Unit 3

WATER TREATMENT

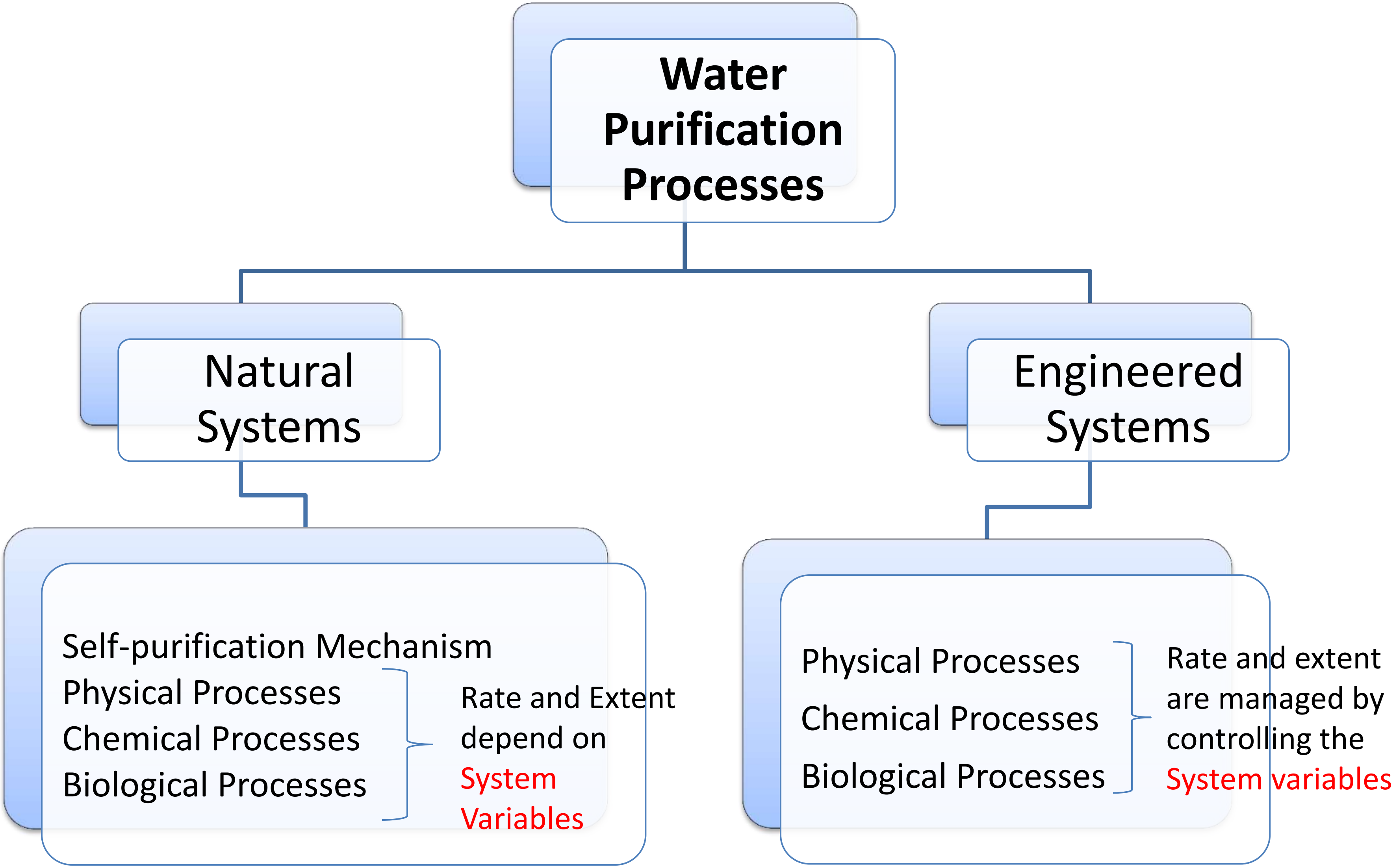
Objective of Water Treatment

- To produce an adequate and continuous supply of water that is chemically, bacteriologically and aesthetically pleasing.

Water treatment must produce water that is :

- Palatable (no unpleasant taste)
- Safe (does not contain pathogens or chemicals harmful to the consumer)
- Clear (free from suspended solids and turbidity)
- Colourless and Odourless (aesthetic to drink)
- Reasonably soft (allows consumers to wash clothes, dishes without excessive quantities of detergents or soap)
- Non-corrosive (to protect pipe work and prevent leaching of metals from tanks or pipes)
- Low organic content

Water Purification Processes



Water Purification Processes

Natural Systems

Self-purification Mechanism
Physical Processes
Chemical Processes
Biological Processes

Rate and Extent depend on System Variables

Engineered Systems

Physical Processes
Chemical Processes
Biological Processes

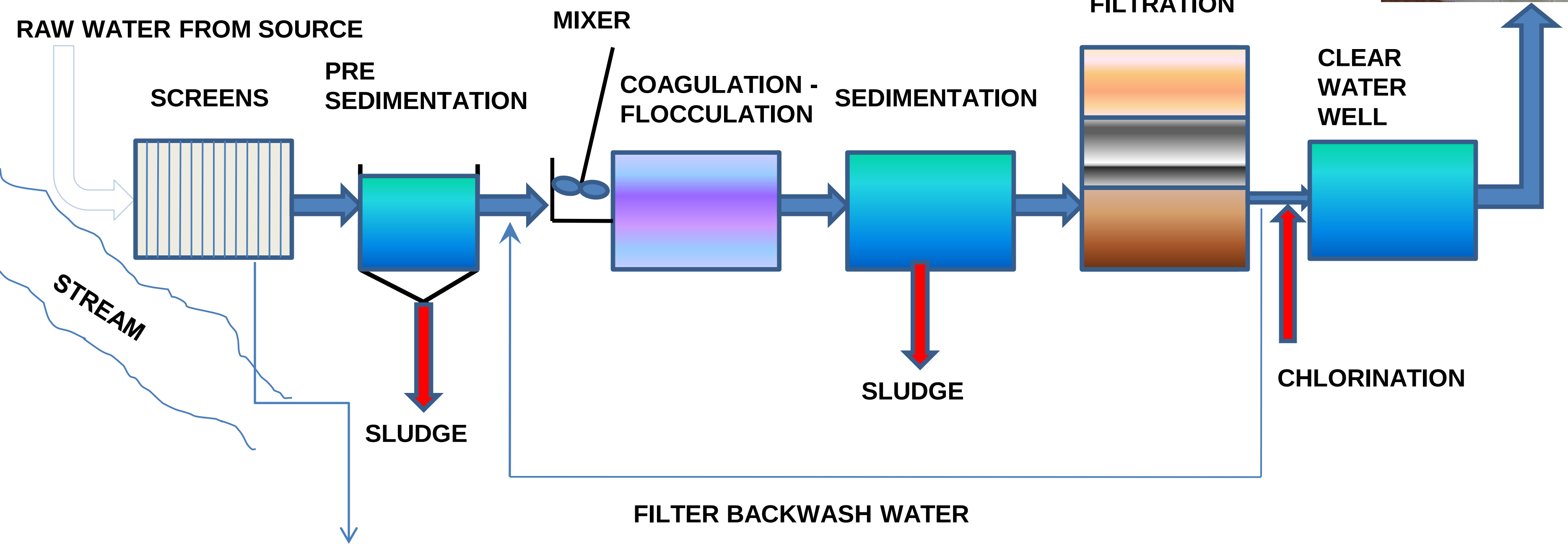
Rate and extent are managed by controlling the System variables

System Variables – Hydraulic Characteristics eg., volume, rate, turbulence of flow; physical characteristics of bottom and bank material; variations in sunlight and temperature; chemical nature of natural water

- Major physical processes involved in self-purification of watercourses are dilution, sedimentation and re-suspension, filtration, gas transfer and heat transfer
- Natural water courses contain many dissolved minerals and gases that interact chemically with one another in complex and varied ways. Oxidation-Reduction, dissolution-precipitation and other chemical conversions may alternately aid or obstruct natural purification processes of natural water systems
- Biological processes have found little use in the treatment of potable water supplies because of the low levels of biodegradable organics in raw water. However biological processes are used extensively in wastewater treatment

CONVENTIONAL WATER TREATMENT PLANT

DISTRIBUTION
RESERVOIR



RAW WATER FROM SOURCE

STREAM

SCREENS

PRE
SEDIMENTATION

SLUDGE

FLASH
MIXER

COAGULATION -
FLOCCULATION

SEDIMENTATION

SLUDGE

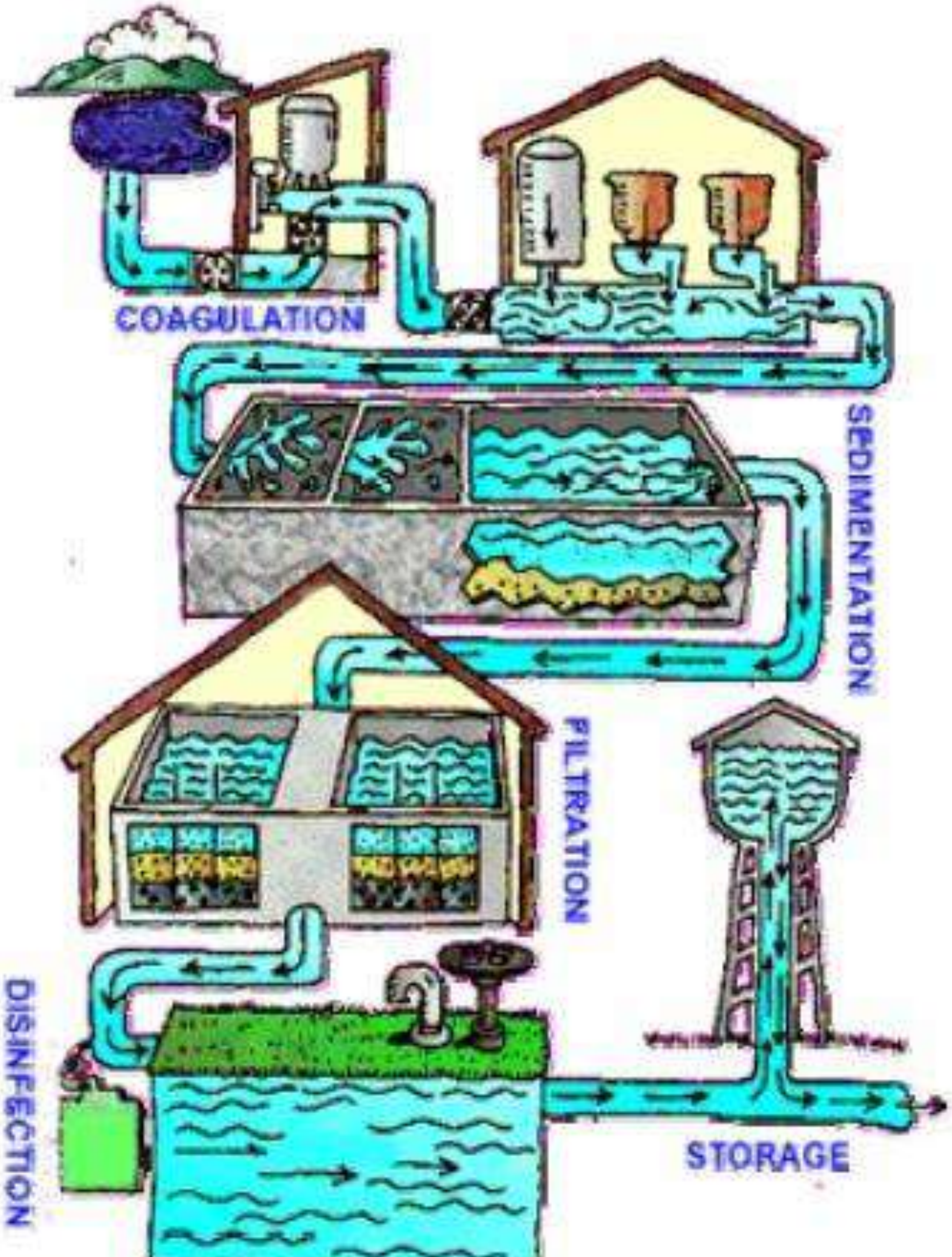
FILTRATION

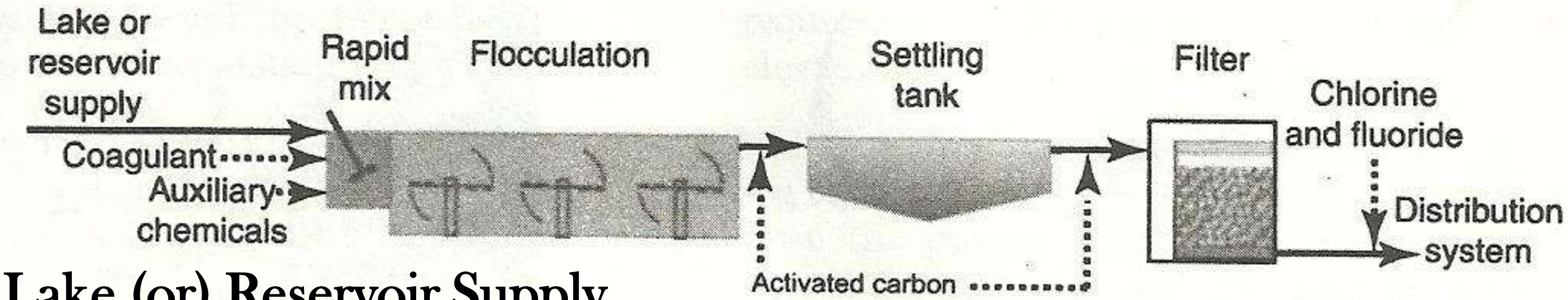
CLEAR
WATER
WELL

CHLORINATION

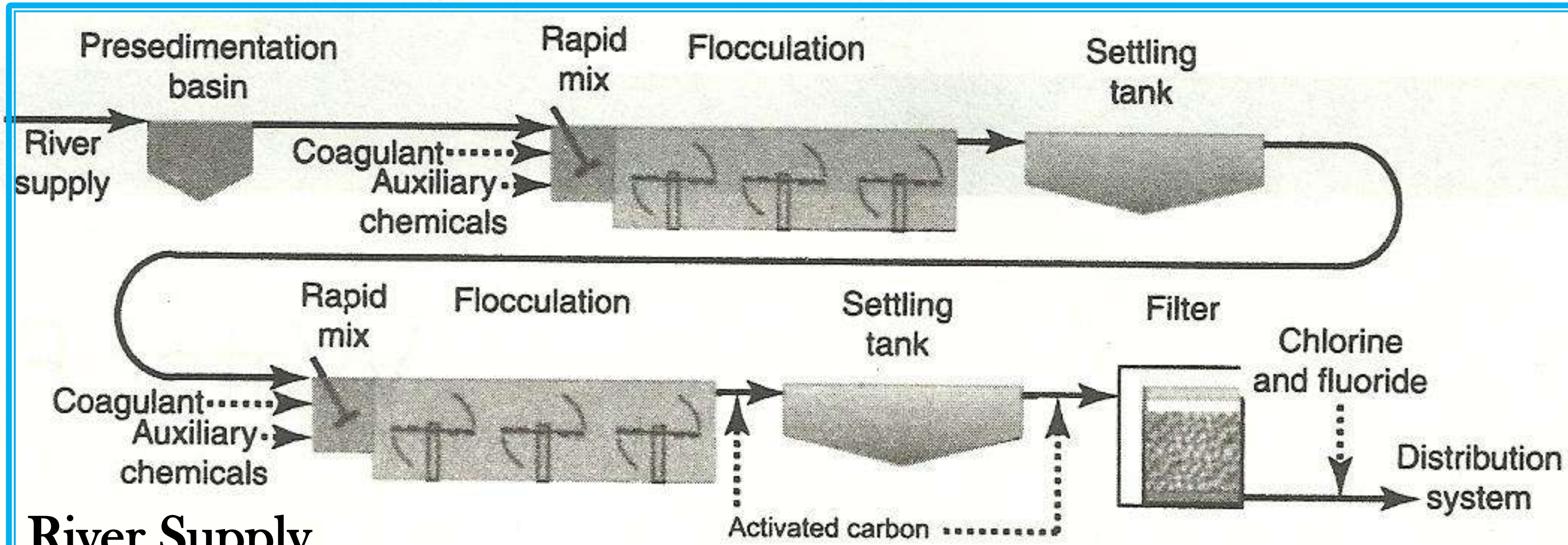
FILTER BACKWASH WATER

SCREENINGS FOR
BURIAL/DISPOSAL/INCINERATION/GRINDING

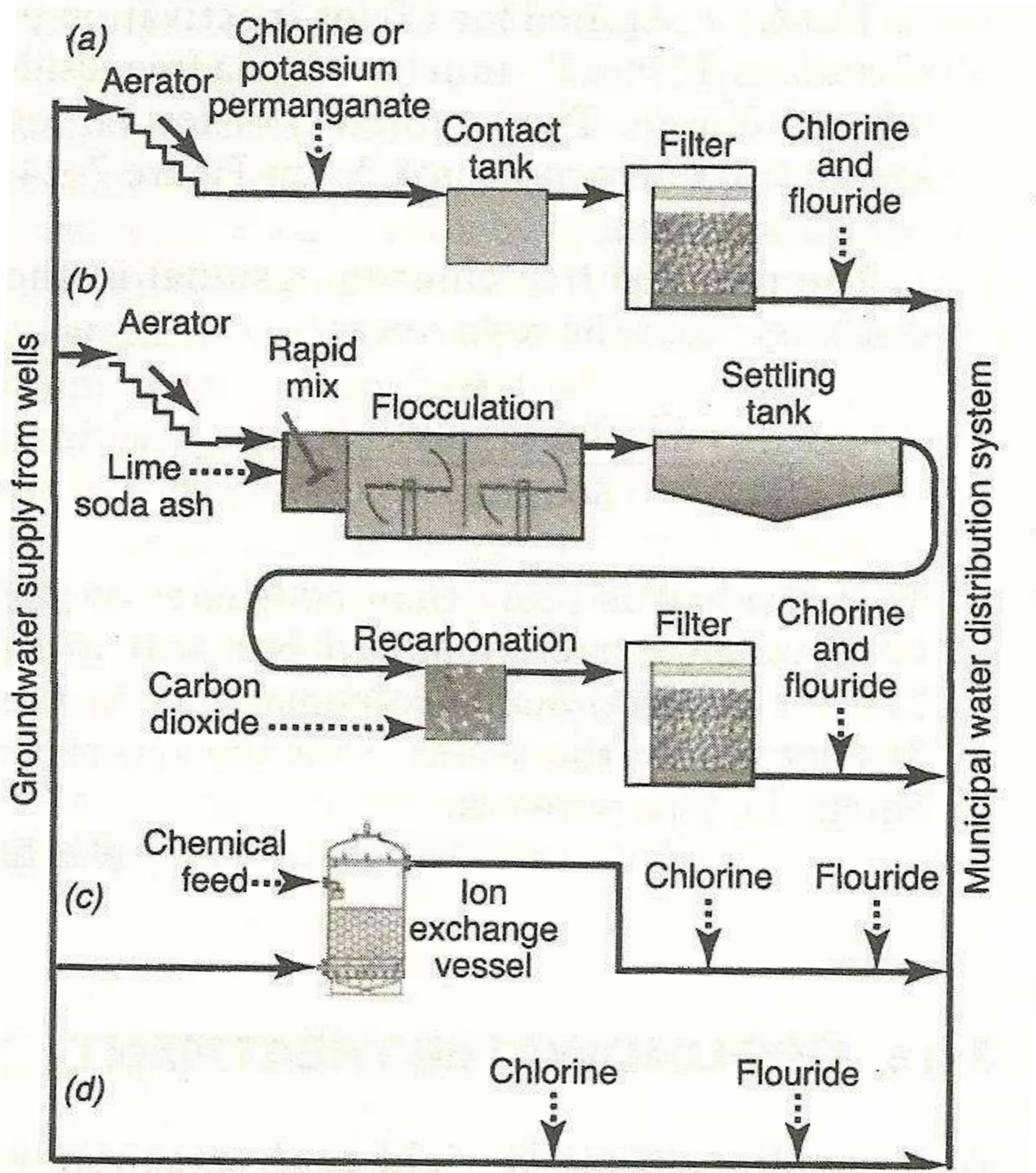

COAGULATION
SEDIMENTATION
FILTRATION
DISINFECTION
STORAGE

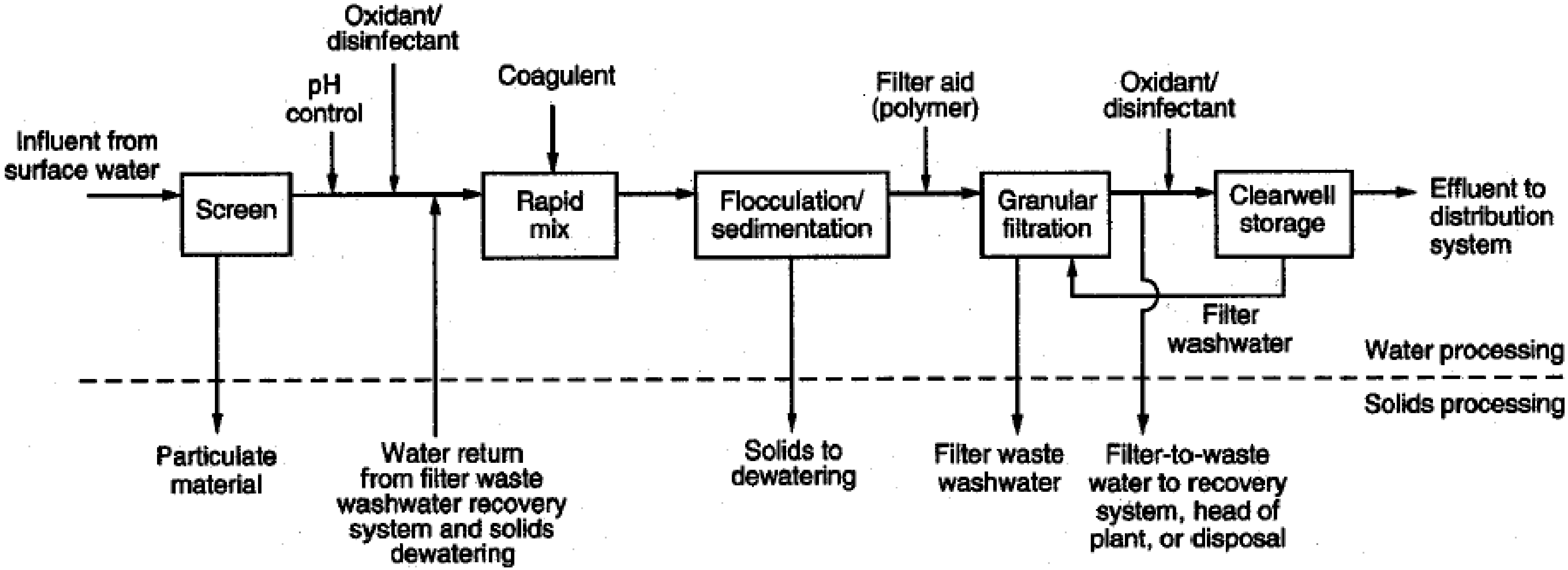


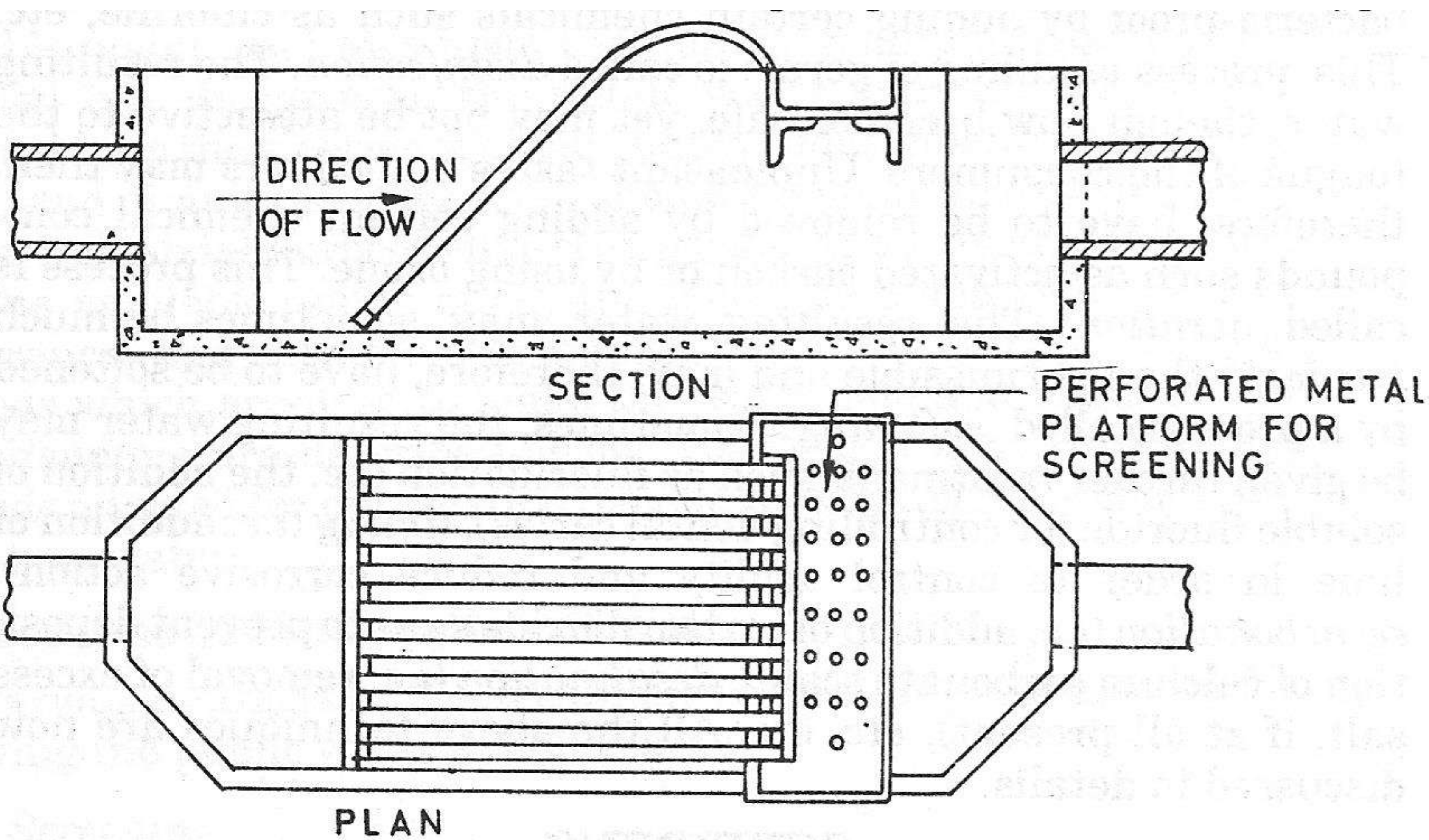
Lake (or) Reservoir Supply

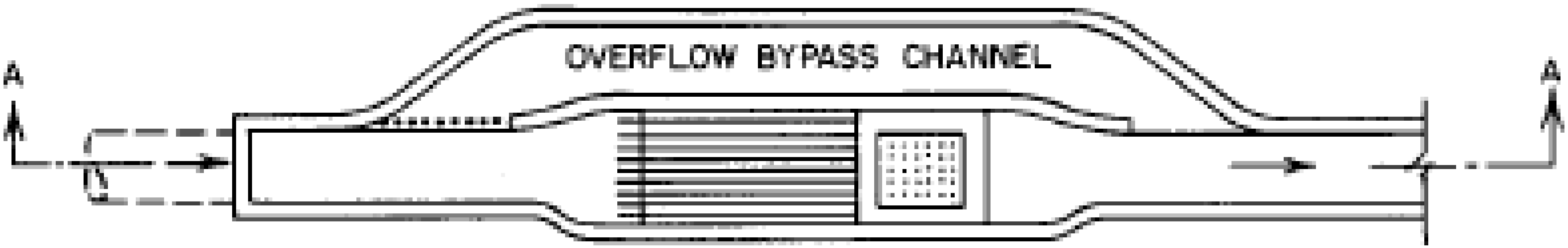


River Supply

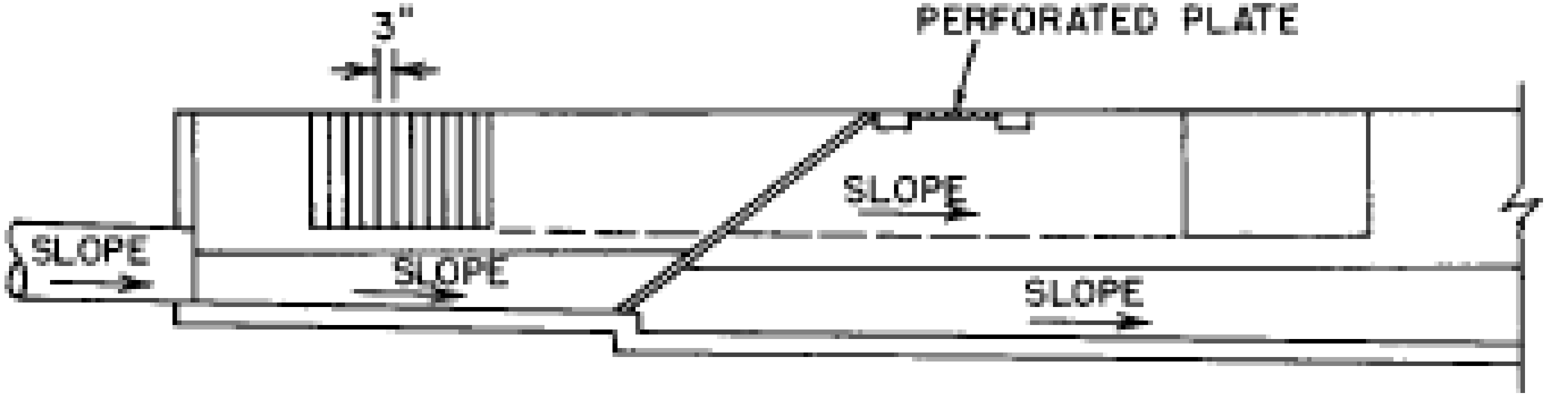

(a)
Aerator
Chlorine or potassium permanganate
Contact tank
Filter
Chlorine and flouride
(b)
Aerator
Rapid mix
Flocculation
Settling tank
Lime soda ash
Recarbonation
Carbon dioxide
Filter
Chlorine and flouride
Chemical feed
(c)
Ion exchange vessel
Chlorine
Flouride
(d)
Chlorine
Flouride
Groundwater supply from wells
Municipal water distribution system


Influent from surface water
Screen
pH control
Oxidant/ disinfectant
Coagulent
Rapid mix
Flocculation/ sedimentation
Filter aid (polymer)
Granular filtration
Oxidant/ disinfectant
Clearwell storage
Effluent to distribution system
Filter washwater
Water processing
Solids processing
Particulate material
Water return from filter waste washwater recovery system and solids dewatering
Solids to dewatering
Filter waste washwater
Filter-to-waste water to recovery system, head of plant, or disposal


DIRECTION
OF FLOW
SECTION
PERFORATED METAL
PLATFORM FOR
SCREENING
PLAN


A
OVERFLOW BYPASS CHANNEL
A

A.


3"
PERFORATED PLATE
SLOPE
SLOPE
SLOPE
SLOPE

B.

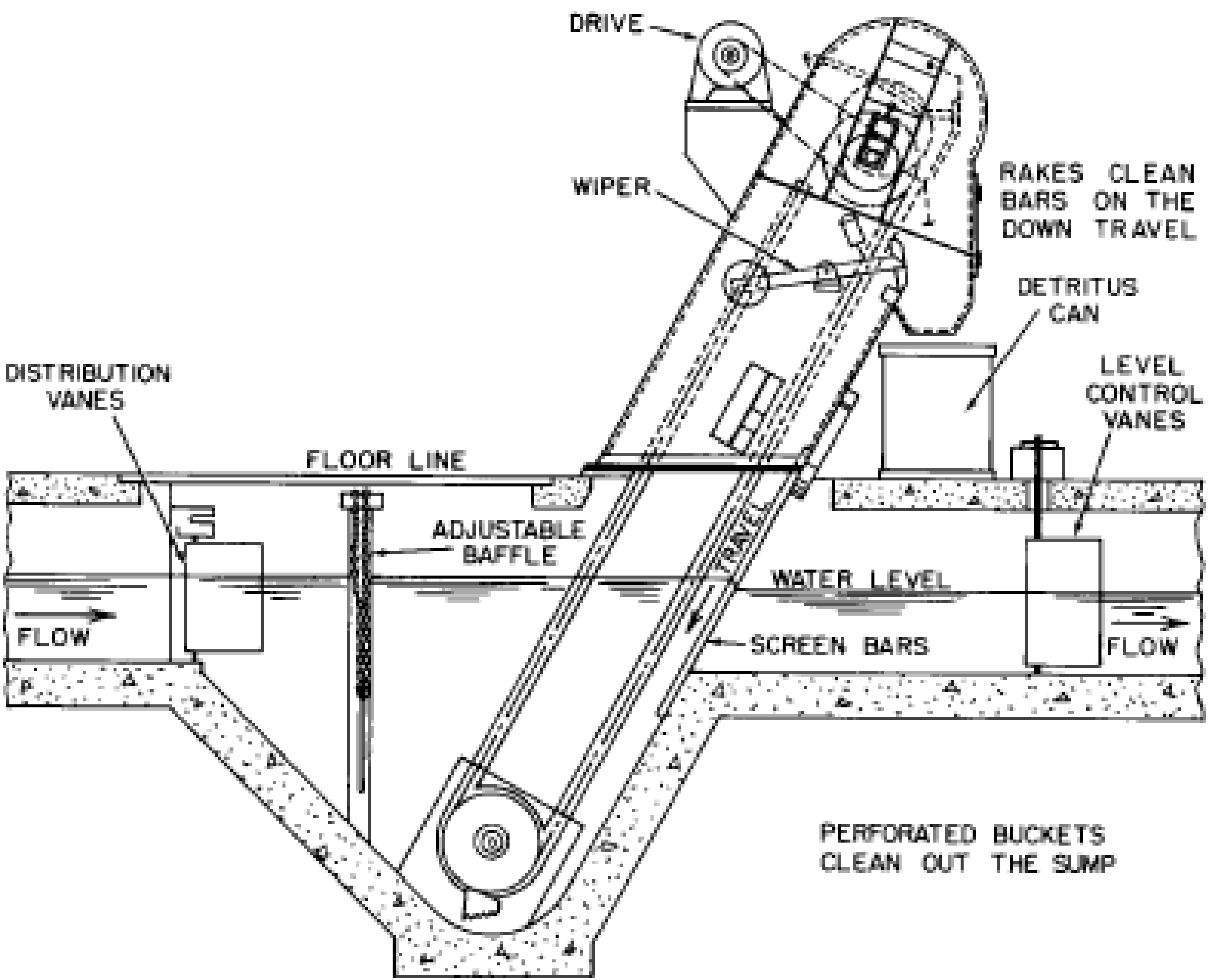


FIG. 7.14.4 Mechanical bar screen and grit collector.

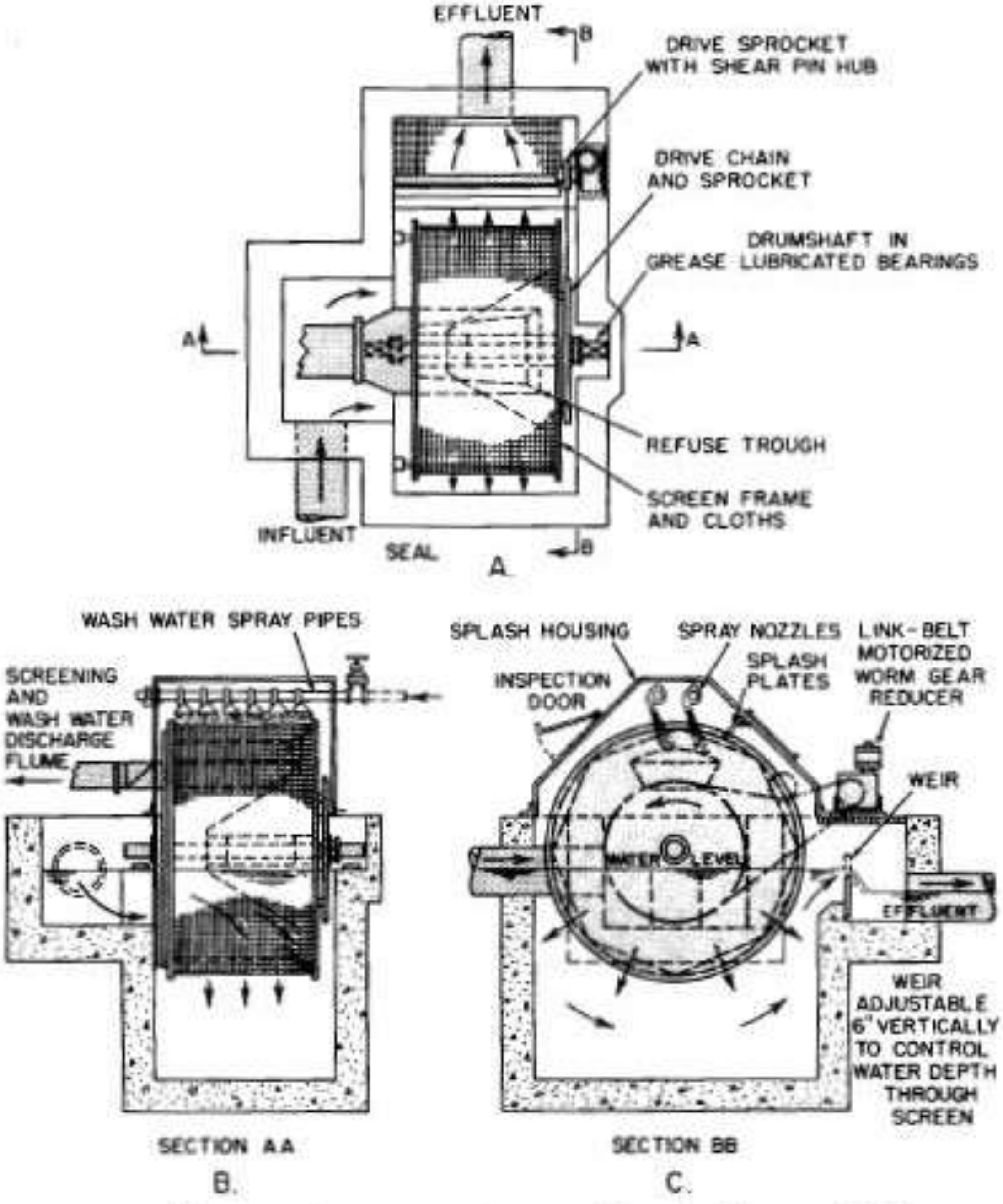


FIG. 7.14.5 Revolving-drum screen with outward flow. A. Plan view; B. Side view; C. End view.

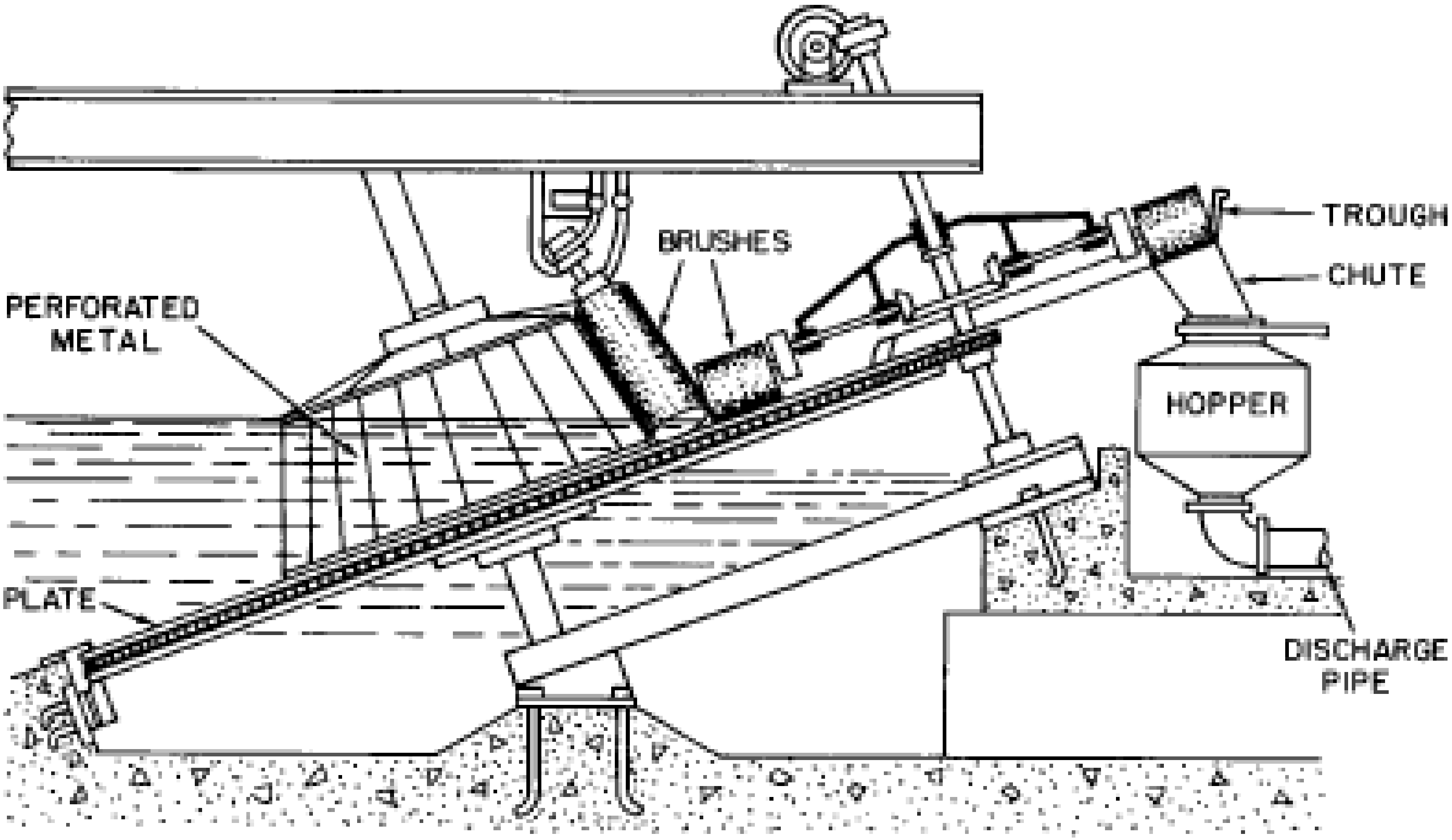
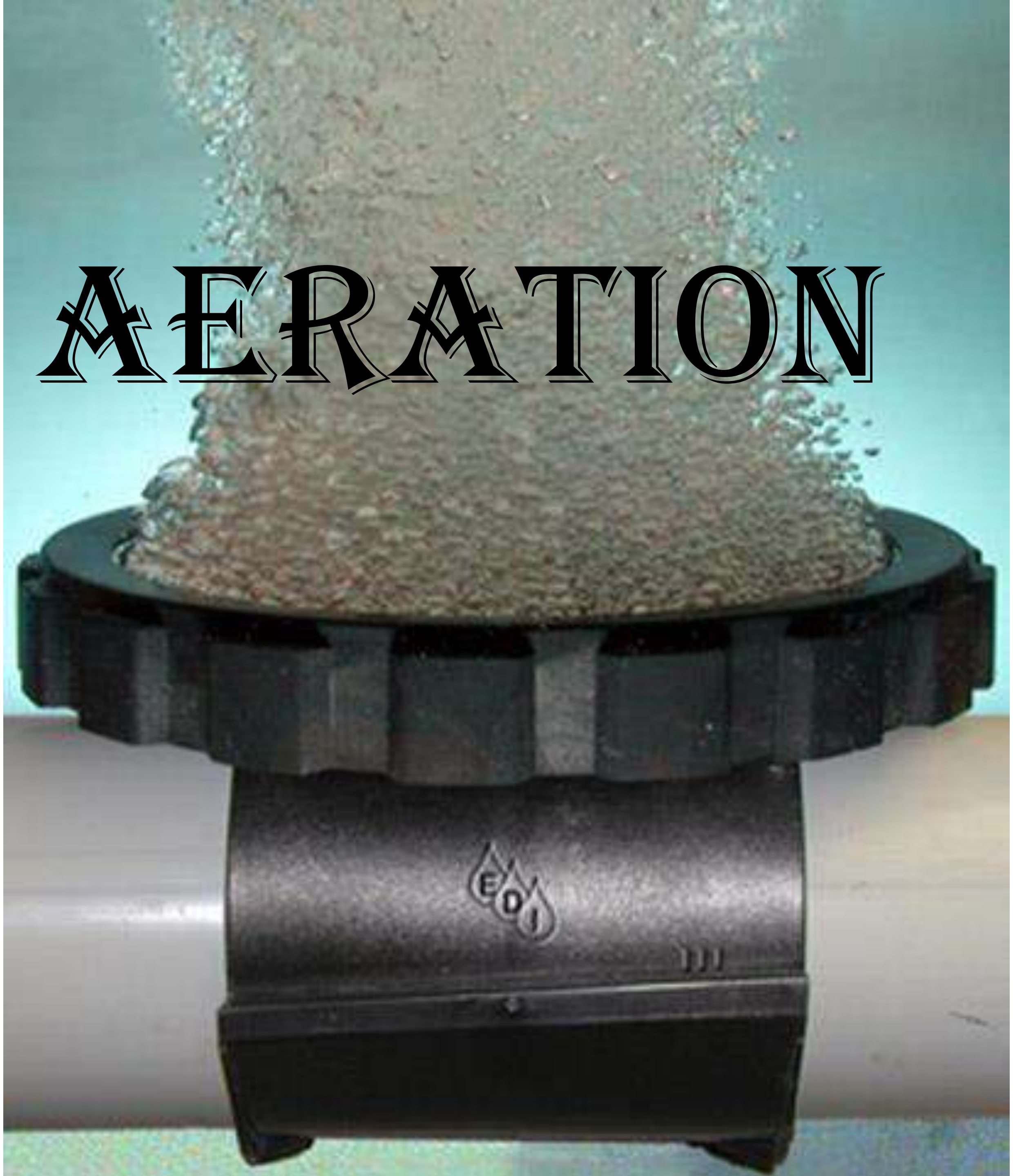


FIG. 7.14.7 Inclined revolving-disk screen.

AERATION



Aeration

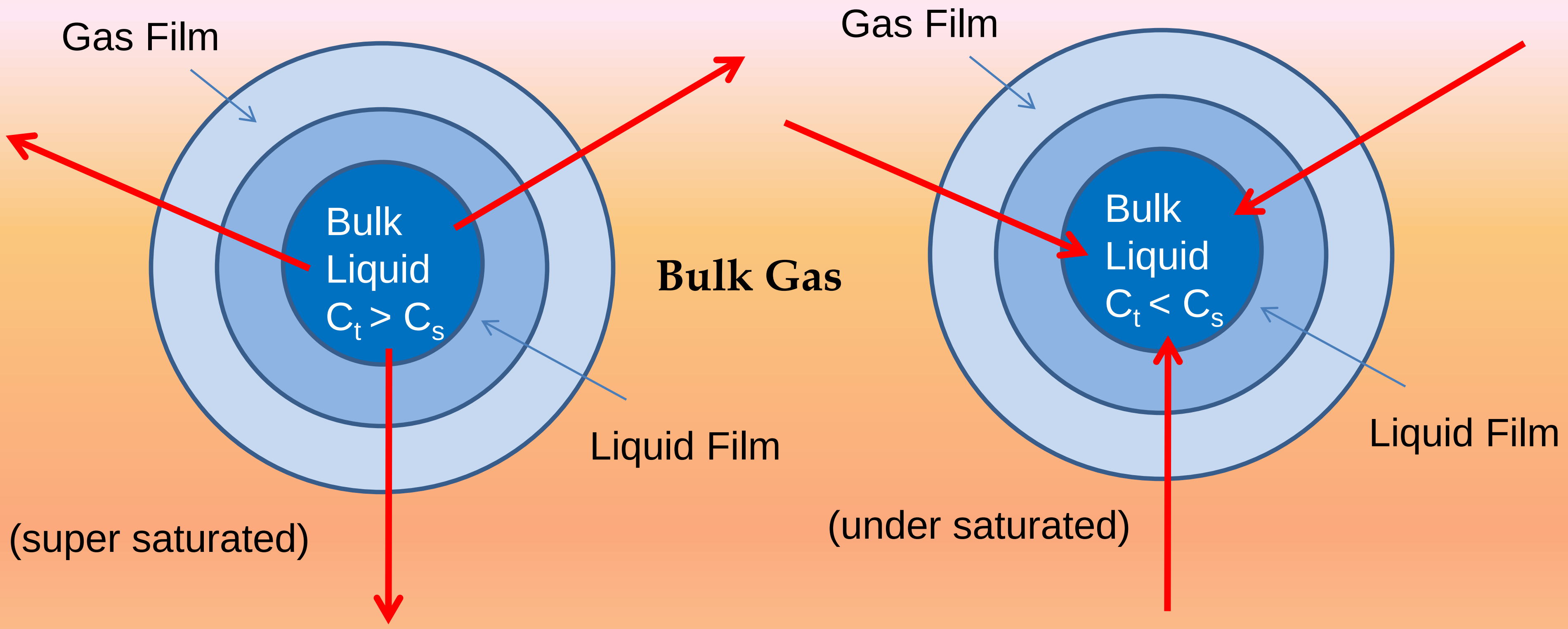
- Process sometimes used in preparing potable water (often groundwater's)
- Removal of
  - undesirable gases dissolved in water (degasification)
  - convert undesirable substances to a more manageable form by addition of Oxygen (oxidation)
- Undesirable gases
  - Carbon Dioxide (CO_2) – excess amount of CO_2 in water results in corrosive water
  - Hydrogen Sulfide (H_2S) – imparts unpleasant taste and odour

- Undesirable Substances
  - Volatile Liquids (e.g., Humic acids), Phenols – excessive quantities can be reduced to more manageable levels
  - Iron and Manganese (both Iron and Manganese are soluble in significant quantities only in +2 oxidation state i.e., Fe+2 and Mn+2). Upon contact with oxygen (or any other oxidizing agents), both Ferrous Iron and Manganese are oxidized to higher valences, forming new ionic complexes that are not soluble and removed as precipitate after aeration

Iron and Manganese

- widely distributed in nature
- present in Groundwater and Hypolimnion of Stratified Lakes
- significantly soluble in +2 state i.e., Fe^{2+} and Mn^{2+}
- Upon contact with oxygen or any other oxidizing agent, Fe^{2+} and Mn^{2+} are oxidized to higher states – new ionic complexes are formed which are not soluble
  - $4Fe^{2+} + O_2 + 10H_2O \longrightarrow 4Fe(OH)_3\downarrow + 8H^+$
  - $2Mn^{2+} + O_2 + 2H_2O \longrightarrow 2MnO_2\downarrow + 4H^+$

Water dispersed in Air



Gas Film

Bulk Liquid $C_t > C_s$

Liquid Film

(super saturated)

Bulk Gas

Gas Film

Bulk Liquid $C_t < C_s$

Liquid Film

(under saturated)

DESORPTION

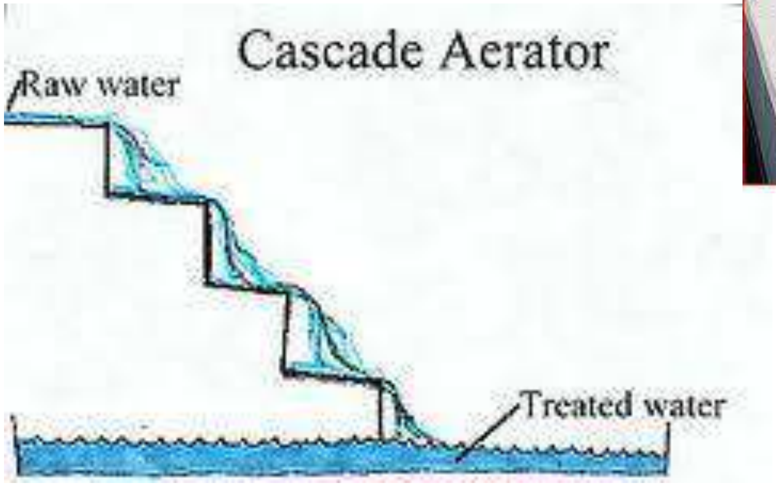
ABSORPTION

Cs – saturation concentration of gas in water at a given temperature (max. conc. In absorption)

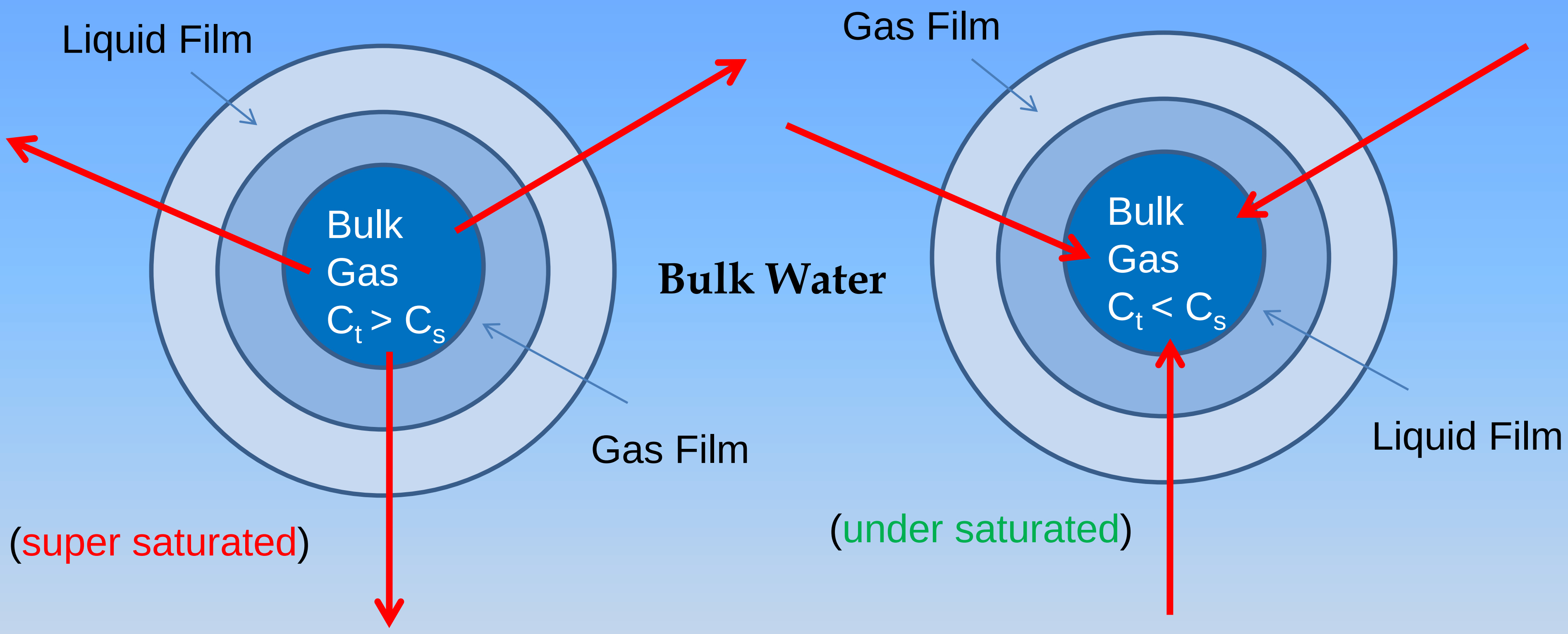
Ct – conc. At time t

FOUNTAINS

Cascade Aerators



Air dispersed in Water



Liquid Film

Bulk Gas $C_t > C_s$

Gas Film

(super saturated)

Bulk Water

Gas Film

Bulk Gas $C_t < C_s$

Liquid Film

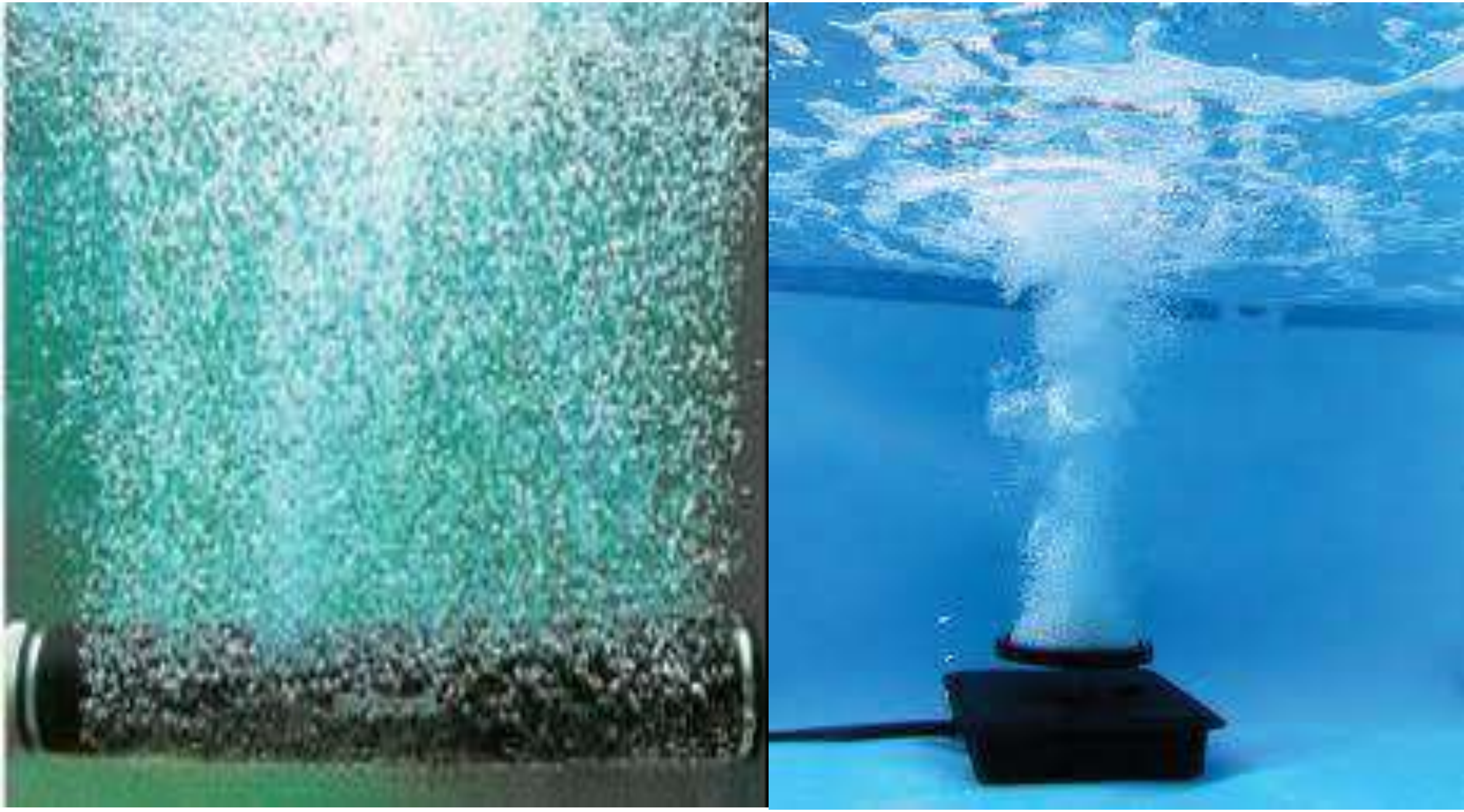
(under saturated)

DESORPTION

ABSORPTION

Cs – saturation concentration of gas in water at a given temperature (max. conc. In absorption)

Ct – conc. At time t



Solids Separation

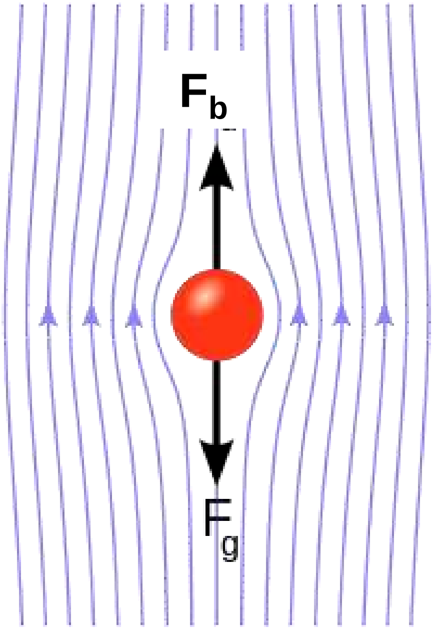
- Discreet Particles – size, shape and specific gravity do not change with time
- Flocculating Particles – Particles whose surface properties are such that they aggregate, or coalesce, with other particles upon contact, thus changing shape, size and perhaps specific gravity with each contact are called flocculating particles.

- Dilute Suspensions – Suspensions in which the concentration of particles is not sufficient to cause significant displacement of water as they settle
- Concentrated Suspensions – Suspensions in which the concentration of particles is too great

Type-I Settling
Discreet Particles in Dilute Suspension

Diameter of Particle	Type of Particle	Settling time through 1 m. of water
10mm	Gravel	1 seconds
1mm	Sand	10 seconds
0.1mm	Fine Sand	2 minutes
10 micron	Protozoa, Algae, Clay	2 hours
1 micron	Bacteria, Algae	8 days
0.1 micron	Viruses, Colloids	2 years
10 nm	Viruses, Colloids	20 years
1 nm	Viruses, Colloids	200 years

Settling Time for Particles of Various Diameters;



Force of Gravity

$$f_g = \rho_p g V_p$$

Density of particle

Buoyant Force

$$f_b = \rho_w g V_p$$

Volume of particle

Driving Force

$$f_{net} = (\rho_p - \rho_w) g V_p$$

Drag Force

$$f_d = C_D A_p \rho_w \frac{v^2}{2}$$

Drag force = Driving Force

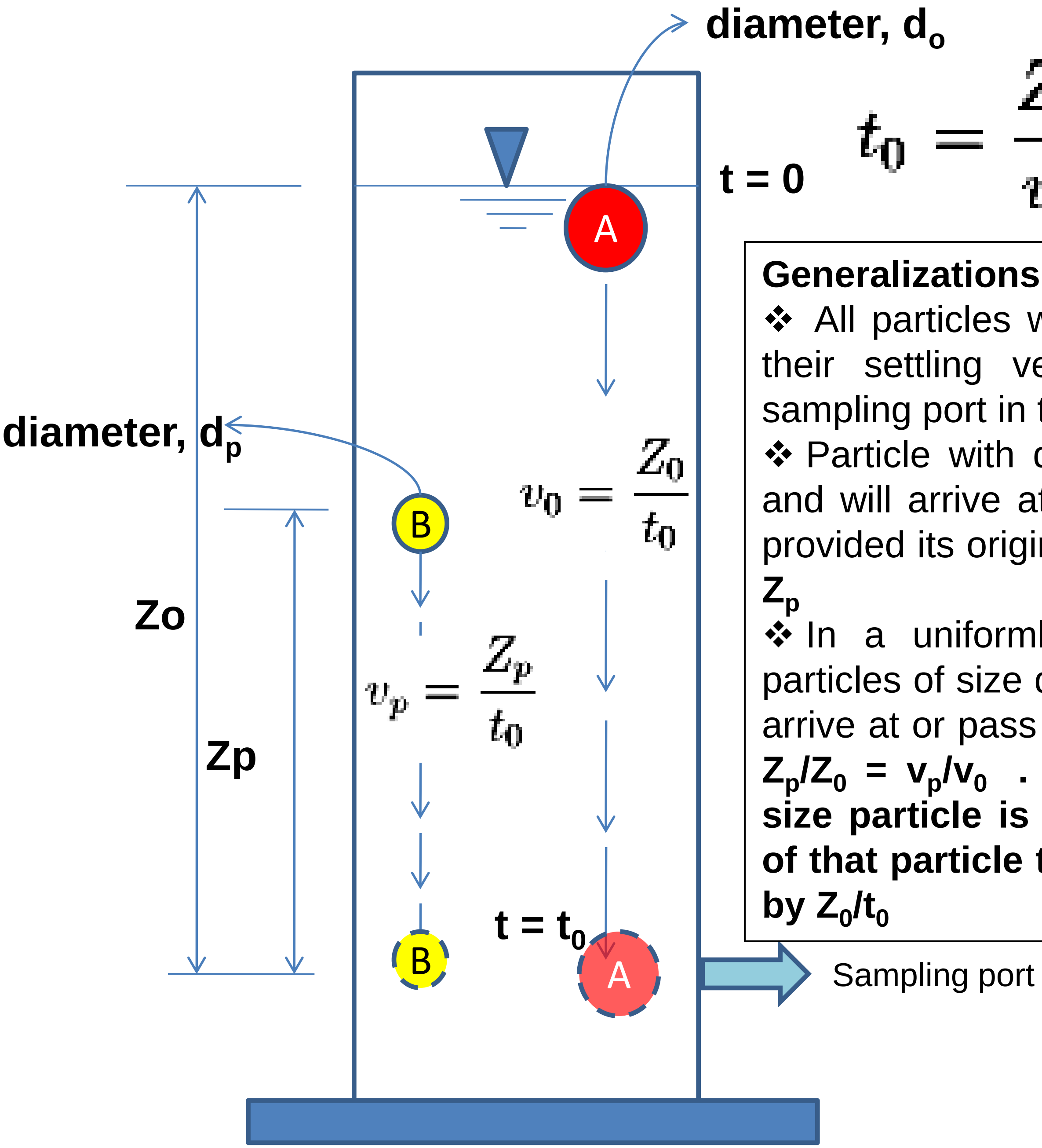
$$\left(\rho_p - \rho_w\right) g V_p = C_D A_p \rho_w \frac{v^2}{2}$$

$$\frac{V_p}{A_p} = \frac{\frac{4}{3}\pi\left(d/2\right)^3}{\pi\left(d/2\right)^2} = \frac{2}{3}d$$

$$v^2 = \frac{4}{3} g \frac{(\rho_p - \rho_w)d}{C_d \rho_w}$$

$$\boxed{\begin{aligned} C_D &= \frac{24}{R_e}(laminar) \\ &= \frac{24}{R_e} + \frac{3}{R_e^{1/2}} + 0.34(transitional) \\ &= 0.4(turbulent)) \end{aligned}} \quad R_e = \frac{\phi v \rho_w d}{\mu}$$

$$\boxed{v_t = \frac{g\left(\rho_p - \rho_w\right) d^2}{18\mu}} \Rightarrow \text{Stoke's Equation}$$



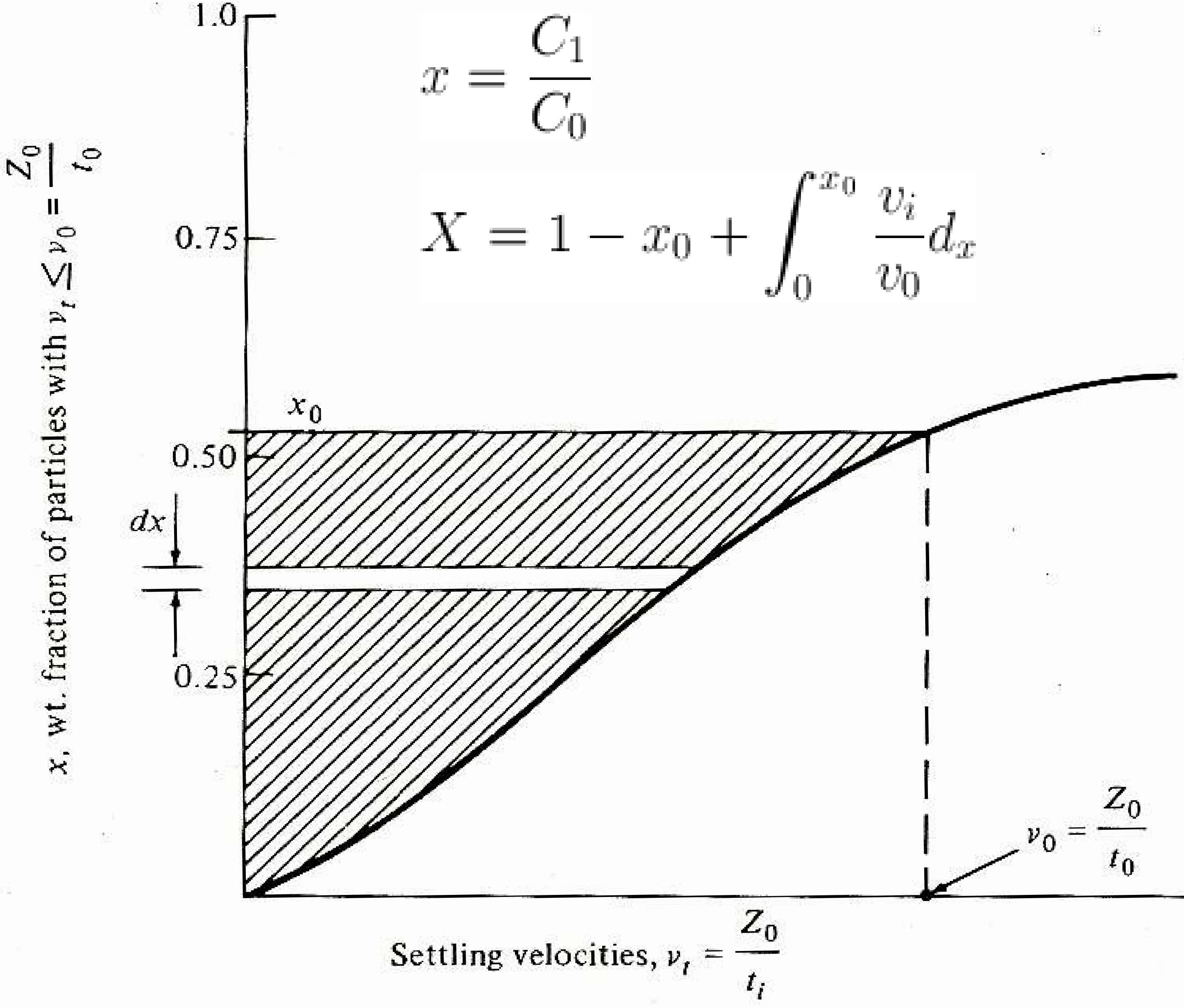
$$t_0 = \frac{Z_p}{v_p} = \frac{Z_0}{v_0} \therefore \frac{v_p}{v_0} = \frac{Z_p}{Z_0}$$

Generalizations:

❖ All particles with diameter **≥ d_0** and such that their settling velocities are **≥ v_0** will arrive at sampling port in time **t_0**.

❖ Particle with diameter **$d_p < d_0$** will have **v_p** < **v_o** and will arrive at or pass sampling port in time t0, provided its original position was at or below a point **Z_p**

❖ In a uniformly mixed suspension, fraction of particles of size d_p with settling velocity v_p which will arrive at or pass the sampling port in time t0 will be **$Z_p/Z_0 = v_p/v_0$. Thus removal efficiency of any size particle is the ratio of the settling velocity of that particle to the settling velocity v_0 defined by Z_0/t_0**



$$x = \frac{C_1}{C_0}$$

$$X = 1 - x_0 + \int_0^{x_0} \frac{v_i}{v_0} d_x$$

Example:

A settling analysis is run on a type-1 suspension. The column is 1.8 m deep, and data are shown below. What will be the theoretical efficiency in a settling basin with a loading rate of 25 $m^3/day.m^2$ (25 m/day)?

Time, min	**0**	**60**	**80**	**100**	**130**	**200**	**240**	**420**
Conc., mg/L	**300**	**189**	**180**	**168**	**156**	**111**	**78**	**27**

C_0

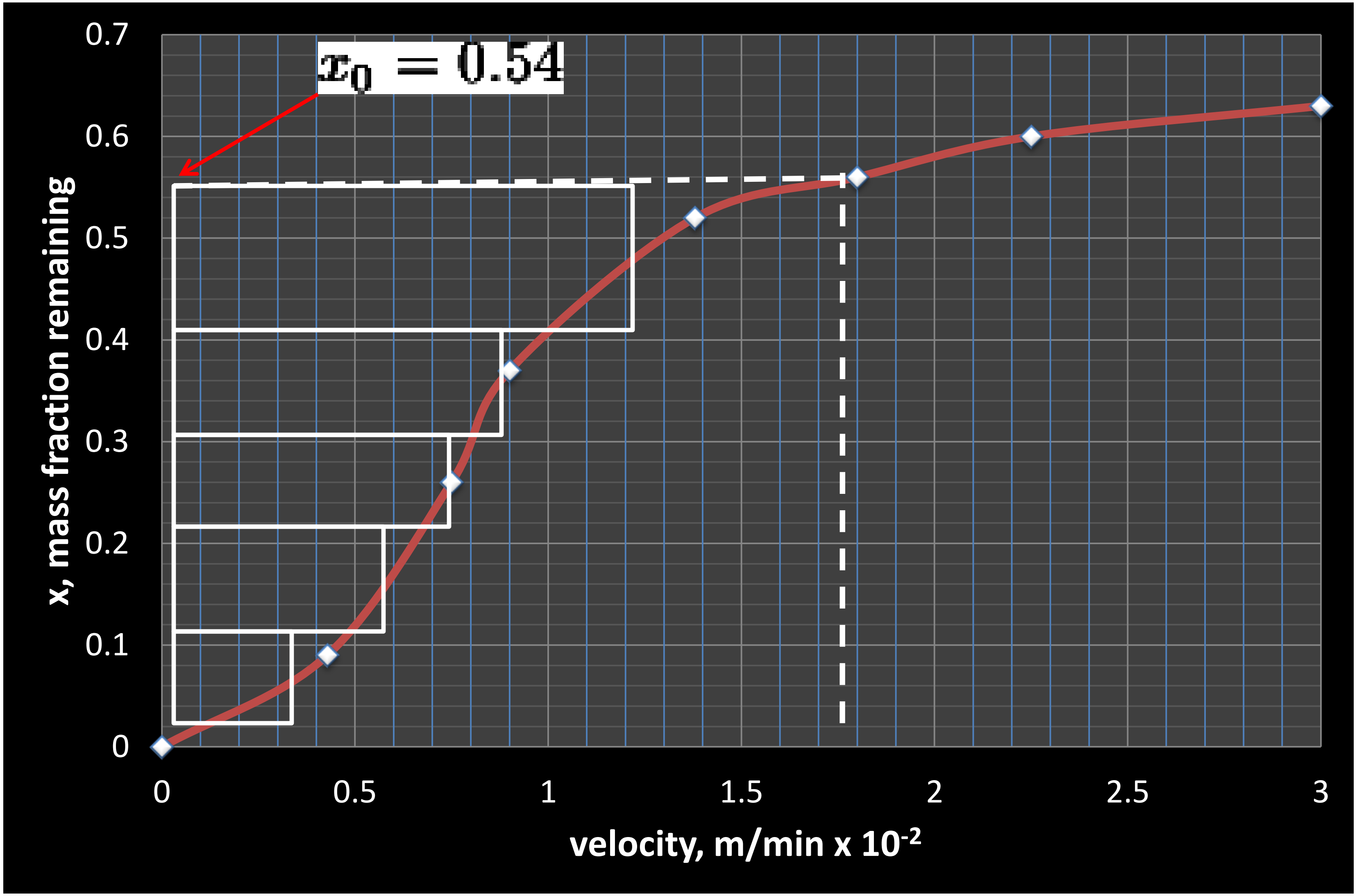
Solution:

Calculate mass fraction remaining and corresponding settling rates

Time, min	60	80	100	130	200	240	420
Mass Fraction Remaining, C_i/C_0	$\frac{189}{300}$ = **0.63**	$\frac{180}{300}$ = **0.6**	$\frac{168}{300}$ = **0.56**	$\frac{156}{300}$ = **0.52**	$\frac{111}{300}$ = **0.37**	$\frac{78}{300}$ = **0.26**	$\frac{27}{300}$ = **0.09**
$v_t = 10^{-2}$, m/min	$\frac{1.8}{60}$ = **3.0**	$\frac{1.8}{80}$ = **2.25**	$\frac{1.8}{100}$ = **1.8**	$\frac{1.8}{130}$ = **1.38**	$\frac{1.8}{200}$ = **0.9**	$\frac{1.8}{240}$ = **0.75**	$\frac{1.8}{420}$ = **0.428**

Determine v_0 = 25 m^3/m^2.day = 1.74 x 10^{-2} m/min.

$$v_0 = 1.74 \times 10^{-2} \text{m/min}$$


$x_0 = 0.54$
x, mass fraction remaining
velocity, m/min x 10^{-2}
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0
0.5
1
1.5
2
2.5
3

Determine Δx.vi

Δx	v_i	$\Delta x \,.\, v_i$
0.1	0.3	0.03
0.1	0.5	0.05
0.1	0.7	0.07
0.1	0.85	0.085
0.14	1.2	0.168
= 0.54	$\sum \Delta x \,.\, v_i$	0.403

$$X = (1 - x_0) + \sum \frac{\Delta x . v_t}{v_0}$$

$$= (1 - 0.54) + \frac{0.379}{1.74}$$

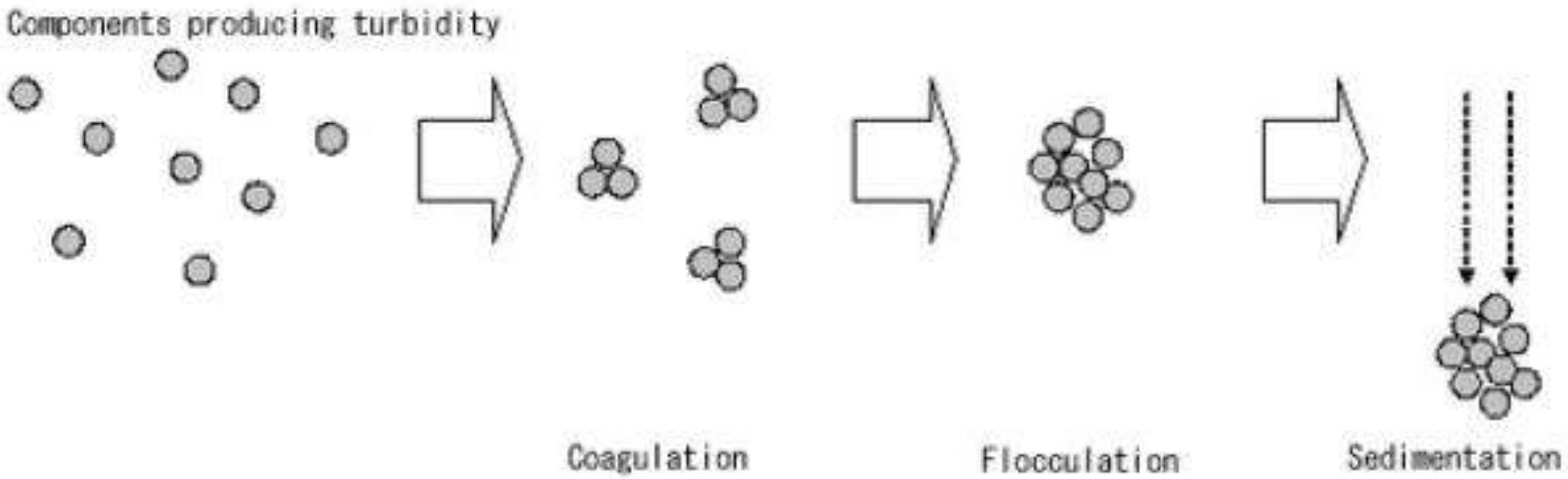
$$= 0.46 + 0.22$$

$$= 0.68$$

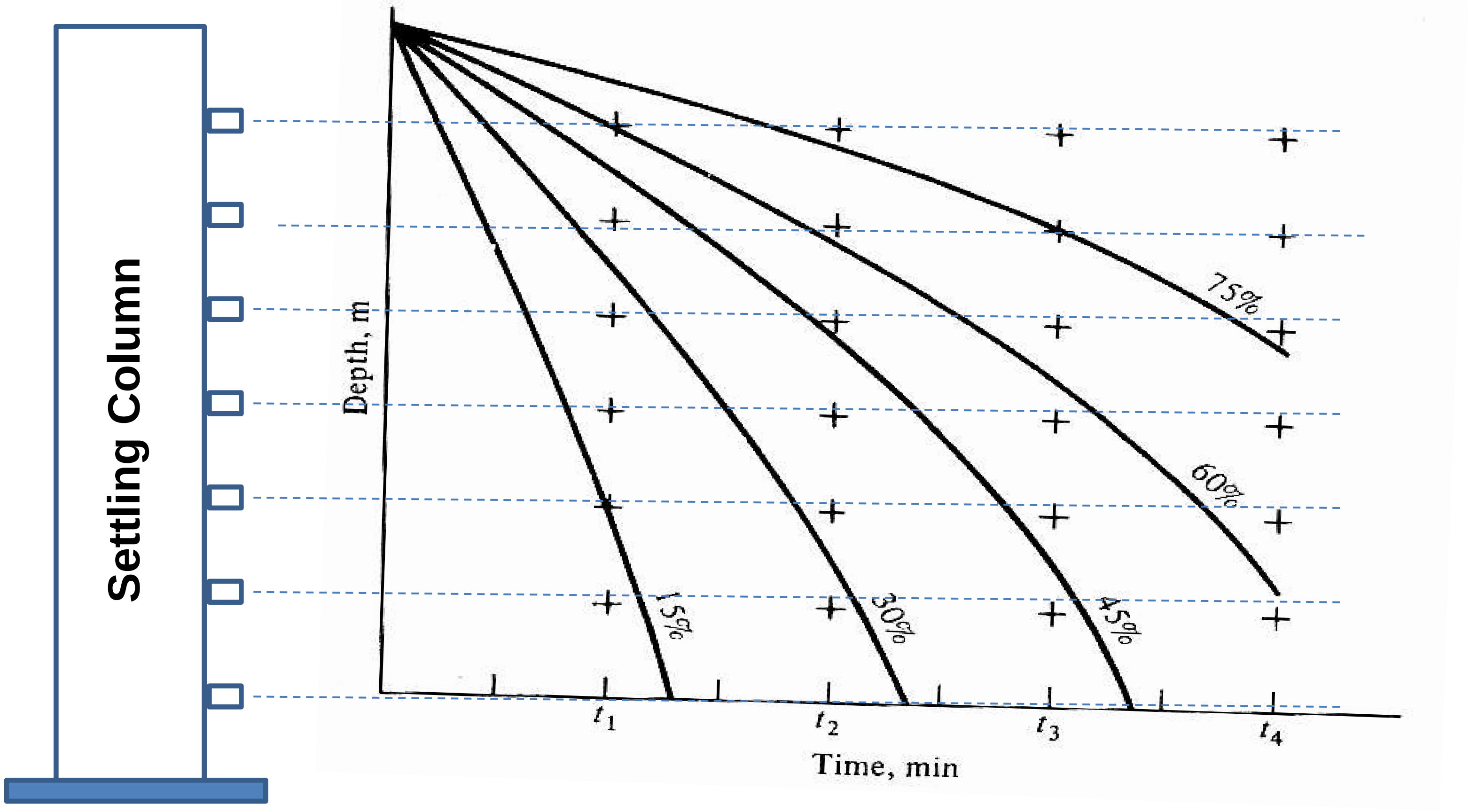
0.46+0.23 = 0.69 = 69%

Therefore, Overall removal efficiency = 68%

Type-2 Settling


Components producing turbidity
Coagulation
Flocculation
Sedimentation

Iso - removal lines from settling analysis



$$x_{ij} = \left(1 - \frac{C_{ij}}{C_0}\right) \times 100$$

Jar Test Apparatus

Sedimentation Tanks

- Plain Sedimentation
- Sedimentation aided with Coagulation

Long rectangular tanks with horizontal flow (or) Circular tanks with radial flow

<u>Long Rectangular Tanks</u>

☐ Commonly used in water treatment for processing large flows

☐ Hydraulically more stable

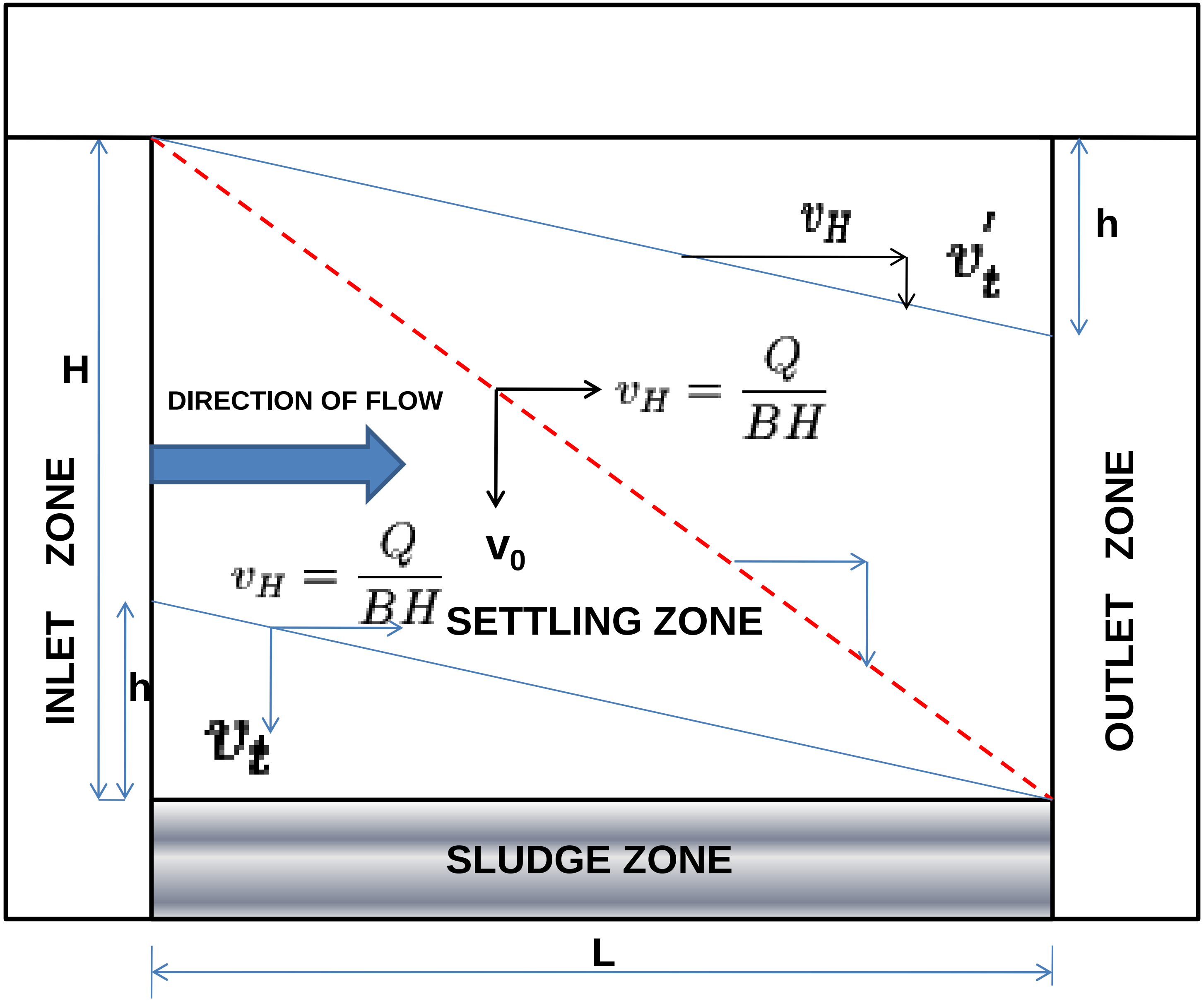
☐ flow control through large volumes is easier with this configuration

Typical designs – Length ~ 2 to 4 times Width and 10 – 20 times depth

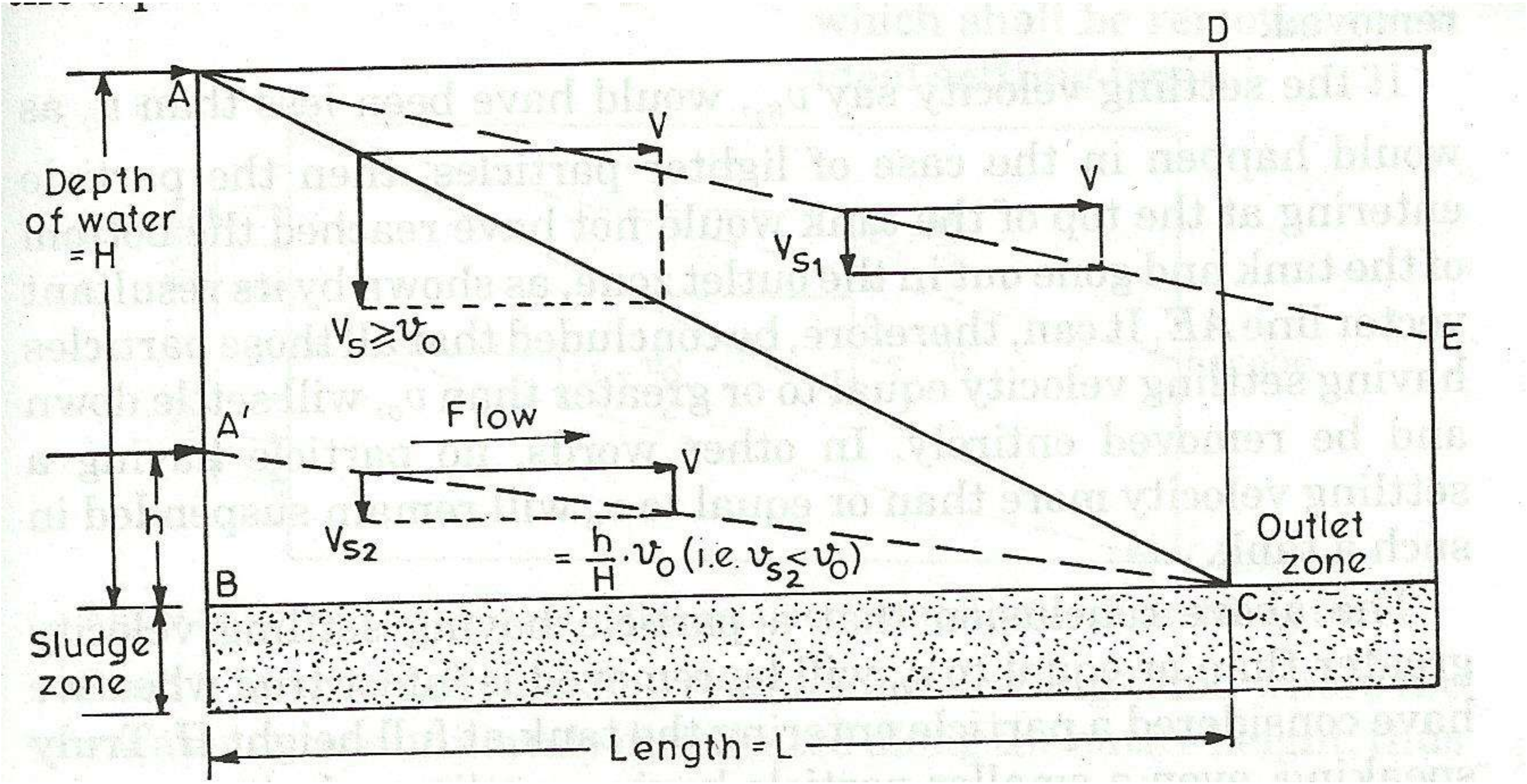
Bottom is slightly sloped to facilitate sludge scraping. A slow moving sludge scraper, continuously pulls the settled material into a sludge hopper where it is pumped out periodically.

Four Functional Zones in a Rectangular tank

- Inlet Zone – Baffles intercept the incoming water and spread the flow uniformly both horizontally and vertically across the tank
- Outlet Zone – water flows upward and over the outlet weir
- Sludge Zone – which extends from the bottom of the tank to just above the scraper mechanism
- Settling Zone – which occupies the remaining volume of the tank



v_H
v_t'
h
H
DIRECTION OF FLOW
$v_H = \frac{Q}{BH}$
INLET ZONE
OUTLET ZONE
$v_H = \frac{Q}{BH}$
v_0
SETTLING ZONE
h
v_t
SLUDGE ZONE
L


D
A
V
Depth
of water
= H
V
V_{S_1}
$V_S \geqslant v_O$
E
A′
Flow
V
h
V_{S2}
$= \frac{h}{H} \cdot v_O$ (i.e. $v_{S_2} < v_O$)
Outlet
zone
B
C
Sludge
zone
Length = L

$$Horizontal\ Velocity\ v_H = \frac{Q}{H \times B}$$

$$\text{where } H = Depth\ of\ the\ tank\ and\ B = Width\ of\ the\ tank$$

$$Time\ t_0\ corresponds\ to\ the\ retention\ time\ in\ the\ settling\ zone$$

$$t_0 = \frac{V}{Q} = \frac{L \times H \times B}{Q}$$

$$Also\ t_0 = \frac{H}{v_t}$$

$$Therefore\ \frac{H}{v_t} = \frac{L \times H \times\ B}{Q}$$

$$v_t = \frac{Q}{L \times B} = \frac{Q}{A_s}$$

$$where\ A_s = Surface\ Area\ of\ the\ settling\ tank$$

Assumptions in the design of horizontal sedimentation tank:

- Particles within the settling or sedimentation zone, settle exactly in the same manner as they do in a quiescent zone
- Flow is horizontal and steady and velocity is uniform in all parts of settling zone for a time equal to the detention period
- Concentration of suspended particles of each size is same at all points of the vertical cross-section at the inlet end
- A particle is removed when it reaches the bottom of the settling zone

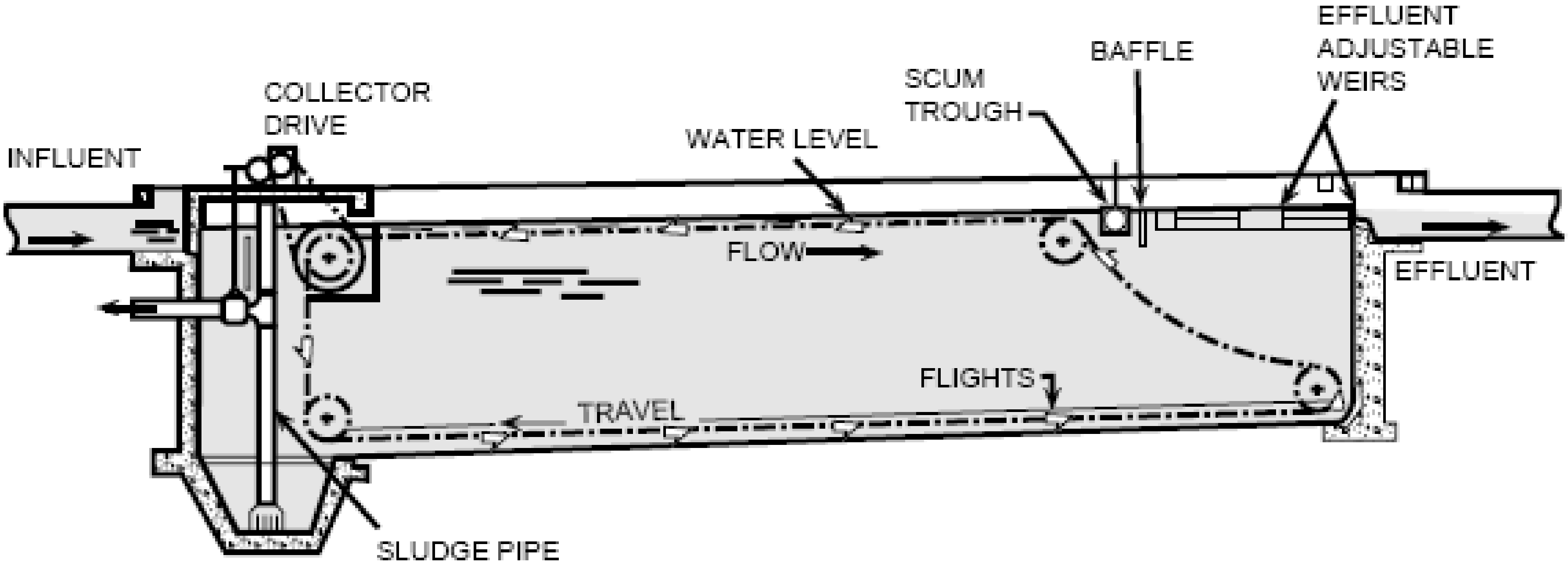


FIG. 7.17.2 Parts of a rectangular basin. (Courtesy of the FMC Corp., Material Handling Systems Division)

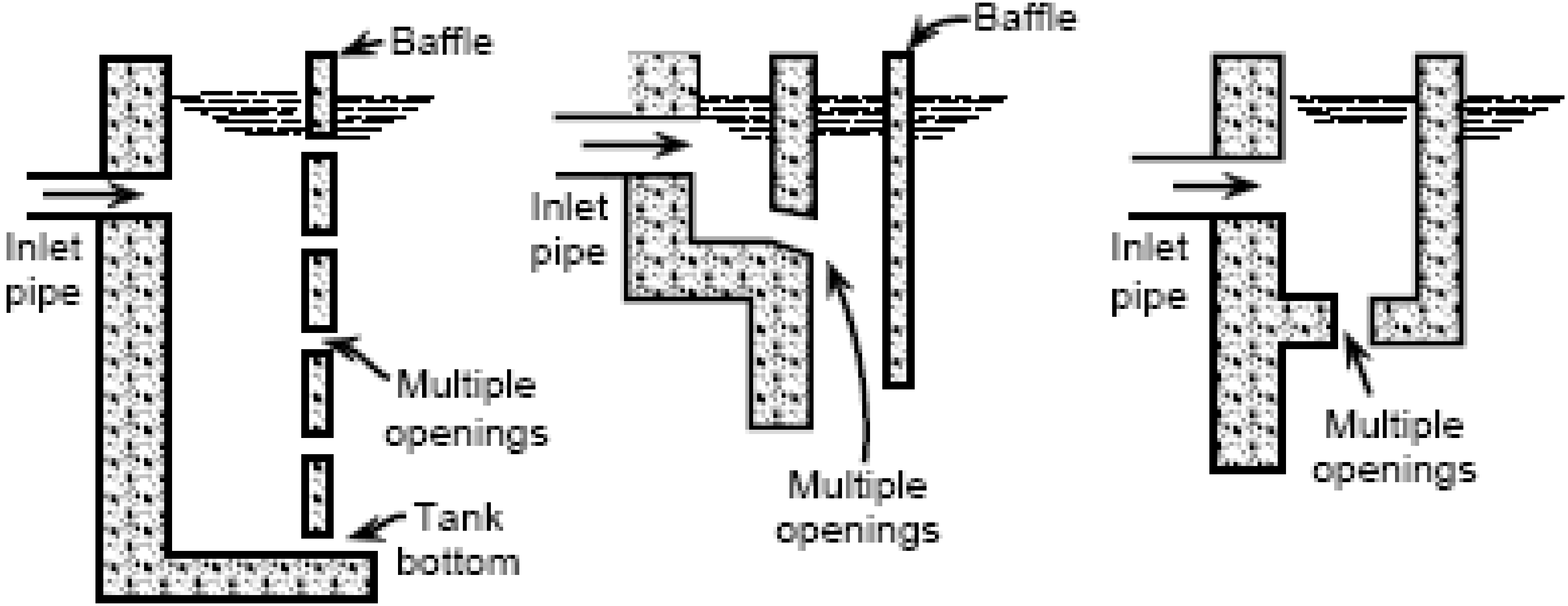


FIG. 7.17.7 Typical sedimentation tank inlets.

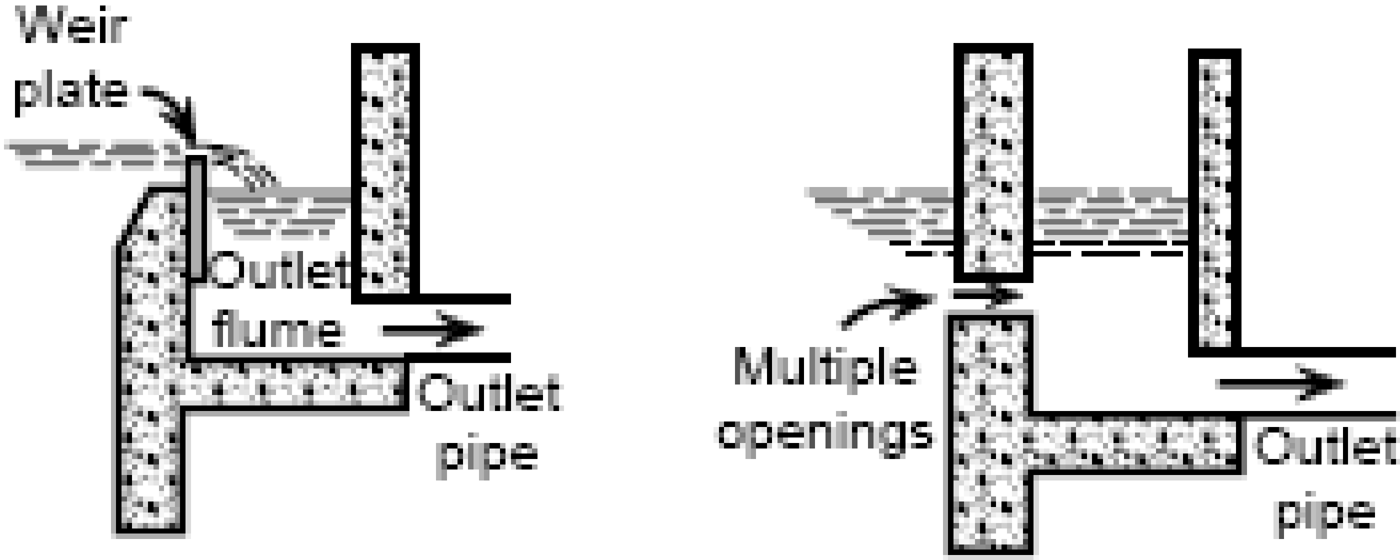


FIG. 7.17.9 Outlet details of sedimentation tanks.

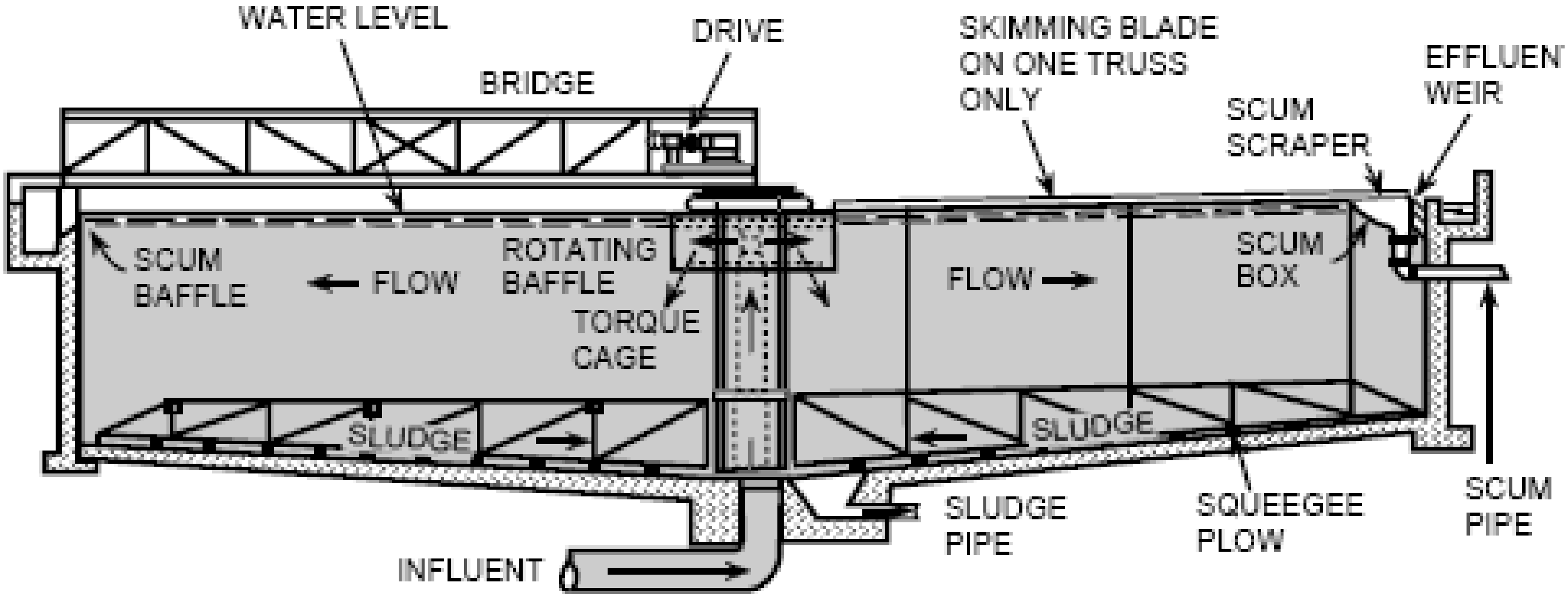
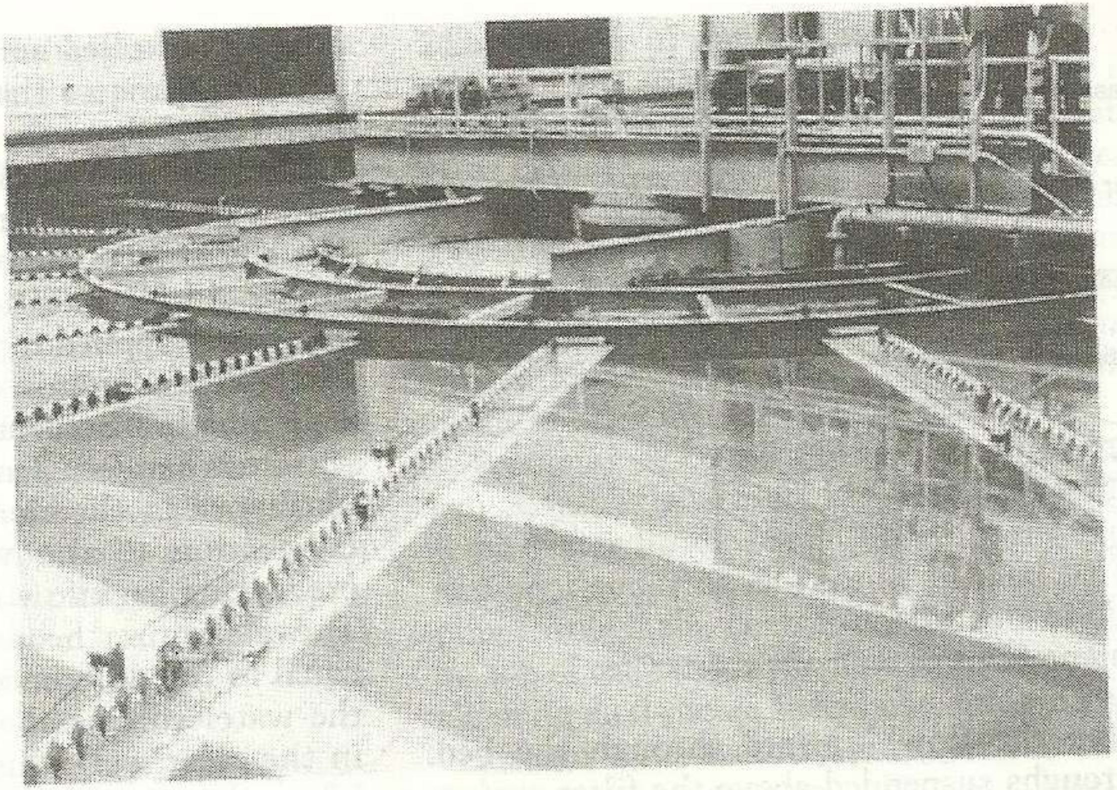
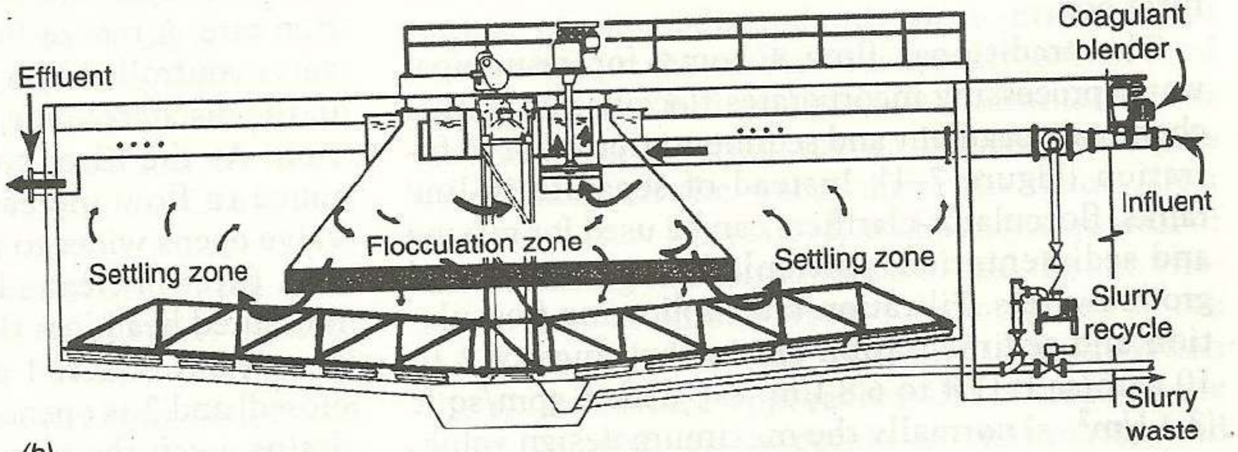


FIG. 7.17.3 Parts of a circular basin. (Courtesy of the FMC Corp., Material Handling Systems Division)



(a)


Coagulant blender
Effluent
Influent
Flocculation zone
Settling zone
Settling zone
Slurry recycle
Slurry waste

(b)

TABLE 7.17.2 DIMENSIONS OF RECTANGULAR AND CIRCULAR BASINS

Clarifier	*Range*	*Typical*
Rectangular		
Length, m	10–100	25–60
Length-to-width ratio	1.0–7.5	4
Length-to-depth ratio	4.2–25.0	7–18
Sidewater depth, m	2.5–5.0	3.5
Width, m[a]	3–24	6–10
Bottom slope, %	1	1
Circular		
Diameter, m[b]	3–60	10–40
Side depth, m	3–6	4
Bottom slope, %	8	8

[a] Most manufacturers build equipment in width increments of 61 cm (2 ft). If the width is greater than 6 m (20 ft), multiple bays may be necessary.

[b] Most manufacturers build equipment in 1.5-m (5-ft) increments of diameter.

TABLE 7.17.1 TYPICAL WATER TREATMENT CLARIFIER DESIGN DETAILS

		Weir Overflow Rate		*Surface Overflow Rate*	
Type of Basin	*Detention Time, hr*	*$m^3/(m \cdot day)$*	*$gal/(ft \cdot day)$*	*m/day*	*$gal/(ft^2 \cdot day)$*
Presedimentation	3–8				
Standard basin following:					
Coagulation and flocculation	2–8	250	20,000	20–33	500–800
Softening	4–8	250	20,000	20–40	500–1000
Upflow clarifier following:					
Coagulation and flocculation	2	175	14,000	55	1400
Softening	1	350	28,000	100	2500
Tube settler following:					
Coagulation and flocculation	0.2				
Softening	0.2				

Sedimentation aided with Coagulation

- Small, less dense particulate matter may be removed by sedimentation only with extended detention times not available in many situations. Colloidal materials form very stable water suspensions. Colloidal particles may consist of clay and silt, color bodies, precipitated iron or manganese oxides, and bacteria and algae.

Diameter of Particle	Type of Particle	Settling time through 1 m. of water
10mm	Gravel	1 seconds
1mm	Sand	10 seconds
0.1mm	Fine Sand	2 minutes
10 micron	Protozoa, Algae, Clay	2 hours
1 micron	Bacteria, Algae	8 days
0.1 micron	Viruses, Colloids	2 years
10 nm	Viruses, Colloids	20 years
1 nm	Viruses, Colloids	200 years

Settling Time for Particles of Various Diameters;

- Coagulation and flocculation are the mechanisms by which particulate and colloidal materials are removed from water in the process of clarification.
- Coagulation can be defined as charge neutralization which results in the destabilization of suspensions of particles in the colloidal size range (1-500 milli microns) allowing agglomeration to occur. Colloidal particles have a negative electrical charge. This net negative charge results in stable suspensions due to the repulsive forces between each particle. As the halo of net negative charge surrounding each particle is neutralized, it is stripped away reducing the effective particle diameter. One may easily visualize the particle agglomeration that may occur through natural particle collision, helped by slow mixing, once the charge is neutralized and the effective diameter of the particles is reduced.

- In most waters requiring charge neutralization, it is a net negative charge that must be neutralized. Therefore, the term coagulant is applied only to cationic chemicals.

- The extent of particle agglomeration due to coagulation cannot be predicted. In some circumstances very small numbers of particles may agglomerate to form microfloc. The coagulated material may or may not be agglomerated enough to achieve good settling without flocculation. A high density macrofloc may not need further treatment, while microfloc and low density macrofloc will likely require flocculation for optimum clarification.

- The extent of particle agglomeration due to coagulation cannot be predicted. In some circumstances very small numbers of particles may agglomerate to form microfloc. The coagulated material may or may not be agglomerated enough to achieve good settling without flocculation. A high density macrofloc may not need further treatment, while microfloc and low density macrofloc will likely require flocculation for optimum clarification.

- Flocculation can be defined as the mechanism by which microfloc or low density macrofloc particles are further agglomerated resulting in rapid settling floc bodies and enhanced finished water quality.
- Inorganic coagulants have been used to clarify water for years. Trivalent ions such as aluminum and ferric iron coagulate colloidal suspensions by charge neutralization and by promoting agglomeration. Therefore, in addition to their coagulating ability, they are also capable of further flocculation through their ability to form hydrated gelatinous hydroxides, at appropriate pH levels. These gelatinous hydroxides entrap destabilized particles as they sweep through the water under the force of gravity.

- Polymeric coagulants, or that class of polyelectrolyte with relatively low molecular weight (compared to flocculant polymers) and a high cationic charge density, are finding wide application as coagulants in water clarification. Polymeric coagulants also have some ability to cause flocculation through a mechanism quite dissimilar to that of the inorganic coagulants. Flocculation by polymer coagulants is brought about via their molecular weight (M.W.) which is very high as compared to inorganic coagulants.

- Molecular weight, for the purposes of this discussion, may be pictured as representing polymer chain length. The greater the M.W., the greater the chain length and the greater the flocculating ability. Longer chain length allows bridging, or attaching to, greater numbers of particles. However, coagulant polymers have relatively low molecular weights and flocculating ability when compared to the high molecular weight cationic, nonionic and anionic flocculant polymers. Therefore, polymeric coagulants have the ability to enhance flocculation only to a limited degree.

Chemicals used for Coagulation

Sl.No	Coagulant	Chemical Formula	Molecular Wt.
1.	Filter Alum (or) Alum	$Al_2(SO_4)_3.18H_2O$	666.42
2.	<u>Iron Salts</u> (i) Ferrous Sulphate (Copperas)	$FeSO_4.7H_2O$	278.0
	(ii) Ferric Chloride	$FeCl_3$	162.2
	(iii) Ferric Sulphate	$Fe_2SO_{4(3)}$	400.0
3	Chlorinated Copperas	$Fe_2SO_{4(3)}.FeCl_3$	562.08
4.	Sodium Aluminate	$Na_2Al_2O_4$	164.0

Alum

$$Al_2(SO_4)_3.18H_2O + 3Ca(HCO_3)_2 \rightarrow 3CaSO_4 + 2Al(OH)_3\downarrow + 6CO_2\uparrow$$

Bicarbonate alkalinity (pH 4.5 – 8.3)

Imparts permanent hardness

Causes corrosiveness

(i) $Al_2(SO_4)_3.18H_2O + 3Ca(OH)_2 \rightarrow 3CaSO_4 + Al(OH)_3\downarrow + 18H_2O$

(ii) $Al_2(SO_4)_3.18H_2O + 3Na_2CO_3 \rightarrow 3Na_2SO_4 + 2Al(OH)_3\downarrow + 3CO_2\uparrow + 15H_2O$

Ferrous Sulphate (Copperas)

When lime is added to water in conjunction with Copperas

(i) $FeSO_4.7H_2O + Ca(OH)_2 \rightarrow CaSO_4 + Fe(OH)_2 + 7H_2O$

Copperas

Hydrated Lime

Ferrous Hydroxide

When Copperas is added earlier to lime

(i) $FeSO4.7H_2O + 3Ca(HCO_3)_2 \rightarrow Fe(HCO_3)_2 + CaCO_4 + 7H_2O$

Alkali present in raw water

(ii) $Fe(HCO_3)_2 + 2Ca(OH)_2 \rightarrow Fe(OH)_2 + 2CaCO_3 + 2H_2O$

Hydrated Lime

Ferrous Hydroxide

(iii) $4Fe(OH)_2 + O_2 + 2H_2O \rightarrow \boxed{4Fe(OH)_3\downarrow}$

Floc

Ferric Hydroxide Precipitate

Chlorinated Copperas

$6(FeSO_4.7H_2O) + 3Cl_2 \rightarrow 2Fe_2(SO4)_3 + 2FeCl_3 + 42H_2O$

Ferric Sulphate Ferric Chloride

$Fe_2(SO_4)_3 + 3Ca(OH)_2 \rightarrow 3CaSO_4 + 2Fe(OH)_3 \downarrow$

Ferric Sulphate Hydrated Lime Ferric Hydroxide
Precipitate

$2FeCl_3 + 3Ca(OH)_2 \rightarrow 3CaCl_2 + 2Fe(OH)_3 \downarrow$

Ferric Chloride Hydrated Lime Ferric Hydroxide
Precipitate

Sodium Aluminate

$Na_2Al_2O_4 + Ca(HCO_3)_2 \rightarrow \underset{\text{Calcium Aluminate}}{CaAl_2O_4\downarrow} + Na_2CO_3 + CO_2\uparrow + H_2O$

$Na_2Al_2O_4 + CaCl_2 \rightarrow CaAl_2O_4\downarrow + 2NaCl$

$Na_2Al_2O_4 + CaSO_4 \rightarrow CaAl2O_4\downarrow + Na_2SO_4$

Comparison of Iron and alum Salts

- Iron salts
  - produce heavy floc – therefore can remove more suspended matter than alum
  - can remove H2S and its corresponding tastes and odours from water
  - used over a wider range of pH values
  - cause staining and promote the growth of iron bacteria in the distribution system
  - impart more corrosiveness than alum
  - Handling and storing of iron salts require more skill and control (iron salts are more corrosive and deliquescent than alum)

Clariflocculator (Coagulation-Flocculation-Sedimentation Basin)

- Consists of four units
  - Feeding Device
    - Dry Feeding Devices
    - Wet Feeding Devices

Depending upon the feeding of chemical coagulant either in powdered form or in a solution form

  - Mixing Device or Mixing Basin
    - Mixing Basins with Baffle Walls
    - Mixing Basins equipped with mechanical devices
  - Flocculation Tank
  - Settling or Sedimentation Tank

Turbidities as low as 10 – 20mg/L and 70% B-Coli index can be removed in Clariflocculator

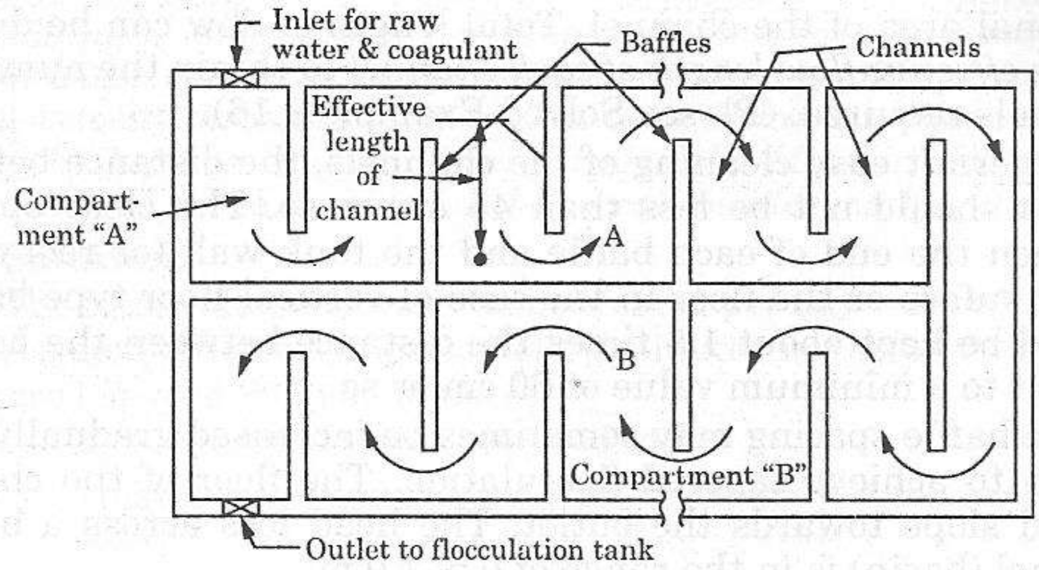


Fig. 9.20. Plan of "Around the end baffle type" (*i.e.* Horizontal flow type) mixing basin.

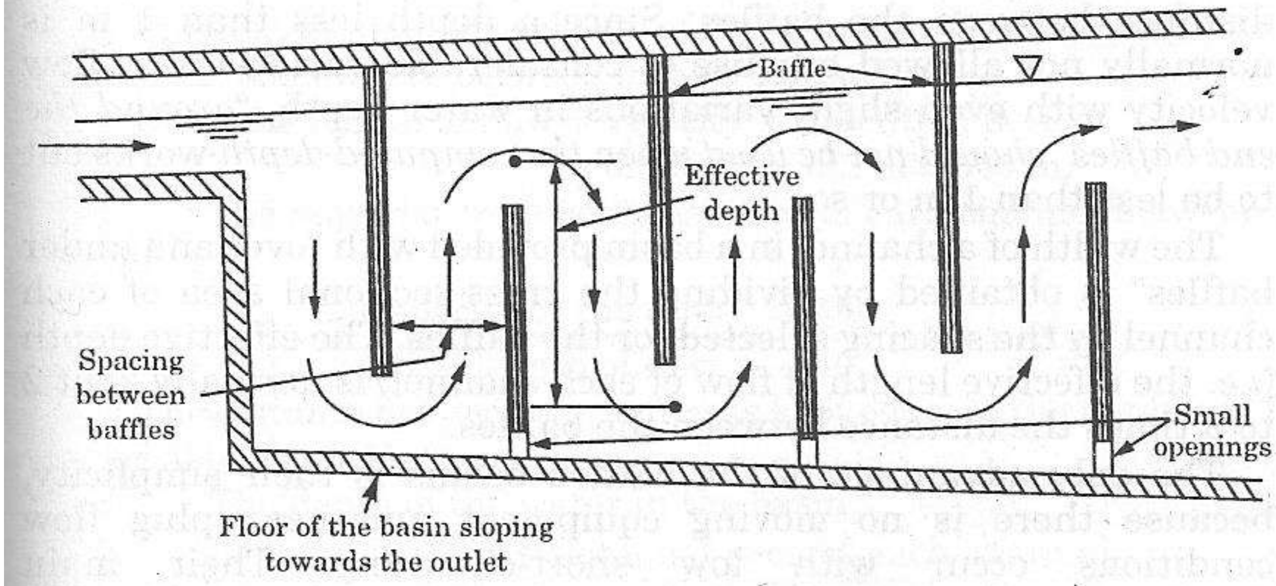
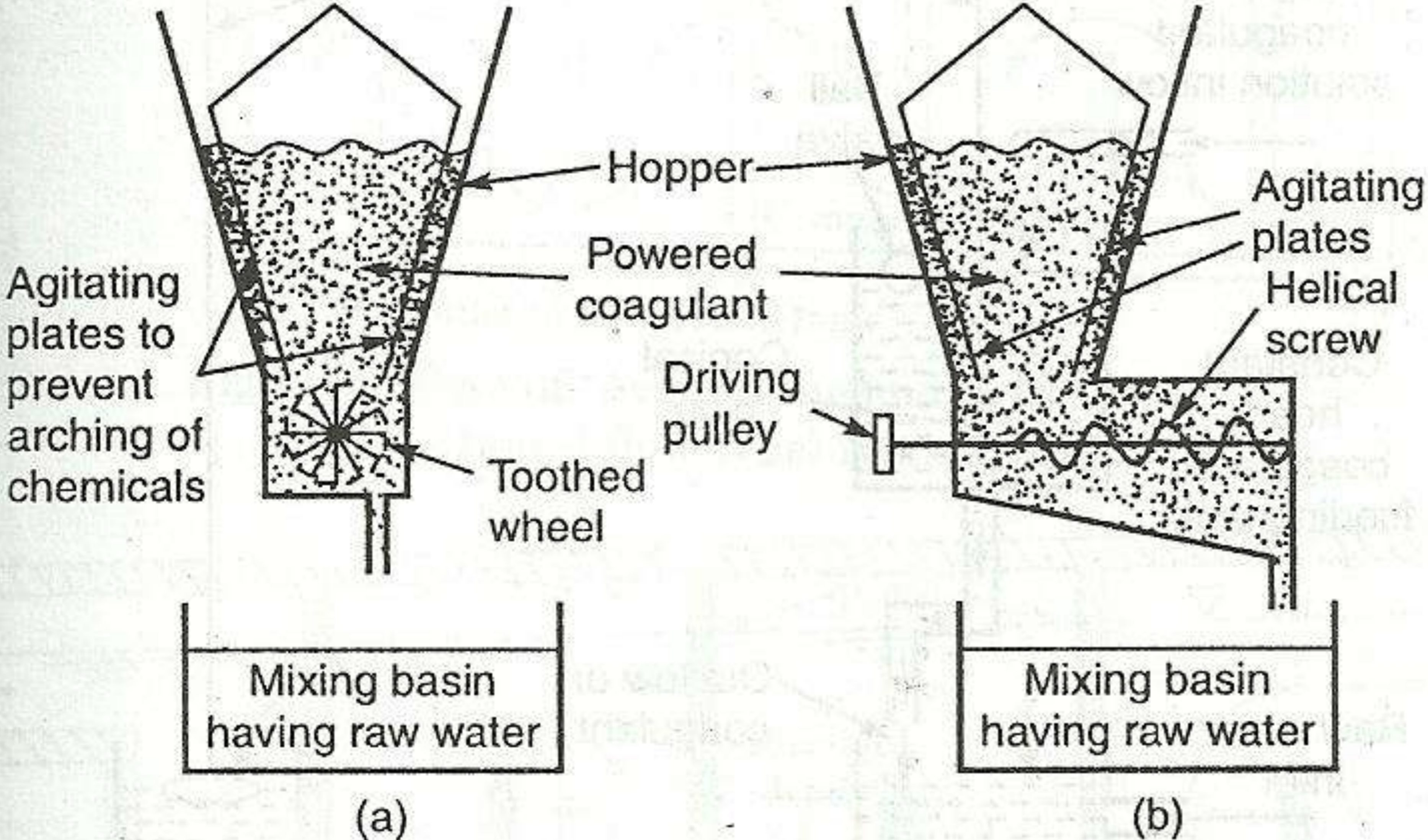


Fig. 9.21. Sectional elevation of "Over and under the baffle type" (vertical flow type) mixing basin.

Dry Feeders


Hopper
Agitating plates
Powered coagulant
Agitating plates to prevent arching of chemicals
Helical screw
Driving pulley
Toothed wheel
Mixing basin having raw water
Mixing basin having raw water
(a)
(b)

Example 1:

Determine the quantity of alum required in order to treat 13 million litres of water per day at a treatment plant, where 12 ppm of alum dose is required. Also determine the amount of carbon dioxide gas which will be released per liter of water treated.

Example 2:

8 mg/L of Copperas is consumed with lime at a coagulation basin. Determine the quantity of copperas and the quick lime required to treat 10 million litres of water.

Example 3:

Raw water with suspended solids concentration of 37mg/L has to be treated in a clariflocculator. The flow into water treatment plant is $0.5 m^3/s$ and alum dosing is found to be 23mg/L. The effluent suspended solids concentration is measured as 12mg/L. The sludge content is 1% and specific gravity of sludge solids is 3.01. What volume of sludge must be disposed of each day?

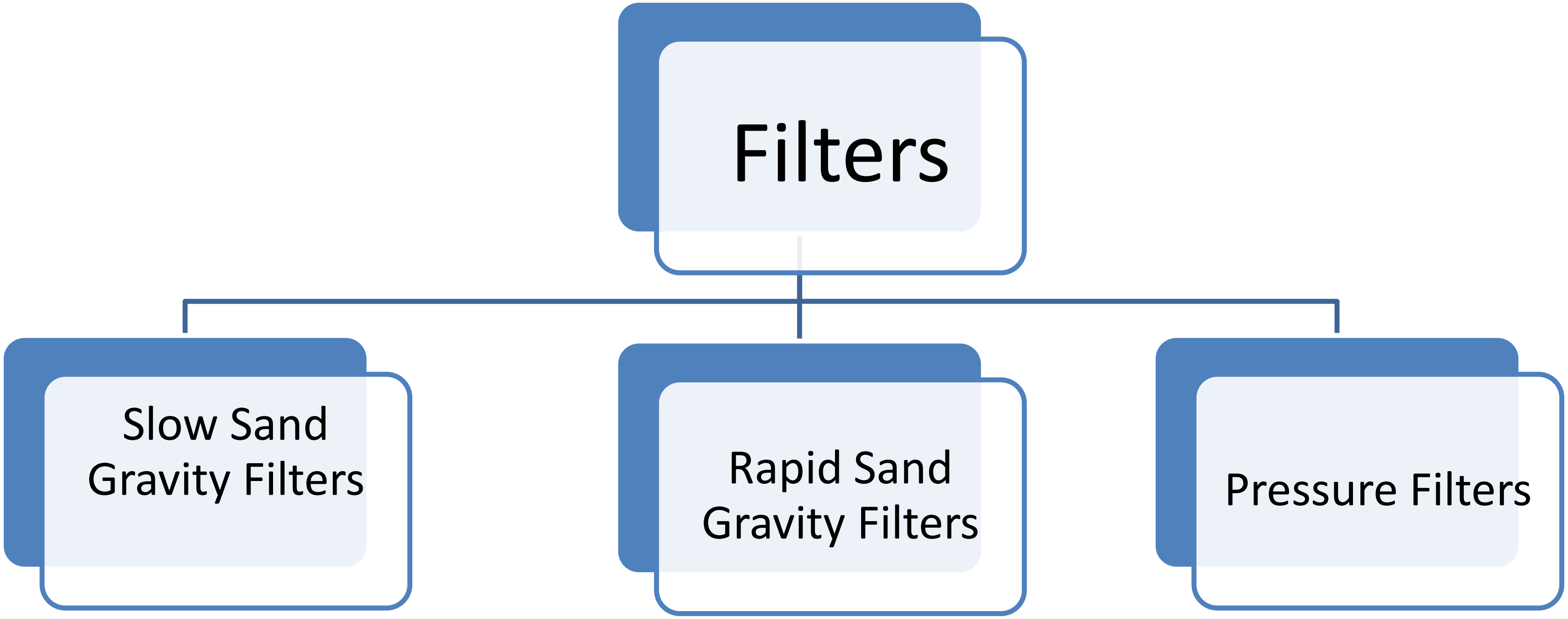
Dewatering and Disposal of Sludge from Water Treatment Plant

- By ponding in Artificial or Natural Lagoons – provided sufficient open land is available in the vicinity of the plant
- Sand Drying Beds
- Gravity Thickeners
- High Speed Centrifuges
- Pressure Filtration
- Vacuum Filtration

Dewatered sludge is finally disposed of either by incineration or by burial in trenches

Filtration - *process where water is filtered through granular material eg. sand*

- Polishing step to remove small flocs (or) precipitant particles not removed in settling basins
- Primary turbidity removal process (under certain conditions)
- Removes Colour, turbidity, odour, pathogenic organisms etc.



Filters

Slow Sand Gravity Filters

- Remove much larger %age of impurities and bacteria as compared to Rapid Sand Gravity Filters
- Very slow rate of Filtration
- require large areas and are costly
- Obsolete in present day

Rapid Sand Gravity Filters

- Universally adopted in modern water treatment plants

Pressure Filters

- Industrial Use
- Swimming Pools

Theory of Filtration

- Mechanical Straining – particles bigger than voids are arrested in sand layers. Most particles are removed in upper sand layers – arrested particles along with flocs form mat which further helps in straining
- Flocculation and Sedimentation – Assumed that void spaces act like coagulation-sedimentation tanks. Remove particles smaller than size of voids. Colloidal particles act as a gelatinous mass and therefore attract other finer particles

- Biological Metabolism – Bacteria present in voids convert organic impurities (eg. algae, plankton etc.) to harmless compounds. There harmless compounds form a layer on top called “Dirty Skin” which further helps in straining out impurities.
- Electrolytic Changes – by changing chemical characteristics of water. Sand grains of filter media and impurities in water carry electrical charges of opposite nature. When these come in contact with each other, they neutralize, thereby changing the character of water

Filter Materials

- Sand (Coarse or Fine)
  - Free from dirt and impurities
  - Uniform in nature and size
  - Hard and resistant
  - Should not loose 5% weight when kept in HCl for 24hours
- Gravel
  - May be used below sand
  - Density 1600kg/m^3
- Anthracite Coal

- Size of sand – expressed as effective size ie., D_{10} – size of sieve in mm through which 10% of sample of sand by mass will pass.
- Uniformity in size or degree of variation in sizes of particles is measured and expressed by Uniformity Coefficient (D_{60}/D_{10})

⑤

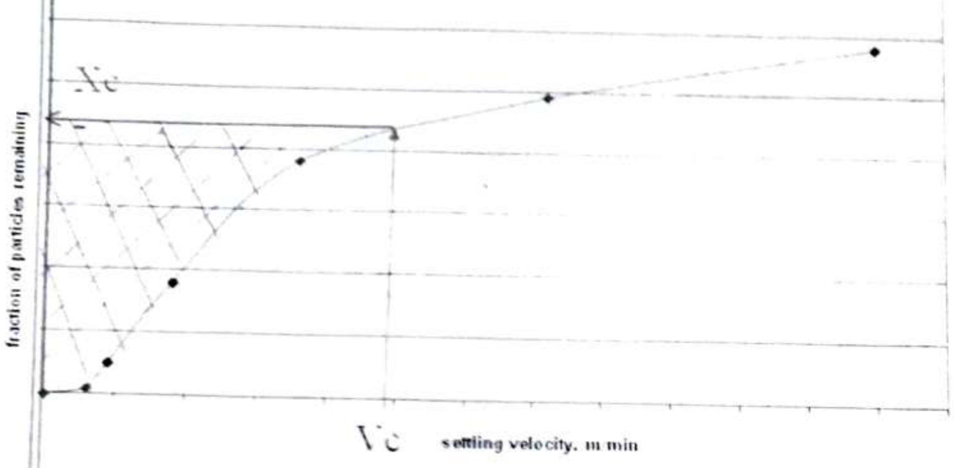
Example 3: A settling analysis is run on a Type 1 suspension in a laboratory column with a port 1.8m below the suspension surface. The data obtained are shown below.

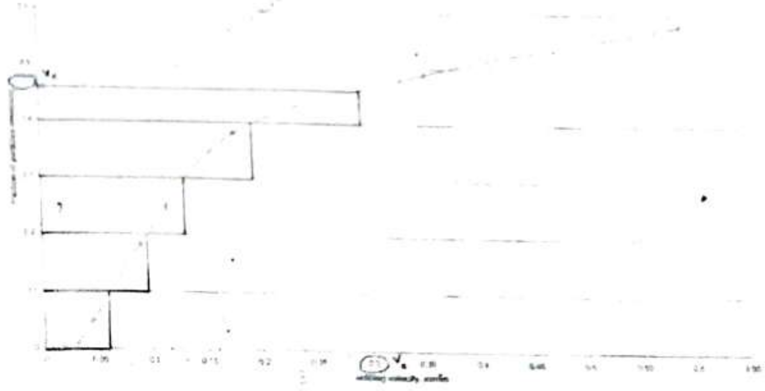
Time (min)	**TSS (conc, mg/L)**
0	220
3	116
5	98
10	75
20	35
40	10
60	2

What will be the theoretical removal efficiency in a settling basin for an overflow of 432 m^2/m.day?

Solution of Example 2 :

Time (min)	TSS (conc, mg/L)	Mass Fraction Remaning	Vs (m/min)
0	200	-	-
3	116	0.58 : 116/200	1.8/3=0.6
5	98	0.49 : 98/200=0.49	1.8/5=0.36
10	75	0.38	1.8/10=0.18
20	35	0.18	1.8/20=0.09
40	10	0.05	1.8/40=0.045
60	2	0.01	1.8/60=0.03





$$\text{Overflowrate}=V_c=432\,\frac{m^2}{m.d}\,\frac{1d}{24hr}\,\frac{1hr}{60\,min}=0.3m/min$$

$X_c=0.46$

$$\text{Total removal}=\left(1-X_c\right)+\frac{1}{V_c}\int_0^{X_c}Vdx$$

$$=0.54+\frac{1}{0.3}(0.0143+0.0158+0.01266+0.0077+0.0044)$$

$$=0.54+\frac{0.05486}{0.3}$$

$$=0.72$$

$$=72\%$$

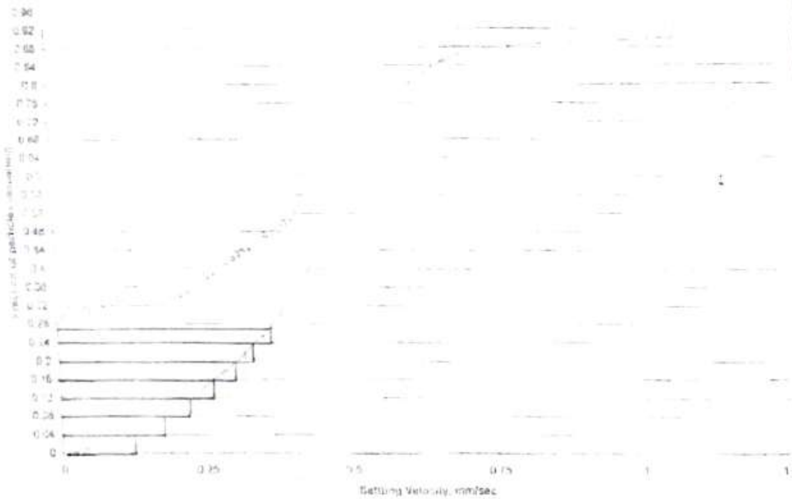
dx	V	V.dx
0.06	0.2388	0.0143
0.1	0.158	0.0158
0.1	0.1266	0.01266
0.1	0.077	0.0077
0.1	0.044	0.0044

Example 4: A settling basin is designed to have a surface overflowrate of 32.6m/d. Determine the overall removal obtained for a suspension with the size distribution given in the table below. The specific gravity of the particles is 1.2 and water temperature is 20°C .

(μ =1.0087x10^{-3} Ns/m2, ρ =998.23 kg/m3)

Particle Size , mm	Weight Fraction Greater Than Size , %
0.1	10
0.08	15
0.07	40
0.06	70
0.04	93
0.02	99
0.01	100

Solution of Example 3:



dx	V	V.dx
0.027	0.35	0.00945
0.04	0.32	0.0128
0.04	0.29	0.0116
0.04	0.25	0.01
0.04	0.20	0.008
0.04	0.15	0.006
0.04	0.065	0.0026

$X_c = 0.267$

$$\text{Total removal} = \left(1 - X_c\right) + \frac{1}{V_c}\int_0^{x_c} V\,dx$$

$$= 0.733 + \frac{1}{0.37}(0.00945 + 0.0128 + 0.0116 + 0.01 + 0.008 + 0.006 + 0.0026)$$

$$= 0.89$$

$$= 89\ \%$$

Example 5 (Type 2 Settling):

A column analysis of a flocculating suspension is run in the apparatus shown below. The initial solids concentration is 250 mg/L. The resulting matrix is shown below. What will be the overall removal efficiency of a settling basin which is 3 m deep with a detention time of 1 h and 45 min.

Depth	Time of sampling, min					
m	30	60	90	120	150	180
0.5	133	83	50	38	30	23
1	180	125	93	65	55	43
1.5	203	150	118	93	70	58
2	213	168	135	110	90	70
2.5	220	180	145	123	103	80
3	225	188	155	133	113	95

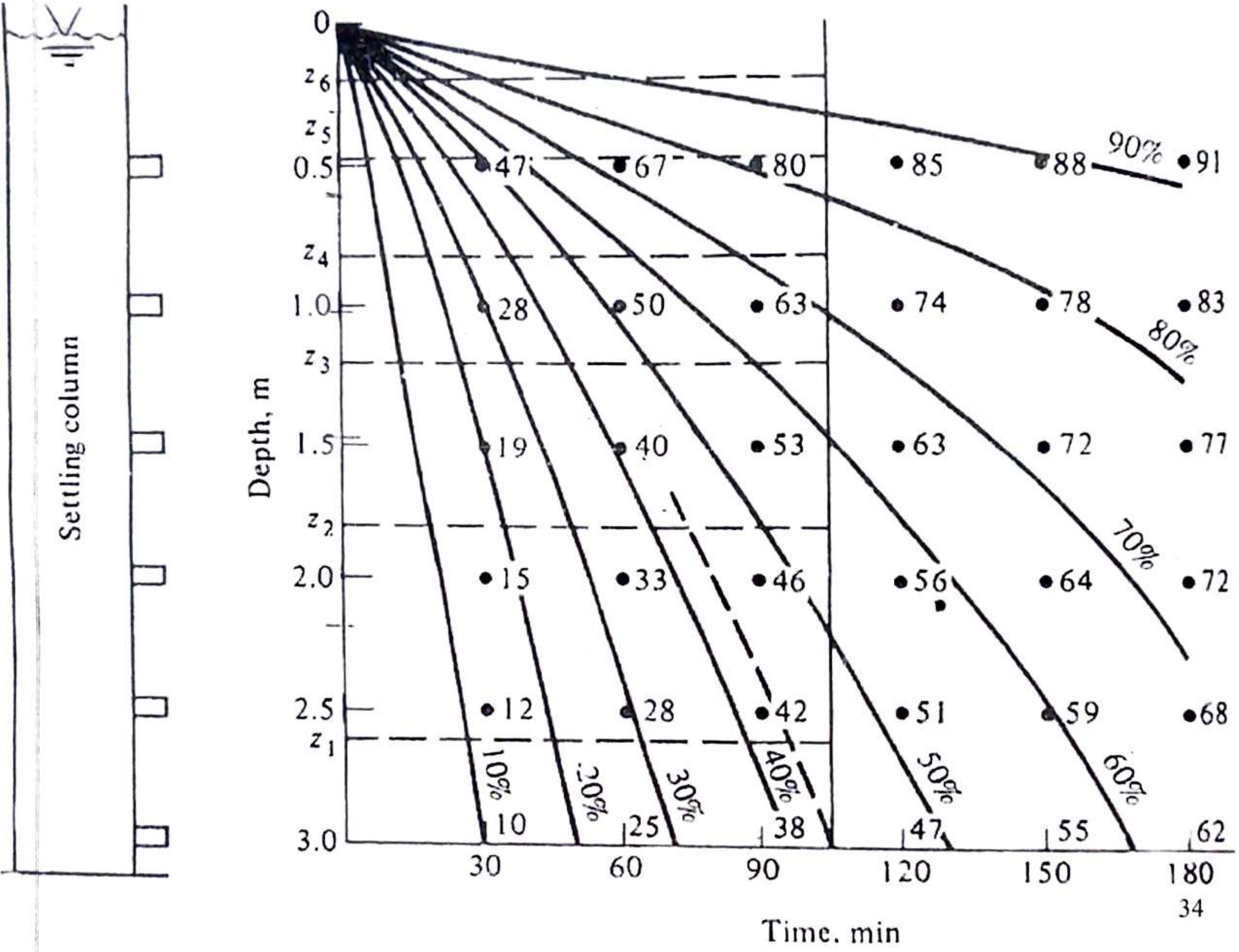
SOLUTION:

Removal at each depth and time: $X_{ij} = \left(1 - \frac{C_{ij}}{C_o}\right)100$

Depth	Time of sampling, min					
m	30	60	90	120	150	180
0.5	47*	67	80	85	88	91
1	28	50	63	74	78	83
1.5	19	40	53	63	72	77
2	15	33	46	56	64	72
2.5	12	28	42	51	59	68
3	10	25	38	47	55	62

$= \left(1 - \frac{133}{250}\right) = 0.47$

→ Plot isoremoval lines by interpolation


Settling column
Depth, m
0
z_6
z_5
0.5
z_4
1.0
z_3
1.5
z_2
2.0
2.5
z_1
3.0
47
67
80
85
88
90%
91
28
50
63
74
78
80%
83
19
40
53
63
72
77
70%
15
33
46
56
64
72
12
28
42
51
59
68
10%
20%
30%
40%
50%
60%
10
25
38
47
55
62
30
60
90
120
150
180
34
Time, min

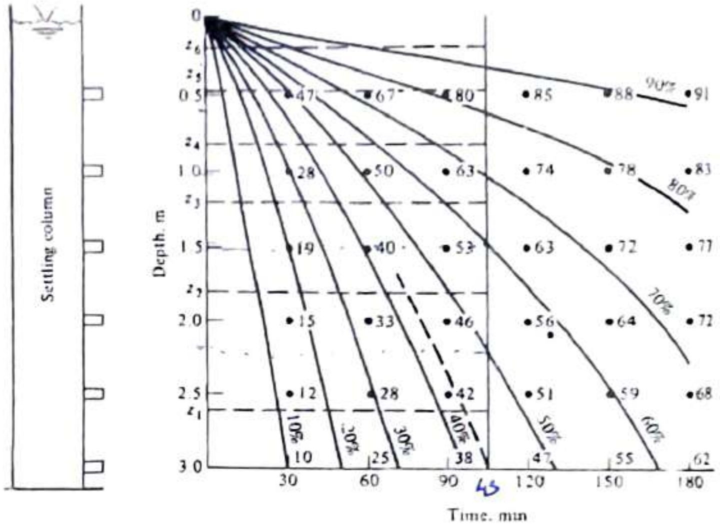
etention time = 1hr 45 min =105min

raw a vertical line from 105 min

removal of completely removed
action at t=105min =43%

rom graph, by interpolation)

removal of partially removed fraction :



v 43-50% → fraction 7%, ave. depth reached by fraction 2.6 $\%\,\text{removal} = \frac{2.6}{3} 7\% = 6.06\%$

v 50-60% → fraction 10%, ave. depth reached by fraction 1.8 $\%\,\text{removal} = \frac{1.8}{3} 10\% = 6\%$

v 60-70% → fraction 10%, ave. depth reached by fraction 1.2 $\%\,\text{removal} = \frac{1.8}{3} 10\% = 6\%$

v 70-80% → fraction 10%, ave. depth reached by fraction 0.8 $\%\,\text{removal} = \frac{0.8}{3} 10\% = 2.66\%$

v 80-90% → fraction 10%, ave. depth reached by fraction 0.45 $\%\,\text{removal} = \frac{0.45}{3} 10\% = 1.5\%$

v 90-100% → fraction 10%, ave. depth reached by fraction 0.15 $\%\,\text{removal} = \frac{0.15}{3} 10\% = 0.5\%$

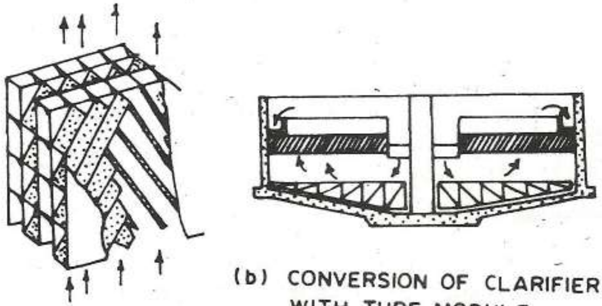
%removal of partially removed particles at time=105 min=6.06 + 6 + 4 + 2.66 + 1.5 + 0.5
=20.72%

tal removal at time=105min = completely removed % + partially removed %
= 43 % + 20.72 %
= 63.72 %

OTE : In applying isoremoval curves to design a tank, scale – up factors of

0.65 → the overflowrate
1.75 → for the detention time

e used to compensate for the side wall effects of the batch settling column.



(a) TUBE MODULE CONSTRUCTION

(b) CONVERSION OF CLARIFIER WITH TUBE MODULE

FIG. 9.29. THE TUBE SETTLER

9.20. ILLUSTRATIVE EXAMPLES

Example 9.9. *Find the dimensions of a rectangular sedimentation basin for the following data :*

(*i*) *Volume of water to be treated = 3 million litres per day.*

(*ii*) *Detention period = 4 hours.*

(*iii*) *Velocity of flow = 10 cm/min.*

Solution :

Detention time = 4 hours = 240 min.

Velocity of flow = 10 cm/min.

$\therefore$ Length of tank = $0.10 \times 240 = 24$ m.

Volume of water in 4 hours

$$= \frac{3 \times 10^6}{10^3} \times \frac{4}{24} = 500 \text{ m}^3$$

$\therefore$ Cross-section area

$$A = \frac{V}{L} = \frac{500}{24} = 20.8 \text{ m}^2$$

Assume a working depth of 3 m.

$$\therefore \quad \text{Width of tank} = \frac{20.8}{3} \approx 7 \text{ m.}$$

Provide an extra depth of 1 m for sludge storage and 0.5 m for free board making a total depth = $3 + 1.5 = 4.5$ m.

Hence provide a settling tank of size 24 m × 7 m × 4.5 m.

Check :

Volume of water per hour

$$= \frac{3 \times 10^6}{24}$$

∴ Surface loading rate

$$= \left(\frac{3 \times 10^6}{24}\right) \times \frac{1}{24 \times 7}$$

$$= 744 \text{ litres/hour/m}^2 \text{ which is satisfactory.}$$

Example 9.10. *Find the dimensions of a circular sedimentation basin for the above data.*

Solution :

A circular sedimentation tank is generally provided with its bottom cone-shaped, with a slope of 1 vertical to 12 horizontal. Under this condition, its volume V, in terms of its diameter D and height H, is given by

$$V = D^2 (0.011\, D + 0.785\, H) \qquad ...(9.36)$$

Now volume of water during detention period

$$= \frac{3 \times 10^6}{10^3} \times \frac{4}{24} = 500 \text{ m}^3$$

Let the effective depth = 3 m.

Substituting the values in the above expression, we get

$$500 = D^2 (0.011\, D + 0.785 \times 3)$$

Solving this by trial, we get

$$D \approx 14.2 \text{ m.}$$

Providing an extra depth of 1.5 m for sludge storage and free board, the size of the tank is 14.2 m dia × 4.5 m depth.

Example 9.11. *Design a plain sedimentation basin to treat 3 million of water litres per day so as to settle at least 75% of the particles of grain size 0.002 cm or more. Assume any suitable data required.*

Solution:

Velocity of settlement of 0.002 cm (or 0.02 mm) diameter at 26°C can be determined from Eq. 9.9 (a) by assuming an average specific gravity of 2.65.

$$v_s = 418\,(2.65 - 1)\,(0.02)^2 \left(\frac{3 \times 26 + 70}{100}\right)$$

$$= 0.408 \text{ mm/sec} = 0.0408 \text{ cm/sec.}$$

From Eq. 9.11 (e), 1 cm per sec. settling velocity corresponds to a surface loading of 8,64,000 litres per day/m^2.

∴ Associated surface loading

$$= 86400 \times 0.0408 = 35278 \text{ litres/day/m}^2.$$

Assume the depth of tank as 3 m. The time required to settle with the above velocity of 0.0408 cm/sec will be

$$= \frac{300}{0.0408 \times 60} = 122.5 \text{ min.}$$

As per past experience, for 75% removal of the particles of the designed category (*i.e.* 0.002 cm), the surface loading should be reduced by an average of 52%, and the detention time increased by 120%.

$\therefore$ Design surface loading

$$= 35278 \times 0.52 = 18345 \text{ litres/day/m}^2$$

Design detention period

$$= 122.5\left(1 + \frac{120}{100}\right) \approx 270 \text{ min.}$$

$$= 4.5 \text{ hours}$$

$$\text{Surface area} = \frac{3 \times 10^6}{18345} = 163.5 \text{ m}^2$$

Volume of water in 4.5 hours

$$= \frac{3 \times 10^6}{10^3} \times \frac{4.5}{24} = 562.5 \text{ m}^3$$

Let $L = 10 \times 3 = 30$ m.

$\therefore$ $B = \dfrac{562.5}{30 \times 3} = 6.25$ m

Displacement velocity of flow

$$= \frac{30}{270} = 0.111 \text{ m/min.,} = 11.1 \text{ cm/min.}$$

which is satisfactory.

Provide an extra depth of 1 m for sludge storage and 0.5 m for free board, making a total depth

$$= 3 + 1.5 = 4.5 \text{ m}$$

Hence provide a tank of 30 m × 6.25 m × 4.5 m size.

Example 9.12. *Design a coagulation sedimentation tank to treat 10 million litres of water per day. Assume suitable data where necessary.*

Solution :

Alum and iron flocs adsorb and entrain water and reduce the specific gravity to as little as 1.002. However, these flocs may be as large as 0.1 cm in diameter, and their settling velocity at 26°C may be assumed to be about 0.12 cm per sec.

Since 1 cm per sec. settling velocity corresponds to a surface loading of 8,64,000 litres per day/ m², the associated surface loading

$$= 864000 \times 0.12$$

$$= 103680 \text{ litres/day/m}^2.$$

Reducing this by about 52%,
Design surface loading

$$= 103680 \times 0.52 \approx 54000 \text{ litres/day/m}^2.$$

$$\therefore \text{ Surface area } = \frac{10 \times 10^6}{54000} = 185.2 \text{ m}^2.$$

Provide a tank of size 20 m× 9.3 m.

Provide an effective depth of tank= 3 m, and an overall depth = 4.5 m, making an allowance of 1 m for sludge and 0.5 m for free board.

Theoretical detention time.

$$= \frac{300}{0.12 \times 60} \approx 42 \text{ min.}$$

Total actual detention time may be

$$= 42\left(1 + \frac{120}{100}\right) \approx 92 \text{ min.}$$

Hence provide the settling basin of size 20 m × 9.3 m × 4.5 m, and give a slope of 1 in 50 to the floor of the tank.

Average velocity of flow in the tank

$$= \frac{\text{Length of tank}}{\text{Detention period}}$$

$$= \frac{20 \times 100}{92} \approx 22 \text{ cm/min.}$$

which is satisfactory.

Design of floc chamber.

The section of tank is shown in Fig. 9.30.

Effective depth of tank near floc chamber

$$= 3 + 1 - \frac{20}{50} = 3.6 \text{ m.}$$

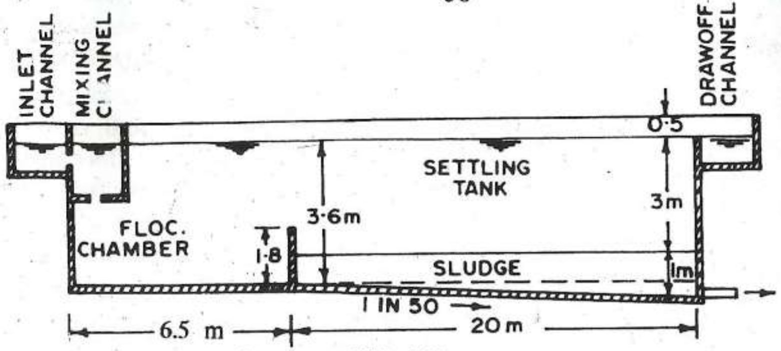


FIG. 9.30

Assume that effective depth of floc chamber is equal to half the effective depth of the settling tank near floc chamber.

∴ Effective depth of floc chamber

$$= \tfrac{1}{2} \times 3.6 = 1.8 \text{ m.}$$

Assume a detention period of 15 minutes for the floc chamber

∴ Total capacity required

$$= \frac{10 \times 10^6}{10^3} \times \frac{15}{24 \times 60} = 104.2 \text{ m}^3.$$

Width of floc chamber = Width of settling tank
= 9.3 m.

∴ Length of floc chamber

$$= \frac{104.2}{9.3 \times 1.8} = 6.22 \text{ m.}$$

Keep length of floc chamber= 6.5 m.

Example 9.13. *A settling basin is designed to have a surface overflow rate of 32.6 m/day. Determine the overall removal obtained for a suspension with size distribution given below. The specific gravity of the particles is 1.2 and the water temperature is 20°C at which dynamic visocity is 1.027 centipoise and the density is 0.997 g/cm³.*

Particle size (mm)	*0.1*	*0.08*	*0.07*	*0.06*	*0.04*	*0.02*	*0.01*
Weight fraction greater than size (per cent)	*10*	*15*	*40*	*70*	*93*	*99*	*100*

(U.P.S.C. Engineering Services 1992)

Solution :

The settling velocities may be calculated from Stoke's law (Eq. 9.6) :

$$v_s = \frac{g}{18\mu}(\rho_s - \rho)\, d^2 = \frac{9810}{18\,(1.027)}\,[1.2 - 0.997]\, d^2$$

$$= 107.73\, d^2 \qquad \text{...}(i)$$

The corresponding Reynolds number can be computed from the relation

$$R = \frac{\rho\, v_s\, d}{\mu} = \frac{0.997}{1.027}\,(107.73\, d^2)\, d$$

or

$$R = 104.56\, d^3 \qquad \text{...}(ii)$$

The computed values of v_s and R for various particle sizes (d) are tabulated below :

Weight fraction greater than stated (%)	10	15	40	70	93	99	100
Weight fraction finer than stated %	90	85	60	30	7	1	0
v_s (mm/s)	1.077	0.689	0.528	0.387	0.172	0.043	0.011
R	0.10	0.05	0.04	0.02	0.007	0.0008	0.0001

Since R is less than 0.5, Stoke's law is applicable and hence calculation for v_s are valid. From the above table, the *cumulative distribution curve* is drawn as shown in Fig. 9.31.

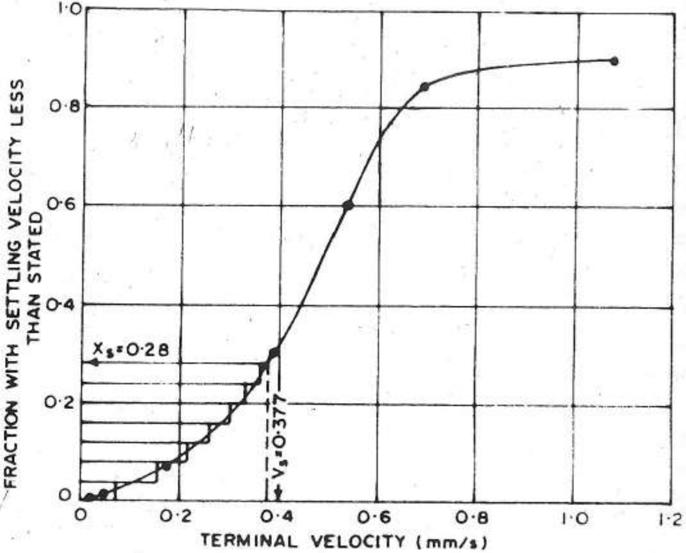


FIG. 9.31. CUMULATIVE DISTRIBUTION CURVE

$$\text{Now SOR} = v_s = 32.6 \text{ m/day} = \frac{32.6 \times 1000}{24 \times 60 \times 60} = 0.377 \text{ mm/s}$$

Hence all particles with setting velocities greater than 0.377 mm/s will be removed. Thus, from the graph (Fig. 9.31), the fraction $(1-x_s)$ is equal to 0.72. The graphical determination of $\Sigma v_s' \, \Delta x$ is tabulated below :

Δx	0.04	0.04	0.04	0.04	0.04	0.04	0.04
v_s'	0.06	0.16	0.22	0.26	0.30	0.33	0.36
$v_s' \, \Delta x$	0.0024	0.0064	0.0088	0.0104	0.0120	0.0132	0.0144

The *overall removal* is given by Eq. 9.13 (*a*)

$$R = (1 - x_s) + \frac{1}{v_s} \Sigma \, v_s' \Delta x$$

$$= 0.72 + \frac{1}{0.377}(0.0676) = 0.899 \text{ or } \mathbf{89.9\ \%}$$

Example 9.14. *A rectangular sedimentation basin is to handle 12 million litres/day of raw water. A sedimentation basin of width to length ratio of $\frac{1}{3}$ is proposed to trap all particles larger than 0.05 mm in size. Assume a relative density of 2.62 for the particles and 20°C as the average temperature, determine the basin dimensions. If the effective depth of tank is 3 m, calculate the detention time.*

Solution.

$$v_s = 418\,(S_s - 1)\, d^2 \left(\frac{3\,t + 70}{100}\right)$$

$$= 418\,(2.62-1)\,(0.05)^2 \left(\frac{3 \times 20+70}{100}\right) = 2.2 \text{ mm/sec}$$

$$= 0.22 \text{ cm/sec } = 7.92 \text{ m/hour}$$

From Eq. 9.11, $v_s = \dfrac{Q}{LB}$

Also, $$t_0 = \frac{L}{v_d} = \frac{LBH}{Q} = \frac{H}{v_s} \qquad \text{...(Eq. 9.11 } d)$$

$$= \frac{H}{7.92} \qquad \text{...}(i)$$

(where t_0 is detention time in hours)

$$Q = 12 \text{ million litres per day } = \frac{12 \times 10^6}{10^3} \times \frac{1}{24} \text{ m}^3\text{/hour}$$

$$= 500 \text{ m}^3\text{/hour}$$

$\therefore$ Capacity of tank, $$V = 500 \times t_0 \qquad \text{...}(ii)$$

$\therefore$ $$LBH = 500 \times \frac{H}{7.92}$$

But $L = 3\,B$

$\therefore$ $$3B^2 = \frac{500}{7.92}$$

From which $B = 4.59 \approx \mathbf{4.60\ m}$

$\therefore$ $L = 4.6 \times 3 = \mathbf{13.8\ m.}$

If $H = 3$ m,

$$t_0 = \frac{H}{7.92} = \frac{3}{7.92} = 0.379 \text{ hours } = \mathbf{22.7\ min.}$$

Example 9.15. *In a continuous flow settling tank, 3.5 m deep and 50 m long, what flow velocity of water would you recommend for effective removal of 0.03 mm particles at 20°C. The specific gravity of particles is 2.65 and the kinematic viscosity of water may be taken as 0.01 cm²/sec.*

Solution :

$$v_s = \frac{g}{18}(S_s - 1)\frac{d^2}{\nu} \qquad ...(9.7\ a)$$

$$= \frac{981}{18}(2.65 - 1)\frac{(0.003)^2}{0.01} = 0.081 \text{ cm/sec.}$$

From Eq. 9.11 (*c*)

$$v_d = \frac{L}{H} v_s$$

Taking 0.5 m F.B., $H = 3.5 - 0.5 = 3$ m ; $L = 50$ m

$$\therefore \qquad v_d = \frac{50}{3} \times 0.081 = \mathbf{1.35\ cm/sec.}$$

Hence the flow velocity or displacement velocity should not be more than 1.35 cm/sec.

Example 9.16. *Two primary settling basings are 25 m in diameter with 2.5 m side water depth. Single effluent weirs are located on the peripheries of the tanks.*

For a water flow of 30000 m³/day, calculate

(i) surface area and volume
(ii) overflow rate in m³/m²/d
(iii) detention time, in hours, and
(iv) weir loading in m³/m/day

Solution :

For a circular sedimentation tank with cone-shaped bottom having a slope of 1 vertical to 12 horizontal,

$$V = D^2(0.011\,D + 0.785\,H)$$

$$= 25^2(0.011 \times 25 + 0.785 \times 2.5) = \mathbf{1398.4\ m^3}$$

$$\text{Now } Q \text{ per tank} = \tfrac{1}{2} \times 30000 = 15000 \text{ m}^3/\text{day}$$

$$= \frac{15000}{24} \text{ m}^3/\text{hr} = 625 \text{ m}^3/\text{hr.}$$

$$\therefore \text{ Detention time } t_0 = \frac{V}{Q}$$

$$= \frac{1398.4}{625} = \mathbf{2.2375\ hours}$$

$$\text{Surface area of tank} = 0.785\,D^2 = 0.785\,(25)^2$$

$$= \mathbf{490.63\ m^2}$$

$$\text{Overflow rate} = \frac{Q}{\text{surface area}} = \frac{15000}{490.63}$$

$$= \mathbf{30.57\ m^3/m^2/day}$$

Length of weir along periphery of the tank

$$= \pi D = \pi \times 25 = 78.54 \text{ m}$$

$$\text{Weir loading per day} = \frac{Q}{\text{Length of weir}}$$

$$= \frac{15000}{78.54} \approx \mathbf{191\ m^3/m/day.}$$

Example 9.17. *A coagulation-sedimentation plant clarifies 50 million litre of water per day. The raw water has an alkalinity equivalent of 4 mg/l of CaCO₃ . The filter alum required at the plant is 20 mg/l. Determine the filter alum and the quick lime (containing 88% to CaO) required per year by the plant. Use the following molecular weights :*

$$(Al = 27 ; S = 32 ; O = 16 ; H = 1 ; Ca = 40 ; C = 12)$$

Solution :

(*i*) Alum required per day $= 50 \times 10^6 \times 20 = 1000 \times 10^6$ mg/day
$= 1000$ kg/day $=$ **365 tonnes/year**

(*ii*) The chemical reaction is indicated by Eq. 9.19, when water has sufficient natural alkalinity. If the natural alkalinity is insufficient, lime is added to water, due to which the reactions are :

$$CaCO_3 + H_2O + CO_2 = Ca\,(HCO_3)_2$$

Mol. wt. of $Al_2(SO_4)_3.18H_2O = (2 \times 27) + 3(32 + 4 \times 16) + 18\,(2 + 16) = 666$

Mol. wt. of $Ca\,(HCO_3)_2 = 40 + 2\,(1 + 12 + 48) = 162$

Mol. wt. of $CaCO_3 = 40 + 12 + 48 = 100$

Mol. wt. of $CaO = 40 + 16 = 56$

From Eq. 9.19, we find that alum requires 3×162 (= 486) parts of natural alkalinity, $Ca\,(HCO_3)_2$ for every 666 parts of alum. Also, 162 parts of natural alkalinity is equivalent to 100 parts of alkalinity as $CaCO_3$.

Hence required alkalinity (as $CaCO_3$), for water containing alum of 20 mg/l

$$= \frac{3 \times 100}{666} \times 20 = 9.01 \text{ mg/l} \qquad ...(i)$$

Natural alkalinity available as $CaCO_3 = 4$ mg/l

∴ Additional alkalinity required to be added in the form of lime

$$= 9.01 - 4 = 5.01 \text{ mg/l as } CaCO_3$$

Since 100 part of $CaCO_3$ produces 56 parts of CaO, quantity of CaO required

$$= \frac{5.01 \times 56}{100} = 2.806 \text{ mg/l.}$$

But market available quick lime contains only 88 per cent of CaO.

$$\therefore \quad \text{Quick lime required} = \frac{2.806 \times 100}{88} = 3.188 \text{ mg/l}$$

$$\therefore \text{ Quantity of quick lime per day} = 50 \times 10^6 \times 3.188 \text{ mg}$$
$$= 159.4 \text{ kg/day} \approx \mathbf{58.2\ tonnes/year}$$

9.21. DESIGN EXAMPLES

Example 9.18. ***Design of rectangular plain sedimentation tank***

Design a rectangular sedimentation tank for the following data:

(i) Required outflow from sedimentation tank $= 300\ m^3/hr$

(ii) Water used in desludging $= 2\ \%$

(iii) Min. size of particles to be removed $= 0.018$ *mm*

(iv) Expected removal efficiency of min. size particle $= 75\ \%$

(v) Nature of particles : Discrete and non-flocculating

(vi) Specific gravity of particles $= 2.65$

(vii) Assumed performance of the settling tank : Good $(n = 1/4)$

(viii) Kinematic viscosity of water at $20\,°C = 1.01 \times 10^{-6}\ m^2/s$

Solution

1. Design elements

We have to design the following elements of the tank

(i) Size (or dimensions) of the tank

(ii) Design of influent structure

(iii) Design of effluent structure.

The tank so designed has to checked against the re-suspension of the deposited particles.

$$\text{Required outflow rate} = 300 \text{ m}^3/\text{hr}$$

$$\text{Water loss} = 2\%$$

$$\therefore \text{ Design average flow} = \left(\frac{100}{100-2}\right) \times 300 \approx 306 \text{ m}^3/\text{hr.}$$

2. Computation of settling velocity of minimum size particles

Assuming Stoke's law to be valid,

$$v_s = \frac{g}{18}(S_s - 1)\frac{d^2}{\nu} \qquad \text{...(Eq. 9.7 } a)$$

$$= \frac{9.81}{18}(2.65 - 1)\frac{(0.018 \times 10^{-3})^2}{1.01 \times 10^{-6}} = 2.88 \times 10^{-4} \text{ m/s}$$

$$\text{Reynold's No.} = \frac{v_s \,.\, d}{\nu} = \frac{2.88 \times 10^{-4}\,(0.018 \times 10^{-3})}{1.01 \times 10^{-6}}$$
$$= 0.00513 < 1$$

Hence stoke's law is applicable and the computations for velocity, done above are valid.

$$\therefore \qquad v_s = 2.88 \times 10^{-4} \text{ m/s.}$$

3. Determination of surface overflow rate

For an ideal tank, for 100% removal of min. size particles, the surface overflow rate will be equal to the settling velocity of min. size particles.

$$\therefore \qquad \text{SOR} = v_s = 2.88 \times 10^{-4} \text{ m/s}$$
$$= 2.88 \times 10^{-4} \times 3600 \times 24 = 24.88 \text{ m/day}$$

However, due to short circuiting, there will be decrease in the efficiency, due to which SOR will have to be *reduced.* For Eq. 9.20 we have

$$\frac{y}{y_0} = 1 - \left[1 + n\frac{v_s}{Q/A}\right]^{-1/n}$$

Here, $\frac{y}{y_0} = 0.75$ and $n = \frac{1}{4}$ (for good performance).

$$\therefore \qquad \frac{v_s}{Q/A} = \frac{1}{n}\left[\left\{1 - \frac{y}{y_0}\right\}^{-n} - 1\right]$$
$$= 4\,[\{1 - 0.75\}^{-1/4} - 1] = 1.657$$

$$\therefore \qquad \text{Design SOR} = \frac{Q}{A} = \frac{v_s}{1.657} = \frac{24.88}{1.657} \approx 15.02 \text{ m/d}$$

The normal values of SOR range between 15 to 30 $m^3/m^2/d$ for plain sedimentation tanks.

4. Determination of dimensions of tank

$$\text{Surface area of tank,} \qquad A = \frac{Q}{Q/A}$$
$$= \frac{306 \times 24}{15.02} = 488.9 \text{ m}^2$$

Let us assume L/B as 4.

Now $\quad L \times B = 488.9$

$$\therefore \quad B = \left(\frac{488.9}{4}\right)^{1/2} \approx 11.1 \text{ m}$$

$$\therefore \quad L = 11.1 \times 4 = 44.4 \text{ m.}$$

Let us assume detention time $t_0 = 4$ hours,

$$\text{Water depth of settling zone} = \frac{Q \times t_0}{A} = \frac{306 \times 4}{11.1 \times 44.4} = 2.48 \text{ m}$$

5. ***Check against scour.***

$$v_d = \sqrt{\frac{8\beta g}{f}(S_s - 1)\,d} \qquad \text{...(9.14)}$$

Taking $\beta = 0.04$ for unigranular sand

$f = 0.03$ for settling tanks.

$$v_d = \sqrt{\frac{8 \times 0.04}{0.03} \times 9.81\,(2.65 - 1)\,(0.018 \times 10^{-3})}$$

$$= 5.57 \times 10^{-2} \text{ m/s}$$

Also, horizontal displacement velocity is given by

$$v_h = \frac{Q}{B \times H} = \frac{306/3600}{11.1 \times 2.48} = 3.09 \times 10^{-3} \text{ m/s}$$

Since $v_h < v_d$, design is OK.

6. ***Design of influent structure.***

The influent structure may consist of (*i*) an influent channel (*ii*) submerged orifices, and (*iii*) baffles in front of orifices (Fig. 9.7 *b*).

Let us provide 0.75 m wide and 0.6 m deep influent channel, running across the width of the tank. Let us assume a depth in of flow of 0.4 m.

$$\therefore \text{ Velocity of flow in channel} = \frac{306/3600}{0.75 \times 0.4}$$

$$= 0.283 \text{ m/s} < 0.3 \text{ m/s.}$$

Let us provide 4 submerged orifices of size 0.20 m × 0.20 m in the inside wall of the influent channel to distribute the flow uniformly in to the settling tank.

Also, provide a baffle 1 m deep, at a distance of 0.8 m from the orifices, to reduce the disturbance.

$$\text{Head loss through the orifices} = \left(\frac{Q}{C_d\, a\sqrt{2g}}\right)^2$$

$$= \left(\frac{306/3600}{0.6 \times 4\,(0.20 \times 0.20)\sqrt{2 \times 9.81}}\right)^2$$

$$= 0.04 \text{ m}$$

7. ***Design of effluent structure***

Various components to be provided in an effluent structure are : effluent weir, effluent launder (or trough), outlet box and outlet pipe or channel.

Let us assume a weir loading of 250 $\text{m}^3/\text{d/m}$ length

$$\therefore \quad \text{Weir length} = \frac{300 \times 24}{250} = 28.8 \text{ m.}$$

Provide effluent weir of 30 m length. Let us provide 90° V-notches @ 200 mm c/c.

$$\text{No. of V-notches} = \frac{30}{0.2} = 150$$

Hence Provide 30 m length of effluent launder with 90° V-notches fixed only to one side of the launder. Let us keep width of launder = 0.3 m.

Hence critical depth h at the other end of the launder is (Eq. 9.18 *b*) :

$$h = \left(\frac{Q^2}{b^2 g}\right)^{1/3}$$

where $\quad Q = \text{discharge from each side} = \frac{300}{2}\ \text{m}^3/\text{hour}$

$$= \frac{300}{2 \times 3600}\ \text{m}^3/\text{s} = 0.04167\ \text{m}^3/\text{hour}$$

b = width of launder = 0.3 m.

$$\therefore \quad h = \left[\frac{(0.04167)^2}{(0.3)^2 \times 9.81}\right]^{1/3} = 0.125 \text{ m}$$

Depth of water at the other end, neglecting the friction losses, is given by Eq. 9.18 (*a*), taking $n = 1$

$$\therefore \quad H = \left[h^2 + \frac{2q^2L^2}{gb^2h}\right]^{1/2},$$

where $\quad q.L = \frac{300}{2 \times 3600} = 0.04167\ \text{m}^2/\text{s}$

$$\therefore \qquad H = \left[(0.125)^2 + \frac{2\,(0.04167)^2}{9.81\,(0.3)^2 \times 0.125} \right]^{1/2}$$

$$= 0.217 \text{ m.}$$

Let us *assume* a head loss of 0.1 m. Providing free board of 0.1 m, depth of the launder below the bottom of weir discharging into it is $= 0.217 + 0.1 + 0.1 = 0.417$ m. If the weir has 50 mm deep V-notches, the resulting depth of channel $= 0.417 + 0.05 = 0.467$ m. Hence provide the effluent launder of width equal to 300 mm and depth equal to 500 mm.

Example 9.19. ***Design of radial circular plain sedimentation tank***

Design a secondary circular sedimentation tank to remove alum floc for the following data :

(*i*) *Required output from the tank* $= 300\ m^3/hr.$
(*ii*) *Water used in desludging* $= 2\%$
(*iii*) *Min. size of alum floc to be removed* $= 0.8\ mm$
(*iv*) *Specific gravity of alum floc* $= 1.002$
(*v*) *Expected removal efficiency of alum floc* $= 75\%$
(*vi*) *Assumed performance of settling tank : very good* $\left(n = \frac{1}{8}\right)$
(*viii*) *Kinematic viscosity of water at* $20°C = 1.01 \times 10^{-6}\ m^2/s.$

Solution.

1. ***Design elements***

This is a typical radial flow circular tank, with central feed, wherein the water enters at the centre of the tank and emanates from the multiple ports of circular well in the centre of tank to flow radially outwards in all directions equally (Fig. 9.12). The effluent launder is provided at the outer peripheri of the tank.

Required outflow rate $= 300\ \text{m}^3/\text{hr}.$
Water loss for desludging $= 2\%$

$$\therefore \text{ Average design flow} = \frac{100}{100-2} \times 300 \approx 306\ \text{m}^3/\text{hr}.$$

2. ***Design of influent pipe***

Let us assume a velocity of flow of 1 m/s in the pipe.

$$\therefore \text{ Influent pipe dia.} = \sqrt{\frac{306}{3600} \times 1 \times \frac{\pi}{4}} = 0.26 \text{ m.}$$

Provide an influent pipe of 300 mm dia.

3. ***Computation of settling velocity of min. size floc***

Assuming Stoke's law to be valid,

$$v_s = \frac{g}{18}(S_s - 1)\frac{d^2}{\nu}$$

$$= \frac{9.81}{18}(1.002 - 1)\frac{(0.8 \times 10^{-3})^2}{1.01 \times 10^{-6}} = 6.907 \times 10^{-4} \text{ m/s}$$

$$\text{Reynolds No.} \qquad R = \frac{v_s d}{\nu} = \frac{6.907 \times 10^{-4} \times 0.8 \times 10^{-3}}{1.01 \times 10^{-6}}$$

$$= 0.55 < 1$$

Hence Stoke's law is applicable and the computations for velocity done above are correct.

$$\therefore \qquad v_s = 6.907 \times 10^{-4} \text{ m/s.}$$

4. ***Determination of surface overflow rate***

For an ideal tank, for 100% removal of min. size flocs, the SOR will be equal to the settling velocity of min. size flocs.

$$\therefore \qquad \text{SOR} = v_s = 6.907 \times 10^{-4} \text{ m/s.}$$

$$= 6.907 \times 10^{-4} \times 24 \times 3600 \approx 59.7 \text{ m/d}$$

However, due to short circuiting, there will be decrease in the efficiency, due to which SOR will have to be reduced.

From Eq. 9.20, we have

$$\frac{y}{y_0} = 1 - \left[1 + n\frac{v_s}{Q/A}\right]^{-1/n}$$

$$\text{Here} \qquad \frac{y}{y_0} = 0.75 \text{ and } n = \frac{1}{8}$$

$$\therefore \qquad \frac{v_s}{Q/A} = \frac{1}{n}\left[\left\{1 - \frac{y}{y_0}\right\}^{-n} - 1\right]$$

$$= 8\left[\left\{1 - 0.75\right\}^{-1/8} - 1\right] = 1.514$$

$$\therefore \qquad \frac{Q}{A} = \frac{v_s}{1.514} = \frac{59.7}{1.514} = 39.43 \text{ m/d.}$$

This is within the permissible range of $30 - 40\ \text{m}^3/\text{m}^2/\text{d}$, and is therefore acceptable.

5. ***Determination of dimensions of the tank***

$$\text{Surface area of tank} = \frac{Q}{Q/A} = \frac{306 \times 24}{39.43} = 186.24 \text{ m}^2$$

$$\therefore \qquad \frac{\pi}{4}(D^2 - D_P^2) = 186.24$$

or
$$\frac{\pi}{4}[D^2 - (0.3)^2] = 186.24$$

From which $D = 15.4$ m.

Providing a detention time $t_0 = 2.5$ hours,

$$\text{Depth of tank} = \frac{Q \times t_0}{A} = \frac{306 \times 2.5}{186.24} = 4.1 \text{ m.}$$

5. ***Design of peripheral effluent launder***

For a peripheral effluent launder, half the flow is assumed to go about each side of the basin.

$$\text{Total length of launder} = \pi D = \pi \times 15.4$$
$$= 43.38 \text{ m.}$$

$$Q = \text{discharge from each side} = \frac{300}{2} = 150 \text{ m}^3/\text{hour}$$

$$= \frac{150}{3600} = 0.04167 \text{ m}^3/\text{s}$$

Let us provide 90° V-notches fixed to the inner side of th launder, all along its perimeter. Let us keep width of launde $= 0.3$ m. Hence critical depth h at the other end of the launde (Fig. 9.18 b) is.

$$h = \left(\frac{Q^2}{b^2 g}\right)^{1/3} = \left[\frac{(0.04167)^2}{(0.3)^2 \times 9.81}\right]^{1/3}$$
$$= 0.125 \text{ m.}$$

The depth of water at the other end, neglecting the friction losses, is given by. Eq. 9.18 (a) taking $n = 1$.

$$\therefore \qquad H = \left[h^2 + \frac{2\,(qL)^2}{g b^2 h}\right]^{1/2}$$
$$= \left[(0.125)^2 + \frac{2\,(0.04167)^2}{9.81\,(0.3)^2 \times 0.125}\right]^{1/2}$$
$$= 0.217 \text{ m.}$$

Let us assume a head loss of 0.1 m. Providing a F.B. of 0.1 m, depth of launder below the bottom of weir discharging into it is $= 0.217 + 0.1 + 0.1 = 0.417$ m. If the weir has 50 mm deep V-notches, the resulting depth of launder$= 0.417 + 0.05 = 0.467$ m. Hence provide the effluent launder of width equal to 300 mm and depth equal to 500 mm.

Check for weir loading

$$\text{Weir loading} = \frac{300 \times 24}{43.38}$$
$$\approx 166 \text{ m}^2/\text{d}/\text{m} < 300 \text{ m}^3/\text{d}/\text{m}.$$

Hence ok.

Example 9.20. ***Design of mechanical rapid mix unit***

Design a mechanical rapid mix unit for a design flow to be treated equal to 300 m³/h. Assume suitable permissible values for various parameters of design. Assume a temperature of 20°C.

Solution

1. ***Design parameters.***

Let us adopt the following values of various design parameters.

(i) *Detention time* : Normal range is 20 to 60 s. Let us provide a detention time of 30 sec.

(ii) *Velocity gradient* : G should be greater than $300\,s^{-1}$. Let us adopt $G = 600\,s^{-1}$.

(iii) *Rotational speed of impeller* : Speed should be greater than 100 rpm. Let us provide a speed of 125 rpm.

(iv) *Ratio of tank height to diameter* The ratio varies from 1:1 to 3:1. Let use provide a ratio of 1.5 : 1.

(v) *Ratio of impeller diameter to tank diameter* : Range of the ratio is from 0.2 :1 to 0.4 : 1. Let us provide a ratio of 0.4:1.

2. ***Determination of dimensions of the tank***

$$\text{Volume of tank} = \text{Flow} \times \text{detention time}$$
$$= \frac{300}{3600} \times 30 = 2.5 \text{ m}^3.$$

$$\text{But volume} = \text{area} \times \text{depth} = \left(\frac{\pi}{4} D^2\right)(1.5\,D)$$

$$\therefore \qquad \frac{\pi}{4} \times 1.5\,D^3 = 2.5$$

From which $D = 1.285$ m.

Provide tank dia. $= 1.3$ m.

$\therefore$ Depth of tank$= 1.3 \times 1.5 = 1.95$ m. Providing a free board of 0.2 m, total depth of tank $= 1.95 + 0.2 = 2.15$ m.

3. ***Computation of power requirements***

From Eq. 9.29, we get

$$P = G^2 . \mu V$$
$$\mu = 1.0087 \times 10^{-3} \text{ at } 20°\text{C}.$$

$$\therefore \quad P = (600)^2 \times 1.0087 \times 10^{-3} \times 2.5$$
$$= 908 \text{ watts.}$$

$\therefore$ Power per unit volume $= \dfrac{908}{2.5} = 363 \text{ watts/m}^3$

Power per unit flow of water $= \dfrac{908}{300}$

$$= 3.03 \text{ watts/m}^3\text{/hr of flow.}$$

4. ***Determination of dimensions of flat blades and impeller***

Dia. of impeller = 0.4× tank dia.

$$= 0.4 \times 1.3 = 0.52 \text{ m}$$

Velocity at the tip of impeller $= \dfrac{2\pi r n}{60}$ m/s

or $$v_p = 2\pi \times \frac{0.52}{2} \times \frac{125}{60} = 3.40 \text{ m/s}$$

Again, from Eq. 9.32,

$$P = \tfrac{1}{2} C_D \,.\, \rho \,.\, A_b \,.\, v_r^3$$

where A_b = area of blades
v_r = relative velocity of blades $= (1-k)\, v_p$
v_p = velocity at the tip of impeller or blade
P_c = power spent = 90.8
ρ = density of water $= 998 \text{ kg/m}^3$ at 20°C

Taking $k = 0.25$,

$$v_r = (1 - 0.25) v_p = 0.75\, v_p = 0.75 \times 3.40$$
$$= 2.55 \text{ m/s}$$

$\therefore$ Taking $C_D = 1.8$ for blades, we get

$$908 = \tfrac{1}{2} \times 1.8 \times 998 \times A_b\, (2.55)^3$$

From which $A_b = 0.06097 \text{ m}^2$

Providing 6 blades, area of each blade $= \tfrac{1}{6} \times 0.06097 \approx 0.01 \text{ m}^2$.
Provide blades of size 0.09×0.12 m (*i.e.* 9 cm × 12 cm).

Example 9.21. ***Design of flocculator***

Design a flocculator for a design flow to be treated equal to 300 m³/h. Assume suitable permissible values of various parameters of design. Assume a temperature of 20°C.

Solution.

1. ***Design parameters.***

The following design parameters are assumed :

(*i*) Detention period = 20 minutes.
(*ii*) Av. value of $G = 40\, s^{-1}$.
(*iii*) Speed of paddles = 4.5 rpm

(*iv*) Area of paddles = 15% of the cross-sectional area of basin.
(*v*) Velocity ratio $k = 0.25$
(*vi*) $\mu = 1.0087 \times 10^{-3}$ N-s/m² at 20°C
(*vii*) $\rho = 998 \text{ kg/m}^3$ at 20°C.
(*viii*) Ratio of Length to width of tank = 2

2. ***Computation of volume of flocculation tank***

Volume of tank V = design flow × detention time

$$= \frac{300}{60} \times 20 = 100 \text{ m}^3$$

Let the depth of tank $= 0.4\, B$
Let the length of tank $= 2B$

$$\therefore \quad B \times 2B \times 0.4B = 100$$

From which
$$B = 5 \text{ m.}$$
$$L = 5 \times 2 = 10 \text{ m}$$
$$H = 0.4 \times 5 = 2 \text{ m}$$

3. ***Computation of power required***

From Eq. 9.29

$$P = G^2 \,.\, V \,.\, \mu$$
$$= (40)^2 \times 100 \times 1.0087 \times 10^{-3}$$
$$= 161.4 \text{ watts.}$$

4. ***Computation of velocity differential between the paddle and water***

Let us provide revolving paddels attached to three horizontal shafts running parallel to the length. Let each shaft be located at mid-depth of the tank. Let us provide four paddles to each shaft, each running parallel to the shaft.

Max. available *outer radial* distance of each paddle

$$= \frac{B}{2 \times 3} = \frac{5}{6} = 0.833 \text{ m.}$$

Assuming 25 cm width of blade, max. value of $r = 0.833 - \dfrac{0.25}{2} = 0.708$ m.

Hence keep distance r from the centre line of paddles to the centre of shaft as 0.7 m.

$$\therefore \quad v_p = \frac{2\pi r n}{60} = \frac{2\pi}{60} \times 0.7 \times 4.5 = 0.3299 \text{ m/s}$$

$$\therefore \quad v_r = (1-k)\, v_p = (1 - 0.25) \times 0.3299$$
$$= 0.2474 \text{ m/s.}$$

5. ***Computation of paddle size.***

From Eq. 9.32

$$P = \tfrac{1}{2} C_D \rho \, . A_p \, . v_r^3$$

Taking $C_D = 1.8$ for flat blades, we have

$$161.4 = \tfrac{1}{2} \times 1.8 \times 998 A_p (0.2474)^3$$

From which $A_p = 11.87 \text{ m}^2$

There area total of $3 \times 4 = 12$ paddles

∴ Area of each paddle= $11.87/12 = 0.989 \text{ m}^2$.

Let the length of each paddle= 4.8 m.

(slightly less than half the length of tank)

∴ Width of paddle $= \dfrac{0.989}{4.8} = 0.206$ m.

Hence provide 21 cm wide paddles, each of length 4.8 m.

PROBLEMS

1. What do your understand by treatment of water ? Why is it necessary ? Give an outline of various processes adopted for treatment of water.

2. Describe in brief various types of screens used for screening water.

3. Working from the fundamentals, derive Stoke's law for settlement of discrete particles in water. How do you modify the law, taking into account the temperature effect ?

4. What do you understand by plain sedimentation ? Describe the design principles of a settling tank.

5. Differentiate between Stoke's law and Newton's law. How is the drag co-efficient related to Reynolds number.

6. Find the settling velocity of silica particles of specific gravity 2.65 at 20°C, if the diameter of particles is (*a*) 0.005 cm, (*b*) 0.02 cm.

Ans. [(*a*) 0.224 cm/sec. ; (*b*) 2.7 cm/sec.]

7. Describe in brief various types of sedimentation tanks generally used.

8. Define and explain the following terms :

(*a*) detention period

(*b*) flow through period

(*c*) displacement efficiency

(*d*) surface loadings.

9. Write an expression for the maximum velocity to prevent bed uplift or scour of the settled particles. Show that for an ideal tank, the ratio of its length to depth should be equal to $\sqrt{8/f}$.

10. Design a plain sedimentation tank to treat 5 million litres water per day. Take a detention period of 8 hours and assume a depth of 3.5 m.

11. Determine the maximum displacement velocity, without the fear of settled particles being lifted up, and the ratio of length to the depth of the settling unit for the following particles :

(*a*) alum floc having $S_s = 1.15$ and diameter 0.08 cm, and

(*b*) anthracite coal dust having $S_s = 1.6$ and diameter = 0.008 cm.

Assume a Weisbach Darcy friction factor $f = 0.025$ and a temperature of 25°C.

Ans. [(*a*) 104.1 cm/sec, $L/H = 17.89$; (*b*) 7.76 cm/sec, $L/H = 33.30$]

12. A settling tank is designed for an overflow rate of 3,000 litres per m² per hour. What percentage of particles of diameter of 0.025 mm (specific gravity 2.65) will be removed at 20°C ?

Ans. [67.3%]

13. Find the diameter of particles with specific gravity of 1.4 removed in a tank having a surface area of 300 m² and treating 10 million litres of water per day. Assume a temperature of 20°C.

Ans. [0.02 mm.]

14. What do you understand by coagulation and flocculation ? why are they necessary ?

15. Describe various types of coagulants commonly used in water treatment.

16. Describe various methods of application of coagulants.

17. (*a*) Prove that theoretically the surface loading (Q/A), and not the depth, is a measure of effective removal of particles in a sedimentation tank.

(*b*) An ideal sedimentation tank with a surface area of 100 m² receives an inflow of 20 millions per day. Referring to tabuler data below for particles of specific gravity 1.20, find out what percentage of each size would settle down.

Dia. in mm	0.5	0.4	0.3	0.2	0.1
settling velocity (cm/s)	0.65	0.49	0.38	0.22	0.08

(A.M.I.E., May 1969)

18. (*a*) Why alum is commonly used coagulant ? Write down the reactions.

(*b*) Explain the mechanism of floc formation.

19. Explain various mixing devices commonly used. What is a flash mixer ?

20. Explain what is meant by flocculation ? Sketch any one type of flocculator with mechanical agitators.

21. Design a coagulation sedimentation tank to treat 16 million litres of water per day. Assume suitable data wherever necessary.

22. Write short notes on the following :

(*ii*) *Free board.* The vertical distance between top of wall and water level in the tank is known as free board and a depth of about 30 *cm* to 60 *cm* should be kept as free board.

(*iii*) *Sludge Capacity.* For the collection of sludge, it is necessary to provide enough space at the bottom of tank. A depth of about 50 *cm* to 60 *cm* is provided for this purpose.

(*iv*) *Sludge Removal.* In order to remove sludge, the working of tank is stopped and water from the tank is drained out. The sludge is removed by gravity or pumping the compressed air and manual labour may be applied, if necessary. The frequency of sludge removal will depend upon the amount of suspended matter retained from water in the settling tank. Generally, the settling tanks are cleaned at an interval of 1 to 3 months.

11.7 SOLVED EXAMPLES

Following solved examples will clearly illustrate the design of simple sedimentation tanks.

Example 11.1. *A water has to purify the water for a town whose daily demand is* 9×10^6 *litres/day. Design the suitable sedimentation tank of the water works fitted with mechanical sludge remover. Assume the velocity of flow in the sedimentation tank as 22 cm/minute and the detention period as 8 hours.*

Solution. Quantity of water to be treated in 24 hours

$$= 9 \times 10^6 \textit{ litres}$$

Quantity of water to be treated by the sedimentation tank during detention period of 8 hours

$$= \frac{9 \times 10^6}{24} \times 8 \textit{ litres} = 3 \times 10^3 \textit{ cu. m}$$

$$= 3000 \textit{ cu. m}$$

Capacity of the sedimentation tank required

$$= 3000 \textit{ cu. m}$$

Velocity of flow inside the tank is given as 22 *cm/minute*

$$= 0.22 \textit{ m/min.}$$

$$\therefore \quad \text{Length of the tank} = \text{Velocity of flow} \times \text{Detention period}$$

$$= 0.22 \times (8 \times 60) = 106 \textit{ m} \text{ say}$$

The cross-sectional area of tank required

$$= \frac{\text{Capacity of the tank}}{\text{Length of the tank}}$$

$$= \frac{3000}{106} = 28.3 \textit{ sq.m.}$$

Assuming the depth of water as 3.5 *m*

$$\text{Required width of the tank} = \frac{28.3}{3.5} = 8.10 \textit{ m.}$$

Providing a free board of 0.5 *m*, overall depth of the tank

$$= 0.5 + 3.5 = 4.0 \textit{ m}$$

$$\therefore \quad \text{Dimension of the tank} = 106 \times 8.10 \times 4.0 \textit{ m} \textbf{ Ans.}$$

The length of the tank may be provided in one, two or three parts depending upon the shape of the are available.

Example 11.2. *Design a sedimentation for a water works, which supplies* 1.4×10^6 *litre/day water to the town. The sedimentation period is 5 hours, the velocity of flow is 12 cm/minute, depth of water in the tank is 4.0 m. Assuming an allowance for sludge is to be made as 80 cm.*

Solution. Quantity of water to be treated

$= 1.4 \times 10^6$ *litres/day*

∴ Quantity of water to be treated during detention period of 5 hours

$$= \frac{1.4 \times 10^6}{24/5} \text{ litres} = 0.292 \times 10^3 \text{ cu.m}$$

$= \mathbf{292\ cu.m}$

The velocity of flow has been given as 12 *cm/min.*

$= 0.12$ *m/min.*

∴ Required length of the tank = Flow velocity × Detention time

$= 0.12 \times (5 \times 60) = 36.0\ m$

$$\text{Cross-sectional area of the tank} = \frac{\text{Tank capacity}}{\text{Tank length}}$$

$$= \frac{292}{36} \text{ sq.m} = 8.12 \text{ sq.m.}$$

Depth of the water in tank = **4.0 *m***

Depth of sludge = 0.80 *m*

∴ Net water depth = 3.20 *m*

$$\therefore \quad \text{Width of the tank} = \frac{\text{Cross-sectional area}}{\text{Water depth}}$$

$$= \frac{8.12}{3.20} = 2.54\ m\ (2.60\ m\ say)$$

Proving free board of 0.5 *m* the size of the tank

$= 36 \times 2.6 \times 4.5$ *m* **Ans.**

Example 11.3. *Design a circular sedimentation tank fitted with mechanical sludge remover for a water work which has to supply daily 4.2 million litres of water to the town. The detention period in the tank for water is 4.5 hours, and the depth of the water in the tank may be assumed as 3.3 m.*

Solution. Quantity of water to be treated

$= 4.2 \times 10^6$ *litres/day*

Quantity of water to be treated by the sedimentation tank during detention period of 4.5 hours

$$= \frac{4.2 \times 10^6}{24/4.5} \text{ litres}$$

$= 0.788 \times 10^3$ *cu.m* = 790 *cu.m. say*

The capacity of circular tank having diameter as '*D*' and water depth as '*H*' is given by

$$Q = D^2(0.011\,D + 0.785\,H)$$

Here

$Q = 790$ *cu.m.*

D = to be determined

$H = 3.3\ m$

$\therefore \quad 790 = D^2(0.011\,D + 0.785 \times 3.3)$

or $\quad 790 = D^2(0.011\,D + 2.59)$

Solving the equation by Hit and Trial method

$= 16.85\ m$ (17 *m* say)

Assuming free board of 0.5 *m* the size of the sedimentation tank

= **3.8 *m* deep × 17.0 *m* dia. Ans.**

Example 11.4. *If a rectangular sedimentation tank is treating 2.5×10^6 litres/day. The size of the tank is $17.5 \times 5.5 \times 3.5$ m. If 80 p.p.m. suspended solids are present in the water, assuming the 75% removal in the basin and the average specific gravity as 2.0, determine the following :*

(*i*) *Average flow of water through tank.*
(*ii*) *Detention time.*
(*iii*) *Deposition of the solids in the tank.*
(*iv*) *Overflow rate.*

Solution. Assuming the free board as 50 *cm*, the capacity of the tank

$$= 17.5 \times 5.5 \times 3.0 = 288.75 \text{ cu.m.}$$

Quantity of water passing through the sedimentation tank

$$= 2.5 \times 10^6 \text{ litres/day}$$

$$= \frac{2.5 \times 10^6}{24} \text{ litres/hour}$$

$$= 0.1041 \times 10^6 \text{ litres/hour}$$

$$= 104.1 \text{ cu.m/hour}$$

(*i*) Average flow of water through tank

$$= \frac{\text{Discharge}}{\text{Cross-sectional area}}$$

$$= \frac{104.1}{5.5 \times 3.0}$$

$$= 6.3 \times \frac{100}{60} \text{ cm/min.} = 10.5 \text{ cm/min.} \textbf{ Ans.}$$

(*ii*) Detention time $= \dfrac{\text{Capacity of tank}}{\text{Discharge}}$

$$= \frac{288.75}{104.1} = 2.744 \text{ hours} \textbf{ Ans.}$$

(*iii*) Quantity of the water passing through the sedimentation tank

$$= 2.5 \times 10^6 \times \frac{80}{10} \text{ litres/day}$$

Concentration of suspended solids

$$= 80 \text{ ppm}$$

Quantity of suspended solids $= 2.5 \times 10^6 \times \dfrac{80}{10^6}$ *litres*

$$= 200 \text{ litres} = 0.20 \text{ cu.m.}$$

The average specific gravity is given as 2.0 the density will be 2000 kg/m^3.

∴ Weight of the suspended solids deposited in the tank with 75% removal

$$= 0.20 \times 0.75 \times 2000 \text{ kg}$$

$$= 300 \text{ kg} \textbf{ Ans.}$$

(*iv*) The overflow rate of the tank

$$= \text{Discharge per unit plan area}$$

$$= \frac{Q}{L.B.}$$

$$= \frac{104.1 \times 10^3 \text{ litres/hour}}{17.5 \times 5.5 \text{ m}^2}$$

$$= \mathbf{1081.56} \textbf{ litres/hour/m}^2$$

12.15 LIMITATIONS OF THE PROCESS

The coagulation process removes the suspended impurities of water and considerably reduces the load on the filtration process. The turbidity of water can be removed less than 20 ppm and if the process is properly controlled it can go upto 5-10 ppm. The floc formed in this process also removes bacteria upto 65%. *B*-coli index is removed by 70%. The efficiency of the coagulation depends on the proper control of various processes.

12.16 SOLVED EXAMPLES

Example 12.1. *Waterworks of a town treat 35 × 10⁶ litres/day. The water is treated by coagulation-sedimentation tanks. The quantity of filter alum is consumed at 20 mg/litres of water. If the alkalinity of the raw water is equivalent to 4.5 mg/litre of* $CaCO_3$*, determine the quantity of filter alum and the quick lime (containing 80% of CaO) required per month by the water works. Molecular weights are given as [Ca = 40, C = 12, S = 32, O = 16, Al = 27 and H = 1].*

Solution. Quantity of water treated

$$= 35 \times 10^6 \textit{ litres/day}$$

∴ Quantity of filter alum required @ 20 *mg/litre*

$$= 20 \times 35 \times 10^6 \; mg/day = \frac{20 \times 35 \times 10^6}{10^6} \; kg/day$$

$$= 700 \; kg/day = 21700 \; kg/month \text{ (of 31 days)}$$

$$= 21.7 \; tonnes/month. \textbf{ Ans.}$$

Following chemical reactions take place during treatment

1. $Al_2(SO_4)_3\,18H_2O + 3\,Ca(HCO_3)_2 \rightarrow 2Al(OH)_3 + 3\,CaSO_4 + 18H_2O + 6CO_2\uparrow$
2. $CaCO_3 + H_2O + CO_2 \rightarrow Ca(HCO_3)_2$
3. $CaCO_3 \rightarrow CaO + CO_2$

The molecular weights shall be

$$Al_2(SO_4)_3 \,.\, 18H_2O = 2 \times 27 + 3\,(32 + 64) + 18\,(2 + 16) = 666$$

$$CaCO_3 = 40 + 12 + 48 = 100$$

$$CaO = 40 + 16 = 56$$

From the above equations, it is clear that 3 × 100 parts of $CaCO_3$ will produce the same alkalinity, which is produced by 666 parts of Alum.

Quantity of $CaCO_3$ required to produce the same alkalinity which is equivalent to 20 *mg/litre*.

$$= \frac{3 \times 100}{666} \times 20 \; mg/litre = 9.009 \; mg/litre$$

The natural alkalinity available as

$$CaCO_3 = 4.5 \; mg/litre \text{ (given)}$$

Additional alkalinity required to be added in the form of the lime

$$= 9.009 - 4.5$$

$$= 4.509 \; mg/litre \; as \; CaCO_3$$

From the above equation , it is also clear that 100 parts of $CaCO_3$ is produced by 56 parts of *CaO*.

∴ Quantity of *CaO* required $= \dfrac{4.509 \times 56}{100} = 2.525 \; mg/litre$

But as the quick lime contains 80% of *CaO*, therefore quantity of the quick lime required

$$= \frac{2.525 \times 100}{80} = 3.156 \; mg/litre$$

The quantity of quick lime required for treating 3.5 × 10⁶ *litres/day*

$$= 3.156 \times 35 \times 10^6 \; mg/day = 110.46 \; kg/day$$

$$= 3424.26 \; kg/month \text{ (of 31 days)}$$

$$= 3.43 \; tonnes/month.$$

12.15 LIMITATIONS OF THE PROCESS

The coagulation process removes the suspended impurities of water and considerably reduces the load on the filtration process. The turbidity of water can be removed less than 20 ppm and if the process is properly controlled it can go upto 5-10 ppm. The floc formed in this process also removes bacteria upto 65%. *B*-coli index is removed by 70%. The efficiency of the coagulation depends on the proper control of various processes.

12.16 SOLVED EXAMPLES

Example 12.1. *Waterworks of a town treat 35 × 10^6 litres/day. The water is treated by coagulation-sedimentation tanks. The quantity of filter alum is consumed at 20 mg/litres of water. If the alkalinity of the raw water is equivalent to 4.5 mg/litre of $CaCO_3$, determine the quantity of filter alum and the quick lime (containing 80% of CaO) required per month by the water works. Molecular weights are given as [Ca = 40, C = 12, S = 32, O = 16, Al = 27 and H =1].*

Solution. Quantity of water treated

$$= 35 \times 10^6 \ litres/day$$

∴ Quantity of filter alum required @ 20 *mg/litre*

$$= 20 \times 35 \times 10^6 \ mg/day = \frac{20 \times 35 \times 10^6}{10^6} \ kg/day$$

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$$= 21.7 \ tonnes/month. \ \textbf{Ans.}$$

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1. $Al_2(SO_4)_3 \, 18H_2O + 3\,Ca(HCO_3)_2 \rightarrow 2Al(OH)_3 + 3\,CaSO_4 + 18H_2O + 6CO_2 \uparrow$
2. $CaCO_3 + H_2O + CO_2 \rightarrow Ca(HCO_3)_2$
3. $CaCO_3 \rightarrow CaO + CO_2$

The molecular weights shall be

$$Al_2(SO_4)_3 \,.\, 18H_2O = 2 \times 27 + 3\,(32 + 64) + 18\,(2 + 16) = 666$$

$$CaCO_3 = 40 + 12 + 48 = 100$$

$$CaO = 40 + 16 = 56$$

From the above equations, it is clear that 3 × 100 parts of $CaCO_3$ will produce the same alkalinity, which is produced by 666 parts of Alum.

Quantity of $CaCO_3$ required to produce the same alkalinity which is equivalent to 20 *mg/litre*.

$$= \frac{3 \times 100}{666} \times 20 \ mg/litre = 9.009 \ mg/litre$$

The natural alkalinity available as

$$CaCO_3 = 4.5 \ mg/litre \text{ (given)}$$

Additional alkalinity required to be added in the form of the lime

$$= 9.009 - 4.5$$

$$= 4.509 \ mg/litre \ as \ CaCO_3$$

From the above equation , it is also clear that 100 parts of $CaCO_3$ is produced by 56 parts of *CaO*.

$$\therefore \quad \text{Quantity of } CaO \text{ required } = \frac{4.509 \times 56}{100} = 2.525 \ mg/litre$$

But as the quick lime contains 80% of *CaO*, therefore quantity of the quick lime required

$$= \frac{2.525 \times 100}{80} = 3.156 \ mg/litre$$

The quantity of quick lime required for treating 3.5 × 10^6 *litres/day*

$$= 3.156 \times 35 \times 10^6 \ mg/day = 110.46 \ kg/day$$

$$= 3424.26 \ kg/month \text{ (of 31 days)}$$

$$= 3.43 \ tonnes/month.$$

Example 12.2. *Determine the quantity of copperas and the lime required per year to treat 4 × 10⁶ litre/day, if 11 mg of copperas is consumed with lime at a coagulation basin. Molecular wright of Fe = 55.85, S = 32, O = 16, H = 1, Ca = 40.*

Solution. Quantity of water to be treated

$$= 4 \times 10^6 \textit{ litres/day}$$

Quantity of copperas required $= 11$ *mg/litre* $= 11 \times 4 \times 10^6$ *mg/day*

$= 44$ *kg/day* $= 44 \times 365$ *kg/year*

$=$ **16.06 tonnes/year Ans.**

The chemical reactions which take place during treatment process are :

$$FeSO_4 \cdot 7H_2O + Ca\,(OH)_2 \rightarrow Fe\,(OH)_2 + CaSO_4 + 7H_2O$$

$$CaO + H_2O \rightarrow Ca\,(OH)_2$$

Now the molecular weight of

$$FeSO_4 \cdot 7H_2O = [55.85 + 32 + 4 \times 16 + 7\,(2 \times 1 + 16)]$$

$$= 277.85\ (= 278 \text{ say})$$

$$CaO = 40 + 16 = 56$$

Now from the above equations it is clear that one molecule of copperas requires one molecule of lime. On the other hand 278 *mg* of copperas will require 56 *mg* of quick lime.

∴ 16.06 tonnes of coppers will require

$$= \frac{56}{278} \times 16.06 \textit{ tonnes} = 3.23$$

∴ Quantity of quick lime required

$= 3.23$ *tonnes/year.* **Ans.**

Example 12.3. *The requirements of a city is 40 × 10⁶ litres/day. The detention period is one hour in the tank, and the flow velocity is 20 cm/sec. Design baffle-wall sedimentation tank. Any data not given may be suitably assumed.*

Solution. Quantity of water to be treated

$$= 40 \times 10^6 \textit{ litres/day}$$

$$= 40 \times 10^3 \textit{ cu.m/day}$$

Quantity of water to be treated by the sedimentation tank in 1.0 hour detention period

$$= \frac{40}{24} \times 10^4 \ mm = 1666.7 \ cu.m.$$

The flow velocity is given as 20 *cm/sec.*

∴ Length of the tank (or flow) = Velocity × Detention time

$$= 0.2 \times (60 \times 60)\, m = 720$$

$$\text{Cross-sectional area of the tank} = \frac{\text{Capacity of tank}}{\text{Length of tank}} = \frac{1666.7}{720}$$

$$= 2.315\, m^2 \qquad (= 2.35\, m^2 \text{ say})$$

Keeping the distance between the baffle walls as 50 *cm*

$$\text{The depth of water in the basin} = \frac{\text{Cross-sectional area of channel}}{\text{Distance between baffles}}$$

$$= \frac{2.35}{0.50} = 4.7\, m$$

The clear opening between the end of each baffle and the wall is kept as 1.5 times the distance between baffles.

$$= 1.5 \times 50\, cm = 0.75\, m$$

Fig. 12.11 shows the plane and section of the designed baffle wall sedimentation tank.

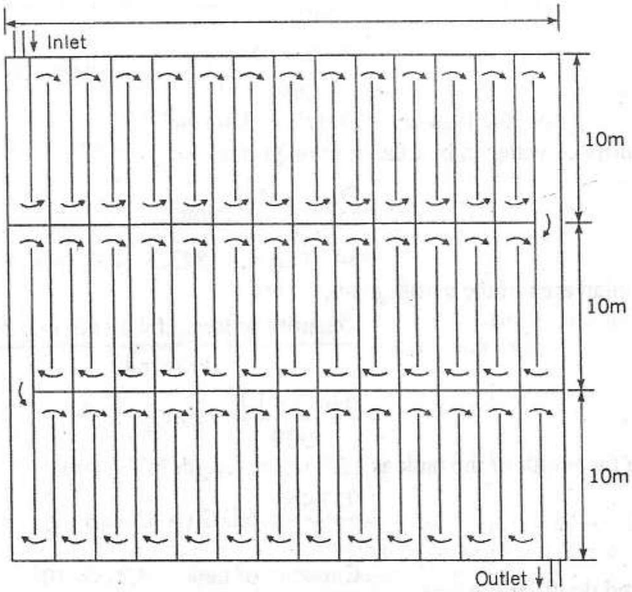


Fig. 12.11.

Assuming the sedimentation tank to have three channels of 10.0 *m* clear width.

The effective length of each channel

= Clear width of one channel – 2 × ½ clear opening at the end of baffle

$$= 10.0 - 2 \times \frac{1}{2} \times 0.75 = 9.25\ m$$

$$\therefore \quad \text{No. of channels required} = \frac{\text{Total flow length}}{\text{Effective length of each channel}}$$

$$= \frac{7.20}{9.25} = 77.837 = 78 \text{ No.}$$

∴ Keeping the thickness of baffle walls = 10 *cm*

∴ Length of the sedimentation tank having three channels

$$= 26 \times 0.75 + (26 - 1) \times 0.10$$

$$= 19.5 + 2.5 = 22.0\ m.$$

Example 12.4. *The population of a town is 1,00,000 and the average per capita demand is 135 litre/day/capita. Design the coagulation cum-sedimentation tank for the water works, supplying water to the town. The maximum demand may be taken as 1.5 times the average demand. Assume the detention periods of 5 hours and 30 minutes for settling tank and floc chamber respectively. Also assume the flow rate as 900 litres/hour/m² of plan area.*

Solution. **Design of Settling Tank**

Average daily requirement of the town

$$= 1{,}00{,}000 \times 135 \text{ litres}$$

$$= 13.5 \times 10^6 \text{ litres}$$

$$\text{Maximum daily requirement} = 1.5 \times 13.5 \times 10^6 \text{ litres}$$

$$= 20.25 \times 10^3 \text{ cu.m.}$$

Capacity of tank = Quantity of water treated in detention period of 5 hours

$$= \frac{20.25 \times 10^3}{24} \times 5 \; cu.m. = 4.219 \times 10^3 \; cu.m$$

Flow rate is given = 900 *litres/hour/m*^{2}

Now quantity of water to be treated in one hour

$$= \frac{20.25 \times 10^3}{24} \; cu.m.$$

$$= 843.7 \; cu.m. \; (= 843.7 \times 10^3 \; litres)$$

Required plan area of the settling tank

$$= \frac{\text{Quantity of treated water in one hour}}{\text{Flow rate}}$$

$$= \frac{843.7 \times 10^3}{900} = 937.45 m^2$$

Providing the width of the tank as 15.0 *m*, the length of the tank

$$= \frac{937.45}{15} = 62.49 \; (= 62.5 \; m)$$

$\therefore$ Required depth of the tank $= \frac{\text{Capacity of tank}}{L \times B} = \frac{4.219 \times 10^3}{62.5 \times 15} = 4.50 \; m$

Providing 50 *cm* for sludge storage, overall depth

$$= 5.0 \; m$$

$\therefore$ Provide settling tank of size 62.5 × 15 × 5.0 *m* **Ans.**

Design of Floc Chamber

In addition to the 62.5 *m* length of the settling tank, floc chamber is to be provided in the beginning. The depth of the floc-chamber may be assumed 2.5 *m*.

Detention period in the floc chamber

$$= 30 \text{ min. (given)}$$

The capacity of the floc chamber

= Flow required in 30 minutes

$$= \frac{20.25 \times 10^3}{24} \times \frac{30}{60} \; cu.m.$$

$$= 421.875 \; cu.m.$$

$\therefore$ Required plan area of the floc chamber

$$= \frac{\text{Capacity}}{\text{Depth}} = \frac{421.875}{2.5} = 168.75 \; m^2$$

Using the same width of 15 *m* of settling tank

Length of floc chamber $= \frac{168.75}{15} = 11.25 \; m$

$\therefore$ Size of floc chamber = 11.25 × 15 × 2.5 *m*. **Ans.**

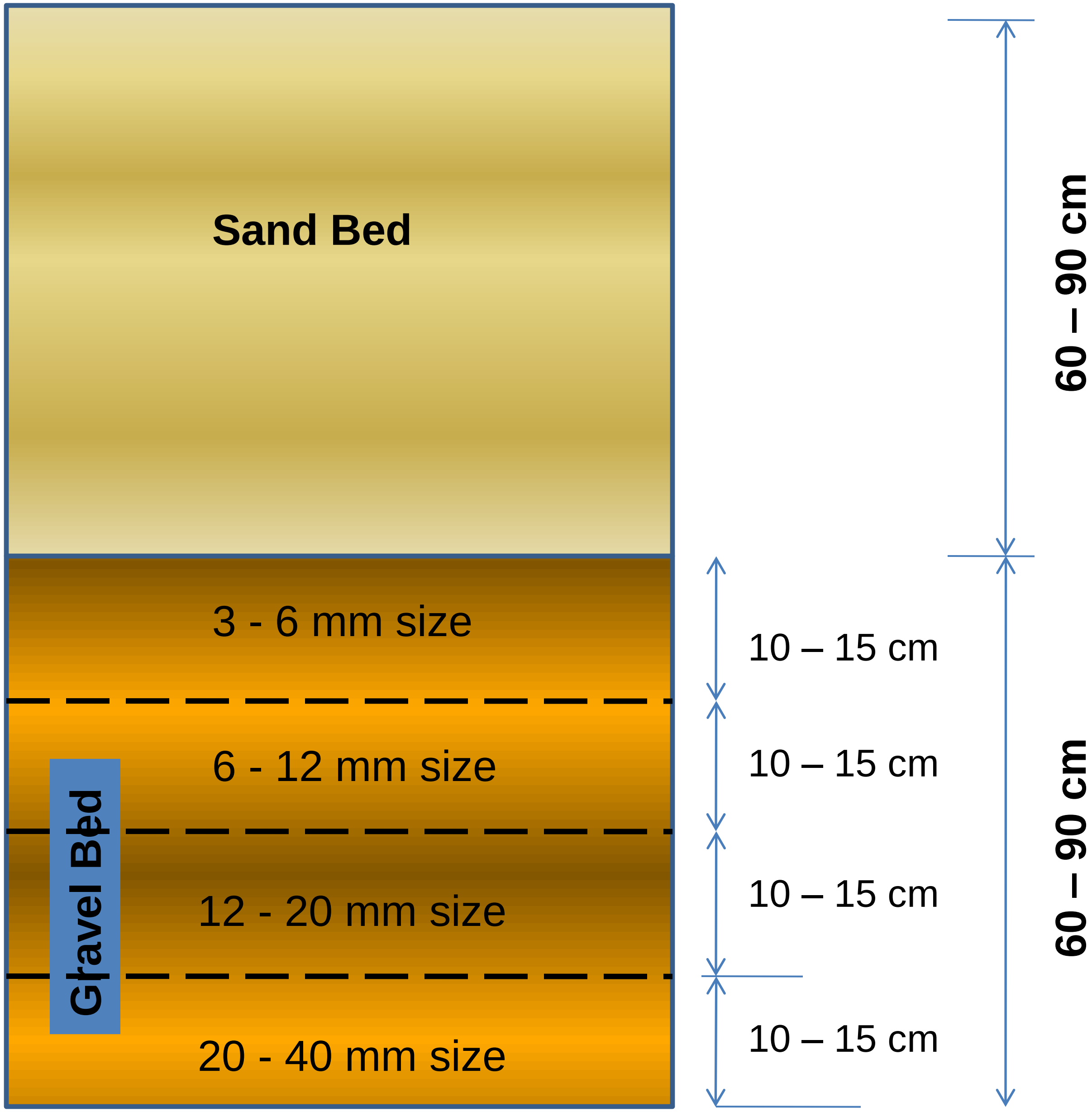
Slow Sand Filter - Components

- Exposure Tank – open water tight rectangular tank made of masonry or concrete
- Depth 2.5 to 3.5m
- Area 100 – 2000sq.m or more
- Bed Slope 1 in 100 towards drain

Filter Media

- Sand layers placed over gravel support
- Depth 90 – 110cm
- D_{10} 0.2 to 0.4mm
- Uniformity Coefficient 1.8 to 2.5
- Top 15cm – sand of finer quality

- Base material – gravel
- 30 – 75 cm thick of different sizes, placed in layers
- 3 to 4 layers of 15 – 20 cm depth
- Coarsest Gravel – Bottommost layer
  - Coarsest layer 40 to 65mm
  - Intermediate layer 20 to 40mm
  - top-most layer 3 – 6mm
- Under drainage system – Central drain and lateral drains
- Inlet and Outlet Arrangements


Sand Bed
60 – 90 cm
3 - 6 mm size
10 – 15 cm
6 - 12 mm size
10 – 15 cm
Gravel Bed
12 - 20 mm size
10 – 15 cm
60 – 90 cm
20 - 40 mm size
10 – 15 cm

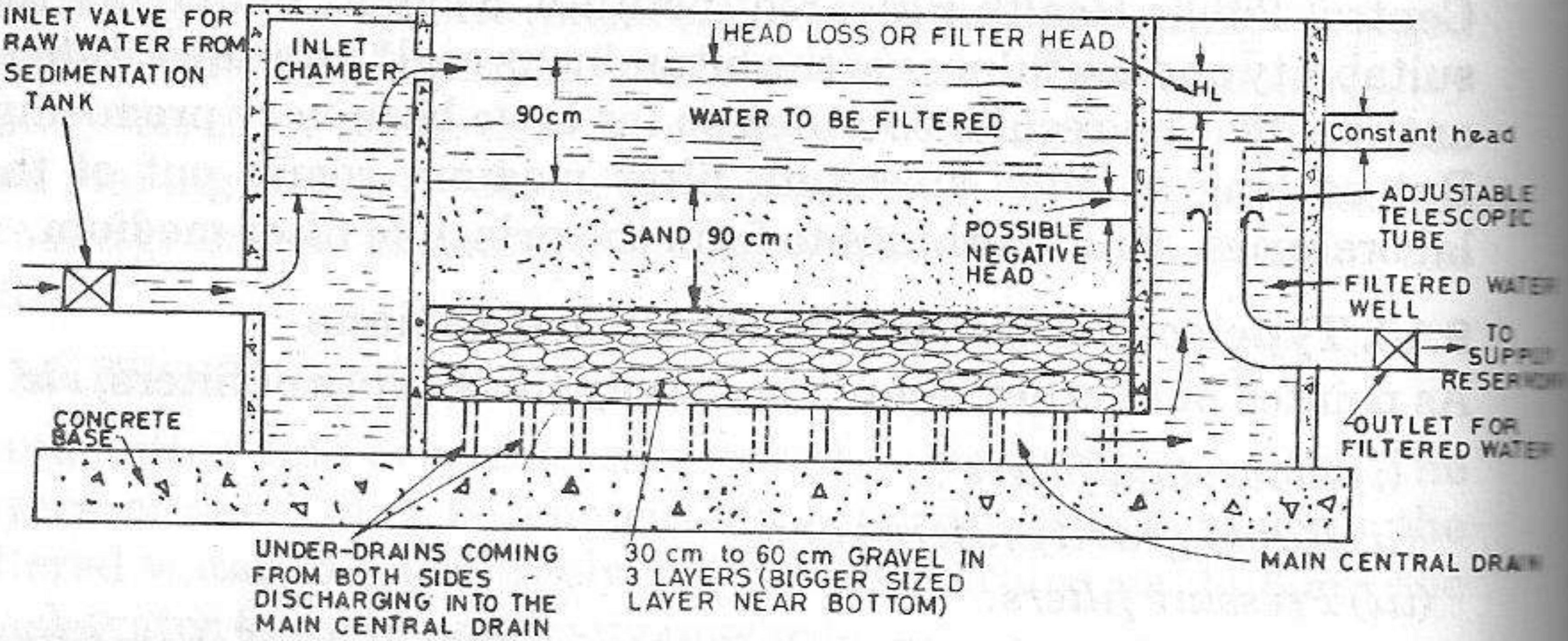


Fig. 9.27. Typical section of a slow sand filter.

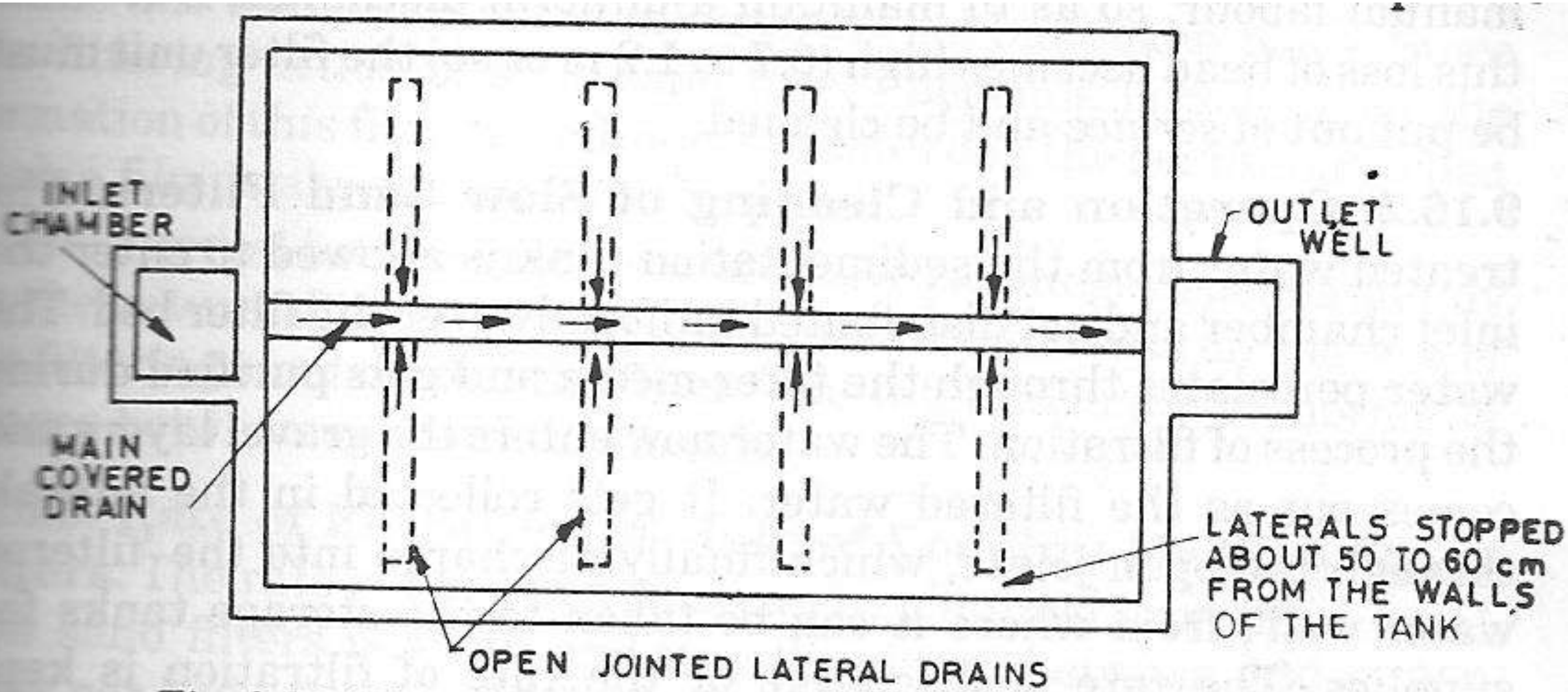


Fig. 9.28. Plan of under drainage system for slow sand filter.

Rapid Sand Gravity Filters

- These filters employ coarser sand, with effective size as 0.5mm or so.
- Yield as high as 30 times the yield given by slow sand filters.

Components of Rapid Sand Gravity Filter

- Enclosure Tank – open water-tight rectangular tank made of masonry or concrete.
  - Depth 2.5 to 3.5m
  - Area of filter 10 – 80 m^2
  - Number of Units at a filter plant (Morrell and Wallace)

$N = 1.22\sqrt{Q}$ where N = No. of Filter Units

Q = Plant capacity in MLD

- Atleast 2 filter units in any plant. For a plant of more than 9 MLD, no single unit should have a capacity > 1/4th of the capacity of that plant

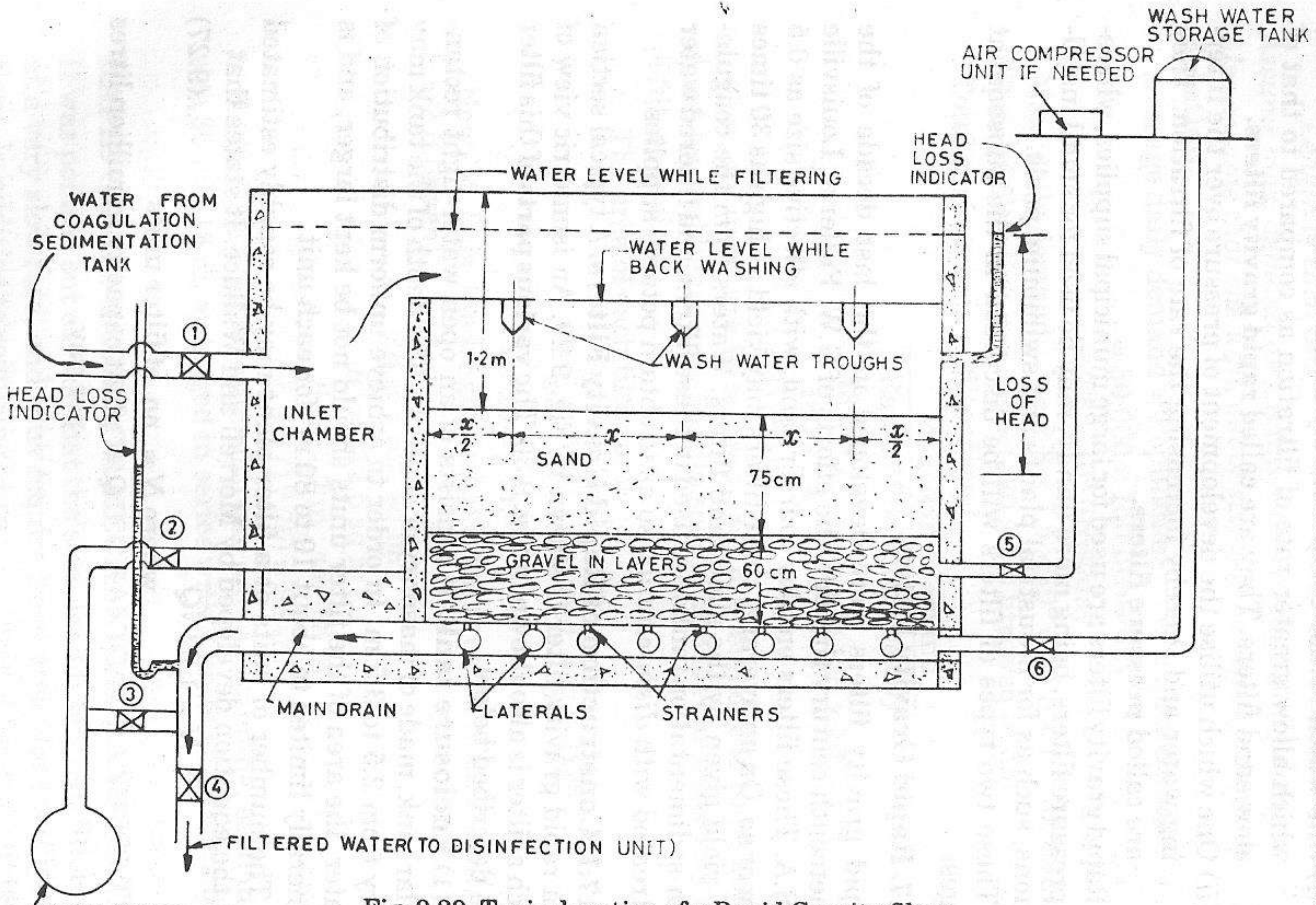


Fig. 9.29. Typical section of a Rapid Gravity filter.

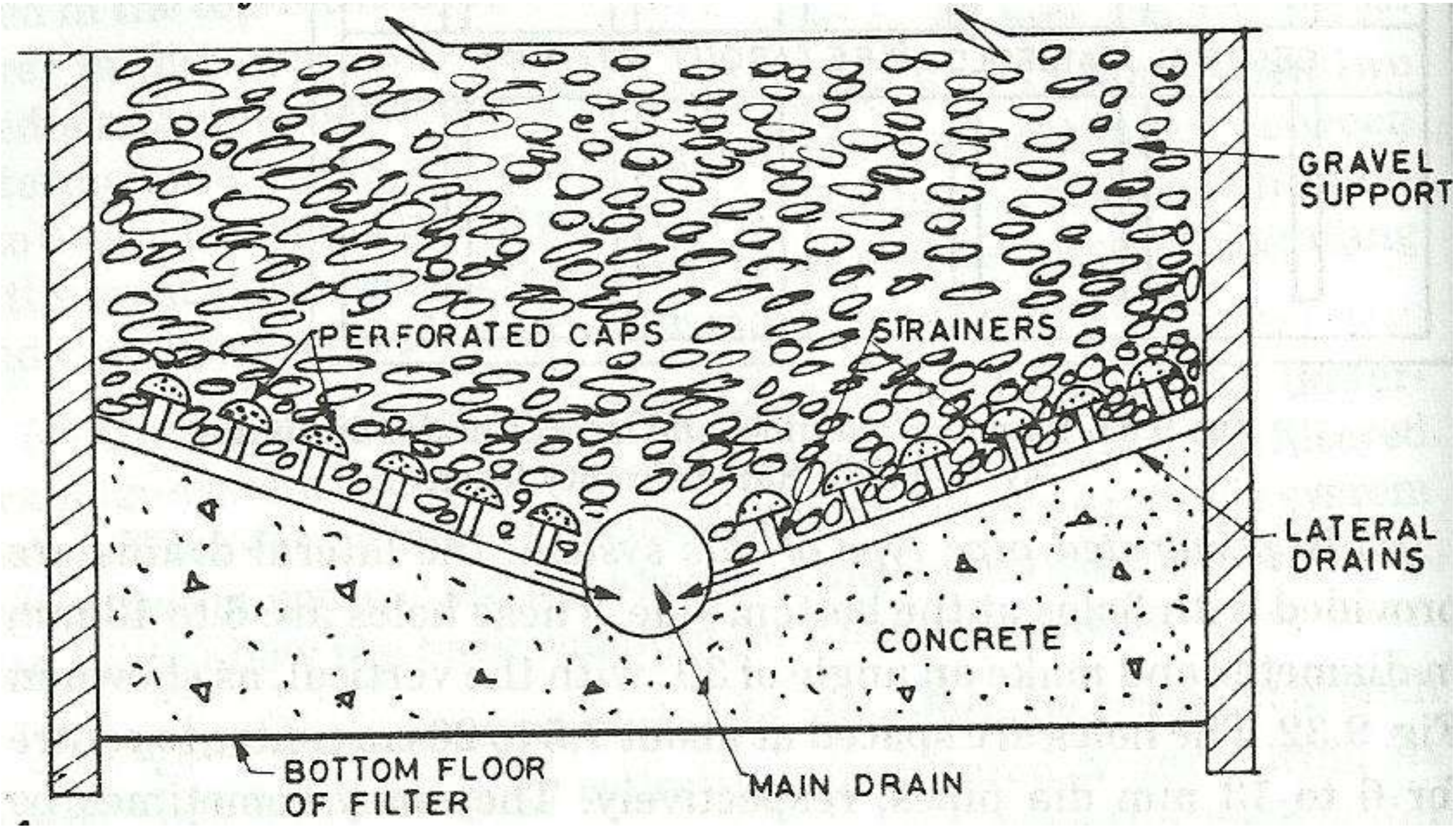
Filter Media

- Sand Layers 60 – 90 cm depth and placed over gravel support
- Effective size of sand (D10) 0.35 to 0.55mm
- Uniformity Coefficient (D60/D10) 1.3 to 1.7
  - Sand must be laid in layers. Finer Quality on top and coarser at the bottom
- Base Material – Gravel. It supports sand.
  - 60 to 90 cm thick of different sizes, placed in layers

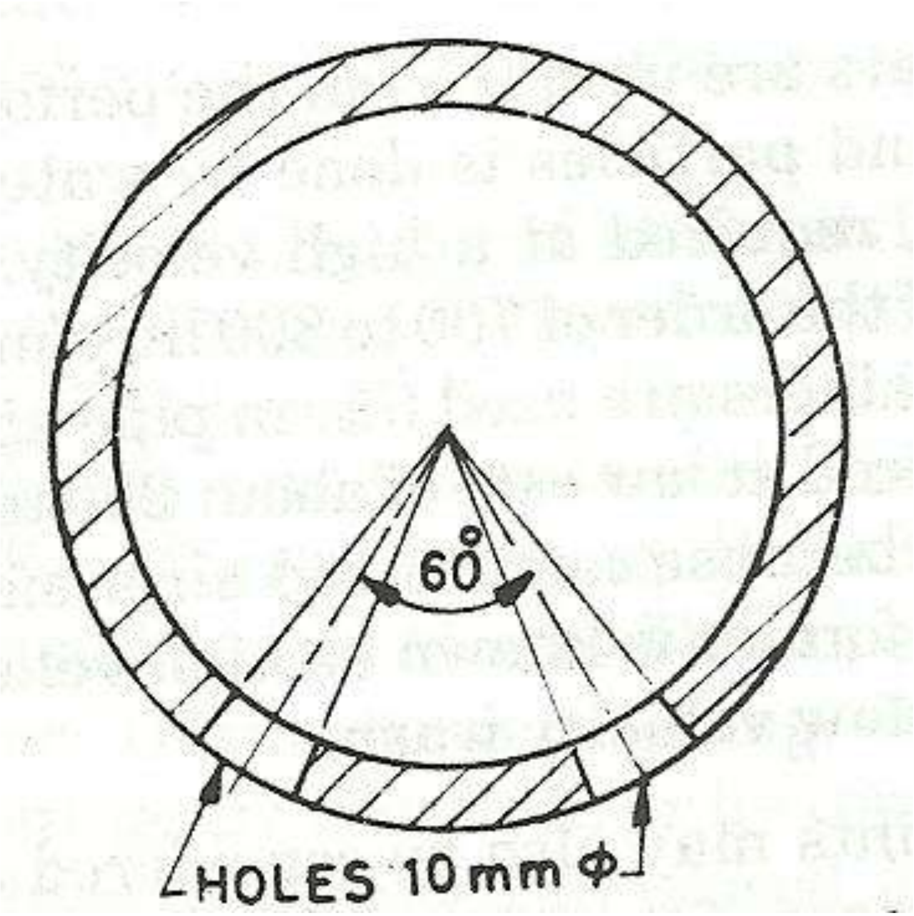
Under-drainage System

- Purpose served
  - Receive and collect filtered water
  - Allow backwashing for cleaning filter
  - Rate of back wash water 300 – 900 litres/min./m^2 of filter area depending on the design. Evenly and uniformly applied over the under portion of the gravel or sand bed.
- Various forms of Under-drainage systems
  - (i) Manifold and lateral system
  - (ii) Wheeler Bottom and
  - (iii) Porous Plate Bottom

Section of the ‘manifold and lateral’ system, in which laterals are provided with strainers



Cross-section of a lateral pipe drain with perforations



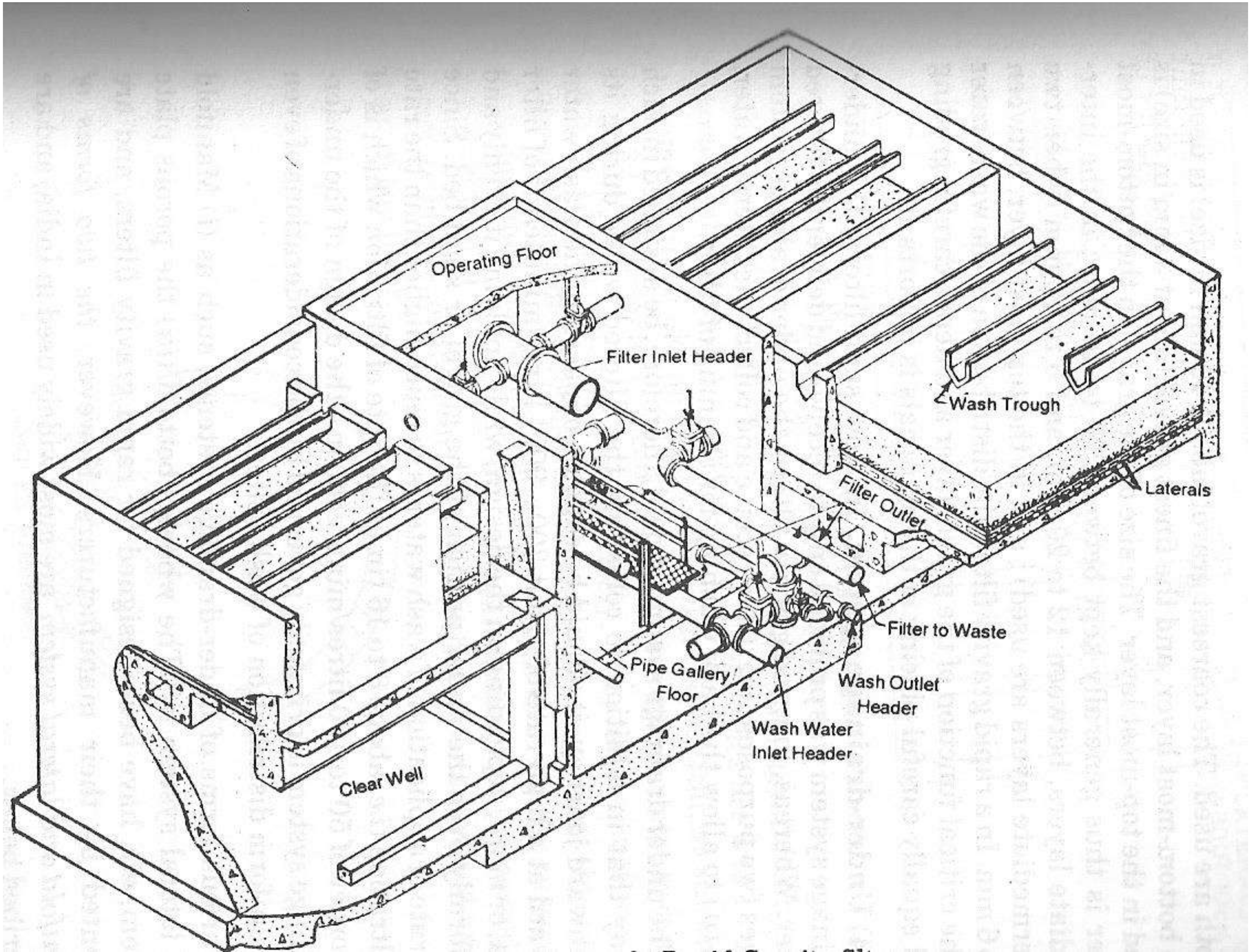
Other Appurtenances

- Wash Water Troughs (or gutters)
  - Square, V shaped or semi-circular
  - Made of cast iron, concrete, steel or wrought iron
- Air Compressor for backwashing – compressed air of 600 to 800 litres/min/sq.m for about 4 mins
- Rate Controller – fitted at outlet end of each filter unit. Most common type is venturi-rate controller
- Miscellaneous Accessories

Backwashing

- When sand becomes dirty (indicates excessive loss of head) the filter must be cleaned and washed.
- Entire backwashing of filters and re-maintenance of filtered supplies takes about 15 mins
- Amount of water required for backwashing vary from 2 – 5% of total amount of water filtered. Frequently washed every 24 to 48 hrs.

Isometric View of a Rapid Gravity Filter



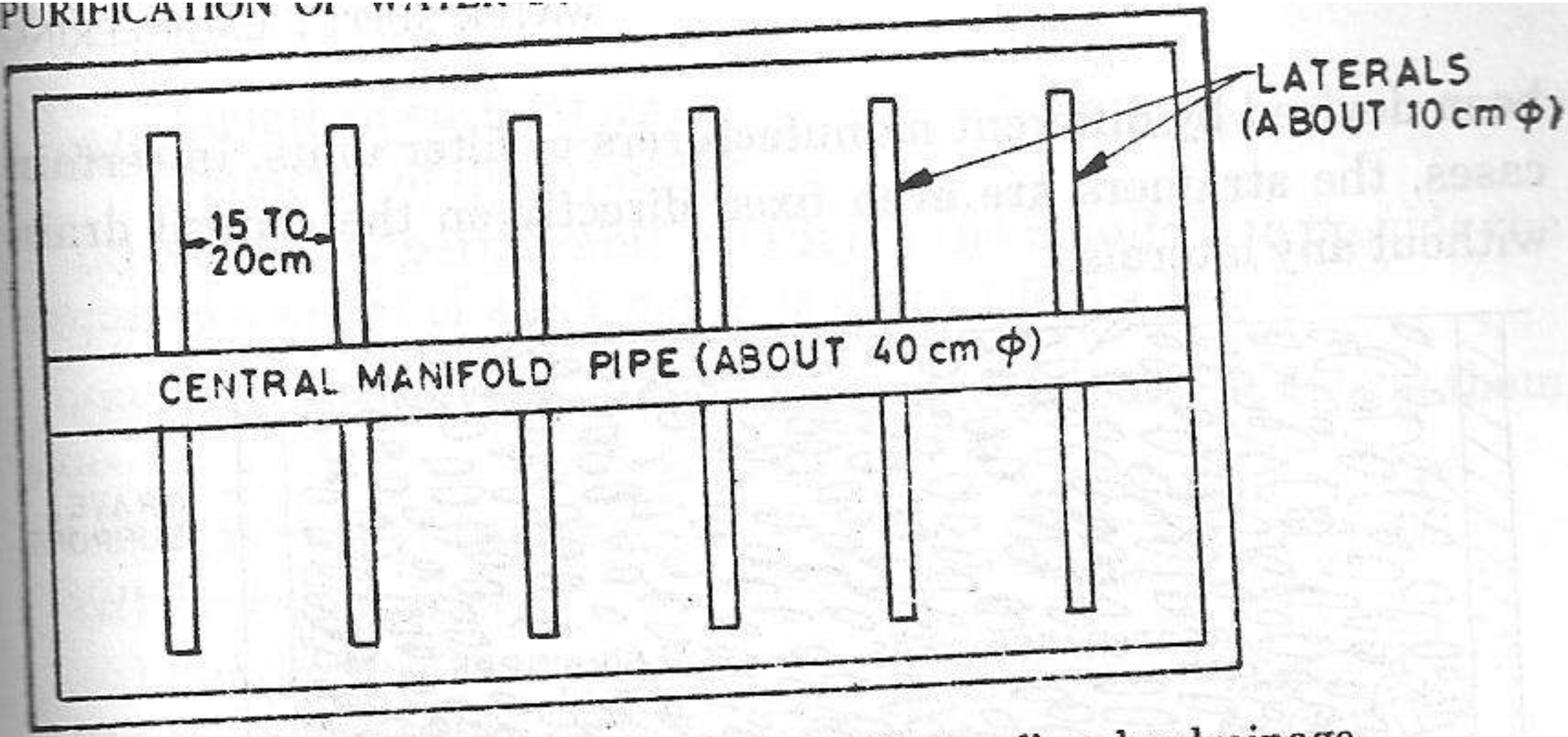
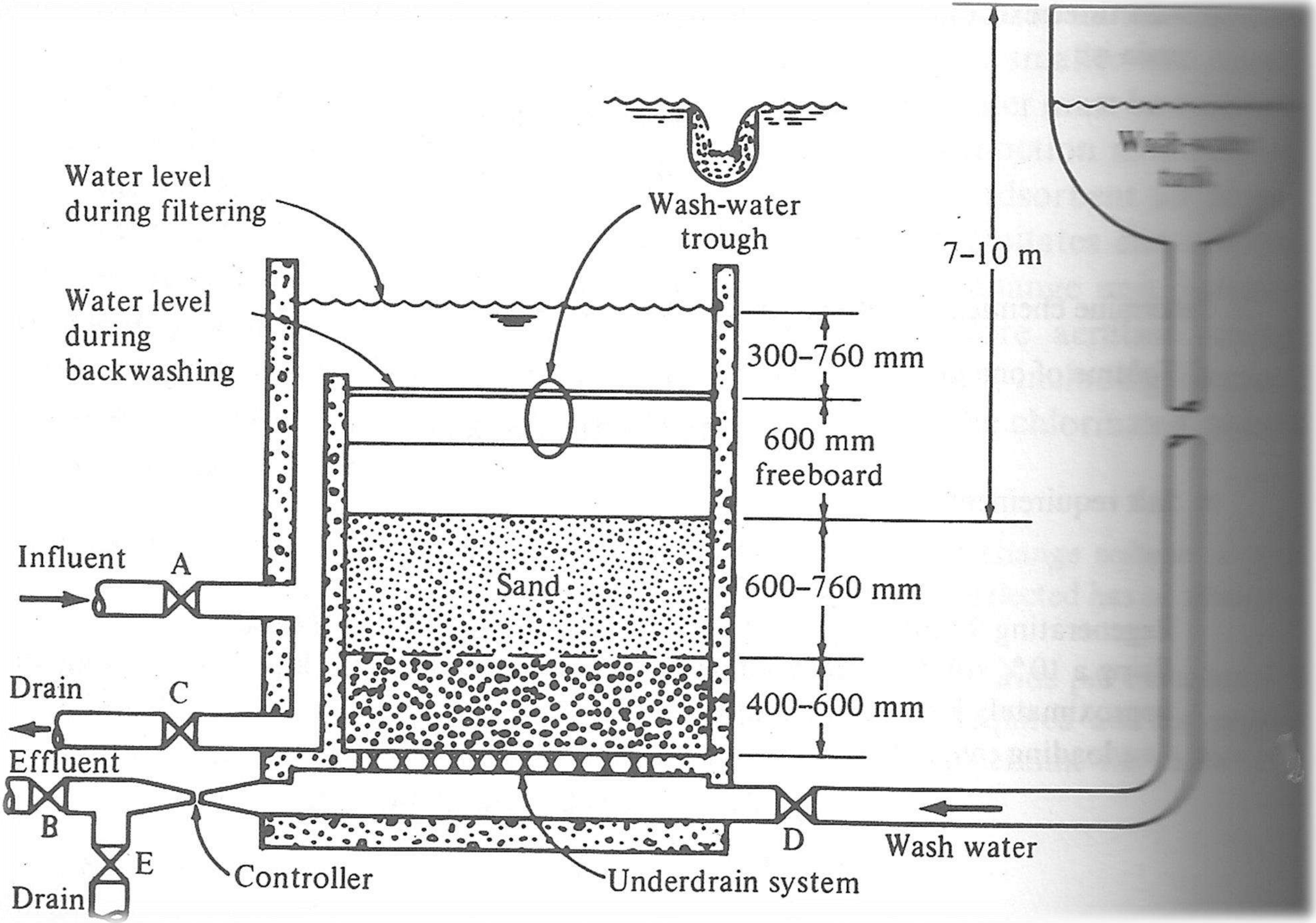


Fig. 9.31. Plan of a 'manifold and lateral' under drainage tem for a Rapid Gravity filter.


Water level during filtering
Wash-water trough
7–10 m
Wash-water tank
Water level during backwashing
300–760 mm
600 mm freeboard
Influent
A
Sand
600–760 mm
Drain
C
400–600 mm
Effluent
B
E
Drain
Controller
D
Wash water
Underdrain system

Operational Troubles in Rapid Gravity Filters

- Formation of Mud Balls
- Cracking of Filters

Example:

Design the approximate dimensions of a set of rapid gravity filters for treating water required for a population of 50,000; the rate of supply being 180 litres per day per person. The filters are rated to work 5000litres per hour per sq.m. Assume whatever data are necessary, and not given.

Solution:

Assuming given rate of supply = average demand

Maximum daily demand = 1.8 x Average Demand

= 1.8 x 180 L x 50,000

= 16.2 Million Litres

Water demand per hour (ignoring time lost in cleaning)

= $(16.2 \times 10^6)/24 = 675 \times 10^3$ L/hr.

Rate of Filtration = 5000 Litres/hr/sq.m

Area of Filter Beds Required = Water Demand/Rate of Filtration

= $(675 \times 10^3)/5000 = 135$ sq.m

Since two units are required to be designed,

area of each unit = 135/2 = 67.5 sq.m

Assuming L = 1.5B, we have

$1.5B^2 = 67.5$ Therefore B = 6.70m

Choose 6.75 m width and 10m length. Hence 2 units of size 10m x 6.75 m are required. One additional unit as stand-by may also be provided for breakdowns, repairs or cleaning operations.

T2 Solution:

Assuming given rate of supply = average demand

Maximum daily demand = 1.8 x Average Demand
= 1.8 x 200 L x 50,000
= 18.0 Million Litres

Rate of Filtration = 300 Litres/day/sq.m

Area of Filter Beds Required = Water Demand/Rate of Filtration
= $(18\text{x } 10^6)/300$ = 60000 sq.m

Since 5 units are required to be designed,
area of each unit = 60000/5 = 12000 sq.m

Assuming L = 1.5B, we have
$1.5B^2 = 12000$ Therefore B = 89.44m
Choose 89.4 m width and 134.2 m length. Hence 5 units of size 134.2m x 89.4 m are required. One additional unit as stand-by may also be provided for breakdowns, repairs or cleaning operations.

penetrate deeply into the bed, even into the gravel, and to impair both the washing of the filter and the efficiency of filtration.

Cleaning Rapid Sand Filters. It may be necessary, at times, to clean the filter sand or the gravel to overcome difficulties from mud balls, cracking and other causes. Usually cleaning serves only as a temporary expedient. The cause of clogging should be corrected. Various chemical treatments have been used, such as caustic soda, sulphuric acid, hydrochloric acid, soda ash, sulphur dioxide, and chlorine. Chlorine may be used where the material to be removed is of biologic origin; copper sulphate may be effective against algae ; caustic soda for alum deposits and organic material ; hydrochloric acid or carbonic acid against calcium carbonate, and sulphur dioxide for dissolved iron, manganese, or alum. H.T.H. (a product of calcium hypochlorite), Perchloron, or Clorox may be used in small filters. Liquid chlorine may be used economically on large filters. Chlorine may be applied as a solution at a concentration of about 50 ppm of available chlorine. The solution should stand in contact with the sand for 4 to 24 hr or longer. Caustic soda can be applied as a solution or, preferably, by spreading 5 to 12 kg of the flaked material per square meter of the filter surface. The soda is dissolved by allowing the wash water to rise 10 to 12 cm above the sand.

Sulphur dioxide has been used, as a patented process, to clean filter sand coated with iron or manganese. A 1 to 2 per cent aqueous solution of sulphur dioxide is pumped into the filter and circulated through the underdrains and the filter for about 24 hr. The method is not recommended where the sand grains are coated with calcium carbonate because of the possibility of cementing the grains together with calcium sulphate or sulphite which might be formed.

Before acids or other chemicals are applied to a filter, samples of the sand to be cleaned should be tested in the laboratory with the chemical to be used. Some concentrations and periods of application of chemicals that have been used with varying degrees of success are indicated in Table 12.1. After chemicals have been used, the filter should be thoroughly washed to remove the loosened substances and to remove the chemicals.

Table 12.1. Chemicals used for Cleaning Filters

Chemical	*Amount of chemical used or strength of solution*	*Period of contact, hr.*
Caustic soda and soda ash	NaOH 8.5 kg/m^2 Na_2CO_3 25 kg/m^2	48
Caustic soda	4.5 kg/m^2	22-72
Soda ash	25 g/l	24-36
Sulphuric acid	0.45 kg/sq m	4
Chlorine	50 ppm	18
Sulphur dioxide	2 per cent aqueous soln.	16

Mechanical methods of cleaning filters in place include raking, scraping, the surface, use of a high-pressure water jet, ejecting the sand, and the use of steam. In some cases the sand may be removed either by hand tools or by a hydraulic ejector and cleaned outside the filter. All these methods have been found helpful in overcoming unexpected difficulties but they should not be depended on as routine procedure. The cause of the difficulty should be discovered and remedied.

Maintaining Filter Runs. The length of the filter run is affected by the character of the influent water, the rate of filtration, the loss of head permitted between washings, and the thoroughness of each wash. The character of the water that is applied to the filter is under the control of the operator in so far as he is able to affect it in the coagulating basin through the application of chemicals, more satisfactory conditioning, or a change in the period of subsidence. Only experience with the particular water being treated can serve as a guide in any particular plant but, in general, an increase in turbidity or in certain microorganisms will shorten filter runs. Experiments reported by Baylis shows the length of filter run at a 80 lpm per sq m rate to be about 2.3 times the length at a 200 lpm per sq m rate, *i.e.* about 9 per cent more water passes through a filter between washings at the higher rate of filtration.

The runs may be lengthened by a more thorough pretreatment of the water or by diminishing the rate of filtration. Where too high a loss of head is permitted between washings, the length of run may be diminished by air binding resulting from the accumulation of gases thrown out of solution as a result of the reduced pressure in the filter. A remedy is the reduction of the loss of head between washings. In some plants the length of run may be increased by "bumping" the filter occasionally between washings. A filter is bumped by preparing it, in the ordinary manner, for washing and turning on the wash water for a moment to break up the schmutzdecke.

Loss of Sand. Sand is carried out of the filter by wash water usually as a result of the increase in the size of the sand grains by material that adheres to them, reducing the specific gravity of the clump and exposing more surface to the rising wash water. The loss of sand is particularly noticeable as the temperature of the wash decreases because of the increased lifting power of the water with increasing viscosity. Since the cause of the difficulty is dirty sand, to decrease the rate of wash may serve ultimately either to aggravated the condition or to cause other troubles in the filter operation. The ultimate effective remedy is to clean the sand.

Sand Incrustation. Increase in the size of sand particulars in a filter is due to the quality of the raw water resulting in the deposition of a scale or of a gelatinous layer on the surface of the sand. A hard scale of calcium carbonate may form in a lime-soda softening plant as a result of incomplete recarbonation, dissolved calcium carbonate being thrown out of solution through the absence of sufficient dissolved carbon dioxide. The increase of carbon dioxide may also serve to dissolve manganese that has deposited on the sand grains. The remedy for the condition lies in the control of the quality of the water being placed on the filter.

Displacement of Gravel. Displacement of the gravel in a filter may result from a break in the underdrainage system, from uneven distribution of the wash water, or from the use of too high a rate of wash. The condition aggravates the unevenness of the wash-water distribution, mixes the gravel with the sand, and may permit the escape of sand into the underdrains.

The condition may be remedied by diminishing the rate of filtration or by repairing or renovating the underdrainage system. The escape of sand into the underdrains is dangerous, because its remedy is expensive, usually involving the shutdown of the filter and the tearing up and replacing of the underdrain pipes. Gravel that has become misplaced in a filter may sometimes be replaced without shutting down the filter by drawing a straightedge across the surface of the gravel while wash water is running through the filter. The fine gravel, which is almost suspended in the rising wash water, is easily shifted about in this manner.

Results. The turbidity of water obtained by rapid gravity filter is less than 1 mg/l. The bacterial removal is about 90% which is lower than slow sand filter. But this filter is more efficient in removing colour and odour.

Though the bacterial removed efficiency is low, rapid gravity filter is more widely used the reason being high rate of filtration possible and much lesser space is required for filtration units.

12.5. COMPARISON OF SLOW SAND AND RAPID GRAVITY FILTERS

	Item	*Slow Sand*	*Rapid Gravity*
1.	Rate of filtration	2400–4800 lit/sq. m/day	120,000–150,000 lit/sq. m/day
2.	Size of unit	2000–4000 sq m in area	4 × 5 to 8 × 10 sq. m
3.	Depth of filter media	Depth of gravel 25 to 30 cm whereas sand 80 to 100 cm.	Gravel 45 to 60 cm and sand 60 to 75 cm.
4.	Size of sand	Effective size 0.20 to 0.35 mm. Uniformity coefficient 3 to 5	Effective size 0.5 to 0.7 mm. Uniformity coefficient 1.3 to 1.7
5.	Grain size distribution	Uniform	Stratified with smallest grain at the top
6.	Under drainage system	Open jointed pipes covered with blocks	(*i*) Manifold and lateral (*ii*) Wheeler bottom (*iii*) Diffuser plate
7.	Loss of head	10 cm initially to 1 metre finally	20 cm initially to 3 m finally
8.	Length of run	20 to 40 days	24 to 48 hours
9.	Penetration of suspended matter	Superficial	Deep
10.	Method of cleaning	Scraping and washing	Back washing
11.	Water required for cleaning	0.2 to 0.6% of water filtered	2% to 6% of water filtered.
12.	Preparatory treatment	Plain-sedimentation	Flocculation and settling
13.	Cost of construction	Higher	Low
14.	Cost of operation	Lower	Higher
15.	Depreciation of plant	Lower	Higher

Example 12.1. *Design a 15 × 10^6 l.p.d. water treatment plant with rapid gravity sand filter. Assume suitable design parameters.*

Solution.

(*a*) *Sand bed.* Assume the filter bed with stratified sand layers. Provide a depth of sand as 60 cm and the effective size of sand as 0.5 mm. The sand layers broadly consist in the following manners.

(*i*) top 20 cm layer with 0.50 mm down sand.
(*ii*) next 20 cm layer with 0.60 mm down sand.
(*iii*) bottom 20 cm layer with 0.75 mm down sand.

Sometimes a layer of sand of 0.80 to 2.0 mm (torpedo sand) is placed below the above mentioned bottom layer and above the supporting gravel bed.

(*b*) *Gravel bed.* The total thickness of gravel provided as 50 cm. Sizes of gravel layer are given below :

(*i*) Top 5 cm layer with 2 mm gravel
(*ii*) 2nd 5 cm layer with 5 mm gravel.
(iii) 3rd 10 cm layer with 10 mm gravel.
(*iv*) 4th 10 cm layer with 15 mm gravel.
(*v*) 5th 10 cm layer with 20 mm gravel.
(*vi*) 6th (bottom) 10 cm layer with 25 mm gravel.

(*c*) *Size of Number of Units*

Total flow $Q = 15 \times 10^6$ litre per day

Assume permissible hydraulic loading = 150000 lit./sq. m/day

Filter area required $= \dfrac{15 \times 10^6}{150000} = 100$ sq. m.

Providing each size of filter unit 4.4 m × 5.7 m.

No. of filters beds required $= \dfrac{100}{4.4 \times 5.7} = 3.98 = 4$ (say)

Provide 2 numbers of additional beds to take care of time required for break-down, repair, maintenance, backwashing etc. So total numbers of filter beds = 6.

(*d*) *Design of under drainage system.* Provide "Central Manifold with laterals" under-drainage system. Assuming size of perforation as 9 mm and area of perforation is 3×10^{-3} times filter area,

Total area of perforations $= 3 \times 10^{-3} \times 4.4 \times 5.7 = 0.0753 \text{ m}^2 = 753 \text{ cm}^2$

Total numbers of perforations of 9 mm dia

$$= \frac{753}{\frac{\pi}{4}(0.9)^2} = 1183$$

Total cross sectional area of laterats= 3 × area of perforation $= 3 \times 753 = 2259 \text{ cm}^2$

Area of central manifold $= 2 \times$ area of laterals $= 2 \times 2259 = 4518 \text{ cm}^2$

Diameter of central manifold $= \sqrt{\dfrac{4518 \times 4}{\pi}} = 75.8$ cm.

Provide a commercially available diameter of 750 mm.

Assume a spacing of 15 cm for laterals,

The number of laterals $= \dfrac{2 \times 5.7 \times 100}{15} = 76$

Cross sectional area of each laterals $= \dfrac{2259}{76} = 29.72 \text{ cm}^2$

Diameter of laterals $= \sqrt{\dfrac{29.72 \times 4}{\pi}} = 6.15$ cm

Provide lateral of diameter 75 mm.

No. of perforations per laterals $= \dfrac{1183}{76} = 16$ (say)

Length of laterals $= \dfrac{1}{2}$ [Width of filter – dia of manifold]

$= \dfrac{1}{2}[4.4 - 0.75] = 1.82$ m.

Spacing of perforations $= \dfrac{1.82 \times 100}{16} = 11.35$ cm

Provide 16 perforations of 9 mm dia @ 113 mm c/c.

(*e*) *Compute wash water trough*

Quantity of wash water required = 6% of water filtered

$= 0.06 \times 15 \times 10^6$ liters: $= 900 \text{ m}^3$

Quantity of wash water per filter bed $= \dfrac{900}{6} = 150 \text{ m}^3$

Assuming back washing time = 10 mins.

Rate of flow $= \dfrac{150}{10 \times 60} = 0.25 \text{ m}^3/\text{s}$.

Provide a spacing of 1.5 m for wash water troughs which will run parallel to the longer dimension of the filter unit.

Number of troughs $= \dfrac{4.4}{1.5} = 3$

Discharge is given by $Q = 1.376 \times bh^{3/2}$

Now if $b = 0.4$ m, $h = 0.3$ m.

Assume a free board of 0.1 m.

Provide 3 troughs of 0.4 m × 0.4 m deep.

(*f*) Depth of Filter Box

Depth of filter box = Depth of (under drains + gravel + sand + water depth + free board)

$$= 0.8 + 0.50 + 0.60 + 1.5 + 0.3 = 3.70 \text{ m.}$$

12.6. FILTER HYDRAULICS

A number of hydraulic processes will be involved in the filtration units. They will broadly cover the (*i*) Influent Feeding System (*ii*) Flow through sand bed (*iii*) Collection of filtered Water (*iv*) Control of Rate of Flow (*v*) Back-Washing of Filter (*vi*) Disposal of Wash Water and (*vii*) Control of Rate of Flow. Excepting the flow through sand bed and backing washing hydraulics, the basic principles of other processes have been covered elsewhere, like pressure flow through pipes, gravity flow through weir etc. So under this head the only flow through sand bed and back washing hydraulics will be discussed.

12.6.1. Flow through Sand Bed. This will take place in actual filtration process by which the water is cleaned and the back washing operation by which the filter is cleaned. Many workers and researchers have worked on it. Here the mattered dealt by flow through packed bed and can be analyzed by classic hydraulic theory. Carmen modified the Darcy-Weisbach equations for head loss in a pipe to reflect conditions in a bed of porous media of uniform size. The development of this equation is presented in several texts like "Water Supply and Pollution Control" by Clark and Viessman, "Elements of Water Supply and Wastewater Control" by Clark and Viessman, "Elements of Water Supply and Wastewater disposal by Fair, Geyer and Okun" "Waste water Treatment" by Sunderstorn and Klei etc and will not be repeated here. The resulting equation, known as the Carmen-Kozeny equation is given by :

$$h_f = \frac{fL\,(1-e)\,V_s^{\,2}}{e^3\,gd}$$

where h_f = friction loss through bed of particles of uniform size d, m

L = depth of filter, m

e = porosity of bed

V_s = filtering velocity, *i.e.* the velocity of water just above the bed (total flow Q to the filter divided by the area of filter), m/s

g = acceleration due to gravity, m/s^2

d = diameter of filter media grains, m

The remaining terms of it is a friction factor related to the coefficient of drag around the particles. In the usual range of filter velocities (laminar flow) this is given by :

$$f = 150\,\frac{(1-e)}{R} + 1.75$$

where R = Reynolds number $= \dfrac{\phi\rho V_s\, d}{\mu}$.

and ρ and μ are the density and dynamic viscosity respectively of water. The units of ρ are kilograms per cubic meter and the units of μ are newton-seconds per square meter. The shape factor ϕ ranges from 0.75 to 0.85 for most filter media.

Example 12.2. *Determine head loss across a bed of uniform size particles, when water is passed through sand bed at a filtering velocity of 0.5 m/h. The sand grains are 0.2 mm in diameter with shape factor of 0.85 and a specific gravity of 2.65. The depth of filter bed is 1.0 m and the porosity is 0.5. Determine the head loss through the filter bed.*

(Dynamic viscosity of water

$$\mu = 1.002 \times 10^{-3} \frac{N.S.}{m^2} = 1.002 \times 10^{-3}\ Kg/m.s)$$

Solution.

Velocity through filter = 0.5 m/h = 1.39×10^{-4} m/s.

Reynolds number

$$R = \frac{0.85 \times 1000 \times 1.39 \times 10^{-4} \times 0.2 \times 10^{-3}}{1.002 \times 10^{-3}} = 0.02774 < 1 \qquad \text{(laminar flow confirmed)}$$

$$\text{Friction factor } f = 150\,\frac{(1-0.5)}{0.02774} + 1.75 = 2707.15$$

$$h_f = \frac{2707.15 \times (1-0.5) \times 1.39 \times 1.39 \times 10^{-4} \times 10^{-4}}{0.5 \times 0.5 \times 0.5 \times 9.81 \times 0.2 \times 10^{-3}} = 0.1066 \text{ m} \quad \textbf{Ans.}$$

Modified equation for non-uniform medium is applicable for sand with varying sizes in stratified bed. From sieve analysis of medium the weight fraction x_i between adjacent sieve sizes is determined. The average particle size d_i is assumed to be halfway between the sieve sizes. The depth of the particles between adjacent sieve sizes can be taken as $x_i\,L$ and the equation can be written as follows :

$$h_f = \frac{L\,(1-e)\,V^2}{e^3\,g} \sum \frac{f_i\,x_i}{d_i}$$

The above equation assumes that the bed is stratified by size but the porosity is constant.

Example 12.3. *Determine the head loss across a bed of non-uniform, stratified particles. Water is passed through filter bed at 1.2×10^{-3} m/s. The bed is 0.75 m deep with sand sp. gr. 2.65 stratified. Porosity and shape factors are 0.4 and 0.85 respectively. Size distribution of sand is given below :*

Particle size-range, mm		*Average size d_i, mm*	*Mass factor in size range x_i*
Passing	*Retained*		
	1.41	1.41	0.01
1.41	0.84	1.13	0.11
0.84	0.71	0.78	0.20
0.71	0.60	0.66	0.32
0.60	0.50	0.55	0.21
0.50	0.42	0.46	0.13
0.42		0.42	0.02

Solution. $$R = \frac{0.85 \times 998.2 \times 1.2 \times 10^{-3}}{1.002 \times 10^{-3}}\,d_i = 1016\,d_i$$

$$f_i = \frac{150\,(1-0.4)}{1016\,d_i\,.} + 1.75 = \frac{0.09}{d_i} + 1.75$$

Determine $f_i \dfrac{x_i}{d_i}$ as follows :

Particle size, d_i, m	x_i	f_i	$f_i \dfrac{x_i}{d_i}$, 1/m
1.41×10^{-3}	0.01	65.5	465
1.13×10^{-3}	0.11	81.5	7924
0.78×10^{-3}	0.20	117.1	30026
0.66×10^{-3}	0.32	138.1	66958
0.55×10^{-3}	0.21	165.4	63153
0.46×10^{-3}	0.13	197.4	55787
0.42×10^{-3}	0.02	216.0	10286

$$\sum f_i \times \frac{x_i}{d_i} = 234{,}599$$

Calculate h_f: $\quad h_f = \dfrac{0.75 \times (1 - 0.4) \times (1.2 \times 10^{-3})^2}{0.4^3 \times 9.81} \times 234{,}599 = 0.24$ m.

12.6.2. Backwash Hydraulics. Backwashing of granulas-medium filters is accomplished by reversing the flow and forcing water move upward through media. To clean the interior of the bed, it is necessary to expand it so that the granules are no longer in contact with each other, thus exposing all surfaces for cleaning. To hydraulically expand a porous bed, the head loss must be at least equal to the buoyant weight of the particles in the fluid. For a unit area of filter this is expressed by

$$h_{fb} = L\,(1 - e)\,\frac{\rho_1 - \rho}{\rho}$$

where h_{fb} = head loss required to initiate expansion, m

L = bed depth, m

e = porosity of medium

ρ_1 = density of the medium, kg/m^3

ρ = density of water, kg/m^3

The head loss through an expanded bed is essentially unchanged because the total buoyant weight of the bed is constant. Therefore :

Weight of the packed bed = Weight of bed fluidized

$$L\,(1 - e)\,\frac{\rho_1 - \rho}{\rho} = L_{fb}\,(1 - e_{fb})\,\frac{\rho_1 - \rho}{\rho}$$

or,
$$L_{fb} = L\,\frac{(1 - e)}{(1 - e_{fb})}$$

where L_{fb} = the depth of fluidized bed

e_{fb} = the porosity of fluidized bed

e_{fb} is a function of the terminal settling velocity of the particles and the backwash velocity. An increased in backwash velocity will result in a grater expansion of the bed. The relation between backwash velocity and particle settling velocity is given by

$$e_{fb} = \left(\frac{V_B}{V_t}\right)^{0.22}$$

where V_B is the backwash velocity and V_t is the terminal settling velocity. The depth of fluidized bed and backwash velocity for a given size medium with know V_t can be related by

$$L_{fb} = \frac{L\,(1-e)}{1-\left(\frac{V_B}{V_t}\right)^{0.22}}$$

This equation can be modified for stratified bed of non uniform particles where

$$X_i L = L_i$$

The expansion of this layer is given by

$$L_{fb,\,i} = X_i\,\frac{L\,(1-e)}{1-\left(\frac{V_B}{V_{t,\,i}}\right)^{0.22}}$$

The total expansion of the sum of the individual layers

$$L_{fb} = L\,(1-e)\sum\frac{X_i}{1-\left(\frac{V_B}{V_{t,\,i}}\right)^{0.22}}$$

The total expanded depth generally should range from 120 to 155 percent of the unexpanded depth and expanded porosity will range from 0.65 to 0.70.

Example 12.4. *Find the expanded depth of a uniform medium. The filter medium described in Example 12.2 is to be expanded to a porosity of 0.7 by hydraulic backwash. Determine the required backwash velocity and the resulting expanded depth.*

Solution. The terminal setting velocity for the medium is first calculated from Stoke's Law

1. $$V_t = \frac{9.81\,(2650-1000)\times(0.2\times10^{-3})^2}{18\times1.002\times10^{-3}} = 0.0359 \text{ m/s}$$

2. Check Reynold's number

$$R = \frac{0.85\times0.0359\times0.2\times10^{-3}\times1000}{1.002\times10^{-3}} = 6.0908$$

3. $$C_D = \frac{24}{R}+\frac{3}{R^{1/2}}+0.34 = 5.496$$

4. $$V_t^2 = \frac{4}{3}g\,\frac{(\rho_1-\rho)\,d}{C_D\,\rho} = \frac{4}{3}\times\frac{9.81\,(2650-1000)\;0.2\times10^{-3}}{1.002\times10^{-3}} = 7.85\times10^{-4}$$

$\therefore$ $V_t = 0.028$ m/s.

Now repeat steps 2, 3 and 4 putting $V_t = 0.028$ m/s,

We get, $R = 4.755 \quad C_D = 6.7$

$V_t = 0.0253$ m/s.

Now $$e_{fb} = \left(\frac{V_B}{V_t}\right)^{0.22}$$

or, $$6.7 = \left(\frac{V_B}{0.0253}\right)^{0.22}$$

$\therefore$ Backwash velocity, $V_B = 5.0\times10^{-3}$ m/s.

The depth of expanded or fluidized bed L_{fb} is given by

$$L_{fb} = \frac{(1-e)}{(1-e_{fb})} = \frac{(1-0.5)}{(1-0.7)} = 1.67 \text{ m}$$

The method of finding the expanded bed of a nonuniform stratified bed has been illustrat in Example 12.5.

Example 12.5. *The bed described in Example 12.3 is to be backwashed to a velocit* 9×10^{-3} *m / s. Determine the depth of the expanded bed.*

Solution. Each layer of particles must be treated separately and the results summed.

For the bottom layer $d_i = 1.41 \times 10^{-3}$ m and $X_i = 0.01$

1. Estimate an initial velocity under turbulent condition with $C_D = 0.4$

(*a*) $V_t = \left(\frac{4}{3} \times 9.81 \times \frac{2560-1000}{1000 \times 0.4} \times 1.41 \times 10^{-3}\right)^{1/2} = 0.276$ m/s

(*b*) $R = \frac{0.85 \times 1000 \times 0.276 \times 1.41 \times 10^{-3}}{1.002 \times 10^{-3}} = 330$ (transitioned flow)

(*c*) $C_D = \frac{24}{330} + \frac{3}{(330)^{0.5}} + 0.34 = 0.58$

(*d*) $V_t = \left(\frac{4}{3} \times 9.81 \times \frac{2650-1000}{1000 \times 0.58} \times 1.41 \times 10^{-3}\right)^{1/2} = 0.229$ m/s.

Repeat Steps *b*, *c* and *d* with $V_t = 0.229$ to get

$$R = 274, \quad C_D = 0.609 \quad V_t = 0.22 \text{ m/s}$$

(*e*) Backwash velocity $V_B = 9 \times 10^{-3}$ m/s.

(*f*) Determine the expanded porosity layer by

$$e_{fb} = \left(\frac{V_B}{V_{ti}}\right)^{0.22} = \left(\frac{9 \times 10^{-3}}{0.223}\right)^{0.22} = 0.493$$

(*b*) Find the term for each layer $\frac{X_i}{(1-e_{fb})} = \frac{0.01}{(1-0.497)} = 0.0198$

2. Repeat the preciding steps for each layer of particles and the results are tabulated below :

Particle size d_i, *m*	V_{ti}, m/s	$\left(\frac{V_B}{V_{ti}}\right)^{0.22}$	$1-\left(\frac{V_B}{V_{ti}}\right)^{0.22}$	*Depth fraction* x_i	$\frac{X_i}{1-\left(\frac{V_B}{V_{ti}}\right)^{0.22}}$
1.41×10^{-3}	0.223	0.493	0.506	0.01	0.0198
1.13×10^{-3}	0.187	0.513	0.487	0.11	0.2259
0.78×10^{-3}	0.135	0.551	0.449	0.20	0.4458
0.66×10^{-3}	0.114	0.572	0.428	0.32	0.7471
0.55×10^{-3}	0.094	0.597	0.597	0.21	0.5206
0.46×10^{-3}	0.077	0.624	0.376	0.13	0.3460
0.42×10^{-3}	0.069	0.640	0.360	0.02	0.0555

Sum total of $\frac{X_i}{1-\left(\frac{V_B}{V_{ti}}\right)^{0.22}} = 2.3606$

3. The expanded depth of the total nonuniform stratified layer is given by

$$L_{fb} = L\,(1-e)\sum \frac{X_i}{1-\left(\frac{V_B}{V_{ti}}\right)^{0.22}} = 0.75 \times (1-0.4) \times 2.3606 = 1.062$$

which is $\frac{1.062}{0.75} \times 100 = 142\%$ of original bed.

12.6.3. Filter Sand. Shape, size and quality of filter sand shall satisfy the following manners :

(a) Sand shall be of hard and resistant quartz or quartzite and free from clay, fine particles, soft grains and dirt of every description.
(b) Effective size shall be 0.45 to 0.70 mm.
(c) Uniformity coefficient shall lie between 1.3 to 1.7.
(d) Ignition loss should not exceed 0.7 per cent by weight.
(e) Soluble fraction in hydrochloric acid shall not exceed 5.0% by weight.
(f) Silica content should be not less than 90%.
(g) Specific gravity shall be in the range between 2.55 to 2.65.
(h) Wearing loss shall not exceed 3%.

IS : 8419 (Part I) - 1977 for "Filtration Media for sand and Gravel" may be referred to for details.

12.6.4. Preparation of Filter Sand. The sand to be used in the filter is specified in terms of effective size and uniformity coefficient. From a sieve analysis of the stock sand, the coarse and fine portion of stock sand that must be removed in order to meet the size specifications, can be computed in terms of p_1 the percentage of stock sand that is finer than the desired effective size d_1 which is also equal to 10% of the usable sand and p_2 the percentage of the stock sand that is smaller than the desired 60 percentile size d_2.

The percentage of suitable stock sand p_3 is then $= 2\,(p_2 - p_1)$ because the sand lying between the d_1 and d_2 sizes will constitute half the specified sand.

To meet the specified composition this sand can contain $0.1p_3$ of a sand below d_1 size. Hence the percentage p_4 below which the stock sand is too fine to use is

$$\begin{aligned} p_4 &= p_1 - 0.1p_3 = p_1 - 0.2\,(p_2 - p_1) \\ &= 1.2\,p_1 - 0.2p_2 \end{aligned}$$

Likewise, the percentage p_5 above which the stock is too coarse for use is

$$\begin{aligned} p_5 &= p_2 + 40\% \text{ of usable sand} \\ &= p_2 + 0.4 \times 2\,(p_2 - p_1) \\ &= 1.8\,p_2 - 0.8\,p_1 \end{aligned}$$

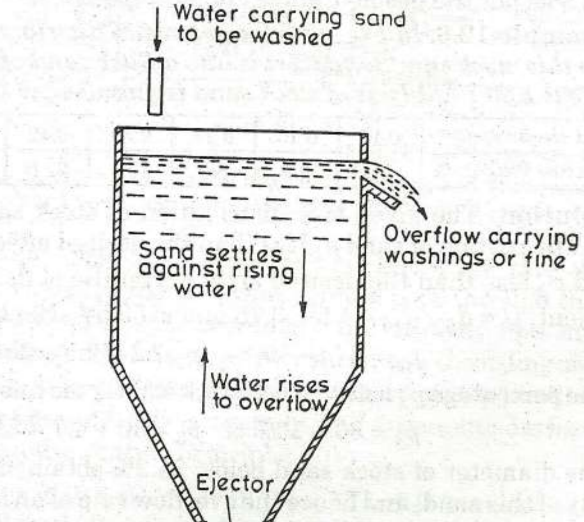


Fig. 12.4. Sand washer.

From the size cumulative frequency curve, the grain sizes of stock sand corresponding to p_4 and p_5 are determined (d_4 and d_5). The sizes below d_4 and above d_5 will have to be separated out from the stock sand to the desired specification. The coarse portion is removed by sieving. The finer portion is removed by a sand washer designed to floate out the particles of sizes smaller than d_4 by maintaining velocity in the upwardflow washers slightly less than the hydraulic subsidence value corresponding to d_4 size, such that all particles less than d_4 size are floated out with the overflowing water. Fig. 12.4 shows the working of a sand washer.

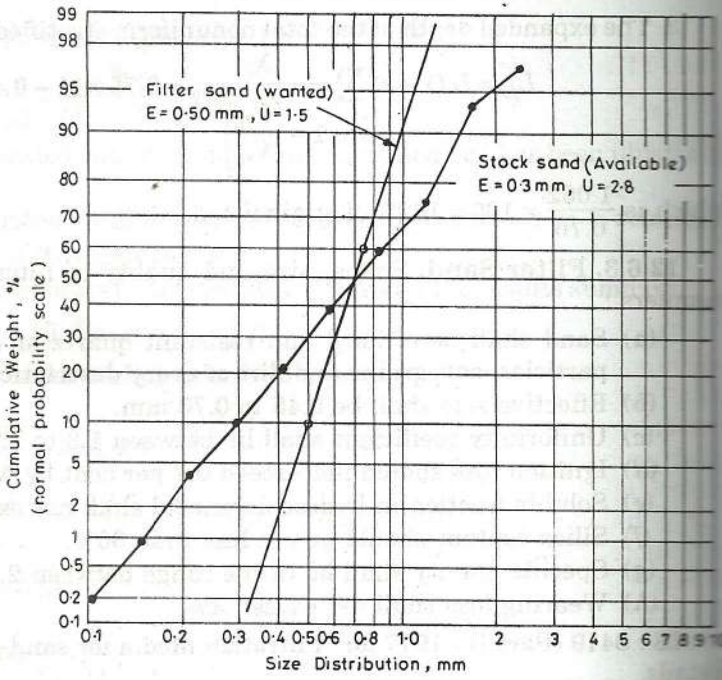


Fig. 12.5. Grain size distribution of a stock sand and required sizing of a filter sand.

On the basis of logarithmic normality, the probability integral the relationship can be established between the effective size and uniformity coefficient and the geometric mean size and geometric standard deviation. The desired filter sand size distribution can be plotted on normal probability scale as given in Fig. 12.5 as per the problem solved in Example 12.6.

Example 12.6. *In Fig. 12.6 the grain size distribution of stock sand is given. What must be done to this stock sand to convert it into a filter sand of effective size of 0.50 mm and uniformity coefficient 1.50 ? Analysis of stock sand is given below :*

Sand size in mm	0.10	0.15	0.21	0.30	0.42	0.59	0.85	1.20	1.69	2.40	3.35
Cumulative Weight, %	0.2	0.9	5.0	10.0	22.0	40.0	60.0	74.4	93.3	97.0	100

Solution. The given size distribution of stock sand is plotted on log-normal probability paper. The portion of sand p_1 less than the desired effective size of .5 mm is 30% and the portion of sand p_2 less than the desired sixty percentile of 0.5 × 1.5 (from the relationship uniformity coefficient, $U = d_{60}/d_{10} = 1.5 = 0.75$ mm is 53.5%. Hence from the relationship.

$$p_3 = 2\,(p_2 - p_1) \quad p_3 = 2\,(53.5 - 30) = 47.0\%$$

The percentage p_4 below which stock sand is too fine is given by

$$p_4 = 30 - 10\% \text{ of } \; p_3 = 30 - 4.7 = 25.3\%$$

The diameter of stock sand below 25.3% obtained from Fig. 12.5 is 0.45 mm. The settling velocity of this sand, and hence the overflow rate of an ideal washer, from Fig. 11.1 (*a*), is 6.5 cm/s approx. at 10°C.

The percentage of sand p_5 above which the stock is too coarse is given by

$$p_5 = 1.8\,p_2 - 0.8\,p_1 = 1.8 \times 53.7 - 0.8 \times 30.0 = 72.3\%$$

and the diameter of this sand from the Fig. 12.5 is 1.10 mm. It follows that all the stock sand finer than 0.45 mm and all the stock sand coarser than 1.10 mm must be wasted in order to create the desired filter sand.

(iii) These filters are very flexible, because the rate of filtration can be changed by change of compressed air pressure over the water.

(iv) These filters require very small area for their installation.

(v) Small number of fittings are required in these filters.

(vi) As the filtered water comes out under pressure no further pumping is required to lift the water.

(vii) No sedimentation and coagulant tanks are required with these filters.

DISADVANTAGES OF PRESSURE FILTERS

(i) Overall capacity of the filter is small, though the rate of filtration per sq.m. is high.

(ii) Due to the heavy cost on water treatment, they cannot be used for treating large quantity of water at the water works.

(iii) As the process of filtration and back washing is done in the closed tank, proper quality control and inspection is not possible.

(iv) The inspection, leaving, change of filter media, gravel and repair of under drainage system is difficult.

(v) Due to circular shape in plan, the design and fixing of back water channels is difficult.

(vi) Their efficiency in removal of bacteria and turbidity is poor.

(vii) They require additional pumps to pump the water in them.

13.16. COMPARISON BETWEEN SLOW AND RAPID GRAVITY FILTERS

S.No.	Item	Slow Sand Filter	Rapid Gravity Filter
1.	Area	Requires very large area	Requires small area
2.	Quantity of sand	Requires Considerable quantity of sand	Requires less quantity of sand
3.	Quality of sand	finer filter media of 0.2 to 0.4 effective size and 2 to 4 uniformity coefficient	Slightly coarser filter media of 0.36 to 0.6 effective size and 1.2 to o 1.8 uniformity coefficient
4.	Quality of raw water	It may not be treated with chemicals, but should not have turbidity more than 50 p.pm.	Treatment with chemicals is essential.
5.	Flexibility in operation	Not possible	Possible
6.	RAte of filtration	100-180 litres/m2/hour	4,000-5,000 litres/m2 hour
7.	Size of one unit	30 m × 60 m	$6\,m \times 8\,m\ 8.m \times 10\,m$
8.	Distribution	Uniform	Smaller at top and coarser in bottom
9.	Underdrainage system	Open jointed pipes or drains covered with blocks	Manifold and pipe laterals. Vitrified tile Blocks; the Wheeler Filter Bottom; the Porous Plate Bottom Concrete Ridge and Valley Bottom etc.
10.	Period of Cleaning	1 to 3 months	24 to 48 hours
11.	Method of Cleaning	Scrapping 2-3 cm sand from the surface and replacing it with new sand.	By back washing with water under pressure, with or with-out compressed air agitation before washing with water Most essential
12.	Skilled Supervision	Not required	Most essential
13.	Loss of head	15 cm to 75 cm	2 m 4 m
14.	Penetration of suspended impurities	Very small, only a dirty layer is formed at the surface	2 m to 4 m Very deep
15.	Amount of water required for washing	0.2 to 0.6% of filtered water	2 to 4% of filtered water
16.	Overall cost of unit	More, because large land and much quantity of materials are required	Cheap and economical
17.	Cost of Maintenance	Small	More
18.	Efficiency	Efficient in removal of bacteria and suspended matter	Cannot remove all bacteria, disinfection necessary. Removes colour, odour and taste.

13.17. DESIGN OF FILTERS

The method of design of filter will be clear from the following solved examples :

Example 13.1. Design five slow sand filter beds from the following data for the water works of a town of population 75,000: Per capita demand = 135 litres/day/capita

Rate of filtration = 210 litres/hour/m^2

Assume maximum demand as 1.5 times the average demand. Out of five units, one is to be kept as stand by and used while repairing other units.

Solution.

Average demand of the town = 13575000 litres/day

$$= 10.125 \times 10^6 \text{ litres/day}$$

Maximum demand of the town = $1.5 \times 10.125 \times 10^6$ litres/day

$$= 15.19 \times 10^6 \text{ litres/day}$$

Rate of filtration = 214×24 litres/day

$$= 5136 \text{ litres/day}$$

Total surface area of filters required

$$= \frac{\text{Maximum daily demand}}{\text{Rate of filtration/day}} = \frac{15.19 \times 10^6}{5136}\ sq.\ m$$

$$= 2957.55 \text{ sq.m.}$$

But as out of five units only four are to be used, therefore, surface area of each unit

$$= \frac{2957.55}{4} = 740 \text{ sq. m}$$

Keeping the length of the slow sand filter as 2.5 times its width

$$2.5B \times B = 740 \quad \text{or} \quad B^2 = \frac{740}{2.5}$$

$$\therefore \quad B = 17.2 \text{ m} \quad \therefore \quad L = 2.5 \times 17.2 = 43$$

$\therefore$ **Size of each slow sand filter unit = 43 × 17.2 m. Ans.**

Example 13.2. Design set of three rapid gravity filters for treating the water at a water works, which has to supply the water to a town of population 1,00,000. Per capita demand of the town is 270 litres/day. The rate of filtration of the rapid gravity filters may be taken as 4500 litres/hour/sq.m.

Solution.

Total quantity of water to be treated by the filters

$$= 100{,}000 \times 270 \text{ litres/day} \quad = 27 \times 10^6 \text{ litres/day}$$

$$= \frac{27 \times 10^6}{4500} \text{ sq.m.} \quad = 1.125 \times 10^6 \text{ litres/hour}$$

total area of filter bed required

$$= \frac{\text{Quantity of water treated}}{\text{Rate of filtration}} = \frac{1.125 \times 10^6}{4500} \; sq.\,m.$$

$$= 250 \text{ sq. m.}$$

$$\therefore \text{ Areas of each rapid gravity filter } = \frac{250}{3} = 83.34 \; sq.\,m.$$

Keeping the length of the filter 1.5 times its width

$$1.5\,B \times B = 83.34 \quad \therefore \quad B = 7.5 \text{ m}$$

$$\therefore \quad L = 1.5 \times 7.5 = 11.25 \text{ m}$$

$\therefore$ Provide three numbers of rapid gravity filters, with the size of each unit as 11.25 x 7.5 m Ans.

Example 13.3 Design a rapid sand filter unit for treating 5×10^6 litres/day supply for a town. The filter are to work day and night. Take 4,500 litres/m^2/hour as the rate of filtration.

Solution- Let half an be lost in washing, draining and returning to the service after 27 hours. Therfore total time will be 23.5 for which filters will work. Now allowing 4% as the allowance for washing of filter.

$$\text{Quantity of water to be treated per hour} = \frac{5.04 \times 10^4}{23.5} \text{ litres.}$$

$$\therefore \text{ Filter area required} = \frac{5.04 \times 10^6}{23.5 \times 4.5 \times 10^3} = 47.8 \text{ sq m} \quad = 48 \text{ m}^2 \text{ (say)}$$

Providing two units, area of each = 24m^2

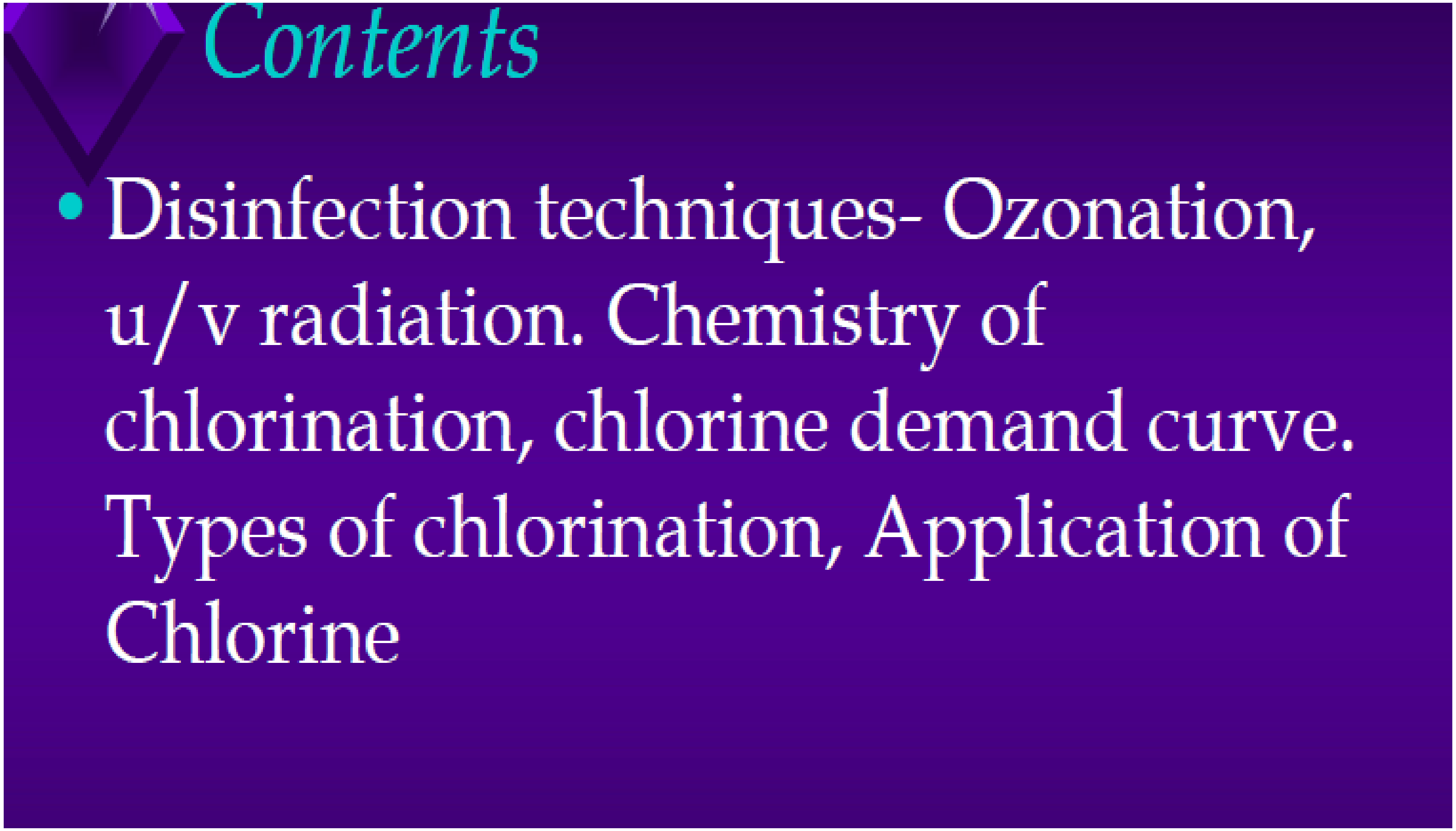
Therefore provide three units of 6 m x 4 m.

Two units of filter will always work. Third is required during repair work of any one of the others, which can also be used during emergency.

QUESTIONS

(a) Explain the theory of filtration as used in the purification of water.
   (b) Sketch and describe an outlet for a slow sand filter. (Pb. Univ., 1965)
2. (a) Explain clearly how does a rapid gravity filter differ in its action from a slow sand filter ?
   (b) What are the merits and demerits of the rapid filters as compared with the slow sand filters ?
3. (a) What is function of sand and gravel in the filters ?
   (b) Why is the underdrain system necessary in sand filters ?
   (C) What precautions should be kept in mind while washing rapid gravity filter ?
4. With the help of a neat sketch describe a pressure filter.
5. Write short notes on the following :
   (i) Effective size and uniformity coefficient of sand.
   (ii) Rate control of filteration.
   (iii) Air-binding.
   (iv) Necessity of under-drainage system.
6. Design a set of rapid gravity filters for treating water required for a population of 50,000, the rate of supply being 180 litres/day/person. The filters are rated to work 5000 litres/hour/sq.m. Assume whatever data are necessary. (U.P.S.C. 1969)
7. Explain the theory of filteration of water. (Ranchi 1966, 1969, Jodhpur 1964)
8. Compare and contrast point by point a slow sand filter with a rapid gravity filter. (U.P.S.c. 1959, A.M.I.E. 1951,1954, Ranchi 1968)
9. Design a slows and filter :
   (a) For a town of population 40,000 and maximum rate of supply 200 l/hour/day. (U.P.S.C., 1963)
   (b) For a population of 5000 and maximum rate of supply 200 l/hour/day. (U.P.S.C. 1968)
10. Write short notes on :
   (a) Mudd ball formation
   (b) Air binding of filters (U.P.S.C., 1952; Bha. 1968)
   (c) Filtering film. (U.P.S.C., 1964)
11. Mention the factors that control the working efficiency of a rapid gravity filter. (Roorkee 1962)
12. Answer with respect to rapid sand filters. (A.M.I.E. 1968)
   (a) What is the maximum permissible head loss ?
   (b) What is the filtration rate?
   (c) Function of rate of flow controller.
   (d) Effective size and uniformity coefficient.
13. (a) Explain the operation of a rapid sand filter with the help of neat sketches.
   (b) A town having a population of 50000 has to be supplied with water at the rate of 200 l.p.c.d. Assume a suitable hydraulic loading, make suitable recommendations for number and size of rapid sand filter beds. (Gorakhpur University, Ist Semester, 1975-76)
14. Compare economics and efficiency of slow sand filter with respect to rapid gravity filter. Work out the size of a rapid gravity filter bed for filtering 162 litres of water per hour at standard rate of filtration and work out the capacity of backwash tank. (Roorkee University, Autumn Semester, 1974-75)
15. The population of a city is 50,000 and the per capita consumption is 130 litres/day. Calculate the following in respect of the rapid sand filter for the above data :
   (i) Total area of filters
   (ii) Number and dimensions of each filter bed
   (iii) Quantity of air for air-wash per filter bed, and
   (iv) Backwash water per filter bed after air-wash. (AMIE, Sec., B., Winter 1975)
16. (a) How does a filter clarify works ?
   (b) Describe a rapid sand gravity filter. Suggest specification for the sand to be used for the filter.
   (c) State the number and size of rapid sand gravity filter to be used to treat 454 X 10^{6}l/d. (AMIE, Sec. B., May 1971)
17. A filter plant has 6 filter units each with a rated capacity of 2.5 ml/d. The rapid sand filter loading rate is 125 l/m^2/min. What percentage of treated water is used for backwashing if each filter is washed every 48 hours at 7.5 times the loading rate for 5 minutes duration ? (AMIE, Sec. B, May 1970)
18. (a) Differentiate between rapid sand and slow sand fitler.
   (b) Mention the design criteria for the underdrain system of the rapid sand filter. (Nagpur Univ. 1981, Eighth Semester)
19. (A) Calculate the number, size and depth of the filter beds of a dual media filter plant to treat 640 m^3/hr of water.
   (b) Mention the merits and demerits of an up flow filter. (Nagpur Univ. 1981, Eighth Semester)
20. (a) Sketch and describe in details the working of slow sand filter.
   (b) Give the advantages and disadvantages of slow sand filter.
   (c) What do you know about negative head in rapid sand filter? (Nagpur Univ. 1981, Ninth Semester)

Disinfection



Contents

- Disinfection techniques- Ozonation, u/v radiation. Chemistry of chlorination, chlorine demand curve. Types of chlorination, Application of Chlorine

Some common water-borne diseases prevented by disinfection

Bacterial	Viral	Protozoan
Typhoid fever	Hepatitis	Amoebiasis
Para-typhoid	Rotavirus diarrhea	Giardiasis
Bacterial diarrhea		Cryptosporidiasis
Cholera		

Overview of the Process

- The purpose of disinfecting drinking water is to destroy organisms that cause diseases in human beings.
- Most pathogenic bacteria are removed from water in varying degrees during the different treatment processes (coagulation, sedimentation, and filtration).
- Disinfection is used to ensure satisfactory removal of pathogens from potable water.

Suitability of the disinfectant

1. Disinfectant shall be effective in killing the microorganisms.
2. Should be cheap and readily available.
3. Should be safe to handle and easy to apply.
4. It should not make water toxic in nature.
5. It should have ability to persist in residual state.

Disinfection Methods

1. Boiling
2. High pH (Excess lime)
3. Silver treatment
4. Potassium Permanganate
5. Chlorination
6. Ozonation
7. UV irradiation
8. Other halogens (Iodine and Bromine)

1. Boiling

- Water shall be boiled at least for 3 mins.
- Most effective
- Can kill bacteria and viruses
- Can not be used on large scale
- During epidemics it is used in domestic scale.

2. High pH Treatment (Excess Lime)

- Lime can destruct bacteria at high pH values
- In this case, no residual will remain after neutralization
- Studies showed that

- higher removal of viruses was obtained with higher pH values
- Optimum pH in the range of 11.2 to 11.3
- Optimum contact time in the range of 1.56 to 2.40 hours
- Complete destruction of viruses was obtained at pH of 11.0 and contact time of 5.0 hours and 10 minutes

3. Silver Treatment

- Silver when immersed in water has inhibiting effect on bacterial life.
- As silver is costly this method can be used in small scale.

4. *Ultraviolet (UV) Irradiation*

- Can disinfect both water and wastewater.
- However, its use in drinking water disinfection is limited to small installations such as aboard ships because it does not produce residual.
- In wastewater treatment, and when compared to

  chlorination and Ozonation, UV was found

– More effective

– More economical

- UV irradiation is gaining prominence.

Ultraviolet (UV) Irradiation

5. Ozonation

- General characteristics of ozone

–– Powerful oxidant

–– More powerful than Hypochlorous acid

–– Unstable in aqueous solutions

–– Has a half of 20 to 30 minutes in distilled water

–– Widely used in drinking water treatment

–– Is produced on -- site and can not be stored site

6. Potassium Permanganate

- Commonly used in rural areas.
- Used for purification of open well water supplies.
- $KMnO_4$ is dissolved in bucket and added in well water.
- Water should not be used during first 48 hours after $KMnO_4$ addition

CHLORINATION

Why Chlorination?

Chlorine is widely used

- Effective at low concentration
- Cheap
- Forms residual if applied in sufficient dosages

Chlorine is applied as:

- Gas (most common), Cl_2
- Hypochlorite (Bleaching Powder) $Ca(OCl)_2$

- Chlorine is a strong oxidizing agent
  - It oxidizes enzymes of microbial cells that are necessary for metabolic processes, thus, inactivating (destroying) the microbial cells.

Chlorination Reaction

- Chlorine gas reacts with water to form
  - Hypochlorous acid (HOCl)
  - Hydrochloric acid (HCl)

$$Cl_2 + H_2O \rightarrow HOCl + HCl$$

- Hypochlorous acid dissociates to hypochlorite ion

$$HOCl \leftrightarrow H^+ + OCl^-$$

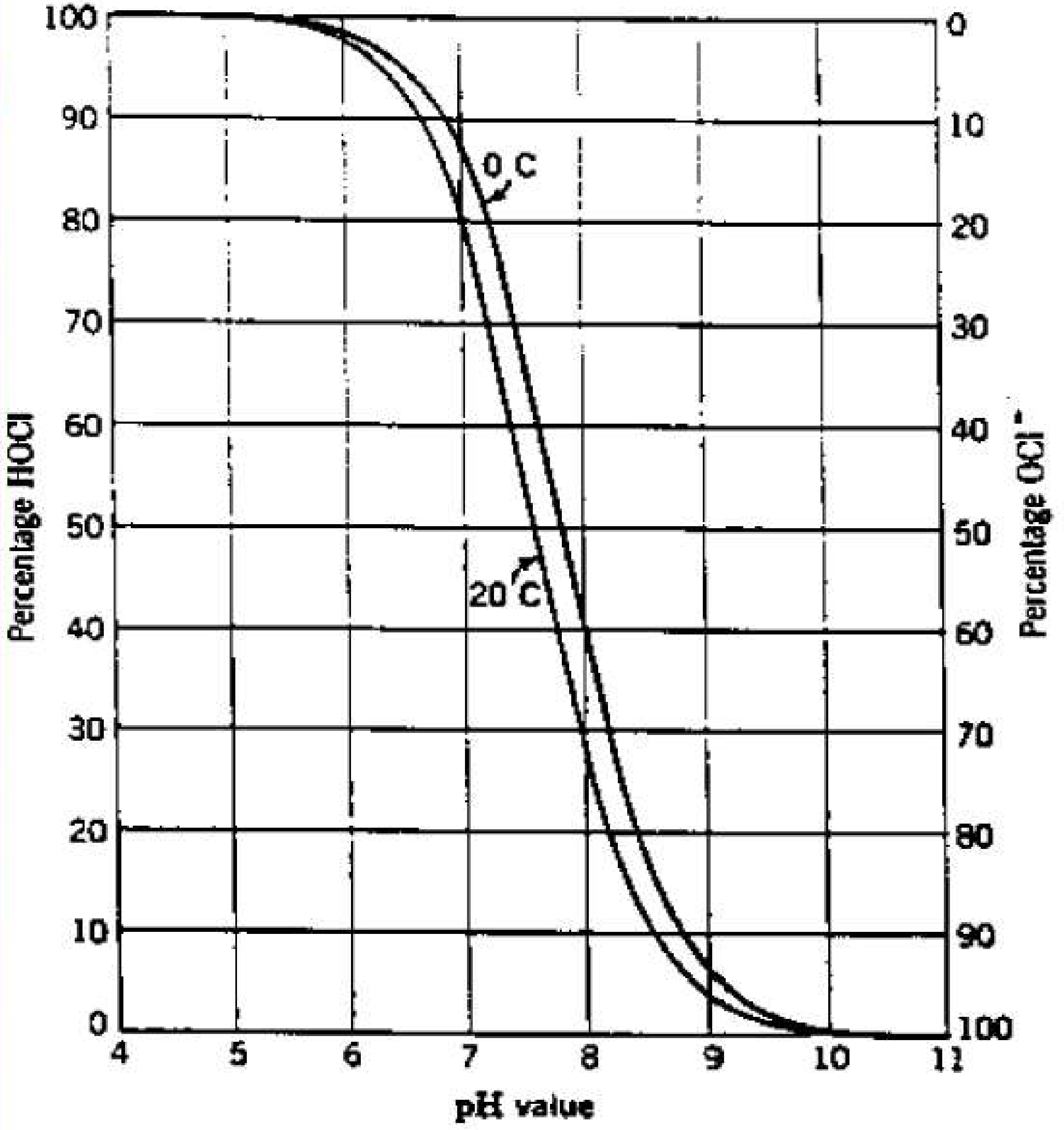
- The dissociation of the HOCl is a function of pH

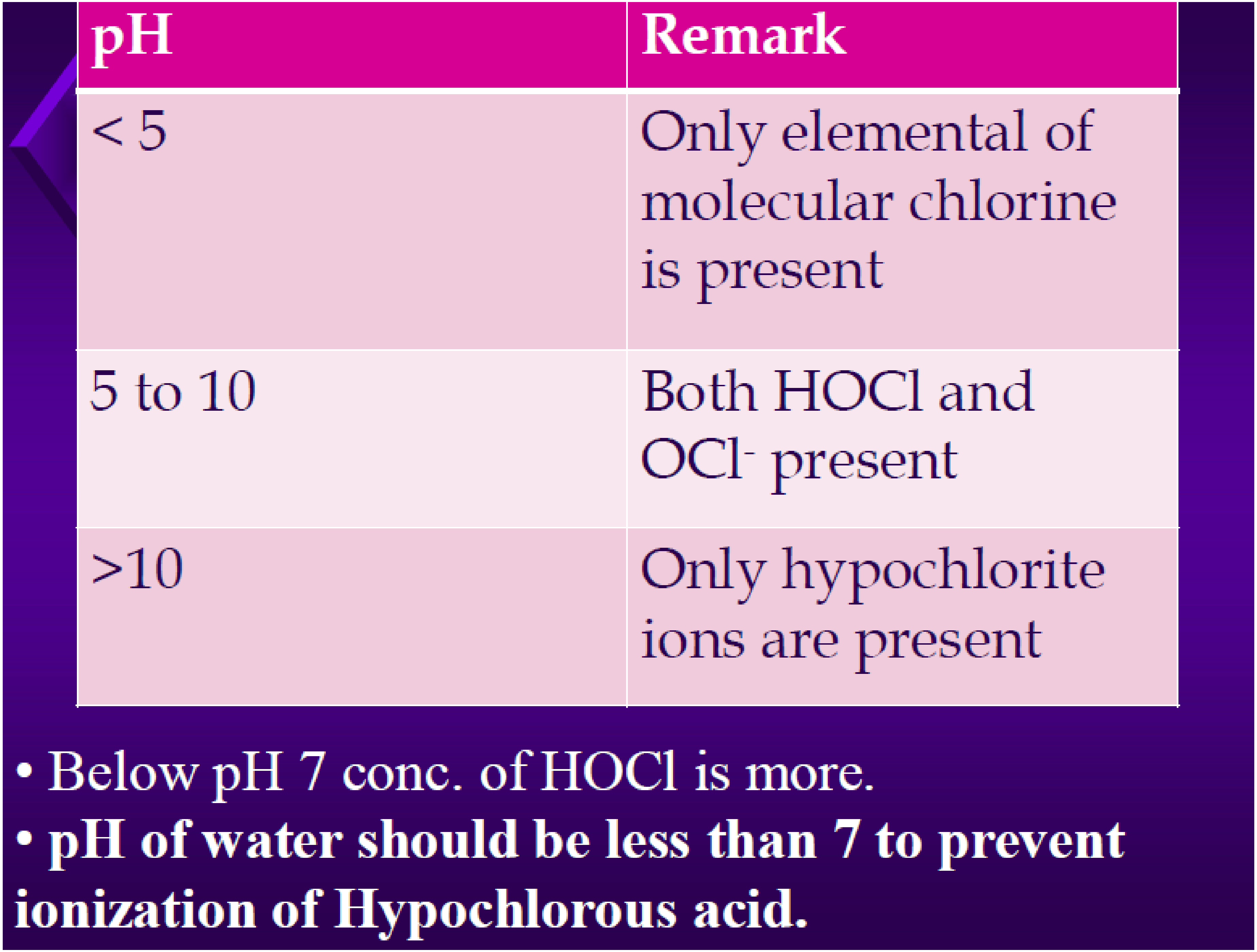
- **Hypochlorous acid is 80 times more effective disinfectant than hypochlorite ions.**
- Chlorine will appear in water in three forms

i. Elemental chlorine (Cl^-)

ii. $HOCl$

iii. OCl^-


100
90
80
70
60
50
40
30
20
10
0
Percentage HOCl
0
10
20
30
40
50
60
70
80
90
100
Percentage OCl⁻
4
5
6
7
8
9
10
11
pH value
0 C
20 C



pH	Remark
< 5	Only elemental of molecular chlorine is present
5 to 10	Both HOCl and OCl^- present
>10	Only hypochlorite ions are present

- Below pH 7 conc. of HOCl is more.
- **pH of water should be less than 7 to prevent ionization of Hypochlorous acid.**

Chlorine demand

- When chlorine is added in water it reacts with organic and inorganic impurities.
- The amount of chlorine utilized for this is known as chlorine demand.
- The remaining chlorine will appear as available residual chlorine.
- Which serves as disinfectant to kill the pathogens

Dosages, Demand and Residuals – Short Summary

- Dosage: the amount of chlorine added
- Demand: the amount of chlorine needed to oxidize materials (reaction)
- Residual: the amount of chlorine remaining after oxidation.

Residual Chlorine = Dosage – Demand

Part-II

FORMS OF APPLICATION OF CHLORINE

Chlorine can be applied in water by following forms

1. As Bleaching powder or Hypochlorite
2. As Chloramines
3. As free Chlorine gas
4. As Chlorine dioxide

1. Bleaching powder

- Chemical formula is Ca $(OCl)_2$.
- Available chlorine is 20 to 40 %.
- When it is added in water following reaction takes place

$$\mathbf{Ca(OCl)_2 + H_2O \rightarrow 2HOCl + Ca(OH)_2}$$

- Hypochlorous acid acts as disinfectant.

- As it contains 20 -40% chlorine it requires higher transportation cost and space for storing.
- Therefore used in small scales i.e. rural water supply schemes

2. Chloramines

- The free chlorine can react with compounds such as **ammonia, proteins, amino acids and phenols** that may be present in water to form chloramines and chloro-derrivatives which constitute as combined chlorine.
- Combined chlorine possesses disinfection properties.

Reaction with Ammonia

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O$$

$$monochlora\min e$$

$$NH_2Cl + 2HOCl \rightarrow NHCl_2 + 2H_2O$$

$$dichlora\min e$$

$$NHCl_2 + 3HOCl \rightarrow NCl_3 + 3H_2O$$

$$nitrogen - trichloride$$

pH range	Chloramines formed
< 4.4	Trichloramine only
4.4 to 5	Dichloramine only
5 to 8	Both Mono and Dichloramine (Equal quantities at pH =7)
> 8.5	Monochloramine only

Advantages

1. More effective than chlorine alone
2. Prevents tastes and odour
3. Water treated with chloramines causes less irritation of eyes and odours.
4. No danger of overdose
5. Less chlorine is required

3. Free chlorine

- Generally applied in gaseous form or liquid form.
- 2.48 times heavier than air and 1.44 times heavier than water.
- Strong oxidant
- Forms residual
- By products are formed e.g. THMs
- Works under specific pH.

4. Chlorine Dioxide (ClO_2)

- Chlorine dioxide is created by mixing solutions of sodium chlorite and chlorine.
- Chlorine dioxide is generated on site

$$2NaClO_2 + Cl_2 \rightarrow 2ClO_2 + 2NaCl$$

- The advantages of chlorine dioxide are:

1. it is a strong bactericide and viricide over a wide pH range.
2. It forms a slight residual in the distribution system.

3. It does not react with nitrogen to form chloramines.
4. It does not react with organic matter to form Trihalomethanes (THMs).

- **Chloroform and Bromoform are THMs**
- The disadvantages of chlorine dioxide are its high cost and its tendency to create chlorate and chlorite, which are potential toxins.
- Chlorine dioxide is used for taste and odor control as well as disinfection.

FORMS OF CHLORINATION

Forms of chlorination

1. **Plain Chlorination**
2. **Pre-Chlorination**
3. **Post Chlorination**
4. **Double of multiple Chlorination**
5. **Break point Chlorination**
6. **Super Chlorination**
7. **De-Chlorination**

1. Plain Chlorination

- Application of chlorine to raw water before it enters into distribution system.
- Sometimes added to reservoirs to check growth of weeds, organic matter , algae etc.
- It can be practiced for water with turbidities 20 to 30 NTU.
- Dose range – 0.5 to 1 ppm

2. Pre Chlorination

- Chlorine is applied before treatment specially before filtration.
- Sometimes added before sedimentation.
- Advantages

i. Reduces bacterial load on filtration
ii. Reduces coagulant required.
iii. Controls growth of algae
iv. Eliminates tastes and odour.

3. Post Chlorination

- Addition of chlorine after treatment i.e. after filtration.
- Added before water enters into distribution system.

4. Double Chlorination

- Application of chlorine at two or more points in purification process.
- Usually added -

i. Before sedimentation tank

ii. After filtration

- This is done if water is more contaminated.

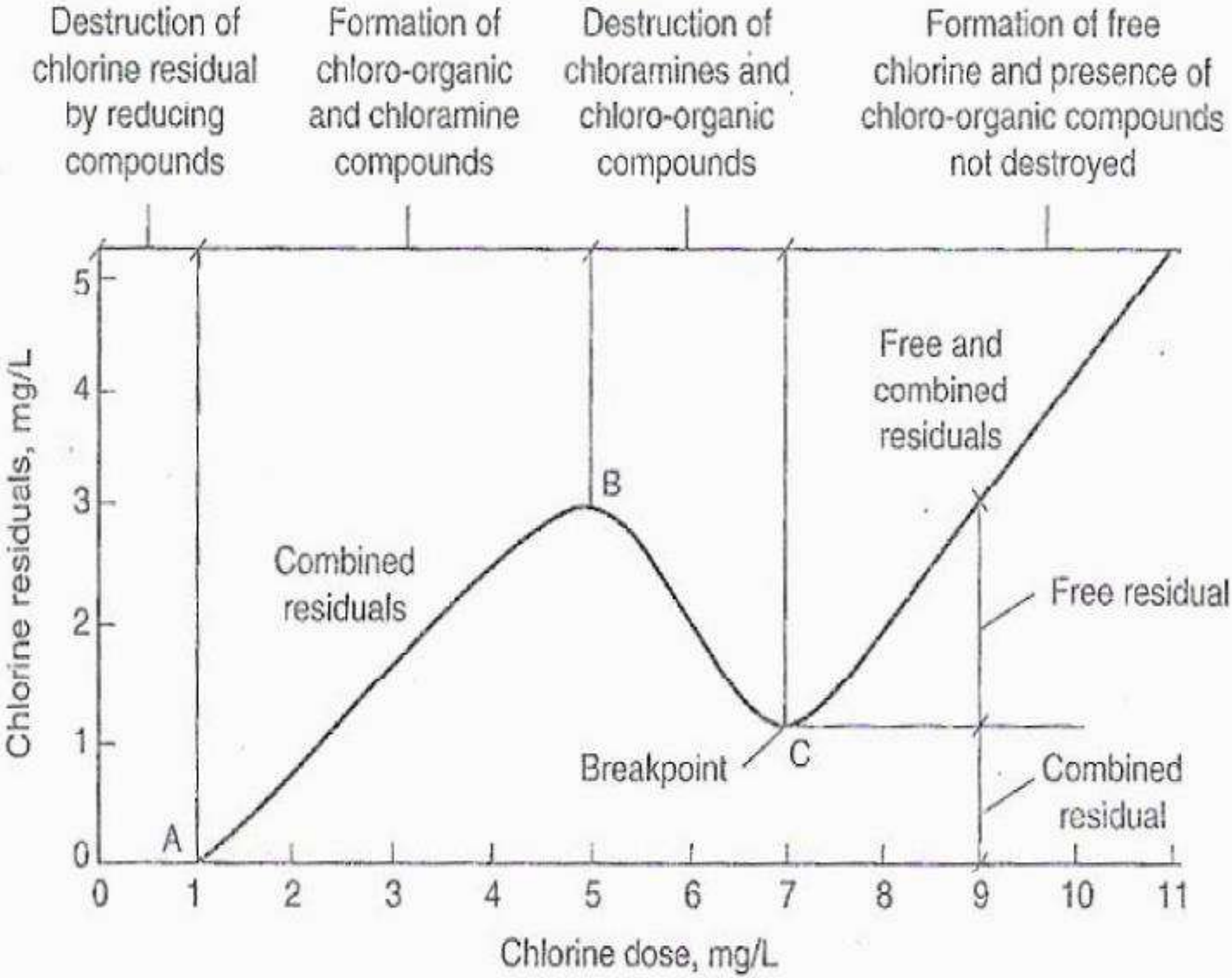
5. Break Point chlorination

-When chlorine is added it reacts first with the reducing compounds such as Fe^{+2}, Mn^{+2}, NO^{-2} , and the chlorine will be reduced to the none effective chloride ion Cl^{-} (from zero to point A on the figure).

- When adding more chlorine it will react with NH_3 to form chloramines as shown in the chlorine chemistry (from point A to B).

-When adding more chlorine some chloramines are oxidized to nitrogen gas and the chlorine is reduced to the none effective Cl^{-} ion.(from point B to C).

-Continued addition of chlorine will produced free available chlorine (at point C). point C is called the break point.


Destruction of chlorine residual by reducing compounds
Formation of chloro-organic and chloramine compounds
Destruction of chloramines and chloro-organic compounds
Formation of free chlorine and presence of chloro-organic compounds not destroyed
Free and combined residuals
B
Combined residuals
Free residual
Breakpoint
C
Combined residual
A
Chlorine residuals, mg/L
0
1
2
3
4
5
0
1
2
3
4
5
6
7
8
9
10
11
Chlorine dose, mg/L

6.Super chlorination

- Chlorine is applied beyond break point.
- It is done after filtration with contact time of 30 to 60 mins.
- It is practiced when there is **epidemics** in the society.

7. De Chlorination

- Process of removing excess chlorine from water is known as De-Chlorination.
- Can be achieved by aeration or using chemicals such as sodium thiosulphate , sodium bi sulphate, sodium sulphite, Activated carbon, potassium permanganate.

FACTORS AFFECTING CHLORINATION

1. Turbidity

- Turbidity affects chlorination process as efficiency of disinfection process decreases if water is more turbid.
- Pathogens can shelter behind solid particles.
- So, chlorine is added after removing turbidity i.e. after filtration.

2. Presence of metallic compounds

- Fe and Mn consumes more chlorine if present in water.
- So, chlorine is added when Fe and Mn are removed from water.

3. Ammonia compounds

- Ammonia forms combined chlorine compounds which are not so powerful disinfectants as compared to free available chlorine.
- Therefore more chlorine is added so that after formation of chloramines, excess free chlorine is available for speedy disinfection

4. pH of water

- Increasing pH reduces effectiveness of chlorine.

pH	**Amount of HOCl**
Up to 6.7	95% of total free chlorine
At 7	80% of total free chlorine
At 8	30% of total free chlorine
At 9	5% of total free chlorine

5. Temperature of water

- Reduction in temperature decreases killing power of both free and combined available chlorine.

6. Time of contact

- Percent kill of pathogens depends upon contact of chlorine and micro-organisms
- Time of exposure or time of contact is needed. Usually for free chlorine 10 min time is required as compared to 60 mins for combined chlorine.

7. Nature and Concentration of bacteria and viruses

- Bacteria spores are more resistant
- Polio virus is most resistant

Disinfection: First-Order Kinetics

Assumes:

- all organisms are identical
- death (inactivation) results from a first-order or "single-hit" or exponential reaction.

Chick's law:

$$-dN/dT = kN$$

where: N = number of organisms

T = time

$$\ln N_t/N_o = -kT$$

Where, N_o = initial number of organisms

N_t = number of organisms remaining at time = T

No = initial number of organisms (T= 0)

Also:

$$N_t/N_o = e^{-kT}$$

Disinfection Kinetics

- Disinfection is a kinetic process
- Increased inactivation with increased exposure or contact time.
  - Chick's Law: disinfection is a first-order reaction.
  - Multi-hit-hit or concave up kinetics: initial slow rate; multiple targets to be “hit”
  - Concave down or retardant kinetics: initial fast rate; decreases over time
    - Different susceptibilities of microbes to inactivation; heterogeneous population
    - Decline of of disinfectant concentration over time
- CT Concept: Disinfection can be expressed at the product of disinfectant concentration and contact time
  - Applies best when disinfection kinetics are first order

Inactivation of *Cryptosporidium* Oocysts in Water by Chemical Disinfectants

Disinfectant	CT_{99} (mg-min/L)	Reference
Free Chlorine	7,200+	Korich et al., 1990
Monochloramine	7,200+	Korich et al., 1990
Chlorine Dioxide	>78	Korich et al., 1990
Mixed oxidants	<120	Venczel et al., 1997
Ozone	~3-18	Finch et al., 1994 Korich et al., 1990 Owens et al., 1994

C. parvum oocysts inactivated by low doses of UV radiation: <10 mJoules/cm^2

Disinfection Kinetics…

- The C x T concept fails if disinfection kinetics do not follow Chick's Law (are not first-order or exponential)
- Chlorine concentration and contact time relationship is often expressed by,

  $C^n t_p = k$, where, n and k are experimentally derived constants

- The effect of temperature variation can be modeled by the following equation

  In $t_1/t_2 = E'(T_2-T_1)/R_{T1,T2}$

  t_1 and t_2 = time required for given kills

  T_2 and T_1 are temperatures corresponding to t_2 and t_1, in °K

  R = gas constant, 1.0 cal/K-mol

  E' = Activation Energy, related to pH

pH ->	7.0	8.5	9.8	10.7
E', Cal ->	8,200	6,400	12,000	15,000

the connection from the downstream side of the orifice having lower pressure is made to the outlet of the rubber bag having dosing solution, as shown in Fig. 9.41. The difference in pressure squeezes the rubber bag injecting the dosing solution into the water line. This injection of solution occurs only during the flow of water in the pipe line. By adjusting the concentration of the dosing solution in the bag, the dosing is made proportional to the water used.

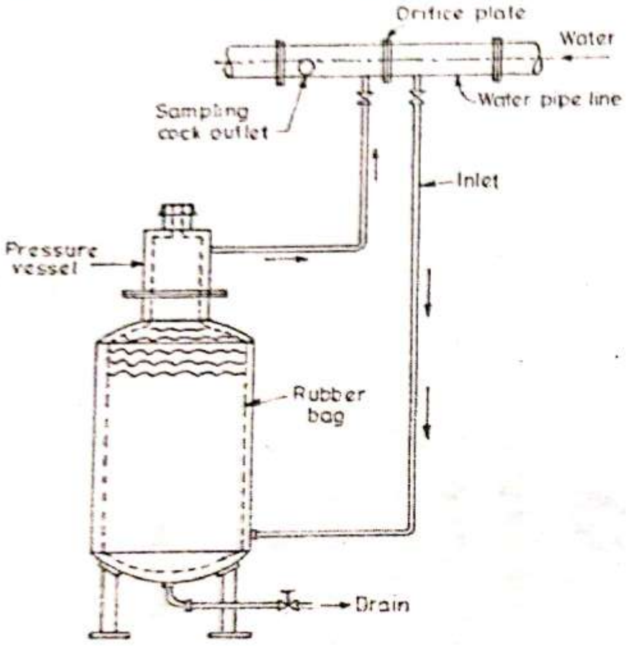


Fig. 9.41. Displacement doser.

Example 9.27. *It is required to supply water to a population of 20,000 at a per capita demand of 150 litres per day. The disinfectant used for chlorination is bleaching powder which contains 30 per cent of available chlorine. Determine how much of bleaching powder is required annually at the waterworks, if 0.3 ppm of chlorine dose is required for disinfection.*

Solution.

Average daily water demand

= Population × Per capita demand

= 20,000 × 150 litres

= 30,00,000 litres

$= 3 \times 10^6$ litres.

∴ Amount of chlorine required *daily* (based on average annual consumption)

$$= 0.3\ \frac{\text{mg}}{l} \times 3 \times 10^6\ l$$

$$= 0.9 \times 10^6 \text{ mg}$$

$$= 0.9 \text{ kg}.$$

Since the chlorine content in bleaching powder is 30%, it means that 30 kg of chlorine is contained in 100 kg of bleaching powder.

∴ Amount of bleaching powder required daily (based on average annual consumption)

$$= \frac{0.9 \times 100}{30} = \mathbf{3\ kg.}$$

∴ Annual consumption of bleaching powder

= 3 × 365 kg

= 1095 kg = 1.095 tonnes. **Ans.**

Example 9.28. *Chlorine usage in the treatment of 20,000 cubic metre per day is 8 kg/day. The residual after 10 min. contact is 0.20 mg/l. Calculate the dosage in milligrams per litre and chlorine demand of the water.* (*Engineering Services, 1980*)

Solution.

Water treated per day = 20,000 cum.

$= 20{,}000 \times 10^3$ litres

$= 20 \times 10^6$ litres = 20 M*l*.

Chlorine consumed per day = 8 kg

= 8 M.mg

∴ Chlorine used per litre of water

$$= \frac{8 \text{ M. mg}}{20 \text{ M}.l} = 0.4 \text{ mg}/l$$

∴ Hence, the given chlorine dosage

= 0.4 mg/*l*.

Now, residual chlorine left

= 0.2 mg/*l* (given)

Hence, actual chlorine dosage, which has reacted in water, *i.e. chlorine demand of water*

= 0.4 – 0.2 = 0.2 mg/*l* . **Ans.**

(3) Use of Chlorine Tablets. Chlorine tablets may also some times be used to disinfect small quantities of water, but they are costly. They are available in the market under various trade names, such as *halazone tablets*. The National Environmental Engineering Research Institute, Nagpur has formulated a new type of chlorine tablet, which is 15 times better than the ordinary *halazone tablets*.

Table 9.7

Colour developed with chlorotex reagent	*Chlorine concentration indicated, in mg/l*
White milky fluorescence	Nil
Faintly pink and milky	0.1
Pink	0.2
Red	0.5
Purple	0.6
Violet	0.8
Blue	1.0 or more

(4) **Starch Iodide test.** In this test, one litre of water sample is collected in a heat-proof earthen ware vessel (called *casserole*), to which 10 ml of potassium iodide solution is added. The contents are thoroughly mixed, and 5 ml of starch solution is added, which produces blue colour. This blue colour is removed by titrating this water sample against sodium thiosulphate solution of normality N/100. The amount of chlorine can then be easily ascertained by using the simplified titration equation :

$$\left[\begin{array}{l}\text{Quantity of chlorine}\\ \text{in mg/l in the original}\\ \text{sample of water}\end{array}\right] = 0.355 \left[\begin{array}{l}\text{Number of ml of}\\ \text{thiosulphate reqd.}\\ \text{to remove the}\\ \text{blue colour}\end{array}\right]$$

Starch iodide test is laborious and costly, and hence generally not used for testing public supplies.

Example 9.29. *Results of chlorine demand test on a raw water are given below* :

Sample No.	*Chlorine dosage mg/l*	*Residual chlorine after 10 min contact (mg/l)*
1	*0.2*	*0.19*
2	*0.4*	*0.36*
3	*0.6*	*0.50*
4	*0.8*	*0.48*
5	*1.0*	*0.2*
6	*1.2*	*0.4*
7	*1.4*	*0.6*
8	*1.6*	*0.8*

Sketch a 'chlorine demand curve'. What is the 'break point dosage' and what is the 'chlorine demand' at dosage of 1.2 mg/l ?

(Civil Services, 1980)

Solution. The given data is used to plot the chlorine demand curve (strictly speaking chlorine residual curve) as shown in Fig. 9.43.

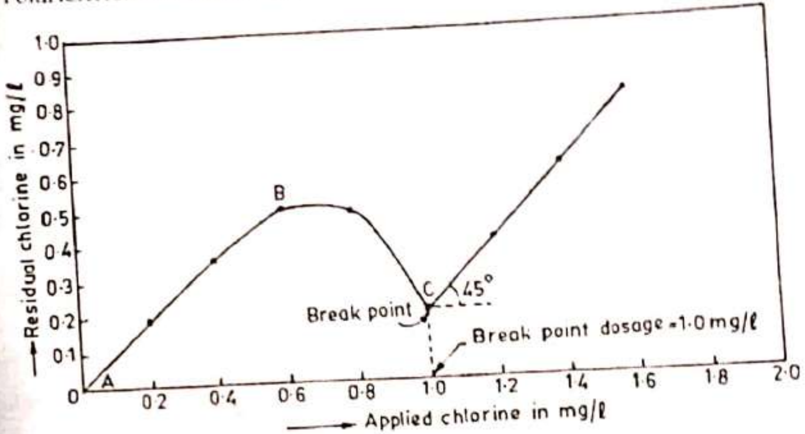


Fig. 9.43

Break point is represented by the point C, and hence the *break point dosage* is 1.0 mg/l. **Ans.**

Chlorine demand is the difference between applied chlorine and residual chlorine, and it becomes equal to 1.0 – 0.2 = 0.8 mg/l at break point. This chlorine demand becomes constant thereafter, and all added chlorine appears as free chlorine.

Thus, at any dosage above 1.0 mg/l, the chlorine demand will remain study, and equal to 0.8 mg/l.

Hence, the chlorine demand at a dosage of 1.2 mg/l will be equal to 0.8 mg/l. This tailies with the given data of chlorine residual of 0.4 mg/l with a dose of 1.2 mg/l, giving chlorine demand = 1.2 – 0.4 = 0.8 mg/l. **Ans.**

WATER SOFTENING

The reduction or removal of hardness* from water is known as water *softening*. It is not essential to soften the water in order to make the water safe for public uses. The advantage of softening lies chiefly in the reduction of soap consumption, lowered cost in maintaining plumbing fixtures, and improved taste of food preparations. Hence whether or not the hardness of a water supply should be reduced, depends on the relation between the cost of treatment and the obtained resultant saving and satisfaction to the consumers. However, for industrial supplies, the softening is more important, because the hard waters are likely to cause scaling troubles in boilers and interfere in the working of dyeing systems.

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*Please refer article 8.2.2 (*iii*).

Q1. The Chick's Law: For disinfection, assume N (0) is initial number of pathogens and N (t) is remaining number of pathogens at time and given by: N (t) =N (0) × exp (-K×t); where K is disinfection rate (unit=1/unit of time) and depends on disinfectant-pathogen interaction and solution characteristics. Here R-log removal: R=-$\log_{10}$ [N (t)/N (0)].

If disinfectant concentration ($C_{disinfectant}$) and contact time (t_c) are related to each other by following equation (the Watson's Law):

$(C_{disinfectant})^n \times (t_c)$ = constant (standard unit: C in mg/L and t_c in minute)

Calculate R-log removal value for 99.9% removal? What is the remaining pathogen concentration at this removal after 1 minutes of contact time? (Assume K=0.046/min) *[3+3=6 points]*

Solution:

R-log removal = $-\log_{10}$ [$1-N_t/N_0$]

Given removal = 99.9%, so N_t/N_0= (1-99.9/100) =0.001

R-log removal = $-\log_{10}$ [0.001] =3 (answer)

For calculating remaining fraction of pathogens after 1 minute of disinfection with K=0.046/min, use the Chick's Law: **N_t/N_0 = exp (-k ×t) = exp (-0.046/min×1 min) =0.9550 (i.e., 95.50%) (answer)**

Q2. An experiment shows that a concentration of 0.1g/m^3 of free available chlorine yield a 99% kill of bacteria in 8 minutes. What contact time is required to achieve a 99.9% kill at a free available chlorine concentration of 0.05 g/m^3? Assume that Chick's Law and Watson's Law hold with n=1. *[2+2=4 points]*

Solution:

Given: For 99% kill: C= 0.1 g/m^3 and time (t) =8 minutes

Chick's Law: $N_t = N_0 \times \exp(-k \times t)$

Calculation of disinfection rate constant:

$N_t/N_0 = (1-99/100) = 0.01$ in 8 minutes

From Chick's Law: $0.01 = \exp(-k \times 8)$ **=> k = - (1/8) ln (0.01) = 0.5756/min (answer)**

Using calculated k value, calculate time for getting 99.9% kill:

$N_t/N_0 = (1-99.9/100) = 0.001$

Using Chick's Law: $0.001 = \exp(-0.5756 \times t)$ **=>t = - (1/0.5756) ln (0.001) =12 min (answer)**

Note: Watson's Law: $C^n \times t = constant$ = > $C \times t = constant$ (as n=1)

For 99.9% kill: C= 0.1 g/m^3 and time (t) =8 minutes. So, C×t value =(0.1 *1000mg/1000L)*(12 minutes) = 1.2 (mg/L)(min.)

To determine contact time using 0.05 **g/m^3**, Ct is equal for both cases.

1.2 (mg/L)(min.) = (0.05 ×1000mg/1000L)*(t minutes)

t= 24 min. (answer)

Q3. For wastewater consists of ammonia, organic matter and microorganisms), draw breakthrough curve using following information and answer following questions:

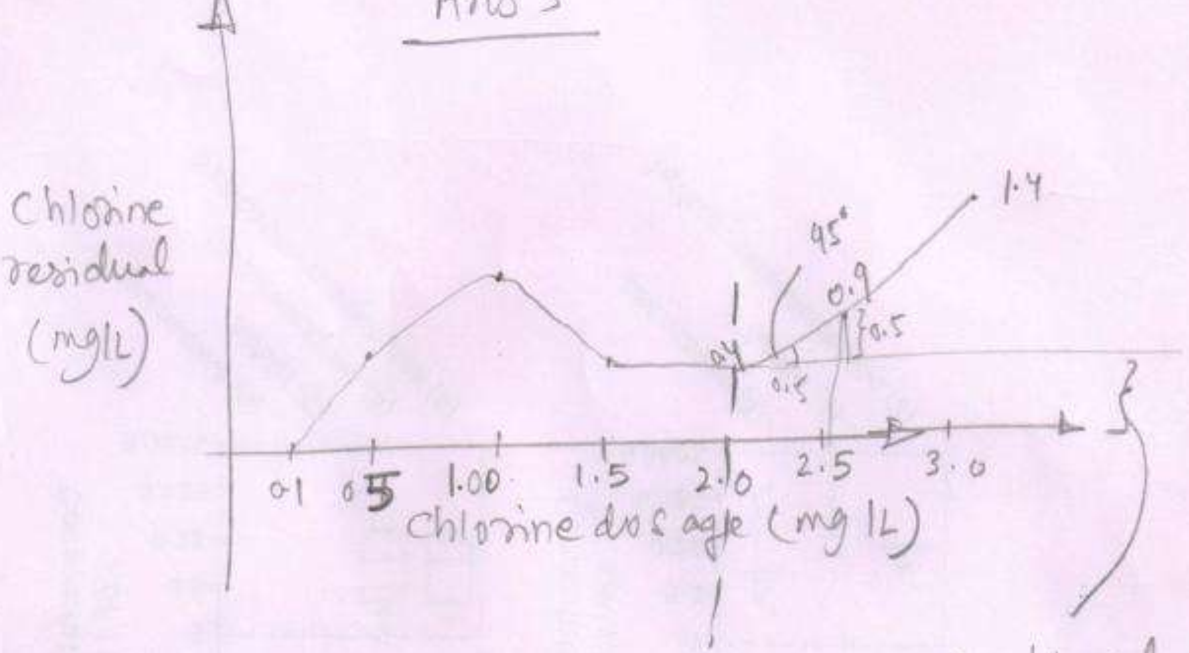
Chlorine dosage (mg/L)	0.1	0.5	1.0	1.5	2.0	2.5	3.0
Chlorine residual (mg/L)	0.0	0.4	0.8	0.4	0.4	0.9	1.4

(i) **Discuss the significance of different regions.** *[8 points]*

(ii) **Calculate chlorine dose to achieve 0.75 mg/L free available chlorine?** *[4 points]*

Solution:

Ans 3



combined chlorine

= 0.4 mg/L

@ chlorine dose = 2.0 mg/L

breakthrough starts @ 2.0 mg/L chlorine dosage.

For getting 0.75 mg/L free available chlorine:

Chlorine dose required

$= 2.0 + 0.75 = 2.75$ mg/L

Q4. Look at the following relationship between concentration of free residual chlorine and contact time required for 99% kill (Watson's Law: $C^{0.86}t_p = \lambda$ (constant) for different pathogens).

Pathogen type	Adenovirus 3	*E.coli*	Coxsackievirus A2
λ (constant)	0.098	0.24	6.3

For given chlorine dose, how long would you like to disinfect to achieve maximum removal of all pathogens? Explain the result. *[6+4=10 points]*

Solution: For given chlorine dose: high contact time is required for high λ value. So we need longest contact time for Coxsackievirus A2 than that for other pathogens. Thus to achieve maximum removal of all pathogens we need contact time equal to that of Coxsackievirus A2.

Q5. Comment on decay of adenovirus (survival = Nt/N0) using low pressure (LP) and medium pressure (MP) UV rays. Note UV dose is given in milli joules/cm^2.

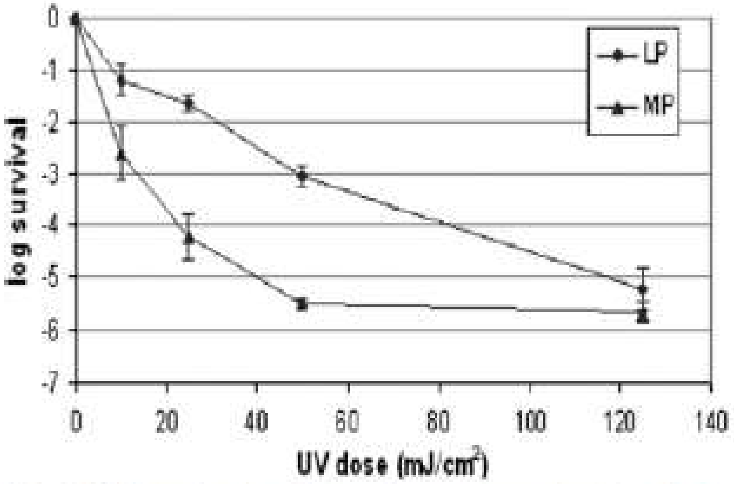


FIG. 2. UV inactivation of adenovirus as determined by cell culture infectivity assay data.

Solution:

Example: Log (survival)= (-4) (i.e., 4-log removal)

Log (Nt/N0)=-4; Nt=N0 (10^{-4}) or Removal = (1-0.0001)100= 99.99%

LP UV rays produce high decay of adenovirus compared to MP UV rays. In initial regions, LP UV rays give high disinfection rate than MP UV rays, but for high dose, both produces similar extent of virus decay.

Q6. Comment on effect of light and dark on concentration of five fecal indicators. Concentration values are shown in box plots where middle line show median value, below 25th percentile value (i.e., 25% of values lower than this) and above 75th percentile.

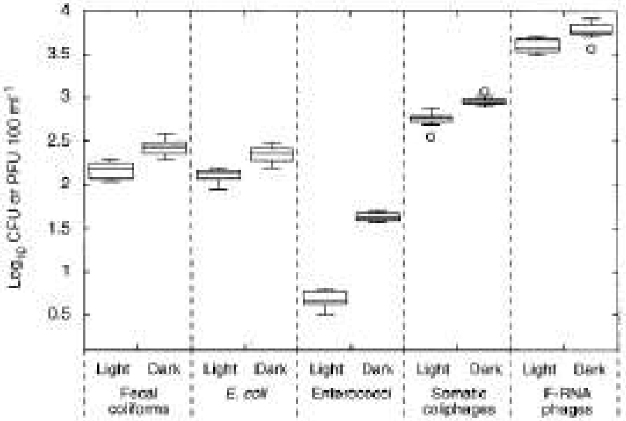


FIG. 1. Variability associated with sample collection, transport, and assay. Each box plot indicates the counts obtained from 10 samples, collected over 6 min from each chamber, after 3 h of sunlight or dark inactivation. The cross-pieces of each box plot represent (from top to bottom) maximum, upper-quartile, median, lower-quartile, and minimum values. An outlier (O) is defined as a point whose value is either above the upper quartile or below the lower quartile by 1.5 times the interquartile distance.

Solution:

Dark conditions give higher CFU values, i.e., lesser removal of pathogens than that due to light conditions.

Highest removal was observed for Enterococci than other microorganisms as lowest remaining concentration.

Lowest removal was observed for F-RNA phages than other microorganisms.

Higher removal for bacterial indicator (i.e., for fecal coliforms) was observed than viral indicators (i.e., for somatic coliphage and F-RNA phages).

Q7. Comment on effect of UV dose on virus survival for two sets of data. Note that two linear models are fit to data (this is first order data).

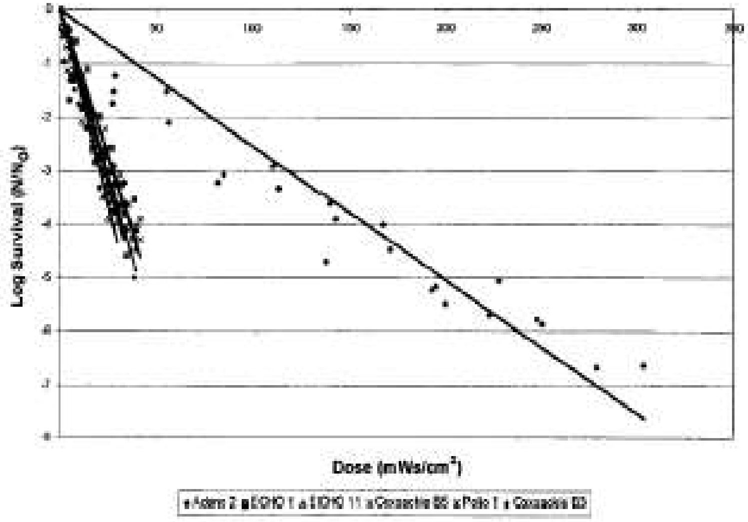


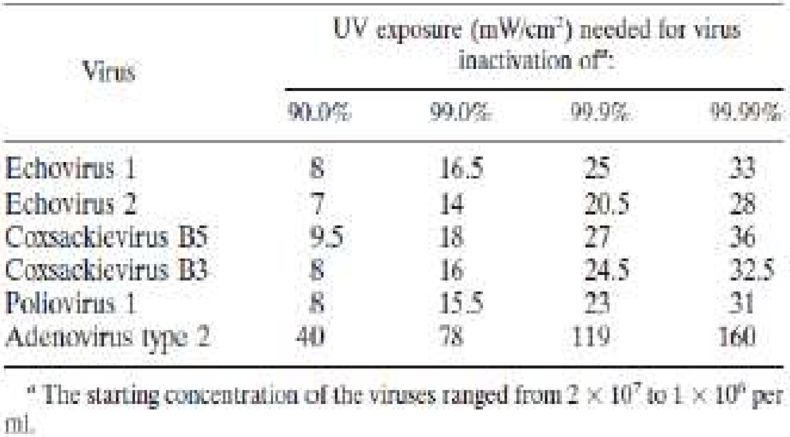
FIG. 1. Comparison of virus survival with UV dose. N, concentration of virus after exposure to the indicated dose; N_0, initial concentration of virus.

Solution:

Adeno 2 virus is disinfected at slower rate than other pathogens with increasing UV dose. For high log-removal, high UV dose is required for adeno 2 virus than other pathogens.

Both linear models appear to fit the observed decay data, however, it is difficult to comment on model fitting due to difficulty in determining goodness-of-fit of fit models to data.

Q8. Comment on dose requirements for inactivation of viruses at different levels by UV light for each of viruses studied.



Virus	UV exposure (mW/cm^2) needed for virus inactivation of^a:			
	90.0%	99.0%	99.9%	99.99%
Echovirus 1	8	16.5	25	33
Echovirus 2	7	14	20.5	28
Coxsackievirus B5	9.5	18	27	36
Coxsackievirus B3	8	16	24.5	32.5
Poliovirus 1	8	15.5	23	31
Adenovirus type 2	40	78	119	160

[a] The starting concentration of the viruses ranged from 2×10^7 to 1×10^8 per ml.

Solution:

With high UV dose, high log-removal can be achieved.

For 99.99% removal (i.e., for 4-log removal), highest UV dose is required for adenovirus type 2 than other pathogens with lowest dose required is for echovirus 2.

These data also indicate that adenovirus type 2 is most resistant to UV dose (i.e., hard to kill). Further, adenovirus 2 requires relatively higher UV dose than other pathogens (at least 4-5 times higher).

Lecture 8 - Lime-soda ash softening, Part 1

Often necessary to remove certain chemicals from water and wastewater:

Water: iron and manganese
arsenic
hardness
nitrate
radionuclides
organic chemicals

Industrial wastewater: virtually any chemical, but especially
metals
organic chemicals

Hardness

A water is "hard" if:

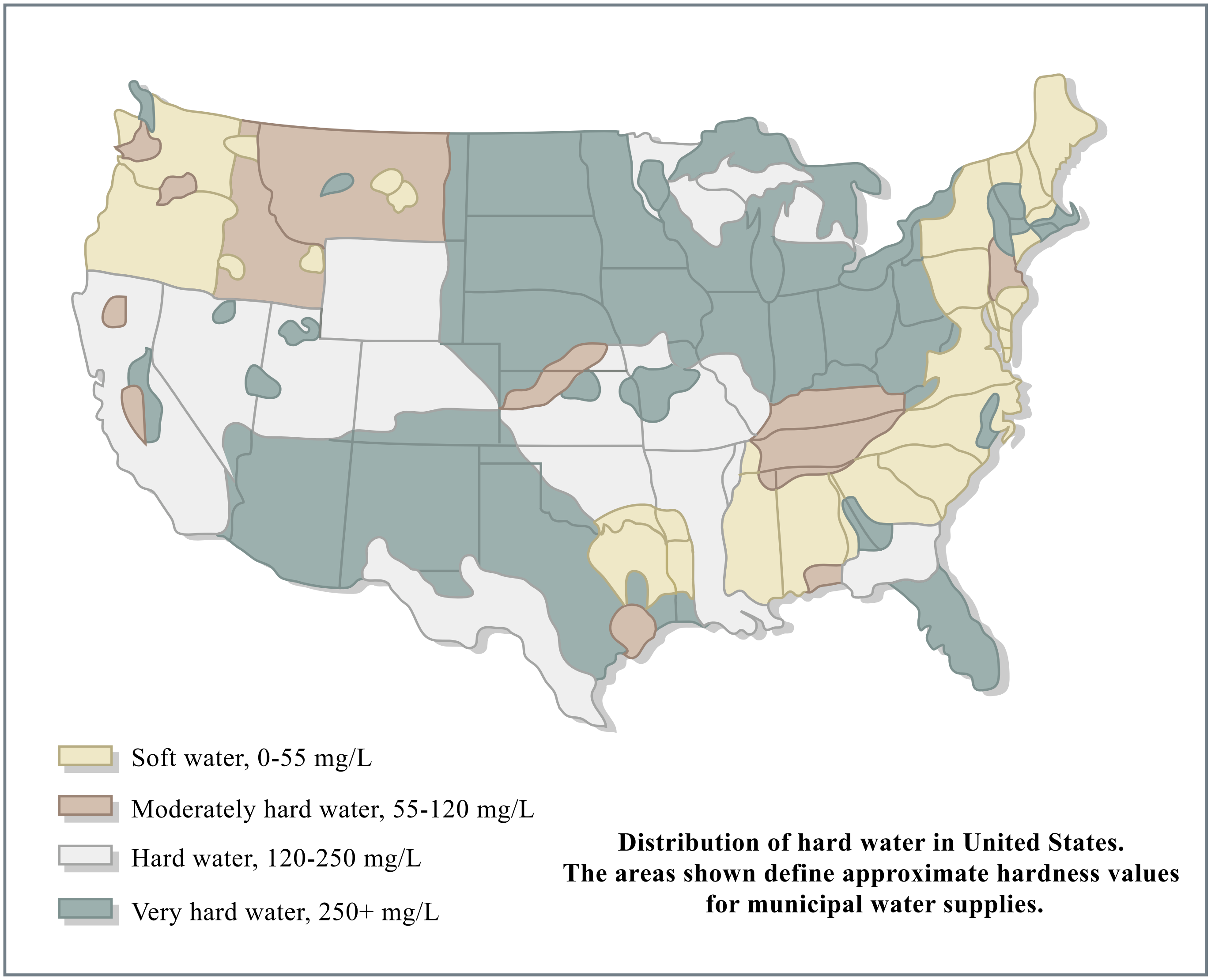
1. soap does not easily form a foam or lather
2. the water leaves scale in hot-water pipes, boilers, etc.

Hardness arises from divalent metal ions in the water:

$$\left.\begin{array}{c} Ca^{++} \\ Mg^{++} \\ Sr^{++} \\ Fe^{++} \\ Mn^{++} \end{array}\right\}$$

come from natural rocks in source area,
especially: limestones:
calcite $CaCO_3$
dolomite $CaMg(CO_3)_2$

Prevalence of hard water nationwide reflects geology - see Figure 19-8 from MWH, 2005



Distribution of hard water in United States. The areas shown define approximate hardness values for municipal water supplies.

Figure by MIT OCW.

Adapted from: Dekker, Marcel. *Water and Water Pollution Handbook*. Edited by L. Ciaccio. New York, NY: 1971.

Different definitions of hardness

Total hardness = $\Sigma\,[M^{++}] \times \frac{50}{\text{eq. wt. of } M^{++}}$

Hardness is expressed in equivalents of $CaCO_3$

Ca − MW = 40 × 1 = 40
C − MW = 12 × 1 = 12
O − MW = 16 × 3 = 48

$\overline{100}$ = MW

Since Ca^{++} and $CO_3^{=}$
have valence of 2,
equivalent weight of $CaCO_3$ = 100/2 = 50

Example water (Ex 11.4, pg. 445 of Y & H):

CO_2 = 8.8 mg/L as CO_2
Ca^{2+} = 70 mg/L
Mg^{2+} = 9.7 mg/L
Na^{+} = 6.9 mg/L

Alk (HCO_3^-) = 115 mg/L as $CaCO_3$
SO_4^{2-} = 96 mg/L
Cl^- = 10.6 mg/L

M^{++}	conc (mg/L)	MW	eq. wt.	hardness
Ca^{2+}	70	40	20	175
Mg^{2+}	9.7	24.4	12.2	39.8
				214.8

pretty hard water!

Most hardness is due to Ca and Mg

Ca hardness = that due to Ca
Mg hardness = that due to Mg

Total hardness $\simeq$ Ca hardness + Mg hardness

Carbonate hardness = part of total hardness equivalent to carbonate plus bicarbonate alkalinity

Refresher on alkalinity [Alk]:

Alk = capacity for solutes to neutralize a strong acid

$= \Sigma$ [strong bases] $-$ Σ [strong acids]
in equivalents be liter

Strong acids are those that completely dissociate in water: HCl, H_2SO_4, HNO_3, HBr, HI, $HClO_4$

Strong bases completey dissociate: $NaOH$, KOH, $Ca(OH)_2$, $Mg(OH)_2$, $LiOH$, $RbOH$, $Sr(OH)_2$, $Ba(OH)_2$

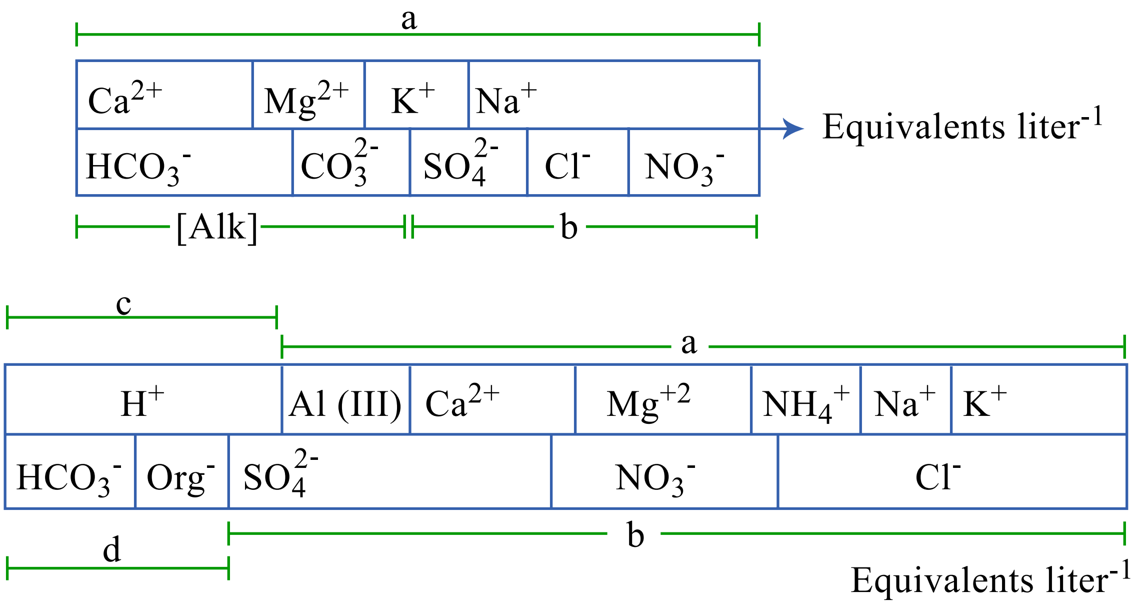
$$[Alk] = [Na^+] + [K^+] + 2[Ca^{++}] + 2[Mg^{++}] - [Cl^-] - 2[SO_4^=] - [NO_3^-]$$

$$= \Sigma[SB] - \Sigma[SA]$$

Can also find charge balance assuming carbonates dominate system

$$\Sigma[SB] + [H^+] = \Sigma[SA] + [OH^-] + 2[CO_3^=] + [HCO_3^-]$$

$$\Sigma[SB] - \Sigma[SA] = [Alk]$$
$$= [OH^-] - [H^+] + 2[CO_3^=] + [HCO_3^-]$$



Natural water charge balance for an alkaline system (Alk = a-b) and an acid system (Alk = a-b = d-c).

Figure by MIT OCW.

Adapted from: Schnoor, J. L. *Environmental Modeling:fate and transport of pollutants in water, air, and soil*. New York, NY: John Wiley & Sons. 1996.

Carbonate hardness (for [Alk] in terms of $CaCO_3$)

If [Alk] < total hardness,

then carbonate hardness = [Alk]

If [Alk] > total hardness,

then carbonate hardness = total hardness

Carbonate hardness causes scaling at high temps:

$$Ca^{++} + 2\,HCO_3^- \xrightarrow{\Delta} CaCO_3\downarrow + CO_2 + H_2O$$

Noncarbonate hardness = total hardness − carbonate hardness

For water treatment, carbonate hardness is removed by adding lime $Ca(OH)_2$:

First lime reacts with any CO_2:

1. $CO_2 + Ca(OH)_2 \rightarrow CaCO_3\downarrow + H_2O$

Then lime reacts to remove carbonate hardness

2. $Ca^{2+} + 2HCO_3^- + Ca(OH)_2 \rightarrow 2CaCO_3\downarrow + 2H_2O$
3. $Mg^{2+} + 2HCO_3^- + 2Ca(OH)_2 \rightarrow 2CaCO_3\downarrow + Mg(OH)_2\downarrow + 2H_2O$

And finally to remove non-carbonate Mg hardness

4. $Mg^{2+} + SO_4^= + Ca(OH)_2 \rightarrow Mg(OH)_2\downarrow + Ca^{2+} + SO_4^=$
5. $Mg^{2+} + 2Cl^- + Ca(OH)_2 \rightarrow Mg(OH)_2\downarrow + Ca^{2+} + 2Cl^-$

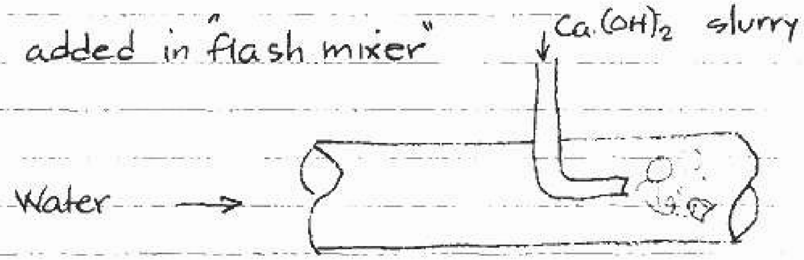
Reactions 4. and 5. simply swap Mg noncarbonate hardness for Ca noncarbonate hardness

Reactions 1 - 5 remove magnesium hardness and calcium carbonate hardness. Calcium noncarb hardness in original solution plus that created removing Mg hardness remain.

For waters low in Mg and with carbonate Ca hardness, this would be sufficent treatment - called "single - stage lime treatment", "single-stage softening", or "undersoftening"

Typical process in water-treatment plant:

$CaCO_3$ added in "flash mixer"



Water/lime mix goes into flocculator, then clarifier to precipitate and remove $CaCO_3$ (T_R = 1-2 hrs)

Water is then "recarbonated"

Recarbonation is needed because addition of $Ca(OH)_2$ raises pH of water to 10.2 to 10.5

Recarbonation consists of bubbling CO_2 through treated water, lowering pH to 8.7 to 9.0:

6. $Ca(OH)_2 + CO_2 \rightarrow CaCO_3\downarrow + H_2O$

7. $CO_3^{2-} + CO_2 + H_2O \rightarrow 2\ HCO_3^-$

Recarbonation is also done to "stabilize" the water

If water is supersaturated with $CaCO_3$, it will precipitate as "scale"

If water is undersaturated, water can be "aggressive" and cause pipe corrosion

Ideal is to keep water slightly oversaturated to maintain thin protective coat of $CaCO_3$ on inside of pipe

Scale precipitation involves two chemical reactions.

$$Ca^{2+} + CO_3^{2-} \rightleftharpoons CaCO_3(s) \qquad 1/K_s$$

$$HCO_3^- \rightleftharpoons H^+ + CO_3^{2-} \qquad K_2$$

$$K_s = \frac{[Ca^{2+}][CO_3^{2-}]}{\underset{1}{[\cancel{CaCO_3(s)}]}}$$

$$K_2 = \frac{[CO_3^{2-}][H^+]}{[HCO_3^-]}$$

Overall reaction:

$$Ca^{2+} + HCO_3^- \rightleftharpoons CaCO_3(s) + H^+$$

$$K = \frac{[Ca^{2+}][HCO_3^-]}{[H^+]} = \frac{K_s}{K_2}$$

Rearrange to get: $$[H^+] = \frac{[Ca^{2+}][HCO_3^-]}{K}$$

$$[H^+] = \frac{[Ca^{2+}][HCO_3^-]}{K}$$

$$\log[H^+] = \log[Ca^{2+}] + \log[HCO_3^-] - \log K$$

$$-\log[H^+] = -\log[Ca^{2+}] - \log[HCO_3^-] + \log(K_s/K_2)$$

$$pH_{eq} = pCa + pHCO_3^- + \log(K_s/K_2)$$
$$\approx pCa + p[Alk] + \log(K_s/K_2)$$

↑ to denote this is pH at $CaCO_3$ equilibrium

$I = pH_{actual} - pH_{eq} \equiv SI$

Langlier Stability Index

$I > 0$ → $CaCO_3$ precipitates ($pH_{actual} > pH_{eq}$)

$I = 0$ → stable

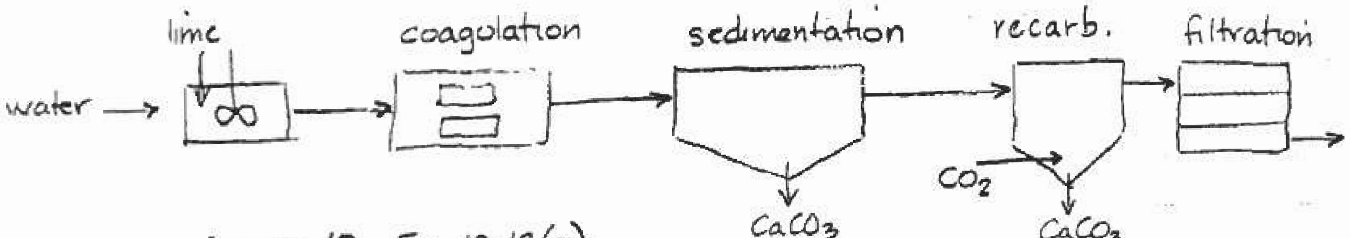
$I < 0$ → $CaCO_3$ dissolves

$I = 0.2$ is desirable – carbonation steps seeks to set $I \doteq 0.2$

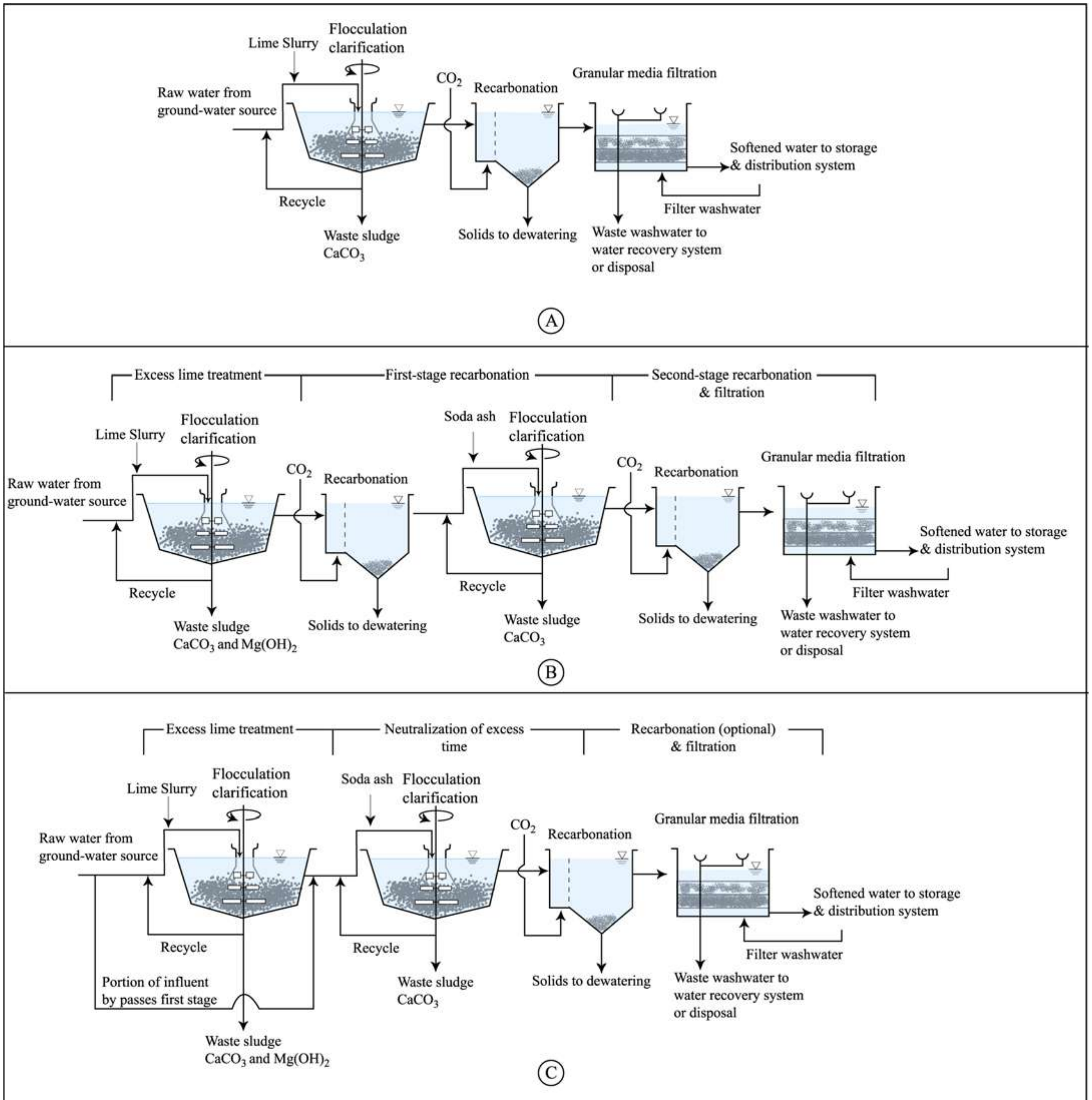
K_2, K_s are functions of temp.

$pK_s = 8.4$ $pK_2 = 10.4$ at $T = 15$ C

conventional lime treatment process looks like:



See pg 10 Fig 19-12(a)



Process flow diagram of common softening treatment techniques: (a) Single- stage lime treatment; (b) two-stage excess lime-soda treatment; (c) split-flow lime treatment

Figure by MIT OCW.

Adapted from: MWH, J. C. Crittenden, R. R. Trussell, D. W. Hand, K. J. Howe, and G. Tchobanoglous. *Water Treatment: Principles and Design*. 2nd ed. Hoboken, NJ: John Wiley & Sons, 2005, p. 1601.

Lecture 9 - Lime-soda ash softening, Part 2

For waters with non-carbonate hardness, single-stage softening is insufficient. Leftover hardness is removed by addition of soda ash (Na_2CO_3)

8. $Ca^{2+} + SO_4^{2-} + Na_2CO_3 \rightarrow CaCO_3\downarrow + 2Na^+ + SO_4^{2-}$

9. $Ca^{2+} + 2Cl^- + Na_2CO_3 \rightarrow CaCO_3\downarrow + 2Na^+ + 2Cl^-$

Practical limits of lime-soda ash softening are dictated by solubility of precipitates: $CaCO_3$ & $Mg(OH)_2$

Ca: 30 mg/L as $CaCO_3$
Mg: 10 mg/L as $CaCO_3$
Total hardness: 40 mg/L as $CaCO_3$

In practice, residual hardness = 50 to 80 mg/L

This water has high pH and needs to be recarbonated

Lime-soda ash treatment is usually treated by "two-stage softening" also called "excess-lime treatment" and "split recarbonation treatment"

See Lecture 8, page 10

Split treatment is similar, except only part of water is treated with lime. Other part by-passes lime treatment and gets soda-ash treatment along with lime-treated water
The CO_2 in untreated water neutralizes high pH in lime-treated water and recarb. is not needed

Water split is computed such that enough Mg is removed in lime-treated water to meet target Mg level in combined finished water

Computing chemical doses for lime soda ash softening - Example 11.4 from Viessman and Hammer, pg. 445 - pg 3 and 4

CO_2 = 8.8 mg/L as CO_2	
Ca^{++} = 70 mg/L	Alk = 115 mg/L as $CaCO_3$
Mg^{++} = 9.7 mg/L	SO_4^{2-} = 96 mg/L
Na^+ = 6.9 mg/L	Cl^- = 10.6 mg/L

Easiest method is to construct a table that converts all concentrations to equivalent concentrations, and then to equivalents of $CaCO_3$

Also use chart from VH Fig 11.8, pg 446

	conc (mg/L)	MW (gm/mole)	equiv (eq/molecule)	eq wt (gm/mole-eq)	meq/L	mg/L as $CaCO_3$
CO_2	8.8	44.0	2	22.0	0.4	20.0
Ca^{2+}	70	40.0	2	20.0	3.5	175.
Mg^{2+}	9.7	24.4	2	12.2	0.80	39.8
Na^+	6.9	23.0	1	23.0	0.30	15.0
					4.6	229.8
Alk	115	100	2	50.0	2.3	115.0
SO_4^{2-}	96	96.0	2	48.0	2.0	100.0
Cl^-	10.6	35.5	1	35.5	0.30	14.9
					4.6	229.9

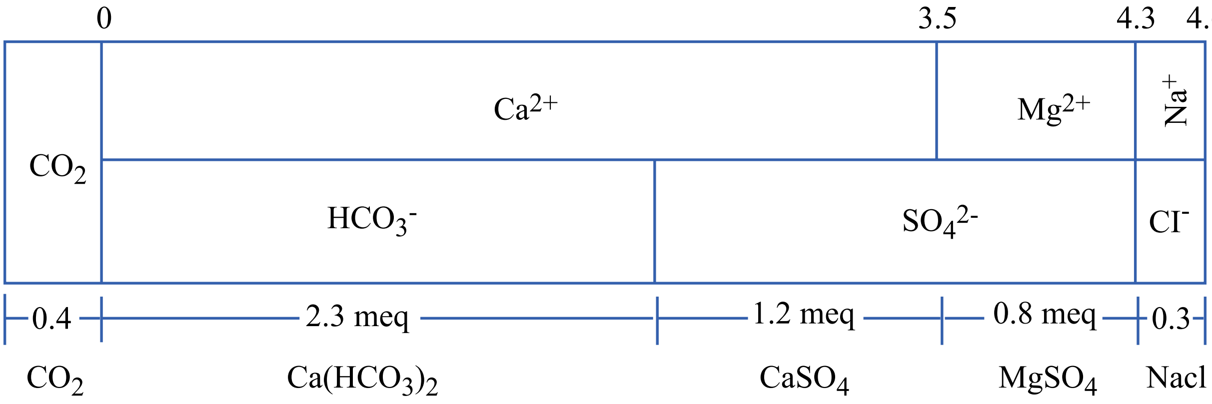
Total hardness = $Ca^{2+} + Mg^{2+}$ = 175 + 39.8 = 214.8 mg/L as $CaCO_3$

Carbonate hardness = [Alk] = 115 mg/L as $CaCO_3$

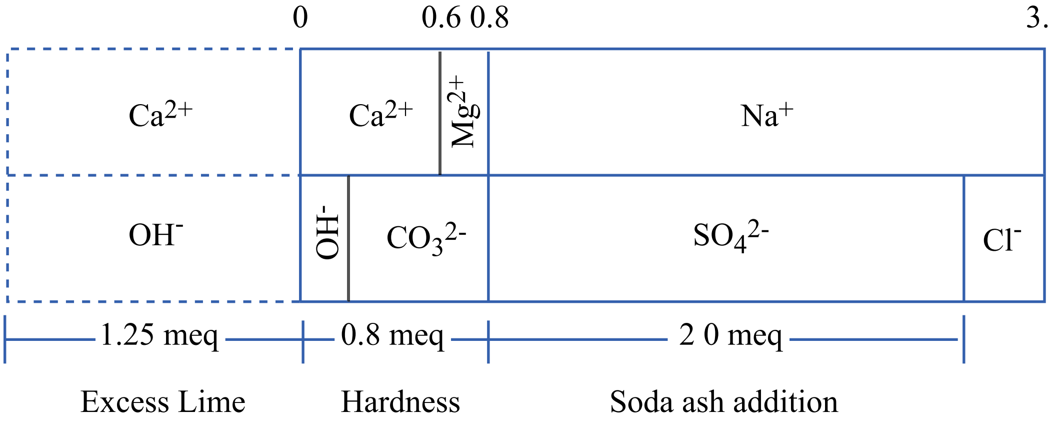
Noncarbonate hardness = TH - CH = 99.8 mg/L as $CaCO_3$

Mg noncarbonate hardness = 39.8 mg/L as $CaCO_3$

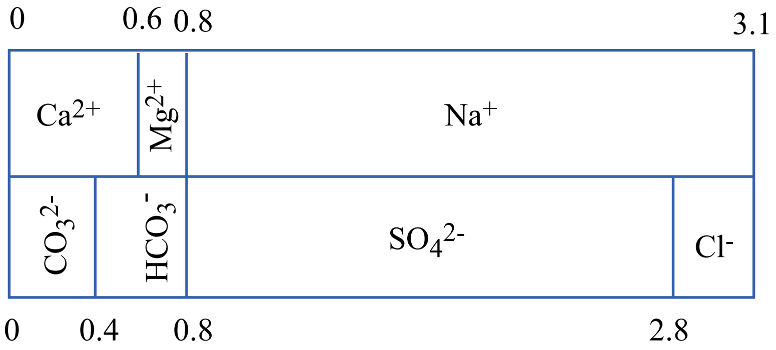
Milliequivalent Bar Graph for Example 11.4



(A) Bar graph & hypothetical chemical combinations in the raw water



(B) Bar graph of the water after lime & soda ash additions & settling but before recarbonation.

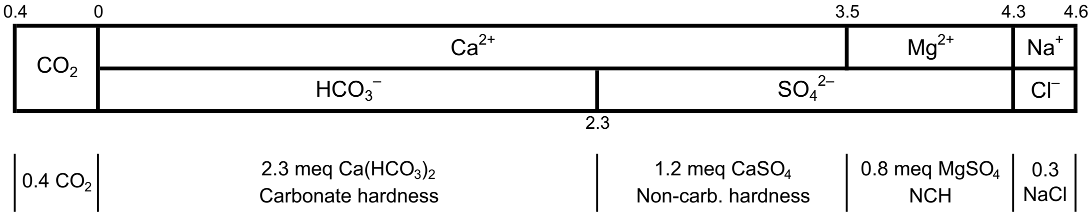


(C) Bar Graph of the water after two-stage recarbonation & final filtration

Figure by MIT OCW.

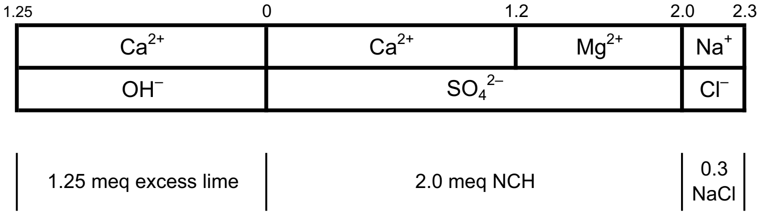
Adapted from: Viessman, W., Jr., and M. J. Hammer. *Water Supply and Pollution Control*. 7th ed. Upper Saddle River, NJ: Pearson Education, Inc., 2005, p. 446.

Before treatment:



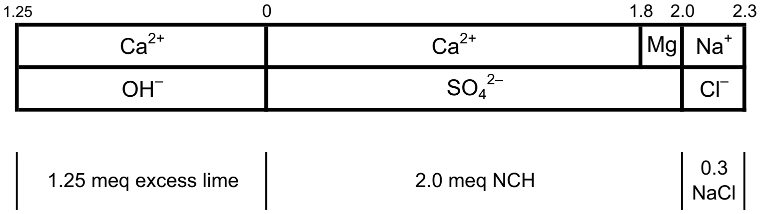
After treatment with lime $Ca(OH)_2$ and intermediate reaction to remove carbonate hardness:

(chemical equations 1, 2, & 3)



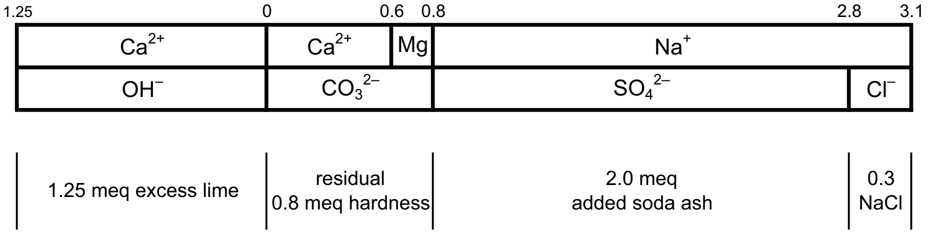
After treatment with lime and intermediate reaction to remove noncarbonate Mg hardness:

(chemical equations 4 & 5)



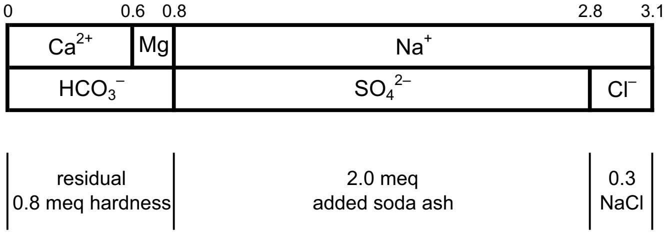
After treatment with soda ash Na_2CO_3:

(chemical equations 8 & 9)



After recarbonation:

(chemical equations 6 & 7)



Lime required

For CO_2 — 20.0 mg/L as $CaCO_3$
For carbonate hardness — 115.0
For Mg noncarbonate hardness — 39.8
174.8 mg/L as $CaCO_3$
(3.5 meq)

Convert from $CaCO_3$ to CaO

$$\frac{CaO}{CaCO_3} = \frac{40+16}{40+12+3\times16} = \frac{56}{100} = \frac{28}{50}$$

174.8 mg/L as $CaCO_3$ = 97.9 mg/L as CaO

Include excess lime of 35 mg/L

Req'd lime = 133 mg/L

Soda Ash for noncarbonate hardness

NCH = 99.8 mg/L as $CaCO_3$ (2.0 meq)

(recall that Mg NCH was treated with lime
but simply swaps Ca for Mg, so still
needs treatment with soda ash)

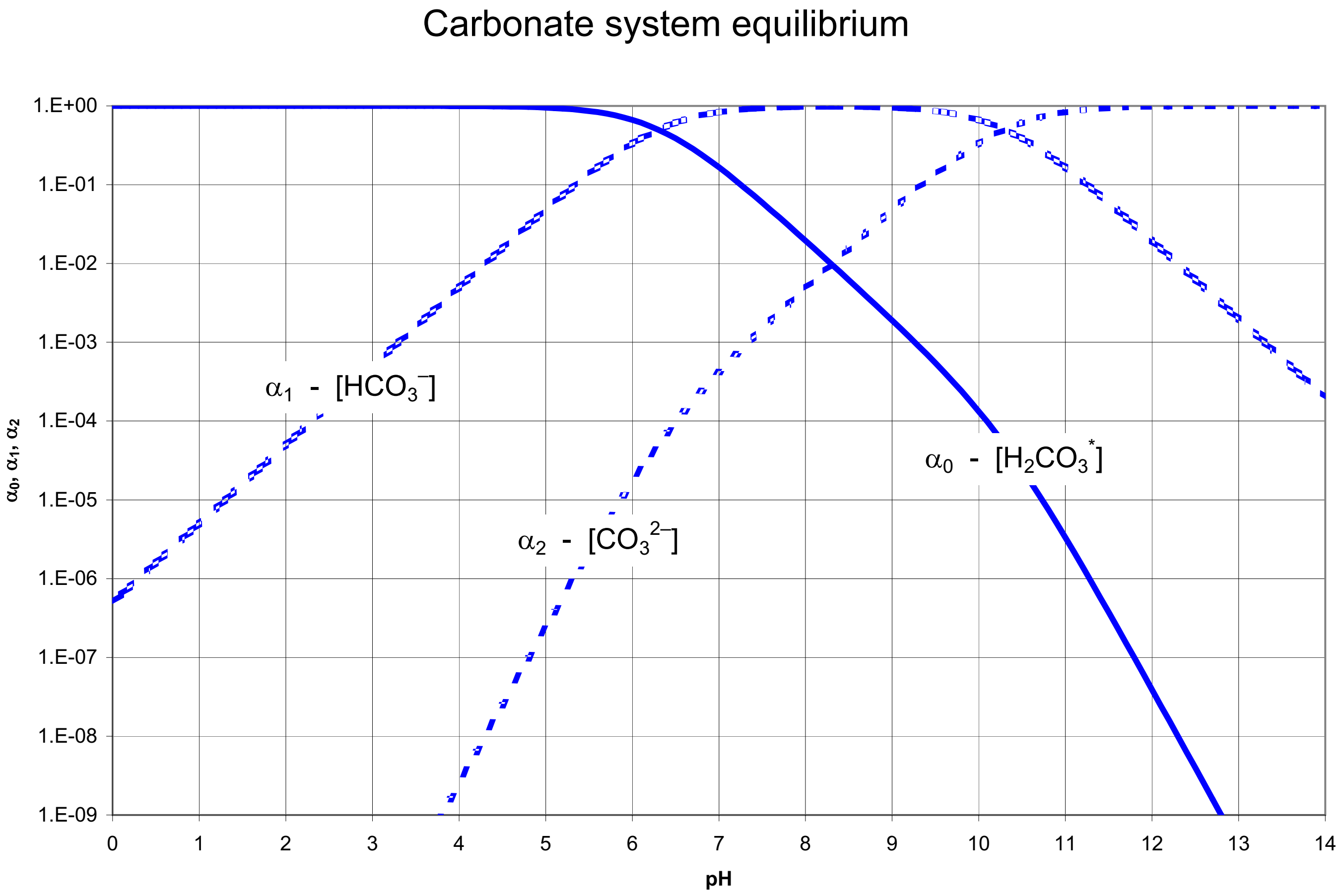
Reqd Soda Ash — 99.8 mg/L as $CaCO_3$

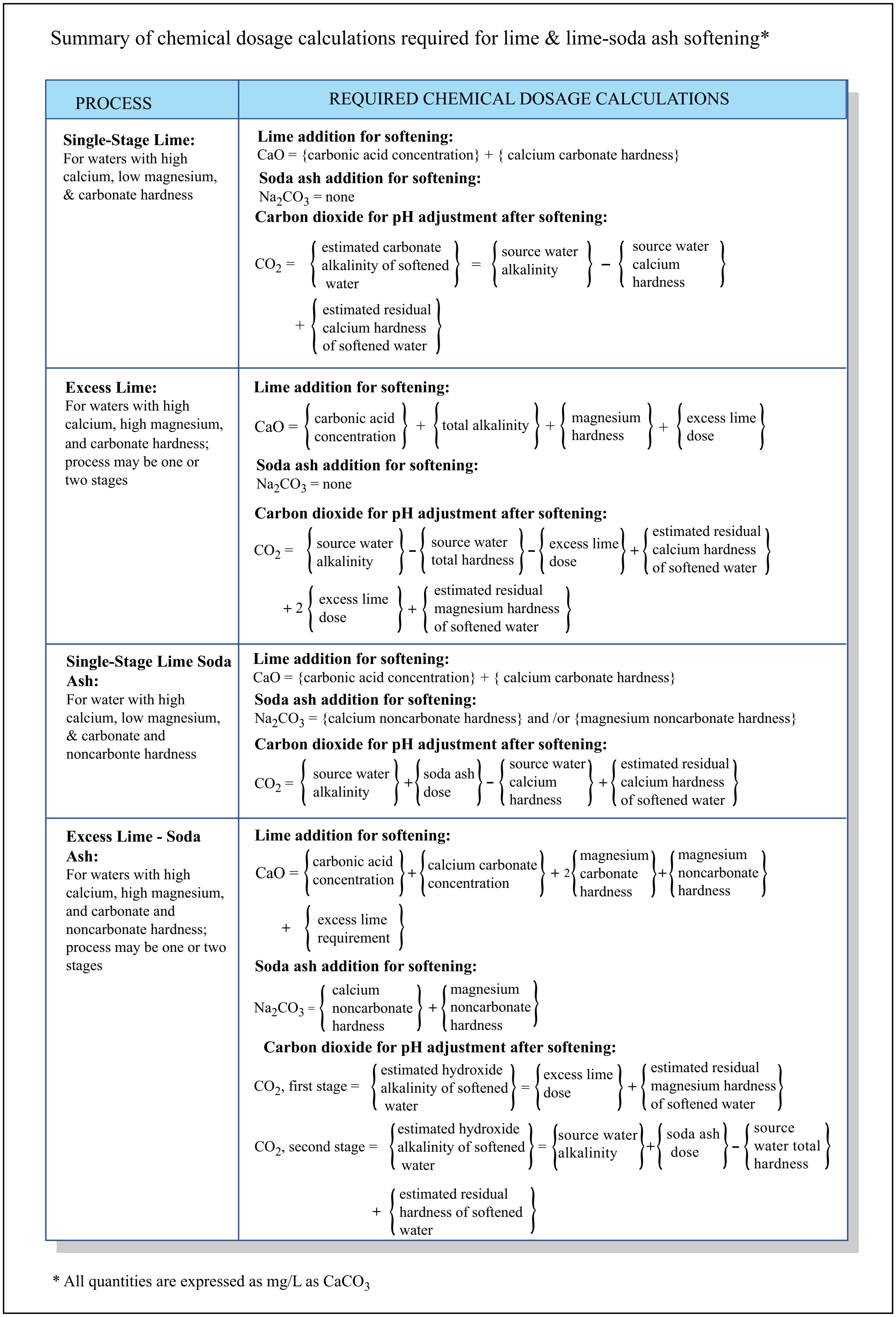
Convert to Na_2CO_3:

$$\frac{Na_2CO_3}{CaCO_3} = \frac{2\times23+12+3\times16}{40+12+3\times16} = 1.06$$

Reqd soda ash = 1.06 × 99.8 = 106 mg/L

Note that pg 14 shows HCO_3^- but will actually be an equilibrium
between CO_2, HCO_3^-, CO_3^{2-} depending on pH per pg 6


Carbonate system equilibrium
1.E+00
1.E-01
1.E-02
1.E-03
1.E-04
1.E-05
1.E-06
1.E-07
1.E-08
1.E-09
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
pH
$\alpha_0, \alpha_1, \alpha_2$
α_1 - $[HCO_3^-]$
α_0 - $[H_2CO_3^*]$
α_2 - $[CO_3^{2-}]$



Summary of chemical dosage calculations required for lime & lime-soda ash softening*

PROCESS	REQUIRED CHEMICAL DOSAGE CALCULATIONS
Single-Stage Lime: For waters with high calcium, low magnesium, & carbonate hardness	**Lime addition for softening:** CaO = {carbonic acid concentration} + { calcium carbonate hardness} **Soda ash addition for softening:** Na_2CO_3 = none **Carbon dioxide for pH adjustment after softening:** $CO_2 = \left\{\begin{matrix}\text{estimated carbonate}\\ \text{alkalinity of softened}\\ \text{water}\end{matrix}\right\} = \left\{\begin{matrix}\text{source water}\\ \text{alkalinity}\end{matrix}\right\} - \left\{\begin{matrix}\text{source water}\\ \text{calcium}\\ \text{hardness}\end{matrix}\right\}$ $+ \left\{\begin{matrix}\text{estimated residual}\\ \text{calcium hardness}\\ \text{of softened water}\end{matrix}\right\}$
Excess Lime: For waters with high calcium, high magnesium, and carbonate hardness; process may be one or two stages	**Lime addition for softening:** $CaO = \left\{\begin{matrix}\text{carbonic acid}\\ \text{concentration}\end{matrix}\right\} + \left\{\text{total alkalinity}\right\} + \left\{\begin{matrix}\text{magnesium}\\ \text{hardness}\end{matrix}\right\} + \left\{\begin{matrix}\text{excess lime}\\ \text{dose}\end{matrix}\right\}$ **Soda ash addition for softening:** Na_2CO_3 = none **Carbon dioxide for pH adjustment after softening:** $CO_2 = \left\{\begin{matrix}\text{source water}\\ \text{alkalinity}\end{matrix}\right\} - \left\{\begin{matrix}\text{source water}\\ \text{total hardness}\end{matrix}\right\} - \left\{\begin{matrix}\text{excess lime}\\ \text{dose}\end{matrix}\right\} + \left\{\begin{matrix}\text{estimated residual}\\ \text{calcium hardness}\\ \text{of softened water}\end{matrix}\right\}$ $+ 2\left\{\begin{matrix}\text{excess lime}\\ \text{dose}\end{matrix}\right\} + \left\{\begin{matrix}\text{estimated residual}\\ \text{magnesium hardness}\\ \text{of softened water}\end{matrix}\right\}$
Single-Stage Lime Soda Ash: For water with high calcium, low magnesium, & carbonate and noncarbonte hardness	**Lime addition for softening:** CaO = {carbonic acid concentration} + { calcium carbonate hardness} **Soda ash addition for softening:** Na_2CO_3 = {calcium noncarbonate hardness} and /or {magnesium noncarbonate hardness} **Carbon dioxide for pH adjustment after softening:** $CO_2 = \left\{\begin{matrix}\text{source water}\\ \text{alkalinity}\end{matrix}\right\} + \left\{\begin{matrix}\text{soda ash}\\ \text{dose}\end{matrix}\right\} - \left\{\begin{matrix}\text{source water}\\ \text{calcium}\\ \text{hardness}\end{matrix}\right\} + \left\{\begin{matrix}\text{estimated residual}\\ \text{calcium hardness}\\ \text{of softened water}\end{matrix}\right\}$
Excess Lime - Soda Ash: For waters with high calcium, high magnesium, and carbonate and noncarbonate hardness; process may be one or two stages	**Lime addition for softening:** $CaO = \left\{\begin{matrix}\text{carbonic acid}\\ \text{concentration}\end{matrix}\right\} + \left\{\begin{matrix}\text{calcium carbonate}\\ \text{concentration}\end{matrix}\right\} + 2\left\{\begin{matrix}\text{magnesium}\\ \text{carbonate}\\ \text{hardness}\end{matrix}\right\} + \left\{\begin{matrix}\text{magnesium}\\ \text{noncarbonate}\\ \text{hardness}\end{matrix}\right\}$ $+ \left\{\begin{matrix}\text{excess lime}\\ \text{requirement}\end{matrix}\right\}$ **Soda ash addition for softening:** $Na_2CO_3 = \left\{\begin{matrix}\text{calcium}\\ \text{noncarbonate}\\ \text{hardness}\end{matrix}\right\} + \left\{\begin{matrix}\text{magnesium}\\ \text{noncarbonate}\\ \text{hardness}\end{matrix}\right\}$ **Carbon dioxide for pH adjustment after softening:** $CO_2\text{, first stage} = \left\{\begin{matrix}\text{estimated hydroxide}\\ \text{alkalinity of softened}\\ \text{water}\end{matrix}\right\} = \left\{\begin{matrix}\text{excess lime}\\ \text{dose}\end{matrix}\right\} + \left\{\begin{matrix}\text{estimated residual}\\ \text{magnesium hardness}\\ \text{of softened water}\end{matrix}\right\}$ $CO_2\text{, second stage} = \left\{\begin{matrix}\text{estimated hydroxide}\\ \text{alkalinity of softened}\\ \text{water}\end{matrix}\right\} = \left\{\begin{matrix}\text{source water}\\ \text{alkalinity}\end{matrix}\right\} + \left\{\begin{matrix}\text{soda ash}\\ \text{dose}\end{matrix}\right\} - \left\{\begin{matrix}\text{source}\\ \text{water total}\\ \text{hardness}\end{matrix}\right\}$ $+ \left\{\begin{matrix}\text{estimated residual}\\ \text{hardness of softened}\\ \text{water}\end{matrix}\right\}$

* All quantities are expressed as mg/L as $CaCO_3$

Figure by MIT OCW.

Adapted from: MWH, J. C. Crittenden, R. R. Trussell, D. W. Hand, K. J. Howe, and G. Tchobanoglous. *Water Treatment: Principles and Design*. 2nd ed. Hoboken, NJ: John Wiley & Sons, 2005, pp. 1610-1611.

Lecture 10 Chemical Removal - Adsorption and Ion Exchange

Definitions:

Adsorption - mass transfer of chemicals in liquid phase onto solid phase

adsorption - chemicals adhere to surface of solid (dominant mechanism)

absorption - chemicals penetrate into solid, forming solid solution

sorption - includes both

Adsorbent - adsorbing phase

Adsorbate - chemical being adsorbed

Adsorption is used in drinking-water treatment to remove organic contaminants:

taste and odor-causing chemicals
synthetic organic chemicals
color forming organics
some disinfection by-product precursors (but not THMs)

Adsorbent of choice is activated carbon
Made in two-step process:

1. Carbonaceous material (wood, coal, coconut shells) is heated in oxygen-starved environment to liberate carbon (carbonization)

2. Carbonized material is exposed to steam or hot CO_2 to cause pores and fissures to form (activation)

1. Creates carbon to which organic chems will sorb,
2. Increases surface area available for sorption

Source: NIOSH, 2003. Guidance for Filtration and Air-Cleaning Systems to Protect Building Environments from Airborne Chemical, Biological, or Radiological Attacks. DHHS (NIOSH) Pub No. 2003-136. National Institute for Occupational Safety and Health, Centers for Disease Control, Cincinnati, Ohio. April 2003. http://www.cdc.gov/niosh/docs/2003-136/2003-136c.html. Accessed February 21, 2006.

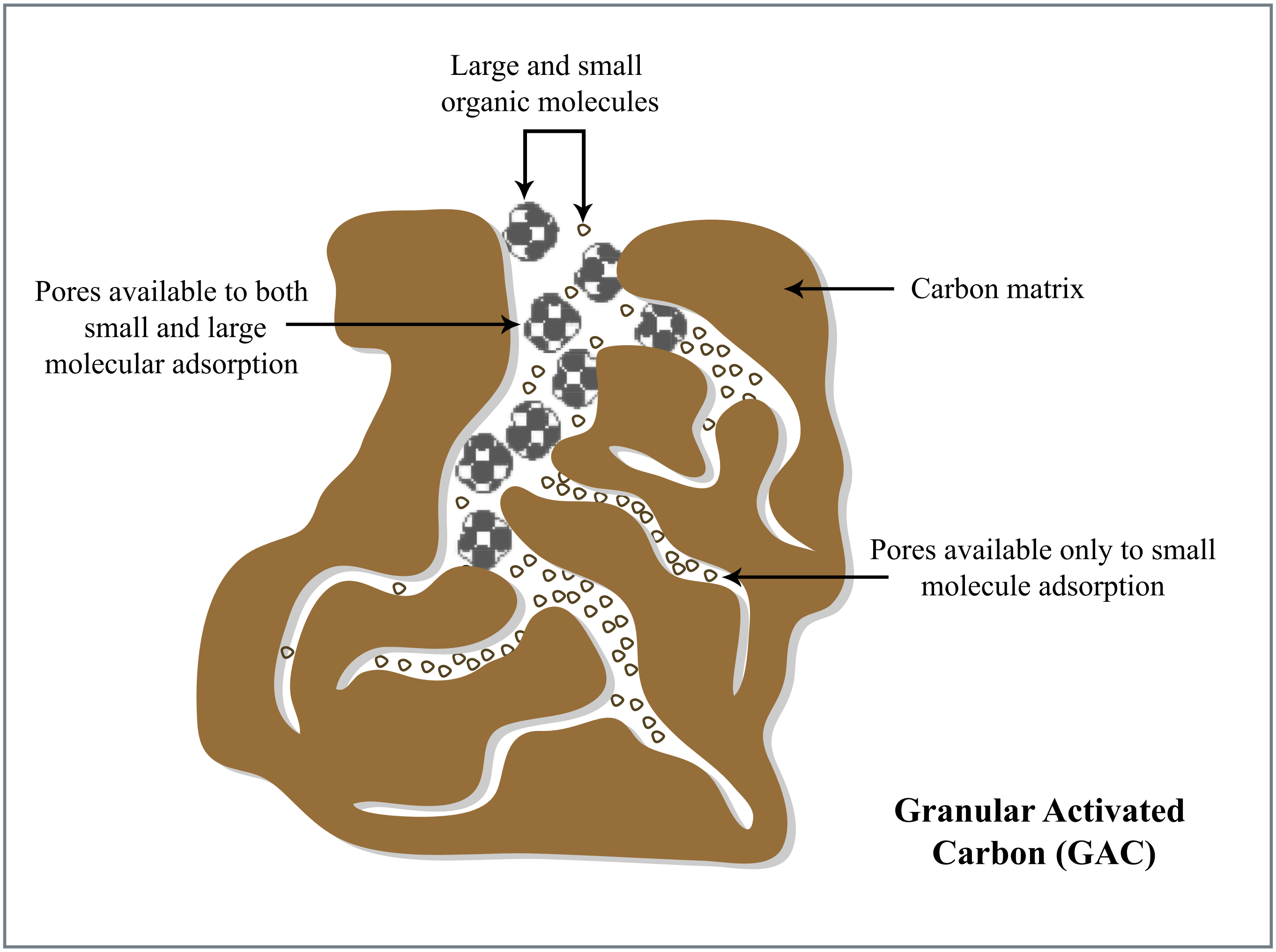


Figure by MIT OCW.

Adapted from Culp, G.L., and R.L. Culp. *New Concepts in Water Purification*. New York, NY: Van Nostrand Reinhold Co. 1974

Adsorption depends on properties of activated carbon, chemistry of adsorbate, pH and temp. of water – each application requires development of adsorption isotherm

Most bonding is physical, occurring by various mechanisms such as van der Waals force

Some bonding is chemical and is generally irreversible.

Adsorption process is quantified via an adsorption isotherm which can take multiple forms

Isotherm relates q_A to C

$$q_A = \text{solid phase conc} = \frac{\text{mass of absorbate}}{\text{mass of absorbent}} = X \text{ in R/R}$$

C_A = liquid phase conc of absorbate

Very simplest is linear isotherm:

$$q_A = K_d C_A \qquad = \quad K C_e \text{ in R/R}$$

$$K_d = \text{partition coeff} \quad \frac{mg/kg}{mg/L} = \frac{L}{kg}$$

Another possible isotherm is the Langmuir

Assumes reaction:

$$S_V + A \rightleftharpoons S \cdot A$$

$S \cdot A$ ← adsorbate bound to absorbent site $\left(\frac{mol}{L}\right)$

A ← adsorbate in solution (mol/L soln)

S_V ← vacant adsorption site on absorbent in mol/L absorbent

At equilibrium

$$K_{ad} = \frac{[S \cdot A]}{[S_V][A]}$$

K_{ad} in L/mg
$[A]$ in mg/L

Consider total number of sites to be fixed:

$$S_T = [S_V] + [S \cdot A] \qquad (\text{moles}/m^2)$$

$$S_T = \frac{[S \cdot A]}{K_{ad}[A]} + [S \cdot A]$$

Solve for $[S \cdot A]$ and use $C_A = [A]$

$$[S \cdot A] = \frac{S_T}{1 + 1/K_{ad}C_A} = \frac{K_{ad} C_A S_T}{1 + K_{ad}C_A}$$

Need expression for q_A mg adsorbate / g adsorbent

$$q_A = [S \cdot A] \cdot A_{ad} \cdot MW_A$$

A_{ad} ← surface area per gm adsorbent m^2/g

MW_A ← molecular wt of adsorbate g/mol

$$q_A = [S \cdot A] \cdot A_{ad} \cdot MW_A$$

$$q_A = \frac{K_{ad}\, C_A\, S_T}{1 + K_{ad} C_A} \cdot A_{ad} \cdot MW_A = \frac{Q_M K_{ad} C_A}{1 + K_{ad} C_A}$$

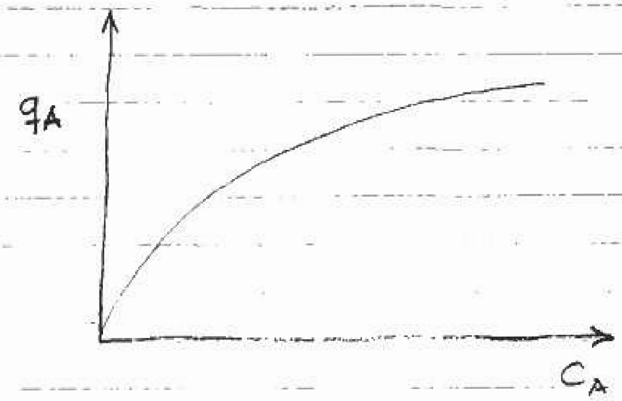
Q_M = max. adsorbent-phase conc of adsorbate,
occurs when all sites $(S_T A_{ad})$ are
saturated with adsorbate

K_{ad} = Langmuir adsorption constant (L/mg)

$\frac{C_A}{q_A}$ plots as straight line vs C_A

slope = $1/Q_M$

intercept = $1/K_{ad} Q_M$

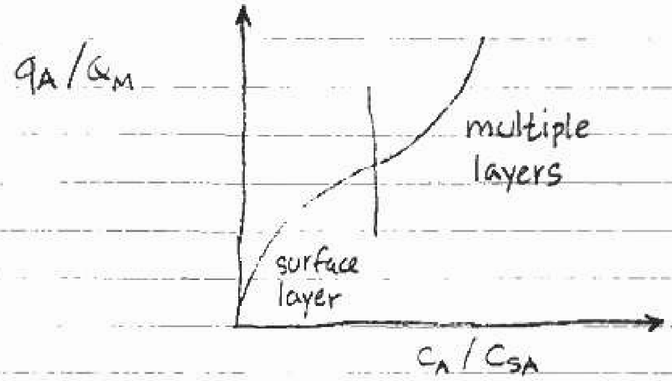


Another alternative isotherm is BET (Brunauer–Emmett–Teller) assumes several molecular layers involved

$$\frac{q_A}{Q_M} = \frac{B_A C_A}{(C_{S,A} - C_A)\left[1 + (B_A - 1)(C_A / C_{SA})\right]}$$

q_A equilibrium adsorbate/adsorbent conc
Q_M max q_A at saturation
C_A equilibrium aqueous conc
C_{SA} saturation C_A

$$B_A = \frac{K_{1,ad}}{K_{i,ad}} = \frac{\text{eq. const for first layer}}{\text{eq. const for subsequent layers}}$$



Alternative most used for activated carbon is Freundlich isotherm (empirically derived)

$$q_A = K_F C_A^{1/n}$$

Can be shown consistent with Langmuir adsorption by heterogeneous sites with distribution of energy of adsorption

Isotherms are found by doing lab tests

Mass of carbon put in bottle (M)

Volume of water $\forall$ w/contaminant conc C_0 added

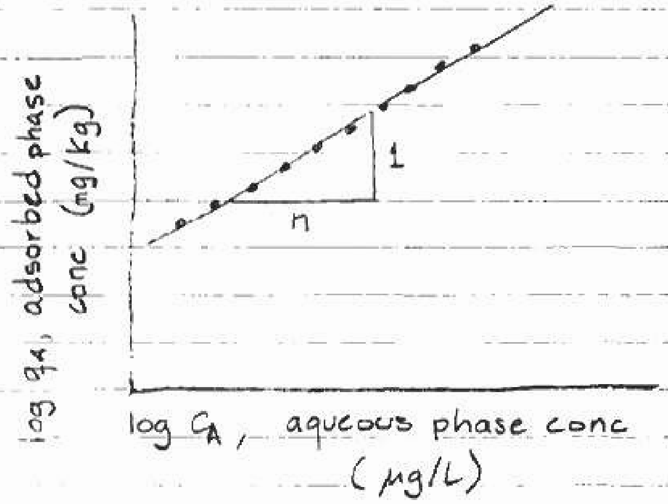
Tumbled gently for 6 days to get equilibrium

Equilibrium C_A in water measured

Compute $q_A = \frac{\forall}{M}(C_0 - C_A)$

Plot q_A vs C_A on log-log graph

If Freundlich isotherm applies plot will be linear:



Get n from slope

Substitute to solve for K_A

Tests are often done by carbon manufacturers (e.g. Calgon carbon)

Freundlich isotherm

$$q_A = K_F \, C_A^{1/n}$$

if q_A is $\frac{mg}{kg}$

C_A is $\frac{\mu g}{L}$

then K_F is in units of $\dfrac{\left(\frac{mg}{kg}\right)}{\left(\frac{\mu g}{L}\right)^{1/n}}$

Example: Assume $1/n = 1/3$

If q_A is $\frac{mg}{kg}$

and C_A is $\frac{mg}{L}$ (1000x larger than above)

what is conversion factor for K_F to units above?

$$\frac{\left(\frac{mg}{kg}\right)}{\left(\frac{mg}{L}\right)^{1/n}} \cdot \frac{\left(\frac{mg}{L}\right)^{1/n}}{\left(\frac{\mu g}{L}\right)^{1/n}} = \frac{\left(\frac{mg}{kg}\right)}{\left(\frac{\mu g}{L}\right)^{1/n}}$$

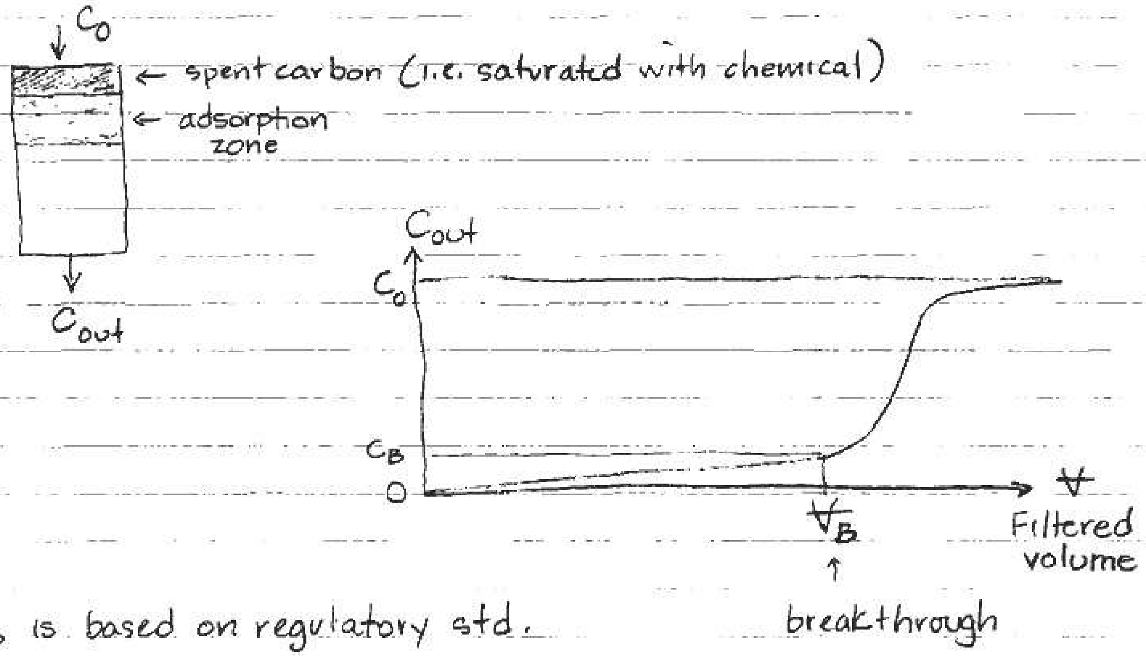
$$= \left(\frac{mg}{\mu g}\right)^{1/n} = 1000^{1/3} = 10$$

Activated carbon comes in two main forms:

PAC - powdered activated carbon (~24 μm) which is suspended in water / wastewater to be treated
PAC is then settled / filtered out and may be recycled

GAC - granular activated carbon (0.6 to 2.4 mm) placed in packed beds usually in pressure tanks

Carbon has fixed adsorption capacity, so chemical eventually "breaks through"



C_B is based on regulatory std.

Contact time is 7 to 20 minutes in typical water treatment plant

GAC Tank

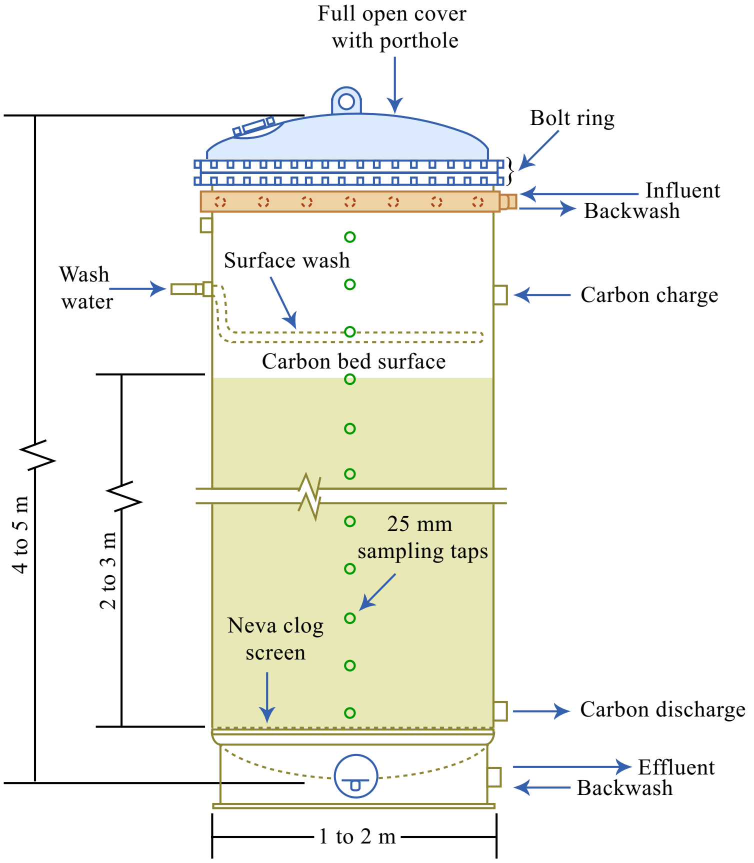


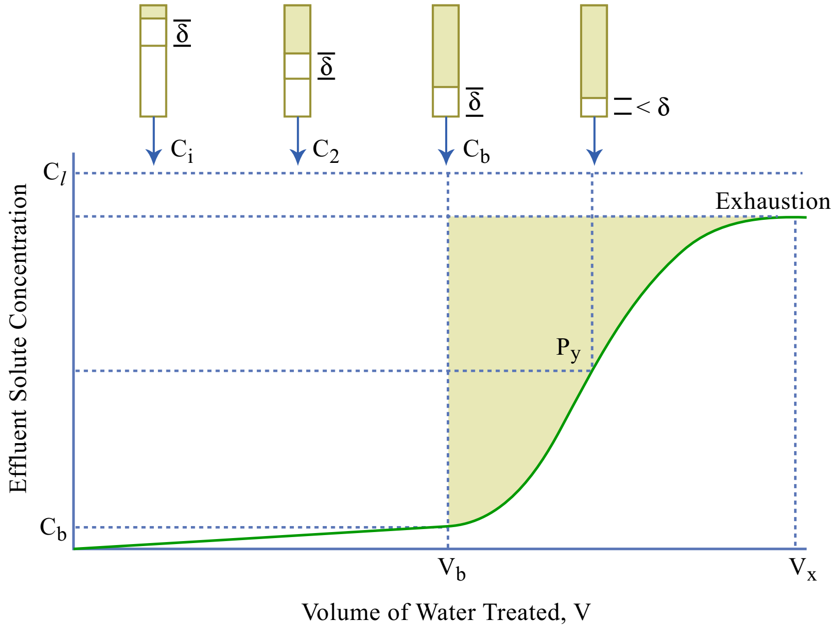
Figure by MIT OCW.

Source: Metcalf & Eddy Inc. *Wastewater Engineering: Treatment, Disposal, Reuse*. McGraw-Hill, New York, 1979.

Granular activated carbon tanks at MMR

Source: MMR, undated. Extraction, treatment and reinjection. Air Force Center for Environmental Excellence, Installation Restoration Program, Massachusetts Military Reservation. http://www.mmr.org/community/guide/extraction.htm. Accessed March 7, 2004.

GAC Breakthrough

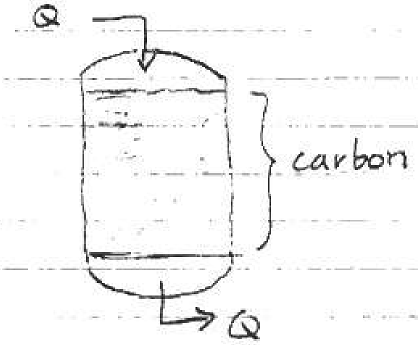


Typical Breakthrough Curve for Activated Carbon

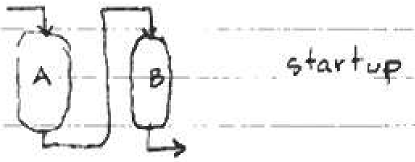
Figure by MIT OCW

Source: Metcalf & Eddy Inc. *Wastewater Engineering: Treatment, Disposal, Reuse*. McGraw-Hill, New York, 1979.

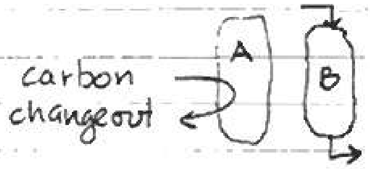
Carbon typically used in pressure vessels much like pressure filtration tanks:



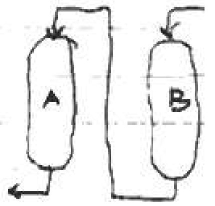
tanks are often placed in series with piping and valving to change order of tanks:



At Tank A breakthrough, Tank A goes off-line for carbon change-out. Fresh Tank B operates alone



Then operation resumes in reverse order, B then A



Continues until breakthrough at B

Second tank is essentially always "fresh", acts as backup to ensure fully treated water even when there is premature breakthrough

Spent carbon can be sent back to manufacturer for regeneration - heat treatment to remove adsorbed organics

Regenerated carbon is not as effective as virgin carbon - some applications specify virgin carbon only

Isotherms are custom developed for type of carbon and water/wastewater and used to predict breakthrough volume $\forall_B$ for certain mass of GAC, M_{GAC}

$$\text{Carbon usage rate} = \frac{M_{GAC}}{\forall_B} = CUR$$

CUR tells how much carbon is needed to treat certain volume of water

In practice, effluent from GAC tank is monitored to also watch for breakthrough

Ion exchange

Similar in many ways to adsorption treatment, but chemical mechanism is ion exchange rather than adsorption

An ion exchanger is a material to which certain ions are sorbed in exchange for ions already bound to exchanger

For example: water can be softened by an ion exchanger that adsorbs Ca and Mg, releasing Na in exchange
Zeolite is a natural mineral that softens water by ion exchange

Reaction looks like:

$$Ca^{+2} + 2Na \cdot Ex \rightleftharpoons Ca \cdot Ex_2 + 2Na^{+}$$

$$Mg^{+2} + 2Na \cdot Ex \rightleftharpoons Mg \cdot Ex_2 + 2Na^{+}$$

↑ exchanger solid

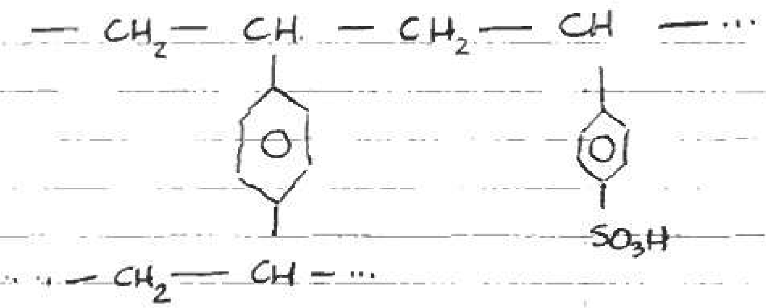
Exchanger is regenerated using strong brine:

$$Ca \cdot Ex_2 + 2Na^{+} \overset{\text{strong brine}}{\rightleftharpoons} 2Na \cdot Ex + Ca^{+2}$$

$$Mg \cdot Ex_2 + 2Na^{+} \overset{\text{strong brine}}{\rightleftharpoons} 2Na \cdot Ex + Mg^{+2}$$

Most exchangers are now synthetic resins

e.g.



sulfonic group $-SO_3H$ is ion exchanger

H^+ swaps with cations

Preference series shows which ions exchange:

$Ba^{+2} > Pb^{2+} > Sr^{2+} > Ca^{2+} > Ni^{2+} > Cd^{2+} >$

$Cu^{2+} > Co^{+2} > Zn^{2+} > Mg^{2+} > Ag^{2+} >$

$Cs^{+} > K^{+} > NH_4^{+} > Na^{+} > H^{+}$

for strong acid resins (e.g. sulfonates
with $-SO_3H$
group)

More preferd ions are swapped for less
preferred – e.g. Ca^{2+} for H^+

For anion exchangers (use carboxylic group $-COOH$)

$SO_4^{-2} > I^- > NO_3^- > CrO_4^{-2} > Br^- >$

$Cl^- > OH^-$

(Preference varies with the resin)

Design

Design procedure and treatment systems are very similar to those for activated carbon

Bench-scale column tests are used to develop curves of breakthrough (C vs. $\forall$)

Resin is generally placed in pressure tanks similar to those for GAC

Overflow rates $\sim$ 6-8 gpm/ft^2

Iron and manganese removal

Iron II (Fe^{2+}) and Manganese II (Mn^{2+}) exist in acidic & reducing environments (e.g. wetlands and in aquifer below wetlands) – see Eh-pH diagrams pp. 14-15

Fe^{2+} and Mn^{2+} are soluble and remain in water following conventional treatment but precipitate at the point of use, causing stains on plumbing fixtures and in laundry
Also support growth of iron bacteria (iron slime) in well screens, distribution systems

Fe and Mn can be addressed in variety of ways:

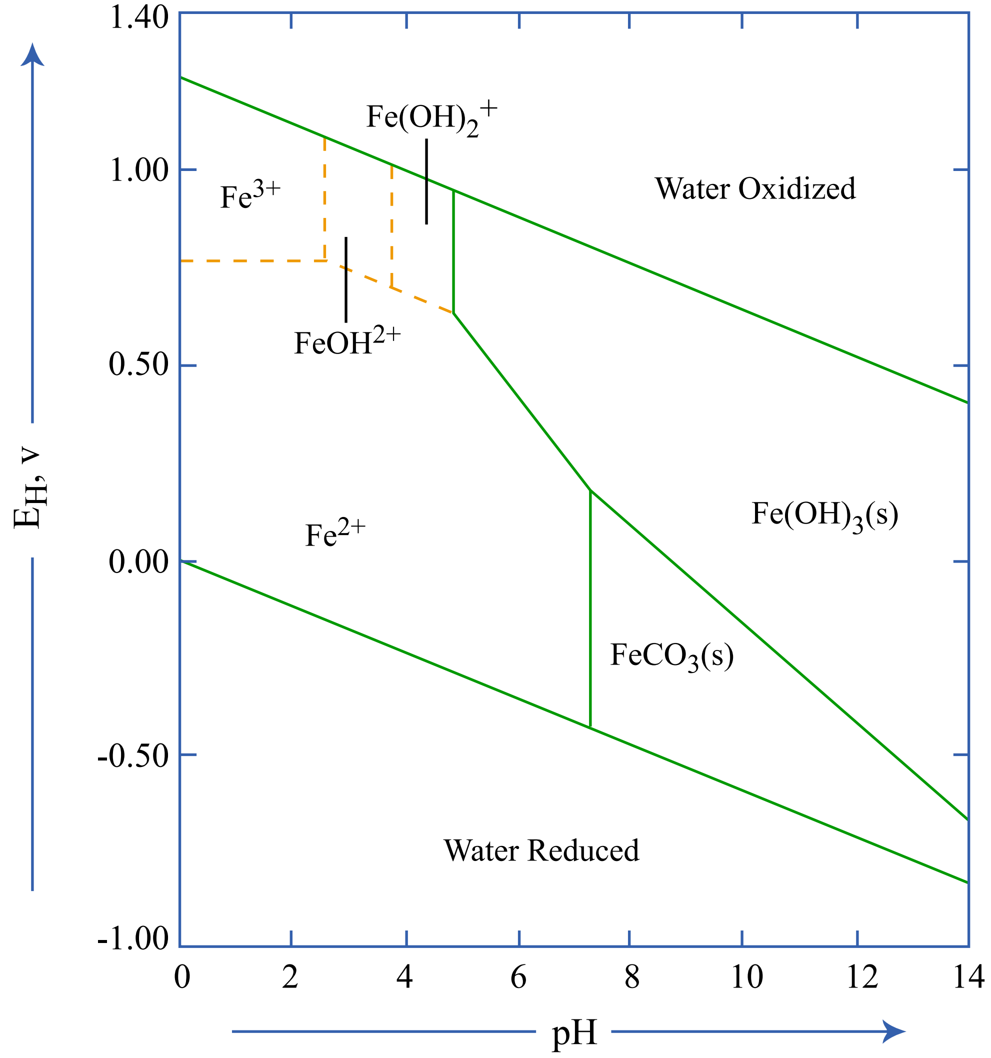
1. In-situ treatment – injection wells around water supply wells to precipitate Fe and Mn in the ground
2. Sequestration – phosphate chemicals added to water to bind with and "sequester" Fe and Mn, preventing later precip
3. Ion exchange treatment with Greensand (glauconite) – natural ion exchanger with Fe and Mn

Removal reaction:

$$\underbrace{Z-MnO_2}_{\text{Mn-coated glauconite}} + \left\{\begin{matrix}Fe^{2+}\\Mn^{2+}\end{matrix}\right\} \rightarrow Z-Mn_2O_3 + \left\{\begin{matrix}Fe^{3+}\\Mn^{3+}\\Mn^{4+}\end{matrix}\right\}\downarrow$$

Regeneration with potassium permanganate

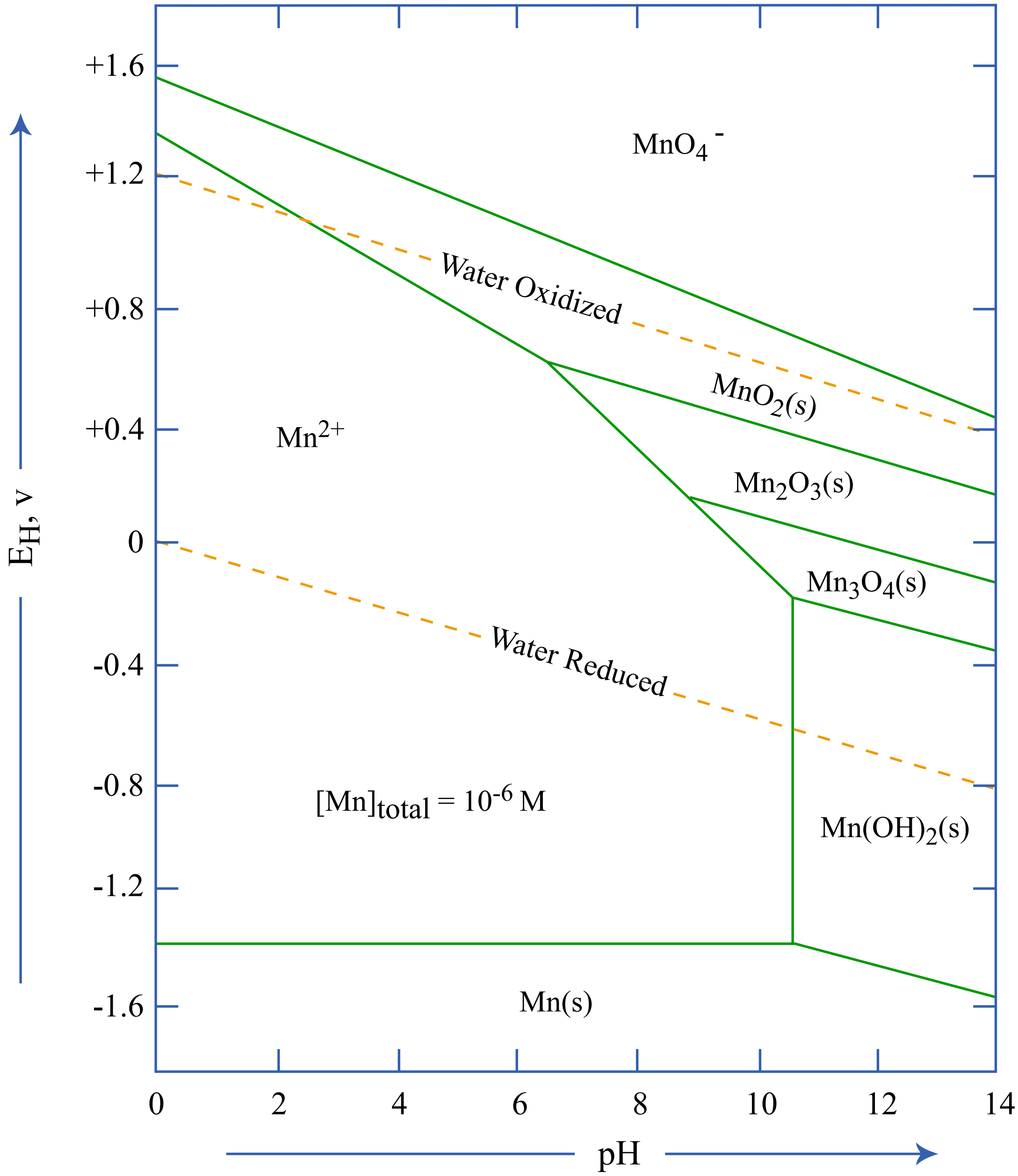
$$Z-Mn_2O_3 + KMnO_4 \rightarrow Z-MnO_2$$



Forms of Iron in Water as Function of Redox Potential Versus pH Constructed with Total Iron Activity 10^{-7}M or 5.6 µg/L, 96 mg/L SO_4^{2-}, CO_2 Species at 1000 mg/L HCO_3^-, Temperature at 25°C, and Pressure of 1 atm.

Figure by MIT OCW.

Adapted from: MWH, J. C. Crittenden, R. R. Trussell, D. W. Hand, K. J. Howe, and G. Tchobanoglous. *Water Treatment: Principles and Design*. 2nd ed. Hoboken, NJ: John Wiley & Sons, 2005, p. 1571.



Forms of Manganese in Water as Function of Redox Potential Versus pH at a Water Temperature of 25°C.

Figure by MIT OCW.

Adapted from: MWH, J. C. Crittenden, R. R. Trussell, D. W. Hand, K. J. Howe, and G. Tchobanoglous. *Water Treatment: Principles and Design*. 2nd ed. Hoboken, NJ: John Wiley & Sons, 2005, p. 1579.

4. Oxidation

Goal is to oxidize Mn^{2+}, Fe^{2+} to get precipitates

Aeration (works for Fe, not for Mn)

$$2Fe^{2+} + 4HCO_3^- + \frac{1}{2}O_2 + H_2O \rightarrow 2Fe(OH)_3\downarrow + 4CO_2$$

Precipitated iron is then settled and filtered
(most removal by filtration)

Chemical oxidation

Addition of strong oxidizer - chlorine
or $KMnO_4$

Precipitated iron settled and filtered

Reaction with permanganate:

$$3Fe^{2+} + 6(HCO_3)^- + KMnO_4 + 2H_2O \rightarrow 3Fe(OH)_3\downarrow + MnO_2 + KHCO_3 + 5CO_2$$

$$3Mn^{2+} + 6(HCO_3)^- + 2KMnO_4 \rightarrow 5MnO_2 + 2KHCO_3 + 2H_2O + 4CO_2$$

5. Lime-soda ash softening

Fe and Mn removed during softening
if pH is raised above 9.8