

**School of Civil and Environmental Engineering
Addis Ababa Institute of Technology**

**WATER AND WASTEWATER
TREATMENT & PROCESS DESIGN**

(CENG 6656)

Course Outline

1. Introduction (1 week)
2. Coagulation and Flocculation (2 weeks)
3. Gravity Separations (2 weeks)
4. Filtration (2 weeks)
5. Disinfection (2 weeks)
6. Miscellaneous treatment processes (2 weeks)
7. Wastewater treatment (2 weeks)
8. Residuals management (1/2 week)
9. Resource oriented wastewater treatment (1/2 week)

1. INTRODUCTION

Water and Health

- 88% of diarrhoeal disease is attributed to unsafe water supply, inadequate sanitation and hygiene.
- 1.8 million people die every year from diarrhoeal diseases (including cholera); 90% are children under 5, mostly in developing countries.
- Every 8 seconds a child dies from drinking contaminated water (that is 10,000 a day).
- 500 million people are at risk from trachoma, 146 million are threatened by blindness and 6 million people are visually impaired from this disease.
- In Bangladesh, between 28 and 35 million people consume drinking water with elevated levels of arsenic.
- Over 26 million people in China suffer from dental fluorosis due to elevated fluoride in their drinking water.

Safe Water and Public health

No other single intervention in the history of medicine has saved as many lives and reduced as much suffering as the provisioning of uncontaminated water.

Ewald, 1994

We shall not finally defeat AIDS, tuberculosis, malaria, or any of the other infectious diseases that plague the developing world until we have also won the battle for safe drinking water, sanitation and basic health care.

Kofi Annan, United Nations Secretary-General

SDG 6 Targets

- **Target 6.1:** By 2030, achieve universal and equitable access to **Safe** and affordable **drinking water** for all
- **Target 6.2:** By 2030, **achieve access to adequate and equitable sanitation hygiene for all and end open defecation**, paying special attention to the needs of women and girls and those in vulnerable situations
- **Target 6.3:** By 2030, **improve water quality by reducing pollution**, eliminating dumping and minimizing release of hazardous chemicals and materials, **halving the proportion of untreated wastewater** and substantially **increasing recycling and safe reuse** globally

Introduction – Water Sector GTP-II Targets of Ethiopia

No.	Goal/Target
1	Increase safe water supply upgrading the service level and improve urban wastewater management system
1.1	Provide safe water supply to 85% of the rural population with 20% having connection to piped schemes
1.2	Provide piped water supply to 75% of the urban population
1.3	Undertake wastewater management studies and design for 36 medium and large towns/cities and build wastewater management infrastructure for 6 large towns/cities
2	Ensure good governance in rural water supply enhancing sustainability, effectiveness, efficiency, and climate change resilience of the service.
2.1-2.5	Improve efficiency and management of rural water supply through community participation and better supply chain, and operation and maintenance services
2.6	Ensure rural water safety through rural water supply water quality monitoring system and water safety planning and implementation
2.7	Establish groundwater monitoring and catchment protection system around water supply sources
3	Ensure good governance in urban water supply enhancing sustainability, effectiveness, efficiency and climate change resilience of the service.
3.1-3.3	Improve the efficiency, reliability and sustainability of urban water supply services
3.4-3.5	Ensure safe and reliable water supply through water safety plan, source and catchment protection, water quality monitoring
4	Build the sub-sectors' overall capacity
4.1-4.6	Enhance the water supply and sanitation service delivery through human resources and institutional capacity building and good governance

Water Sector Short, Mid- and Long-Term Targets- Ethiopia

Short Term (GTP-II 2020)

- Improved Water Supply as per GTP-II standard
 - Rural: 85 %
 - Urban: 75%
 - National: 83%
- Improved latrine and ODF: 82%
- Improved urban wastewater mgt

Mid Term (2025)

- Universal access to as per GTPII standard
- Leaving no one behind

Long Term (SDG 2030)

- Safely managed water supply and sanitation for all as per SDG standard

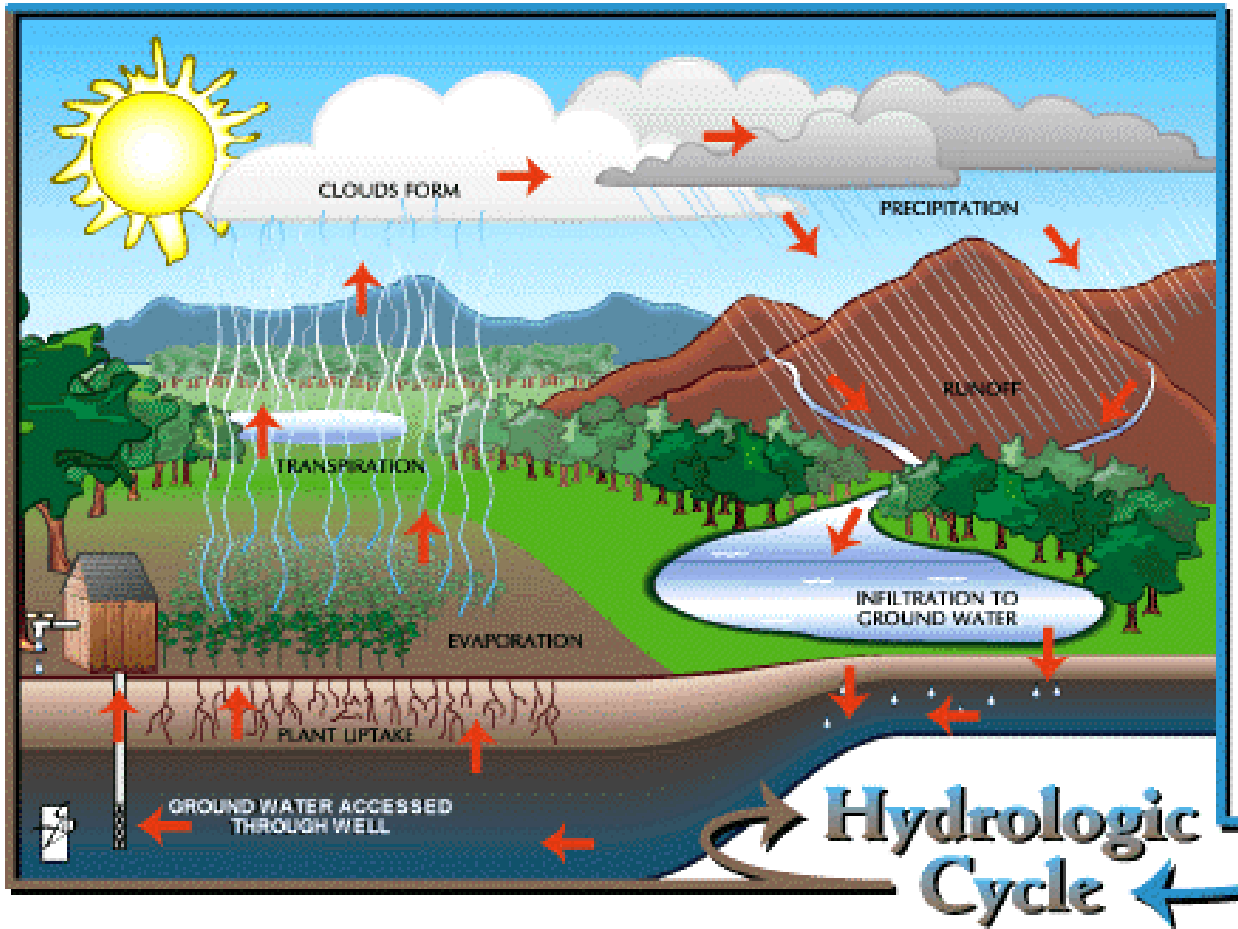
Water Supply System Objectives

- **Safe and wholesome water**
- **Adequate quantity**
- **Readily available to encourage
personal and household hygiene**

Components of water demands

- Water demand is defined as the volume of water required by users to satisfy their needs.
- Demand is the theoretical while consumption is actual
- Design of a water supply scheme requires knowledge of water demand and its timely variations.
- Various components of a water demand are residential, commercial, industrial, institutional, agricultural, public water uses, fire demand and unaccounted for system losses.

Sources of Water

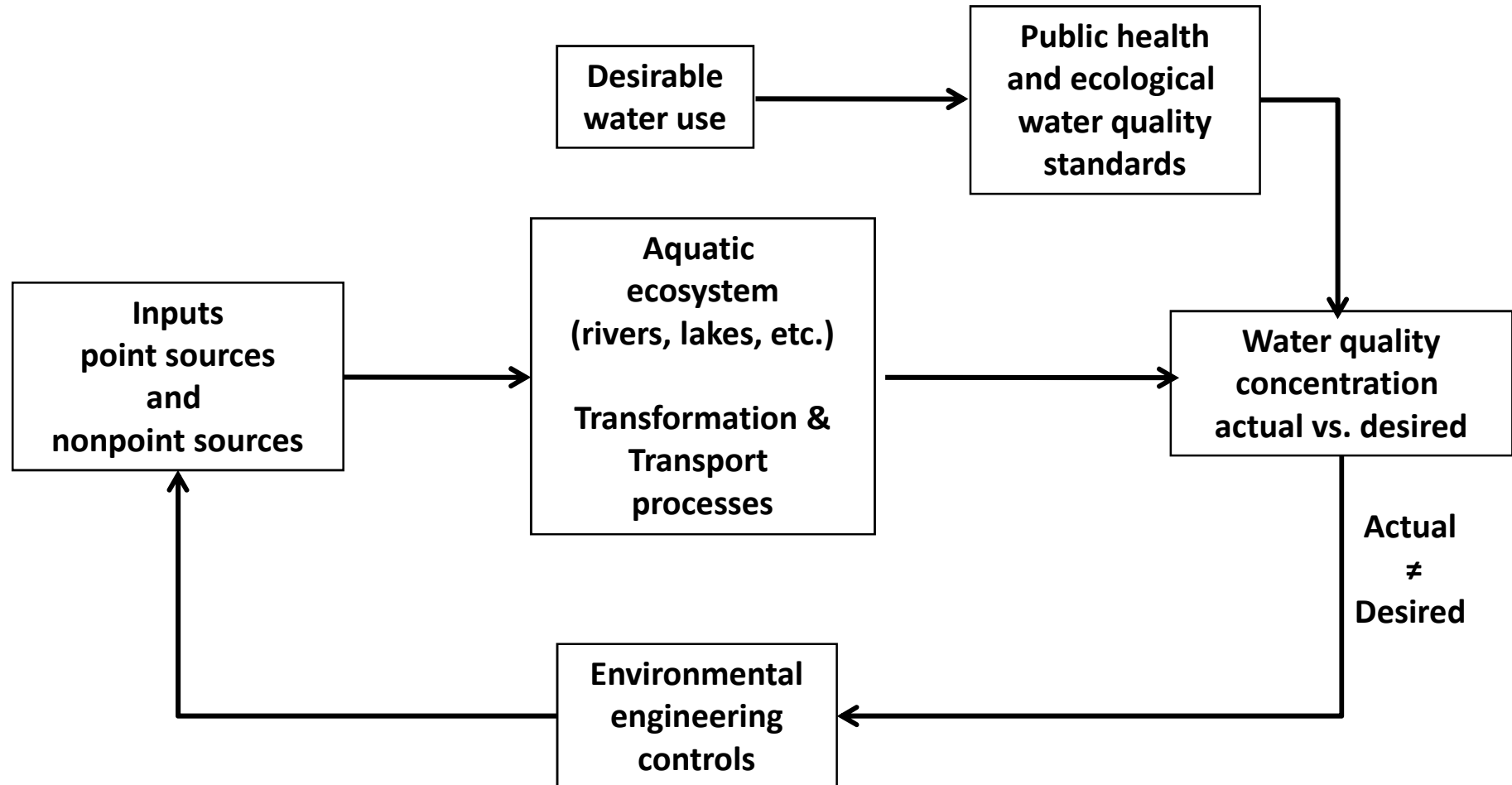


100% Precipitation;
50% Percolation;
30% Evaporation;
40% Evapotranspiration;
20% Surface Runoff ;
10% Groundwater Flow

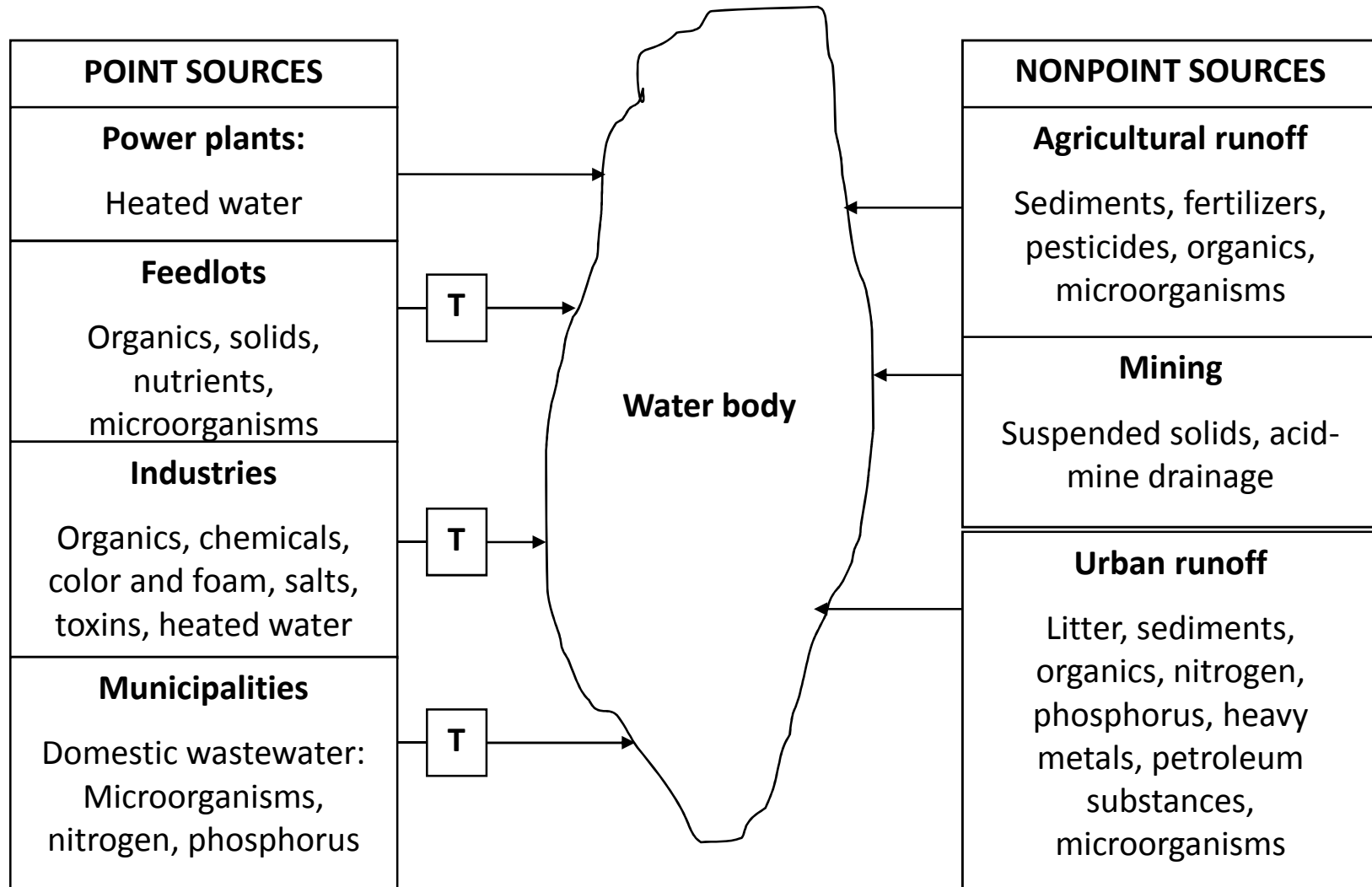
Sources of water:

- Surface sources (upland rivers, lowland rivers, lakes & reservoirs)
- Groundwater aquifers

Water Quality Engineering

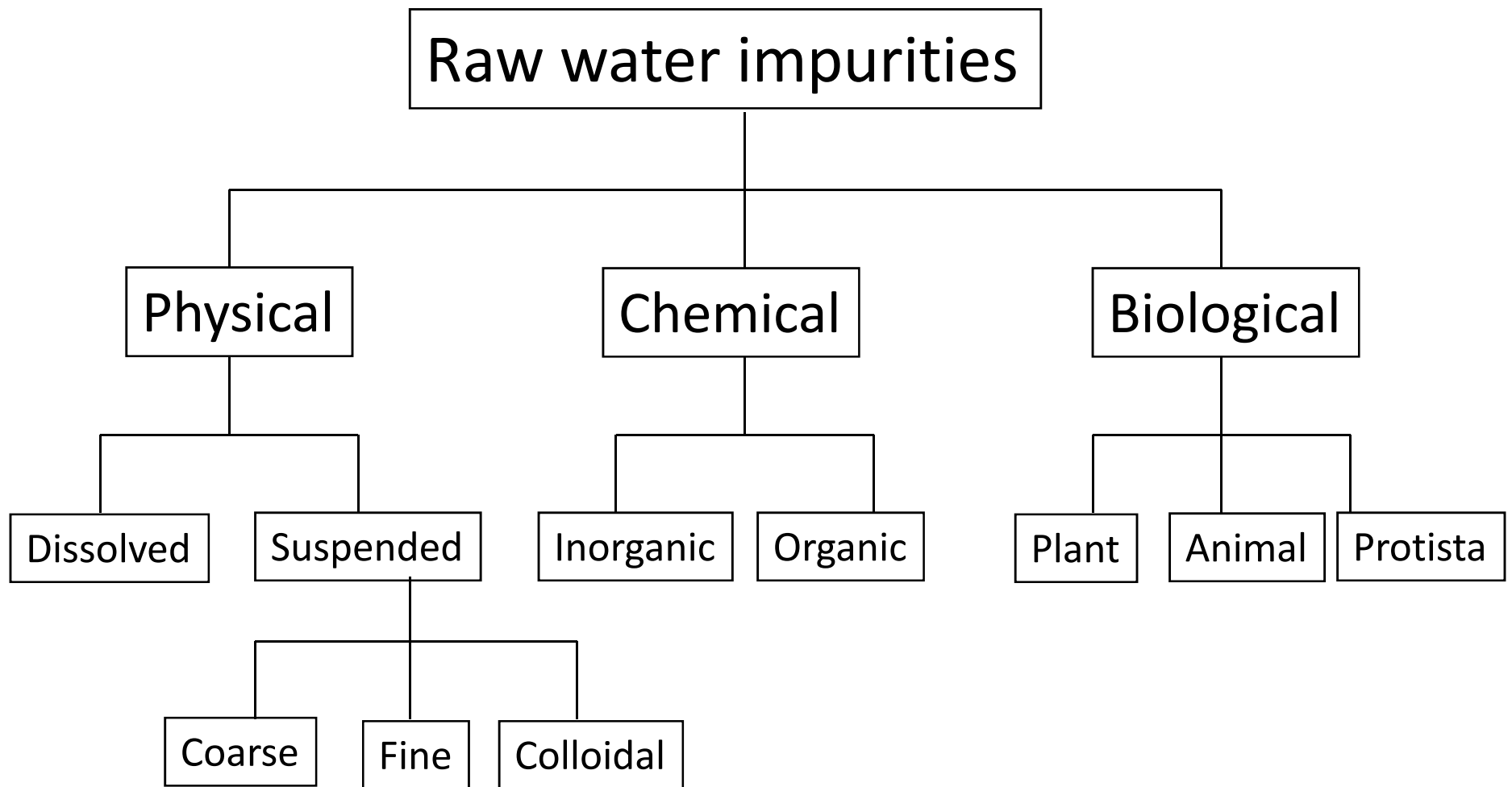


Sources of water pollution



Importance of water quality considerations

- To evaluate and classify raw water quality
- To identify sources of pollution
- To assess the treatment required for beneficial uses



Some raw water quality parameters

Physical

- Suspended solids
- TDS
- Color
- Turbidity
- Conductivity
- Taste
- Odor

Chemical

- Oxygen demanding organics
- Dissolved oxygen
- Principal inorganic ions
- Trace constituents
- Synthetic organic matters
- pH
- Alkalinity
- Hardness

Biological

- Total coliforms
- E.coli
- Plants and animals

Comparison of surface water and groundwater quality

Parameter	Surface water	Groundwater
Temperature	Varies with season	Relatively constant
Turbidity and SS	Varies and is sometimes high	Usually low or nil
Mineral content	Varies with soil, rainfall, effluents, etc	Relatively constant and high
Iron and manganese	Some	Always high
Carbon dioxide	None	Always high
Dissolved oxygen	Often near saturation, if not polluted	Usually low
Ammonia	Only in polluted water	Increasing trend
Hydrogen sulphide	None	Usually some
Nitrate	Generally none	Increasing trend
Living organisms	May be high	Usually none

Typical raw water analysis

Parameter	Deep well water	Moorland water	River water	Arid zone water	Seawater
Color	Clear	Slightly yellow	Turbid	Turbid	
Conductivity,ms/cm	580	150	915	1000-7000	51000
pH	7.3-7.9	6.5-7.2	7-8	7.5-8.5	7.9
TDS, ppm	410	105	640	700-5000	36200
Cations, mg/L as CaCO ₃					
Ca ²⁺	250	30	200	250-1500	350
Mg ²⁺	75	15	75	150-2000	1330
Na ⁺	25	35	200		10300
K ⁺					350
Anions, mg/L as CaCO ₃					
Cl ⁻	40	30	125	<2000	20500
SO ₄ ²⁻	500	15	175	<1500	2850
F ⁻					
Si ⁻		6	10	10-20	20
HCO ₃ ⁻	250	30	125		170
NO ₃ ⁻	10	5	50		

Microorganisms

- Bacteria: may cause disease, taste and odors, pipe corrosion, pipe blockage
- Viruses: are more resistant to disinfection than bacteria
- Fungi and molds: frequently form dense, slimy mats that clog filters and other units; they can produce musty taste and odors as well as color and turbidity
- Algae:
 - may cause turbidity and color
 - interferes with coagulation and sedimentation causing filter clogging
 - Chemicals produced by algae are precursors of and are associated with different taste and odor problems

Microorganisms

- Protozoa: most of them cause disease
- Helminths (parasitic worms): cause many diseases
- Nuisance-causing organisms: many organisms, snails, crustaceans, and slime growth cause serious problems in raw water conveyance

WATER-RELATED INFECTIONS

PRIMARY PUBLIC HEALTH CONCERN

Waterborne diseases

- Cholera
- Poliomyelitis
- Diarrhoeal diseases
- Roundworm
- Enteric fevers: typhoid
- Whipworm
- Hepatitis A
- Cryptosporidium
- Giardia

Water-washed diseases

- Scabies
- Typhus
- Trachoma
- Louse infestation

Water-based diseases

- Schistosomiasis
- Dracunculiasis (guinea-worm)

Diseases transmitted by water-related insect vectors

- Malaria
- Onchocerciasis
- Yellow fever
- Dengue
- Filariasis
- African trypanosomiasis
- Leishmaniasis

INORGANIC CHEMICAL: ARSENIC (As)

- ❖ From natural erosion, pesticide run-off, coal burning, smelting, glass and electronic production waste
- ❖ Skin lesions and cancer, vascular and neurological disease, increased risk of cancer
- ❖ WHO guideline: 0.01 mg/L (ppm)

NGO: EARTH IDENTITY PROJECT



PHOTO: EIP
Chandra Banu (Hands)
AN ARSENICOSIS PATIENT (Before)

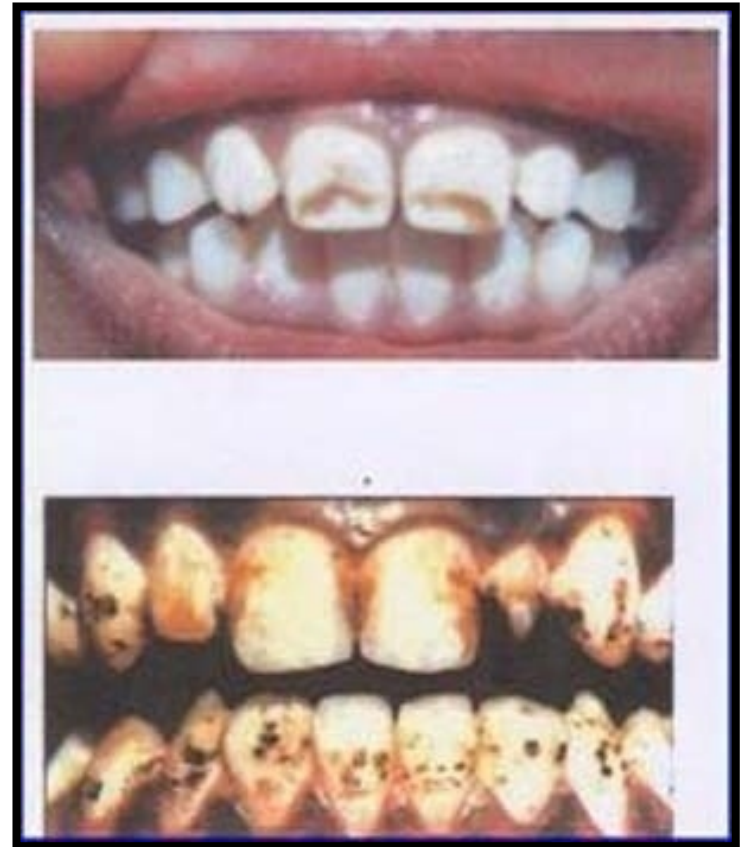
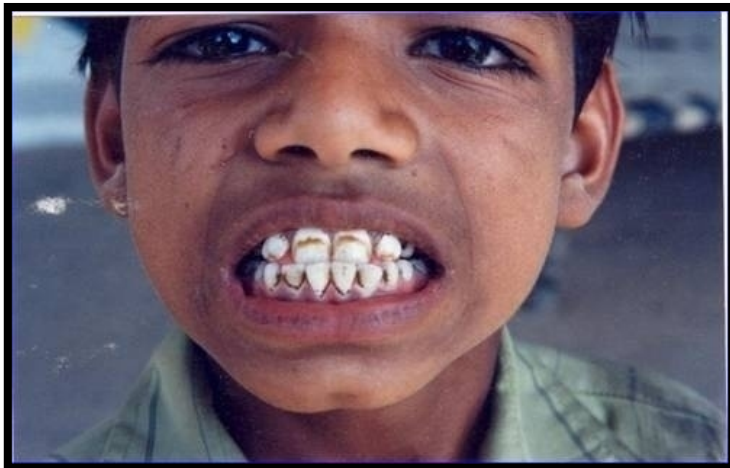


PHOTO: EIP
Chandra Banu (Feet)
AN ARSENICOSIS PATIENT (Before)

N. Karim, NGO: Earth Identity Project, Bangladesh

INORGANIC CHEMICAL: FLUORIDE (FI)

- ❖ From natural erosion, discharge from fertilizer and aluminum factories, or added to drinking water
- ❖ Bone disease, mottled teeth
- ❖ WHO guideline: 1.5 mg/L (ppm)



A. K. Susheela. Fluorosis Research & Rural Development Foundation of India

INORGANIC CHEMICAL: LEAD

- ❖ From plumbing, solder, lead-glazed ceramics, old paint, deposits from leaded petrol
- ❖ Impaired growth and development, behavioural problems, kidney damage
- ❖ WHO guideline: 0.01 mg/L (ppm)

INORGANIC CHEMICALS: NITRATES/NITRITES

- ❖ Introduced by run-off from fertilized land, septic tanks, sewers, erosion from natural deposits
- ❖ Methaemoglobinaemia in young infants
- ❖ WHO guidelines:
 - nitrate 50 mg/L (ppm)
 - nitrite 3 mg/L (ppm)

ORGANIC CHEMICALS: METHYLMERCURY (MeHg)

- ❖ Hg is emitted into the atmosphere and circulated globally
- ❖ Ultimately enters water bodies
- ❖ Deposition is:
 - Close to source
 - Far from source
- ❖ Biotransformed by bacteria
- ❖ Bioconcentrated in fish

ORGANIC CHEMICALS – PESTICIDES

- ❖ Extensive and intensive use, a worldwide concern
- ❖ Pesticides used in agriculture may contaminate water sources
- ❖ Chronic, low-level exposure may lead to subtle effects
- ❖ Persistent organic pollutants (POPs) are the subject of an international Convention (*Stockholm Convention*)

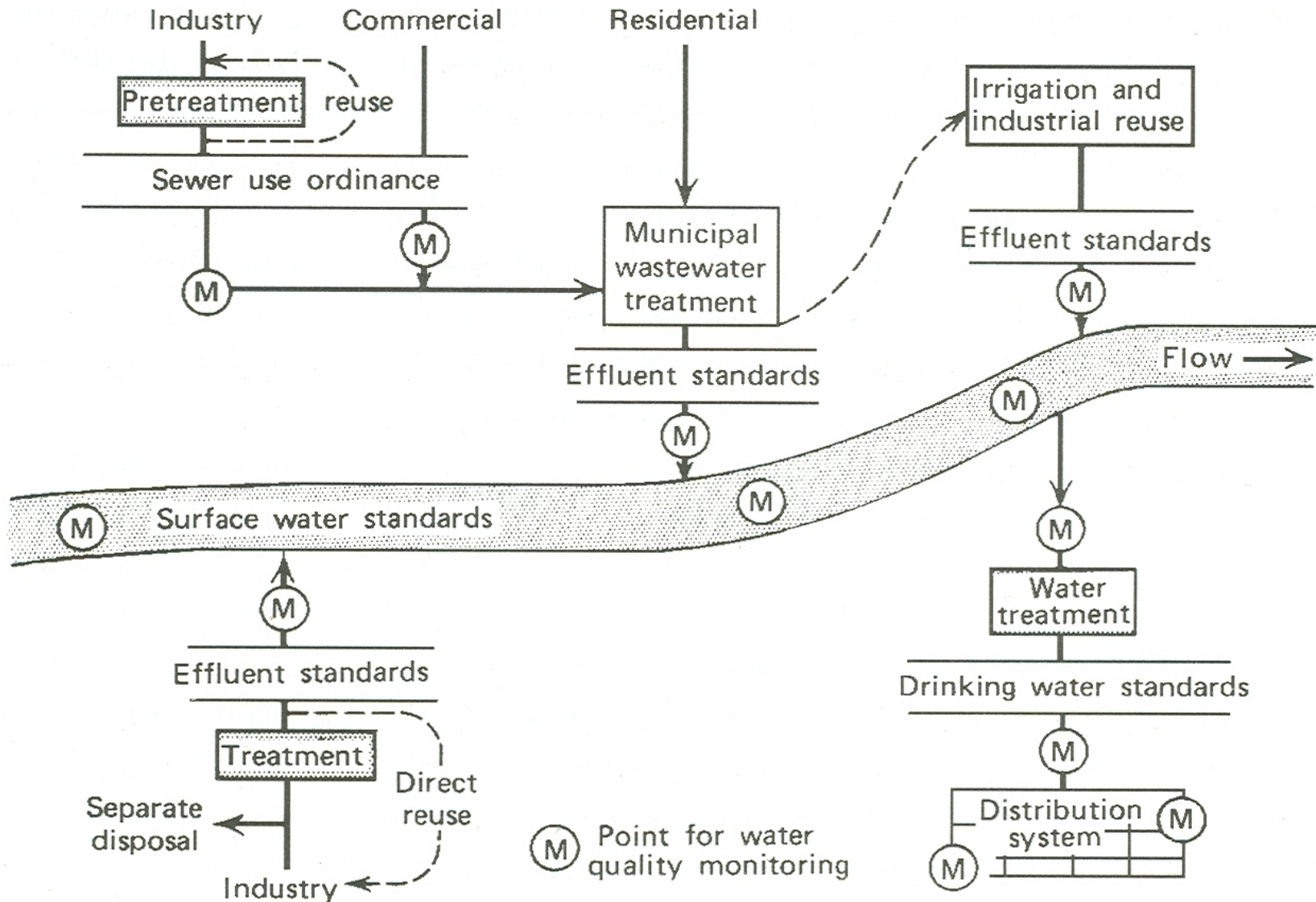
ORGANIC CHEMICALS – DISINFECTION BYPRODUCTS

Disinfection by-products	<i>Potential</i> health effects
Bromate	Increased risk of cancer
Chlorite	Anaemia; infants and young children: nervous system effects
Haloacetic acids (HAA5)	Increased risk of cancer
Total trihalo-methanes (TTHMs)	Liver, kidney or central nervous system problems; increased risk of cancer

Basic quality requirements of drinking water

- Free from diseases causing organisms
- Contain no compounds that have adverse effect on human health
- Fairly clear (i.e., of low turbidity, little color)
- Contain no compounds that cause offensive taste and odor
- Free of substances and organisms that cause corrosion or encrustation of the water supply system, staining of clothes washed in it or food items cooked with it.

Water quality standards



Drinking water quality standards

- *Primary*
 - protect public health—maximum permissible levels
 - enforced
- *Secondary*
 - aesthetic qualities—color, odor, taste
 - recommended but not enforced

Primary MCLs

- *Maximum contaminant levels* (MCLs)
 - Safe levels —no health effect
- *treatment technique requirements*
 - for contaminants difficult or costly to measure
- *maximum contaminant level goals* (MCLGs)
 - Level at which there are no known or anticipated adverse health effects.
 - it is a goal, not an enforceable
 - set at zero for known carcinogens—safe level not known

Primary MCLs: inorganic chemicals

<i>Contaminants</i>	<i>Health effects</i>	<i>MCL (mg/L)</i>
Arsenic	Nervous system effects	0.05
Asbestos	Possible cancer	7 MFL
Cyanide	Nervous system effects	0.2
Lead	Nervous system and kidney effects, toxic to infants	TT
Mercury	Kidney, nervous system effects	0.002

Primary MCLs: organic chemicals

<i>Contaminants</i>	<i>Health effects</i>	<i>MCL (mg/L)</i>
Benzene	Possible cancer	0.005
Total trihalomethanes (TTHM)	Possible cancer risk	0.1
Carbon tetrachloride	Possible cancer	0.005
Endrin	Nervous system, liver, kidney effects	0.002
Vinyl chloride	Possible cancer	0.002

Secondary MCLs

<i>Contaminant</i>	<i>MCL (mg/L)</i>	<i>Health effects</i>
Aluminum	0.05-0.2	Discoloration of water
Fluoride	2.0	Dental fluorosis (a brownish discoloration of the teeth)
Iron	0.3	Bitter metallic taste; staining of laundry, rusty color, sediment
Manganese	0.05	Taste; staining of laundry, black to brown color, black staining
Odor	3 TO	Rotten egg, musty, or chemical smell
Color	15 CU	Visible tint

WHO Drinking Water Quality Guideline

Parameter	WHO guideline
pH	6.5-8.5
Total solids, mg/L	1000
Total hardness, mg/L	500
Chloride, mg/L	250
Sulphate, mg/L	400
Fluoride, mg/L	1.5
Iron, mg/L	0.3
E. Coli, MPN/100 ml	10
Nitrate , mg/L	10

Ethiopian DWQ Standard

Biological

Organism	Maximum permissible level	Test method
Total viable organisms, colonies per ml	must not be detectable	ES ISO 4833
Faecal streptococci per 100ml	must not be detectable	ES ISO 7899-1 ES ISO 7899-2
Coliform organisms, number per 100 ml	must not be detectable	ES ISO 9308-1
E. Coli, number per 100 ml	must not be detectable	ES ISO 9308-1 ES ISO 9308-2

Chemical

Substance or characteristic	Maximum permissible level	Test method
Total hardness (as CaCO ₃)	300	ES 607
Total dissolved solids mg/l, Max	1000	ES 609
Total Iron (as Fe) mg/l, Max	0.3	ES ISO 6332
Manganese (as Mn) mg/l, Max	0.5	ES ISO 6333
Ammonia (NH ₃ +NH ₄ ⁺)* mg/l, Max	1.5	ES ISO 7150-2
Residual, free chlorine mg/l, max	0.5	ES ISO 7393
Anionic surfactants, as mass concentration of MBAS mg/l, Max	1.0	ES ISO 7875-1
Magnesium (as Mg) mg/l, Max	50	ES ISO 7980
Calcium (as Ca), mg/l, Max	75	ES ISO 7980
Copper (as Cu) mg/l, Max	2	ES ISO 8288
Zinc (as Zn) mg/l, Max	5	ES ISO 8288
Sulfate (as SO ₄) mg/l, max.	250	ES ISO 9280
Chloride (as Cl), mg/l, Max	250	ES ISO 9297
Total alkalinity (as CaCO ₃) mg/l, Max	200	ES ISO 9963-1
Sodium (as Na), mg/l, Max	200	ES ISO 9964-1
Potassium (as K), mg/l, max	1.5	ES ISO 9964-2
pH value, units	6.5 to 8.5	ES ISO 10523
Aluminium (as Al) mg/l, Max	0.2	ES ISO 12020

Ethiopian DWQ Standard

Substance or characteristic	Maximum permissible level	Test method
Barium (as Ba) mg/l, Max	0.7	ES 606
Total mercury (as Hg) mg/l, Max	0.001	ES ISO 5666-3
Cadmium (as Cd) mg/l, Max	0.003	ES ISO 5961
Arsenic (as As) mg/l, Max	0.01	ES ISO 6595
Cyanide(as CN) mg/l, Max	0.07	ES ISO 6703-1
Nitrite (as NO ₂), Mg/l, Max	3	ES ISO 6777
Nitrate as NO ₃ Mg/l, Max	50	ES ISO 7890-3
Phenolic compound as phenols , mg/l, Max.	0.002	ES ISO 8165-1
Lead (as Pb) mg/l, Max	0.01	ES ISO 8288
Boron (as B) mg/l, Max	0.3	ES ISO 9390
Selenium (as Se) mg/l, Max	0.01	ES ISO 9965
Fluoride (as F) Max	1.5	ES ISO 10359-1
Chromium (as Cr) mg/l, Max	0.05	ES ISO 11083
Pesticides and Organic constituents, Mg/l, Max		
a) DDT	2	ES ISO 6468
b) Heptachlor and heptachlor epoxide	0.03	
c) Hexachlorobenzene	1	
d) Lindane (Gamma – BHC)	2	
e) Methoxychlor	20	
f) Aldrin/Dieldrine	0.03	
g) 1,2 Dichloro ethane	30	ES ISO 10301
h) 1,1,1- Trichloro ethane	2001	
i) – Trichloro ethene	70	
j) Trichlorobenzenes (total)	20	
k) Hexachlorobutadiene	0.6	

Physical

Characteristic	Maximum permissible level	Test method
Odour*	Unobjectionable	ES 605
Taste	Unobjectionable	
Turbidity, NTU	5	ES ISO 7027
Colour, TCU	15	ES ISO 7887

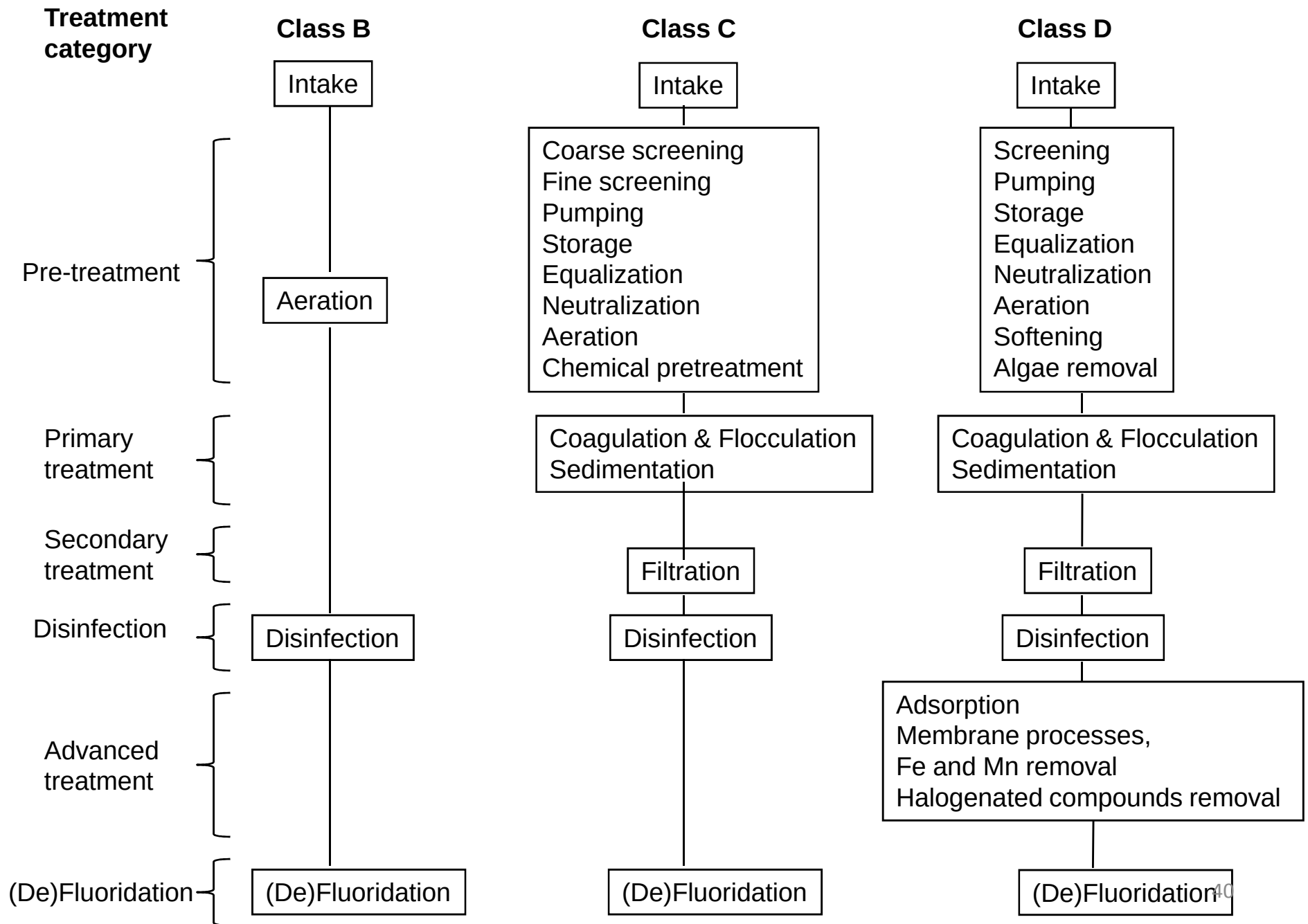
Typical characteristics of domestic wastewater

Constituent	Weak (mg/L)	Medium (mg/L)	Strong (mg/L)
Alkalinity (as CaCO_3)	50	100	200
BOD_5	100	200	300
Suspended solids	100	200	350
TDS	200	500	1000
TKN (as N)	20	40	80
Total phosphorus (as P)	5	10	20

Classes of water treatment

Class	Description	Source/requirement
A	No treatment	Some borehole water Occasional upland water
B	Disinfection only	Some borehole water Occasional upland water
C	Conventional treatment	Lowland rivers and reservoirs
D	Special treatment	Some rural supplies (Fe and Mn) Color removal Trace element removal Algae removal Organics removal Industrial water

Treatment processes for different raw water classes



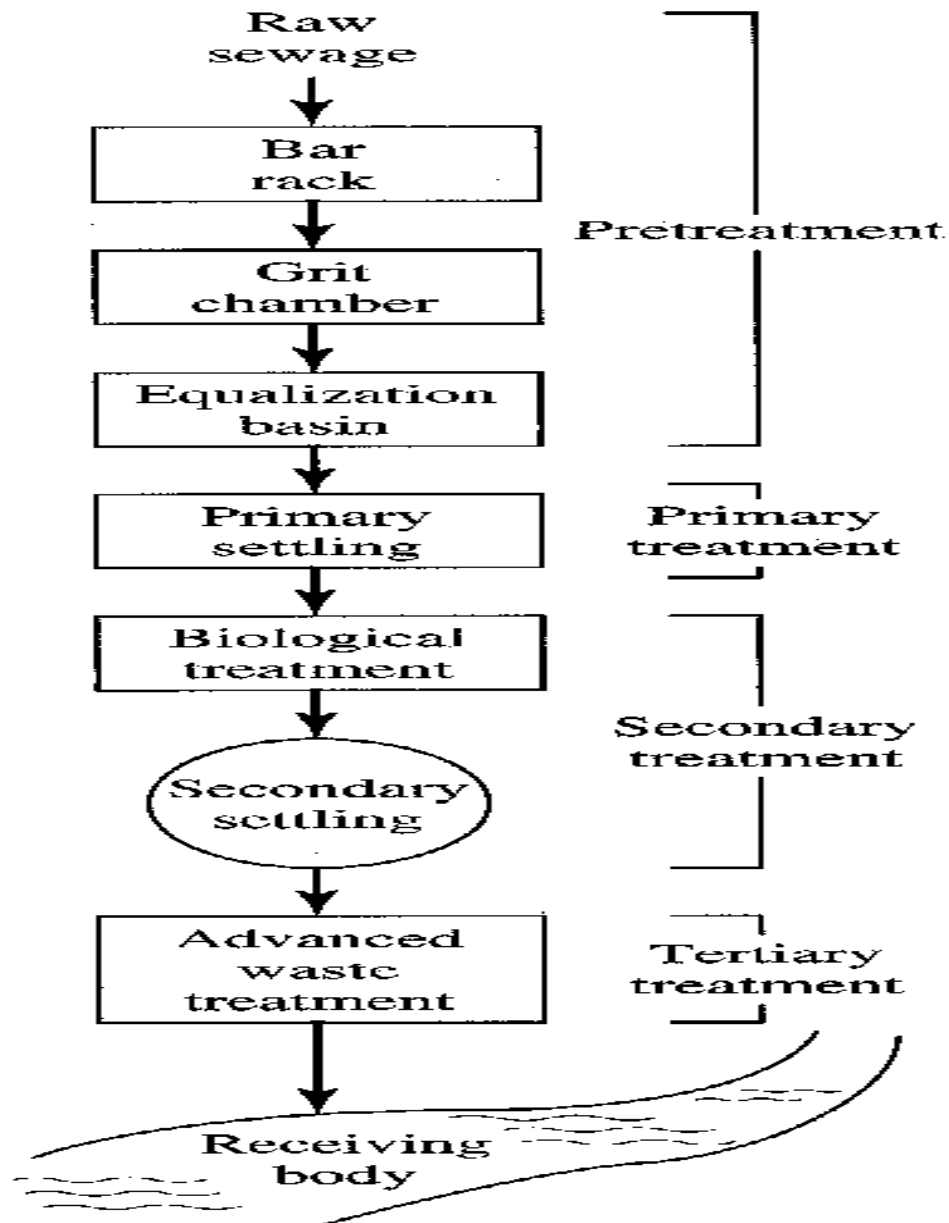
Factors for selection of treatment processes

- Raw water quality
- Availability and reliability of water supply
- Site suitability
- Land availability
- Proximity to consumers
- Economics
- Environmental impact assessment

Recommended water treatment for specific impurities

Parameter	Treatment process
Floating matter	Coarse screens, fine screens
Suspended matter	Microscreens
Algae	Microscreens, pre-chlorination, carbon adsorption, rapid filtration
Turbidity	Coagulation, sedimentation, post-chlorination
Color	Flocculation, coagulation, filtration
Taste and odor	Activated carbon
Hardness	Coagulation, filtration, lime softening
Iron and manganese	
>1 mg/L	Pre-chlorination
<1 mg/L	Aeration, coagulation, filtration
Pathogens, MPN/100 ml	
< 20	Post-chlorination
20-100	Coagulation/filtration/post-chlorination
>100	Pre-chlorination
Free ammonia	Post-chlorination, adsorption

Conventional wastewater treatment



Pretreatment – removes materials that can cause operational problems, equalization optional

Primary treatment – remove ~60% of solids and ~35% of BOD

Secondary treatment – remove ~85% of BOD and solids

Advanced treatment – varies: 95+ % of BOD and solids, N, P

Laboratory methods for WQ analysis

- Gravimetric methods
 - Weighting solids obtained by evaporation, filtration or precipitation (e.g. Total solids, Total suspended solids, etc.)
- Volumetric (titrimetric) methods
 - Requires a liquid reagent of known strength and indicators (e.g. alkalinity, acidity, DO, chloride, etc.)
- Spectrometric (colorimetric) methods
 - Relation between electromagnetic waves and chemicals concentration (e.g. Nitrate, nitrite)
- Chromatographic methods
 - Can separate individual components from a complex (e.g. volatile organic compounds, disinfection by-products, etc.)
- Electrochemical methods
 - Relates electric potential, current or resistance to concentration of ions (e.g. pH, anions and cations, metals, conductivity etc.).
 - They are good for in-situ measurements

Assignment-1

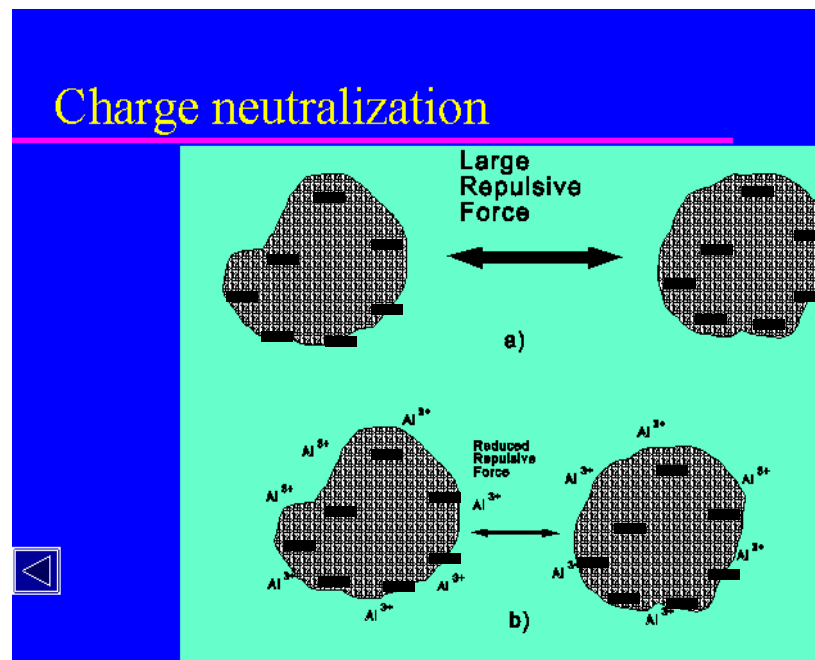
- Reading assignments on
 - Water quality and public health
 - Fundamental principles of environmental engineering that have relevance for water treatment

(Refer File: [Principles of Water Treatment, Chapters #2 & 4.pdf](#))

2. Coagulation & Flocculation

What is Coagulation?

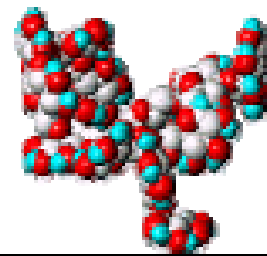
- ❑ **Coagulation** is the destabilization of colloids by addition of chemicals that neutralize the negative charges
- ❑ The chemicals are known as coagulants, usually higher valence cationic salts (Al^{3+} , Fe^{3+} etc.)
- ❑ Coagulation is essentially a chemical process



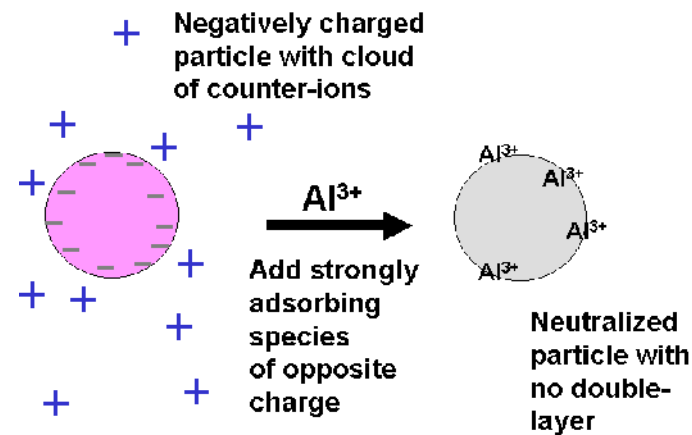
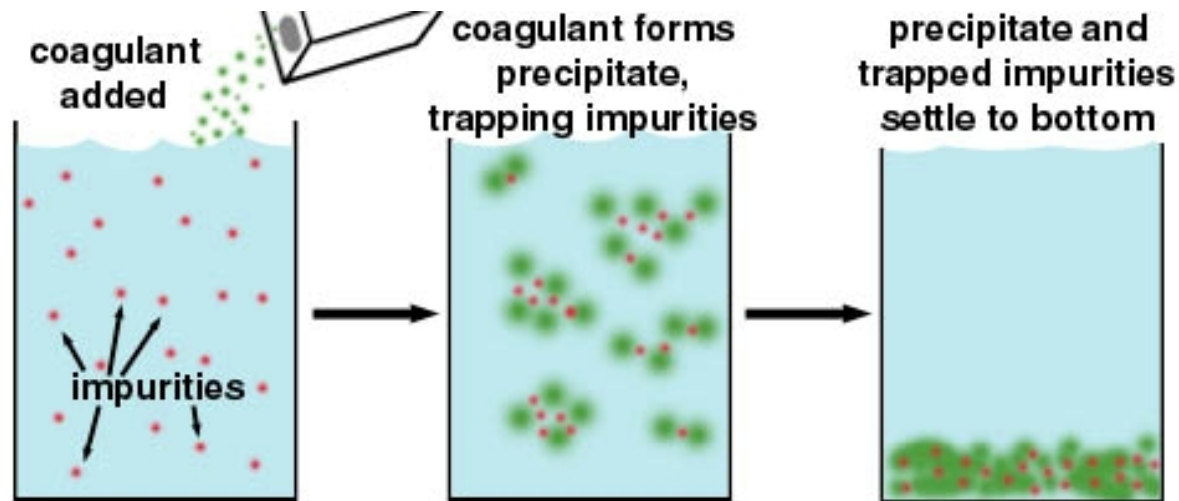
What is Flocculation?

Flocculation is the agglomeration of destabilized particles into a large size particles known as flocs which can be effectively removed by sedimentation or flotation.

- Gentle mixing or *flocculation*, then causes the destabilized (reduced charge) colloids to cluster.
- Another method of enhancing agglomeration is to add organic polymers.
- These compounds consist of a long carbon chain with active groups such as amine, nitrogen, or sulfate groups along the chain.



Coagulation aim



M. Hubbe

Why coagulation and flocculation?

Various sizes of particles in raw water

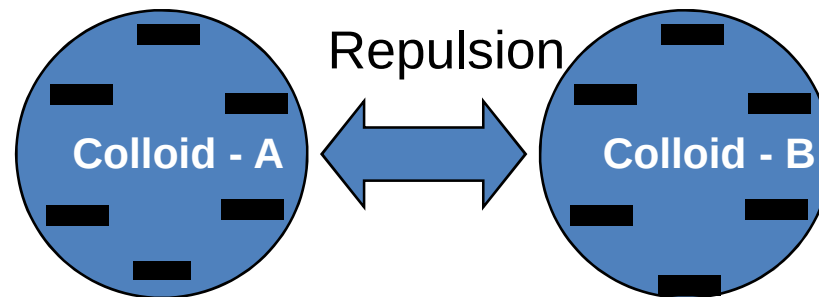
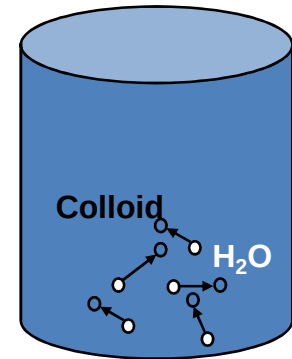
Particle diameter (mm)	Type	Settling velocity
10	Pebble	0.73 m/s
1	Coarse sand	0.23 m/s
0.1	Fine sand	0.6 m/min
0.01	Silt	8.6 m/day
0.0001 (10 micron)	Large colloids	0.3 m/year
0.000001 (1 nano)	Small colloids	3 m/million year

Gravity settling

Colloids – so small: gravity settling not possible

Colloid Stability

- ✓ Colloids have a net negative surface charge
- ✓ Electrostatic force prevents them from agglomeration



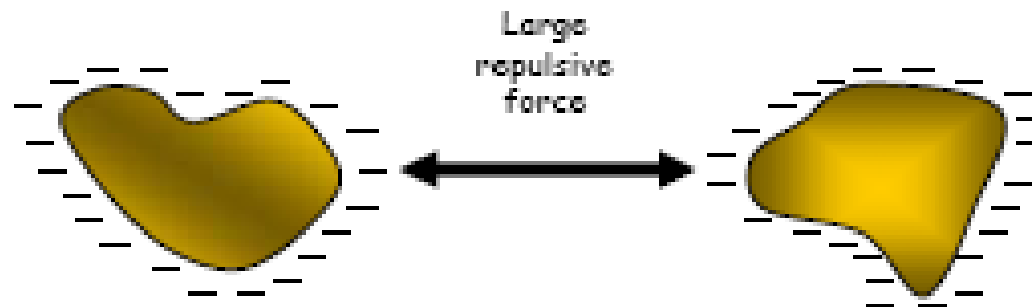
- ✓ Brownian motion keeps the colloids in suspension
- ✓ Impossible to remove colloids by gravity settling

Colloidal interaction

- There are two major forces acting on colloids:

1) electrostatic repulsion

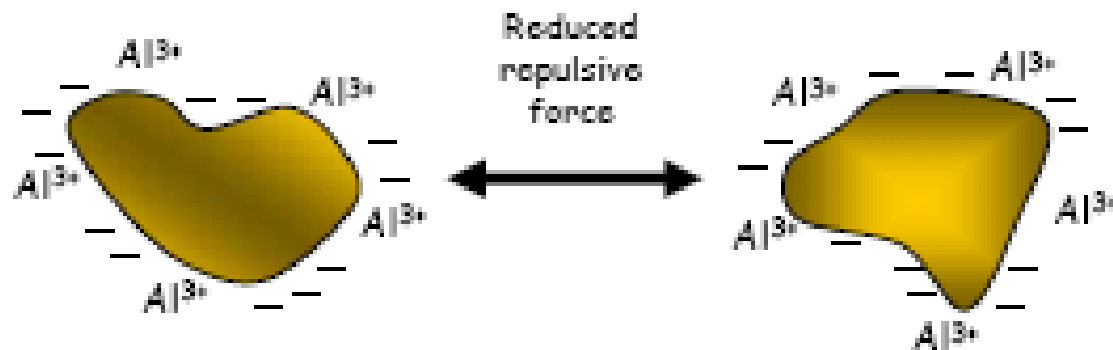
(simply, negative colloids repel other negatively charged colloids)



2) intermolecular, or van der Waals, attraction.

Charge reduction

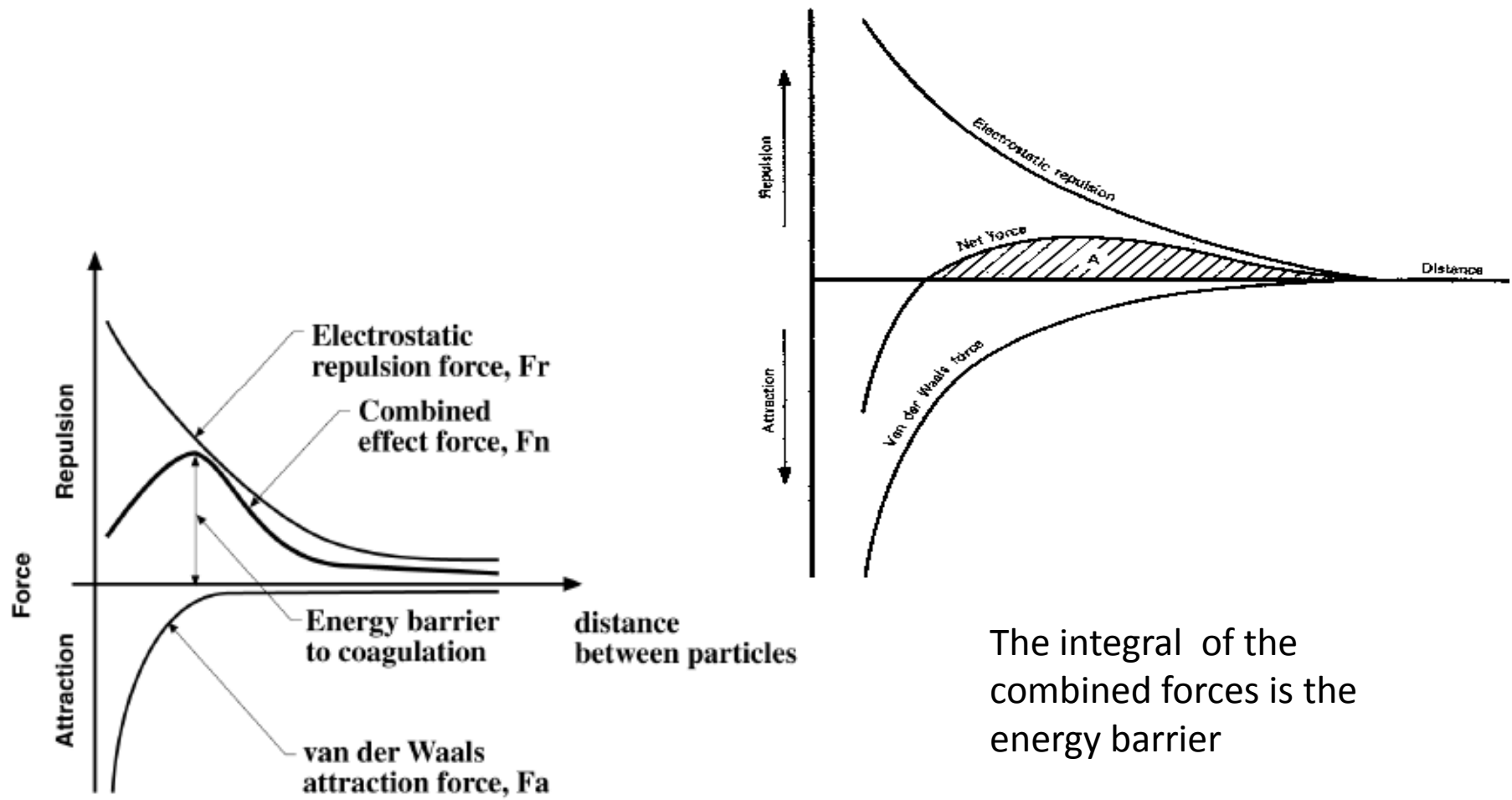
- Coagulants can be used to reduce the electrostatic repulsive forces
- The electrostatic repulsion reduced by the addition of countercharged ions [A^{3+}]



Colloid Destabilization

- Colloids can be destabilized by charge neutralization
- Positively charged ions (Na^+ , Mg^{2+} , Al^{3+} , Fe^{3+} etc.) neutralize the colloidal negative charges and thus destabilize them.
- With destabilization, colloids aggregate in size and start to settle

Force analysis on colloids

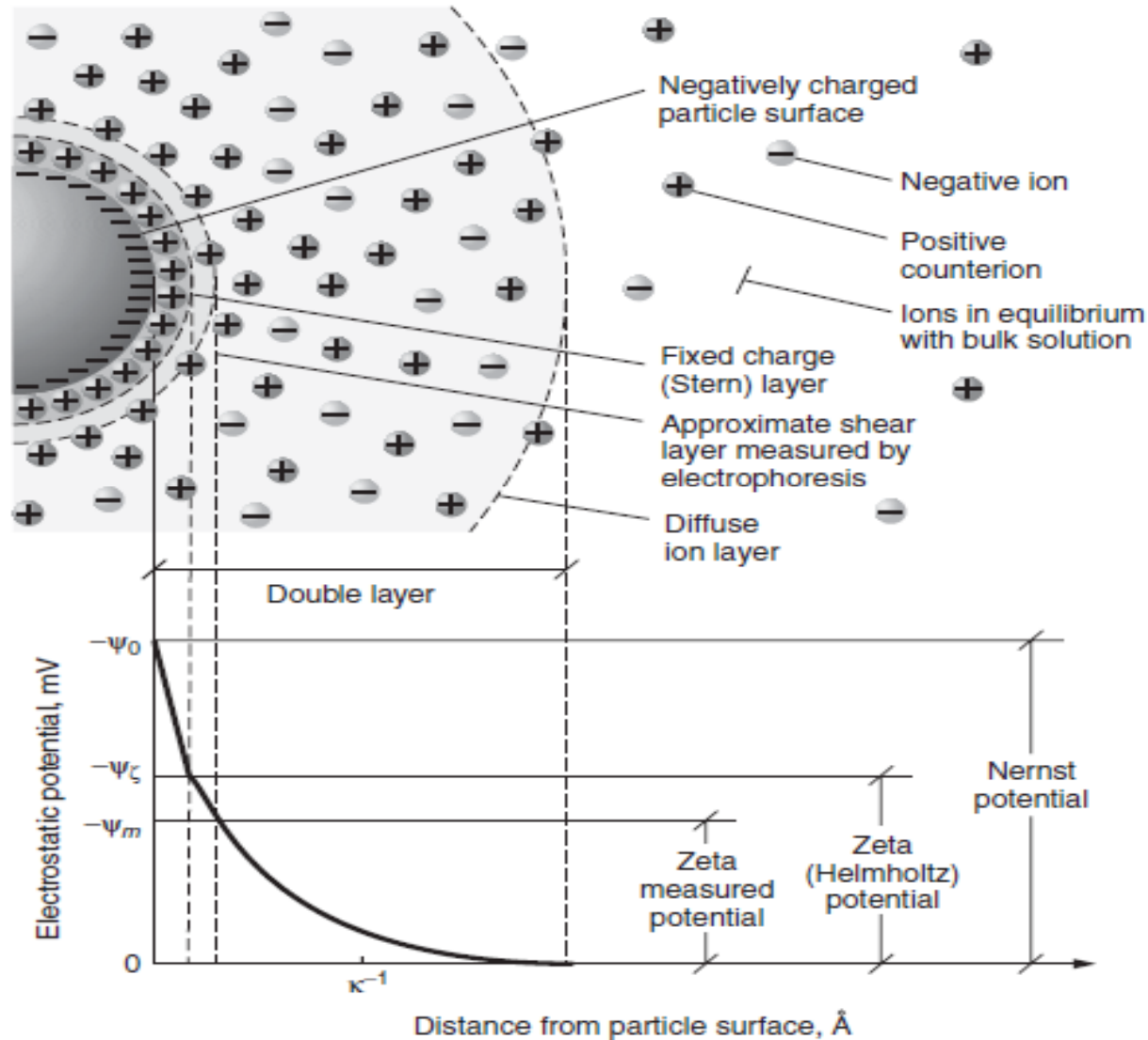


The integral of the combined forces is the energy barrier

Mechanisms of coagulation

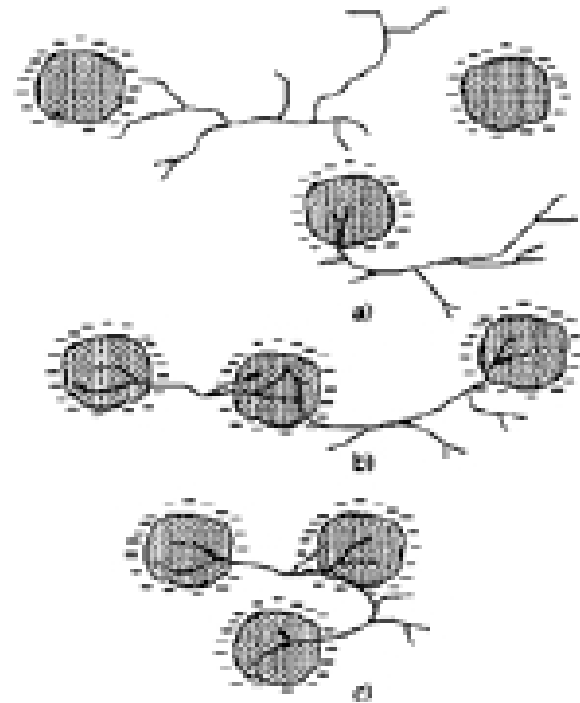
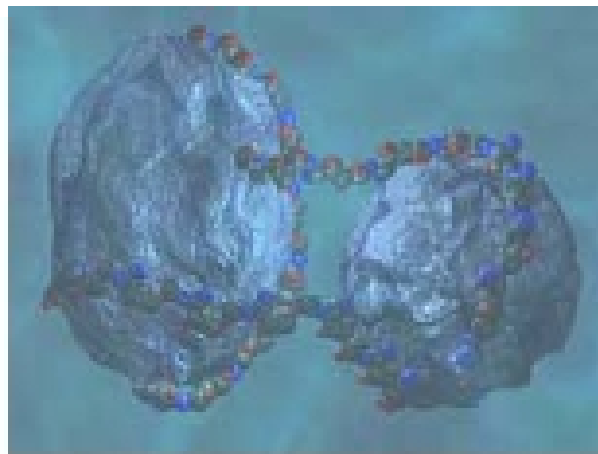
- Compression of the double layer (ionic layer surrounding the colloidal particle)
- Adsorption and charge neutralization
- Enmeshment of colloids in a precipitate
- Interparticle bridging

The Electrical Double Layer



Flocculation aids

- The chain is long enough to allow active groups to bond to multiple colloids



Floc formation with polymers

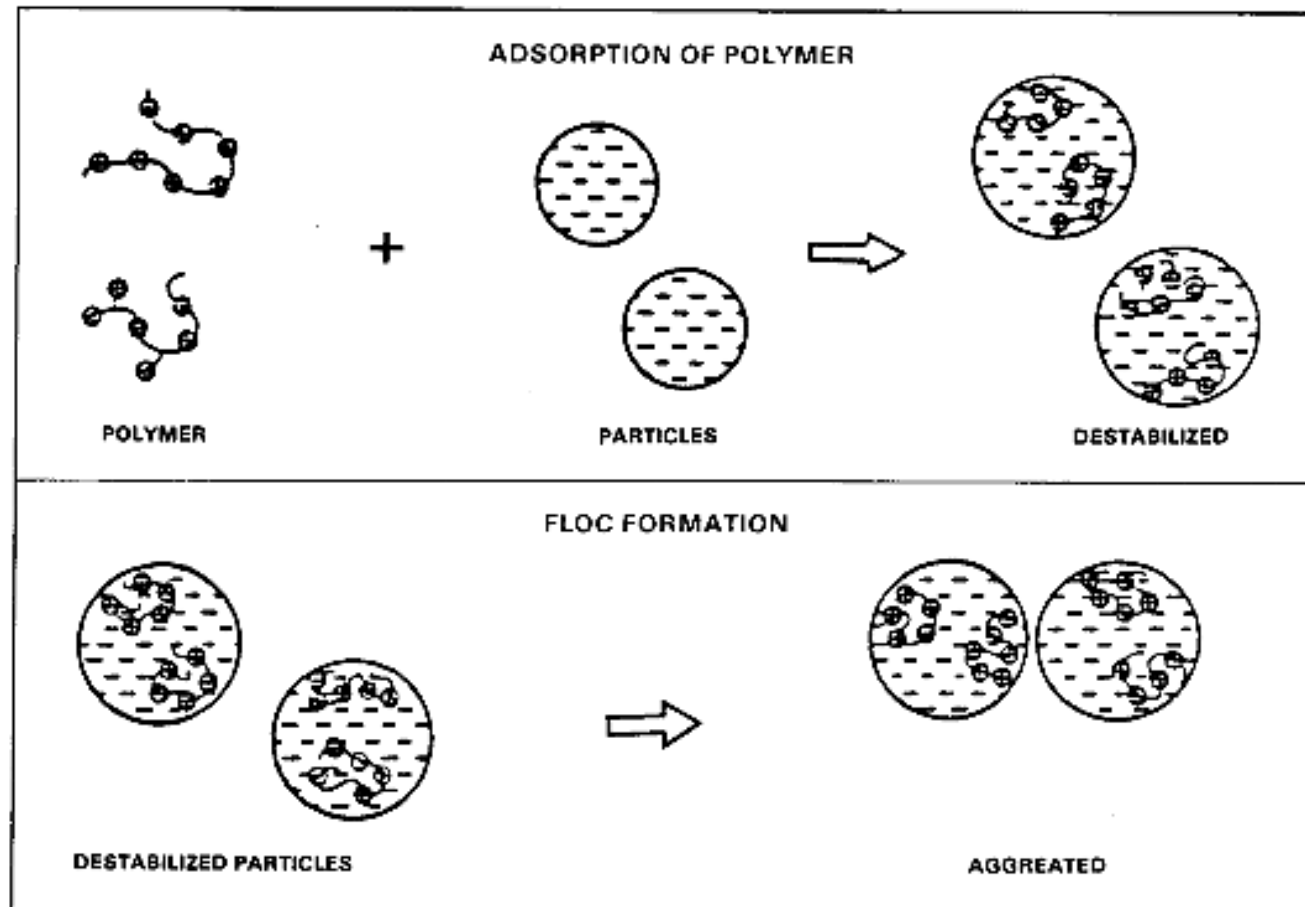


Figure 1-2. Floc formation process

Jar Tests

- ❑ The jar test – a laboratory procedure to determine the optimum pH and the optimum coagulant dose
- ❑ A jar test simulates the coagulation and flocculation processes

Determination of optimum pH

- ❑ Fill the jars with raw water sample (500 or 1000 mL) – usually 6 jars
- ❑ Adjust pH of the jars while mixing using H_2SO_4 or NaOH /lime (pH: 5.0; 5.5; 6.0; 6.5; 7.0; 7.5)
- ❑ Add same dose of the selected coagulant (alum or iron) to each jar (Coagulant dose: 5 or 10 mg/L)

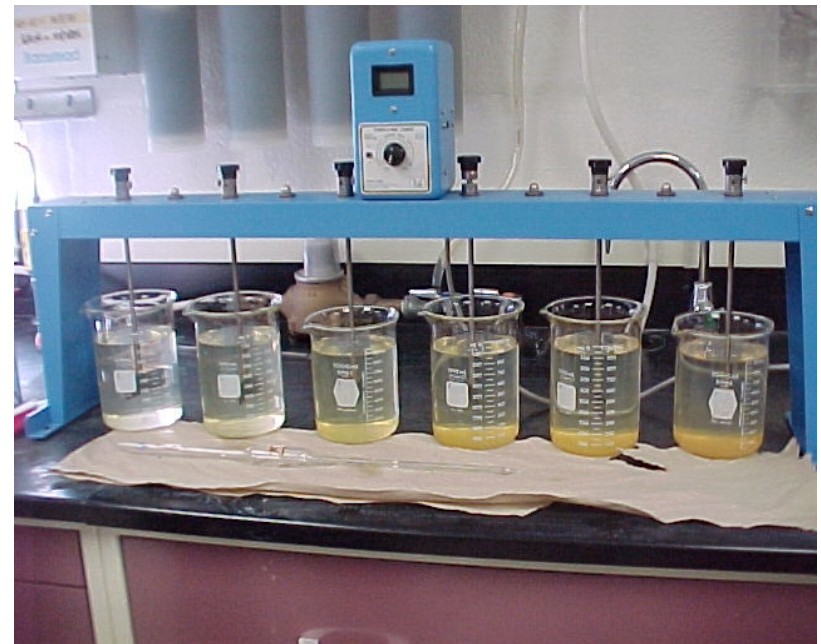


Jar Test

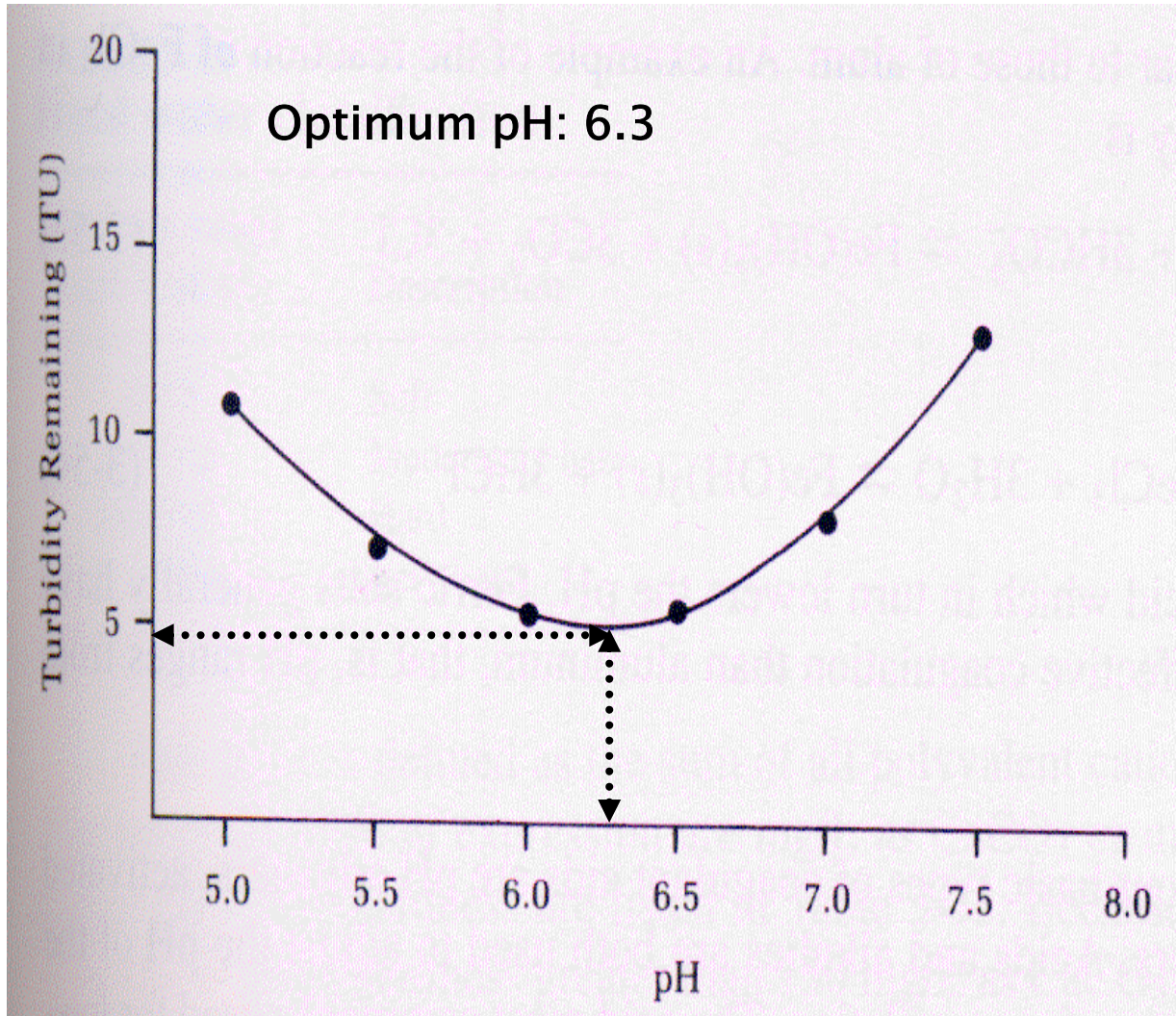
Jar Tests – determining optimum pH

- ❑ Rapid mix each jar at 100 to 150 rpm for 1 minute. The rapid mix helps to disperse the coagulant throughout each container
- ❑ Reduce the stirring speed to 25 to 30 rpm and continue mixing for 15 to 20 mins. This slower mixing speed helps promote floc formation by enhancing particle collisions, which lead to larger flocs
- ❑ Turn off the mixers and allow flocs to settle for 30 to 45 mins
- ❑ Measure the final residual turbidity in each jar
- ❑ Plot residual turbidity against pH

Jar Test set-up

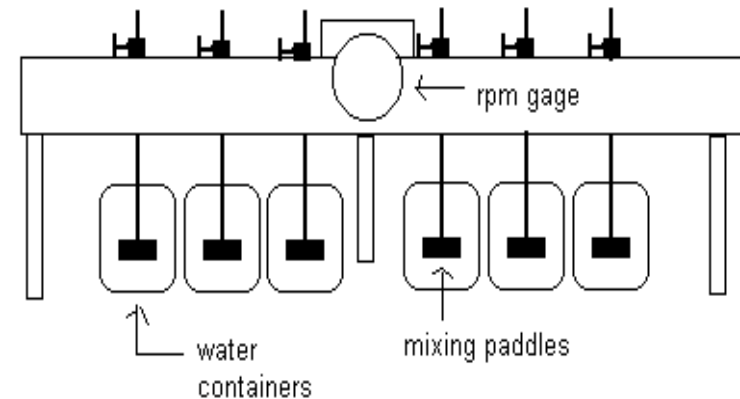


Jar Tests – optimum pH



Optimum coagulant dose

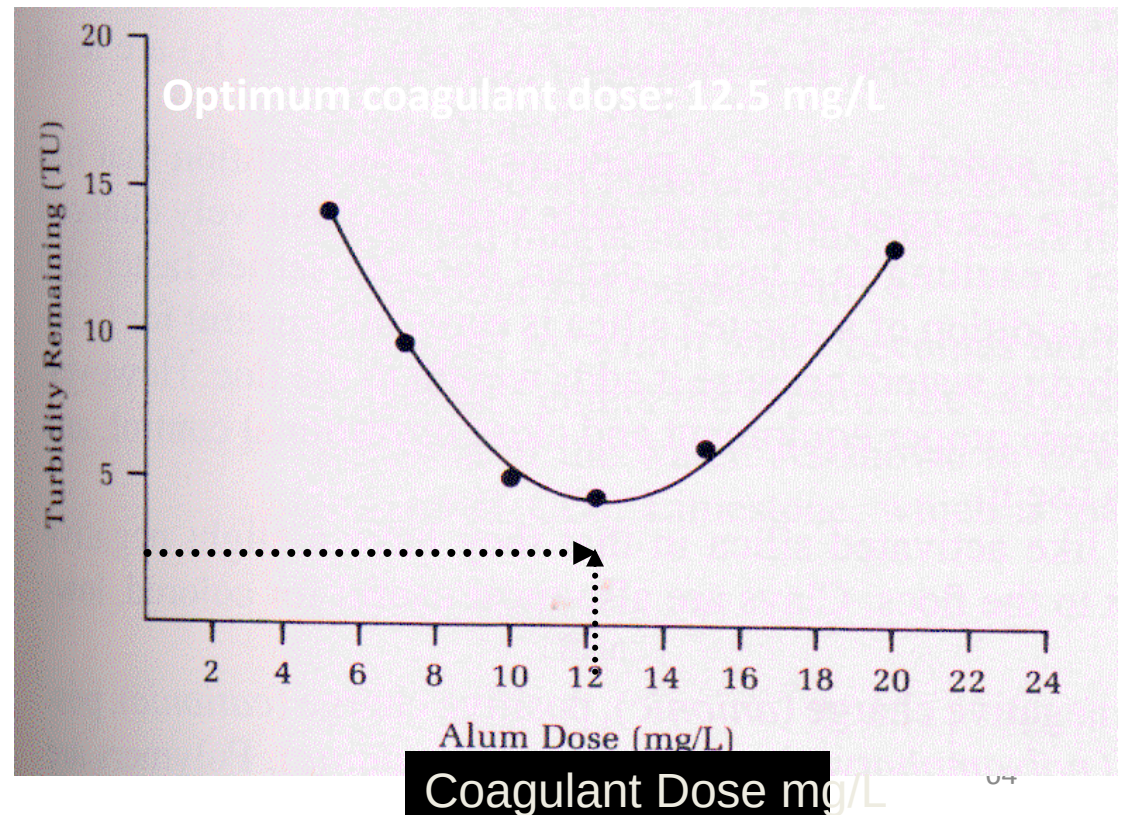
- ☐ Repeat all the previous steps
- ☐ This time adjust pH of all jars at optimum (6.3 found from first test) while mixing using H_2SO_4 or NaOH/lime
- ☐ Add different doses of the selected coagulant (alum or iron) to each jar (Coagulant dose: 5; 7; 10; 12; 15; 20 mg/L)
- ☐ Rapid mix each jar at 100 to 150 rpm for 1 minute. The rapid mix helps to disperse the coagulant throughout each container
- ☐ Reduce the stirring speed to 25 to 30 rpm for 15 to 20 mins



Optimum coagulant dose

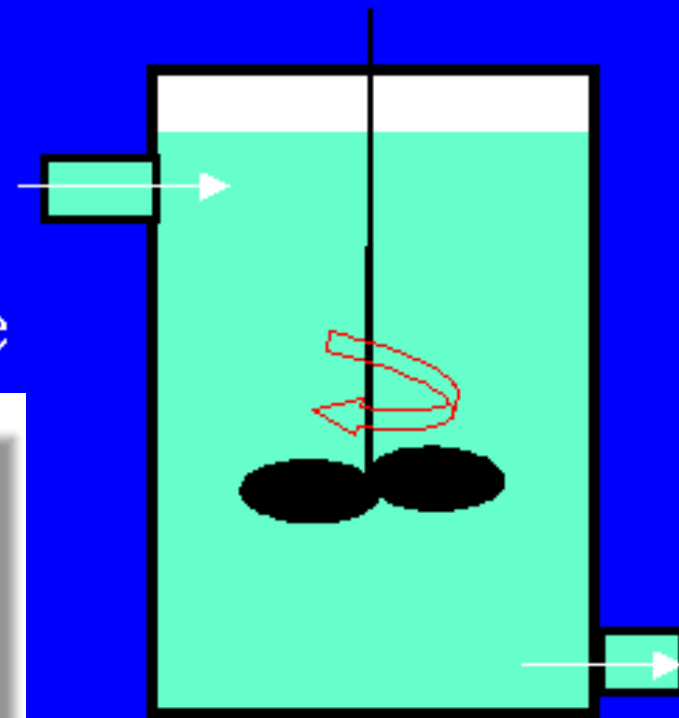
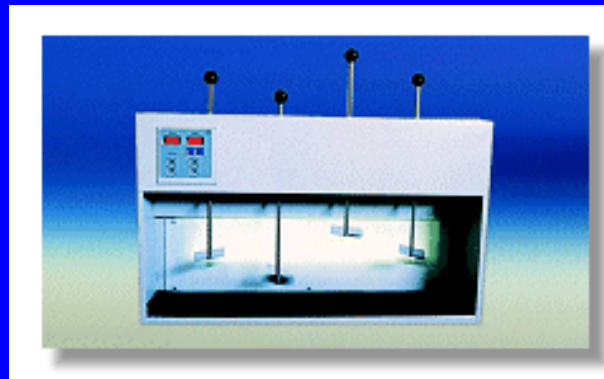
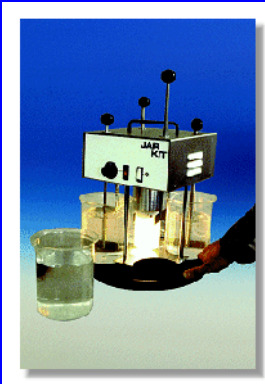
- ❑ Turn off the mixers and allow flocs to settle for 30 to 45 mins
- ❑ Then measure the final residual turbidity in each jar
- ❑ Plot residual turbidity against coagulant dose

The coagulant dose with the lowest residual turbidity will be the optimum coagulant dose



Coagulant Addition: Rapid Mix

- ◆ Mixer
 - vertical shaft turbine impeller
- ◆ Tank
 - 3 to 10 ft diameter
 - flow through, top to bottom
 - 30 to 60 second detention time

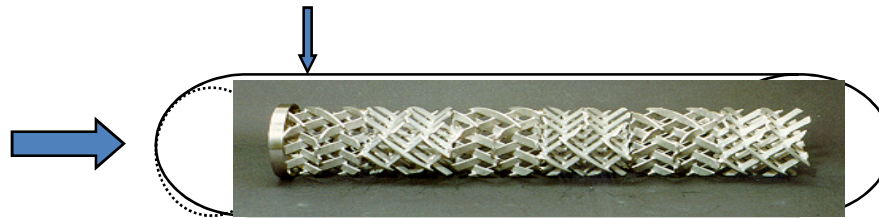


- *Hydraulic Jump*: Hydraulic Jump creates turbulence and thus help better mixing.



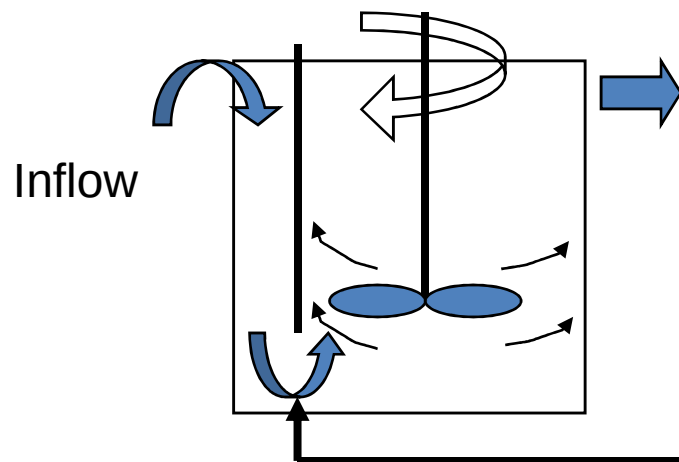
Coagulant

- *In-line flash mixing*

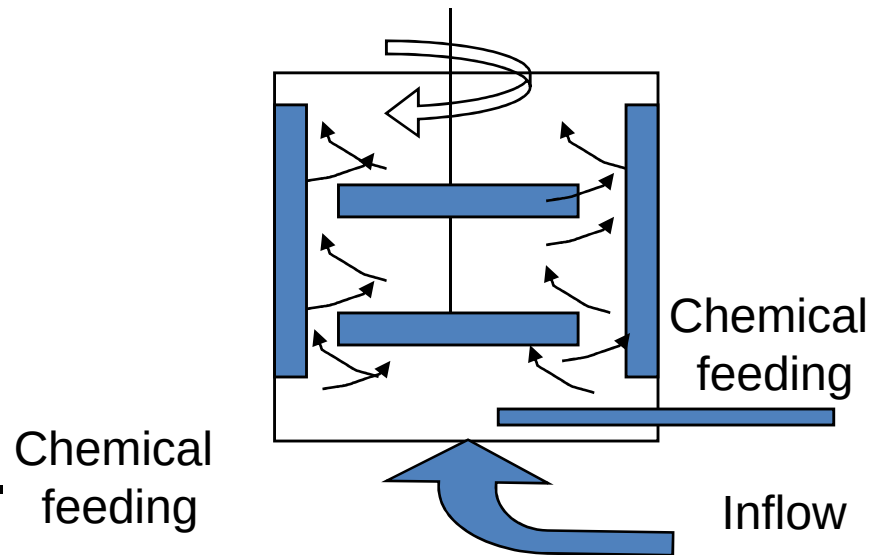


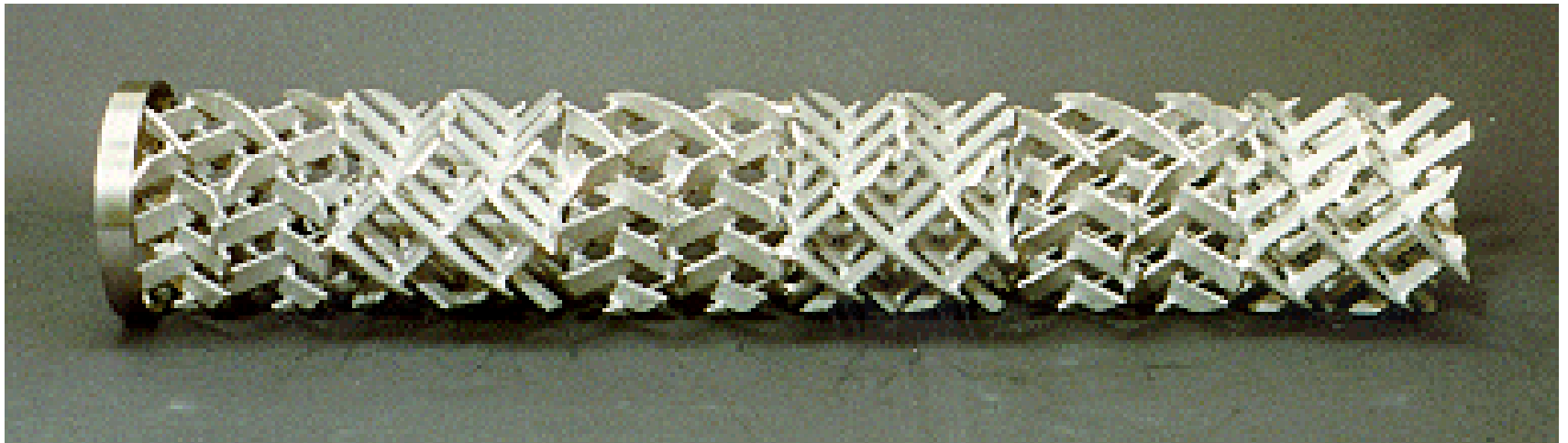
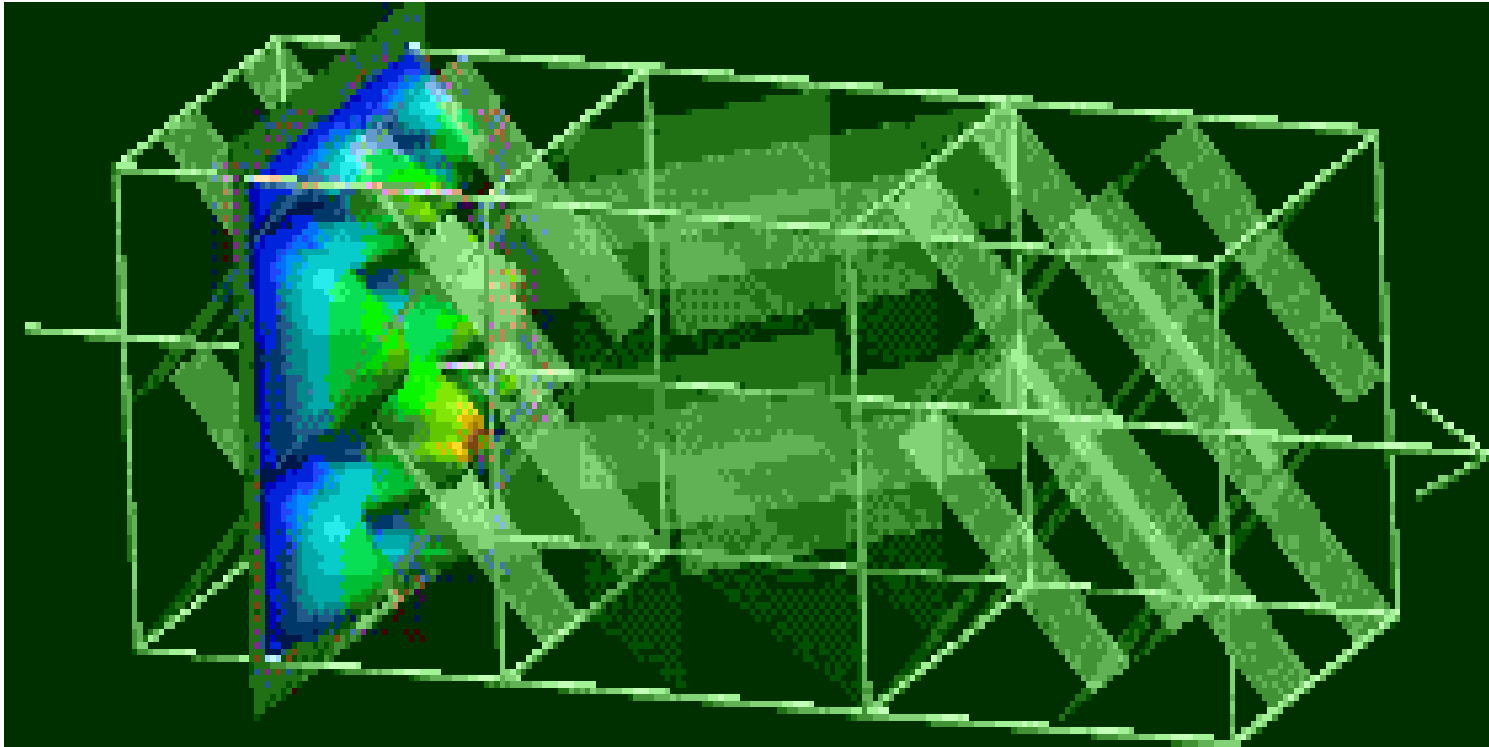
- *Mechanical mixing*

Back mix impeller



flat-blade impeller







❑ Relative coagulating power

$\text{Na}^+ = 1;$ $\text{Mg}^{2+} = 30$
 $\text{Al}^{3+} > 1000;$ $\text{Fe}^{3+} > 1000$

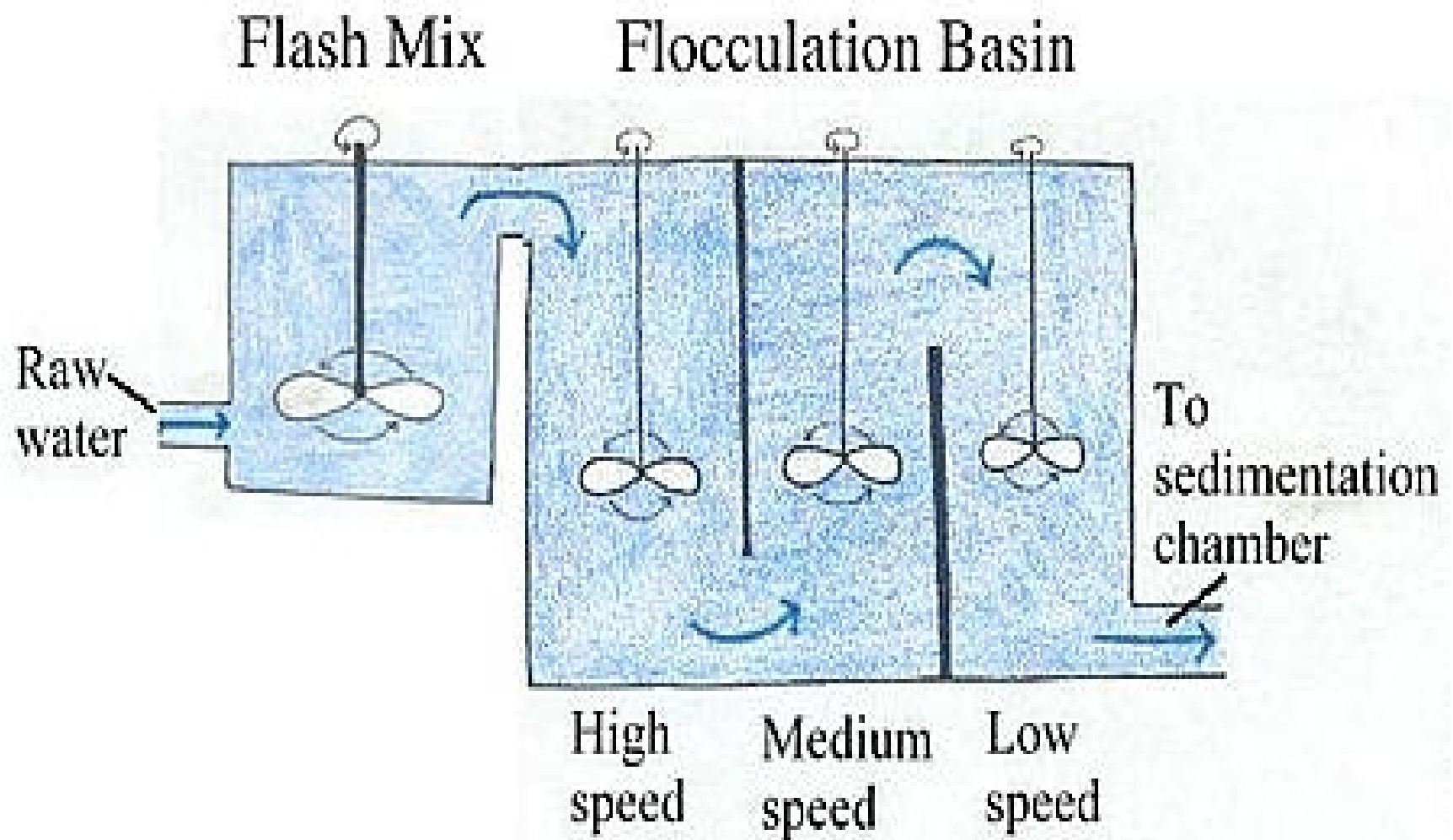
❑ Typical coagulants

Aluminum sulfate: $\text{Al}_2(\text{SO}_4)_3 \cdot 14 \text{H}_2\text{O}$

Iron salt- Ferric sulfate: $\text{Fe}_2(\text{SO}_4)_3$

Iron salt- Ferric chloride: Fe_2Cl_3

Polyaluminum chloride (PAC): $\text{Al}_2(\text{OH})_3\text{Cl}_3$



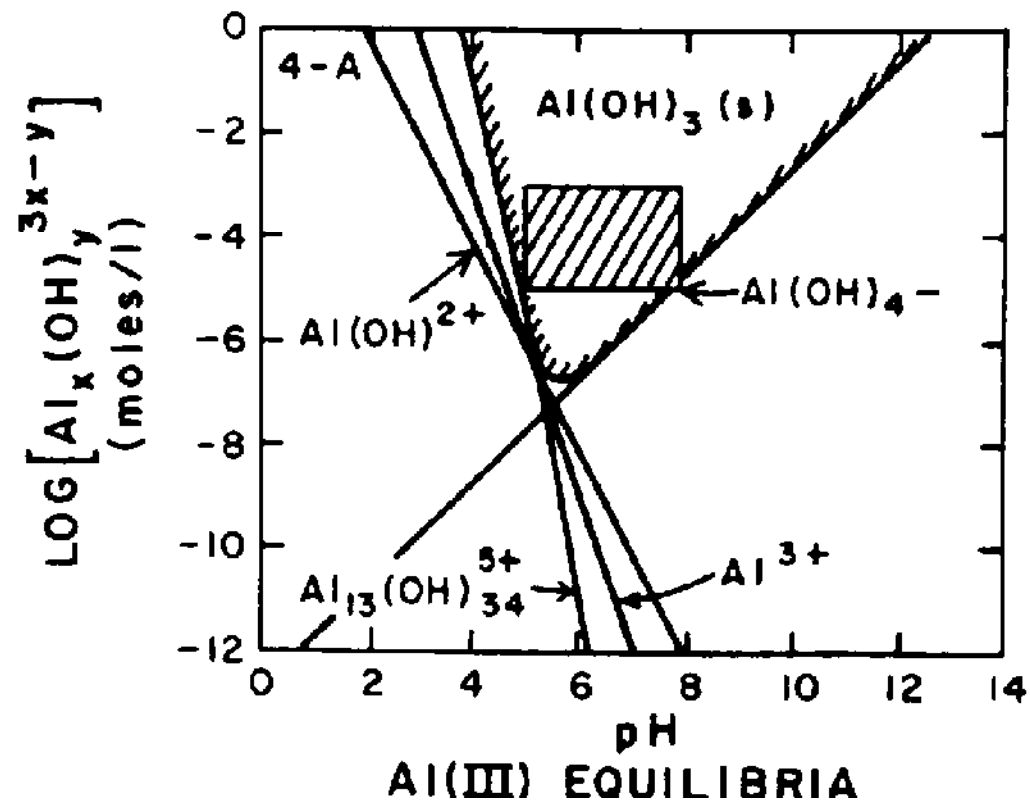
Stoichiometry of metal ion coagulants



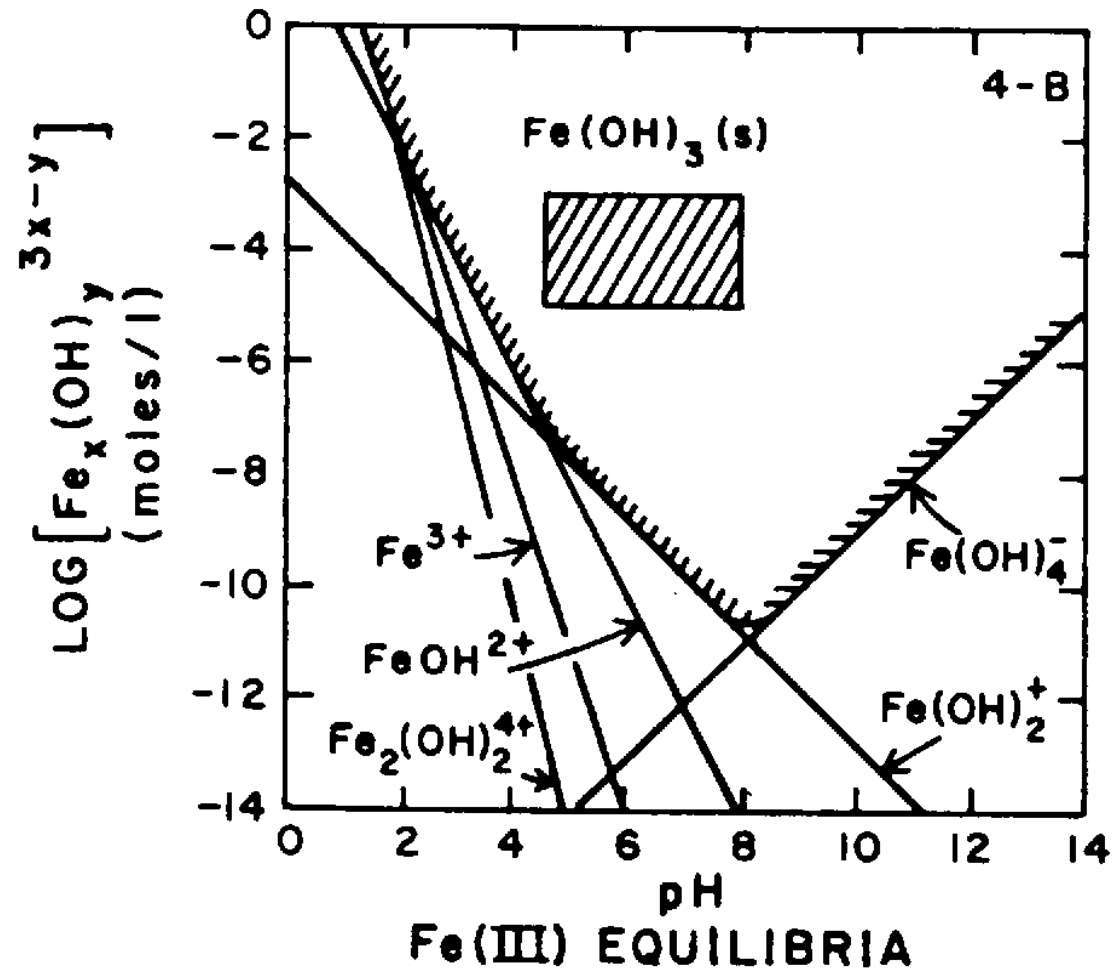
If alkalinity is not enough, pH will reduce greatly. Caustic soda, Lime or Soda ash may be needed to neutralize the acid.



Al^{3+} species as a function of pH



Fe species as a function of pH



Examples

1. For 1 g of alum calculate
 - a) The amount of insoluble $\text{Al}(\text{OH})_3$ produced
 - b) The amount of alkalinity consumed as CaCO_3

2. If 200 mg/L of ferric chloride is to be added for coagulation, how much alkalinity is consumed in mg/L as CaCO_3 ?

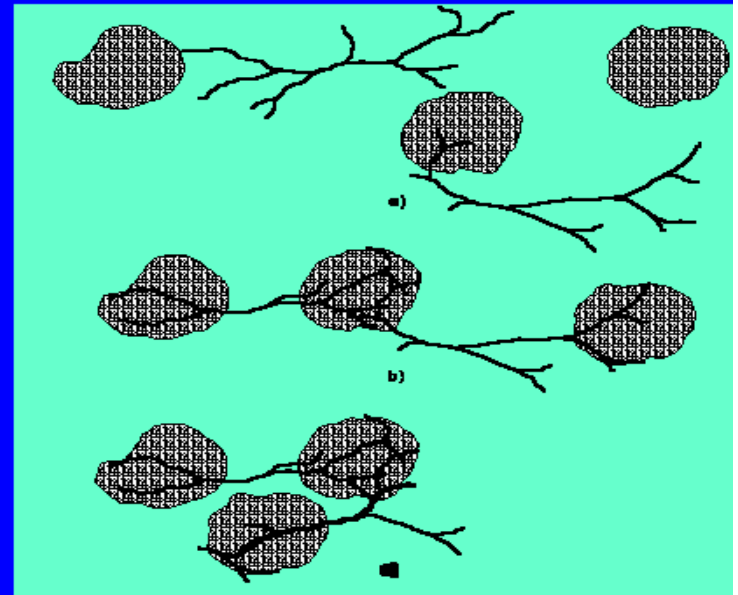
COAGULANT AIDS

Other substances than
coagulants used:

- Clay minerals
- Silicates
- Polymers

Polymers are often
either anionic or
cationic to aid
coagulation.
Polymers also
reinforce flocs

Destabilization with Polymers



Design of rapid mix

- ***Dimensions of the tank:***
 - Determine the tank volume for given detention time
 - Assume a depth.
 - Calculate the tank diameter/width
- ***Power requirements***
 - Calculate water horsepower.
 - Calculate electric horsepower.
 - Estimate power costs
 - Calculate impeller size and speed

Geometry of rapid-mix basin & Detention time

- Should provide uniform mixing
- Should minimize dead areas and short-circuiting
- Usually square basins for mechanical mixers
- Depth-to width ratio: about 2
- Detention time should provide sufficient time for homogenization of the chemicals with water
- Typical detention time: 10 s to 5 min

Average detention time

$$t_d = V/Q$$

t_d : detention time, min; V: volume of tank, m³; Q: flow rate, m³/min

Agitation requirements

$$G = \frac{dv}{dy} = \left(\frac{P}{V \mu} \right)^{1/2}$$

Where,

G: velocity gradient, s⁻¹ (100 to 1000s⁻¹)

P: power imparted to the water, watt

V : tank volume, m³ ; μ: absolute viscosity, Pa-s

The motor power can be calculated if the motor efficiency is known (motor efficiency = 80-90%).

Power imparted to the water, P

It can be calculated by

$$P = 2\pi nT$$

$$P = N_p \mu n^2 d^3 \text{ (for laminar flow, } NR < 10)$$

$$P = N_p \rho n^3 d^5 \text{ (for turbulent flow, } NR < 10,000)$$

$$N_R = d^2 n \rho / \mu$$

Where,

n: impeller speed, rps

T: impeller shaft torque, N-m

N_p : power number of the impeller (given for different impellers)

d: impeller diameter

ρ : density of fluid, kg/m³

Recommended values for coagulants

- Dosage
 - Alum: 10-150 mg/L
 - Ferric sulfate: 10-250 mg/L
 - Ferric chloride: 5-150 mg/L
- pH
 - Alum: 5.5-7.7
 - Fe³⁺: 5-8.5
- Initial mixing time (Al³⁺ and Fe³⁺)
 - 1 s and preferably less than 0.5 s

FLOCCULATION

Flocculation - agglomeration of colloids by collisions to form separable flocs

Two stages processes:

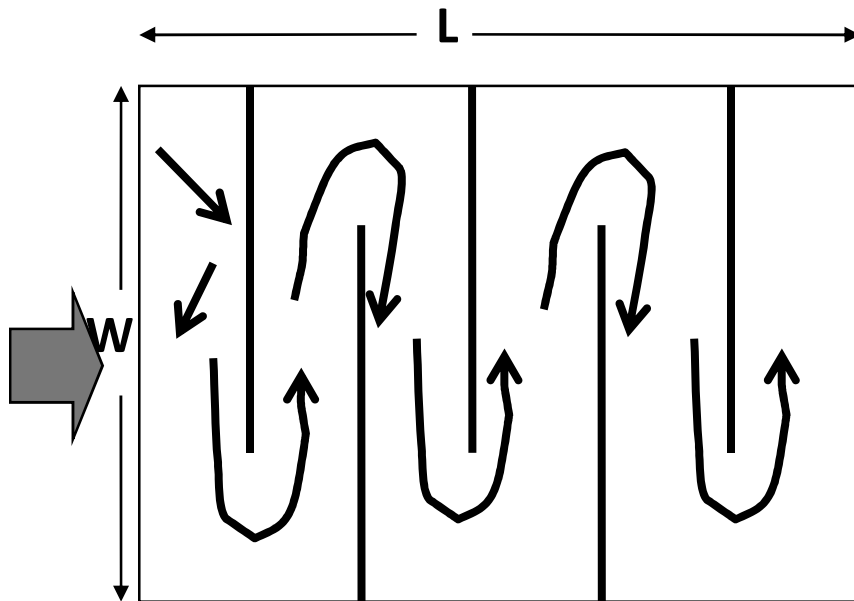
- **1st stage: Perikinetic, collisions from Brownian motion**
- **2nd stage: Orthokinetic, collisions through stirring**

Flocculators:

- **Hydraulic (baffled channel basin)**
- **Mechanical Mixing**
 - **Paddles**
 - **Vertical-shaft turbines**

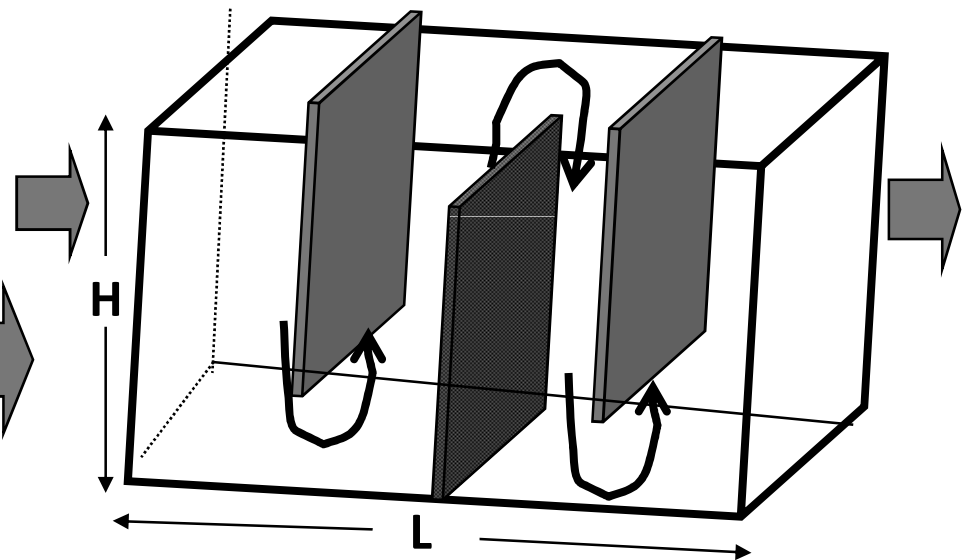
Baffled channel basin

Horizontal baffled tank



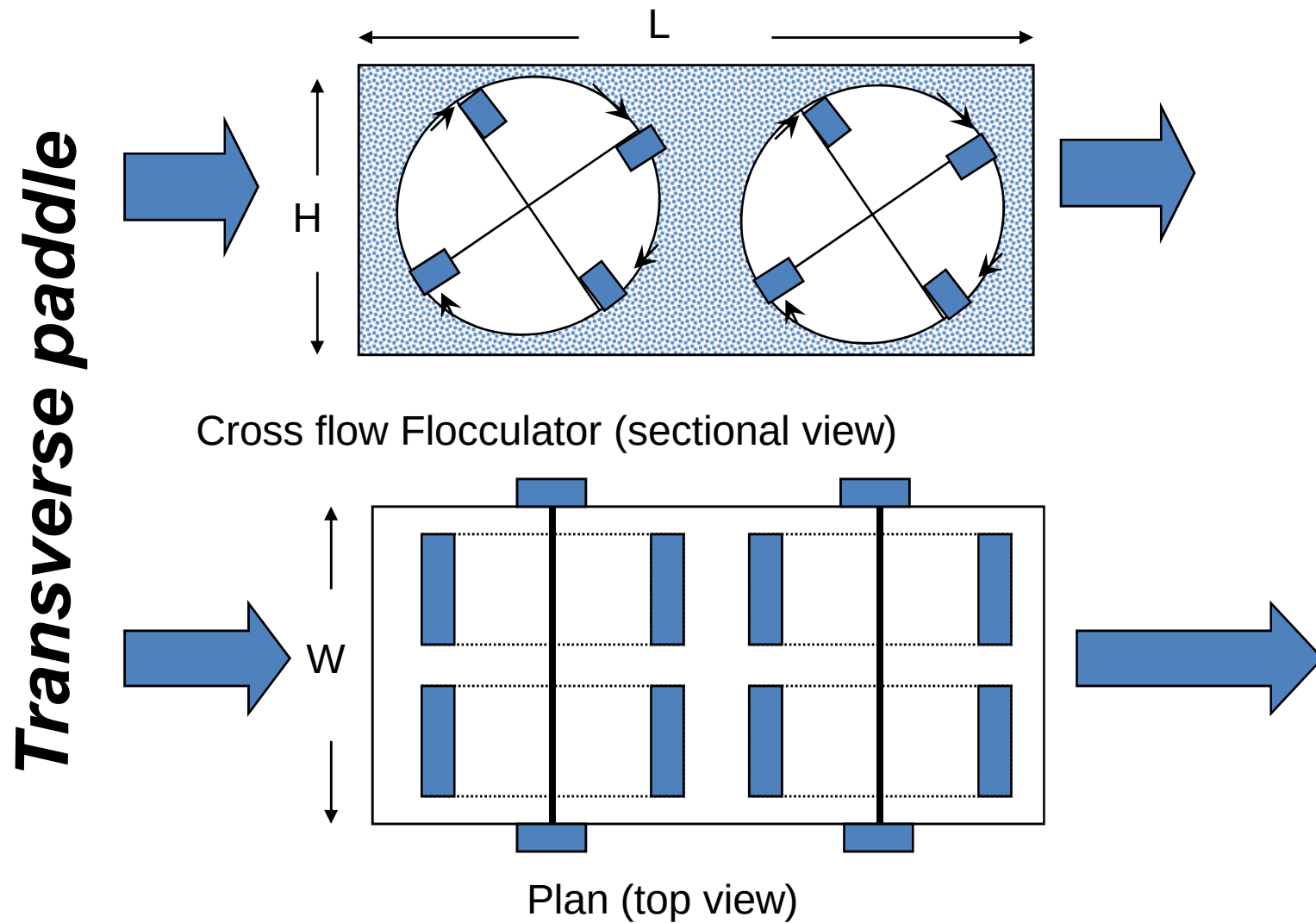
Plan view (horizontal flow)

Vertical baffled tank

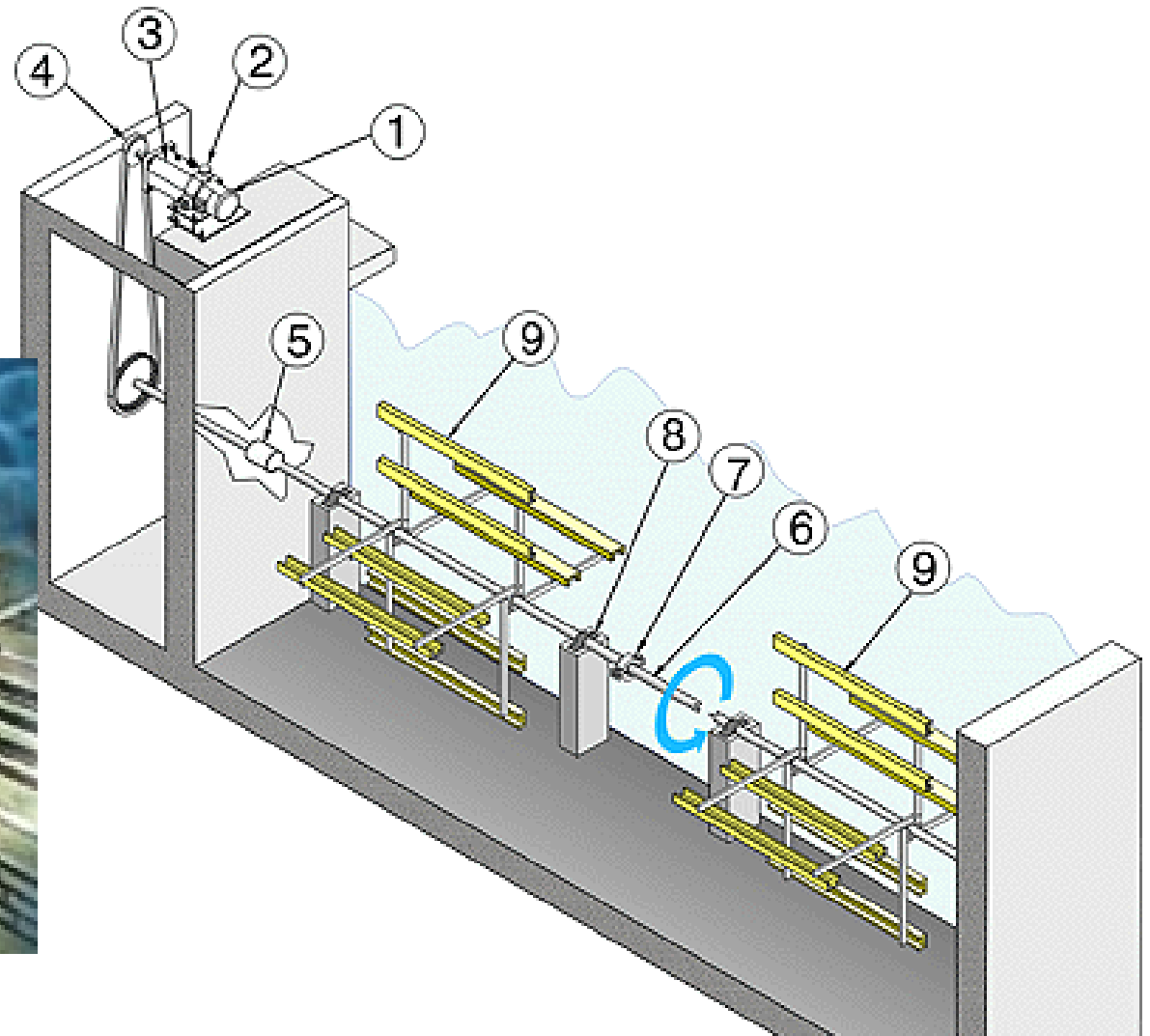


Isometric View (vertical flow)

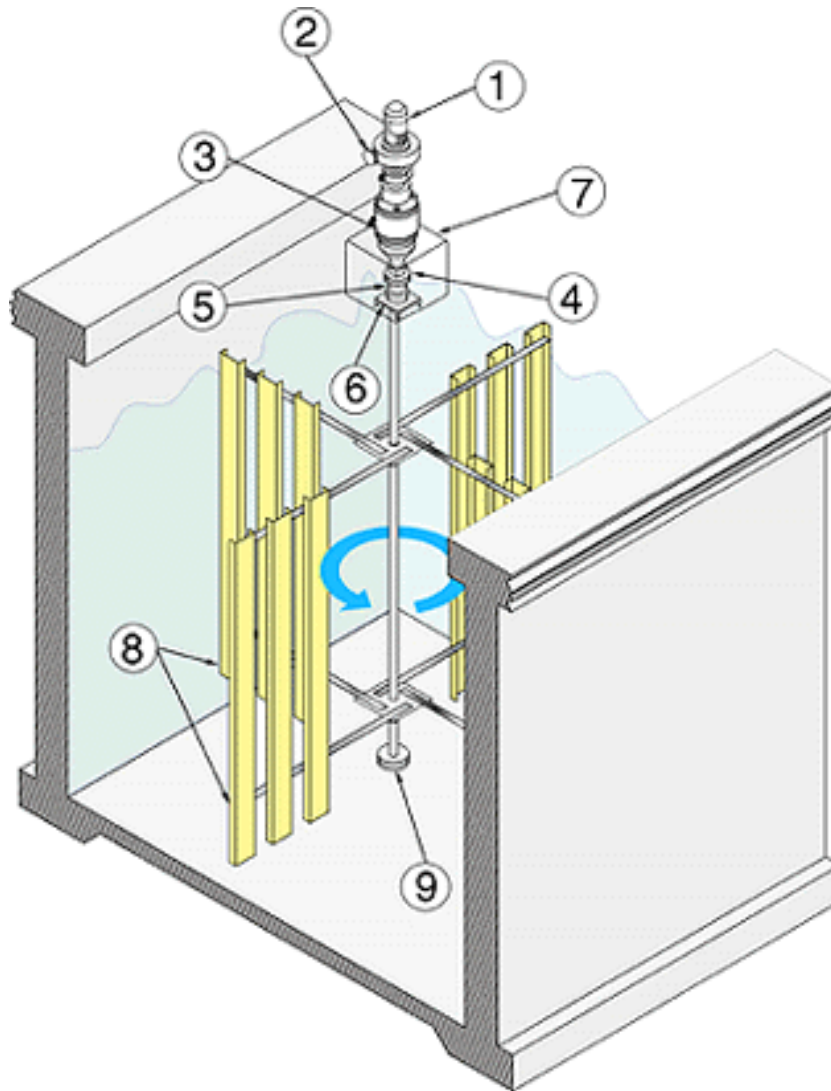
Mechanical Flocculator- Paddle



Paddle- horizontal shaft



Paddle- vertical shaft



Comparison of flocculators

Process issue	Paddles	Vertical-shaft turbine	Hydraulic
Floc type	Large and fluffy	Small to medium floc	Very large floc
Head loss	None	None	0.05-.015 m
Operational flexibility	Good, limited to low G	Excellent	Moderate to poor
Capital cost	Moderate to high	Moderate	Low to moderate
Construction difficulty	Moderate	Easy to moderate	Easy to difficult
Maintenance effort	Moderate	Low to moderate	Low to moderate
Compartmentalization	Moderate	Excellent	Excellent
Advantages	Large floc; Reliable; No head loss; One shaft for several mixers	No shutdown during maintenance; no head loss; flexible; reliable; high energy	Simple and effective; easy, low cost maintenance; no moving parts; large flocs
Disadvantages	Replacement and some maintenance requires shutdown	Difficult to specify proper impeller and reliable gear drives	Little flexibility

Flocculator Diffuser Wall



Design of Paddle Flocculator

- ***Flocculator's dimensions***
 - Decide on the number of channels.
 - Calculate the flow in one channel.
 - Calculate the volume of one channel.
 - Assume a depth.
 - Calculate the width and length
- ***Check the flow through velocity***
 - Calculate the cross-sectional area of one channel.
 - Calculate the velocity in one channel.
 - Determine whether the flow through velocity is acceptable.
- ***Calculate the power requirements:***
 - Calculate the volume of the entire flocculator.
 - Calculate water horsepower for the flocculator.
 - Calculate electric horsepower for the flocculator.
 - Estimate power costs for the flocculator.

Agitation requirement

$$G = \frac{dv}{dy} = \left(\frac{P}{V \mu} \right)^{1/2}$$

- P for paddle flocculator is calculated as:

$$P = \frac{C_D A \rho v^3}{2}$$

Where,

C_D = dimensionless drag coefficient for plates moving with faces normal to direction of motion

A = cross-sectional area of the paddles, m^2

v = velocity of paddle relative to fluid, m/s (70-80% of the paddle speed)

ρ = density of water, kg/m^3

Detention time and geometry of flocculation basin

- Typical detention time: 20 to 60 minutes
- Key design factor is the Camp number, Gt (typical value: 10, 000 to 150, 000)
- Use multiple compartments in series to minimize short-circuiting
- Separate each compartment by diffuser(baffle) wall
- Typical velocity criteria:
 - Rapid mixing unit to flocculation basin: 0.45-0.9 m/s
 - Flow through the flocculation basin: 0.15-0.45 m/min
 - Baffle-wall: 0.3-0.45 m/s
 - Pipe between flocculation and sedimentation basins: 0.15-0.45 m/s
 - End baffle-wall b/n flocculation & sedimentation basins: 0.15 m/s

Typical values for paddle wheel flocculator

Parameter	Value
Diameter of wheel, m	3-4
Paddle board section, mm	100 x 200
Paddle board length, m	2-3.5
$A_{\text{paddle boards}}/\text{tank section area, \%}$	< 20
Cd, L/W = 1	Cd = 1.16
L/W = 5	Cd = 1.20
L/W = 20	Cd = 1.50
L/W > 20	Cd = 1.90
Paddle tip speed, m/s	Strong floc, 4 Weak floc, 2
Spacing between paddle wheels on the same shaft, m	1
Clearance from basin walls, m	0.7
Minimum basin depth, m	1 m > diam of paddle wheel
Minimum clearance between stages, m	1

Design criteria for hydraulic flocculation

Parameter	Value
Average G, s ⁻¹	30-40
Channel velocities, m/s	0.15-0.45
Minimum residence time, min	20
Head loss at 180° turn, velocity head	3.2-3.5
Head loss through slit or port, velocity head	1.5
Minimum distance between baffles, m	0.75
Minimum water depth, m	1

$$\overline{G} = \frac{\rho g \Delta H}{\mu t}$$

Head loss from turbulence and friction on the sides of the channel $\Delta H = \frac{Lv^2}{C^2 R_h}$

Head loss resulting from each 180° turn $\Delta H = k \left(\frac{v^2}{2g} \right)$

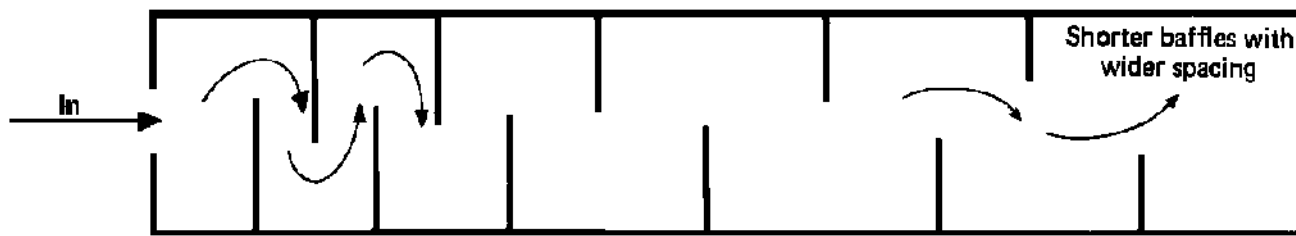
Where, ρ = density of water, kg/m³; g = acceleration due to gravity, m/s²; ΔH = head loss through baffles, m; μ = dynamic viscosity, Pa.s; t = detention time, s; L = length of channel, m; v = velocity of flow, m/s; C = Chezy coefficient, unitless; R_h = hydraulic radius of channel, m; k = empirical constant (2.5 to 4)

Design guides for diffuser walls

Parameter	Guideline
Opening area, % of flow cross-section	2-5
Velocity through orifice	
Dividing first and second floc basins, m/s	0.55
Dividing floc and sedimentation basins, m/s	0.35
Headloss across baffle	
Dividing first and second floc basins, mm	7-9
Dividing floc and sedimentation basins, mm	3-4
Submergence of highest port, mm	15
Clearance below baffle for sludge, mm	25

Tapered flocculation

- Rapid flocculation rate near the inlet
- Agitation decreases progressively towards the outlet
- Allows floc growth without disruption
- Methods (e.g. varying baffle spacing or size of the flocculator compartments)



Example 1- Power Calculation

What horsepower level do we need to supply to a flocculation basin to provide a G value of 100s^{-1} and a Gt of 100,000 for $0.438\text{ m}^3/\text{s}$ flow?
(Given: $\mu = 0.89 \times 10^{-3}\text{ Pa.s}$; $1\text{ hp} = 745.7\text{ watts}$)

Examples 2

A water treatment plant of design capacity 378500 m³/d requires 100 mg/L of alum to remove 25 mg/L suspended solids. Estimate the mass of alum required every day (kg/day) and the maximum mass of dry solids (in kg/day) that are removed from the plant.

Examples 3

Design a mechanical rapid mix tank used for treating 75700 m³/d of water that has a temperature of 10°C. The items you need to determine are: 1) tank volume 2) tank dimensions and 3) power consumption. Use a detention time of 2 minutes, velocity gradient of 850 s⁻¹ and cubic tank geometry.

Examples 4

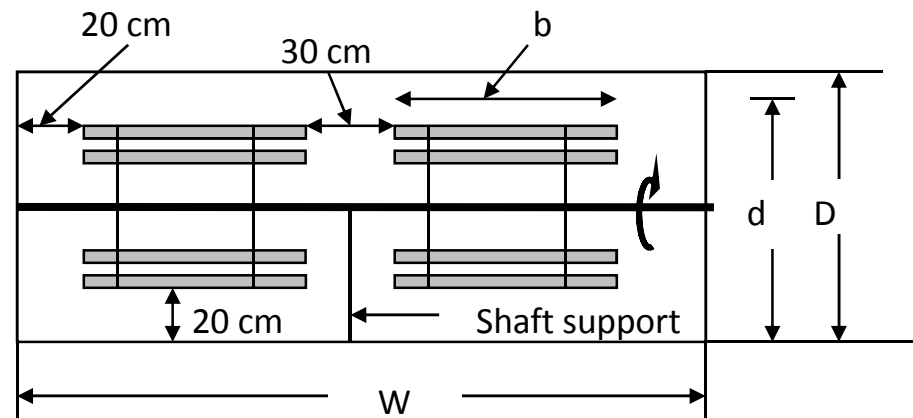
A water treatment plant is designed to process 100×10^6 L/d. The flocculator is 30 m long, 15 m wide, and 5 m deep. Revolving paddles are attached to four horizontal shafts that rotate at 1.5 rpm. Each shaft supports four paddles that are 200 mm wide, 15 m long and centered 2 m from the shaft. Assume the mean water velocity to be 70% less than paddle velocity and $C_D = 1.8$. All paddles remain submerged all the time. Find:

- a) the difference in velocity between paddles and water
- b) the value of G and
- c) the Camp number.

Example 5

Design a three-compartment flocculator with the configuration shown below to treat a design flow of $15000 \text{ m}^3/\text{d}$. The design G values are 45, 20, and 10 s^{-1} in each successive compartment and the total detention time in the flocculator will be 30 min. The maximum length of an individual paddle should not exceed 3.5 m and the width of a paddle should be between 10 and 20 cm. The minimum clear space between the outer paddle and the floor and water surface should be 20 cm. Shaft supports require a 30-cm spacing. The distance of a paddle from a wall should be at least 20 cm. The lowest temperature expected is 10°C , which is the design temperature. The maximum depth (D) of the flocculator is 4.3 m. Assume $C_D = 1.8$ and relative velocity of paddle = $0.75 \times$ paddle tip velocity, respectively.

Also, design baffle walls to be placed between compartments and at the exit end of the flocculation basin assuming maximum velocity to be 0.3 m/s .



Assignment-2

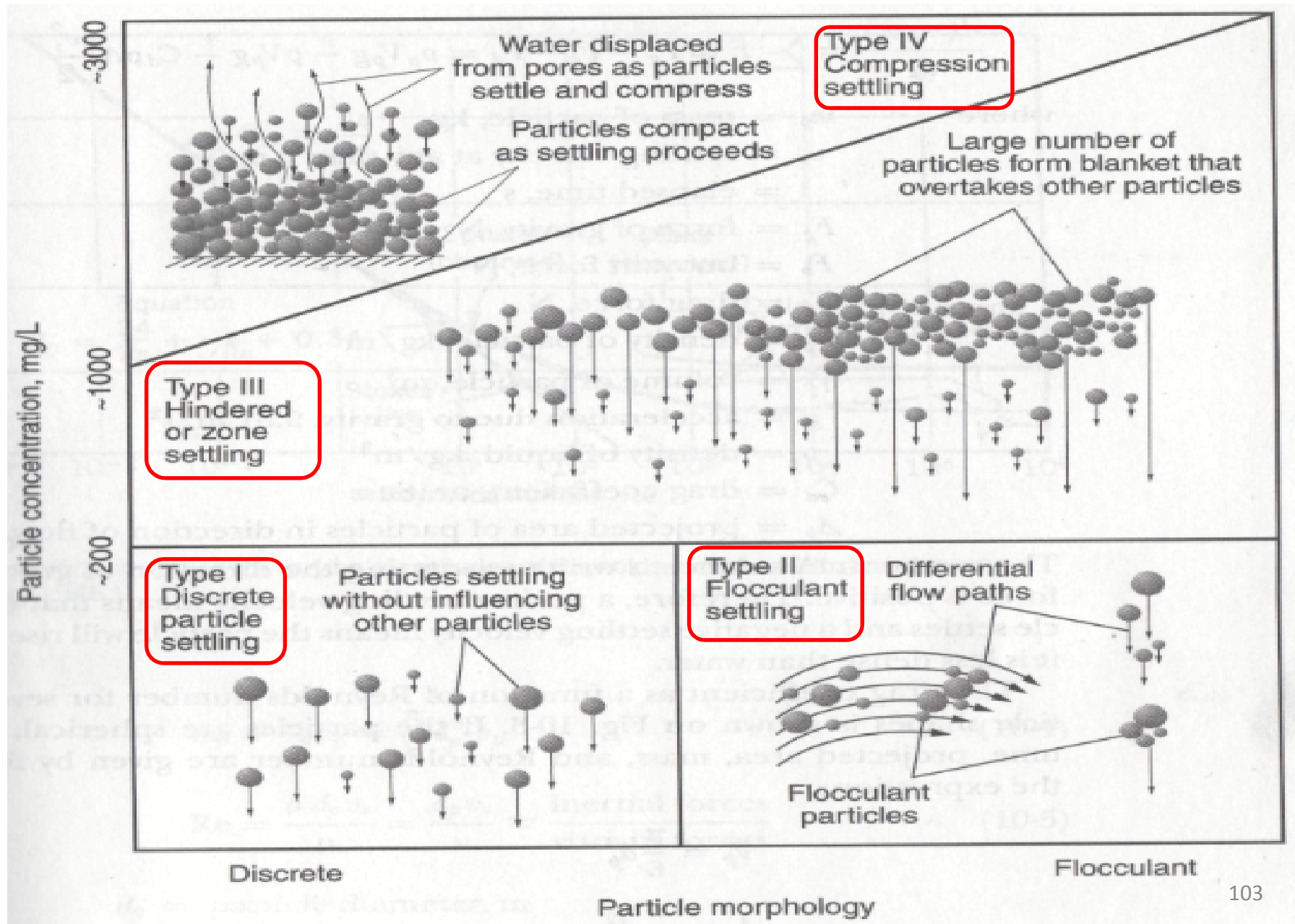
Make a Literature Review on Plant-based Natural Coagulants and report the key lessons drawn (maximum 10 pages)

3. GRAVITY SEPARATIONS

3.1 Sedimentation

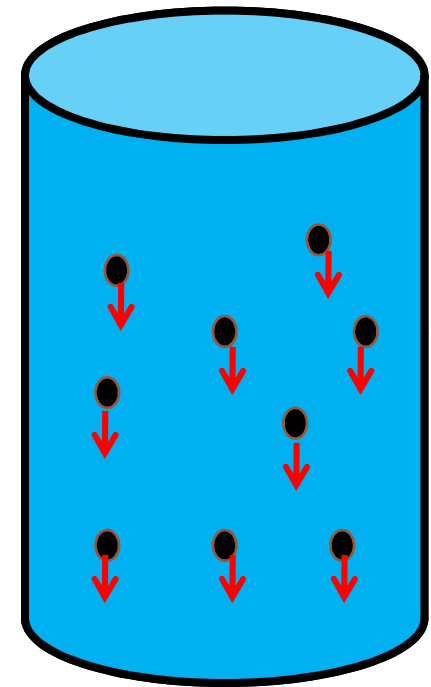
- **Sedimentation** and to a lesser extent, **Dissolved Air Flotation**, are commonly used solid/liquid separation processes that rely on gravity as the driving force.
- Sedimentation is removal of particulate materials suspended in water by quiescent settling due to gravity.
- It is a commonly used unit operation in water and wastewater treatment plants.
- *Water Treatment:*
 - Plain Sedimentation (Presedimentation)
 - Sedimentation after Flocculation
 - Sedimentation after Softening
- *Wastewater Treatment:*
 - Grit Removal
 - Primary Sedimentation
 - Biomass Sedimentation

Types of Sedimentation



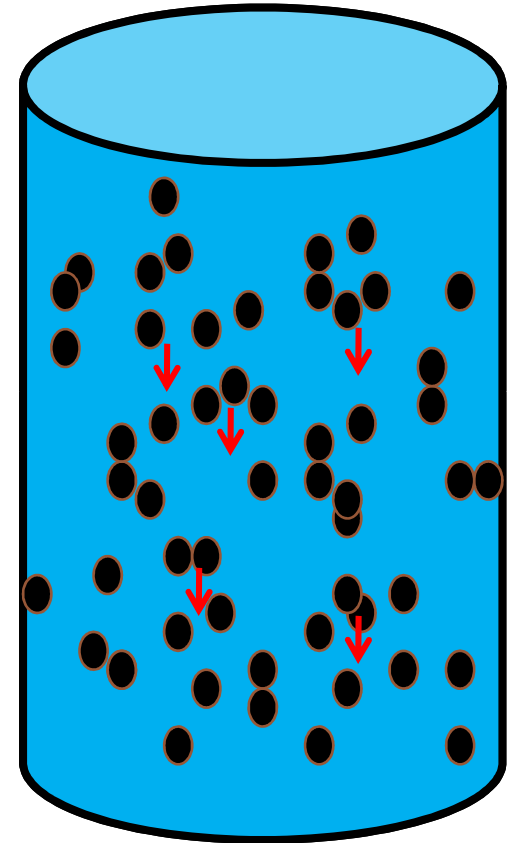
Type I: Discrete particle settling

- No interaction between particles
- Settling velocity is constant for individual particles
- Dilute solid's concentration
- Examples: presedimentation in water treatment, settling of sand particles during backwashing of rapid sand filters, grit removal in wastewater

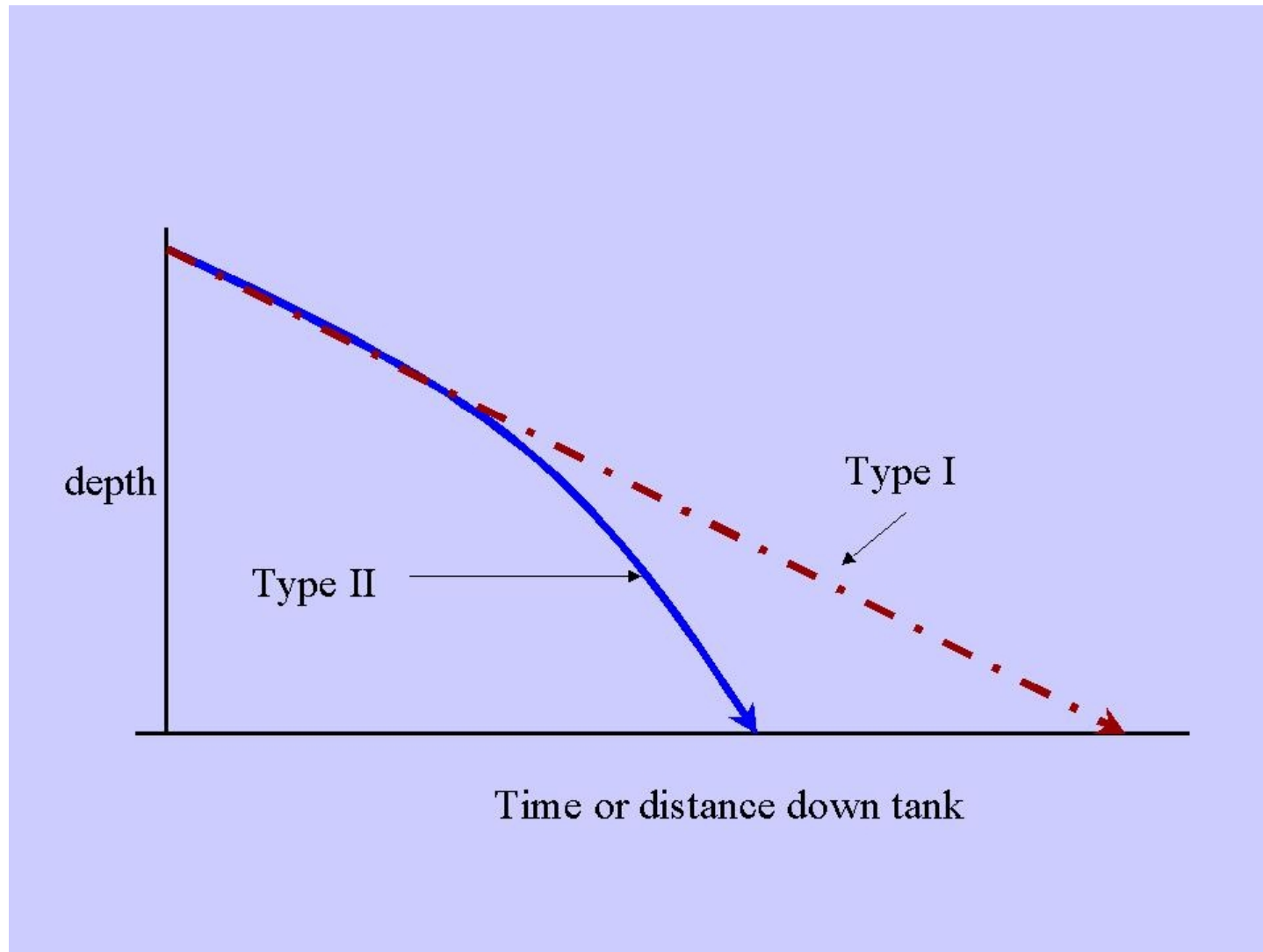


Type II: Flocculent settling

- Particles collide and adhere to each other resulting in particle growth
- Dilute solid's concentration
- Examples: coagulation/flocculation settling in water treatment and primary sedimentation in wastewater treatment



Comparison of Type I and II sedimentation

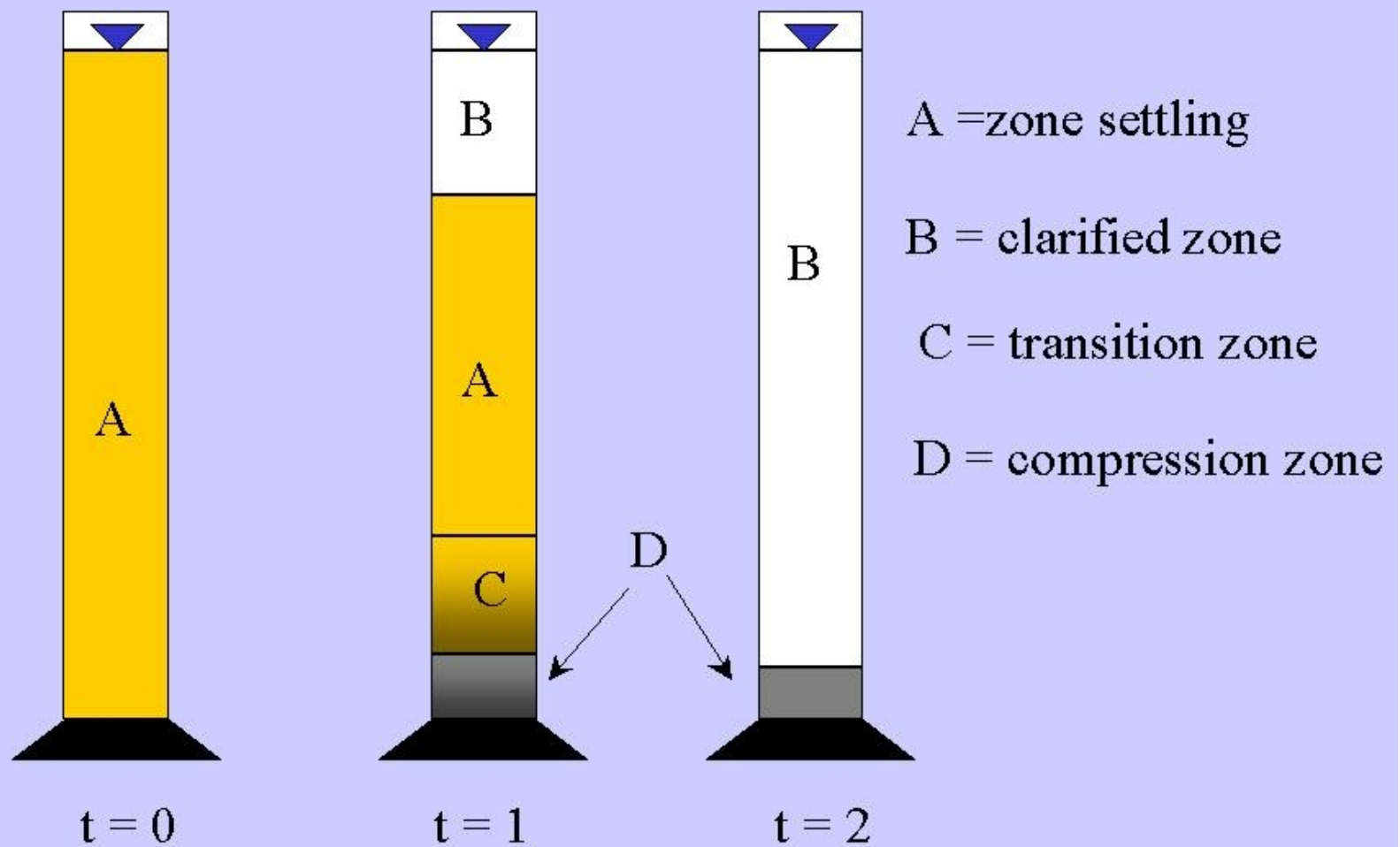


Type III: Hindered or zone settling

- Particles are so close together movement is restricted
- Intermediate solids concentration
- Solids move as a block rather than individual particles
- Fluidic interference causes a reduction in settling velocity
- Distinguishable solids liquid interface
- Intermediate solids concentration
- Example: settling of secondary effluents

Type IV: Compression settling

- Particles physically in contact
- Water is squeezed out of interstitial spaces
- Volume of solids may decrease
- High concentration of solids (sludge)



Modeling Discrete Settling

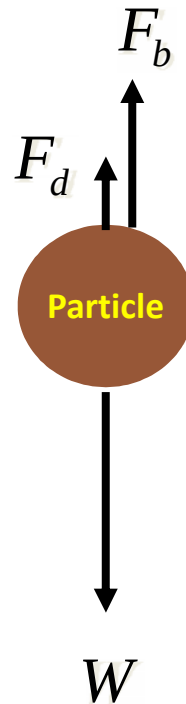
$$\sum F = ma$$

$$F_d + F_b - W = 0$$

$$F_b = \rho_w \nabla_p g$$

$$W = \rho_p \nabla_p g$$

$$F_d = C_d \rho A_p \frac{v_t^2}{2}$$



W = force of gravity, N

F_b = bouyant force, N

F_d = drag force, N

∇_p = Volume of particle, m^3

ρ_p = density of particle

ρ_w = density of water

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

C_d = drag coefficent, unitless

A_p = Projected area of particle in the direction of flow, m^2

v_t = settling velocity at any time, m/s

Terminal settling velocity

When forces are in equilibrium

$$F_d = W - F_b$$

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

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

for a speherical particle

$$V_p = \frac{4}{3} \pi r^3 \quad A_p = \pi r^2$$

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

$$V_t = \sqrt{\frac{4}{3} \frac{g d (\rho_p - \rho_w)}{C_D \rho_w}}$$

- d = particle diameter
- C_D is a function of Reynolds number, Re

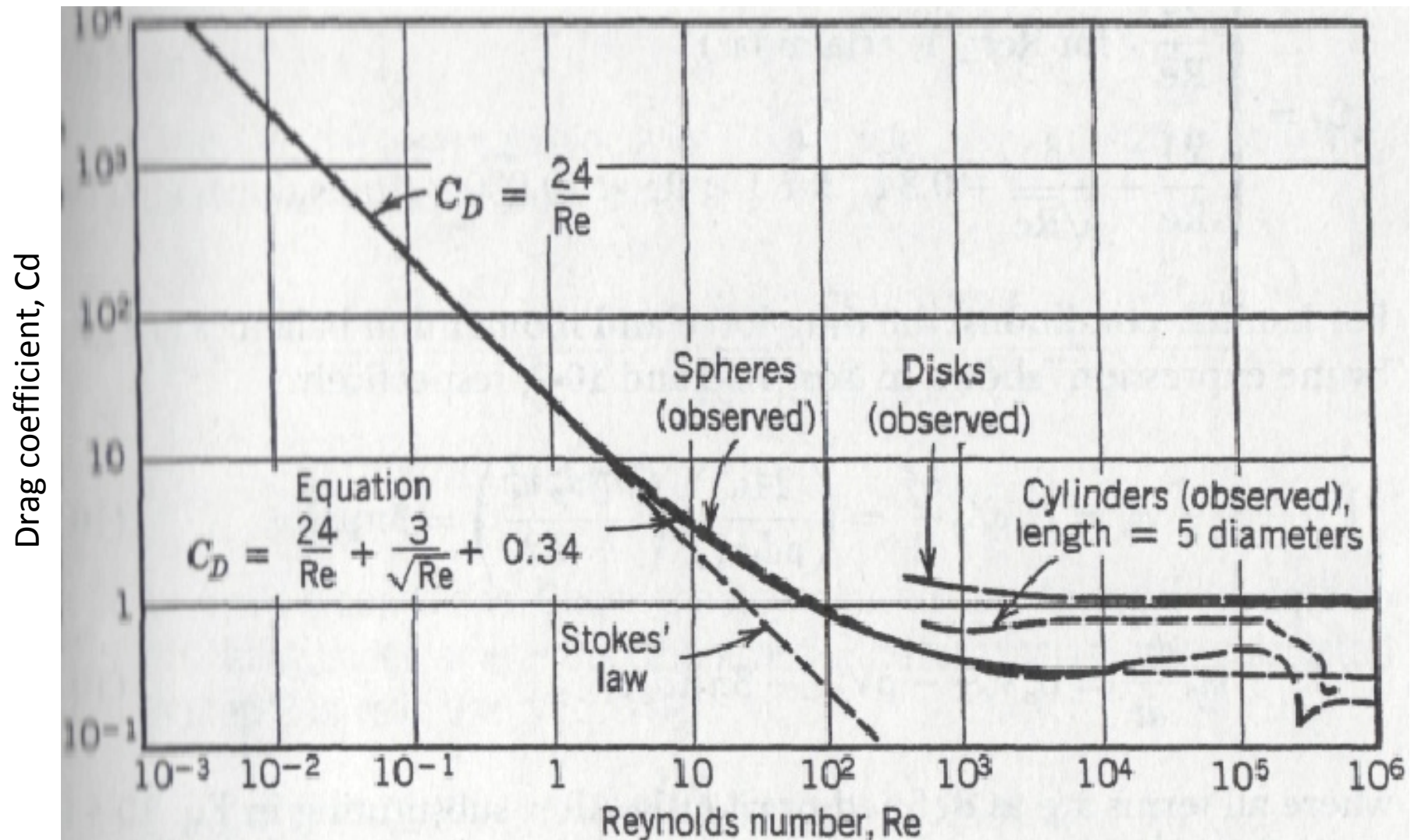
$$Re = \frac{V_t d \rho}{\mu} \quad \text{OR} \quad Re = \frac{V_t d}{\nu}$$

$$Re < 1, C_D = \frac{24}{Re} \quad \text{laminar}$$

$$1 < Re < 10^4, C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34 \quad \text{transition}$$

$$Re > 10^4, C_D \approx 0.4 \quad \text{turbulent}$$

Drag coefficients as a function of Reynolds number



Stokes Law

- Using $C_d = 24/R_e$:

$$V_t = \frac{d^2 g (\rho_p - \rho_w)}{18 \mu}$$

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

sg_p : specific gravity of particle
 ν : kinematic viscosity, m^2/s

- Stokes law has limited applications in water treatment
- V_t should be calculated using the general formula by trial and error

$$V_t = \sqrt{\frac{4 g d (\rho_p - \rho_w)}{3 C_D \rho_w}}$$

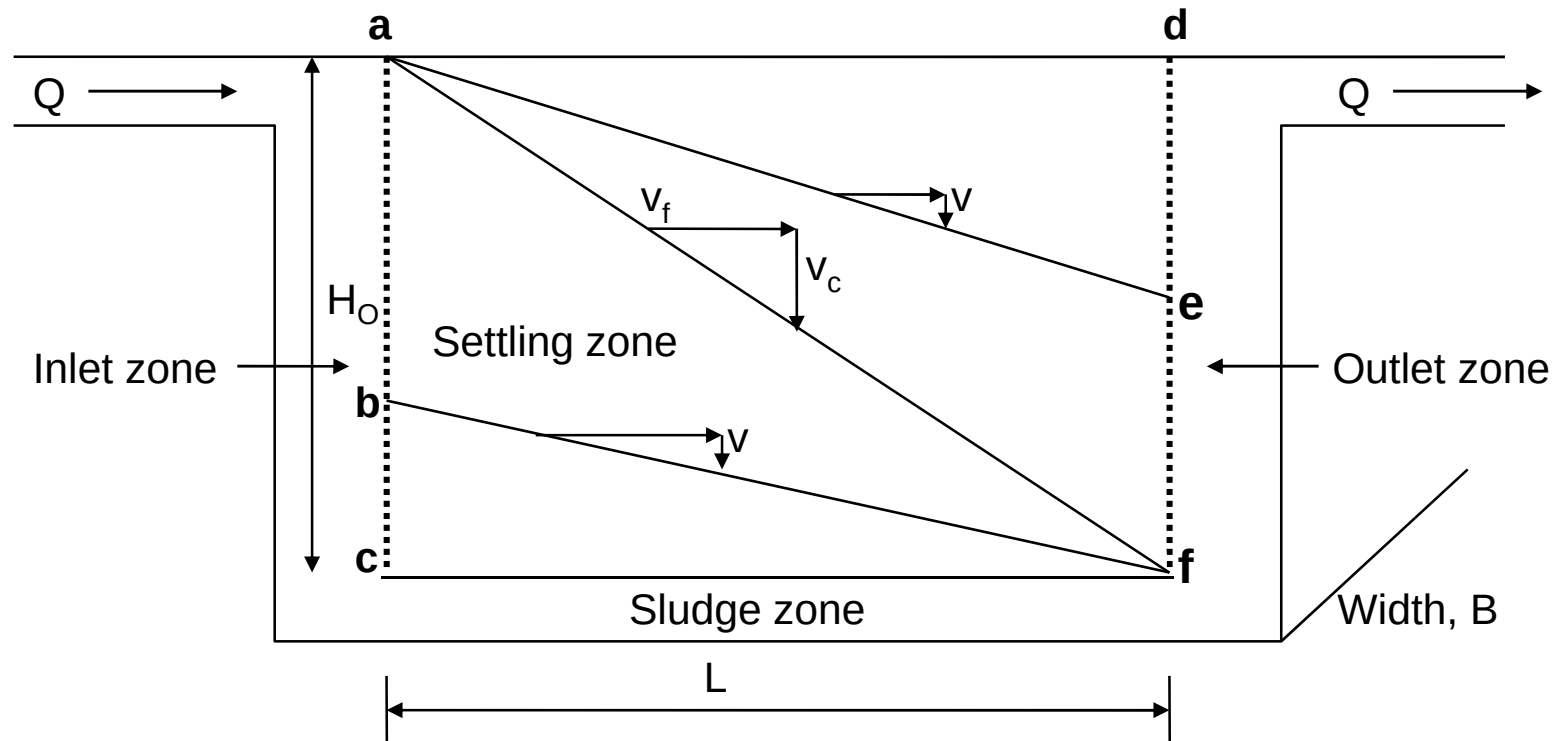
Example calculation of

Calculate the terminal settling velocity for sand in water at 10°C having particle diameters of 100 and 200 μm and density of 2650 kg/m^3 . For water at 10°C, $\nu = 1.306 \times 10^{-6} \text{ m}^2/\text{s}$

Ideal horizontal flow sedimentation basin

- Ideal sedimentation basin has four distinct zones
- Uniform dispersion of water and suspended particles in the inlet zone
- No particle settling in the inlet and outlet zones
- Continuous flow at constant rate
- Once a particle enters the sludge zone, it remains there (No resuspension of settled particles)
- The flow-through period is equal to the detention time
- Particles move forward with the same velocity as the liquid

Ideal horizontal flow sedimentation basin



$$v_c = \frac{H_o}{t_d} \quad \text{or} \quad v_c = \text{SOR} = \frac{Q}{A}$$

v_c : critical velocity, m/h

SOR: surface overflow rate, $\text{m}^3/\text{m}^2.\text{h}$

T_d : detention time, h

Q : flow rate, m^3/h

A : area of top of basin settling zone, m^2

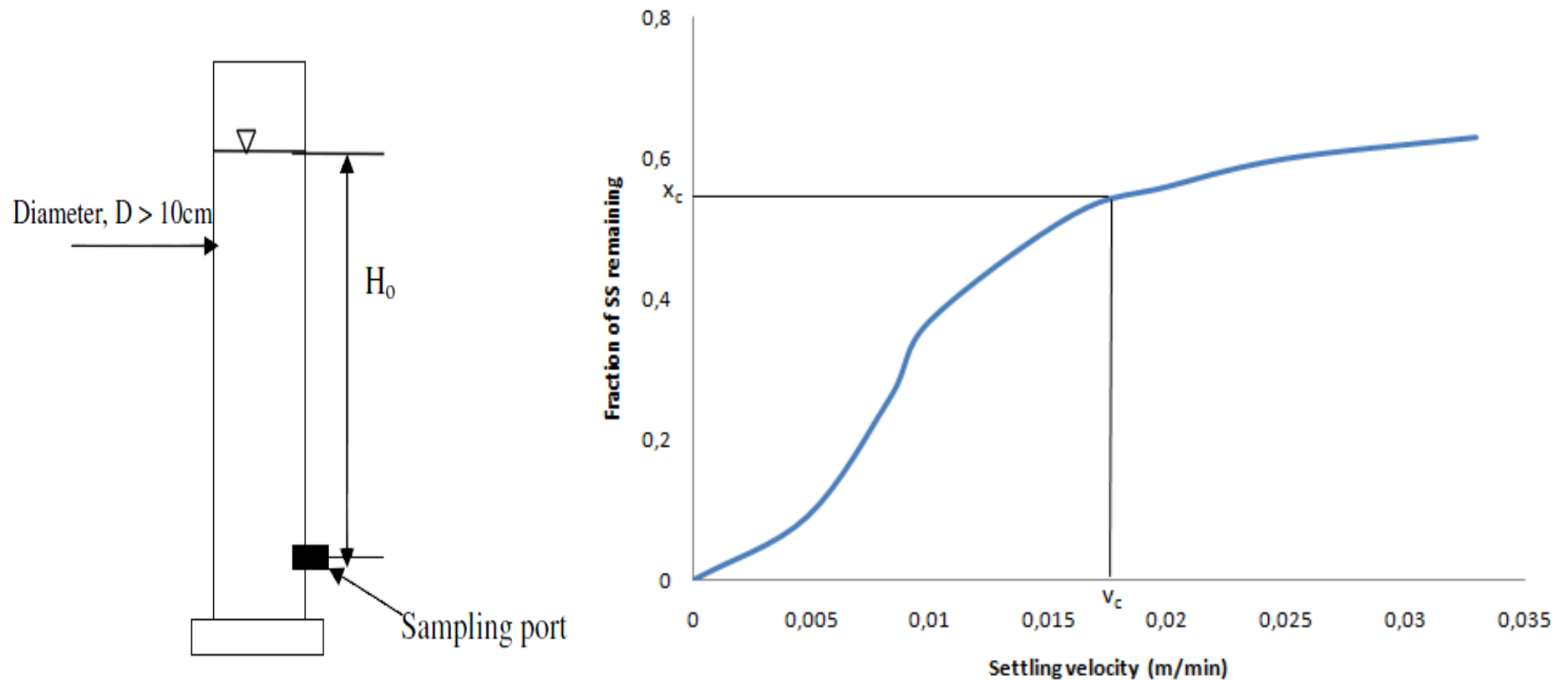
Ideal settling characteristics

- The particle that takes the longest time to remove is the one that enters at the top
- The flow-through velocity is v_f
- The design settling velocity is v_c (also called critical settling velocity)
- Particles with $v > v_c$ will be removed independent of entry position.
- Particles with $v < v_c$ will only be partially removed depending on their entry position.
- A particle entering at (a) that settles to (e) will not be removed.
- If the same particle entered at (b) it would settle to (f) and be removed:
- The fraction removed for a given settling velocity is then computed as:

$$r = \frac{bc}{ac} = \frac{v}{v_c}$$

- Total removal of particles can be calculated if the settling velocity distribution of the suspension is known. The settling velocity distribution can be obtained from sieve analysis or a settling column test.

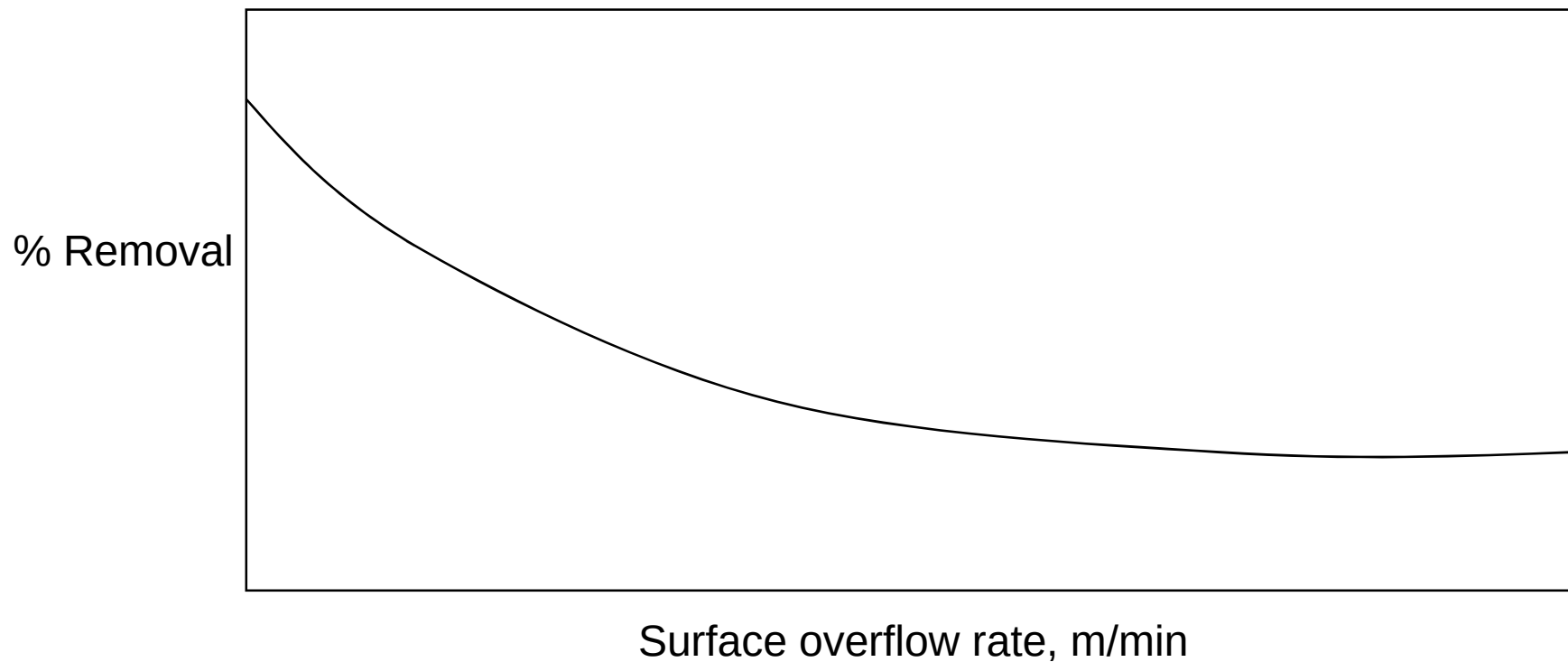
Discrete settling column test



$$\underbrace{R_{total}}_{\text{Total SS Removal}} = \underbrace{1 - x_c}_{R_{100\%}} + \underbrace{\frac{1}{v_c} \int_0^{x_c} v dx}_{R_{\text{partial}}}$$

Design curve

The design SOR can be estimated by multiplying the theoretical SOR (obtained from the column test) by 1.25-1.67

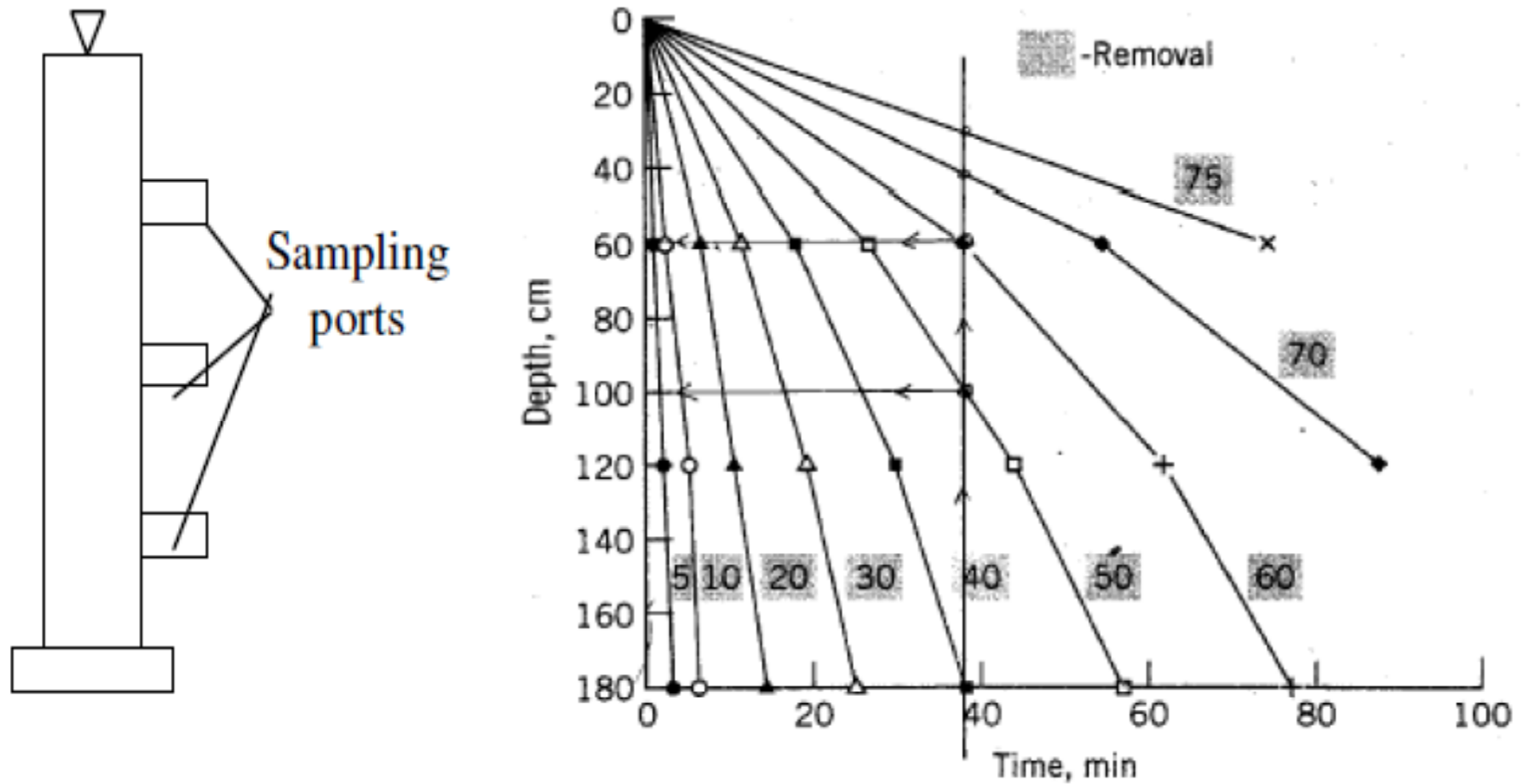


Example- Particle settling analysis

Calculate the theoretical suspended solids removal efficiency for a clarifier with a surface overflow rate of 25 m/d. A settling column analysis that was performed in a cylinder with sample port located 2 m below the water surface gave the following results.

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

Flocculent settling column analysis



$$R_{total} = r_o + \sum_i^n r_i \frac{h_i}{H_o}$$

Design of Presedimentation Basin

- Purpose: remove easily settleable sand and silt
- Location: upstream of any raw water pump facility and as close as possible to the intake structure
- Rectangular basin
 - with hopper bottoms
 - equipped with continuous sediment removal facility

Typical presedimentation tank design

Parameter	Value
Type	Horizontal flow, rectangular
Minimum number of tank	2
Depth (without automated sediment removal), m	3.5-5
Depth (with automated sediment removal), m	3-4
Minimum length-to-depth ratio	6:1
Length-to-width ratio	4:1-8:1
Surface overflow rate, m ³ /m ² .d	200-400
Horizontal mean flow velocity (at Q _{max} -day), m/s	0.05
Detention time, min	6-15
Minimum size of particle to be removed, mm	0.1
Bottom slope, m/m	Minimum 1:100 longitudinal

Assuming ideal design criteria, the length of the presedimentation basin can be estimated as follows

$$L = K \left(\frac{h_o}{v_c} \right) v_f$$

L: length, m

K: safety factor, 1.5-2

h_o : effective water depth

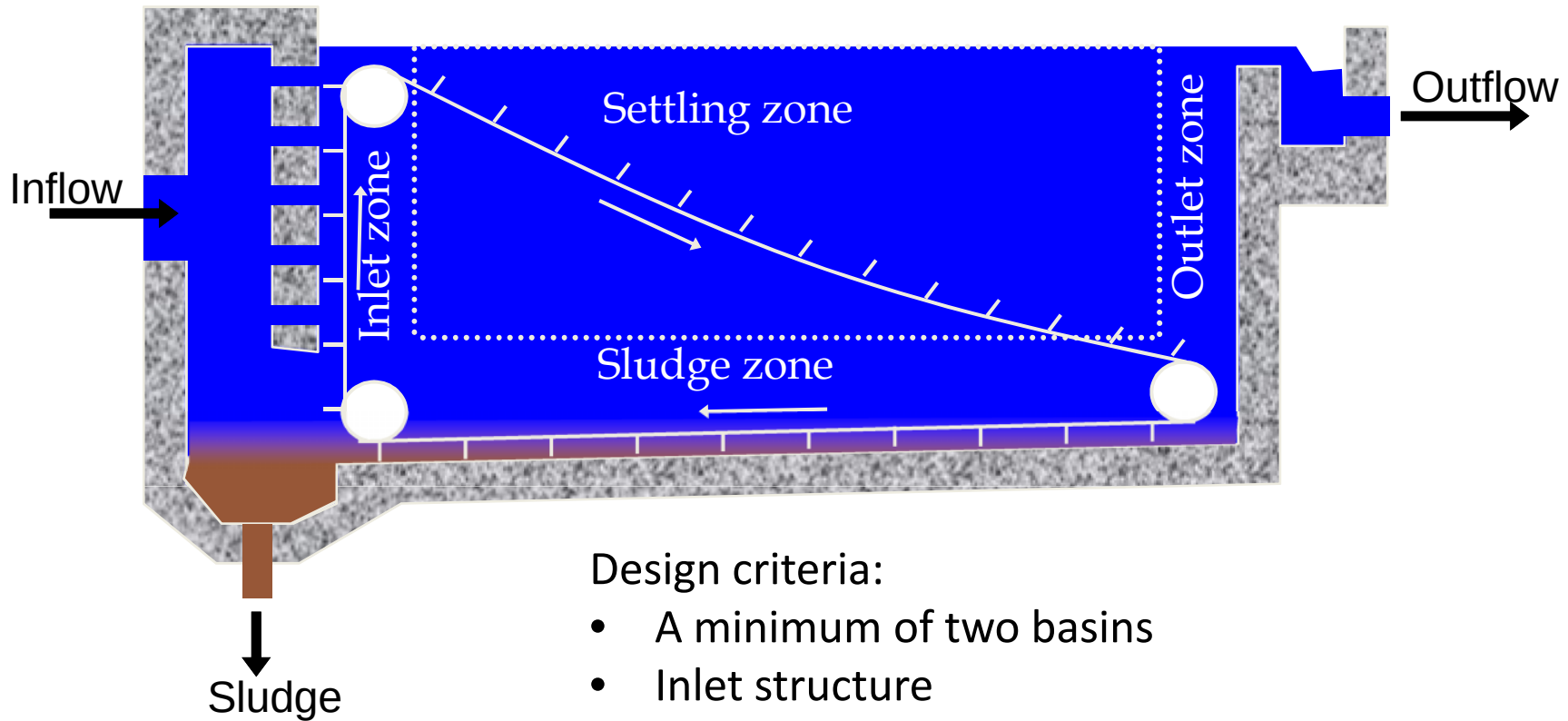
v_c : settling velocity, m/s

v_f : mean water velocity at average Q, m/s

Example- presedimentation tank design

A grit chamber (one concrete tank divided into two sections) is designed to remove sand of 0.08 mm and larger for a average flow of $1.1 \text{ m}^3/\text{s}$. the maximum flow rate is to be 1.6 times the average flow and the water temperature is 10°C assuming typical water depth of 3.5 m and a factor of safety of 1.5, determine the length of and width of each tank and check that the surface overflow rate and the detention time are within the recommended design criteria ranges. Use the typical design criteria and a settling velocity of 6 mm/s for 0.08 mm particle size.

Design of Rectangular Sedimentation Basin



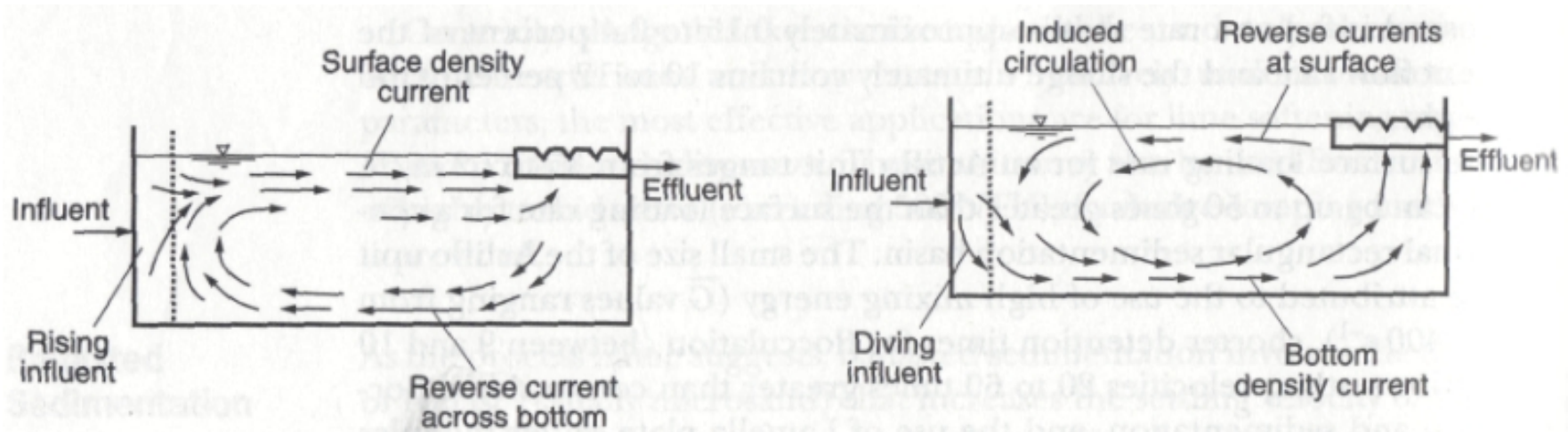
Design criteria:

- A minimum of two basins
- Inlet structure
- Settling zone:
 - Surface overflow rate
 - Effective water depth
 - Horizontal flow velocity
 - Minimum length-to-width ratio
- Outlet structure
- Sludge zone

Typical design criteria for horizontal-flow rectangular tanks

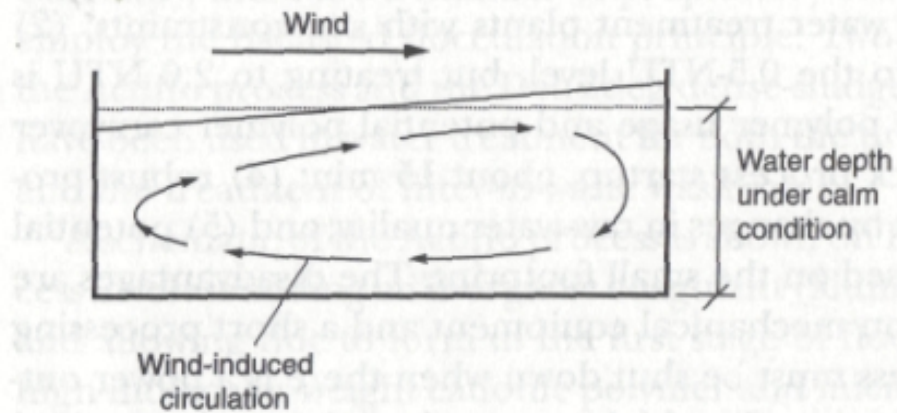
Parameter	Value
Type	Horizontal flow rectangular
Minimum number of tank	2
Water depth, m	3-5
Minimum length-to-depth ratio	15:1
Width-to-depth ratio	3:1-6:1
Minimum length-to-width ratio	4:1-5:1
Surface overflow rate, m/h	1.25-2.5
Horizontal mean flow velocity (at $Q_{\text{max-day}}$), m/min	0.3-1.1
Detention time, h	1.5-4
Launer weir loading, m ³ /m.h	9-13
Reynolds number	<20,000
Froude number	>10 ⁻⁵
Bottom slope for manual sludge removal systems, m/m	1:600
Bottom slope for mechanical sludge scraper equipment	1:300
Sludge collector speed for collection path, m/min	0.3-0.9
Sludge collector speed for the return path, m/min	1.5-3

Some physical factors that affect sedimentation



(a) Surface density currents

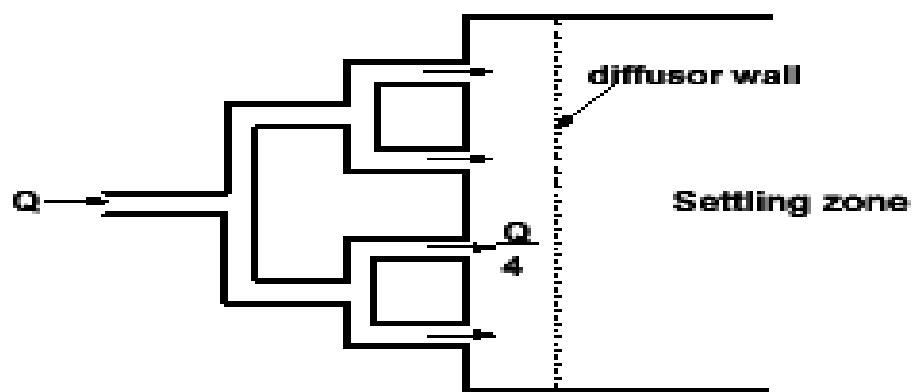
(b) Bottom density currents



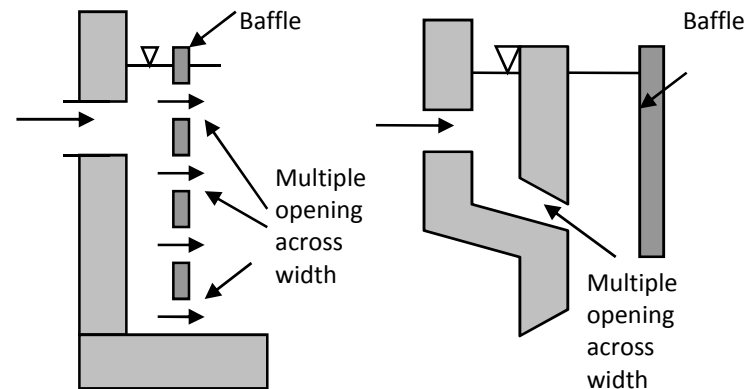
(c) Wind induced currents

Inlet structure

- *Inlets* should be designed to dissipate the momentum and accurately distribute the incoming flow in such a way as to establish the required flow pattern in the tank.
- For an even distribution of the water over the width and depth of the tank at low velocity, several inlet channels followed by a diffuser wall (baffle wall) may be provided.



Plan view



Vertical sections

Inflow velocity: 0.15-0.60 m/s

Baffle openings: 100-200 mm diameter

Settling zone

- Depth of tank should give adequate space for mechanical sludge removal equipment and sludge deposit and minimize scouring effect of flow velocity and wind speed.
- Long, narrow and deep basins are preferred to minimize short-circuiting. A freeboard of tank 0.5 m should be provided to act as a wind barrier
- Ensure minimum turbulence ($Re < 20,000$) and no backmixing ($Fr > 10^{-5}$).

$$Re = \frac{v_f R_h}{\nu}$$

$$Fr = \frac{v_f^2}{g R_h}$$

Re = Reynolds number

V_f = average horizontal fluid velocity, m/s;

R_h = hydraulic radius, m

ν = kinematic viscosity, m^2/s

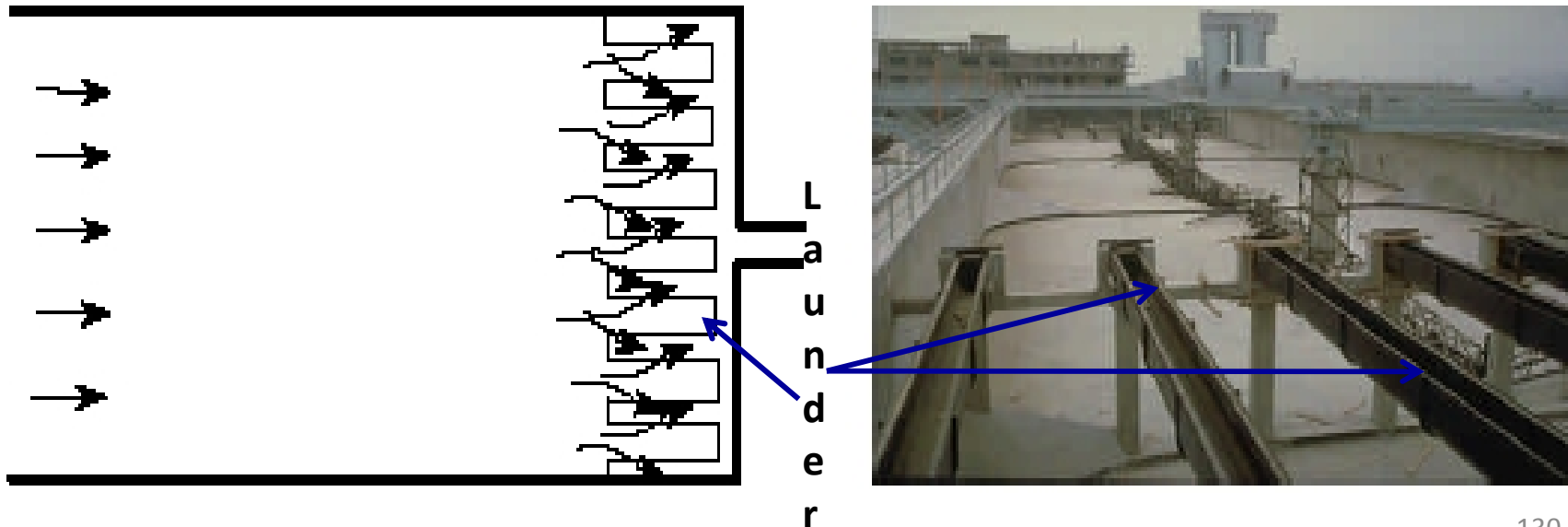
Fr = Froude number

g = gravitational acceleration, m^2/s

Longitudinal baffles placed parallel to the flow direction reduces the Reynolds number and increases the Froude number

Outlet structure

- Located at the end of the settling tank
- Generally consist of an overflow weir and a launder.
- re-suspension of settled solids must be prevented and the flow velocity in upward direction thus be limited.
- The length of the overflow weir must be several times the width of the tank.

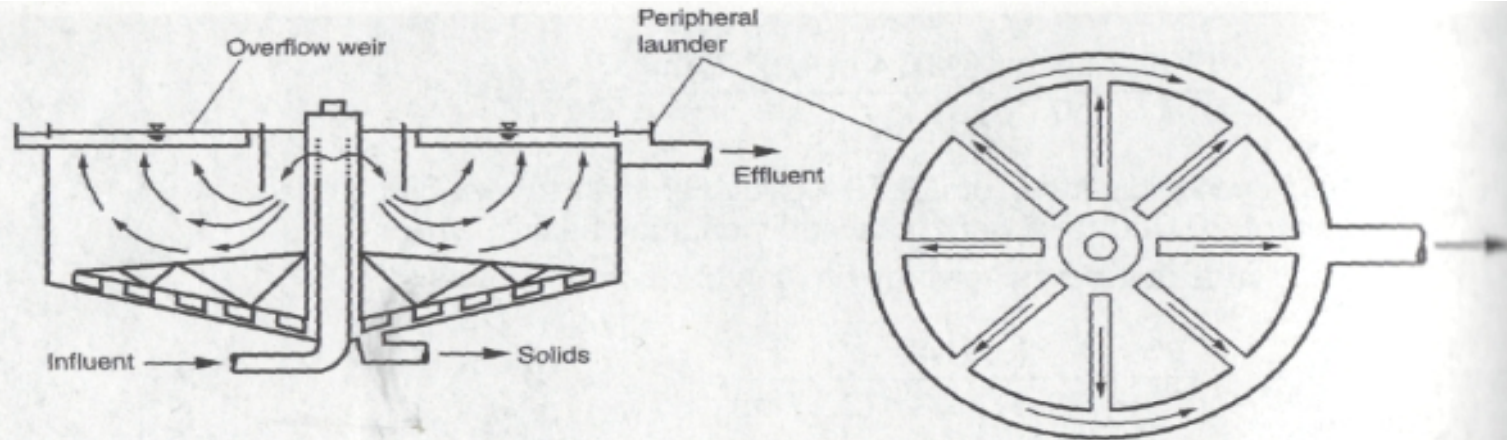


Sludge zone

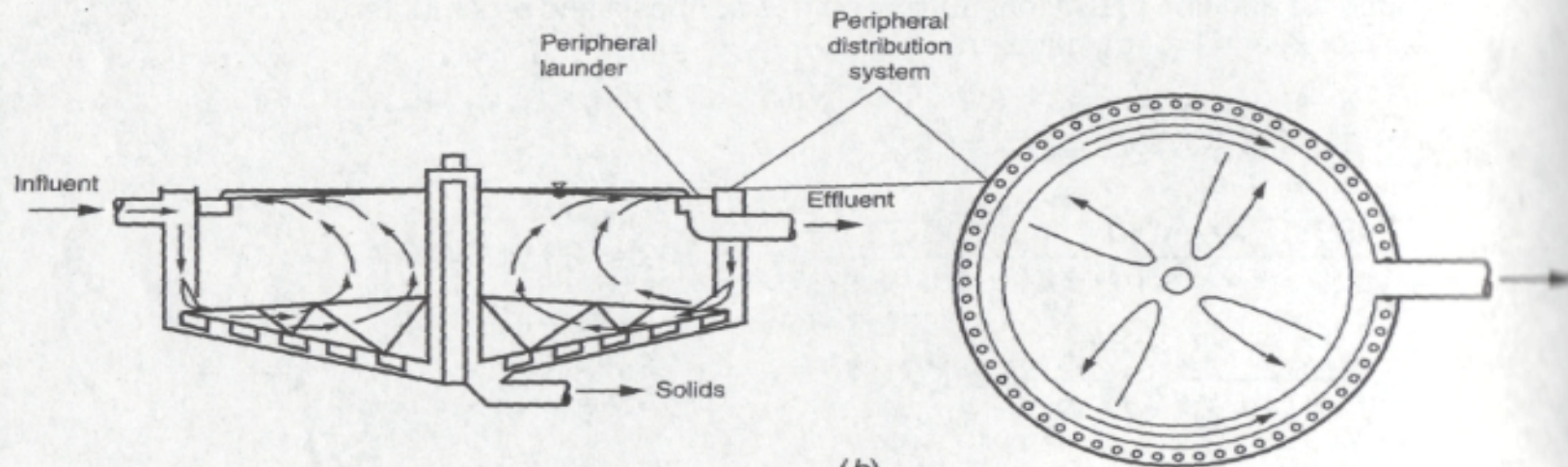
- The removal of the sludge can be done hydraulically and mechanically.
- Hydraulic sludge removal is done at regular intervals by dewatering the tank and flushing the sludge with pressured water (from hydrants) to a hopper at the bottom of the tank.
- Mechanical sludge removal is frequently applied when sludge volumes are large or the sludge is unstable, resulting in anaerobic decomposition during storage in the sludge zone.
 - Chain-and-flight type
 - Traveling-bridge type
 - Circular type

Circular Sedimentation Basins

- Enable use of trouble-free circular sludge removal mechanism
- Require a lower capital cost per unit surface area
- Need more piping for water and sludge conveyance



(a) Center-feed

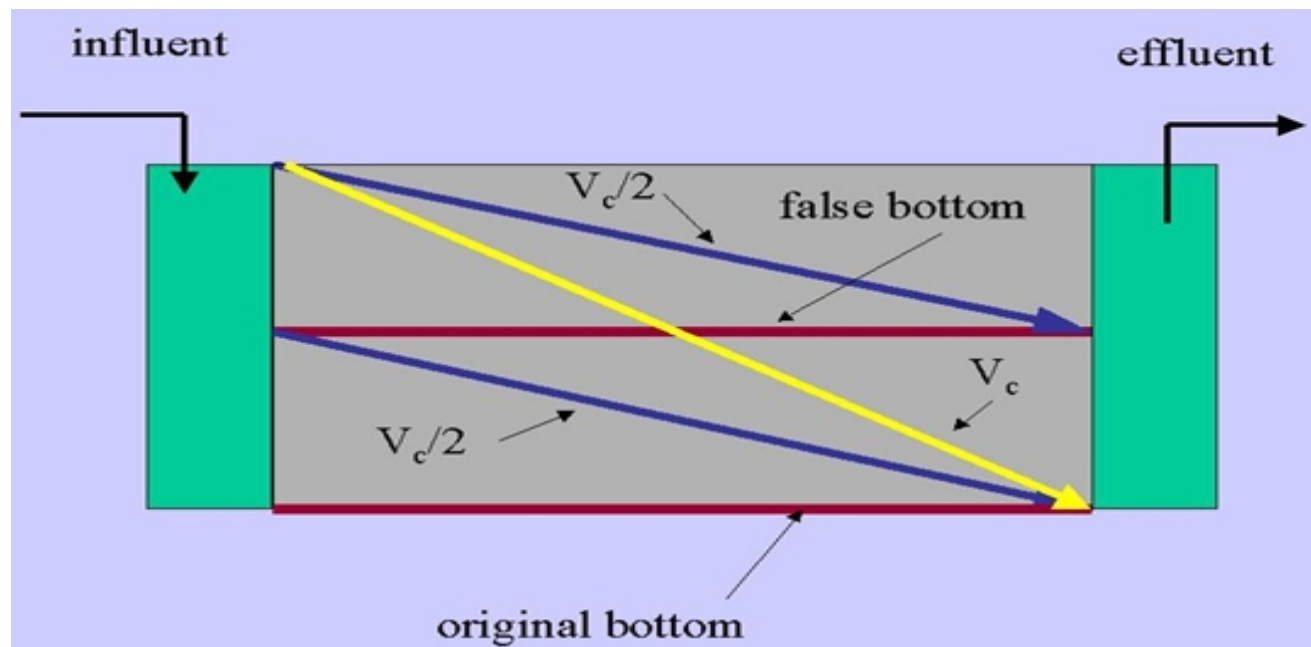


(b) Peripheral -feed

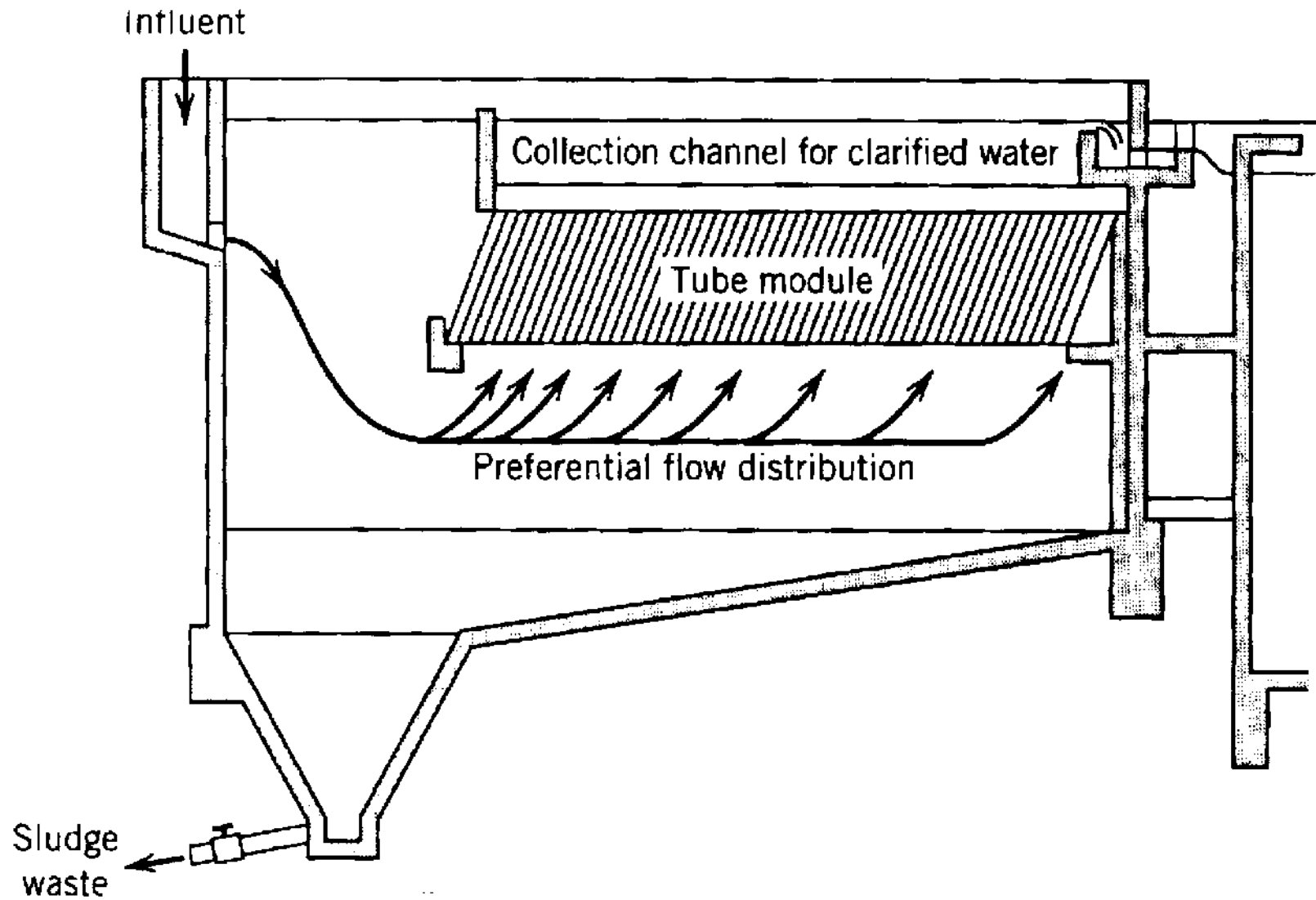
Enhanced Sedimentation

One method to increase the efficiency or increase the capacity of clarifiers is to install "false bottoms" in the clarifiers. Commonly used devices are tube or plate settlers inclined at 45° to 60° angles from horizontal. The enhanced sediment removal is due to:

- Reduced settling distance
- Existence of laminar flow
- Absence of density and temperature currents and other wave actions



Tube settlers



Tube settlers



Design values for tube/plate settlers

Parameter	Value
Inclined length, m	1-2
Angle of inclination from horizontal	7-60°
Tube diameter or plate spacing, cm	near 5
SOR based on plan area, $\text{m}^3/\text{m}^2/\text{d}$	88-78
Depth, m	6-7

Assignments

Assignment-3

Problems on Coagulation and Flocculation and Sedimentation (Separate word file)

Assignment-4

Reading assignment on plate and tube settlers

3.2 Dissolved Air Flotation

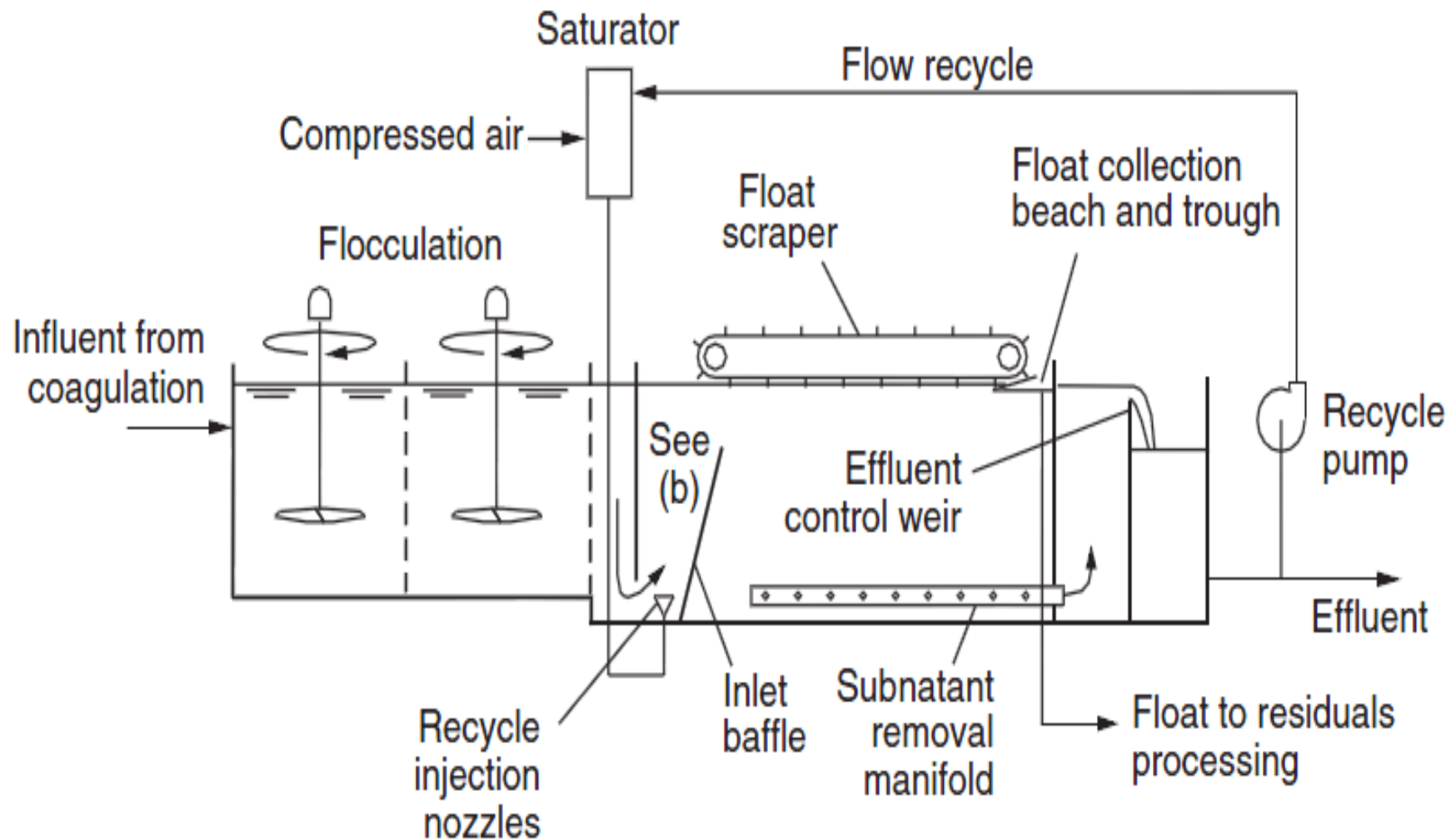
- Dates back to 1920s; introduced in USA the early 1980s
- Placed after coagulation and flocculation units
- Involves the use of fine air bubbles for the separation of solid and semisolid (floc) particles from a liquid.
- Requires introduction of fine air bubbles near the bottom of the basin containing the water to be treated
- As the bubbles move upward through the water, they become attached to particulate matter and floc particles, and the buoyant force of the combined particle and air bubbles causes the aggregated particles to rise to the surface.

Applications

Used to remove:

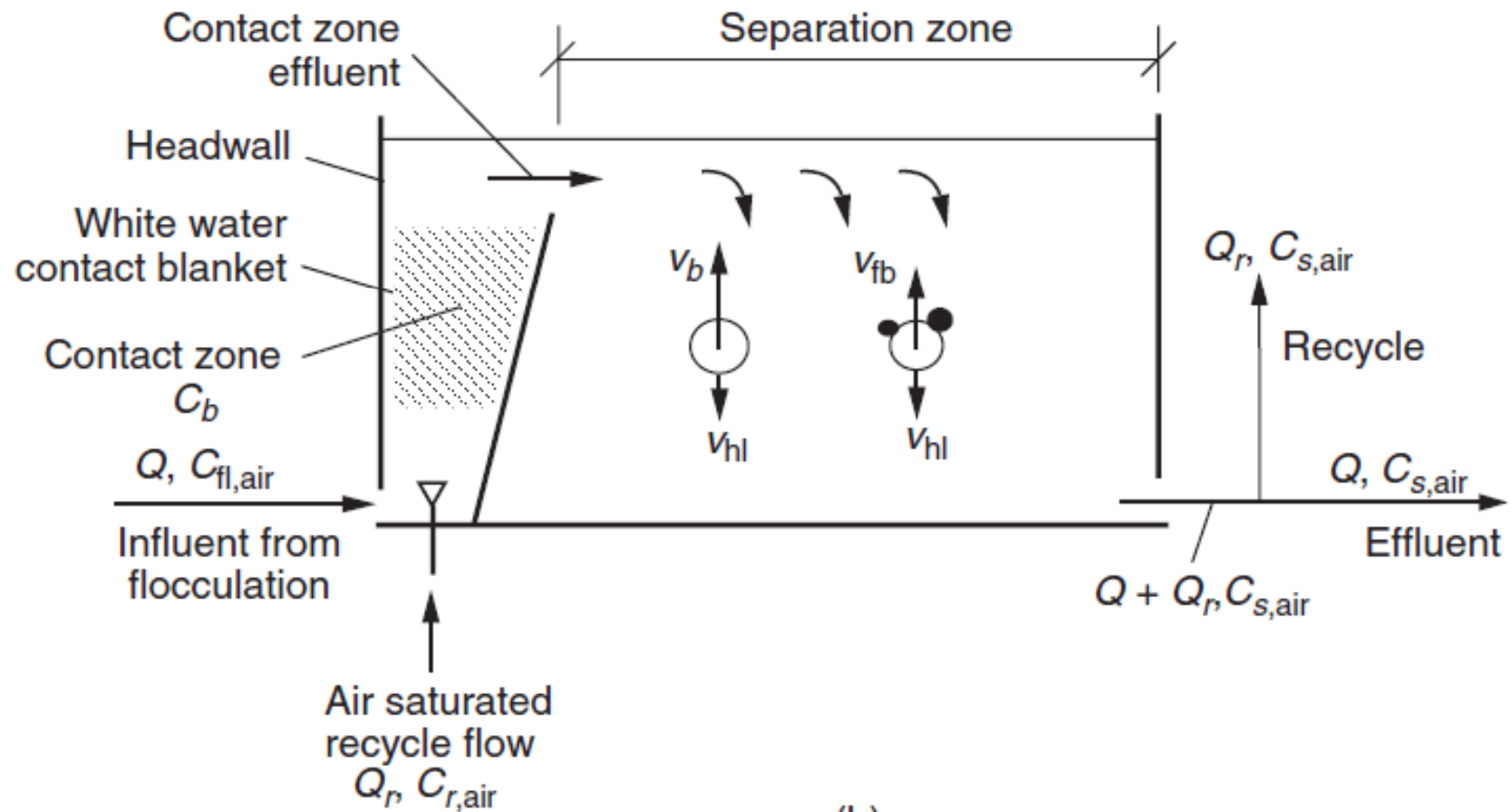
- Low-density particulate matter such as algae
- Dissolved organic matter (natural color)
- Particulate matter from low- to moderate-turbidity waters
- Algae, dissolved organic matter, and turbidity from low-temperature waters
- Suspended material from filter waste washwater

DAF Process Schematic



(a)

DAF Process Schematic



(b)

Advantages and Disadvantages of DAF

Advantages

- High loading rate: Typically 10–20 m/h. New process variants have operated successfully up to 40–45 m/h.
- Very thick float (sludge) product: Typically 2–3% total solids float can be achieved using hydraulic or mechanical skimming devices. Float can be dewatered without intermediate thickening.
- Often, no polymer is required, as DAF does not require a large, dense floc. Coagulant dosages may also be reduced in some circumstances.
- Shorter flocculation times, as compared to gravity separation, are possible, because a smaller floc particle size is required.
- Rapid startup, typically <30–60 min to reach steady state, depending on size.
- Excellent algae removal efficiencies.
- Excellent *Giardia* and *Cryptosporidium* removal efficiencies (~2–2.5 log), depending on temperature.
- Smaller footprint required as compared to conventional flocculation and gravity sedimentation

Disadvantages

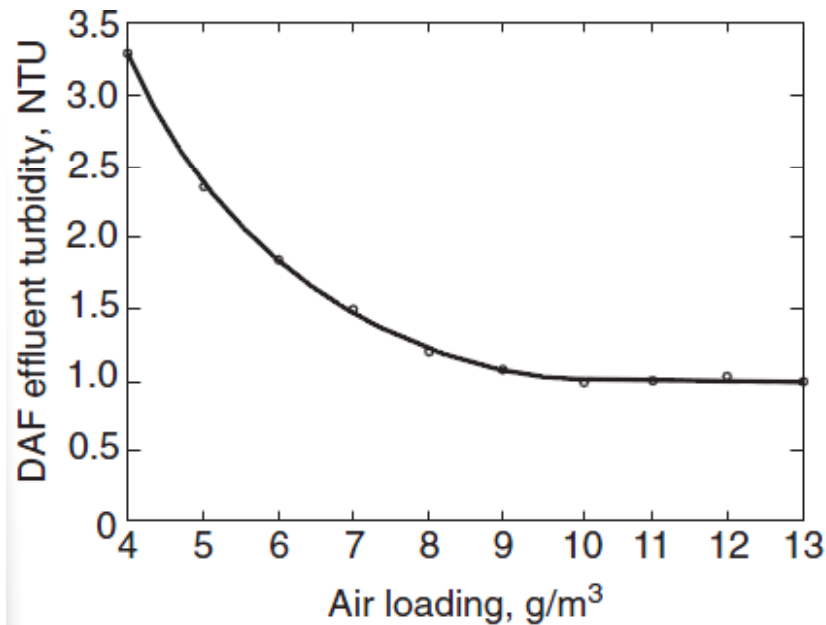
- Requires a cover or housing to protect the float layer from wind and precipitation.
- Mechanically more complex than conventional gravity clarifiers.
- More power intensive as compared to conventional flocculation and sedimentation (2.5–3 to 0.75–1 kWh/ $10^3 \text{ m}^3 \cdot \text{d}$).
- Generally not well suited for clarification of high-turbidity silt-laden waters.
- Because DAF is more mechanically intensive, may not be suitable for locations where equipment maintenance is likely to be neglected.

Factors that affect DAF performance

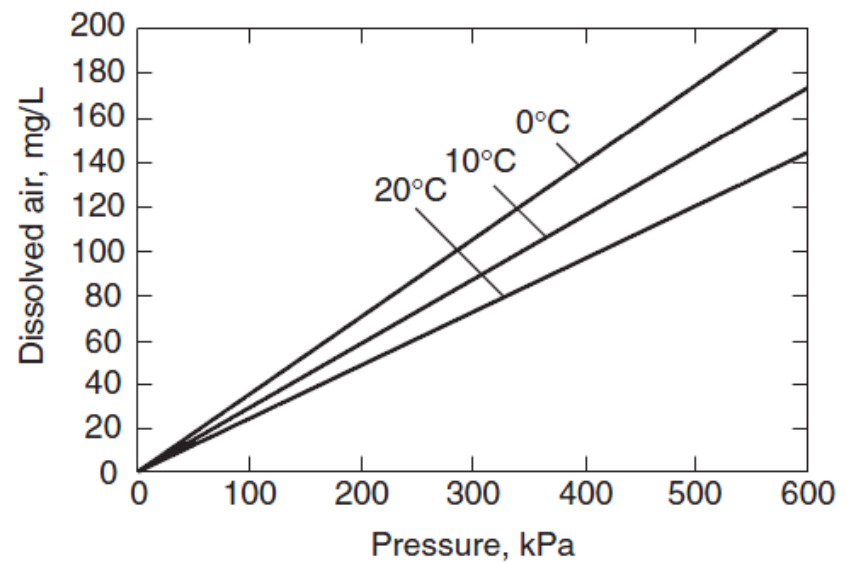
- Proper coagulation
 - Should produce destabilized flocs
- Floc characteristics
 - Small and low-density floc is preferred
- Bubble size and rise velocity
 - 10-100 μm floc bubble size
 - Laminar flow regime
- Air loading
 - Ensure dense bubble cloud
- Floc-bubble attachment
 - Important to know the mechanisms for floc-bubble attachment
- Solubility of gases
 - Ensure sufficient air delivery

Factors that affect DAF....

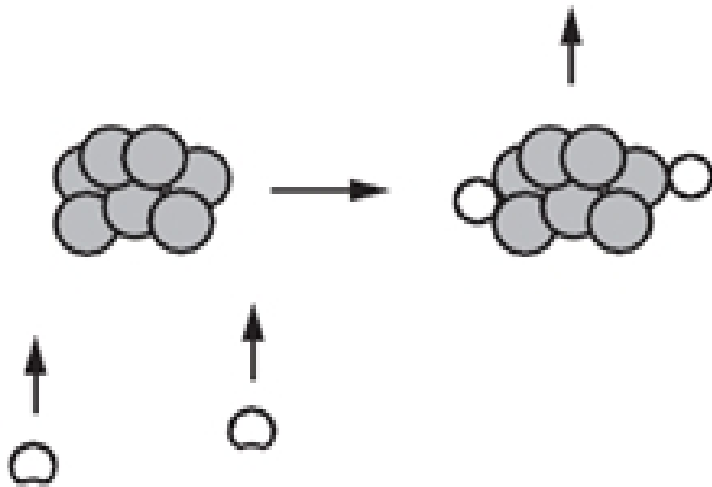
Effect of air loading on turbidity



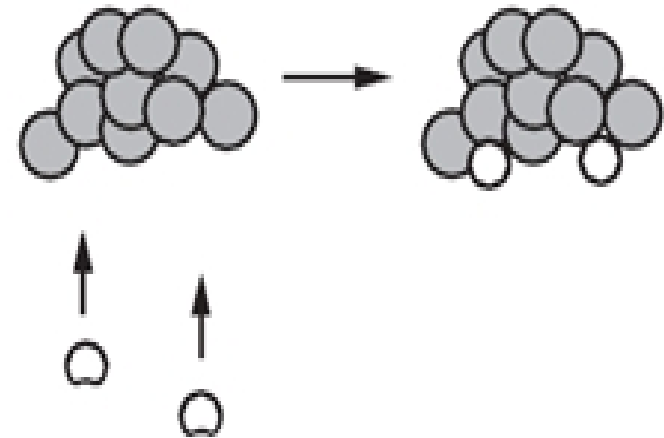
Solubility of air



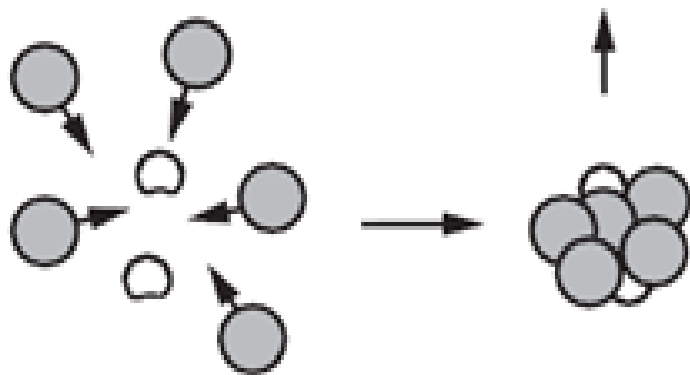
Possible mechanisms for floc–bubble attachment



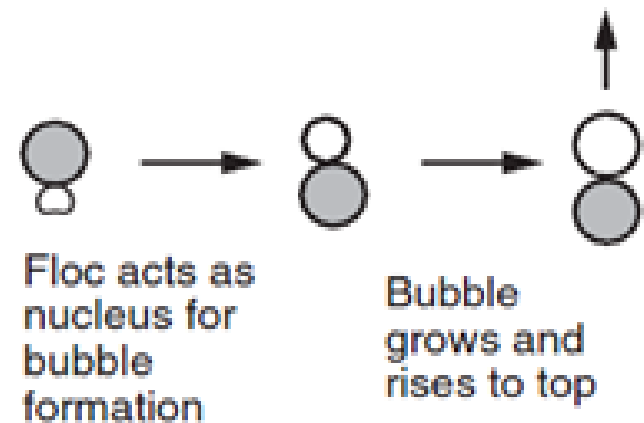
Bubbles adhere to preformed floc



Bubbles trapped in preformed floc



Bubbles trapped as floc forms



Floc acts as nucleus for bubble format

Design considerations

- Basin layout and geometry
- Recycle systems
- Subnatant removal systems
- Float removal systems.

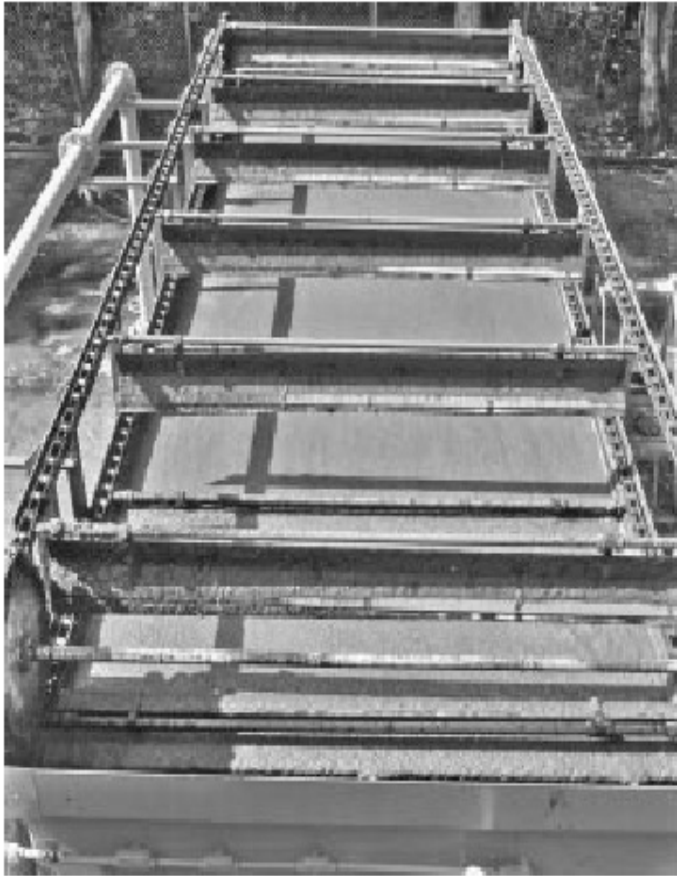
Typical design criteria for DAF

Design Parameter	Unit	Value
Flocculation Process		
Number of basins	Minimum number	2
Number of stages	Number	2
Water depth	m	3.5–4.5
Basin length-to-width ratio	Ratio	1
Detention time, large plants	min	10–15 ^a
Detention time, small package plants	min	20
Energy input range, G	s ⁻¹	50–100
Basin Design for Rectangular Configuration		
Number of basins	Minimum number	2
Surface hydraulic loading rate, conventional, based on separation zone	m/h	10–20
Surface hydraulic loading rate, high rate, based on separation zone	m/h	40–45
Basin length, from headwall to float beach (separation zone)	m	<11
Basin length-to-width ratio	—	1–1.25
Surface area	m ²	90–110
Maximum hydraulic capacity for single basin	m ³ /s	0.25–0.5
Basin cross-flow velocity	m/h	18–100
Basin depth	m	2.5–3
Contact zone detention time	s	60–240
Contact zone hydraulic loading rate	m/h	35–100
Baffle clearance velocity	m/h	55
Contact zone baffle angle	deg to horizontal	60–90
Recycle System		
Recycle ratio	% of influent flow	6–12
Recycle system pressure	kPa (gauge)	400–700
Nozzle spacing	m	0.2–0.3
Saturator hydraulic loading rate	m/h	60–80
Saturator packing depth	m	1.0–1.5
Injection nozzles	Fixed-orifice nozzles recommended	
Air loading	g air/m ³ raw water	6–10
Air bubble size	μm	10–100
Bubble concentration	bubbles/mL	1.0–2.0 × 10 ⁵
Raw-water bubble number–particle number ratio	Dimensionless	10:1–200:1
Bubble volume concentration	ppm	3500–8000

Typical design criteria....

Design Parameter	Unit	Value
Flocculated water bubble–volume to particle volume ratio	Dimensionless	350:1–8000:1
Recycle pump	Centrifugal	Provides one stand-by pump per recycle system
Air compressor	Oil-lubricated rotary screw type, with good posttreatment for particulate and oil removal; oil-free compressors not recommended due to maintenance requirements	
Float Removal		
System type	Mechanical or hydraulic. Choice of type is site specific, but should include cost implications on residuals-handling systems. For mechanical skimming, reciprocating scrapers are recommended.	
Subnatant Removal		
Type of removal system	Perforated pipe laterals or underflow end wall. Special underdrain systems consisting of a false bottom and plenum	

Typical dissolved air flotation units



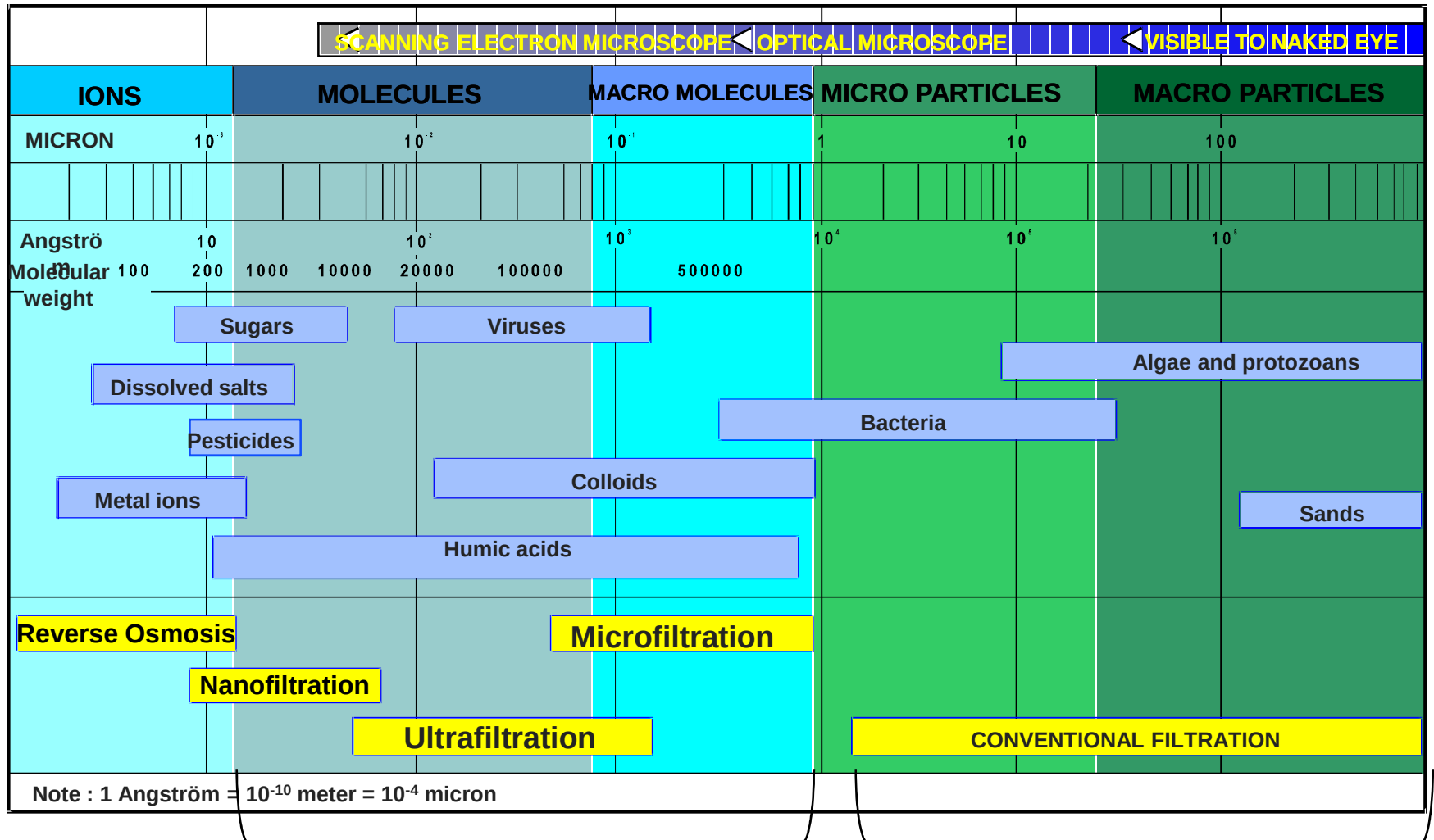
4. FILTRATION

Definition

Filtration: Removal of colloidal (usually destabilized) and suspended material from water by passage through layers of porous media.



Filtration Spectrum



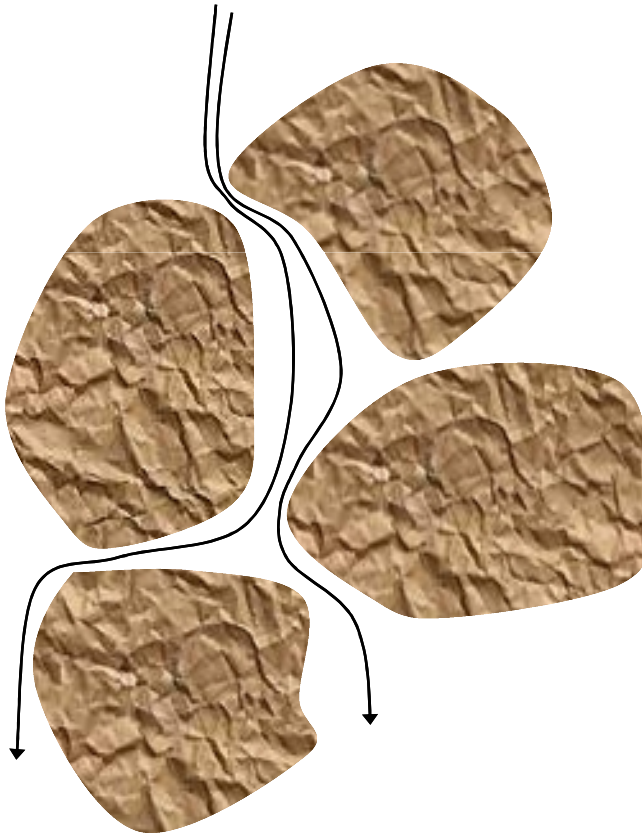
Membrane filtration

Granular filtration

4.1 Granular Filtration

- **Based on filter media**
 - Slow sand filtration
 - Rapid filtration
 - High-rate filters
- **Based on driving force**
 - Gravity filters
 - Pressure filters
- **Based on flow direction**
 - Downflow filters
 - Upflow filters

Particle Removal Mechanisms in Filters



Transport

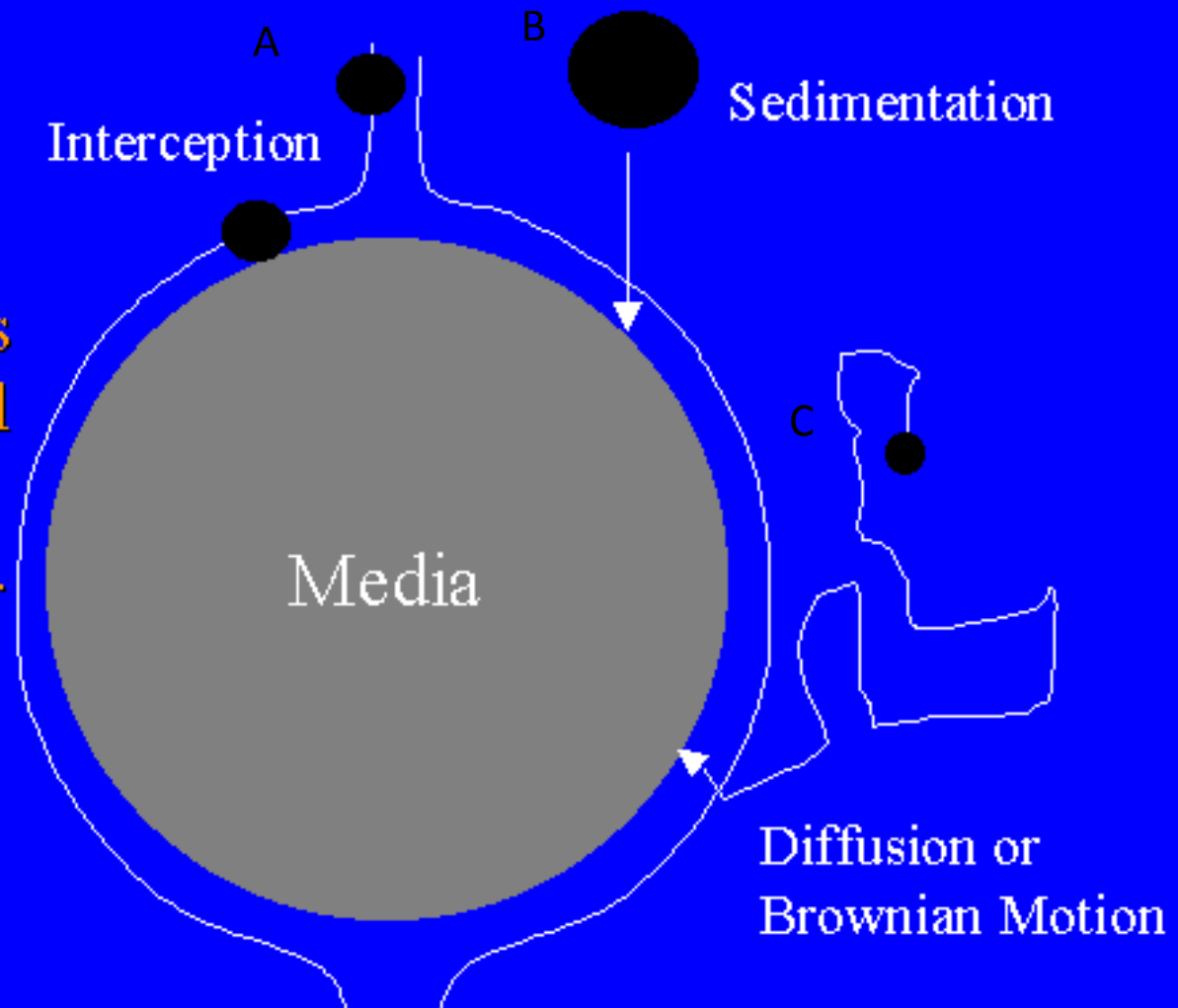
Molecular diffusion
Inertia
Sedimentation
Interception

Attachment

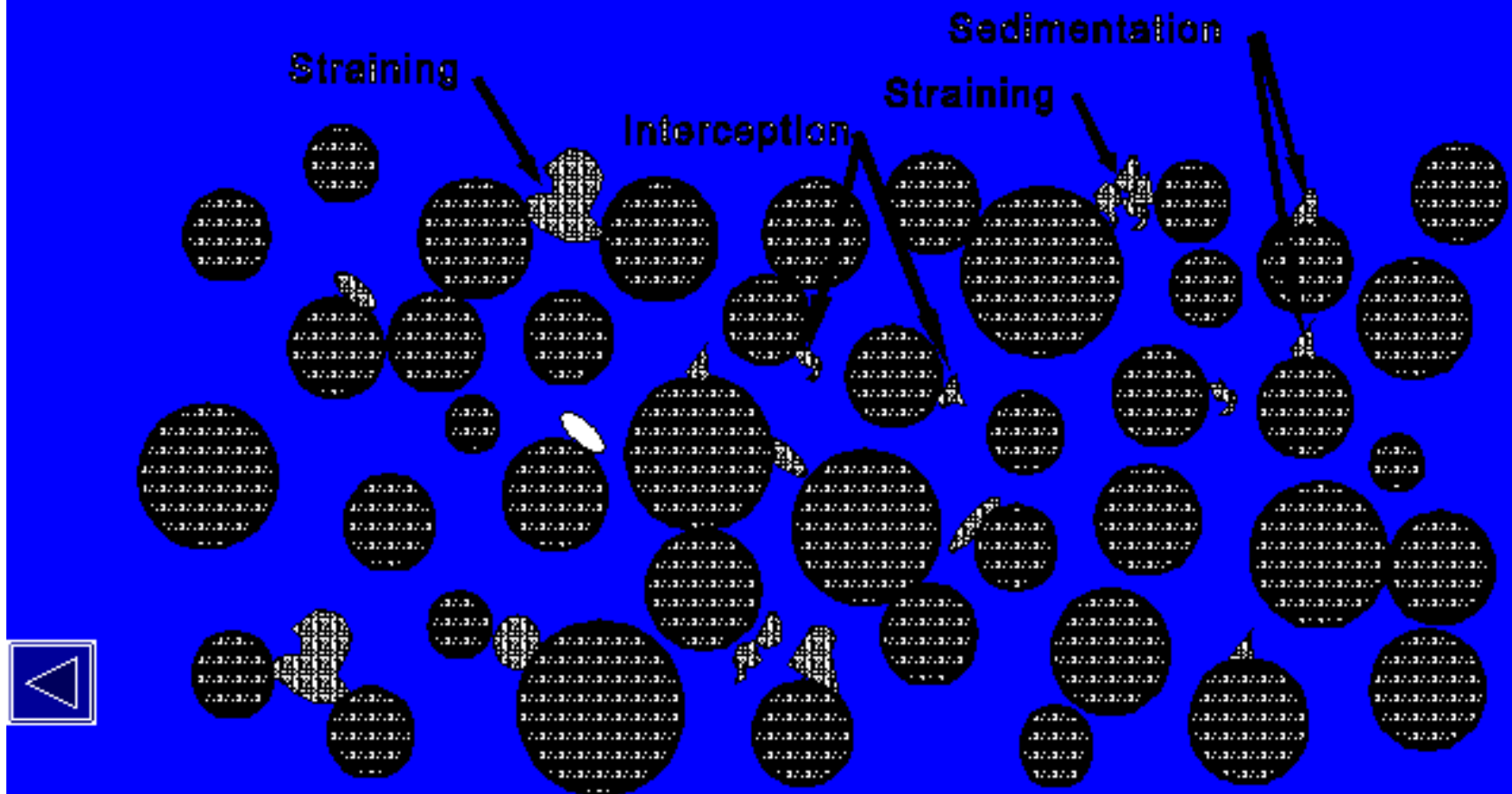
Straining
Surface forces

Filtration transport mechanisms

In addition, particles must be able to stick. This requires chemical destabilization (i.e. coagulation).



Deposition in a Filter



Filter media size parameters

- Effective size (d_{10}): the size of standard sieve opening that will pass 10% by weight of the media
- Uniformity coefficient (UC): the ratio of the standard sieve opening that will pass 60% by weight of the media (d_{60}) to its effective size.

$$UC = \frac{d_{60}}{d_{10}}$$

Comparison of grain size distributions of sand

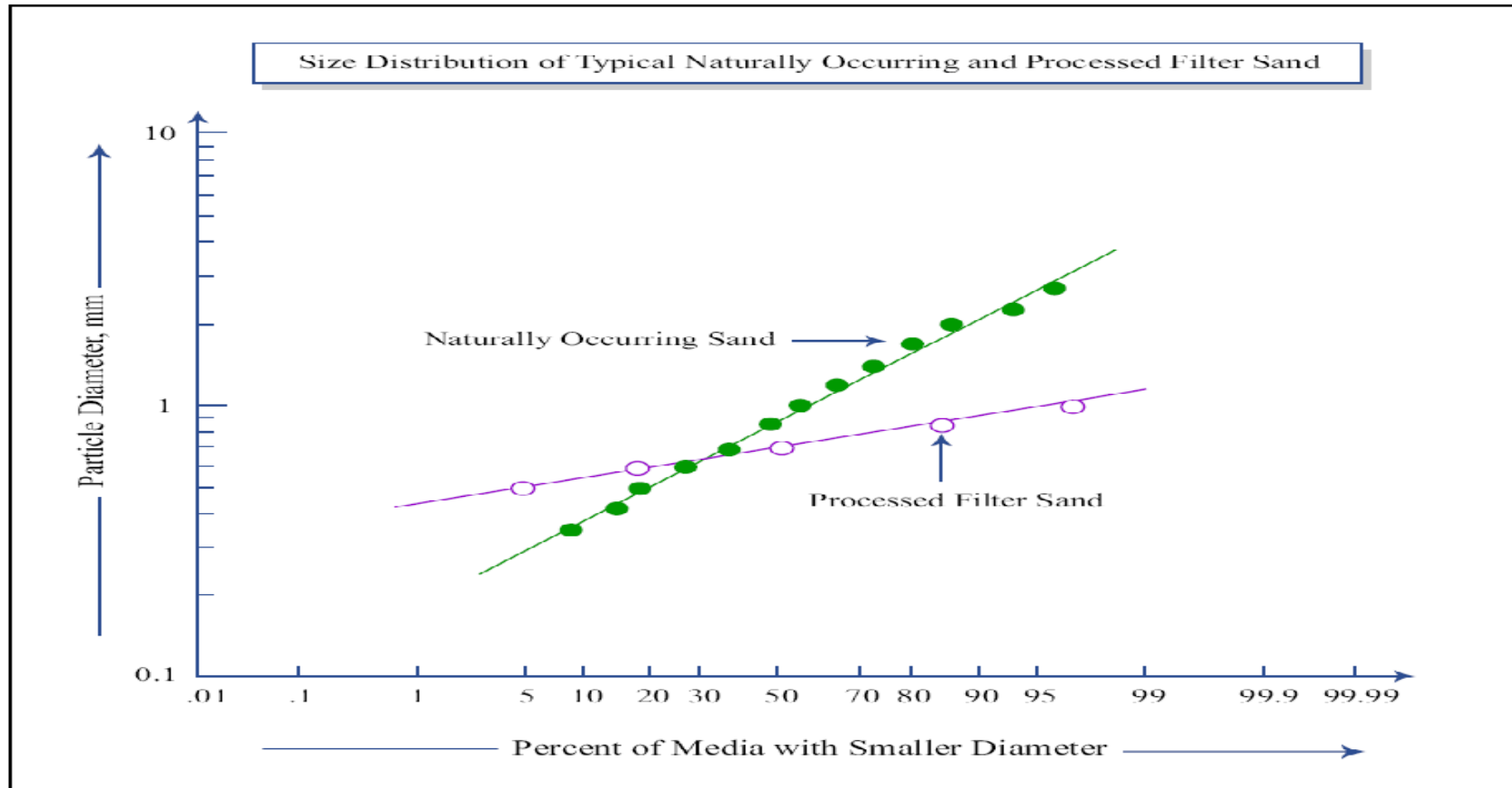
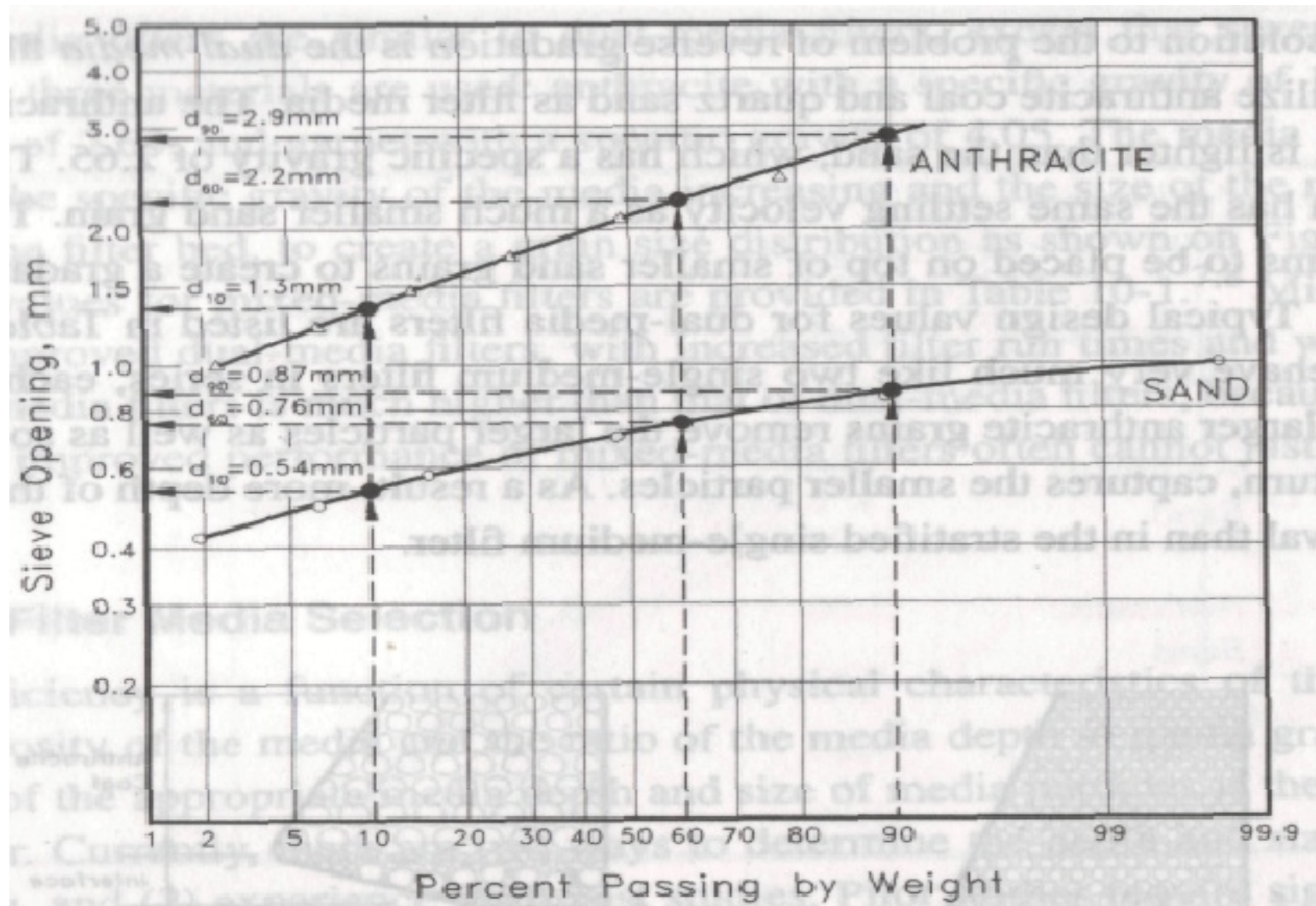


Figure by MIT OCW.

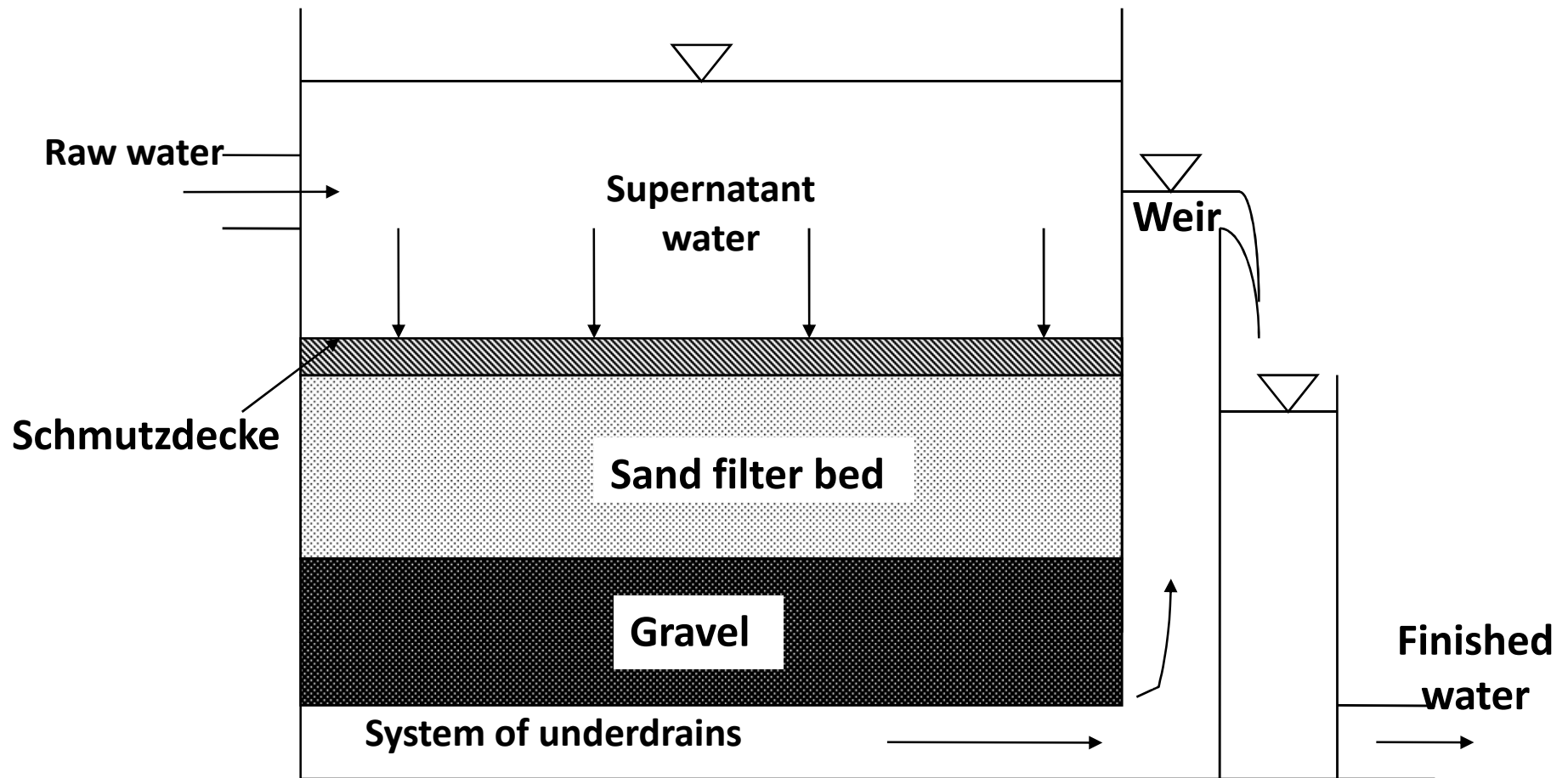
Adapted from: MWH, J. C. Crittenden, R. R. Trussell, D. W. Hand, K. J. Howe, and G. Tchobanoglous. *Water Treatment: Principles and Design*. 2nd ed. Hoboken, NJ: John Wiley & Sons, 2005, p. 881.

Typical sieve analysis results

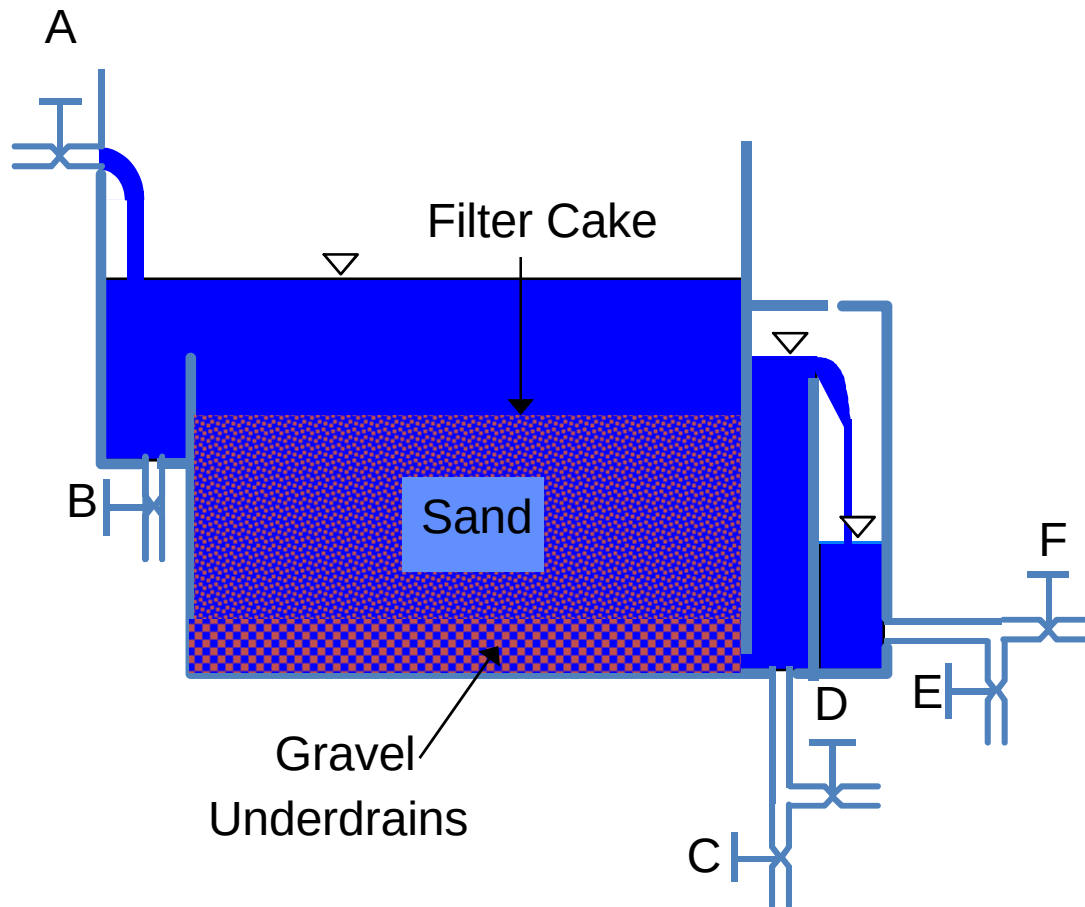


Slow Sand Filter

Slow sand filter consists of concrete/brick work rectangular basin containing carefully selected graded sand supported on gravel and stones.



Slow Sand Filter Schematic



- A. Valve for raw water inlet and regulation of filtration rate
- B. Valve for draining unfiltered water
- C. Valve for back-filling the filter bed with clean water
- D. Valve for draining filter bed and outlet chamber
- E. Valve for delivering treated water to waste
- F. Valve for delivering treated water to the clear-water reservoir

Mechanisms of impurities removal in SSF

- **Physical:** Mechanical straining/sedimentation
- **Chemical:** Oxidation of organic matter by aerobic bacteria
- **Biological:** Occurs through *Schmutzdecke* or “Vital layer”. *Schmutzdecke* is a layer of dirt, debris, and microorganisms build up on the top of the sand

Slow Sand Filter Cleaning

- Periodic raking and cleaning of the filter by removing the top two inches of sand. After a few cleanings, new sand must be added to replace the removed sand.
- After a cleaning the filter must be operated for two weeks, with the filtered water sent to waste, to allow the schmutzdecke layer to rebuild.
- Two slow sand filters should be provided for continuous operation.



Advantages and Disadvantages of SSF

- Advantages:
 - Simple to construct and operate
 - Cost of construction cheaper than rapid sand filter
 - Do not usually require coagulation/flocculation before filtration
 - Bacterial count reduction is 99.9% to 99.99% and E.coli reduction is 99% to 99.9%
- Disadvantages:
 - Initial cost is low but maintenance cost is much more than rapid sand filter
 - These filters need a lot of space

Design criteria for slow sand filters

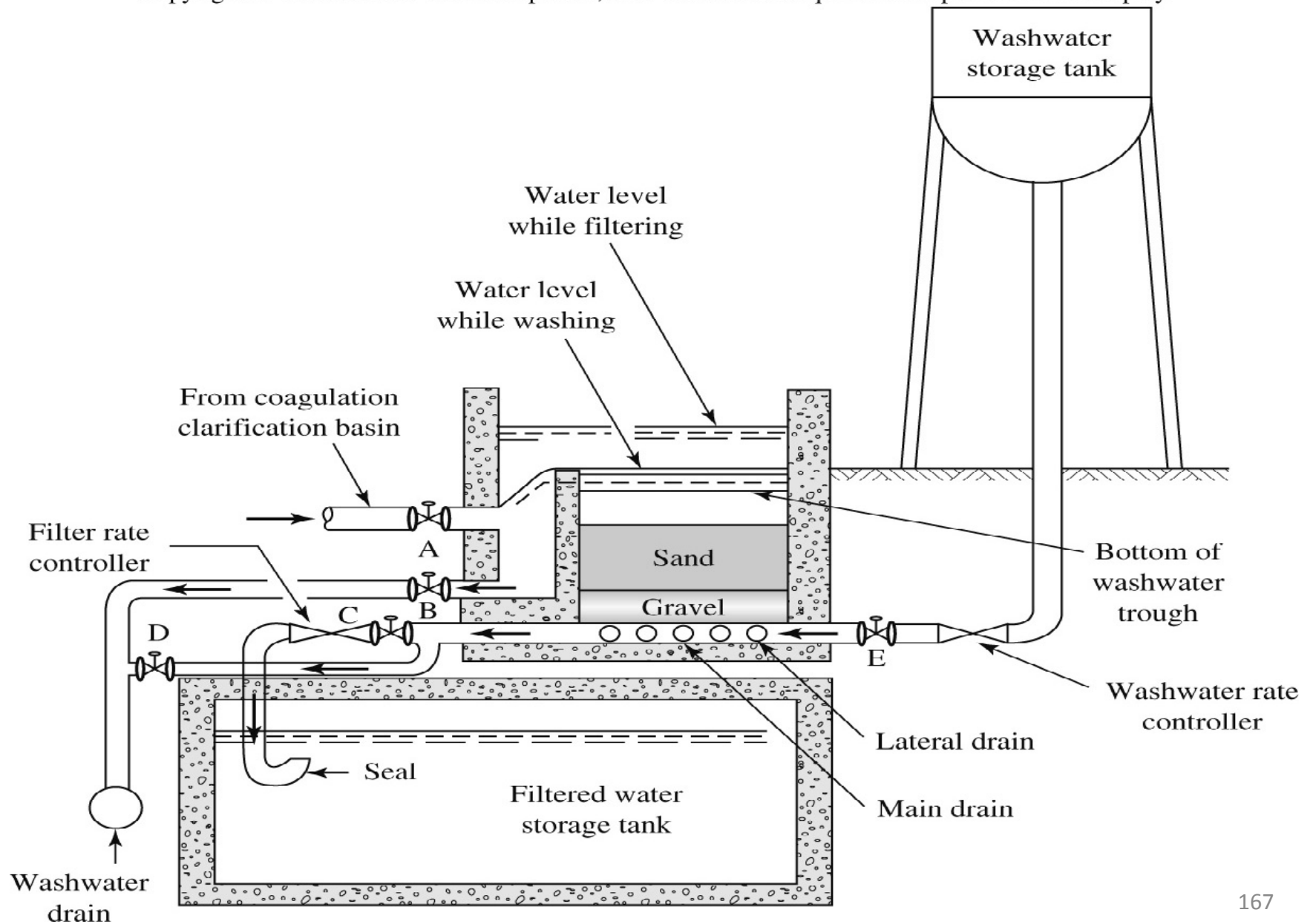
Parameter	Recommended level
Design life	10-15 year
Period of operation	24 h/day
Filtration rate	0.1 – 0.2 m/h
Filter bed area	5-200 m ² /filter
Height of filter bed	
Initial	0.8-0.9 m
Minimum	0.5-0.6 m
Effective size	0.15-0.3 mm
Uniformity coefficient	< 3
Height of under drains including gravel layer	0.3-0.5 m
Height of supernatant water	1 m

Rapid Filter

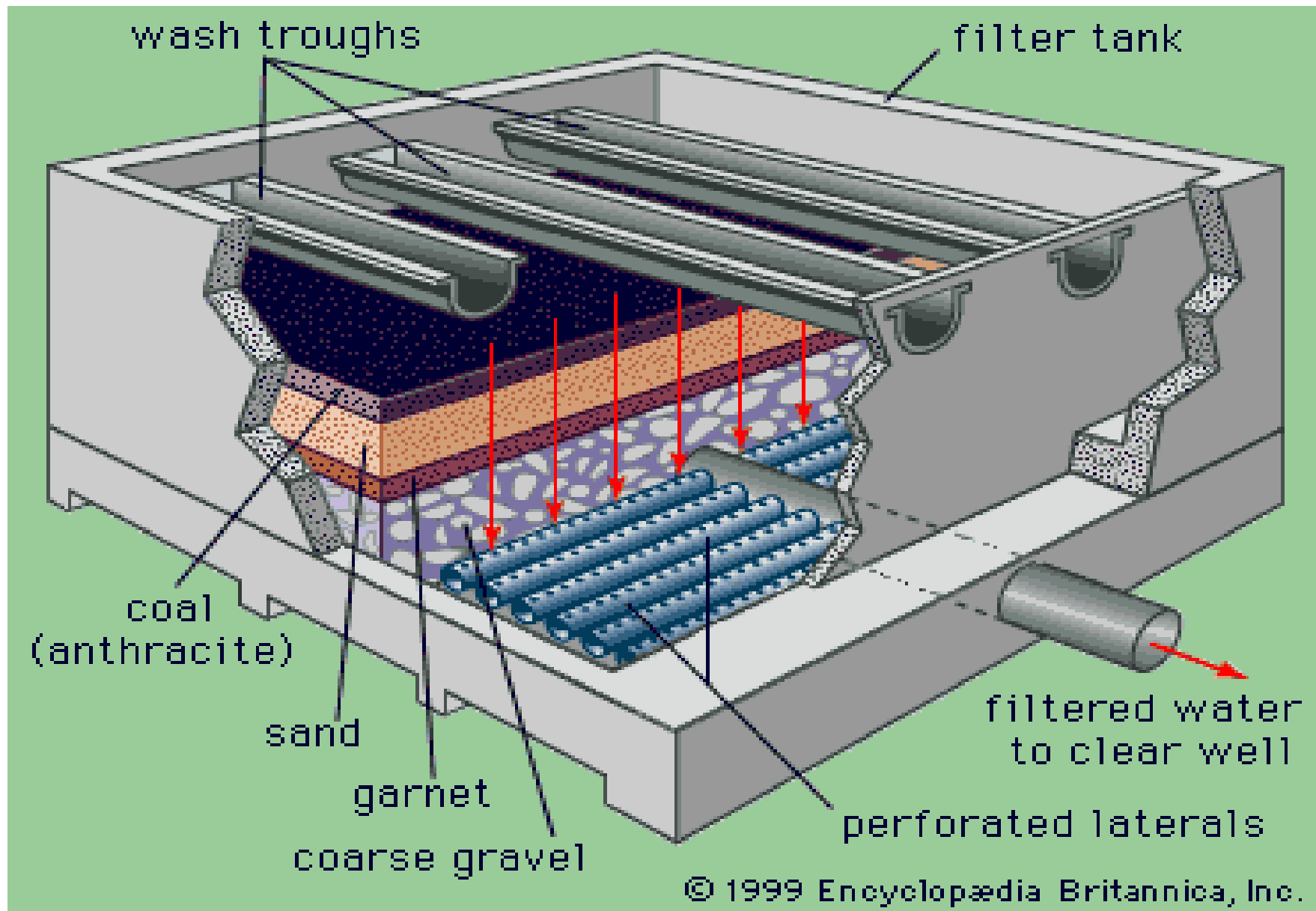
- Filtration rate is greater
- Backwashing is incorporated
- Straining is not important removal mechanism
- Particles adhere to media grains and are removed
- Each grain is a collector
- Water must be pre-treated to destabilize negatively charged particles

Rapid Filter- Essential parts

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Components of Rapid Filters



Parts of a rapid filter

- Filter tank: contains all system equipment except the control equipment. Usually rectangular and made of concrete and constructed side by side on either side of a control pipe gallery to minimize piping
- Filter sand: specially manufactured or selected sand and other filter media
- Gravel support bed: prevents sand from being disturbed
- Under drain system: carries filtered water & distributes the backwash water uniformly
- Wash water trough: collects back wash water
- Filter bed agitator: agitates sand layer for cleansing (air-scour or surface wash systems)
- Control equipment: maintains fairly constant flow.

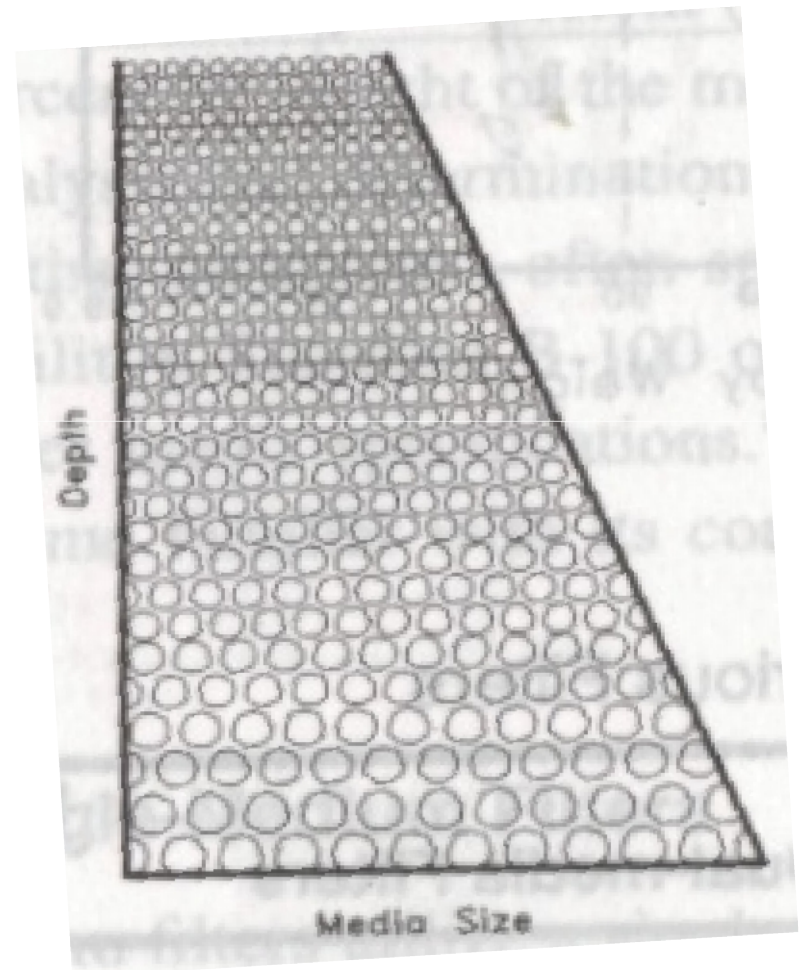
Rapid Filter Media

- Filter media control
 - the solids holding capacity of the filter bed
 - the hydraulic loading rate of the filters
 - The finished water quality
- Types
 - Single medium (usually sand)
 - Dual media (usually sand and anthracite)
 - Multi-media (usually sand, anthracite, garnet)

-

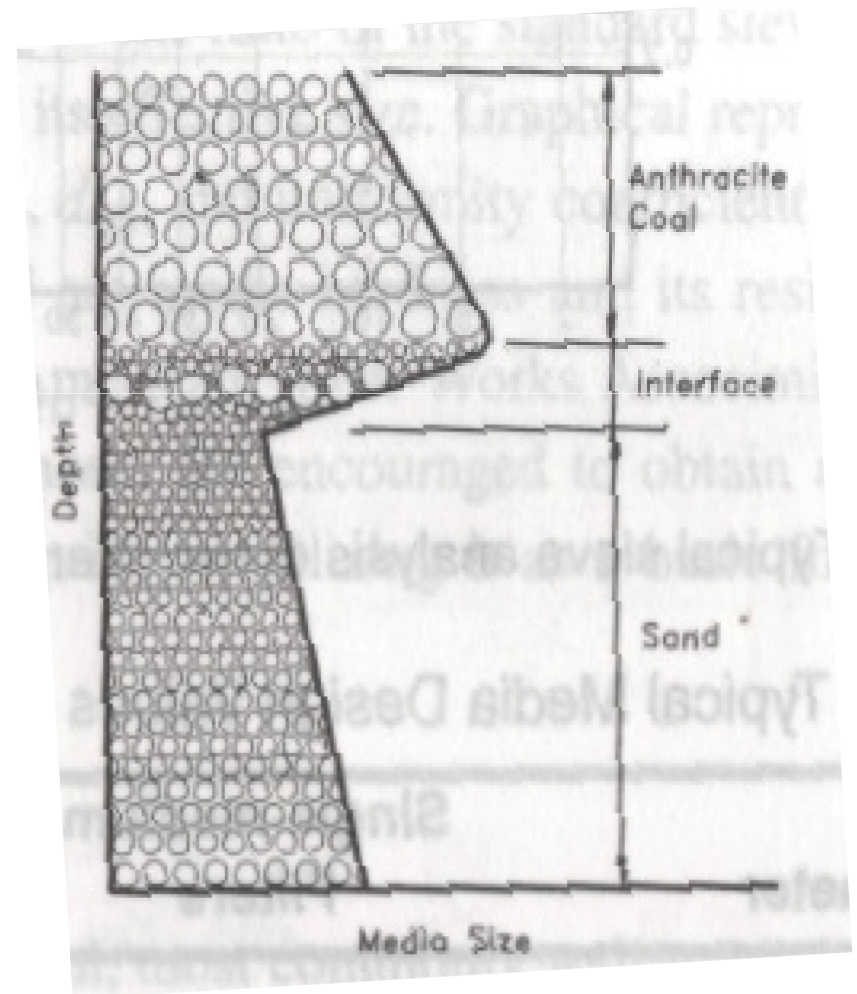
Single-medium filters

- Utilize single medium- usually well graded sand
- Reverse gradation after backwashing is the major drawback of these filters
- Only the top 4-5 cm of the filter bed are used for filtration
- Filter runs are short due to smaller solids-holding capacity
- Solution: use large diameter medium with a uniformity coefficient close to unity and deep medium bed



Dual-media Filters

- Avoids the problem of reverse gradation
- Top: larger and lighter filter media (e.g. anthracite)
- Bottom: Smaller and denser material (e.g. sand)
- Greater depth of the filter media is utilized

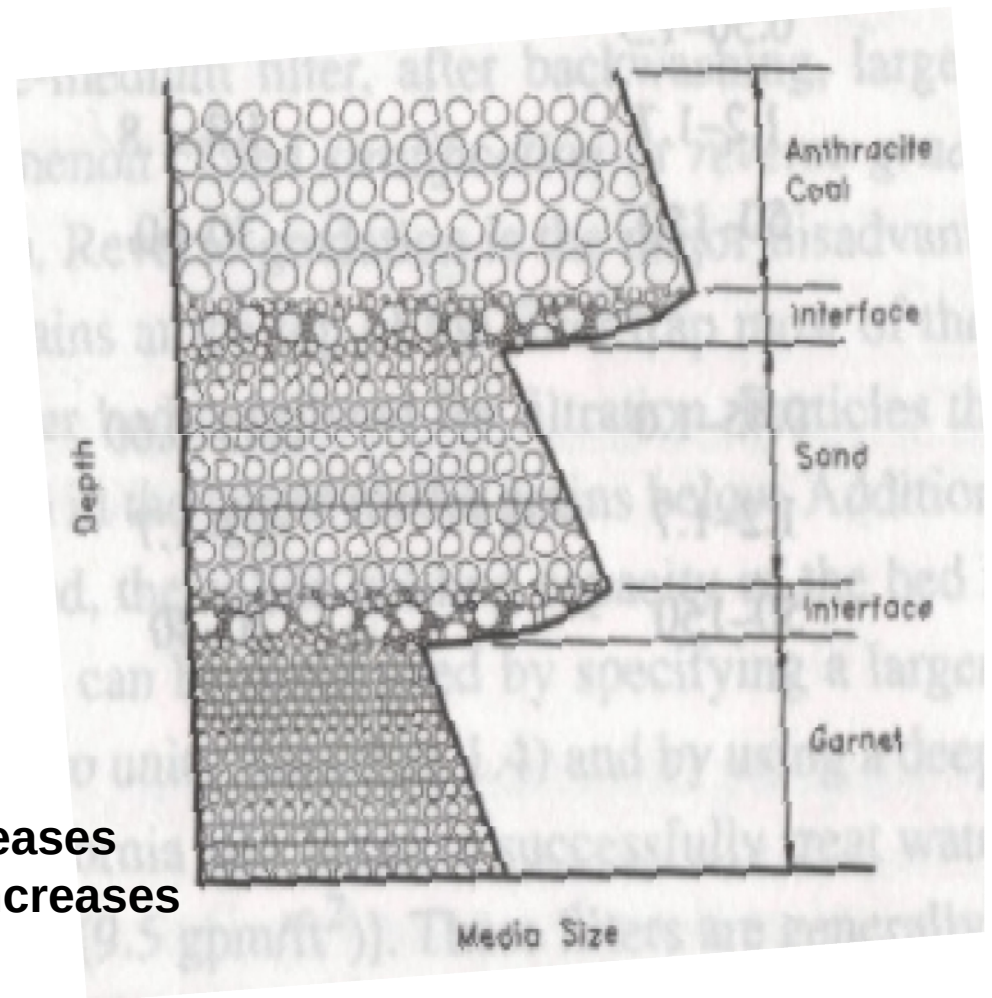


Multi-media Filters

Common materials used:

- Anthracite ($S_g = 1.55$)
- Sand ($S_g = 2.65$ and)
- Garnet ($S_g = 4.05$)
- Increased filter run time
- Improved water quality
- Costly

- ↓
- Size decreases
 - Density increases



Typical Filter Media Design Values

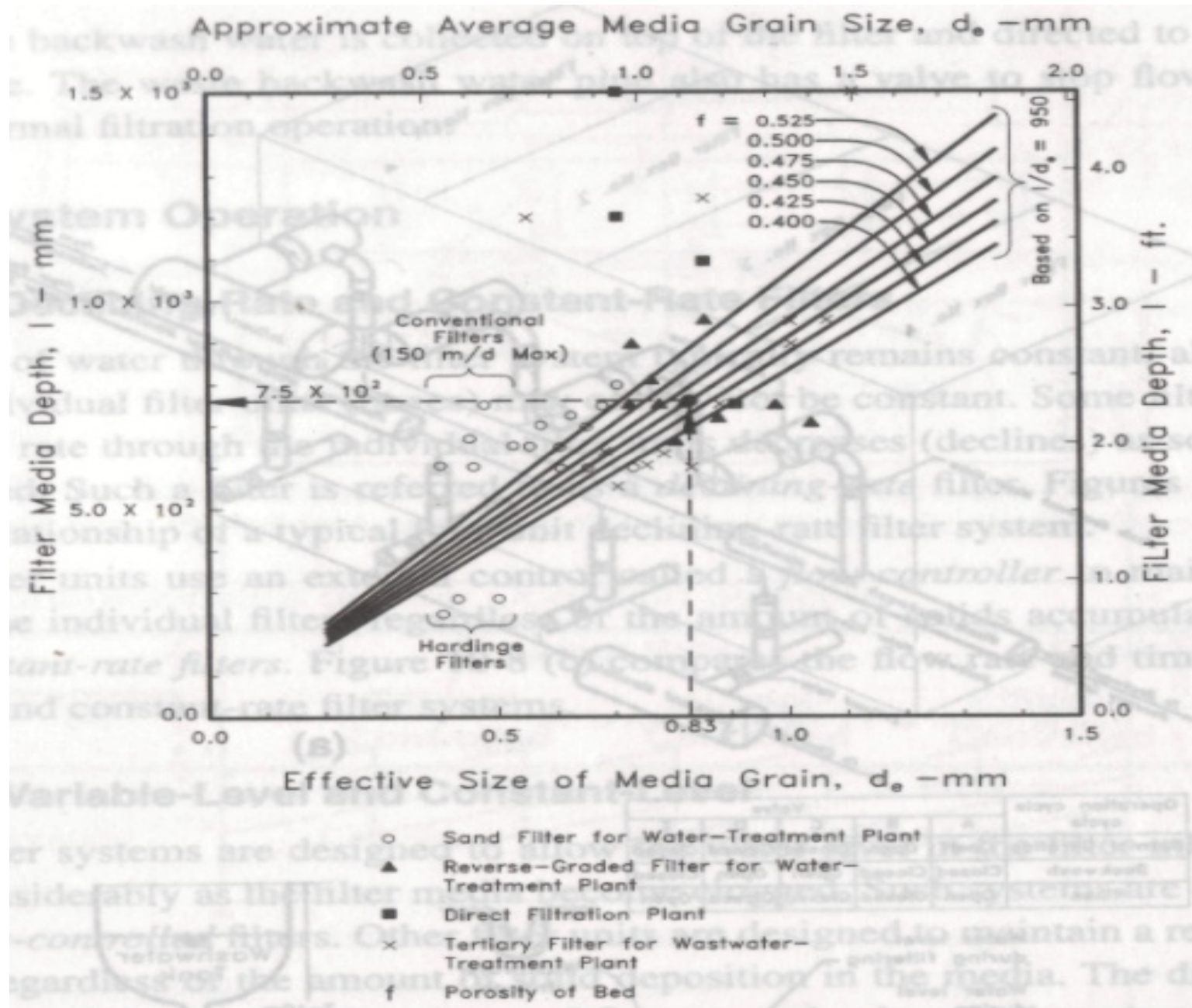
Parameter	Single-medium	Dual-media	Multi-media
Anthracite layer			
Effective size, mm	0.50-1.5	0.70-2.0	1.0-2.0
Uniformity coeff.	1.2-1.7	1.3-1.8	1.4-1.8
Depth, cm	50-150	30-60	50-130
Sand layer			
Effective size, mm	0.45-1.0	0.45-0.60	0.40-0.80
Uniformity coeff.	1.2-1.7	1.2-1.7	1.2-1.7
Depth, cm	50-150	20-40	20-40
Garnet layer			
Effective size, mm			0.20-0.80
Uniformity coeff.			1.5-1.8
Depth, cm			5-15

Selection of Filter Media

- Methods
 - Pilot testing: takes long time and costly
 - Experience from past studies (e.g. use media depth to effective size ratio)

Filter	Material	Effective size (de), mm	Media depth (l), cm	Uniformity coefficient	l/de
Small dual media	Anthracite Sand	1.00	50.8	1.5	1016
		0.50	25.4	1.3	
Intermediate-dual media	Anthracite Sand	1.48	76.2	1.5	1023
		0.75	38.1	1.2	
Large-dual media	Anthracite Sand	2.00	101.6	1.5	1016
		1.00	50.8	1.3	
Multi-media	Anthracite Sand Garnet	1.00	45.7	1.5	1306
		0.42	22.9	1.5	
		0.25	7.6	1.3	
Single-medium	Anthracite	1.00	101.6	1.4	1016

Relationship between depth and size of media



Head loss through clean filter beds

Equation	
Carmen-Kozeny	$h_L = \frac{f}{\phi} \frac{1-e}{e^3} \frac{L}{d} \frac{v^2}{g}$
Fair-Hatch	$h_L = k^2 v S^2 \frac{(1-e)^2}{e^3} \frac{L}{d^2} \frac{v}{g}$
Rose	$h_L = \frac{1.067}{\phi} C_d \frac{1}{e^4} \frac{L}{d} \frac{v^2}{g}$
Hazen	$h_L = \frac{1}{C} \frac{5.2 \times 10^6}{T + 10} \frac{L}{d_{10}^2} v$
Friction factor	$f = 150 \frac{1-e}{N_R} + 1.75$
Reynolds number	$N_R = \frac{dv}{\mu} \rho_w$
Coefficient of drag	$C_d = \frac{24}{N_R} + \frac{3}{\sqrt{N_R}} + 0.34$

C = coefficient of compactness (600–1200)
 C_d = coefficient of drag
 d = media grain diameter, m
 d_{10} = media effective size, mm
 e = porosity ratio (usually 0.4 to 0.5)
 f = friction factor (See Eq. (10.5).)
 g = acceleration due to gravity (9.81 m/s²)
 h_L = head loss, m (ft)
 k = filtration constant
 L = media depth, m (ft)
 N_R = Reynolds number
 S = shape factor (6.0 to 8.5)
 T = temperature °F
 v = filtration velocity, m/s
 μ = absolute viscosity, N-s/m² (kg/m·s)
 ν = kinematic viscosity, m²/s
 ρ_w = density of water, kg/m³
 ϕ = particle shape factor (usually 0.85 to 1.0)

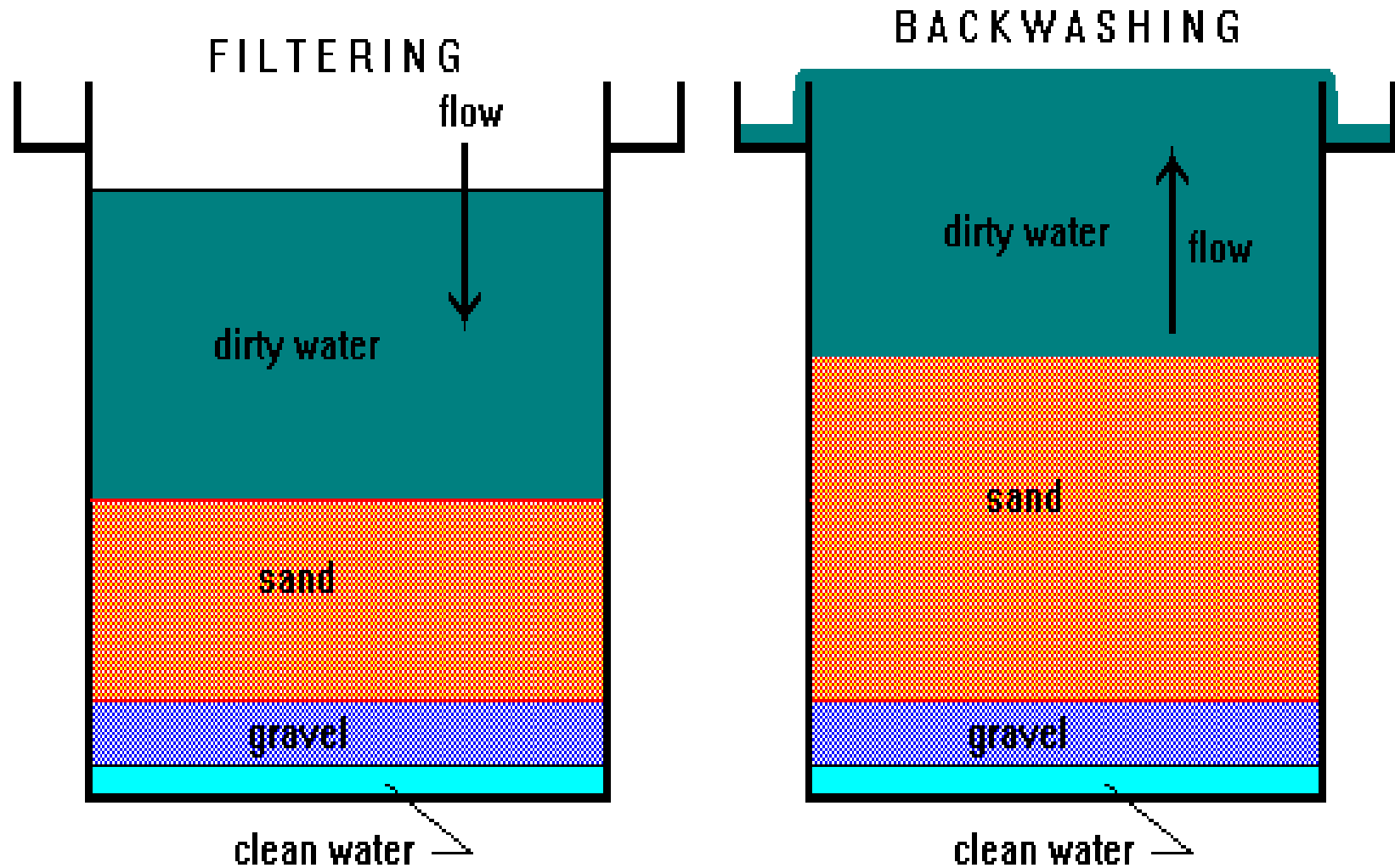
Exercise 1

Calculate the head loss through a clean bed of uniform sand that has a grain diameter of 0.5 mm, bed depth 0.3 m and porosity ratio of 0.4. The shape factor is 0.85 and the specific gravity of sand is 2.65. The water temperature is 20°C and the filter has a loading rate of 4.0 m³/m²-h. Use the Carmen-Kozeny equation.

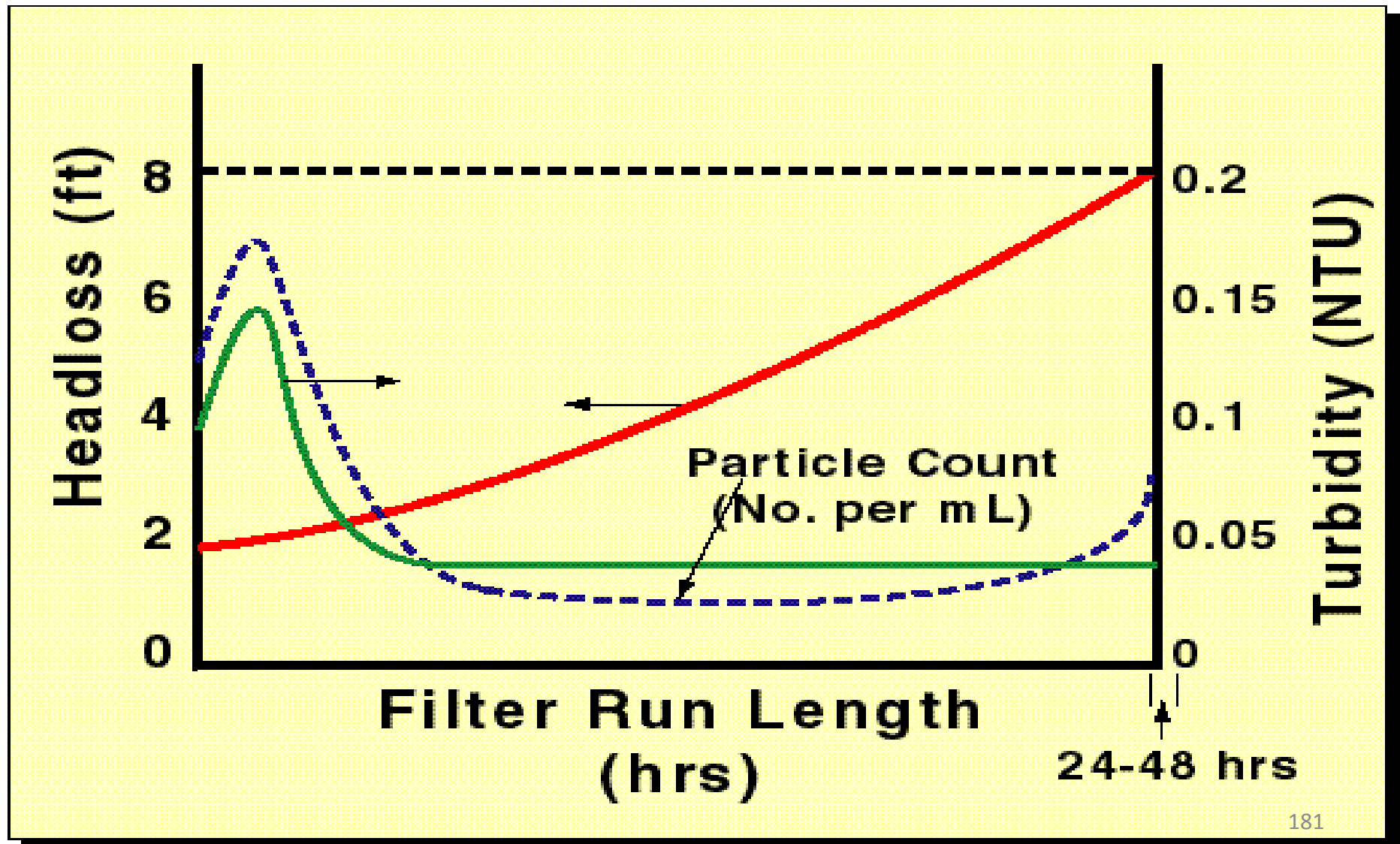
Backwashing

- Involves passing water upward through the filter media at a velocity sufficient to expand (fluidize) the bed and wash out the accumulated solids
- Done when:
 - The head loss through the filter exceeds the design value
 - Turbidity breakthrough causes the effluent quality to be less than a minimum acceptable level
 - A pre-selected maximum filter run time has passed since it was last cleaned
- Filtered water is used which consumes 1-5% of the product water
- Bed expansions are achieved in backwashing

Filter media during filtration and backwashing



Ideal Filter Run





Backwashing Design Considerations

- Settling velocity of the media
- The backwash flow rate
- The head loss during backwash
- The duration of backwash cycle
- The quantity of water required to backwash each filter

Settling velocity of the filter media

- Backwash system should operate without washing out the filter media
- Dual or multi-media systems should be designed such that the entire filter bed has approximately the same settling velocity

$$d_2 = d_1 \left(\frac{S_{g1} - 1}{S_{g2} - 1} \right)^{2/3}$$

Where,

d_2 =effective size of the media with specific gravity of S_{g2} , mm

d_1 =effective size of the media with specific gravity of S_{g1} , mm

Procedure:

- Select the sand medium that would yield acceptable finished water quality
- Use the above formula to select the effective size of the other media

Exercise 2

- A dual-media filter bed is being designed. The media used are sand and anthracite. The effective size and specific gravity of sand are 0.60 mm and 2.65. The specific gravity of anthracite is 1.5. The anthracite layer is $\frac{2}{3}$ of the total bed depth. Calculate
 - The effective size of the anthracite that has the same settling velocity as the sand
 - The depth of the sand and the anthracite layer

Backwash flow rate

- Vertical velocity of water

Vertical velocity of water

during backwash

> floc settling velocity (2.5-25 cm/min)

< media settling velocity

- Minimum backwash rise rate = 30 cm/min
- The terminal settling velocity for discrete settling can be used
- Settling velocities at 20°C can also be estimated by:

$$V_s = \left\{ \frac{0.018 (\rho_s - \rho_w)^2 g^2}{\rho_w \mu} \right\}^{1/3} d_{60}$$

v_s in m/s and d_{60} m.

- $V_s(\text{m/min}) = 10d_{60}$ for sand ($S_g = 2.61$)
- $V_s(\text{m/min}) = 4.7d_{60}$ for anthracite ($S_g = 1.5$)

Backwash velocity

- Effective backwash rate is approximately equal to 10% of media settling velocity.

$$V_b = d_{60} \text{ for sand at } 20^\circ\text{C}$$

$$V_b = 0.47d_{60} \text{ for anthracite at } 20^\circ\text{C}$$

At temperature other than 20°C

$$V_{bT} = V_{b20} \times \mu_T^{-1/3}$$

V_{bT} = backwash rate at temperature $T^\circ\text{C}$, m/min

V_{b20} = backwash rate at 20°C , m/min

μ_T = dynamic viscosity at temperature $T^\circ\text{C}$, Pa.s

Filter bed expansion

- Increase in backwash velocity will result in a greater expansion of bed

$$e_{ab} = \left(\frac{V_b}{V_s} \right)^{0.22}$$

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

Where,

e_{ab} = porosity of expanded bed

V_s , media settling velocity, m/s

L = bed depth at rest, m

L_{fb} = expanded bed depth, m

Exercise 3

3. Determine the terminal settling velocity and the minimum fluidization velocity of a sand filter of effective size 0.60 mm, with a uniformity coefficient 1.6, a specific gravity 2.65 and a porosity of 0.44. Determine also the backwash rate at 10°C and 20°C.
4. The filter bed in Problem 3 is expanded to a porosity of 0.7 by backwashing. Calculate the required backwashing velocity and the resulting expanded bed depth.

Duration of backwash and quantity of water

Backwash duration:

- Should be long enough to clean the media bed
- It is a function of:
 - Nature of the floc
 - Whether air-scour (or surface wash) is used
 - The geometry of the filter bed and the washwater troughs
- Usually lasts from 8 to 15 minutes

Quantity of backwash water:

- Depends on washwater flow rate and the duration of the backwash cycle
- Must also include water used to operate the surface wash systems

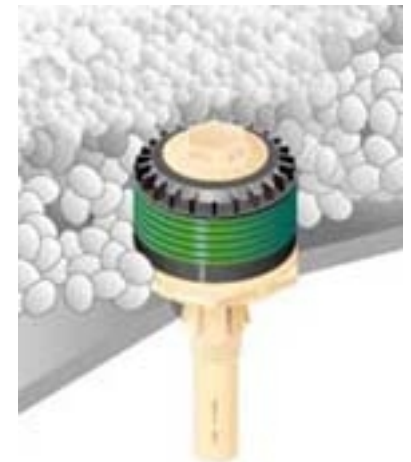
Filter bed agitators

- Surface wash systems:
 - Provide maximum agitation at the top of the filter bed
 - Done by spraying water through nozzles
 - Located on fixed piping manifolds or rotating arms
 - Nozzles spaced 5-8.6 cm apart
 - discharge rate : 1.2-2.4 m³/m².h for rotating pipe systems
: 5-10 m³/m².h
 - Operating heads: 350-520 kPa
 - Placed 2.5-5 cm below the media bed at rest or just above sand/antracite interface in dual media filters.
 - Started 1-3 min before the start of the backwash cycle
 - Stopped 5-10 min before the end of the backwash cycle

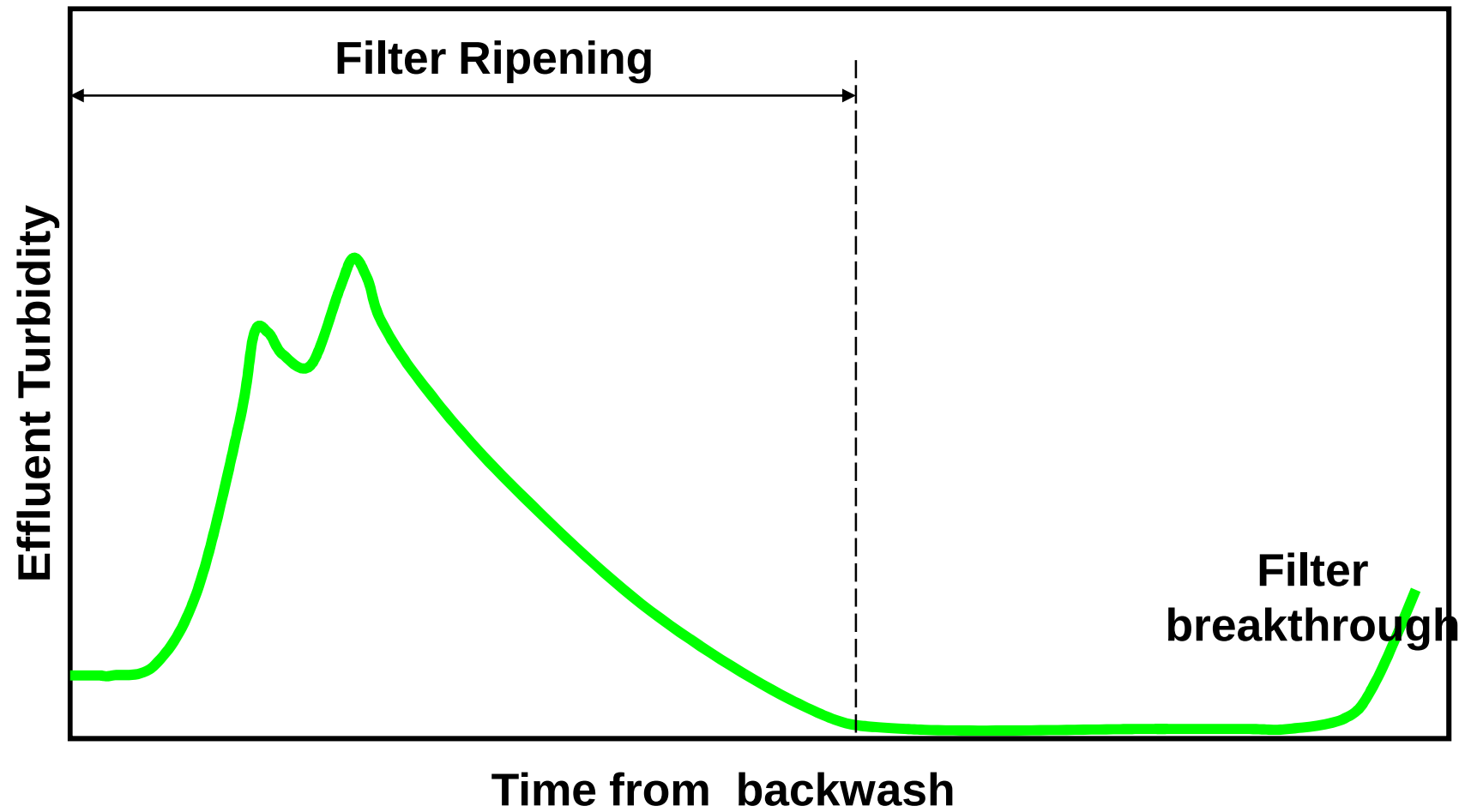
Filter bed agitators

Air-scour:

- Air is introduced at the bottom
- More effective than surface wash systems
- Two alternative methods of application:
 - Air-scour applied before (2-5 min) the backwash
 - Simultaneous application of air-scour and backwash



Controlling Turbidity Spikes

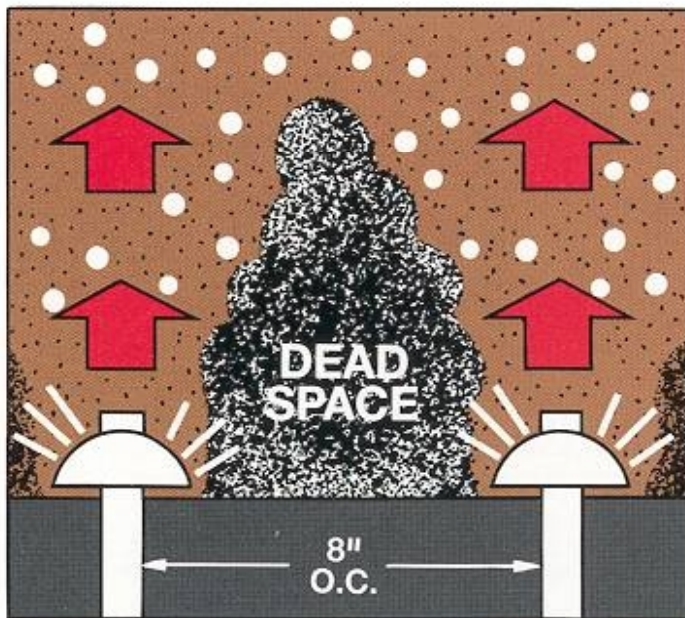


Controlling Turbidity after backwashing

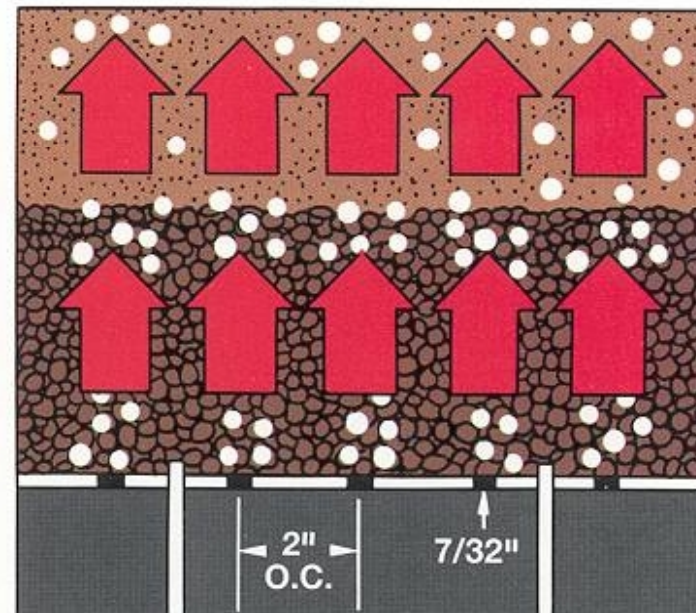
- Some strategies:
 - Waste-to-drain
 - Results in wastage of a large amount of water
 - Its application has become less common
 - To use ripening filtration rates
 - Filter is run at a reduced rate after backwash
 - Can be implemented at little or no cost
 - Usually effective
 - Preconditioning the filter
 - Involves use of a coagulant

Washwater Trough

- Placed above the filter media to collect the backwash water and carry it to the drain system
- Should be closely spaced and set high above the expanded media

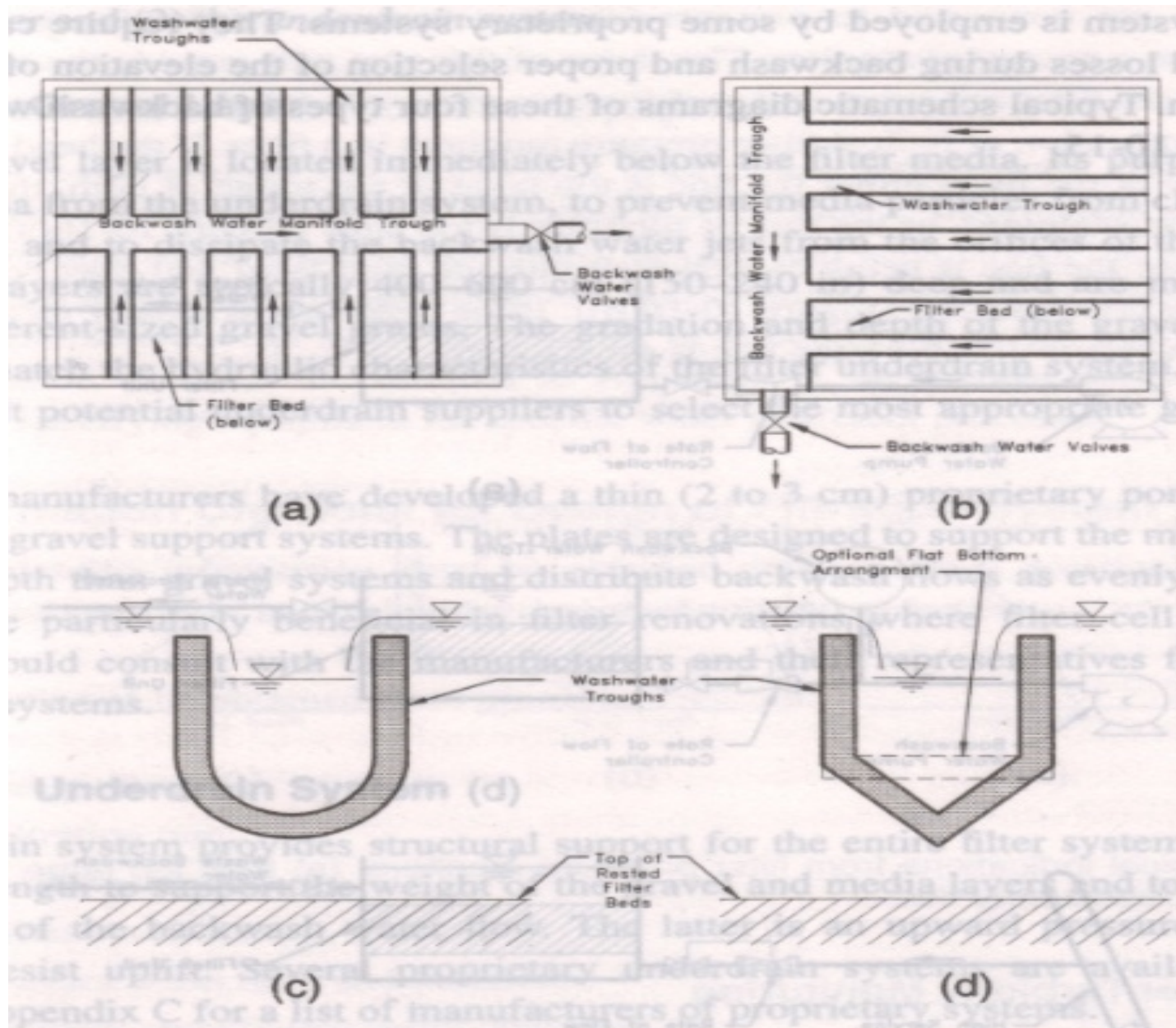


- 55 nozzles/m² - acceptable
- 40 nozzles/m² – large dead zones



- 268 nozzles/m² - good

Typical backwash trough arrangement



Backwash water systems and head loss

- Water storage: should be equal to the maximum amount needed to backwash each filter once during a 24-hour period
- Backwash water delivery system
- Head loss during backwash:

$$h_L = L(1 - e)(S_g - 1)$$

Where,

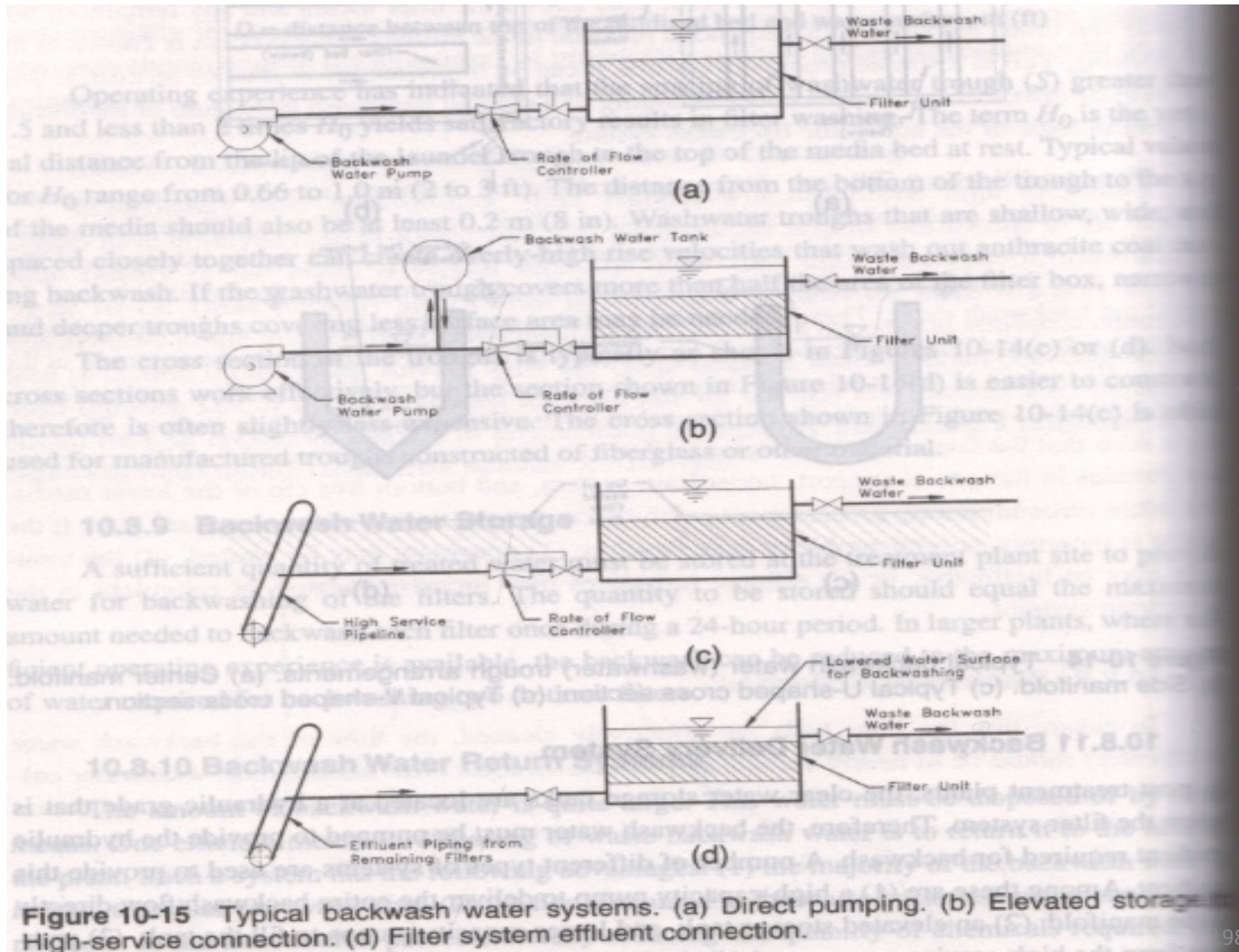
h_L = head loss through the media during backwash, m

E = porosity of the clean stratified bed at rest

L = depth of the stratified bed at rest, m

S_g = specific gravity of the media

Backwash water delivery system

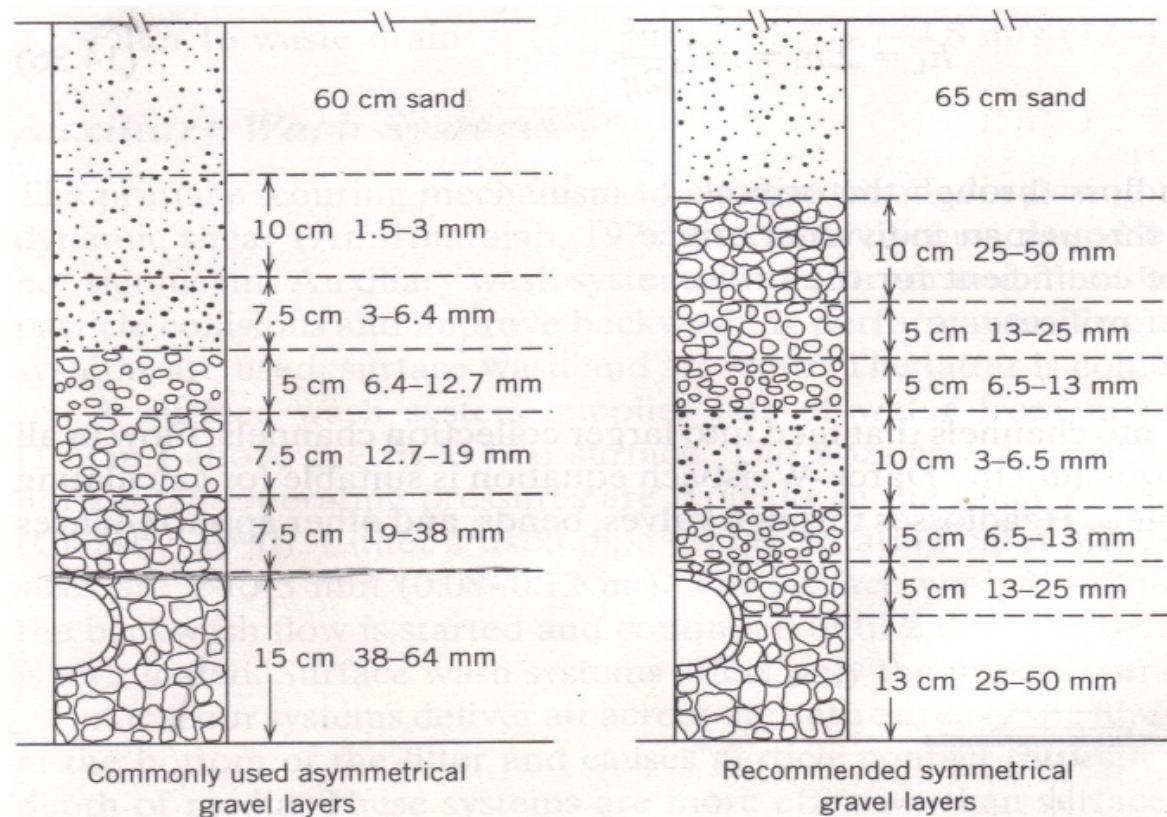


Backwash water delivery system

Water Supply	Description	Advantages	Disadvantages
Backwash pumps	Pumps, sized to provide the entire backwash flow, withdraw water from the filter effluent channel or finished water clearwell and provide it directly to the filter bottom.	Provides the maximum amount of control over backwash flow rates.	Large pumps are required.
Elevated tank	Small pumps withdraw water from the filter effluent channel or finished water clearwell and send it to an elevated tank. The <i>minimum water</i> level in the tank is typically 9–12 m (30–40 ft) above the filter media. During backwash, water flows from the tank to the filters by gravity.	Smaller pumps required because the volume of water required for backwash can be pumped to the tank over a period of hours.	Backwash flow rate can decline as water level in the elevated tank declines.
Effluent channel (self-backwashing filter)	Filter effluent flows to a common effluent channel whose water elevation is controlled by a weir set several feet above the top of the media. During backwash, the water level in the filter box drops so that the head in the effluent channel is sufficient to provide the necessary backwash flow. This type of filter is often called a self-backwashing filter.	Simplicity of design and operation; no pumps required.	Less control over backwash flow rates. Deep filter box required to provide sufficient head for filtering (maximum water level in filter to water level in effluent channel) and sufficient head for backwashing (water level in effluent channel to minimum water level in filter).

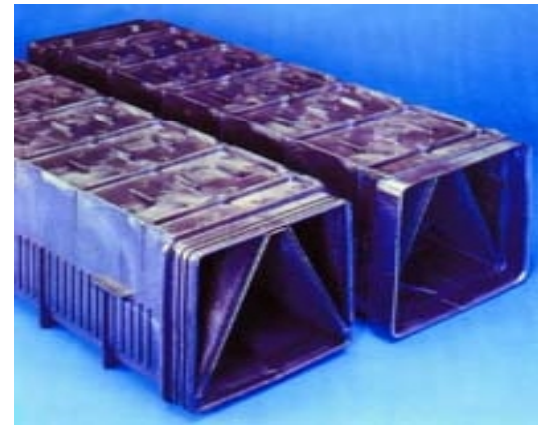
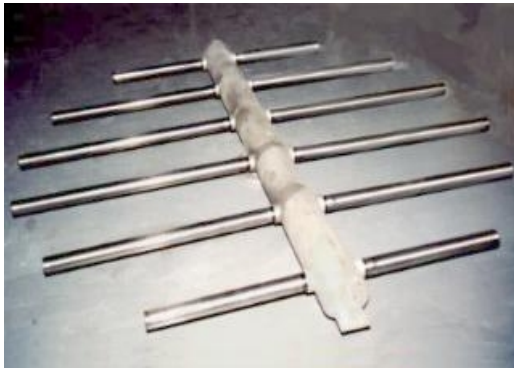
Filter media support system: Gravel layer

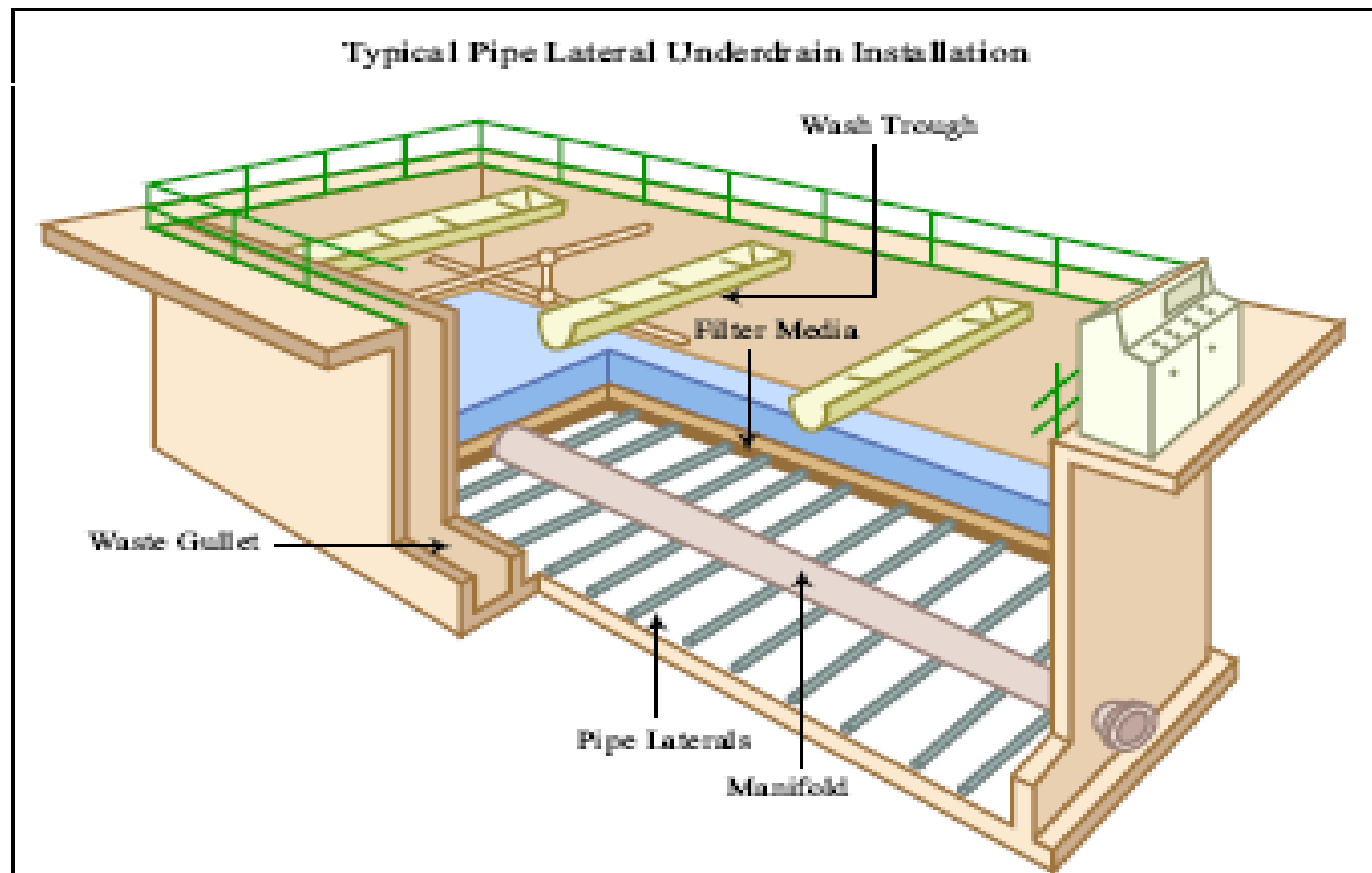
- Purpose:
 - To separate filter media from underdrain system
 - To prevent clogging of underdrain orifices by filter media particles
 - To dissipate the backwash jets
- Typical gravel layer



Filter media support system: underdrain system

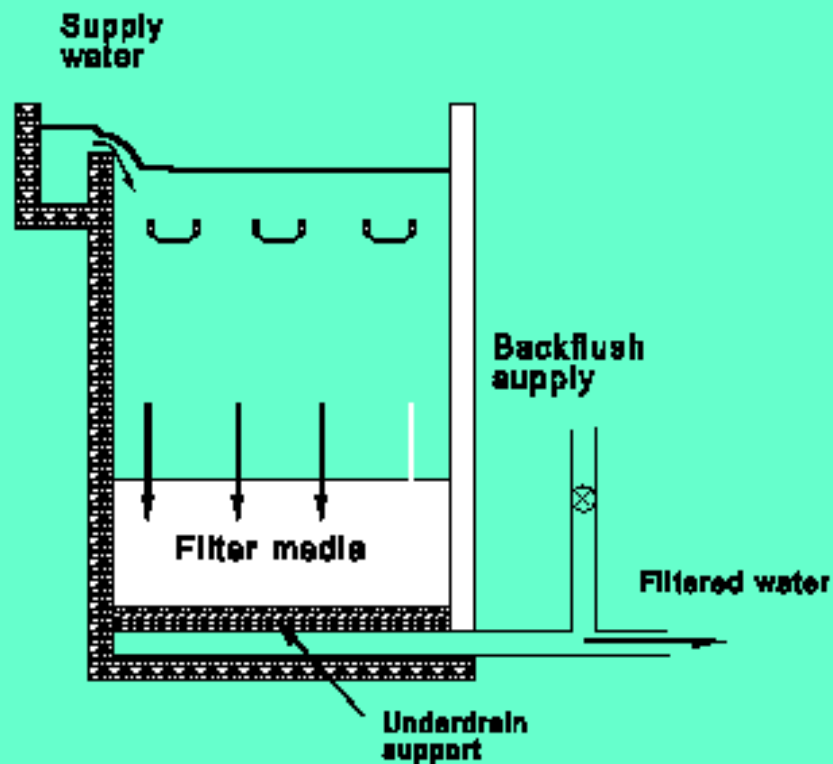
- Provides structural support for the entire filter system
- Must be strong enough to support the overlying weight and resist uplift by the backwash water flow
- Examples of underdrain systems



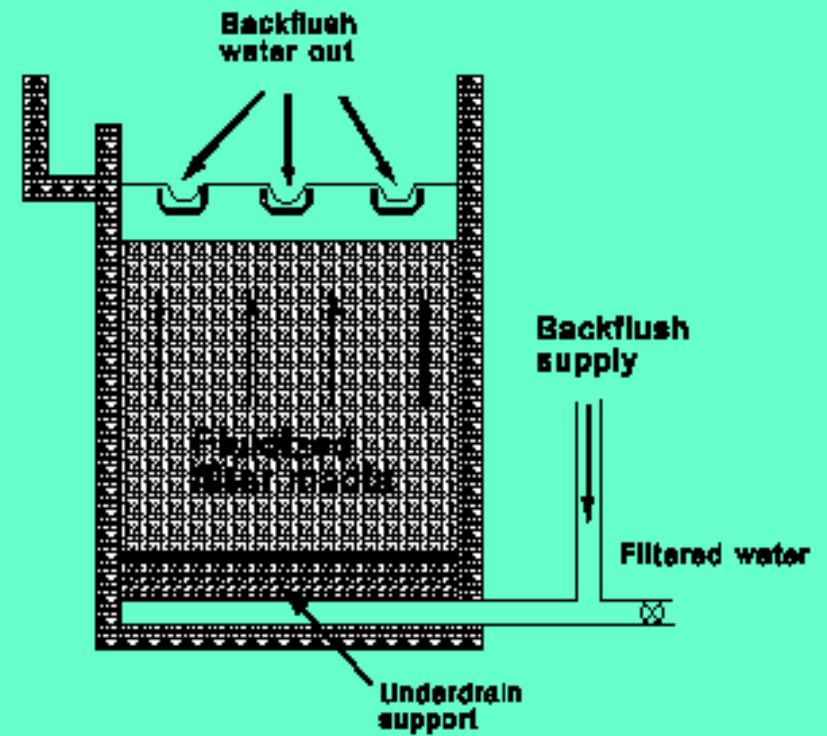


Source: F.B. Leopold Company, 2003. Filtration, The Process, Underdrain Types.
<http://www.fbleopold.com/water/filtration/underdrain.htm>. Accessed February 21, 2005.

Filter Operation



a) Operation during filtration



b) Operation during cleaning

Design of Rapid Filters

- Performance criteria
 - Effluent water quality (Turbidity: 0.1-0.3 NTU)
 - Filter run time (1-4 days)
 - Recovery (r): > 95%

$$r = \frac{V_F - V_{BW} - V_{FTW}}{V_F} = \frac{UFRV - UBWV - UFWV}{UFRV}$$

Where,

$$UFRV = \frac{V_F}{a}; \quad UBWV = \frac{V_{BW}}{a}; \quad UFWV = \frac{V_{FTW}}{a}$$

UFRV = unit filter run volume (m³/m²); UBWV = unit backwash volume (m³/m²)

UFWV = unit filter-to-waste volume (m³/m²)

V_F = volume of water filtered during one filter run, m³

V_{BW} = volume of water required to backwash one filter, m³

V_{FTW} = volume of water discharged as filter-to-waste, m³

Design of Rapid Filters

- Process design criteria
 - Filter type
 - Number and dimensions of filters
 - Filtration rate
 - Available head
 - Filter media

Design criteria for typical filters

Parameter	Value
Filter type	Conventional, dual media
Flow control	Influent weir split, constant level
Number	12
Inside dimensions (m x m)	4.3 x 11.6 x cells
Media surface area (each filter), m ²	100
Media surface area (total area), m ²	1200
Maximum available head, m	2.5
Filtration rate- one filter offline, m/h	15
Filtration rate- all filters in service, m:h	13.75
Filter media- Anthracite:	
• Depth, m	1.5
• Effective size, mm	1.0
• Uniformity coefficient	<1.4
• Specific gravity	1.5-1.6
Filter media- Sand:	
• Depth, m	0.3
• Effective size, mm	0.5
• Uniformity coefficient	<1.4
• Specific gravity	2.65
Backwash criteria:	
• Maximum rate, m/h	56
• Normal rate, m/h	45
• Duration, min	15

Design of Rapid Filters

- Flow control
 - Filtration rate control and flow apportionment
 - Head loss accommodation
- Backwashing
 - Alternatives for backwashing
 - Backwash delivery systems

Design criteria for auxiliary backwash systems

Criteria	Fixed-nozzle surface wash	Rotating nozzle surface wash	Air scour
Surface wash water flow rate, m/h	7-10	1.2-1.8	-
Air flow rate, m ³ /m ² .h	-	-	36-72
Pressure at discharge point, bar	0.5-0.8	5-7	0.3-0.5
Duration of washing, min	4-8	4-8	8-15
Backwash water flow rate, m/h	30-60	30-60	15-45

Filter conduits

- Influent pipe to filters: 0.6-1.8 m/s
- Effluent pipe carrying filtered water: 0.9-1.8 m/s
- Drains carrying spent backwash water: 1.2-2.4 m/s
- Wash water line (influent): 2.4-3.7 m/s
- Filter to waste drain: 3.7-4.8 m/s

Assignments

Assignment-5

- Make review of key planning, design and construction considerations for water treatment plants and report

(Refer File: [AWWA_Water Treatment Plant Design.pdf](#), Chapters 2 & 3)

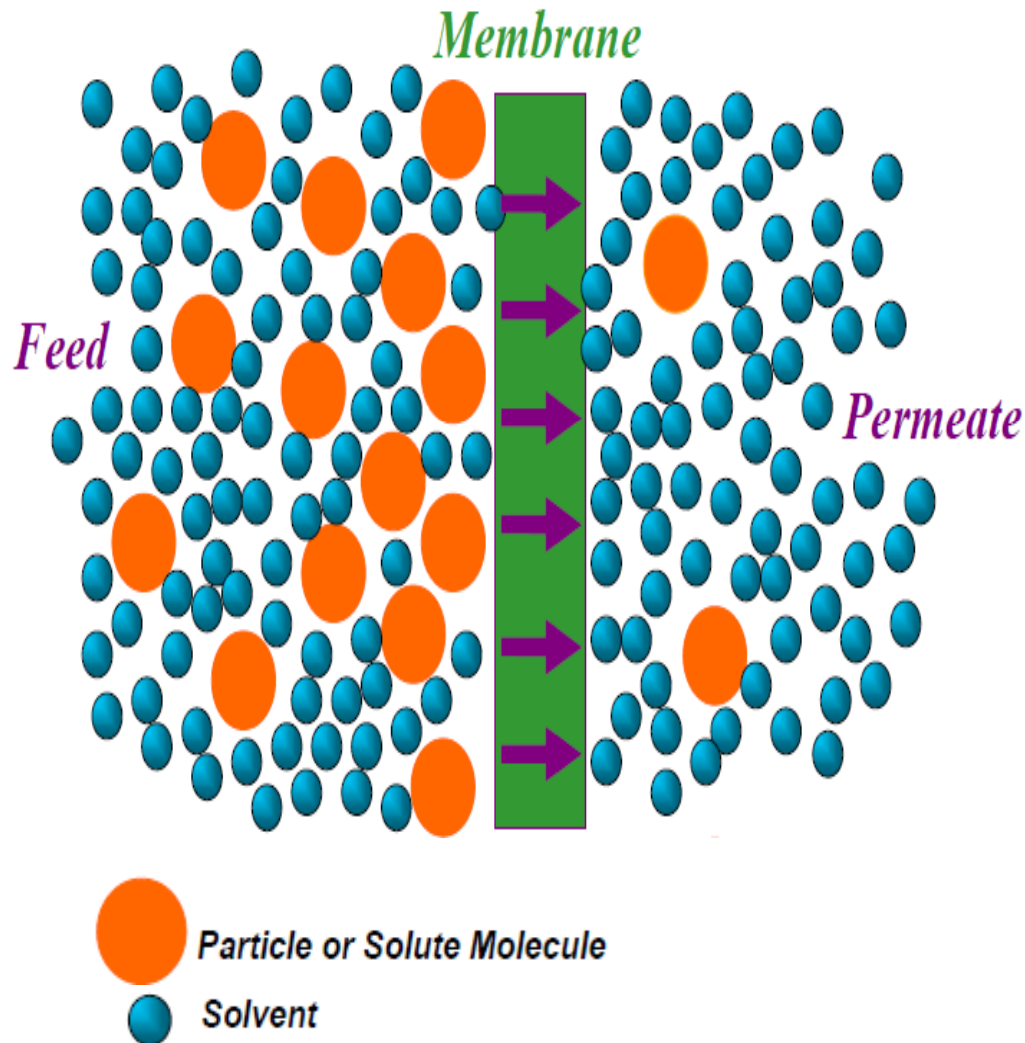
Assignment-6

Reading assignment on Roughing Filters

4.2 Membrane Processes

- A membrane is a selective barrier that permits the separation of certain species in a fluid by combination of sieving and diffusion mechanisms
- Membranes can separate particles and molecules, over a wide particle size range and molecular weights

Membrane Processes



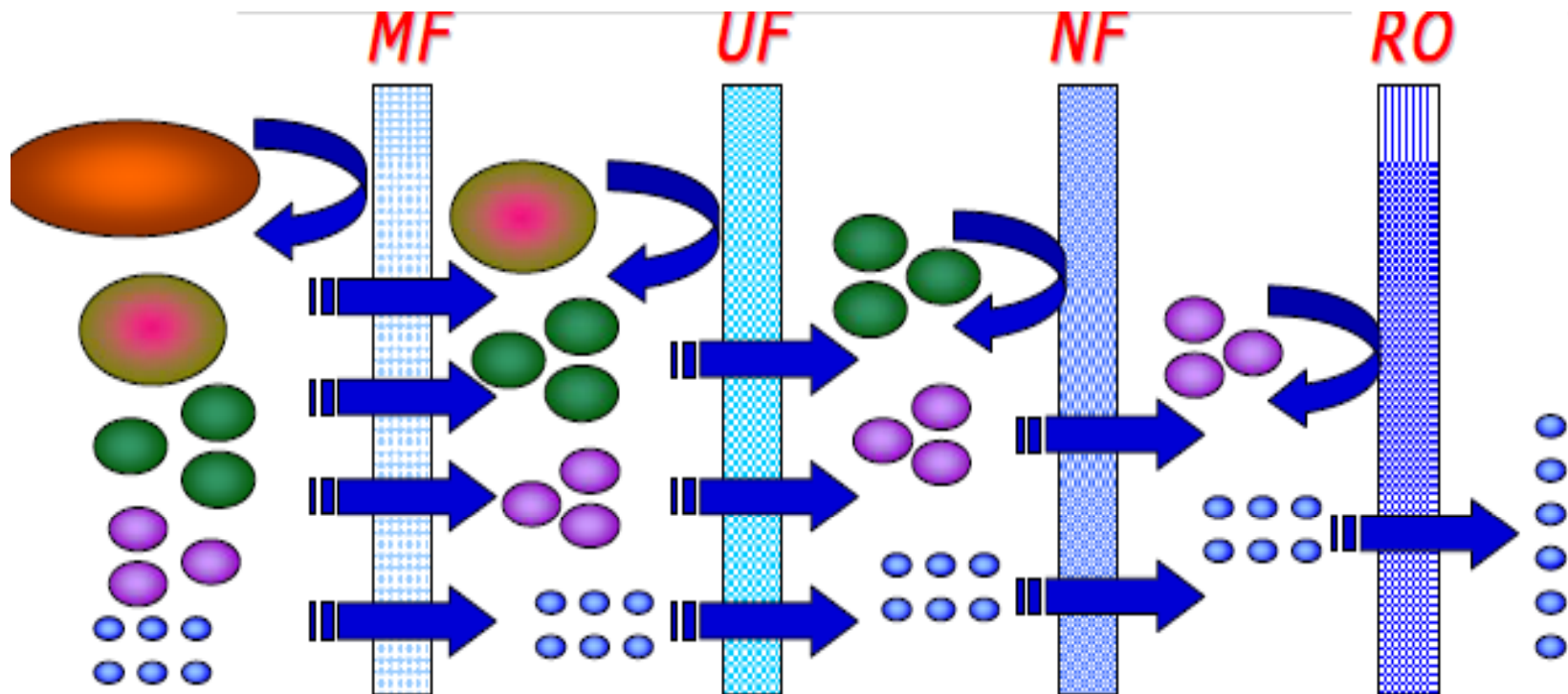
- **Feed**: the influent water to the membrane.
- **Permeate**: the portion of the feed stream that passes through the membrane that contains lower TDS than the feed water.
- **Concentrate or Retentate**: the portion of the feed water that does not pass the membrane that contains higher TDS than the feed stream.

Membrane Process Types

The main membrane processes used in water treatment are:

- **Pressure membranes**
 - Microfiltration (MF)
 - Ultrafiltration (UF)
 - Nanofiltration (NF)
 - Reverse Osmosis (RO)
- **Electrodialysis (ED)**

Pressure-Driven Membrane Processes



 *Suspended Solids (Particles)*

 *Macromolecules (Humics)*

 *Multivalent Ions (Hardness)*

 *Monovalent Ions (Na^+ , Cl^-)*

 *Water Molecules*

Pressure Membranes

They are membranes that are used to separate solids from fluid by application of pressure on the membrane

Membrane	Pore Size	Pressure
MF	Larger than UF	70 kPa
UF	0.001 – 10 μm	100 – 500 kPa
NF	Between UF and RO	500 – 1400 kPa
RO	0.0001 – 0.001	1400 – 8300 kPa

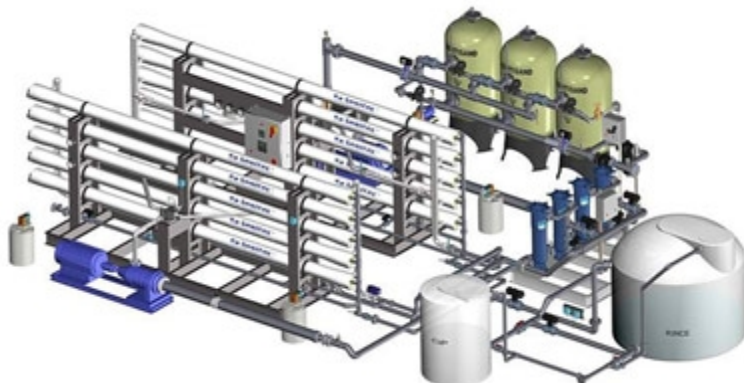
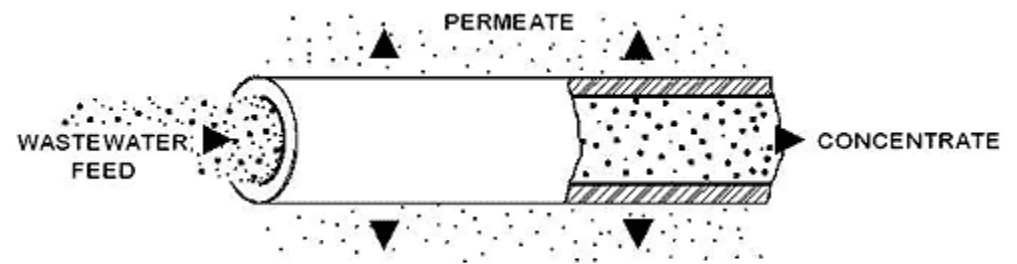
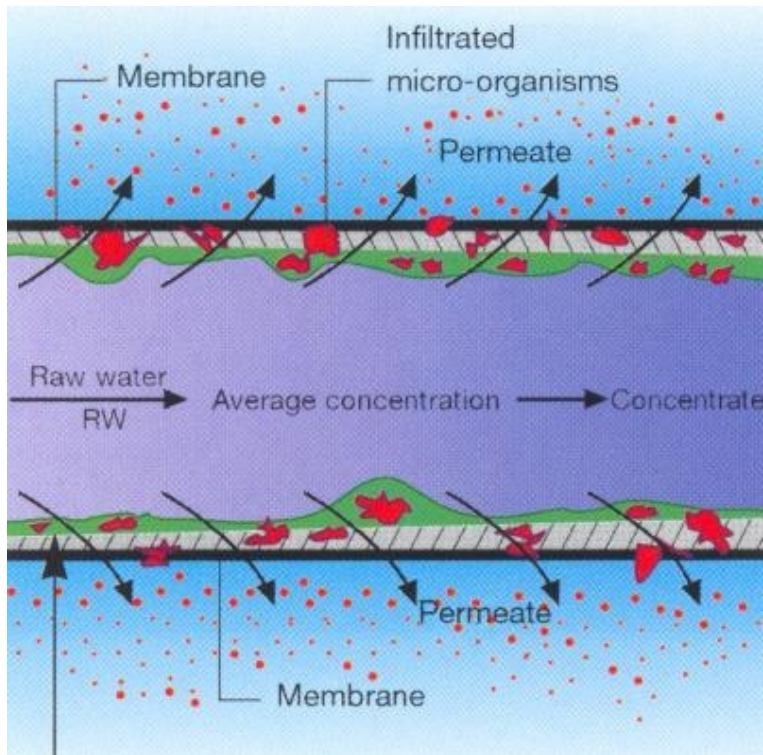
Module Design

- Tubular
- Hollow fibers
- Spiral wound
- Plate and frame

Tubular Membranes

Tubular modules consist of a set of parallel tubes all penetrating a circular plate at either end of a tube bundle housed inside a larger shell, or shroud. Feed material is pumped through the tubes in a cross-flow manner. Permeate is collected in the shroud while the retentate passes out the other end of the tubes. Advantages include turbulent flow (providing good membrane/solution contact and removing retentate film buildup), relatively easy cleaning, easy handling of suspended solids and viscous fluids and ability to replace or plug a failed tube while the rest of the system runs. Disadvantages include high capital cost, low packing density, high pumping costs, and limited achievable concentrations.

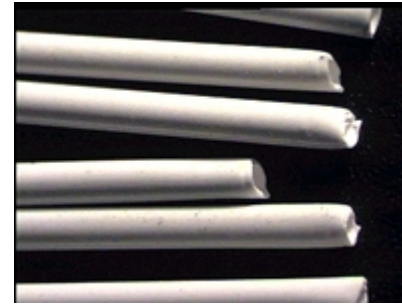
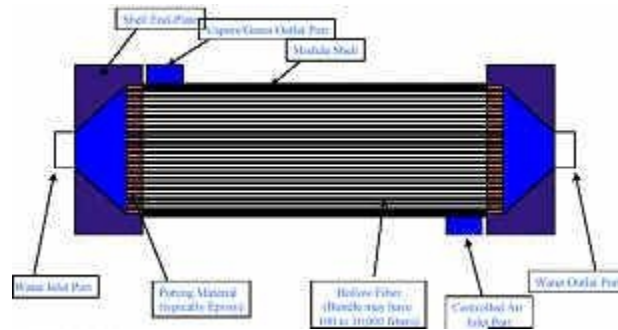
Tubular Membranes



Hollow Fiber Membranes

Hollow-fiber membranes, another cross-flow arrangement, consist of hollow, hair-like fibers bundled together into either a U-shape or straight-through configuration. Tube bundles are inside a pressure vessel and feed material normally flows inside the tubes. Fibers in the straight-through design are somewhat larger and allow low levels of suspended solids. The finer strands in the U-tube cannot tolerate suspended solids. U-tubes tend to be used for reverse osmosis, and the straight-through design for ultrafiltration. Advantages include low pumping power, very high packing density, cleaning can be accomplished with backflushing, and ability to achieve high concentrations in the retentate. Disadvantages include the fragility of the fibers, inability to handle suspended solids well, difficult cleaning and, in the straight-through design, damage of one fiber requires replacement of the entire module.

Hollow Fibers Membranes



Spiral Wound Membranes

Spiral membranes consist of two layers of membrane, placed onto a permeate collector fabric. This membrane envelope is wrapped around a centrally placed permeate drain (see picture below). This causes the packing density of the membranes to be higher. The feed channel is placed at moderate height, to prevent plugging of the membrane unit. Spiral membranes are only used for nano filtration and Reverse Osmosis (RO) applications.

A spiral wound membrane module comprises a spiral wound membrane element including a separation membrane having high back pressure strength. The flow rate of permeate in filtration of the spiral wound membrane element is preferably set to 0.5 to 2.0 m³ /m² /day, and the filtration time is preferably set to 10 to 300 minutes. The flow rate of wash water in washing is preferably set to 1.0 to 4.0 m³ /m² /day, and the washing time is preferably set to 10 to 300 seconds. In such ranges, the ratio of the permeate volume in filtration to the permeate volume in back wash reverse filtration is set to be not more than 600.

Spiral Wound Membranes

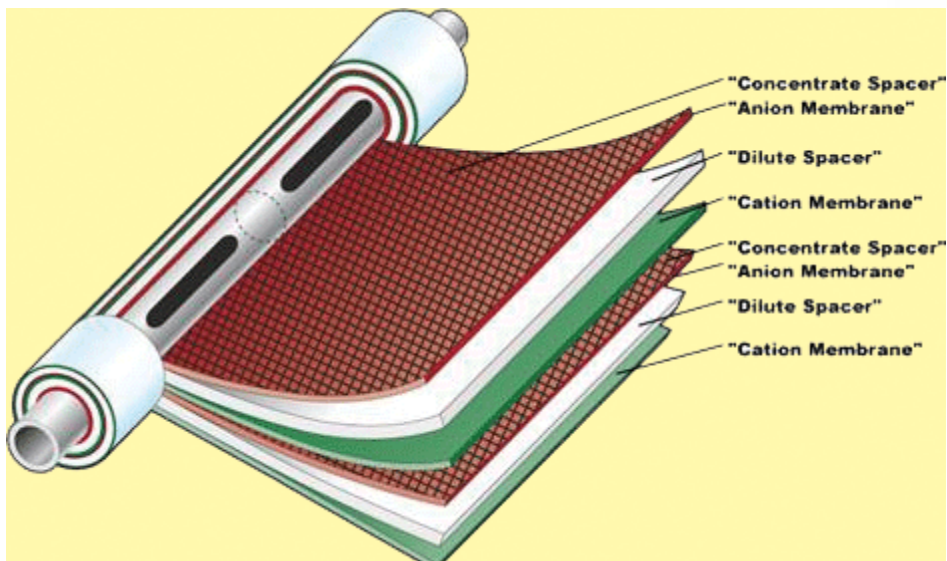
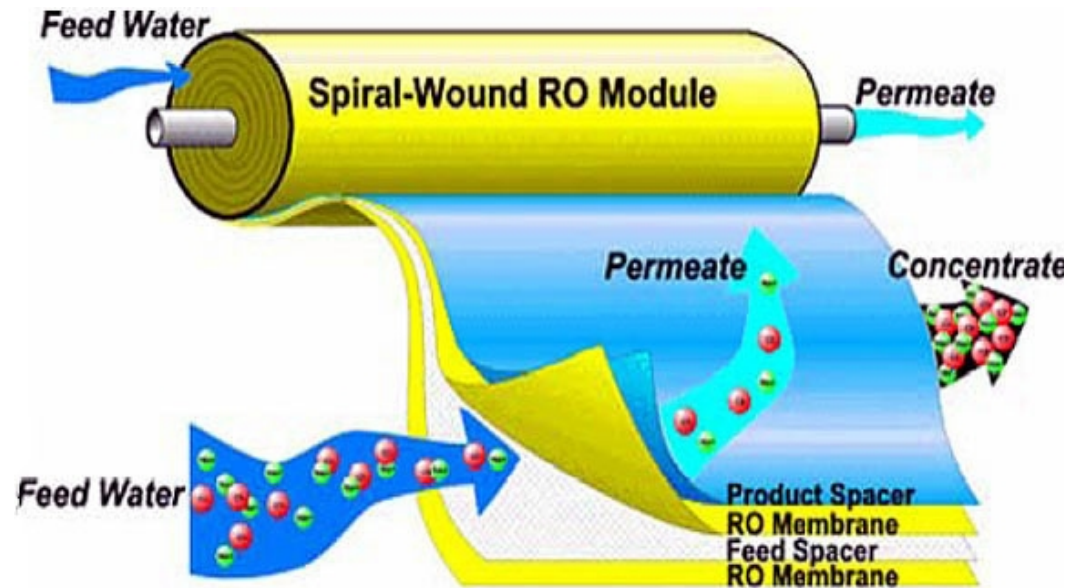
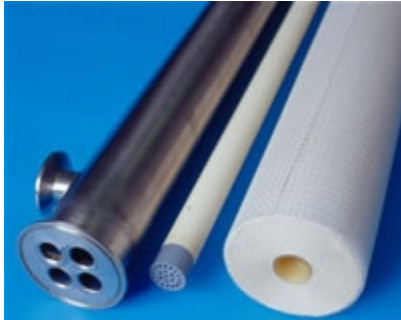


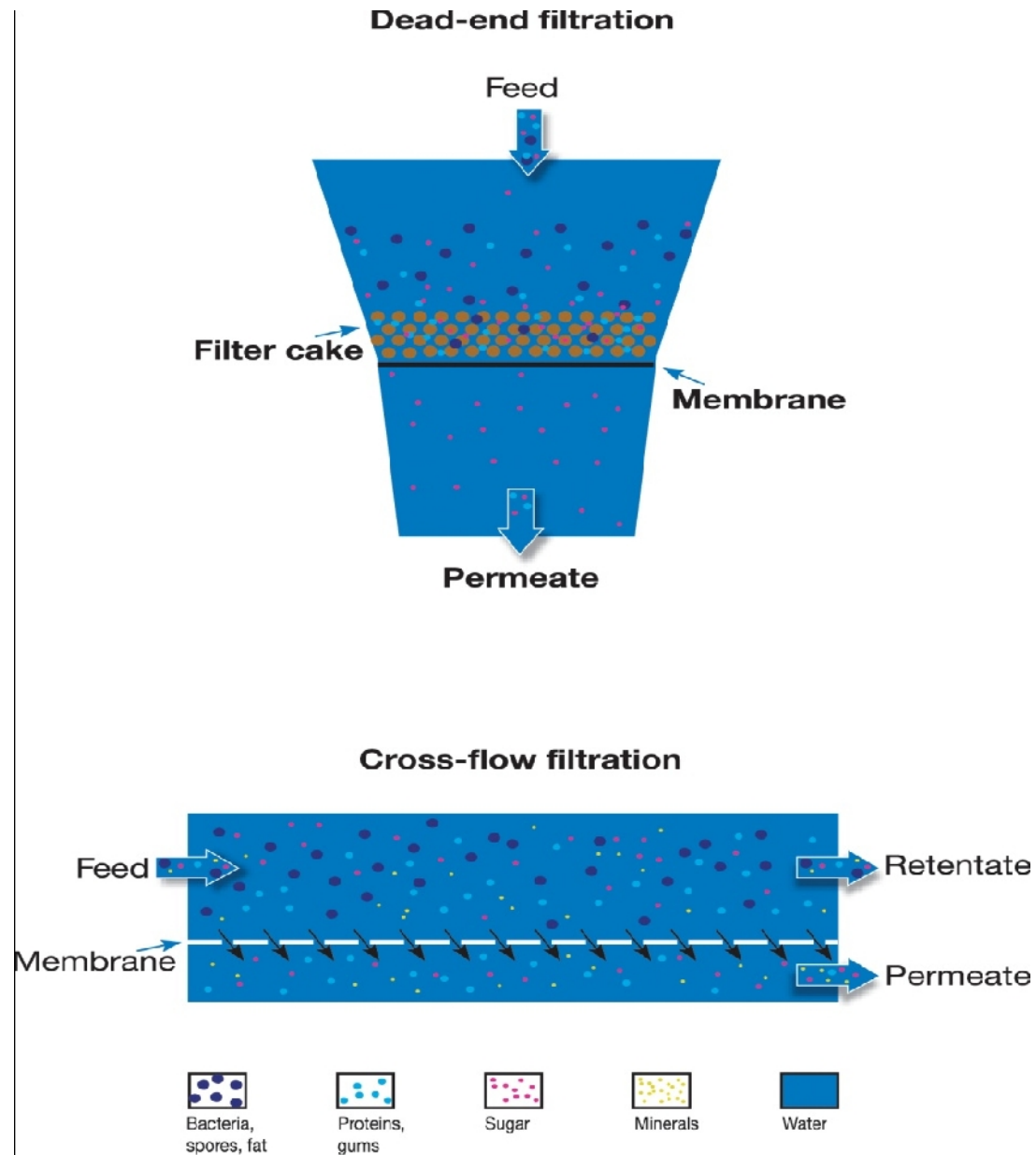
Plate and Frame Membrane

Plate and frame membranes were among the earliest configurations in the market. Plate and frame devices use flat membrane sheets with permeate collection between the sheets. The sheets are sealed around the edges but with a provision for permeate removal (usually by a tube). Several of these plates are stacked on top of each other and clamped together to form a module or cartridge. Many plate and frame systems are based on dead-end flow and are more subject to plugging. The most commercially significant application of the plate and frame design is in electrodialysis modules (or stacks), although some microfiltration units and one reverse osmosis module design are also based on this configuration.

Plate and Frame Membranes



Dead-End vs Crossflow Filtration



Microfiltration

Membranes with a pore size of 0.1 – 10 μm perform microfiltration. Microfiltration membranes remove all bacteria. Only part of the viral contamination is caught up in the process, even though viruses are smaller than the pores of a microfiltration membrane. This is because viruses can attach themselves to bacterial biofilm. Microfiltration can be implemented in many different water treatment processes when particles with a diameter greater than 0.1 mm need to be removed from a liquid.

Examples of micro filtration applications are:

- Cold sterilisation of beverages and pharmaceuticals**
- Clearing of fruit juice**
- Separation of bacteria from water (biological wastewater treatment)**
- Effluent treatment**
- Separation of oil/ water emulsions**
- Pre-treatment of water for nano filtration or Reverse Osmosis**
- Solid-liquid separation for pharmacies or food industries**

Ultrafiltration

For complete removal of viruses, ultra filtration is required. The pores of ultra filtration membranes can remove particles of $0.001 - 0.1 \mu\text{m}$ from fluids.

Examples of fields where ultra filtration is applied are:

- The dairy industry (milk, cheese)**
- The food industry (proteins)**
- The metal industry (oil/ water emulsions separation, paint treatment)**
- The textile industry**

Nanofiltration

Nanofiltration is a technique that has prospered over the past few years. Today, nanofiltration is mainly applied in drinking water purification process steps, such as water softening, decoloring and micro pollutant removal. During industrial processes nanofiltration is applied for the removal of specific components, such as coloring agents. Nanofiltration is a pressure related process, during which separation takes place, based on molecule size. Membranes bring about the separation. The technique is mainly applied for the removal of organic substances, such as micro pollutants and multivalent ions. Nanofiltration membranes have a moderate retention for univalent salts.

Other applications of nanofiltration are:

- **The removal of pesticides from groundwater**
- **The removal of heavy metals from wastewater**
- **Wastewater recycling in laundries**
- **Water softening**
- **Nitrates removal**

Reverse Osmosis

Reverse Osmosis is based upon the fundamental pursuit for balance. Two fluids containing different concentrations of dissolved solids that come in contact with each other will mix until the concentration is uniform. When these two fluids are separated by a semi permeable membrane (which lets the fluid flow through, while dissolved solids stay behind), a fluid containing a lower concentration will move through the membrane into the fluids containing a higher concentration of dissolved solids. After a while the water level will be higher on one side of the membrane. The difference in height is called the osmotic pressure. By pursuing pressure upon the fluid column, which exceeds the osmotic pressure, one will get a reversed effect. Fluids are pressed back through the membrane, while dissolved solids stay behind in the column. Using this technique, a larger part the salt content of the water can be removed.

Reverse Osmosis

The applications of Reverse Osmosis application are:

- Water softening
- Drinking water production
- Process water production
- Ultra pure water production (electronic industries)
- Concentration of molecular solvents for food and dairy industries

Feed Pre-Treatment

The pre-treatment of feed water for nanofiltration or Reverse Osmosis installations greatly influences the performance of the installation. The required form of pre-treatment depends on the feed water quality. The purpose of pre-treatment is reducing the organic matter content and the amount of bacteria, as well as lowering the MFI.

The organic matter content and the amounts of bacteria should be as low as possible to prevent the so-called biofouling of membranes. The application of a pre-treatment has several benefits:

- Membranes have a longer life-span when pre-treatment is performed
- The production time of the installation is extended
- The management tasks become simpler
- The employment costs are lower

Main Pre-treatment Techniques

This is an overview of the main pre-treatment techniques and the substances, which are reduced during these processes.

[illegible]

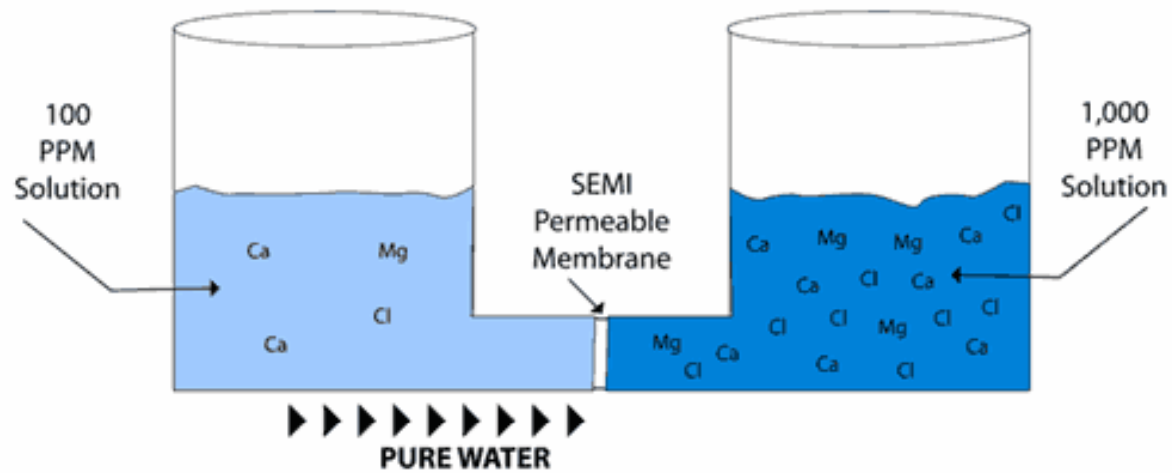
Table 3. Membrane configurations.

Configuration	Pleated cartridge	Plate-and-frame	Spiral-wound	Tubular	Hollow fibre
Area/volume ratio (m ² /m ³)	800 – 1000	400 – 600	800 – 1000	20 – 30	5000 – 40000
Cost	Low	High	Low	Very high	Very low
Turbulence promotion	Very poor	Fair	Poor	Very good	Very poor
Advantages	Robust construction, compact design	Can be dismantled for cleaning	Low energy costs, robust and compact	Easily mechanically cleaned, tolerates high TSS waters	Can be backflushed, compact design, tolerates high colloid levels
Disadvantages	Easily fouled, cannot be cleaned	Complicated design, cannot be backflushed	Not easily cleaned, cannot backflush	Replacement cost	Sensitive to pressure shocks
Applications	Dead end MF	ED, UF, RO	RO, UF	Cross-flow filtration, high TSS waters	MF, UF, RO

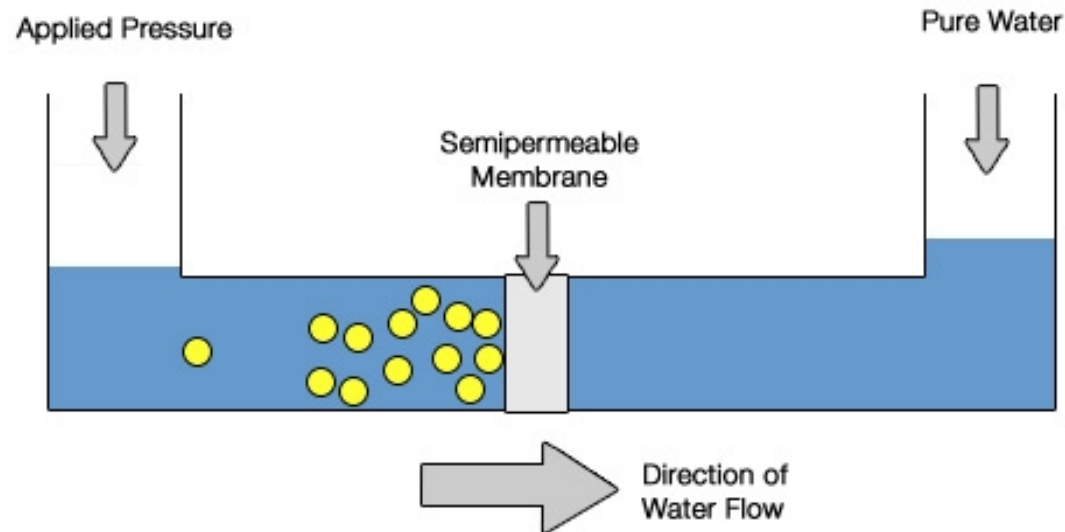
Type	Typical Applications	Industries	Energy Use (kWh/10 ³ gal)	By-Product Uses
MF	1) removal of coarser suspended solids, bacteria 2) pretreatment before NF & RO	Wide Applications	medium (15 - 25)	1) animal feed 2) recovery of valuable solids
UF	1) removing oil 2) recycling emulsions used in metal forming 3) recovery of caustic solutions or acid for reuse in cleaning systems 4) concentration of whey solids 5) juice and wine clarification 6) recycling of stack scrubber water 7) E-coat paint recovery 8) recycling of bacteria in membrane bio reactors 9) pretreatment before NF & RO	food processing, textile, metal working pharmaceutical pulp & paper, bio-industries	high (15 - 30)	1) animal feed 2) reuse of paint 3) recovery of valuable solids 4) recovery of metals 5) reuse of starches
NF	1) BOD reduction in sugar containing streams 2) separation of sugars 3) desalting of whey products 4) metal finishing acid recovery 5) dye removal from textile waste water 6) ethylene glycol reclamation	food processing, textile metal working pharmaceutical pulp & paper, bio-industries	high (15 - 30)	1) animal feed 2) recovery of valuable solids 3) reuse of sugars
RO	1) polishing evaporator condensate before reuse 2) preconcentration of fruit, tomato juices before evaporator 3) sugar recovery in candy mkg. 4) landfill leachate treatment 5) hardness, sulfates and nitrates removal 6) water softening 7) boiler feed water treatment	food processing, pharmaceutical, bio-industry, electronic industries	high (20 - 40)	1) animal feed 2) recovery of valuable solids 3) reuse of sugars

Table 1: Typical Applications of Membrane Technology.

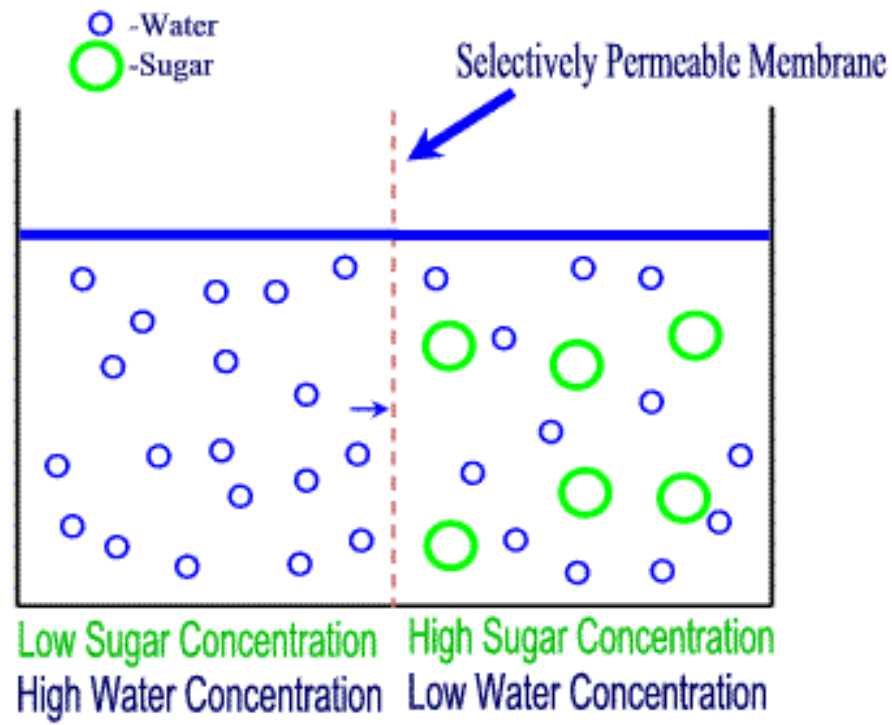
Osmosis



Reverse Osmosis



Osmosis



Factors affecting solute rejection in RO

- functional groups present in the membrane
- nature of membrane surface
- size of solute molecule
- pH
- pressure of other solutes

Types of Membranes

- Cellulose Acetate
- polyethylene Amine (PA-100)
- Polyether Amine (PA – 300)
- Film Tec (FT – 30) [meta-phenylene diamine + trimesoyl chloride]
- NS – 200 (2-hydroxyl-methyl furan + H₂SO₄)
- PEC – 1000 (2-hydroxyl-methyl furan)

Membrane Performance is measured by:

- Flux
- Quality of product

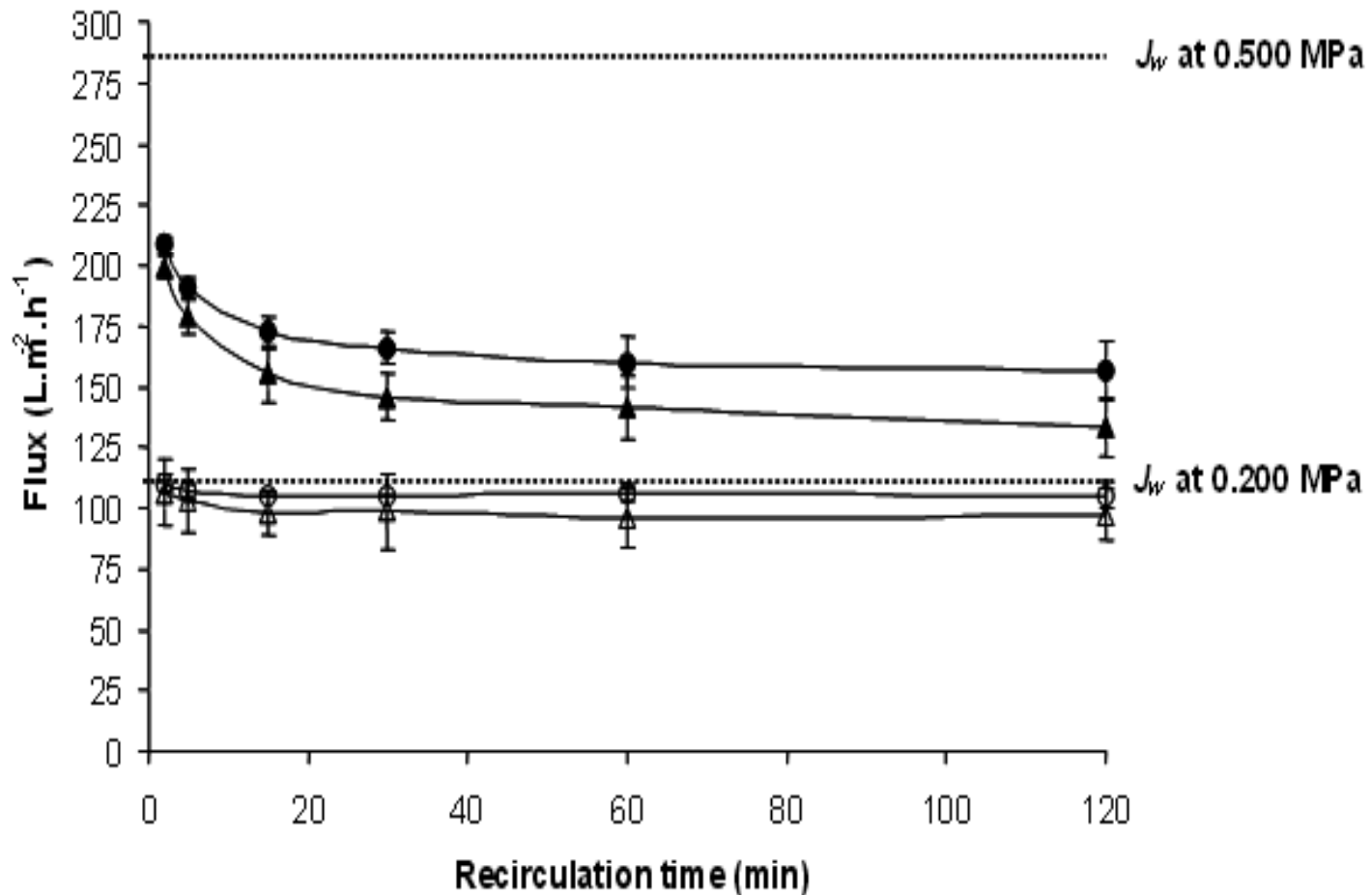
Flux is a function of:

- Membrane thickness
- Chemical composition of feed
- Membrane porosity
- Time of operation
- Pressure across the membrane
- Water temperature

Quality of product is a function of:

- rejection ability of the membrane

Flux Decline



The general trend of flux curve can be represented by:

$$F = Kt^m$$

$$\ln(F) = m \ln(t) + \ln K$$

Where

F = flux

t = time

K = constant

This is a straight-line equation, so, at least two data points are necessary to calculate m and K.

Flux as a Function of Pressure

$$F = \frac{1}{\mu R_m} (-\Delta P)$$

Where,

R_m = resistance due to membrane

μ = viscosity

ΔP = transmembrane pressure

Flux as a Function of Temperature

Since F is a function of μ and μ is a function of temperature. Therefore, F is a function of temperature.

Percent Solute Rejection or Removal

This is a very important parameter in designing an RO process

$$R = \frac{\sum Q_0 [C_{oi}] - \sum Q_p [C_{pi}]}{\sum Q_0 [C_{oi}]} \times 100$$

$$R = \frac{\sum Q_c [C_{ci}]}{\sum Q_0 [C_{oi}]} \times 100$$

Where,

R = percent rejection

Q_0 = feed flow rate

$[C_0]$ = feed concentration of solutes

Q_p = permeate flow rate

$[C_p]$ = permeate concentration of solutes

Q_c = concentrate flow rate

$[C_c]$ = concentrate concentration of solutes

i refers to the i^{th} solute species

Fouling of Membrane

- **Fouling** of a membrane **increase resistance** to flow and reduces the flux of water through a membrane
- **Backwashing** or **chemical treatment** may be applied to remove **foulants**.
- **Irreversible fouling** of membranes is the most serious problems.
- **Oxidizing agents** such as chlorine or ozone attack membranes and change their structures.

5. DISINFECTION

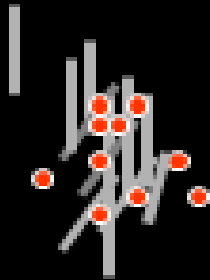
Purpose and Types

- Treatment used for destruction or removal of pathogens
- Widely used disinfectants
 - Oxidizing agents (halogens, halogen compounds, ozone)
 - Physical agents (Ultraviolet radiation)
- Factors that affect efficiency of disinfection
 - Type and concentration of microorganisms
 - Type and concentration of disinfectant
 - Contact time provided
 - Character and temperature of the water

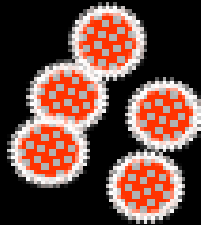
Some diseases associated with contaminated water

Disease	Symptoms
• Viruses ○ Hepatitis ○ Poliomyelitis Bacteria ○ Cholera ○ Gastroenteritis ○ Shigellosis (Bacillary dysentery) • Protozoa ○ amebiasis (amoebic dysentery) ○ Giardiasis ○ Cryptosporidiosis	✓ Fever, nausea, loss of appetite, fatigue, jaundice ✓ Headaches, nausea, vomiting, stiff neck, nasal voice, runny nose, difficulty in swallowing ✓ Extremely heavy diarrhea, rice-water stool, vomiting, thirst, pain, dehydration, high death rate ✓ Abdominal discomfort, diarrhea, nausea, indigestion, vomiting, cramps, fever ✓ Diarrhea, fever, mucus and blood in stool ✓ Prolonged diarrhea, abdominal discomfort, blood and mucus in stool, abscesses of liver and small intestine ✓ Mild to severe diarrhea, nausea, indigestion, flatulence ✓ Diarrhea, abdominal cramps, fever, nausea, vomiting

Bacteria
(*E.coli*)



Viruses
(Hepatitis, Polio)

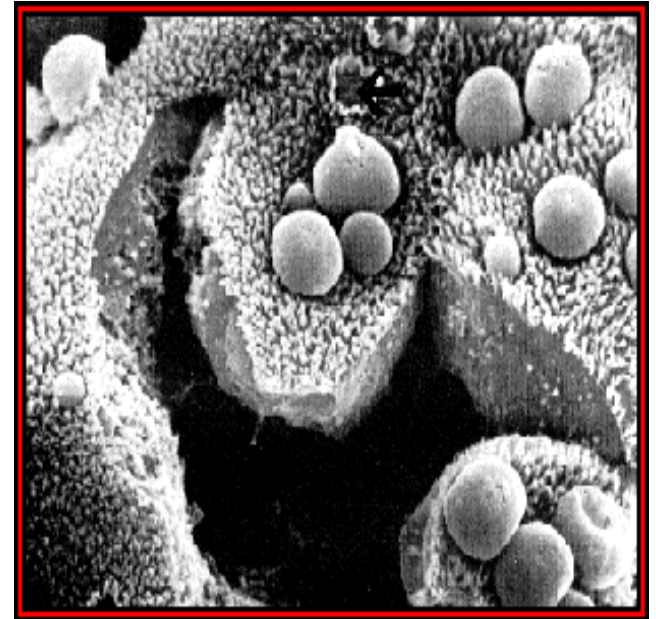


Protozoa

(*Giardia*)



(*Cryptosporidium*)



Objectives of Disinfection

- Assure 99.9% (3-log) and 99.99% (4-log) inactivation of *Giardia lamblia* cysts and viruses, respectively
- Do not impart toxicity and taste and odor to the disinfected water
- Minimize the formation of undesired disinfection by-products
- Meet the maximum contaminant levels for the disinfectants used and the by-products that can form

Characteristics of an Ideal Disinfectant

- Kill or disable pathogens
- Nontoxic to consumers
- Cheap and easy to use
- Fast acting and long lasting
- Improve water aesthetics

Disinfectants in common use

- Chlorine (Cl_2)
- Chloramines (NH_2Cl , NHCl_2)
- Chlorine dioxide (ClO_2)
- Ozone (O_3)
- Ultraviolet (UV) radiation.

Disinfection Kinetics

Chick's Law:

$$\frac{dN}{dt} = -kN$$

Rearranging and integrating,

$$N = N_0 e^{-kt}$$

Where,

N = number of microbes at any time t

N₀ = initial number of microbes

The concept of Ct and Log-inactivation

- In disinfection, the ability to inactivate microorganisms is a function of both disinfectant concentration and time (Ct).

$$\text{Fractional removal, } N/N_0 = C^n t^m$$

This is generally simplified to

$$N/N_0 = Ct$$

- Log inactivation: the number or percent of microorganisms inactivated through the disinfection process
 - 1 log = 90% inactivation
 - 2 log = 99% inactivation
 - 3 log = 99.9 inactivation
 - 4 log = 99.99 inactivation

Table 11.2 *Ct* values for achieving 1-log inactivation of *Giardia Lamblia*

Disinfectant	pH	<i>Ct</i> values at water temperature					
		0.5 °C	5 °C	10 °C	15 °C	20 °C	25 °C
Free residual chlorine of 2 mg/l	6	49	39	29	19	15	10
	7	70	55	41	28	21	14
	8	101	81	61	41	30	20
	9	146	118	88	59	44	29
Ozone	6–9	0.97	0.63	0.48	0.32	0.24	0.16
Chlorine dioxide	6–9	21	8.7	7.7	6.3	5	3.7
Chloramines	6–9	1270	735	615	500	370	250

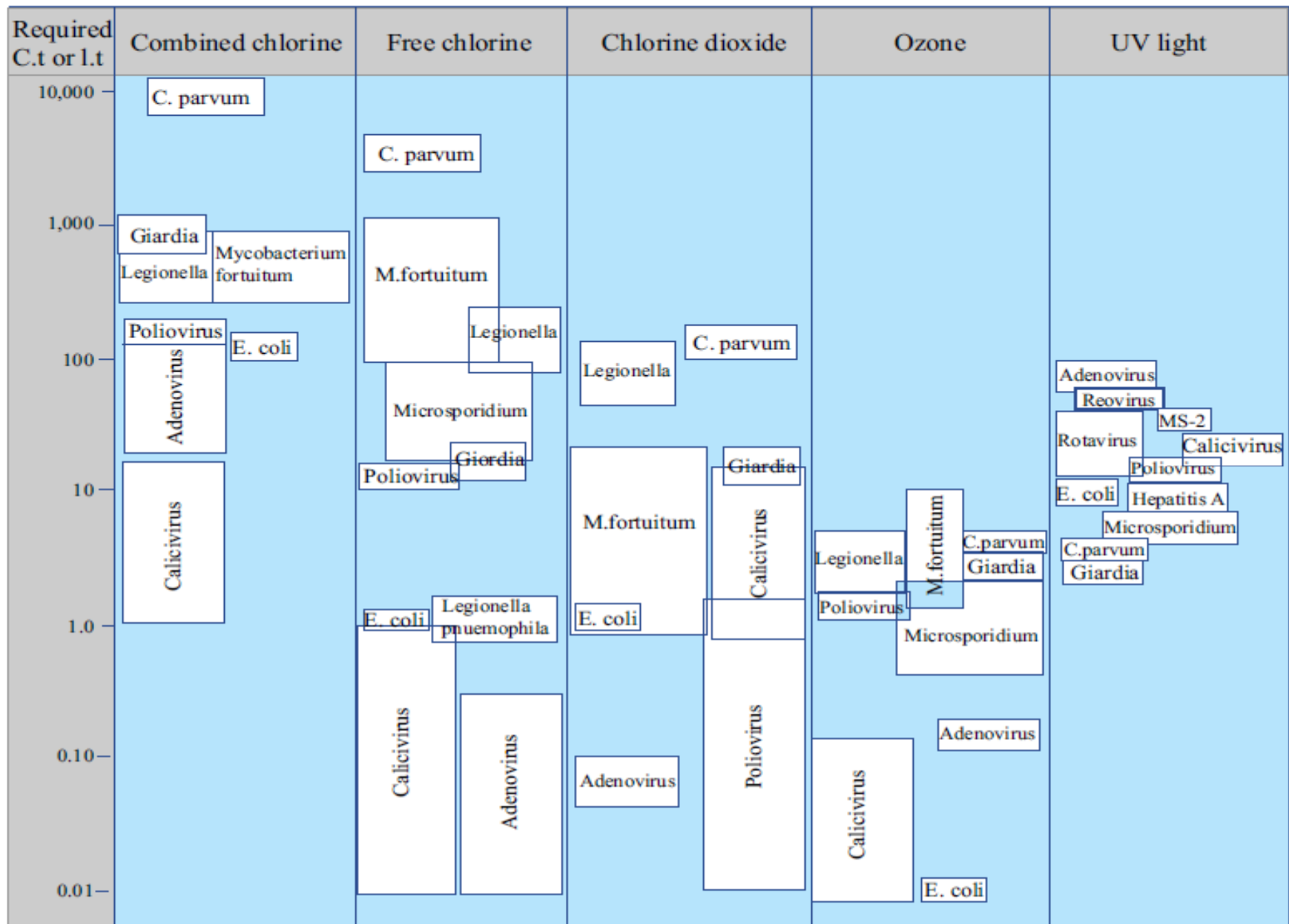
Source of information: Surface Water Treatment Rule (SWTR) (US EPA, 1991)

Table 11.3 *Ct* values for achieving 2- and 3-log inactivation of enteric viruses at pH values 6–9

Disinfectant	Log inactivation	<i>Ct</i> values at water temperature					
		0.5 °C	5 °C	10 °C	15 °C	20 °C	25 °C
Free residual chlorine	2	6	4	3	2	1	1
	3	9	6	4	3	2	1
Ozone	2	0.9	0.6	0.5	0.3	0.25	0.15
	3	1.4	0.9	0.8	0.5	0.4	0.25
Chlorine dioxide	2	8.4	5.6	4.2	2.8	2.1	1.4
	3	25.6	17.1	12.8	8.6	6.4	4.3
Chloramines	2	1243	857	643	428	321	214
	3	2063	1423	1067	712	534	356

Source of information: Surface Water Treatment Rule (SWTR) (US EPA, 1991)

Overview of disinfection requirements for 99 percent inactivation



Chlorination

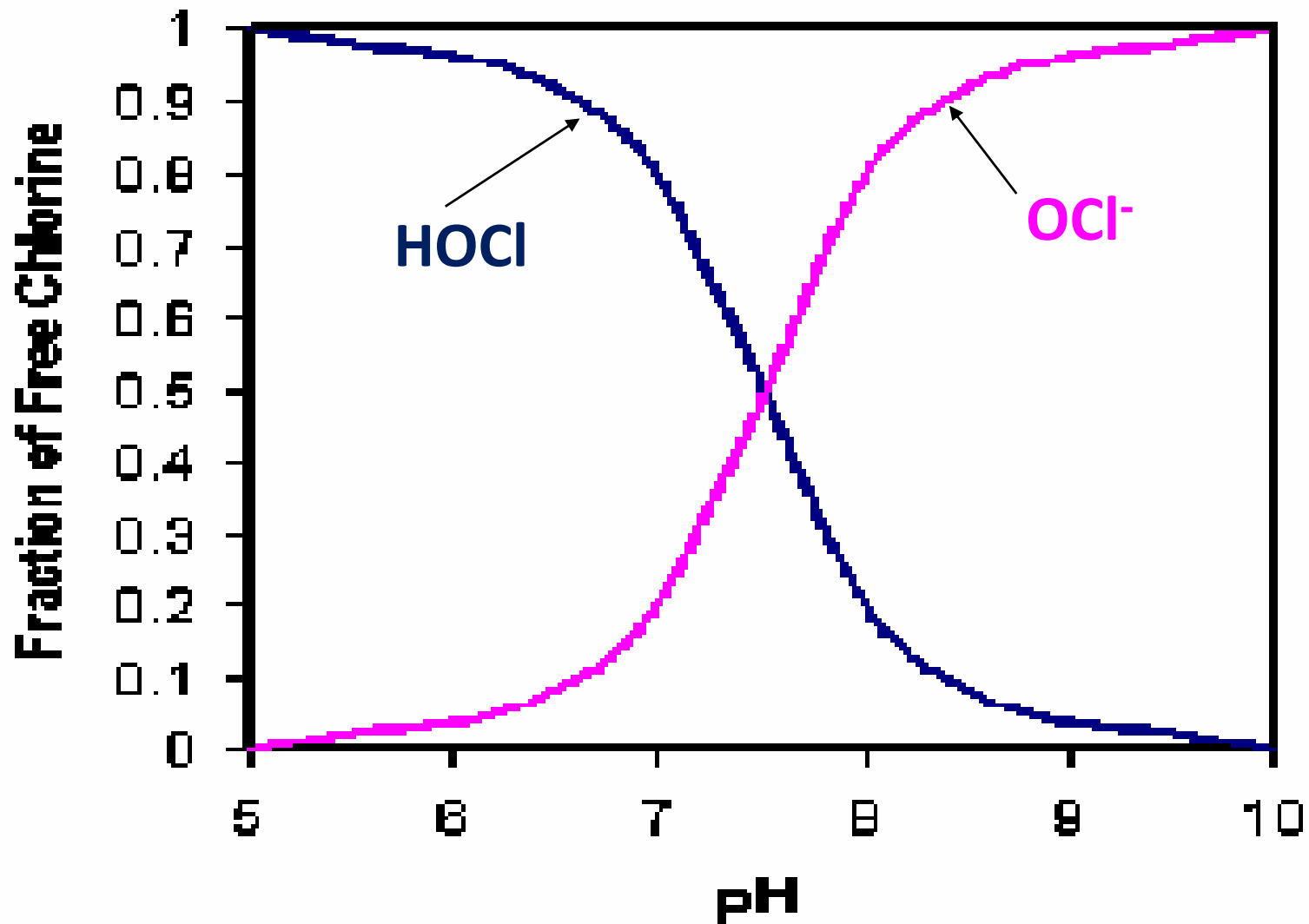
- Widely used disinfectant
- Advantages: readily available as gas, liquid or solid; cheap; easy to apply; toxic to most microorganisms; provides protection in the distribution system
- Disadvantages: Fatal at high concentrations; Toxic; Irritant; Corrosive; Formation of harmful disinfection by-products (DBPs) like Trihalomethanes (THMs)

- Reactions:



- Hypochlorous acid (HOCl) and Hypochlorite ion (OCl⁻) are known as “free chlorines” and are the most effective forms of chlorine.
- HOCl is 80-200 times as strong as OCl⁻.

Relative concentrations of HOCl and OCl⁻

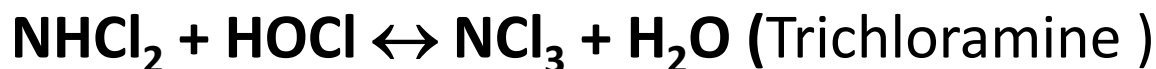
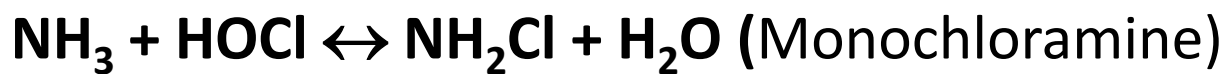


Other substances with which chlorine reacts

- Reducing agents such as S^{2-} , Fe^{2+} , Mn^{2+} , NO^{2-}



- Organic matter
- Ammonia: HOCl reacts with ammonia to produce chloramines which are known as “combined chlorine residuals”



Free residual chlorine = $Cl_{2(l)} + HOCl + OCl$

Combined residual chlorine = $NH_2Cl + NHCl_2 + NCl_3$

Total residual chlorine = Free residual chlorine + Combined residual chlorine

Chloramines

- Advantages
 - Less corrosive
 - Less toxicity and chemical hazards
 - Relatively tolerable to inorganic and organic loads
 - No known formation of DBP
 - Relatively long-lasting residuals
- Disadvantages
 - Not so effective against viruses, protozoan cysts, and bacterial spores

Chlorine gas cylinder



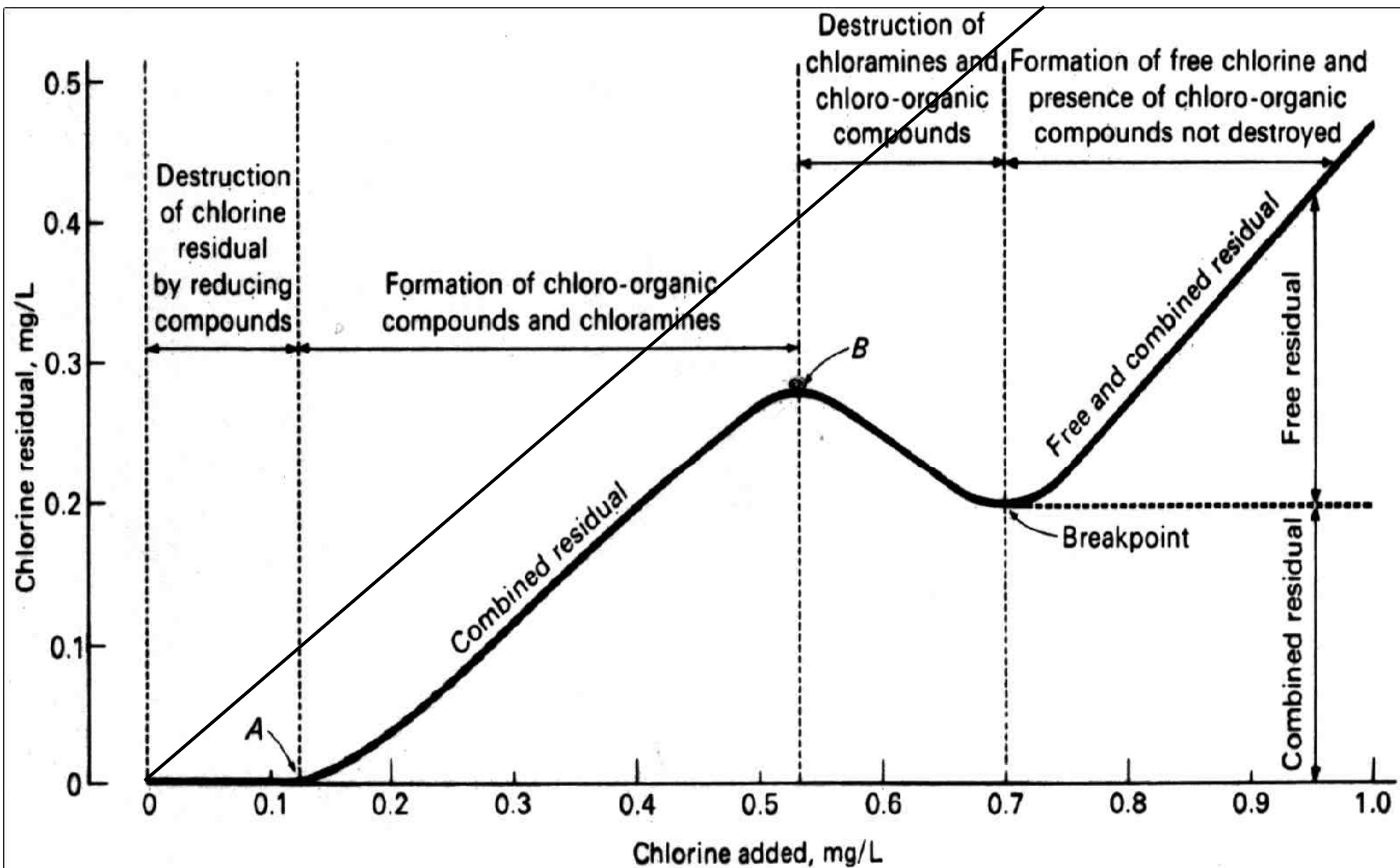
Hypochlorites

- Less pure and dangerous than chlorine gas
- Strength decreases with time while in storage
- Safer to handle but expensive
- Types:
 - Sodium hypochlorite (NaOCl) solution
 - Calcium hypochlorite (CaO(Cl)₂)

- Reactions:

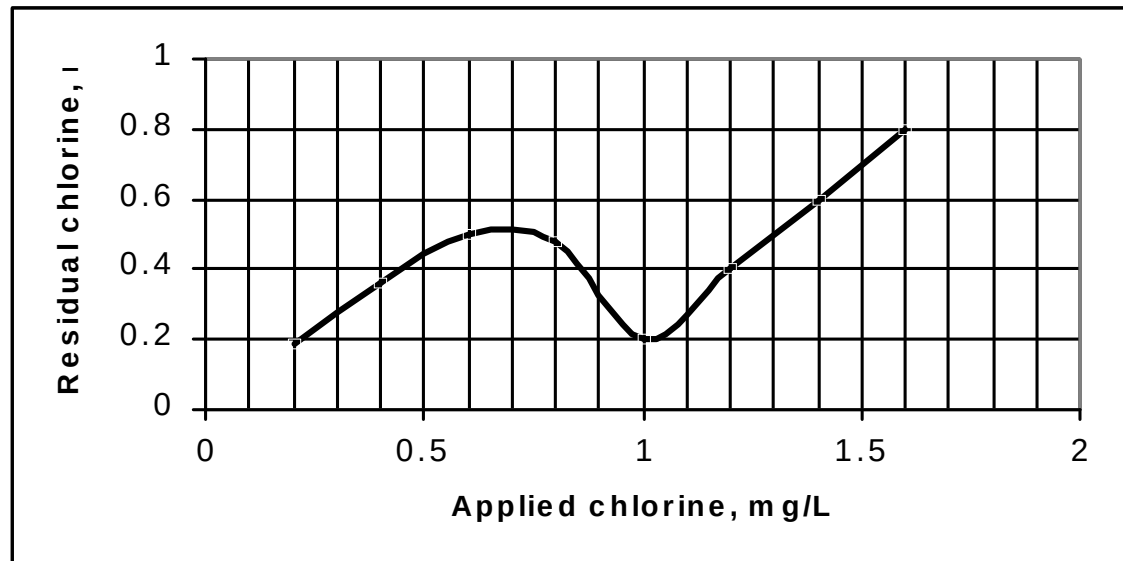


Breakpoint Chlorination



Example 1

Referring to the breakpoint-chlorination curve shown below, calculate the daily chlorine requirement in kg to attain a 0.4 mg/L free chlorine residual. The design discharge is 7500 m³/d.



Example 2

Disinfection of the groundwater from a well field is required for virus inactivation of 99.9 percent (3-log). The proximity of the river that recharges the well field does not provide adequate natural disinfection based on site evaluation. The transmission main from the well field to the first customer is 3.0 km, the velocity of flow at peak pumping capacity is 1 m/s, water temperature is 5°C, and the pH is 7.5. If chlorine is applied at the well field, is disinfection adequate before the water arrives at the first customer?

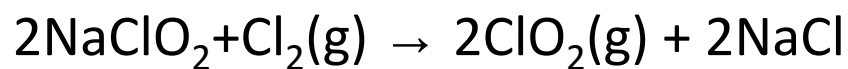
Ozonation

- strong oxidant but no residual
- no trihalomethanes but other (non-chlorinated) DBPs possible
- often used as a primary disinfectant



Chlorine Dioxide

- strong oxidant, but not as powerful as ozone
- dose limited to 1.0 mg/L due to health concerns of chlorite and chlorate
- residual is not long lasting
- Produced on site:



OR



Advantages and Disadvantages of ClO₂

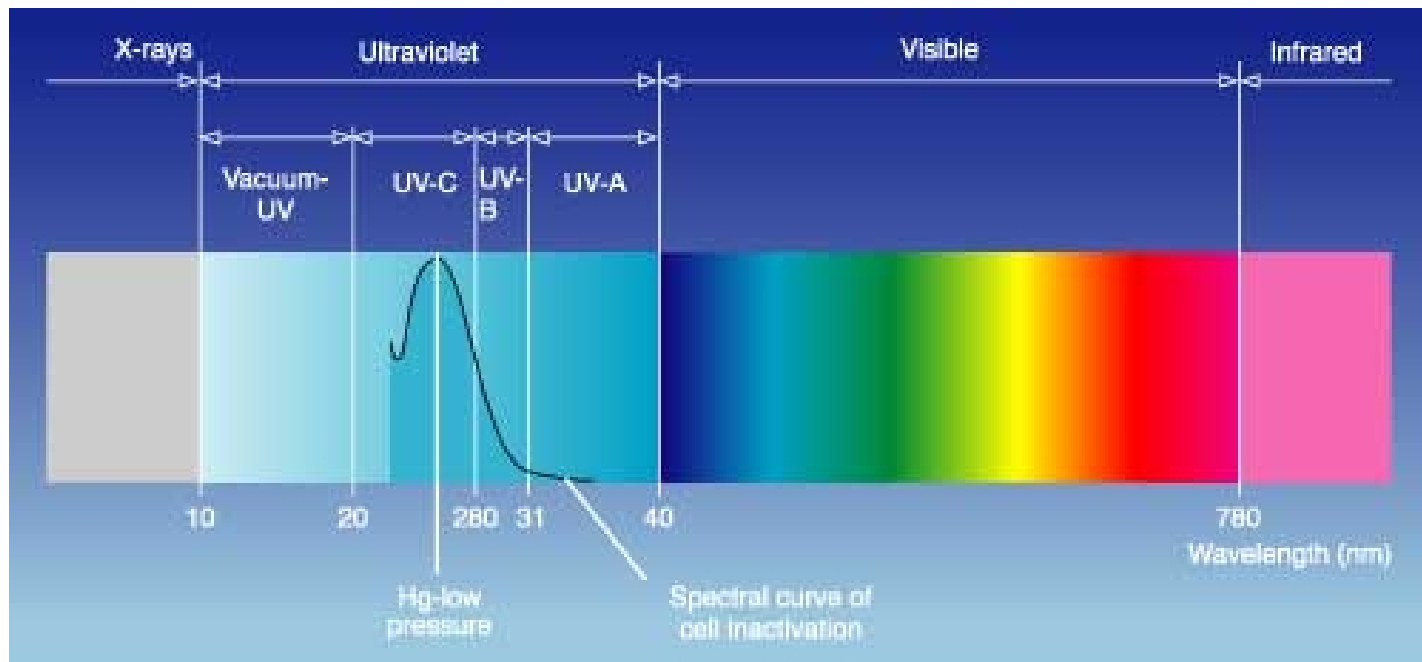
Advantages	Disadvantages
• Powerful bactericide and viricide • No reaction with ammonia nitrogen • No reaction with organic compounds to form THMs • Reactions to destroy 30-40% of THM precursors • Efficiency of disinfection relatively unaffected by pH levels between 6 and 10	• High chemical costs • Must be generated on site • Possibility of health hazards from by-products of production • Pronounced metallic taste in treated water at high dosages • Lack of reliable and practical techniques for evaluation of chlorine dioxide from other chlorine compounds

UV Disinfection

- Physical method of inactivating pathogens.
- Mechanism of UV Disinfection:
 - Radiation with an wavelength of around 260 nm penetrates the cell wall and cell membrane of microorganisms and is absorbed by cell material such as DNA and RNA and promotes changes that prevents replication to occur

UV Disinfection - Mechanisms

- Emissions from Low Pressure Hg lamps as compared to spectral curve for cell inactivation. Maximum of absorption (265 nm) coincides to wavelength of radiation emitted (254 nm).



UV Disinfection - The Concept of Dosage

- For chlorine disinfection, presented the concept of Ct product as an indication of disinfecting potential.
- For UV disinfection, a similar quantity is UV dosage:

$$D = I \times t$$

$$D = \text{UV Dosage (mW} \bullet \text{s/cm}^2\text{)}$$

$$I = \text{Radiation Intensity (mW)}$$

$$t = \text{Contact Time (seconds)}$$

UV Disinfection

- Advantages of UV disinfection over chlorine disinfection:
 - No by-products of disinfection are known to be formed. With chlorine, there is strong potential for the formation of trihalomethanes (THM).
 - Short detention times: UV disinfection requires a six-to-10-second contact time, compared to a 15-to-30-minute contact time for chlorine.
 - UV disinfection presents no dangers in terms of handling chemicals . Chlorine highly toxic to humans.

6. MISCELLANEOUS TREATMENT PROCESSES

6.1 Softening

The presence of multivalent cations, particularly calcium and magnesium, is referred to as water hardness.

Carbonate (temporary) hardness causing compounds	Non-carbonate (permanent) hardness causing compounds
Calcium carbonate (CaCO_3)	Calcium sulfate (CaSO_4)
Magnesium carbonate (MgCO_3)	Magnesium sulfate (MgSO_4)
Calcium bicarbonate ($\text{Ca}(\text{HCO}_3)_2$)	Calcium chloride (CaCl_2)
Magnesium bicarbonate ($\text{Mg}(\text{HCO}_3)_2$)	Magnesium chloride (MgCl_2)
Calcium hydroxide ($\text{Ca}(\text{OH})_2$)	
Magnesium hydroxide ($\text{Mg}(\text{OH})_2$)	

Carbonate hardness can be removed by boiling.

Total hardness = Carbonate hardness + Non-carbonate hardness

Conventional units for hardness

- Milliequivalent per liter (meq/L)

$$\text{meq/L of X} = \frac{\text{concentration of X (mg/L)}}{\text{Equivalent weight of X (mg/meq)}}$$

- mg/L as CaCO_3

$$\text{mg/L of X as CaCO}_3 = \frac{\text{concentration of X (mg/L)} \times 50.0 \text{ mg CaCO}_3/\text{meq}}{\text{Equivalent weight of X (mg/meq)}}$$

Hardness Classification of Water

Degree of hardness	mg/L as CaCO_3	meq/L
Soft	< 50	< 1
Moderately hard	50-150	1-3
Hard	150-300	3-6
Very hard	> 300	> 6

Problems associated with hardness:

- Results in use of excessive soap and detergents
- May have taste and odor problems
- Shorten the life of fabrics when washed with them
- Leads to scale formation

Alkalinity

$$\text{Alkalinity (meq/L)} = (\text{HCO}_3^-) + (\text{CO}_3^{2-}) + (\text{OH}^-) - (\text{H}^+)$$

$$\text{Alkalinity (meq/L)} = (\text{HCO}_3^-) \text{ for } 6 \leq \text{pH} \leq 8.5$$

$$\text{Total hardness (TH)} = \text{Ca}^{2+} + \text{Mg}^{2+}$$

$$\text{TH} = \text{Carbonate Hardness (CH)} + \text{Noncarbonate hardness (NCH)}$$

If $(\text{HCO}_3^-) + (\text{CO}_3^{2-}) \geq \text{TH}$, then $\text{CH} = \text{TH}$ and $\text{NCH} = 0$

If $\text{Alkalinity} < \text{TH}$, $\text{CH} = \text{Alkalinity}$ and $\text{NCH} = \text{TH} - \text{CH}$

Example 1

The analysis of a sample of water with pH 7.5 has produced the concentrations (mg/L) shown below. Find the total hardness, the carbonate hardness, the noncarbonate hardness, and the alkalinity as CaCO_3 .

Cations		Anions	
Ca^{2+}	80	Cl^-	100
Mg^{2+}	30	SO_4^{2-}	201
Na^+	72	HCO_3^-	165
K^+	6		

Solution

Ion	mg/L	mg/meq	mg/L as CaCO ₃
Ca ²⁺	80	20.0	200.0
Mg ²⁺	30	12.2	123.0
Na ⁺	72	23.0	156.5
K ⁺	6	39.1	7.7
Cl ⁻	100	35.5	140.8
SO ₄ ²⁻	201	48.0	209.4
HCO ₃ ⁻	165	61.0	135.2

Check on the accuracy of the chemical analysis

$\Sigma \text{Cations} = 487.2 \text{ mg/L as CaCO}_3$ $\Sigma \text{Anions} = 485.4 \text{ mg/L as CaCO}_3$ OK!

Answers:

TH = 323 mg/L as CaCO₃

CH = 135.2 mg/L as CaCO₃; Ca-CH = 135.2 mg/L as CaCO₃

NCH = 187.8 mg/L as CaCO₃; Ca-NCH = 64.8 mg/L as CaCO₃;

Mg-NCH = 123.0 mg/L as CaCO₃

Alkalinity = (HCO₃⁻) = 135.2 mg/L as CaCO₃

Softening

- Softening is removal of hardness from water
- Water softening is needed when hardness is above 150-200 mg/L as CaCO_3
- Can be achieved through lime-soda softening or ion exchange

Lime-Soda Softening

Goal is to change the soluble calcium and magnesium compounds to calcium carbonate and magnesium hydroxide precipitates

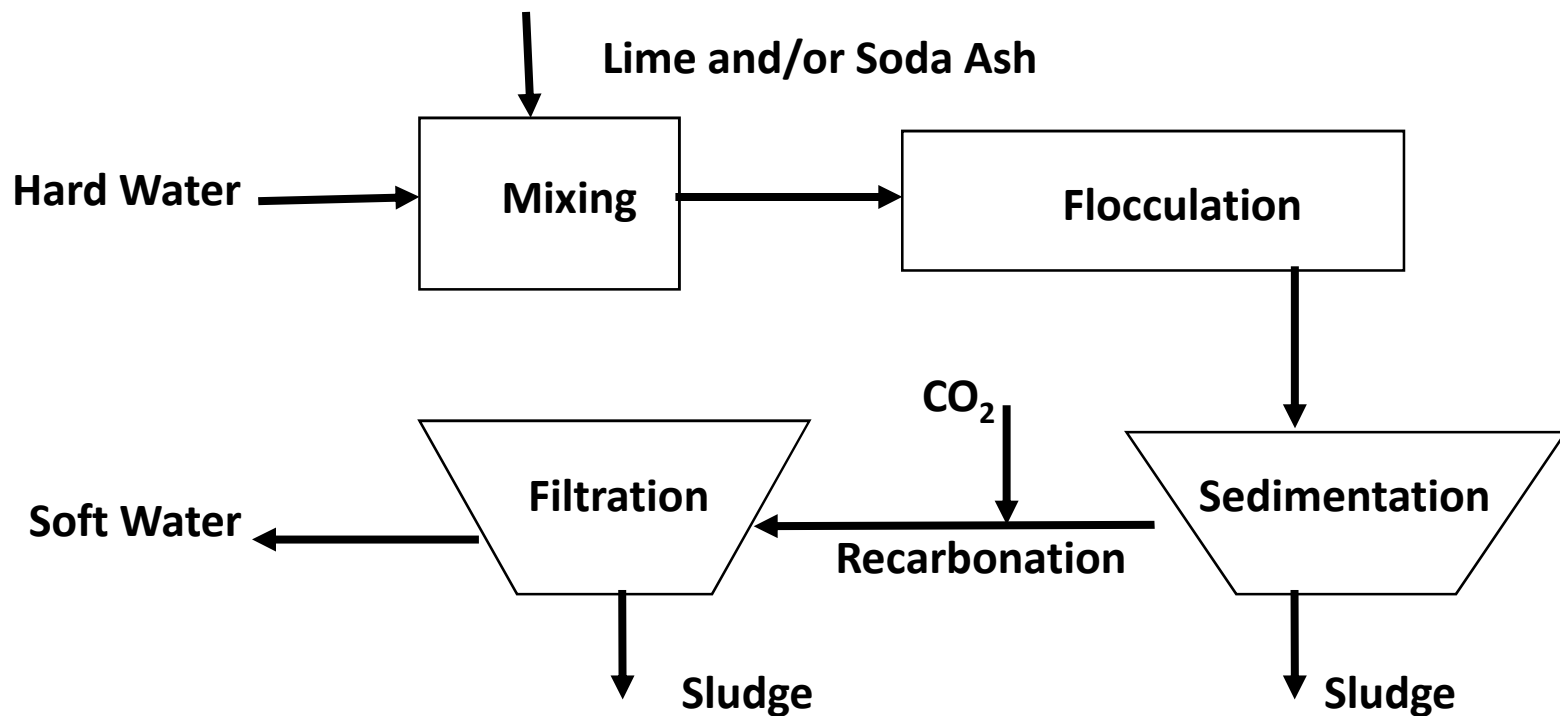


Figure 7-21 Schematic flow diagram for a two-stage excess lime softening plant.

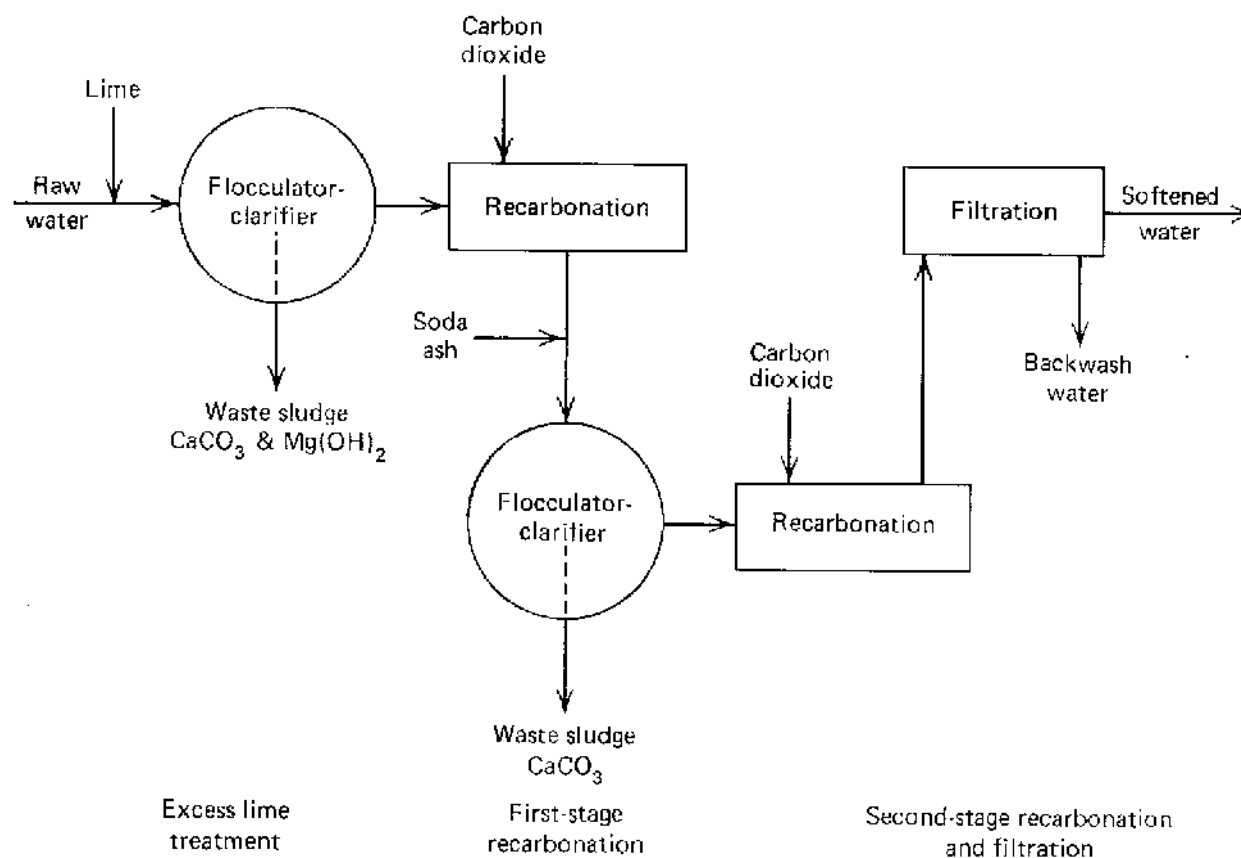
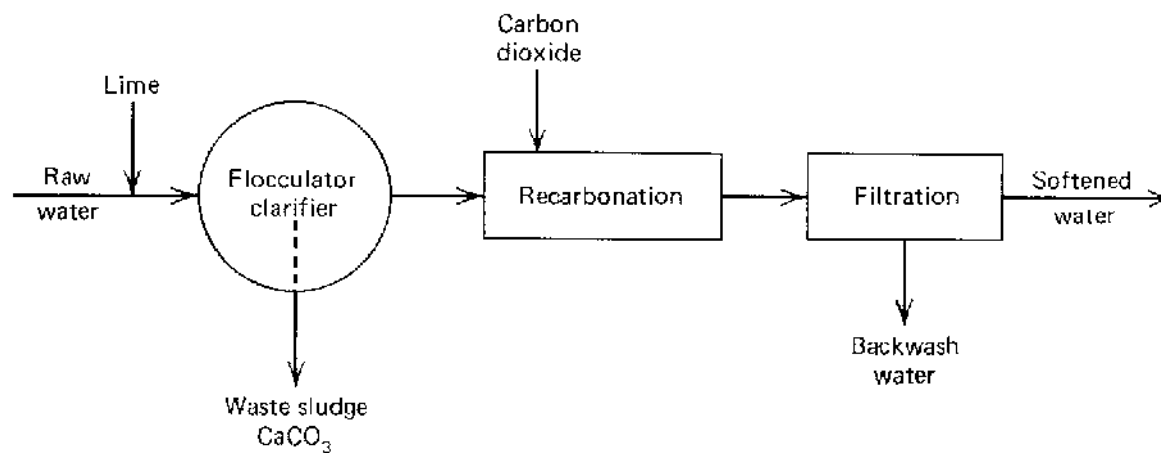


Figure 7-22 Schematic flow diagram for a single-stage calcium carbonate softening plant.



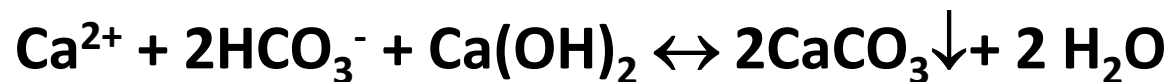
Softening chemistry

- Higher pH value is required 9.0-9.5 for calcium compounds and 10.0-10.5 for magnesium compounds

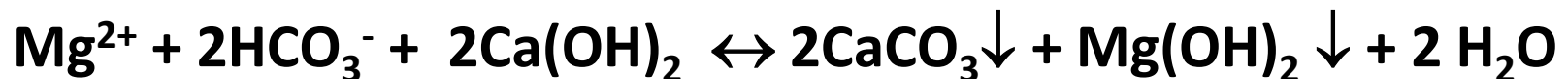
- Removal of $\text{CO}_{2(\text{aq})}$:



- Removal of Calcium carbonate hardness:

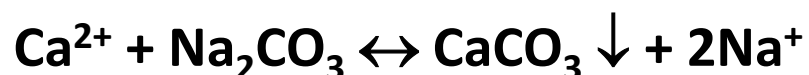


- Removal of Magnesium carbonate hardness:

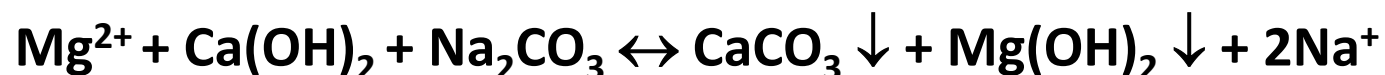


Softening chemistry

- Removal of Calcium noncarbonate hardness:



- Removal of Magnesium noncarbonate hardness:

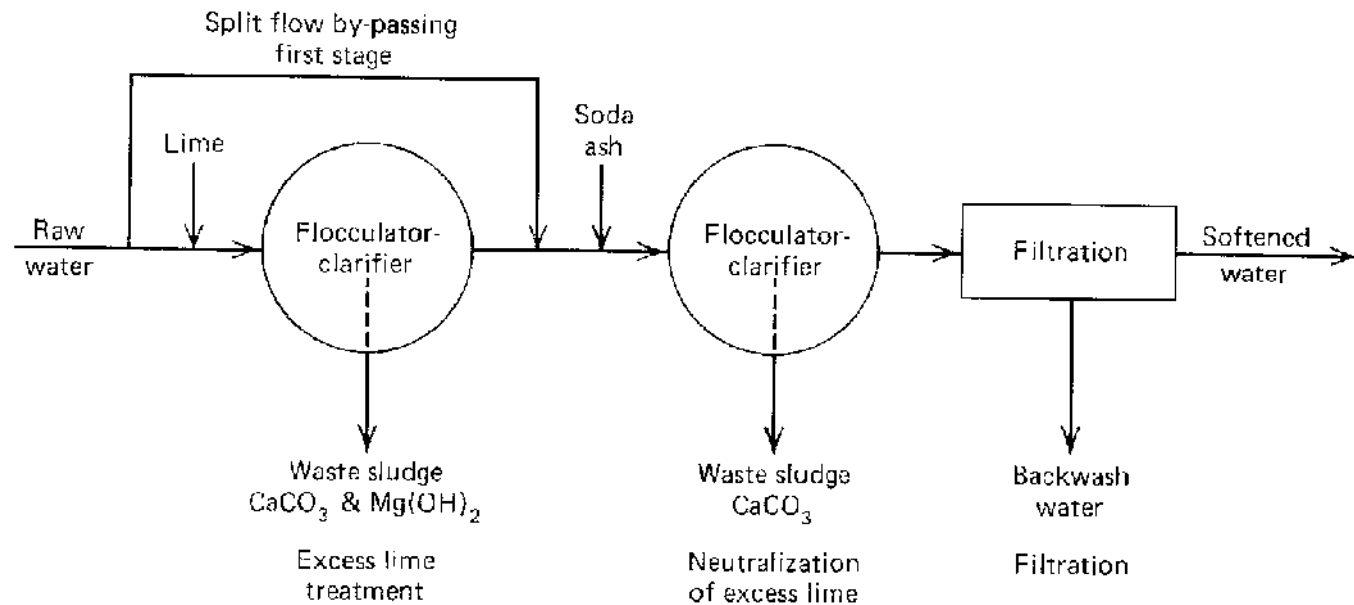


- In practice, excess lime (20-40 mg/L as CaCO_3) has to be added to maintain elevated pH and good softening.
- The practical lower limits for removal of calcium and magnesium using lime-soda softening processes are 30 mg/L as CaCO_3 and 10 mg/L as CaCO_3 , respectively.

Split-Treatment Softening

- Split-treatment softening is done by splitting the flow of the raw water into two lines, of different portions, for softening in a two-stage system
- the larger portion is given excess lime treatment in the first stage
- water is flocculated and settled
- treated water is mixed with split flow
- excess lime from the first stage reacts with calcium hardness of the split water
- soda ash is added to the second stage
- in this case, excess lime is used and not wasted, so recarbonation might not be necessary.
- Recarbonation is recommended to produce stable water
- Advantages:
 - lime and recarbonation costs are lower than excess lime treatment
 - possibility of reducing magnesium hardness to less than 40 mg/l

Figure 7-23 Schematic flow diagram for a split-treatment softening plant.



Bypass flow fraction

$$X = \frac{(Mg)_e - (Mg)_t}{(Mg)_r - (Mg)_t}$$

Where

X = Fraction of total flow to be bypassed

(Mg)e = Desired magnesium concentration in plant effluent (mg/L as CaCO₃)

(Mg)t = Magnesium concentration in stream from lime treatment process (mg/L as CaCO₃)

(Mg)r = Magnesium concentration in source water (mg/L as CaCO₃)

Example 2

A water treatment plant treats 50, 000 m³/d of the water in Example 1 to its maximum softness before distribution.

- How much lime and soda-ash in mg/L as CaCO₃ must the utility add?
- What mass of softening sludge will the plant generate daily?

Answers:

TH = 323 mg/L as CaCO₃

CH = 135.2 mg/L as CaCO₃ ; Ca-CH = 135.2 mg/L as CaCO₃

NCH = 187.8 mg/L as CaCO₃ ; Ca-NCH = 64.8mg/L as CaCO₃ ;

Mg-NCH = 123.0 mg/L as CaCO₃

Alkalinity = (HCO₃⁻) = 135.2 mg/L as CaCO₃

Solution

- 1st determine the amount of CO_{2(aq)} that must be neutralized

$$\frac{[H^+][HCO_3^-]}{[CO_{2(aq)}]} = K_1 = 4.47 \times 10^{-7} \text{ mol/L}$$

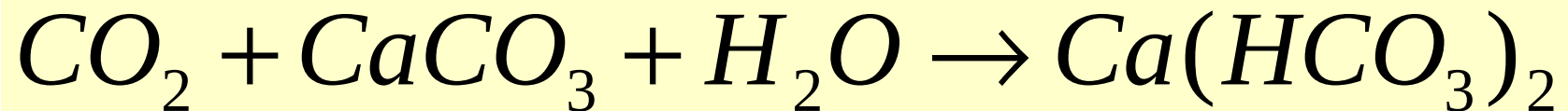
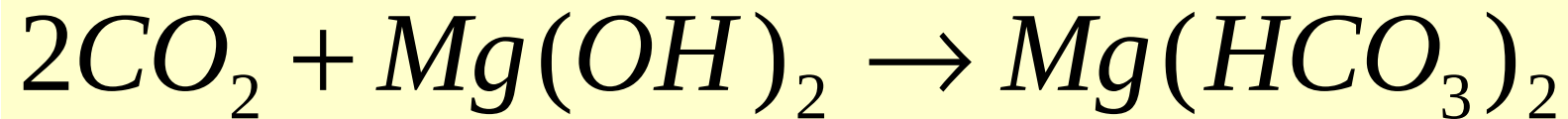
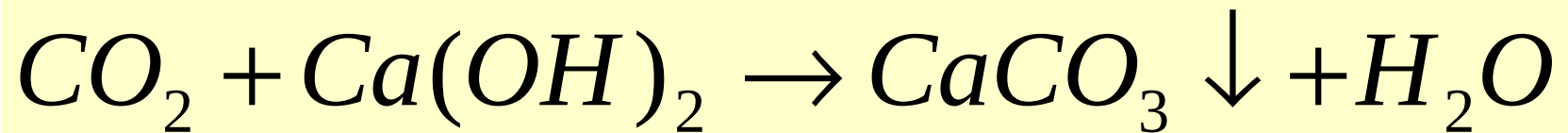
From which, CO_{2(aq)} = 1.91 x 10⁻⁴ M = 19.14 mg/L as CaCO₃

Component	Concentration , mg/L as CaCO ₃				
	Analysis	Lime	Soda-ash	CaCO ₃	Mg(OH) ₂
CO ₂ (aq)	19.1	19.1	0	19.1	0
Ca-CH	135	135	0	270	0
Mg-CH	0	0	0	0	0
Ca-NCH	64.8	0	64.8	64.8	0
Mg-NCH	123	123	123	123	123
Excess		20			
Total		297	188	477	123

Recarbonation

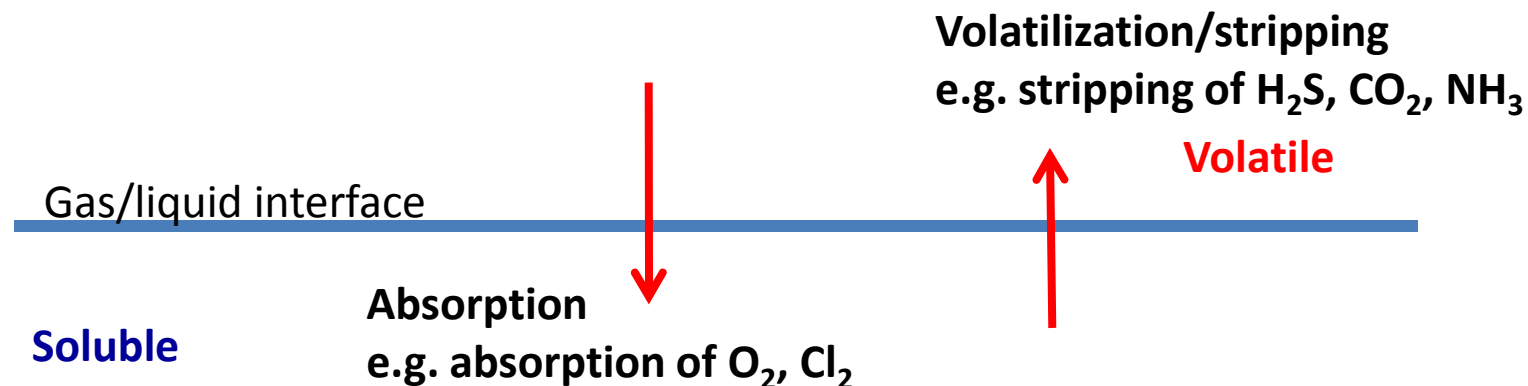
- After softening, the water is at high pH and too corrosive to distribute without pH adjustment
- Recarbonation is done to stabilize (to lower the pH) of the water so that scale formation and corrosion problem are minimized
- After recarbonation the water is filtered to remove residual turbidity and particles that form during recarbonation

Recarbonation Reactions



6.2 Aeration and Air Stripping

A physical phenomenon in which gas molecules are exchanged between a liquid and a gas at a gas-liquid interface.



Gas Transfer in Water Treatment

1. Absorption of reactive gases for water stabilization or disinfection

- Aeration or bubble systems are used for the absorption of reactive gases (O_2 , O_3 , Cl_2)
- Ozone is used for disinfection and oxidation of total organic carbon (TOC)
- Chlorine is used for disinfection and sometimes preoxidant for the oxidation of Fe and Mn

Gas Transfer in Water Treatment

2. Oxidation/precipitation of Iron and Manganese

Adverse effects

- Discoloration of water (iron: reddish water, manganese: brown or black water)
- Staining of plumbing fixtures
- Impartation of a bitter taste to the water
- Stimulation of the growth of microorganisms.
- Damage to textiles, dye, paper, and food.
- Iron residue (tuberculation) in pipes

Removal mechanisms

- $4\text{Fe}^{2+} + \text{O}_2 + 10 \text{H}_2\text{O} \longrightarrow 4\text{Fe}(\text{OH})_3\downarrow + 8\text{H}^+$
- $2\text{Mn}^{2+} + \text{O}_2 + 2\text{H}_2\text{O} \longrightarrow 2\text{MnO}_2\downarrow + 4\text{H}^+$

Gas Transfer in Water Treatment

3. Air stripping of volatile organic compounds, NH_3 , CO_2 , H_2S , O_2
- Diffused-aeration systems are used for the stripping of odor causing chemicals and VOCs
 - Surface aerators primarily used for VOC removal
 - Packed-tower or spray nozzle systems are used for the removal of NH_3 , CO_2 , H_2S , VOCs

Theory of Gas Transfer

To design and operate aeration/air stripping devices understanding of:

- Equilibrium partitioning of chemicals between air and water and
- Mass transfer rate across the air-water interface are required

Both equilibrium and mass transfer equations are incorporated into mass balance equations

Henry's Law

Equilibrium partitioning of a gas or organic contaminant between air and water is described by Henry's Law

Form of equation	H	Liquid phase conc.	Gas phase conc.
$P = H \cdot C$	$\frac{\text{atm} \cdot \text{m}^3}{\text{mol}}$	mol/m^3	atm
$C_g = H_c \cdot C_l$	unitless	mol/m^3	mol/m^3
$P = H_a \cdot X$	atm	mol/mol	atm

H is Henry constant

TABLE 5.1 Unit Conversions for Henry's Law Constants

$$H\left(\frac{\text{L}_{\text{H}_2\text{O}}}{\text{L}_{\text{Air}}}\right) = \frac{H\left(\frac{\text{L} \cdot \text{atm}}{\text{mol}}\right)}{RT}$$

$$H\left(\frac{\text{L} \cdot \text{atm}}{\text{mol}}\right) = H\left(\frac{\text{L}_{\text{H}_2\text{O}}}{\text{L}_{\text{Air}}}\right) \times RT$$

$$H\left(\frac{\text{L}_{\text{H}_2\text{O}}}{\text{L}_{\text{Air}}}\right) = \frac{H(\text{atm})}{RT \times 55.6 \frac{\text{mol H}_2\text{O}}{\text{L}_{\text{H}_2\text{O}}}}$$

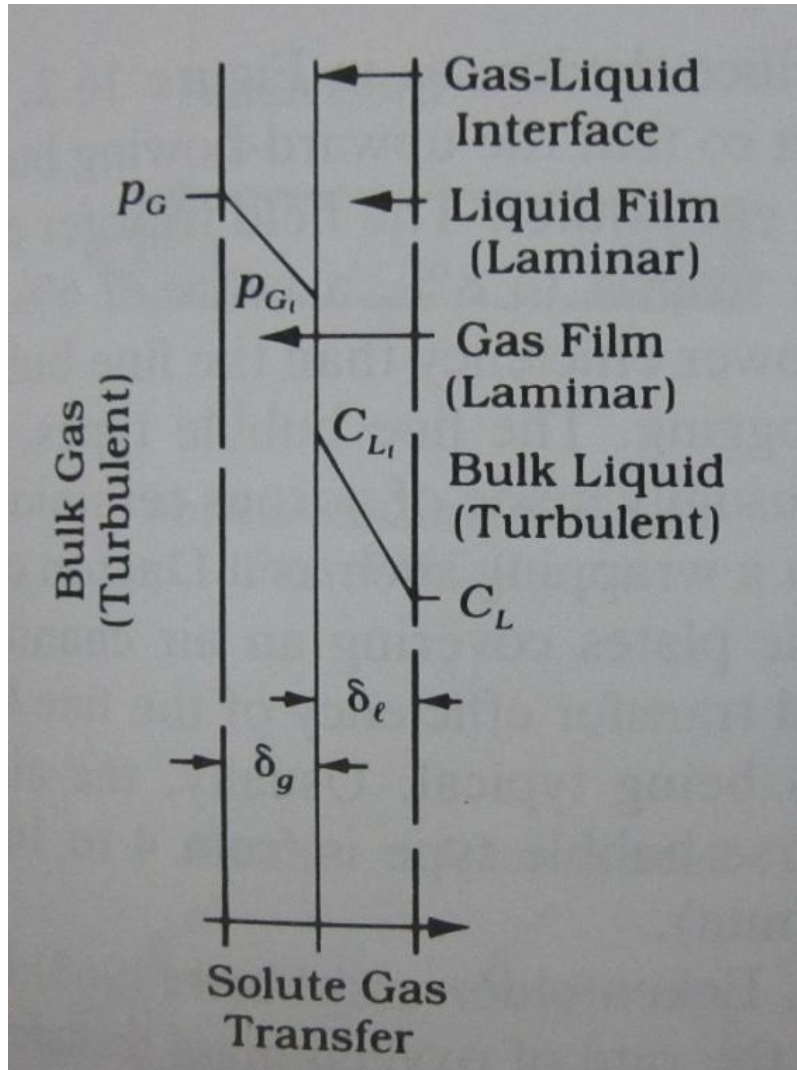
$$H\left(\frac{\text{L} \cdot \text{atm}}{\text{mol}}\right) = \frac{H(\text{atm})}{55.6 \frac{\text{mol H}_2\text{O}}{\text{L}_{\text{H}_2\text{O}}}}$$

$$H(\text{atm}) = H\left(\frac{\text{L} \cdot \text{atm}}{\text{mol}}\right) \times 55.6 \frac{\text{mol H}_2\text{O}}{\text{L}_{\text{H}_2\text{O}}}$$

$$H(\text{atm}) = H\left(\frac{\text{L}_{\text{H}_2\text{O}}}{\text{L}_{\text{Air}}}\right) \times RT \times 55.6 \frac{\text{mol H}_2\text{O}}{\text{L}_{\text{H}_2\text{O}}}$$

$$R = 0.08205 \frac{\text{atm} \cdot \text{L}}{\text{mol} \cdot ^\circ\text{K}} \quad T = ^\circ\text{K}$$

Mass Transfer



$$\frac{dC}{dt} = K_L a (C_s - C_L)$$

- dC/dt : rate of oxygen transfer, mass/volume.time
- K_L : diffusion transfer coefficient
- a : interfacial bubble area per unit volume of water
- $K_L a$: overall liquid mass transfer coefficient, time⁻¹
- C_s : Saturation dissolved oxygen concentration, mass/volume
- C_L : dissolved oxygen concentration in the liquid, mass/volume

Gas Absorption

$$\frac{dC}{dt} = K_L a (C_s - C_L)$$

$$\int_{C_o}^{C_t} \frac{-dC}{C_s - C} = K_L a \int_0^t -dt$$

$$\ln(C_s - C_t) = \ln(C_s - C_o) - K_L a t$$

Absorption of gas into a solution $C < C_s$

Desorption of Gas

$$\frac{dC}{dt}h = -K_L(C - C_s) \quad \text{h: depth of reactor}$$

$$\int_{C_0}^{C_t} \frac{dC}{C - C_s} = \frac{K_L}{h} \int_0^t -dt$$

$$\ln(C_t - C_s) = \ln(C_0 - C_s) - \frac{K_L}{h} t$$

Desorption of gas out of a solution $C > C_s$

Types of aerators

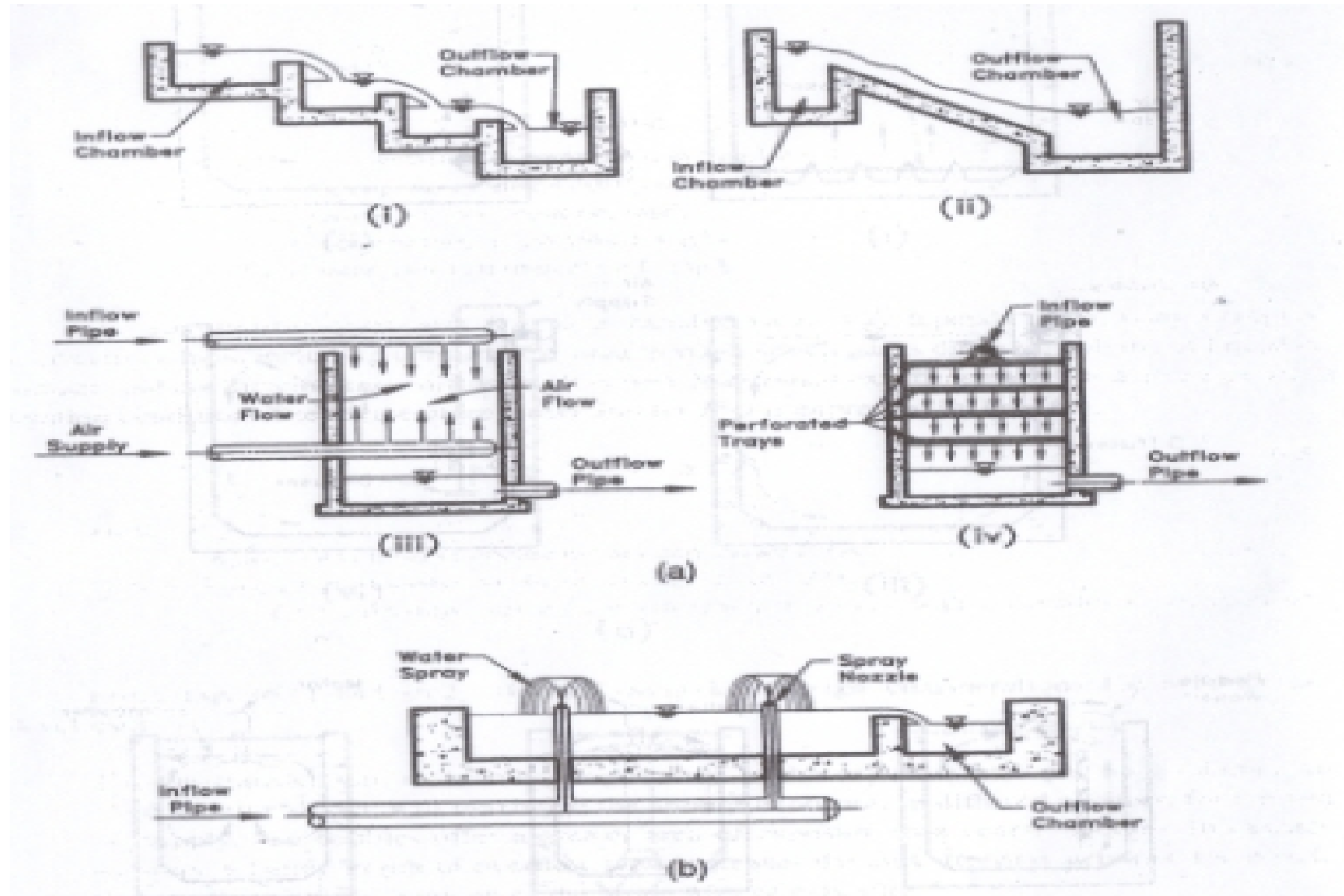
- Gravity aerators: e.g. waterfalls, cascades, perforated tray towers

- Spray aerators: spray droplets of water into the air from orifice or nozzles

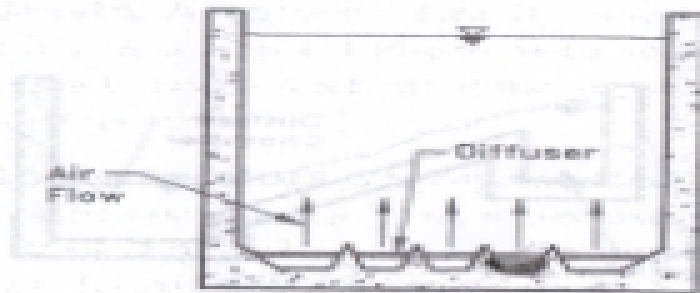
- Diffused-air aerator: compressed air is injected into the tank through porous diffuser plates, or tubes

- Mechanical aerators: employ either motor-driven impellers or a combination of impeller with air-injection devices

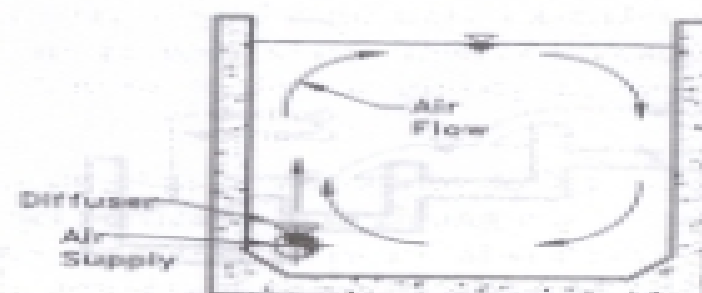
Gravity and Spray aerators



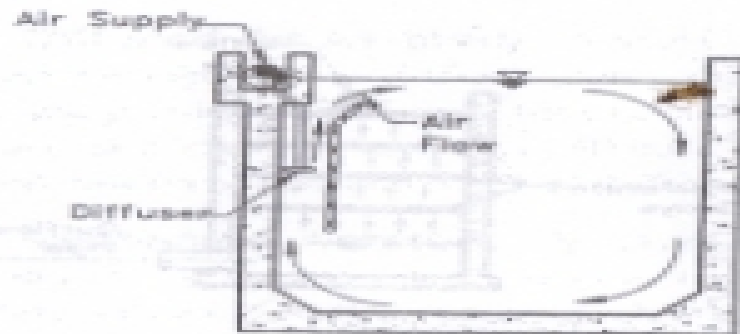
Diffused-air and Mechanical aerators



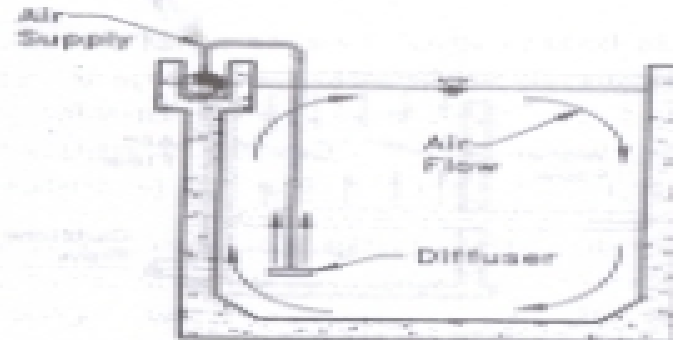
(i)



(ii)

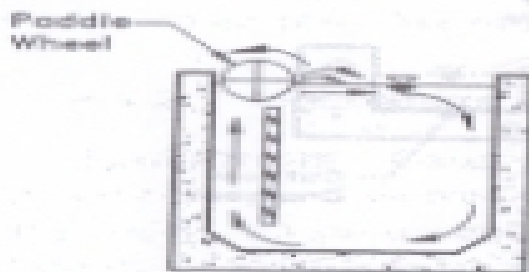


(iii)

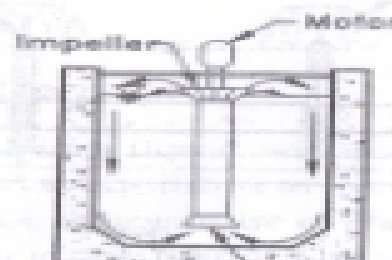


(iv)

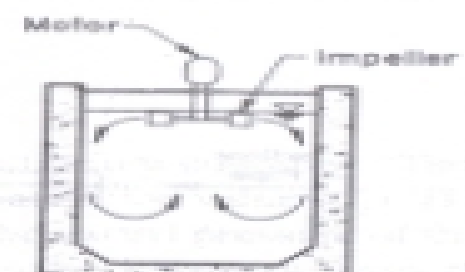
(a)



(i)



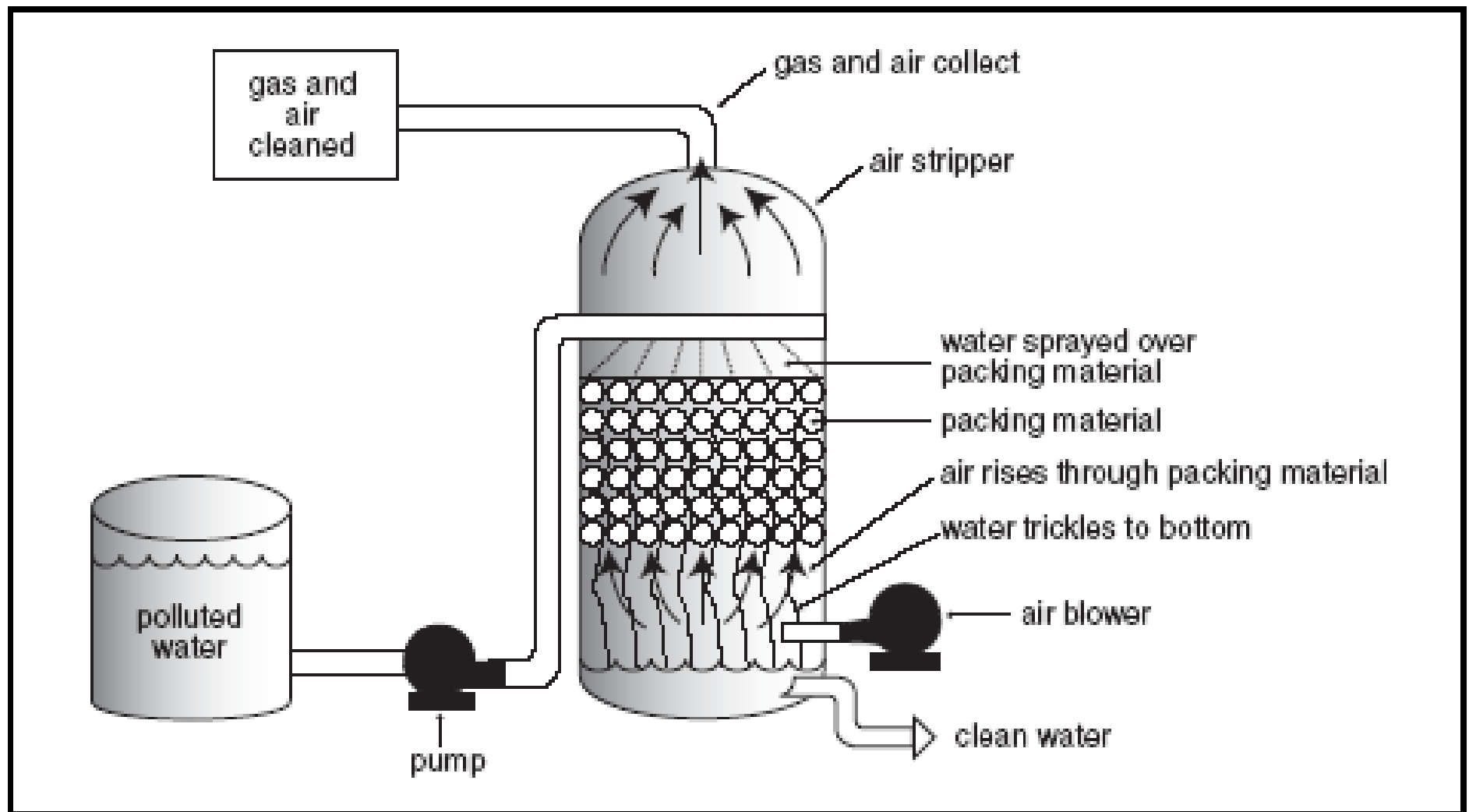
(ii)



(iii)

(b)

Air Stripping Tower Use



Design and operational characteristics of aerators

Aerator	Removal Rates	Design Specifications	Equipment
Gravity aerators			
<i>Cascade</i>	20–45% CO ₂	Head: 1.0–3.0 m Area: 85–105 m ² /m ² ·s Flow velocity of approx. 0.3 m/s	Pump and concrete, or other corrosion-resistant flow surface baffles. Energy required to raise water
<i>Packing Tower</i>	>95% VOC ^a >90% CO ₂	Max. column diameter 3 m Hydraulic loading: 2000 m ³ /m ² ·d	Tower structure, packing material, pump, and air blowers. Energy required to raise water and provide counter-current air flow.
<i>Tray</i>	>90% CO ₂	Water rate: 0.8–1.5 m ³ /m ² /min. Air requirement: 7.5 m ³ /m ³ water Tray spacing: 30–75 cm Area: 50–160 m ² /m ³ ·s	Corrosion-resistant trays, tray media, pump, air blowers. Energy required to raise water and provide counter current air flow.
<i>Spray</i>	70–90% CO ₂ , 25–40 H ₂ S	Head: 1.2–9 m Nozzle diameter: 2.5–4.0 cm Nozzle spacing: 0.6–3.6 m Nozzle discharge: 5–10 L/s Basin area: 105–320 m ² /m ³ ·s Spray pressure: around 70 kPa	Bronze or cast iron nozzles, piping, meters and valves. Energy required to operate pressurized spray pump.
Diffused aerators	80% VOC ^a	Detention time: 10–30 min. Air: 0.7–1.1 m ³ /m ³ water Tank: 2.7–4.5 m deep, 3–9 m wide Width/depth < 2 Maximum volume = 150 m ³ Diffuser orifice: 2–5 mm diameter	Diffusers, air piping, air compressor, air filter, valves, meters, basin with drains. Energy for air compressor around 11–45 kW/m ³ ·s.
Mechanical aerators	50–80% CO ₂	Detention time: 10–30 min. Tank: 2–4 m deep	Motor-driven impellers turbine aerators or draft tube aerators.

6.3 Adsorption and Ion Exchange

- Adsorption is a mass transfer operation in which substances present in a liquid phase are adsorbed on a solid phase and thus removed from the liquid
- Adsorbent: the solid media on which adsorption occurs
- Adsorbate: the molecule that accumulates or adsorbs on the adsorbent material

Use of adsorption in water treatment

- Removal of odor- and taste- causing compounds
- Removal of synthetic organic chemicals (e.g. pesticides, herbicides, etc;)
- Removal of color-forming substances
- Removal of disinfection byproduct precursors
- Removal of inorganic constituents (e.g. mercury, arsenic, etc)
- Dechlorination

Adsorption Process in Water Treatment

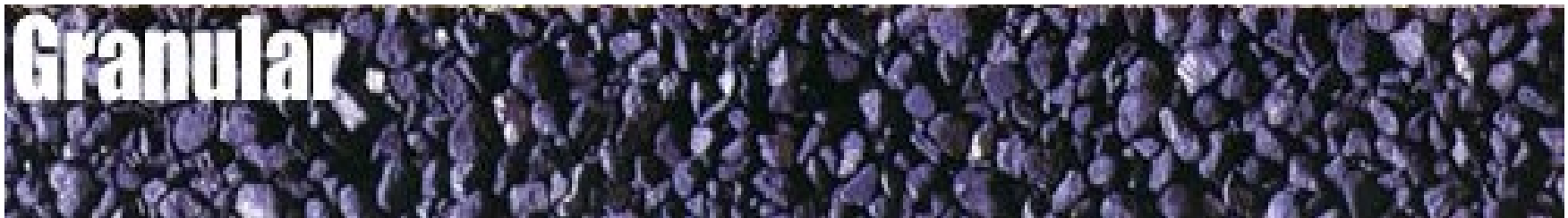
Application field	Objective	Adsorbent
Drinking water treatment	Removal of dissolved organic matter	Activated carbon
	Removal of organic micropollutants	Activated carbon
	Removal of arsenic	Aluminum oxide, iron hydroxide
Urban wastewater treatment	Removal of phosphate	Aluminum oxide, iron hydroxide
	Removal of micropollutants	Activated carbon
Industrial wastewater treatment	Removal or recycling of specific chemicals	Activated carbon, polymeric adsorbents
Swimming-pool water treatment	Removal of organic substances	Activated carbon
Groundwater remediation	Removal of organic substances	Activated carbon
Treatment of landfill leachate	Removal of organic substances	Activated carbon
Aquarium water treatment	Removal of organic substances	Activated carbon

Primary Adsorbents

- Powdered Activated Carbon (PAC)
 - It is added directly to water
 - can be applied at various locations within a water treatment plant
 - Usually removed by sedimentation or filtration



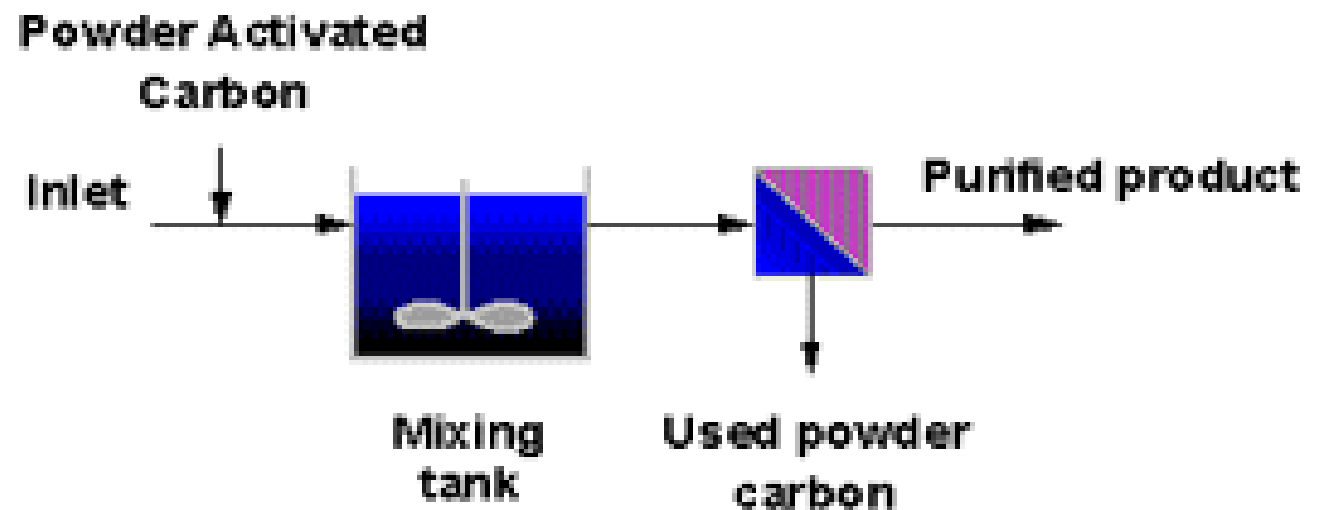
- Granular Activated Carbon (GAC)
 - Usually employed after filtration just prior to postdisinfection
 - Is operated in a fixed-bed mode





Principle of a granular activated carbon adsorption system

Application Methodology



Principle of a powder activated carbon adsorption system

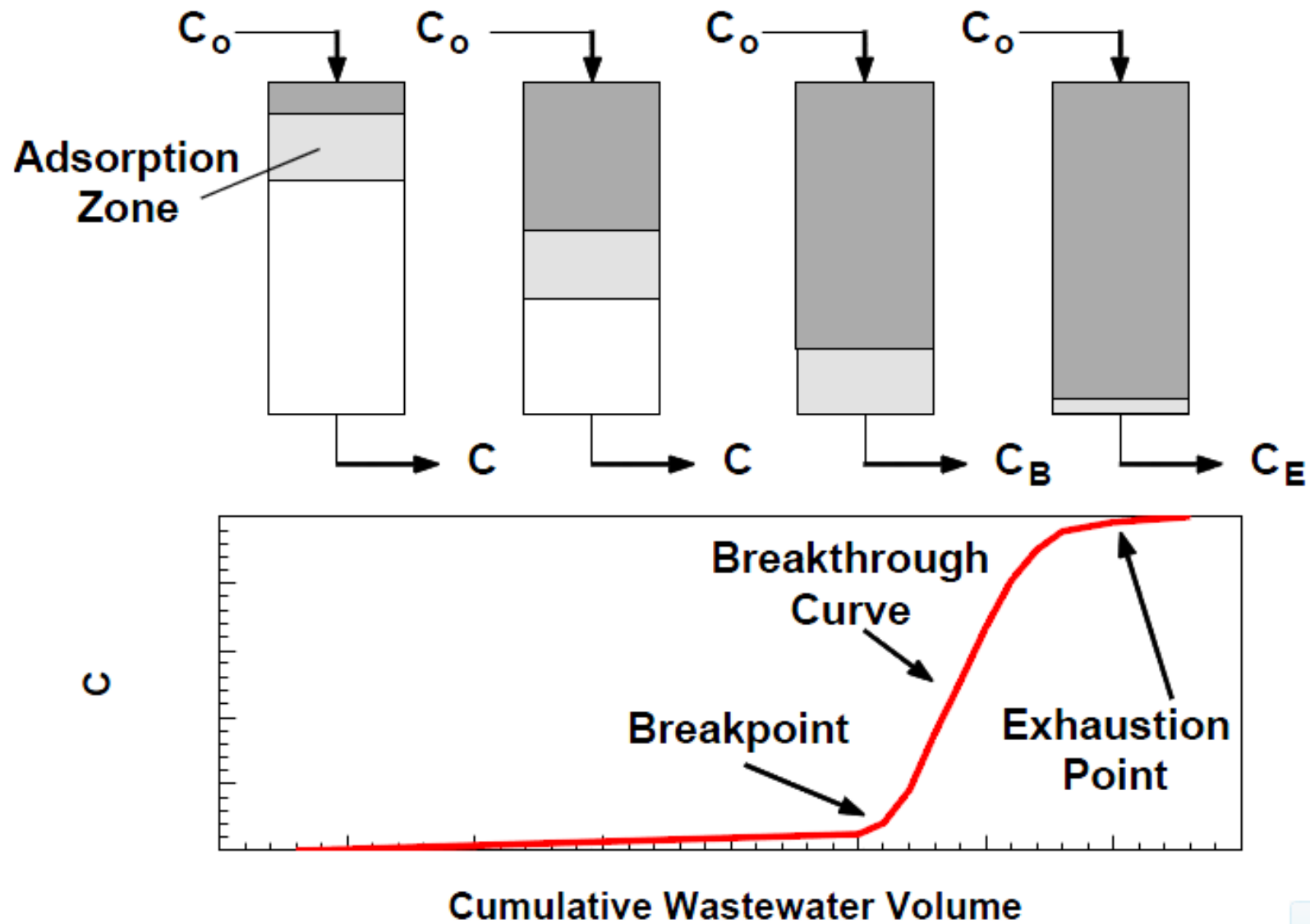
Adsorption phenomena

- Mainly physical adsorption mechanism
 - Important factors
 - Surface area
 - Pore size
- They have inverse relationship
- Surface area of commercial GAC: 600-1600 m²/g

Comparison between GAC & PAC

Parameter	GAC	PAC
Principal use	• Control of toxic organic compounds that are present in groundwater • Barrier to occasional spikes of toxic organics in surface waters and control of taste and odor compounds • Control of disinfection byproduct precursors or DOC	• Seasonal control of taste and odor compounds and strongly adsorbed pesticides and herbicides at low concentrations ($< 10 \mu\text{g/L}$)
Advantages	• Easily regenerated • Lower carbon usage rate per volume of water treated as compared to PAC	• Easily added to existing coagulation facilities for occasional control of organics
Disadvantages	• Need contactors and yard piping to distribute flow and replace exhausted carbon • Previously adsorbed compounds can desorb and in some cases appear in the effluent at concentrations higher than present in influent	• Hard to regenerate and impractical to recover from sludge of coagulation facilities • much higher carbon usage rate per volume of water treated as compared to GAC

Breakpoint and Breakthrough Curve



Adsorption Isotherms

Isotherm is an equation relating the amount of solute adsorbed onto the solid and the equilibrium concentration of solute in solution at a given temperature.

Most common isotherms in Environmental Engineering Applications are:

- Freundlich
- Langmuir

Freundlich Isotherm

$$\frac{X}{M} = k [C]^{\frac{1}{n}}$$

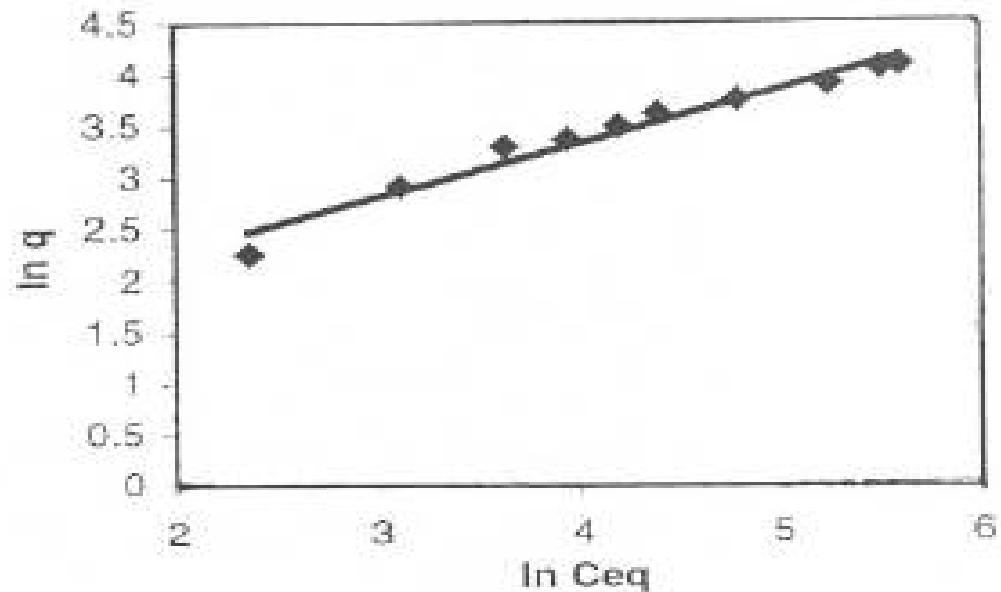
X = mass of adsorbate

M = mass of adsorbent

[C] = concentration of adsorbate
in solution in equilibrium

k and **n** = constants

$$\ln \frac{X}{M} = \ln k + \frac{1}{n} \ln [C]$$



Langmuir Isotherm

$$\frac{X}{M} = \frac{ab [C]}{1 + b [C]}$$

a and b = constants

$$\frac{[C]}{X/M} = \frac{1}{ab} + \frac{1}{a}[C]$$

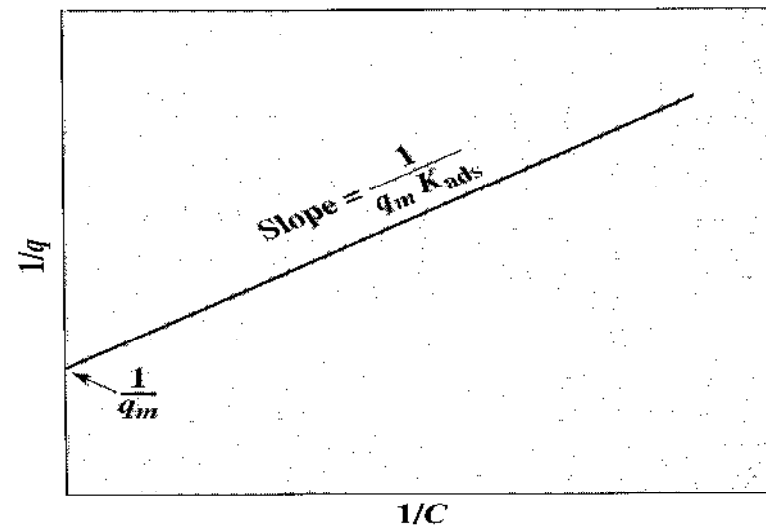


Figure 3.14
Straight-line form of the Langmuir isotherm.

Adsorption Capacity

$$\text{Adsorption Capacity} = a = \left(\frac{X}{M} \right)_{ult}$$

$$X = ([C_0] - [C]) \times V$$

- $[C_0]$ = solute concentration before adsorption onto mass of adsorbent (M)
- V = sample volume

Example 8.5 A wastewater containing $[C_o] = 25$ mg/L of phenol is to be treated using PAC to produce an effluent concentration $[C]_{eff} =$ of 0.10 mg/L. The PAC is simply added to the stream and the mixture subsequently settled in the following sedimentation tank. The constants of the Langmuir equation are determined by running a jar test producing the results below. The volume of waste subjected to each test is one liter. If a flow rate of Q_o of 0.11 m³/s is to be treated, calculate the quantity of PAC needed for the operation. What is the adsorption capacity of the PAC? Calculate the quantity of PAC needed to treat the influent phenol to the ultimate residual concentration.

Test	PAC Added M (g)	$[C]$ (mg/L)
1	0.25	6.0
2	0.32	1.0
3	0.5	0.25
4	1.0	0.09
5	1.5	0.06
6	2.0	0.06
7	2.6	0.06

Solution:

$$a = \left\{ \frac{l \sum_{l+1}^m [C] - (m-l) \sum_1^l [C]}{l \sum_{l+1}^m \frac{[C]}{X/M} - (m-l) \sum_1^l \frac{[C]}{X/M}} \right\}$$

$$b = \frac{l}{a \sum_1^l \frac{[C]}{X/M} - \sum_1^l [C]}$$

Neglect tests 6 and 7, because the additional values of 0.06's would not conform to the Langmuir equation.

Test No.	PAC Added M (g)	$[C]$ (mg/L)	X/M	$[C]/X/M$
1	0.25	6.0	0.076	78.95
2	0.32	1.0	0.075	13.33
3	0.5	0.25	0.0495	5.05
4	1.0	0.09	0.0249	3.61
5	1.5	0.06	0.0166	3.61

Lct $l = 3$; $m = 5$

$$a = \frac{3(0.09 + 0.06) - 2(6.0 + 1.0 + 0.25)}{3(3.61 + 3.61) - 2(78.95 + 13.33 + 5.05)} = \frac{0.45 - 14.5}{21.66 - 194.66} = 0.081$$

$$b = \frac{l}{a \sum_1^l \frac{[C]}{X/M} - \sum_1^l [C]} = \frac{3}{0.081(78.95 + 13.33 + 5.05) - (6.0 + 1.0 + 0.25)}$$

$$= \frac{3}{7.88 - 7.25} = 4.76$$

$$\frac{X}{M} = \frac{ab[C]}{1 + b[C]} = \frac{0.081(4.76)(0.10)}{1 + (4.76)(0.10)} = 0.026 \frac{\text{kg phenol to be removed}}{\text{kg C}}$$

Total phenol to be removed = $0.11(0.025 - 0.0001) = 0.00274$ kg/s.

Therefore,

$$PAC \text{ required} = \frac{0.00274}{0.026}(60)(60)(24) = 9,105 \text{ kg/d} \quad \text{Ans}$$

$$\left(\frac{X}{M}\right)_{ult} = a = 0.081 \frac{\text{kg phenol to removed}}{\text{kg C}} \quad \text{Ans}$$

The lowest concentration of phenol is 0.06 mg/L; at $[C] = 0.06$ mg/L,

$$\frac{X}{M} = \frac{0.081(4.76)(0.06)}{1 + (4.76)(0.06)} = 0.018 \frac{\text{kg phenol}}{\text{kg C}}$$

Total phenol to be removed to the ultimate residual concentration = $0.11(0.025 - 0.00006) = 0.00274$ kg/s.

Therefore,

$$PAC \text{ required} = \frac{0.00274}{0.018}(60)(60)(24) = 13,152 \text{ kg/s} \quad \text{Ans}$$

Example 8.6 Solve the previous example using the Freundlich isotherm.

Solution:

$$n = \frac{l \sum_{l+1}^m \ln[C] - (m-l) \sum_1^l \ln[C]}{l \left(\sum_{l+1}^m \ln \frac{X}{M} - (m-l) \sum_1^l \ln \frac{X}{M} \right)}$$

$$k = \exp \left(\frac{\sum_1^l \ln \frac{X}{M} - \frac{1}{n} \sum_1^l \ln[C]}{l} \right)$$

Neglect tests 6 and 7, because the additional values of 0.06's would not conform to the Langmuir equation.

Test No.	PAC Added M (g)	$[C]$ (mg/L)	$\ln [C]$	X/M	$\ln X/M$
1	0.25	6.0	1.79	0.076	-2.58
2	0.32	1.0	0	0.075	-2.59
3	0.5	0.25	-1.39	0.049	-3.02
4	1.0	0.09	-2.41	0.025	-3.69
5	1.5	0.06	-2.81	0.017	-4.07

^a 0.076 = (25 - 6)(1)/0.25(1000); other values in the column are computed similarly.

Let $l = 3$; $m = 5M$

$$n = \frac{l \sum_{l+1}^m \ln[C] - (m-l) \sum_1^l \ln[C]}{l \left(\sum_{l+1}^m \ln \frac{X}{M} - (m-l) \sum_1^l \ln \frac{X}{M} \right)} = \frac{3(0.09 + 0.06) - 2(6.0 + 1.0 + 0.25)}{3(-3.69 - 4.07) - 2(-2.58 - 2.59 - 3.02)}$$

$$= \frac{0.45 - 14.5}{-23.28 + 16.38} = 2.04$$

$$k = \exp \left(\frac{\sum_1^l \ln \left(\frac{X}{M} - \frac{1}{n} \right) \sum_1^l \ln[C]}{l} \right)$$

$$= \exp \left[\frac{(-2.58 - 2.59 - 3.02) - \frac{1}{2.04}(1.79 + 0 - 1.39)}{3} \right]$$

$$= \exp \left[\frac{-8.19 - 0.196}{3} \right] = 0.061$$

$$\frac{X}{M} = k[C]^{1/n} = 0.061(0.1)^{1/2.04} = 0.020 \frac{\text{kg phenol to be removed}}{\text{kg } C}$$

Total phenol to be removed = $0.11(0.025 - 0.0001) = 0.00274 \text{ kg/s.}$

Therefore,

$$PAC \text{ required} = \frac{0.00274}{0.020}(60)(60)(24) = 11,837 \text{ kg/d} \quad \text{Ans}$$

From $X/M = k[C]^{1/n}$, as $[C]$ increases, X/M increases, such that, theoretically, X/M could become infinite. Practically, this means that equilibrium concentrations of C must be determined experimentally until found. The data do not reflect this. Therefore, the adsorption capacity would have to be solved using the Langmuir isotherm; thus,

$$\left(\frac{X}{M}\right)_{ult} = a = 0.081 \frac{\text{kg phenol to be removed}}{\text{kg } C} \quad \text{Ans}$$

The lowest concentration of phenol is 0.06 mg/L; at $[C] = 0.06 \text{ mg/L}$,

$$\frac{X}{M} = 0.061(0.06)^{1/2.04} = 0.015 \frac{\text{kg phenol}}{\text{kg } C}$$

Total phenol to be removed to the ultimate residual concentration = $0.11(0.025 - 0.00006) = 0.00274 \text{ kg/s}$.

Therefore,

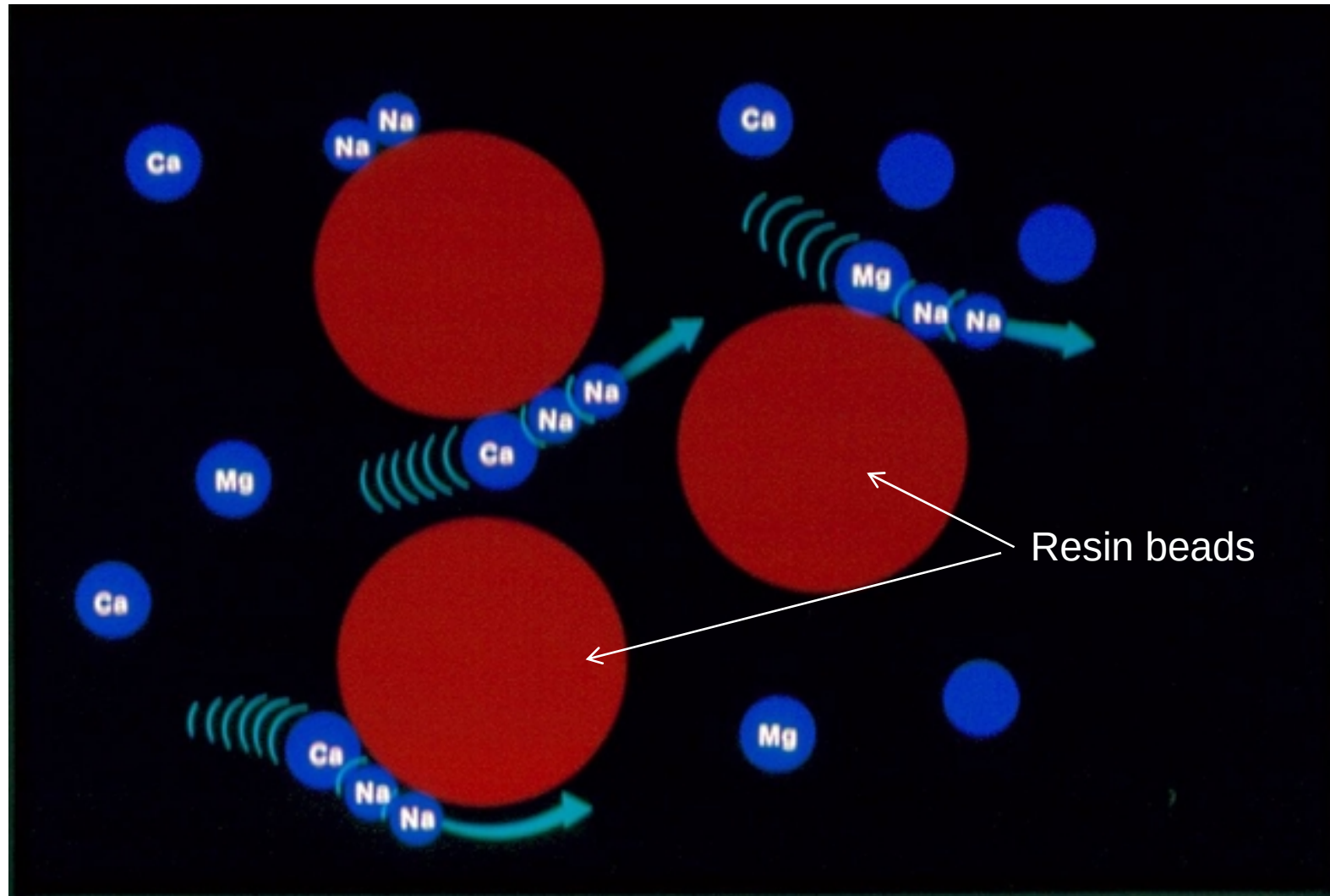
$$PAC \text{ required} = \frac{0.00274}{0.015}(60)(60)(24) = 15,782 \text{ kg/s} \quad \text{Ans}$$

Ion Exchange

- Involves replacement of the unwanted ions in solution with ions of different species that are attached to an insoluble resin
- Can be used to remove nitrate ions, metal ions, hardness causing ions, etc.
- High quality water can be obtained, but too expensive to use for municipal water treatment

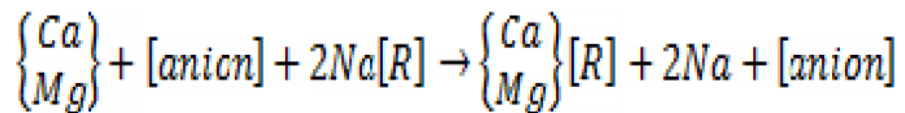
Hardness Removal by Ion Exchange

Resin beads attract Ca^{2+} and Mg^{2+} and release Na^+

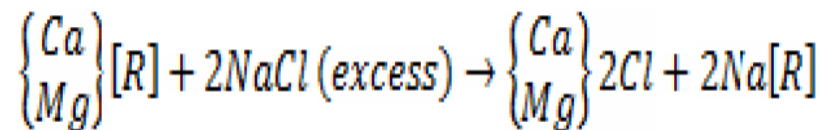


Ion Exchange Reactions

- Removal reaction



- Regeneration reaction



LARGE-SCALE ION EXCHANGE COLUMNS

Exercise on PAC Adsorption

A water supply district has recognized watershed development as the cause of increasing taste and odor problems from algae blooms. Blue-green algae and actinomycetes have been identified as the most common source, with geosmin a representative odorous organic compound they produce. The district has no power to regulate development and expects decreased water quality. Accordingly it has tested a commercial grade of PAC to determine if it removes geosmin effectively.

In the tests, raw water was placed in 1-L bottles and spiked with geosmin to reach a constant concentration of 40 mg/L (geosmin concentration in the raw water prior to spiking was less than 1 mg/L). a different dosage of PAC was applied to each bottle. After three hours, the samples were analysed to determine the equilibrium concentration of geosmin. The results are given on the next slide.

Given the information, produce adsorption isotherms based on the Langmuir and Freundlich models to describe the removal of geosmin.

Problem on PAC...Cont'd

<u>Carbon dosage (M)</u> mg/L	Initial <u>Geosmin (Co)</u> µg/L	Final concentration (C) µg/L
0	40	40.00
0.05	40	35.10
0.5	40	15.85
1.0	40	9.50
2.0	40	4.90
4.0	40	2.35
10.0	40	0.90
20.0	40	0.42
30.0	40	0.25

Assignment-7

- Problems on Filtration, Disinfection, Softening and Adsorption (Separate word file)

7. WASTEWATERTREATMENT

Definition of Wastewater

- Wastewater is simply that part of the water supply to the community or to the industry which has been used for different purposes and has been mixed with solids either suspended or dissolved.
- Wastewater is 99.9% water and 0.1% solids. The main task in treating the wastewater is simply to remove most or all of this 0.1% of solids.

Goals of Wastewater Treatment

Protecting the public health

- Wastewater (WW) contains pathogenic microorganisms which lead to dangerous diseases to humans and animals.
- Hazardous matter such as heavy metals that are toxic.
- Produces odorous gases and bad smell.

Protecting the environment

- Raw (untreated) WW leads to septic conditions in the environment and consequently leads to the deterioration of surface and groundwater quality and pollutes the soil.
- Raw WW is rich with nitrogen and phosphorus (N,P), and leads to the phenomena of "Eutrophication" if disposed in surface water. Eutrophication is the growth of huge amounts of algae and other aquatic plants leading to the deterioration of the water quality.
- Raw WW is rich with organic matter which consumes oxygen when disposed in rivers leading to killing aquatic creatures such as fish.
- Raw WW may contain toxic gases and volatile organic matter (VOCs) and toxic air contaminants (TACs).

Reuse of the treated WW and solids

- Treated water is reused for
 - Agriculture
 - Industry
 - Recharge to ground water
- Separated and treated solids are reused for
 - Agriculture
 - Solids Landfill
 - Energy source

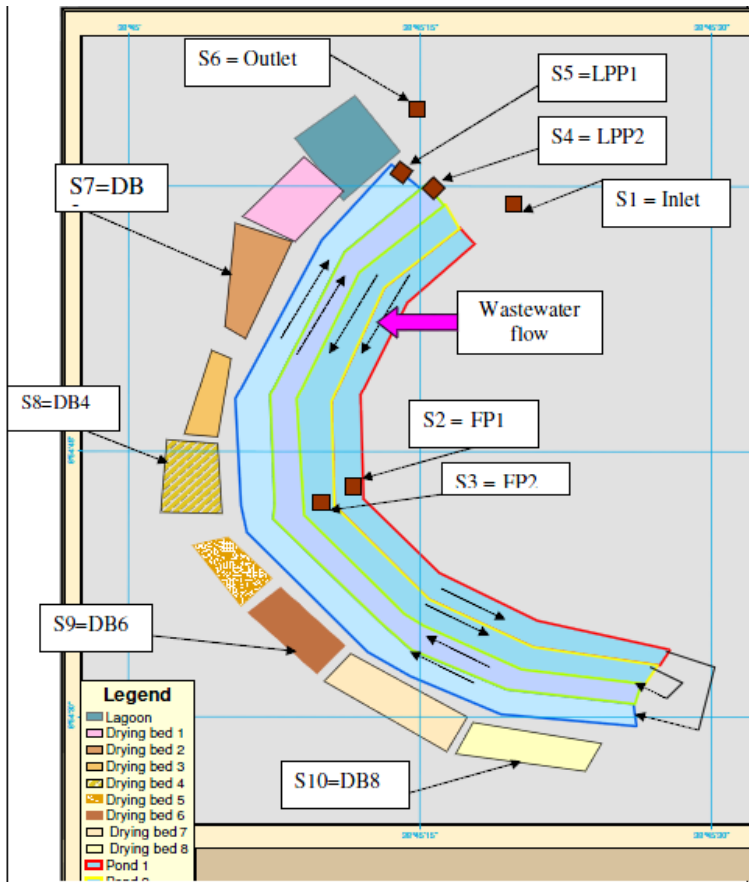
Properties of Wastewater

Surfactants	Domestic, commercial, and industrial wastes
Volatile organic compounds	Domestic, commercial, and industrial wastes
Other	Natural decay of organic materials
Inorganic: Alkalinity	Domestic wastes, domestic water supply, groundwater infiltration
Chlorides	Domestic wastes, domestic water supply, groundwater infiltration
Heavy metals	Industrial wastes
Nitrogen	Domestic and agricultural wastes
PH	Domestic, commercial, and industrial wastes
Phosphorus	Domestic, commercial, and industrial wastes natural runoff
Priority pollutant Sulfur	Domestic water supply; domestic, commercial. And industrial wastes
Gases: Hydrogen sulfide	Decomposition of domestic wastes
Methane	Decomposition of domestic wastes
Oxygen	Domestic water supply , surface- water infiltration
Biological constituents: Animals	Open watercourses and treatment plants
Plants	Open watercourses and treatment plants
Eubacteria	Domestic wastes, surface water infiltration, treatment plants .
Archaeobacteria	Domestic wastes, surface-water infiltration, treatment plants
Viruses	Domestic wastes

Typical characteristics of domestic wastewater

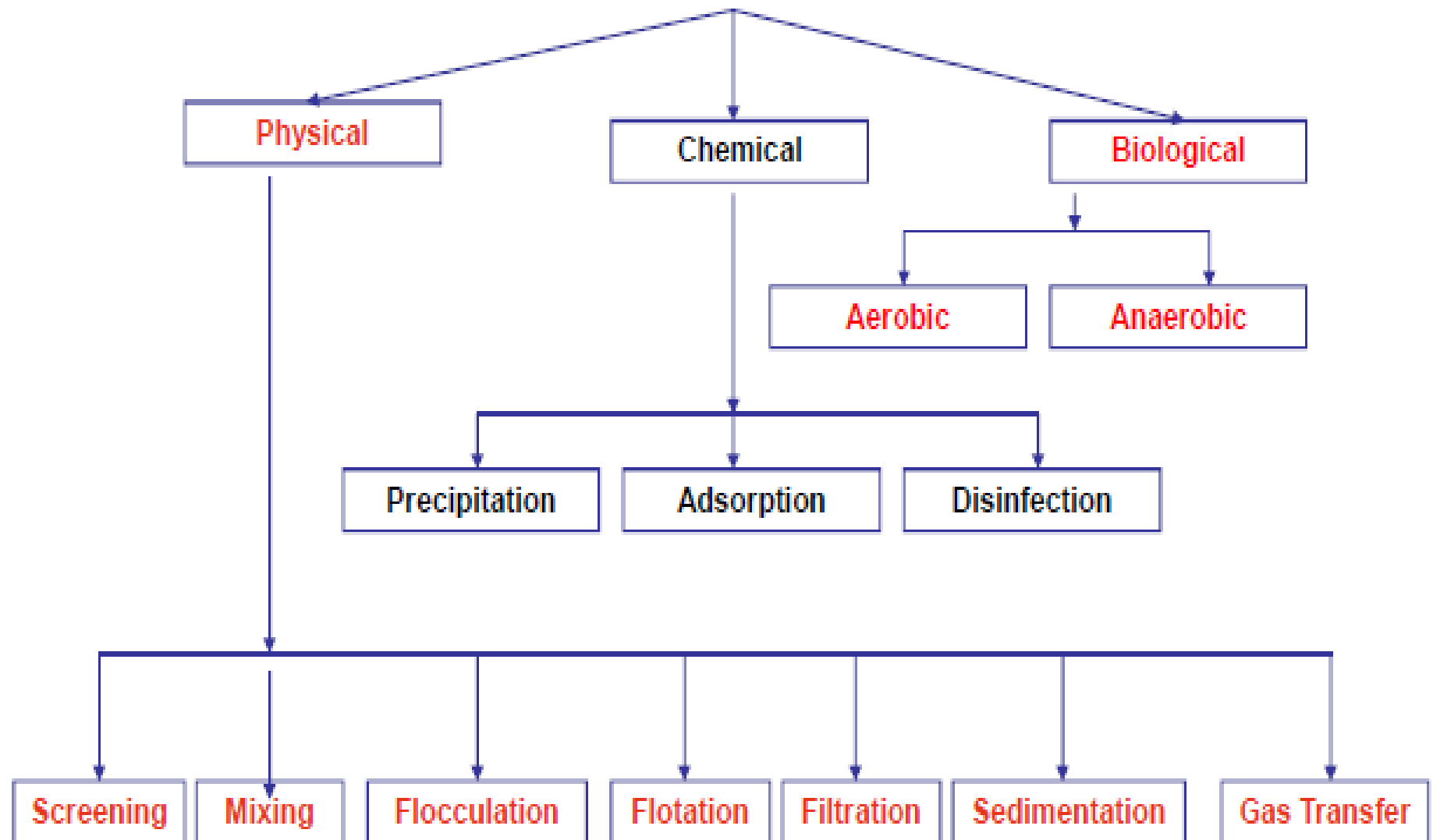
Constituent	Weak (mg/L)	Medium (mg/L)	Strong (mg/L)
Alkalinity (as CaCO_3)	50	100	200
BOD_5	100	200	300
Suspended solids	100	200	350
TDS	200	500	1000
TKN (as N)	20	40	80
Total phosphorus (as P)	5	10	20

Wastewater characteristics at Kality Stabilization Pond, Addis Ababa

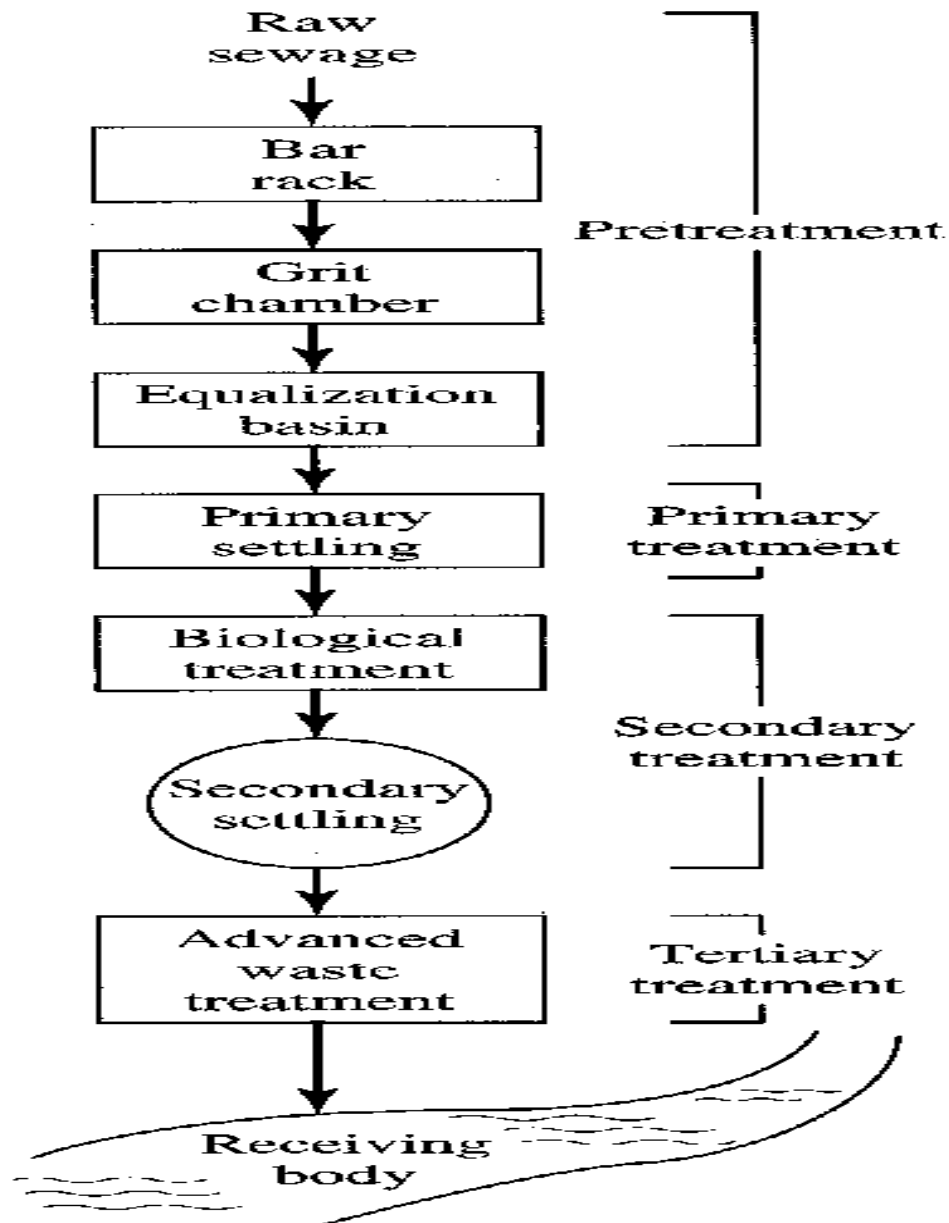


Parameter	Influent quality	Effluent quality
BOD5, mg/L	229-324	41-55
COD, mg/L	480-697	123-142
TSS, mg/L	188-237	63-79
TN, mg/L	85-95	30-43
TP, mg/L	13-16	9-12
T. Coliform, MPN/100ml	28-90 billion	0.33-0.34 billion

Wastewater treatment methods



Conventional wastewater treatment



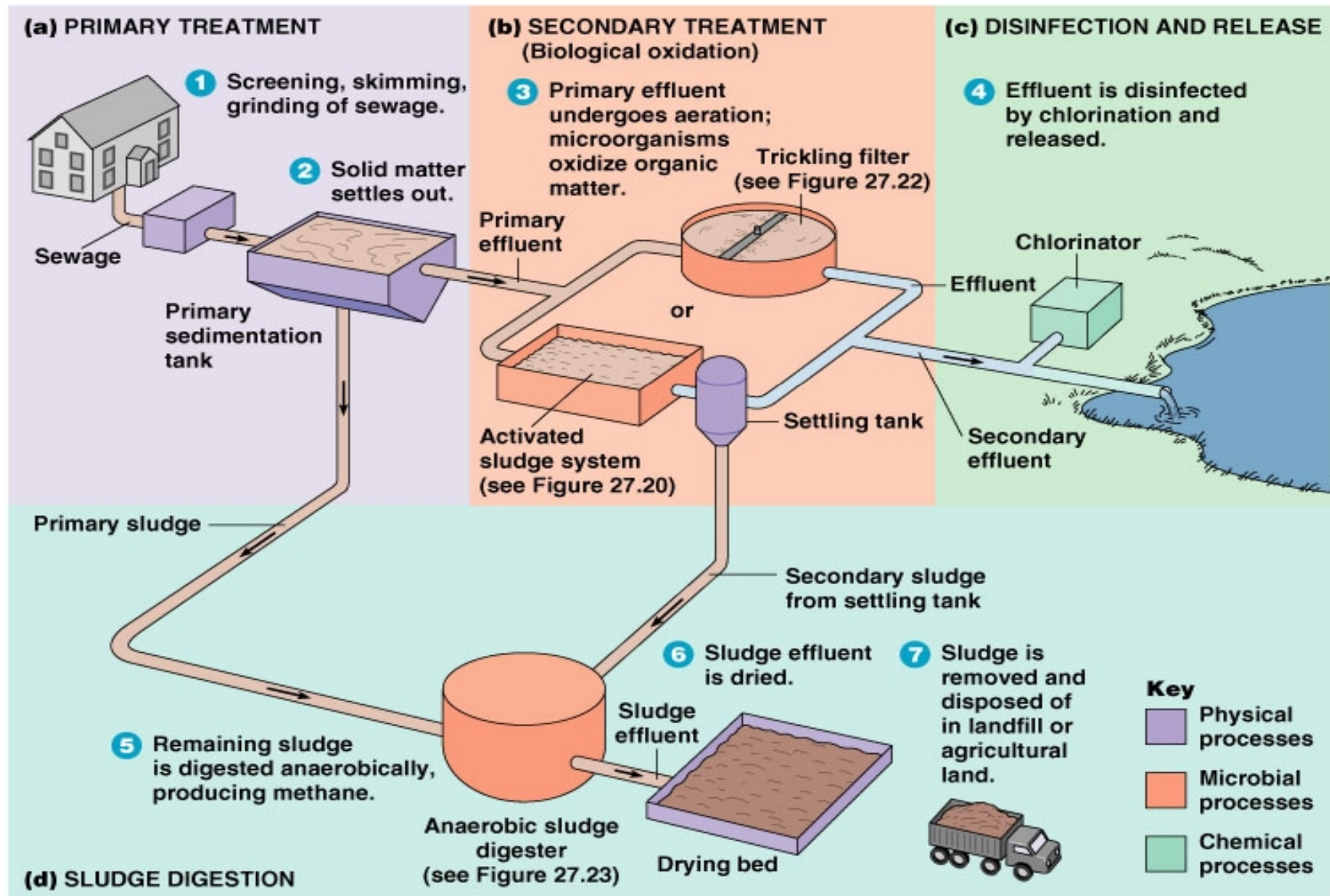
Pretreatment – removes materials that can cause operational problems, equalization optional

Primary treatment – remove ~60% of solids and ~35% of BOD

Secondary treatment – remove ~85% of BOD and solids

Advanced treatment – varies: 95+ % of BOD and solids, N, P

Typical Municipal Sewage Treatment



Biological Wastewater Treatment

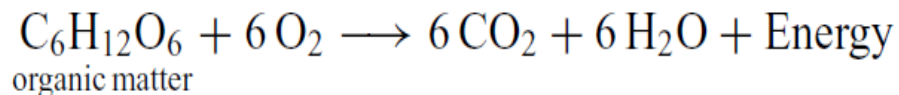
- Objectives of biological treatment:
 - To stabilize the organic content
 - To remove nutrients such as nitrogen and phosphorus

Types:

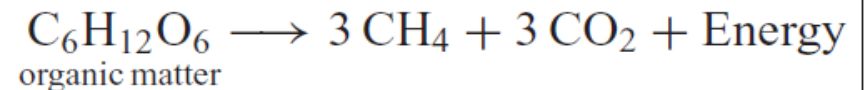
- ✓ Aerobic Processes
- ✓ Anoxic Processes
- ✓ Anaerobic Processes
- ✓ Combined Aerobic-Anoxic-Anaerobic Processes
- ✓ Pond Processes

- ✓ Attached Growth
- ✓ Suspended Growth
- ✓ Combined Systems

- ✓ Aerobic
- ✓ Maturation
- ✓ Facultative
- ✓ Anaerobic

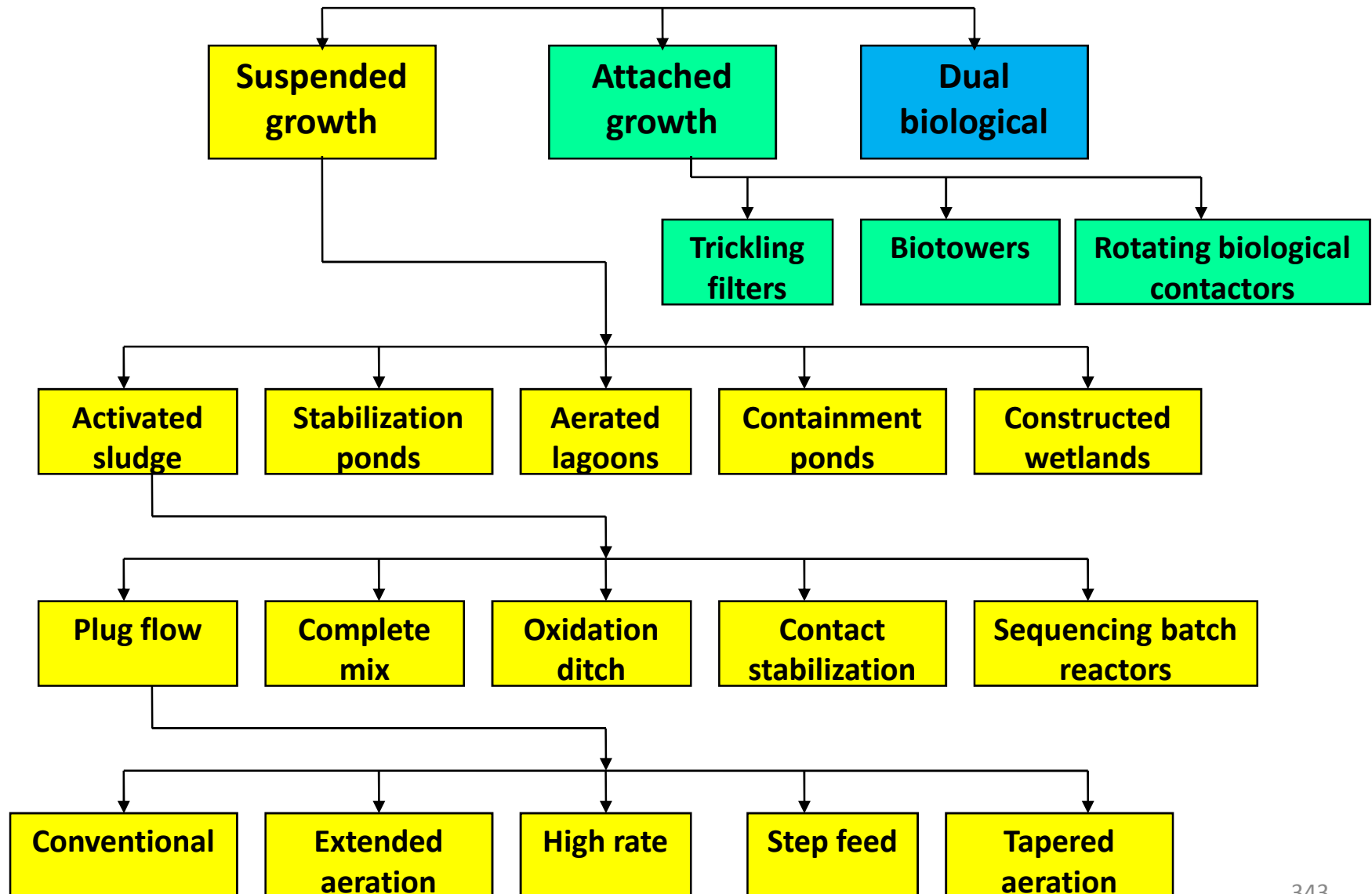


Aerobic



Anaerobic

Examples of Treatment Technologies



Major Aerobic Biological Processes

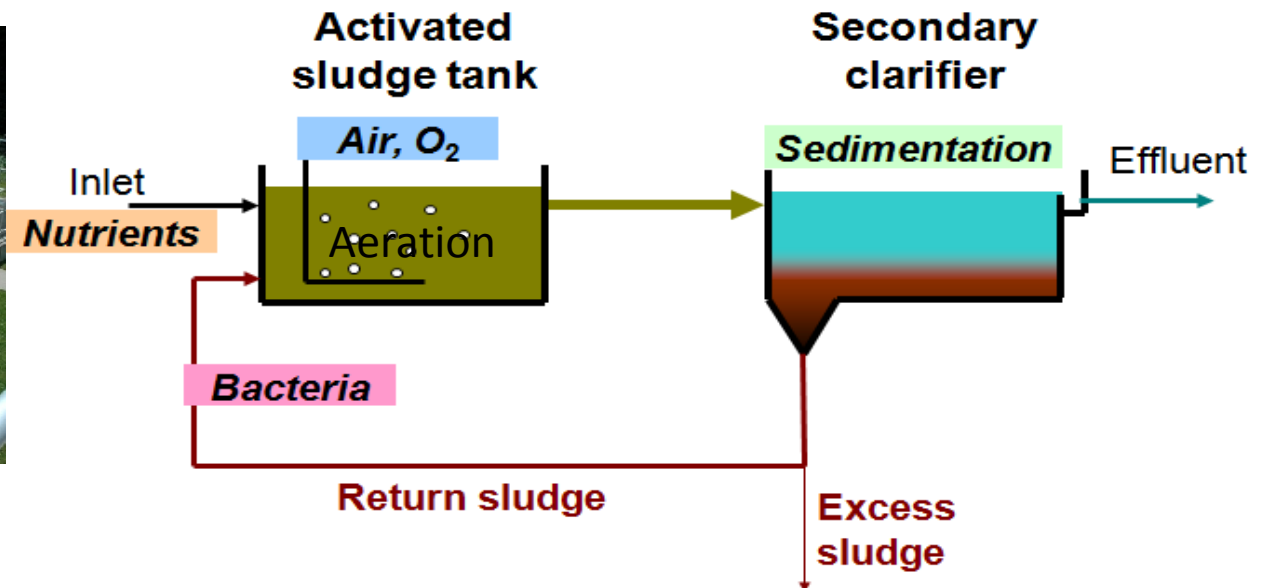
Type of Growth	Common Name	Use
Suspended Growth	Activated Sludge (AS)	Carbonaceous BOD removal (nitrification)
	Aerated Lagoons	Carbonaceous BOD removal (nitrification)
Attached Growth	Trickling Filters	Carbonaceous BOD removal. nitrification
	Roughing Filters (trickling filters with high hydraulic loading rates)	Carbonaceous BOD removal
	Rotating Biological Contactors	Carbonaceous BOD removal (nitrification)
	Packed-bed reactors	Carbonaceous BOD removal (nitrification)
Combined Suspended & Attached Growth	Activated Biofilter Process ✓Trickling filter-solids contact process ✓Biofilter-AS process ✓Series trickling filter-AS process	Carbonaceous BOD removal (nitrification)

Activated Sludge Process

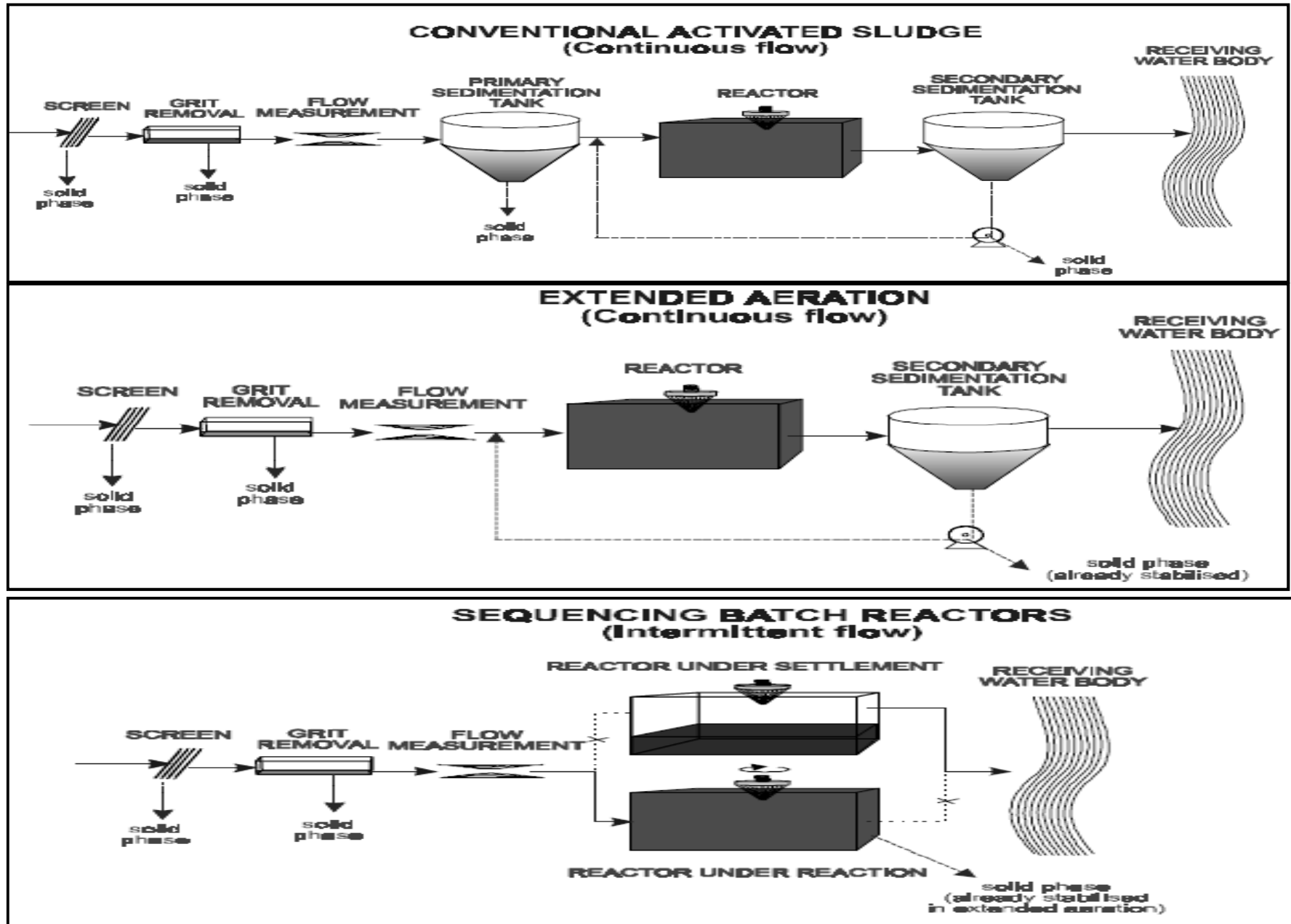
- The aeration tank contains a suspension of the wastewater and microorganisms, the mixed liquor. The liquor is mixed by aeration devices (supplying also oxygen)
- A portion of the biological sludge separated from the secondary effluent by sedimentation is recycled to the aeration tank



Aeration Basin



Some Activated Sludge Configurations



CAS: Advantages/Disadvantages

Advantage

- High BOD removal efficiency
- Nitrification usually obtained
- Biological removal of N and P is possible
- Low land requirements
- Reliable process, as long as it is supervised
- Reduced possibilities of bad odours, insects and worms
- Operational flexibility

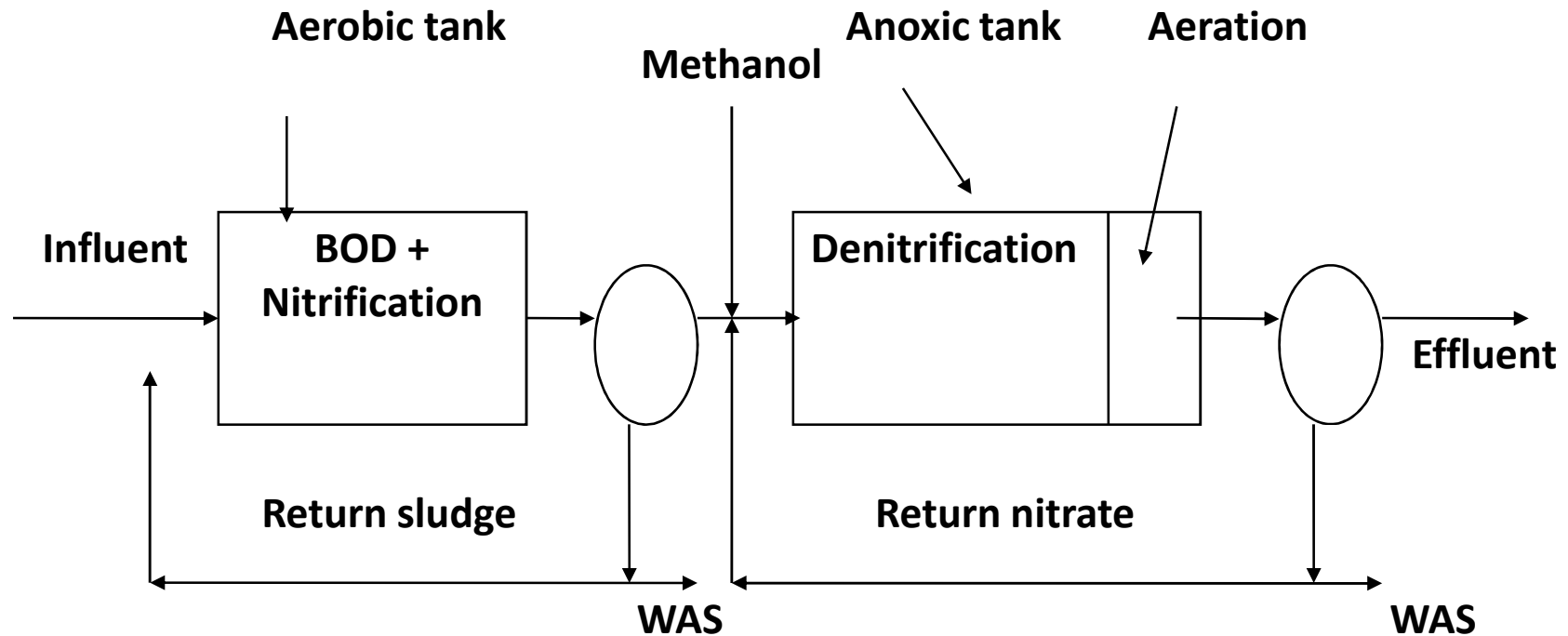
Disadvantage

- Low coliform removal efficiency
- High construction and operational costs
- High energy consumption
- Sophisticated operation required
- High mechanisation level
- Relatively sensitive to toxic discharges
- Requires complete treatment and final disposal of the sludge
- Possible environmental problems with noise and aerosols

Main characteristics of activated sludge systems

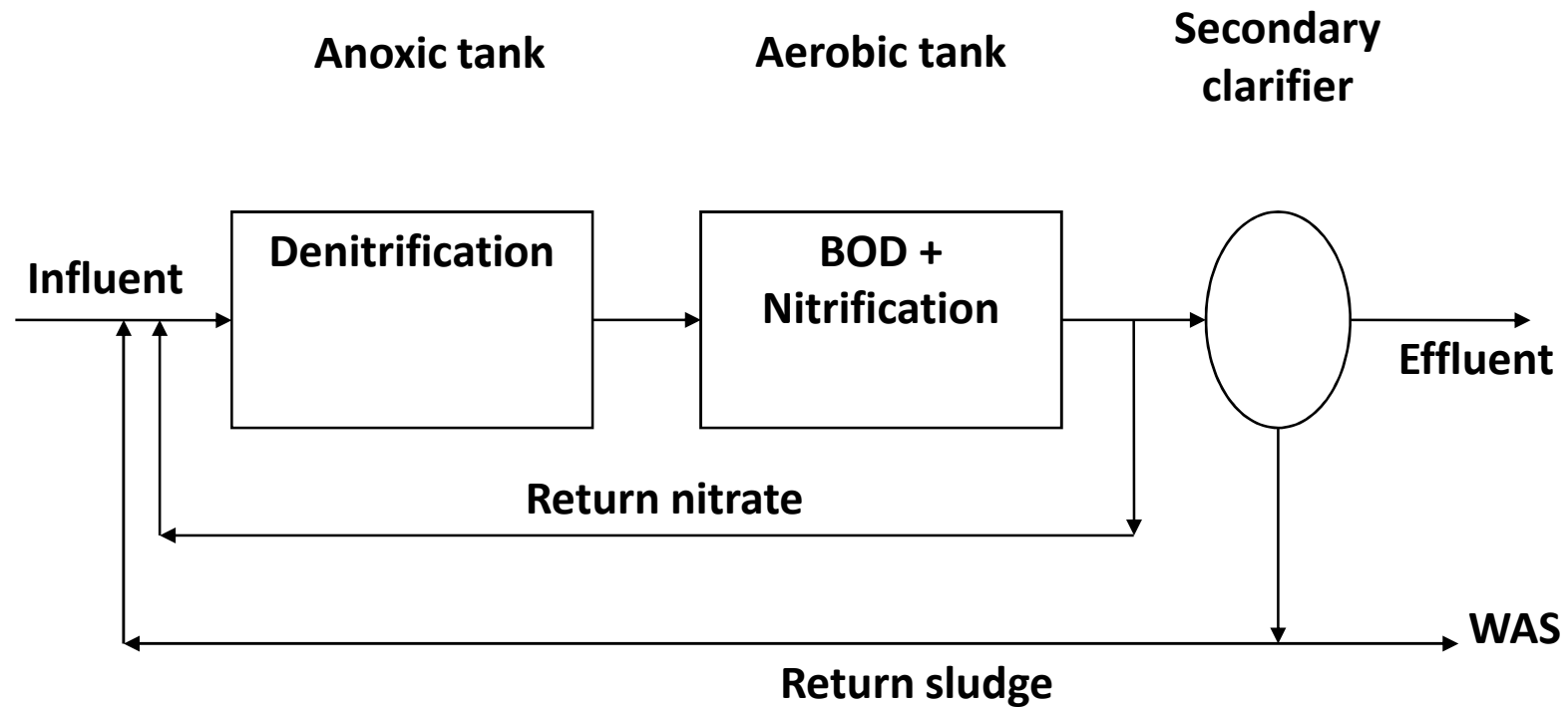
General item	Specific item	Type		
		Conventional	Extended aeration	UASB-activated sludge
Sludge age	Sludge age (day)	4–10	18–30	6–10
F/M ratio	F/M ratio (kgBOD/kgMLVSS·d)	0.25–0.50	0.07–0.15	0.25–0.40
Removal efficiency	BOD (%)	85–95	93–98	85–95
	COD (%)	85–90	90–95	83–90
	Suspended solids (%)	85–95	85–95	85–95
	Ammonia (%)	85–95	90–95	75–90
	Nitrogen (%) ⁽¹⁾	25–30	15–25	15–25
	Phosphorus (%) ⁽¹⁾	25–30	10–20	10–20
	Coliforms (%)	60–90	70–95	70–95
Area required	Area (m ² /inhabitant) ⁽²⁾	0.2–0.3	0.25–0.35	0.2–0.3
Total volume	Volume (m ³ /inhabitant) ⁽³⁾	0.10–0.12	0.10–0.12	0.10–0.12
Energy ⁽⁴⁾	Installed power (W/inhabitant)	2.5–4.5	3.5–5.5	1.8–3.5
	Energy consumption (kW·hour/inhabitant·year)	18–26	20–35	14–20
Volume of sludge ⁽⁵⁾	To be treated (L sludge/inhabitant·d)	3.5–8.0	3.5–5.5	0.5–1.0
	To be disposed of (L sludge/inhabitant·d)	0.10–0.25	0.10–0.25	0.05–0.15
Sludge mass	To be treated (gTS/inhabitant·d)	60–80	40–45	20–30
	To be disposed of (gTS/inhabitant·d)	30–45	40–45	20–30

Biological Nitrogen removal



- **Nitrification in aerobic environment**
 $\text{Organic matter} + \text{O}_2 \rightarrow \text{NH}_3 + \text{CO}_2 + \text{microorganisms}$
 $\text{NH}_3 + 2\text{O}_2 \rightarrow \text{NO}_3^- + \text{H}^+ + \text{H}_2\text{O} + \text{microorganisms}$
- **Denitrification in an anoxic environment (no free oxygen)**
 $\text{Organic matter} + \text{NO}_3^- \rightarrow \text{N}_2 + \text{microorganisms}$

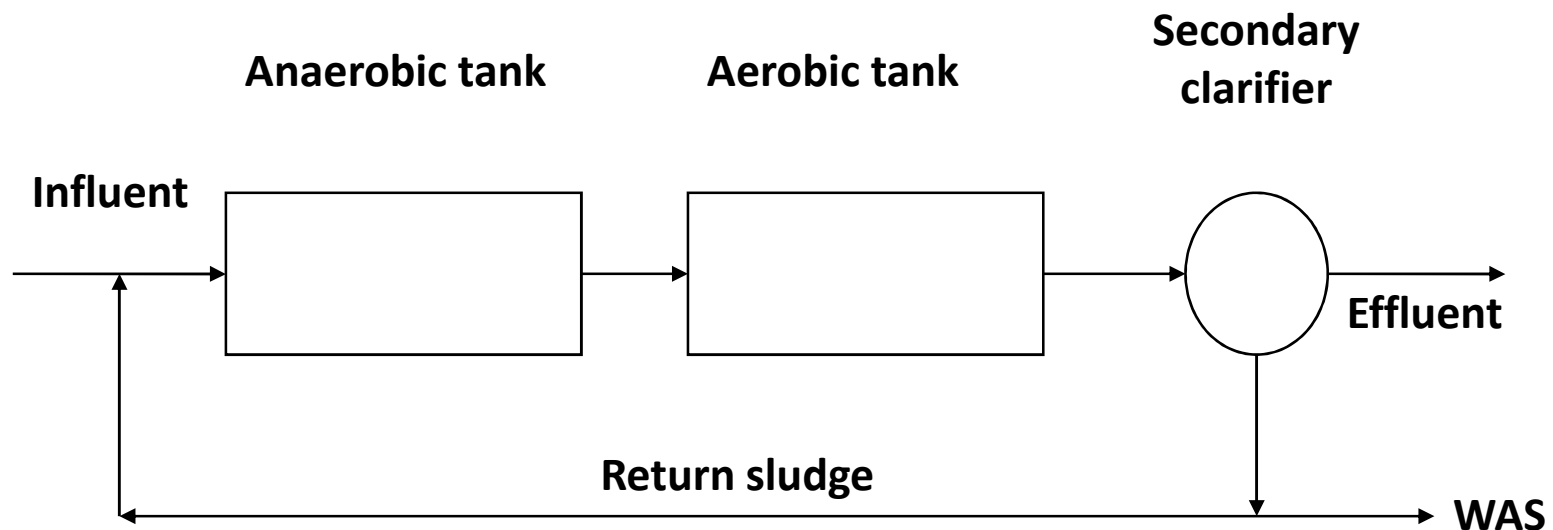
Biological Nitrogen removal



Design criteria for biological nitrogen removal

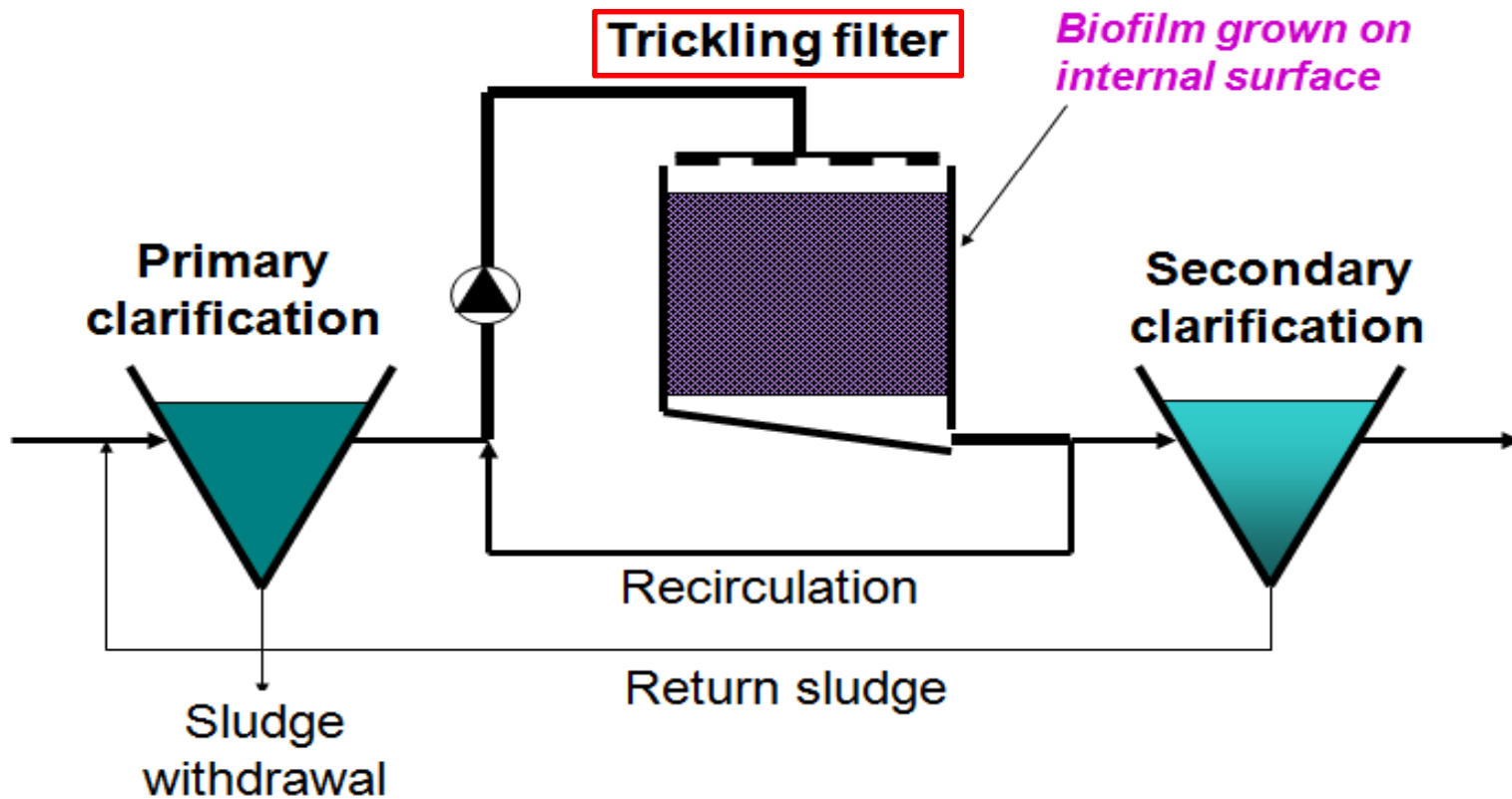
Parameter	System with pre-denitrification	System with post-denitrification	Four-stage Bardenpho
MLVSS (mg/L)	1500–3500	1500–3500	1500–4000
Total θ_c (d)	6–10	6–10	10–30
Aerobic θ_c (d)	≥ 5	≥ 5	≥ 8
HDT – 1 st anoxic zone (hour)	0.5–2.5	–	1.0–3.0
HDT – aerobic zone (hour)	4.0–10.0	5.0–10.0	5.0–10.0
HDT – 2 nd anoxic zone (hour)	–	2.0–5.0	2.0–5.0
HDT – final aerobic zone (hour)	–	–	0.5– 1.0
BOD removal ratio – anoxic zone/ aerobic zone	0.7	0.7	0.7
Sludge recirculation ratio R_{sludge} (Q_r/Q) (%)	60–100	100	100
Internal recirculation ratio R_{int} (Q_{int}/Q) (%)	100–400	–	300–500
Power level in the anoxic zone (W/m^3)	5–10	5–10	5–10
Average DO in the aerobic zone	2.0	2.0	2.0

Two-stage biological P removal



Trickling Filters

- The trickling filter or biofilter consists of a bed of permeable medium of either rock or plastic
- Microorganisms become attached to the media and form a biological layer or fixed film. Organic matter in the wastewater diffuses into the film, where it is metabolized. Periodically, portions of the film slough off the media



Advantages/Disadvantages

Advantages

- Good quality (80-90% BOD₅ removal) for 2-stage efficiency could reach 95%
- Moderate operating costs (lower than activated sludge)
- Withstands shock loads better than other biological processes

Disadvantages

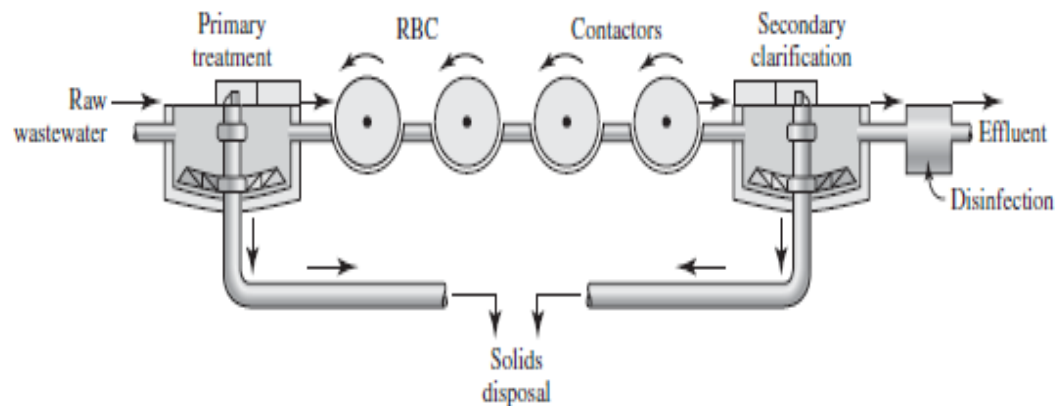
- High capital costs
- Clogging of distributors or beds
- Snail, mosquito and insect problems

Comparison of Different Types of Trickling Filters

Design characteristics	Trickling filter classification				
	Low or standard rate	Intermediate rate	High rate (stone media)	Super rate (plastic media)	Roughing
Hydraulic loading, m/d	1 to 4	4 to 10	10 to 40	15 to 90 ^a	60 to 180 ^a
Organic loading, kg BOD ₅ /d · m ³	0.08 to 0.22	0.26 to 0.51	0.36 to 1.8	0.32 to 1.0	Above 1.0
Recirculation ratio	0	0 to 1	1 to 3	1 to 2	0 to 2
Filter flies	Many	Varies	Few	Few	Few
Sloughing	Intermittent	Varies	Continuous	Continuous	Continuous
Depth, m	1.5 to 3	1.5 to 2.5	1 to 2	Up to 12	1 to 6
BOD ₅ removal, %	80 to 85	50 to 70	40 to 80	65 to 85	40 to 85
Effluent quality	Well nitrified	Some nitrification	No nitrification	No nitrification	No nitrification

Rotating Biological Contactors

- It consists of a series of circular disks of polystyrene or polyvinyl chloride that are submerged in wastewater and rotated slowly through it
- The disk rotation alternately contacts the biomass with the organic material and then with atmosphere for adsorption of oxygen
- Excess solids are removed by shearing forces created by the rotation mechanism



RBC: Advantages/Disadvantages

Advantages

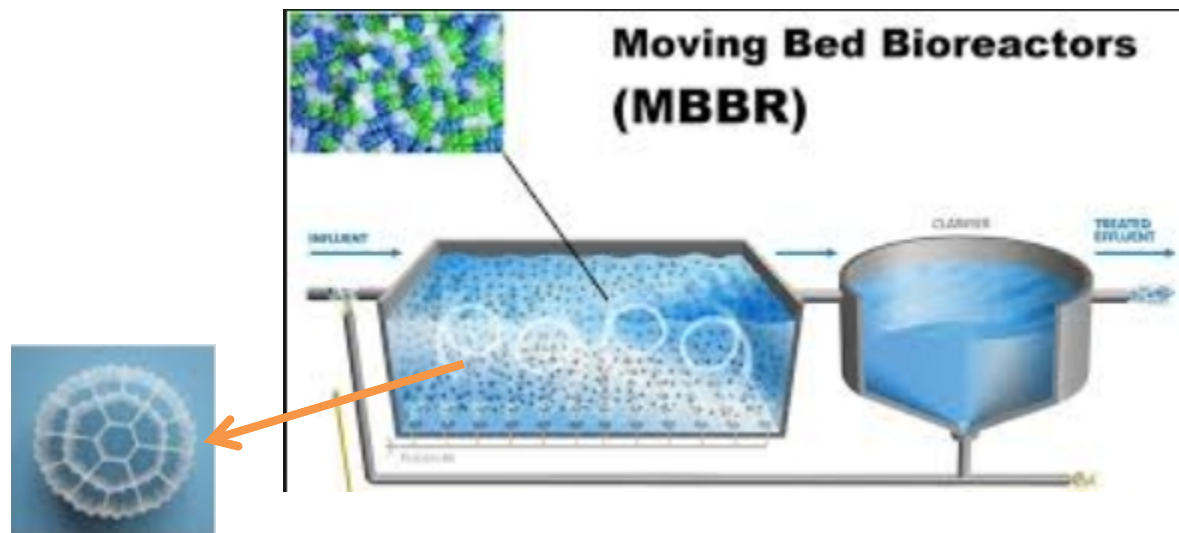
- Short contact periods
- Handles a wide range of flows
- Easily separates biomass from waste stream
- Low operating costs
- Short retention time
- Low sludge production
- Excellent process control

Disadvantages

- Need for covering units installed in cold climate to protect against freezing
- Shaft bearings and mechanical drive units require frequent maintenance

Moving Bed Bioreactor

- The Moving Bed Bio Reactor (MBBR) process utilizes the attached bio-film and provides a smaller footprint solution for wastewater treatment with lower capital and operating costs.
- The suspended biomass carriers are designed to create a large surface area for biofilm growth.
- Uses small, plastic elements to support the growth of biofilm in the reactor
- Benefits include:
 - An enhanced biological wastewater treatment process without increasing the plant footprint
 - MBBR process is ideally suited for retrofit/upgrade of existing installation with minimum changes in the existing setup.
- **Typical design criteria:**
 - MLSS: 100 to 250 mg/L
 - MCRT: 0.15 days
 - HRT: 3 hours

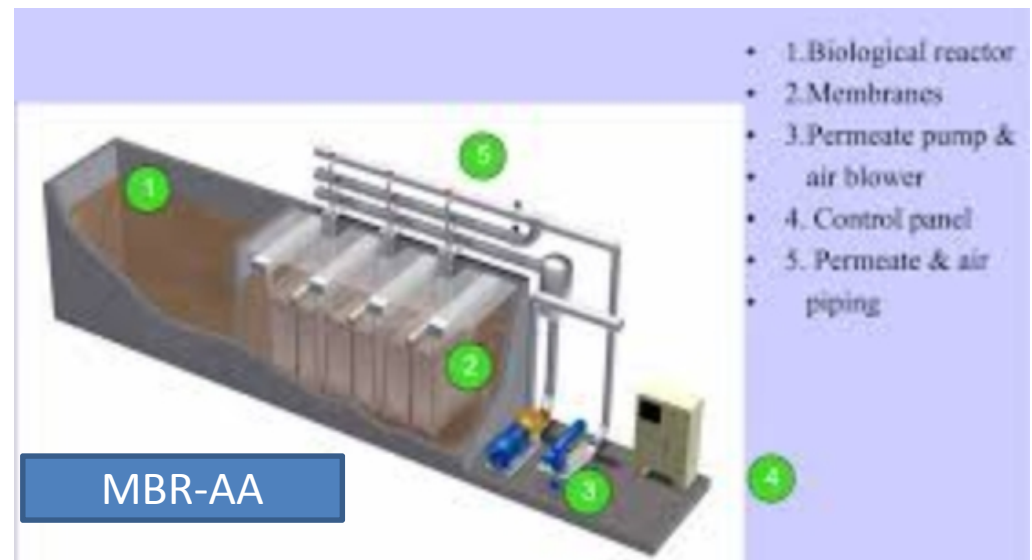


Membrane Bio-Reactor (MBR)

- A new method for wastewater treatment, integrates membrane separation and biotechnology
- Rejects activated sludge and macromolecular organic matter in aerobic tank/MBR tank with membrane separation plant, thus saving the use of secondary sedimentation tank.
- Consequently, the concentration of activated sludge rises greatly, the HRT and the SRT could be controlled separately, and difficult degraded matters are constantly degraded and reacting in reactor.

Effluent quality

BOD ₅ :	< 3	mg/L
TSS:	< 1	mg/L
NH ₃ -N:	< 0.5	mg/L
Total Nitrogen:	< 3	mg/L
Total Phosphorus:	< 0.05	mg/L
Turbidity:	< 0.2	NTU



Advantages/Disadvantages of MBRs

Advantages & Disadvantage

■ **The major ADVANTAGE of MBR:-**

- ✓ High quality effluent.
- ✓ High loading rate capacity.
- ✓ Reducing tertiary filtration and secondary clarifiers.
- ✓ Reducing plant budget.
- ✓ Easy operation
- ✓ Easy modulation

■ **The major DISADVANTAGE of MBR:-**

- ✓ Increase capital cost (Membrane cost)
- ✓ Relatively expensive to install and operate.
- ✓ Membrane may be sensitive some chemical.
- ✓ Require pre-treatment.
- ✓ Frequent membrane monitoring and maintenance.
- ✓ Membrane fouling.

Major Anaerobic Biological Processes

Type of Growth	Common Name	Use
Suspended Growth	Anaerobic Contact Process	Carbonaceous BOD removal
	Upflow Anaerobic Sludge-Blanket (UASB)	Carbonaceous BOD removal
Attached Growth	Anaerobic Filter Process	Carbonaceous BOD removal, waste stabilization (denitrification)
	Expanded Bed	Carbonaceous BOD removal, waste stabilization

Anaerobic Contact Process

- Untreated wastewater is mixed with recycled sludge solids and then digested in a sealed reactor
- The mixture is separated in a clarifier
- The supernatant is discharged as effluent, and settled sludge is recycled

ACP: Advantages/Disadvantages

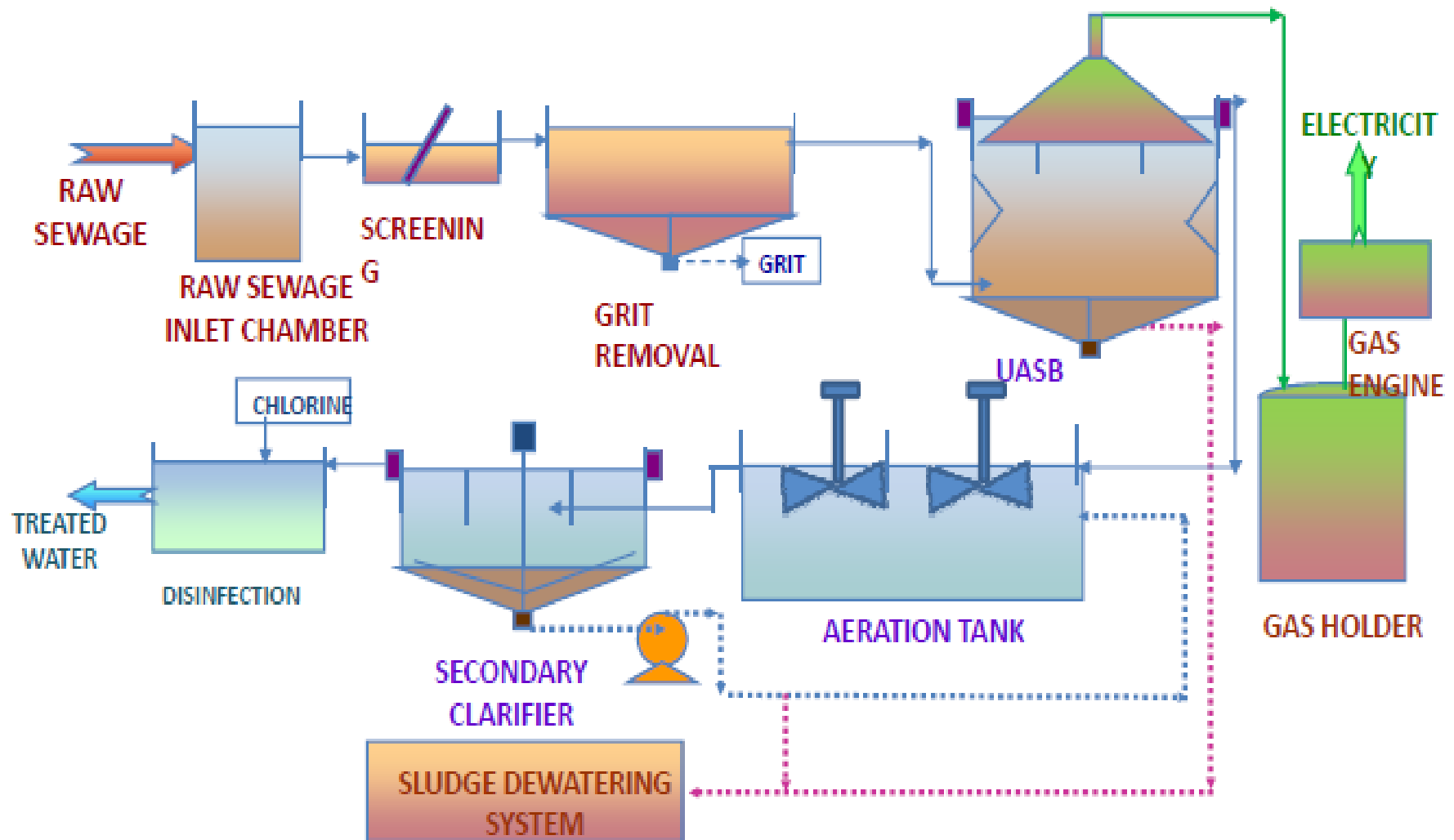
Advantages

- Methane recovery
- Small area required
- Volatile solids destruction

Disadvantages

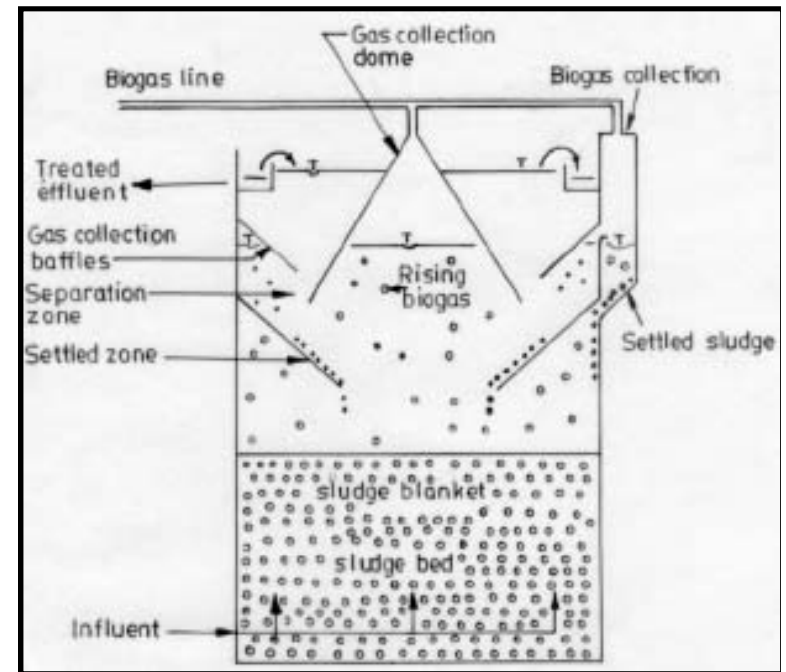
- Heat required
- Effluent in reduced chemical form requires further treatment
- Requires skilled operation
- Sludge to be disposed off is minimal

Upflow Anaerobic Sludge Blanket Reactor



UASB: Upflow Anaerobic Sludge Blanket

- Wastewater flows upward through a sludge blanket composed of biological granules that decompose organic matter
- Some of the generated gas attaches to granules that rise and strike degassing baffles releasing the gas
- Free gas is collected by special domes
- The effluent passes into a settling chamber



Recommended Design Criteria for UASB

COD conc. (mg/l)	OLR $\frac{\text{kg COD}}{\text{m}^3 \text{d}}$	SLR $\frac{\text{kg COD}}{\text{kg VSS} \times \text{d}}$	HRT (m)	Liquid upflow velocity (m/h)	Expected efficiency (%)
<750	1-3	0.1-0.3	6-18	0.25-0.7	70-75
750-3000	2-5	0.2-0.5	6-24	0.25-0.7	80-90
3000-10,000	5-10	0.2-0.6	6-24	0.15-0.7	75-85
>10,000	>10,000	0.2-1	>24	0.15-0.7	75-80

UASB

Advantages

- Requires less power than aerobic processes
- Biogas generated can be used as fuel or electricity

Disadvantages

- UASB alone does not treat the sewage to desirable limits; therefore, downstream aerobic treatment is compulsory
- Requires very large space due to post treatment
- Recovery of biogas is not sufficient to produce substantial electricity in case of municipal wastewater

UASB: Advantages/Disadvantages

Advantages

- Low energy demand
- Low land requirement
- Low sludge production
- Less expensive than other anaerobic processes
- High organic removal efficiency

Disadvantages

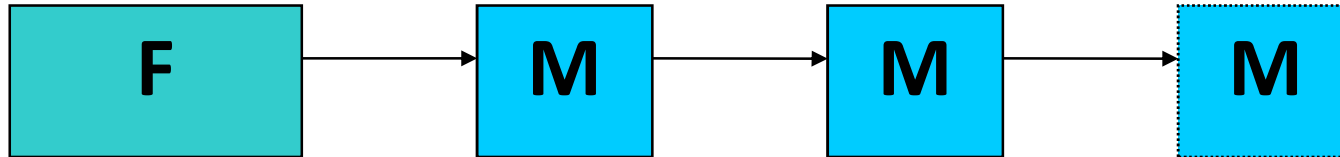
- Long start-up period
- Requires sufficient amount of granular seed sludge for faster start-up
- Significant wash out of sludge during initial phase of process
- Lower gas yield than other anaerobic processes

Alternative Treatments

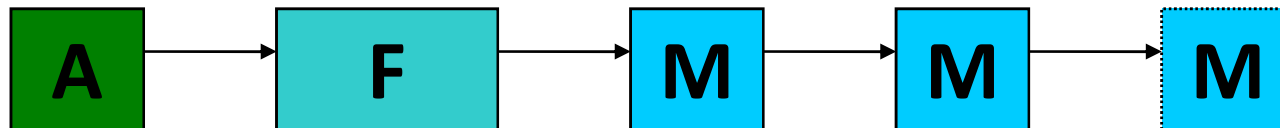
Waste Stabilization Ponds

- **Comprises series of man-made shallow ponds**
- **They can achieve any required degree of treatment at the lowest cost and with minimum of maintenance by unskilled operators**
- **The removal of pathogens is considerably greater than that in other methods of sewage treatment**
- **They are well able to withstand both organic and hydraulic shock loads**
- **They can effectively treat a wide variety of industrial and agricultural wastes**
- **They can easily be designed so that the degree of treatment is readily altered**
- **The method of construction is such that, should at some future time the land be required for some other purpose, it is easily reclaimed**
- **The algae produced in the pond are potential sources of high-protein food which can be conveniently exploited by fish farming**

Waste stabilization ponds



a



b

Pond layouts (a) Weak-medium strength wastes

(b) Strong wastes

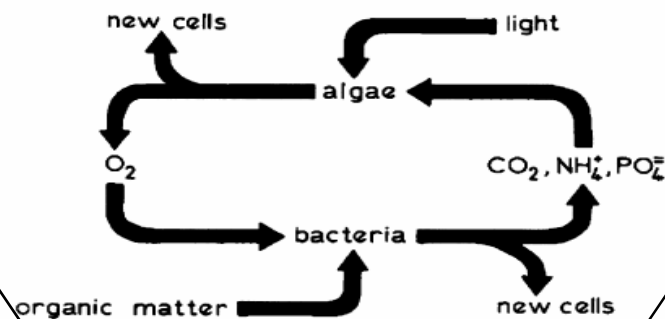
A: Anaerobic pond

F: Facultative pond

M: Maturation pond

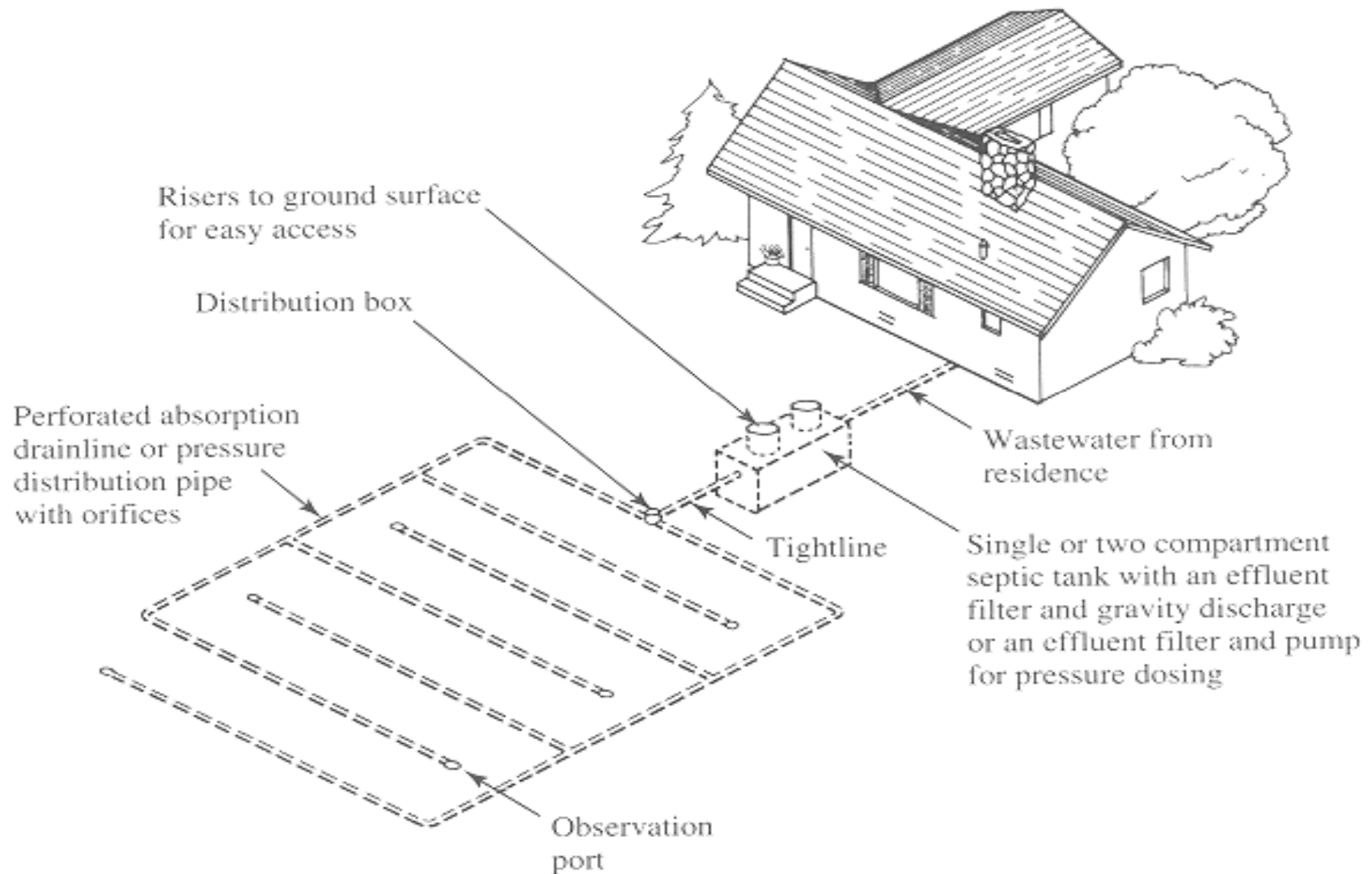
Functions and requirements of ponds

Pond	Main function	Design factors
Anaerobic	Treat strong organic waste (70% reduction in BOD5)	Depth: 2-4 m Detention time: 5 days Temp/pH: $> 15^{\circ}\text{C}/>6$
Facultative	BOD removal	Depth: 1-1.5 m Exposed to wind Large land area Desludging: 10-15 years
Maturation	Fecal coliform removal (99.9%)	Depth: 1-1.5 m



Processes in a facultative pond

Septic Systems



Land Treatment Methods

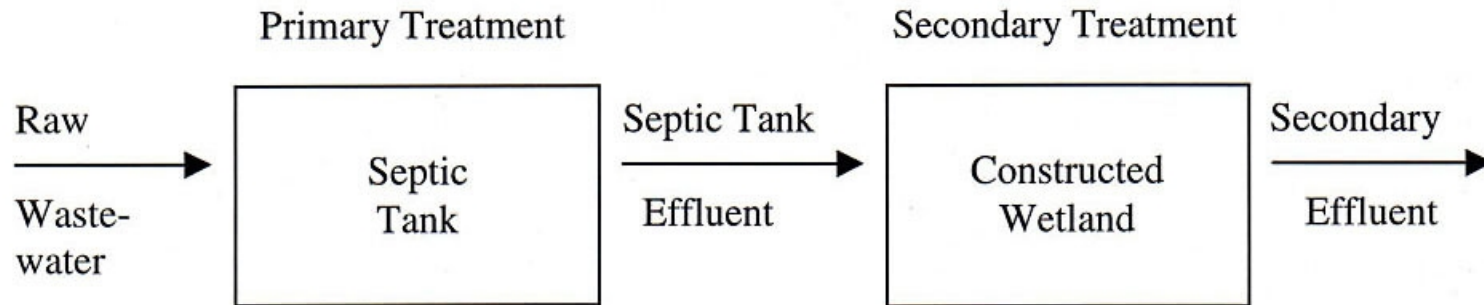
- Application of wastewater on the land
- Plants and soil play roles in pollutant removal
- Three types of application:
 - Slow rate infiltration
 - Rapid infiltration
 - Overland flow

Characteristics of land treatment systems

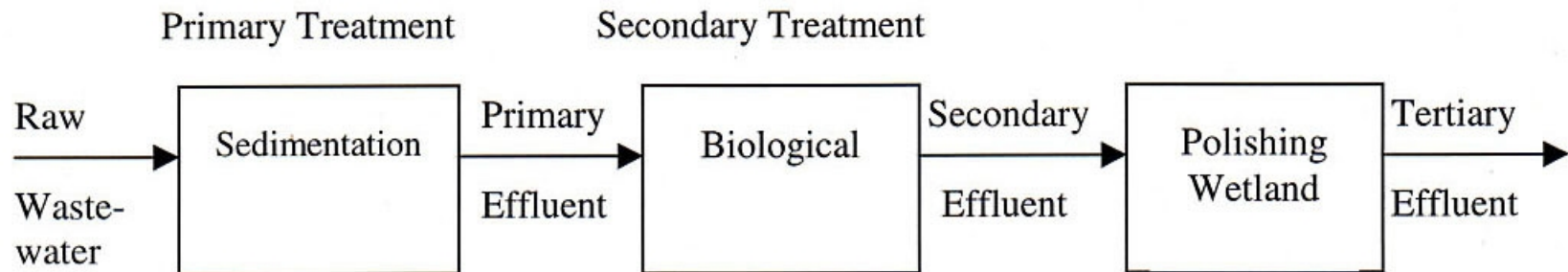
Feature	Slow-rate irrigation	Rapid infiltration	Overland flow
Hydraulic loading rate (cm/d)	0.2-1.5	1.5-3.0	0.6-3.6
Land required for 1000 m ³ /d (1000 x m ²)	63.4-396	3.2-63.4	26.4-159
Soil type	Loamy sand to clay	Sand	Clay to clay loam
Soil permeability	Moderately slow to moderately rapid	Rapid	slow

Constructed Wetlands

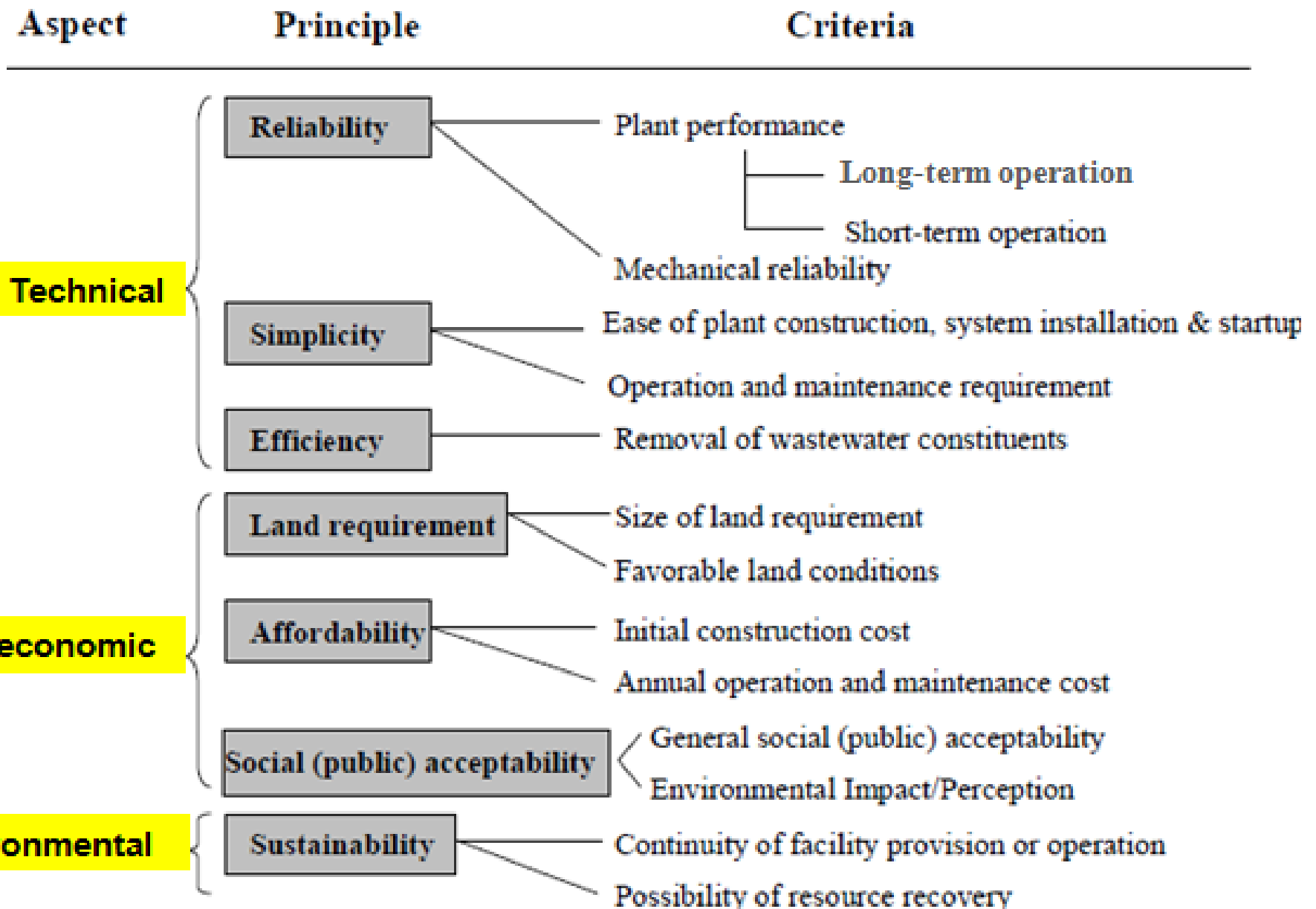
CONSTRUCTED WETLANDS TREATMENT TRAIN



LARGE POLISHING WETLAND TREATMENT TRAIN



Criteria for WWT Technology Selection



Relative Evaluation of Main Sewage Treatment Technologies

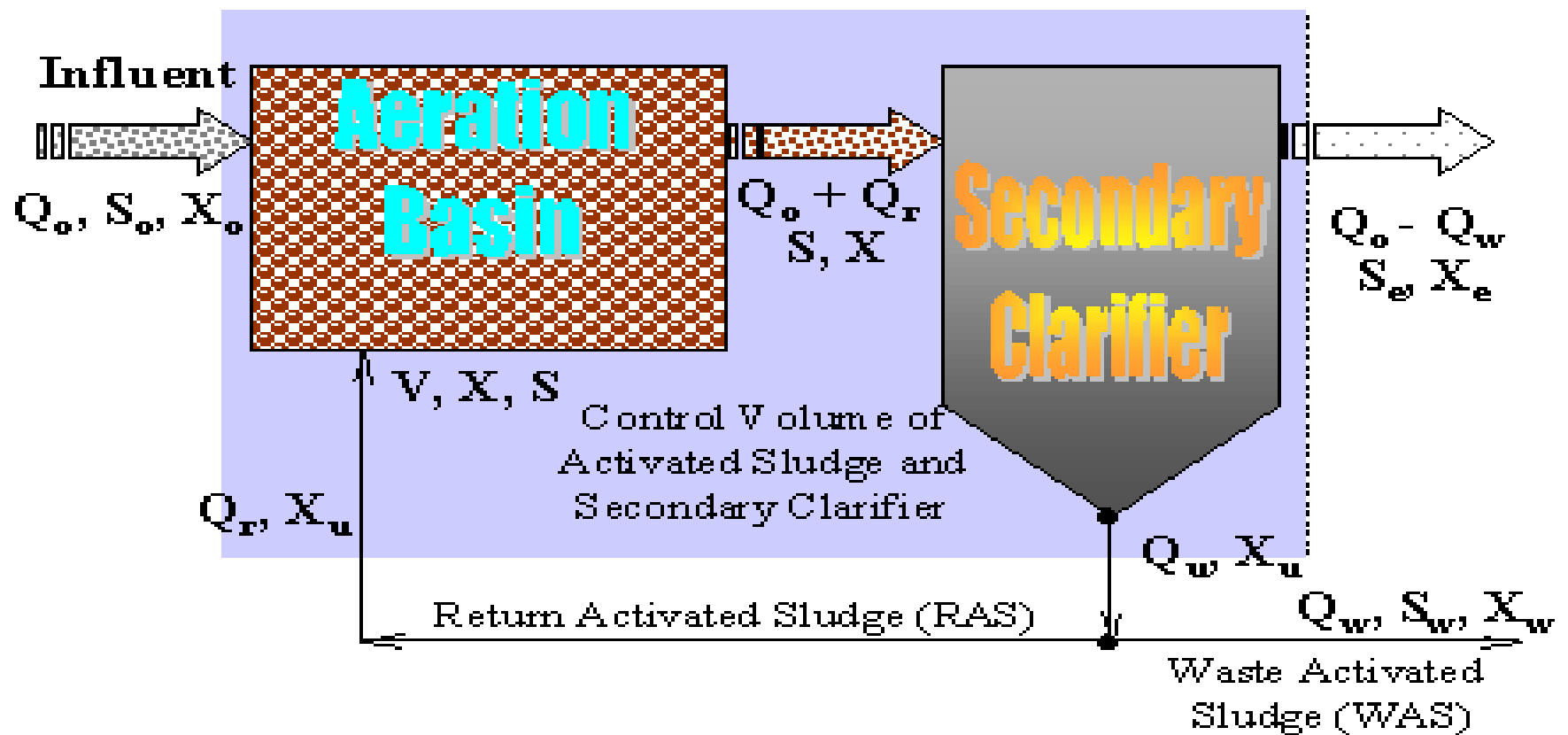
	Removal efficiency			Economy					Resistance capacity to influent variations and shock loads			Reliability	Simpli- city in O&M.	Independence of other charact.for good perform.		Lower possibility of environmental problems			
	BOD	Nutrients	Coliforms	Requirements		Costs		Gener- ation	Flow	Quality	Toxic			Climate	Soil	Bad odours	Noise	Aerosols	Insects and worms
Preliminary treatment	0	0	0	++++	++++	++++	+++	++++	++++	++++	++++	++++	+++	++++	++++	+	+++	++++	+++
Primary treatment	+	+	+	++++	++++	++++	+++	+++	++++	++++	++++	++++	+++	++++	++++	++	+++	++++	+++
Advanced primary treatment	++	+ /++++	++	++++	++++	+++	++	+	++++	++++	++++	++++	+++	++++	++++	+++	+++	++++	+++
Facultative pond	+++	++	++ /++++	+	++++	+++	++++	++++	++++	++++	+++	++++	++++	++	+++	+++	++++	++++	++
Anaerobic pond – facultative pond	+++	++	++ /++++	++	++++	++++	++++	++++	++++	++++	+++	++++	++++	++	+++	+	++++	++++	++
Facultative aerated lagoon	+++	++	++ /++++	++	+++	+++	++++	++++	++++	++++	+++	++++	++++	+++	+++	++++	+	+	+++
Compl. mix aerated – sedim. pond	+++	++	++ /++++	+++	+++	+++	+++	+++	+++	++++	+++	+++	+++	+++	++++	+++	+	+	++
Pond – maturation pond	+++	+++	++++	+	++++	+++	++++	++++	++++	++++	+++	++++	++++	++	+++	+++	++++	++++	++
Pond –high rate pond	+++	++++	+++	++	+++	+++	+++	++++	++++	++++	+++	++++	+++	+++	+++	+++	++	++	++
Pond – algae removal	++++	++	++ /++++	++	++++	+++	++++	+++	++++	++++	+++	++++	+++	+++	+++	+++	++++	++++	++
Slow rate treatment	++++	+++	+++	+	++++	+++	++++	++++	++++	++++	+++	++++	++++	++	+	++	++++	+ /++++	++
Rapid infiltration	++++	+++	+++	+	++++	++++	++++	++++	++++	++++	+++	++++	++++	++	+	++	++++	++++	++
Overland flow	++++	+++	++ /+++	+	++++	+++	++++	++++	++++	++++	+++	++++	++++	+++	++	++	++++	+ /++++	++
Constructed wetlands	++++	++	+++	+	++++	+++	++++	++++	++++	++++	+++	++++	++++	++	++	++	++++	++++	++
Septic tank – anaerobic filter	+++	+	++	++++	++++	+++	+++	+++	+++	+++	++	+++	+++	++	++++	++	+++	++++	+++
UASB reactor	+++	+	++	++++	++++	++++	+++	+++	++	++	++	+++	+++	++	++++	++	+++	++++	+++
UASB reactor – post-treatment	(a)	(a)	(a)	(a)	(a)	(a)	(a)	(a)	(b)	(b)	(b)	(a)	(a)	(a)	(a)	(b)	(a)	(a)	(a)
Conventional activated sludge	++++	++ /++++	++	++++	++	+	++	+	+++	+++	++	++++	+	+++	++++	++++	+	+ /++++	+++
Activated sludge (extended aeration)	++++	++ /++++	++	++++	+	++	+	++	++++	++++	+++	++++	++	++++	++++	++++	+	+ /++++	+++
Sequencing batch reactor	++++	++ /++++	++	++++	+ /++	+	+	+ /++	++++	++++	+++	++++	+++	++++	++++	+++	+	+ /++++	+++
Trickling filter (low rate)	++++	++ /++++	++	+++	++++	+	+++	++	+++	++	++	++++	+++	++	++++	++++	+++	+++	++
Trickling filter (high rate)	++++	++ /+++	++	++++	+++	++	+++	+	++++	+++	+++	++++	+++	++	++++	++++	+++	+++	++
Submerged aerated biofilter	++++	++ /+++	++	++++	++		+++	+	+++	+++	++	++++	++	++++	++++	++++	++	++++	+++
Rotating biological contactor	++++	++ /+++	++	++++	+++	+	+++	+	+++	+++	++	+++	+++	++	++++	++++	+++	++++	++

Activated Sludge Design (Complete Mix Reactor)

Activated Sludge Principles

- Wastewater is aerated in a tank
- Bacteria are encouraged to grow by providing
 - Oxygen
 - Food (BOD)
 - Nutrients
 - Correct temperature
 - Time
- As bacteria consume BOD, they grow and multiply
- Treated wastewater flows into secondary clarifier
- Bacterial cells settle, removed from clarifier as sludge
- Part of sludge is recycled back to activated sludge tank, to maintain bacteria population
- Remainder of sludge is wasted

Schematic of activated sludge unit

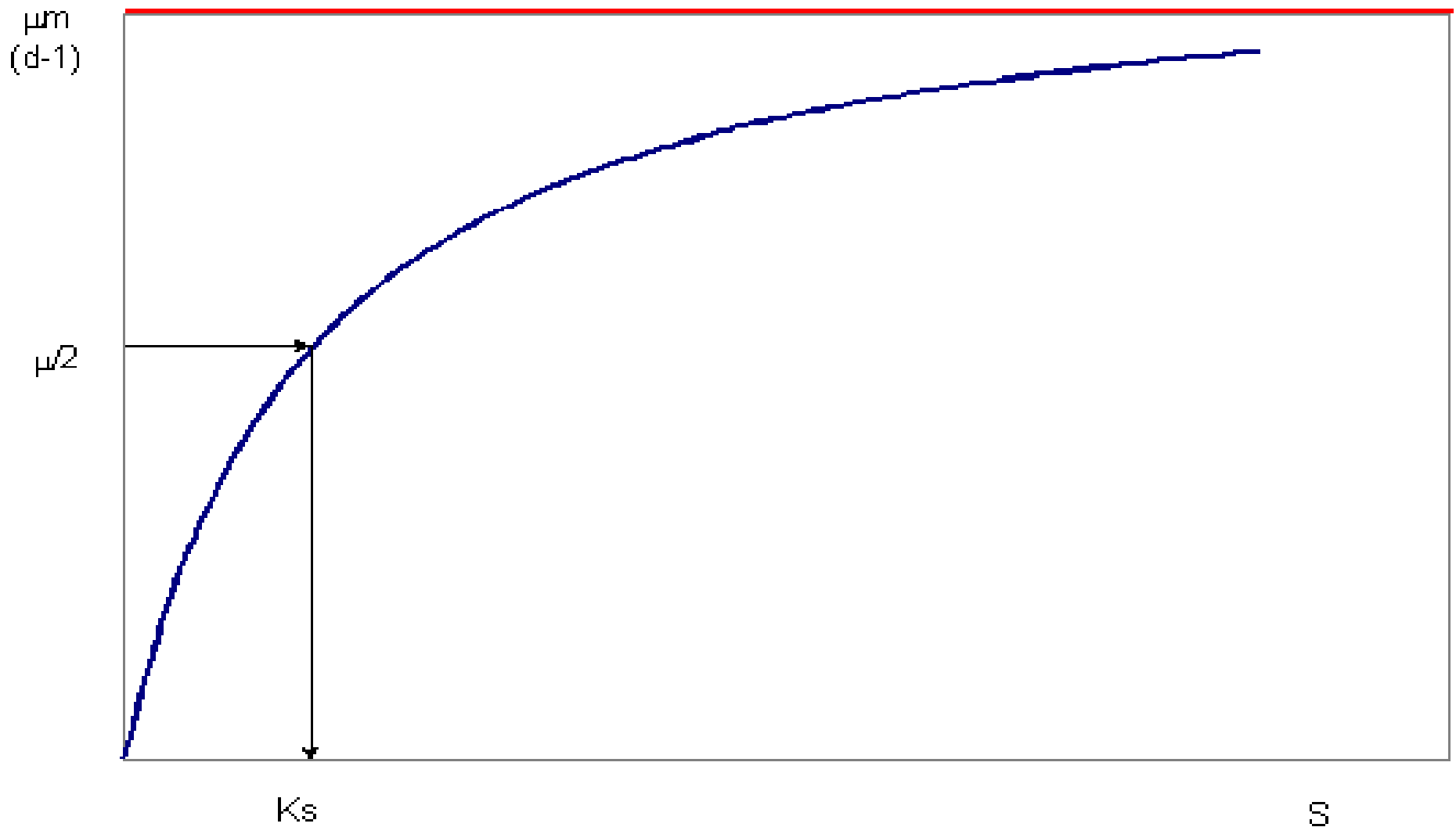


Kinetics of Microbial Growth

- Biochemical reaction
- $\text{Food} + \text{O}_2 + \text{nutrients} \xrightarrow{\text{Bacteria}} \text{CO}_2 + \text{NH}_3 + \text{new biomass} + \text{other end products}$
- Biomass concentration.
- The concentration of biomass, X (mg/L), increases as a function of time due to conversion of food to biomass:
- $\frac{dX}{dt} = \mu X$

Where μ is the specific growth rate constant (d^{-1}). This represents the mass of cells produced/mass of cells per unit of time.

Effect of substrate concentration on growth rate constant



Monod Kinetics

- Growth rate

- Growth rate constant, μ , is a function of the substrate concentration, S .
- Two constants are used to describe the growth rate
 - μ_m (mg/L) is the maximum growth rate constant (the rate at which the substrate concentration is not limiting)
 - K_s is the half-saturation constant (mg/L) (i.e., concentration of S when $\mu = \mu_m/2$)

-

$$\mu = \mu_m \frac{S}{K_s + S}$$

- Biomass production

$$\frac{dX}{dt} = \text{growth rate} - \text{death rate} = \mu X - k_d X$$

Where k_d represents the endogenous decay rate (d^{-1}) (i.e., microorganism death rate).

Substituting the growth rate constant:

$$\frac{dX}{dt} = \left(\frac{\mu_m S}{K_s + S} \right) X - k_d X$$

- Substrate utilization

$$\frac{dS}{dt} = \frac{1}{Y} \frac{dX}{dt}$$

Where Y is the yield factor (mg of biomass produced/mg of food consumed)

- Y range:
 - Aerobic: 0.4 - 0.8 mg/mg

$$\frac{dS}{dt} = \frac{1}{Y} \frac{dX_{\text{growth}}}{dt} = \frac{1}{Y} \left(\frac{\mu_{\text{max}} SX}{K_s + S} \right)$$

- Food to microorganism ratio (F/M)
 - Represents the daily mass of food supplied to the microbial biomass, X, in the mixed liquor suspended solids, MLSS
 - Units are Kg BOD₅/Kg MLSS/day

$$\frac{F}{M} = \frac{BOD_5 (Kg / m^3) \times Influent \cdot flow (m^3 / d)}{ReactorSolids (Kg / m^3) \times ReactorVolume (m^3)}$$

$$\frac{F}{M} = \frac{S_o Q_o}{XV}$$

- Since the hydraulic retention time, $\theta = V/Q_0$, then

$$\frac{F}{M} = \frac{S_0}{\theta X}$$

Typical range of F/M ratio in activated sludge units

<i>F/M</i> <i>Kg BOD₅/Kg MLSS/day</i>	
<i>Extended aeration</i>	0.03 - 0.8
<i>Conventional</i>	0.8 - 2.0
<i>High rate</i>	> 2.0

Design parameters for activated sludge processes

Process	θ_x (d)	θ (h)	F/M	Q_r/Q	X (mg/L)
Conventional	5-15	4-8	0.2-0.4	0.25-5	1,500-3,000
Complete-mix	5-15	3-5	0.2-0.6	0.25-1	3,000-6,000
Step-aeration	5-15	3-5	0.2-0.4	0.25-0.75	2,000-3,500
Modified-aeration	0.2-0.5	1.5-3	1.5-5.0	0.05-0.15	200 – 500
Contact-stabilization	5-15	0.5-1 3-6	0.2-0.6	0.25-1	1,000-3,000 4,000-10,000
Extended-aeration	20-30	18-36	0.05-0.15	0.75-1.5	3,000-6,000
High-rate aeration	5-10	0.5-2	0.4-1.5	1-5	4,000-10,000
Pure-oxygen	8-20	1-3	0.25-1.0	0.25-0.5	6,000-8,000

Operational characteristics of activated sludge processes

<i>Process</i>	<i>Flow model</i>	<i>Aeration system</i>	<i>BOD₅ removal efficiency (%)</i>
<i>Conventional</i>	Plug-flow	Diffused air, mechanical aerators	85-95
<i>Complete-mix</i>	Complete-mix	Diffused air, mechanical aerators	85-95
<i>Step-aeration</i>	Plug-flow	Diffused air	85-95
<i>Modified-aeration</i>	Plug-flow	Diffused air	60-75
<i>Contact-stabilization</i>	Plug-flow	Diffused air, mechanical aerators	80-90
<i>Extended-aeration</i>	Complete-mix	Diffused air, mechanical aerators	75-95
<i>High-rate aeration</i>	Complete-mix	Diffused air, mechanical aerators	75-90
<i>Pure-oxygen</i>	Complete-mix	Mechanical aerators	85-95

Activated Sludge Design Equations

Mass balance of biomass production

- Influent biomass + biomass production = effluent biomass + sludge wasted

$$Q_o X_o + V \frac{dX}{dt} = (Q_o - Q_w) X_e + Q_w X_w$$

- Substitute biomass production equation

$$Q_o X_o + V \left(\frac{\mu_m S}{K_s + S} \right) X - k_d X = (Q_o - Q_w) X_e + Q_w X_w$$

- Assume that influent and effluent biomass concentrations are negligible and solve

$$\frac{\mu_m S}{K_s + S} = \frac{Q_w X_w}{VX} + k_d$$

Mass balance of food substrate

- Influent substrate + substrate consumed = effluent substrate + sludge wasted substrate

$$Q_o S_o + V \frac{dS}{dt} = (Q_o - Q_w) S_e + Q_w S_w$$

- Substitute substrate removal equation

$$Q_o S_o + \frac{V}{Y} \left(\frac{\mu_m X S}{K_s + S} \right) = (Q_o - Q_w) S_e + Q_w S_w$$

- Assume that no biochemical action takes place in clarifier. Therefore the substrate concentration in the aeration basin is equal to the substrate concentrations in the effluent and the waste activated sludge. Solve:

$$\frac{\mu_m S}{K_s + S} = \frac{Q_o Y}{V X} (S_o - S)$$

Overall equations

- Combine the mass balance equations for food and biomass:

$$\frac{Q_w V_w}{VX} + k_d = \frac{Q_o Y}{VX} (S_o - S)$$

- The cell residence time is:

$$\theta_c = \frac{VX}{Q_w X_w}$$

- and the hydraulic retention time is $\theta = V/Q_o$; Substitute and rearrange:

$$X = \frac{\theta_c}{\theta} Y \left(\frac{S_o - S}{1 + k_d \theta_c} \right)$$

- Compute the F/M ratio

$$\frac{F}{M} = \frac{S_o}{\theta X} \qquad \frac{F}{M} = \frac{S_o}{(V/Q_o)X} = \frac{Q_o S_o}{VX}$$

Exercise

- Given:
 - Design flow = 11250 m³/d
 - Influent BOD₅ = 200 mg/L
 - Required effluent BOD₅ < 10 mg/L
 - Yield coefficient = 0.6
 - Decay rate = 0.06 /d
- Assumptions:
 - MLSS in aeration basin (X) = 3500 mg/L
 - MLSS in the clarifier sludge (X_w) = 15000 mg/L
 - Mean cell residence time = 10 days

Exercise- Required

- Determine
 - The aeration basin volume (V)
 - The hydraulic retention time (θ)
 - The sludge volume wasted daily (Q_w)
 - The mass of sludge wasted daily ($Q_w X_w$)
 - The fraction of sludge recycled (Q_r/Q_o)
 - The F/M ratio

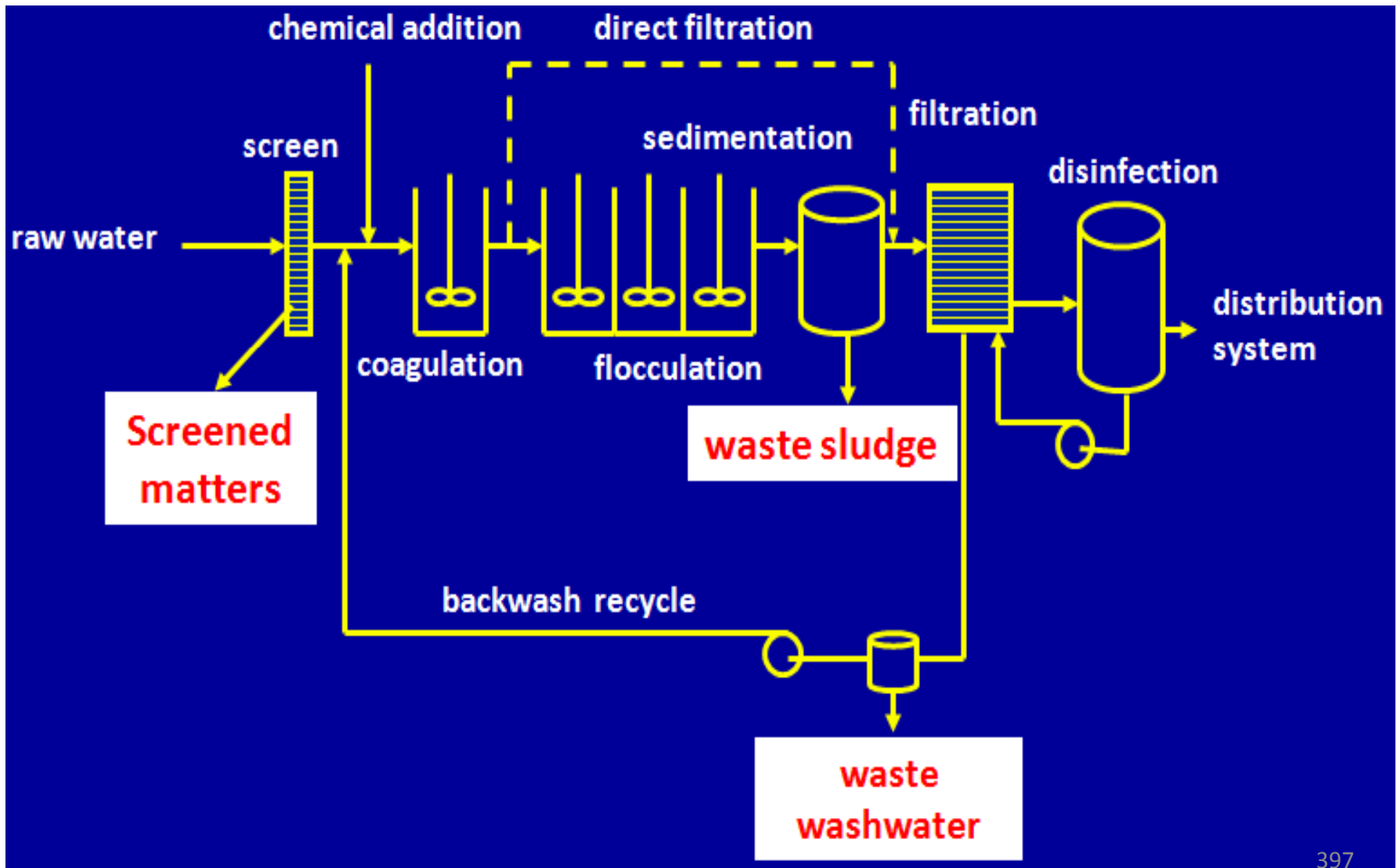
Assignment-8

- Make a review on analysis and selection of wastewater treatment processes

(Refer File: [UN_WWR_2017_Wastewater_Untapped Resource.pdf](#), Chapter #4.7)

8. RESIDUALS MANAGEMENT

Sources of Water Treatment Plant Residuals



Sources of WTP Residuals

- Screens
- Pre-sedimentation
- Coagulation sludges
- Softening sludges
- Slow sand filter scrapings
- Filter waste washwater
- Filter-to-waste water

Typical production of residuals in WTP

Type of residual	Percent of plant flow	
	Range	Typical
Alum sludge	0.08-0.3	0.1
Iron sludge	0.08-0.3	0.1
Filter backwash water	2-5	4
Lime softening sludge	0.3-6	4
Microfiltration backwash water	2-8	6
Reverse osmosis/nanofiltration concentrate	10-50	10-50
Ion exchange brine	1.5-10	5-8

- 3-5 % of the volume of the raw water entering a conventional WTP may end up as a solid and liquid residual.
- Major portion of the cost with residual management is associated with transport and ultimate disposal.

Properties used to characterize WTPR

Physical	Chemical	Biological
Total solids	BOD	Bacteria
Dry density	COD	Protozoan cysts and oocysts
Wet density	pH	Helminths
Specific gravity of dry solids	Alum content	Viruses
Specific resistance	Calcium, magnesium content	
Dynamic viscosity	Iron content	
Initial settling velocity	Silica and inert material	
	Trace constituents	

$$\text{Volume of sludge} = \frac{W_s}{\rho_w S_{gs} P_s}$$

Where,

W_s = weight of total dry solids, kg

ρ_w = density of water, kg/m³

S_{gs} = specific gravity of sludge

P_s = percent of solids expressed as a decimal

Coagulation Sludges

Process	Unit	Range	Typical values
Coagulation			
• Alum, $\text{Al}_2(\text{SO}_4)_3 \cdot 14\text{H}_2\text{O}$	kg sludge/kg coagulant	0.33-0.44	0.33
• Ferric sulfate, $\text{Fe}_2(\text{SO}_4)_3$	Kg sludge/kg coagulant	0.59-0.8	0.59
• Ferric chloride, FeCl_3	Kg sludge/kg coagulant	0.48-1.0	0.48
• PACl	Kg sludge/kg PACl	$(0.0372-0.0489) \times \text{Al}\%$	$0.0489 \times \text{Al}\%$
Polymer addition	Kg sludge/kg coagulant	1.0	1.0
Turbidity removal	mg TSS/NTU removed	0.9-1.5	1.25
Softening			
• Ca^{2+} (as CaCO_3)	Kg sludge/kg Ca^{2+} removed	2.0	2.0
• Mg^{2+} (as $\text{Mg}(\text{OH})_2$)	Kg sludge/kg Ca^{2+} removed	2.6	2.6

Total sludge = Sludge from turbidity + sludge from coagulant + sludge from other chemical or materials

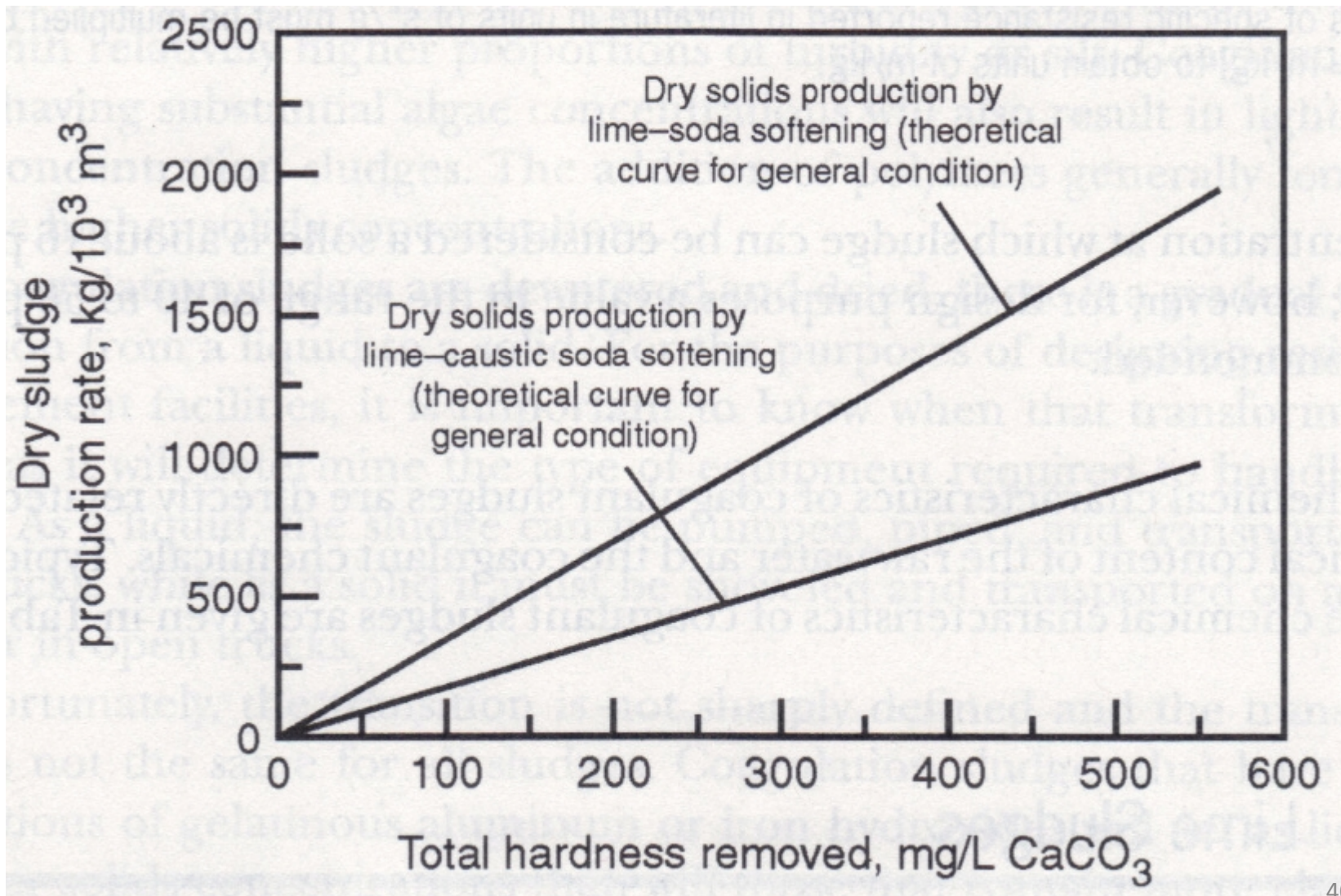
Example- Sludge determination

Determine the mass and volume of sludge produced from alum precipitation for the removal of turbidity. Assume the following conditions apply: flow rate = $0.5 \text{ m}^3/\text{s}$; average raw water turbidity is 25 NTU; average alum dose is 30 mg/L; the sludge solids concentration is 5% with a corresponding specific gravity of 1.05, and the temperature is 15°C ($\rho_w = 998.2 \text{ kg/m}^3$). Assume the ratio between total suspended solids and turbidity expressed as NTU is 1.25, that 0.44 kg of alum sludge is produced per kilogram of alum added.

Typical properties of coagulant sludges

Item	Alum sludge	Iron sludge
Physical properties		
• Volume (% water treated)	0.1-0.3	0.1-0.3
• Total solids (%)	0.1-4	0.25-3.5
• Dry density (kg/m ³)	1200-1500	1200-1800
• Wet density (kg/m ³)	1025-1100	1050-1200
• Specific resistance (m/kg)	10-50 x 10 ¹¹	40-50 x 10 ¹¹
• Viscosity at 20°C (N.s/m ²)	2-4 x 10 ⁻³	2-4 x 10 ⁻³
• Initial settling velocity (m/h)	2.2-5.5	1-5
Chemical properties		
• BOD (mg/L)	30-300	30-300
• COD (mg/L)	30-5000	30-5000
• pH	6-8	6-8
• Solids		
• Al ₂ O ₃ .5.5H ₂ O (%)	15-40	
• Fe (%)		4-21
• Silicates and inert materials (%)	35-70	35-70
• Organics (%)	5-15	5-15

Lime Sludges



Typical properties of lime-softening sludges

Item	Range of values
Physical properties	
• Volume (% water treated)	0.3-6
• Total solids (%)	2-15
• Dry density (kg/m ³)	1100
• Wet density (kg/m ³)	1920
• Specific resistance (m/kg)	12×10^{10}
• Viscosity (N.s/m ²)	$5-7 \times 10^{-3}$
• Initial settling velocity (m/h)	0.4-3.6
Chemical constituents	
• BOD (mg/L)	0-low
• COD (mg/L)	0-low
• pH	10.5-11.5
• Total dissolved solids (%)	2-15
• Solids	
• CaO ₃ (%)	40-80
• Silicates and inert materials (%)	8-12
• Organics (%)	5-8

Granular Media Waste Washwater

- Types:
 - Backwash water
 - Filter-to-waste washwater
- Could be 2-5 % of the total amount of water processed in the treatment plant

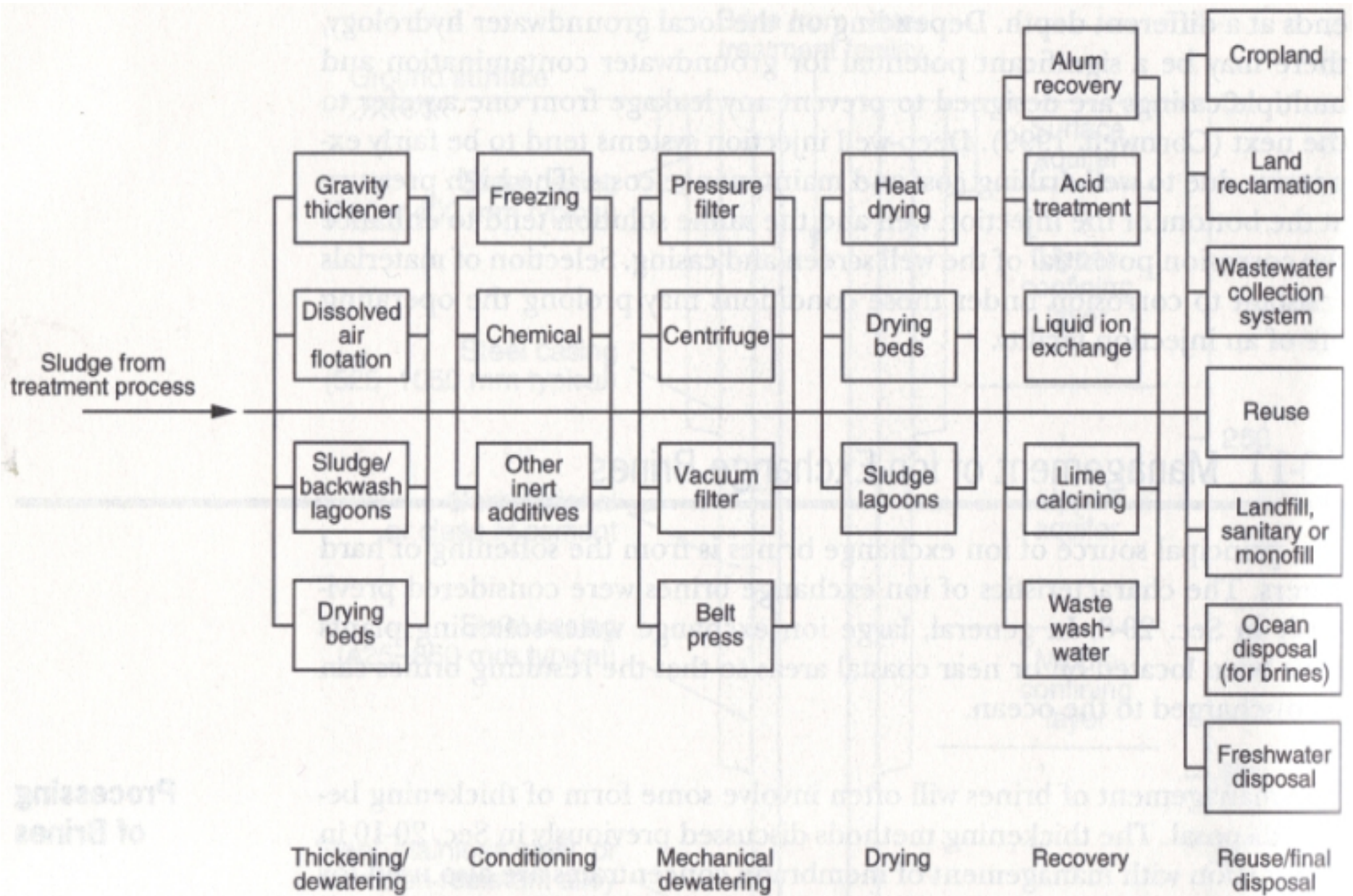
Typical properties of Granular Filter Waste Washwater

Item	Range of values
Physical properties	
• Volume (% water treated)	0.3-6
• Total solids (%)	2-15
• Specific gravity, sg	1.00-1.025
• Specific resistance (m/kg)	$11-120 \times 10^{10}$
• Viscosity (N.s/m ²)	$5-7 \times 10^{-3}$
• Initial settling velocity (m/h)	0.06-0.15
Chemical constituents	
• BOD (mg/L)	2-10
• COD (mg/L)	20-200
• pH	7.2-7.8
• Solids	
• CaO ₃ (%)	20-50
• Silicates and inert materials (%)	30-40
• Organics (%)	15-22

Options for WTP Residuals Management

- Treatment
- Disposal to surface water
- Discharge to wastewater collection system

Unit Operations and Process for WTP Sludges Management



Typical Sludge Management Processes

- Alum sludge

Gravity thickening → Chemical conditioning →
Centrifugation → Sanitary landfill or monofill
Sludge lagoons → Decant recovery & recycle →
Sanitary or monofill landfill or sewer system

- Lime sludge

Gravity thickening → Chemical conditioning →
Centrifugation → Sanitary or monofill landfill
Sludge lagoons → Drying beds → cropland
application or monofill landfill

Wastewater Treatment Plant Sludges

- A. Grit sludge:
 - It is all the solids collected in the grit removal chamber, because grit can be easily drained and is relatively stable in terms of biological activity, it does not need treatment and is generally disposed directly to landfills.
- B. Primary or raw sludge:
 - Primary sludge is the sludge taken from the bottom of the primary settling tanks. It contains from 3-8% solids. This sludge is composed of settleable raw solids. Almost 7% of the primary sludge is organic.
- C. Secondary Sludge:
 - This sludge consists of microorganisms and inert materials that has been wasted from the secondary treatment processes. Thus 9% of this sludge is organic and contains from 0.8 to 2% solids.

Composition of Wastewater TP Sludge

- Predominantly water
- Micro-organisms
- Viruses, pathogens, germs in general
- Organic particles, heavily bio-degradable
- Organic compounds, inert, adsorbed to sludge flocs
- Heavy metals
- Micro-pollutants

Physical Characteristics of Wastewater Sludges of Different Units

Treatment operation or process	Specific gravity of solids	Specific gravity of sludge	Dry solids, lb/10 ³ gal		Dry solids, kg/10 ³ m ³	
			Range	Typical	Range	Typical
Primary sedimentation	1.4	1.02	0.9–1.4	1.25	110–170	150
Activated sludge (waste biosolids)	1.25	1.005	0.6–0.8	0.7	70–100	80
Trickling filter (waste biosolids)	1.45	1.025	0.5–0.8	0.6	60–100	70
Extended aeration (waste biosolids)	1.30	1.015	0.7–1.0	0.8 ^a	80–120	100 ^a
Aerated lagoon (waste biosolids)	1.30	1.01	0.7–1.0	0.8 ^a	80–120	100 ^a
Filtration	1.20	1.005	0.1–0.2	0.15	12–24	20
Algae removal	1.20	1.005	0.1–0.2	0.15	12–24	20
Chemical addition to primary tanks for phosphorus removal						
Low lime (350–500 mg/L)	1.9	1.04	2.0–3.3	2.5 ^b	240–400	300 ^b
High lime (800–1600 mg/L)	2.2	1.05	5.0–11.0	6.6 ^b	600–1300	800 ^b
Suspended growth nitrification	—	—	—	—	—	— ^c
Suspended growth denitrification	1.20	1.005	0.1–0.25	0.15	12–30	18
Roughing filters	1.28	1.02	—	— ^d	—	— ^d

Typical Chemical Composition of Untreated Sludge & Digested Bio-solids

Item	Untreated primary sludge		Digested primary sludge		Untreated activated sludge
	Range	Typical	Range	Typical	Range
Total dry solids (TS),%	5-9	7	2-5	4	0.8-1.2
Volatile solids (% of TS)	60-80	65	30-60	40	59-88
Grease and fats (% of TS):					
Ether soluble	6-30	—	5-20	18	—
Ether extract	7-35	—	—	—	5-12
Protein (% of TS)	20-30	25	15-20	18	32-41
Nitrogen (N, % of TS)	1.5-4	2.5	1.6-3.0	3.0	2.4-5.0
Phosphorus (P_2O_5 , % of TS)	0.8-2.8	.6	1.5-4.0	2.5	2.8-11
Potash (K_2O , % of TS)	0-1	0.4	0-3.0	1.0	0.5-0.7
Cellulose (% of TS)	8-15	10	8-15	10	—
Iron (not as sulfide)	2.0-4.0	2.5	3.0-8.0	4.0	—
Silica (SiO_2 , % of TS)	15-20	—	10-20	—	—
pH	5.0-8.0	6.0	6.5-7.5	7.0	6.5-8.0
Alkalinity (mg/L as $CaCO_3$)	500-1500	600	2500-3500	3000	580-1100
Organic acids (mg/L as HAc)	200-2000	500	100-600	200	1100-1700
Energy content, kJ TS/kg	23,000-29,000	25,000	9000-14,000	12,000	19,000-23,000

Goals of WWTP Sludge Treatment

Volume reduction

- Thickening
- Dewatering

Elimination of pathogenic germs

- If used in agriculture as fertiliser or compost

Stabilisation of organic substances

- Gas production
- Reduction of dry content
- Improvement of dewatering
- Reduction of odour

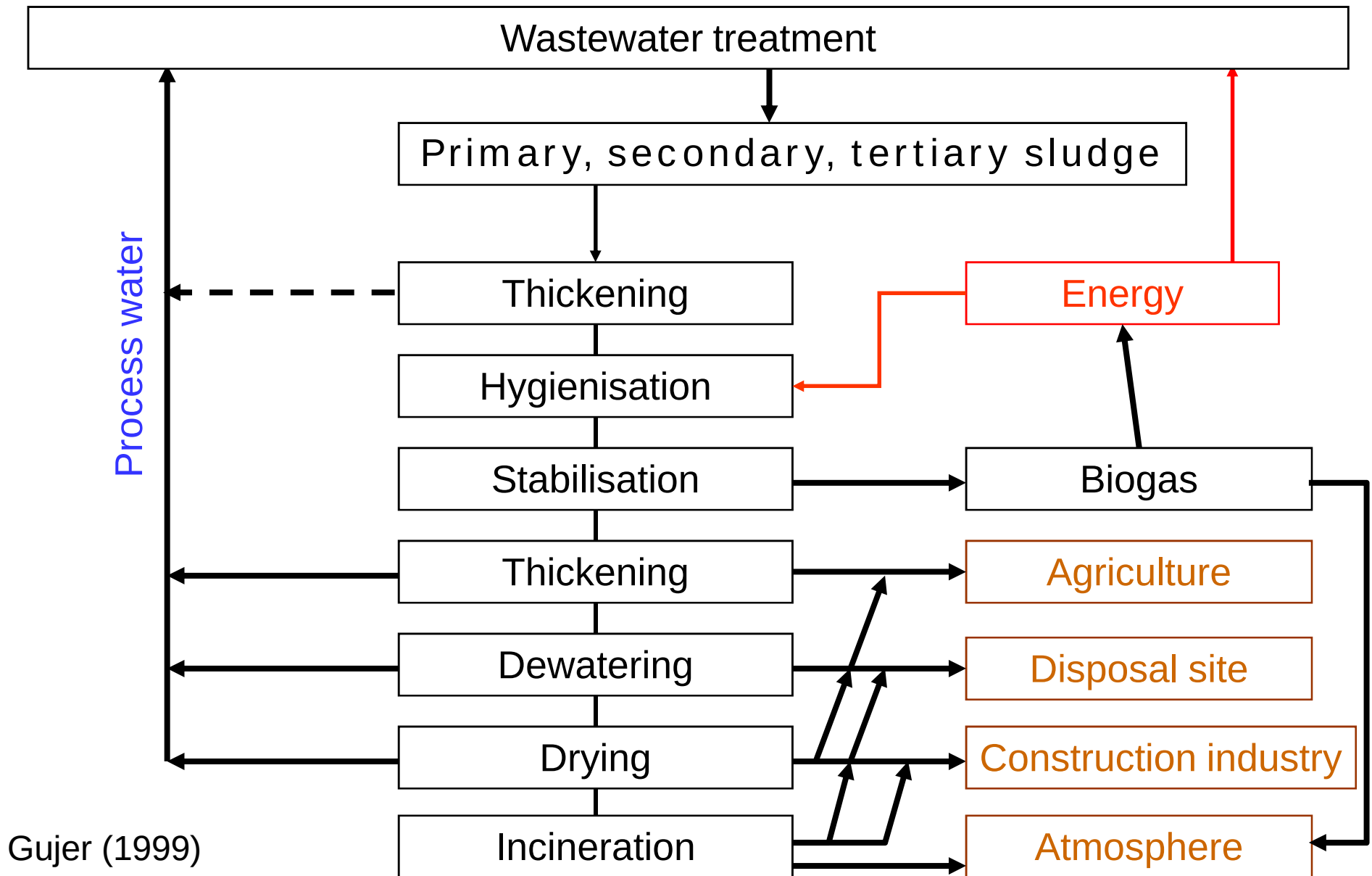
Recycling of substances

- Nutrients, fertiliser
- Humus
- Biogas

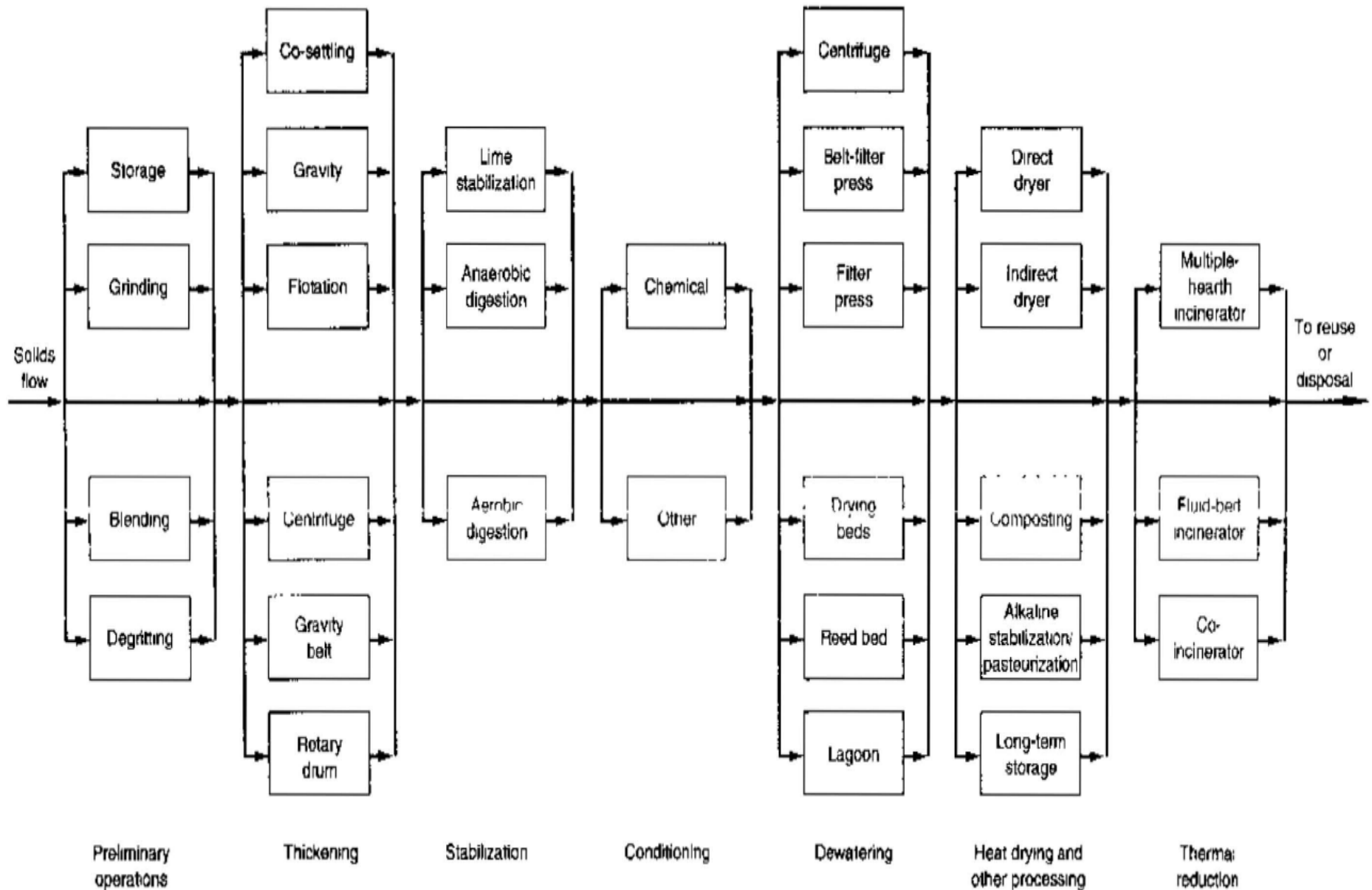
Sludge Treatment Processes

- Sludge thickening: The main goal is to reduce the sludge volume by removing as much as possible of the water content of sludge, leading to the increase of the solids content of sludge.
- Sludge conditioning: is a pretreatment step to increase the efficiency of sludge thickening and dewatering of sludge.
- Sludge stabilization: is the process in which the biodegradable organic matter is converted to non-biodegradable (or inert) matter. The main purpose of this process is to produce a sludge that will not undergo any further decomposition when disposed to the environment (i.e stable sludge). If sludge is disposed without stabilization it will create bad odor and health hazards. The most common method used for sludge stabilization is the anaerobic digestion.
- Sludge dewatering: Dewatering is a physical process used to reduce to water content of sludge after stabilization.

WWTP Sludge Treatment Processes



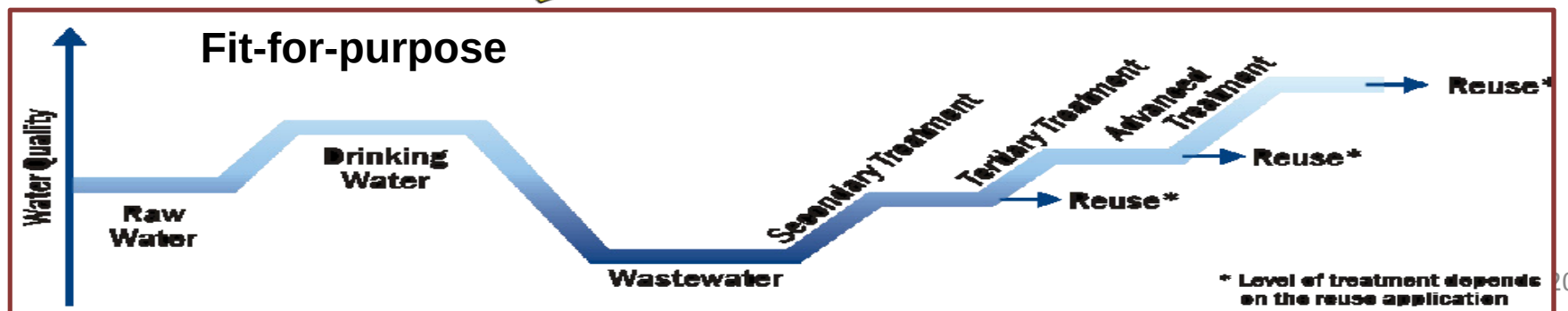
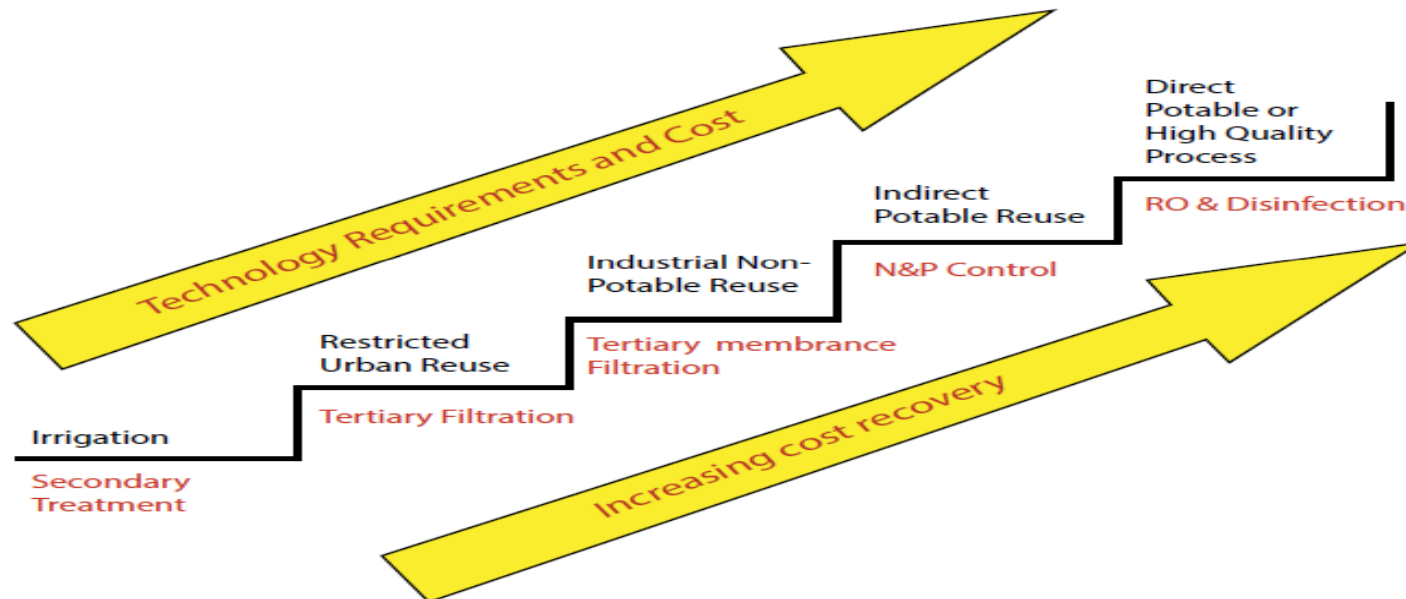
Generalized Sludge Treatment Flow Diagram



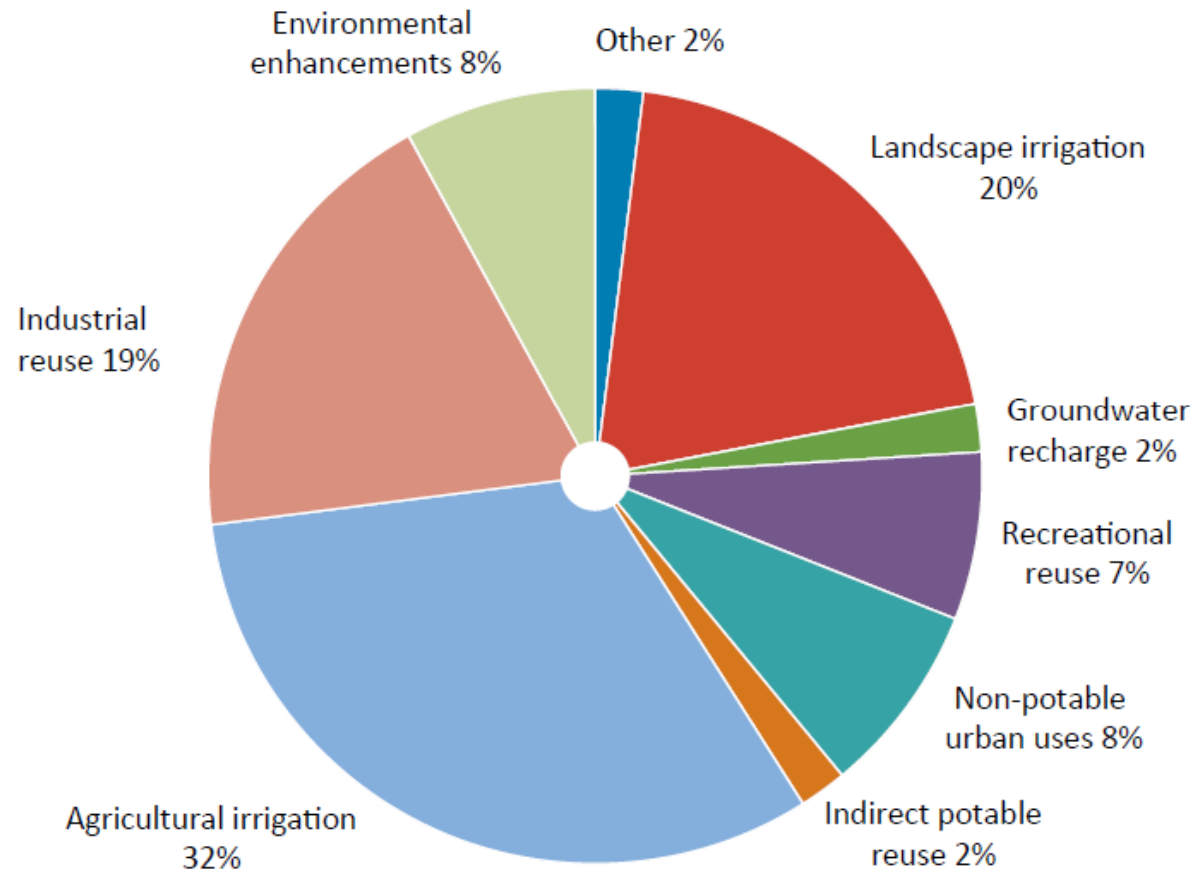
9. RESOURCE ORIENTED WASETWATER TREATMENT

Introduction

- Water reuse & resource recovery have become focus areas in wastewater management
- They have environmental and socio-economic benefits
- Need to match technology options with the treated wastewater use



Overview of Global Treated Wastewater Reuse



Assignment-9

- Review the UN World Water Development Report 2017 on Water Reuse and Resource Recovery (Chapters 16 & 17) and report the key lessons drawn

(Refer File: [UN_WWR_2017_Wastewater_Untapped Resource.pdf](#))