CE412 A
Water Supply & Wastewater Disposal Systems

Part I_Water

**Instructor :
Dr Vinod Tare**

**CE412A: WATER SUPPLY AND WASTEWATER DISPOSAL SYSTEMS
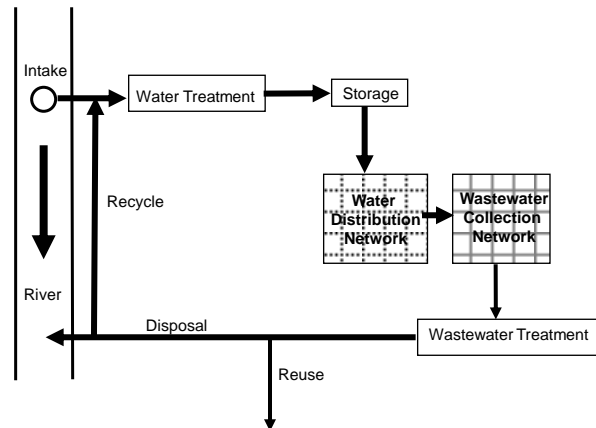2019-2020, 2nd Semester**

Instructor: Dr Vinod Tare

Lectures: M 12:00 - 13:15 & Tu 09:00 – 10:15; Venue: L-13

Tutorial: M 14:00 - 15:50; Venue: L-14

Marks Distribution: Quiz (Four) 20; Mid Semester: 30; End Semester: 50

Urban Water Cycle


Disposal, Reuse and Recycling

Due to the addition of impurities during beneficial use, water is transformed to wastewater.

This wastewater may be discharged to aquatic or terrestrial environments provided the relevant discharge standards are attained through wastewater treatment. This is known as wastewater disposal.

Alternatively, the wastewater may be used for certain other beneficial purpose, provided adequate treatment is given to the wastewater to conform to the water quality standards for that beneficial use. This procedure of using water for multiple beneficial uses is known as reuse.

Alternatively, the wastewater may be used for the same beneficial purpose as before, provided adequate treatment is given to the wastewater such that it again conforms to the standards for the original beneficial use. This procedure for using water for the same beneficial use is known as recycling.
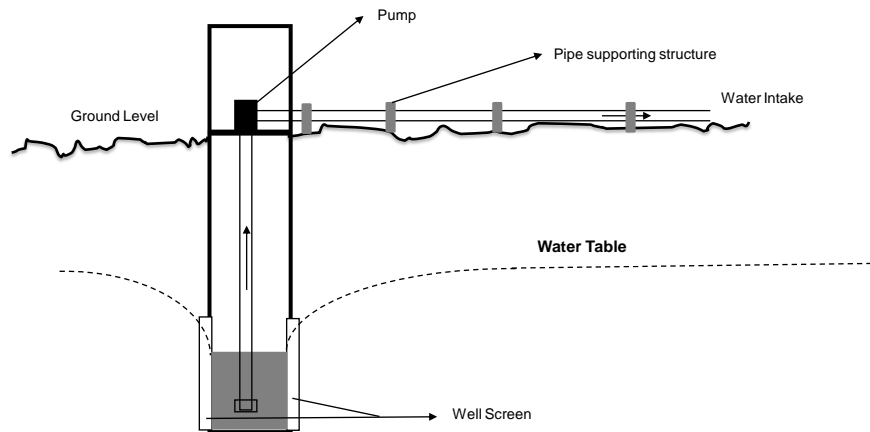
Water Intake Structure: Perennial River
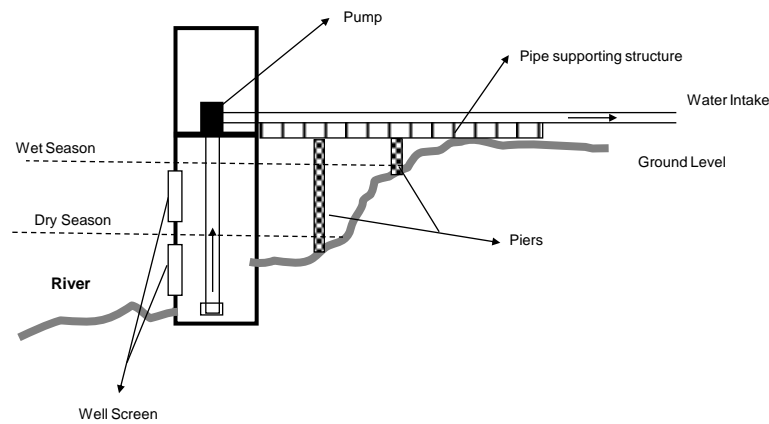
Water Intake Structure: Dry River

3
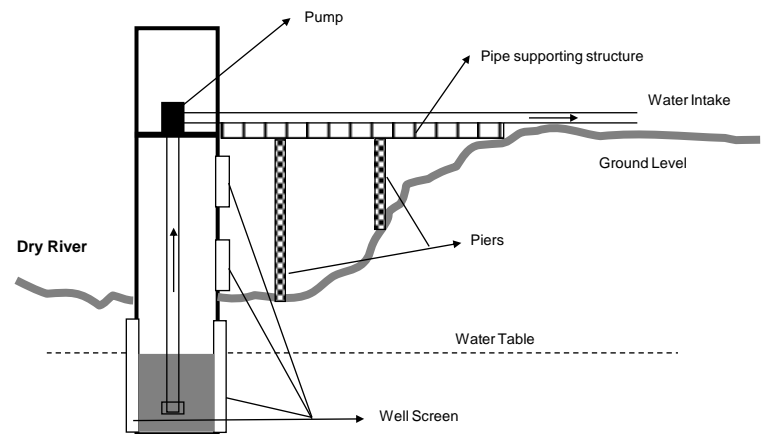
Water Intake Structure: Deep Tube Wells

Water Quantity

Determine present resident population
Present average domestic water demand = 135 lpcd

Estimate present temporary population
Present temporary water demand = 40 lpcd

Estimate future population
Future average domestic water demand = 235 lpcd

Estimate future temporary population
Future temporary water demand = 60 lpcd

Commercial demand = 50 percent of domestic demand

Horticultural demand = 0.5 cm-acre / d

Average Daily Demand = Domestic + Temporary + Commercial + Horticultural

Water Quantity

Determine present resident population
Present average domestic water demand = 135 lpcd

Estimate present temporary population
Present temporary water demand = 40 lpcd

Estimate future population
Future average domestic water demand = 235 lpcd

Estimate future temporary population
Future temporary water demand = 60 lpcd

Commercial demand = 50 percent of domestic demand

Horticultural demand = 0.5 cm-acre / d

Average Daily Demand = Domestic + Temporary + Commercial + Horticultural

Natural Contaminants in Surface Water and Ground Water Samples

The concentration of inorganic particulate matter, i.e., sand, silt and clay is high in surface water samples. Surface water may also contain dissolved natural organic matter (NOM)

Concentration of dissolved contaminants, i.e., anions (e.g., chloride, sulfate, nitrate, bicarbonate, fluoride, etc.) and cations (e.g., sodium, calcium, magnesium, iron, arsenic, etc.) are high in ground water samples.

Origin of the Contaminants

Eroded soil is carried by surface runoff into natural water bodies resulting in high sand, silt and clay concentration in surface water. Surface runoff over decaying vegetation contributes to dissolved NOM

Passage of water through rock formations containing various minerals result in leaching of cations and anions into ground water samples.

Anthropogenic Impacts

Fecal contamination of surface water results in the addition of organic matter (responsible for color, odor etc.) and bacteria/viruses. Surface runoff may add nitrates, pesticides etc,. To surface water.

Dissolved contaminants such as nitrates, pesticides, other synthetic organic compounds, cations/anions, fecal organic matter etc. are added to ground water (only the top aquifer) due to anthropogenic impacts.

Main Contaminants in Surface Water	**Main Contaminants in Ground Water**
Natural Origin • Sand, Silt and Clay • Natural Organic Matter (NOM)	**Natural Origin** • Dissolved Cations and Anions
Anthropogenic Origin • Fecal organic matter • Bacteria, viruses	**Anthropogenic Origin (only top aquifer)** • Agricultural pollutants • Other synthetic organic pollutants • Nitrates, Ammonia (from fecal pollution) • Bacteria, viruses

Surface Water vs Ground Water

Surface water is available at a higher level than ground water. Hence pumping costs for surface water supply are lower.

To get a reliable supply of ground water, a deep tube well must be dug. This is costly. Further, there is no guarantee that a deep tube well dug at a certain location will give sufficient yield. Further, the yield from a well generally decreases with time. Hence, multiple deep tube wells must be operated simultaneously to ensure reliable water supply. In addition, new wells must be dug every now and then, as the old wells must be abandoned when the yield from them becomes unsatisfactory. All these issues make the supply of groundwater more difficult and expensive than surface water.

Turbidity and microbial quality are major issues with surface water. However, these problems can be solved through conventional water treatment and potable water supplied.

Pollutants in groundwater are dissolved in nature. Such pollutants are generally more difficult and expensive to remove by water treatment. Hence wells yielding polluted groundwater are generally abandoned and new wells must be dug.

Contaminants vs Pollutants

Contaminants:	Impurities present in the surface and ground water.
	(By this definition, all water in contaminated or contains impurities)

Pollutants: When the concentration of a contaminant (impurity) in the water is higher than the maximum allowable limit for a particular beneficial use, that contaminant is called a pollutant.

Example: say, maximum allowable limit of copper (a contaminant) in drinking water is 5 mg/L. So a surface or groundwater sample with 10 mg/L copper is unfit for drinking and copper will be considered a pollutant vis-vis this beneficial use.

say, the maximum allowable limit of copper in water used in a particular industry is 50 mg/L. So a surface or groundwater sample with 10 mg/L copper is fit for this industrial use and will not be considered a pollutant vis-à-vis this beneficial use.

Water Quality Standard: A list of maximum allowable level of various contaminants in water for a beneficial use is known as the Water Quality Standard for that Beneficial Use. Example: Drinking Water Quality Standard, Water Quality for various Industries; Irrigation Water Quality Standard, River Water Quality Standard, etc.

Standard

Objective of Water Treatment: To remove various contaminants from water, such that the water is suitable for the intended beneficial use, i.e., the Water Quality Standard for that Beneficial Use is met.
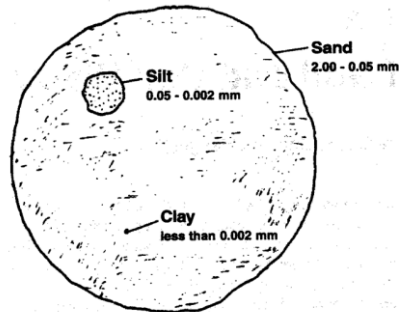
Surface Water Treatment Objectives: for Potable Water

Treated water must be potable, i.e., free from

• Color
• Odor
• Taste
• Turbidity (< 1 NTU)
• Pathogens (absent)
• Other harmful substances (as per drinking Water Quality Standards)

Treatment Process Train

UO – Unit Operation
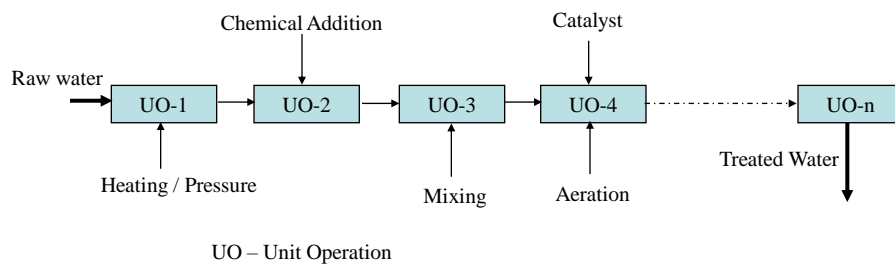
Sand, Silt and Clay

Transport of Sand, Silt and large organic particles in flowing water
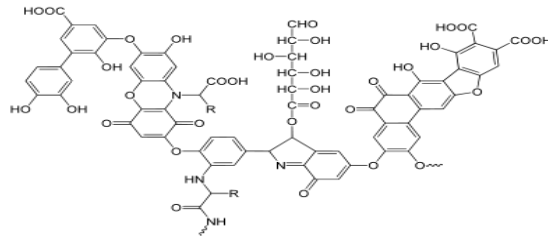
Transported through
• Advection
• Interplay of settling and scouring.

> **Settling:** Downward motion of particles suspended in water due to gravitational forces
> **Scouring:** Re-suspension of settled particles by flowing water

**Humic Substances
(Natural Organic Matter)**

Transport of Clay, Bacteria/virus and NOM/fecal organic matter and other dissolved contaminants in flowing water
• No settling and scouring

Transported through
• Advection
• Dispersion

Fate of Contaminants in Water
•Large particles removed/added through settling scouring
•Small particles removed through coagulation
•Fecal organic matter is biodegradable and removed through biodegradation
•NOM is not easily biodegradable
•Inorganic cations/anions are non-biodegradable
•Bacteria/virus may die off / grow in flowing water depending on conditions

Particles in Water: Settling

GRAVITY + BOUYANT FORCE

$$F_G = (\rho_P - \rho)\ g\ V_P \quad \text{volume of particle}$$

density of particle — density of fluid — gravitational constant

DRAG FORCE

$$F_D = \frac{C_D\ A_P\ \rho\ v_S^2}{2}$$

drag coefficient — area of particle — fluid — settling velocity

COMBINING:

$$v_S = \sqrt{\frac{2\ (\rho_P - \rho)\ g\ V_P}{C_D\ A_P\ \rho}}$$

For Spherical particles:

$$v_S = \left[\frac{4}{3}\cdot g \cdot \frac{(\rho_p - \rho)}{\rho}\cdot\frac{d}{C_D}\right]^{1/2}$$

$$C_D = \frac{24}{N_R} \qquad \text{(For } N_R < 1)$$
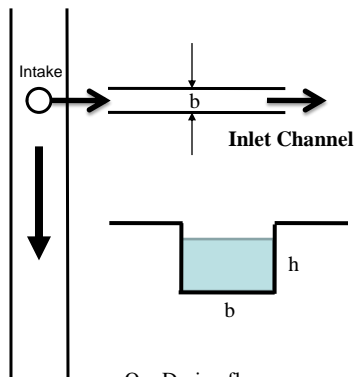
$$N_R = \frac{v_S\cdot d}{\nu}$$

$$v_S^2 = \frac{4}{3}\cdot g\cdot\frac{(\rho_p - \rho)}{\rho}\cdot\frac{d}{C_D} = \frac{4}{3}\cdot g\cdot\frac{(\rho_p - \rho)}{\rho}\cdot\frac{d}{24.\nu}\cdot v_s d$$

$$v_S = \frac{g}{18}\frac{(\rho_p - \rho)}{\rho}\cdot\frac{d^2}{\nu}$$

$$C_D = \frac{18.5}{(N_R)^{0.6}} \quad \text{For } N_R \text{ values of 1 - 1000}$$
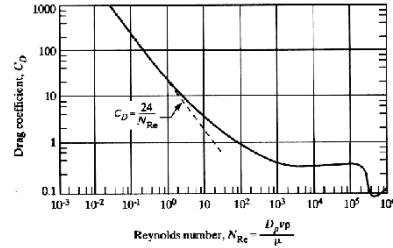
Particles in Water: Scouring: Re-suspension of settled particles in flowing water

$$v_{sc} = 4.\sqrt{g.\left(\frac{\rho_s - \rho}{\rho}\right).d}$$

Intake

Inlet Channel

Q = Design flow

b = Channel width

h = depth of flow

S = channel slope

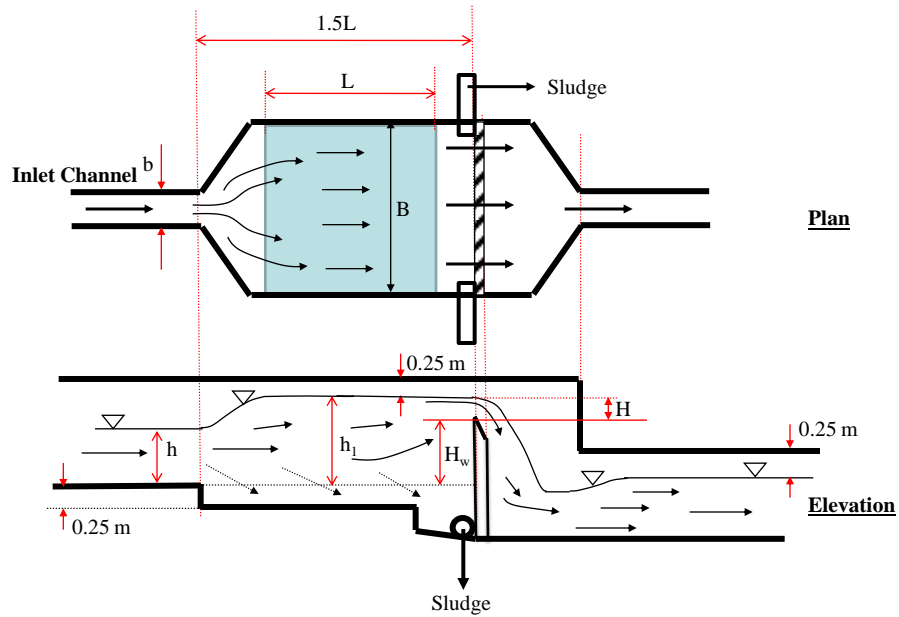A = flow cross section

V_H = flow velocity

Inlet Channel:

No particle <2.5 mm in diameter must settle in the inlet channel

$$V_{sc} \text{ of 2.5 mm particle} = 4.\sqrt{g.\left(\frac{\rho_s - \rho}{\rho}\right).d}$$

$$= 4.\sqrt{9.81.\frac{(2650-1000)}{1000}.(0.0025)} = 0.805 \text{ m/s}$$

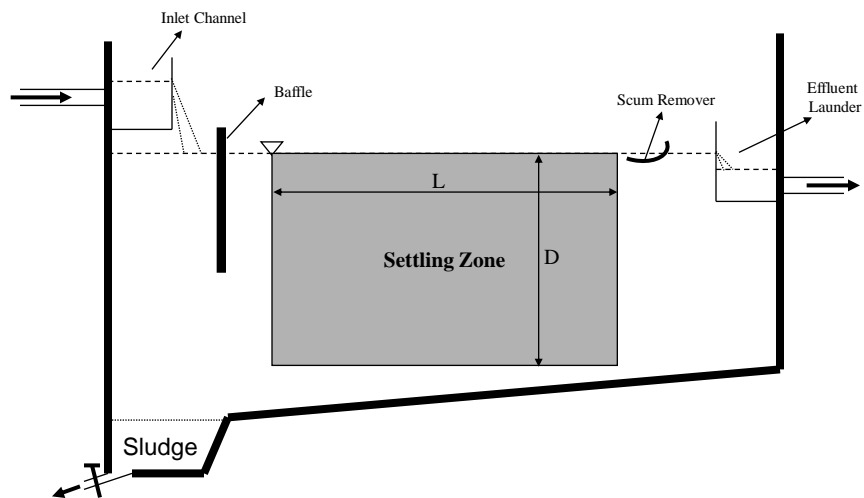$$\boxed{V_H > V_{sc}}$$

Silt Excluder: To remove particles up to 0.25 mm in diameter

V_{sc} (0.25 mm particles) = 0.254 ms^{-1};   Thus, $V_H < V_{sc}$

1.5L

L

Sludge

Inlet Channel

b

B

Plan

0.25 m

H

0.25 m

h

h$_1$

H$_w$

0.25 m

Elevation

Sludge

Primary Sedimentation Tank
(To remove particles up to 20 μm size completely)

Inlet Channel

Baffle

Scum Remover

Effluent
Launder

L

Settling Zone

D

Sludge

Primary Sedimentation Tank (PST): Design Theory

Objective: To remove particles up to 20 μm size completely.

$V_{sc} = 0.072$ m s^{-1}; $V_s = 3.56 \times 10^{-4}$ m s^{-1}

Detention Time (T) = average time for which water stays in the tank = V/Q

V = Volume of the tank;   Q = Flowrate

Surface overflow rate (SOR) = Q/A_s; A_s = surface area of the tank

For a rectangular tank, L = length, B = width; D = depth

SOR = Q/(L.B);   Detention time (T) = V/Q = D/V_s = (L.B.D)/Q
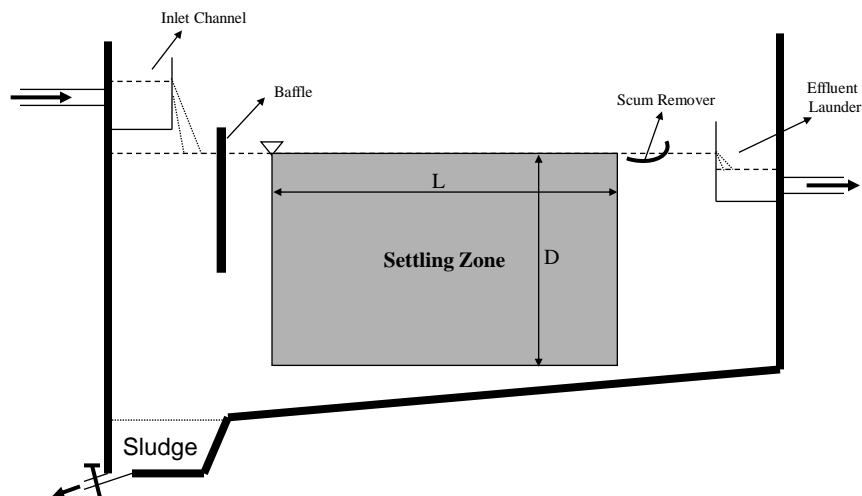
$V_s = Q/(L.B) = Q/A_s$;

In a rectangular sedimentation tank, the critical settling velocity is always numerically equal to the SOR

For a 20 μm particle, $V_s = 3.56 \times 10^{-4}$ m s^{-1}, i.e., SOR = 30.77 m^3/m^2/d

A PST is generally designed with SOR of 30-50 m^3/m^2/d

$V_H = Q/(B.D)$; $V_H < V_{sc}$

Primary Sedimentation Tank
(To remove particles up to 20 μm size completely)

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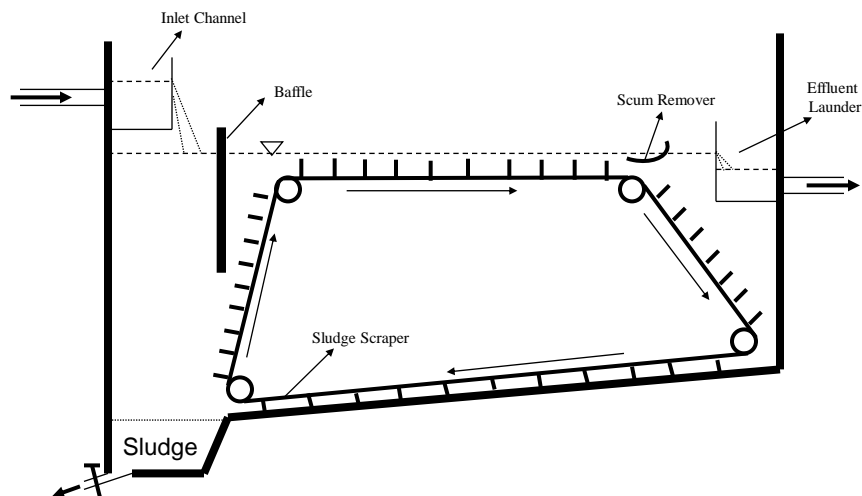
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Primary Sedimentation Tank
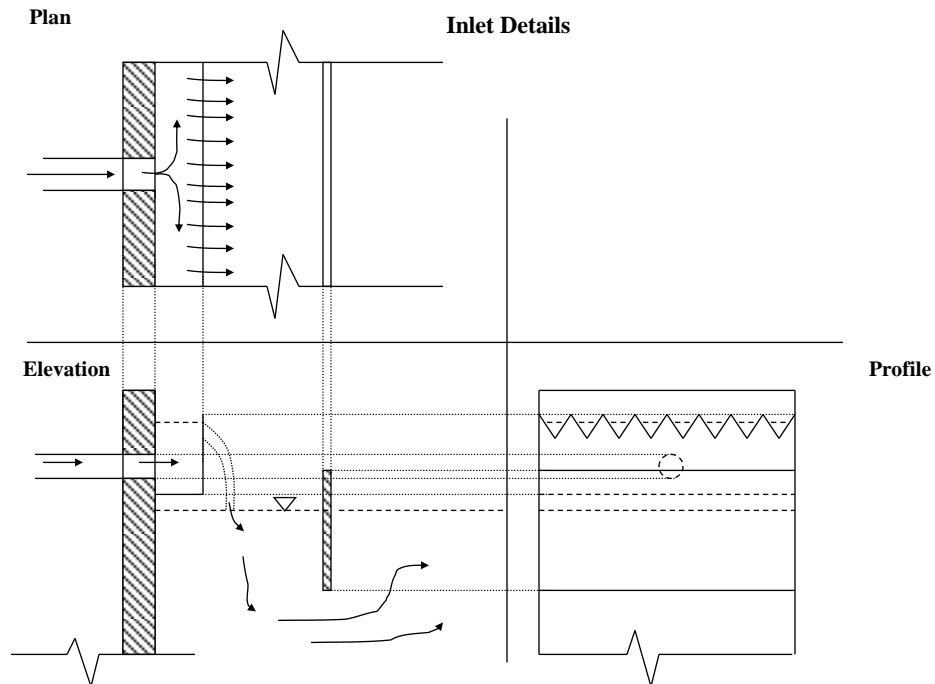**(Sectional Elevation)**

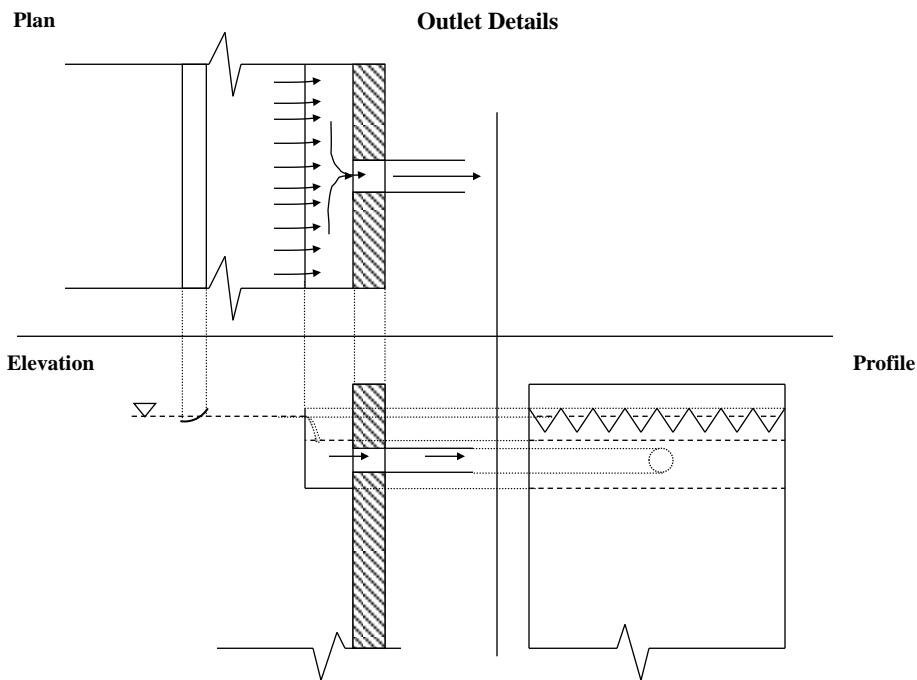

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Apparent Irrelevance of Depth in PST

Assume flow (Q), length (L) and breath (B) of the PST is constant. Critical settling velocity (v_s), is the settling velocity of the smallest particle that will be completely removed in the PST. We know, $v_s = SOR = Q/(L.B)$. Thus, v_s is not a function of depth (D). Hence from purely settling considerations, the depth of PST has no effect on the PST efficiency.
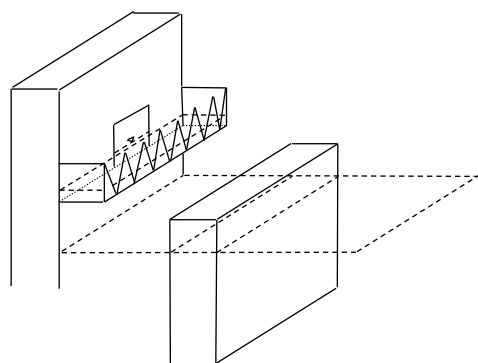
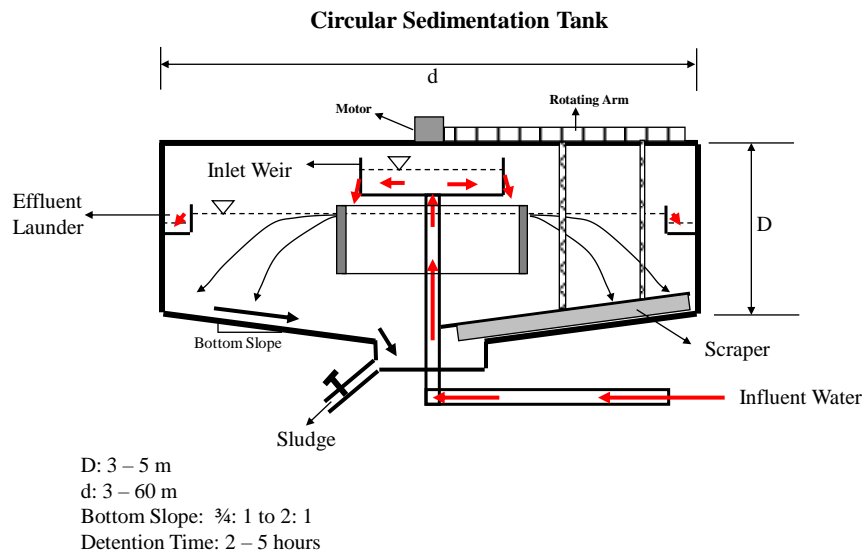**However in Practice…….** 3-5m depth is provided,

Why??
• Assuming flow (Q), length (L) and breath (B) of the PST is constant, provision of lower depth (D) will mean that the horizontal velocity of flow, $v_H = Q/(B.D)$ will be high. Higher the value of v_H, higher the size of the largest particle that will be scoured. Hence the settling tank efficiency goes down when D is low. Hence the minimum depth provision in the PST.

• A shallow PST will be susceptible to wind effects, i.e., the calm conditions required for efficient PST operations cannot be maintained in shallow tanks when there is a stiff breeze.

• However, a PST should be made as shallow as practicable, considering density currents.
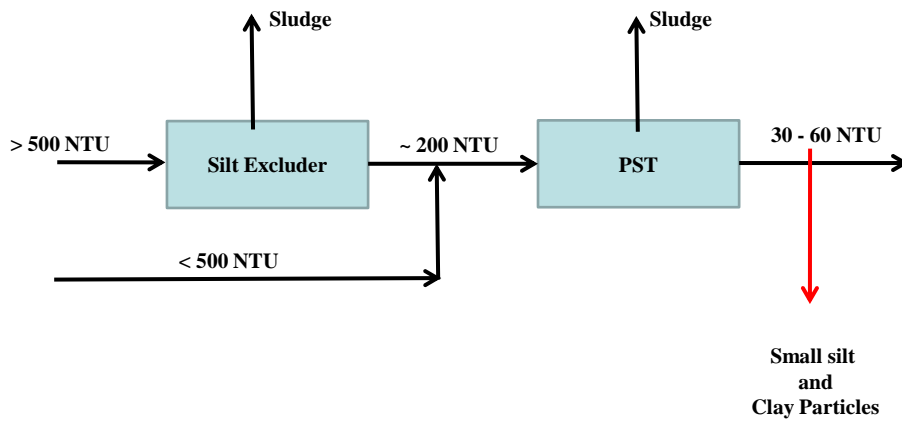
Plan

Inlet Details

Elevation

Profile

Plan

Outlet Details

Elevation

Profile

Inlet Weir

Circular Sedimentation Tank

D: 3 – 5 m
d: 3 – 60 m
Bottom Slope: ¾: 1 to 2: 1
Detention Time: 2 – 5 hours

Preliminary and Primary Treatment

Coagulation-Flocculation

- Particles > 20 μm size have been completely removed through PST

- Turbidity of water after Primary Sedimentation is still 30 – 60 NTU

- Turbidity caused by particles < 20 μm size, i.e., small silt particles and clay particles.

- Such particles do not settle. They exhibit Brownian (random) motion in water.

How can they be removed from water ????

Solution 1:
Suppose these particles can be made to agglomerate (stick to each other) by some means (e.g., changing solution chemistry) and form bigger particles, which will then settle.
Problem: these particles do not normally agglomerate. They are stabilized. **Why?? Can they be made to agglomerate??**

Solution 2:
Suppose, we add a chemical to water, which can attach to one or more stabilized silt/clay particles, resulting in the combined entity being destabilized. These destabilized entities can then agglomerate to form settleable particles.

Solution 3:
Suppose we add stabilized/destabilized particles to water, which interact with stabilized silt/clay particles, such the combined entity is destabilized. These destabilized entities can then agglomerate to form settleable particles.

Coagulation-Flocculation

The process of destabilization of stable particles is called **<u>Coagulation</u>**

The process of agglomeration of destabilized particles is called **<u>Flocculation</u>**

Important Questions:

Why are silt/clay particles stable??

How can they be destabilized, i.e., coagulated??

If destabilized, why and how will they flocculate??

How can the coagulation – flocculation process be incorporated in water treatment?? Design issues.

The process of settling of agglomerated particles is called **<u>Secondary Sedimentation</u>**

Origin of Clay Particles

The contact of rocks and water produces clays, either at or near the surface of the earth.

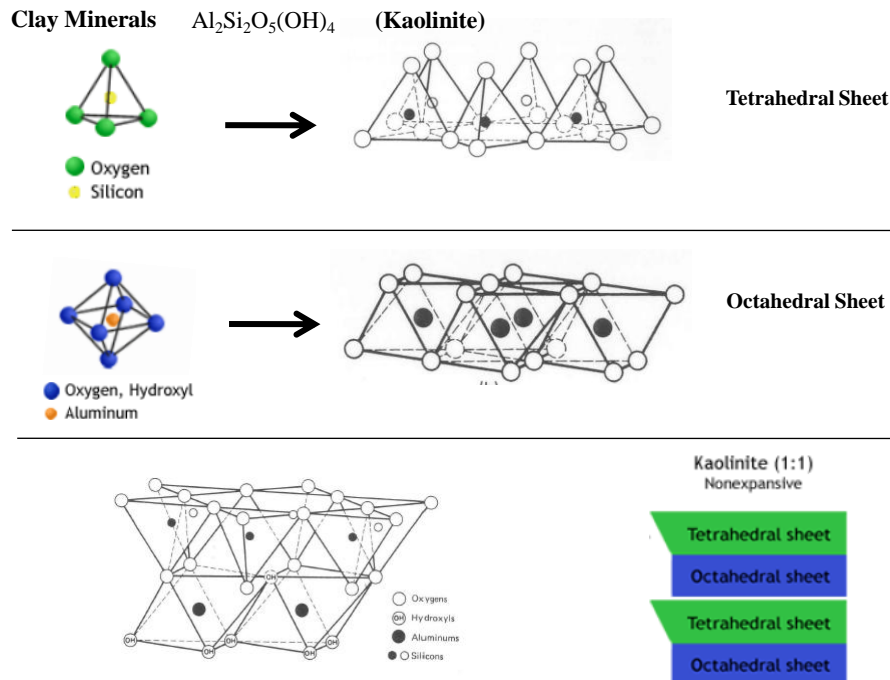$$\textbf{Rock + Water} \rightarrow \textbf{Clay}$$

For example,

The CO_2 gas can dissolve in water and form carbonic acid, which will become hydrogen ions H^+ and bicarbonate ions, and make water slightly acidic.

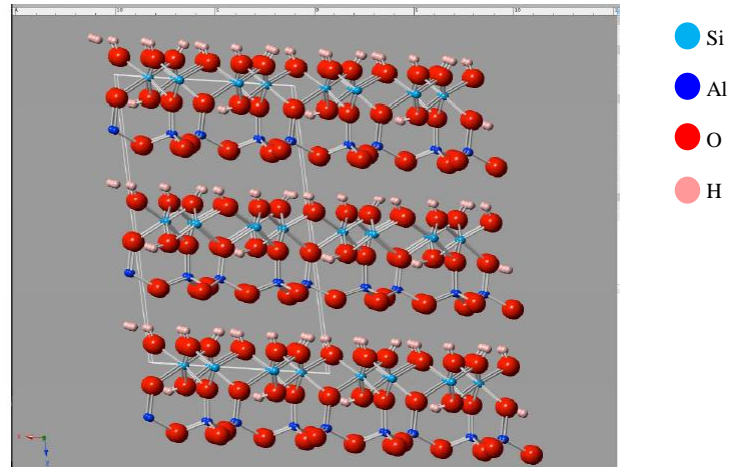$$CO_2 + H_2O \rightarrow H_2CO_3 \rightarrow H^+ + HCO_3^-$$
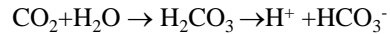
The acidic water will react with the rock surfaces and tend to dissolve the K ion and silica from the feldspar (a constituent of granite). Finally, the feldspar is transformed into kaolinite.

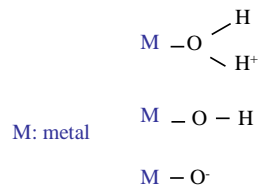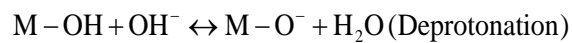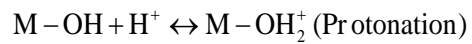Feldspar + hydrogen ions + water $\rightarrow$ clay (kaolinite) + cations, dissolved silica

$$2KAlSi_3O_8 + 2H^+ + H_2O \rightarrow Al_2Si_2O_5(OH)_4 + 2K^+ + 4SiO_2$$

Clay Minerals $\quad Al_2Si_2O_5(OH)_4 \quad$ **(Kaolinite)**

Tetrahedral Sheet

○ Oxygen
○ Silicon

Octahedral Sheet

● Oxygen, Hydroxyl
● Aluminum

○ Oxygens
OH Hydroxyls
● Aluminums
● ○ Silicons

Kaolinite (1:1)
Nonexpansive

Tetrahedral sheet
Octahedral sheet
Tetrahedral sheet
Octahedral sheet

**Permanent Charge on Clay Particles**

Si
Al
O
H

Atoms in crystal structure are often substituted
during the formation of the mineral.

(Al^{+3} for Si^{+4} ; Mg^{+2} for Al^{+3} ; Fe^{+2} for Al^{+3})
This results in permanent –ve charge

pH-Dependent Charge on Clay Particles

$$M - OH + H^+ \leftrightarrow M - OH_2^+ \,(Pr\,otonation)$$
$$M - OH + OH^- \leftrightarrow M - O^- + H_2O\,(Deprotonation)$$

$$M - O \begin{array}{c} \diagup H \\ \diagdown H^+ \end{array}$$

M: metal

$$M - O - H$$

$$M - O^-$$

Kaolinite particles are positively charged on their outer periphery when in a low pH environment, but negatively charged in a high pH (basic) environment.
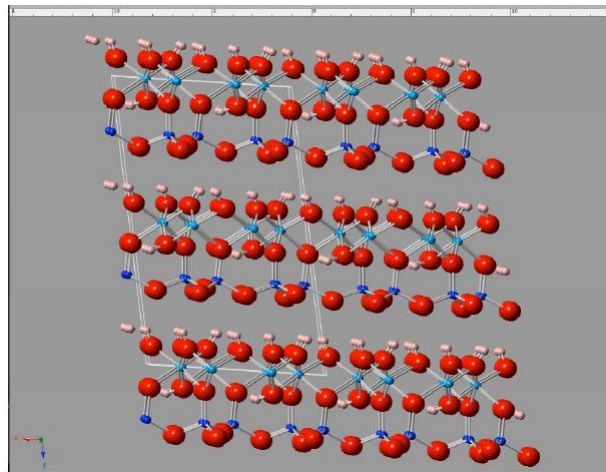
Why are silt/clay particles stable??

The negatively charged clay particle is surrounded by a water layer containing excess positive charge.

This water layer is called the diffused layer.

The negatively charged clay particle along with its diffused layer is suspended in the bulk phase, which is electrically neutral.
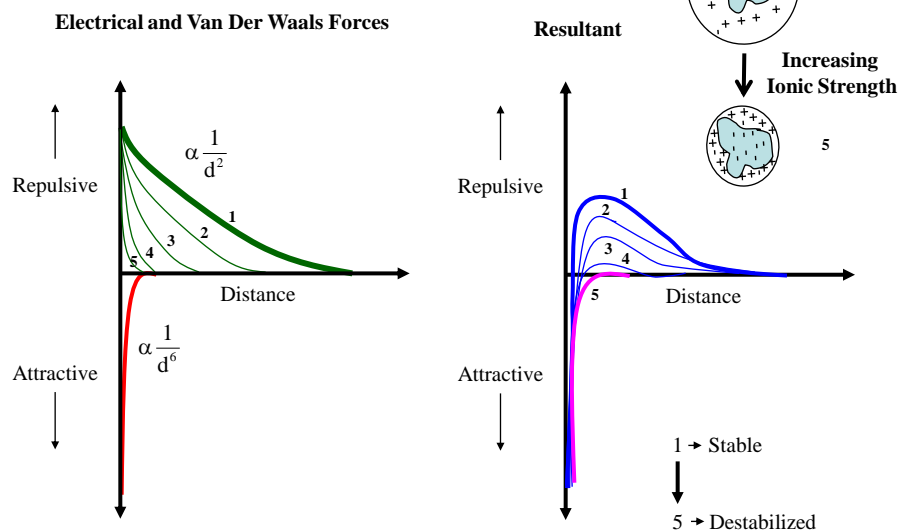
When two such particles approach each other, their diffuse layers repel each other due to electrostatic forces, hence the particles cannot agglomerate. A suspension of such particles in water is known as a stabilized suspension
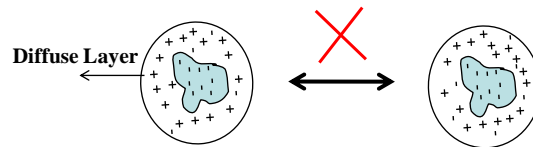
Solution 1:
Suppose these particles can be made to agglomerate by some means (e.g., changing solution chemistry) and form bigger particles, which will then settle
- Increasing ionic strength
- Reducing pH

Electrical and Van Der Waals Forces

Resultant

$\alpha \dfrac{1}{d^2}$

$\alpha \dfrac{1}{d^6}$

1 → Stable

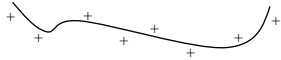5 → Destabilized

Solution 2:
Suppose, we add a chemical to water, which can attach to one or more stabilized silt/clay particles, resulting in the combined entity being destabilized. These destabilized entities can then agglomerate to form settleable particles.
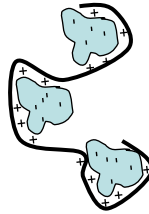
Cationic Polymers as Coagulants

• Charge Neutralization
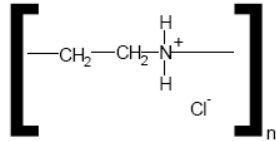
• Inter-particle Bridging

Polyethyleninine
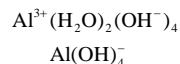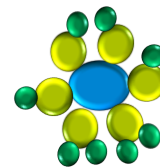
Optimal Polymer dose is important to prevent re-stabilization

Solution 3:
Suppose we add stabilized/destabilized particles to water, which interact with stabilized silt/clay particles, such the combined entity is destabilized. These destabilized entities can then agglomerate to form settleable particles.

Aluminium / Ferric Salts as Coagulants

Alum: $Al_2(SO_4)_3.18H_2O$;

$$Al_2(SO_4)_3.18H_2O \rightarrow 2Al^{3+} + 3SO_4^{2-} + 18H_2O$$

$Al^{3+}(H_2O)_6$	$Al^{3+}(H_2O)_5(OH^-)$	$Al^{3+}(H_2O)_4(OH^-)_2$	$Al^{3+}(H_2O)_3(OH^-)_3$	$Al^{3+}(H_2O)_2(OH^-)_4$
Al^{3+}	$AlOH^{2+}$	$Al(OH)_2^+$	$Al(OH)_3 \downarrow$	$Al(OH)_4^-$

Ferric Chloride: $FeCl_3$; $\quad FeCl_3 \rightarrow Fe^{3+} + 3Cl^-$

Fe^{3+}	$Fe(OH)^+$	$Fe(OH)_2^+$	$Fe(OH)_3$ (ppt),	$Fe(OH)_4^-$
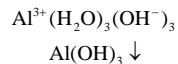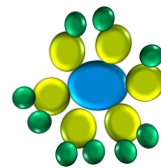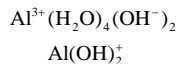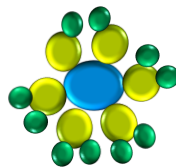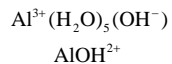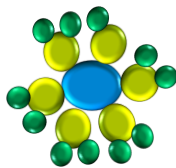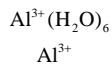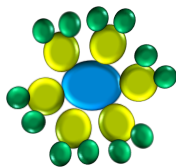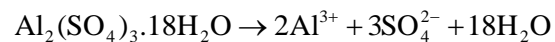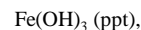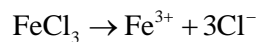
Aluminum Speciation
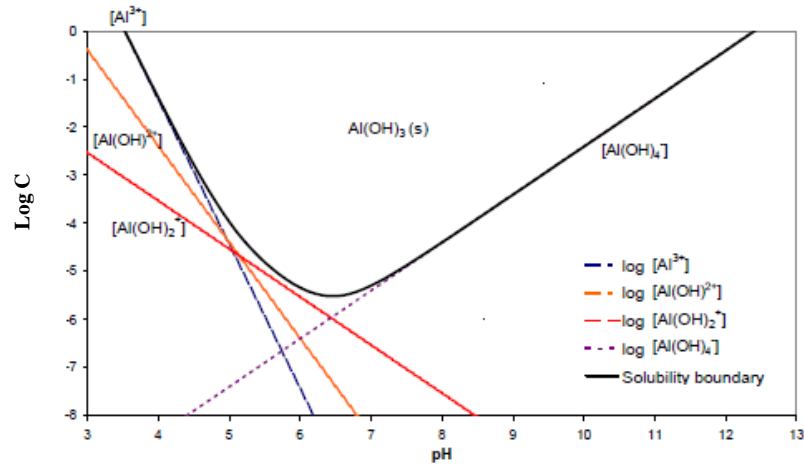
$$K_{sp}\,[Al(OH)_3] = [Al^{3+}].[OH^-]^3 = 3 \times 10^{-34}$$

- Al(OH)$_3$ / Fe(OH)$_3$ particles are either uncharged (destabilized) or positively charged (stabilized)

- Al(OH)$_3$ / Fe(OH)$_3$ particles can interact with silt/clay particles in water, such that the combined entity is destabilized
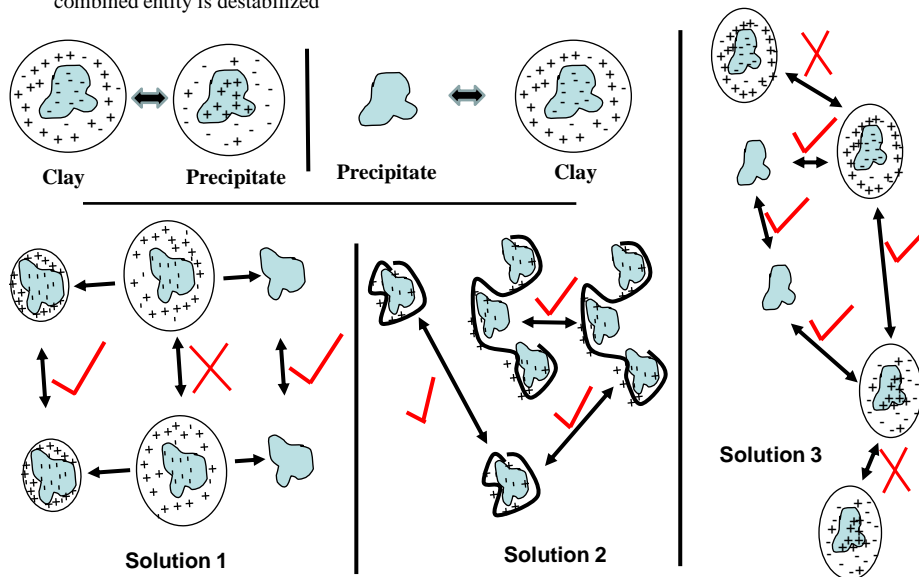
pH decline due to the formation of Al(OH)$_3$

$$Al_2(SO_4)_3.18H_2O \rightarrow 2.Al^{3+} + 3.SO_4^{2-} + 18.H_2O$$

$$Al^{3+} + 6H_2O \overset{\rightarrow}{\leftarrow} Al^{3+}(H_2O)_5(OH^-) + H^+$$

$$Al^{3+} + 6H_2O \overset{\rightarrow}{\leftarrow} Al^{3+}(H_2O)_4(OH^-)_2 + 2H^+$$

$$Al^{3+} + 6H_2O \overset{\rightarrow}{\leftarrow} Al^{3+}(H_2O)_3(OH^-)_3 + 3H^+$$

$$Al^{3+} + 6H_2O \overset{\rightarrow}{\leftarrow} Al^{3+}(H_2O)_2(OH^-)_4 + 4H^+$$
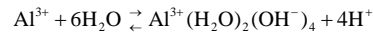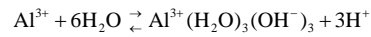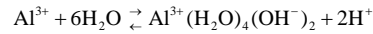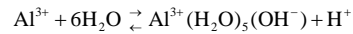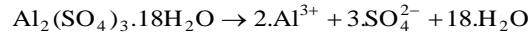
Soda is added to arrest pH decline (if the water has low alkalinity)

$$Na_2CO_3 \rightarrow 2Na^+ + CO_3^{2-}$$
$$CO_3^{2-} + H^+ \overset{\rightarrow}{\leftarrow} HCO_3^-$$
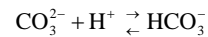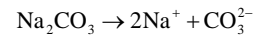
Flocculation Mechanisms

Perikinetic flocculation:
Effective only if one of the interacting particle is < 1 μm in size, irrespective of the other particle. Flocculation through Brownian motion.

Differential settling:
Effective only if both interacting particles are > 1 μm in size and vary in size by more than one order of magnitude.

Orthokinetic Flocculation:
Effective only if both interacting particles are > 1 μm in size and within one order of magnitude in size. Flocculation through shear velocity.

External velocity gradient (G) must be provided for orthokinetic flocculation. In practice, G is provided through slow mixing in a flocculation tank.

Effect of Coagulation Flocculation
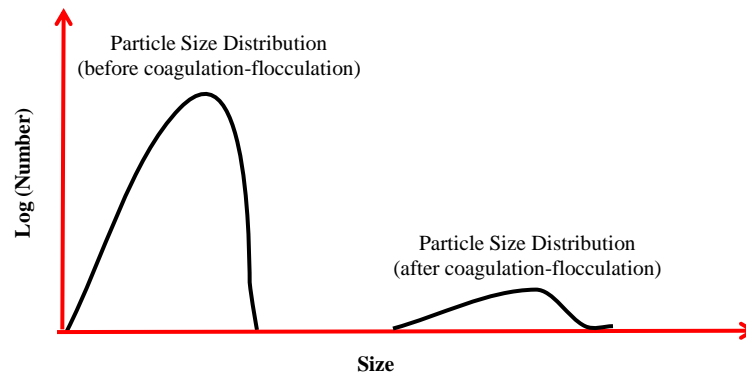
Most particles are large enough to be removed by secondary sedimentation after coagulation flocculation.

Residual turbidity after efficient coagulation-flocculation-secondary sedimentation is ~ 5-10 NTU

Coagulant dose varies depending on the concentration and nature of particles in water. Hence optimal coagulant dose must be determined experimentally for different water samples.

Adding excess coagulant may cause re-stabilization.

Determination of Optimal Coagulant Dose: Jar Test Apparatus
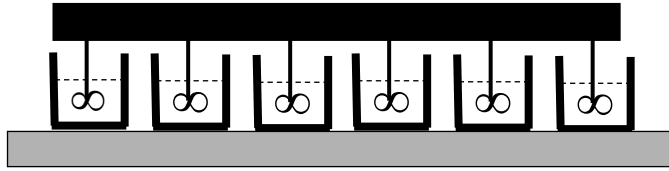
- Take 1 L of the primary settled water in each jar
- Add alum / Ferric and/or polymer and soda (if required for pH adjustment)
- Rapid mix for 2 minutes at 100 rpm
- Slow mix for 30 minutes at 20 rpm
- Stop mixing and allow settling for 2 hours.
- Measure turbidity of the supernatant
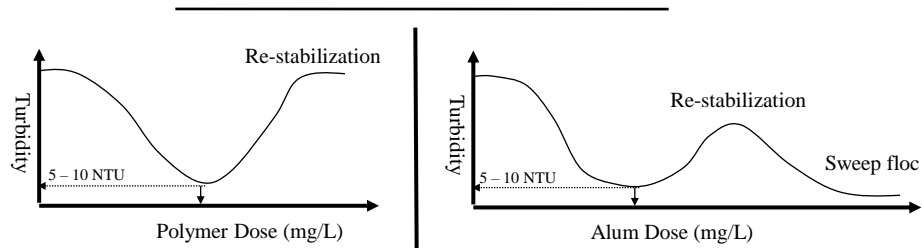
Coagulation – Flocculation – Secondary Sedimentation
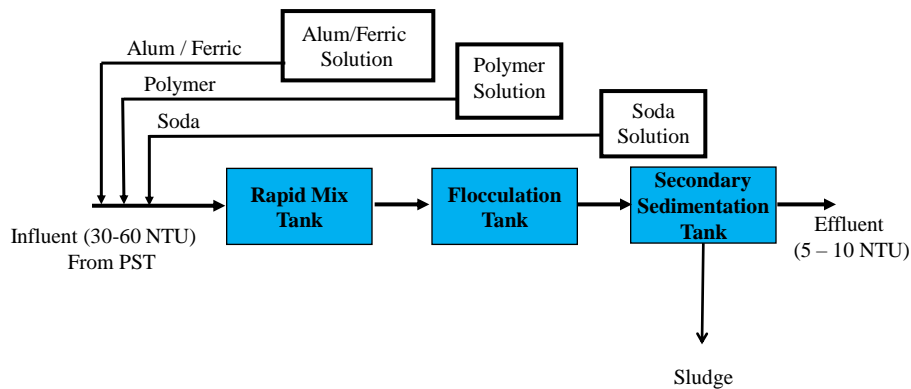
Design of a Rapid Mix Tank: Vertical Shaft Impeller

Plan

Elevation
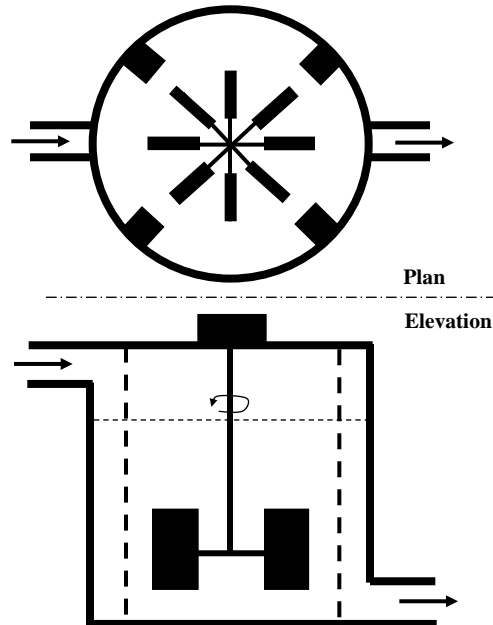
Vertical Shaft Impeller Design Problem:

Design a conventional vertical-shaft rapid mix tank unit for uniformly dispersing coagulant in 11.75 MLD of settled raw water as per design parameters given below:

Detention time (t):	20 – 60 s
Ratio of tank height (H) to diameter (D):	(1:1 to 1:3)
Ratio of impeller diameter (D_I) to tank diameter (D):	(0.2:1 to 0.4:1)
Velocity gradient (G):	>300 /s
Gt:	10000 – 20000
Tank diameter (D):	< 3 m
Blade tip speed (v_p):	1.75 – 2.0 m/s
Velocity of blade relative to water (v):	0.75 x paddle tip speed
Blade area (A_p)/Tank section area (A_T):	10:100 – 20:100
Coefficient of drag on impeller blade (C_D):	1.8
Maximum length of each impeller blade (L):	0.25 x impeller diameter
Maximum width of impeller blade (B):	0.20 x impeller diameter
Blade thickness:	5 cm
Impeller height from bottom (H_B):	1.0 x impeller diameter
Kinematic viscosity (v):	1.003 x 10^{-6} m^2/s
Dynamic viscosity of water (μ):	1.002 x 10^{-3} N.s/m^2

Determine tank dimensions (provide a freeboard of 0.5 m), impeller diameter, blade dimensions, number of blades, clearance of the impeller from tank bottom, blade rotation speed and power input requirement.
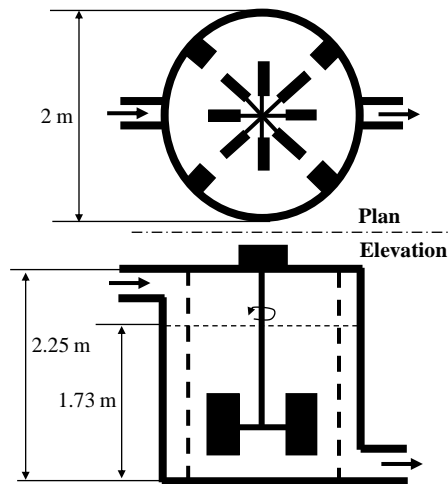
Solution:

Q = 11.75 MLD = 0.14 m^3/s;
Let t = 40 s (between 20 – 60 s)

V = t.Q = 5.44 m^3; Let D = 2 m; A_{cs} = 3.14 m^2;

H = V/A$_{cs}$ = 1.73 m; Freeboard = 0.5 m; H_T = 2.25 m

H / D = 0.865 (in the range 0.33 – 1.0, hence okay)

Let, G = 400 s^{-1} (> 300 s^{-1}, hence okay);

Gt = 16000 (in the range 10000 – 20000, hence okay)

Let the blade tip speed (v_p) = 1.8 m/s
    (in the range, 1.75 – 2.00 m/s, hence okay)

Velocity of the blade relative to water (v)

$$G = \left\{ \frac{C_d.A_p.v^3}{2.v.V} \right\}^{1/2} \qquad = 0.75.(1.8) = 1.35 \text{ m/s}$$

$; A_p = 0.393 \text{ m}^2$

Tank sectional area = D.H = 3.46 m^2

A_p / (D.H) = 0.393 / 3.46 = 0.113
        (between 0.10 – 0.20, hence okay)
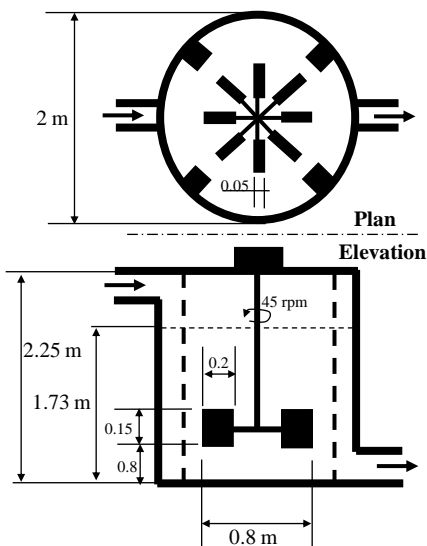
Solution (continued):

Let the impeller diameter be 0.8 m, i.e.,

$\left(\dfrac{D_I}{D} = \dfrac{0.8}{2} = 0.4 \right)$, (between 0.2 – 0.4, hence okay)

Choose length of each impeller blade (L) as 0.20m, i.e.,

$\left(\dfrac{L}{D_I} = \dfrac{0.2}{0.8} = 0.25 \right)$, (up to 0.25, hence okay)

Choose breadth of each impeller blade (B) as 0.15m, i.e.,

$\left(\dfrac{B}{D_I} = \dfrac{0.15}{0.8} = 0.187 \right)$, (up to 0.20, hence okay)

Area of each blade = (L.B) = (0.2).(0.15) = 0.03 m^2

Blade thickness = 5 cm

Therefore, number of blades to be provided = 13.1 (say 14)

Clearance of the paddles from the tank bottom = 0.8 m

Paddle rotation speed (w) = 2.v$_p$/D$_I$ = 4.5 radians/s,
        (or ~ 45 revolutions/ min)

Power requirement (P): $G = \left[\dfrac{P}{V.\mu} \right]^{1/2}$ P = 870 W
Provide 1 kW motor

Design of Flocculation Tank: Horizontal Shaft Paddle

Plan

Elevation
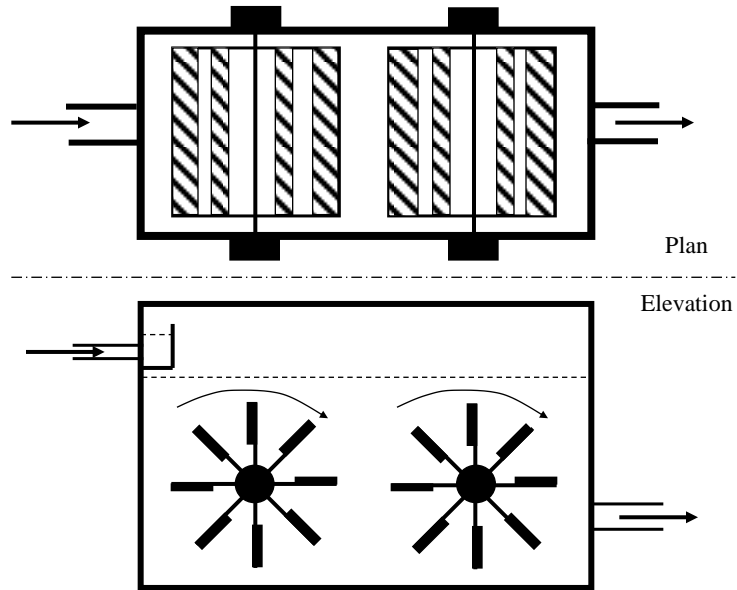
Using a Hydraulic Jump for Rapid Mixing

Alum / Ferric

Polymer

Soda
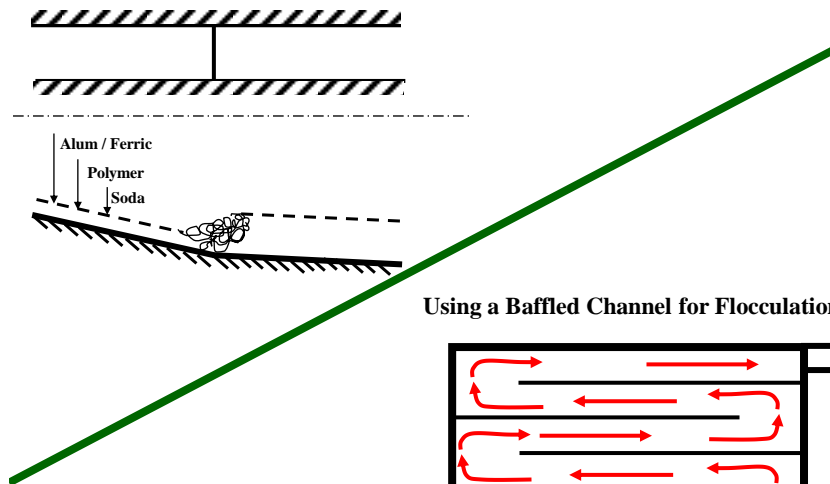
Using a Baffled Channel for Flocculation

Effluent

Influent
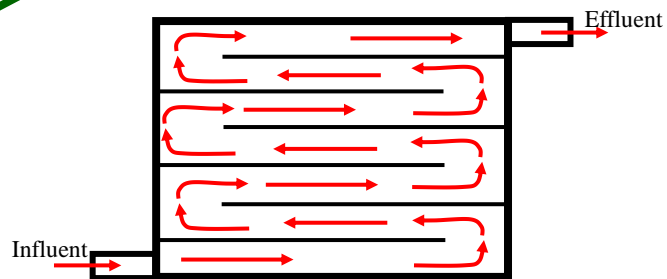
Secondary Sedimentation Tank (SST)

Secondary sedimentation tanks are generally circular

Circular sedimentation tanks are also called **clarifiers**

Type I Settling: Discrete Settling (PST)

Design Parameters:

Type II Settling: Flocculant Settling (SST)
[particles flocculate and grow bigger as they settle]

SOR: 30 – 40 $m^3/m^2/d$
Depth: 3 – 4.5 m
Dia: 3 – 60 m
Bottom Slope (cm/m): 6.3 - 17
Detention Time: 2 - 5 hours

Type I Settling

Type II Settling
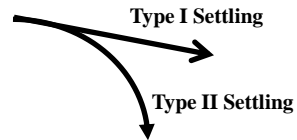
Clari-flocculator:
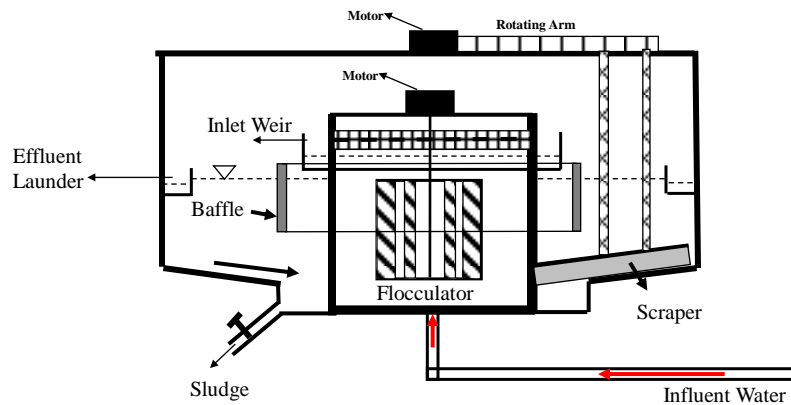
Rapid Sand Filtration

Water containing de-stabilized particles (turbidity: 5 – 10 NTU) is applied to a sand bed.

Almost all particles in the influent water are retained in the sand bed, such that the water effluent from the sand bed has a turbidity of < 1 NTU.
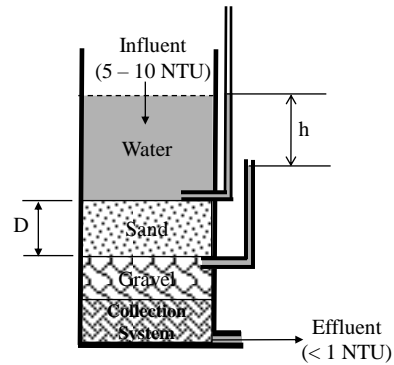
Flow of water through the sand bed results in head loss

Magnitude of head loss increases are more and more particles are retained in the sand bed

Hence the sand bed must be cleaned periodically to ensure water flow at the required rate

Schematic Representation: Flow through porous media
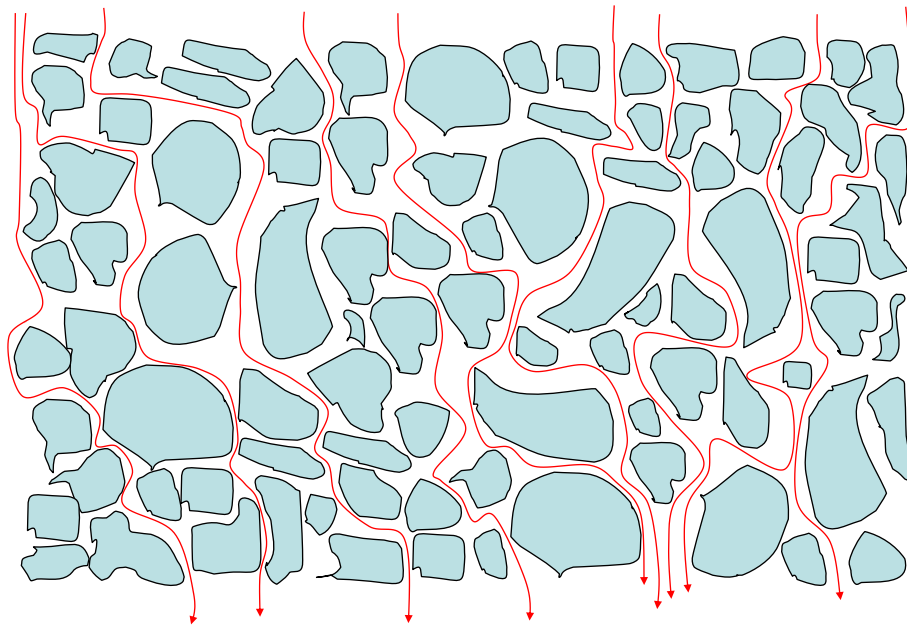
Schematic Representation: Rapid Sand Filtration
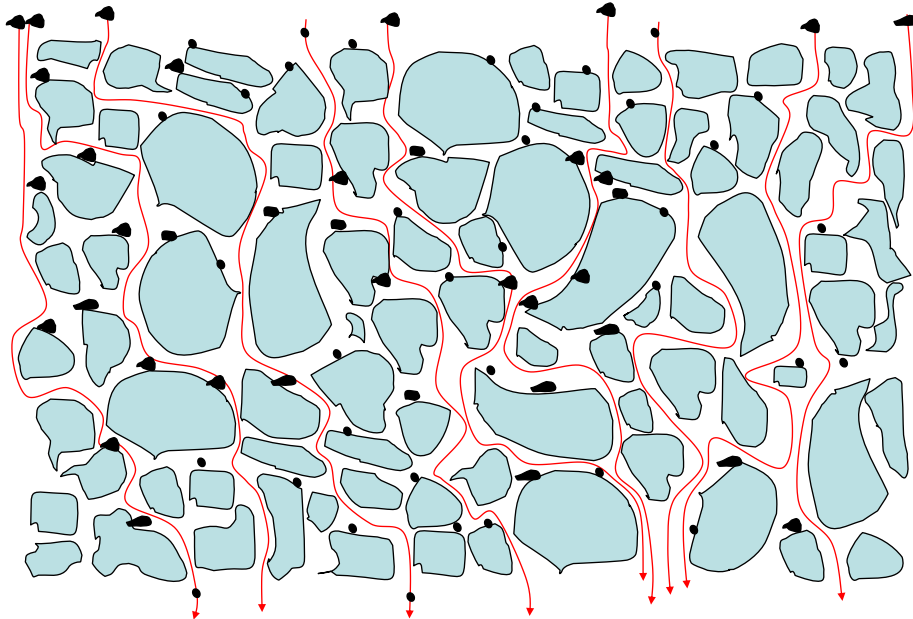
Rapid Sand Filtration: Mechanism of Particle Removal

Rapid sand filtration (RSF) is **NOT** Straining.

Why??

Filtration Media: Sand
Sand Size (d): ~ 0.5 mm

Porosity (α) ~ 0.4 – 0.5;
this means that nearly half of the sand bed is empty.

Thus pore size ~ sand grain size

For particles to be removed by straining, their size should be similar to pore size, i.e., ~ 0.5 mm

However, particles applied to the filter are more than an order of magnitude lower in size, ~ 10 μm

Hence the mechanism(s) of particle removal in the RSF must be something else
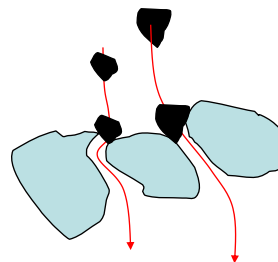
Particle Removal
by Straining

Filtration: Collision Mechanisms

| Collection by Brownian Diffusion | Collection by Settling | Collection by Interception |
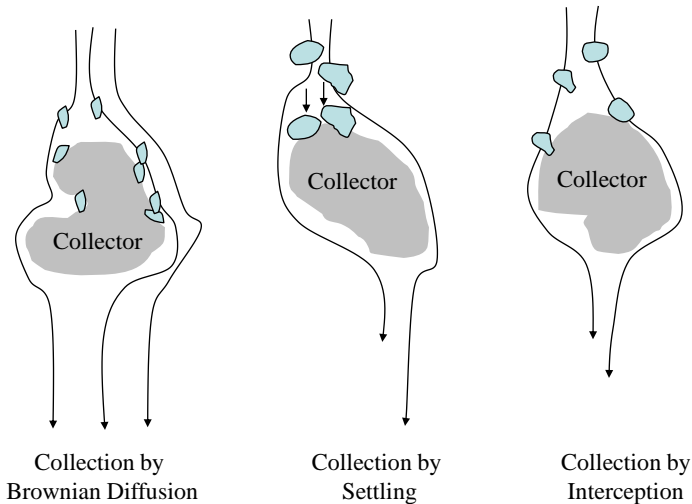
Attachment Mechanisms:
1. Electrostatic attraction between particle and collector
2. Van der Waals forces between particles and collector

Rapid Sand Filtration
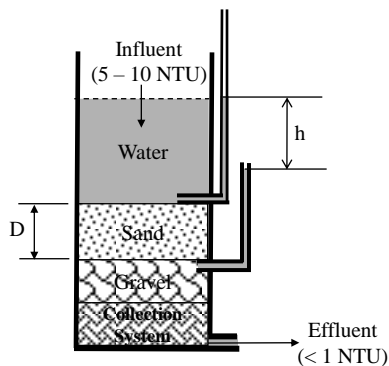
Sand: Filtration Media
D: Depth of Filter Media
Sand Size (d): ~ 0.5 mm
Porosity (α) ~ 0.4 – 0.5

Under-Drainage System
Gravel: Support Media
Collection System: Perforated pipes

h: Head loss through filter media

Filtration Rate (V_s): 8 – 12 m³/m²/hr

Carmen-Kozeny Equation: (Clean Bed head loss)

$$h_c = \frac{f}{\phi}\cdot\frac{1-\alpha}{\alpha^3}\cdot\frac{D}{d}\cdot\frac{V_s^2}{g}$$

ϕ: Shape factor (1 for spheres)

$$f: \quad 150.\frac{(1-\alpha)}{N_R}+1.75$$

$$\text{Re ynold's No. } (N_R) = \frac{\phi.d.V_s}{\nu}$$

Calculation of h_c

$V_s = 10$ m³/m²/hr $= 2.77 \times 10^{-3}$ m/s; $d = 5 \times 10^{-4}$ m

$N_R = 1.(5 \times 10^{-4}).(2.77 \times 10^{-3})/(1.003 \times 10^{-6}) = 1.385$

$\alpha = 0.45$; $f = [150.(1-0.45)/(1.385)]+1.75 = 61.32$

$h_c = (61.32/1).[(1-0.45)/(0.45)^3].[0.6/(5 \times 10^{-4})].$
$\qquad [(2.77 \times 10^{-3})^2/9.81] = 0.347$ m

Rapid Sand Filter Performance

Reason for Particle Breakthrough ??

Influent Turbidity ~ 5 – 10 NTU

↑ Turbidity

Effluent Turbidity

1 NTU

Time →

T_B

3.0 m

$(V_s)_1 \geq (V_s)_2 \geq (V_s)_3$

↑ Head loss

Time →

T_H ~ 8 hr

3.0 m

↑ Head loss

~ 0.31 m

T_H ~ 8 hr

Time →

Filter Depth

Solids Concentration →

Backwash
(to Flocculation Tank)

Rapid Sand Filter Schematic

Backwash Tank

Plan

Backwash Tank

Pump

Front Sectional View

31

Rapid Sand Filter Schematic

Backwash
(to Flocculation Tank)

Backwash
Tank

Plan

Backwash
Tank

**Front
Sectional
View**

Pump

Backwashing

A filter has to be stopped every few hours due to excessive head loss build up.

After stopping the filter is drained.

Then filtered water stored in the backwash tank is passed through the filter in an up flow mode.

Back wash Rate:	1 $m^3/m^2/min$
Backwash Time:	5 minutes

The backwash rate is such that the sand bed is fluidized during backwashing.

Shear forces during fluidization cause the particles attached to sand during filtration to be detached and removed with the backwash water. Sand bed is thus cleaned.

Filter down time due to backwashing: 30 minutes per cycle

The backwash water is conveyed to the head of the flocculation tank.

Dual Media Filtration

The 60 cm sand bed used for filtration is replaced by a dual media containing 30 cm of Anthracite coal on top, followed by 30 cm of sand.

Size of the coal particles: ~ 1 mm;	Sp. Gr.: 1.20
Size of the sand particles: ~0.5 mm	Sp. Gr.: 2.65

Advantages:

Head loss development in a dual media filter bed is slower, resulting in longer filter run times.

Performance of a dual media filter is indistinguishable from that of a single media filter.
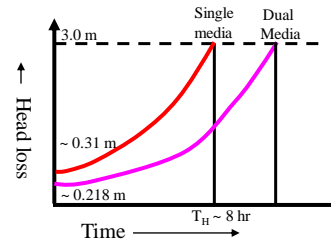
Clean bed head loss in a dual media filter

Calculation of h_c (coal layer):	Calculation of h_c (sand layer):
$V_s = 10$ m^3/m^2/hr $= 2.77 \times 10^{-3}$ m/s; d = 1 x 10^{-3} m	$V_s = 10$ m^3/m^2/hr $= 2.77 \times 10^{-3}$ m/s; d = 5 x 10^{-4} m
$N_R = 1.(1 \times 10^{-3}).(2.77 \times 10^{-3})/(1.003 \times 10^{-6}) = 2.762$	$N_R = 1.(5 \times 10^{-4}).(2.77 \times 10^{-3})/(1.003 \times 10^{-6}) = 1.385$
$\alpha = 0.45$; f = [150.(1-0.45)/(2.762)]+1.75 = 31.62	$\alpha = 0.45$; f = [150.(1-0.45)/(2.762)]+1.75 = 61.32
$h_c = (31.62/1).[(1-0.45)/(0.45)^3].[0.3/(1 \times 10^{-3})].$ $[(2.77 \times 10^{-3})^2/9.81] = 0.045$ m	$h_c = (61.32/1).[(1-0.45)/(0.45)^3].[0.3/(5 \times 10^{-4})].$ $[(2.77 \times 10^{-3})^2/9.81] = 0.174$ m

Total clean bed head loss = 0.218 m

Effect of Various Parameters on Filter Operation: A Review

What is the effect of increase in the following filter and influent particle characteristics on time (T_B) for reaching particle breakthrough (turbidity > 1.0 NTU) in the effluent, and time (T_H) of reaching terminal head-loss (>3 m) across filter bed,

1. Filter Depth (D)
2. Superficial Velocity (V_s)
3. Influent Turbidity (C_o)
4. Collector Diameter (d)

Explain the answers based on your understanding of filter operation and particle removal mechanisms operative in rapid sand filters. Assume that when one characteristic is changed, all other characteristics remain fixed.
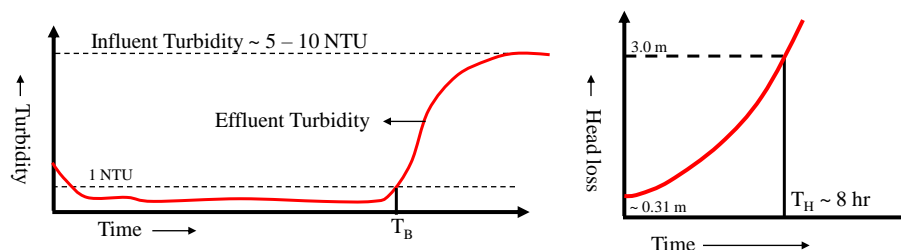
Effect of Increase in Filter Depth (D):

T_B will **increase** since particles in water will have the opportunity to interact with more collectors.

Since bed depth is more, water is encounter more resistance is passing through the bed. Hence T_H will **decrease**.

Effect of Increase in Superficial Velocity (V_s):

Since pore velocity is more, the shear forces experienced by collected particles will be more. Hence particle detachment will be more, leading to a **decrease** in T_B.

Since particle loading rate on the filter and rate of particle collection in the filter will increase, the porosity of the filter bed will decrease more rapidly, leading to more rapid head-loss buildup and hence a **decrease** in T_H.

Effect of Increase in Influent Particle Concentration (C_o):

Particle loading rate on the filter, rate of particle collection and hence porosity of the filter bed will decrease more rapidly. Thus pore velocity in the filter bed will increase more rapidly, leading to more rapid particle detachment. This will lead to **decrease** in T_B.

Since particle loading rate on the filter and rate of particle collection in the filter will increase, the porosity of the filter bed will decrease more rapidly, leading to more rapid head-loss buildup and hence a **decrease** in T_H.

Effect of Increase in Collector Diameter (d):

Increase in collector diameter will mean the presence of a lesser number of collectors in the filter media. This will result is lesser number of interaction between a particle and a collector. Thus T_B is expected to **decrease**.

Since lesser number of particles will be collected as mentioned above, the rate of head-loss buildup will be lower, leading to **increase** in T_H.

Rapid Sand Filter Design

Design Parameters:

Filter Depth:	60 cm
Filtration Rate:	8-12 $m^3/m^2/hr$
Filter Run Time:	7.5 hours
Terminal Head loss:	~3 m
Length:	< 7 m
Length: Width Ratio:	1.3-1.5: 1
Water Depth on top of filter:	up to 3 m
Freeboard:	0.5 m
Under drainage Depth:	1.5 m (Including gravel support and collection chamber)
Back wash Rate:	1 $m^3/m^2/min$
Backwash Time:	5 minutes
Filter Down Time:	30 minutes per cycle (Due to Backwashing)

Modifications to Conventional Water Treatment Train

Case A: Low Turbidity (~ 5-10 NTU) water
                  Raw water source: Lake Water

Coagulant/polymer addition ⟶ Rapid Mixing ⟶ Rapid Sand Filtration

In-Line Filtration

Case B: Medium turbidity (~ 10 – 20 NTU) water
                  Raw water source: 1) River water in dry season; 2) Autumn turnover

Coagulant/polymer addition ⟶ Rapid Mixing ⟶ Flocculation
                                                                                          ↓
                                                                            Dual Media Filtration

Direct Filtration

Case C: Applicable to all waters from low to high turbidity

Coagulant (high dose) and Soda addition ⟶ Rapid Mixing ⟶ Flocculation
                                                                                                        (Sweep Floc formation)
                                                                                                        ↓
Rapid Sand Filtration ⟵ Secondary Sedimentation

Sweep Floc Formation

Surface Water Treatment Objectives

Treated water must be potable, i.e., free from

- Color
- Odor
- Taste
- Turbidity (< 1 NTU)
- Pathogens (absent)
- Other harmful substances (as per relevant standards)

Treatment Process Train

Raw water ⟶ | SE | ⟶ | PS | ⟶ | RM | ⟶ | F | ⟶ | SST |
                                              ↑ |
                                    Chemical   ↓
                                    Addition | RSF |
                                                                                    ↓
                                                                                    ?
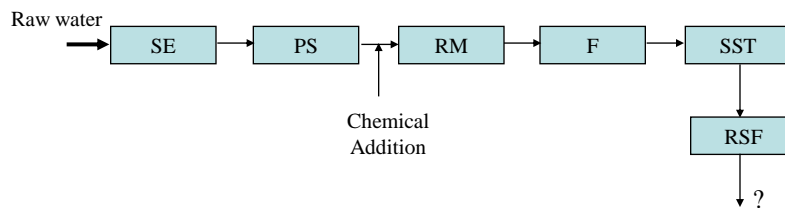
Chlorination

Chlorination helps to achieve multiple objectives in water treatment,

1. Removal of color, taste and odor (caused by the presence of dissolved inorganic and organic substances)

2. Removal of ammonia (present in water due to anthropogenic pollution)

3. Microbial disinfection of water

Application of Chlorine to Water:

1. Chlorine Gas:

$$Cl_2(g) \underset{\leftarrow}{\rightarrow} Cl_2(aq)$$

$$Cl_2(aq) + H_2O \underset{\leftarrow}{\rightarrow} HOCl + H^+ + Cl^-$$

$$HOCl \underset{\leftarrow}{\rightarrow} H^+ + OCl^- \qquad pK = 7.5$$

2. Sodium Hypochlorite:

$$NaOCl \rightarrow Na^+ + OCl^-$$

$$HOCl \underset{\leftarrow}{\rightarrow} OCl^- + H^+$$

3. Bleaching Powder:

$$Ca(OCl)Cl \rightarrow Ca^{2+} + OCl^- + Cl^-$$

$$HOCl \underset{\leftarrow}{\rightarrow} OCl^- + H^+$$

Free Chlorine Residual (mg/L as Cl_2) = $\left([HOCl] + [OCl^-]\right) 71000$

Common Oxidation States of Chlorine: -1, 0, +1

Oxidation states of Carbon: -4, -3, -2, -1, 0, +1, +2, +3, + 4

Inorganic carbon is almost exclusively in +4 oxidation state
Organic carbon is almost exclusively in oxidation states lower than +4

Free chlorine is a strong oxidizing agent.

It can oxidize organic carbon present in water, i.e., increase its oxidation state (subject to kinetic limitations).

Sometimes (rarely) it completely mineralizes organic carbon to inorganic carbon (i.e., +4 oxidation state).

Some cations like Fe^{2+}, Mn^{2+} etc. are also oxidized by chlorine.

Chlorine Demand in Pure Water

45°

Free Chlorine Residual (mg/L)
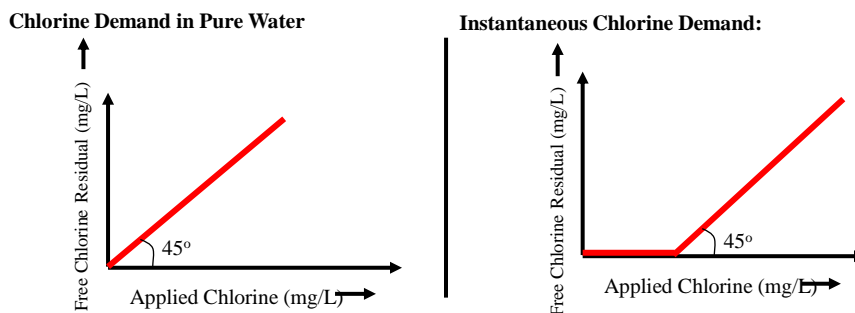
Applied Chlorine (mg/L)

Instantaneous Chlorine Demand:

45°

Free Chlorine Residual (mg/L)

Applied Chlorine (mg/L)

Instantaneous Chlorine Demand

As soon as chlorine is added to water, it reacts with organic matter (and also some inorganic matter) present in water.

These reactions occur in a matter of minutes and results in chlorine being reduced. The taste, color and odor causing compounds present in water are oxidized during this time.

Generally 1 mg/L chlorine is added per mg/L BOD_5 present in the water to care of the demand by organic compounds present in water

If oxidisable inorganic ions are present (i.e., Fe^{2+} and Mn^{2+} ions, etc.), additional chlorine will be required to satisfy the demand from these ions.

Satisfying instantaneous chlorine demand of the water will take care of the taste, color and odor problems in most cases.

Example (odor causing compounds):
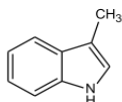
3-methylindole
(Skatole)

An odor (fecal odor) causing compound, can be partly oxidized by chlorine to compounds which do not smell

Example (Taste and Odor causing compounds):
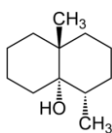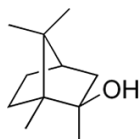
Geosmin   **2-Methylisoborneol (MIB)**

These compounds (excreted by algae in water) impart earthy / grassy taste to water

Oxidation of Inorganic Constituents:

$$2Fe^{2+} + HOCl + 5H_2O \leftrightarrow 2Fe(OH)_3(s) + Cl^- + 5H^+$$

$$Mn^{2+} + HOCl + H_2O \leftrightarrow MnO_2(s) + Cl^- + 3H^+$$

$$H_2S + 4HOCl \leftrightarrow SO_4^{2-} + 4Cl^- + 6H^+$$
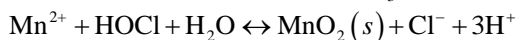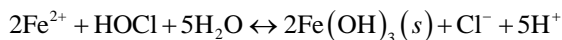
Example (Color causing compounds):

Model structure of fulvic acid by Buffle
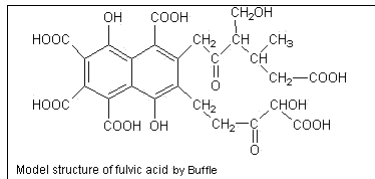
Fulvic acids impart yellow/brown color to water. Such compounds can be partly oxidized by chlorine to compounds which are colorless (or at least have less color)

Chlorine Demand: In water with Ammonia
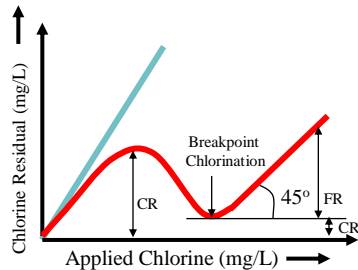
The chloramines together constitute
The combined chlorine residual (CR)

$NH_4^+ \rightleftarrows NH_3(aq) + H^+$ \quad pK = 9.5

$NH_4^+ + OCl^- \rightleftarrows NH_2Cl + H_2O$ \quad Monochloramine Formation
$NH_2Cl + OCl^- \rightleftarrows NHCl_2 + OH^-$ \quad Dichloramine Formation
$NHCl_2 + OCl^- \rightleftarrows NCl_3 + OH^-$ \quad Trichloramine Formation

$2NH_4^+ + HOCl \rightarrow 3Cl^- + N_2 + 5H^+ + 3H_2O$
$2NH_2Cl + HOCl \rightarrow N_2 + H_2O + 3H^+ + 3Cl^-$
$NHCl_2 + 2HOCl + H_2O \rightarrow NO_3^- + 5H^+ + 4Cl^-$ \quad Breakpoint Chlorination
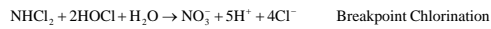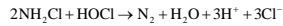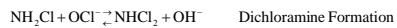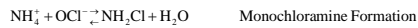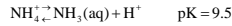
These reaction take ~30 minutes to complete

Chloramines are formed at low chlorine: ammonia ratios. At higher chlorine doses, the chloramines and residual ammonia is oxidized. This is known as breakpoint chlorination.

How much Chlorine must be added to water ??

Instantaneous chlorine demand of water must be satisfied

Breakpoint chlorination must be achieved, such that Free Residual Chlorine can exist in water

Sufficient Free Residual Chlorine must be present in water for effective disinfection and maintenance of free chlorine residual in the water distribution system
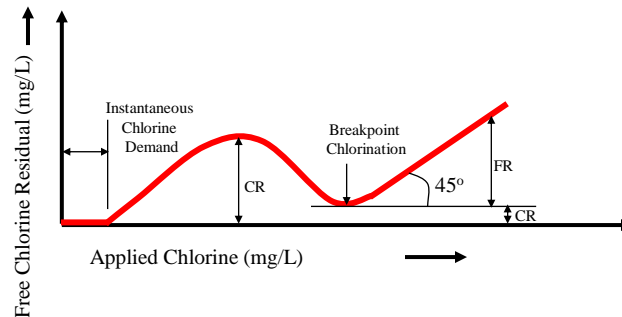
How much Free Residual Chlorine is required for effective disinfection ??

Kinetics of Disinfection

Chick's Law: $\quad \text{Log}\left[\dfrac{N}{N_o}\right] = -K.t$

N_o = Number of micro-organisms at time, t = 0
N = number of microorganisms at time t
K = Constant dependent on,
        1. Type of disinfectant
        2. Concentration of disinfectant
        3. Type of microorganism

Concept of Log Removals:

$\text{Log}\left[\dfrac{N_o}{N}\right] = 1; \ 1\text{-Log Removal, i.e., 90 \% removal}$

$\text{Log}\left[\dfrac{N_o}{N}\right] = 2; \ 2\text{-Log Removal, i.e., 99 \% removal}$

$\text{Log}\left[\dfrac{N_o}{N}\right] = 3; \ 3\text{-Log Removal, i.e., 99.9 \% removal}$

and so on.

Chick-Watson's Law:

$\text{Log}\left[\dfrac{N}{N_o}\right] = -\Lambda.C^n.t$

Λ = Coefficient of specific lethality, dependent on
        1. Type of disinfectant
        2. Type of microorganism

C = Concentration of the disinfectant
n = constant dependent upon
        1. Type of disinfectant
        2. Type of microorganism

Simplified Chick-Watson's Law

Assuming n = 1 in Chick-Watson's Law,

$$\text{Log}\left[\dfrac{N_o}{N}\right] = \Lambda'.(C.t)$$

Need to know, Λ' values, so that the 'Ct' value corresponding to a desired 'log-removal' may be calculated for the specified microorganism-disinfectant combination

Disinfectant of interest: Free chlorine

Target microorganism??
    Many microorganisms cause water borne diseases,

<u>Some Examples:</u>

Bacteria: *Vibrio cholerae, Escherichia coli* (certain strains)*, Shigella, Salmonella typhi,*

Viruses: *Poliovirus, Hepatitis A virus*, *Astrovirus, Calicivirus, Enteric Adenovirus, and Parvovirus*

Protozoa: *Entamoeba hystolytica, Giardia lamblia, Cryptosporidium parvum*

Giardia lamblia exists as dormant microbial cysts in contaminated water. These are the most difficult to kill. Hence if the objective of disinfection is to target *Giardia lamblia*, then we may be reasonably sure that all other pathogens in water are also inactivated.
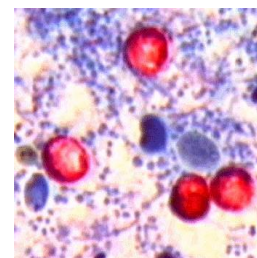
'CT' values (in mg/L-min) of the removal of *Giardia lamblia* cysts at various pH values (T = 20°C) (using free chlorine as disinfectant)

	1- Log	2-Log	3-Log
pH = 6	14	28	42
pH = 7	20	39	59
pH = 8	30	59	89
pH = 9	43	86	129

Why are the 'CT' values pH dependent??

Free chlorine consists of two species, [HOCl] and [OCl⁻]. Among these, [HOCl] is by far more effective for disinfection. In fact, disinfection by [OCl⁻] may be neglected in comparison to that by [HOCl]

Concept of 'CT' Credit

Bacteria, viruses and other microorganisms are particles. Hence they are removed substantially by particle removal processes, i.e., coagulation-flocculation and filtration commonly used in water treatment. It may be assumed that 2.5-log removal of *Giardia lamblia* cysts occur during conventional water treatment, while 2-log removal may be assumed in case of direct filtration. Hence, if 5-log removal of *Giardia lamblia* cysts is desired during conventional water treatment, the disinfection system should be designed corresponding to a 'CT' value of only 2.5.

(Solved example of a disinfection problem will be described in Lab 6)

Reactions of Chlorine with Organics (NOM)
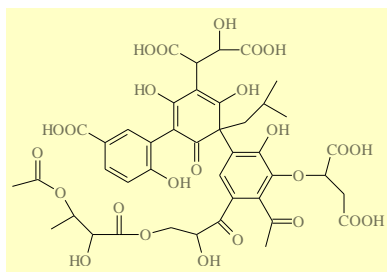
A NOM Molecule

Free Cl_2 reacts with NOM molecules in two ways: (1) oxidation, generating Cl⁻, and (2) substitution to form chlorinated organics.
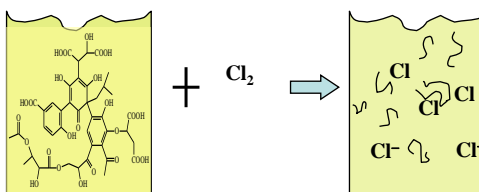
Incorporation of Chlorine into Organics

A portion of the added free Cl_2 (usually <10%) becomes incorporated into NOM molecules. As with substitution into NH_3, multiple substitutions at a single site seem common.

40
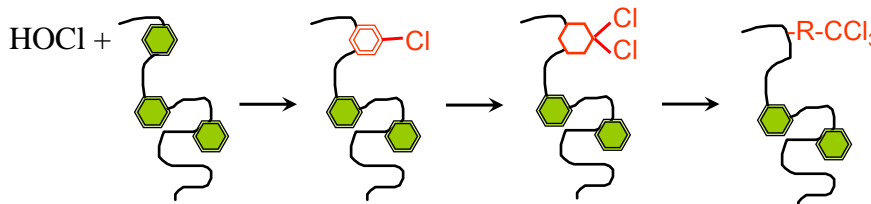
Disinfection by-products (DBPs)

Addition products of chlorine with organic matter present in water

- Trihalomethanes: 4 kinds
- Haloacetic acids: 9 kinds

These are the most common chlorination by-products, though many others are also formed.

Some of the DBPs are carcinogenic
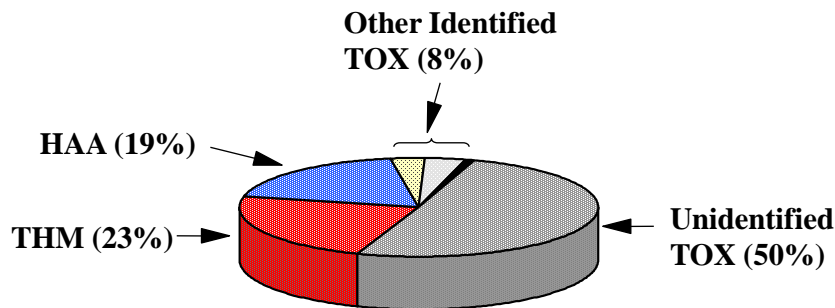
Reactions of Chlorine with NOM: in the presence of Bromine
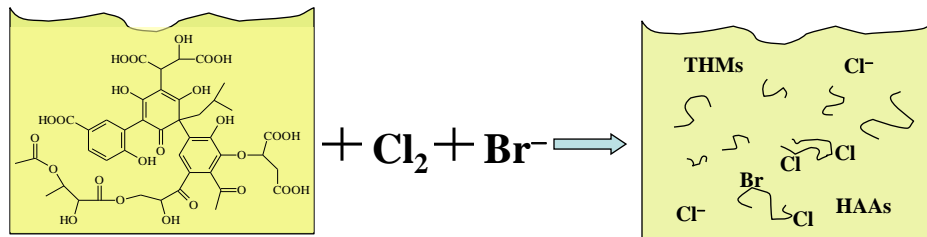
Risk assessment: Immediate risk of illness from water borne diseases versus long term risk of cancer

Of course, best option will be to prevent formation of DBPs. For this, DBPs precursors, i.e., dissolved organic matter must be destroyed or removed from water before chlorination. Another option will be to use alternative disinfectants.

Alternative Disinfectants
- Ozone
- ClO_2
- UV

Big disadvantage: they do not leave behind any residual for protection against re-contamination of water in the distribution system.

Different Modes of Disinfection

Post-chlorination

After Primary Treatment → [RM] → [F] → [SST] → [RSF] → [UGT] → To Distribution

Chemical Addition

Pre-chlorination / Post-chlorination

After Primary Treatment → [RM] → [F] → [SST] → [RSF] → [UGT] → To Distribution

Chemical Addition

Ozonation/UV

After Primary Treatment → [RM] → [F] → [SST] → [RSF] → [UGT]

Chemical Addition / Chlorination → To Distribution
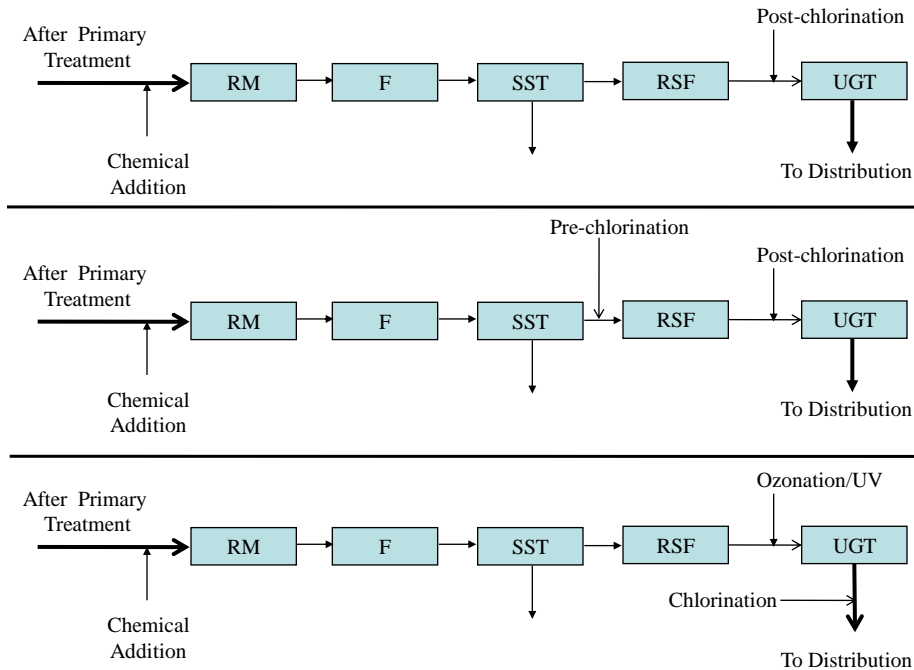
Estimation of Water Demand in 2036

Average per capita domestic water demand = 235 lpcd
Average domestic water demand = $235 \cdot (30345)/10^6 = 7.13$ MLD

Temporary population = 10000
Average per capita temporary water demand = 60 lpcd
Average temporary water demand = $60 \cdot (10000)/10^6 = 0.60$ MLD

Commercial water demand = $0.5 \cdot (7.13) = 3.57$ MLD

Horticultural demand = $(0.5/100) \cdot (60) \cdot (4046)/1000 = 1.21$ MLD

Average daily demand = 7.13 + 0.60 + 3.57 + 1.21 = 12.51 MLD

Maximum Daily demand = (1.8).(12.51) = 22.52 MLD   [say in May/June]
        (The water intake pumps and the water treatment plant must be designed for this value)
                (water treatment facilities may be built in 3 stages @7.5 MLD/stage)

Peak hourly demand = 3.(12.51) = 37.53 MLD   (the distribution system must be designed for this value)

Fire Demand = $(100/1000) \cdot (30345/1000)^{0.5} = 0.55$ ML

Duration of storage = 6 hours
Total Storage = (37.53 – 22.52).(6/24) + 0.55 = 4.30 ML
                (provided in underground or overhead tanks or a combination of these)

Minimum Demand = 0.5*(Average Demand) = 6.255 MLD   [Say in December / January]

Underground Storage Tank
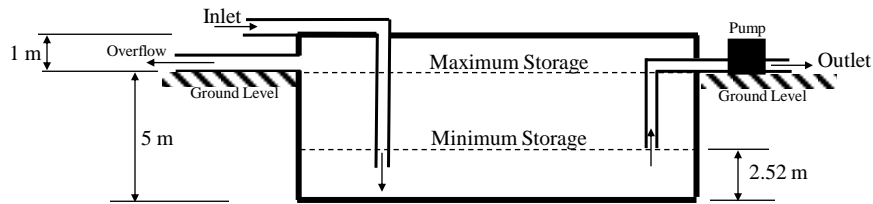
Maximum Water demand = 22.52 MLD

Peak water demand = 37.53 MLD

Total storage capacity (in UG and OH tanks combined) = 4.30 ML

Minimum detention time in UG tank = t_{min} = 44.4 minutes = V_{min}/Q_{max}

$V_{min} = t_{min} \cdot Q_{max} = (44.4/60)(37.53/24) = 1.16$ ML

Let maximum storage (V_{max}) = 2.30 ML = 2300 m^3

Let maximum depth of storage (d_{max}) = 5 m; Freeboard = 1 m

Surface area = 2300/5 = 460 m^2; Length = 23 m; Breadth = 20 m

Minimum depth of storage (d_{min}) = 1160/460 = 2.52 m

Water Distribution Network

Water Distribution Network

Input Node

Output

House Connections

Solving a pipe network

L_i = Length of ith pipe (known)
D_i = Diameter of ith pipe (known)
f_i = Friction factor of ith pipe (known)

Q_A, Q_B, Q_C, Q_D, Q_E and Q_F are known
$Q_A = Q_B + Q_C + Q_D + Q_E + Q_F$

h_a is known
At any node, $\sum Q_i = 0$
Flow into a node is +ve,
Flow going away for a node is –ve

$Q_A - Q_1 - Q_3 = 0$
$Q_1 - Q_B - Q_2 = 0$
$Q_2 + Q_4 - Q_C - Q_5 = 0$
$Q_5 - Q_D - Q_7 = 0$
$Q_7 + Q_6 - Q_E = 0$
$Q_3 - Q_4 - Q_F - Q_6 = 0$

Use any 5

In any loop, $\sum h_i = 0$

$h_3 + h_4 - h_2 - h_1 = 0$
$h_6 - h_7 - h_5 - h_4 = 0$

Use both

Solve the 7 equations simultaneously to get values of Q_1 to Q_7

Change the direction of flow if any Q_i is -ve

Then calculate corresponding h_1 to h_7

$h_1 = h_b - h_a$
$h_2 = h_c - h_b$
$h_3 = h_f - h_a$
$h_4 = h_c - h_f$
$h_5 = h_d - h_c$
$h_6 = h_e - h_f$
$h_7 = h_e - h_d$

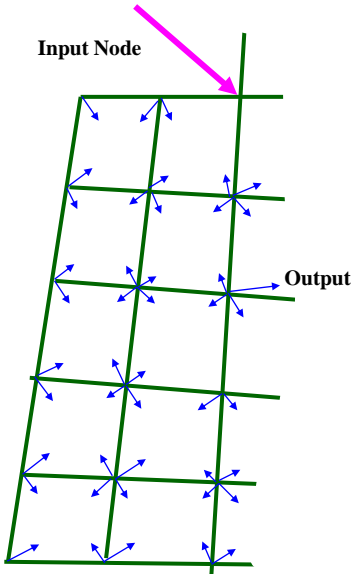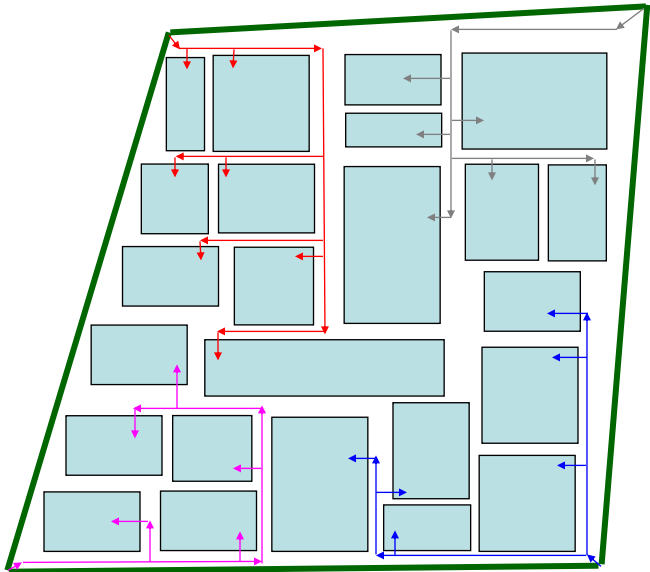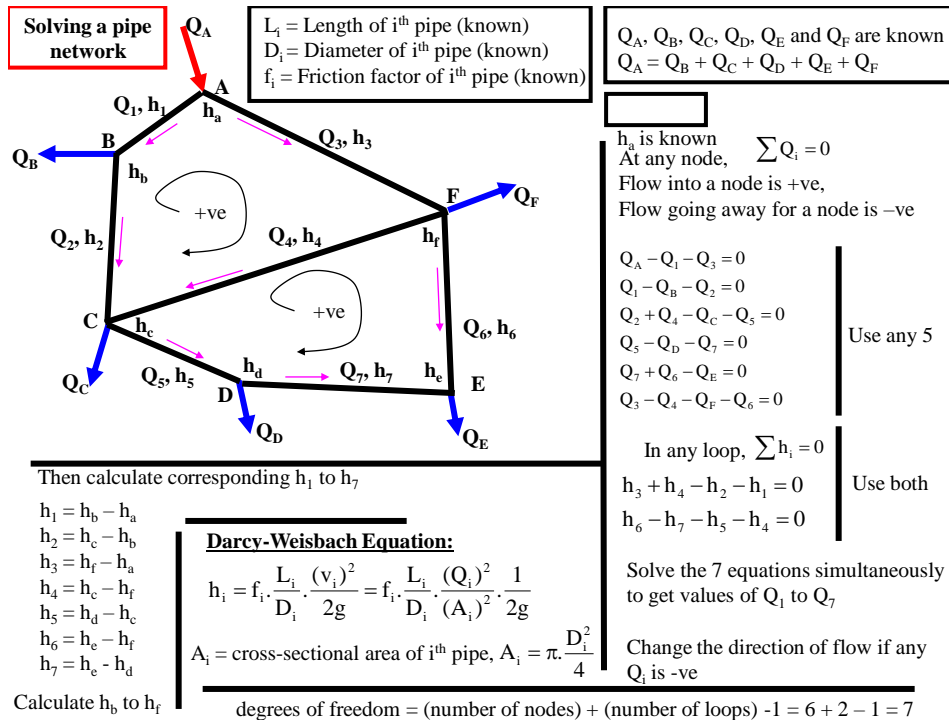Calculate h_b to h_f

Darcy-Weisbach Equation:

$$h_i = f_i \cdot \frac{L_i}{D_i} \cdot \frac{(v_i)^2}{2g} = f_i \cdot \frac{L_i}{D_i} \cdot \frac{(Q_i)^2}{(A_i)^2} \cdot \frac{1}{2g}$$

A_i = cross-sectional area of ith pipe, $A_i = \pi \cdot \frac{D_i^2}{4}$

degrees of freedom = (number of nodes) + (number of loops) -1 = 6 + 2 – 1 = 7

Cost of Conventional Water Treatment

The cost of producing and supplying water (excluding capital cost) by conventional water treatment is approximately Rs 5 / m³ at current prices. This comes to 0.5 paisa / L.

(This does not include the cost of water itself)

Thus for a middle class family of 4 using ~ 700 L of piped water / d, the water charges (only considering the cost of production and supply) per month are approximately Rs. 105.

However, even this modest cost of piped water supply cannot be recovered in many urban areas of our country.

Hence government subsidy is required for the construction and for the operation and maintenance of water supply systems.

Due to insufficient subsidy and other reasons including unavailability of raw water, water supply system in many urban areas are in a very bad shape. Insufficient water is supplied to many urban areas. Many areas are not covered at all by the water supply network.

People often have to get water by other means, i.e., by constructing their tube wells or by buying was from private tankers to survive.

Rural Water Supply

In rural areas, extensive treatment of water before supply is not feasible. Hence, in such areas, a clean water source must be identified, from which water can be supplied with little or no treatment.

The clean water sources in rural settings is deep-tube well water (in plains) and mountain springs (in hilly areas). This water is naturally aerated (to precipitate iron and remove odors) passed through a roughing filter (containing large stone media) and chlorinated, before being pumped to overhead tanks for supply. The distribution system consists of stand posts at various localities in the village.

Supply of surface water and shallow tube well water is generally discouraged due to the poor water quality and the possibility of microbial contamination, though this may be an option where such water is of good quality. The water should always be supplied after chlorination.

Treated surface water is only supplied in areas where the available groundwater is unfit for human consumption due to high salinity (coastal areas) or contamination with natural pollutants such as arsenic, fluoride, etc.

There are still many villages in India, where water supply is not assured and the residents are dependent on shallow tube wells, dug wells and other seasonal sources for their water supply. In the dry season, women often have to travel long distances to collect water.
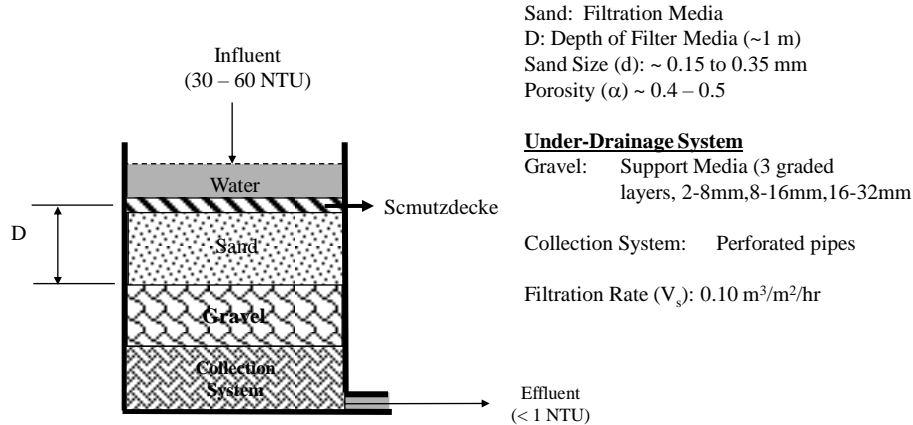
Rural / Semi-Urban Water Treatment Options

Rural / Semi-urban communities often cannot afford the cost and the complexities involved in operation and maintenance of conventional water supply system. The options open to such communities are the following,

1. Go for a high quality water supply like deep tube well (in plains) and mountain springs (in the hills), which can be supplied without treatment.

2. Use slow sand filtration systems if surface water is the raw water source

3. Use River-Bank filtration systems wherever possible

All water supplied must be microbiologically safe. Hence disinfection is always desirable, even for rural / semi-urban water supply systems, particularly as a check for contamination during distribution

Slow Sand Filtration

Sand: Filtration Media
D: Depth of Filter Media (~1 m)
Sand Size (d): ~ 0.15 to 0.35 mm
Porosity (α) ~ 0.4 – 0.5

Under-Drainage System
Gravel: Support Media (3 graded
             layers, 2-8mm,8-16mm,16-32mm

Collection System: Perforated pipes

Filtration Rate (V_s): 0.10 m^3/m^2/hr

Influent
(30 – 60 NTU)

D

Water

Scmutzdecke

Sand

Gravel

Collection
System

Effluent
(< 1 NTU)

The water passes slowly through the sand from top to bottom.   The sand particles at the top of the filter are colonized by microscopic organisms including bacteria and protozoans. In time, a 'slime' layer (Scmutzdecke) is formed on top of the sand. This layer is responsible for filtering the water. The mechanism of filtration is predominantly 'straining'.

The processes that occur in the schmutzdecke are enormously complex and varied, but the principal one is the mechanical straining of most of the suspended matter in a thin dense layer in which the pores may be very much less than a micron. The thickness of this layer increases with time from the initial installation to the point where the flow rates become unacceptably small.

After the initial installation of the SSF, the formation of the Schmutzdecke in the bed may take days or weeks depending strongly on the ambient temperature. During this period the processed water is unsafe for human consumption

When the filtration rate drops to an unacceptable level (~ 6 months of filter operation), the SSF must be cleaned by carefully removing about 25 mm of the top layer, which includes most of the existing Schmutzdecke, after dropping the water level to slightly below the latter.

In time the depth of the original bed of sand will have been reduced by the cleaning processes to about 0.75 meters (30"), when the original depth must restored. As the new sand will be almost devoid of biological activity, placing it on top of the existing sand would require an excessively long time to develop a new schmutzdecke. To accelerate this process, the schmutzdecke is first removed and discarded, and then most of the existing bed is removed and set aside for re-use. The new sand is placed in position and is then covered with the original bed. In this way the biological activity and new schmutzdecke are more rapidly restored.

Process Schematic:

Disinfection

| Storage | | SSF | | UGT |

River Bank Filtration

Riverbank filtration (RBF) operates by extracting water from wells located near rivers (~20 to 200 m away). If engineered correctly, most of the extracted RBF water originates from the river. As this river water passes through the riverbed sediments, contaminants are removed by overlapping biological, physical, and chemical processes.
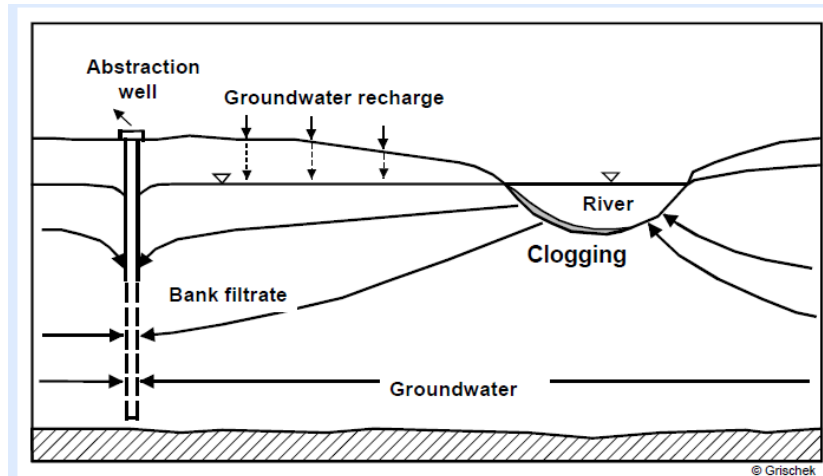

River Bank Filtration

Large-scale water abstraction by riverbank filtration is possible by installing multiple wells are shown in the figure below.

The water produced through riverbank filtration will generally have low turbidity and low concentrations of pathogenic microorganisms.

However, dissolved contaminants may sometimes be present in such waters. The objective of riverbank filtration is to supply this water for human consumption with just disinfection.
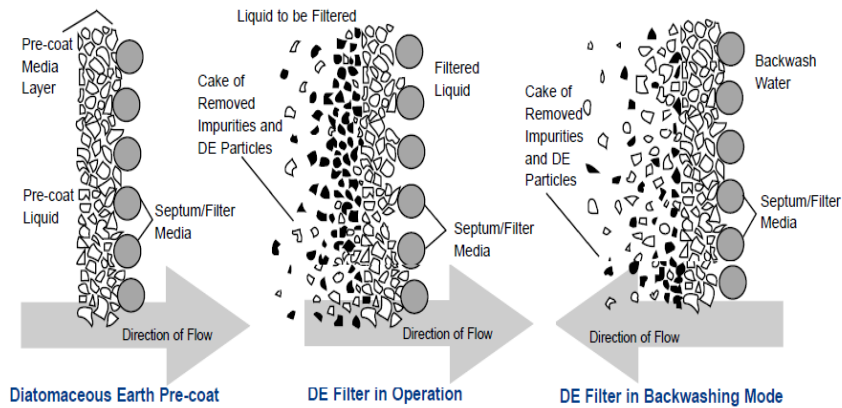
2/7/2020
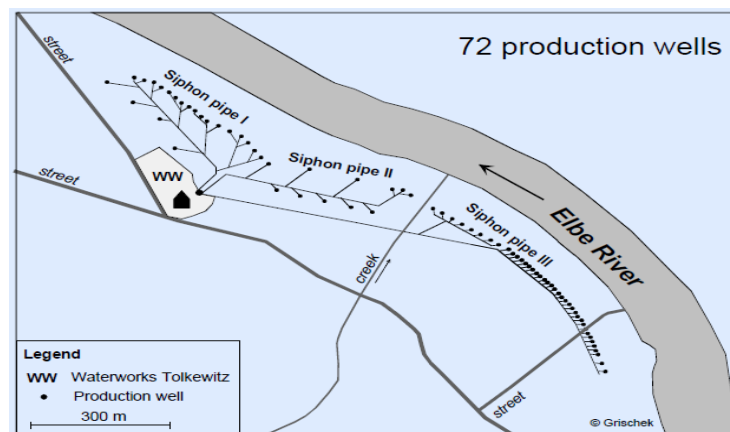
Emergency / Temporary Compact Water Treatment Option
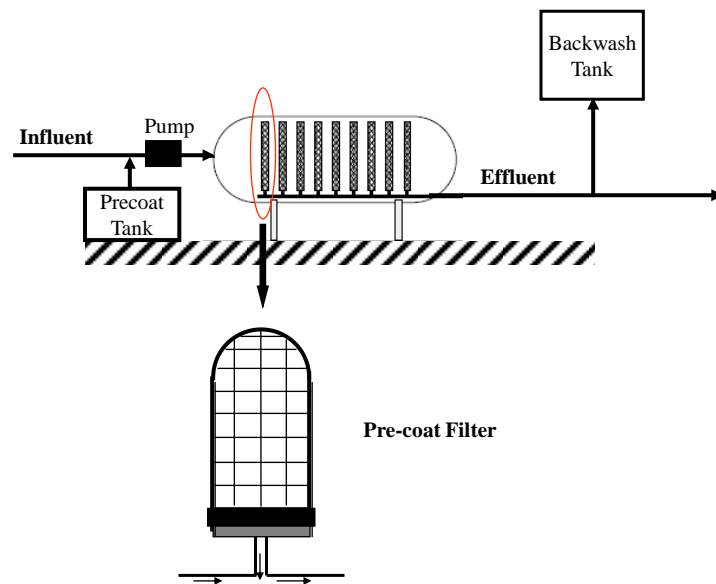**Precoat Filtration**
Pre-coat filtration, also known as diatomaceous earth filtration is popular for low cost supply of treated water in temporary / emergency situations.

Diatomaceous Earth is composed of skeleton of microscopic plants. Diatomaceous Earth is a good filter media because of its high permeability.
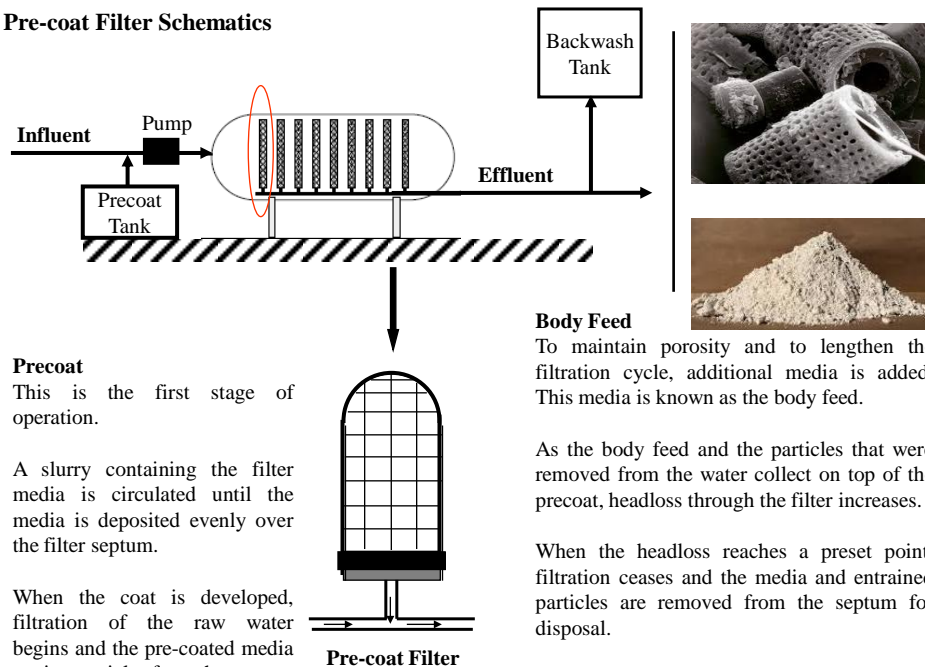
D.E. filtration is a process that removes particles from water by passing water through a layer of finely ground media (the D.E) that is deposited on a fine mesh screen, called a septum.
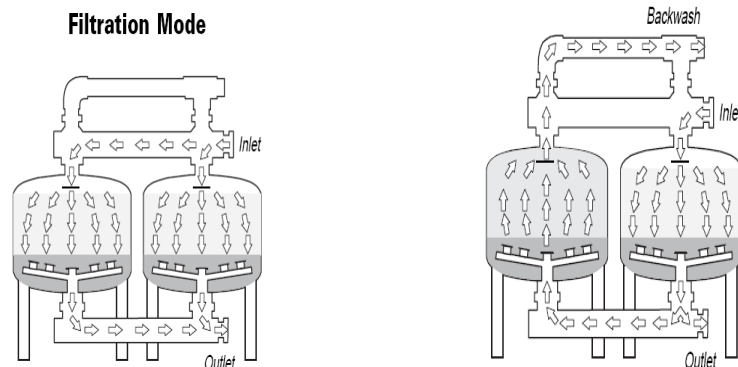

Source: Fulton, George P. 2000. *Diatomaceous Earth Filtration for Safe Drinking Water*

Pre-coat Filter Schematics

Pre-coat Filter Schematics

Backwash Tank

Influent

Pump

Effluent

Precoat Tank

Precoat

This is the first stage of operation.

A slurry containing the filter media is circulated until the media is deposited evenly over the filter septum.

When the coat is developed, filtration of the raw water begins and the pre-coated media strains particles from the water.

Pre-coat Filter

Body Feed

To maintain porosity and to lengthen the filtration cycle, additional media is added. This media is known as the body feed.

As the body feed and the particles that were removed from the water collect on top of the precoat, headloss through the filter increases.

When the headloss reaches a preset point, filtration ceases and the media and entrained particles are removed from the septum for disposal.

When the filter is cleaned, the precoat stage begins again.

Pressure Filtration

Filtration Mode

Backwash

Inlet

Outlet

Inlet

Outlet

Operating principle is the same as in a rapid sand filter

Due to higher pressure application, throughput in pressure filters are higher

These filters can be operated with higher headloss and hence filter run times are also longer

Compact systems, which can be purchased off the shelf and installed quickly

Advanced Water Treatment

Removal of all particles from water
Removal of dissolved contaminants from water
Removal of Residual color, odor and taste from water
Render a sparkling quality to the water

In India: mostly used in	**Bottled Water Industry**
	Home Water Treatment Systems

|	
Micro/ultra/nano filtration:	Filtration through membranes for particle and dissolved contaminant removal
Reverse osmosis:	Complete or partial removal of Na^+ and Cl^- from water
Cation exchange column:	Exchange all other cations in water with Na^+ ions
Anion exchange column:	Exchange all anions in water with Cl^- ions
Activated carbon adsorption:	Removal for dissolved organic matter from water
Ozonation:	Disinfection and removal of any residual color / taste / odor from water. Sparkling appearance.

Membrane Filtration

Water is passed through membranes of various pore sizes. Depending on the pore-size of the membrane, some impurities in water are 'rejected' by the membrane.

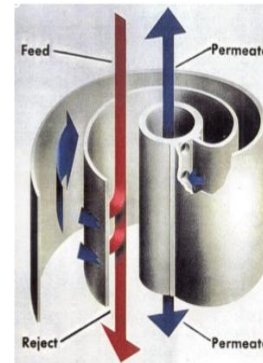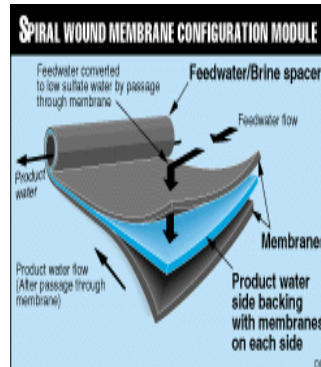The water passing through the membrane is free of contaminants.
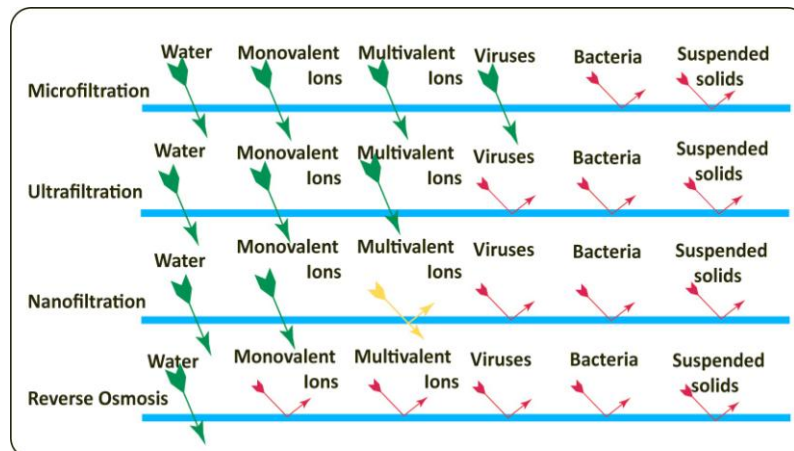
Membrane Filtration
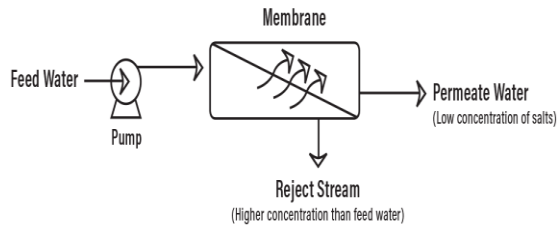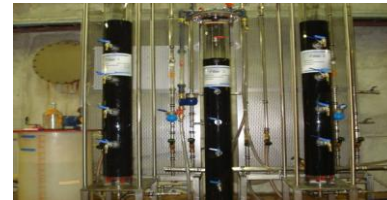

Hollow Fiber Membrane
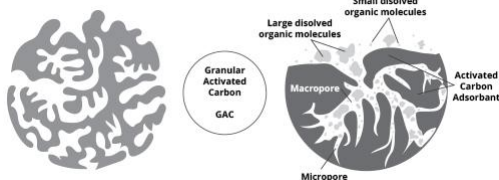

Spiral Wound Membrane

Granular Activated Carbon (GAC) Adsorption

Granular Activated Carbon (GAC)
(> 1 mm size)

Lab-Scale GAC Columns

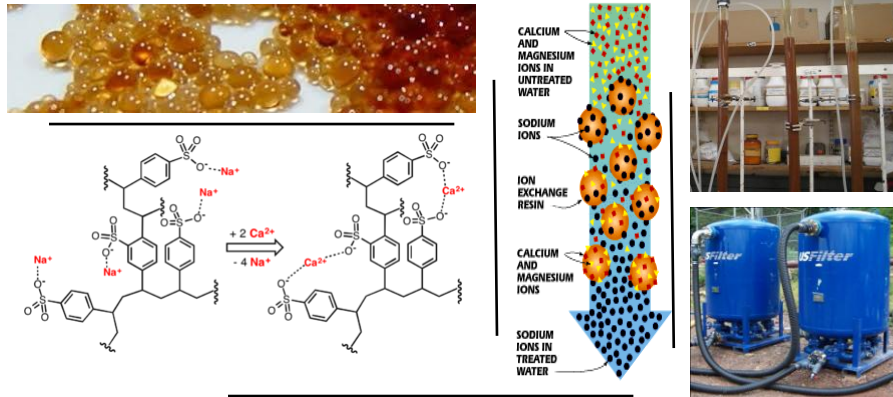Very Porous: Specific Surface Area, 1000 – 3000 m²/g

Full-Scale GAC Columns

GAC columns are very efficient in removing relatively non-polar dissolved organic carbon molecules from water. Once exhausted, GAC can be heated for regeneration

Ion Exchange: Hardness, Arsenic and Fluoride Removal

Cation Exchange Resins remove Hardness from Water

Anion Exchange Resins remove Fluoride, Arsenic and Nitrate from Water

Arsenic Removal Resin **Fluoride Removal Resin** **Nitrate Removal Resin**

Ozonation