structure base.

1. *The dead weight of the pipe*, according to Eq. (24/92),

$$P'_s = + \Sigma G'_s \sin \beta_2 \quad [\text{kg}]. \qquad (1/93)$$

(Forces acting in a direction towards the anchor will be denoted hereafter as positive, and opposite ones as negative.)

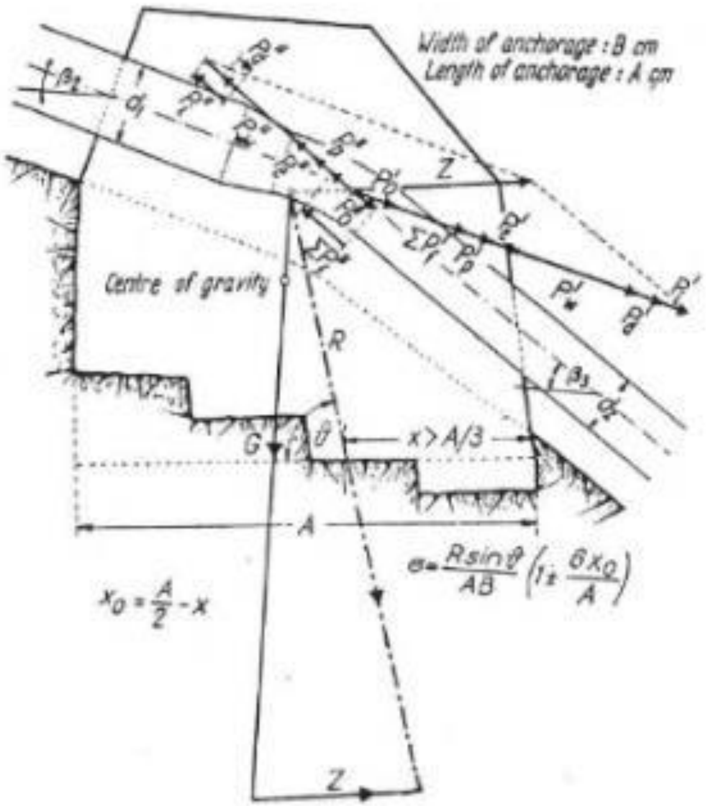


Fig. 3/93. Forces acting upon the anchor block

2. *The friction force over the supports*, according to (Eq. 25/92),

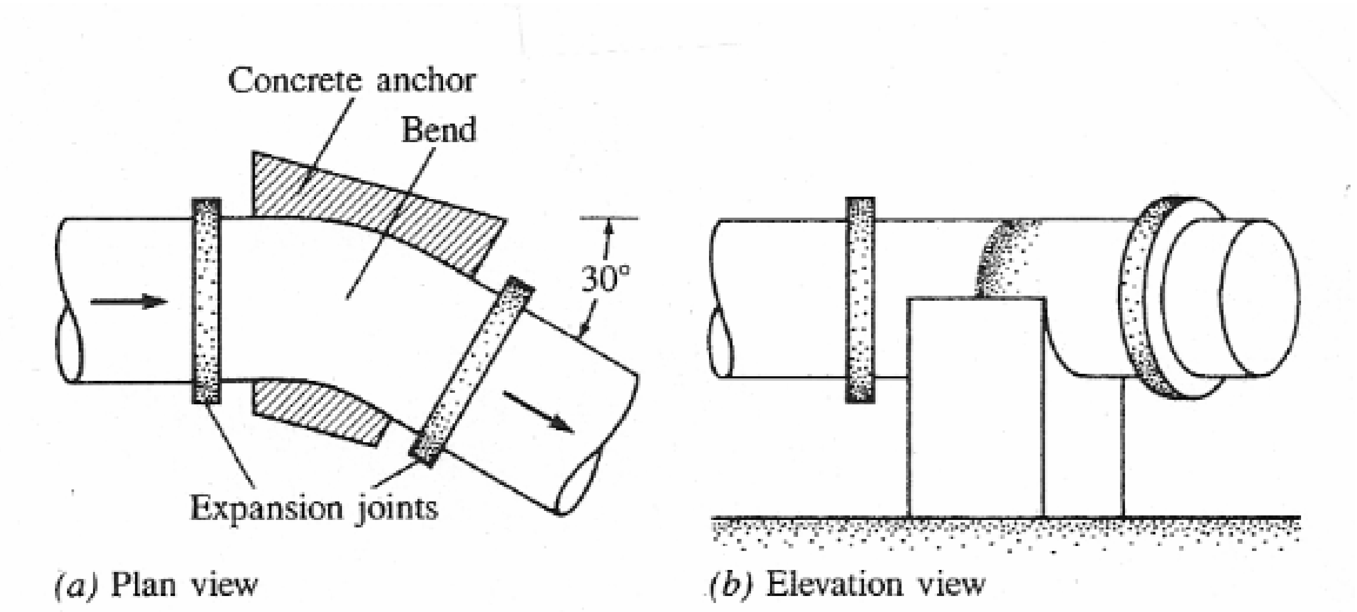
$$\Sigma P'_f = \pm \mu \Sigma (G'_s + G'_w) \cos \beta_2 \quad [\text{kg}] \qquad (2/93)$$

where the "+" sign applies to forces due to temperature increase, while the "—" sign indicates forces due to a temperature drop. These forces, as pointed out earlier, act along a line below the centreline of

- The pressure transmitted to the foundation must be within the safe bearing capacity of the foundation material. This can be expressed as:

$$\left\{\begin{matrix} Maximum\ \ pressure \\ by\ the\ structure \end{matrix}\right\} = \frac{\sum V}{A_{base}}\left(1+\frac{6e}{L_{base}}\right) < \left\{\begin{matrix} Bearing\ capacity\ of \\ the\ \ foundation\ \ soil \end{matrix}\right\}$$

Example: A 1 m diameter pipe has a 30^0 horizontal bend in it, and carries water at a rate of 3 m^3/sec. If we assume the pressure in the bend is uniform at 75 kPa gauge pressure, the volume of the bend is 1.8 m^3, and the metal in the bend weighs 4 kN, what forces must be applied to the bend by the anchor to hold the bend in place? Assume expansion joints prevent any force transmittal through pipe walls of the pipes entering and leaving the bend.



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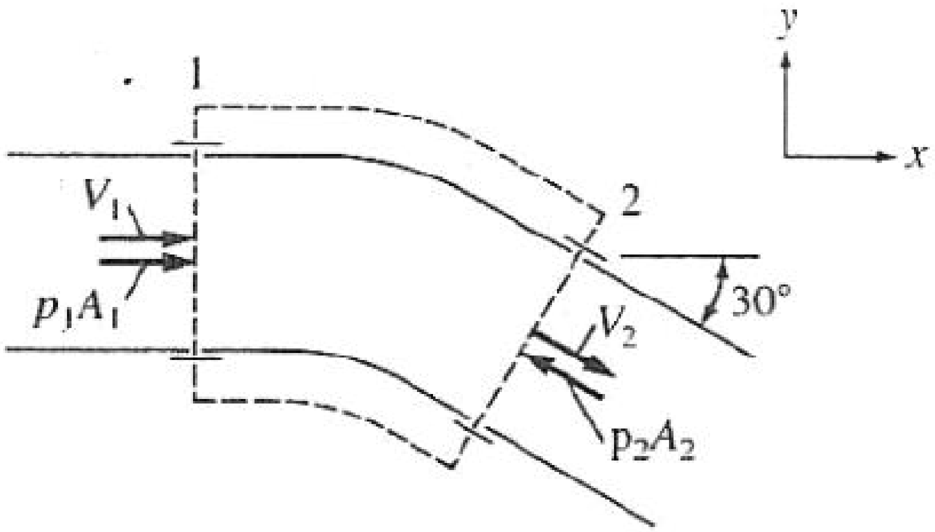
Solution: Consider the control volume shown in figure, and first solve for the x component of force,



$$\sum F_x = \rho Q(V_2 - V_1)$$

$$p_1 A_1 - p_2 A_2 \cos 30^0 + F_{anchor,x} = 1000 \times 3 \times (V_2 \cos 30^0 - V_1)$$

Where,

$$p_1 = p_2 = 75000 Pa$$

$$A_1 = A_2 = \frac{\pi D^2}{4} = 0.785 m^2$$

$$V_1 = V_2 = \frac{Q}{A} = \frac{3}{0.785} = 3.82\, m/\sec$$

$$F_{anchor,x} = 1000 \times 3.82 \times (\cos 30^0 - 1) + 75000 \times 0.785 \times (\cos 30^0 - 1)$$

$$F_{anchor,x} = -9423 N$$

Solve for F_y:

$$\sum F_y = \rho Q\left(V_2 \sin 30^0 - V_1\right)$$
$$p_2 A_2 \sin 30^0 + F_{anchor,y} = \rho Q\left(-3.82 \sin 30^0\right)$$
$$F_{anchor,y} = -1000 \times 3 \times 3.82 \times \sin 30^0 - 75000 \times 0.785 \times \sin 30^0$$
$$F_{anchor,y} = -35168 N$$

Solve for F_z:

$$\sum F_z = \rho Q\left(V_{2z} - V_{1z}\right)$$
$$W_{bend} + W_{water} + F_{anchor,z} = 0$$
$$F_{anchor,z} = 4000 + 1.8 \times 9810 = 21680 N$$

Then the total force that the anchor will have to exert on the bend will be,

$$F_{anchor} = -9420\vec{i} - 35168\vec{j} + 21658\vec{k}$$