NASKAH TUTORIAL

TLB-305
DESAIN PENGOLAHAN FISIKA KIMIA 2

Semester Ganjil 2024/2025

Disusun Oleh:

Dr. Ir. Rachmawati Sugihhartati Djembarmanah, M.Env.Stud

PROGRAM STUDI TEKNIK LINGKUNGAN

FAKULTAS TEKNIK SIPIL DAN PERENCANAAN

INSTITUT TEKNOLOGI NASIONAL

2024

YAYASAN PENDIDIKAN DAYANG SUMBI
INSTITUT TEKNOLOGI NASIONAL
FAKULTAS TEKNIK SIPIL DAN PERENCANAAN
PROGRAM STUDI TEKNIK LINGKUNGAN
Jl. PHH. Mustafa No. 23 Bandung Indonesia. Phone **+62 22 7272215; Fax +62 22 7202892**
www.itenas.ac.id

S U R A T K E T E R A N G A N

Yang bertandatangan dibawah ini Ketua Program Studi Teknik Lingkungan Fakultas Teknik Sipil dan Perencanaan ITENAS, menerangkan bahwa :

Dr. Ir. Rachmawati Sugihhartati Djembarmanah, M.Env.Stud

Adalah **Pembuat Naskah Tutorial Desain Pengolahan Fisika Kimia 2** Prodi Teknik Lingkungan Periode Semester Ganjil Tahun Ajaran 2024/2025.

Demikian surat keterangan ini kami buat untuk digunakan sebagaimana mestinya.

Bandung, 30 September 2024
Ketua Program Studi Teknik Lingkungan

Prof. Dr. M. Rangga Sururi, S.T., M.T.

LEMBAR PENGESAHAN

NASKAH TUTORIAL TLB-305

DESAIN PENGOLAHAN

FISIKA KIMIA 2

Mengetahui
Ketua Jurusan
Teknik Lingkungan

Prof. Dr. M. Rangga Sururi, S.T., M.T.

TLB-305
DISAIN PENGOLAHAN FISIKA KIMIA 2

PERTEMUAN PERTAMA

PENDAHULUAN

1. PERKEMBANGAN TEKNOLOGI IPAL DAN APLIKASINYA DI LAPANGAN
2. PENGARUH IPAL TERHADAP KUALITAS AIR BAKU
3. PENGARUH KUALITAS AIR BAKU AIR MINUM TERHADAP PROSES PENGOLAHAN DI IPAM

RACHMAWATI S. DJ

SEMESTER GANJIL 2024/2025

ISI

1. Pendahuluan

2. Perkembangan teknologi IPAL dan aplikasinya di lapangan
3. Pengaruh IPAL terhadap kualitas air baku
4. Pengaruh kualitas air baku air minum terhadap proses pengolahan di IPAM
5. TUGAS 1 (kelompok)

1. Pendahuluan

KONTRAK PERKULIAHAN

- Kuliah – 2 x 50 menit
- Kehadiran minimum 80%
- Toleransi keterlambatan: 15 menit setelah kuliah berjalan
- Tidak ada toleransi untuk keterlambatan pengumpulan tugas

Sub CPMK 4.1 & MATERI

Sub CPMK 4.1

Mahasiswa mampu merumuskan masalah kualitas air baku dan air limbah, serta baku mutu air limbah dan air minum

MATERI

- Perkembangan teknologi IPAL dan aplikasinya di lapangan
- Pengaruh IPAL terhadap kualitas air baku
- Pengaruh kualitas air baku air minum terhadap proses pengolahan di IPAM

2. Perkembangan teknologi IPAL dan aplikasinya di lapangan
3. Pengaruh IPAL terhadap kualitas air baku

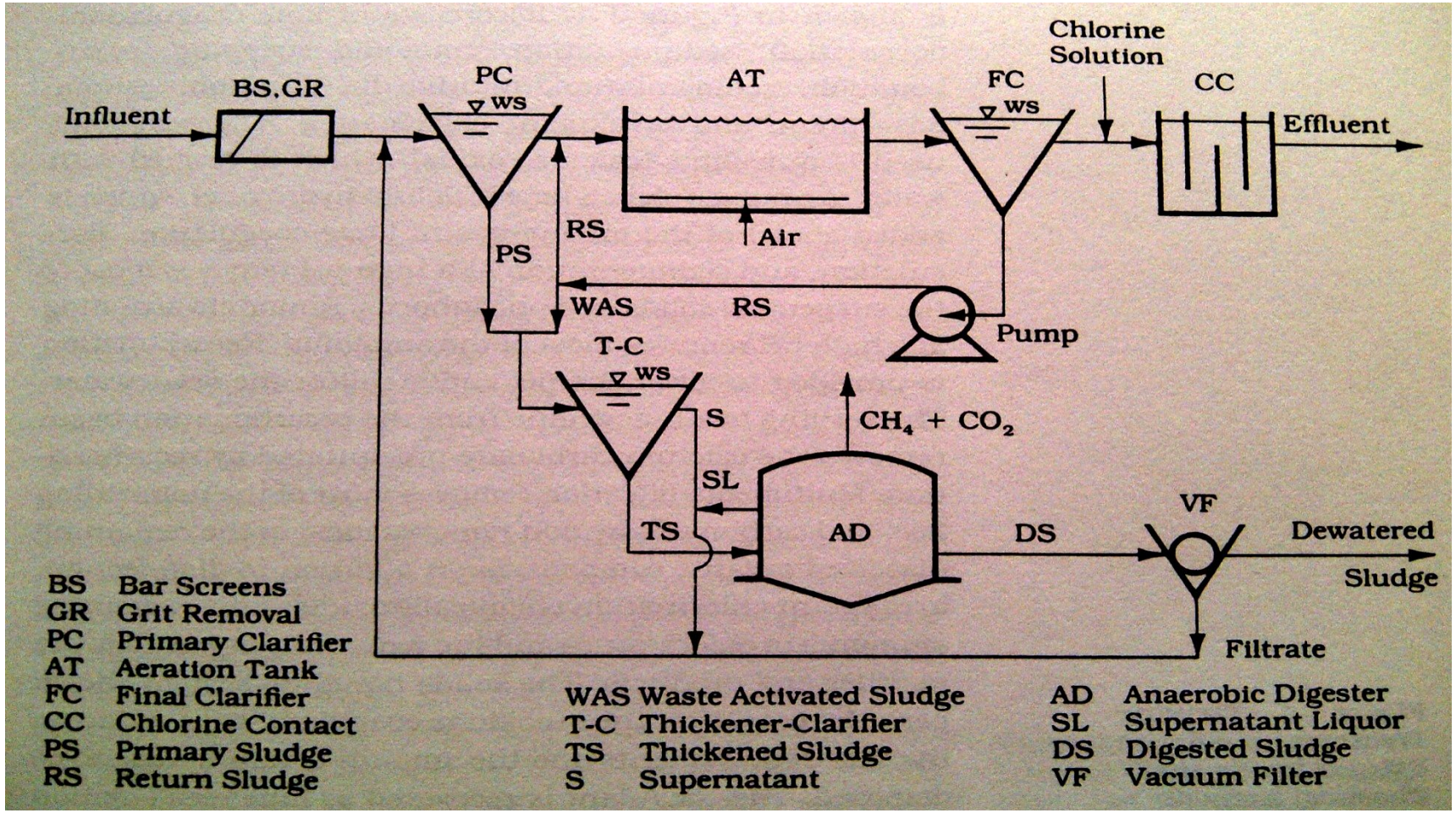
INTRODUCTION

- Unit operations and processes:
  - Physical
  - Chemical
  - Biological treatments
- Unit operation: a physical treatment
- Unit process: chemical & biological treatment

WWTP

- Most common municipal WWTP:
  - Primary & secondary treatment plants
  - Tertiary treatment plants
  - Physical chemical treatment plants

Activated sludge plant for a municipal waste water



Activated sludge plant for a municipal waste water

- Screening
- Grit removal
- Primary clarification:
- Activated sludge treatment
- chlorination

Activated sludge plant for a municipal waste water

- Screening: coarse solids
- Grit removal: sand and silt
- Primary clarification:
  - remove suspended solids
  - Primary effluent is mixed with the return activated sludge
- Mixed liquour flows to aeration tank
- Aeration tank: bio-oxidation of most of the remaining organic matter

Activated sludge plant for a municipal waste water

- Final clarifier:
  - Remove the biological solids
  - Which are returned to mix with the incoming primary effluent
- Chlorination:
  - Disinfected the effluent from the final clarifier
  - To kill pathogenic organisms
- Effluent discharged to the receiving body of water

Activated sludge plant for a municipal waste water

- The primary clarifier sludge and the waste activated sludge are mixed together
- Then thickened to increase the solids content
- The thickened sludge is sent to the anaerobic digester for bio-oxidation of the organic solids
- The digested sludge is digested by facum filtration
- The dewatered sludge is disposed of in a SL

Activated sludge plant for a municipal waste water

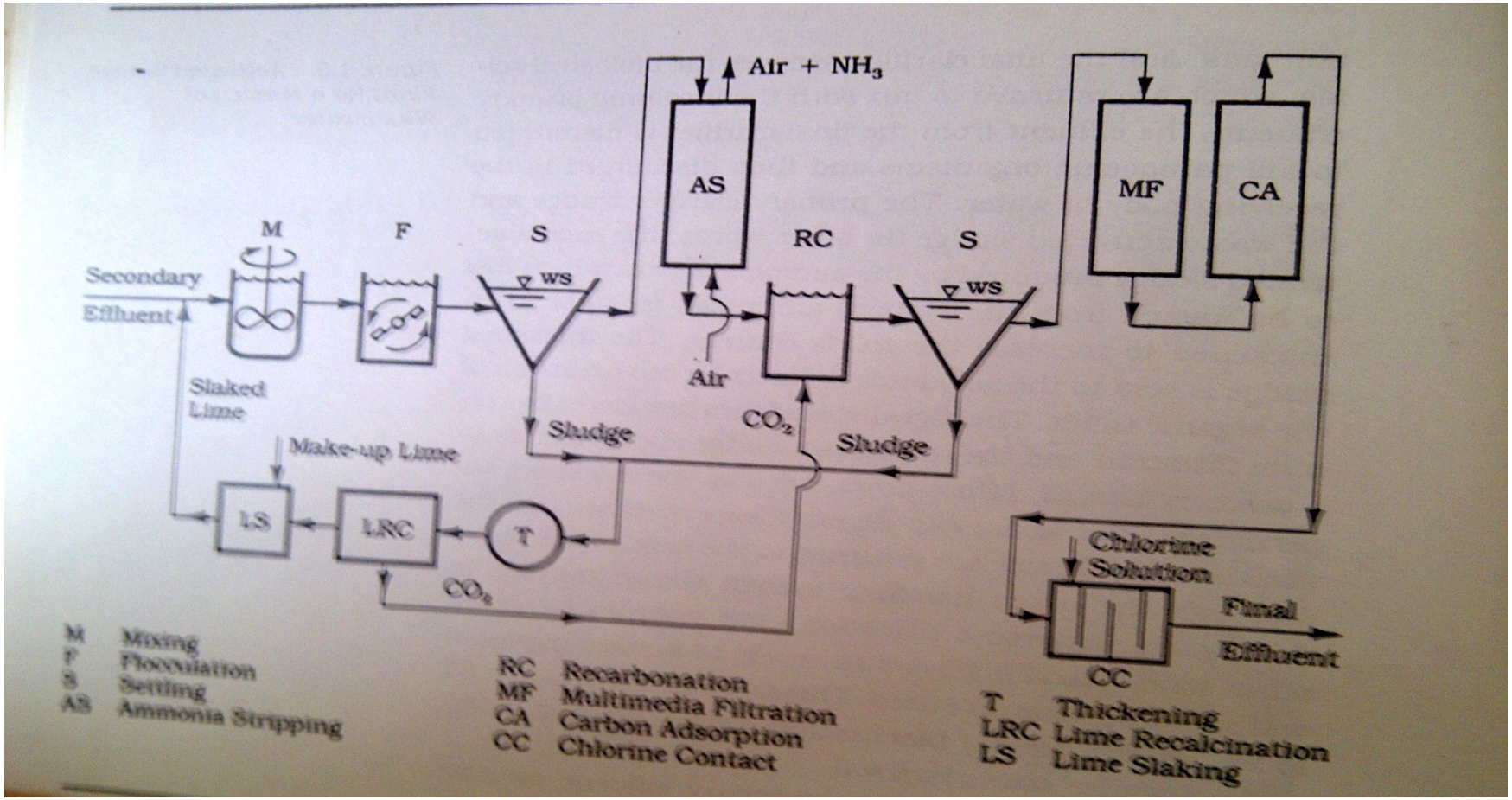
- Minor flows:
  - The thickener supernatant
  - The anaerobic digester supernatant
  - The vacuum filtrate supernatant
- Returned to the head of the plant
- Other solids handling system:
  - Aerobic digestion
  - Centrifugation

Activated sludge plant for a municipal waste water

Removal rate:

85-95% BOD5 and SS

Tertiary treatment of secondary effluent by physical and chemical methods



Tertiary treatment of secondary effluent by physical and chemical methods

- Providing further treatment to increase the quality of the effluent
- Consists of:
  - Lime coagulation
  - Flocculation
  - Sedimentation
  - Ammonia stripping
  - Recarbonation
  - Sedimentation

Tertiary treatment of secondary effluent by physical and chemical methods

- Consists of:
  - Multi media filtration
  - Carbon adsorption
  - Breakpoint chlorination
- Coagulant:
  - quick lime (calcium oxide) reacted with water to produce slaked lime (calcium hydroxide)
  - Added ahead of the mixing basin

Tertiary treatment of secondary effluent by physical and chemical methods

- Lime coagulation
- Flocculation
- Sedimentation
- At a high pH removes most of the SS and phosporus
- Ammonia stripping at a high pH removes most of the ammonia

Tertiary treatment of secondary effluent by physical and chemical methods

- Recarbonation:
  - to lower the pH
  - and stabilize the waste water
- Settling basin after RC: removes calcium carbonate precipitated by RC
- Multi media filtration: most of the nonsettling flocs
- Carbon adsorption: most of the remaining dissolved organic compounds

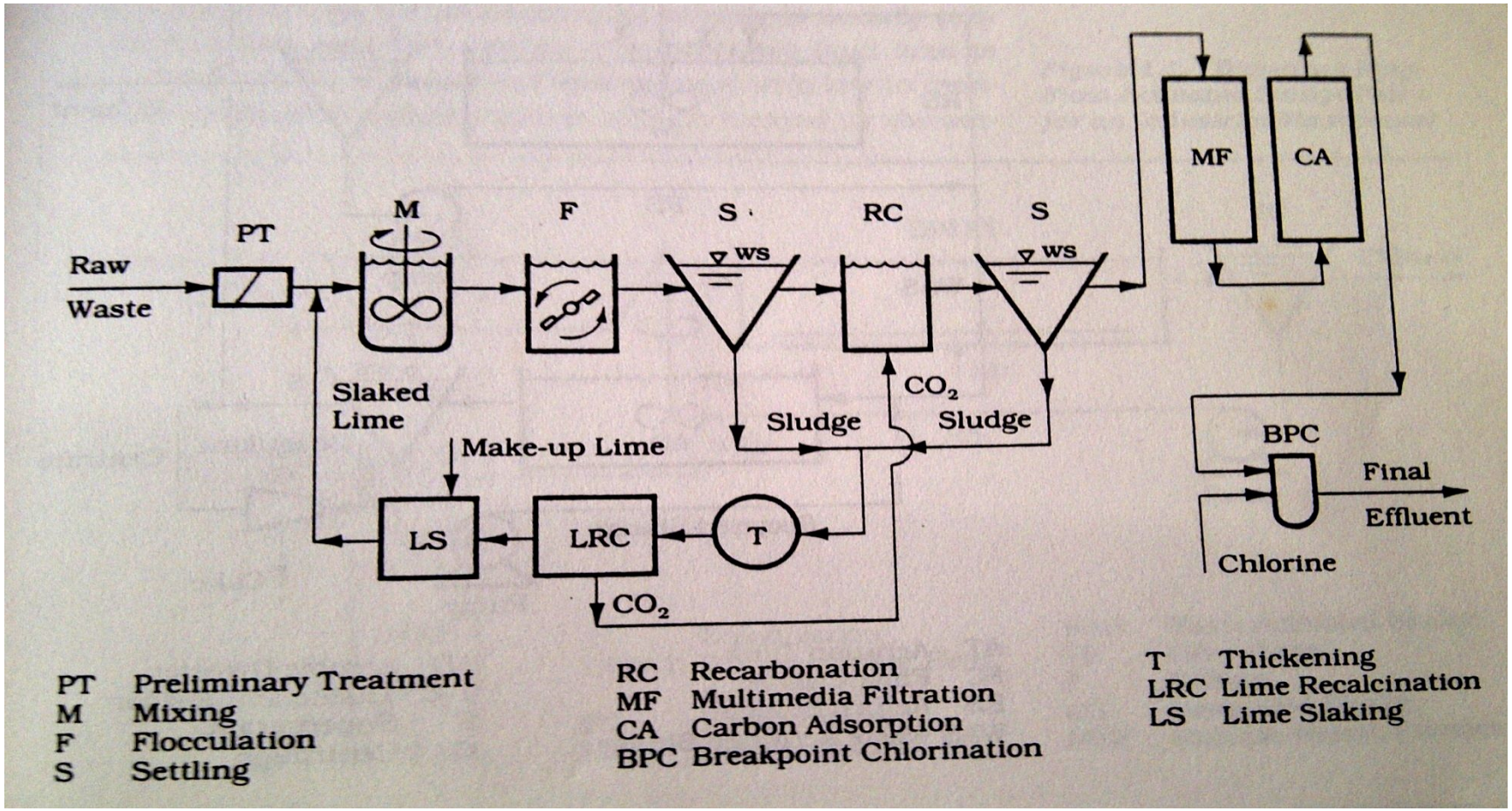
Tertiary treatment of secondary effluent by physical and chemical methods

- Break point chlorination:
  - Disifection
  - chemically oxidizes the remaining ammonia to chloramines
  - The remaining organic matter to other end products
- The solid handling system:
  - Permits recovery of the quick lime coagulant
  - Reducing the lime requirements
  - And the lime sludge to be disposed of

Tertiary treatment of secondary effluent by physical and chemical methods

- Lime recalcination:
  - Recovered coagulant
  - Calcium carbonate precipitate in the sludge is heated at a high temperature to produce calcium oxide (coagulant)
- The organic solids in the sludge are incinerated
- Removal rate:
  - Produce an effluent approaching drinking water quality (when treating municipal secondary effluents)

Physical-chemical treatment of raw municipal wastewater



Physical-chemical treatment of raw municipal wastewater

- Consists of:
  - Lime coagulation
  - Flocculation
  - Sedimentation
  - Recarbonation
  - Sedimentation
  - Multimedia filtration
  - Carbon adsorption
  - Breakpoint chlorination

Physical-chemical treatment of raw municipal wastewater

- Lime coagulation
- Flocculation
- Sedimentation
- Remove most of the SS, phosphorus & organic nitrogen
- RC:
  - Lower the pH
  - Stabilizes the wastewater

Physical-chemical treatment of raw municipal wastewater

- Sedimentation basin after RC: removes most of the calcium carbonate precipitated by RC
- Multimedia filtration: Most of the fine, nonsettling floc
- Carbon adsorption: most of the remaining organic compounds

Physical-chemical treatment of raw municipal wastewater

- Break point chlorination:
  - Disifection
  - chemically oxidizes the remaining ammonia to chloramines
  - The remaining organic matter to other end products
- The solid handling system:
  - Permits recovery of the quick lime coagulant
  - Reducing the lime requirements
  - And the lime sludge to be disposed of
  - Incinerating the organic solids in the lime sludge

- Removal efficiency:

96-99% of the BOD_5 and SS

4. Pengaruh kualitas air baku air minum terhadap proses pengolahan di IPAM

WTP

- For surface waters:
  - Rapid sand filtration (RSF) plants
  - Lime-soda softening plants
- For ground waters:
  - Gas stripping
  - Chlorination plants
  - Softening plants (lime-soda or IE plants)

RSF Plant

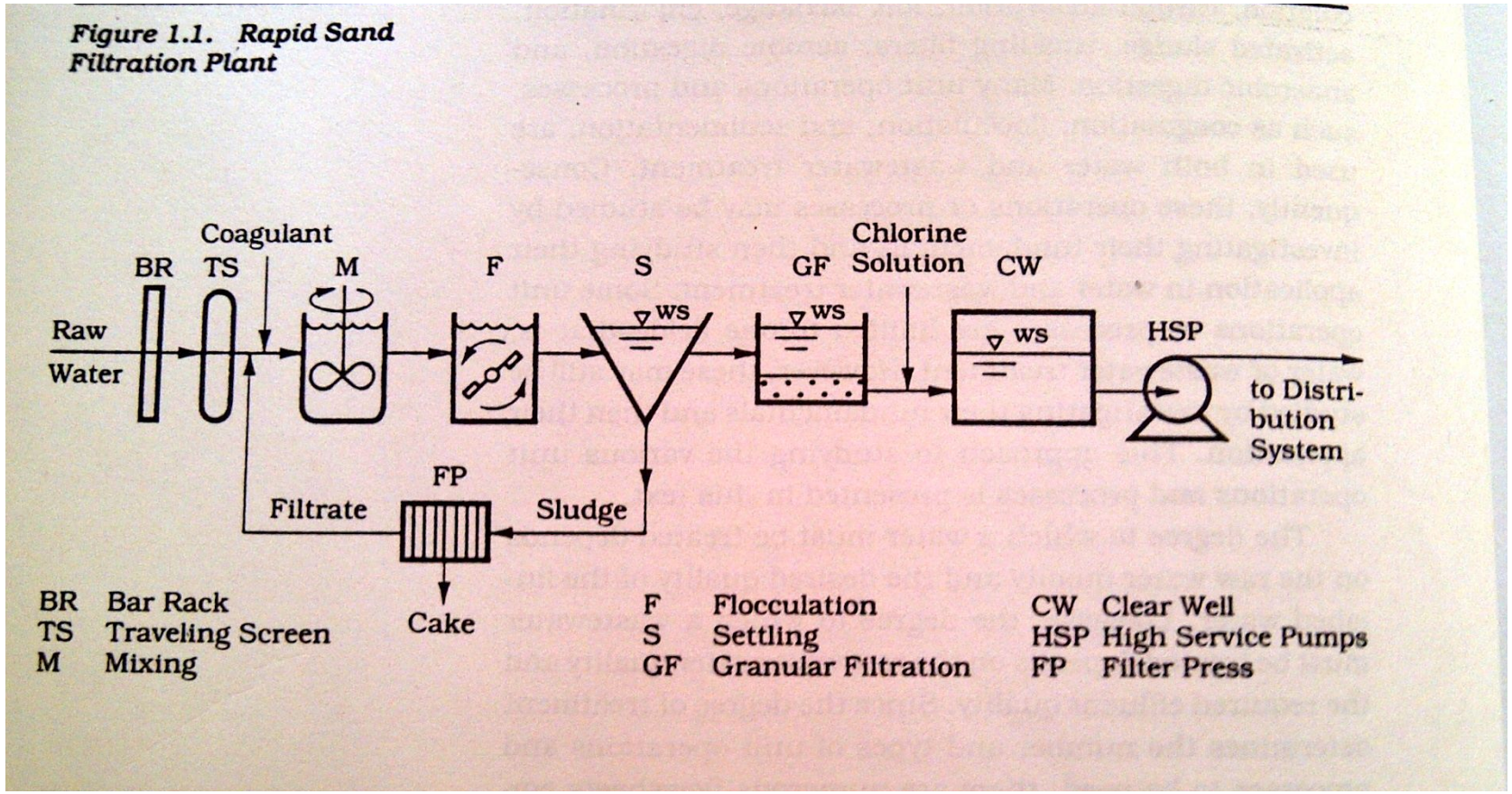
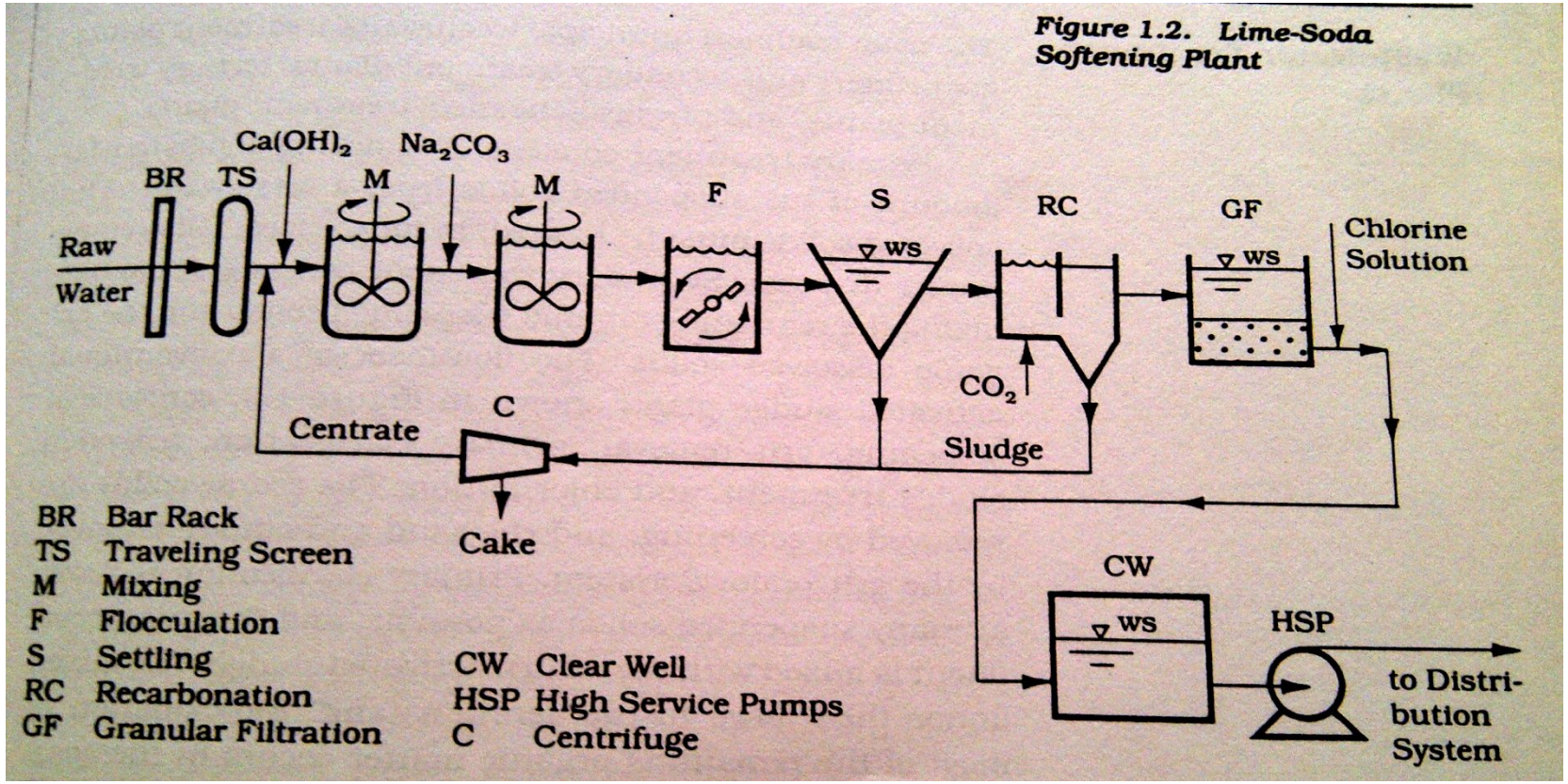


Figure 1.1. Rapid Sand Filtration Plant
Coagulant
BR
TS
M
F
S
GF
Chlorine Solution
CW
Raw Water
ws
ws
ws
HSP
to Distri-bution System
FP
Filtrate
Sludge
Cake
BR Bar Rack
TS Traveling Screen
M Mixing
F Flocculation
S Settling
GF Granular Filtration
CW Clear Well
HSP High Service Pumps
FP Filter Press

RSF Plant

- Coarse screen: large debris
- Finer travelling screen: smaller debris
- Chemical coagulation & flocculation: colloidal solids (flocs)
- Settling basin: flocs
- Granular media filters: fine non settling flocs
- Desinfection: pathogenic organisms
- Clear well: store water
- Solids handling system: Filter press dewater the sludge

Lime-soda softening plant



Lime-soda softening plant

- Lime (calcium hydroxide): precipitates the calcium ions as calcium carbonate
- Soda ash (sodium carbonate): precipitates the magnesium ions as magnesiumhydroxide
- Flocculation: flocs from colloidal solids
- Settling basin: flocs
- Recarbonation (by CO2): lower the pH, stabilizes the water further precipitation does not occur.
- Filters: fine non settling flocs
- Desinfection: pathogenic organisms

Lime-soda softening plant

- Removing hardness
- Produces coagulation and settling of colloidal solids
- To improve performance: a coagulant is added
- Solids handling system: centrifugation & disposal of dewatered cake by SL
- modification:
  - 2 flocculators, 2 settling basins (1^{st} flocculator, 1^{st} settling basin, 2^{nd} flocculator, 2^{nd} settling basin)
  - 2 stages stabilization (dual units)

Ground waters

- High quality waters
- Gas stripping: remove gasses (CO_2)
- Chlorination: provide residual in distribution system
- Hardness: softening by lime-soda or IE
- Iron or manganese: special treatment

5. TUGAS 1 (Kelompok)

TUGAS 1 (Kelompok)

- Buatlah ringkasan (tulisan) terkait studi kasus *performance* IPAL dan pengaruhnya terhadap kesehatan dan lingkungan

TERIMA KASIH

TLB-305
DISAIN PENGOLAHAN FISIKA KIMIA 2

PERTEMUAN KEDUA:

- STANDAR BAKU MUTU AIR LIMBAH DAN KUALITAS AIR BAKU
- STANDAR KUALITAS AIR MINUM
- PENENTUAN DERAJAT PENGOLAHAN
- FUNGSI PENGOLAHAN TK-2 DAN TK-3

RACHMAWATI S. DJ

SEMESTER GANJIL 2024/2025

ISI

1. Pendahuluan
2. Standar baku mutu air limbah dan kualitas air baku
3. Standar kualitas air minum
4. Penentuan derajat pengolahan
5. Fungsi pengolahan tk-2 dan tk-3
6. TUGAS 2 (kelompok)

1. Pendahuluan

Sub CPMK 4.2 & MATERI

Sub CPMK 4.2

Mahasiswa mampu menganalisis kualitas dan kuantitas air yang akan diolah dan menelaah jenis pengolahan yang diperlukan

MATERI

- Standar baku mutu air limbah dan kualitas air baku
- Standar kualitas air minum
- Penentuan derajat pengolahan
- Fungsi pengolahan tk-2 dan tk-3

2. Standar baku mutu air limbah dan kualitas air baku

Baku Mutu Air Limbah

LAMPIRAN I
PERATURAN MENTERI LINGKUNGAN HIDUP
REPUBLIK INDONESIA
NOMOR 5 TAHUN 2014
TENTANG
BAKU MUTU AIR LIMBAH

BAKU MUTU AIR LIMBAH BAGI USAHA DAN/ATAU KEGIATAN
INDUSTRI PELAPISAN LOGAM DAN GALVANIS

Parameter	Kadar Paling Tinggi Pelapisan Logam (mg/L)	Beban Paling Tinggi Pelapisan Logam (gr/m^2)	Kadar Paling Tinggi Galvanisasi (mg/L)	Beban Paling Tinggi Galvanisasi (gr/m^2)
TSS	20	0,4	20	0,04
Cu	0,5	0,01	0,5	0,001
Zn	1,0	0,02	1,0	0,0005
Cr^{6+}	0,1	0,002	-	-
Cr	0,5	0,01	-	-
Cd	0,05	0,001	0,05	0,0001
Pb	0,1	0,002	0,1	0,0002
Ni	1,0	0,02	1,0	0,002
CN	0,2	0,004	0,2	0,0004
Ag	0,5	0,01	0,5	0,001
pH	6 – 9		6 – 9	
Kuantitas air limbah paling tinggi	20 L per m^2 produk pelapisan logam		2 L per m^2 produk pelapisan logam	

Catatan:
Penghitungan Beban Paling Tinggi atau Beban Maksimum (BM):

$$BM = \text{Kadar Paling Tinggi (mg/L)} \times \text{Kuantitas Air Limbah Paling Tinggi } (L/m^2)/1000$$

MENTERI LINGKUNGAN HIDUP
REPUBLIK INDONESIA,

ttd

BALTHASAR KAMBUAYA

LAMPIRAN II
PERATURAN MENTERI LINGKUNGAN HIDUP
REPUBLIK INDONESIA
NOMOR 5 TAHUN 2014
TENTANG
BAKU MUTU AIR LIMBAH

BAKU MUTU AIR LIMBAH BAGI USAHA DAN/ATAU KEGIATAN
INDUSTRI PENYAMAKAN KULIT

Parameter	Proses Penyamakan Menggunakan Krom		Proses Penyamakan Menggunakan Daun-daunan	
	Kadar Paling Tinggi (mg/L)	Beban Pencemaran Paling Tinggi (kg/ton)	Kadar Paling Tinggi (mg/L)	Beban Pencemaran Paling Tinggi (kg/ton)
BOD_5	50	2,0	70	2,8
COD	110	4,4	180	7,2
TSS	60	2,4	50	2,0
Krom Total (Cr)	0,60	0,024	0,10	0,004
Minyak dan Lemak	5,0	0,20	5,0	0.20
Nitrogen Total (sebagai N)	10	0,40	15	0,60
Amonia Total	0,5	0,02	0,50	0,02
Sulfida (sebagai S)	0,8	0,032	0,50	0,02
pH	6,0 - 9,0		6,0 - 9,0	
Debit limbah paling tinggi	40 m^3 per ton bahan baku		40 m^3 per ton bahan baku	

Catatan:
1. Kadar paling tinggi untuk setiap parameter pada tabel di atas dinyatakan dalam miligram parameter per liter air limbah.
2. Beban pencemaran paling tinggi pada tabel di atas dinyatakan dalam kg per ton bahan baku hasil dari kegiatan penggaraman kulit mentah.
3. Nitrogen Total = Nitrogen Organik + Amonia Total + NO_3 + NO_2.

MENTERI LINGKUNGAN HIDUP
REPUBLIK INDONESIA,

ttd

BALTHASAR KAMBUAYA

Baku Mutu Air Limbah Industri Tekstil

LAMPIRAN I
PERATURAN MENTERI LINGKUNGAN HIDUP DAN KEHUTANAN REPUBLIK INDONESIA
NOMOR P.16/MENLHK/SETJEN/KUM.1/4/2019
TENTANG
PERUBAHAN KEDUA ATAS PERATURAN MENTERI LINGKUNGAN HIDUP NOMOR 5 TAHUN 2014 TENTANG BAKU MUTU AIR LIMBAH

BAKU MUTU AIR LIMBAH BAGI USAHA DAN/ATAU KEGIATAN INDUSTRI TEKSTIL PERIODE PERALIHAN

Parameter	Kadar Paling Tinggi (mg/L)	Beban Pencemaran Paling Tinggi (kg/ton)
BOD_5	60	6
COD	150	15
TTS	50	5
Fenol Total	0,5	0,05
Krom Total (Cr)	1,0	0,1
Amonia Total (NH_3-N)	8,0	0,8
Sulfida (sebagai S)	0,3	0.03
Minyak dan Lemak	3,0	0,3
pH	6,0 – 9,0	
Debit Limbah Paling Tinggi	100 m3/ton produk tekstil	

Salinan sesuai dengan aslinya
KEPALA BIRO HUKUM,

ttd.

KRISNA RYA

MENTERI LINGKUNGAN HIDUP DAN KEHUTANAN REPUBLIK INDONESIA,

ttd.

SITI NURBAYA

Baku Mutu Air Limbah Industri Tekstil

LAMPIRAN II

PERATURAN MENTERI LINGKUNGAN HIDUP DAN KEHUTANAN REPUBLIK INDONESIA

NOMOR P.16/MENLHK/SETJEN/KUM.1/4/2019

TENTANG

PERUBAHAN KEDUA ATAS PERATURAN MENTERI LINGKUNGAN HIDUP NOMOR 5 TAHUN 2014

TENTANG BAKU MUTU AIR LIMBAH

BAKU MUTU AIR LIMBAH BAGI USAHA DAN/ATAU KEGIATAN INDUSTRI TEKSTIL

Debit	BOD	COD	TSS	Fenol Total	Krom Total	Amonia Total	Sulfida	Minyak Lemak	pH	Warna	Suhu	Debit Maksimum
≤100	60	150	50	0,5	1	8	0,3	3	6 - 9	200	Deviasi 2*	100
100 < x < 1000	45	125	40	0,5	1	8	0,3	3	6 - 9	200	Deviasi 2*	100
≥1.000	35	115	30	0,5	1	8	0,3	3	6 - 9	200	Deviasi 2*	100
m^3/hari	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L		Pt-Co	°C	m^3/ton produk

Keterangan:

Pt-Co: *true colour*

*: temperatur udara sekitar

Salinan sesuai dengan aslinya
KEPALA BIRO HUKUM,

ttd.

KRISNA RYA

MENTERI LINGKUNGAN HIDUP DAN
KEHUTANAN REPUBLIK INDONESIA,

ttd.

SITI NURBAYA

Standar kualitas air baku

- PP No. 22 Tahun 2021

3. Standar kualitas air minum

PERATURAN MENTERI KESEHATAN REPUBLIK INDONESIA

NOMOR 2 TAHUN 2023

TENTANG

PERATURAN PELAKSANAAN PERATURAN PEMERINTAH NOMOR 66 TAHUN 2014 TENTANG KESEHATAN LINGKUNGAN

4. Penentuan derajat pengolahan

Tahapan Pengolahan

Treatment level	Description
Preliminary	Removal of wastewater constituents such as rags, sticks, floatables, grit, and grease that may cause maintenance or operational problems with the treatment operations, processes, and ancillary systems
Primary	Removal of a portion of the suspended solids and organic matter from the wastewater
Advanced primary	Enhanced removal of suspended solids and organic matter from the wastewater. Typically accomplished by chemical addition or filtration
Secondary	Removal of biodegradable organic matter (in solution or suspension) and suspended solids. Disinfection is also typically included in the definition of conventional secondary treatment
Secondary with nutrient removal	Removal of biodegradable organics, suspended solids, and nutrients (nitrogen, phosphorus, or both nitrogen and phosphorus)
Tertiary	Removal of residual suspended solids (after secondary treatment), usually by granular medium filtration or microscreens. Disinfection is also typically a part of tertiary treatment. Nutrient removal is often included in this definition
Advanced	Removal of dissolved and suspended materials remaining after normal biological treatment when required for various water reuse applications

The degree of WT & WWT

- The degree to which a water must be treated depends on:
  - The raw water quality
  - The desired quality of the finished water
- The degree to which a waste water must be treated depends on:
  - The raw waste water quality
  - The required effluent quality

The degree of treatment

- The degree of treatment determines:
  - the number and types of unit operations
  - and processes to be used
- Numerous flowsheets employed in WT and WWT

WWTP

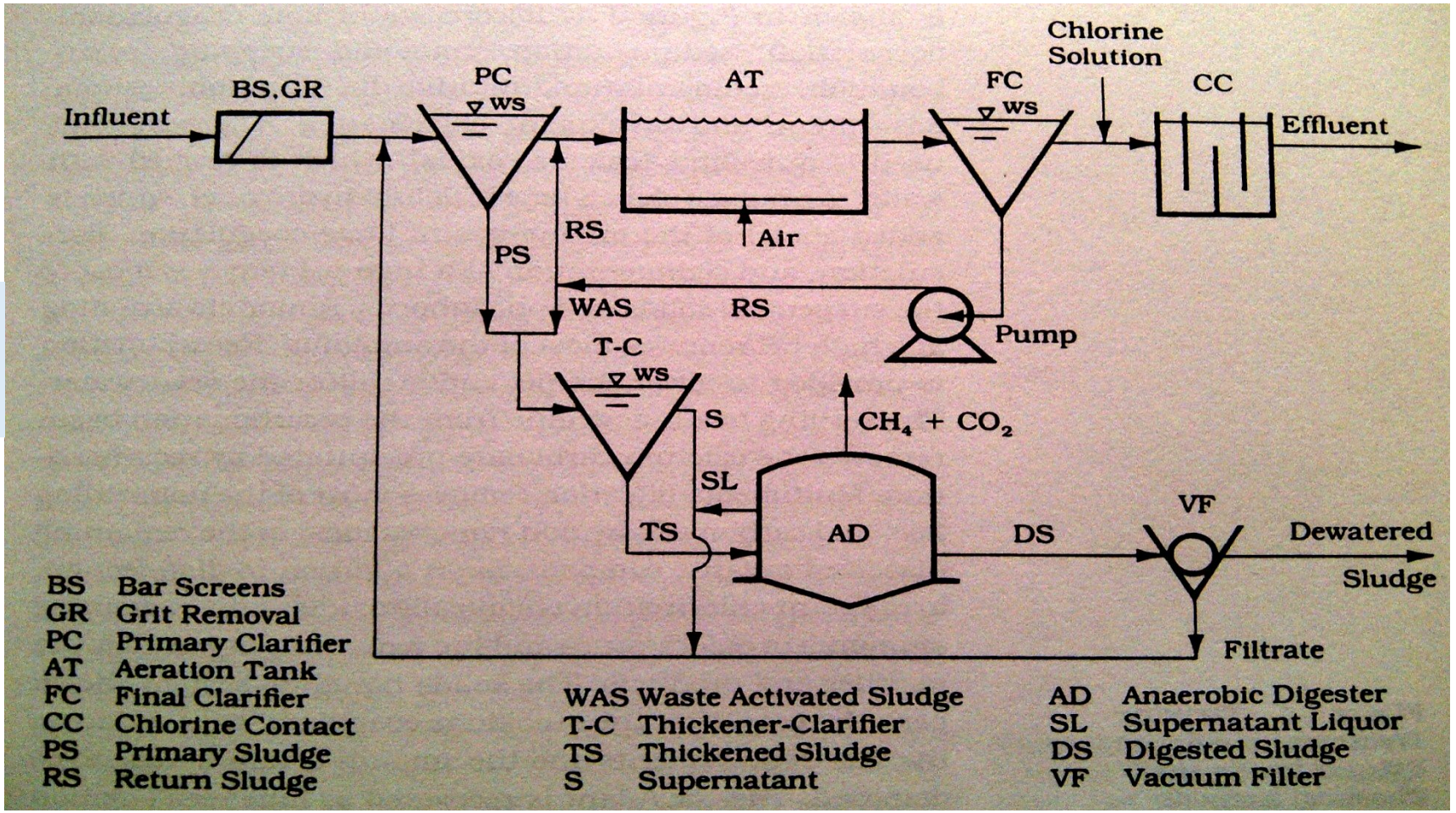
- Most common municipal WWTP:
  - Primary & secondary treatment plants
  - Tertiary treatment plants
  - Physical chemical treatment plants

- Primary treatment:
  - remove a substantial amount of the suspended solids from waste water
  - The collected solids must be treated, followed by proper disposal
- Secondary treatment:
  - Bio-oxidizing the remaining oganic suspended solids and the organic dissolved solids

Activated sludge plant for a municipal waste water

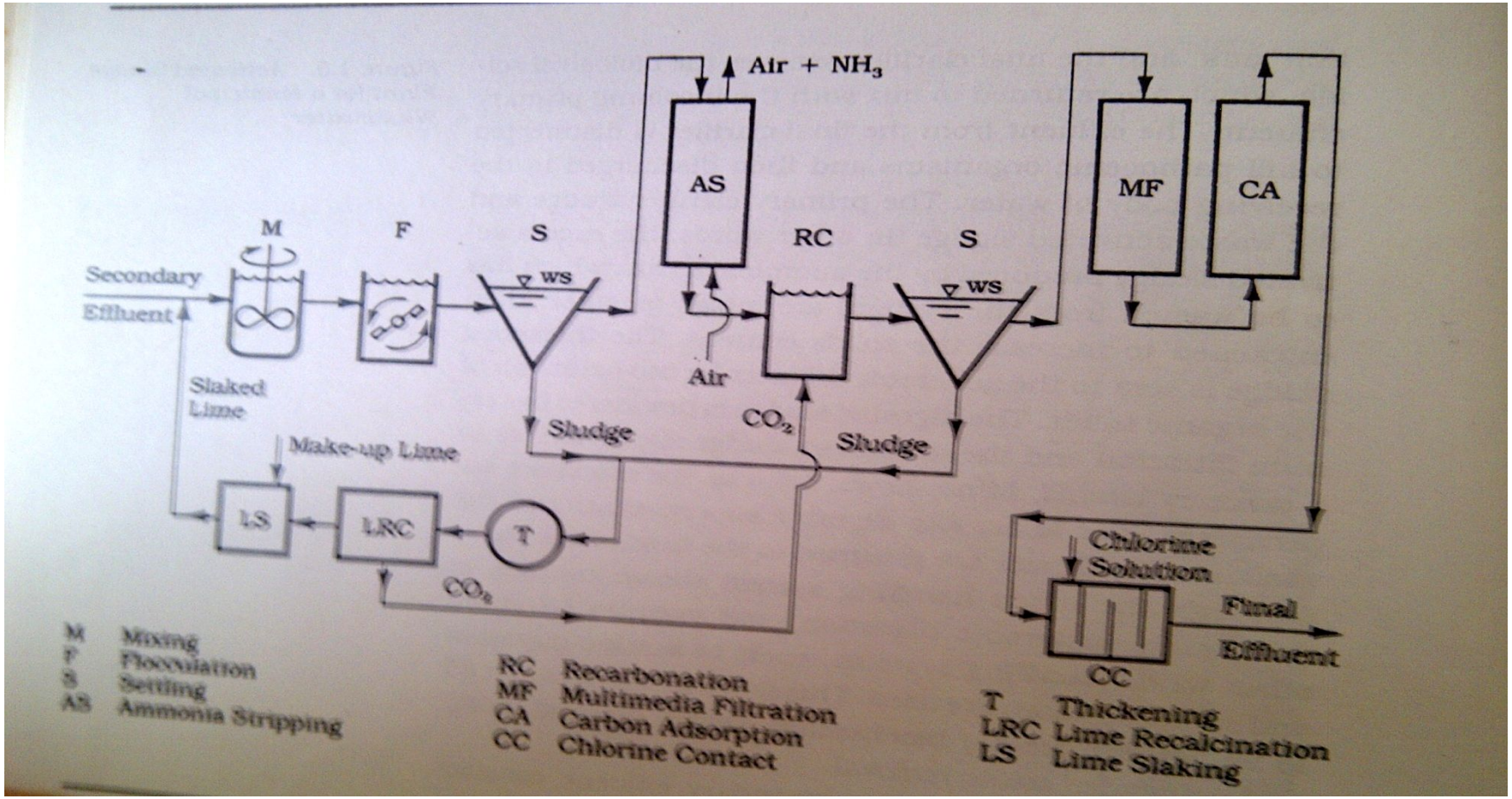
Removal rate:

85-95% BOD5 and SS



Tertiary treatment of secondary effluent by physical and chemical methods

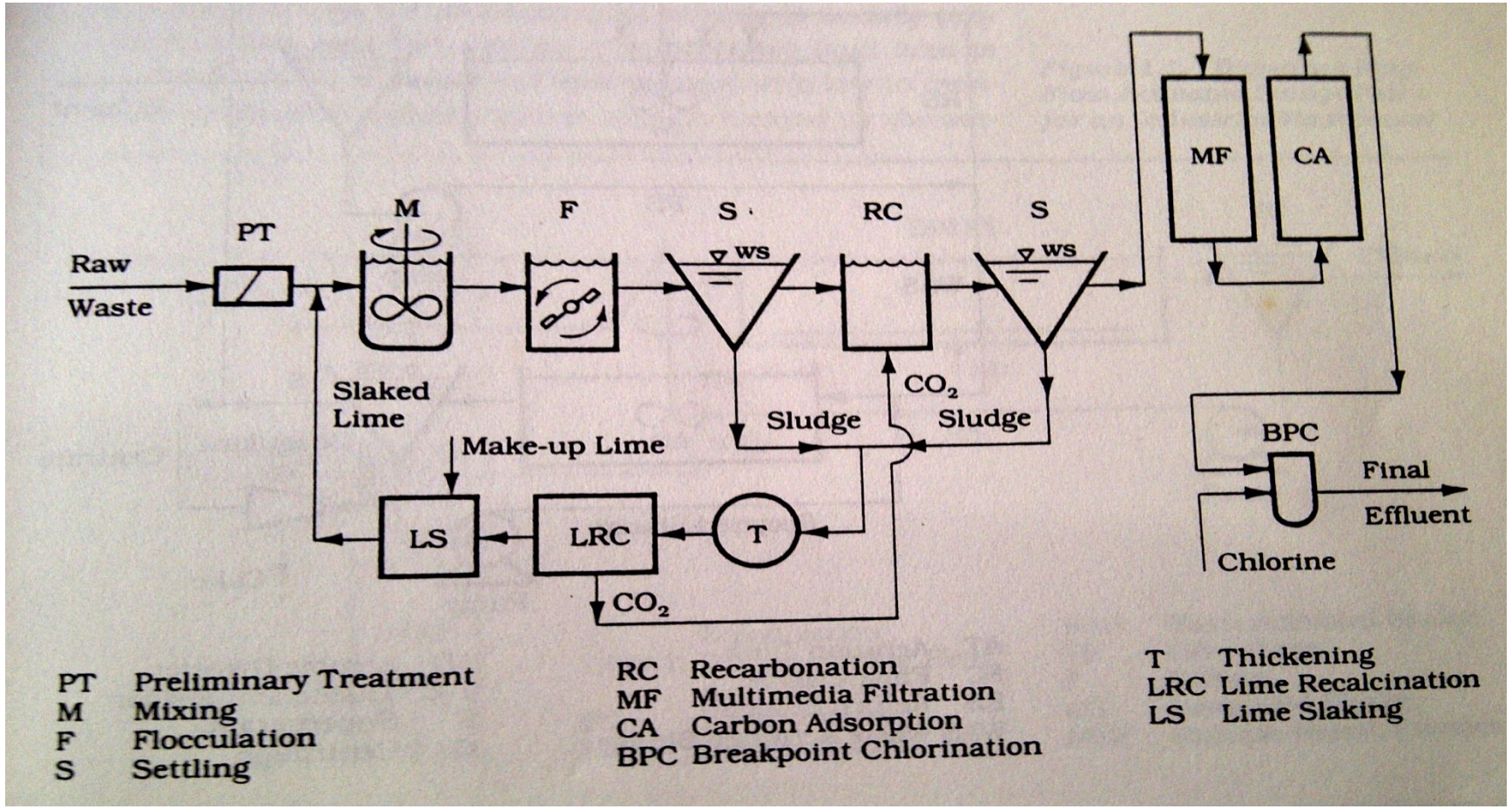
Providing further treatment to increase the quality of the effluent



Physical-chemical treatment of raw municipal wastewater

Removal efficiency:

96-99% of the BOD_5 and SS



5. TUGAS 2 (Kelompok)

TUGAS 2 (Kelompok)

- Buatlah ringkasan (tulisan) terkait permasalahan kualitas air baku di Indonesia

TERIMA KASIH

WEEK 3

COAGULATION

SEMESTER GANJIL 2024/2025

01.
Introduction

02.
Sub Topics

03.
About the Sub Topics

01.

Introduction

Sub CP MK 5.1 & Materi

Sub CPMK 5.1

Mahasiswa mampu menerapkan prinsip kerekayasaan, merinci formulasi, dan merancang unit *mixing (fast mixing*), termasuk peluang untuk *resource recovery*, sehingga melalui pengolahan air dapat melindungi lingkungan dan kesehatan masyarakat

Materi

- Introduction
- Colloidal Behavior
- Zeta Potential
- Colloid Stability
- Coagulation in Water Treatment
- Coagulation in Waste Treatment

COAGULATION

Extracted from:

Rich, L.G.,

Unit Processes of Sanitary Engineering,
John Wiley & Sons, Inc., New York, London, 1963

What Is This Topic About?

It’s about the …..

Definition of Concepts

Coagulation

Process in which chemicals are added to an aqueous system for the purpose of creating rapid-settling aggregates out of finely divided, dispersed matter with slow or negligible settling velocities.

Colloid Particles

- > atoms & small molecules
- Small enough to pass through the pores of ordinary filters
- 10^{-6} mm (1 mμ)- 10^{-3} mm (1 μ)

Zeta Potensial

Responsible for the electrokinetic behavior exhibited by colloid particles

Water / Waste Treatment

Pengolahan air maupun limbah untuk memenuhi baku mutu. Salah satunya dengan cara melakukan pengendapan partikel dengan cara koagulasi.

02.

SUB TOPICS

02 SUB TOPICS

Introduction

Colloidal Behavior

Zeta Potensial

Colloid Stability

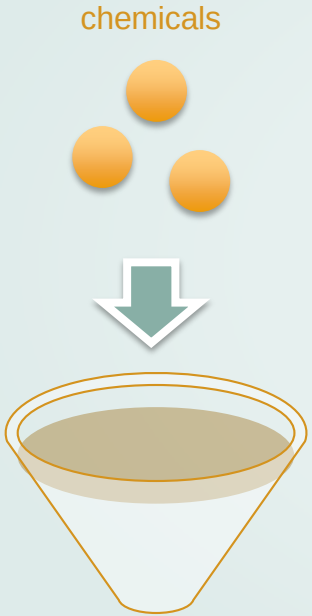
Coagulation in Water Treatment

Coagulation in Waste Treatment

03.

Introduction

03 Introduction



creating rapid-settling aggregates out of finely divided

dispersed matter with slow or negligible settling velocities.

COAGULATION

Widely used in the treatment of domestic & industrial waters.

Limited used in the treatment of sewage & industrial wastes.

Size of particles & its separation

Dispersed Matter

Surface waters may contain dispersed matter, varying in size: 10^{-7} to 10^{-1} mm

Particles

- ≥ 10^{-4} mm → turbidity
- < 10^{-4} mm → color, taste & odor

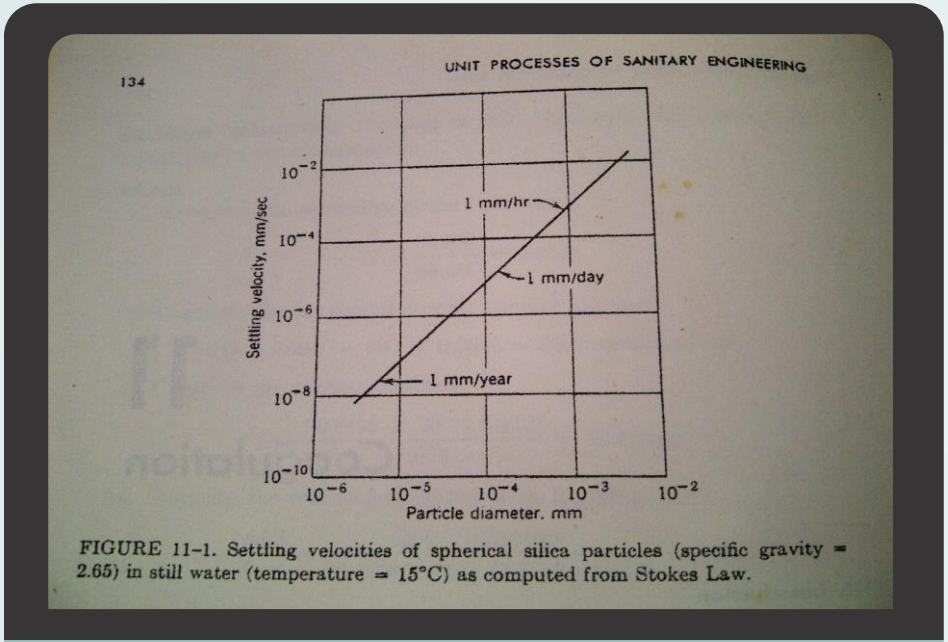
Particulate Matter

In natural waters, much of the particulate matter is composed of silica (ρ = 2.65).

Silica

- sand grains: 0.1 – 2 mm → settles rapidly
- In a finely divided state → gravity separation is relatively slow → Fig. 11-1

Settling velocities of spherical silica particles



Rich, 1963

Fig. 11-1:

d = 10^{-3} mm □ V_s = 1 mm/hr

d = 10^{-5} mm □ V_s = 1 mm/year

COAGULATION

The removal of these particles from water by sedimentation is practical only if:

They are incorporated into particle structures of a size large enough to result in V_s = several hundred mms/hr

03.

Colloidal behaviour

COAGULATION

The effective application of coagulation requires an understanding of the:

- Properties
- Behaviour of colloids

COLLOID

A system in which particles of relatively small size (the disperse phase) are dispersed in a homogenous medium (the dispersed medium).

Colloidal particles

- > atoms & small molecules
- Small enough to pass through the pores of ordinary filters Fig. 11-2
- 10^{-6} mm (1 mμ)- 10^{-3} mm (1 μ)
- Larger particles exhibit certain colloidal properties

Classification of dispersed systems

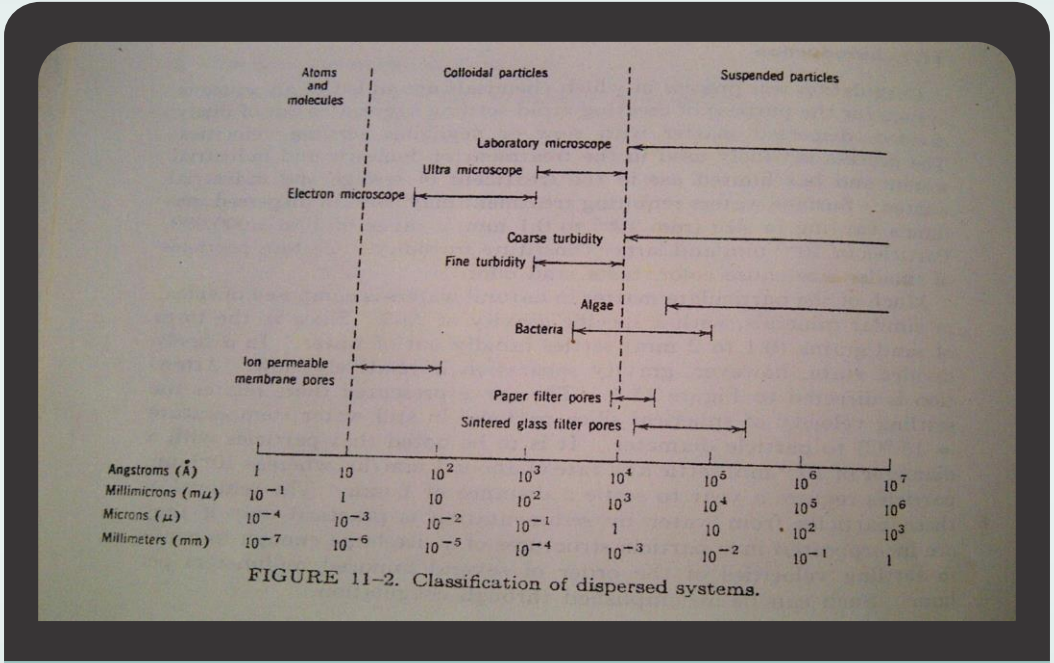


FIGURE 11–2. Classification of dispersed systems.

Rich, 1963

Types of Colloid Systems

- Several different types of colloid systems are possible
- **Table 11-1** : 8 known types of colloid systems
- Consists of two media that do not mix:
  1. Disperse medium
  2. Disperse phase

8 types of colloid systems

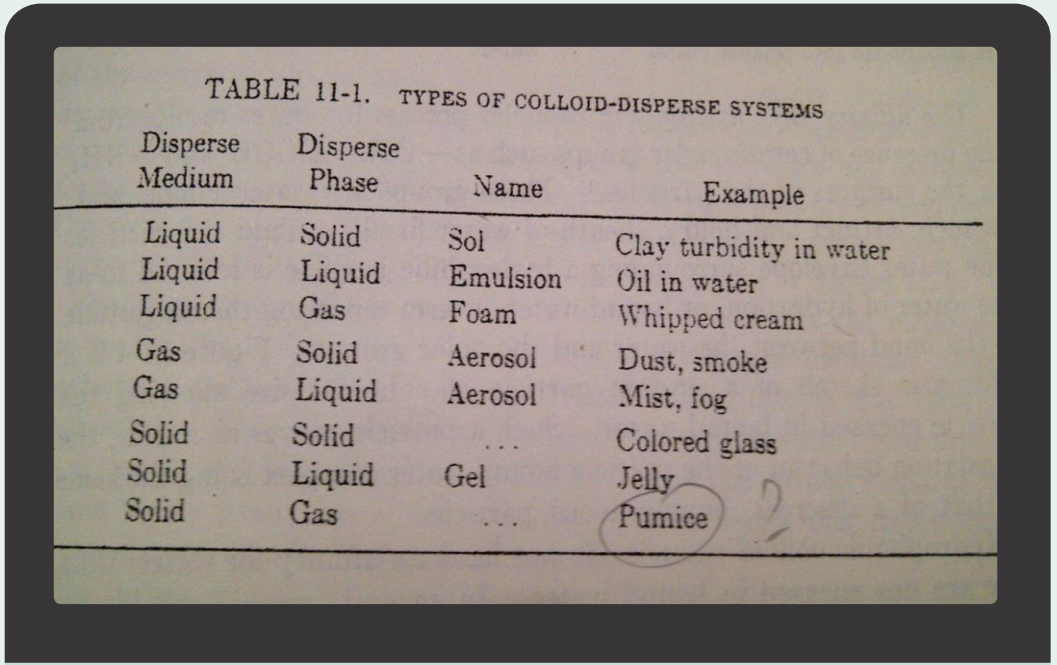


TABLE 11-1. TYPES OF COLLOID-DISPERSE SYSTEMS

Disperse Medium	Disperse Phase	Name	Example
Liquid	Solid	Sol	Clay turbidity in water
Liquid	Liquid	Emulsion	Oil in water
Liquid	Gas	Foam	Whipped cream
Gas	Solid	Aerosol	Dust, smoke
Gas	Liquid	Aerosol	Mist, fog
Solid	Solid	...	Colored glass
Solid	Liquid	Gel	Jelly
Solid	Gas	...	Pumice

Rich, 1963

8 types of colloid systems

Environmental Engineers

- Are required to have an understanding of:
  - Sols
  - Emulsions
  - Aerosols
- The following deals particularly with the properties of solids dispersed in liquids (sols), but some of the properties apply to emulsions & aerosols as well

TABLE 11-1. TYPES OF COLLOID-DISPERSE SYSTEMS

Disperse Medium	Disperse Phase	Name	Example
Liquid	Solid	Sol	Clay turbidity in water
Liquid	Liquid	Emulsion	Oil in water
Liquid	Gas	Foam	Whipped cream
Gas	Solid	Aerosol	Dust, smoke
Gas	Liquid	Aerosol	Mist, fog
Solid	Solid	...	Colored glass
Solid	Liquid	Gel	Jelly
Solid	Gas	...	Pumice

Rich, 1963

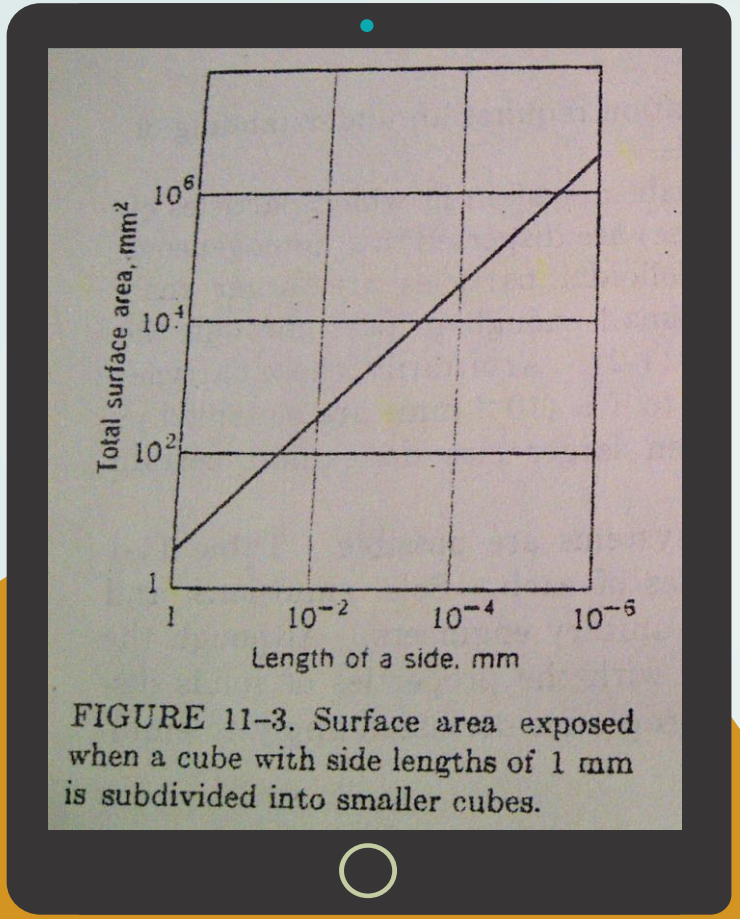


FIGURE 11–3. Surface area exposed when a cube with side lengths of 1 mm is subdivided into smaller cubes.

Surface Phenomena

- The unique behaviour of colloid particles is the result of surface phenomena.
- The large area exposed by a given mass of particles Fig. 11-3
- If a cube with l = 1 mm, A_s = 6 mm^2 is divided into smaller cubes:
  - l = 10^{-6} mm A_s = 6 m^2

Affinity for the Dispersed Medium

Sols

- Lyophilic
- Lyophobic

Depend upon whether or not the colloidal particles have an affinity for the dispersed medium

Hydrosols

- Disperse medium: water
- Hydrophilic
- hydrophobic

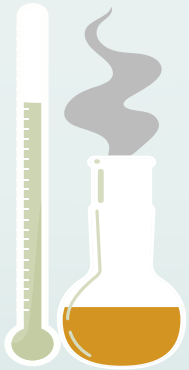
Affinity for the Dispersed Medium

The affinity that **hydrophilic** particles possess for water results from the presence of certain polar groups such as :

- –OH (hydroxide)
- – COOH (carboxylic acid)
- –NH_2 (amino)

on the surfaces of the particles.

These groups are water soluble attract and hold a sheath of water firmly around the particle



Bound water or the water of hydration

The water envelope surrounding a hydrophilic particle

the water of hydration or bound water

(reflecting the magnitude of the bound between the water and the polar groups)

03 Colloidal Behavior

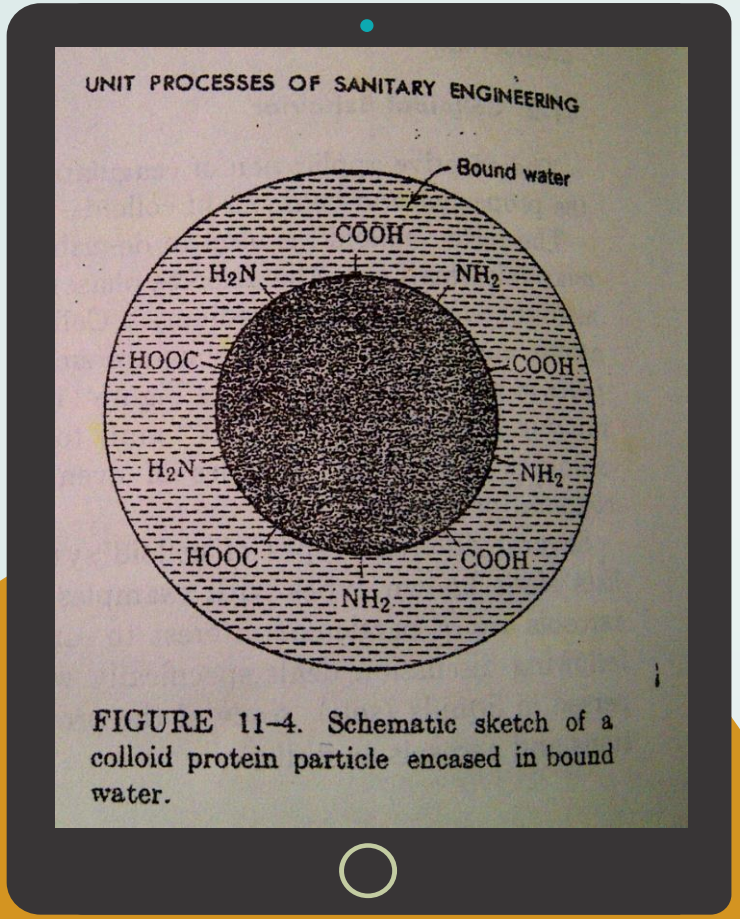


Fig. 11-4: a protein particle of colloidal size showing the particle encased in bound water

Colloid protein particle encased in bound water

such a particle moves as a unit, the translation behaviour of the particle-bound water complex being the same as a discreet, homogenous particle

Hydrophobic & hydrophilic

- **Hydrophobic** particles do not have an affinity for water
- not encased in bound water

Generally:

Organic colloids → hydrophilic

Inorganic colloids → hydrophobic

03.

Zeta Potential

03 Zeta Potential

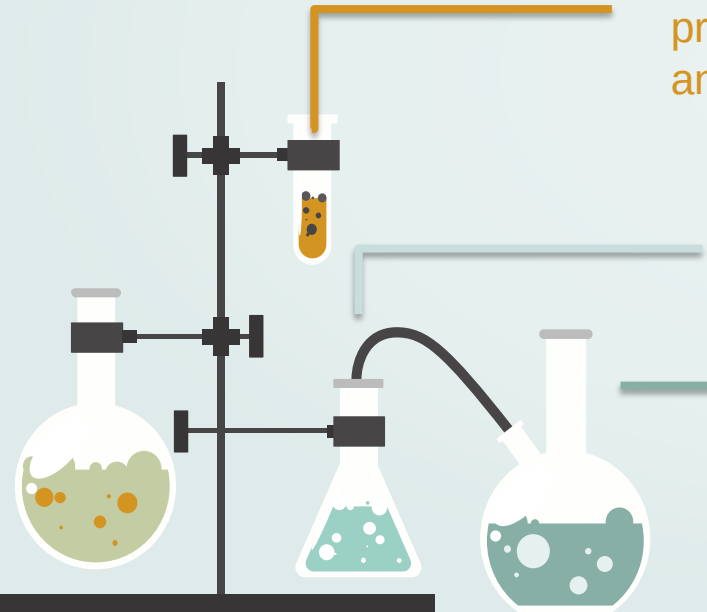
Colloid particles possess electrical properties that strongly influence their behaviour.

Charges located on particles surfaces establish an electrostatic field that is a major factor in determining the stability of the colloid system

These called primary charges, that result from one or both of two phenomena:

1. The **dissociation** of reactive groups on the ends of the molecules comprising the particle structure
2. The preferential **adsorption** of ions from the disperse medium

Hydrophilic colloid particles primary charge - Dissociation of polar groups



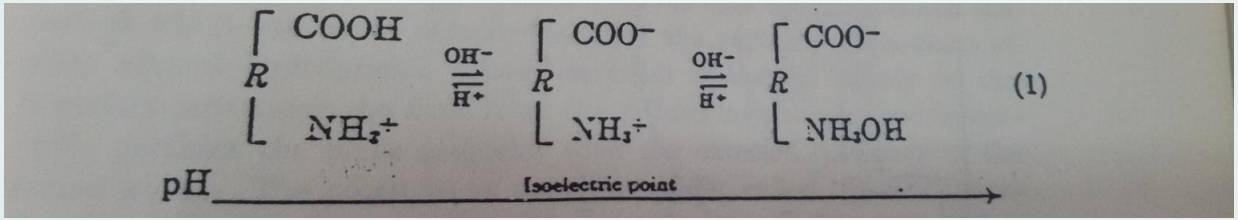
The primary charge on hydrophilic colloid particles is due primarily to the dissociation of polar groups such as –COOH and $–NH_2$.

Eg. Protein molecules & their degradation products contain carboxylic acid & amino groups on opposite ends of the molecule from each other.

In water, the amino group hydrolizes and one or both of the groups dissociates depending upon the pH of the system.

Dissociation of proteins

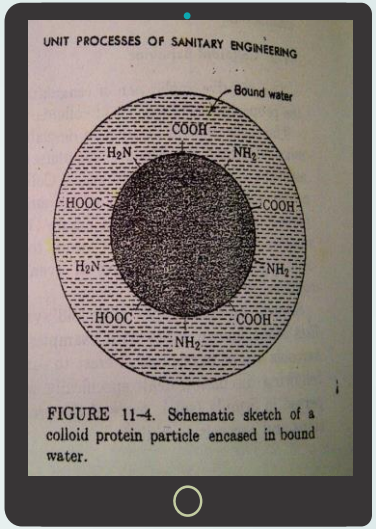
- With the central molecular structure represented by the symbol R, the dissociation is shown as:



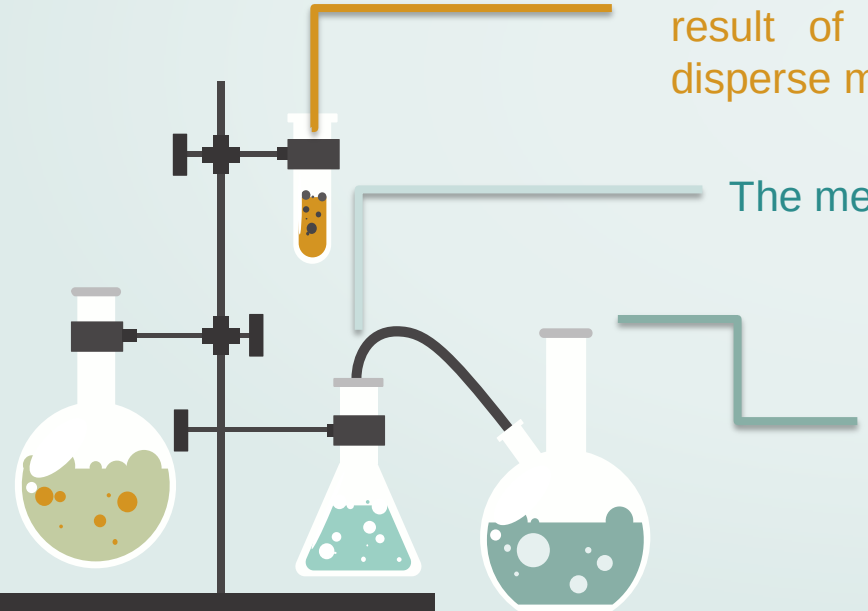
- The reactions are reversible
- At a certain pH (isoelectric point), both groups are ionized and the net charge on the molecule is zero.
- A molecule in this neutral state a zwitter ion

Effect of pH on the charge

- **pH > from the isoelectric point**
  - depressed the ionization of the hydrated amino group
  - results in a net negative charge
- **pH < from the isoelectric point**
  - depressed the ionization of the carboxylic group
  - results in a net positive charge
- The same relatioships are involved between the polar groups located on the surface of a colloid particle shown in **Fig. 11-4**.



Hydrophobic colloid particles primary charge – Preferential adsorption of ions from the disperse medium



The primary charge on hydrophobic colloid particles is the result of the preferential adsorption of ions from the disperse medium.

The mechanism is not clearly understood.

Since the change on hydrophobic particles can be reversed by a change in pH, it has been suggested **that the primary charge is caused by the adsorption of either hydrogen or hydroxyl ions** depending on the relative numbers of each in solution.

A double layer around a colloid particle

Primary Charge

The primary charge on a colloid particle attracts solution ions of opposite charge.

Compact Layer

If the primary charge is sufficiently large, it will be partially balanced by the formation of a compact layer of counterions located adjacent to the particle.

Gouy Layer

The compact layer (the fixed or Stern layer) is surrounded by a second layer of counter-ions (the diffuse or Gouy layer)

Concentration of counterions in the diffuse layer

The concentration of counterions in the diffuse layer varies from the relatively **high concentration** existing at the boundary between the fixed & diffuse layers to that of the bulk of the solution.

Ions having the same sign as the primary charge are present in **minor concentrations** in both layers.

See Fig. 11-5.

A double layer around a colloid particle

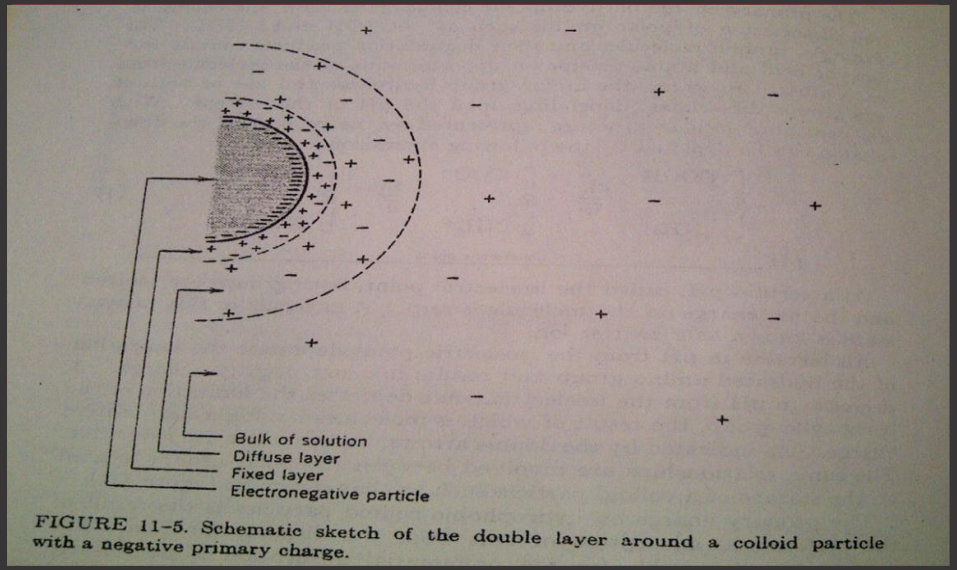


FIGURE 11-5. Schematic sketch of the double layer around a colloid particle with a negative primary charge.

Rich, 1963

Electrostatic Fields

- Concentration differences between anion and cation species result in the establishment of electrostatic fields.
- If the bulk of the solution is considered to have zero potential, the potential at any point across the double layer can be measured by the work required to transport a unit charge from the solution bulk to the particle surface.
- See Fig. 11-6

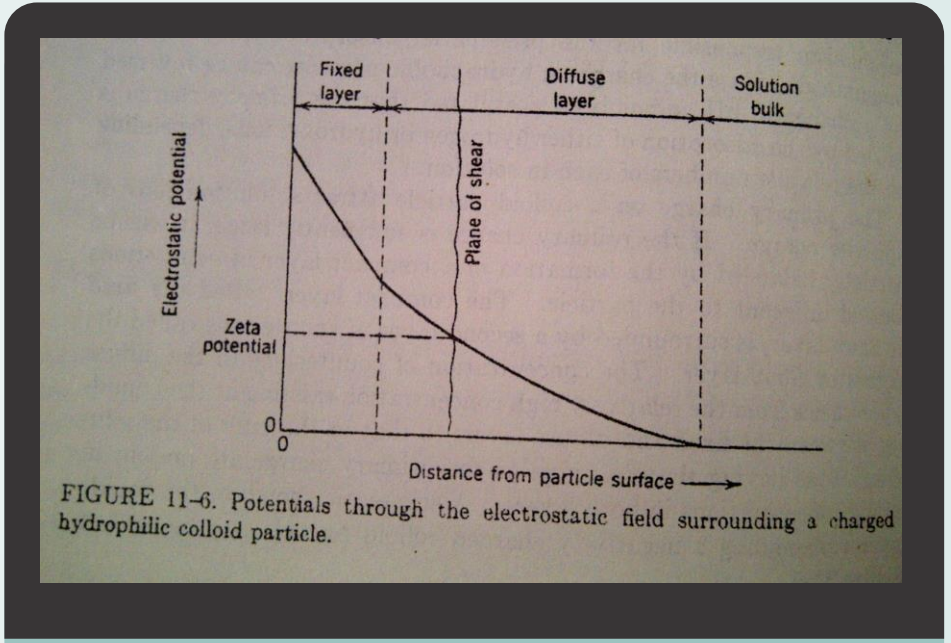


FIGURE 11-6. Potentials through the electrostatic field surrounding a charged hydrophilic colloid particle.

Rich, 1963

Plane of shear (bidang geser)

- The potential existing at the plane of shear is of particular importance.
- **A shear plane:** the two dimensional surface in which shear stress (tegangan geser) occurs
- This plane forms a boundary between:
  - that portion of the solution around the particle that moves with the particle, and
  - The portion which can move independently of the particle

The plane of shear of hydrophillic & hydrophobic colloid particles

The plane of shear around

hydrophobic

located closely to the boundary separating the fixed from the diffuse layer

hydrophillic

coincides with the exterior boundary of the bound water

zeta potential

responsible for the electrokinetic behavior exhibited by colloid particles

Thanks!

Do you have any questions?

slidesgo

TLB 305 DESAIN PENGOLAHAN FISIKA KIMIA II

WEEK 3

COAGULATION

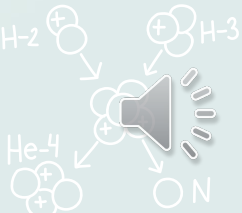
SEMESTER GANJIL 2024/2025

By
GINA MAULUDDINA

01.
Introduction

02.
Sub Topics

03.
About the Sub Topics



01.

Introduction

Sub CP MK 5.1 & Materi

Sub CPMK 5.1

Mahasiswa mampu menerapkan prinsip kerekayasaan, merinci formula, dan merancang unit *mixing (fast mixing*), termasuk peluang untuk *resource recovery*, sehingga melalui pengolahan air dapat melindungi lingkungan dan kesehatan masyarakat

Materi

- Introduction
- Colloidal Behavior
- Zeta Potential
- Colloid Stability
- Coagulation in Water Treatment
- Coagulation in Waste Treatment

COAGULATION

Extracted from:

Rich, L.G.,

Unit Processes of Sanitary Engineering,

John Wiley & Sons, Inc., New York, London, 1963

What Is This Topic About?

It’s about the …..

Definition of Concepts

Coagulation

Process in which chemicals are added to an aqueous system for the purpose of creating rapid-settling aggregates out of finely divided, dispersed matter with slow or negligible settling velocities.

Colloid Particles

- > atoms & small molecules
- Small enough to pass through the pores of ordinary filters
- 10^{-6} mm (1 mμ)- 10^{-3} mm (1 μ)

Zeta Potensial

Responsible for the electrokinetic behavior exhibited by colloid particles

Water / Waste Treatment

Pengolahan air maupun limbah untuk memenuhi baku mutu. Salah satunya dengan cara melakukan pengendapan partikel dengan cara koagulasi.

02.

SUB TOPICS

02 SUB TOPICS

Introduction

Colloidal Behavior

Zeta Potensial

Colloid Stability

Coagulation in Water Treatment

Coagulation in Waste Treatment

03.

Colloid Stability

03 Colloid Stability

A colloid system is said to be stable if the colloidal condition is more or less permanent.

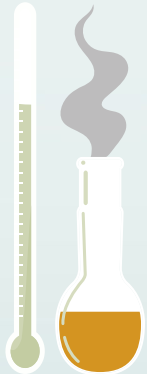
Since the interest in colloids in water and waste treatment centers about the separation of the disperse phase from the disperse medium, colloid

The stability of colloids depend upon the net resultant of the attraction and repulsion forces acting on the colloid particles.

Attraction force

Van der Waal's force

The attraction forces (van der Waal's forces) are caused by the interaction of particle dipoles (permanent or induced).



Repulsion force

zeta potential

bound water

Hydrophobic

furnished by the **zeta potential**

Hydrophilic

In a hydrophilic system the stability depends on:

- Zeta potential
- The bound water : elastic barrier to keep the particles from coming together

inter-particle forces as a function of inter-particle distance

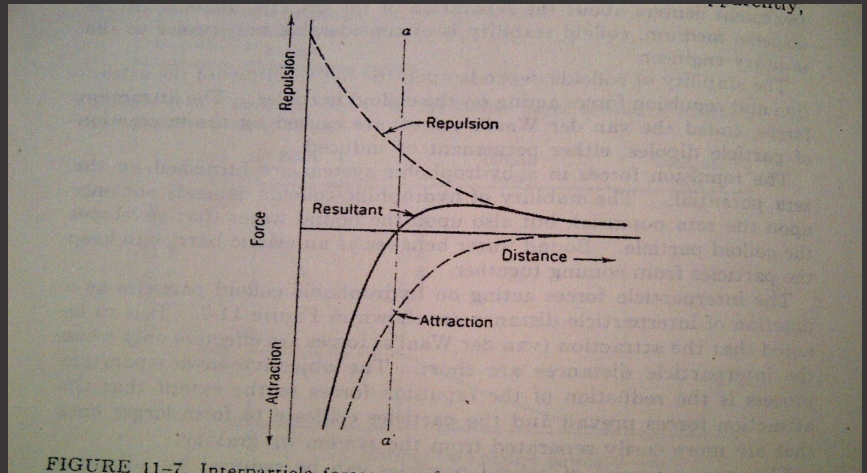


FIGURE 11–7. Interparticle forces as a function of interparticle distance. After Fig. 4–4a, p. 87, K. J. Myseis. *Introduction to Colloid Chemistry*, Interscience, New York, 1959.

Rich, 1963

- The inter-particle forces acting on hydrophobic colloid particles **Fig 11-7**
- The attraction (van der Waal's) forces are effective only when the inter-particle distances are short.

The objective in a separation process

the reduction of the repulsion forces

The attraction forces prevail

The particles coalesce to form larger ones more easily separated from the system by gravity

Reducing the zeta potential

Adjusting the pH of the system toward the isoelectric point

Adding ions or colloids of opposite charge to the colloid system

At the iso electric point:

- The primary charge is zero
- No double layer exists to produce a zeta potential

Adding ions or colloids of opposite charge to the colloid system.

- Precipitation with colloids (mutual coagulation) appears to be a reaction similar to the formation of insoluble precipitates through a combination of solution ions.
- The addition of counterions serves to increase the concentration of counterions in both the fixed and diffuse layers □ zeta potential is reduced.
- See Fig. 11-8

Reduction of zeta potential of hydrophobic colloid through addition of counterions

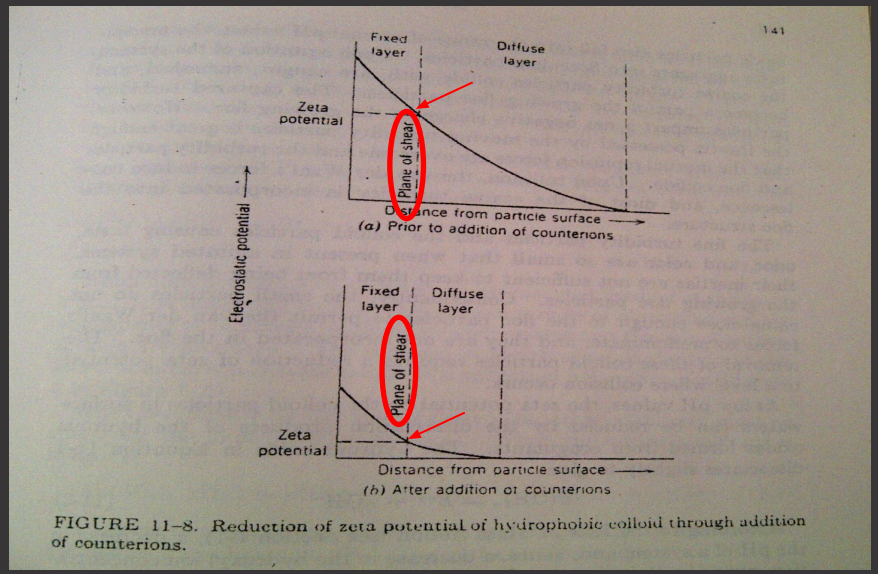


FIGURE 11–8. Reduction of zeta potential of hydrophobic colloid through addition of counterions.

Rich, 1963

The plane of shear in hydrophobic systems coincides closely with the boundary between the fixed & diffuse layers

The valence of counterion

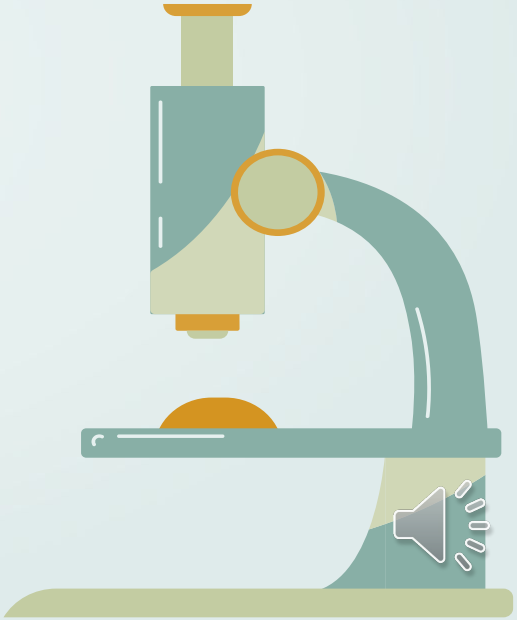
The valence of counterion added is quite important:

A bivalent ion

50x more effective than a monovalent ion

A trivalent ion

1,000x more effective than a monovalent ion



Salting Out

- The stability of hydrophilic colloids depend on the zeta potential & bound water
- Bound water can be reduced by adding salts in high concentration (salting out).

the Hofmeister series

- The effectiveness of salting out depends on the nature of the anion rather than the cation added.
- The effectiveness of the anions in a descending order (the Hofmeister series):

  SO_4^-, Cl^-, NO_3^-, I^-
- Apparently, the ions added to induce precipitation of colloid particles compete with the above anions for the bound water

03.

Coagulation in Water Treatment

03 Coagulation in Water Treatment

The colloidal & suspended particles found in natural waters

negatively charged

The large particles constituting coarse turbidity (majority: clay & silt) □ hydrophobic nature

are economically aggregated through the addition of coagulants

- alum
- ferric sulfate

These materials react with the alkalinity in the water to form hydrous oxides.

Task 1g

What is a/an:
Clay
Silt
alkalinity

Coagulation process of large particles

Where trivalent cations are involved:

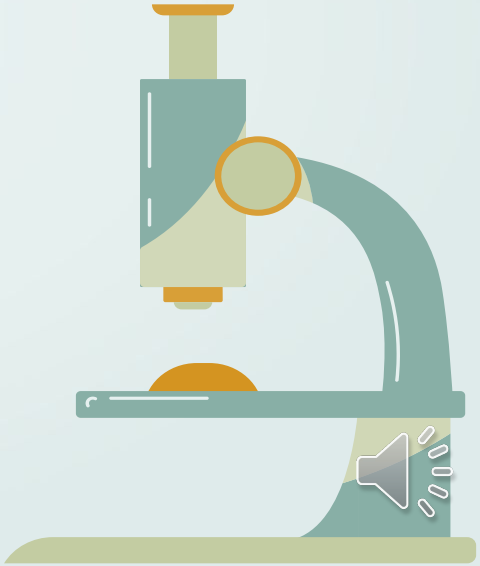
$$B_mA_n + mnOH^- \square mB(OH)_n + nA^{m-} \quad (11\text{-}2)$$

The hydrous oxides are relatively insoluble at normal pH values

precipitate out

Since the isoelectric points of most hydrous oxide particles also fall into the range of normal pH values

the precipitates aggregate into flocculent particles



Coagulation process of large particles

Agitation

the coarse turbidity particles collide with, are caught, enmeshed and become a part of the growing floc particles

The captured turbidity particles impart a net negative charge to the growing floc

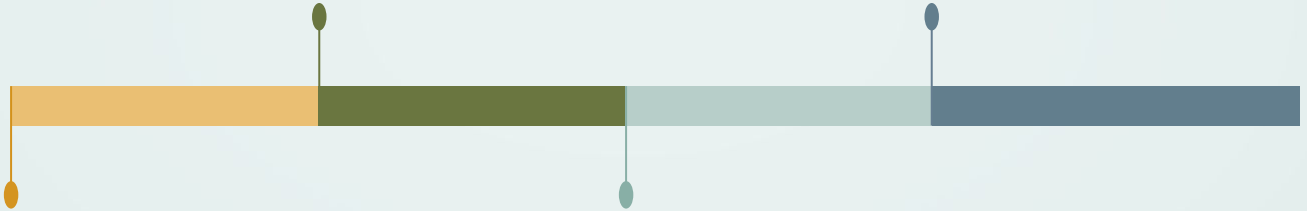
The inertia possesed

The inertia possesed by the moving turbidity particles is great enough that:

- the mutual repulsion forces are overcome,
- and the turbidity & floc collide.

Upon collision

- the van der Waal's forces induce coalescence (peleburan) , and
- most of the coarse turbidity is incorporated into the floc structure



Coagulation process of small particles

The fine turbidity & colloid particles causing taste, odor & color are so small that when present in agitated systems, their inertias are not sufficient to keep them from being deflected from the growing floc particles

- the small particles do not come close enough to the floc particles to permit the van der Waal's forces to predominate
- they are not incorporated in the floc

The removal of these colloid particles requires a reduction of zeta potential to a level where collision occurs.

Reduction of zeta potential at low pH values

low pH

the zeta potential on the colloid particles in surface waters can be reduced by the dissociation products of the hydrous oxides formed from coagulants.

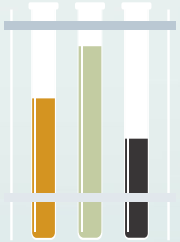
The hydrous oxide in Eq 11-2 dissociates slightly to give

$$B(OH)_n \text{ <--> } B^{n+} + nOH^- \quad (11\text{-}3)$$

Reduction of zeta potential at low pH values

Law of Mass Action (Sec 7-1):

- a decrease in the pH of a system
- a decrease in the hydroxyl ion concentration
- the dissociation of the hydrous oxide
- increase in the concentration of the free cation B^{n+}



The free cations enter the double layer of the colloid particles

Reduce or neutralize the zeta potential

The neutralized colloid particles can make contact with and be adsorbed on the surface of the floc

Addition of Poly-Electrolytes

Colloid particles can be coagulated through the addition of

poly-electrolytes

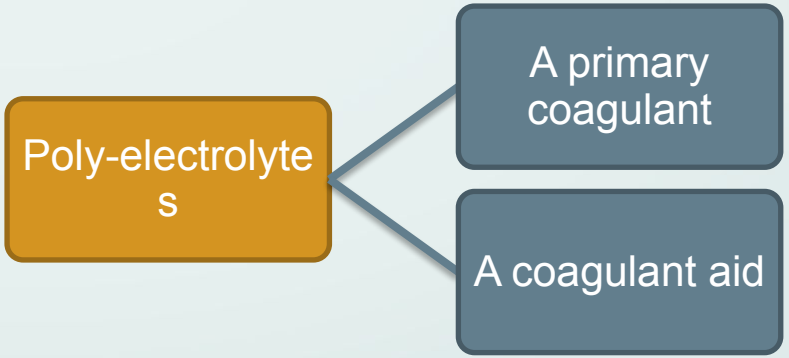
high molecular weight, long-chain, organic polymers with a multitude of ionizable sites along the chain length.

Once a poly-electrolyte is added to a colloid, the ionizable sites dissociate to form charges on the polymer, the sign of which depends upon the nature of the poly-electrolyte

Addition of Poly-Electrolytes

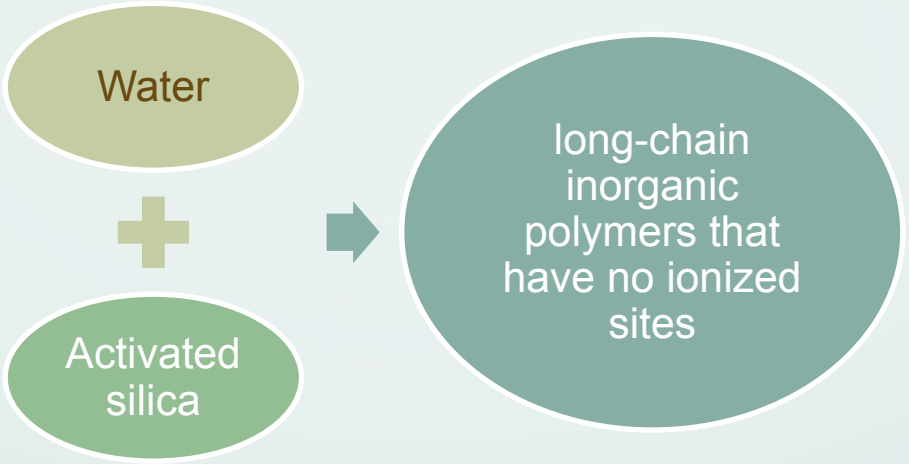
The charged sites attract colloid particles of opposite charge and the polymer structure provides a link between numerous particles forming larger particles more easily separated

Since colloid particles in natural waters normally have a negative charge, cationic poly-electrolytes are popular in water treatment:



Addition of Activated Silica

In many waters low in natural silica, coagulation is facilitated through the addition of activated silica



Polymers function as a coagulant aid by forming bridges through adsorption between the hydrous oxide precipitates produced by the solution of primary coagulants

Alum

Alum (aluminum sulfat) is the most popular coagulant utilized for water treatment

The stoichiometric equation between alum & the natural alkalinity in water can be written as:

$$Al_2(SO_4)_3.xH_2O + 6\,HCO_3^- \rightarrow 2\,Al(OH)_3\,! + 3\,SO_4^- + 6\,CO_2 + x\,H_2O \quad (11\text{-}4)$$

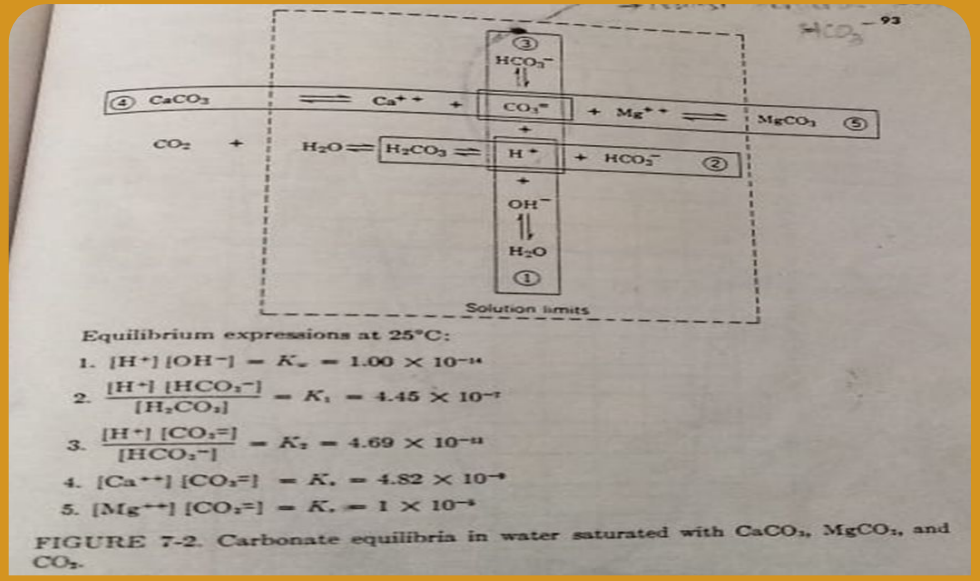
For every mole of alum reacting

- Six equivalents of bicarbonate alkalinity are required
- 6 moles of carbon dioxide are produced

Aluminum hydroxide constitutes the precipitated product of the reaction

Alum

Fig 7-2



Equilibrium expresions at 25°C:

1. $[H^+][OH^-] = K_w = 1.00 \times 10^{-14}$
2. $\frac{[H^+][HCO_3^-]}{[H_2CO_3]} = K_1 = 4.45 \times 10^{-7}$
3. $\frac{[H^+][CO_3^=]}{[HCO_3^-]} = K_2 = 4.69 \times 10^{-11}$
4. $[Ca^{++}][CO_3^=] = K_s = 4.82 \times 10^{-9}$
5. $[Mg^{++}][CO_3^=] = K_s = 1 \times 10^{-5}$

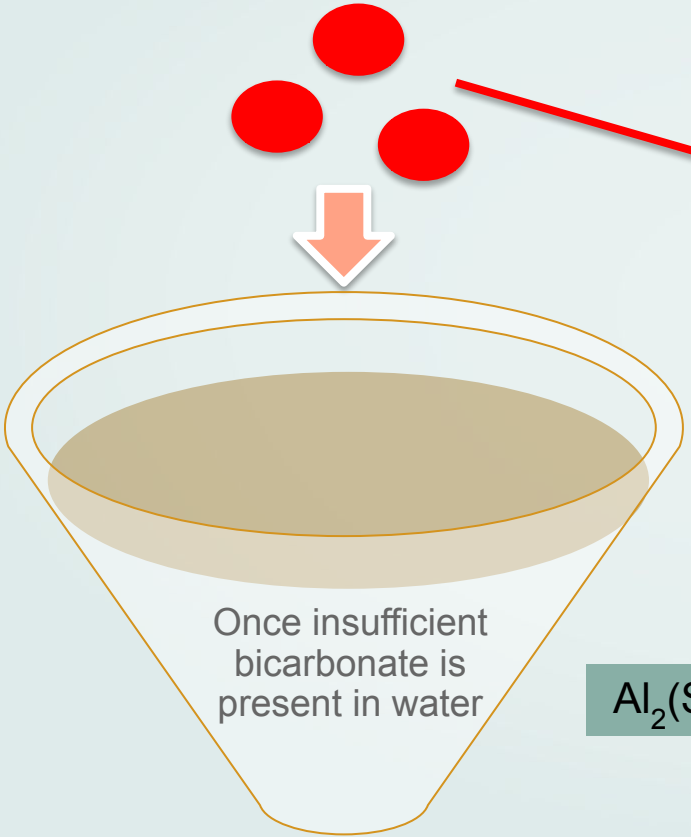
FIGURE 7-2. Carbonate equilibria in water saturated with $CaCO_3$, $MgCO_3$, and CO_2.

Rich, 1963

$$Al_2(SO_4)_3.xH_2SO_4 + 6\,HCO_3^- \rightarrow 2\,Al(OH)_3\,! + 3\,SO_4^- + 6\,CO_2 + x\,H_2O \quad (11\text{-}4)$$

The decrease in bicarbonate alkalinity & the increase in carbon dioxide → a reduction pH (See equilibrium 2 in Fig 7-2)

Hydroxyl as Added Alkalinity



alkalinity must be added for the alum reaction to occur

When the added alkalinity is hydroxyl

$$Al_2(SO_4)_3.xH_2SO_4 + 6OH^- \rightarrow 2Al(OH)_3\downarrow + 3SO_4^- + xH_2O \quad (11\text{-}5)$$

Copperas

Copperas (ferrous sulfate) is also used as a coagulant

When copperas is added to water having a pH falling in the neutral range

$$FeSO_4.7H_2SO_4 + 2OH^- \rightarrow Fe(OH)_2\,! + 3SO_4^{-2} + 7H_2O \quad (11\text{-}6)$$

The formation of feerous hydroxyde (a precipitate) is promoted by increasing the pH

For copperas to be useful as a coagulant in systems devoid of oxygen, the pH must be greater than 9.5

obtained by adding lime, Ca(OH)2.

Copperas once oxygen is present

When oxygen is present in a system, the ferrous hydroxyde is oxidized to ferric hydroxide:

$$4Fe(OH)_2 + O_2 + H_2O \square 4Fe(OH)_3 \downarrow \qquad (11\text{-}7)$$

Normal pH ferric hydroxide is more insoluble than ferrous hydroxide

copperas has greater utility as a coagulant when oxygen is available.

Adding chlorine with copperas

Occasionally, chlorine is added with copperas to oxidize the iron to the ferric form:

$$6FeSO_4.7H_2SO_4 + 3Cl_2 \rightarrow 6Fe^{3+} + 6SO_4^{-2} + 6\ Cl^- + 42H_2O \quad (11\text{-}8)$$

The ferric ion hydrolizes to ferric hydroxide:

$$Fe^{3+} + 3OH^- \rightarrow Fe(OH)_3\downarrow \quad (11\text{-}9)$$

Eq 11-9 also expresses the hydrolyisis ferric sulfate & ferric chloride, two other coagulants which have been used in water treatment.

Solubilities of the hydrous oxides of aluminum & iron as a function of pH

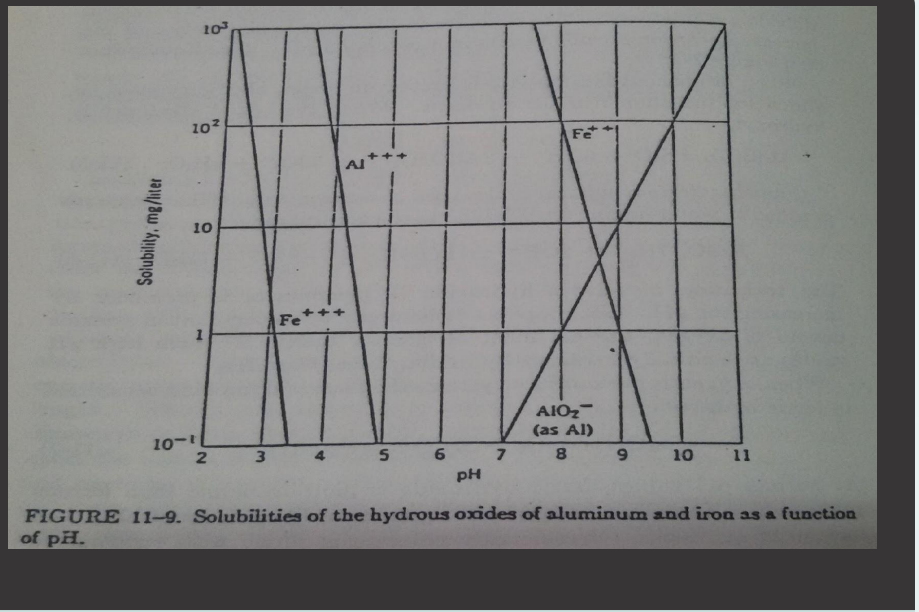


FIGURE 11–9. Solubilities of the hydrous oxides of aluminum and iron as a function of pH.

Rich, 1963

Aluminum hydroxide

- Aluminum hydroxide is the most insoluble between the pH values of 5-7.
- At pH < 5, aluminum hydroxide dissociates to form the aluminum ion
- At pH > 7, aluminum hydroxide dissociates to form the aluminate ion
- $Al^{3+} + 3OH^- \rightleftharpoons Al(OH)_3 \rightleftharpoons AlO_2^- + H^+ + H_2O$ (11-10)

Ferric hydroxide

- Ferric hydroxide does not exhibit an amphoteric nature.
- Fig 11-9: the solubilities of the hydrous oxides of aluminum & iron as a function of pH.

Task 1h

What is a an amphoteric nature?

Coagulant & dosage required

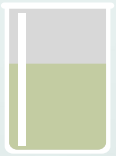
Coagulant

The type & quantity of coagulant needed vary with the characteristics of the water being treated

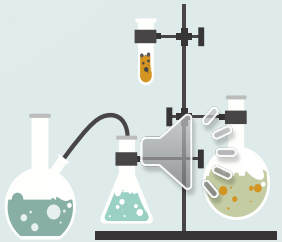
Dosage Required

the coagulant & dosage required to effect optimum coagulation of a specific water must be determined in the laboratory

jar test



electrophetic techniques

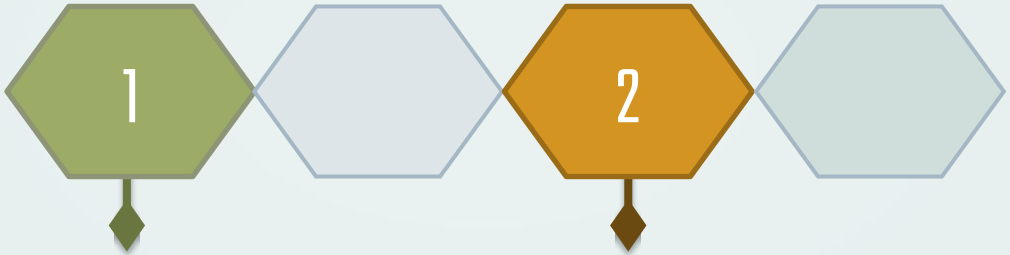


Task 1i

What is the electrophetic techniques?

Coagulation process in water treatment

Is normally carried out ahead of sedimentation & filtration



flash mix

- Water is agitated violently for 1-3 mins
- to disperse the chemicals evenly

flocculation

- Gentler agitation for 30-60 mins
- promotes the agglomeration of precipitates & suspended materials into floc structures easily removed through sedimentation.

03.

Coagulation in Waste Treatment

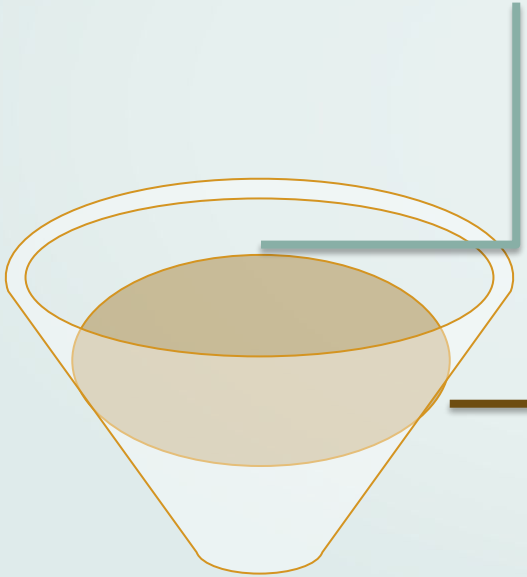
03 Coagulation in Waste Treatment

For the treatment of a variety of industrial wastes:

Factors governing colloid & suspended particle stability are the same as previously discussed

But, the specific coagulant & technology used are vary

Sewage sludges



Digested & undigested sewage sludges are coagulated:

- to improve their filtering properties
- to agglomerate the particles together in a structure that facilitates the separation of much of the water contained within.

Consist primarily of hydrophilic colloids & suspended particles having a wide variety of composition & size.

Mechanism in sludge conditioning

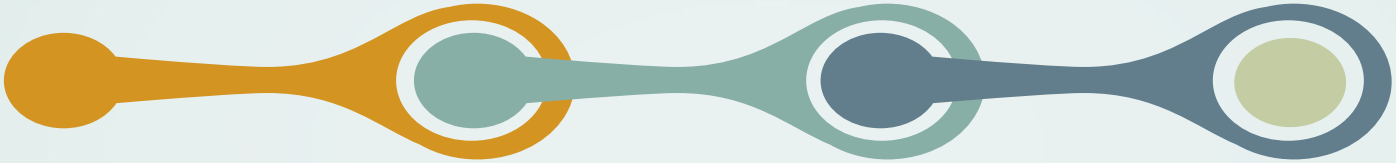
Is not clearly understood.

pH

Conditioning chemicals are generally acid salts of heavy metals

The net charge on the hydrophilic particles is depressed as the pH of the sludge system is shifted towards the isoelectric point

Bicarbonate ions of digested sludges



Digested sludges contain high concentration of bicarbonate ion

exert a demand upon the coagulant

For ferric chloride as a coagulant:

$$FeCl_3 + 3\ NH_4HCO_3 \rightarrow Fe(OH)_3\downarrow + 3NH_4Cl + 3CO_2 \quad (11\text{-}11)$$

Addition of Hydroxyl Alkalinity

bicarbonate demand must be met before a reduction in pH can result

The quantity of coagulant needed for the proper conditioning of sludge can be reduced by decreasing the buffering capacity of the sludge through the addition of hydroxyl alkalinity

$OH^- + HCO_3^- \rightarrow H_2O + CO3^-$ (11-12)

Bicarbonate demand can also be reduced by washing out a portion of the bicarbonate concentration → elutriation

Thanks!

Do you have any questions?

slidesgo

Flocculation (Fair, et al, 1968) 1. Objectives

- Flocculation:
  - Flocculent or flocculated particles conjoin while the suspended fluid is strirred.
- To reduce the load on:
  - the suspending or carrying fluid
  - On subsequent treatment operations

2. Flocculation

- The stirring or agitation of water or ww containing flocs.
- Chemical flocculation:
  - Stirring or agitation soon after the coagulating chemicals has been added -□ mixing
- Mixing□ quick response
- Stirring/agitation □protacted actions □to form the conjunction of suspended particles or flocs

3. Mixing & stirring devices

- Sources of power:
  1. Gravitational
  2. Pneumatic
  3. Mechanical
- Pneumatic & mechanical ☐ flexible
- Gravitational ☐ inflexible ☐ seldom used in large plants

4. Gravitational mixing & stirring
a. Baffled channel

- Differs from:
  - an unobstructed open channel
  - A pipeline
- Shear gradients or turbulence are not merely functions of frictional resistance to flow.
- Velocity gradients are purposely intensified by enforced changes in the direction of flow.

- For baffled channels:
  - Of capacity C
  - In which a loss of head h is incurred when the rate of flow is Q
- The useful power input $P = Q\rho gh$
  - Where ρg = the weight density of water

- The power dissipation function (Eq 1):

$$\frac{P}{C} = \mu G^2$$

- Important concept in WTP & WWTP
- Where:
  - P = power input
  - C = relative capacity
  - μ = Viscosity
  - G = the mean temporal shear or velocity gradient (sec^{-1})

- If G = the mean temporal shear or velocity gradient, is combined with
- The mean or displacement time, t_d = C/Q
  - Where Q = the rate of flow
- Then (Eq 2):

$$Gt_d = \left(\frac{C}{Q}\right)\sqrt{\frac{P}{\mu C}} = \frac{\sqrt{PC/\mu}}{Q}$$

- For a given value of G t_d, the loading of flocculation units (Eq 3):

$$\frac{Q}{C} = \frac{1}{t_d} = \frac{\sqrt{P/\mu C}}{Gt_d}$$

- The hydraulic loading of flocculation units, Q/C, is a function of:
  - Detention time, t_d
  - Power input, P-□ manageable variable in treatment operations
  - Viscosity, μ

- Thus the permissible channel loading at a given value of Gt_d (Eq 4):

$$\frac{Q}{C} = \frac{\sqrt{Q\rho gh / \mu C}}{Gt_d} = \frac{\sqrt{Qgh / \upsilon C}}{Gt_d}$$

- g = gravity constant
- υ = kinematic viscosity of the fluid

Unit conversion

- 1 cuft of water = 62.43 lb = 7, 481 gall
- 1 mgd = 1.547 cuft/sec
- 1 HP = 550 ft lb/sec
- 1 KW = 737.6 ft lb/sec
- Each foot of lost head = 62.43 x 1.547/550 = 0.175 HP/mgd
- Or 62.43 x 1.547/737.6 = 0.131 KW/mgd

- In practice:
- h = 0.5 – 2 ft
- V = 0.5 – 1.5 fps
- t_d = 10 – 60 min

- For (n-1) equally spaced over & under or around-the end-baffles, and
- For velocities v_1 in the channels and v_2 in the baffle slots,
- The loss of head approaches:

$$n\, v_1^2/2g + (n-1)\, v_2^2/2g + \text{normal channel friction}$$

- Assumption: necessary velocities must be redeveloped at each change in direction of flow
- Substitutional estimate: normal channel friction increased by a reduction of the Chezy or Hazen & Williams coefficient of discharge to 20 and 50

- Normal channel friction:
- $h = (n^2 L v^2)/R^{4/3}$
- Where:
- n = Manning coeff = 0.015
- L = channel length per unit
- v = straight channel velocity
- R = hydraulic radius = A/P
- A = wet area
- P = wet perimeter

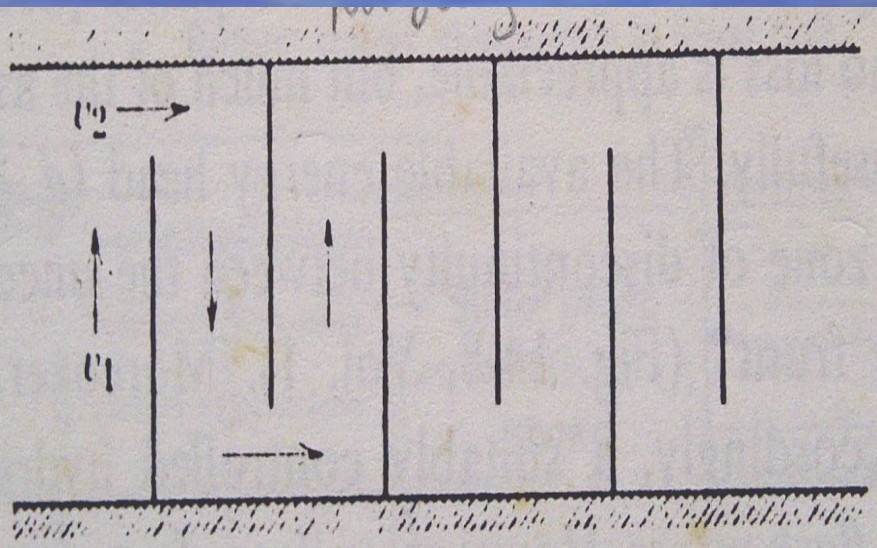


Fig. 26-1. Baffled channel (schematic diagram). Plan of round-the-end baffles, or vertical section of over-and-under baffles.

Example:

Water zigzags through a baffled channel at a velocity of 0.5 fps and speeds up to 1.5 fps in the slots. There are 19 around-the end-baffles. The tank consists of 20 channels each 2.5 ft wide, 8 ft deep on average and 45 ft long. Estimate:

1. The loss of head, neglecting normal channel friction
2. The power dissipated
3. The G and Gt_d values for a flow of 6.46 mgd (10 cfs), with a displacement time of 30 min. Assume a water temperature of 50^0F, i.e., $\mu = 2.7 \times 10^{-5}$ (lb force) (sec)/(sq ft)
4. Calculate the channel loading

1. $h = n\, v_1^2/2g + (n-1)\, v_2^2/2g$

   $h = 20 \times (0.5)^2/2g + 19 \times (0.5 + 1.5)^2/2g$

   $= 1.26$ ft

2. $P = Q\rho gh$

   $P = 10 \times 62.4 \times 1.26 = 790$ ft-lb/sec

3. $\frac{P}{C} = \mu G^2$

$G^2 = P/\mu C$ --> $G = \sqrt{P/\mu C}$

C = 10 x 30 x 60 = 18000 cu ft

G = $(790/2.7 \times 10^{-5} \times 18000)^{1/2} = 40\ sec^{-1}$

$G\ t_d = 40 \times 1800 = 7.2 \times 10^4$

4. $Q/C = 6.46 \times 10^6 / (1.8 \times 10^4) = 360$ gpd/cu ft

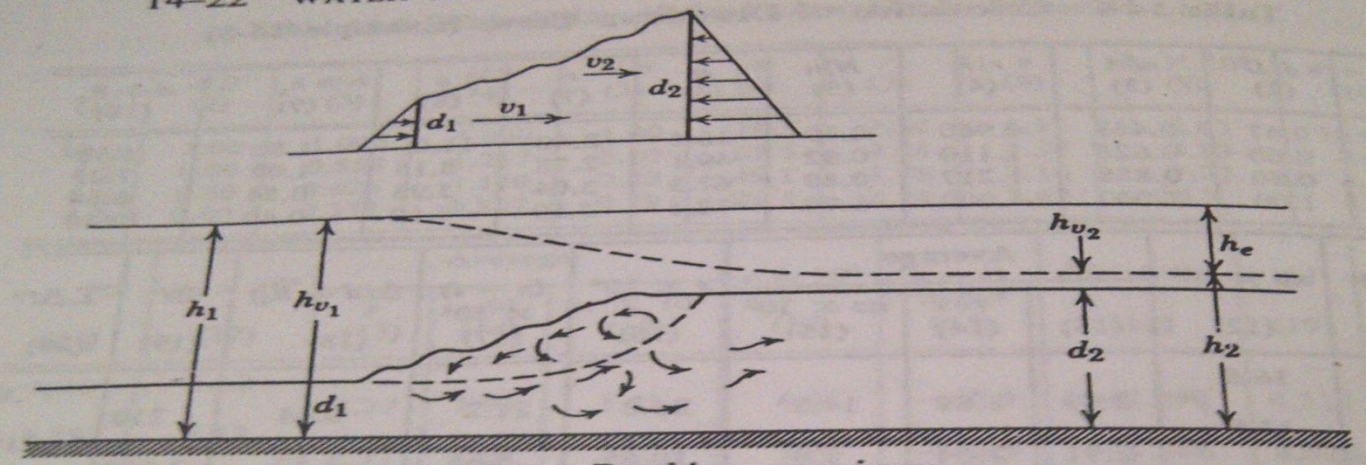
Hydraulic jump mixer

- Includes:
  - A chute
  - Followed by a channel
  - With or without a sill
- The chute ☐ super critical flow
- The gently sloping channel☐ the jump
- The sill ☐ the location of the jump

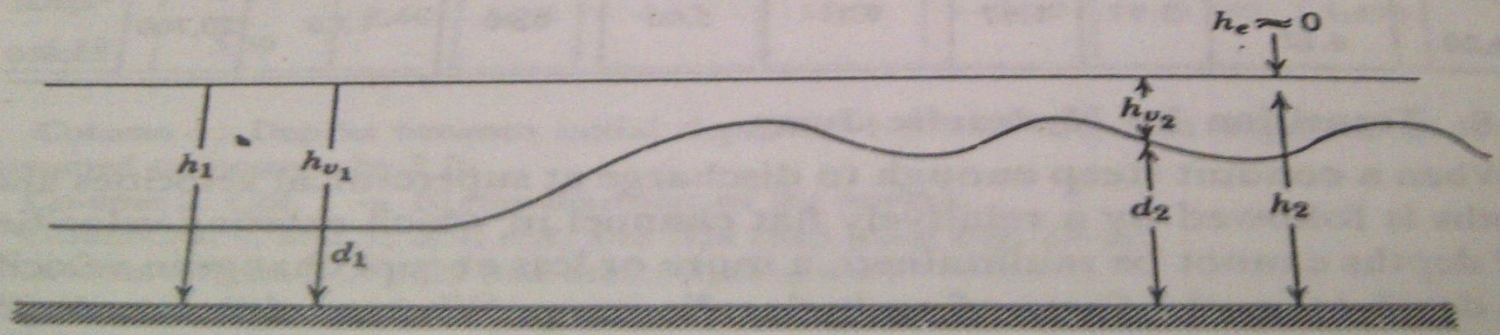
- Hounter Rouse:
- The hydraulic jump □ a breaking wave or turbulent surface roller when:
  - The depth ratio, $d_2/d_1 > 2.375$
  - The Froude number, $F > 2$
- (Eq 5)

$$\frac{d_2}{d_1} = \left[\left(\frac{1}{2}\sqrt{1+8F_r^{\,2}}\right)-1\right] > 2.375$$

- Below the breaking wave, the oncoming flow expands
- Head loss is appreciable
- But much of the associated power is not dissipated fully
- The available energy head ($d + V^2/2g$) is used up largely at the zone of discontinuity between the oncoming flow and the breaking wave front



(a) F > 2. Breaking-wave jump



(b) 2 > F > 1. Undulating jump

Fig. 14-8. Profiles of hydraulic jumps.

- A controlled hydraulic jump □flash mixer
- The jump occurs in a series of undulations with substantially no loss of head and no power of dissipation of any kind when $1<F<2$

5. Pneumatic mixing & stirring

- When air is injected or diffused into water after suitable compression □ expands isothermally
- Thus the work done by the air= $\int P\, \partial V$
- Where:
- P = the absolute pressure intensity
- V = the volume of air

- Because PV = constant (ex. P_a V_a)
- (Eq 6)

$$V_a \int_{V_c}^{V_a} \partial V / V = P_a V_a \ln\left(V_a / V_c\right) = P_a V_a \ln\left(P_c / P_a\right)$$

- a =free or atmospheric condition
- c = compressed condition

- If Q_a cu ft per min of free air are injected into water from a diffuser situated h ft below the water surface, the power dissipated usefully by the rising air bubbles is essentially
- $P = (14.7 \times 144 \times 2.303/60) Q_a \log [(h + 34)/34]$ ft-lb per sec (Eq 7), or
- $P = 81.5 Q_a \log [(h + 34)/34]$ (Eq 8)

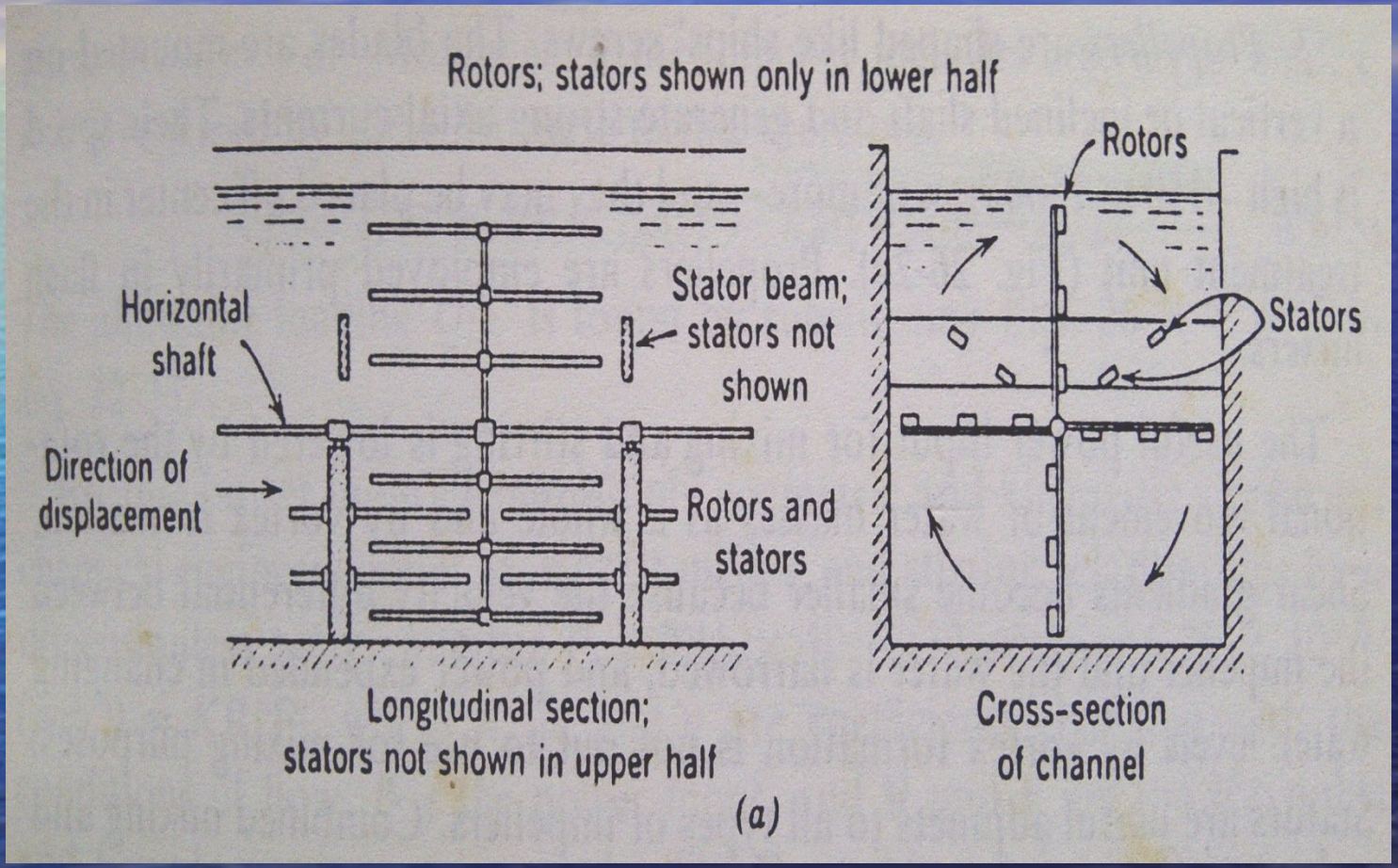
- Area: 1 sq ft = 144 sq inch
- Velocity: 1 miles per hour = 1.467 ft per sec
- Pressure: 1 lb per square inch = 2.307 feet of water

- The allowable loading Q/C of an aerated flocculating tank or channel is found by substituting Eq 3 in Eq 4.

- Compressed air is diffused into treatment units also in:
  - Aeration for gas exchange
  - Cleaning granular filters by air scour
  - Aerating & stirring activated sludge unit

6. Mechanical mixing & stirring

- The impellers employed in mechanical mixing & stirring generate:
  - Mass flow
  - Turbulence
- 3 types of impellers that are commonly used:
  1. Paddles
  2. Turbines
  3. Propellers


Rotors; stators shown only in lower half
Horizontal shaft
Stator beam; stators not shown
Direction of displacement
Rotors and stators
Longitudinal section; stators not shown in upper half
Rotors
Stators
Cross-section of channel
(a)

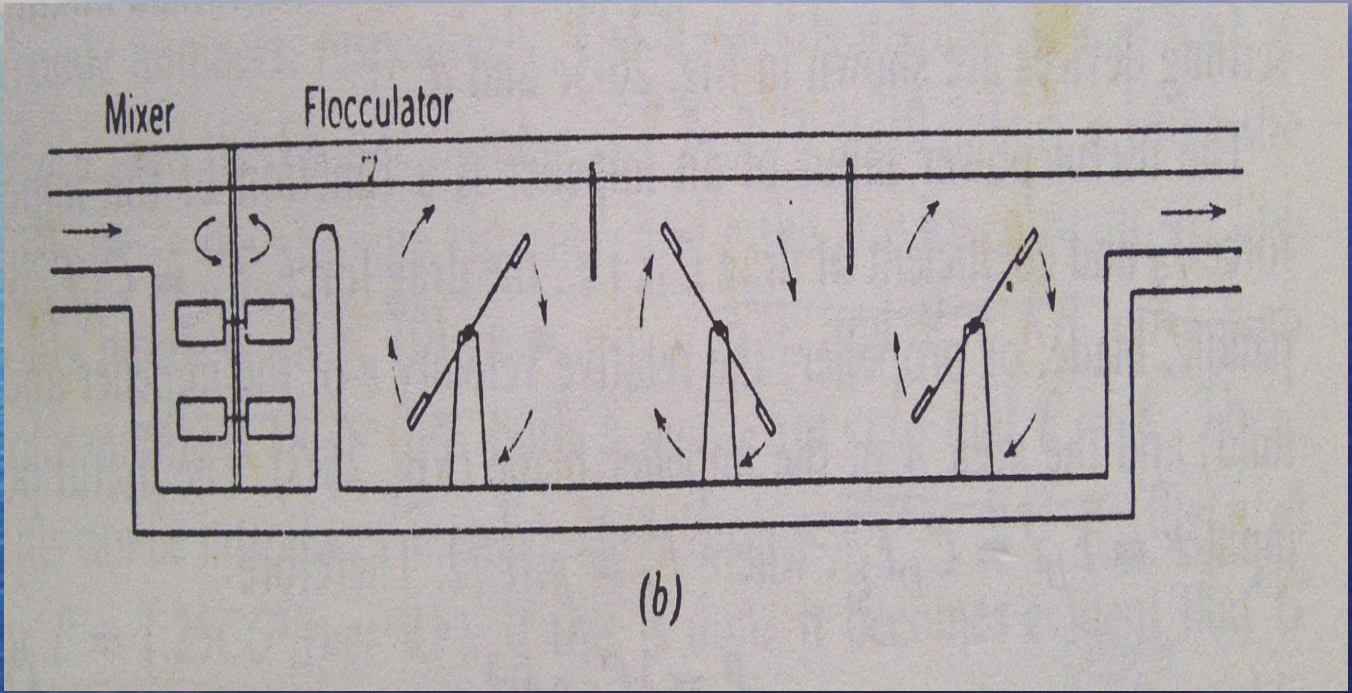
1. Paddles

- Consist of blades attached directly to vertical or horizontal shafts.
- The moving blades (rotors) may be complemented by stationary blades (stators):
  - ☐ oppose rotational movement of the entire mass of water within the treatment unit
  - ☐ help to suppress vortex formation

- Paddles are rotated at slow to moderate speeds of 2 – 15 rpm
- The currents generated by them are both radial & tangential.

2. Turbines

- Comprise flat or curve blades attached by a connecting radius arm to a vertical or horizontal shaft.
- Operating in the middle range of speeds (10 – 150 rpm)
- Generate much of the same currents as paddles



(b)

3. Propellers

- Shaped like ships' screws
- The blades are mounted on a vertical or inclined shaft
- □ generate strong axial currents
- High speed 150 – 1500 rpm or more
- Maybe placed off center in the treatment unit
- Employed primarily in flash mixers

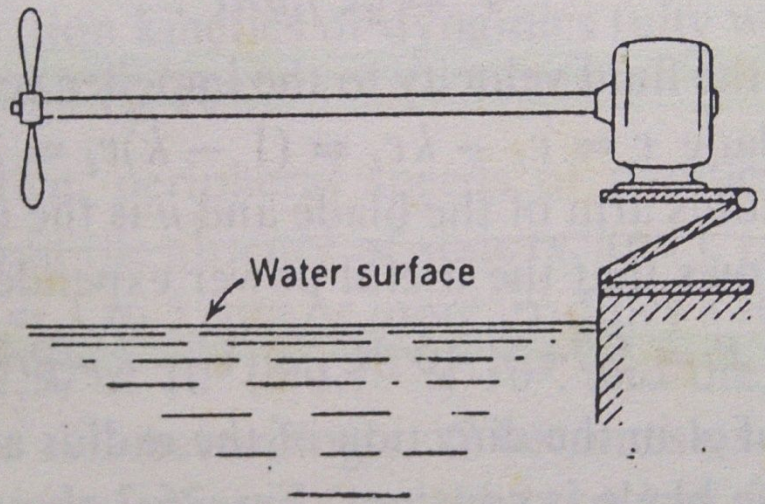
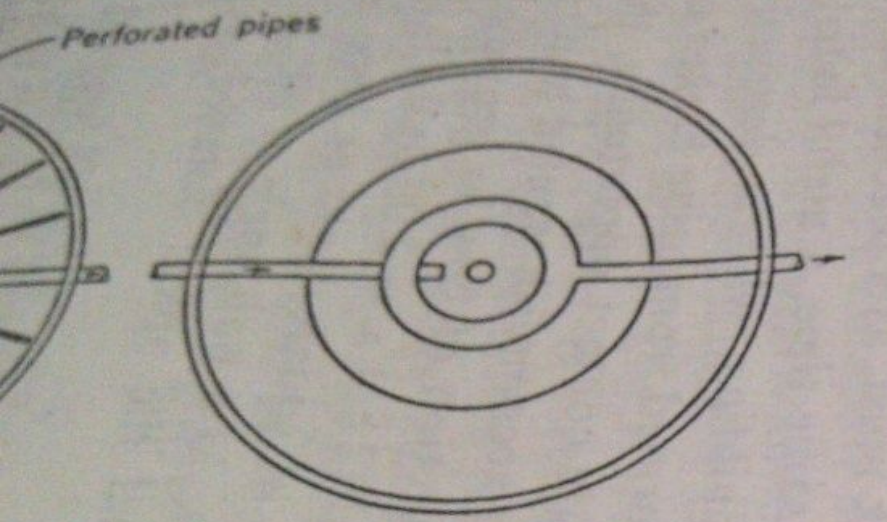


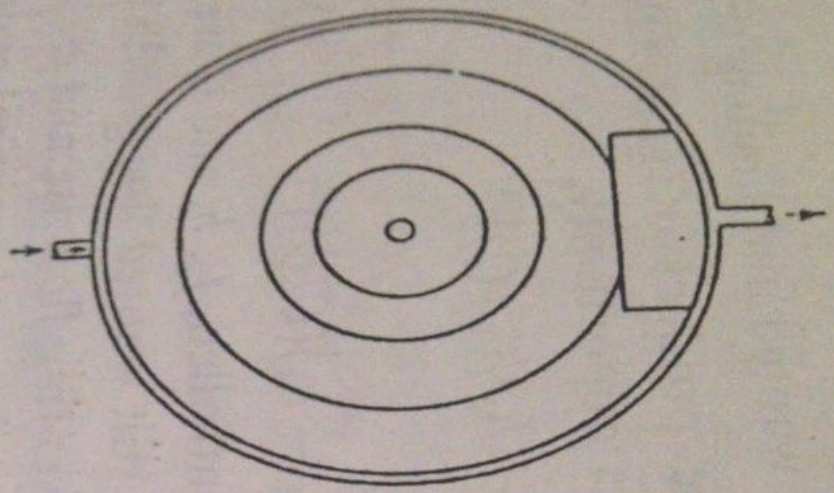
Fig. 26-2. Mixing and stirring impellers. (*a*) Paddle or blade mixer with horizontal, longitudinal shaft, and rotor and stator blades. (*b*) Turbine mixer with vertical shaft followed by cross-channel, horizontal-shaft stirring and resuspension of settling floc in countercurrent fashion. (*c*) Mixing propeller tilted into horizontal position.

- The useful power input for mixing & stirring is lowered by the rotational movement of water masses as a whole and by vortex formation.
- Shear gradients become smaller because:
  - The velocity differential between the impeller and the water is narrowed
  - Power expended in changing water levels by vortex formation is not put to use for mixing purposes.

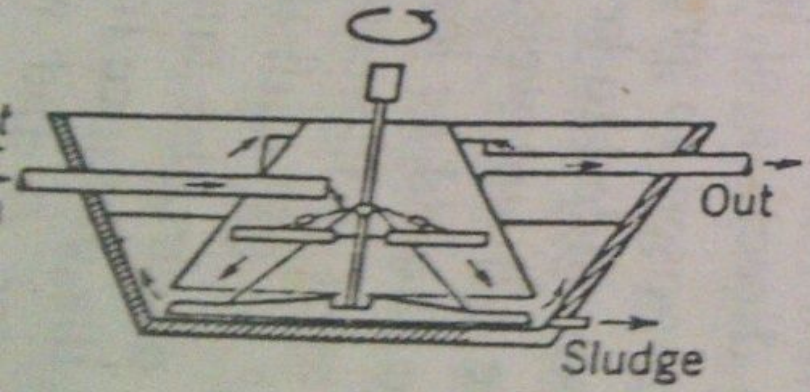
- Stators are useful adjuncts to all types of impellers.
- Combines mixing & settling devices: Fig 26-5c & d



Plan

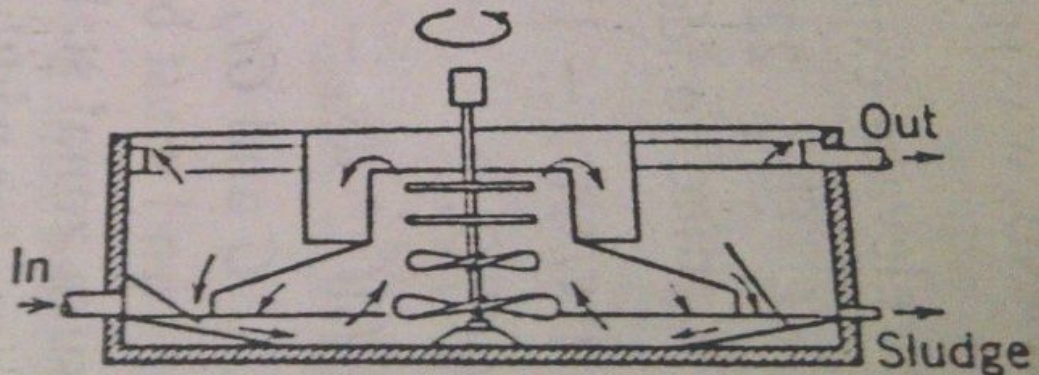


Plan



Elevation

(c)



Elevation

(d)

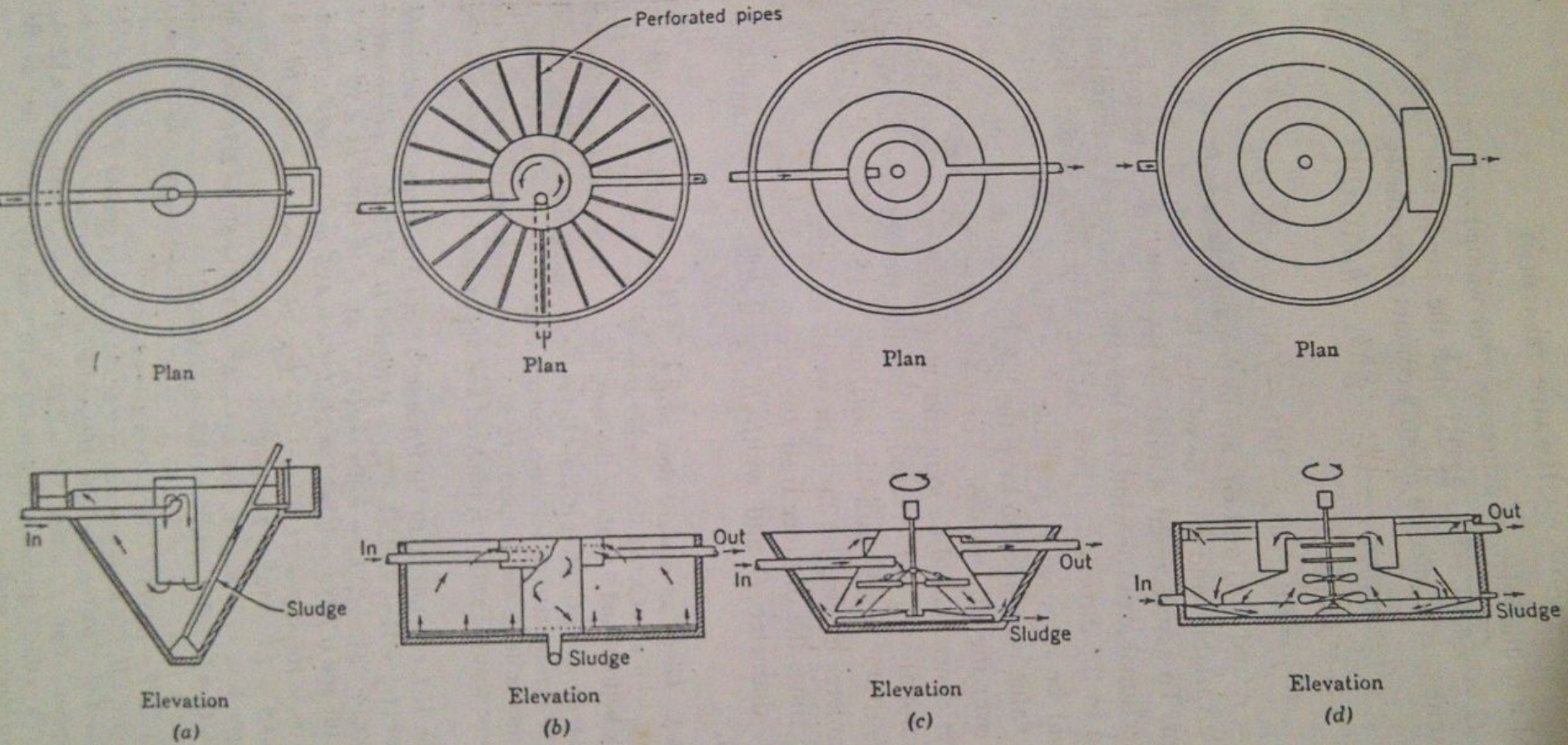


Fig. 26-5. Representative designs of upflow settling tanks. (*a*) Flaring circular or square tank. Sludge is removed during operation; in this case hydrostatically. (*b*) Circular tank with central mixing and flocculating chamber. Pipe grid distributes flow and intermittently collects sludge. (*c*) Circular, flaring tank with central mixing and flocculating chamber. *Spaulding Precipitator.* (*d*) Circular tank with central mixing and flocculating chamber and built-in sludge recirculation. *Accelator.*

- The useful power input of an impeller is a function of:
- The impelling force, F_I
- Coefficient drag, C_D
  - i.e. the drag force, $F_D = C_D F_I$
  - of the paddle, blade or propeller
- The relative velocity, v of the impeller & the fluid

- The useful power input:
- $P = F_D\, v = C_D\, F_I\, v$ (Eq 9)
- Where $F_I = \rho\, A\, v^2/2$ (Eq 10)
- Thus: $P = \frac{1}{2}\, C_D\, \rho\, A\, v^3$ (Eq 11)

- If k is the ratio of the fluid velocity to the impeller velocity (v_i),
- The relative velocity of the blade (Eq 12):
- $v = v_i - k\, v_i = (1 - k)\, v_i = 2\pi\, (1 - k)\, r\, n/60$
- Where:
- r = the effective radius arm of the blade
- n = the number of revolutions per minute

- The useful power expended by a single blade:
- $P = 5.74 \times 10^{-4} C_D \rho [(1 - k) n]^3 r^3 A$ (Eq 13)
- If the dimension of A in the direction of the radius arm is substantial, but the width b is constant

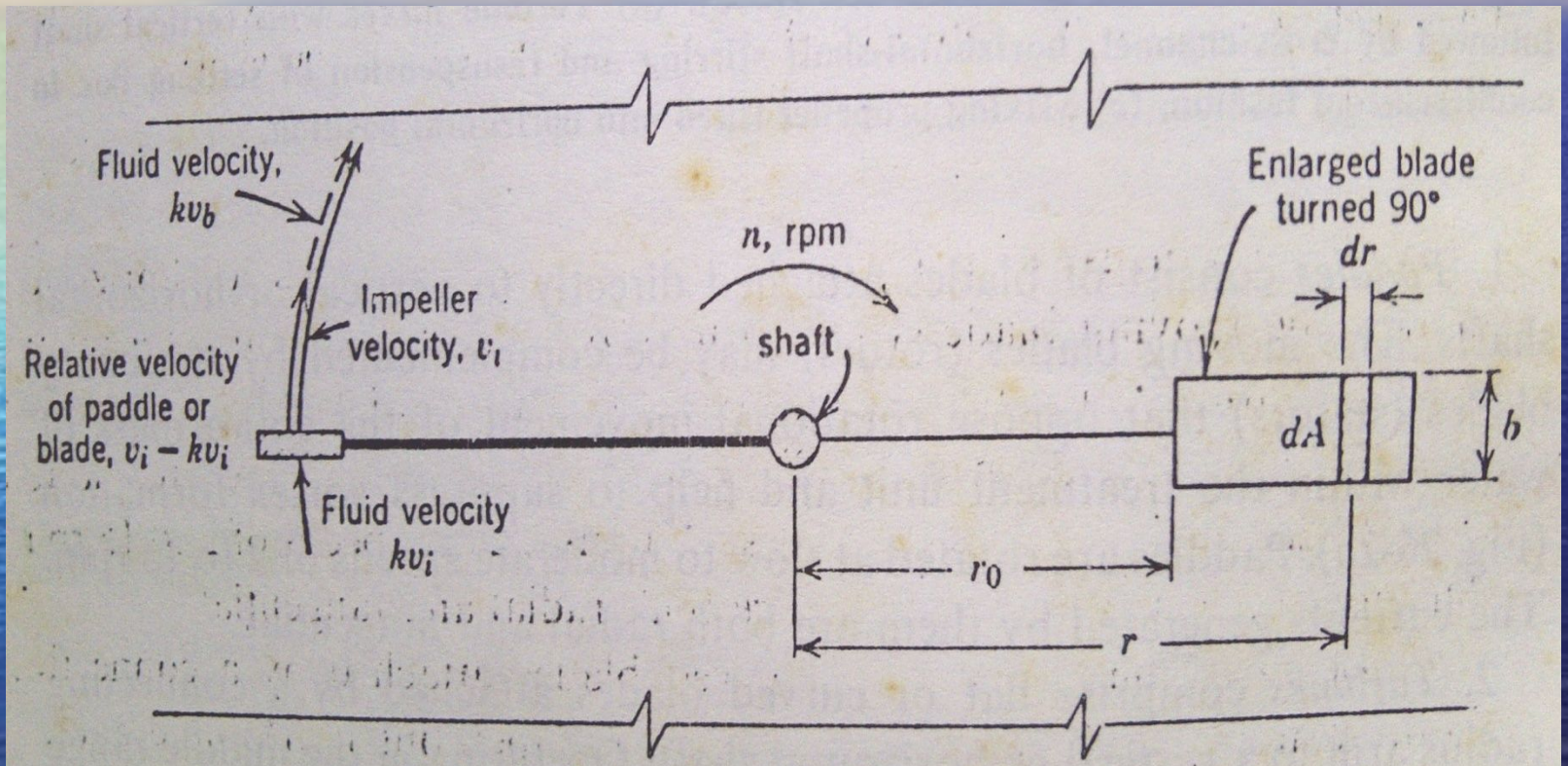


Fig. 26-3. Velocity and power relationships of mechanical mixers or stirrers (schematic diagram).

- Fig 26-3 shows that (Eq 14):

$$r^3 A = \int_{r_o}^{r} r^3 \partial A = b \int_{r_o}^{r} \partial r = \frac{1}{4} b\left(r^4 - r_o^4\right)$$

Because ∂A = b∂r

If the impeller includes a series of blades, r^3A is replaced by $\Sigma\, r^3A$. Thus (Eq 15):

$$P = 1.44 \times 10^{-4} C_D \rho [(1-k)n]^3 b \Sigma\left(r^4 - r_o^4\right)$$

The allowable loading Q/C is found by substituting Eq (15), (13) or (Eq 11) in Eq (3)

- Because power input identifies only geometric & kinematic similarity, chemical engineers prefer to express mixing & stirring relationships by a dimensionless power number (Eq 16):
- $P = 2.16 \times 10^5 \, P/(32\rho n^3 r^5) = 6.75 \times 10^3 P/ /(\rho n^3 r^5) = K R^p F^q$
- Where:
- K, p and q are coefficients changing with the conditions of flow
- P = power input
- R = Reynolds number
- F = Froude number

- $R = 6.67 \times 10^{-2} nr^2 \rho/\mu$
- $F = 2.78 \times 10^{-3} nr^2/g$
- Where:
- ρ = mass density
- μ = absolute viscosity
- g = gravity constant

- At low R number, $R \leq 10$, $p = -1$, $q = 0$ and $R = KP^{-1}$
- At high R number, $R \geq 10^5$, $p = 0$, $q = 0$ and $P = K$
- Can be put it into Eq 13 because
- $n^3r^3 = 5.41 \times 10^3 (\mu/\rho) RF$ or
- $P = 1.25 CG^2/(\rho g r^2 RF)$
- ☐ G and Gt_d identify conjunction kinetics or dynamics fully when R is high ($R \geq 10^5$)
-

In flocculation practice:

- Peripheral speeds of paddles range from 0.3 to 3 fps
- k = 0.25 in the absence of stators
- C_D = 1.8 for flat plates

At n = 1 to 5 rpm or more, paddles 8 ft in diameter have R = 7.57×10^4 to 3.79×10^5 and $P \approx G^2$

Example

A flocculator designed to treat 20 mgd is 100 ft long, 40 ft wide and 15 ft deep. It is equipped with 12-in paddles supported parallel to and moved by four horizontal shafts which rotate at a speed of 2.5 rpm. The center line of the paddles is 6 ft from the shaft, which is at mid-depth of the tank. Two paddles are mounted on each shaft, one opposite the other. If the mean velocity of the water is approximately one-fourth the velocity of the paddles and their drag coefficient is 1.8, find:

1. The velocity differential between the paddles and the water
2. The useful power input and the energy consumption
3. The detention time
4. The value of G and the product G t_d
5. The flocculator loading

Assume a water temperature of 50^0F, μ =

answers

1. $v_i = 2\pi rn = 2\pi \times 6 \times 2.5/60 = 1.57$ fps

$v = (1 - k)\, v_i = (1 - 0.25)\, 1.57 = 1.18$ fps

2. $A = 40 \times 2 \times 4 \times 1 = 320$ sq ft

$P = \frac{1}{2}\, C_D\, \rho\, A\, v^3 = \frac{1}{2} \times 1.8 \times (62.4/32.2) \times 320 \times (1.18)^3 = 918$ ft-lb/sec

And $918/550 = 1.67$ HP or $1.67 \times 0.7457 = 1.24$ KW

- The energy consumption per million gallon = 1.67 x 24/20 = 2 HP-hr/mg or 2 /1.341 =1.5 KW-hr/mg treated. For electrical drive, there must be added the energy required to overcome mechanical friction and to provide for electrical losses in the lines and motor (in practice, flocculators consume 2 to 6 kwhr/mg treated).

3. $V = 40 \times 100 \times 15 = 6 \times 10^4$ cu ft

$t_d = 6 \times 10^4$ cuft x 7.48 gall/cuft x 24 hr/d x 60 min/hr /(20×10^6) mgd= 32.5 min

4. $G = (P/\mu C)^{1/2} = [918/(2.74 \times 10^{-5} \times 100 \times 40 \times 15)]^{1/2} = 23.7$ fps/ft = 23.7/sec

$G\ t_d$ = 23.7/sec x 32.5 min x 60 sec/min= 4.64×10^4

5. $Q/C = 20 \times 10^6 / 6 \times 10^4 = 333$ gpd/cu ft

SOAL

- Diketahui suatu BPAM dengan Qav = 300 l/det. Fm = 1,4. Hasil uji jar test:
- Koagulasi: G = 300/det, td = 90 det
- Flokulasi : G = 90/det, 50/det, 20/det, td utk 3 bak = 10 mnt
- Koagulasi: hydraulic jump, 1 bak
- Flokulasi: baffled channel, 2 bak paralel
- Koagulasi & flokulasi merupakan satu

- Dit:

1. Dimensi bak
2. Tek/energi yang diperlukan
3. Gambar 3D, denah
4. 1 potongan memanjang
5. 1 potongan melintang

- $\mu = 0.273 \times 10^{-5}$ lb sec/ft^2
- Pg = 62.4 lb sec/cu ft
- Q total yang diolah=
- + air yang akan hilang selama proses pengolahan (7.5 – 15)% -☐ ambil: 10 %
- Free board: (10-20)% x kedalaman air -☐ ambil 15%

TLB 305 DEFIKI 2

Dosen: Rachmawati S. Dj.

Week 5

Sedimentasi

Source

- Fair, Geyer, Okun, 1968
- Reynolds, T. D.
- Rich, L. G., 1963

SEDIMENTASI

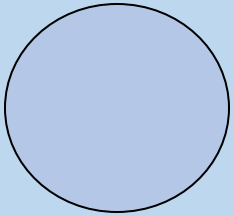
Merupakan pemisahan partikel padat dari suatu suspensi dengan menggunakan gaya gravitasi dimana suspensi dipisahkan menjadi cairan yang jernih (*clarified liquid*) dan suspensi yang terkonsentrasi (*concentrated suspension*) (Fair, Geyer, Okun, 1968)

SEDIMENTASI

- Prasedimentasi (pengendapan partikel diskrit)
- Sedimentasi (pengendapan flokulen)
- Pengendapan Lanjutan
  - Pengendapan hasil pengolahan proses pelunakan
  - Pengendapan pada penghilangan besi dan mangan

Bentuk bak sedimentasi

- *Rectangular*
- *Circular*
- *Square*



Pemisahan zat padat secara individu dengan
cara pengendapan secara gravitasi

Clarification & thickening

Istilah penjernihan (*clarification*) dan pemekatan (*thickening*) juga digunakan untuk menggambarkan proses ini.

Clarification □ Jika tujuannya untuk memperoleh cairan yang jernih

Thickening □ Jika yang dimaksud untuk memperoleh suspensi yang terkonsentrasi di bawah cairan yang jernih

Tujuan proses sedimentasi

Proses sedimentasi digunakan untuk menghilangkan partikel diskrit, materi flokulen, dan *precipitates* dari air yang terbentuk selama operasi pengolahan air.

Kategori sedimentasi

Proses ini dibagi menjadi 4 kategori tergantung dari konsentrasi suspensi dan kondisi/sifat (flok) partikel:

1. pengendapan diskrit (*discrete settling*)/kelas I,
2. pengendapan flokulen (*flocculent settling*)/kelas II,
3. zona pengendapan (*zone settling*) dan
4. pengendapan kompresi (*compression settling*)/pemadatan.

Pembagian zona tipe sedimentasi

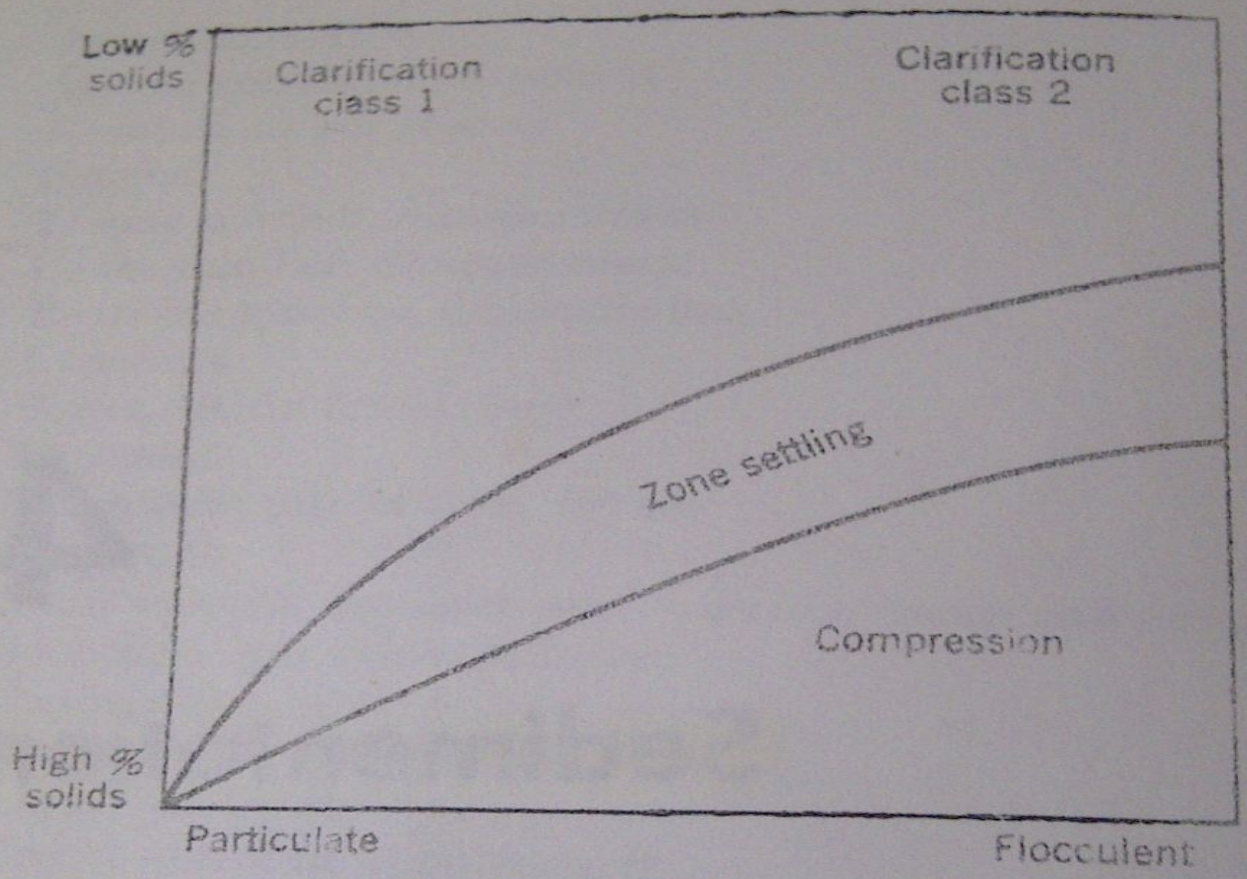


FIGURE 4-1. Paragenesis diagram. [From Figure 2-1, Fitch, E. B., in McCabe, J., and W. W. Eckenfelder, Jr. (eds.), *Biological Treatment of Sewage and Industrial Wastes*—Vol. 2: *Anaerobic Digestion and Solids-Liquid Separation*, Reinhold Publishing Corp., New York (1958), 169.]

Kondisi/sifat (flok) partikel

1. Partikel diskrit

   Mis: butir pasir, butir batu bara, butir tanah

   Yg apabila bertemu satu sama lain tidak akan bergabung

2. Partikel flokulan

   Mempunyai sifat bisa bergabung satu sama lain menjadi 1 partikel yg lebih besar

Konsentrasi/sifat suspensi

1. Suspensi encer:
   kadar solid ≤ 500 ppm
2. Suspensi intermediate/peralihan:
   kadar solid > 500 ppm
3. Suspensi kental:
   kadar solid > 10.000 ppm

Suspensi terlarut

Dua kategori pertama berhubungan dengan suspensi terlarut (*dilute suspensions*).

Partikel pada kategori pertama merupakan partikel diskrit, dan

pada kategori kedua berupa flokulen.

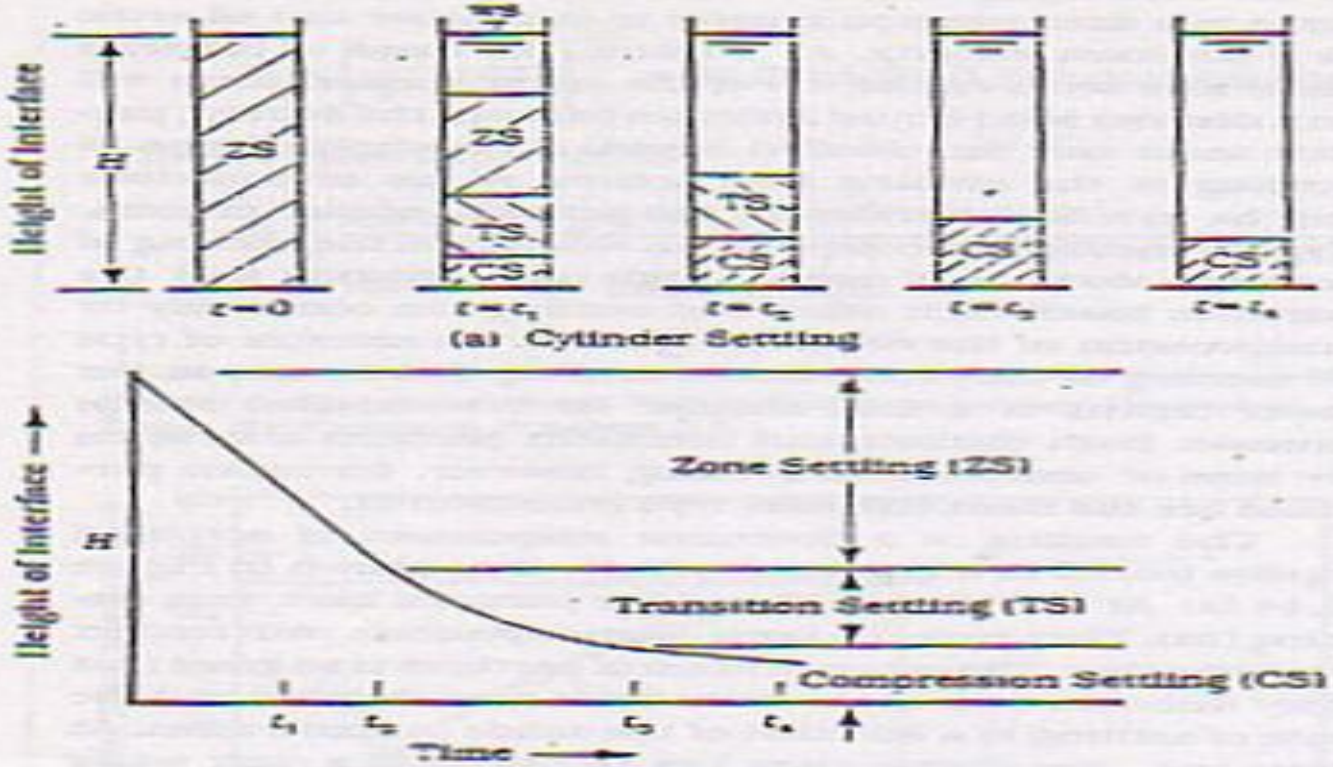
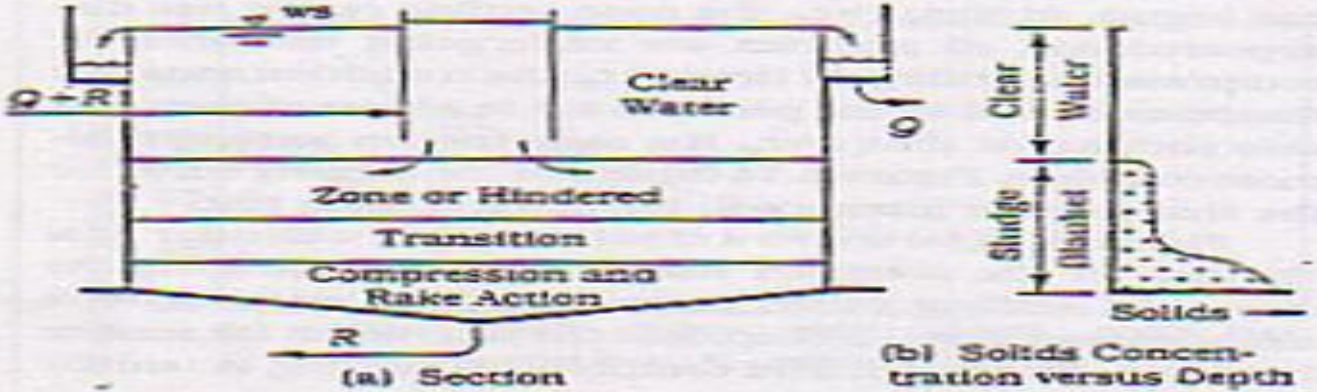


Figure 3.15. Settling in a Final Clarifier for the Activated Sludge Process



Kelas I: suspensi encer-partikel diskrit

Merupakan pengendapan individu (partikel tunggal, bebas, tidak terhalang) partikel satu sama lain berjauhan karena konsentrasi yang rendah

- digunakan pada bak prasedimentasi
- Kecepatan konstan

Grafik pengendapan partikel diskrit dan flokulan

Grafik pengendapan partikel diskrit dan flokulan

Ket:

- Kec horisontal disebabkan oleh aliran air.
- Kec mengendap disebabkan oleh gaya gravitasi.
- Kec mengendap partikel diskrit merupakan garis lurus
- Utk flokulen, kec pertama-tama rendah, makin lama makin berat karena sudah berbentuk flok, dmn kec sebanding dg kwadrat diameter partikel
- $V = k\ d^2$

Kelas II

- partikel flokulan (kumpulan partikel-partikel tunggal), suspensi encer

 digunakan pada bak sedimentasi

 Kecepatan tidak konstan ada percepatan

Selama pengendapan terjadi perubahan bentuk dan ukuran akibat gaya kohesi

Zona *settling*

Partikel flokulan, suspensi intermediate (> 500 ppm).

Pada zona *settling*, sebagian besar partikel yang berupa materi flokulen dalam suspensi dengan konsentrasi *intermediate* mengendap sebagai kesatuan, karena partikel2 tsb sangat berdekatan dimana gaya antar partikel menahan mereka dalam posisi yang tetap relatif satu sama lain.

Zona *settling*

Zone *settling* tdpt hanya pd pengolahan lumpur

Pd zone *settling*, kec pengendapan secara individu tdk ada lagi, ttp pengendapan berlangsung sec zone dmn terjadi penurunan bidang *interface* (=bidang yg menghub 2 zone/suspensi yg berbeda)

- Zone settling terjadi akibat jarak partikel dalam suspensi berada pada jarak yang demikian dekat satu sama lain kohesi
- Posisi seluruh partikel saat mengendap relatif tetap terhadap satu sama lain
- Akibatnya partikel-partikel mengendap sebagai suatu keseluruhan (zona) sehingga dengan jelas suatu garis batas yang bergerak turun antara daerah lumpur dan cairan jernih
- Contoh pada kedalaman menengah pada bak pengendap (clarifier) activated sludge

Gambar zone *settling*

Pd zone *settling* terjadi 2 peristiwa:

1. Clarifikasi *effluent (overflow*)
2. Pengendapan lumpur (*underflow*)

Gambar penurunan *interface*

Kompresi

Ketika konsentrasi amat tinggi dimana partikel2 ada dalam kontak fisik satu sama lain dan didukung oleh masa padat (*compacting mass*), maka proses kompresi yang terjadi.

Cairan dalam masa suspensi menghilang secara perlahan (suspensi pekat) ketika kompresi terjadi.

kompresi

Thickening = pemekatan lumpur

Yaitu utk memperkecil volume lumpur yg akan diolah

Lumpur = solid + air

- *zone settling*
- *Thickening*

Yg mengalami hanya air buangan

Gambar kompresi

Pengendapan klas I

Diterapkan dlm proses prasedimentasi

Ciri:

- Partikel yg berada dlm air bersifat diskrit/non flokulan/partikel tersuspensi yang tidak mengalami perubahan ukuran, bentuk dan berat.
- Suspensi bersifat encer
- Utk air baku yg berupa air sungai/permukaan

Tujuan pengendapan kls I

- Mengurangi bahan kimia yg diperlukan pd proses selanjutnya
- Menurunkan kekeruhan spy air mudah diolah

Lama sedimentasi tgt pd:

- BJ partikel
- Diameter partikel

Gambar prased

Ket gbr prased

- Efisiensi:
- $\eta = \frac{C_{in} - C_{out}}{C_{in}}$ x 100%
- Tujuan: C_{out} atau $C_{effluent}$ memenuhi persyaratan, yg tgt dari musim:
  - Kemarau: $C_{effluent} \leq 500$ ppm
  - Hujan: $C_{effluent} \leq 200$ ppm
- Shg:
  - Bahan kimia yg diperlukan tdk terlalu boros

Pengendapan partikel diskrit

Sebuah partikel diskrit yang bergerak vertical ke bawah melalui cairan *quiescent* akan bertambah kecepatannya sampai gaya friksi *drag* (*the frictional resistance of drag*) cairan sama dengan gaya *impelling* yang beraksi thd partikel, dan setelah itu partikel tsb akan mengendap dengan kecepatan yang *uniform*.

Impelling force (Fi)	=	Drag friction force (Fd)

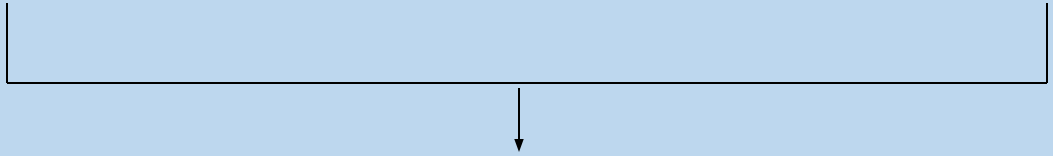
gaya seimbang

 partikel mengendap dengan kecepatan konstan

berat efektif partikel = gaya lawan

↓ F_t ↓ F_D

gaya luar yang bekerja perlawanan gaya geser



Gaya *impelling* F_i

- Gaya impelling Fi sama dengan berat partikel dalam cairan suspensi,

  $F_i = (D_p - D)\, g\, V$…….(3.1)
- Dimana:
- Dp: density partikel
- D: density cairan
- V: volume partikel.

Gaya friksi *drag* Fd

- Gaya friksi drag Fd merupakan sebuah fungsi dari
  - viskositas dinamis, μ,
  - densitas cairan D,
  - kecepatan pengendapan V_s, dan
  - diameter karakteristik partikel, d
- $Fd = f(V_s, d, D, \mu)$

Rumus F_d

- Dengan analisa dimensional dan eksperimen, F_d dapat dinyatakan sebagai:

  $Fd = ½ C_d A_c DV_s^2$……..(3.2)

- Dimana:
- C_d : koefisien *drag Newton*
- A_c : *cross sectional area* dari proyeksi partikel pada sudut yang tepat terhadap arah pengendapan.

Koefisien drag, C_d

- Untuk partikel spherik (*spherical particles*) C_d tergantung dari angka *Reynold* dan dinyatakan sebagai:

  $C_d = 24/Re + 3/(Re)^{1/2} + 0{,}34$…….(11.3)
- Persamaan 11.3 berlaku untuk daerah perbatasan antara transisi dan turbulen (sampai dengan $Re = 1 \times 10^4$).

Kec mengendap partikel, V_s (= kec mengendap terminal, *terminal settling velocity*, Ut)

- Dengan menyamakan F_i dan F_d serta memecahkan persamaan2 di atas, maka:

  $V_s = (2g (D_p – D) V/ (C_d D A_c))^{1/2}$ …..(11.4)

Untuk partikel spherik $V = \Pi (d)^3/6$ dan

$A_c = \Pi (d)^2/4$, maka $V/A_c = 2/3\ d$

Kec mengendap partikel, V_s

- Dengan demikian kecepatan mengendap partikel menjadi:
- $V_s = (4\ g/\ 3\ C_d)\ ((D_p - D)\ d/D)^{1/2}$.....(11.5)
- Utk:
  - daerah perbatasan antara transisi dan turbulen (sampai dengan Re = 1 x10^4).
  - Daerah transisi (Re = 1 – 2000)
- Vs tsb dlm keadaan tenang (*quisance*)

Hukum *Stoke* (laminar, Re < 1,0)

- Untuk wilayah laminar (Re < 1,0) $C_d = 24/Re$ dan $Re = D V_s d/\mu$, Maka dari pers 11.5:
- $Vs = g (D_p – D)d^2/18\mu$……(11.6)

- Pers 11.6 terkenal sebagai hukum *Stoke*.

catatan

- Kecepatan pengendapan partikel dapat dicari dengan pers 11.5 dan 11.6 jika partikel merupakan pertikel spherik dan tidak berubah ukuran dan bentuknya selama pengendapan.
- Tetapi sebenarnya pertikel tidak selamanya diskrit selama pengendapan juga tidak benar2 spherik.

catatan

- Umumnya materi tersuspensi mempunyai luas permukaan (surface area) yang besar dibandingkan dengan volumenya.
- Hal ini mengakibatkan tegangan (resistance) bertambah selama pengendapan.
- Walaupun begitu, persamaan2 ini dapat digunakan untuk mencek disain tanki pengendapan.

V_s yg mana?

- Hukum Stoke dapat dipakai untuk partikel berdiameter kecil (sampai dengan 0,1 cm) dan angka Reynold bervariasi antara 10^{-4} dan 1,0.
- Pada wilayah transisi dimana Re = 1 – 2000, pers 11.5 dapat digunakan.
- Partikel pasir dan flok yang berat umumnya mengendap pada wilayah transisi.

Kecepatan overflow

- Kecepatan overflow pada tanki pengendapan umumnya antara 0,85 – 1,7 m3/m2/ jam.
- Melihat kecepatan pengendapan ini, secara teoritis partikel diskrit dengan gaya gravitasi spesifik (specific gravity) sebesar 1,2 dengan diameter < 0,005 cm tidak bisa dihilangkan.
- Tetapi dalam tanki pengendapan bahkan partikel dengan diameter yang lebih kecil dapat dihilangkan karena agglomeration.

SEDIMENTASI Type 1

- Bentuk partikel diskrit/tunggal
- Kecepatan mengendap konstan ☐ F_D = Fi

$$F_D = C_D \times Ac \times \rho_w \times \frac{Vs}{2}$$

$$Ft = (\rho s - \rho) g V$$

Penurunan rumus

$$F_D = Fi$$

$$C_D \times \frac{\rho_w}{2} \times V_s^2 \times A = (\rho_s - \rho) \times g \times V$$

- V = Vol. Partikel = $\frac{\pi}{6} \times d^3$
- A = Luas proyeksi bola partikel

$$\pi \times r^2 = \frac{1}{4} \times \pi \times d^2$$

$$V_s^2 = \frac{2 \times (\rho_s - \rho) \times g \times \frac{\pi}{6} \times d^3}{C_D \times \rho \ \frac{\pi}{4} \times d^2}$$

$$V_s^2 = \frac{4 \times \rho_s - \rho}{3 \times C_D \times \rho} \times g \times d$$

Rumus UMUM Sedimentasi Type 1

$$V_s = \sqrt{\frac{4 \times g}{3 \times C_D} \ \frac{\rho_s - \rho}{\rho} \ d}$$

$$V_s = \sqrt{\frac{4 \times g}{3 \times C_D} (Ss - 1) d}$$

Nilai Drag Coeficient (C_D)

- tergantung bilangan Reynold (NRe)

1. Laminer (NRe < 1) □

   (Hk. Stokes)

$$C_D = \frac{24}{NRe} = \frac{24}{Vd/\upsilon}$$

2. Transisi 1 <NRe < 10^4 □

$$C_D = \frac{24}{NRe} + \frac{3}{(NRe)^{1/2}} + 0{,}34$$

3. Turbulen NRe > 10^4 □ CD = 0,4

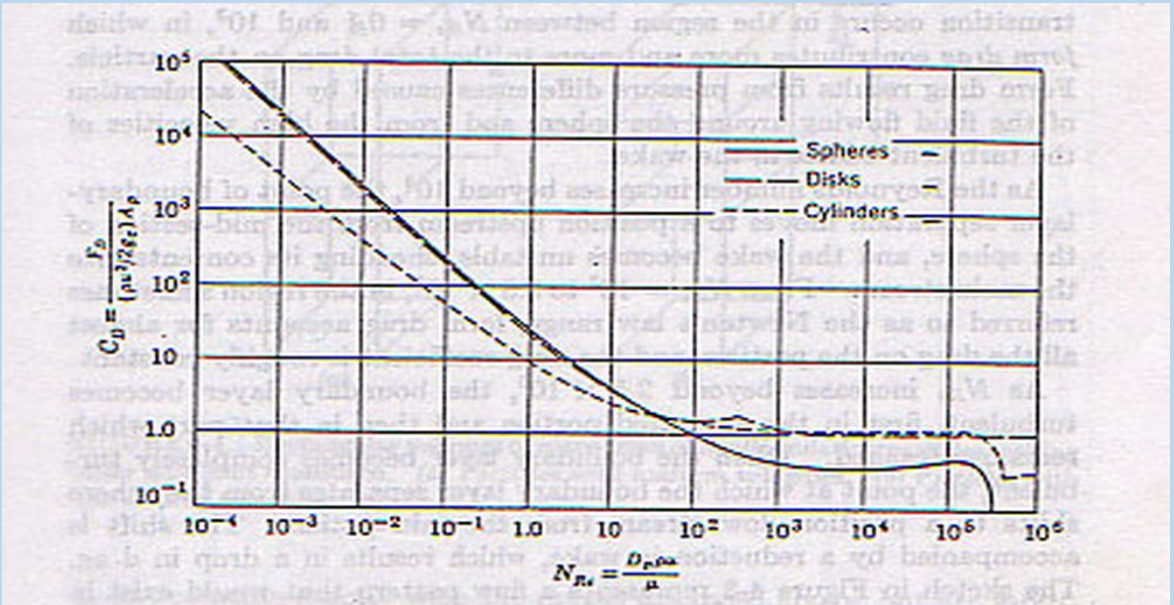


FIGURE 4-2. Drag coefficient for spheres, disks, and cylinders. [By permission from *Chemical Engineers Handbook* (J. H. Perry, ed.), by C. E. Lapple. Copyright, 1950, New York, McGraw-Hill Book Company, Inc.]

Menghitung V settling

- Laminer ke persamaan A

$$V_s = \frac{1}{18} \times \frac{g}{\upsilon} \times \frac{\rho_s - \rho}{\rho} \times d^2$$

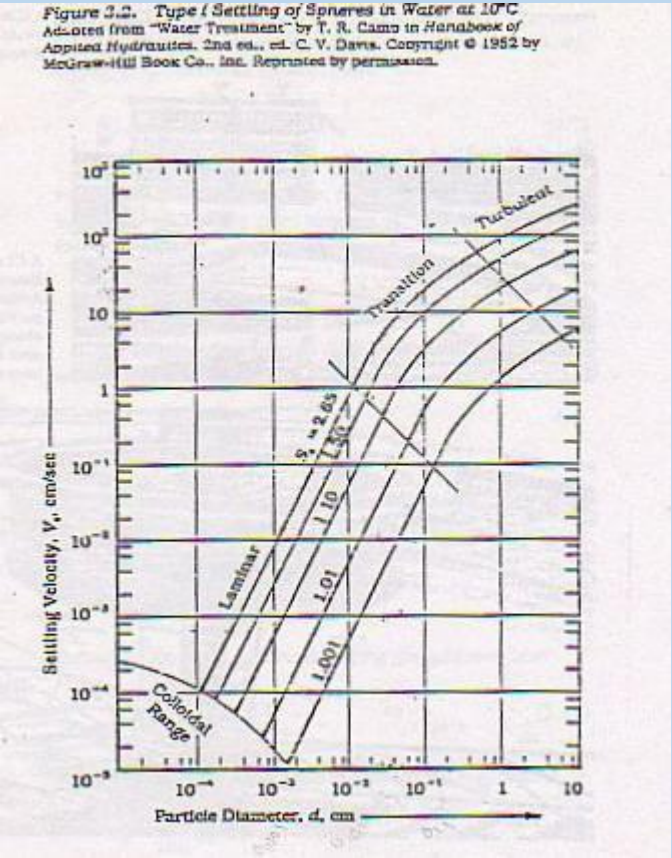
$$V_s = \frac{1}{18} \times \frac{g}{\upsilon} \times (Ss - 1) \times d^2$$

- Turbulen ke persamaan **B**

$$V_s = \sqrt{3{,}3 \times g \times \frac{\rho_s - \rho}{\rho} \times d}$$

$$V_s = \sqrt{3{,}3 \times g \times (Ss - 1) \times d}$$

Grafik Pebandingan Settling Velocity dengan Diameter Partikel



Sedimentasi dalam tanki ideal

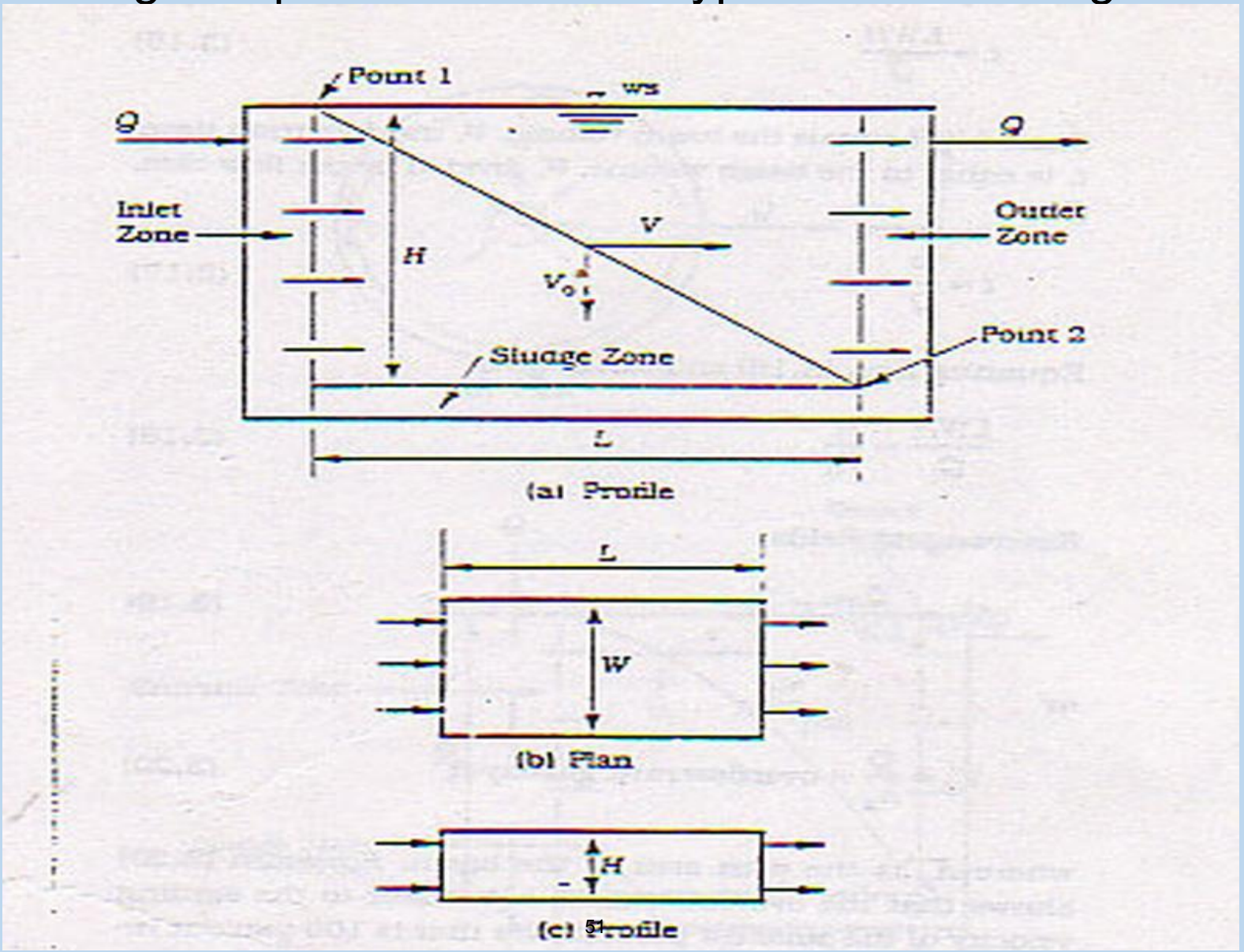
Sebuah tanki yang ideal terdiri dari (Rich, 1963):

1. Sebuah zona inlet dimana air didistribusikan sepanjang *cross section*.
2. Sebuah zona pengendapan (*settling zone*) yang menghilangkan partikel tersuspensi dan yang berada dalam kondisi *quiescent state*.
3. Sebuah zona Lumpur yang mengumpulkan partikel yang mengendap.
4. Sebuah zona *outlet* yang merupakan tempat keluar air dengan partikel yang tidak mengendap (*non settleable particle).*

Teori ideal Basin (Bak ideal) Camp Theory 1946

1. Partikel yang mengendap harus partikel diskrit
2. Aliran terdistribusi merata memasuki bak dan keluar bak
3. Terdapat 3 zone bak (inlet, outlet dan zone lumpur)
4. Partikel yang dihilangkan (*removed*) menyentuh dasar
5. Partikel yang memasuki zone lumpur tetap berada disana sedangkan yang memasuki zone outlet terbawa ke unit selanjutnya

Pola Pengendapan Sedimentasi Type 1 Bak Rectangular



- V_s : Kecepatan mengendap
- V_0 : kecepatan mengendap suatu partikel yang paling kecil yang akan dihilangkan dalam suatu bak (trayek terjauh)
- V_h : kecepatan horisontal

Bila Vs > V_0 maka pasti diendapkan didasar tangki

$$td = \frac{H}{V_0} \quad (\text{vertikal}) \qquad\qquad td = \frac{L}{V_h} \quad (\text{horisontal})$$

RUMUS UNTUK BAK RECTANGULAR

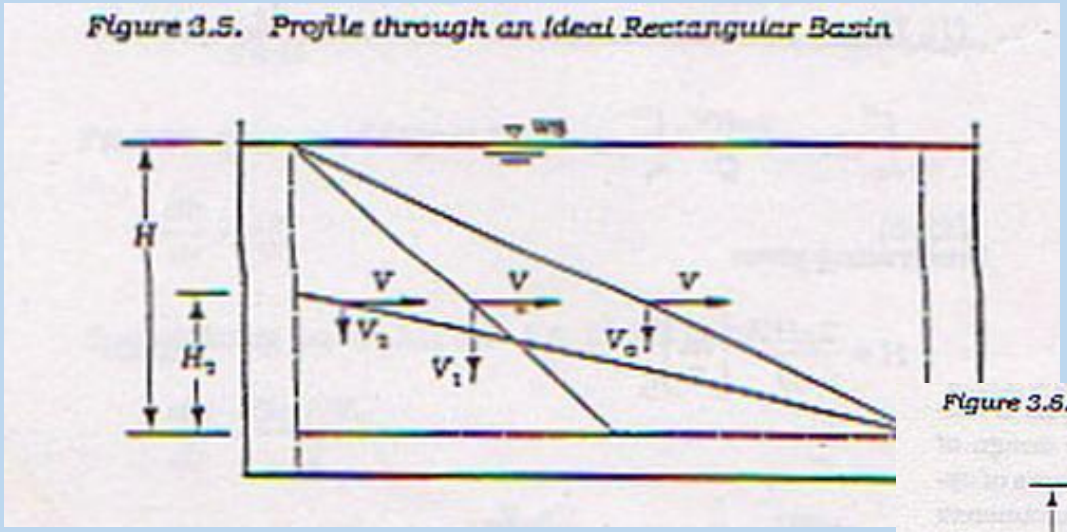


Figure 3.5. Profile through an Ideal Rectangular Basin

$$\frac{H}{V_0} = \frac{L}{Q/H \times W}$$

$$V_0 = \frac{Q}{L \times W}$$

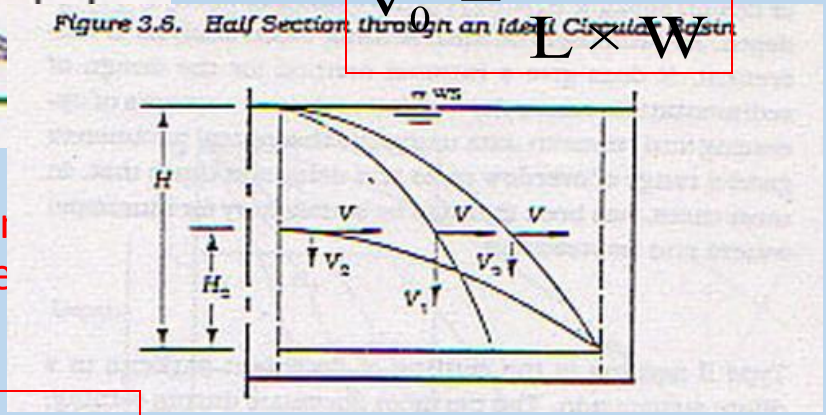


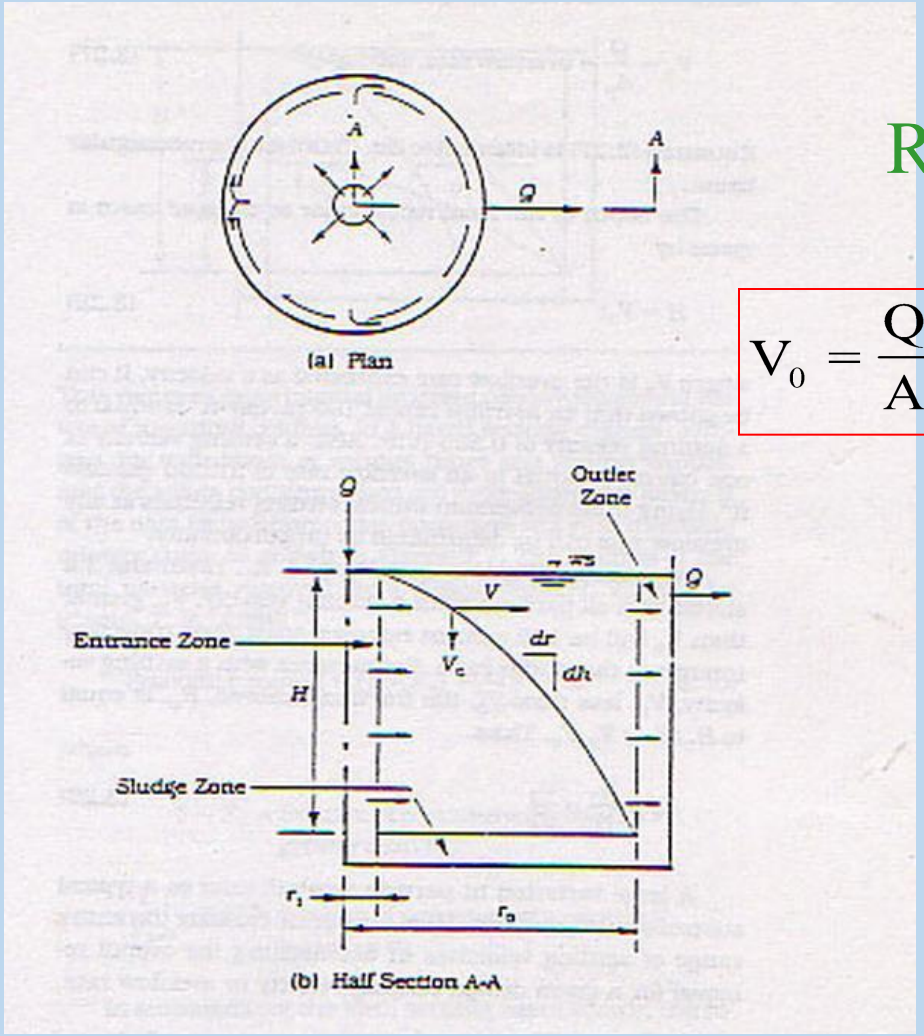
Figure 3.6. Half Section through an Ideal Circular Basin

Pola Pengendapan Untuk Berbagai Ukuran Partike

$$V_0 = \frac{Q}{A_p} = \text{over flow rate (gal/day - ft}^2\text{)}$$

$$V = \frac{Q}{H \times W}$$

Pola Pengendapan Sedimentasi Type 1 Bak Circular



Rumus Untuk Bak Circular

$$V_0 = \frac{Q}{A} = \frac{Q}{\pi \times (r_2 - r_1)^2} \quad \square \quad \frac{Q}{\pi \times R^2}$$

$$H = V_0 \times t$$

Clarification rate
(= kapasitas bak pengendap)

- Gambar

Clarification rate
(= kapasitas bak pengendap)

- Partikel seragam pd (gbr a), yi diameter & BJ sama □ Vs sama, terbagi merata pd seluruh cairan.
- Pd wkt t (gbr b), air yg keruh menjd jernih krn partikel tsb telah turun sedalam z dari permukaan.

Clarification rate
(= kapasitas bak pengendap)

- $Q = \frac{z}{t} A = Vs.\ As$
- As = luas bak pengendap
- Q tgt dr Vs & A,
- Q tdk tgt dr z

Surface loading (=beban permukaan)

- Q/A

Efisiensi pengendapan, η

η tdk mgkn 100% krn dlm praktek partikel tdk seragam.

Utk partikel yg tdk seragam		
P1 → Ut1		Ps sama
P2 → Ut2	Ut1 > Ut2 > Ut3	Pd t=0 → z=0 utk ketiga partikel
P3 → Ut3		

Banyaknya partikel yg terendapkan

- Dibiarkan dlm wkt t sd ketinggian z.
- Gbr
- Z1 > z2 > z3
- Ut1= z1/t
- Ut2 = z2/t
- Ut3 = z3/t

Banyaknya partikel yg terendapkan

- Pd wkt diperiksa, Ceff sdh terbebas dari P1.

Dlm wkt t			
kondisi	**η**	**sebesar**	**∑ partikel yg terendapkan**
Seluruh P1 sdh terendapkan	100%	P1	P1 = 100%
Sebagian besar P2 sdh terendapkan	< 100%	(Ut2/Ut1)P2	P2 =(Ut2/Ut1) x 100%
Sebagian kecil P3 sdh terendapkan	<< 100%	(Ut3/Ut1)P3	P3 =(Ut3/Ut1) x 100%

contoh

Jumlah masing partikel = 5. Ut1 = 2 cm/mnt, Ut2 = 1,5 cm/mnt, Ut3 = 1 cm/mnt. Brp butir yg terambil dan tersisa?

Efisiensi pengendapan, η

- Pd wkt t, semua partikel yg ≥ P1 pd ketinggian endapan z, 100% terambil.
- Ttp partikel yg < P1, yg sehrsnya tdk terambil, kenyataannya sebgan terambil.
- Gbr
- Jd outlet msh mengandung partikel yg < yg terendapkan.
- Remaining particles, bkn tertinggal di bak pengendap, ttp ikut bersama aliran, keluar melalui outlet.
- Semakin bnyk partikel yg terambil, smkn efisien pengendapan yg dilakukan.

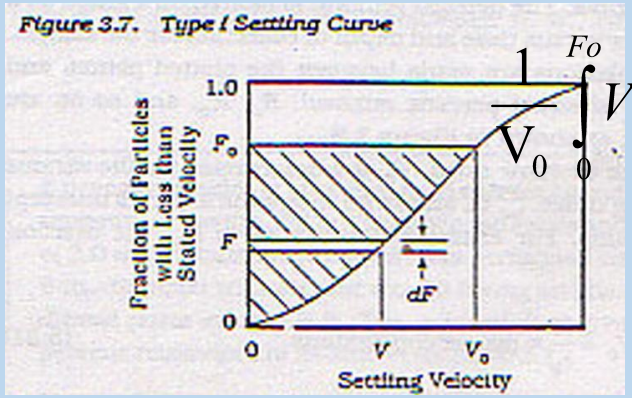
Jumlah partikel yg terambil (total removal solid)

- $Q = \frac{z}{t} A = Vs.\ As$
- As = luas bak pengendap
- Q tgt dr Vs & A,
- Q tdk tgt dr z

Removal Efisiensi

$$\% \text{ penyisihan} = (1 - F_0) + \frac{1}{V_0} \int_0^{Fo} V dF$$

$(1 - F_0)$ Fraksi partikel dengan kecepatan partikel yang lebih besar dari Vo



$\frac{1}{V_0} \int_0^{Fo} V dF$ Fraksi partikel dengan kecepatan partikel yang lebih kecil dari Vo

Keuntungan posisi partikel

- Gbr
- Dr gbr terlihat, pd wkt t, part P2 & P3 yg terletak/berada pd posisi dekat dg grs bts pengendapan, ikut terendapkan, walaupun Ut2 & Ut3 < Ut1

Coloumn settling analysis

Percobaan pengendapan yg dilakukan utk menentukan overflow rate, yg berbentuk kolom/tabung.

Gbr

Pd wkt	Ceff	ket	
t_0	C_0		Kons mkn menurun krn sdh bnyk yg mengendap
t_1	C_1		
t_2	C_2		
t_3	C_3		
t_n	C_n		

$$\square_{\square\square} = \frac{\square\square_0 - \square\square_{\square}}{\square\square_0} \square 100\%$$

Contoh soal

Dari percobaan coloumn settling analysis, diperoleh hasil sbb:

Brp RT, jika diinginkan fraksi yg mempunyai Ut > Ut1 adalah sebesar 70%, serta jarak yg ditempuh (kedalaman), z = 30 cm.

Wkt (mnt)	0	0,5	1	2	4	6	12	20
$C_{e,i}$ (%)	100	85	60	40	30	20	10	5

Clarification rate (=Kec penjernihan, beban perm, overflow rate)

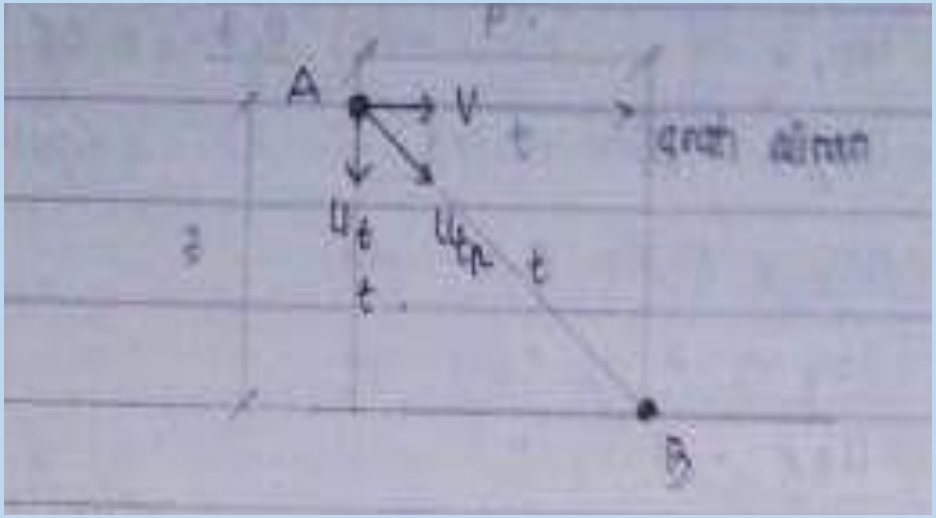
- $Q = \frac{z}{t} A = Vs.\ As$
- As = luas bak pengendap
- Q tgt dr Vs & A,
- Q tdk tgt dr z

(Sifat) aliran dlm bak pengendap

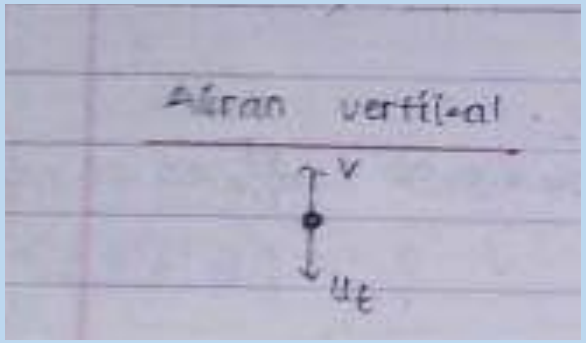
1. (arah) horisontal
2. Vertikal

Gambar

t=z/Ut=P/V=AB/UtR



Aliran vertikal



Gambar

UtR = (Ut-V) > 0 □ mengendap

= 0 □ melayang

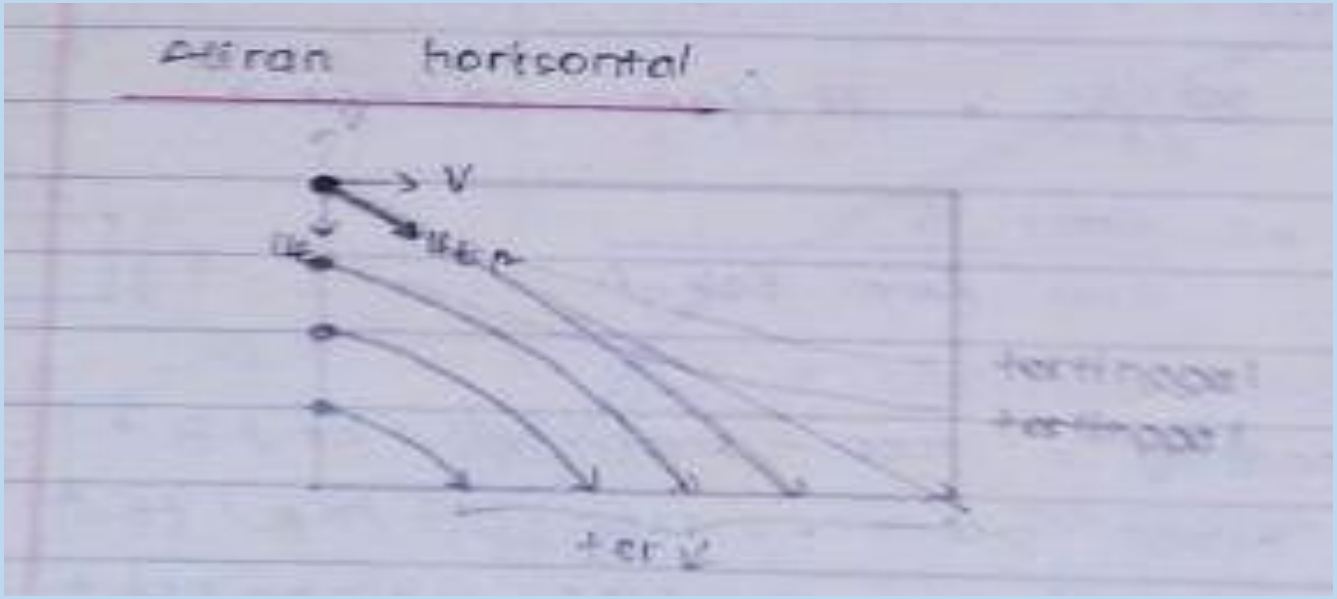
< 0 □ tertinggal

Utk partikel diskrit, aliran vertikal tdk menguntungkan, krn partikel dg UtR <0

tdk terendapkan sama sekali

Aliran horisontal

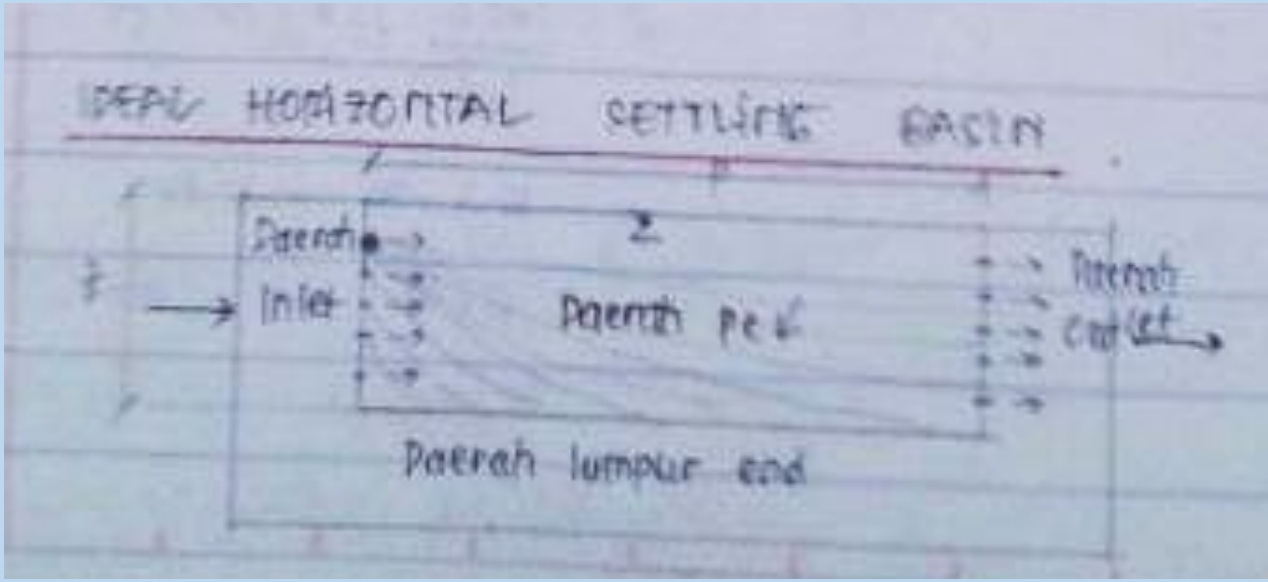
Gambar



Utk partikel diskrit, sebaiknya arah aliran horisontal spy partikel yg seharusnya tdk terendapkan dpt terendapkan, krn adanya V horisontal yang menyebabkan UtR >

Ideal horisontal settling basin

Gambar



Ciri:

- Memp 4 zone
- Arah aliran sejajar/merata di seluruh bak pengendap
- Inlet & outlet dibagi merata utk seluruh kedalaman bak
- Digunakan utk partikel diskrit spy partikel yg seharusnya tdk terendapkan dpt mengendap
- Perbandingan: p= 3-5 l

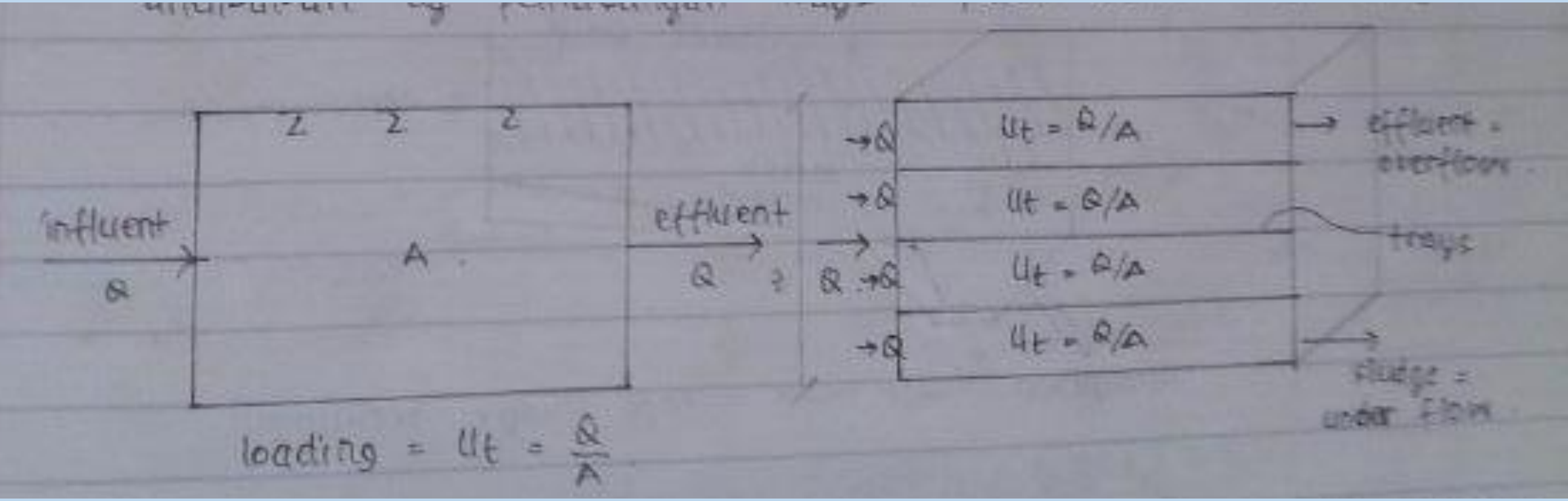
Unhindered settling

Pengendapan kls I disebut unhindered settling (=tdk saling terganggu oleh partikel lain), krn suspensi encer shg masih tdpt jarak yg besar ant partikel satu sama lain

Unhindered x hindered settling = saling terganggu.

Peningkatan kapasitas bak pengendap

Dilakukan dg pemasangan trays = false bottom = dasar semu



trays

Dg pemasangan trays, kapasitas dpt ditingkatkan menjadi nx, sebanyak trays yg dipasang, krn dg pemasangan trays berarti menambah luas penampang bak

trays

Pemasangan trays bukan utk meningkatkan efisiensi ttp utk meningkatkan kapasitas bak pengendap (=banyaknya air yg diolah/dialirkan ke dlm bak pengendap), sedangkan efisiensi berhubungan dg kualitas efluen.

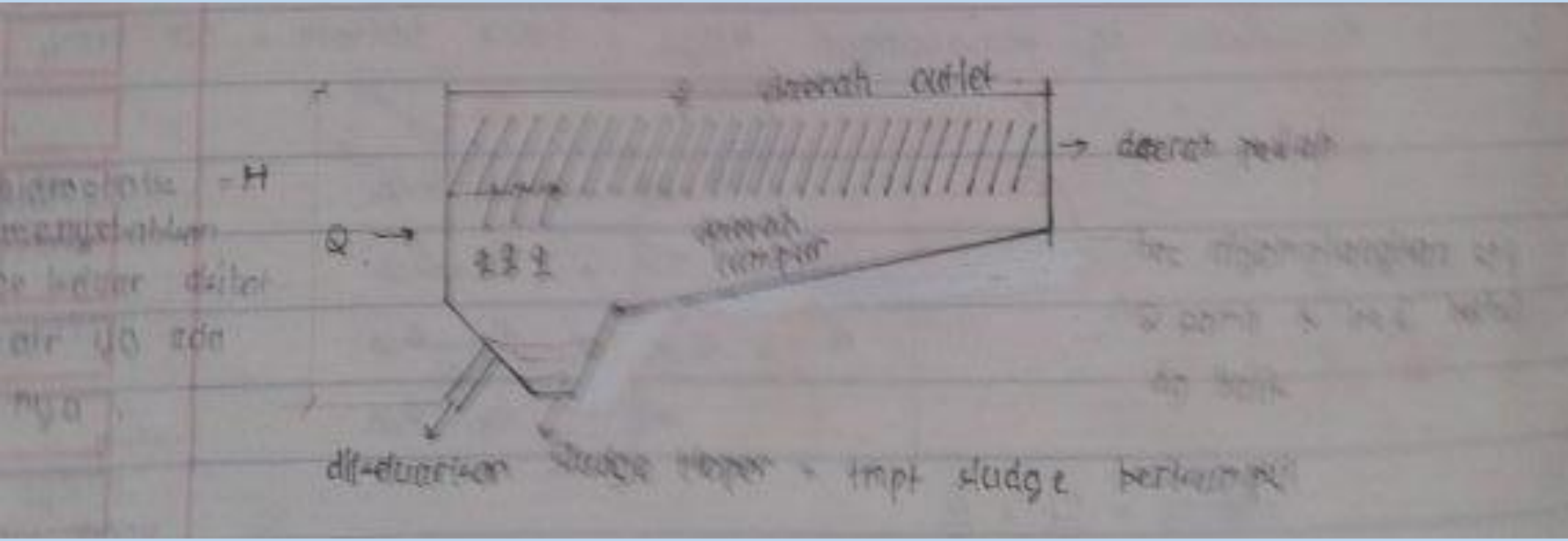
Efisiensi = kemampuan bak pengendap utk memisahkan partikel (solid)

$\eta=((Co-Ce)/Co) \times 100\%$

Multi trays (plates) settler

Utk menghindarkan penumpukan partikel pd tray (yg dpt mengakibatkan kons eff meningkat krn adanya pertikel yg terbawa oleh aliran air), maka posisi dari tray dijomplang/miringkan

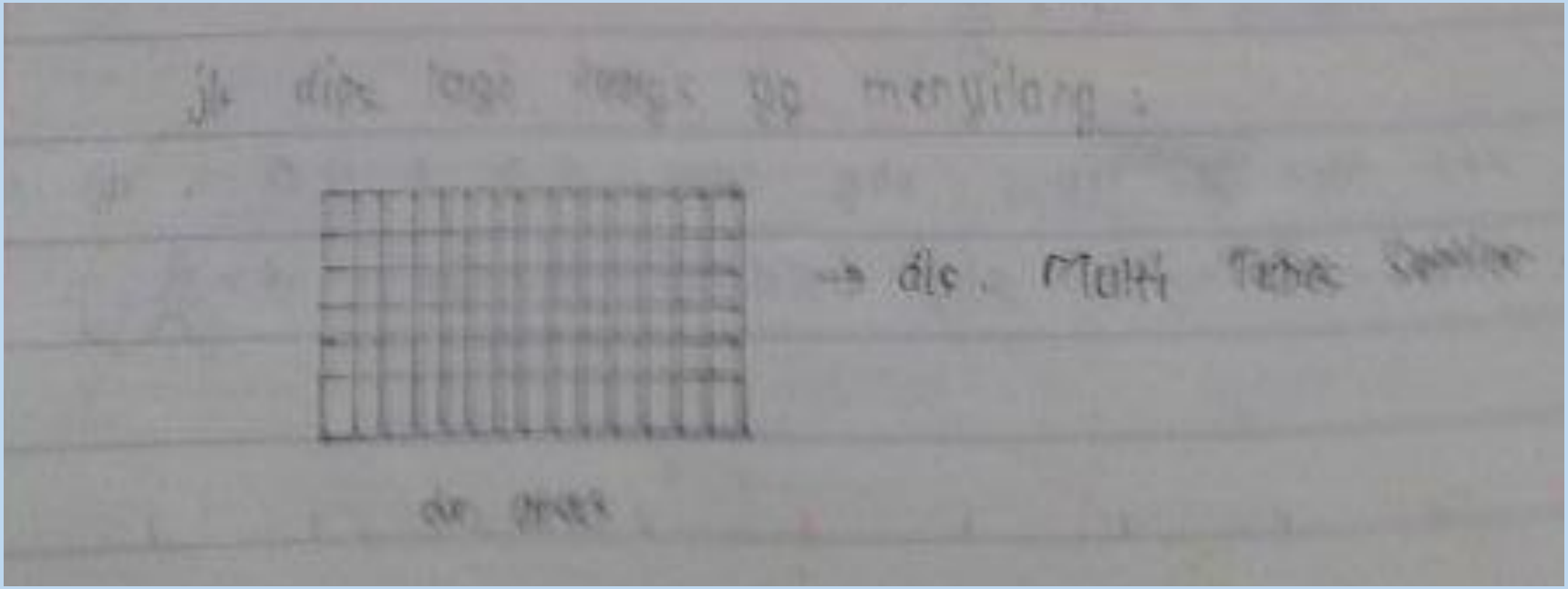
Gbr



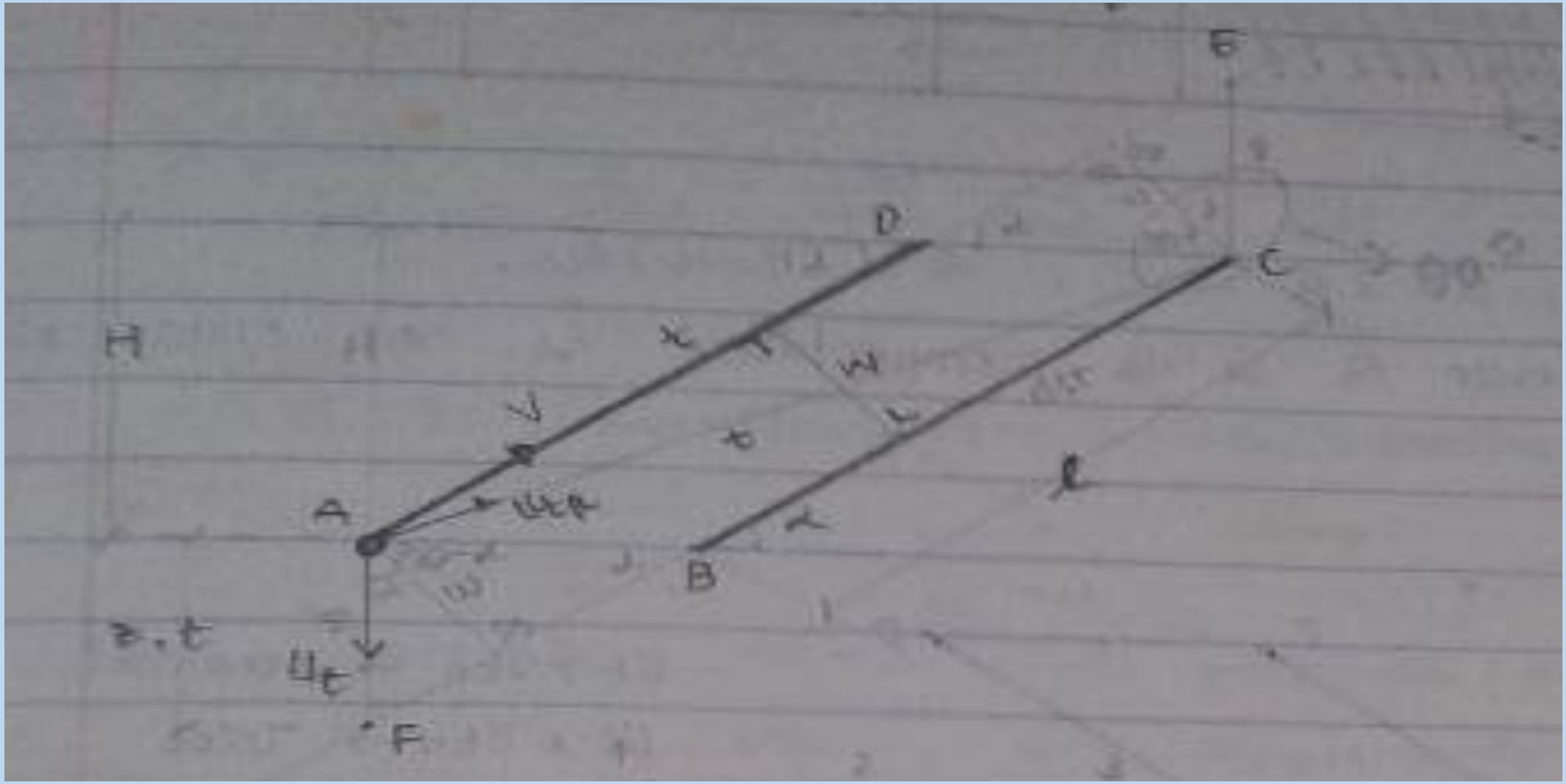
Multi Tubes settler

Jk dipasang lagi trays yg menyilang

Dis Multi Tubes settler



Tinjauan per segmen tray



Tinjauan per segmen tray

Partikel yg terletak pd kemungkinan posisi yg terjelek yi A, dpt bergerak ke:

1. C, jk ada aliran dg kec Ut_R
2. F, jk tdk mengalir, dg kec Ut (akan turun sd dasar)
3. E, jk tdk memp berat atau ringan dg kec V

Pd wkt t yg sama

Tinjauan per segmen tray

- $Q = \frac{z}{t} A = Vs.\ As$
- As = luas bak pengendap
- Q tgt dr Vs & A,
- Q tdk tgt dr z

Tinjauan per segmen tray

- $Q = \frac{z}{t} A = Vs.\ As$
- As = luas bak pengendap
- Q tgt dr Vs & A,
- Q tdk tgt dr z

- $Q = \frac{z}{t} A = Vs.\ As$
- As = luas bak pengendap
- Q tgt dr Vs & A,
- Q tdk tgt dr z

Perbedaan clay Ss dan flok:

- Clay Ss :
  - ukuran cukup besar sampai dengan koloidal
  - Untuk prasedimentasi - ukurannya berbeda-beda dan perbedaannya cukup besar.

- Flok : ukuran homogen karena sudah dikondisikan.
  - Clarifier : dibuat dalam bentuk bulat

Clarifier

- Removal lebih kecil daripada 100% tetapi lebih baik daripada plain sedimentasi.
- Jarak sama karena flok homogen.
- Surface loading akan berlaku betul

Prasedimentasi (PS)

- atau plain sedimentation
- baknya dibuat persegi panjang karena flok ukurannya tidak homogen tetapi berbeda-beda dan perbedaannya cukup besar.

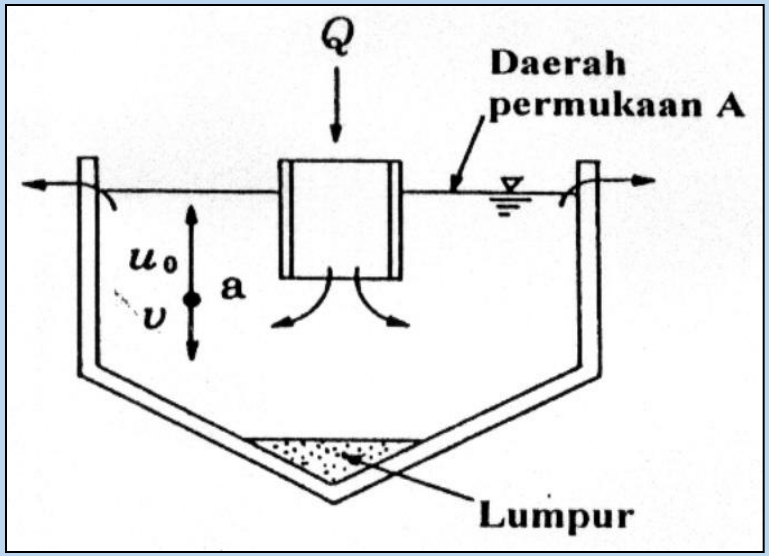
SEDIMENTASI

Tujuan : Untuk menyisihkan padatan tersuspensi melalui pengendapan

Prinsip pengendapan secara gravitasi

$P_s > 1$

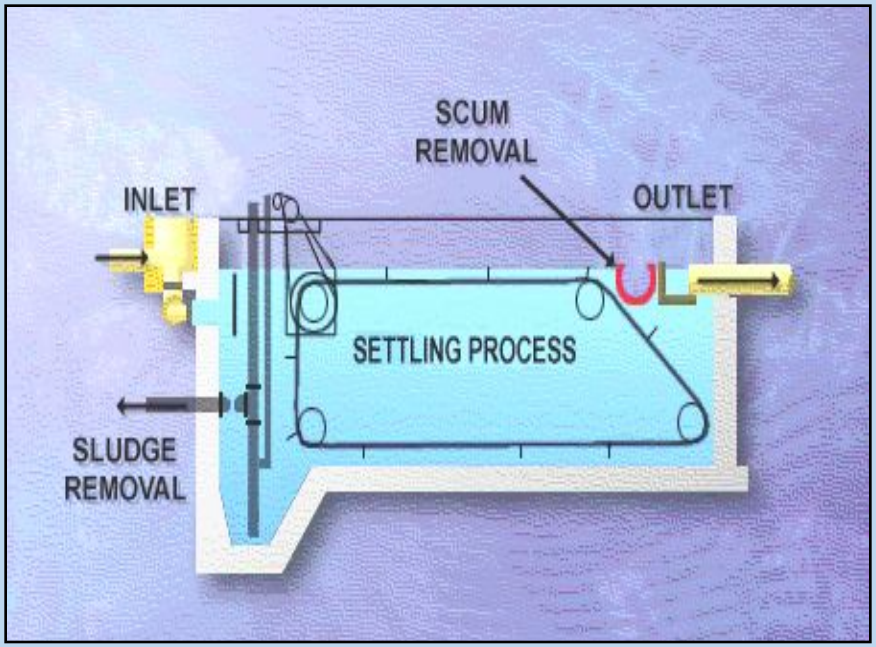
$V > V_o$ [perbedaan arah aliran]



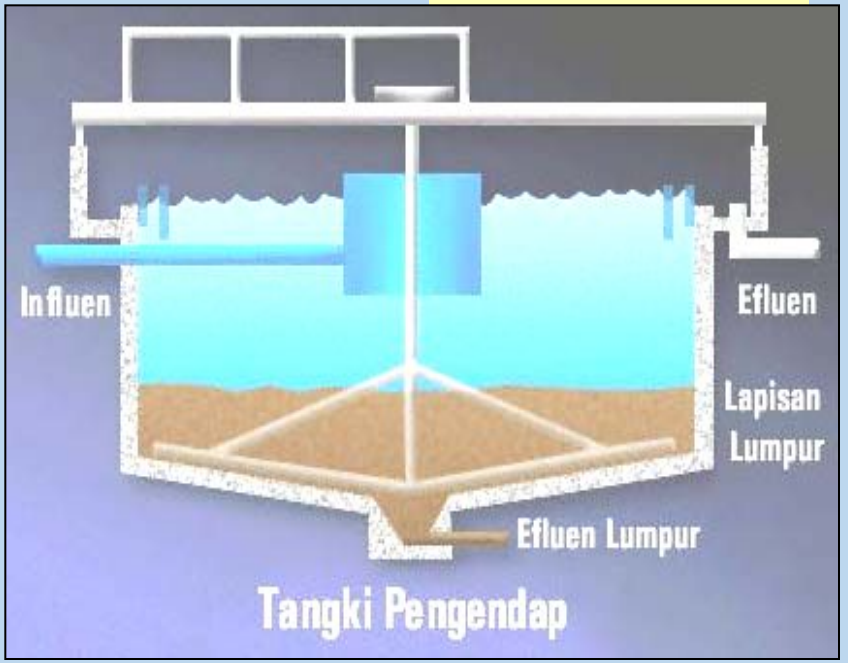
Gambar Pengaruh arus terhadap sedimentasi

TIPE TANGKI SEDIMENTASI

1. Persegi



2.Lingkaran



Parameter-parameter operasi

1. Efisiensi [%]
2. Beban permukaan [So] □ rumus:
3. Waktu tinggal [td]

SEDIMENTATION (Reynolds, Tom D., 1982)

- It is a solid-liquid separation utilizing gravitational settling to remove suspended solids.
- Commonly used in water treatment, wastewater treatment and advanced wastewater treatment.
- In water treatment its applications are:
  - Plain settling of surface water priors to treatment by a RSF plant
  - Settling of coagulated and flocculated waters prior to RSF.
  - Settling of coagulated and flocculated waters in a lime soda softening plant
  - Settling of treated waters in an iron or manganese removal plant

- In wastewater treatment:
  - Grit or sand and silt removal
  - Suspended solids removal in primary clarifiers
- It is one of the earliest unit operations used in water and wastewater treatment.
- The principles are the same for basins used in water and wastewater treatment;
- The equipment and operational methods are also similar.

- Basins are usually constructed of reinforced concrete.
- Maybe circular, square or rectangular in plan view.
- Circular tanks:
  - d = 15-300 ft
  - h = 6-16 ft
- Commonly:
  - D = 35-150 ft
  - H = 10-14 ft
- Standard size tanks have d with 5 ft intervals
  accommodate commercially built sludge rake mechanism.

- Square tanks:
  - w= 35-200 ft
  - h = 6-19 ft
- Freeboard for circular or square tanks: 1-2.5 ft
- Standard size tanks have w with 5 ft intervals.
- Rectangular tanks:
- W= 5-20 ft, 80-100 ft,10-120 ft
- L = up to 250 ft, 40-300 ft
- H > 6 ft

Classification of the type of settling

- 4 general type of settling
- Based on:
  - The concentration of the particles
  - The ability of the particles to interact.

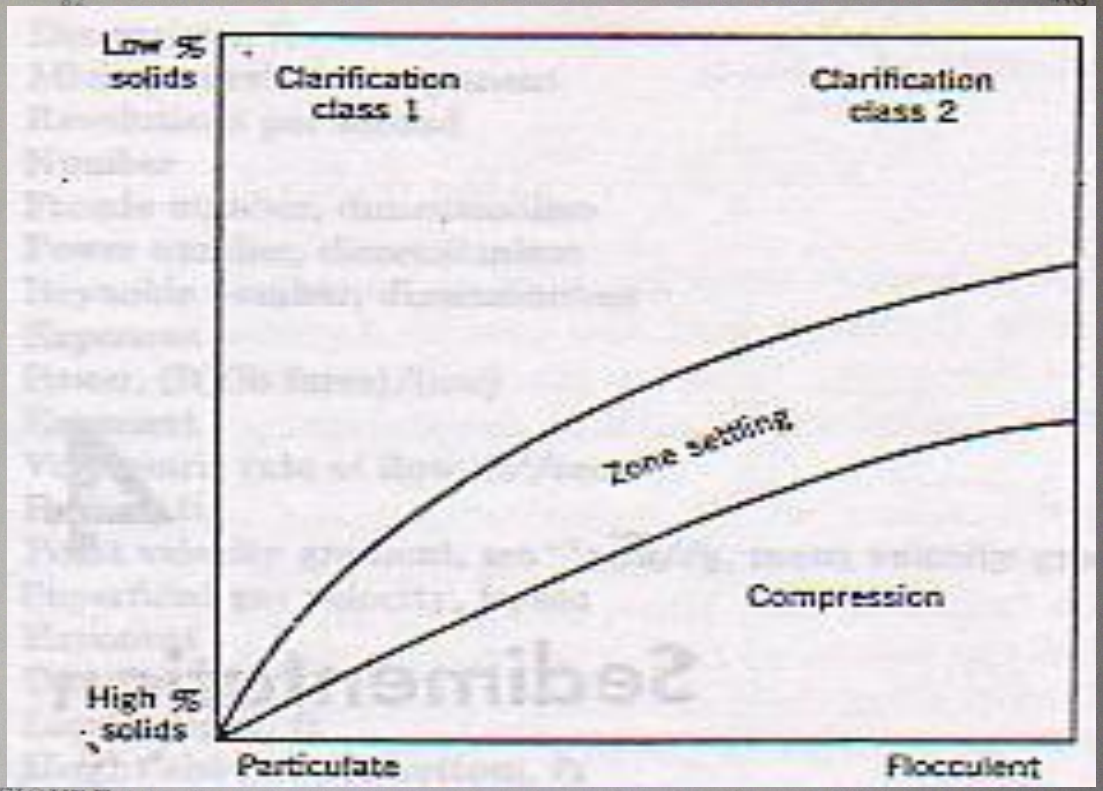


FIGURE 4-1. Paragenesis diagram. [From Figure 2-1, Fitch, E. B., in McCabe, J., and W. W. Eckenfelder, Jr. (eds.), *Biological Treatment of Sewage and Industrial Wastes*—Vol. 2: *Anaerobic Digestion and Solids-Liquid Separation*, Reinhold Publishing Corp., New York (1958), 169.]

Type I settling

- Or free settling
- It is the settling of discrete, nonflocculent particles in a dilute suspension
- The particles settle as separate units
- There is no apparent flocculation or interaction between the particles
- Example:
- The plain sedimentation of surface waters
- The settling of sand particles in grit chambers

- In type I settling, a particle will accelerate until the drag force, F_D , equals the impelling force, F_I ; then
- Settling occurs at a constant velocity, V_s
- The impelling force, F_I :
- $F_I = (\rho_s - \rho)\, g\, V$ ….(3-1)
- Where:
  - F_I = impelling force
  - ρ_s = mass density of the particle
  - ρ = mass density of liquid
  - V = volume of particle
  - g = acceleration due to gravity

- The drag force is given by Newton's law:
- $F_D = C_D A_C \rho (V_s^2/2)$ .....(3-2)
- Where:
  - F_D = drag force
  - C_D= coefficient of drag, which is a function of the Reynolds number, N_{Re}
  - A_C = area in cross section at right angle to the velocity
  - ρ = mass density of liquid
  - V_s = settling velocity

- Combining Eq (3-1) and (3-2) gives:
- $(\rho_s - \rho)\, g\, V = C_D\, A_C\, \rho\, (V_s^2/2)$
- Or (3-3):

$$V_s = \sqrt{\frac{2g}{C_D}\left(\frac{\rho_s - \rho}{\rho}\right)}\frac{V}{A_c}$$

- For spheres of diameter, d, the volume, V is:
- $V = (\pi/6)\, d^3$
- The cross section area, A_c is:
- $A_c = (\pi/4)\, d^2$

- From the preceding two equations it follows that for spheres
- $V/A_c = (\pi/6)\, d^3\, (4/\pi)\, 1/d^2 = 2/3\, d$
- Substituting the above expression for V/A_c into Eq (3-3): (3-4a)

$$V_s = \sqrt{\frac{4g}{3C_D}\left(\frac{\rho_s - \rho}{\rho}\right)d}$$

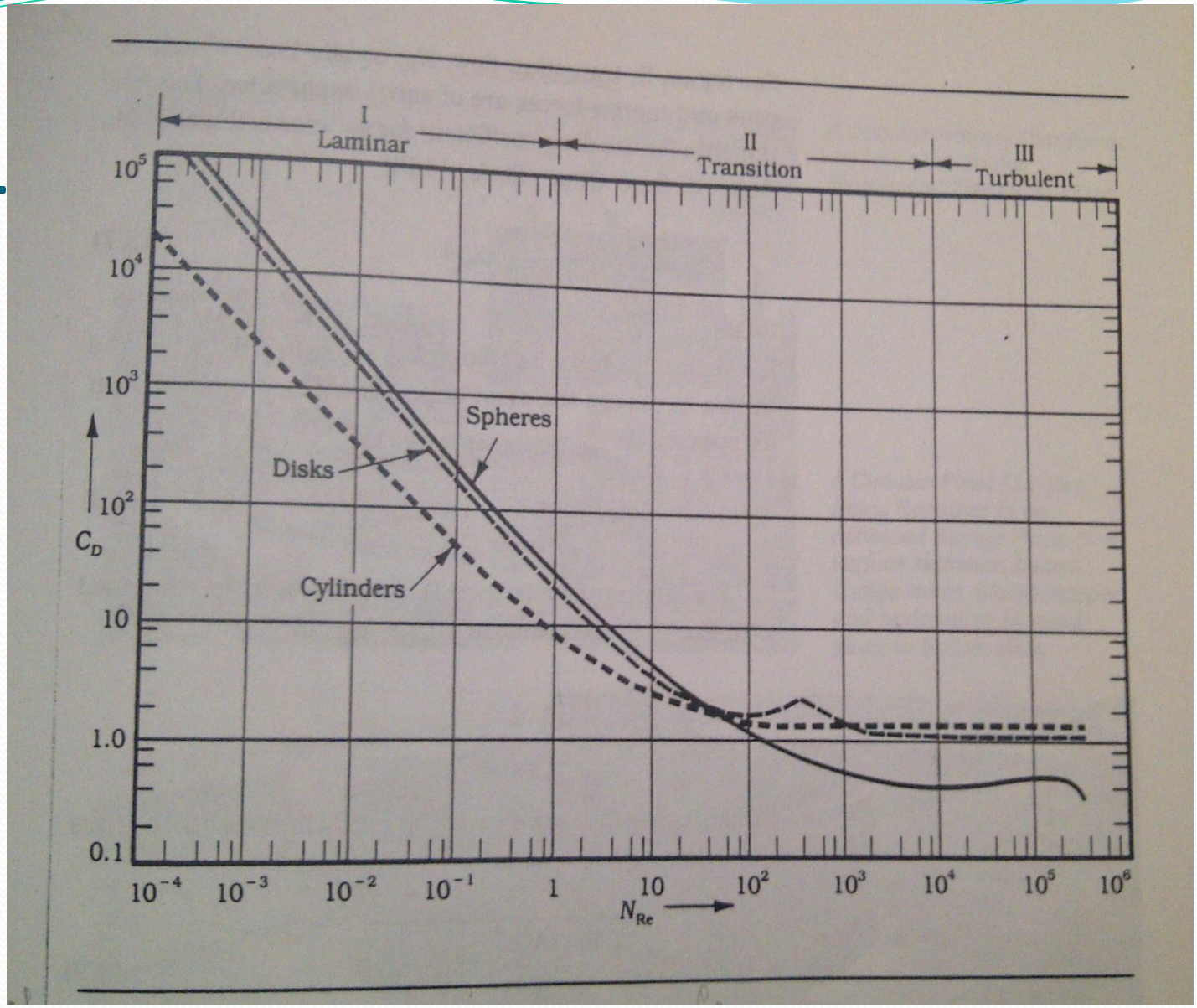
- Or (3-4b)

$$V_s = \sqrt{\frac{4g}{3C_D}}(S_s - 1)d$$

- Where: S_S = specific gravity of the particles

- The numerical value of the drag force depends on whether the flow regime around the particle is laminar or turbulent.
- Fig 3-1 shows the drag coefficient for various shapes as a function of the Reynolds number.
- the Reynolds number for the spheres:
- $N_{Re} = V_s d/\nu = V_s d\rho/\mu$ .....(3-5)
- Where:
  - ν = kinematic viscocity
  - μ = dynamic viscocity
  - ρ = mass density

Fig3-1


I
Laminar
II
Transition
III
Turbulent
10^5
10^4
10^3
10^2
10
1.0
0.1
C_D
Spheres
Disks
Cylinders
10^{-4}
10^{-3}
10^{-2}
10^{-1}
1
10
10^2
10^3
10^4
10^5
10^6
N_{Re}

- In Figure 3-1 - 3 distinct regions:
  - I= laminar flow
  - II = transition flow
  - III = turbulent flow
- I = laminar flow:
  - $N_{Re} < 1$
  - The viscous forces are more important than inertia forces
- The relationship for the drag coefficient for spheres:
- $C_D = 24/N_{re}$ ….(3.6)

- II = transition flow:
  - N_{Re} = 1-10^4
  - The viscous and inertia forces are of equal importance
- The relationship for the drag coefficient for spheres (3-7)

$$C_D = \frac{24}{N_{\mathrm{Re}}} + \frac{3}{\sqrt{N_{\mathrm{Re}}}} + 0.34$$

- III = turbulent flow:
  - $N_{Re} > 10^4$
  - The inertia forces are the most important
- The drag coefficient for spheres (3-8):
- $C_D = 0.4$
- For laminar flow (region I), Eq (3-5) maybe combined with Eq (3-6) to eliminate N_{Re} giving:
- $C_D = 24\nu/V_s d$

- Substituting the above equation for C_D into Eq (3-4b) gives the following equation (3-9):

$$V_s = \sqrt{\frac{4g}{3}\left(\frac{V_s d}{24\upsilon}\right)}(S_s - 1)d$$

- Squaring Eq (3-9) and rearranging gives Stokes' law:
- $V_s = g/18\nu\ (S_s - 1)\ d^2$ .....(3-10)
- Or since $\nu = \mu/\rho$, substitution into Eq (3-10) yields:
- $V_s = g/18\ \mu\ (\rho_s - \rho)\ d^2$ .....(3-11)
- Which is another form of Stokes' law
- Much of the dilute suspensions in water and wastewater treatment follows Stokes' law

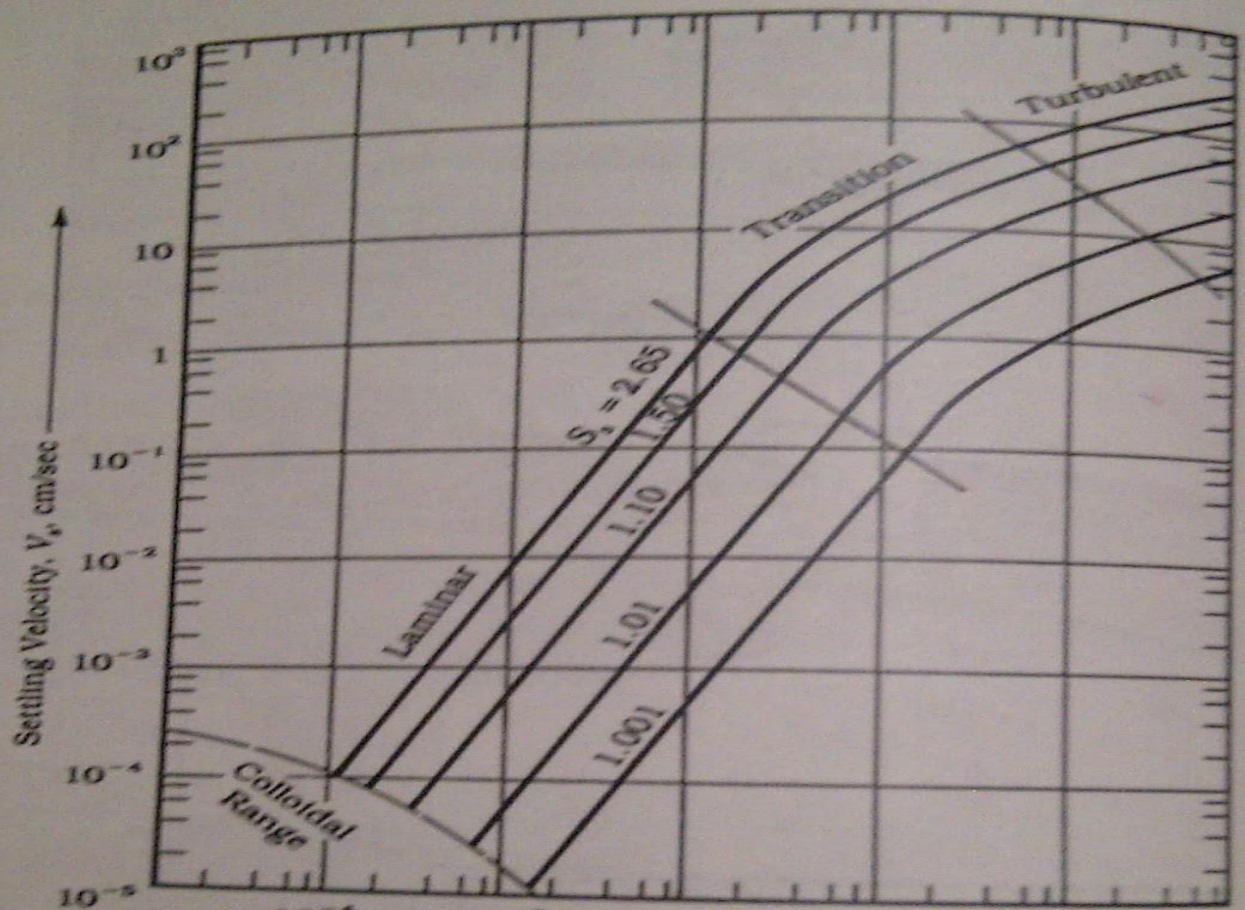
- For transistion flow (region II),the determination of the settling velocity is a trial and error solution using Eq (3-4a) or (3-4b) and eq (3-7)
- For turbulent flow (region III), substituting eq (3-8) into eq (3-4b) gives (3-12):

$$V_s = \sqrt{3.3g(S_s - 1)d}$$

- The settling velocity of sand in grit removal chambers can be determined using Eq (3-12).

- The solution for the settling velocity in regions I, II and III can also be done using the graph in Fig 3.2.
- The graph gives a direct solution for the settling velocity V_s, if the diameter, specific gravity and temperature are known.

Figure 3.2. Type I Settling of Spheres in Water at 10°C
Adapted from "Water Treatment" by T. R. Camp in *Handbook of Applied Hydraulics*, 2nd ed., ed. C. V. Davis. Copyright © 1952 by McGraw-Hill Book Co., Inc. Reprinted by permission.



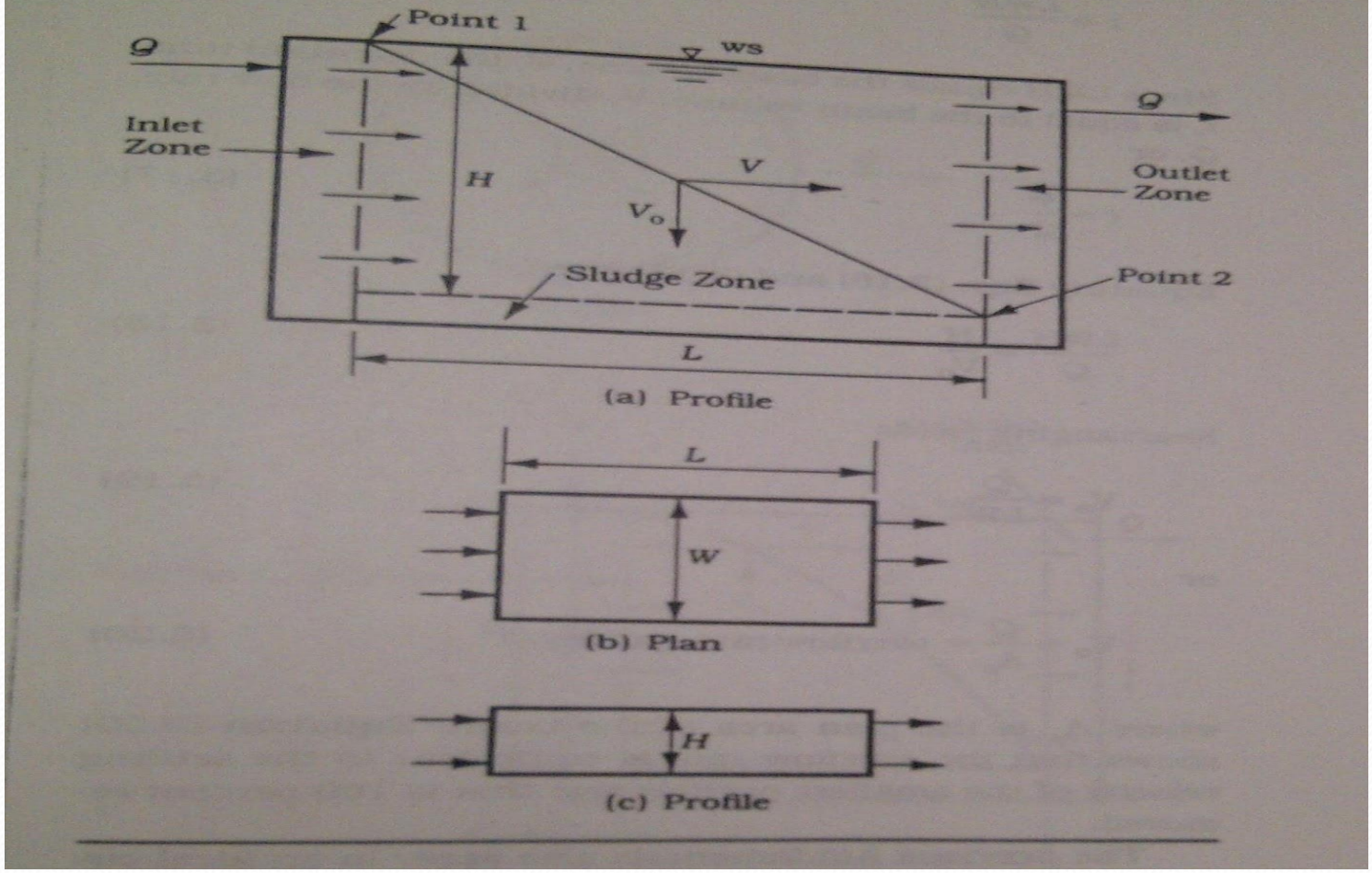
Ideal basin

- The ideal basin theory by Camp (1946) assumes:
  - The settling is type I settling (or discrete particles)
  - There is an even distribution of the flow entering the basin
  - There is an even distribution of the flow leaving the basin
  - There are 3 zones in the basin: (1) the entrance zone, (2) the outlet zone, (3) the sludge zone
  - There is a uniform distribution of particles throughout the depth of the entrance zone
  - Particle that enter the sludge zone remain there, and particles that enter the outlet zone are removed

- Fig 3-3 shows an ideal rectangular settling basin of a length, L, a width, W and a depth H.
- V_o is the settling velocity of the smallest particle size that is 100% removed
- When a particle of this size enters the basin at the water surface, point 1,it has a trajectory as shown and intercepts the sludge zone at point 2, which is at the down-stream end.
- The detention time, t is equal to the depth H divided by the settling velocity V_o or
- $t = H/V_o$ ….(3.13)

- The detention time t is also equal to the length L divided by the horizontal velocity V or
- t= L/V ….(3-14)
- The horizontal velocity V is equal to the flow rate Q divided by the cross-sectional area HW or
- V = Q/HW ….(3-15)
- Combining Eq (3-14) and (3-15) to eliminate V gives:
- t = LWH/Q ….. (3-16)

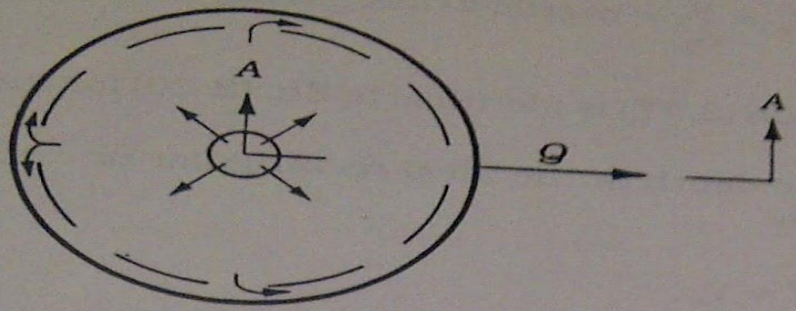
Figure 3.3. Ideal Rectangular Settling Basin



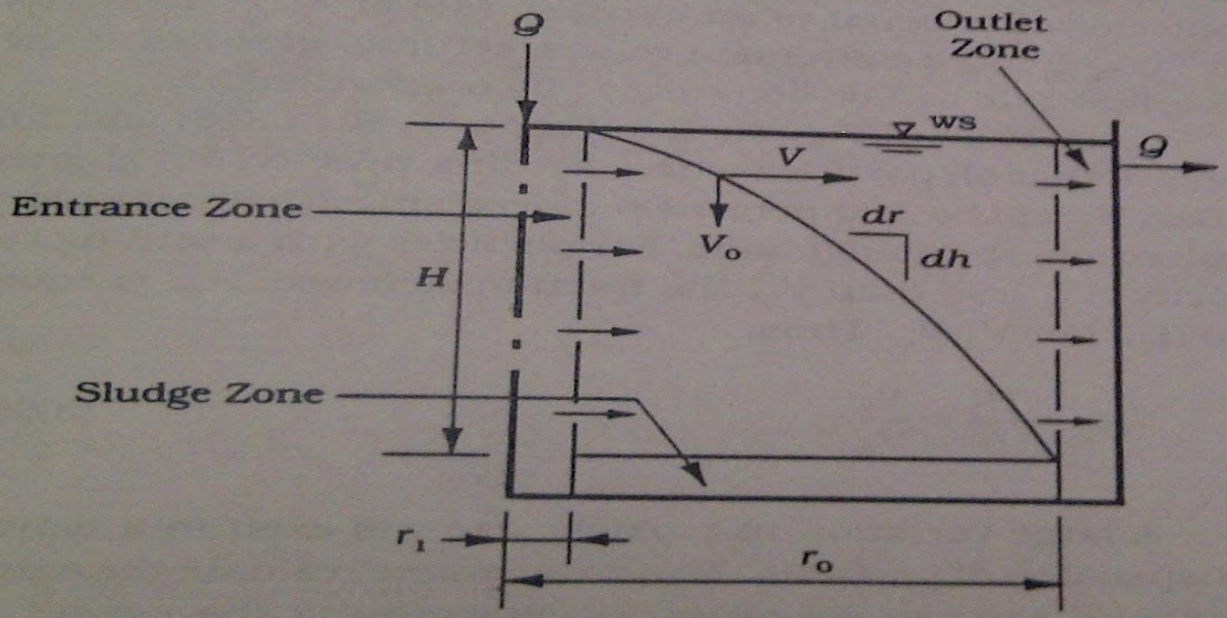
- Since LWH equals the basin volume V, the detention time t is equal to the basin volume V divided by the flow rate Q or $t = V/Q$ …… (3-17)
- Equating Eq (3-13) and (3-16):
- $LWH/Q = H/V_o$ ….. (3-18)
- Rearranging gives:
- $V_o = Q/LW$ ….. (3-19)
- Or
- $V_o = Q/A_p$ = overflow rate, gal/day-ft² ….. (3-20)
- Where A_p is the plan area of the basin
- Eq (3-20) shows that the flow rate is equivalent to the settling velocity of the smallest particle size that is 100% removed.

- The previous fundamentals also apply to an ideal circular settling basin (Fig 3-4).
- The horizontal velocity is given by
- $V = Q/2\pi rH$ ....(3-21)
- From Fig 3-4:
- $dH/dr = V_o/V$ ..... (3-22)
- Substituting Eq (3-21) into (3-22):
- $dH/dr = 2\pi rHV_o/Q$ ....(3-23)

Figure 3.4. Ideal Circular Settling Basin



(a) Plan



(b) Half Section A-A

- Rearranging Eq (3-23) and setting the integration limits (3-24):

$$\int_0^H dh = \frac{2\pi H V_0}{Q} \int_{r_1}^{r_0} r\,dr$$

- Integrating gives (3.25):

$$H = \frac{2\pi H V_0}{Q} \left[\frac{r^2}{2} \right]_{r_1}^{r_0}$$

- Or (3-26):

$$H = \frac{\pi H V_0}{Q} \left(r_0^2 - r_1^2 \right) = \frac{H A_p V_0}{Q}$$

- Where: A_p =plan area of the basin

- Cancelling the H terms in Eq (3-26) and rearranging yields:
- $V_o = Q/A_p$ = overflow rate, gal/day-ft^2 ……(3-27)
- Eq (3-27) is identical Eq (3-20) for the rectangular basin.
- The depth of the ideal rectangular or circular basin is given by:
- $H = V_o t$ ……(3-28)
- Where V_o = overflow rate expressed as velocity.

- It can be shown that:
  - An overflow rate of 100 gal/day-ft^2 = a settling velocity of 0.555 ft/hr.
  - a settling velocity of 1 cm/sec = an overflow rate of 21,200 gal/day-ft^2.
- Using these coversion values, settling velocities at any overflow rate can be determined proportionally.

- Inspection of Fig. 3.5 and 3.6:
- All particles with a settling velocity, $V_1 > V_0$ □ 100% removed since their trajectory intercepts the sludge zone.
- For particles with a settling velocity, $V_2 < V_0$, the fraction removed R_2 :
- $R_2 = H_2/H = V_2/V_0$ .....(3-29)

Figure 3.5. Profile through an Ideal Rectangular Basin

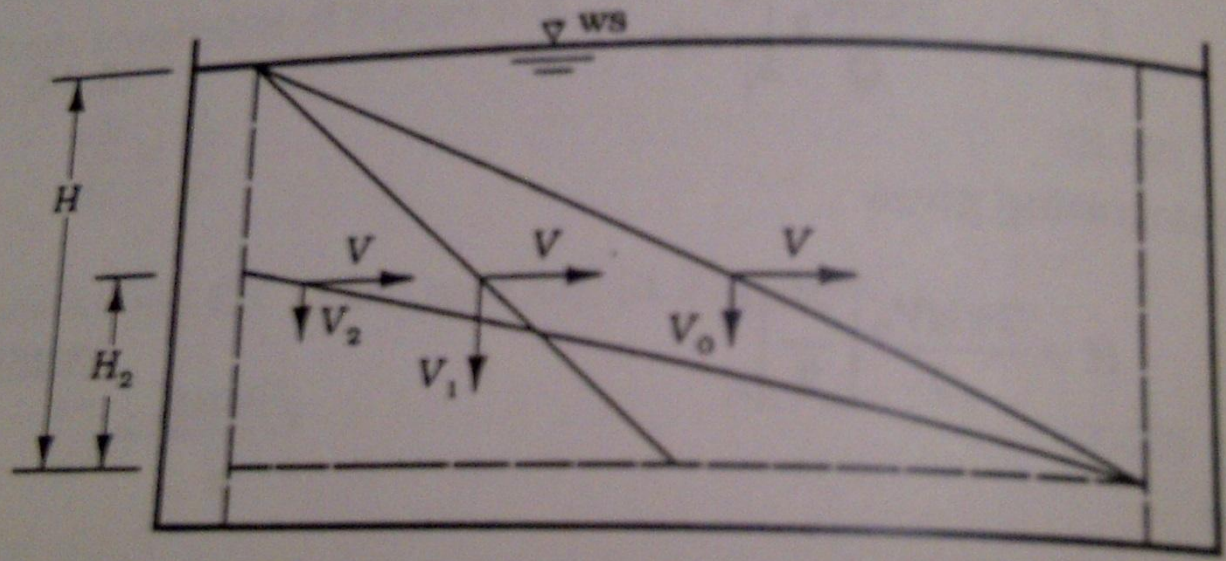
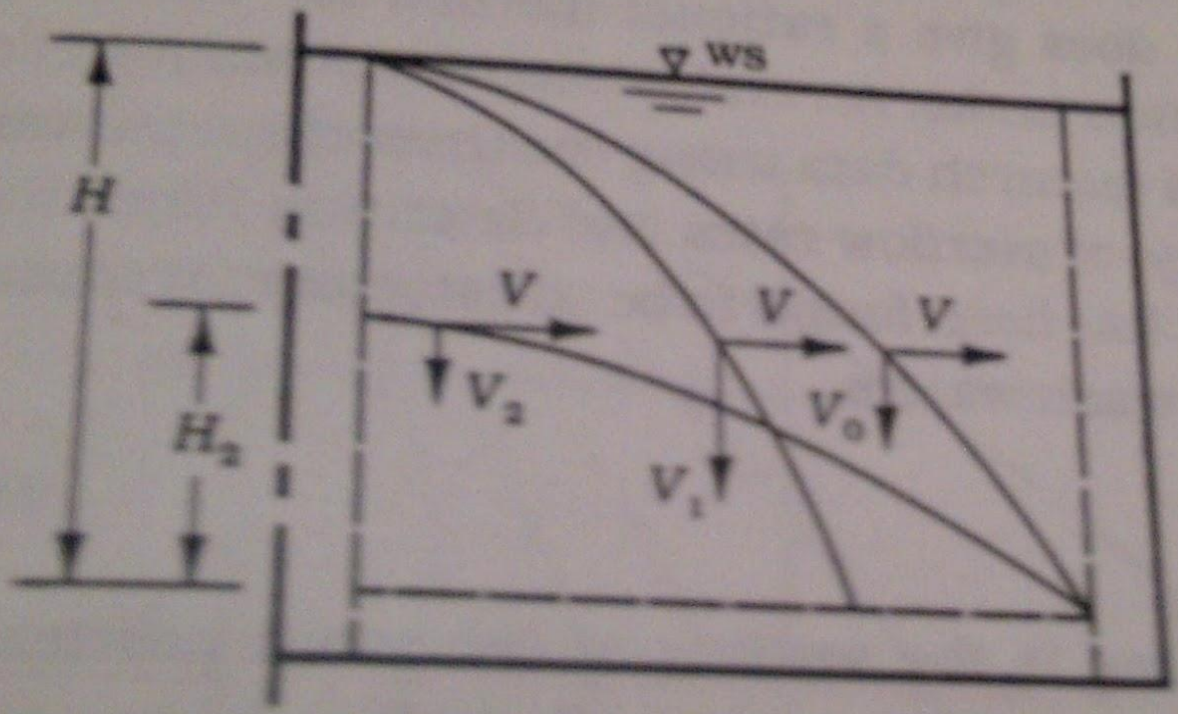


Figure 3.6. **Half Section through an Ideal Circular Basin**



- A large variation in particle size will exist in a typical suspension of particles.
- Thus, the entire range of settling velocities must be evaluated in determining the overall removal for a given design settling velocity or overflow rate.
- This requires experimental analysis the use of a settling coloumn.
- In a batch settling coloumn, samples are withdrawn at various times & depths, and the solids concentrations are determined.

- An analysis of the data by an appropriate procedure wil yield a settling velocity curve Fig. 3.7
- The fraction of the total particles removed for a design velocity V_o will be (3.30):
- Fraction removed =

$$(1 - F_0) + \frac{1}{V_0}\int_0^{F_0} V dF$$

- Where:
- $1\text{-}F_o$ = fraction of particles with velocity $V > V_o$

$$\frac{1}{V_0}\int_0^{F_0} V dF$$

- = fraction of particles with velocity $V < V_o$

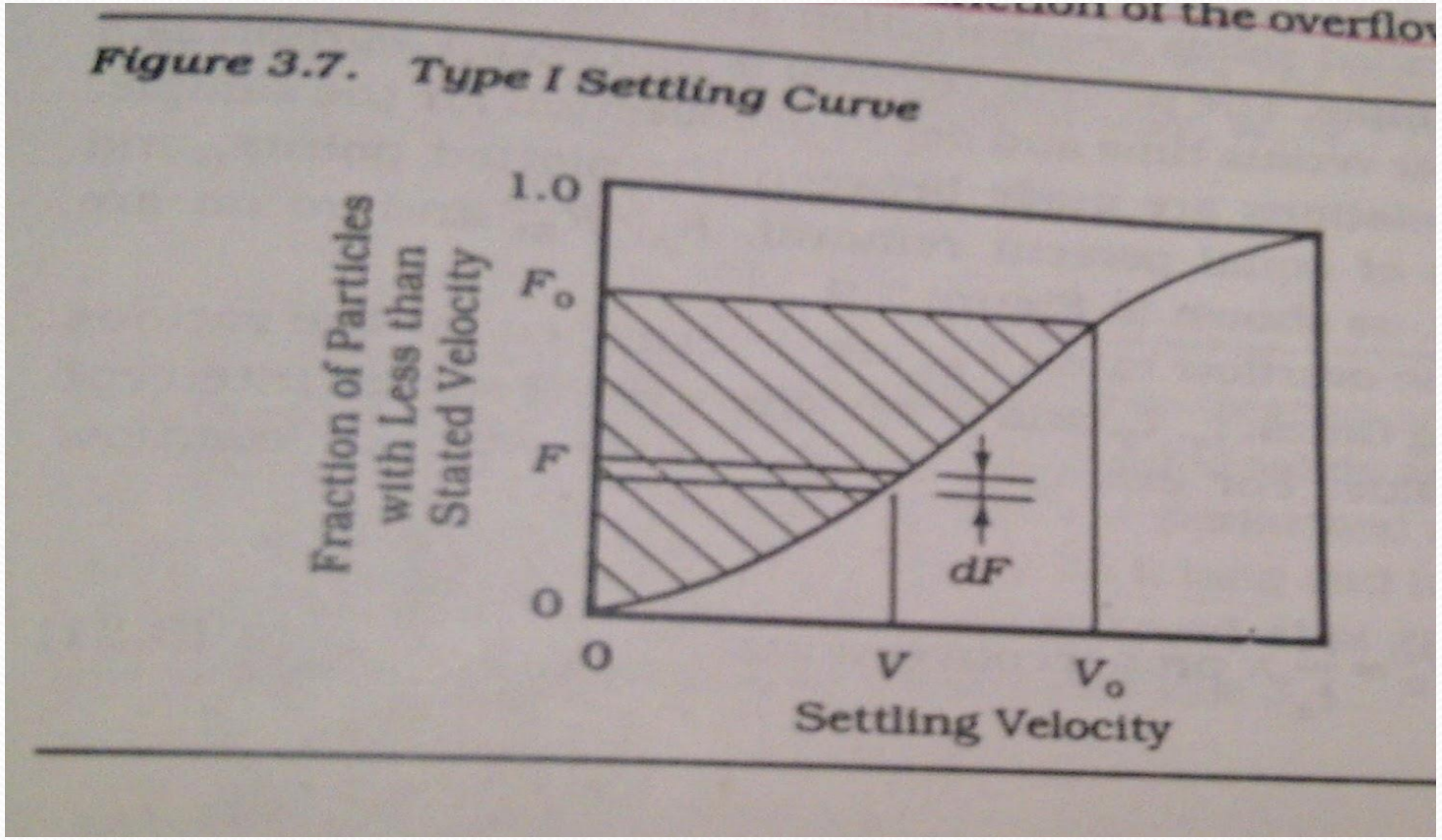


Figure 3.7. Type I Settling Curve
1.0
F_o
F
0
0
V
V_o
dF
Fraction of Particles with Less than Stated Velocity
Settling Velocity

Summary of the ideal settling basin theory

- the removal of suspended solids is a function of:
  - The overflow rate or design settling velocity, V_o
  - The detention time, t
  - The depth, H

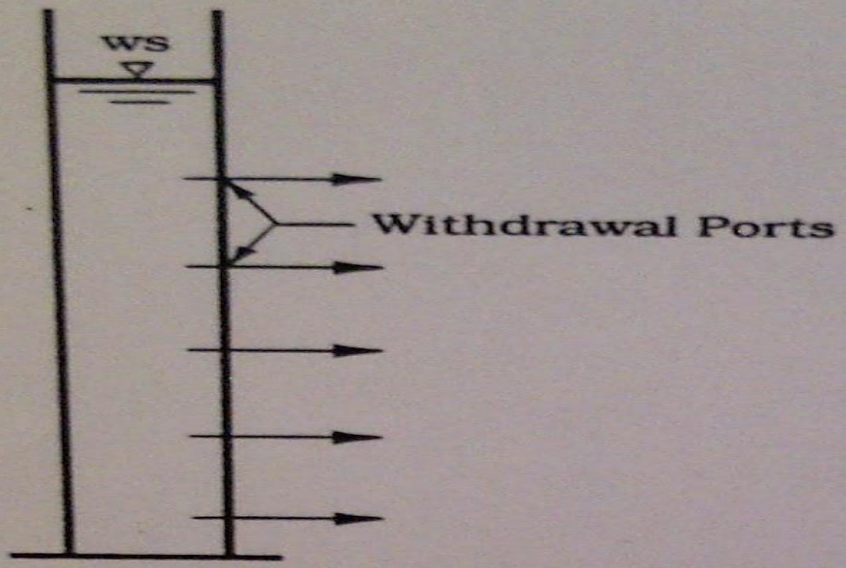
- Although the ideal settling basin analysis is theoretical, it gives a rational method for the design of the sedimentation tanks.
- An evaluation of vast amounts of operating & research data using the theoretical parameters gives a range of overflow rates & detention times that, in most cases, has been found to be satisfactory for municipal waters and wastewaters.

Type II settling

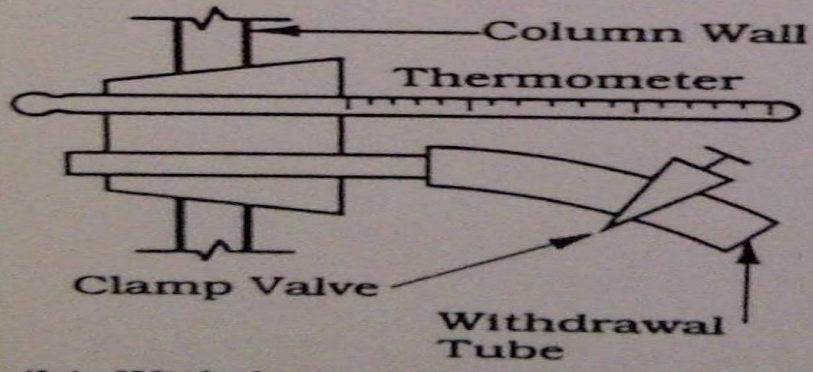
- Is the settling of flocculent particles in a dilute suspension.
- The particles flocculate during settling
  - they increase in size
  - And settle at a faster velocity
- Examples:
  - The primary settling of wastewaters
  - The settling of chemically coagulated waters and wastewaters

- To evaluate the characteristics of a flocculent suspension - batch settling tests
- Fig 3.8: a schematic drawing of a batch settling column:
  - The coloumn diameter: min = 5-8 inches to minimize side-wall effects
  - The height: min = the depth of the proposed settling tank
  - Sampling ports: at equal intervals in height

Figure 3.8. Batch Settling Column Details for Type II Settling



(a) Column Elevation



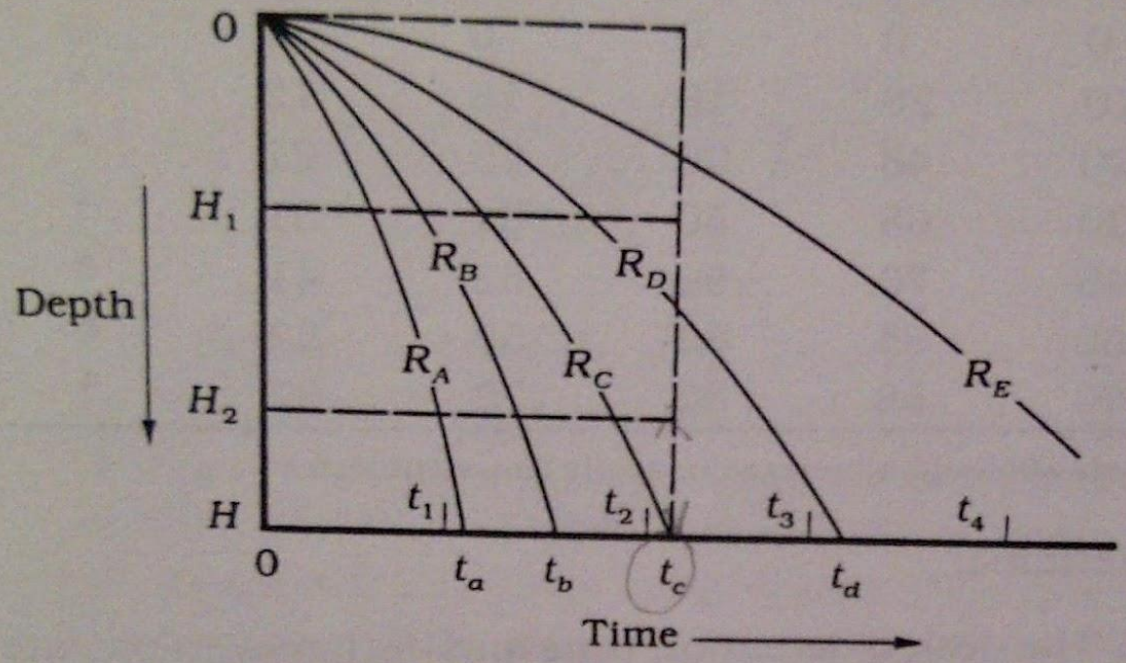
(b) Withdrawal Port Detail

- The suspension must be mixed thoroughly
- And poured rapidly into the coloumn
- □ a uniform distribution of the particles occurs throughout the height of the coloumn.
- To be representative:
- The test must be under quiescent conditions
- The temperature should not vary > 1°C throughout the coloumn height □to avoid convection currents

- Samples are removed through the ports at periodic time intervals
- And the suspended solids concentration are determined
- The % removal is calculated for each sample knowing the initial SS concentration and the concentration of the sample
- The % removal is plotted on a graph: a number vs time and depth of collection for the sample
- Interpolations are made between the plotted points
- Curves of equal % removal, R_A, R_B etc are drawn (Fig. 3.9)

- The overflow rates, Vo are determined for the various settling times t_a,t_b etc where the R curves intercept the x-axis.
- Ex:
- For the curve R_C the overflow rate:
- Vo = H/tc x proper conversions……(3-31)
- Where H = height of the coloumn
- t_c = the intercept of the R_C curve and the x axis

Figure 3.9. Settling Diagram for Type II Settling



- The fractions of solids removed, R_T for the times t_a,t_b etc are thus determined.
- Ex:
- For the time t_c, the fraction removed R_T (3-32):

$$R_T = R_C + \frac{H_2}{H}\left(R_D - R_C\right) + \frac{H_1}{H}\left(R_E - R_D\right)$$

- Where:
- H_2 = the height that the particles of (R_D-R_C) size would settle during t_c.
- These would intercept the sludge zone in a basin as in Fig 3.10.

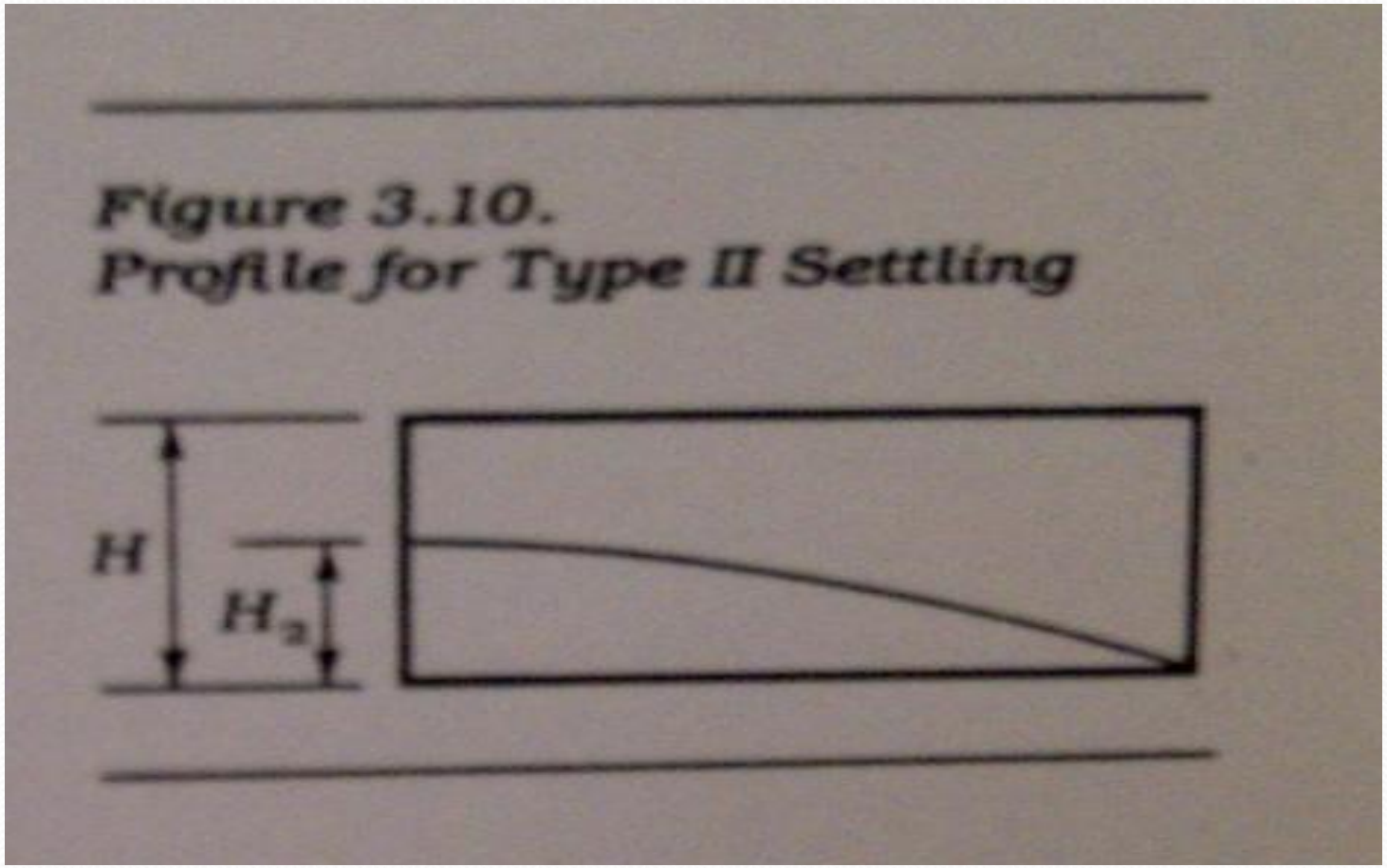


Figure 3.10.
Profile for Type II Settling

- By using the various times t_a, t_b etc
- The various overflow rates V_o
- And the various fractions removed R_T
- A graph of the:
  - overflow rates vs fractions removed
  - And fractions removed vs detention times
- can be made.

Scale up factors

- In applying the curves to design a tank,
- Scale up factors of:
  - 0.65 for the overflow rate
  - And 1.75 for detention time
- Are used to compensate for the side-wall effects of the settling coloumn

Example 3.1

A primary clarifier is to be designed to treat an industrial wastewater having 320 mg/l suspended solids and a flow of 2 mgd. A batch settling test was performed using an 8 inch diameter coloumn that was 10 ft long and had withdrawals ports every 2 ft. The reduced data giving the % removals are shown in Table 3.1

Table 3.1. Percent Suspended Solids Removal at Given Depths

Time (min)	2 ft	4 ft	6 ft	8 ft	10 ft
0	0	0	0	0	0
10	28	18	18	12	[a]
20	48	39	25	27	[a]
30	68	50	34	31	[a]
45	70	56	53	41	[a]
60	85	66	59	53	[a]
90	88	82	73	62	

a. Data showed an increase in solids concentration

Determine:

1. The design detention time and design flow rate if 65% of the SS are to be removed.
2. The design diameter and depth

Figure 3.11. Graph Showing Suspended Solids Removal (as a Percent) at Various Depths and Settling Times, for Example Problem 3.1.

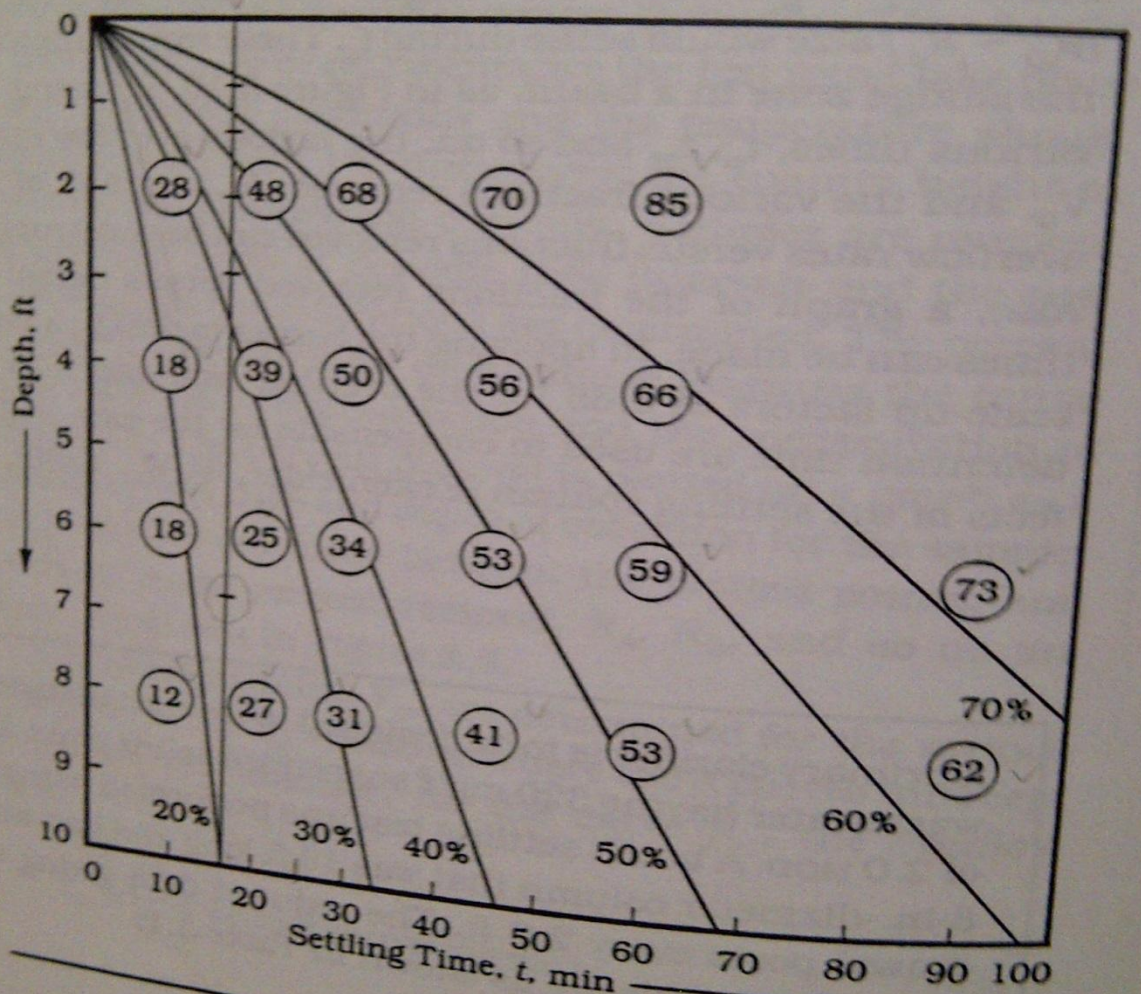


Table 3.2. Reduced Data for 30, 40, 50, and 60 Percent Curves

Time t (hr)	Overflow Rate V_0 (gal/day-ft^2)	Fraction Removed R_T (%)
0.27	6730	33.7
0.55	3260	48.7
0.77	2340	56.7
1.13	1590	63.8
1.60	1120	68.6

Figure 3.12. Suspended Solids Removal versus Detention Time, for Example Problem 3.1

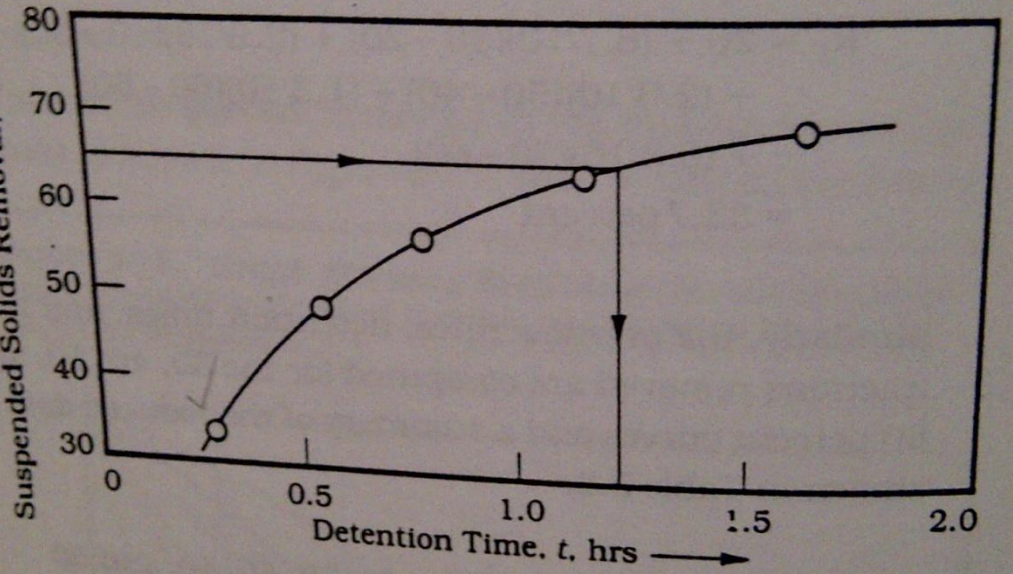
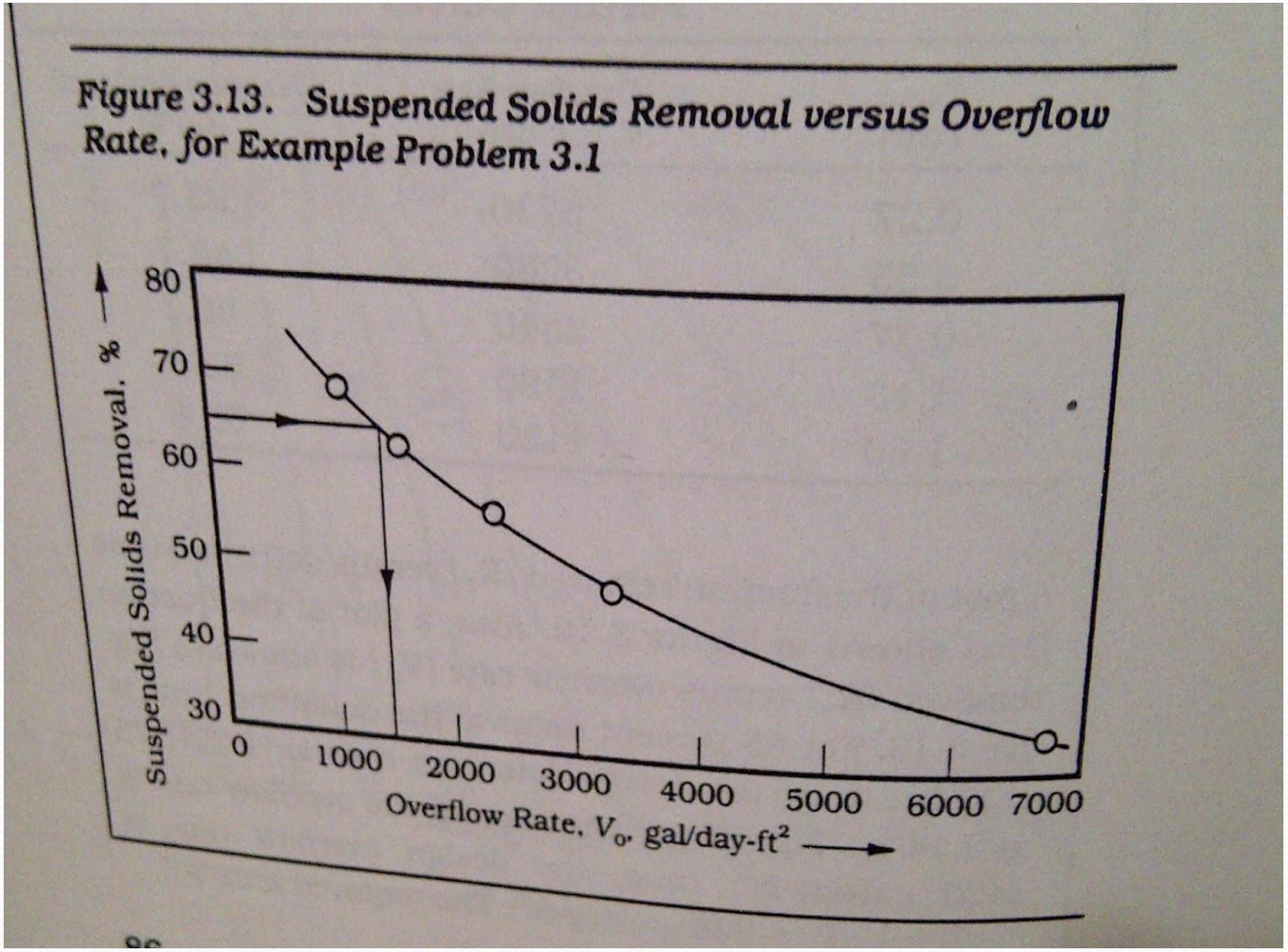


Figure 3.13. Suspended Solids Removal versus Overflow Rate, for Example Problem 3.1



- Gbr dr e-mail

Type III settling

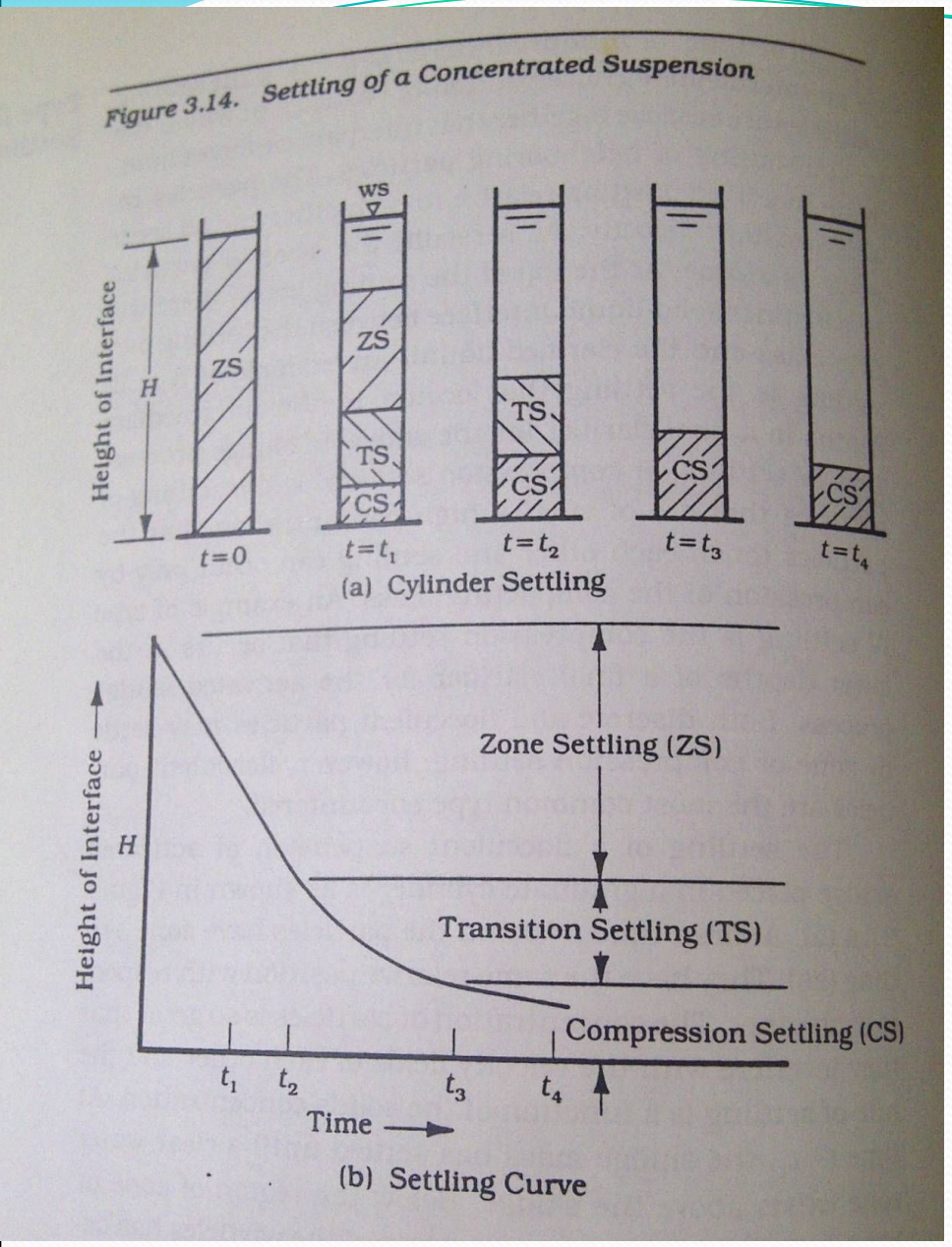
- Or zone or hindered settling
- Is the settling of an intermediate concentration of particles in which the particles are so close together that interparticle forces hinder the settling of neighboring particles.
- The particles remain in a fixed position relative to each other and all settle at a constant velocity.
- As a result, the mass of particles settle as a zone.

- At the top of the settling mass, there will be a distinct solid-liquid interface between the settling particle mass and the clarified liquid.
- Ex:
- The settling that occurs in the intermediate depths in a final clarifier for the activated sludge process.

Type IV settling

- Or compression settling
- Is the settling of particles that are of such high concentration that the particles touch each other and settling can occur only by compression of the compacting mass.
- Ex:
- The compression settling that occurs in the lower depths of a final clarifier for the activated sludge process

- Both discrete and flocculent particles may settle by zone or compression settling;
- But flocculent particles are the most common type encountered.

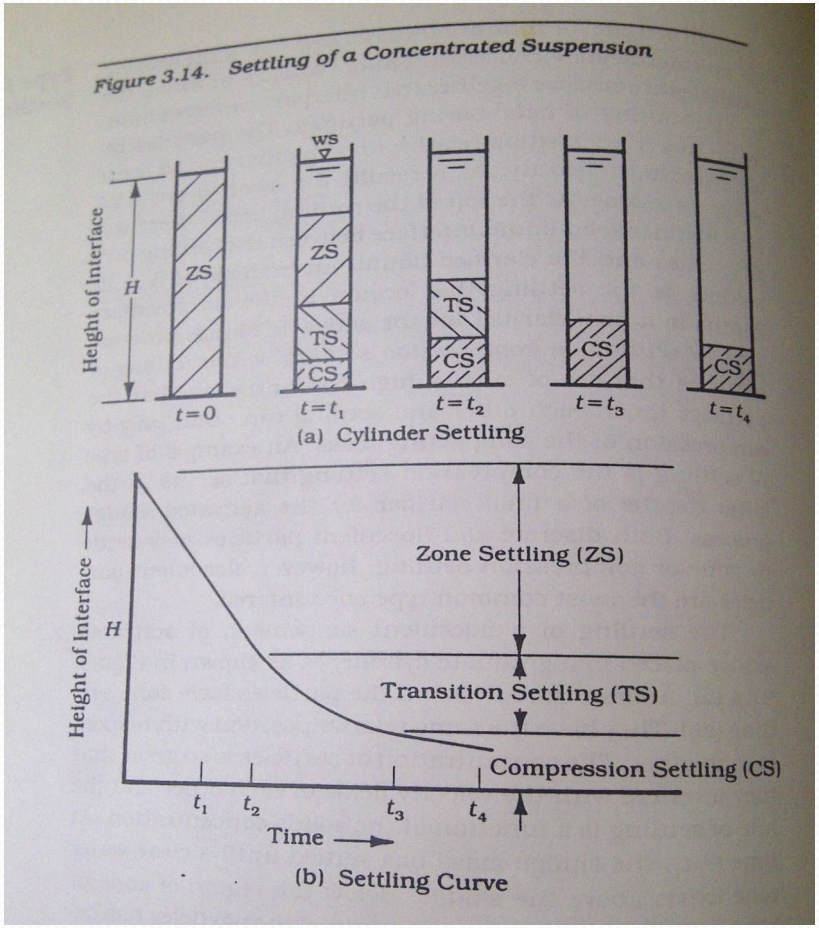


The settling of a flocculent suspension of activated sludge placed in a graduate cylinder Fig. 3.14 (a)

At first, time t = 0 and the particles have zone settling (ZS).

They have the same relative position with respect to each other.

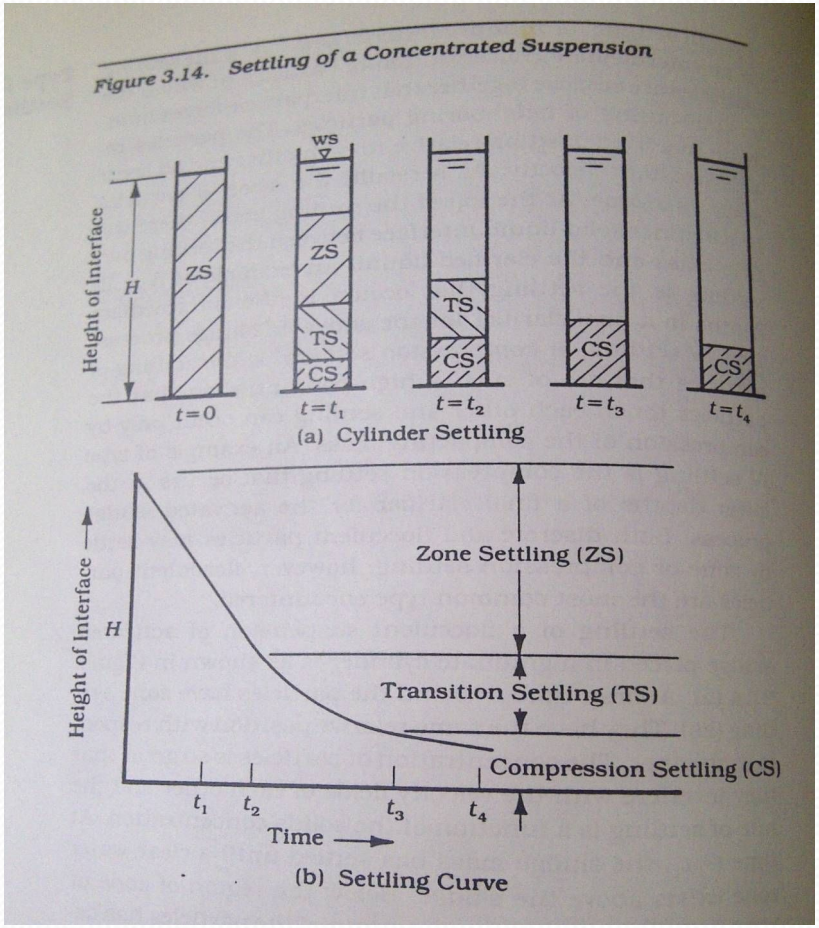
The concentration of particles is so great that they interfere with the velocity fields of each other and the rate of settling is a function of the solids concentration.



At time $t = t_1$, the sludge mass has settled until a clear water zone exists above the sludge. Below the region of zone or hindered settling, the concentration of the particles has become so great that many of the particles have made physical contact with each other.

- transition settling (TS) from zone settling to compression settling (CS).

Below the TS zone is the compression settling zone where all of the particles are in contact with each other and compression has begun.



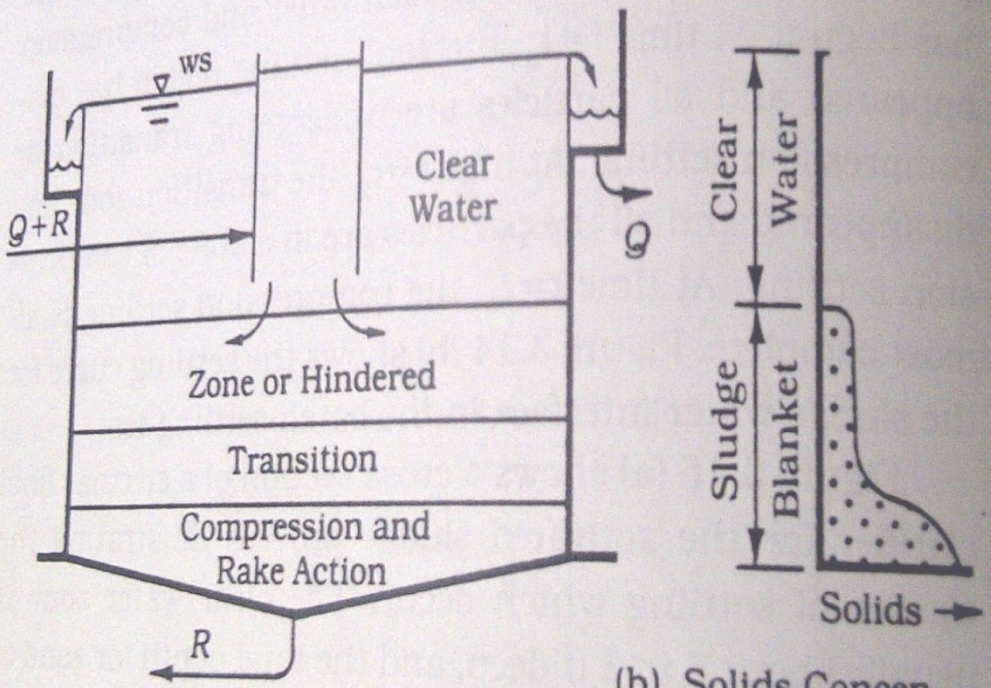
At time t = t2, the zone settling region has disappeared and all particles are undergoing transition or compression settling.

At time t = t3, the transition zone has disappeared and all particles are in a state of compression settling.

At time t = t4, the compression settling is almost complete.

Fig 3.14 (b) the settling curve for the sludge water interface in the batch settling test.

Figure 3.15. Settling in a Final Clarifier for the Activated Sludge Process



- Fig. 3.15 (a) a cross section of a circular final clarifier for the activated sludge process illustrating the classes of settling which occur.
- The clear water zone is usually about 5-6 ft deep.
- The total depth for zone or hindered, transition and compression settling is usually = 5-7 ft.
- Batch settling test as previously described can be used to obtain the parameters needed for the design of an activated sludge final clarifier by a method presented by Talmage and Fitch (1955).
- Another method based on solids flux is available (Dick, 1970).

- In final clarifiers, both:
  - Clarification of the liquid, and
  - The thickening of the solids
- Must be accomplished.
- Batch settling tests for design are usually done in a 1-L graduate cylinder equipped with a slow stirring device rotating at 4-6 rph to stimulate the raking action of the trusswork of the mechanical sludge rakes.

Determining the area of a final clarifier

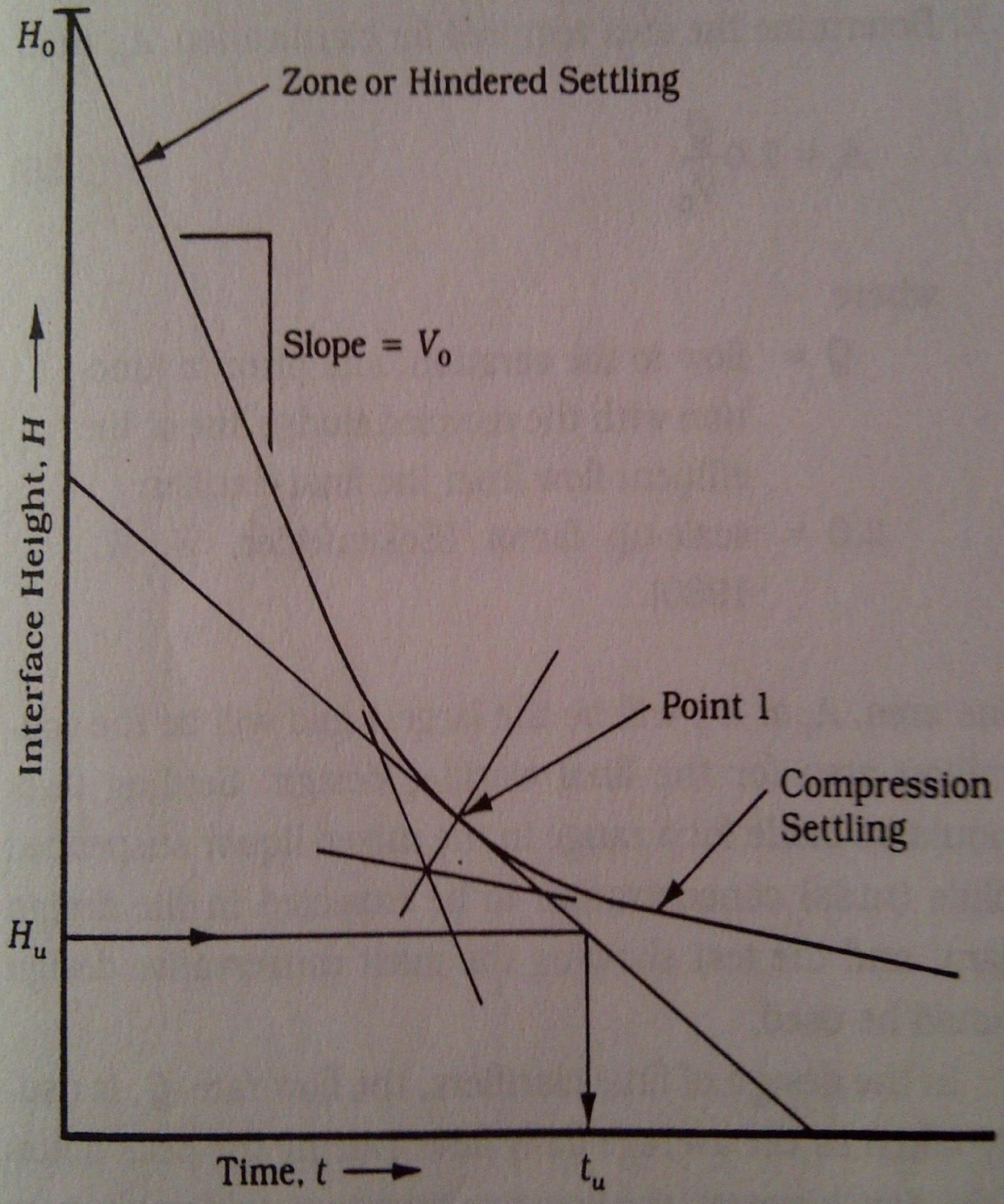
The procedure for determining the area of a final clarifier after obtaining a settling curve (Fig. 3.16):

1. Determine the slope of the hindered settling region, Vo the settling velocity required for clarification.
2. Extent tangents from the hindered settling region and the compresion region, and bisect the angle form to locate point 1.
3. Draw a tangent to the curve at point 1.
4. Knowing the initial sludge concentration, Co, and the initial sludge height, Ho, select a design underflow concentration, Cu, and determine the interface height, Hu.

- Since $C_u.H_u = C_o.H_o$ (3.33)
- Then Hu = Co.Ho/Cu (3.34)
- Draw a horizontal line from Hu to intersect the tangent line and determine the time, tu time to reach the desired underflow concentration, Cu.

Figure 3.16. Analysis of a Batch Settling Curve, Concentrated Suspension

Adapted and reprinted with permission from "Determining Thickener Unit Areas" by W. P. Talmage and E. B. Fitch. In *Industrial and Engineering Chemistry* 47, no. 1 (January 1955):38. Copyright © 1955 American Chemical Society.



- Determine the area required for thickening, At, from:
- $At = 1.5\ (Q + R)\ t_u/H_o$
- Where:
- Q = flow to the aeration tank prior to junction with the recycled sludge line
- R = recycled sludge flow
- Q + R = total flow to the final clarifier
- 1.5 = scale-up factor (Eckenfelder, 1980).

- Determine the area required for clarification, Ac, from:
- $Ac = 2.0\ Q/Vo$
- Where:
- Q = flow to the aeration tank prior to junction with the recycled sludge line or the effluent flow from the final clarifier
- 2.0 = scale-up factor (Eckenfelder, 1980).

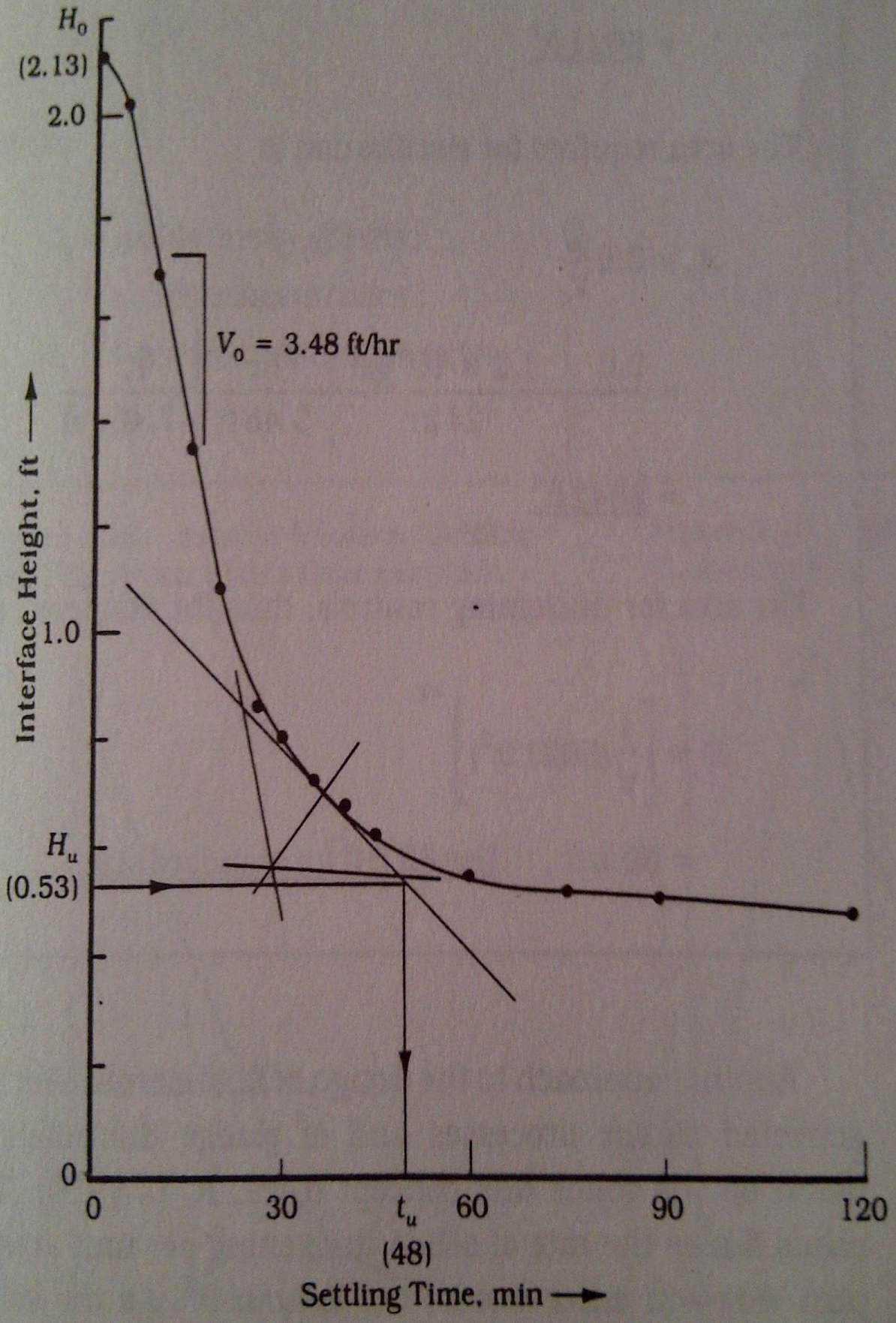
- One area, At or Ac, will be the largest and controlling area for the final clarifier design.
- Settling test should be made for a range in the mixed liquor suspended solids (MLSS) concentration to be expected in the design plant, and the test showing the most conservative design should be used.
- In the design of final clarifiers, the flow rate, Q, is usually taken as the average daily flow. During the peak hours of the day, however, the flow rate may be considerably more and could be up to about five times the average flow for municipal ww.

- - solids spill over the effluent weirs during the peak of the day.
- At and Ac for the peak hour of the day should be checked.
- the peak flow condition controls the design area for the final clarifier for an activated sludge process.

Example

- A final clarifier is to be designed for an activated sludge plant treating an industrial waste water having a design flow of 1.2 MGD. Batch settling studies have been performed in the laboratory using an acclimated culture of activated sludge and a graduate cylinder with a very slow rotating stirrer. The MLSS in the test was 2500 mg/l. The interface height vs settling time is shown in Fig 3.17. The design MLSS is 2500 mg/l and the design underflow concentration is 10,000 mg/l.

Figure 3.17. Graph for Example Problem 3.2



- Determine:
- The area required for clarification.
- The area required for thickening
- The design diameter.

Solids flux concept (Dick, R.I, 1970)

- Solids flux: the rate of solids thickening per unit area (lb/hour-ft^2).
- As the solids settle in clarifiers and thickeners, they must be thickened form the initial concentration, Co, to the underflow concentration, Cu, at the bottom of the tank.
- As the solids move downward, at some level in the tank, a limiting solids flux, G_L, occours.
- This flux must not be exceeded or solids will build up and spill over with the effluent from the tank.

- The movement of the solids downward occurs by:
- Hindered settling,
- The bulk flow downward due to the underflow.
- The data required for the flux design method are determined from batch settling tests.
- Numerous concentrations of the sludge are allowed to settle to obatin the hindered settling velocities.
- The hindered or zone settling velocities, Vo, are measured as illustrated in the previous example using a slowly stirred graduate cylinder.

- a plot is made of the hindered settling velocity vs the solids concentration, C (Fig. 3.18).
- At numerous concentrations, the solids flux is computed since it is obtained by multiplying the velocity by the solids concentration. The resulting curve of flux vs concentration is shown in Fig. 3.19.

- At any level in the settling tank, the movement of solids by settling is:
- Gs = Ct. Vt
- Where:
- Gs = solids flux by gravity
- Ct = solids concentration
- Vt = hindered settling velocity

- The movement of the solids due to bulk flow is:
- $G_b = C_t . V_b$
- Where:
- G_b = bulk flux
- V_b = bulk velocity
- The total solids flux for gravity settling and bulk movement is:
- $G_t = G_s + G_b = C_t V_t + C_t V_b$
- Where G_t = total flux

- The bulk velocity:
- $V_b = Q_u/A$ ....(3.40)
- Where
- Q_u = flow rate of the under flow
- A = plan area of the tank

- The mass rate of solids settling (the weight of the solids settling per unit time):
- $M_t = Q_o C_o = Q_u C_u$.....(3.41)
- Where:
- M_t = rate of solids settling
- Q_o = influent flow rate to the tank
- C_o = influent solids concentration

- The limiting cross sectional area:
- A = Mt/GL = QoCo/GL.....(3.42)
- Where
- GL = limiting flux

- Rearranging Eq 3.41 gives Qu = Mt/Cu
- Combining this with Eq 3.40 and 3.42:
- Vb = Qu/A = Mt/(CuA) = GL/Cu
- - Fig 3.20

- Selecting an underflow concentration, Cu, and drawing a tangent to the flux gives the y-axis intercept as GL, the limiting flux value.
- The slope of the tangent = Vb, the bulk velocity.
- The value of the gravity flux = Gs
- The value of the bulk flux = GL-Gs.

Example

- Batch settling tests have been performed using an acclimated activated sludge to give the data in Table 3.3.
- The design mixed liquor flow to the final clarifier is 2530 gpm, the MLSS = 2500 mg/l, and the underflow concentration = 12,000 mg/l. Determine the diameter of the final clarifier.

Example

- Batch settling tests have been performed using an acclimated activated sludge to give the data in Table 3.4.
- The design mixed liquor flow to the final clarifier is 160 L/s, the MLSS = 2500 mg/l, and the underflow concentration = 12,000 mg/l. Determine the diameter of the final clarifier.

Contoh soal

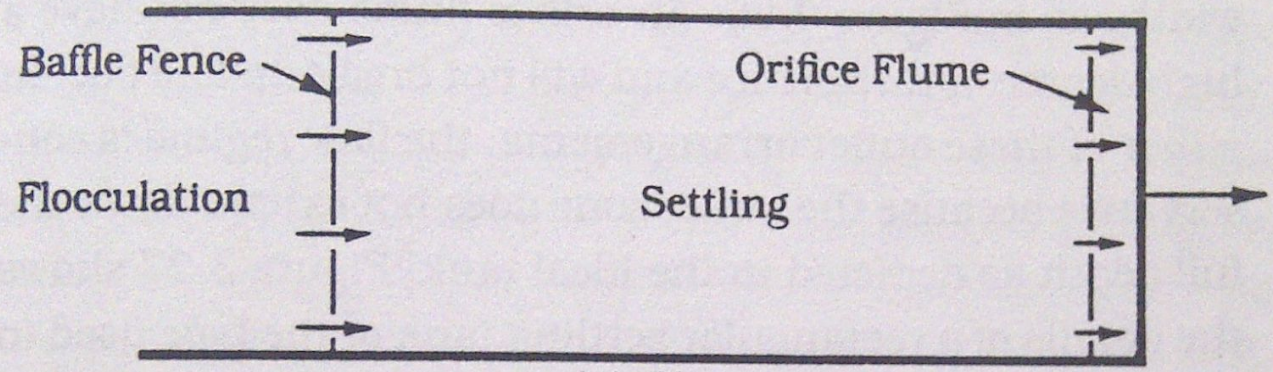
- Q = 1,8 MGD
- Cu = 13,500 mg/L
- Co = 3500 mg/l
- Tentukan:
- Diameter clarifier
- Tinggi clarifier

Waktu (menit)	Ketinggian permukaan lumpur (cm)
0	75,5
10	57,5
20	40
25	34
30	27
35	24
40	22
50	18,5
60	17,5
70	17,25

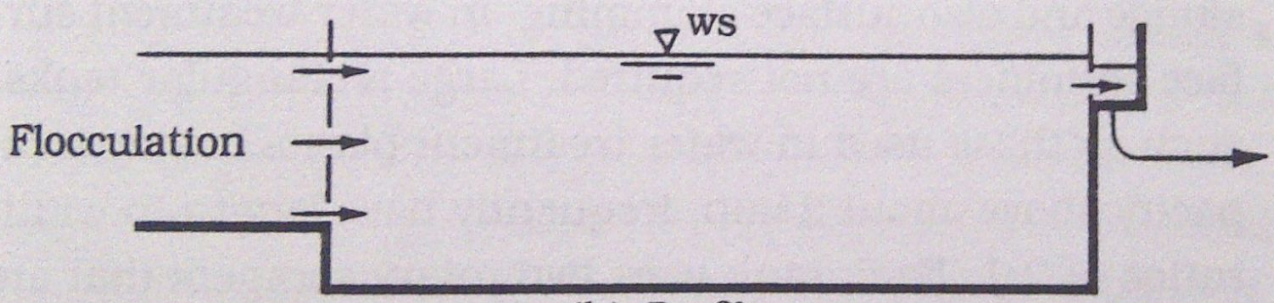
Actual sedimentation basins

- Are rectangular, square or circular in plan area.
- A single rectangular basin will cost more than a circular basin of the same size.
- But if numerous tanks are required the rectangular units can be constructed with common walls and be the most economical.
- Fig. 3.25 □ a schematic of the inlet to a rectangular tank if the unit is adjacent to a flocculation basin.
- This occurs frequently in the chemical coagulation of waters and wastewaters.

Figure 3.25. Inlet and Outlet Details for a Rectangular Settling Tank with Orifice Flume Outlet Preceded by Flocculation



(a) Plan

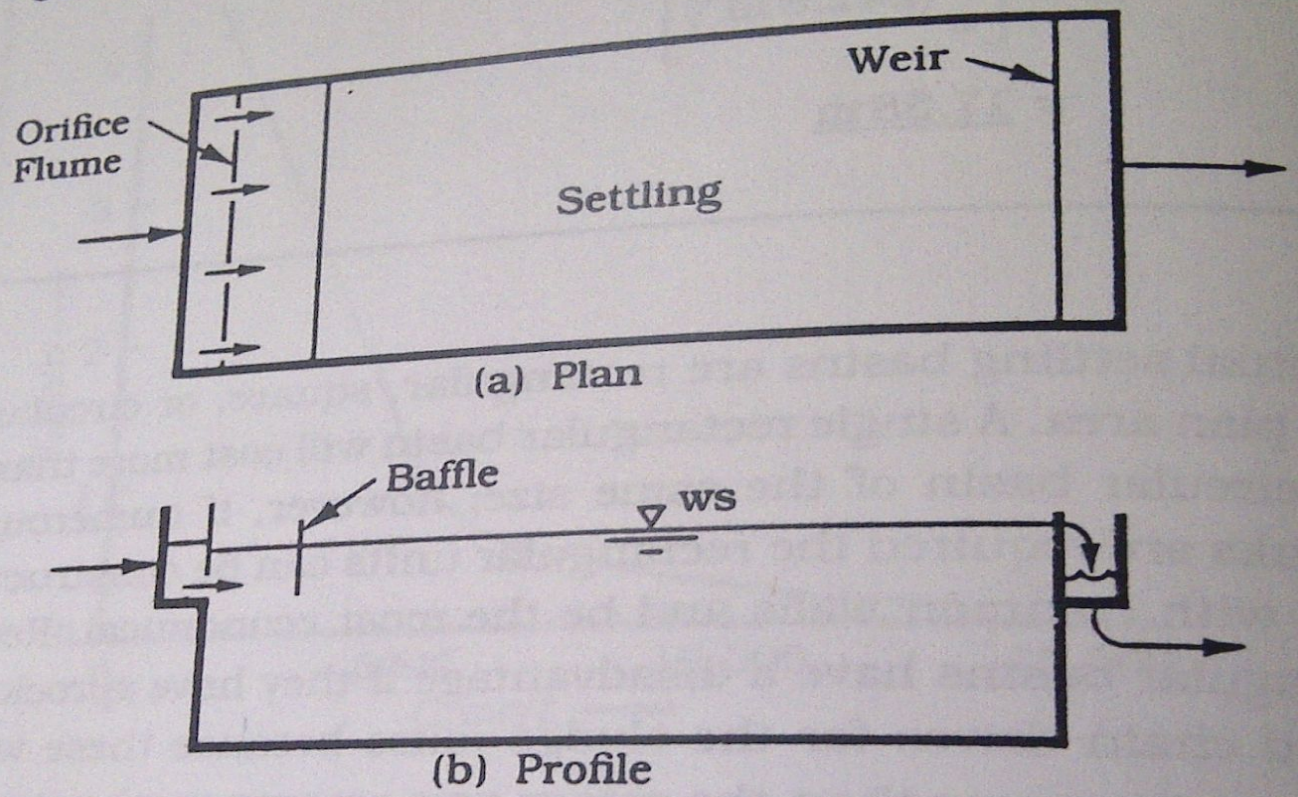


(b) Profile

- The flocculation basin will be the same width as the settling tank but is usually not as deep.
- The two basins are separated by a wood baffle fence or a concrete wall with numerous ports.
- The inlet water will enter uniformly across the basin.
- This inlet arrangement closely approaches that of an ideal rectangular tank;
- The only difference is that the inlet zone does not extend down to the full depth of the settling tank but extends down to the depth of the flocculator.

- If the rectangular basin does not adjoin a flocculator (Fig. 3.26), the inlet water is distributed uniformly across the basin by a flume with ports into the tank, and
- The inlet zone does not extend down to the full depth of the tank as depicted in an ideal tank.
- But, a baffle in front of thee flume will disperse the water downward to give a deeper inlet zone.
- Fig. 3.26 shows one type of outlet for a rectangular basin.

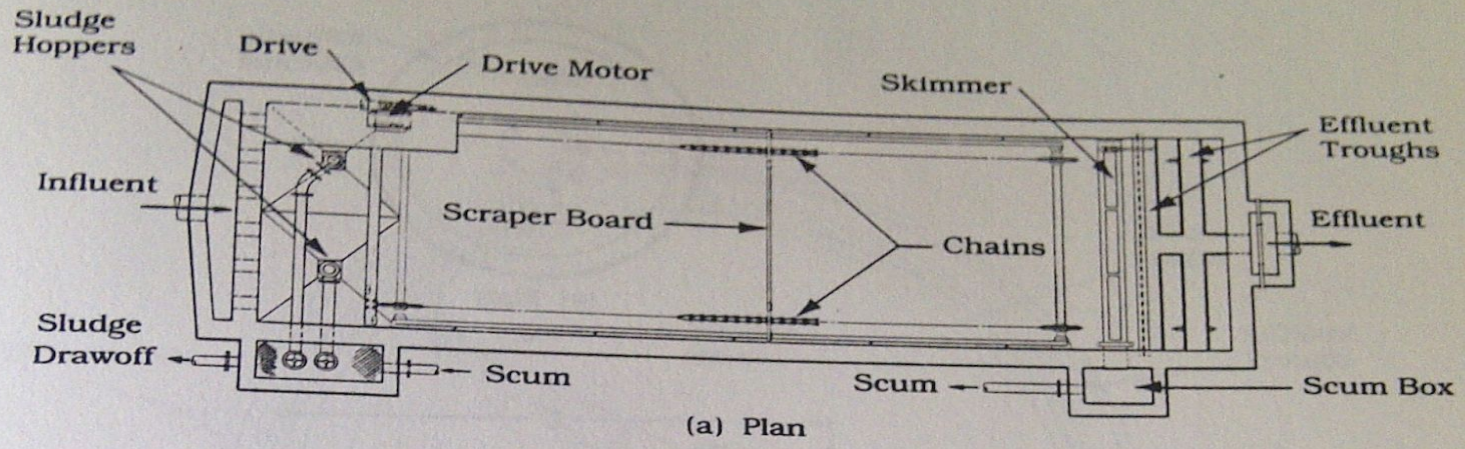
Figure 3.26. Inlet and Outlet Details for a Rectangular Settling Tank with Orifice Flume Inlet and Weir Channel Outlet



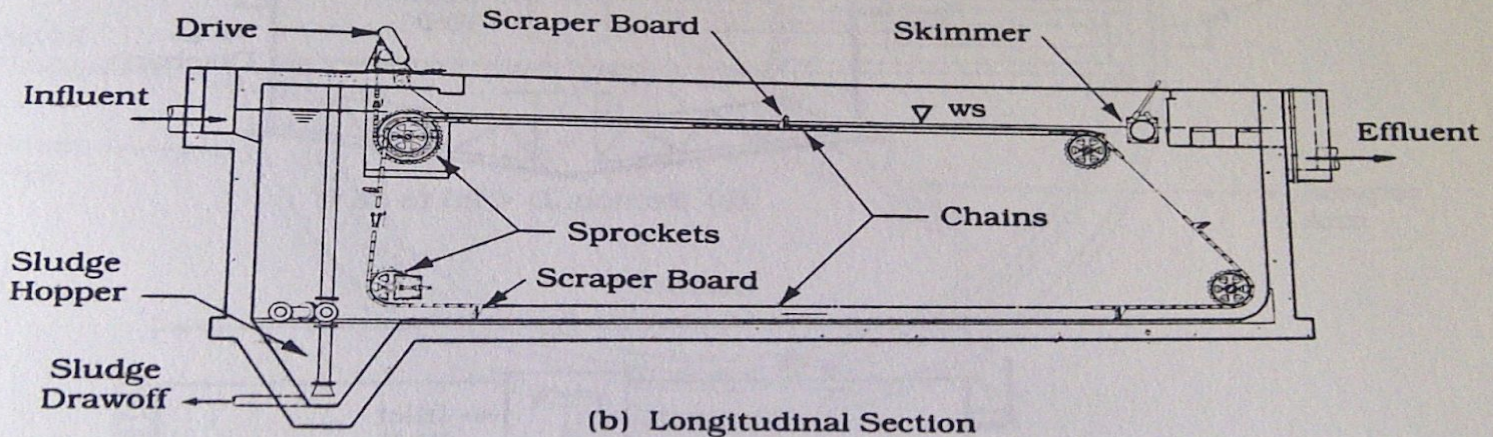
- A weir is used with spills into the effluent flume and extends across the entire width of the basin.
- If, however, the water is a chemically coagulated water, a weir should be avoided because the turbulence will break up much of the fine floc and result in poor filter performance.
- For chemically treated waters, it is best to have an orifice flume across the basin width (Fig.3.25).
- An orifice flume does not have a high degree of turbulence and will not break up fine floc.

- In either of these outlet arrangement, the flow regime is conservative because the outlet zone does not extend down the full depth as depicted in the ideal tank.
- Fig 3.27 □ the details of a rectangular tank of the type used in wastewater treatment.
- It has mechanical collection of the sludge and surface skimming.
- In water treatment surface skimmers are not required.

Figure 3.27. Rectangular Settling Tank
Courtesy of Walker Process, Inc.



(a) Plan

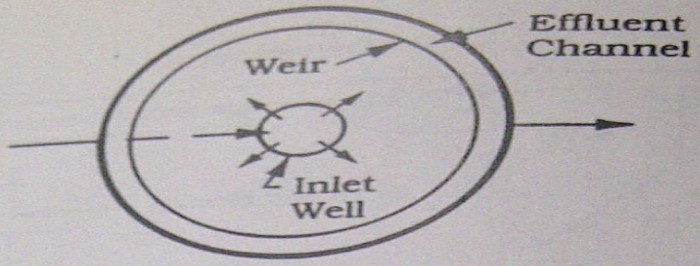


(b) Longitudinal Section

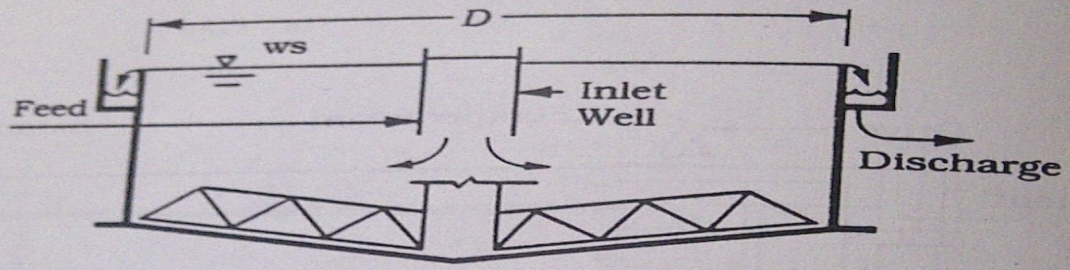
- Large rectangular tanks, such as those used in WTP having a capacity > 2 mgd, frequently have length to width ratios of 2:1.
- Each tanks uses two rotary scrapers that are designed for square basins.
- In circular tanks, the flow either enters the center of the tank (center feed) or the periphery of the tank (side feed).
- Fig. 3,28 □ the inlets to center feed tanks

- If the tank < 30 ft in diameter, the inlet pipe will enter through the wall and discharge into the baffle well (Fig. 3.28 (b)).
- Then the flow enters in a downward direction.
- If the tank > 30 ft in diameter, the inlet pipe will run underneath the tank and discharge vertically in the center at the baffle well (Fig.3.28 (c)).

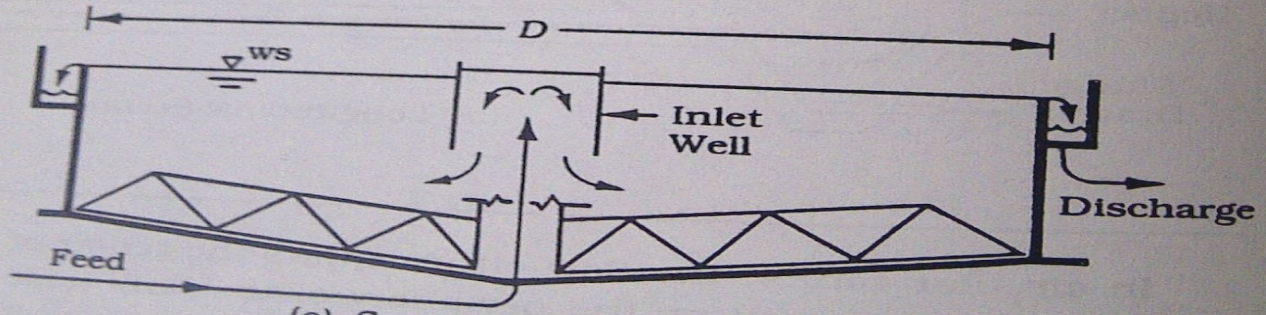
Figure 3.28. Inlet and Outlet Details for Circular Tanks (Center Feed)



(a) Plan



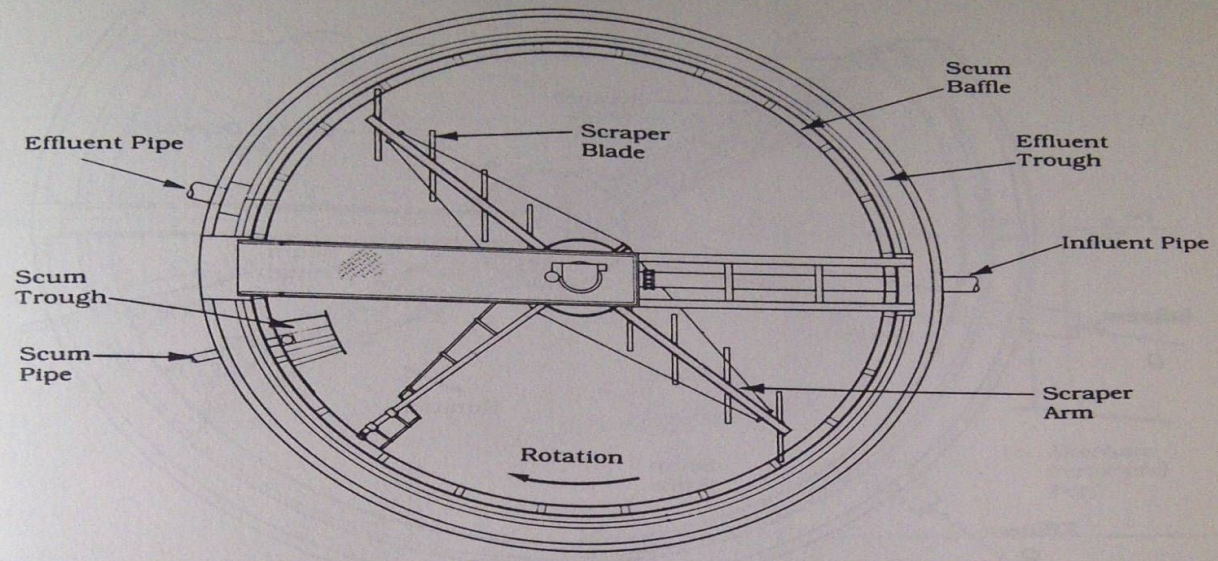
(b) Section, $D < 30$ to 35 ft



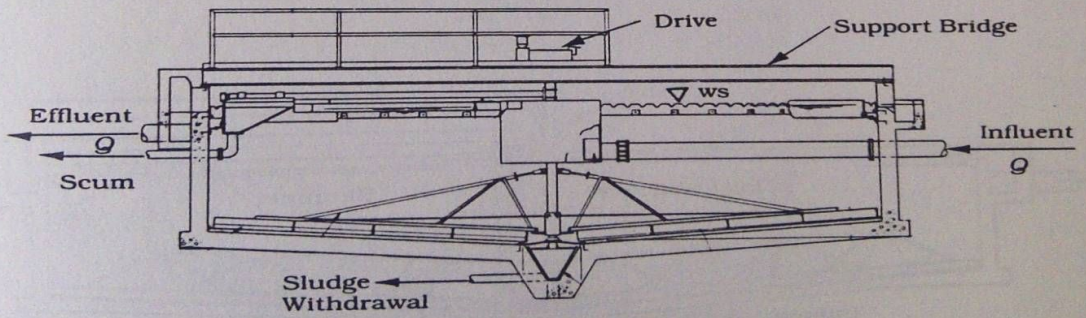
(c) Section, $D > 30$ to 35 ft

- The outlet for both tanks consist of a weir channel around the periphery giving a uniform flow removal.
- The depth of the outlet zone is not as great as for an ideal basin; thus it is conservative.
- Fig 3.28 & 3.29 the details of the center feed circular clarifiers that are used in WWTP.
- They have both mechanical sludge rakes & surface skimming.

Figure 3.29. ***Circular Settling tank (Center Feed by Pipe through Wall)***
Courtesy of Infilco Degremont, Inc.



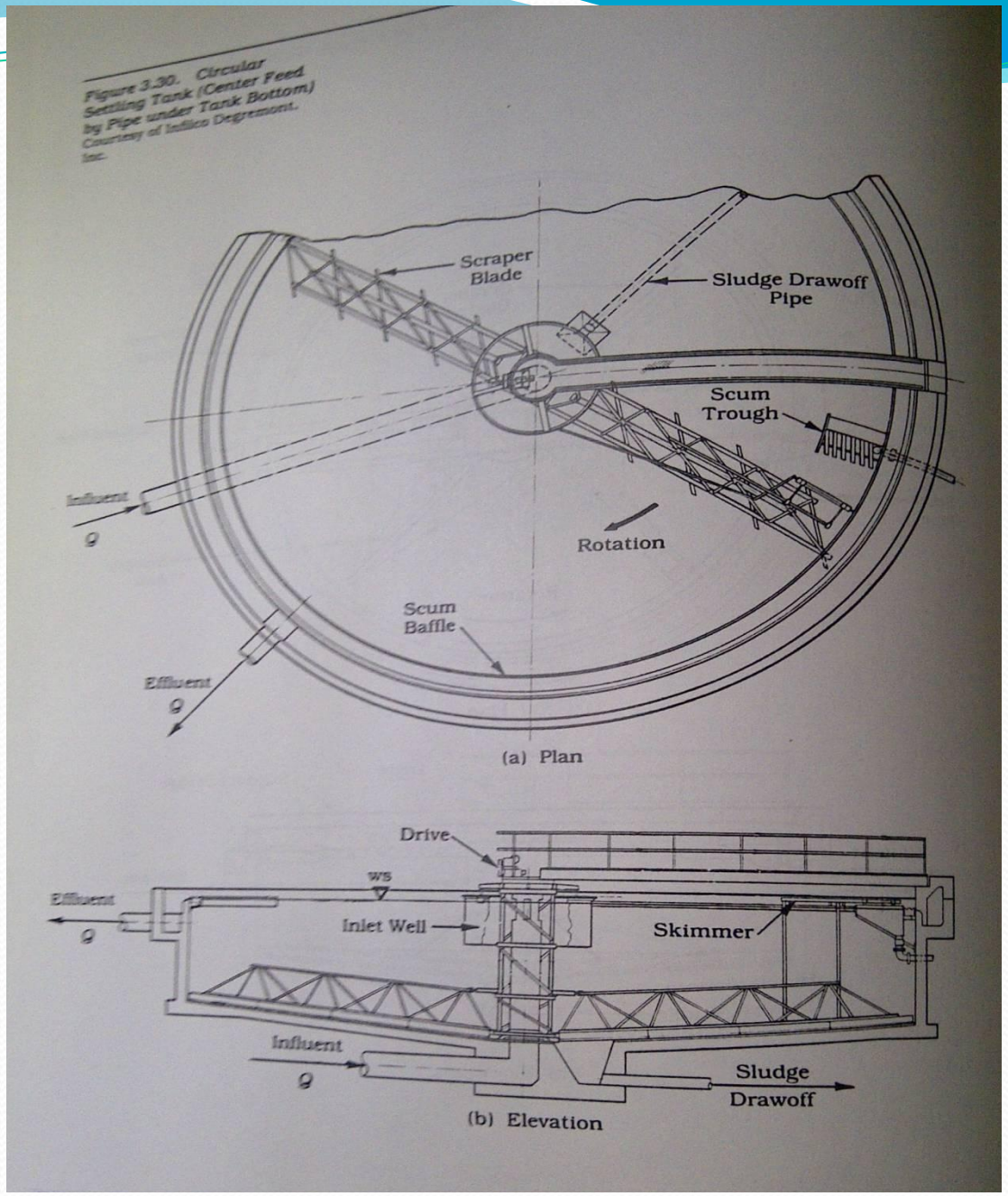
(a) Plan



(b) Elevation

- Fig 3.29: the unit has the influent pipe entering through the clarifier wall and extending to the baffle well.
- Fig.3.30: a unit where the influent pipe runs under the clarifier, rises at the center and discharges into the inlet well.
- In WTP, circular tanks used are similar o those used in WWTP except that surface skimmers are not needed.

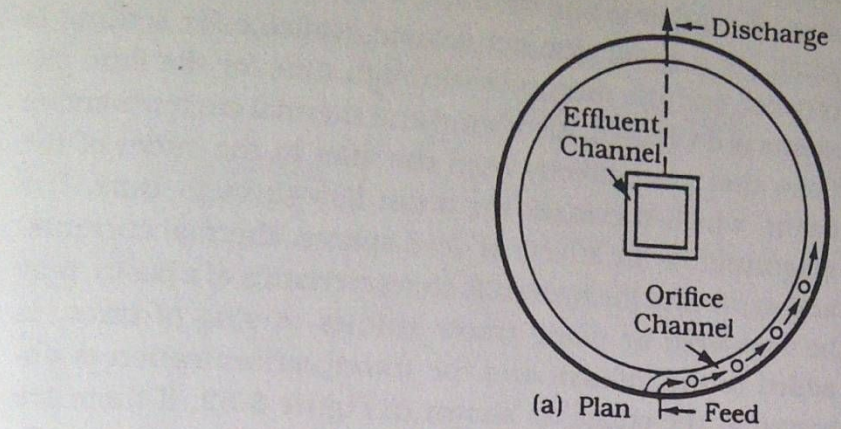
Figure 3.30. Circular Settling Tank (Center Feed by Pipe under Tank Bottom) Courtesy of Infilco Degremont, Inc.



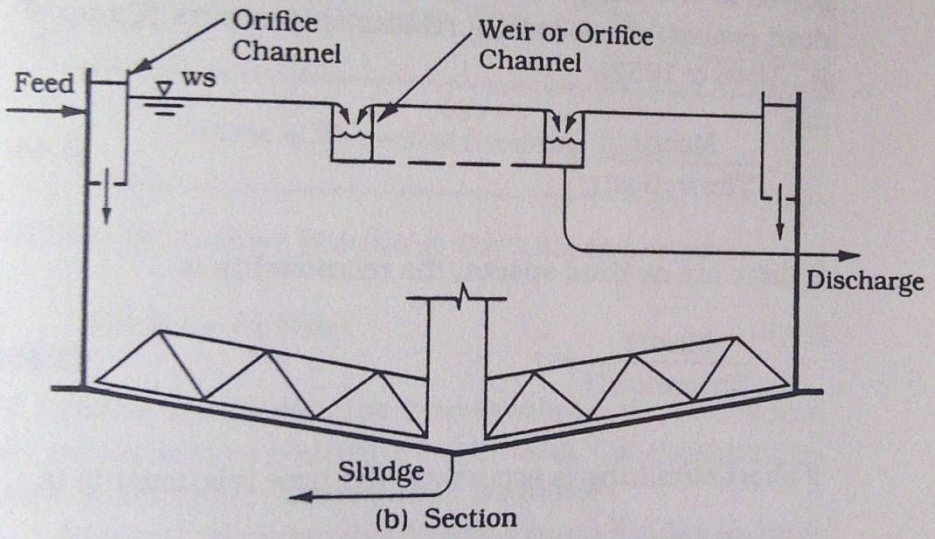
- The bottom of circular tank slopes to the center at a slope which is usually 1:12; thus it forms a flat inverted cone.
- In design, the volume of the cone is not considered in the design volume, which is taken as being the plan area times the depth of the water at the side of the tank.
- The sludge is usually collected in a hopper near the center of the tank.

- Fig 3.31 □ the inlet details for a periphery feed tank.
- As the flow enter, it is deflected so that it moves around the periphery in an orifice channel (Fig 3.31 (a)).
- From the channel, the flow discharges through the orifices into the clarifier (Fig.3.31 (b)).
- Sometimes, instead of an orifice channel, there will be simply a skirt surrounding the inside of the tank, and the liquid flows out under the skirt (Fig. 3.31 (c)).

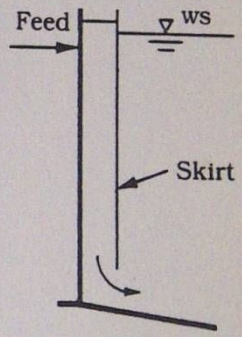
Figure 3.31. Inlet and Outlet Details for a Circular Tank (Peripheral Feed)



(a) Plan



(b) Section



(c) Alternate Peripheral Feed

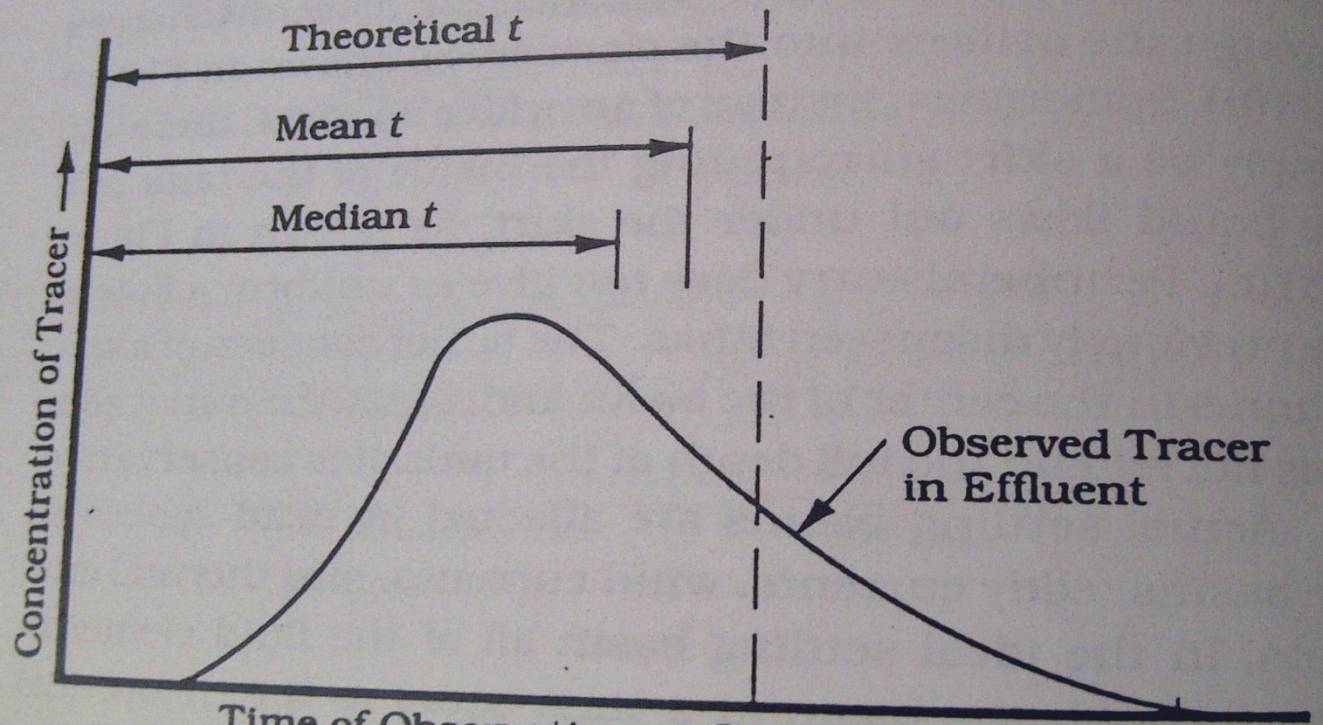
- Peripheral entry does not give as uniform a flow as the previously discussed tanks.
- The outlets consist of a weir channel in the center of the basin, and
- Since the outlet zone does not extend to the full depth of the tank, it is conservative.

- Actual settling basins are affected by dead spaces in the basins, eddy currents, wind currents, and thermal currents.
- In the ideal settling basin all of the fluid elements pass through the basin at a time equal to the theoretical detention time, t, which is equal to V/Q.
- Actual basins, however, have most of the fluid elements passing through at a time shorter than the theoretical detention time, although somefluid elements take longer.

- Dead spaces and eddy current have rotational flow and do very little sedimentation since the inflow to and outflow from these spaces is very small.
- Thus:
- □ the net volume available for settling is reduced
- □ the mean-flow through time for the fluid elements is decreased.

- Wind and thermal currents create flows that pass directly from the inlet to the outlet of the basin, which decreases the mean-flow through time.
- The magnitude of the effects of dead spaces, thermal currents, etc., and the hydraulic characteristics of a basin - tracer studies.
- A slug of tracer is added to the influent and the tracer concentration is observed at the outlet Fig.3.32

Figure 3.32. Settling Basin Characteristics as Shown by Tracer Studies



- If there are dead spaces, the following relationship occur (3.44):

$$\frac{mean_t}{theoretical_t} < 1$$

- If there are no dead spaces (3.45):

- $$\frac{mean_t}{theoretical_t} = 1$$

- If short circuiting occuring (3.46):

$$\frac{median_t}{mean_t} < 1$$

- If there is no short circuiting (3.47):
- Mean t = median t

- If a basin is unstable, the time concentration plot cannot be reproduced in a series of tracer test erratic basin performance can be expected.
- Fig 3.33 the results of tracer studies on 3 types of settling basins:
- the rectangular basin approached the ideal more closely than the circular type.
- Of the circular type, the peripheral feed had better performance than the center-feed tank.

Figure 3.33. Tracer Studies on Circular and Rectangular Settling Tanks

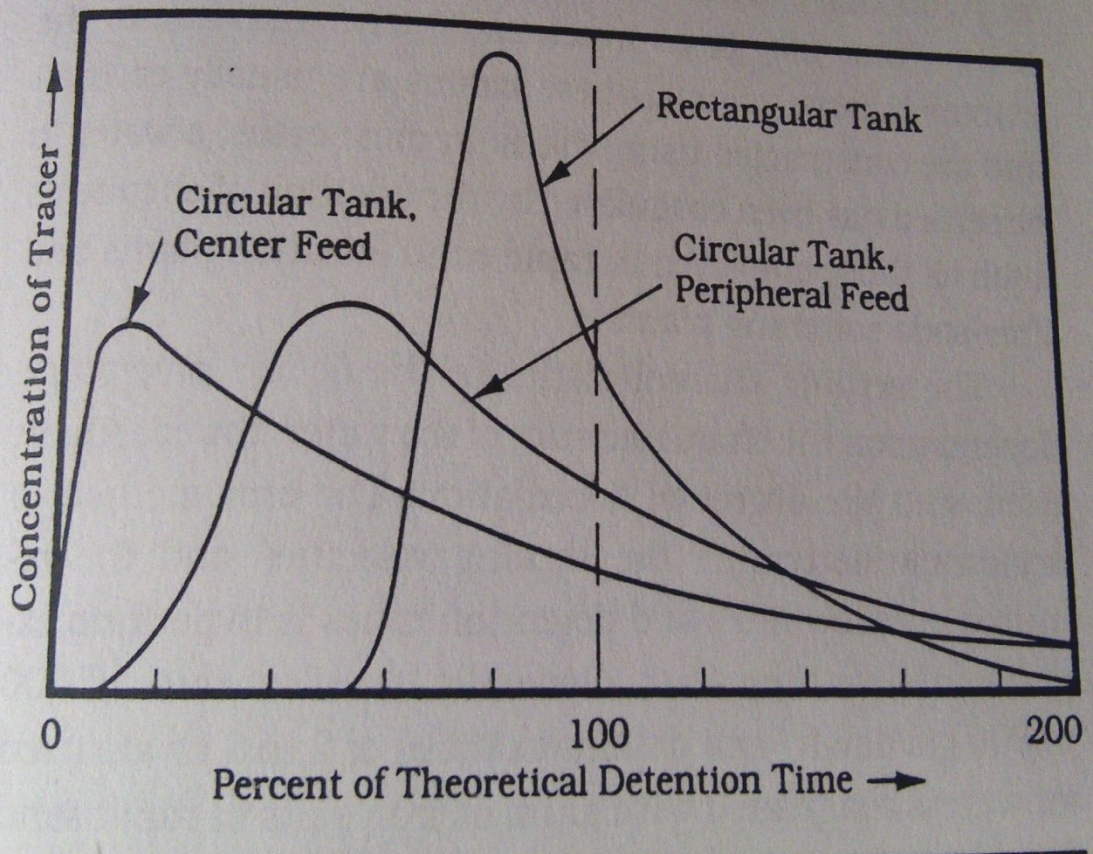
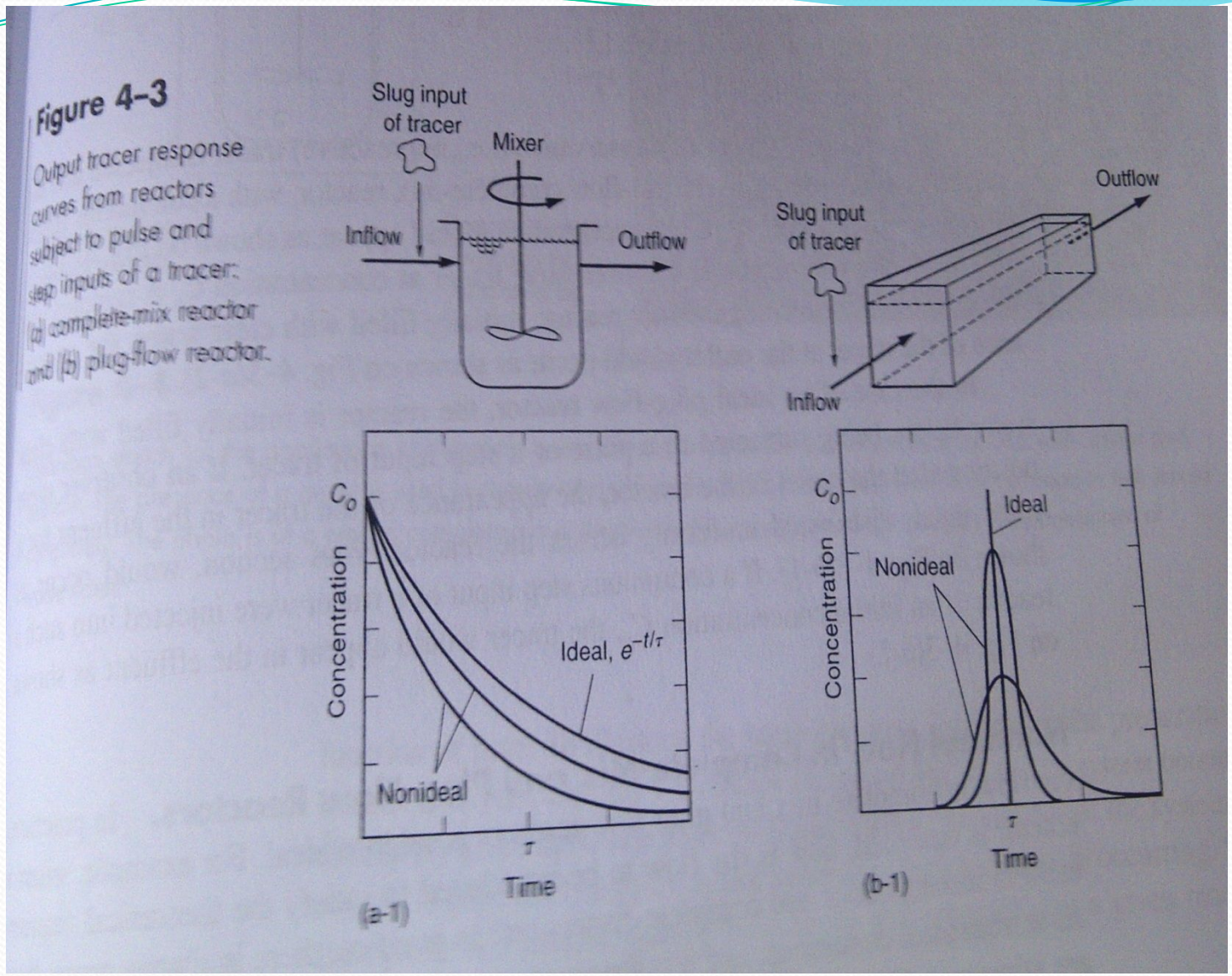


Figure 4–3

Output tracer response curves from reactors subject to pulse and step inputs of a tracer: (a) complete-mix reactor and (b) plug-flow reactor.



- In water & ww engineering, most of the suspensions are flocculant.
- Flocculant particles of the same initial size & density as discreet particles will intercept the sludge zone in a shorter time due to flocculation and more rapid settling.
- □ if the ideal settling basin theory is applied to slightly flocculant particles, it will be conservative.

- Athough there are differences between the ideal & actual basins, the ideal settling basin theory gives the most rational approach to design,
- And reveals that the most important design parameters are:

1. The overflow rate or design settling velocity
2. Detention time
3. depth

Sedimentation in WTP

- Sedimentation is practiced for:

1. Untreated water - plain sedimentation (PS)
2. Chemically coagulated waters

- If a water has a high turbidity due to silt, PS may be used to reduce the turbidity.
- PS is frequently used for waters having consistent turbidity > 1000 mg/L

- The settling characteristics of the floc or precipitate depend on:

1. the characteristic of the waters
2. The coagulant used
3. The degree of flocculation

The only method to accurately determine the settling velocities, the required overflow rates & detention times perform experimental settling tests

- For waters coagulated with alum or iron salts in RSF plants, usually:

1. Overflow rate = 500-800 gal/day-ft2
2. Detention time = 2-8 hr
3. Weir loading = 12,000-22,000 gal/day-ft

these are based on average daily flow.

In lime-soda softening plant:

1. Overflow rate = 500-1000 gal/day-ft2
2. Detention time = 4-8 hr
3. Weir loading = 22,000-26,000 gal/day-ft

Since turbulence due to weirs may shear apart fine floc

- use orifice channels for the outlets
- or low-weir loadings

Shearing of floc will lead to:

- poor filter performance
- Short filter runs.

example

A rectangular clarification basin is to be designed for a RSF. The flow is 8 mgd. The overflow rate = 600 gall/day-ft2. the detention time = 6 hr. 2 sludge scraper mechanism for square tanks are to be used in tandem to give a rectangular tank with a length to width ratio of 2:1. Determine the dimensions of the basin.

Sedimentation in WWTP

- In conventional WWTP, primary sedimentation is used to remove as much settleable solids as possible from raw wastewaters.
- Secondary settling in:
  - active sludge plants is employed to remove the MLSS
  - Trickling filter plants to remove any growth that may slough off the filters.
- □ good secondary settling produces a high quality effluent low in suspended solids.

Primary sedimentation

- Recommended criteria for primary clarifiers treating municipal wastewaters Table 3.5
- The detention times based on the average daily flow= 45 min- 2hr.
- The depth & overflow rates in Table 3.5 should controll in design.
- Once the flow > 1 mgd:
  - □ use multiple tanks.
  - □ peak weir loadings should not exceed 30,000 gal/day-ft.
- Once the flow < 1 mgd □ peak weir loadings should not exceed 20,000 gal/day-ft.

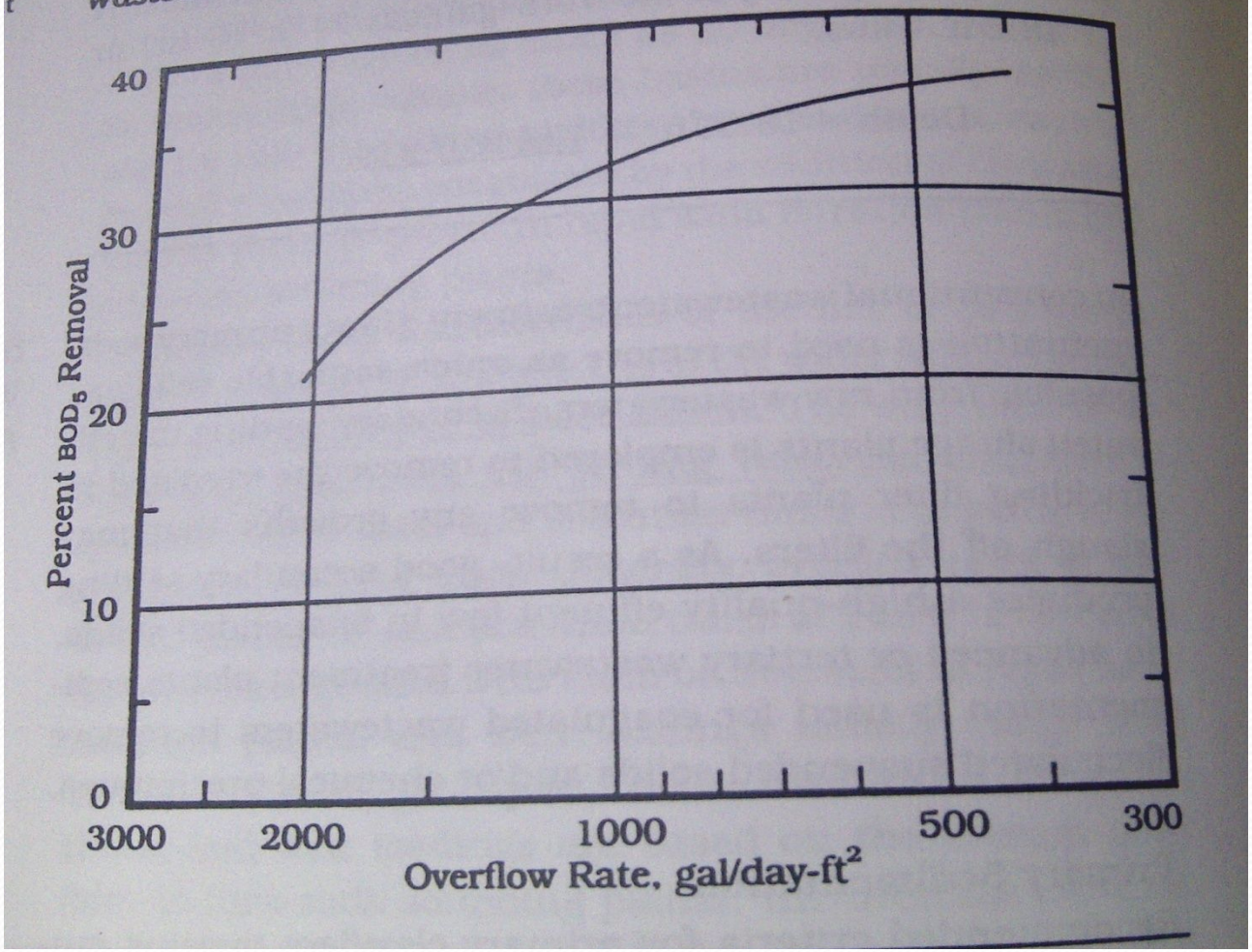
Table 3.5. Overflow Rates and Depths for Primary Clarifiers

Type Treatment	Overflow Rate (gal/day-ft^2)		Depth (ft)
	Average	Peak	
Primary Settling Followed by Secondary Treatment	800–1200	2000–3000	10–12
Primary Settling with Waste Activated Sludge	600–800	1200–1500	12–15

Taken from EPA, *Suspended Solids Removal*, EPA Process Design Manual, January 1975.

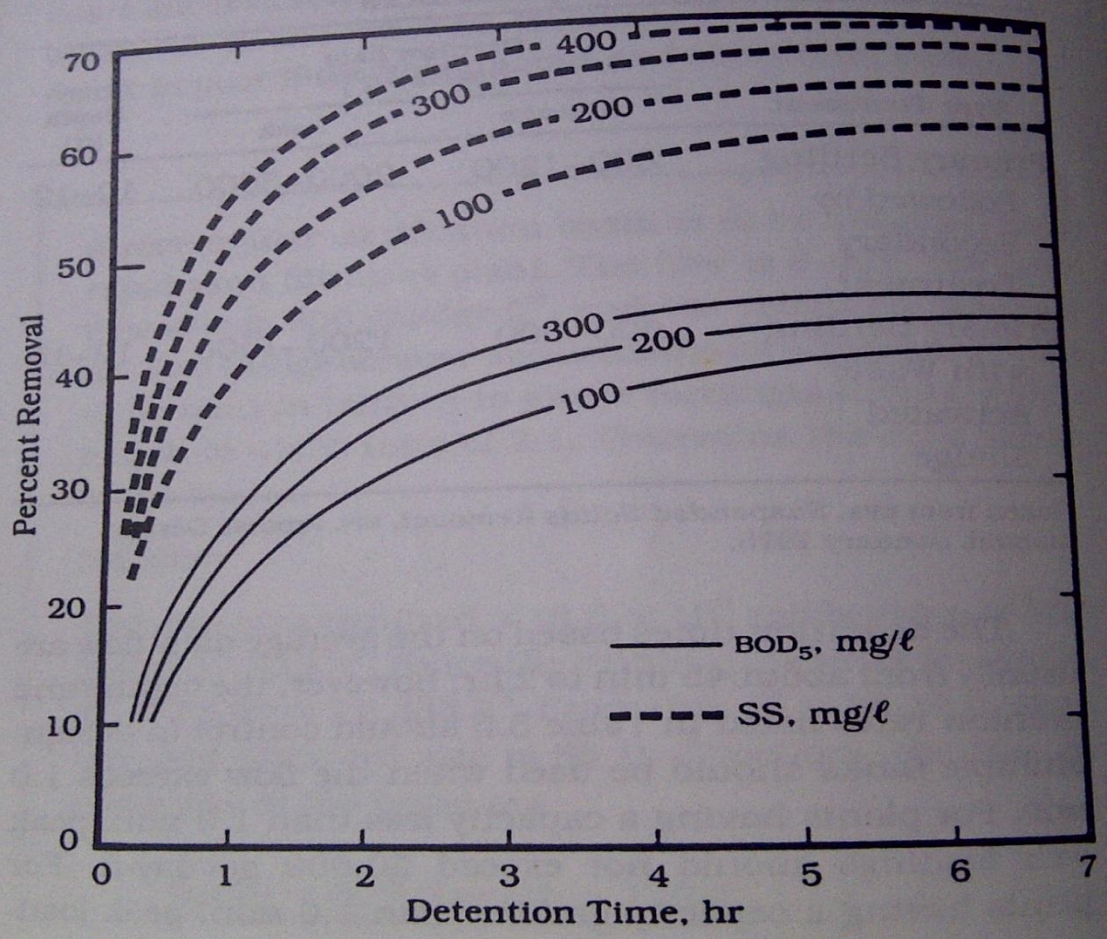
- A surface skimmer & a baffle are necessary for primary clarifiers to remove scum from the water surface.
- Although individual tank peformance varies, an estimate of the BOD5 & SS removal can be made using the detention time & overflow rate based on the average daily flow & Fig. 3.34 – 3.35.

Figure 3.34. Percent BOD_5 Removal versus Overflow Rate in Performance of Primary Clarifiers Treating Municipal Wastewaters.



Adapted from *Water Supply and Sewage*, 4th ed., by E. W. Steel. Copyright © 1960 by McGraw-Hill Book Co., Inc. Reprinted by permission.

Figure 3.35. Percent BOD_5 and Suspended Solids Removal versus Detention Time in Performance of Primary Clarifiers Treating Municipal Wastewaters



example

- A primary clarifier for a municipal WWTP is to be designed for an average flow of 2 mgd. The state's regulatory agency criteria for primary clarifiers are: peak overflow rate=2,200 gal/day-ft2, average overflow rate=900 gal/day-ft2, and peak weir loading = 30,000 gal/day-ft. The ratio of the peak hourly flow to the average hourly flow = 2.75.
- Determine:

1. The clarifier diameter
2. The weir loading

Secondary sedimentation

- Table 3.6 recommended criteria for secondary clarifiers treating wastewaters.
- The detention time based on the average daily flow= 1-2.5 hr;
- But, the depths, overflow rates, solids loadings in Table 3.6 should control in design.
- If Q > 1 mgd:
  - multiple tanks should be used
  - peak weir loadings similar to those used for primary clarifiers should not be exceeded.

Table 3.6 Overflow Rates, Solids Loadings, and Depths for Secondary Clarifiers

Type Treatment	Overflow Rate (gal/day-ft^2) Average	Peak	Solids Loading (lb/day-ft^2) Average	Peak	Depth (ft)
Activated Sludge (except Extended Aeration)	400–800	1000 –2000	20–30	50	12–15
Activated Sludge, Extended Aeration	200–400	800	20–30	50	12–15
Activated Sludge, Pure Oxygen	400–800	1000 –2000	25–35	50	12–15
Trickling Filters	400–600	1000 –2000	–	–	10–12

Taken from EPA, *Suspended Solids Removal*, EPA Process Design Manual, January 1975.

Inclined settling devices

- Include:
  - Inclined tube settlers
  - Lamella separator
- All have overflow rates much higher than conventional settling basins.

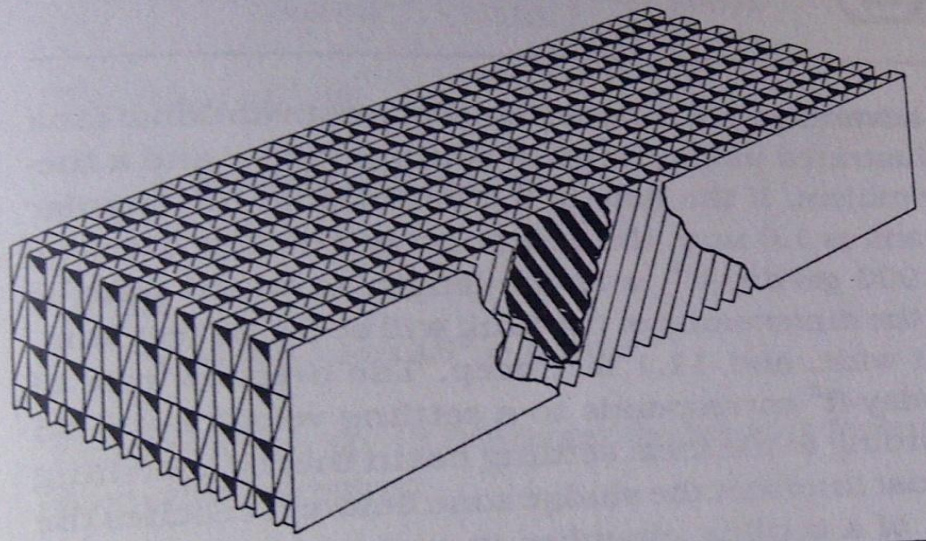
Inclined tube settlers

- Fig 3.36 (a) a module of inclined tube settlers
- Fig 3.37 the modules installed in a circular clarifier
- Fig 3.38 a rectangular clarifier

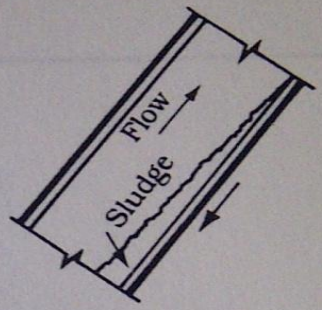
- The water to be clarified passes upward through the tubes,
- And as settling occurs the solids are collected on the bottom of the tubes - Fig 3.36 (b)

Figure 3.36. Inclined-Tube Settler

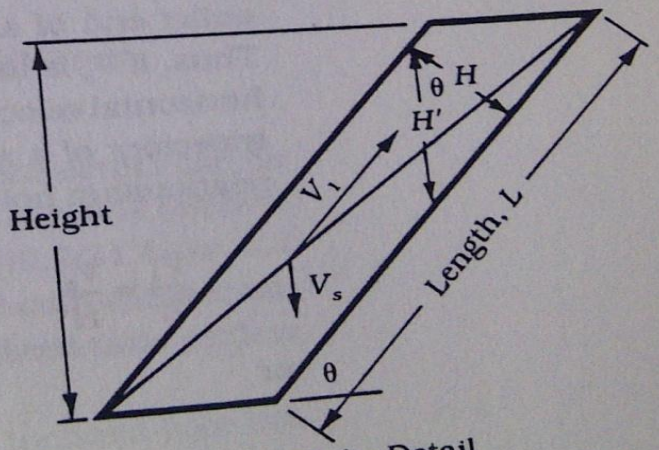
Part (a) courtesy of Neptune Microfloc, Inc.



(a) Module of Inclined Tubes



(b) Inclined-Tube Detail



(c) Inclined-Tube Detail

- The tubes are inclined at an angle of 45-60 degrees, which is steep enough to cause the settled sludge to slide down the tubes.
- The sludge fall from the tubes to the bottom of the clarifier, where it is removed by the sludge rakes.
- A square or rectangular cross section is the most common type.

- The advantage of a tube settler over a conventional tank can be illustrated using the ideal settling theory and a theoretical problem.
- If for a conventional tank:
  - Flow = 1 mgd
  - Detention = 2 hr
  - Overflow rate = 1,000 gal/day-ft2
  - Length to width ratio = 4:1

- Then the dimensions will be:
  - Length = 63.25 ft
  - Width = 18.81 ft
  - Depth = 11.1 ft
- The overflow rate of 1,000 gal/day-ft2 corresponds to a settling velocity of 5.55 ft/hr
- According to the ideal settling basin theory:
- A particle must intersect the sludge zone before it reaches the outlet end of a settling chamber in order to be removed.

- Thus, if:
  - Vs = the settling velocity
  - H = the depth
  - V1 = the horizontal velocity
  - L = the chamber length
- The critical trajectory of a settling particles is such that the following relationship holds true (3.48):

$$\frac{V_1}{L} = \frac{V_s}{H}$$

- Or (3.49):
- V1 = VsL/H
- Applying Eq 3.49 to the hypothetical problem gives:

$$V_1 = \left(5.55 \frac{ft}{hr}\right)\left(\frac{63.25\,ft}{11.10\,ft}\right) = 31.6\,ft/hr$$

- Thus the horizontal velocity= 31.6 ft/hr
- For an inclined tube settler, a particle must settle through the distance H' (Fig 3.36 (c)).
- Thus, for an inclined tube Eq 3.48 becomes (3.50):

$$\frac{V_1}{L} = \frac{V_s}{H}$$

- Assume, for the hypothetical problem, that modules of tube settlers are to be placed in the rectangular tank.
- The modules:
  - Height = 3 ft
  - Tubes:
  - Depth = 2 inches
  - Inclined at a 45 angle

- The value of H’ (3.51):

$$H' = \frac{H}{\cos\theta}$$

- Or

$$H' = \left(\frac{2in}{\cos 45^0}\right)\left(\frac{ft}{12in}\right) = 0.236\,ft$$

- The value of L:
- L= module height/sin θ ….(3.52)
- Or L = 3 ft/sin 45 = 4.24 ft

- Now applying Eq (3.50) to get the velocity through the tube gives the following:

$$V_1 = \left(\frac{L}{H'}\right)V_s = \left(\frac{4.24}{0.236}\right)\left(\frac{5.55\,ft}{hr}\right) = 99.7\,ft/hr$$

- Thus the velocity through the inclined tube settler can be 99.7 ft/hr and still have the same degree of solids removal as the horizontal settling unit.
- 99.7/31.6 or 3.2 times as much flow as the conventional basin can accommodate
- the advantages of the inclined tube settlers are readily apparent

- Usually: the overflow rates used for inclined tube settlers = 3-6 six times as great as those used for conventional settling tanks.
- Laminar flows is necessary for efficient settling since turbulent flow would scour the settled solids.
- Laminar conditions are made possible by the use of tubes with small hydraulic radii.
- Fig 3.37 & 3.38 □ modules of inclined tube settlers installed in a new or existing circular or rectangular clarifier.

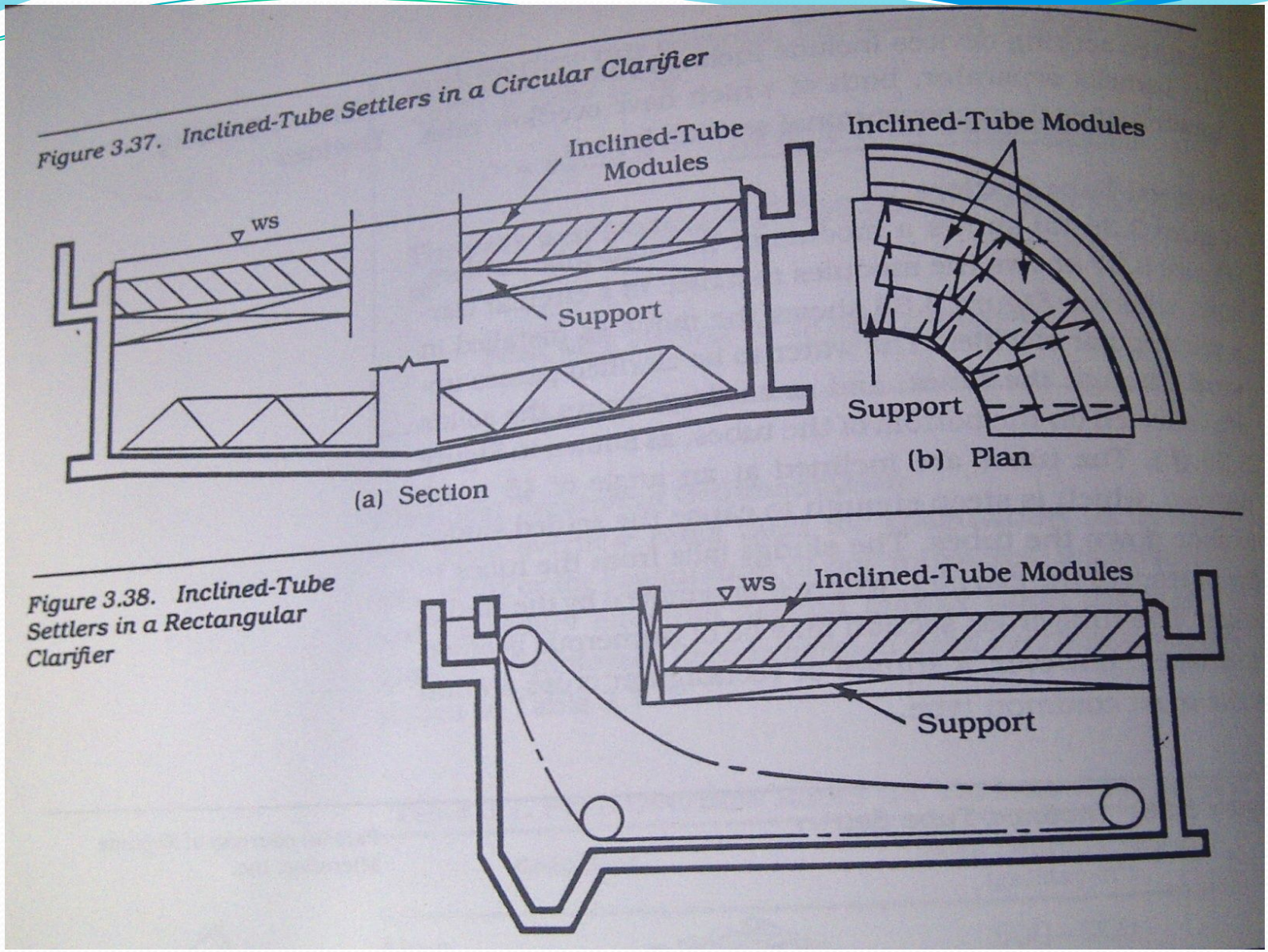


Figure 3.37. Inclined-Tube Settlers in a Circular Clarifier
Inclined-Tube Modules
WS
Support
(a) Section
Inclined-Tube Modules
Support
(b) Plan
Figure 3.38. Inclined-Tube Settlers in a Rectangular Clarifier
WS
Inclined-Tube Modules
Support

- In either type clarifier, a large portion of the plan area (=67-80%) is occupied by the tube modules.
- New settling tanks using inclined tube settlers will have much less area requirements than conventional settling tanks;
- But, one of the most common uses of inclined tubes is to increase the capacity of existing clarifiers.
- Since the settling velocity is dependent upon the water temperature, better results are obtained with warm waters.

- The usual overflow rates based upon the area covered by the tube modules: 3600-6000 gal/day-ft2.
- The tube settlers are ideal for increasing the capacity of existing clarifiers.
- In wastewater treatment, tube settlers have been successfully used in:
  - Secondary settling for activated sludge
  - & trickling filter plants
  - And for settling of coagulated wastewaters

- They are not well suited as primary clarifiers biological growths develop within the tubes.
- In particular, they are useful in increasing the capacities of existing final clarifiers.

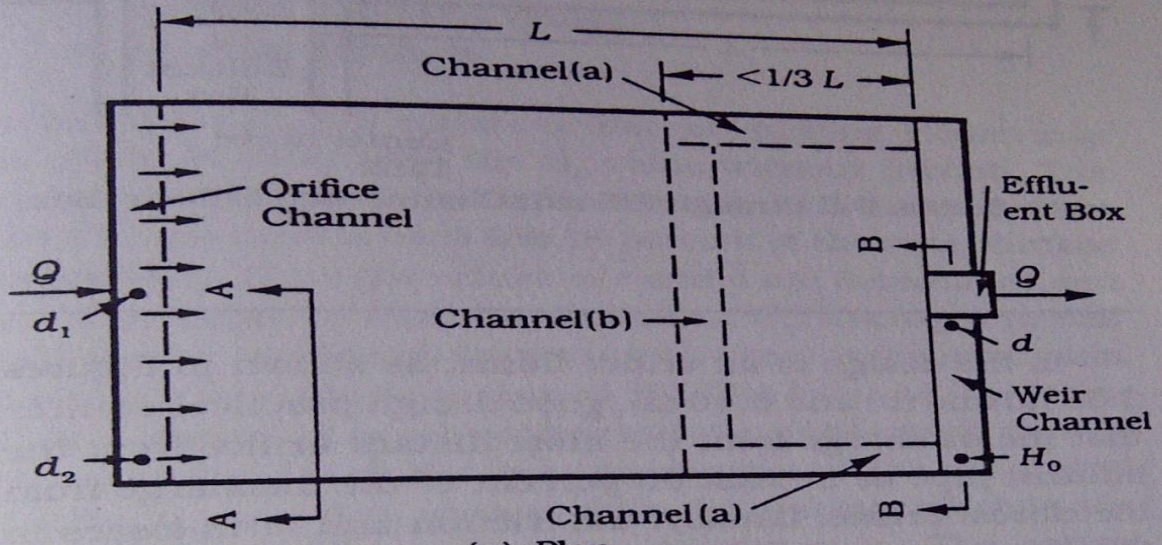
Lamella separator

- Is similar to the inclined tube settlers except that inclined plates are used to form the settling compartments and the sludge and water flow is cocurrent instead of countercurrent.
- The manufacturer recommends it only for use with coagulated waters and wastewaters.

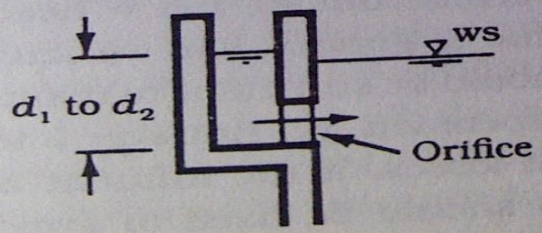
Inlet & outlet hydraulics

- To prevent short circuiting & basin instability:
  - The influent flow enter a sedimentation basin uniformly
  - The effluent flow leave uniformly
- Fig 3.39 (a) the plan of a typical rectangular basin with an influent orifice flume and an effluent weir channel.
- Fig 3.39 (b) a cross-section through the influent flume
- Fig 3.39 (c) - a cross section through the effluent channel

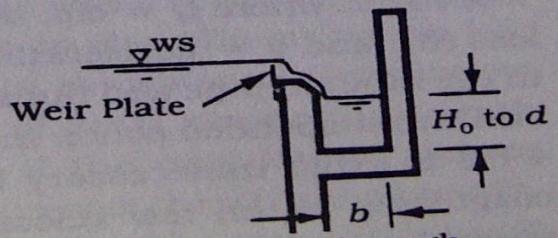
Figure 3.39. Inlet and Outlet Details for a Rectangular Tank



(a) Plan



(b) Section through Orifice Flume



(c) Section through Effluent Channel

- Fig 3.40 (a) a profile of the influent flume
- Fig 3.40 (b) a profile of the effluent channel

- Influent orifices:
  - Circular
  - Rectangular
- Effluent weir:
  - A surpressed weir
  - A series of 90 degree V notch weirs 8 inch

- The elevation of the water surface in the effluent box is set by the elevation of the water surface in the next downline treatment unit and the total headloss between the two water surfaces.

Orifice flume design

- The discharge from the most distant orifice from the influent pipe be = min 90% of the discharge from the closest orifice.
- This criterion is easy to satisfy as the friction and form losses in the flume are very small compared to the head losses through the orifices.

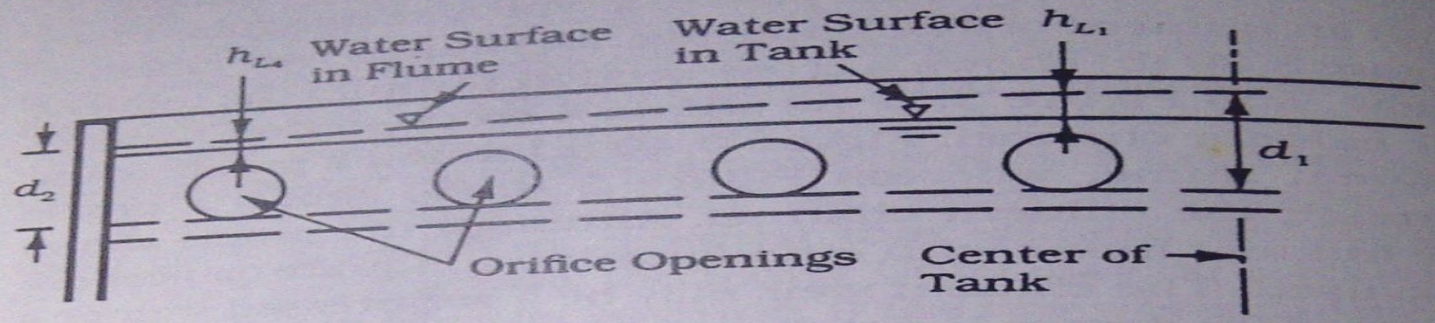
- The discharge form the orifice:

$$Q = 0.6A\sqrt{2gh}$$

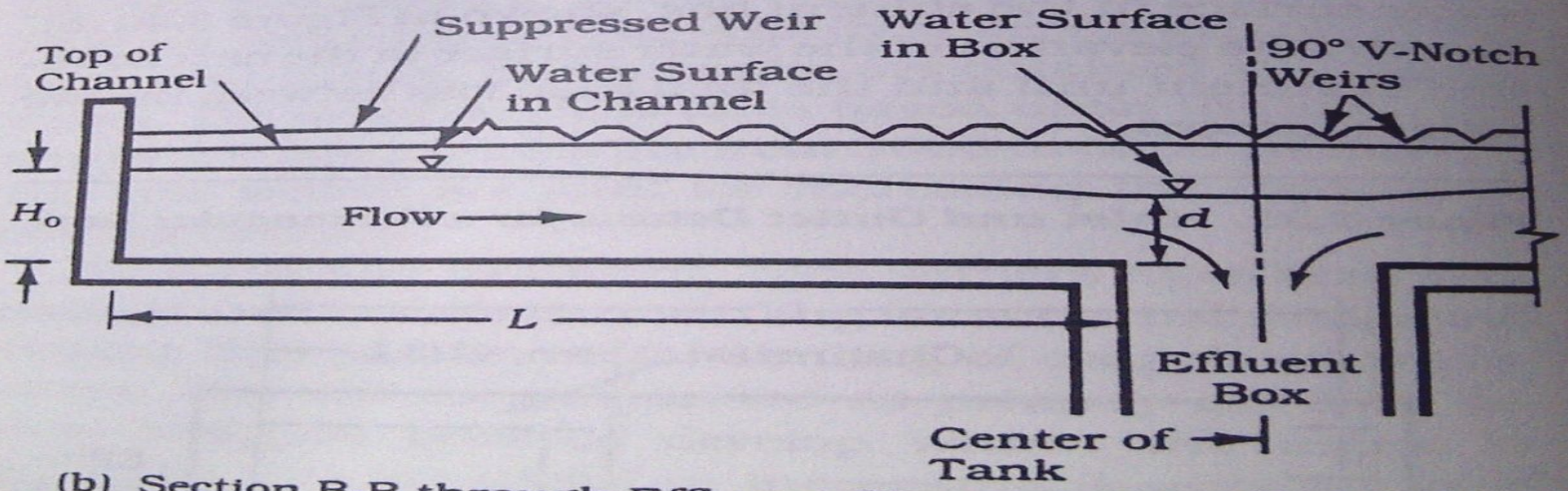
- Q=cfs
- A = orifice area (ft2)
- H = head loss (ft)
- G = acceleration due to gravity

- The velocity in the influent pipe and flume should be sufficient to maintain the suspended solids in suspension
- - not > 3-4 fps to avoid unnecessary head losses.
- If the influent is coagulated water low velocity should be used to avoid shearing the floc.

Figure 3.40. Sections through Orifice Flume and Effluent Channel



(a) Section A-A through Tank Showing Orifice Flume



(b) Section B-B through Effluent Channel and Effluent Box

TLB-305
DISAIN PENGOLAHAN FISIKA KIMIA 2

PERTEMUAN KETUJUH
FILTRASI 1

RACHMAWATI S. DJ

SEMESTER GANJIL 2024/2025

ISI

1. Pengertian
2. Jenis-jenis filter
3. Siklus penyaringan
4. F*ilter headloss*
5. Karakteristik filter
6. Sistem *underdrain*
7. *Filter backwashing*
8. *Hydraulics of filtration*

1. Pengertian

FILTRATION

- Is a solid-liquid separation in which the liquid passes through a porous medium or other porous material to remove as much fine suspended solids as possible.

In WTP:

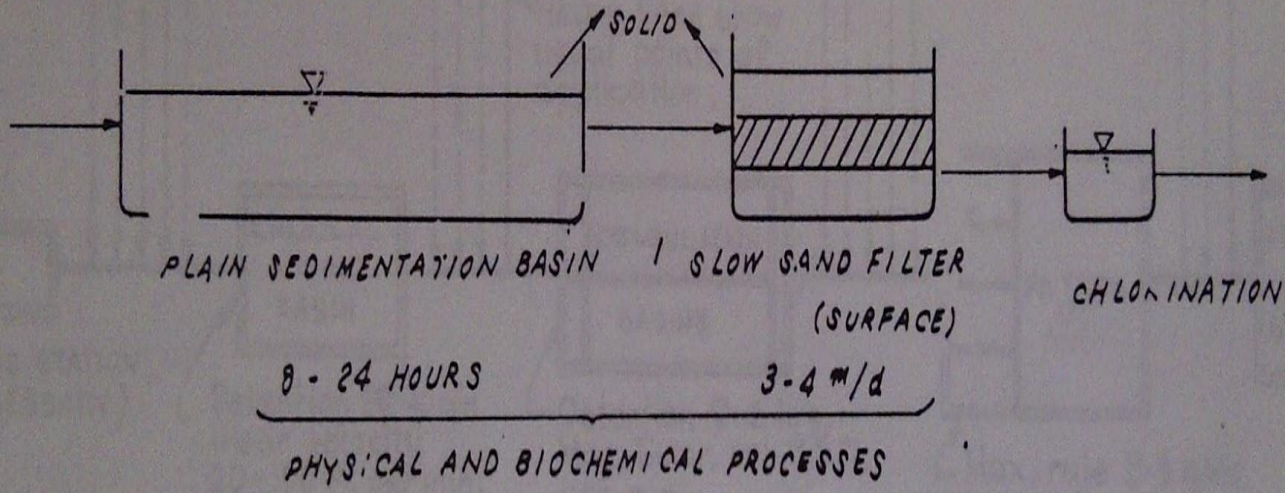
- To filter chemically coagulated and settled waters to produce a high quality drinking water

In WWTP:

- To filter:
  - Untreated secondary effluent
  - Chemically treated secondary effluent
  - Chemically treated raw wastewaters

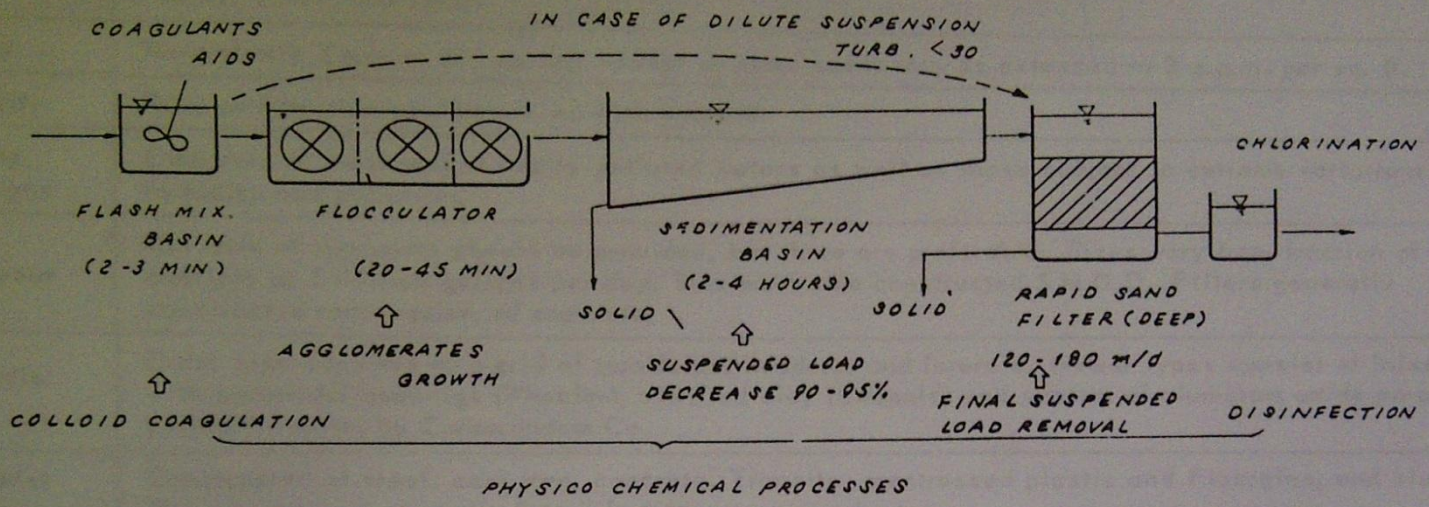
- to produce a high quality effluent

Slow sand filtration system .....



* Mechanism : Coarse suspended matter is removed in the plain sedimentation basin and colloidal (organic and inorganic) matters are arrested on the biological surface filter layer (Schmutzdecke).

Rapid sand filtration system .....



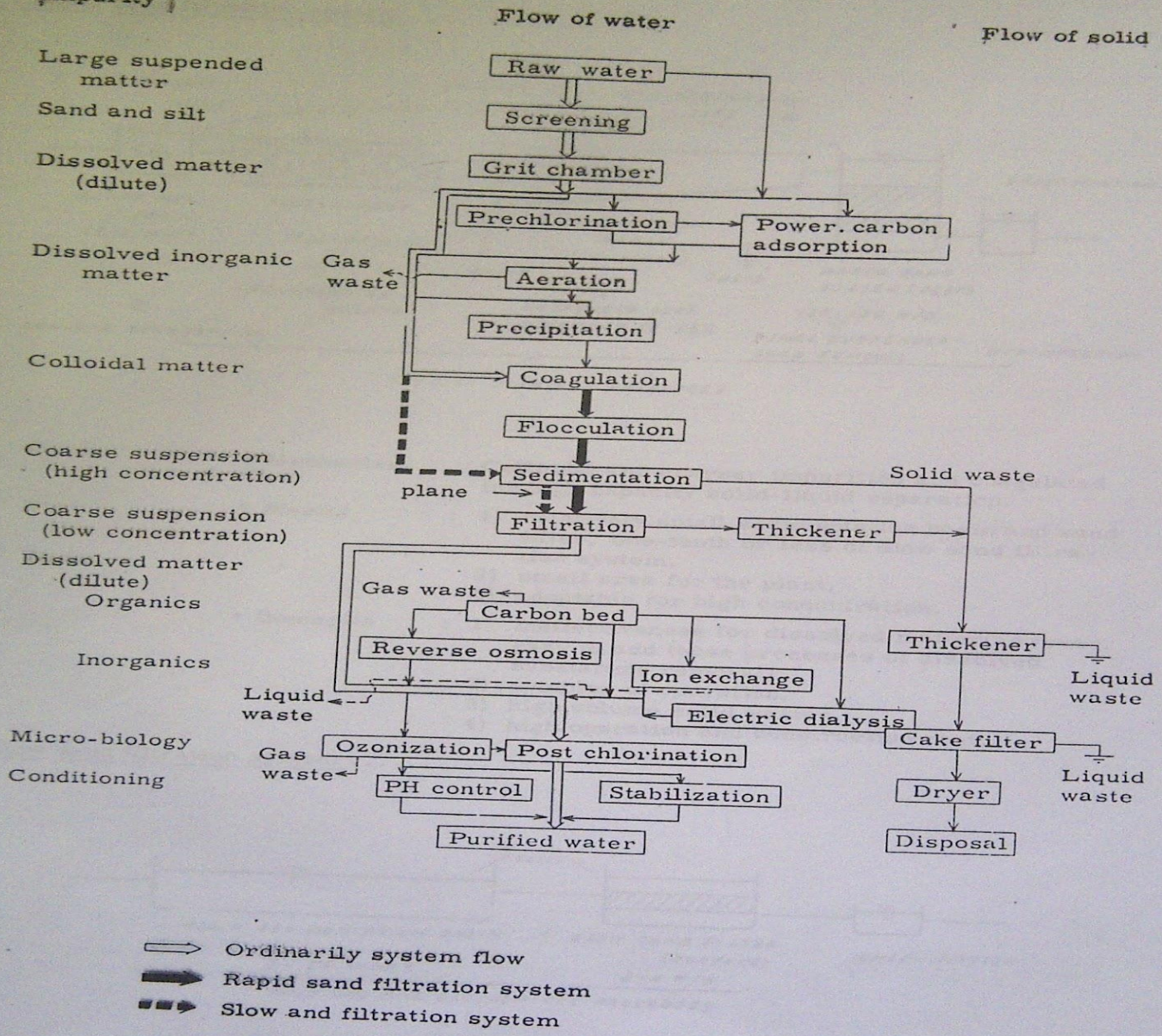
* Mechanism : Colloidal and coarser impurities are coagulated for high capacity solid-liquid separation.

* Merits :
1) relatively small sedimentation basin and sand filter, one-tenth or less of slow sand filtration system.
2) small area for the plant,
3) adoptable for high concentration.

* Demerits :
1) ineffectiveness for dissolved matter(however, easy to add those processes of dissolved substances removal),
2) complex of operation,
3) high volume solid wastes,
4) high operation and construction cost.

Slow sand filtration system

A flow sheet of purification system .....



2. Jenis-jenis filter

classification

- Single-medium filters:
  - One type medium:
    - Sand, or
    - Crushed anthracite coal
- Dual media filters:
  - two types of media:
    - Crushed anthracite and sand
- Multimedia filters:
  - three types of media:
    - Crushed anthracite, sand and garnet

In WTP:

- All three types are used
- Popular: dual & multi media filters

In advanced & tertiary WWTP:

 Dual or multi media types

Both in WTP & WWTP:

- The principles
- Filter structures, equipment, accessories & method of operation

- similar

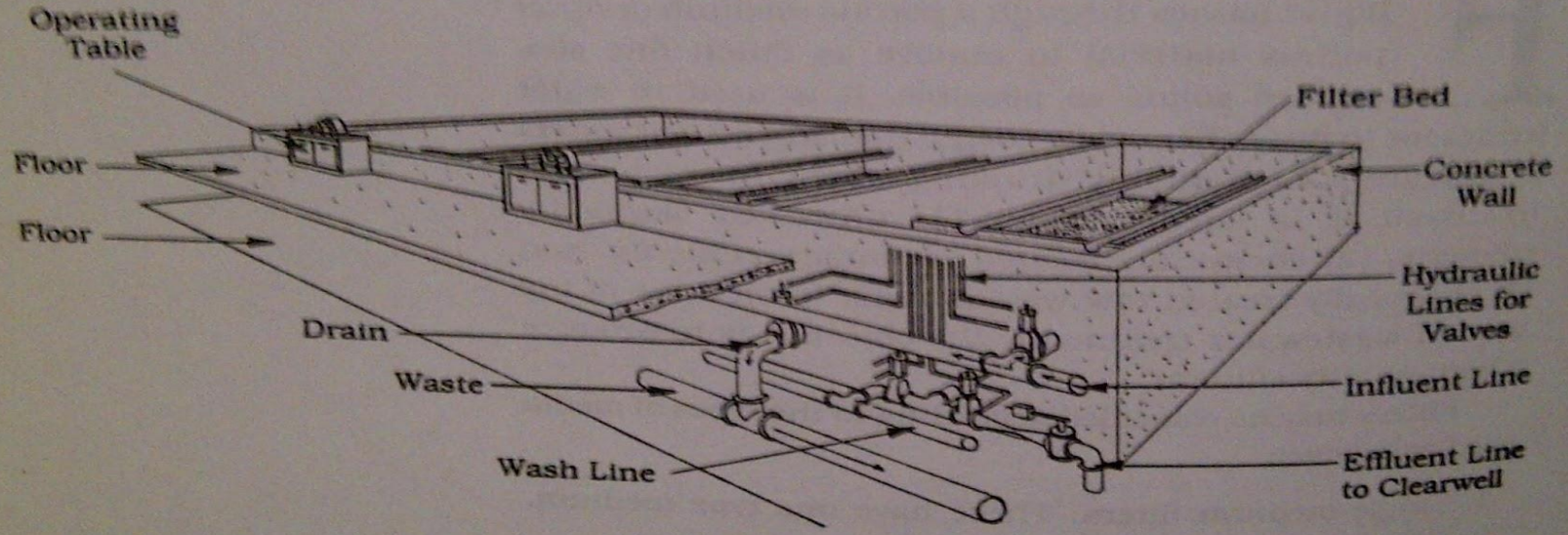
Single medium filter

RSF in WTP:

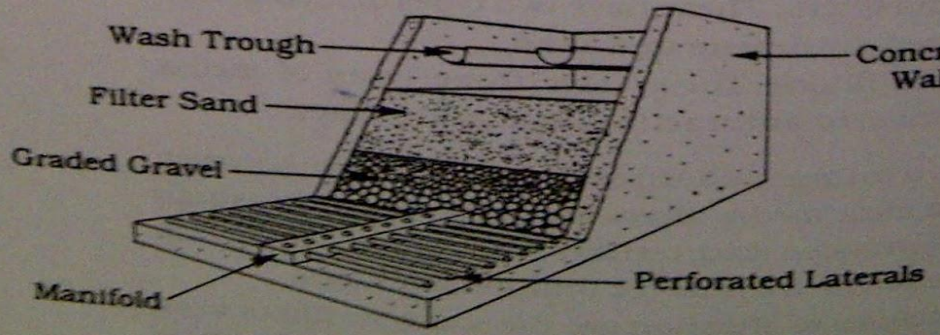
- Gravity type
- Housed in open concrete basins

- Fig 4.1 – 4.8
- The most common: open gravity filters
- □ pressured filters are also used (Fig 4.5 -4.7)

Figure 4.1. Gravity Filters and Accessories
Courtesy of the National Lime Association.



(a) Perspective of a Battery of Filters



(b) Perspective through a Filter

Figure 4.2. Section through a Rapid Sand Filter

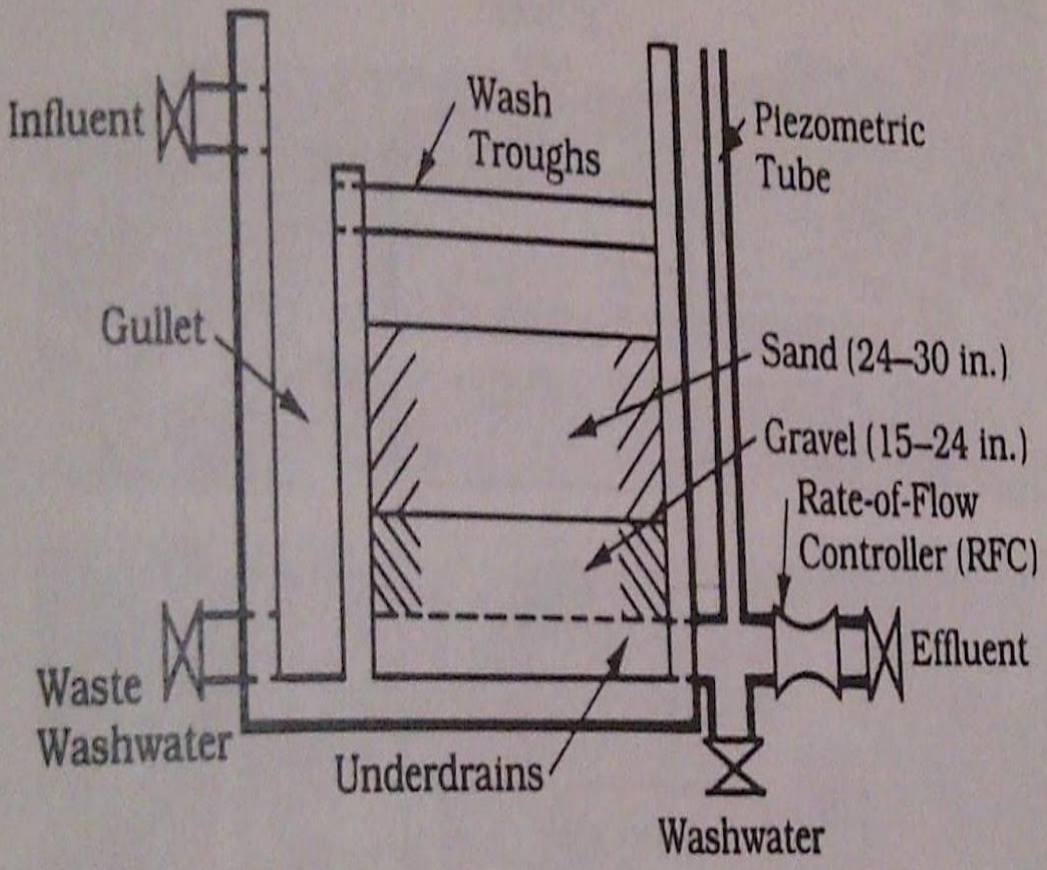


Figure 4.3. Wash System Layout

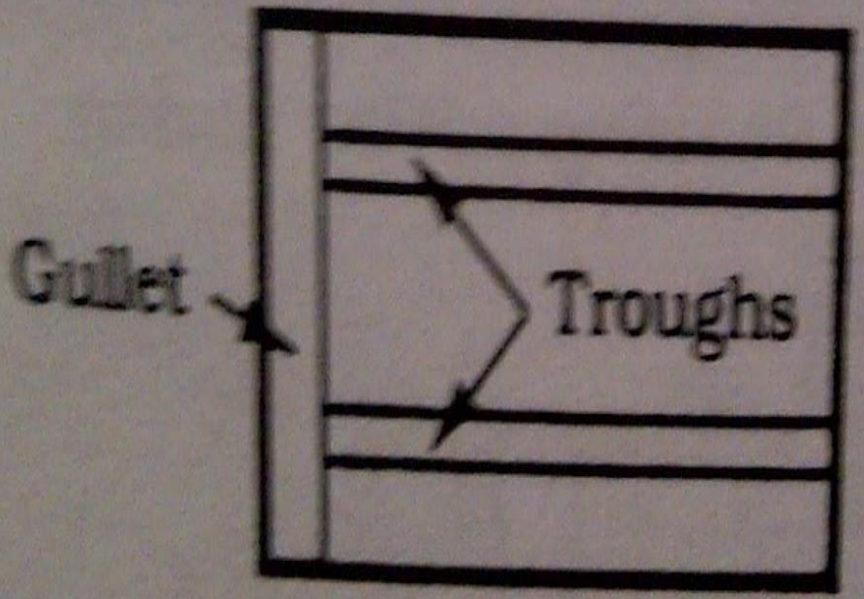


Figure 4.4. Filter Piping Layout

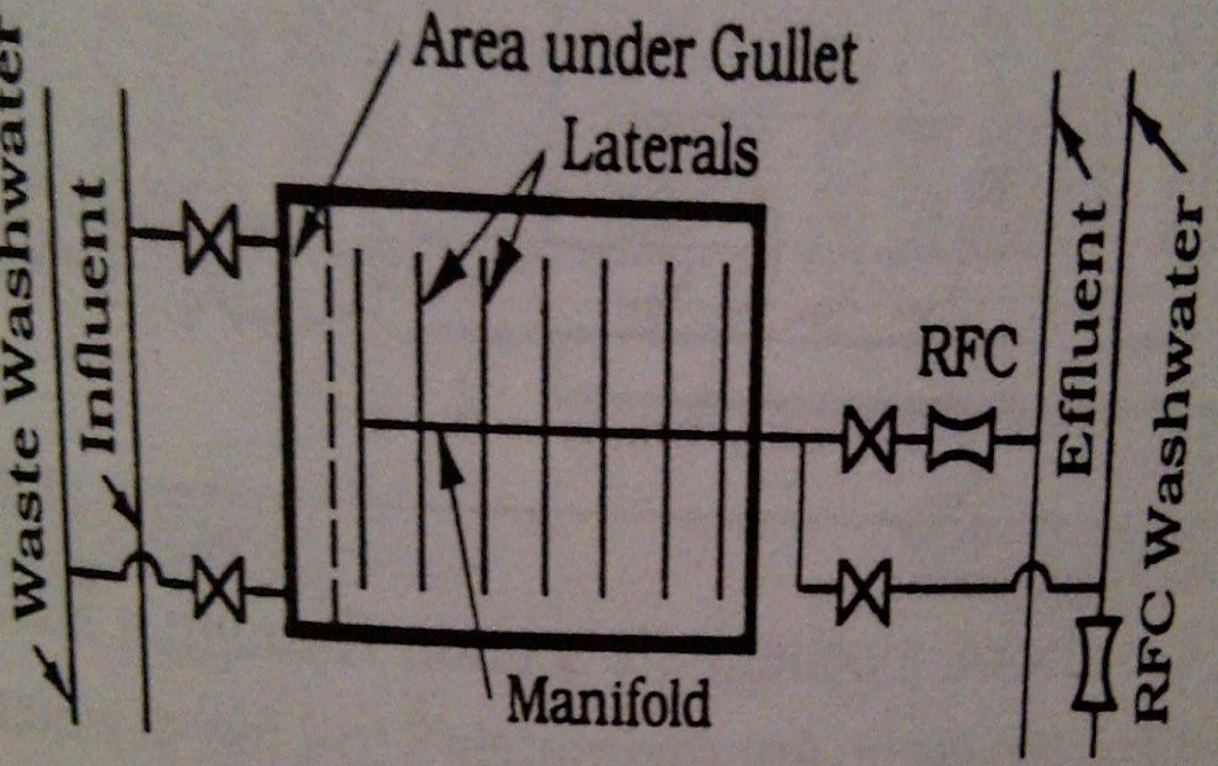


Figure 4.5. Profile through a Horizontal Pressure Filter

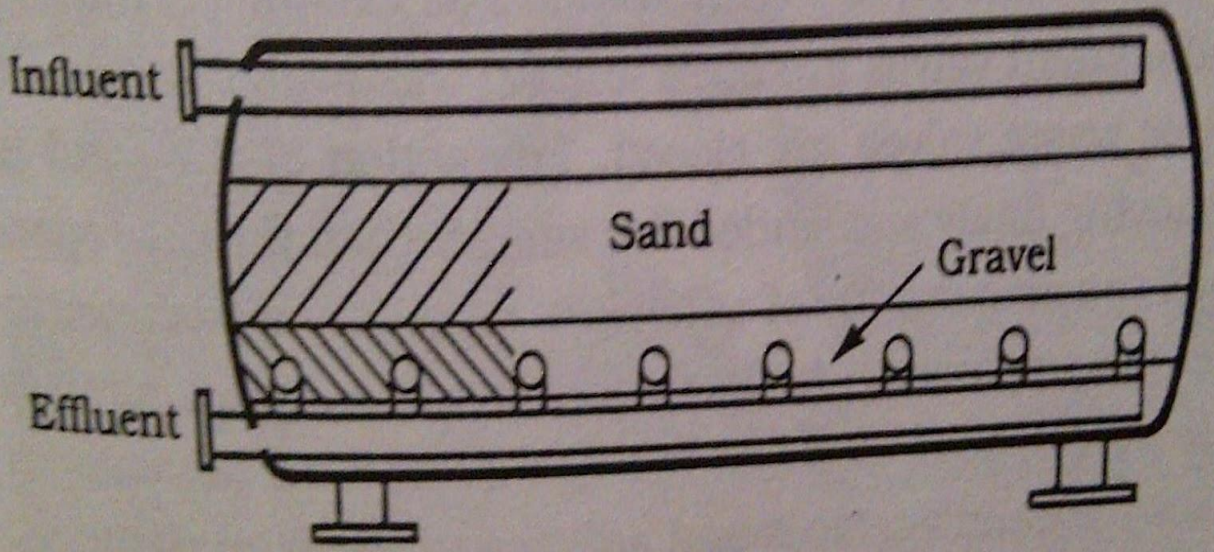


Figure 4.6. Cross Section through a Horizontal Pressure Filter

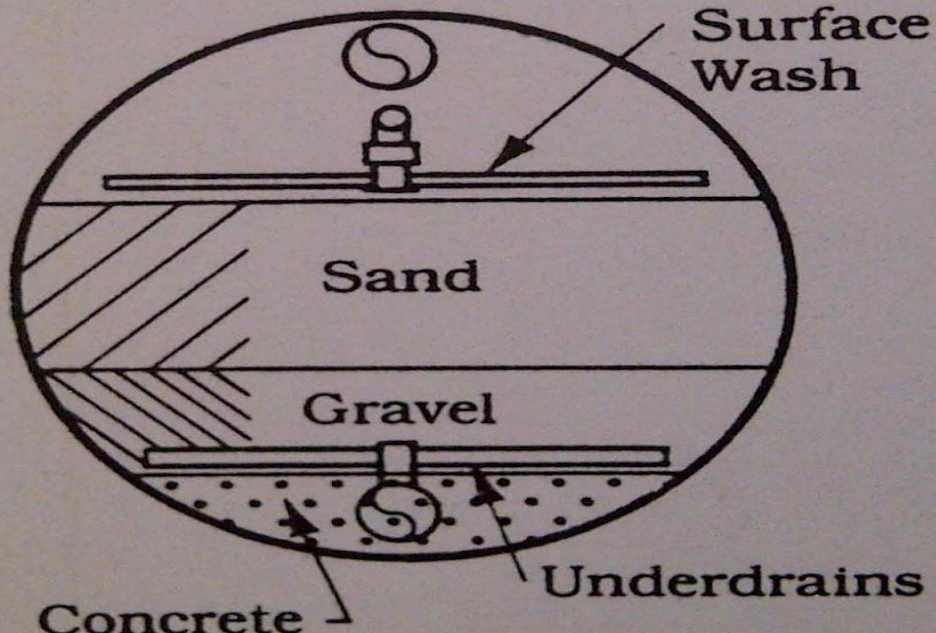
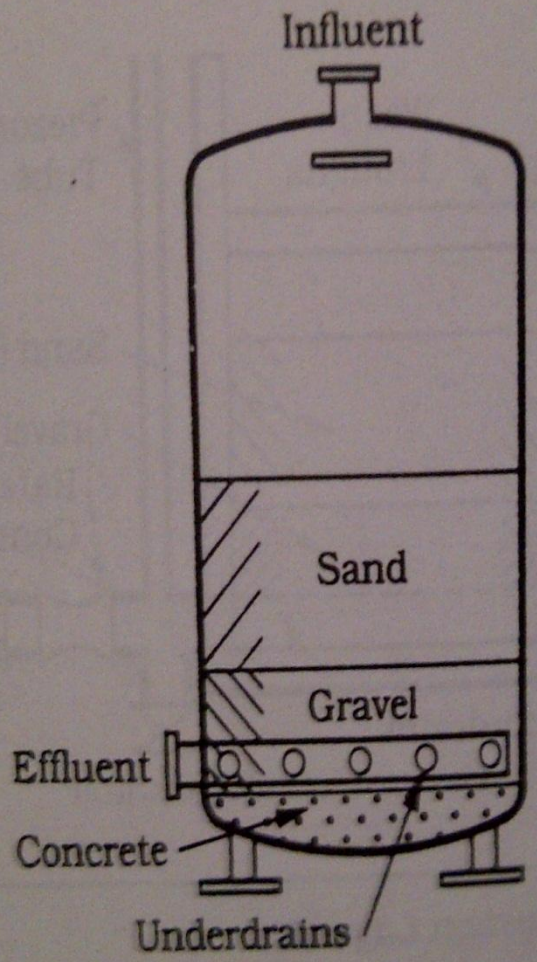


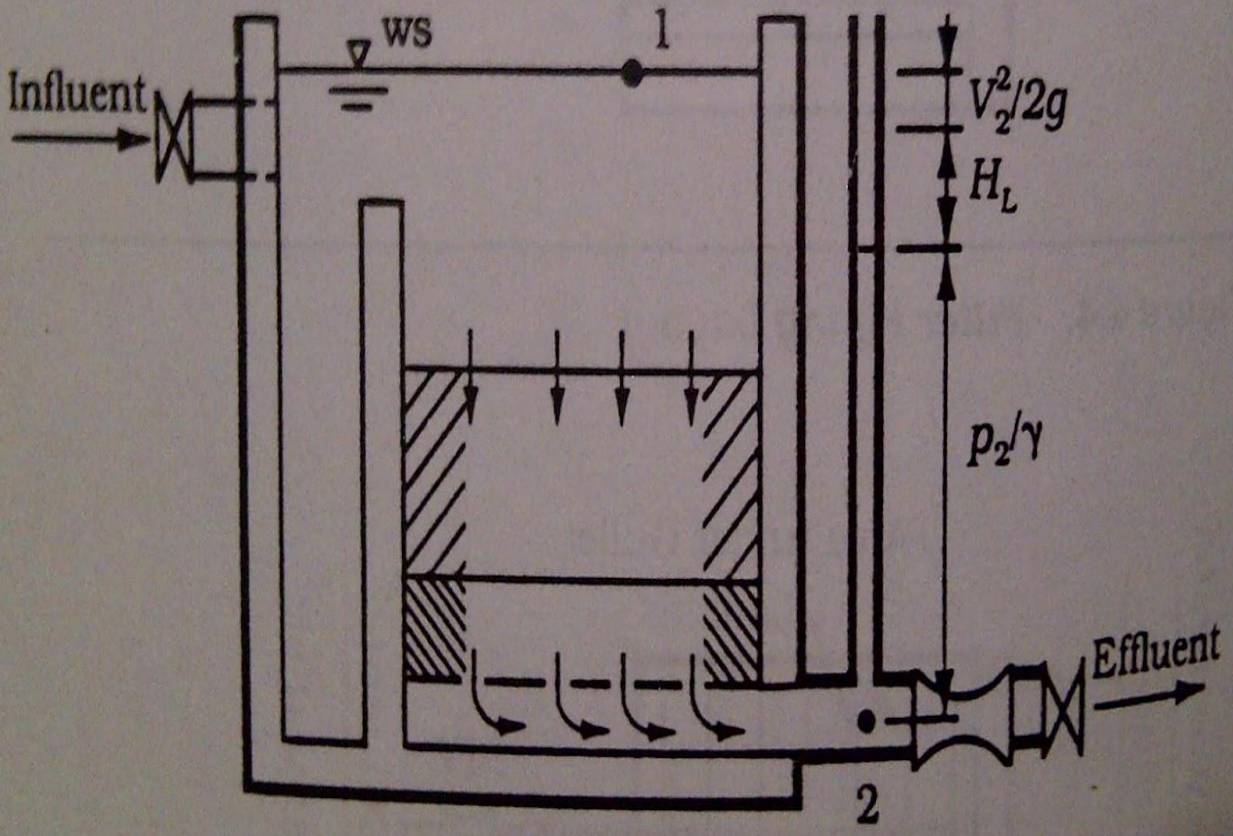
Figure 4.7. Cross Section through a Vertical Pressure Filter



- Frequently crushed anthracite coal is used
- Sand bed depth: 24-30 inches
- Underlaying gravel: 15-24 inches

- Fig 4.8: filtration cycle

Figure 4.8. Schematic Section Showing a Filter during Filtration



3. Siklus penyaringan

During filtration cycle

- Approximately 3-4 ft of water is above the sand.
- The water passes downward through the media into the underdrain system.
- Then it flows through the rate of flow controller which controls the rate at which the water is filtered.

During filtration cycle

The valves position (Fig 4.4):

- Influent & effluent valves are open
- Wash water & wash water waste valves are closed

The action of the sand in removing finely suspended floc smaller than the pore openings consists mainly of:

- Adhesion
- Flocculation
- Sedimentation
- straining

- As the water moves downward through the pore spaces, some of the fine suspended flocs collides with the sand surfaces and **adheres** to the sand particles.
- As the water passes through pore constrictions, some of the fine floc is brought together, **flocculation** occurs, and the enlarged floc **settles** on the top of the sand particles immediately below the constrictions.

- The build up of floc that has been removed in the filter creates a straining action and some of the incoming floc is removed by **straining**.

- During a filter run:
  - The accumulated floc causes the pore spaces to become smaller
  - The velocities to increase
  - Some of the removed floc to be carried deeper within the filter bed.

- Straining may also occur at the surface of the filter if large particles of floc are strained and form a compressible cake that assists in filtering smaller particles.

To sum up:

- The removal of SS is by:
  - Surface removal at the top of the bed
  - And depth removal within the filter bed itself

- For WTP:
- RSF depth removal is the most important

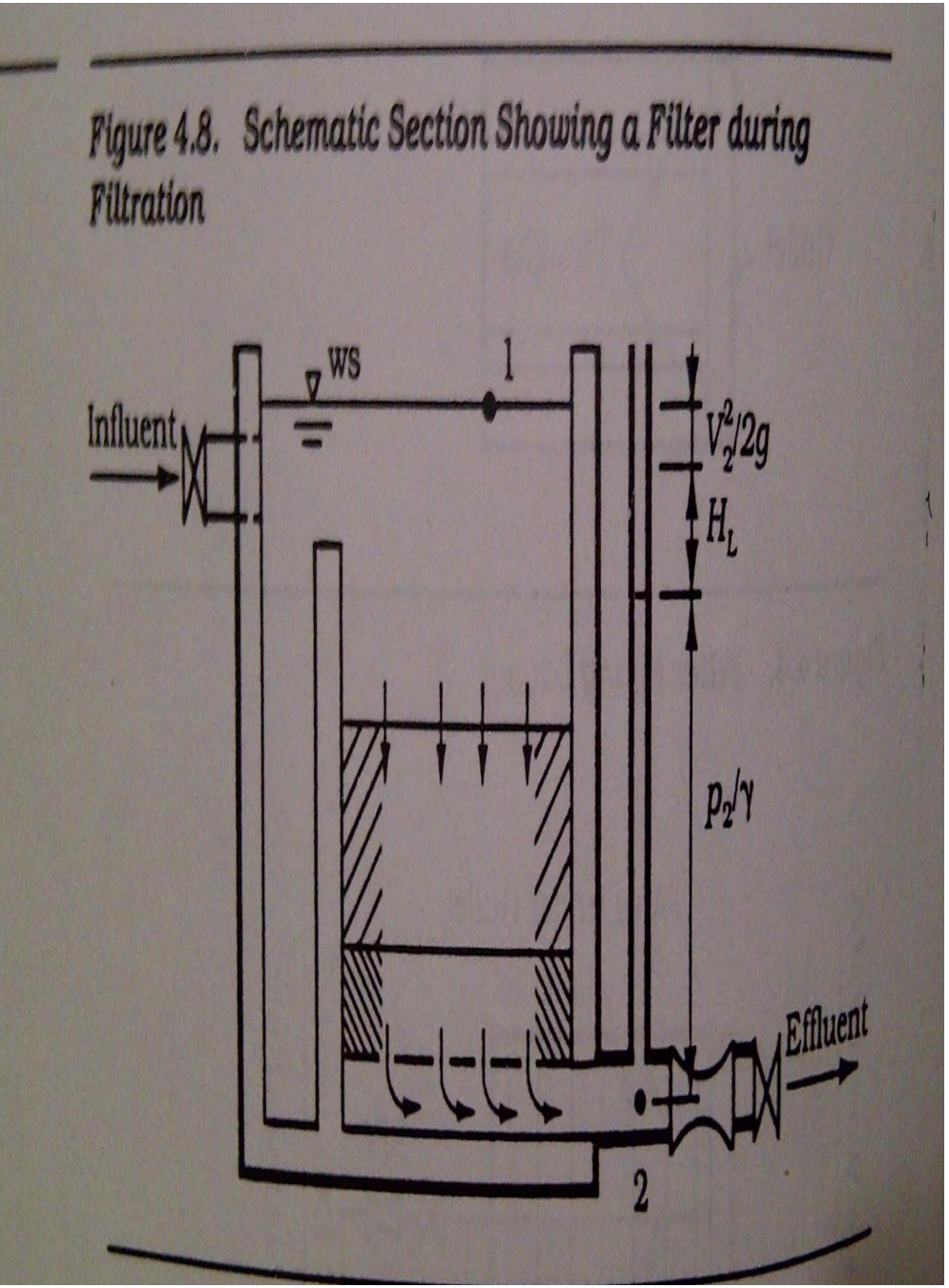
4. Headloss filter

When a clean filter is put into operation:

- The floc accumulation is in the first few inches of the sand

As the time of operation increases

- The floc accumulation extends deeper into the filter bed.



- The accumulated floc causes an increase in the hydraulic head loss.
- The magnitude of the head loss, HL is illustrated by writing Bernoulli's energy equation between point 1 on the water surface (Fig 4.8) and point 2 at the center of the effluent line.

Bernoulli's energy equation

- (4.1):

$$\frac{V_1^2}{2g} + \frac{p_1}{\gamma} + z_1 = \frac{V_2^2}{2g} + \frac{p_2}{\gamma} + z_2 + H_L$$

- V1, V2 = respective velocities
- P1,P2 = respective pressures
- z1, z2 = respective elevation heads
- γ = specific weight of water
- g = acceleration due to gravity
- HL = head loss in ft

- If the relative pressure is used, p1 = 0.
- Also v1 = 0, and
- The datumn may be selected so that z2 = 0.
- Incorporating these values in Eq 4.1 and rearranging gives (4.2)

$$\frac{p_2}{\gamma} = z_1 - \frac{V_2^2}{2g} - H_L$$

Since it is common to have:

- 4 ft of water over the sand
- 2 ft 6 inch of sand
- 1 ft 6 inch of gravel
- 1 ft depth for the underdrains

- z1 = 4 + 2.5 + 1.5 +1 = 9 ft

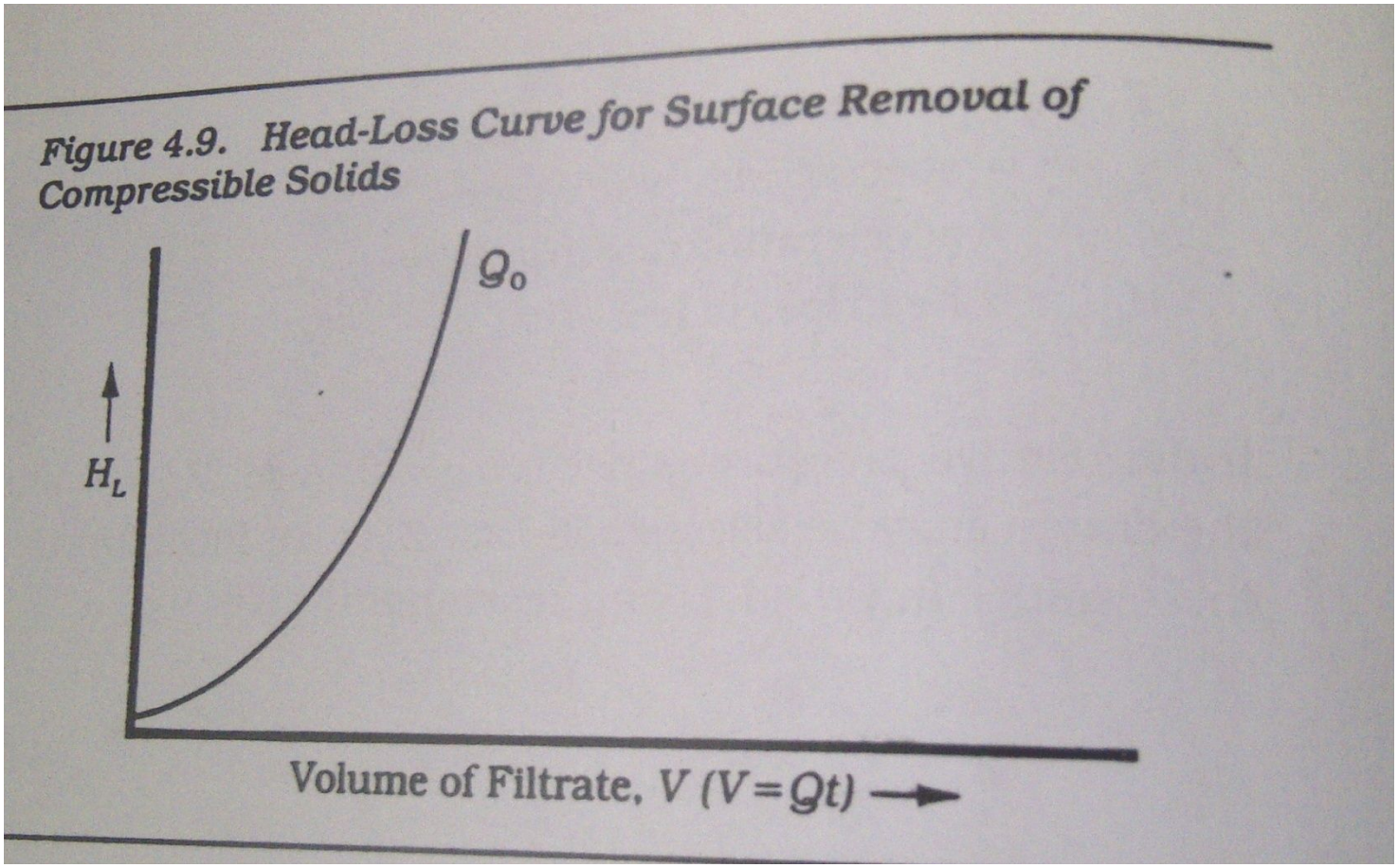
- And pipes are usually designed for= 4 fps

- Thus Eq 4.2 yields an expression for the gauge pressure head at point 2 (4.3):

$$\frac{p_2}{\gamma} = 9\,ft - \frac{4^2}{2g} - H_L = 8.75\,ft - H_L$$

- When a clean filter is put on line, the HL = 0.5 -1.5 ft depending upon the filtration rate,
- But as the filter run progresses, the HL increases.
- Eq 4.3:
- Once the HL = 8.75 ft - P2 = 0
- HL >>> - a negative pressure - undesirable

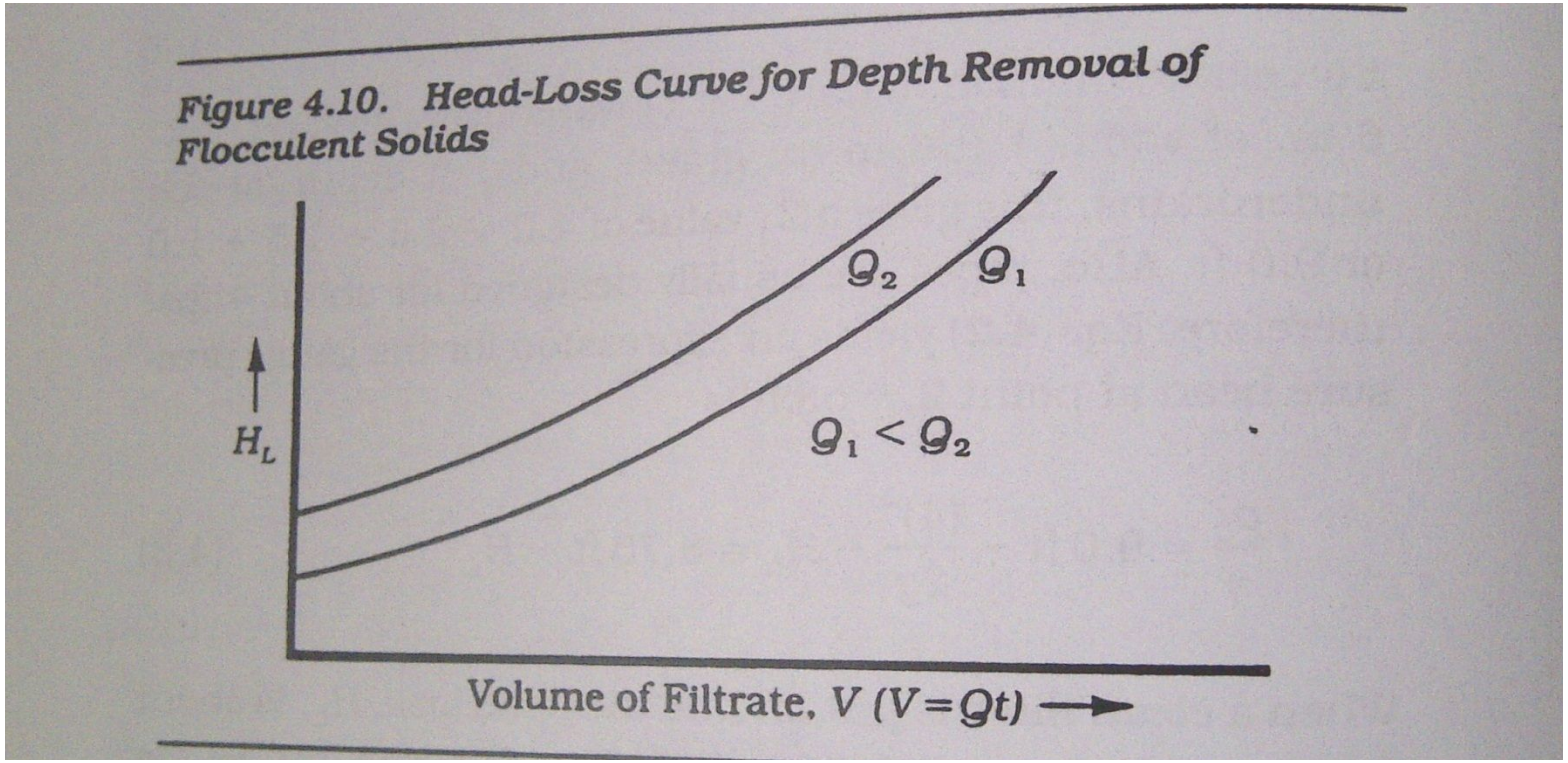
- In practice: once HL = 6-8 ft - □ back washed
- Brp pun kriteria (tinggi air di atas filter, h sand, gravel n underdrain), backwash hrs dilakukan ktk HLnya = 75%
- The amount of washwater= 1-5% of the filtered water (typical value=2-3%)
- The shape of the curve showing the HL as a function of filtrate volume for a particular filter is dependent upon the type of filter action (Fig 4.9-4.11)



If the filter action is by surface removal of compressible solids, the HL curve will be exponential.

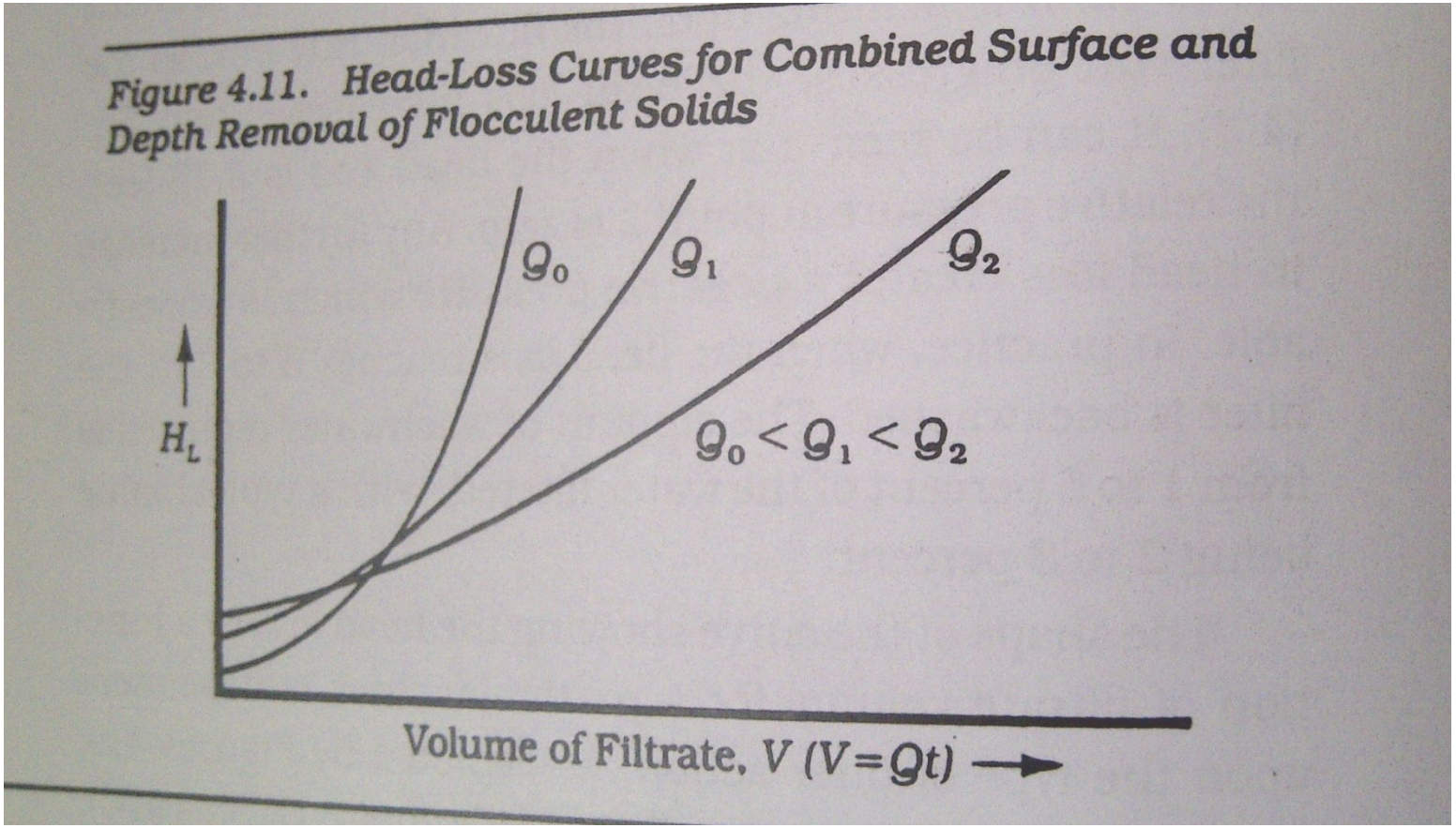
For: fine grained media and low filtration rates

Ex: micro-screens and diatomaceous earth filters



If the filter action is depth removal of flocculated suspended solids, the HL curve will be rather flat.

For: deep granular filters at relatively high filtration rate



If the filter action is by surface removal and depth removal of flocculated suspended solids, the HL curves will be as shown above.

- At low filtration rates, surface removal is predominant, the curve is similar to Fig 4.9.
- At higher filtration rates, the solids penetrate deep within the filter. The principal action is depth removal, the curves are similar to Fig 4.10.

- In RSF, at usual filtration rates:
  - □ depth removal □the main filter action
  - □ a flat HL curve

Surface removal

Will result when the feed water contains:

- Large floc, and
- High turbidity

The top pores will rapidly become clogged

- short filter runs

Depth removal

Will result when the feed water contains:

- small floc, and
- low turbidity

Deeper penetration within the filter bed

- long filter runs

5. Karakteristik filter

Characterization of filter sand

1. The ES:

The sieve size in mm that will pass 10% (by weight) of the sand

2. The UC:

The sieve size passing 60% of the sand divided by that size passing 10%

UC & ES

- Most RSF have sand with an ES = 0.35-50 mm.
- Some have sand with an ES = 0.70 mm
- The UC (= a measure of gradation) = 1.3 <UC< 1.7.

gravel

- Serves to support the sand bed
- Usually is placed in several layers
- Total depth = 6-24 inches (typical=18 inches)
- The size of :
  - the top layer of gravel depends on the sand size
  - The bottom gravel depends on the type of the underdrain system

6. Sistem underdrain

Underdrain system

- Serves to:
  - collect the filtered water from the bed during the filtration cycle.
  - Distribute the backwash water during the washing cycle
- The rate of flow of the backwash >> the filtration time The rate of flow of the backwash governs the hydraulic design of the filter

Underdrain system

- Two types:

1. A manifold with perforated lateral pipes
2. A false bottom

the manifold with perforated lateral pipe system

- the perforations are directed downward
- so that the high velocity of the backwash water is dissipated by:
  - the filter water bottom and
  - the surrounding gravel.

The false bottom

- Consists of a perforated bottom
- With a water way underneath:
  - that removes the filtered water
  - And permits the backwash to enter the filter bed

The standard rate of filtration

- 2 gal/min-ft2 of filter bed area
- This is a common rate at which the first RSF were operated
- Present coagulation & sedimentation practice allows the use of higher filtration rates
- Frequently, plants are rated at 2 gal/min-ft2
- But, provisions are made for operation at rates up to 5 gal/min-ft2

- Most filters are operated at a constant filtration rate
- But a declining rate of filtration is sometimes employed

a declining rate of filtration

- The rate of filtration is decreased:
  - as the filter run progresses
  - And the degree of clogging increases
- Result in:

 longer filtration runs

 Better effluent quality

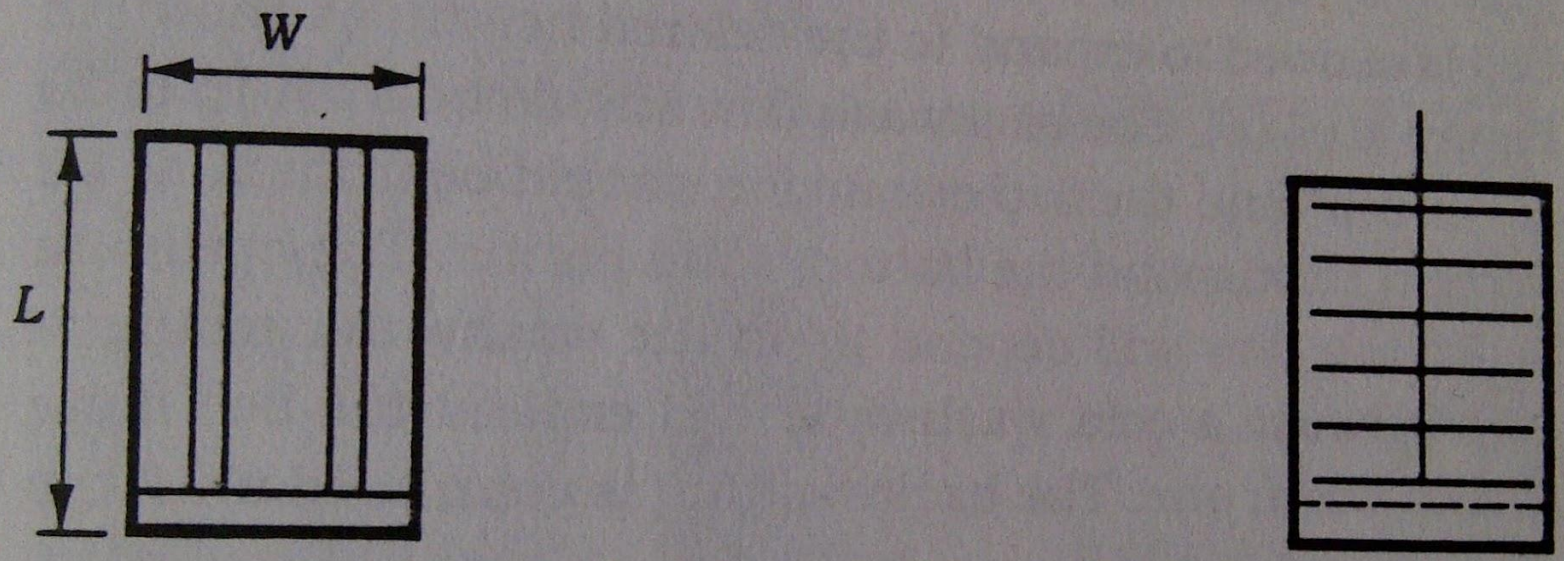
- It is limited to medium to large plants because the filters must be staggered in the degree of clogging to permit a constant rate in the total water production from the plant.

Gravity filters

- A single filter (fig 4.12) or
- A double filter within each concrete basin

(Fig 4.13)

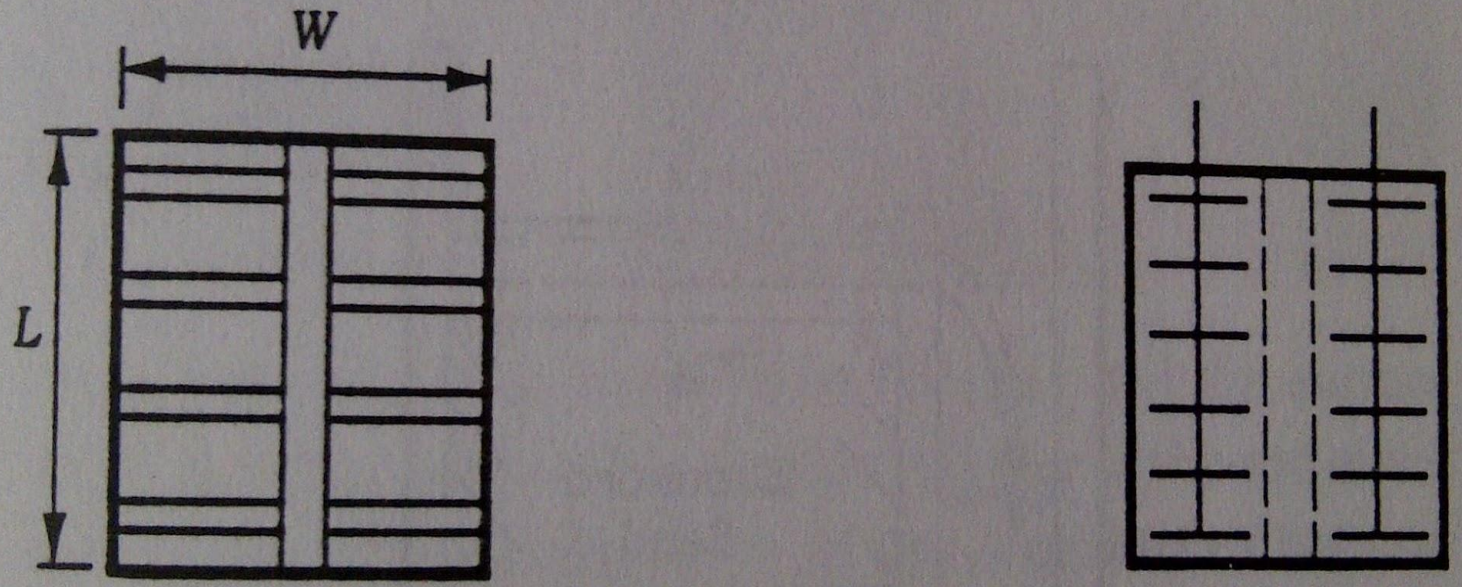
Figure 4.12. Layout and Underdrains for a Single Filter



most popular

The length to width ratio = 1:1.5 to 1:2

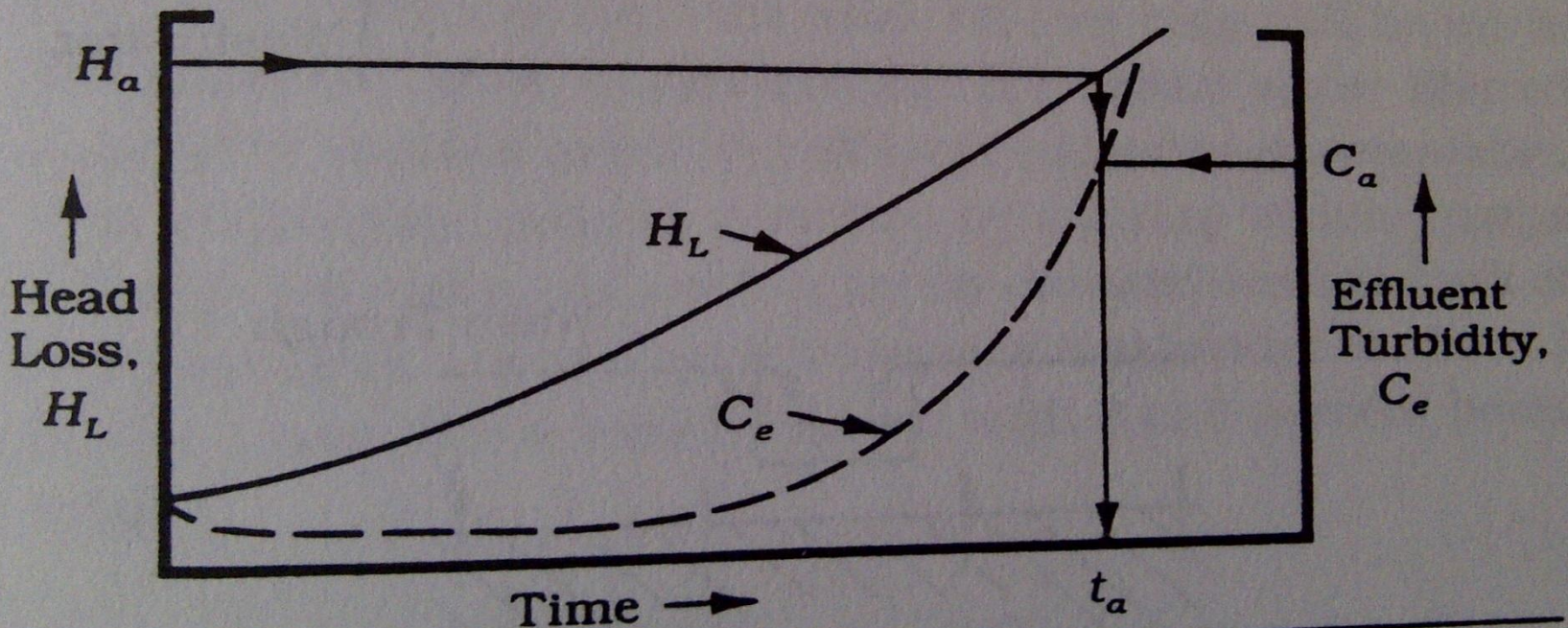
Figure 4.13. Layout and Underdrains for a Double Filter



Almost square

Length to width ratio = 1:1

Figure 4.14. Head Loss and Effluent Turbidity versus Filter Run Time for Optimum Performance



If the filter operation is optimum:

The maximum allowable Ha

Occurs simultaneously with

The maximum allowable effluent turbidity Ca

Filter run

- In may cases, this does not occur and the termination of the filter run is controlled by whichever occurs first, Ha or Ca.
- The length of the filter run:
  - Will depend on the quality of the feed water
  - May range from less than a day to several days

7. Filter backwashing

backwashing

- Removes the floc that has accumulated upon and within the filter bed.
- A surface wash or
- Air scour system
- Is considered essential for high filter performance

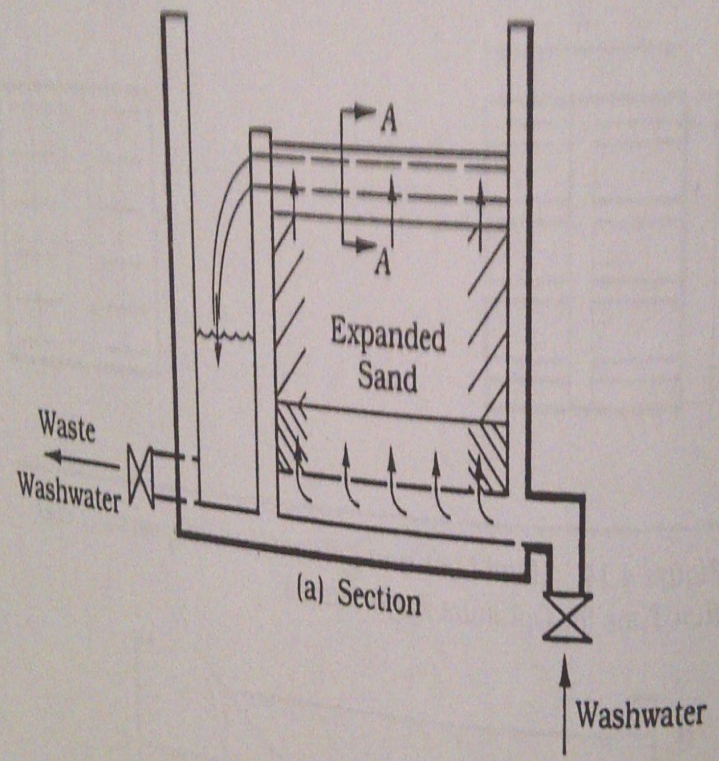
backwashing

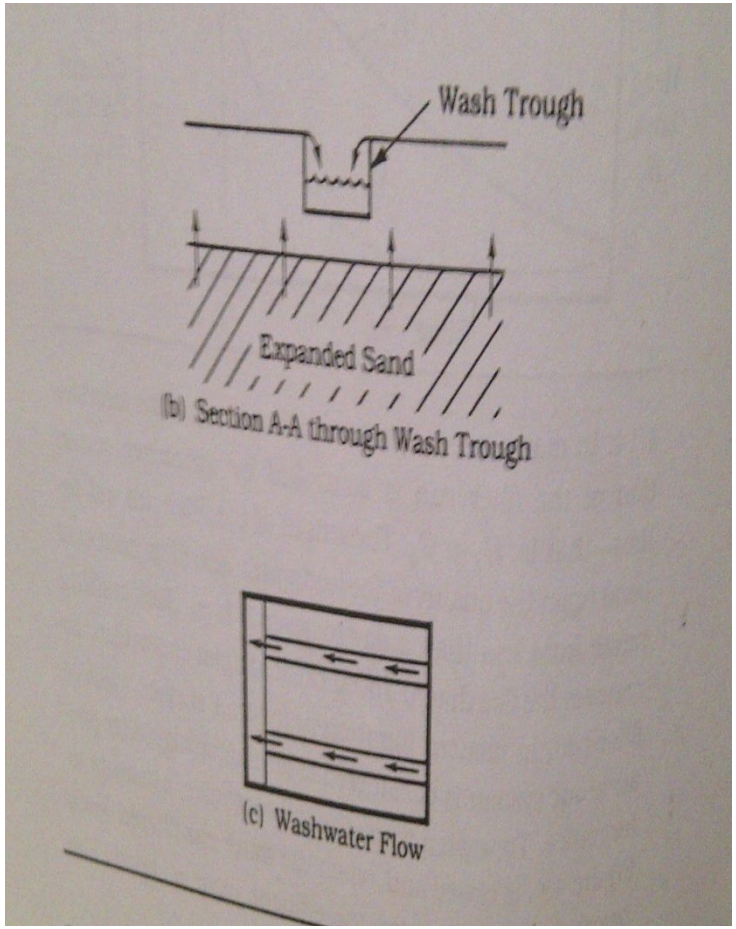
- To wash a filter (Fig 4.4):
- The influent pipe is closed
- And once the water is filtered down below the wash troughs,
- The effluent valve is closed.
- The waste washwater is opened and
- The surface wash is started at a rate= 0.5 gal/min-ft2.

backwashing

- After about 1 min of surface washing
- The backwash flow is initiated by gradually opening the washwater influent valve and
- The bed is allowed to expand to the desired height (Fig 4.15 (a)).
- The backwash flow should be: 15-20 gal/min-ft2
- And the bed expansion should be: 20-50%
- To suspend the bottom sand grains

Figure 4.15. Schematic Showing Filter during Backwashing





backwashing

- The optimum backwash flow will depend on the washwater temperature because a cold washwater will expand the bed more than a warm one.
- The backwashing is continued till the wastewash water appears relatively clear, and
- The surface wash is terminated at 1-2 min prior to the end of the backwash.

backwashing

Surface wash:

- Washes the filter surface prior to the backwashing
- Scours the expanded bed during the backwash

 An auxiliary scour system

Complete washing: 3-10 min of backwash flow

Total off-line time: up to 20min

backwashing

- After backwashing: the initial water filtered should be wasted until the effluent turbidity is acceptable.
- Washwater supply:
- A pump
- An elevated storage tank
- The volume of washwater=1-5% of the water filtered (2-3% typically)

8. Hydraulics of filtration

Hydraulics of filtration

- The HL through a clean bed of porous media having a relatively uniform diameter, as given by the Carman-Kozeny equation,maybe developed starting with the Darcy-Weisbach equation (4.4):

$$h_L = f \frac{LV^2}{D_c 2g}$$

h_L = frictional head loss

f = dimensionless friction factor

L = conduit length

D_c= conduit diameter

V= mean conduit velocity

g = acceleration due to gravity

- The flow channels in a porous bed are irregular, thus the diameter Dc may be replaced by the term 4r, where r is the hydraulic radius for a conduit diameter.
- If D=the bed depth, substituting this an Dc=4r into Eq 4.4 gives (4.5):
-

$$h_L = f \frac{DV^2}{8rg}$$

- If there are n particles in the bed, and
- The particle volume = v_p,
- The total volume of particle = nv_p.
- If the porosity = ε,
- The total bed volume = $nv_p / (1- \varepsilon)$
- The total channel volume = the void space = ε $nv_p / (1- \varepsilon)$

- If the wetted surface is considered as the total surface of the particles = ns_p,
- where s_p=the surface area per particle.
- The hydraulic radius r = the total channel volume divided by the wetted surface or (4.6):

$$r = \left(\frac{\varepsilon}{1-\varepsilon} \right) \frac{v_p}{s_p}$$

- For spherical particles (4.7):

$$\frac{v_p}{s_p} = \frac{\pi d^3 / 6}{\pi d^2} = \frac{d}{6}$$

- For irregularly shaped particles (4.8):

$$\frac{v_p}{s_p} = \phi \frac{d}{6}$$

- Where φ = the shape factor

Shape factors

- Spheres = 1
- Crushed coal and angular sand = 0.73
- Rounded sand = 0.82
- Average sand = 0.75

- The approach velocity, Va = the flow Q divided by the filter surface A.
- Thus, the velocity through the pore spaces (4.9):

$$V = \frac{V_a}{\varepsilon}$$

- Substituting Eq (4.6), (4.8), (4.9) into (4.5), gives (4.10):

$$h_L = f' \frac{D}{\phi d} \frac{1-\varepsilon}{\varepsilon^3} \frac{V_a^2}{g}$$

- □ Carman-Kozeny relationship
- Where: f’ = dimensionless friction factor

- The friction factor f’(4.11):

$$f' = 150\left(\frac{1-\varepsilon}{N_{\mathrm{Re}}}\right)+1.75$$

- The Reynolds number (4.12):

$$N_{\mathrm{Re}} = \frac{\phi d V_a}{v} = \frac{\phi \rho d V_a}{\mu}$$

- Where:
- ρ= mass density
- μ= dynamic viscosity
- v=kinematic viscosity

- The HL through a clean bed of porous media having a relatively uniform diameter is also given by the Rose equation (4.13):

$$h_L = \frac{1.067}{\phi} \frac{C_D}{g} D \frac{V_a^2}{\varepsilon^4} \frac{1}{d}$$

- Where C_D = the coefficient of drag

- For $N_{Re} < 1$ (4.14):

$$C_D = \frac{24}{N_{\mathrm{Re}}}$$

- For $10^4 > N_{Re} > 1$ (4.15):

$$C_D = \frac{24}{N_{\mathrm{Re}}} + \frac{3}{\sqrt{N_{\mathrm{Re}}}} + 0.34$$

- For beds with varying particle size, the Rose equation (4.16):

$$h_L = \frac{1.067}{\phi} \frac{C_D}{g} D \frac{V_a^2}{\varepsilon^4} \sum \frac{x}{d}$$

- Where:

x =the weight fractions for particle sizes, d

- For stratified beds with uniform porosity, the Rose equation (4.17):

$$h_L = \frac{1.067}{\phi}\frac{D}{g}\frac{V_a^2}{\varepsilon^4}\sum\frac{C_D x}{d}$$

- In both equations, the summation terms maybe obtained from computation using sieve analysis.

- The HL through the underdrain system is usually negligible compared to the HL through the bed.
- The Carman-Kozeny & Rose equation are limited to clean filter beds, but they also illustrate the relationship between the HL and the degree of clogging.
- As a filter bed cloggs, the effective porosity ε decreases the HL increases.

9. Soal *Clean Filter*

example

A RSF has a sand bed 24 inch in depth. Specific gravity of the sand = 2.65, shape factor φ = 0.82, porosity ε = 0.45, filtration rate = 2.5 gpm/ft2, and operating temperature 50^0F (10^0C). The sieve analysis of the sand is shown in the table below. Determine the H_L for the clean filter bed using the Rose equation for a stratified bed.

Soal 1

Suatu saringan pasir cepat mempunyai ketebalan 25 inch. Specific gravity pasir = 2.65, shape factor φ = 0.82, porosity ε = 0.45, filtration rate = 2.6 gpm/ft^2, temperatur pada saat beroperasi = 50^{0}F (10^{0}C). Sieve analysis untuk pasir diperlihatkan pada tabel di bawah ini. Tentukan H_L untuk media filter yang masih bersih dengan menggunakan persamaan Rose untuk suatu media yang terstratifikasi.

Table. Sieve analysis

(1) Sieve Size	(2) Weight Retained (%)
14–20	0.87
20–28	8.63
28–32	26.30
32–35	30.10
35–42	20.64
42–48	7.09
48–60	3.19
60–65	2.16
65–100	1.02

Sieve size

Sieve size	d (ft)
14-20	0.003283
20-28	0.002333
28-32	0.001779
32-35	0.001500
35-42	0.001258
42-48	0.001058
48-60	0.000888
60-65	0.000746
65-100	0.000583

No ayakan	Ukuran bukaan (mm)	% kumulatif lolos
140	0,105	0,4
100	0,149	1,2
70	0,210	3,6
50	0,297	9,2
40	0,420	20,8
30	0,590	38,4
20	0,840	58,6
18	1,00	66,2
16	1,19	78,4
12	1,68	92,8
8	2,38	96,2
6	3,36	100,0

No ayakan	Kumulatif Berat Tertahan (%)
40-50	2,4
30-40	37,1
20-30	44,3
18-20	92,9
16-18	100,0

At 10^0C:

- v = 1.3101 centistokes
- 1 centistoke = 10^{-2} cm^2/sec
- 1 centistoke = 1* 1.075 x 10^{-5} sqft/sec

1 cuft = 7.481 gall

1 ft = 12 inch

10. Kuis

TERIMA KASIH

TLB-305
DISAIN PENGOLAHAN FISIKA KIMIA 2

PERTEMUAN KEDELAPAN

FILTRASI 2

RACHMAWATI S. DJ

SEMESTER GANJIL 2024/2025

Hydraulics of expanded bed

- May be analyzed for:
  - Uniform
  - Stratified beds

For a uniform bed

- For a uniform bed of depth, D,
- Backwashing will expand the bed to expanded depth D_e.
- During backwashing, the frictional resistance of the particles equals the head loss of the liquid expanding the bed.
- thus (4.18):

$$h_L \rho g = (\rho_s - \rho)(1 - \varepsilon_e)(D_e)g$$

- ρ_s = mass density of the particles
- ε_e = porosity of the expanded bed

- Cancelling g and rearranging Eq 4.18 gives (4.19):

$$h_L = \left(\frac{\rho_s - \rho}{\rho} \right) (1 - \varepsilon_e)(D_e)$$

- The value of ε (4.20):

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

- V_b = upflow velocity of the backwash water
- V_S = settling velocity of the particles

- Consequently, a bed of uniform particles will expand when (4.21):

$$V_b = V_s \varepsilon_e^{4.5}$$

- The volume of the sand in an unexpanded bed will equal the volume of sand in an expanded bed (4.22):

$$(1-\varepsilon)AD = (1-\varepsilon_e)AD_e$$

- A = bed area

- Rearranging gives (4.23):

$$D_e = \left(\frac{1-\varepsilon}{1-\varepsilon_e} \right) D$$

- Substituting Eq (4.20) for ε_e in Eq (4.23) gives (4.24):

$$D_e = \left[\frac{1-\varepsilon}{1-\left(V_b/V_s\right)^{0.22}} \right] D$$

For stratified beds

- The smaller particles in the upper layer expand first.
- Once V_b is sufficient to fluidize the largest particles, the entire bed will be expanded.
- The expansion of the bed is illustrated by a modification of Eq 4.23 (4.25):

$$D_e = (1-\varepsilon)D\sum\frac{x}{1-\varepsilon_e}$$

- X = the weight fraction of the particles with an expanded porosity, ε_e.

Example

A RSF having the same sand analysis as in the previous example is to be backwashed. Determine:

1. The backwash velocity required to expand the bed.
2. The backwash flow required to expand the bed.
3. The headloss at the beginning of the backwash.
4. The depth of the expanded sand bed.

-
- $V_s = \left[\frac{4}{3}\ \frac{g}{C_D}\ (S_s\ -1)\ d\right]^{1/2}$

Operational problems

- Mud accumulation or mudballs
- Bed shrinkage
- Air binding

Mud accumulation or mudballs

- May occur when:
  - the filter feed contains a muddy floc and
  - The filter is not adequately backwashed
- The muddy floc will accumulate on the surface of the sand bed forming a muddy mat that will penetrate any craks in the top of the sand.
- If a surface wash is not used, some of the mud may be pressed together to form small muddy balls during the backwash.

mudballs

- With subsequent cycles of filtration & backwashing these balls enlarge and become caked with sand and may eventually settle to the gravel layer.
- They interfere with uniform filtration and cause inadequate backwashing.
- May be minimized by the use of surface wash that breaks up any muddy mat formation.

Bed shrinkage

- May occur if the sand grains become covered with a soft slime coating.

 Causes the bed to compact as the filter run progresses, and

 Results in cracks in the bed surface & along the side walls of the filter.

- These cracks are undesirable:

 They may allow improperly filtered water to pass through the bed, and

 Fine muddy floc may accumulate in them to start mudball formation.

- May be minimized by the used of a surface wash system.

Air binding

- Caused by the release of air gases dissolved in the water, i.e. nitrogen & oxygen
- Creating air bubbles in the sand bed
- Results when a filter is operated under a negative head.
- May interfere with the rate of filtration.
- At the beginning of the backwash, the violent agitation due to the rising air bubbles may cause a loss of sand.
- To control avoid negative head or pressures

Multi media filters

- Maybe:
  - Open gravity filters (Fig 4.1) or
  - Pressure filters (Fig 4.5 & 4.7)
- In WTP become more popular
- Dual media:
  - Usually employ Anthracite & sand
  - others Activated carbon & sand

Multi media filters

- Generally use Anthracite, sand & garnet
- Others activated carbon, sand & garnet
- Or dual & multi media filters IE resin as one of the media.
- The media may have additional characteristics other than removing particulates.
- Ex: activated carbon removes dissolved organic substances.

Main advantages of multi media filters

- Compared to single medium filters:
  - Longer filter runs
  - Higher filtration rates
  - The ability to filter a water with high turbidity and SS.
- These advantages are due to:
  - The media particle size
  - The different specific gravities of the media
  - The media gradation
- These result in: a filter with a larger percent of the pore volume being available for solid storage.

The pore available for solids storage

- In the single medium filter: in the top portion of the bed.
- In the multi media filters: is extended deep within the filter bed.
- Due to the deep penetration of accumulated floc deep bed filters.

usage

- Single medium filter are rarely used in WWTP or advanced WWTP due to short filter runs
- Due to the large volume being available for floc storage, multi media filters can be used in advanced or tertiary WWTP and still have a reasonable filter run.

Dual media filters

- Consisting of :
  - A layer of coarse anthracite coal above
  - A layer of fine sand

 To increase the pore volume of a filter.

Pore size profile

- The available pore volume of the dual media filter > the single-medium filter
- The available pore volume, won’t be as large as the total pore volume due to the fine to coarse gradation within each layer.
- Ideally, the available pore volume would be maximum at the top of the filter, and gradually decrease to a minimum at the bottom of the filter.

A dual media filter

- Consists of:
  - An 18-24 inch layer of crushed anthracite coal overlaying
  - A 6-12 inch layer of sand.
- Specific gravity:
  - Coal = 1.2 -1.6
  - Sand = 2.65

A dual media filter

- During the first backwash:

 The sand layer remains below the coal due to:

  - Its higher specific gravity and
  - Its grain size relative to the coal particles

- After the first backwash:

 There won't be a distinct interface between the two layers, but instead

 There will be a blended region of both coal particles & sand grains.

A dual media filter

- The size & characteristics of the anthracite & sand media, and
- The thickness of the layers depend on:
- The usage of the filter:
  - WTP
  - WWTP

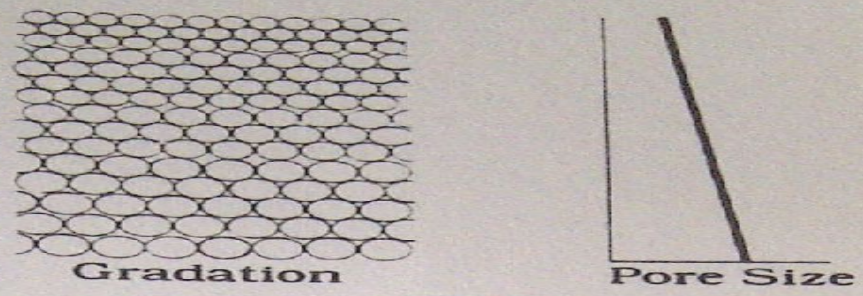
Filtration rate:

- 2 -10 gpm/ft2 or
- 3-6 gpm/ft2

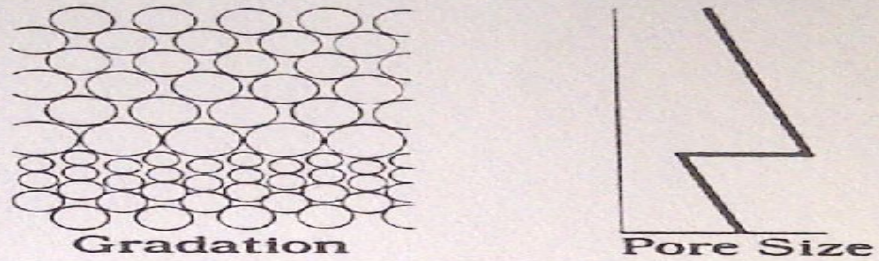
Mixed media filter

- The ideal filter has a pore size and gradation as □ Fig 4.16 (c).
- The pore size is:
  - greatest at the top of the bed,
  - And gradually decreases to a minimum at the bottom.
- The available pore volume (pore space) is:
  - maximum at the top of the bed, and
  - Decreases to a minimum at the bottom.

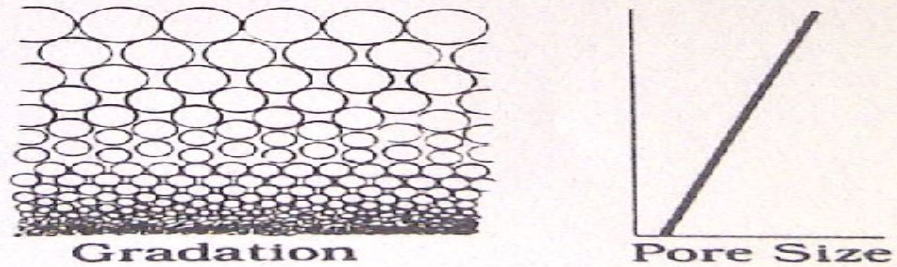
Figure 4.16. Gradation and Pore Size in Various Filters
Courtesy of Neptune Microfloc, Inc.



(a) Single-Medium Filter



(b) Dual-Media Filter



(c) Ideal Filter

- The media have a gradation which is:
  - from coarse at the top to
  - Fine at the bottom.
- The ideal filter maybe approached by using:
- A dual-media filter of:
  - Crushed anthracite coal above sand, and
  - Placing a third very dense medium below the sand

A third medium

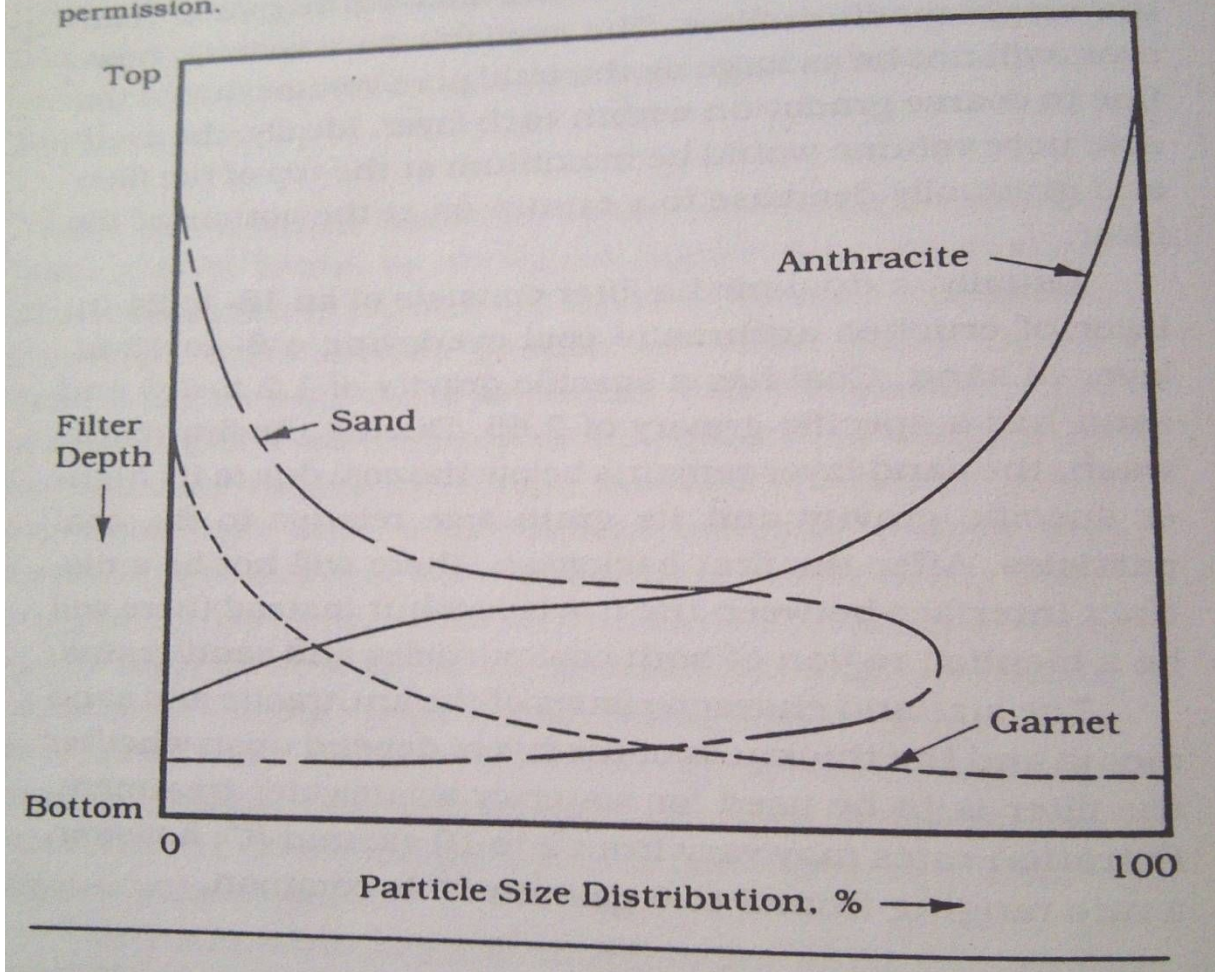
- This allows the third medium to be very fine and still remain in the lower depths during backwashing.
- The resulting filter is referred to as a mixed media filter since there is some intermixing between the layers during backwashing.

Third medium

- Garnet: Ss =4.2
- Illeminite: Ss= 4.5
- The anthracite, sand, and garnet or ilmenite are properly sized to allow some intermixing of the media during backwash.
- After backwash no distinct interface between media layers.
- The filter bed the ideal (Fig 4.16 c) a gradual decrease in pore size with increasing depth

- Since the pore size decreases from the top to the bottom of the filter the filter will have a large available pore volume extending throughout the depth of the filter bed.
- Fig 4.17
- 3 inch of coarse garnet or ilmenite are placed under the third layer to prevent fine particles to enter the underlaying gravel.

Figure 4.17. ***Media Distribution in a Mixed-Media Filter***
Adapted from *Mixed Media*, Bulletin no. KL 4206, by Neptune Microfloc, Inc. Copyright 1975 by Neptune Microfloc, Inc. Reprinted by permission.



- Filter:
  - Size and
  - Characteristics of media
  - The thickness of the layers
- □ depend on the type of service for the filter
- □ WTP or WWTP?
- WTP or advanced WWTP: 2-12 gpm/ft2
- Sometimes: 3-6 gpm/ft2

Filter layout, appurtenances, details

- Gravity filters □reinforced concrete.
- Min no. 2 □ 4 are preferable
- Placed side by side in a row.
- Pipe gallery (contains all the necessary piping, valves etc) runs parallel to the filter row.
- Pipe gallery & operating floor □ always enclosed □ protect personnel & equipment.
- A dehumidified pipe gallery □ reduce maintenance on controls, valves etc.

Pressure filters

- Usually cylindrical
- Prefabricated of steel
- A max diameter = 10-12 ft
- Max length = 60 ft.
- Should be equipped with a sight glass observe the bed during backwashing
- Should have an access manhole for maintenance.

Control systems

- Manual or
- Fully automatic
- Equipment to measure effluent turbidity should be accurate & reliable:
  - For a continuous record of filter performance
  - Assists in filter operation

- Fully automatic systems a programmer activated by:
  - Effluent turbidity
  - Head loss
- Once the effluent turbidity or head loss preset level the progammer takes the filter off line backwash it place it back on line.

A rate of flow controller

- To control the flow of the filter.
- Maintains a uniform flow rate with a constant water depth over the filter depth.
- Uniform flow is maintained by varying the head loss between:
  - the filter bed surface and
  - The downstream side of the rate of flow controller.

A rate of flow controller

- Usually consists of a ventury
  - with a variable opening diaphgram or
  - A butterfly valve on the downstream side
- The valve is activated by the difference in pressure between:
  - The upstream side
  - And the throat of the venturi

A weir

- Another method to control the flow of the filter.
- Consists of:
  - A weir in the inlet to the filter
  - A weir in the effluent channel discharging into the clear well.

The weir

- The weir downstream:
- □ keeps a minimum water depth over the filter bed.
- The weir in the inlet:
- □ maintains a constant rate of flow to the filter
- The filter flow rate is independent of:
- □ the depth of water over the filter bed since the weir crest is above the water surface

The weir

- as the filter run progresses, the depth of water over the filter bed increases
- □ due to the increase in head loss.
- Disadvantage:
- The filter walls must be from 5-6 feet deeper than required when a rate of flow controller is used.

By pumps

- For pressure filter
- Pump at a relatively constant rate to the filters.

Air scouring

- For surface wash
- The water is filtered down to about 6 inches above the bed.
- Air is applied to the underdrain system at 2-5 cfm/ft2 for 3-10 min
- Then the backwash is initiated at 2-5 gpm/ft2

Air scouring

- Once the water level is about 1 feet below the washwater throughs:
- □ the air is stopped
- □ the backwash is operated at the normal rate for the usual period of time.

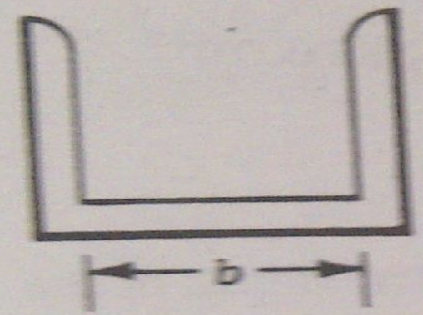
Washwater troughs

- Are usually placed at a clear distance of 5-6 ft from each other.
- Function:
  - To remove the backwash
  - To maintain a uniform backwash, since the distance from the top of all troughs to the underdrains is a constant value.

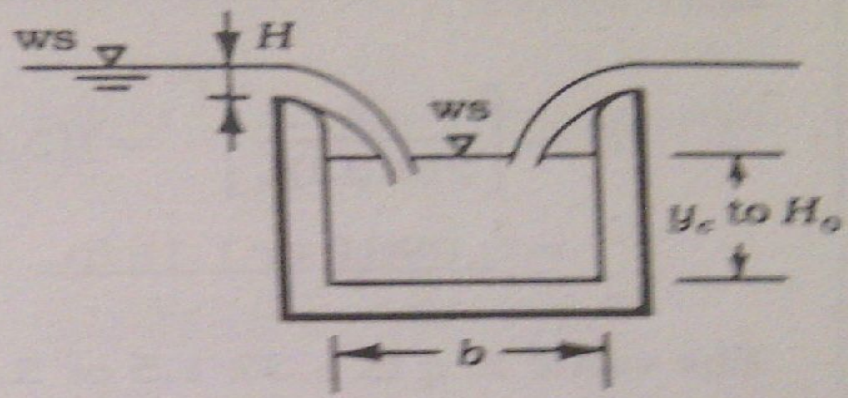
Washwater troughs

- Trough bottoms: min 6 inch above the expanded bed during backwashing.
- Material:
  - (precast) Reinforced concrete
  - Fiberglass
  - Enameled steel
- Rectangular cross section Fig 4.18
- Fig 4.18 b the washwater overflowing into a trough

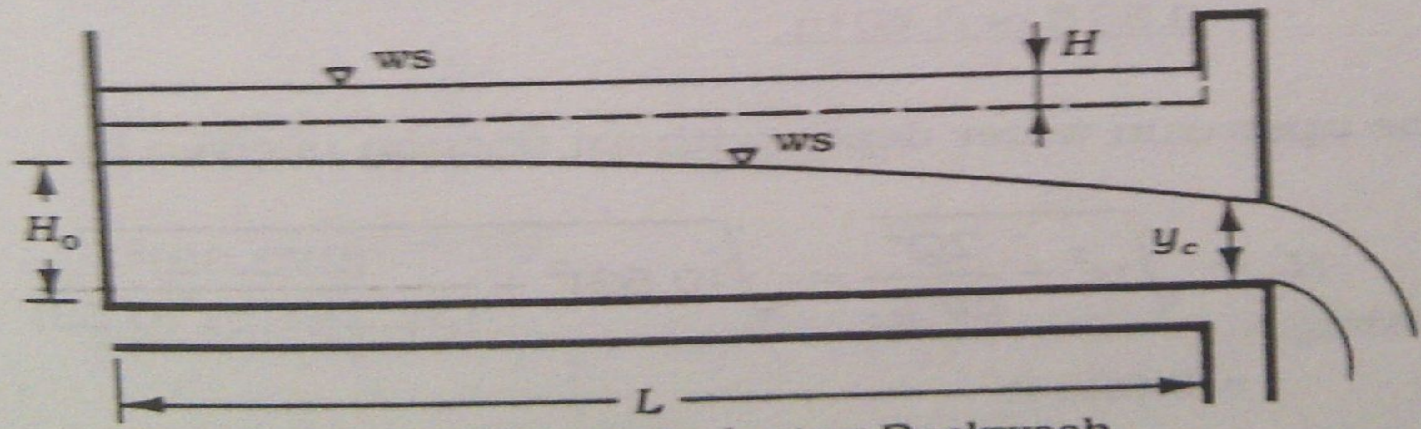
Figure 4.18. Wash Trough Details



(a) Section through Trough



(b) Section through Trough during Backwash



(c) Profile of Trough during Backwash

Total discharge of washwater overflowing into a trough

- Is twice that of a supressed weir (4.26):

$$Q = 2(3.33)LH^{3/2}$$

- Q= total discharge, cfs
- L= trough length, ft
- H= head on weir, ft

Critical depth

- Fig 4.18 c □ profile of a trough during backwashing
- Shortly before the freefall into the gullet, the water is at the critical depth, y_c.
- Critical depth can be assumed at the freefall.

Critical depth

- For a rectangular channel the critical depth (4.27):

$$y_c = \sqrt[3]{\frac{q^2}{g}}$$

- y_c = critical depth, ft
- q = discharge per ft of width, q = Q/b, cfs/ft
- g = acceleration by gravity

Upstream depth

- The upstream depth, H_0, for a level, rectangular, lateral spillway with a freefall and no friction (4.28):

$$H_0 = \sqrt{y_c^2 + \frac{2Q^2}{gb^2 y_c}}$$

- H_0 = upstream depth, ft
- Q = total discharge,cfs
- b = trough width, ft

- The friction loss will increase H_0 by 6-16% of the water surface drawdown.
- A distance of min 3 inch should be provided from the maximum water depth to the top of the trough.

Example

A RSF has washwater troughs that are 16 ft long and spaced 6.20 ft from center to center. The precast concrete troughs are rectangular in cross section with an inside width of 18 inch. The backwash flow is 15 gpm/ft2. It is desired to determine the water depth over the trough sides, H, the depth at the freefall, y_c, the upstream water depth, H_0, and the trough depth if the top of the trough is min 3 inch above the maximum water surface in the trough. Assume the friction loss is 16% of the surface drawdown.

Soal 3

Suatu filter cepat mempunyai washwater troughs yang panjangnya 6.0 m, dan berjarak 2.0 m dari pusat ke pusat washwater troughs. Troughs ini terbuat dari beton, yang bentuknya rectangular jika dipotong secara melintang, dengan lebar dalam sebesar 0.5 m. Aliran backwash = 11 L/det-m^2. Tentukan ketinggian air di atas sisi trough (H), ketinggian air pada saat tinggi jatuh ($y_{c)}$, ketinggian air di hulu (H_0), dan ketinggian air di dalam trough, jika bagian teratas trough minimal sebesar 7.5 cm di atas ketinggian muka air maksimum di dalam trough. Asumsikan *friction loss* = 16% dari penurunan tinggi muka air.

Filtration in water treatment

- SSF first type used in WTP
- PS SSF
- Single medium filter
- ES = 0.2-0.4 mm
- V = 0.05-0.15 gal/min-ft2
- Cleaned manually usually every 4-6 weeks, by:
  - By scraping off the top layers of clogged sand
  - And cleaning the sand with a scouring device

SSF

- Disadvantages:
  - Large land area requirement
  - Manual labour involved
- □ replaced by RSF

RSF

- Always preceded by:
  - Coagulation
  - Flocculation
  - Sedimentation
- First RSF:
  - 2 gal/min-ft2
  - A quartz sand bed
  - Overlaying a gravel layer

RSF

- Turbidity removals= 90-98%
- Standard rate= 2gpm/ft2
- Most:
  - operated at 3-5 gpm/ft2
  - Coarse sand beds
- Primary action: depth removal
- Table 4.3

Table 4.3. Single-Medium Filter Characteristics for Water Treatment

Characteristic	Value	
	Range	Typical
Sand Medium:		
Depth, in.	24–30	27
Effective size, mm	0.35–0.70	0.60
Uniformity coefficient	<1.7	<1.7
Filtration rate, gpm/ft^2	2–5	4
Anthracite Medium:		
Depth, in.	24–30	27
Effective size, mm	0.70–0.75	0.75
Uniformity coefficient	<1.75	<1.75
Filtration rate, gpm/ft^2	2–5	4

Calcium carbonate encrustations

- May occur on the sand grains once lime-soda softening is used.
- Enlarge the sand grains - undesirable
- Controlled by:
  - Lowering the pH by carbonation prior to filtration to:
    - precipitate excess lime and
    - Stabilize the water
  - Stabilization using sodium hexametaphosphate or other such chemicals.

encrustations

- Crushed anthracite coal less susceptible to encrustation
- is used in many RSF
- Typical anthracite characteristics Table 4.3
- To support the anthracite bed:
  - gravel or
  - graded anthracite of 12 in thickness

Dual media & mixed media filters

- Used by most new plants
- Primary filter action depth removal
- Table 4.4 & 4.5
- Principal advantages over sand filters:
  - High filtration rates
  - Longer filter runs due to the increased volume for floc storage within the filter
  - Thus, less backwash water is required per unit volume of filtrate produced.

Table 4.4. Dual-Media Filter Characteristics for Water Treatment

Characteristic	Value	
	Range	Typical
Anthracite:		
Depth, in.	18–24	24
Effective size, mm	0.9–1.1	1.0
Uniformity coefficient	1.6–1.8	1.7
Sand:		
Depth, in.	6–8	6
Effective size, mm	0.45–0.55	0.5
Uniformity coefficient	1.5–1.7	1.6
Filtration rate, gpm/ft^2	3–8	5

Table 4.5. Mixed-Media Filter Characteristics for Water Treatment

Characteristic	Value	
	Range	Typical
Anthracite:		
Depth, in.	16.5–21	18
Effective size, mm	0.95–1.0	1.00
Uniformity coefficient	1.55–1.75	<1.75
Sand:		
Depth, in.	6–9	9
Effective size, mm	0.45–0.55	0.50
Uniformity coefficient	1.5–1.65	1.60
Garnet:		
Depth, in.	3–4.5	3
Effective size, mm	0.20–0.35	0.20
Uniformity coefficient	1.6–2.0	<1.6
Filtration rate, gpm/ft^2	4–10	6

Filtration in WWTP

- In advanced WWTP may be employed to filter:
  - Secondary effluents
  - Chemically treated secondary effluents
  - Chemically treated primary or raw wastewaters
- In advanced or tertiary WWTP are usually:
  - Dual media or
  - Mixed media
- Table 4.6 & 4.7

Table 4.6. Dual-Media Filter Characteristics for Advanced or Tertiary Wastewater Treatment

Characteristic	Value	
	Range	Typical
Anthracite:		
Depth, in.	12–24	18
Effective size, mm	0.8–2.0	1.2
Uniformity coefficient	1.3–1.8	1.6
Sand:		
Depth, in.	6–12	12
Effective size, mm	0.4–0.8	0.55
Uniformity coefficient	1.2–1.6	1.5
Filtration rate, gpm/ft^2	2–10	5

Table 4.7. Multimedia or Mixed-Media Filter Characteristics for Advanced or Tertiary Wastewater Treatment

Characteristic	Value	
	Range	Typical
Anthracite:		
Depth, in.	8–20	16
Effective size, mm	1.0–2.0	1.4
Uniformity coefficient	1.4–1.8	1.5
Sand:		
Depth, in.	8–16	10
Effective size, mm	0.4–0.8	0.5
Uniformity coefficient	1.3–1.8	1.6
Garnet:		
Depth, in.	2–6	4
Effective size, mm	0.2–0.6	0.3
Uniformity coefficient	1.5–1.8	1.6
Filtration rate, gpm/ft^2	2–10	5

Tables 4.6 and 4.7 adapted from *Wastewater Engineering. Treatment. Disposal and Reuse* by Metcalf and Eddy, Inc. Copyright © 1979 by McGraw-Hill Book Co., Inc. Reprinted by permission.

difference

- Principal difference between filters used in WTP & WWTP the size of media.

For WWTP:

- The granules must be larger
- the filter will have:
  - The desired flow rate capacity
  - And the required volume for the accumulated floc.

TLB 305 Desain Pengolahan Fisika dan Kimia 2

Dosen: Rachmawati S. Dj.

Week 13

Disinfection

SEMESTER GANJIL 2024/2025

Table of Contents

01.
Introduction

02.
Sub Topics

03.
About Sub-Topics

Introduction

Sub CP MK *Week 13*

Minggu ke	SubCPMK	Metode Penilaian			Materi Pembelajaran	Ruang belajar daring			Waktu Pembelajaran
		Indikator	Komponen	Bobot %		Sinkronus virtual	*Self directed* Asinkronus mandiri	*Peer learning* Asinkronus Kolaboratif	
13, 14	Mahasiswa mampu menjelaskan teori, prinsip dan penggunaan proses desinfeksi secara lancar dan tepat (C3 P3) menentukan, menghitung dan menganalisis penggunaan bahan kimia secara lancar dan tepat (C4P3)	Ketepatan menjelaskan teori, prinsip dan penggunaan proses desinfeksi; kebenaran menentukan, menghitung dan menganalisis penggunaan bahan kimia		11	• *Introduction* • *Heating* • *Ultraviolet Irradiation* • *Chlorination*	Vicon	Teks/audio /ppt/video /url	Forum diskusi	2 x 50 menit

Extracted from:

Rich, L.G.,
Unit Processes of Sanitary Engineering,
John Wiley & Sons, Inc., New York, London, 1963

WHO, 2017
(Principles and Practices of Drinking-water Chlorination, A guide to strengthening chlorination practices in small-to medium-sized water supplies)

What is this topic about?

It’s about the chlorination process aiming to make the water save to be consumed by the customers

Definition of concept

Disinfection

Process in which pathogenic microorganisms (not their spores) are destroyed

Chlorine Demand

Chlorine consumed during reaction with organic and inorganic material present

Chlorination

Process where chlorine is added to drinking-water to kill or inactivate microorganisms, including harmful pathogens

Residual Chlorine

Free chlorine available for disinfection (may prevent recontamination of water)

Definition of concept

Disinfection

Process in which pathogenic microorganisms (not their spores) are destroyed

Chlorine Demand

Chlorine consumed during reaction with organic and inorganic material present

Chlorination

Process where chlorine is added to drinking-water to kill or inactivate microorganisms, including harmful pathogens

Residual Chlorine

Free chlorine available for disinfection (may prevent recontamination of water)

Sterilization

Total destruction or total removal of all microorganisms from the treated medium

Disinfection

Process in which pathogenic microorganisms (not their spores) are destroyed

Used in a broader sense:
The destruction of a particular species of microorganism at some stage of its development

Rate of kill

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

Eq 12-1

- dN/dt = time rate of kill
- k = rate constant
- N = number of living microorganism

Rate of kill

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

Eq 12-1

- dN/dt = time rate of kill
- k = rate constant
- N = number of living microorganism

Eq 12-1

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

Time to kill

- Integrating Eq 12-1 between the limits of t= 0 and t =t (Eq 12-2 – Eq 12-4),
- N_1, N_2 = number of microorganisms living initially and at time t respectively

$$\int_{N_1}^{N_2} \frac{dN}{N} = -k \int_{0}^{t} dt$$ Eq 12-2

$$\ln \frac{N_2}{N_1} = -kt$$ Eq 12-3

$$t = \frac{2.3}{k} \log \frac{N_1}{N_2}$$ Eq 12-4

N_2

- can never reach zero
- □ probability levels are assumed in the application of Eq 12-4 to disinfection processes.
- Ex: in the destruction of a species of microorganism with only one chance in one hundred of failing to destroy all microorganisms of the species, N_2 should be set to 0.01.
- Higher levels of confidence result from the use of smaller N_2.

$$t = \frac{2.3}{k} \log \frac{N_1}{N_2}$$

Eq 12-4

The rate constant k

depends on:

The concentration of the disinfectant

Temperature

Other environmental conditions

$$t = \frac{2.3}{k} \log \frac{N_1}{N_2}$$

Eq 12-4

Concentration and time

The relationship b/w the concentration of disinfectant and the time required for the killing process:

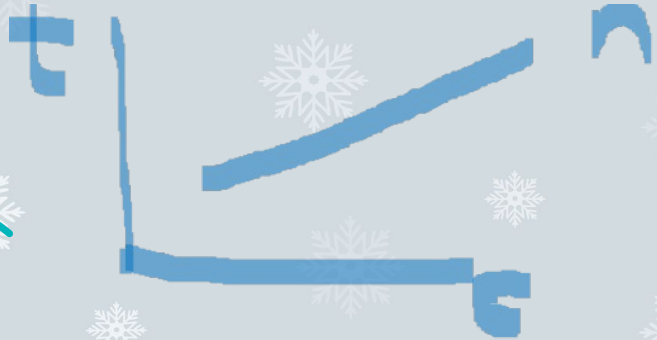
$$t = \frac{K''}{c^n}$$ Eq 12-5

t = time required to kill a given % of microorganisms
c = concentration of disinfectant
n = coefficient of dilution
K" = constant

Coefficient of dilution, n

1. The value of n for a particular disinfectant can be determined from the slope of the straight-line relationship obtained when
   a. The log of contact time is plotted as a function of
   b. The log of the concentration

2. The effect of the disinfectant is strongly dependent upon its:
   a. Concentration □ Once: n > 1
   b. Contact time □ n < 1



$$t = \frac{K''}{C^n}$$

Eq 12-5

Coefficient of dilution, n

1. The value of n for a particular disinfectant can be determined from the slope of the straight-line relationship obtained when
   a. The log of contact time is plotted as a function of
   b. The log of the concentration

2. The effect of the disinfectant is strongly dependent upon its:
   a. Concentration □ Once: n > 1
   b. Contact time □ n < 1

$$t = \frac{K''}{C^n}$$

Eq 12-5

The rate constant, k

1. For a given % kill, Eq 12-4 becomes:

$$t = \frac{K'}{k}$$ Eq 12-6

2. By equating Eq 12-5 & 12-6:

$$k = Kc^n$$ Eq 12-7

$$t = \frac{2.3}{k} \log \frac{N_1}{N_2}$$ Eq 12-4

$$t = \frac{K''}{c^n}$$ Eq 12-5

Arrhenius equation

- The effect of temperature on the rate constant:

$$k = Ce^{-(\Delta H_a / RT_A)}$$ Eq 12-8

- C = constant
- ΔH_a = energy activation, cal
- R = gas constant, 1.99 cal/°C
- T_A = absolute temperature, °K

Arrhenius equation

Through a logarithmic transformation Eq 12-8 becomes:

$$\log k = \log C - (0.434 \Delta H_a / R)(1/T_A)$$

Eq 12-9

A plot of log k vs ($1/T_A$) enables us to evaluate:

- The constant C
- The energy activation ΔH_a.

$$k = Ce^{-(\Delta H_a / RT_A)}$$

Eq 12-8

Environmental factors

The effect of such env factors:

- pH
- Osmotic pressure
- Nutrient concentration

On the rate constant is variable

 Must be expressed for the specific env to which the mo are exposed

$$k = Kc^n$$ Eq 12-7

$$k = Ce^{-(\Delta H_a / RT_A)}$$ Eq 12-8

Processes used in disinfection

Heating

UV

Chlorination

Ozonation

Sub Topics

Sub Topics

Heating

UV

Chlorination

Heating

Heating

too expensive
to use in large-scale water treatment,

carried out in a bach basis

wide application
in the food & beverage industry

process is reliable

easy to control

Heating

Thermal resistance

- a particular microorganisms or
- its spore

Table 12-1

the relative resistance of a microorganism to moist heat

Bacterial spores

have a very high thermal resistance

Sterilization

heat must be applied sufficient to kill bacterial spores

Table 12-1

en sterilization is the objective, heat must be appl
l bacterial spores.

12-1. RELATIVE RESISTANCES OF MICROORGANISMS TO STERILIZATION BY MOIST HEAT*

Organism	Relative Resistance
herichia coli	1
terial spores	3,000,000
ld spores	2–10
ses and bacteriophages	1–5

n, "Physical Methods of Sterilization of Microorganis
iews, 9 (1945), 1–47.
acteria often used as a test species.

UV

UV radiation

Water, air & food stuffs can be disinfected by exposure to the UV radiation emitted by a quartz-mercury vapor lamp

Such radiation destroys bacteria, bacterial spores, molds, mold spores, viruses etc.

Destruction is thought to result from a toxic condition induced by radiation energy.

Electromagnetic spectrum

Fig 12-4

spectrum
UV occupies a band along the spectrum extending from 30-3,650 A^0

lethal effectiveness
of UV energy varies greatly with its wavelength

bactericidal waves
extend from 2,000-2,950 A^0 with a maximum effect at about 2, 537 A^0

Figure 12-1

UV occupies a band along the spectrum extending from 30-3,650 A^0

bactericidal waves extend from 2,000-2,950 A^0 with a maximum effect at about 2, 537 A^0

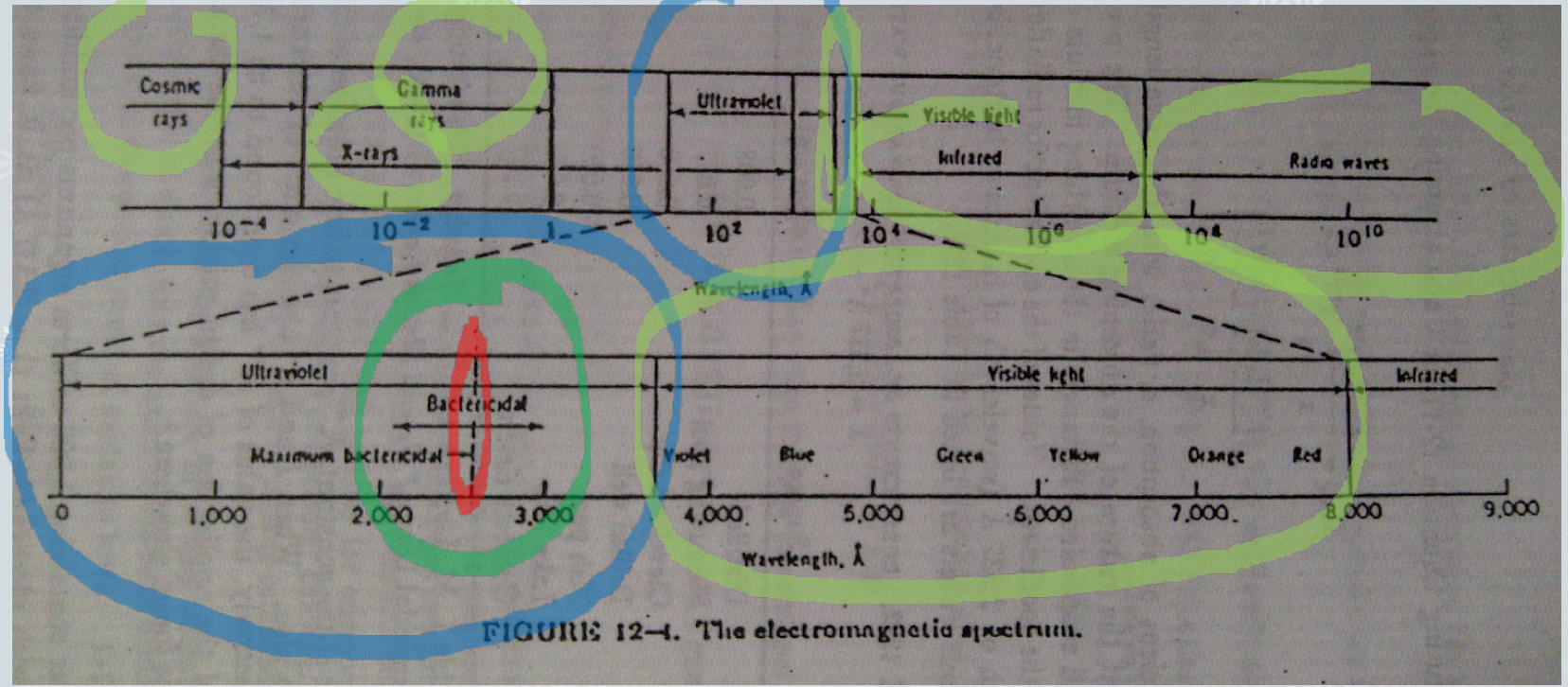


FIGURE 12–4. The electromagnetic spectrum.

The rate of kill

- The rate of kill by UV radiation can be expressed by Eq 12-3 & 12-4.
- The relationship between the rate constant, k, and the radiation intensity, I, is found to be analogous to Eq 12-7:

$$k = K''' I^n$$ Eq 12-11

- K ‘’’ = constant
- I= radiation intensity
- n= exponent
- In most applications, n is found to be closely to unity.

Eq 12-3

$$\ln \frac{N_2}{N_1} = -kt$$

Eq 12-4

$$t = \frac{2.3}{k} \log \frac{N_1}{N_2}$$

Eq 12-7

$$k = Kc^n$$

The reduction in intensity

- The intensity of radiation is reduced as it passes through an absorbing medium.
- The reduction in intensity is predicted by:

$$I = I_0 e^{-ax}$$ Eq 12-12

- I_0 = initial intensity
- a =coefficient absorption
- x = distance between points where the intensities are I_0 and I

In WTP

1. UV radiation is applied to water as it rests in a reservoir or flows through a tank or a pipe.
2. Thus, the rate of kill varies throughout the depth of flow.
3. The average rate may be computed using an average-rate constant:

$$k_{avg} = \frac{\int_0^x k\,dx}{x} \qquad \text{Eq 12-13}$$

The average rate

By substituting Eq 12–11 for k and assuming the exponent n to be unity:

Eq 12-14

$$k_{avg} = \frac{K''' I_0}{x} \int_0^x e^{-ax} dx = \frac{K''' I_0}{x} \left[\frac{1}{a} - \frac{e^{-ax}}{a} \right]$$

$$k = K''' I^n$$ Eq 12-11

$$I = I_0 e^{-ax}$$ Eq 12-12

$$k_{avg} = \frac{\int_0^x k dx}{x}$$ Eq 12-13

The coefficient of absorption, a

Varies with:
- The wavelength of the radiation, and
- The nature of the absorbing medium

The presence of dissolved & suspended substances in the absorbing medium increases the value of the coefficient.

a for radiation having a wavelength of 2,537 A^0 (wavelength of max bactericidal effect) & for various water □ Table 12-2.

TABLE 12-2. COEFFICIENTS OF ABSORPTION FOR VARIOUS WATERS, $\lambda = 2{,}537$ Å*

Source	a(per cm)
Distilled	0.008
Swimming pool	0.031
Cleveland tap	0.050
Drilled well	0.056
Fish pool	0.070
Lake Erie	0.083
Concrete cistern	0.297

* From Table I, M. Luckiesh and L. L. Holladay, "Disinfecting Water by Means of Germicidal Lamps," *General Electric Review*, 47 (1944), 45–50.

The value of the constant K‴

Appears to be a function of the absorbing medium also.

Once I = cal/cm^2/min, and k = min^{-1}
□ K‴ = 1,740

$$k_{avg} = \frac{K''' I_0}{x}\int_0^x e^{-ax}dx = \frac{K''' I_0}{x}\left[\frac{1}{a} - \frac{e^{-ax}}{a}\right]$$

Eq 12-14

Thanks!

CREDITS: This presentation template was created by Slidesgo, including icons by Flaticon, and infographics & images by Freepik.

Please keep this slide for attribution.

slidesgo

TLB 305 Desain Pengolahan Fisika dan Kimia 2

Dosen: Rachmawati S. Dj.

Week 14

Disinfection

SEMESTER GANJIL 2024/2025

Table of Contents

01.
Introduction

02.
Sub Topics

03.
About Sub-Topics

Introduction

Sub CP MK *Week 14*

Minggu ke	SubCPMK	Metode Penilaian			Materi Pembelajaran	Ruang belajar daring			Waktu Pembelajaran
		Indikator	Komponen	Bobot %		Sinkronus virtual	*Self directed* Asinkronus mandiri	*Peer learning* Asinkronus Kolaboratif	
13, 14	Mahasiswa mampu menjelaskan teori, prinsip dan penggunaan proses desinfeksi secara lancar dan tepat (C3 P3) menentukan, menghitung dan menganalisis penggunaan bahan kimia secara lancar dan tepat (C4P3)	Ketepatan menjelaskan teori, prinsip dan penggunaan proses desinfeksi; kebenaran menentukan, menghitung dan menganalisis penggunaan bahan kimia		11	• *Introduction* • *Heating* • *Ultraviolet Irradiation* • *Chlorination*	Vicon	Teks/audio /ppt/video /url	Forum diskusi	2 x 50 menit

Extracted from:

Rich, L.G.,
Unit Processes of Sanitary Engineering,
John Wiley & Sons, Inc., New York, London, 1963

WHO, 2017
(Principles and Practices of Drinking-water Chlorination, A guide to strengthening chlorination practices in small-to medium-sized water supplies)

Reynolds, Tom D.,
Unit Operations and Processes in Environmental Engineering, Brooks/Cole Engineering Division Monterrey, California, 1982

What is this topic about?

It’s about the chlorination process aiming to make the water save to be consumed by the customers

Definition of concept

Chlorine Dose

Amount of chlorine added to the water

Combined Chlorine

Chlorine bound to organic and nitrogen compounds (weak disinfection capacity)

Total Chlorine

Chlorine remaining after chlorine demand has been satisfied and disinfection has occurred

Free/Residual Chlorine

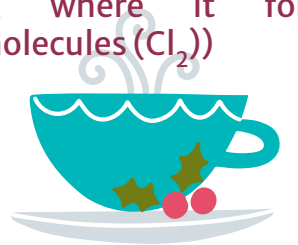
- Free chlorine available for disinfection (may prevent recontamination of water)
- Effective disinfectant

Chlorination

Chlorine gas

Popular disinfectants

- Chlorine & chlorine compounds are popular as disinfectants.
- Chlorine (Cl) (element, yellow green gas under standard conditions, where it forms diatomic molecules (Cl_2))



Chlorine gas

Chlorine gas (pale yellow, green gas) wide application in WTP

Why?

- In concentrations that are tasteless & non poisonous to humans has a specific high toxicity for mo responsible for wbd
- Inexpensive
- Easy to apply
- Can be used to establish temporarily residuals

Chlorine compounds

Atomic chlorine
can exist in any of several oxidation states

Chlorine compounds
The higher the oxidation level, the more powerful its oxidizing power

Chlorine compounds
containing chlorine at higher levels of oxidation maybe more effective towards minimizing tastes & odours, that often develop when chlorine compounds are used as disinfectants

bactericidal efficiency
little difference is noted between the oxidation levels, with the exception of chlorides (Cl^-)

Chlorides (Cl^-)
in small concentrations do not behave as disinfectants.

Chlorides
anion Cl^- , formed once:
- the element chlorine (Cl) (a halogen) gains an electron, or
- when a compound such as hydrogen chloride (HCl) dissolved in water or other polar solvent,

Chlorine gas

- Is quite soluble in water (7,160 mg/L at 20C & 1 atm)
- When dissolved in water, chlorine hydrolyses rapidly to form hypochlorous acid
- $Cl_2 + H_2O \Leftrightarrow HOCl + H^+ + Cl^-$ (12–15)
- For chlorine concentrations of < 1,000 mg/L and pH > 3, the hydrolysis goes virtually (almost) to completion.

Hypochlorous acid

- Hypochlorous acid in water ionizes to form the hypochlorite ion
- $HOCl \Leftrightarrow OCl^- + H^+$ (12-16)
- The dissociation of hypochlorous acid in water is strongly dependent upon the hydrogen ion concentration.
- The dissociation constant varies linearly: 2.0×10^{-8} at 0^0C to 3.7×10^{-8} at 25^0C

$Cl_2 + H_2O \Leftrightarrow HOCl + H^+ + Cl^-$ (12-15)

Free available chlorine

The effect of pH on the distribution of hypochlorous acid & hypochlorite ion in water at 2 different temperatures: Fig 12-5

The chlorine species HOCl & OCl^- in water constitute
 free available chlorine.

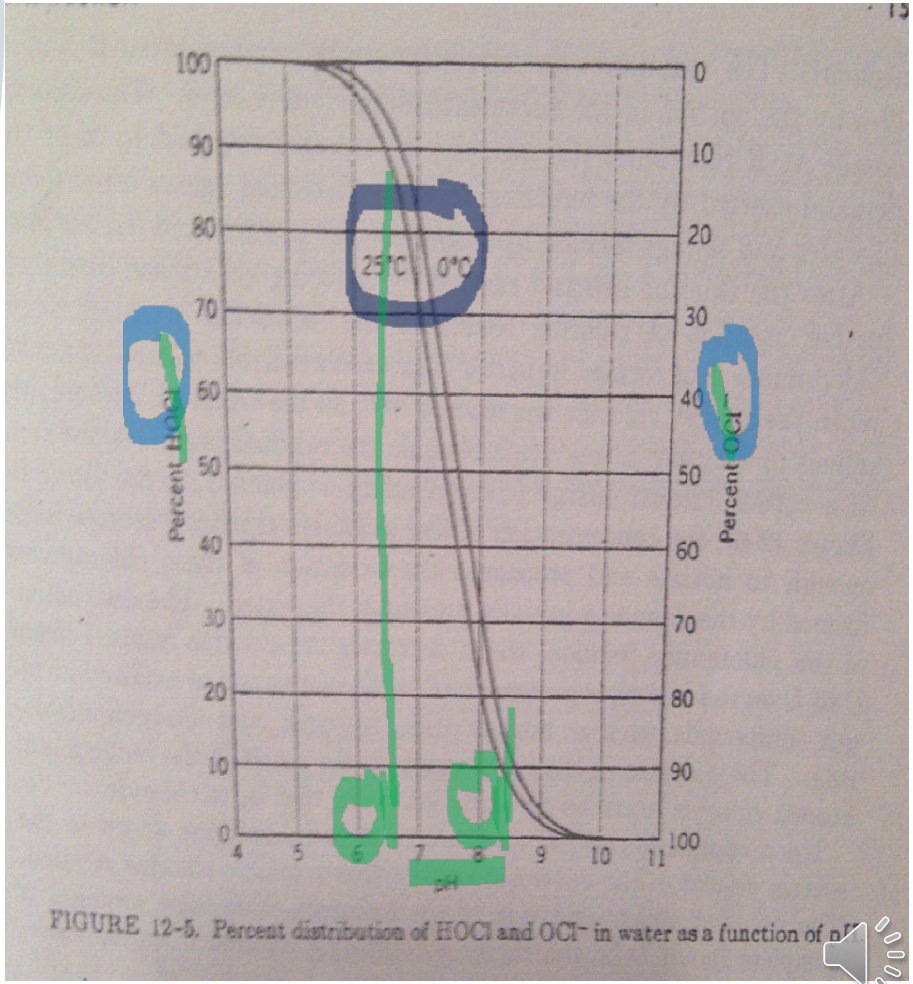


FIGURE 12-5. Percent distribution of HOCl and OCl^- in water as a function of pH

Hypochlorite ions

- Hypochlorites, such as calcium hypochlorite, when in water ionize to form the hypochlorite ion
- $Ca(OCl)_2 \rightarrow Ca^{++} + 2OCl^-$ (12–17)
- The hypochlorite ions thus produced along with the hydrogen ions in solution establish an equilibrium with hypochlorous acid in accordance with Eq 12–16

$HOCl \Leftrightarrow OCl^- + H^+$ (12-16)

Hypochlorous acid

In water reacts with ammonia to produce monochloramine, dichloramine, and trichloramine.

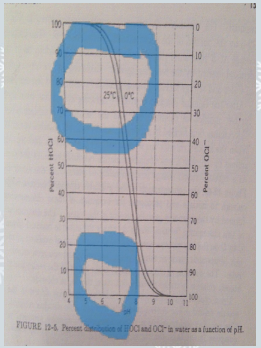
$NH_3 + HOCl \rightarrow NH_2Cl + H_2O$ (12-18)

$NH_3 + 2HOCl \rightarrow NHCl_2 + 2H_2O$ (12-19)

$NH_3 + 3HOCl \rightarrow NCl_3 + 3H_2O$ (12-20)

Combined available chlorine

- The amount of each species of chloramine produced is dependent on:
  - The relative quantities of hypochlorous acid & ammonia present
  - pH
  - Temperature
- Chloramines are also formed by the reaction of hypochlorous acid with any organic amines present in solution.
- Chlorine present in water as chloramines
  - combined available chlorine



$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \quad (12\text{-}18)$$

$$NH_3 + 2HOCl \rightarrow NHCl_2 + 2H_2O \quad (12\text{-}19)$$

$$NH_3 + 3HOCl \rightarrow NCl_3 + 3H_2O \quad (12\text{-}20)$$

Chlorine residual curve

Once chlorine is added to water containing reducing agents, ammonia & organic amines, residuals develop which, if plotted vs dosages required to attain the residuals, yield a curve similar to the one shown in Fig 12-6

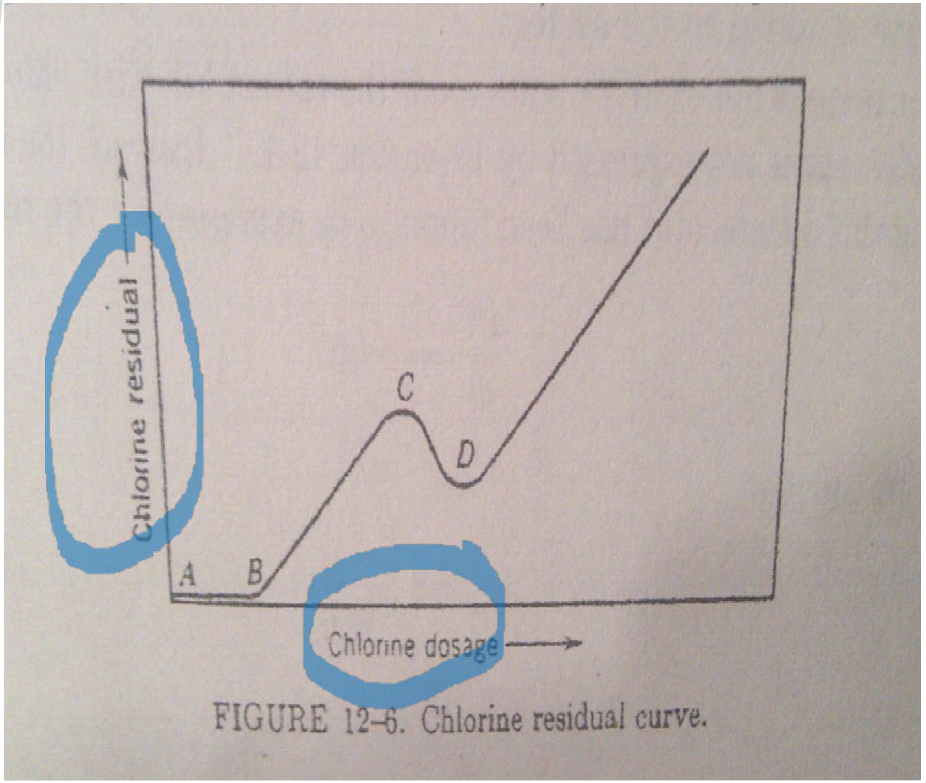


FIGURE 12-6. Chlorine residual curve.

A-B

- Chlorine added initially reacts with the reducing agents present, is reduced to chlorides, and develops no measurable residual.
- The reaction with reducing agents is represented in Fig 12-6 by the portion of the curve extending from A to B.

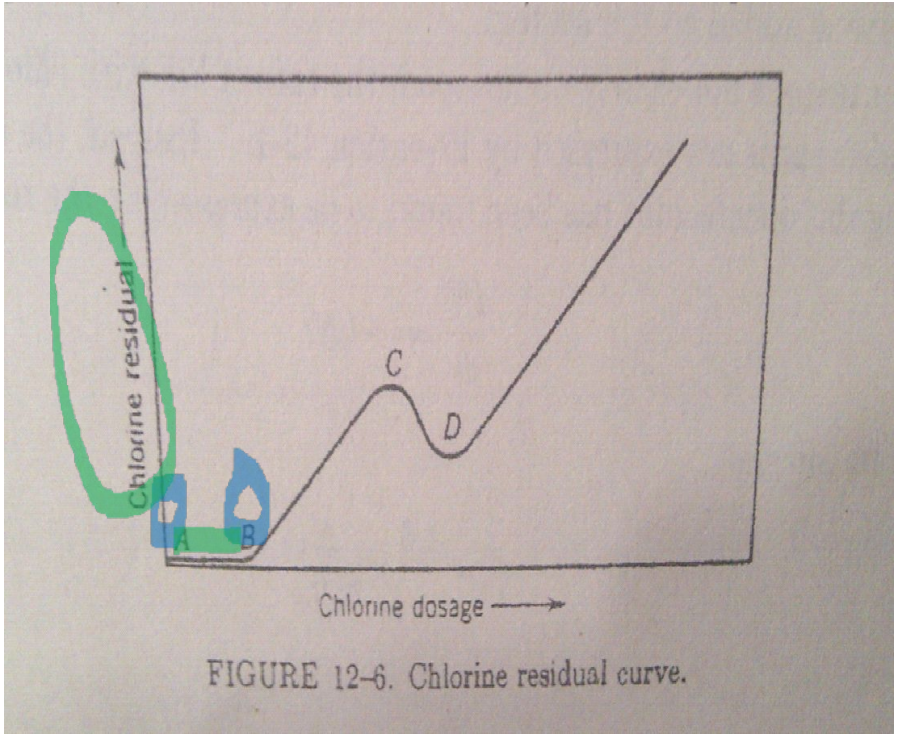


FIGURE 12-6. Chlorine residual curve.

B, chlorine demand

- The chlorine dosage at B represents the quantity of chlorine required to meet the demand exerted by the reducing agents.
- Reducing agents often found in water & sewage include hydrogen sulfide, nitrites & ferrous ions.

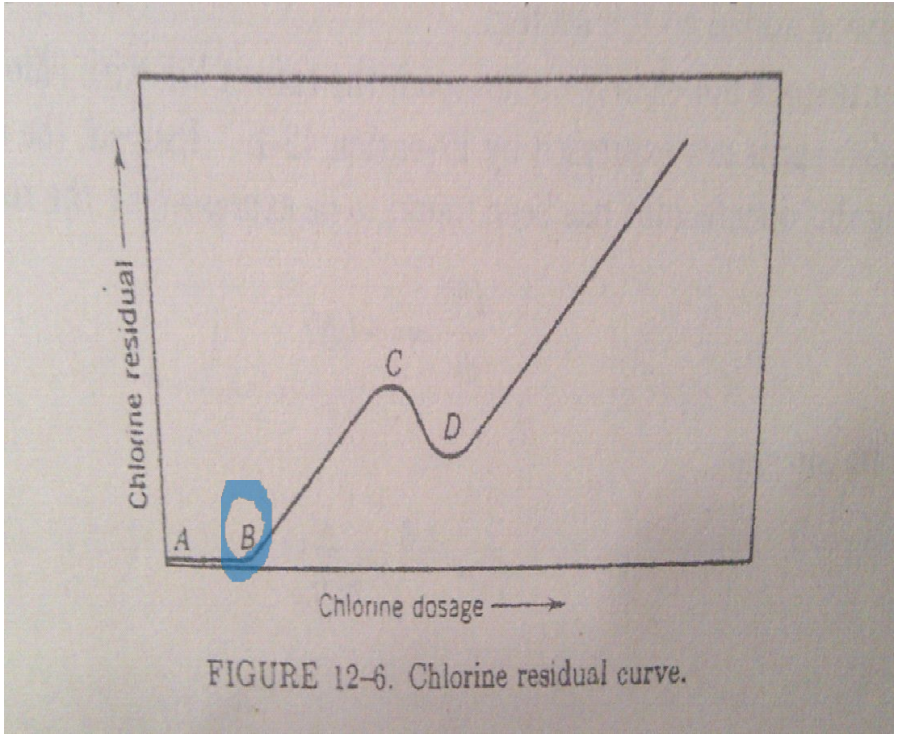


FIGURE 12-6. Chlorine residual curve.

B-C

- After the chlorine demand exerted by reducing agents has been met, further addition of chlorine results in the formation of chloramines.
- Chloramines thus formed impart a combined available chlorine residual.
- When the applied chlorine has reacted with all the ammonia & organic amines in solution, a free available chlorine residual begins to develop.

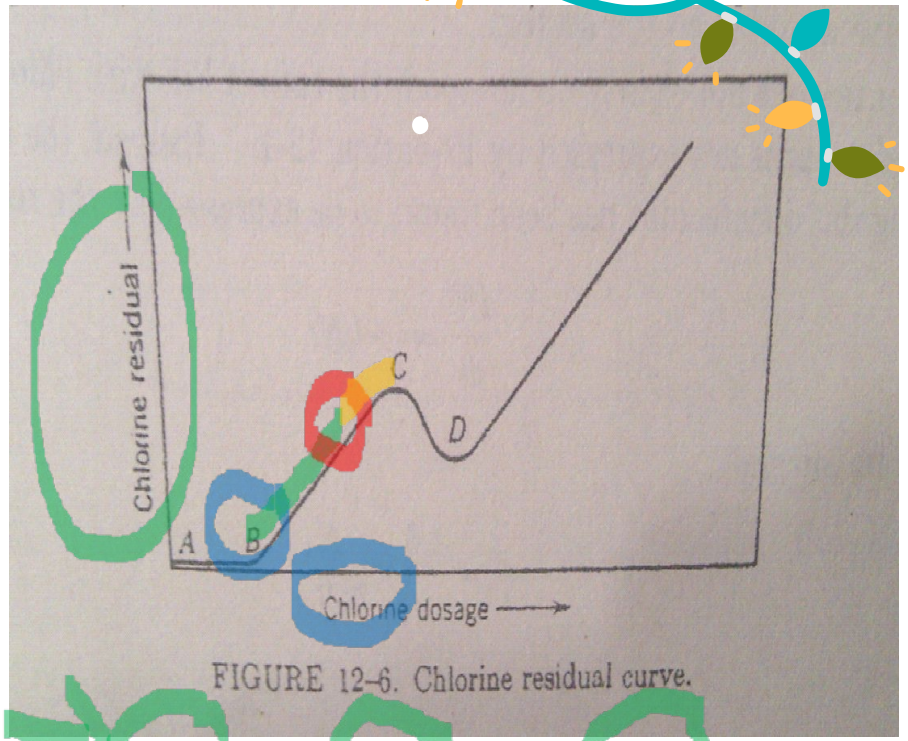


FIGURE 12-6. Chlorine residual curve.

$NH_3 + HOCl \rightarrow NH_2Cl + H_2O$ (12-18)

$NH_3 + 2HOCl \rightarrow NHCl_2 + 2H_2O$ (12-19)

$NH_3 + 3HOCl \rightarrow NCl_3 + 3H_2O$ (12-20)

04 CHLORINATION

C

- At a certain critical dosage (point C), the concentration of free available chlorine becomes large enough to initiate and propagate the oxidation of those chloramines formed by the ammonia initially present in the water.

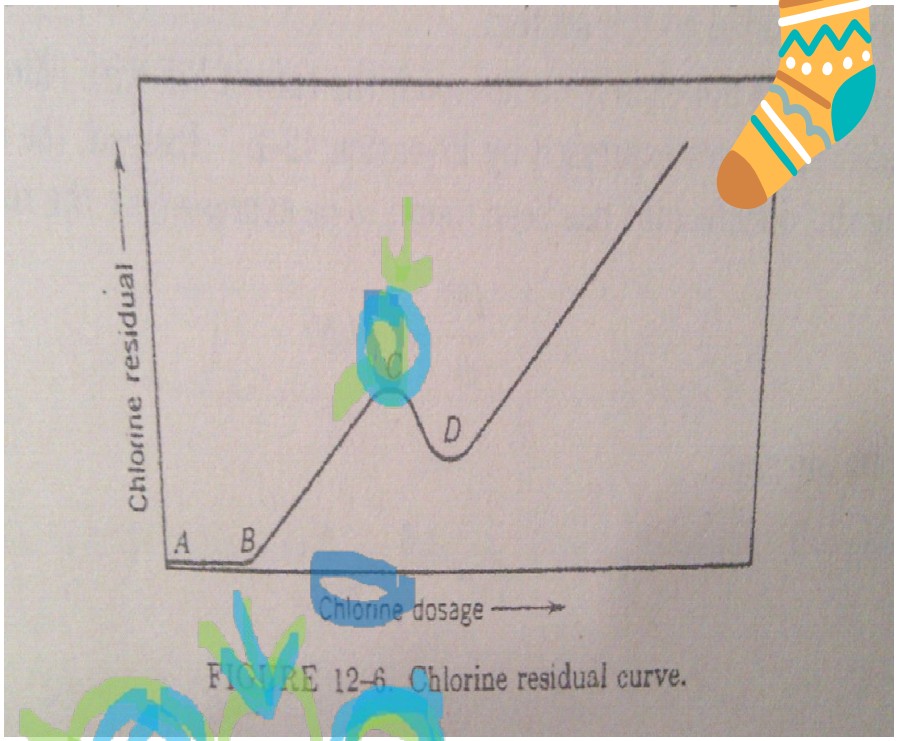


FIGURE 12-6. Chlorine residual curve.

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \quad (12\text{-}18)$$

$$NH_3 + 2HOCl \rightarrow NHCl_2 + 2H_2O \quad (12\text{-}19)$$

$$NH_3 + 3HOCl \rightarrow NCl_3 + 3H_2O \quad (12\text{-}20)$$

C-D

- The destruction of this chloramine fraction, which decreases the chlorine residual (from C to D on the curve), is accompanied by the formation of oxidized nitrogen compounds, such as nitrous oxide (N_2O, colourless gas), nitrogen (N_2), and nitrogen trichloride (NCl_3).
- The decrease in chlorine residual is the result of the reduction of atomic chlorine (Cl) to its lowest oxidation state □ chloride (Cl^-)

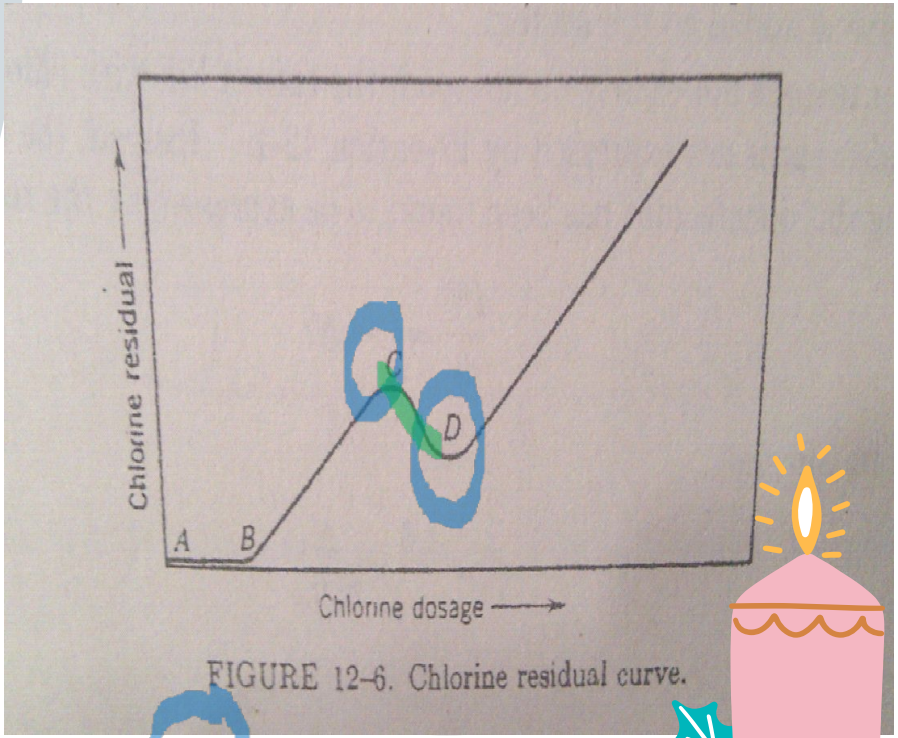


FIGURE 12-6. Chlorine residual curve.

$NH_3 + HOCl \rightarrow NH_2Cl + H_2O$ (12-18)

$NH_3 + 2HOCl \rightarrow NHCl_2 + 2H_2O$ (12-19)

$NH_3 + 3HOCl \rightarrow NCl_3 + 3H_2O$ (12-20)

D, breakpoint

- Upon completion of the oxidation, additional chlorine to the system results in the development of a free available chlorine residual.
- The point at which oxidation of the ammonia-chloramine products is complete (point D) □ breakpoint

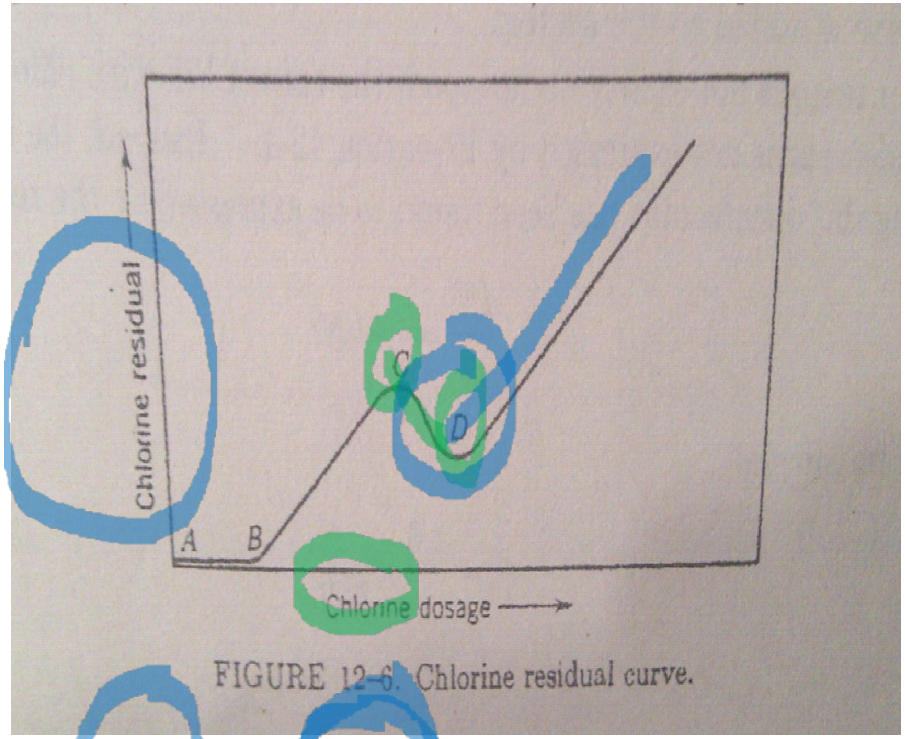


FIGURE 12-6. Chlorine residual curve.

$NH_3 + HOCl \rightarrow NH_2Cl + H_2O$ (12-18)

$NH_3 + 2HOCl \rightarrow NHCl_2 + 2H_2O$ (12-19)

$NH_3 + 3HOCl \rightarrow NCl_3 + 3H_2O$ (12-20)

Chlorine residual curve

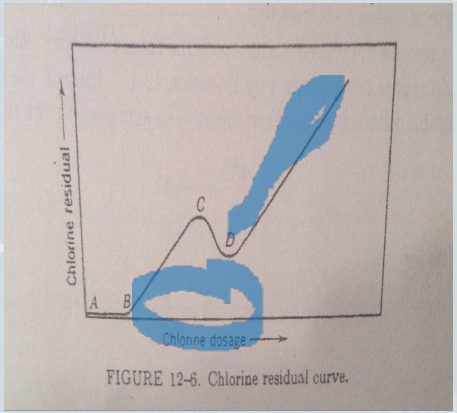


FIGURE 12-6. Chlorine residual curve.

B–C

- The chlorine residual is predominantly in the combined available state with some free available residual developing as C is approached

C–D

The chloramines resulting from the chlorination of ammonia are destroyed

D

The residual remaining consists of the resistant chloramines produced by the chlorination of organic amino compounds.

Beyond D

The residual is composed of the resistant chloramines and the free available chlorine formed as additional chlorine is added to the system.

Chlorine residual curve

Typical

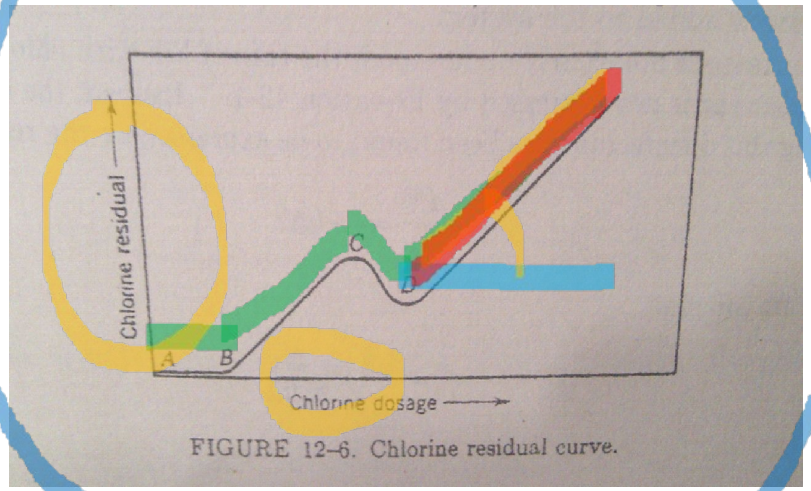


FIGURE 12–6. Chlorine residual curve.

The rate of kill

- For reasons not clearly understood, the rate of kill with chlorine as the disinfectant is not expressed by Eq 12-1.
- The rate of kill using this disinfectant:

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

Eq 12-21

- Or its integral (12-22):

$$t^2 = \frac{4.6}{k} \log \frac{N_1}{N_2}$$

Eq 12-22

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

Eq 12-1

$$t = \frac{2.3}{k} \log \frac{N_1}{N_2}$$

Eq 12-2

k, n

The rate constant k

can be determined from a semi-logarithmic plot of the percent surviving vs the square of the contact time

$$t^2 = \frac{4.6}{k} \log \frac{N_1}{N_2}$$

Eq 12-22

The coefficient of dilution n

For pH = 7-10 and
For the organism E.coli
n, in Eq 12-7, ≈ 1.3, for both:
Free available and
Combined available chlorine residuals

$$k = Kc^n$$

Eq 12-7

K

K at a given temperature varies with:

- The type of residual
- The pH of the system

The influence of these factors on the constant Fig 12-7

The curves found are based on limited data. But, they do provide a comparison between the values of K for the two types of residual.

The larger values of K found for the free available residuals reflect the greater disinfectant power of this type of residual.

Eq 12-22

$$t^2 = \frac{4.6}{k} \log \frac{N_1}{N_2}$$

Eq 12-7

$$k = Kc^n$$

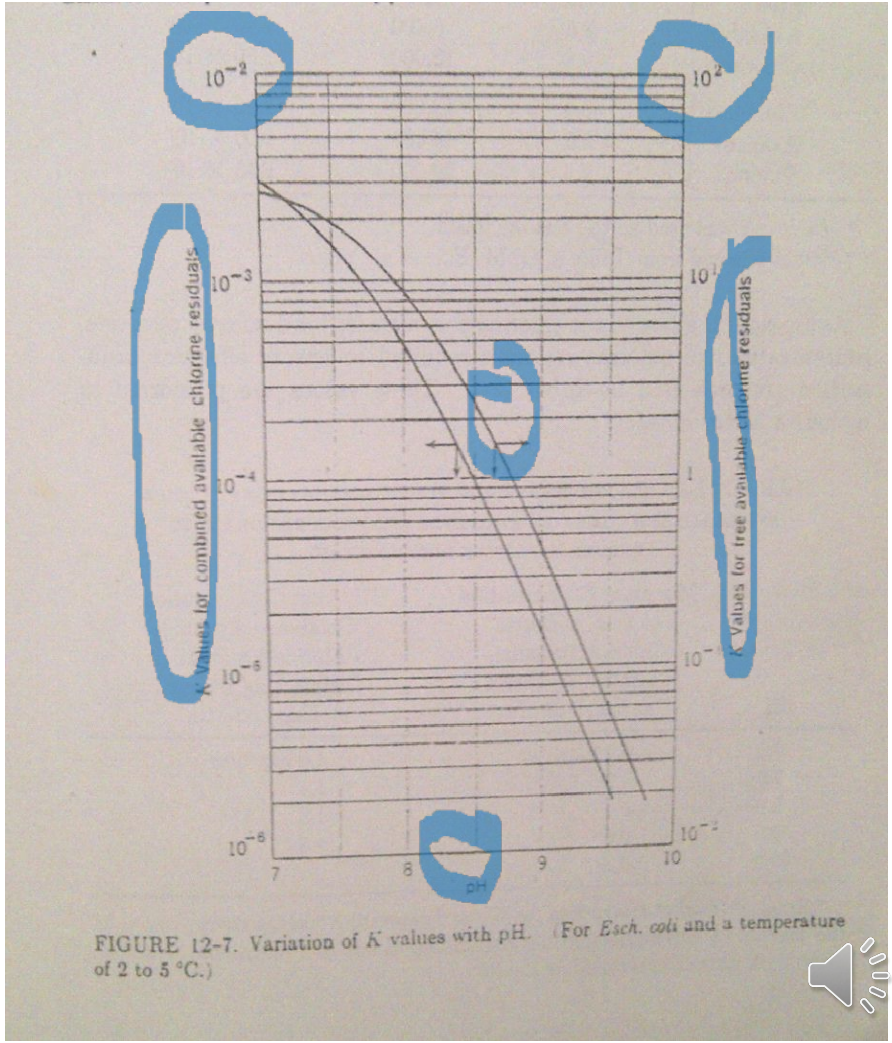


FIGURE 12-7. Variation of K values with pH. (For *Esch. coli* and a temperature of 2 to 5 °C.)

T, ΔH_a, C

The effect of temperature on the rate constant is predicted by Eq 12-8.

Values of the energy of activation, ΔH_a, and the constant, C, when chlorine is the disinfectant → Table 12-3.

These values, specific for E.coli, are based on limited data → must be used with caution.

TABLE 12-3. VARIATION OF THE ENERGY OF ACTIVATION AND THE CONSTANT IN EQUATION 12-8 WITH pH*

	pH	ΔH_a† (calories)	C
Free Available Chlorine	7.0	8,200	8.16×10^5
	8.5	6,400	1.40×10^4
	9.8	12,000	2.60×10^6
Combined Available Chlorine	7.0	12,000	8.95×10^6
	8.5	14,000	1.00×10^7
	9.5	20,000	1.45×10^{10}

* (*Esch. coli* was used as the test organism.)

† Values selected from Table 4, G. M. Fair et al., *op. cit.*

$$t^2 = \frac{4.6}{k} \log \frac{N_1}{N_2}$$ Eq 12-22

$$k = Ce^{-(\Delta H_a / RT_A)}$$ Eq 12-8

$$\log k = \log C - (0.434 \Delta H_a / R)(1/T_A)$$ Eq 12-9

Chlorine residuals

As a practical guide to the chlorination of water for domestic purposes, concentrations of chlorine residuals required to ensure effective disinfection □ Table 12-4.

These values are purported (claimed) to include a factor of safety.

TABLE 12-4. RECOMMENDED MINIMUM CONCENTRATIONS OF FREE AVAILABLE AND COMBINED AVAILABLE CHLORINE RESIDUALS TO ENSURE EFFECTIVE DISINFECTION*

pH	Minimum Concentration of Free Available Chlorine Residual, Disinfecting Period at Least 10 Min	Minimum Concentration of Combined Available Chlorine Residual, Disinfecting Period at Least 60 Min
6.0	0.2 mg/liter	1.0 mg/liter
7.0	0.2	1.5
8.0	0.4	1.8
9.0	0.8	1.8
10.0	0.8	. . .

*From *Manual of Instruction for Water Treatment Plant Operators*, issued by the Bureau of Environmental Sanitation and Office of Professional Training, New York State Department of Health.

Ammonia

The use of ammonia in conjunction with chlorine has been practiced in WTP for the purpose of inducing the formation of combined available residuals.

Due to:

- The minimization of tastes & odours
- A more persistent chlorine residual

WWTP

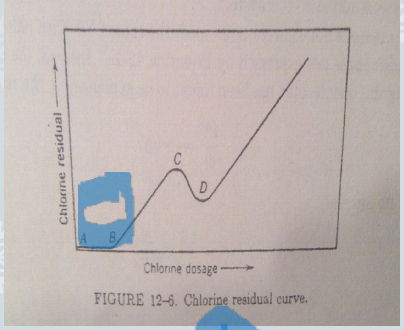


FIGURE 12–6. Chlorine residual curve.

is generally deemed to be adequate when sufficient chlorine has been added to obtain a combined available residual of 0.5 mg/L after a 15-min contact period.

The quantity of chlorine added to obtain a measurable residual □ the chlorine demand

And is represented by the dosage at point B

Ozonation

Ozone

- an allotrope of oxygen.
- A powerful oxidant
- More powerful than hypochlorous acid
- Is relatively unstable in aqueous solution
- Having a half-life of 20-30 min at 20^0C.
- The presence of oxidant demanding materials in solution □ the half-life even shorter.

Ozone

1. Usage:
   a. Widely used in WTP
   b. Very limited used in WWTP
2. Ozone must be produced on-site because it cannot be stored as chlorine.
3. Ozone is produced by passing air:
   a. Between oppositely charged plates or
   b. Through tubes in which a core and the tube walls serve as the oppositely charged surfaces.

Thanks!

CREDITS: This presentation template was created by Slidesgo, including icons by Flaticon, and infographics & images by Freepik.

Please keep this slide for attribution.

slidesgo