Arba Minch Water Technology Institute

Wastewater Treatment

WSEE-3152

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Introduction

- **What is Wastewater?**
- **What is wastewater treatment?**
  - o Removing contaminants/pollutants from wastewater.

General About WWT

Some contaminants in WW.

- Suspended solids: lead to the development of sludge deposits and anaerobic conditions
- **Pathogens: cause diseases**
- Nutrients: essential for growth (N, P,…).
- Refractory organics: resist conventional methods of wastewater treatment.
- Heavy metals :may have to be removed if the wastewater is to be reused
- Dissolved inorganic solids (calcium, sodium, and sulfate): may have to be removed if the wastewater is to be reused
- Organic matters

Therefore, Wastewater should be collected and treated before its ultimate disposal in order to :

- Reduce spread of **communicable diseases**
- Prevent surface and ground water **pollution**.

wastewater can be broadly divided into two categories:

1. **Biodegradable wastewater**:

- The wastes in general have a predominance of biodegradable organic matter
- The stabilization of organic matter is accomplished biologically using a variety of microorganisms.
- Based on bacterial relationship to oxygen the microorganisms can be:

I. obligate aerobes: utilise oxygen

II. obligate anaerobes: without oxygen

III. facultative anaerobes: utilise oxygène if présent and not if not present

IV. denitrifiers

2. Non-biodegradable wastewater

- The wastewater are rich in non-biodegradable matter consisting of solids and liquids in suspended or dissolved form, including various inorganic and organic, many of which may be highly toxic.

General About WWT

- WWT comprises of the following stages of treatment:
  1. Conventional treatment
     I. Preliminary treatment
     II. Primary treatment
     III. Secondary/biological
  2. Advanced/tertiary treatment

General About WWT

- Methods of treatment in which the application of physical forces dominates are called **unit operation**
- Methods of treatment in which chemical or biological activity are involved are known as **unit process**.
- WWT applies any of this operations, processes or combination of both.
- WWT is the combination of physical, chemical and biological processes.

Objectives of WWT

- Removal of pollutants and the protection and preservation of our natural resources.
- protection of human health by the destruction of pathogenic organisms prior to treated effluent being discharged to receiving water bodies and land.

Wastewater Treatment Standards

- Environmental standards are developed to ensure that the **impacts of treated wastewater discharges into ambient waters are acceptable.**

Effluents from different establishments should be:

1. Free from materials and heat in quantities, concentrations or combinations which are toxic or **harmful to human, animal, aquatic life.**
2. Free from anything that will **settle in receiving waters forming putrescence** or that will adversely affect aquatic life.
3. Free from floating debris, oil, scum and other materials;

Wastewater Treatment Standards

4. Free from materials and heat that **produce color, turbidity, taste or odor** in sufficient concentration to create a nuisance or adversely affect aquatic life in receiving waters;
5. Free from **nutrients** in concentrations that create nuisance growths of aquatic weeds or algae in the receiving waters.

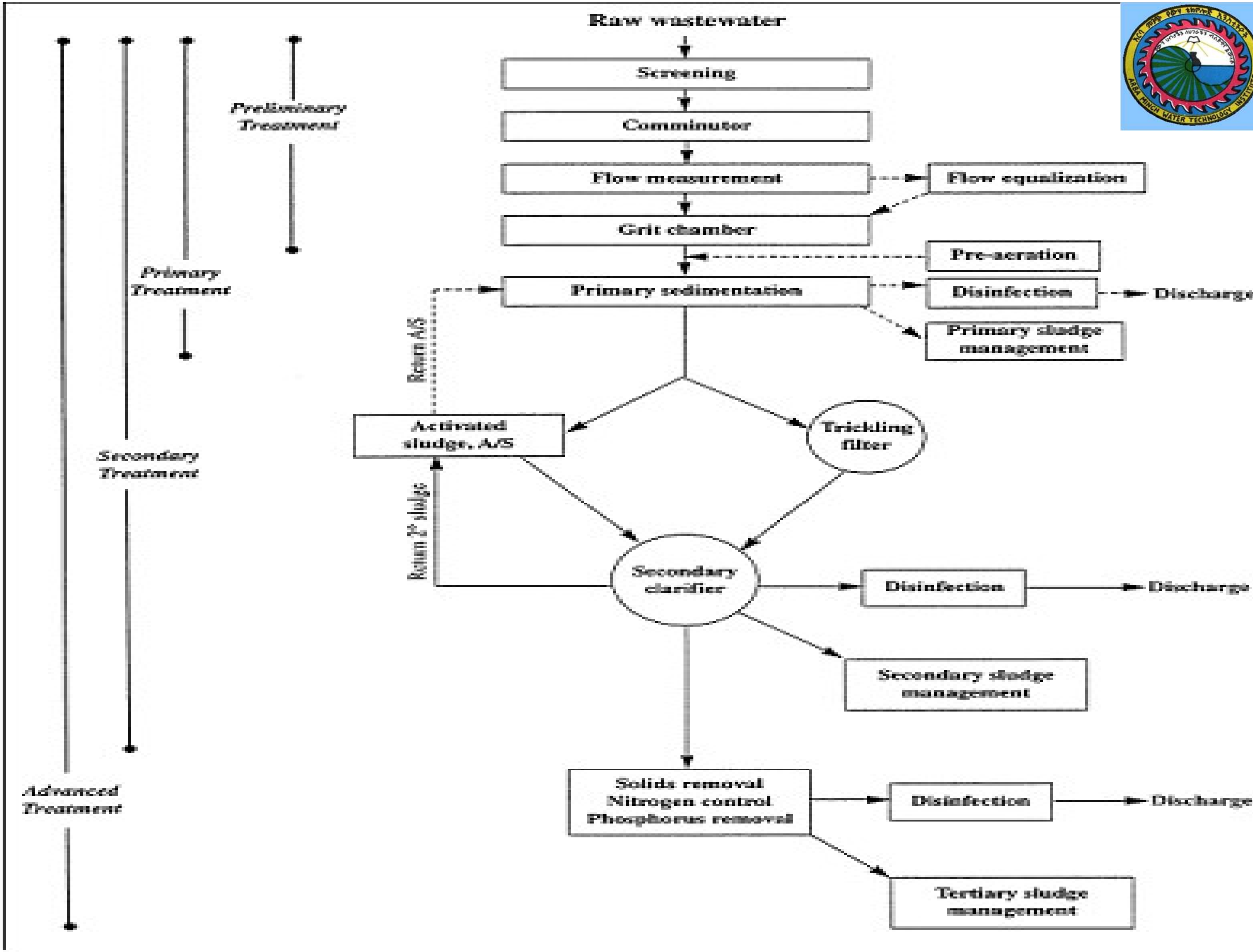
Flow Sheets for WWT Systems

- The term "flow sheet" is used to describe a particular **combination of unit operations and processes** used to achieve a specific treatment objective.
- Is the combination **preliminary treatment**

  primary

  secondary

  tertiary treatment system



Raw wastewater
Screening
Comminuter
Flow measurement
Flow equalization
Grit chamber
Pre-aeration
Primary sedimentation
Disinfection
Discharge
Primary sludge management
Preliminary Treatment
Primary Treatment
Secondary Treatment
Advanced Treatment
Return A/S
Activated sludge, A/S
Trickling filter
Return 2° sludge
Secondary clarifier
Disinfection
Discharge
Secondary sludge management
Solids removal
Nitrogen control
Phosphorus removal
Disinfection
Discharge
Tertiary sludge management

CHARACTERISTICS OF WASTEWATER

Physical, Chemical and Bacteriological Characteristic of Wastewater

Physical Characteristics

- Turbidity , color, odor, temperature, solids

Chemical Characteristics

- Alkalinity, pH, Chloride Contents, Dissolved gases, Nitrogen compounds, Phosphorus, Presence of Fats, Oils and Greases, Sulphides, Sulphates and Hydrogen Sulphide Gas, Dissolved Oxygen (DO), Bio-Chemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), Total Organic Carbon

Biological Characteristics

Physical, Chemical and Bacteriological Characteristic of Wastewater

What is the turbidity, color, odor, temperature of sewage?

❖Turbid, grey/soapy/black after petrification, musty (not offensive)/bad smell after H2S released, warmer

Solids

- sewage only contains about 0.05 to 0.1 percent solids
- Solids present in sewage may be in any of the 4 forms:
  - suspended solids: solids which remain floating in sewage.
  - dissolved solids: remain dissolved in sewage just as salt in water
  - colloidal solids: are finely divided solids remaining either in solution or in suspension.
  - settleable solids: solid matter which settles out.

Physical, Chemical and Bacteriological Characteristic of Wastewater

- The proportion of these different types of solids is generally found as:
- **Inorganic matter** consists of minerals and salts, like: sand, gravel, debris, dissolved salts, chlorides, sulphates, etc.
- **Organic matter** consists of:
  - I. **Carbohydrates** such as cellulose, cotton, fiber, starch, sugar, etc.
  - II. **Fats and oils** received from kitchens, laundries, garages, shops, etc.
  - III. **Nitrogenous compounds** like proteins and their decomposed products, including wastes from animals, urea, fatty acids, hydrocarbons, etc.

Physical, Chemical and Bacteriological Characteristic of Wastewater

pH

- is a method of expressing the acid condition of the WW.
- For proper treatment, wastewater pH should normally be in the range of 6.5 to 9.0.
- the fresh sewage is generally alkaline in nature (pH > 7); but as time passes, its pH tends to fall due to production of acids by bacterial action in anaerobic or nitrification processes.

Chloride Contents

- derived from the kitchen wastes, human feces, and urinary discharges, etc.
- when the chloride content of a given sewage is found to be high, it indicates the presence of industrial wastes or infiltration of sea water

Nitrogen compounds

- The presence of nitrogen in sewage indicates the presence of organic matter, and may occur in:
  - a) Free ammonia, called ammonia nitrogen;
  - b) Albuminoid nitrogen, called Organic nitrogen;
  - c) Nitrites; and
  - d) Nitrates

Phosphorus

- is essential to biological activity and must be present in at least minimum quantities or secondary treatment processes will not perform.
- Excessive amounts can cause stream damage and excessive algal growth.

Physical, Chemical and Bacteriological Characteristic of Wastewater

Presence of Fats, Oils and Greases

- are derived in sewage from the discharges of animals and vegetable matter, or from the industries like garages, kitchens of hotels and restaurants, etc.
- Such matter form scum on the top of the sedimentation tanks and **clog the voids of the filtering media.**
- Hence, need proper detection and removal.

Physical, Chemical and Bacteriological Characteristic of Wastewater

Sulphides, Sulphates and Hydrogen Sulphide Gas

- reflects aerobic, and/or anaerobic decomposition.
- Sulphides and sulphates are formed due to the decomposition of various sulphur containing substances
- this, decomposition also leads to evolution of H2S gas which cause: bad smells and odours,
  - corrosion of concrete sewer pipes.
- in aerobic digestion of, the aerobic and facultative bacteria oxidize the sulphur and its compounds to sulphides, which ultimately break down to form sulphate ions (SO_4^{-2}), which is a stable and an unobjectionable end product
- In anaerobic digestion of, the anaerobic and facultative bacteria reduce the sulphur and its compounds into sulphides, with evolution of H2S gas along with methane and CO2, thus causing very obnoxious smells and odours.

Physical, Chemical and Bacteriological Characteristic of Wastewater

Dissolved Oxygen (DO)

- DO is very important for aquatic life like fish,…
- the treated sewage should ensure at least **4ppm of DO in it before discharging it to river stream**;
- otherwise, fish are likely to be killed, creating nuisance near the vicinity of disposal.
- very fresh sewage contains some DO, which is soon depleted by aerobic decomposition.

Physical, Chemical and Bacteriological Characteristic of Wastewater

Bio-Chemical Oxygen Demand (BOD)

- used as a measure of the quantity of oxygen required for oxidation of biodegradable organic matter by aerobic biochemical action.
- The rate of oxygen consumption is affected by a number of variables:
  - temperature, pH, the presence microorganisms, and the type of organic and inorganic material.
- **The greater the BOD, the more rapidly oxygen is depleted in the water body.**
- The consequences of high BOD are the same as those for low DO:
  - aquatic organisms become stressed, suffocate, and die.

Physical, Chemical and Bacteriological Characteristic of Wastewater

Chemical Oxygen Demand (COD)

- measures the total quantity of oxygen required for oxidation of organics into carbon dioxide and water.

Physical, Chemical and Bacteriological Characteristic of Wastewater

Biological Characteristics

- ✓are due to the presence of bacteria and other living microorganisms, such as algae, fungi, protozoa, etc.
- ✓Most of the vast number of bacteria present in sewage is harmless non-pathogenic bacteria.
- ✓They are useful and helpful in bringing oxidation and decomposition of sewage.

Measurement of Concentration of Contaminants in Wastewater

- The most important standard methods for analysis of organic contaminants/oxygen demand are:
  1. Theoretical Oxygen Demand (ThOD)
  2. Chemical Oxygen Demand (COD)
  3. Biochemical Oxygen Demand (BOD)

Measurement of Concentration of Contaminants in Wastewater

1. **Theoretical Oxygen Demand (ThOD)**

- This is the theoretical amount of oxygen required to oxidize the organic fraction of the wastewater completely to carbon dioxide and water.
- E.G. calculate the amount of oxygen required to oxidize 300mg/l of glucose is:

$$C_6H_{12}O_6 + 6O_2 \rightarrow 6CO_2 + 6H_2O$$

(C = 12, H = 1 and O = 16), $C_6H_{12}O_6$ is 180 and $6O_2$ is 192; then, ThOD of, $\frac{192}{180} * 300 = 321$ mg/l .

- Because WW is so complex in nature its ThOD cannot be calculated, but in practice it is approximated by the chemical oxygen demand.

Measurement of Concentration of Contaminants in Wastewater

2. Chemical Oxygen Demand (COD)

- is determined by performing a lab. test with a strong oxidant like dichromate solution.
- In order to perform this test, a known quantity of WW is mixed with a known quantity of standard solution of potassium dichromate, and the mixture is heated.
- The organic matter is oxidized by $K_2Cr_2O_7$ (in the presence of H_2SO_4 (helps to digest/break down the complex molecules).
- COD used more of to measure **non-biodegradable matter.**
- The advantage of COD measurements is that they are obtained very quickly (within 3 hours)
- the disadvantages, they do not give any information on the proportion of the WW that can be oxidized by bacteria.

Measurement of Concentration of Contaminants in Wastewater

3. Biochemical Oxygen Demand (BOD)

- Oxygen demand of WW is exerted by three classes of materials:

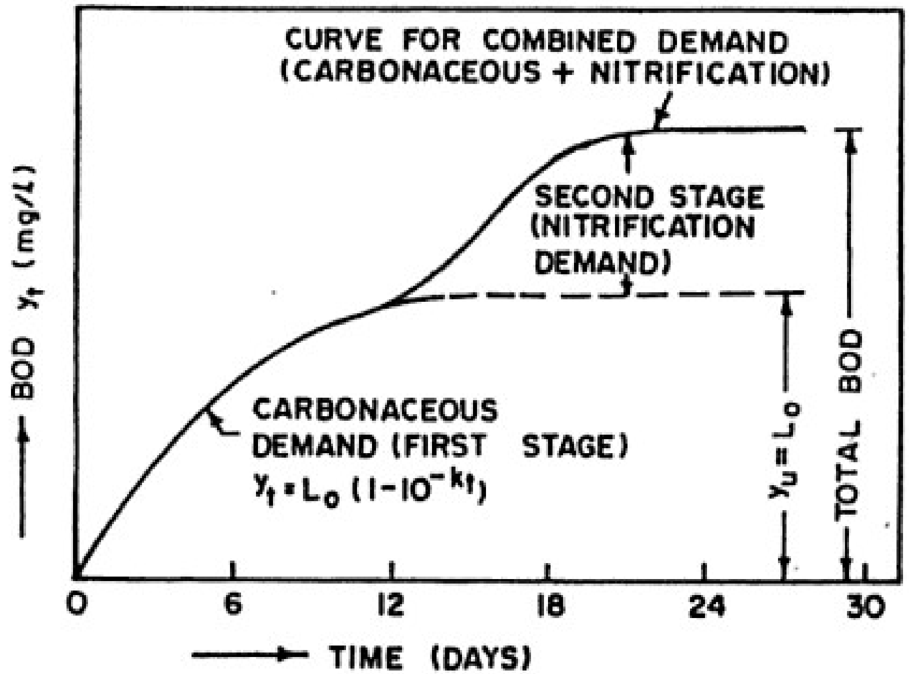
1. **Carbonaceous organic materials** usable as a source of food by aerobic organisms
2. oxidizable nitrogen derived from nitrite, ammonia, and organic nitrogen compounds which serve as food for specific bacteria (e.g., Nitrosomonas and Nitrobacter).
3. Chemical reducing compounds, e.g., ferrous ion (Fe2+), sulfites (SO32-), and sulfide (S2-) which are oxidized by dissolved oxygen.

- For domestic sewage, nearly all oxygen demand is due to carbonaceous organic materials.

Measurement of Concentration of Contaminants in Wastewater

- The carbonaceous BOD is the amount of oxygen required by microorganisms to decompose carbonaceous material that are subject to microbial decomposition.
- This is the first stage of oxidation and the corresponding BOD is known as the first stage demand.

In the 2nd stage, nitrogenous matter oxidized, and the corresponding BOD is called second stage BOD or nitrifying demand.



Measurement of Concentration of Contaminants in Wastewater

- The BOD test results are used for the following purpose:
  1. Determination of the quantity of oxygen required
  2. Determination of size of WW treatment facilities
  3. Measurement of efficiency of some treatment methods
  4. Determination of strength of sewage
  5. Determination of quantity of clear water required for dilution during disposal.

BOD test: dilution test method procedures

1. Sample is diluted with specially prepared dilution water
2. The water is aerated to saturate it with oxygen before mixing it with sample
3. Initial DO of diluted sample is measured
4. Then, incubated for 5days at 20^{oc}, after 5days DO measured again

Measurement of Concentration of Contaminants in Wastewater

Then, oxygen consumed = $DO_{initial} - DO_{final}$

$$\boldsymbol{BOD_5} = (oxygen\ consumed) * dilution\ factor$$

Where, dilution factor (Df) = $\frac{volume\ of\ diluted\ sample}{volume\ of\ undiluted\ WW\ sample}$

e.g., if 1ml of sewage diluted to make 100ml of test sample, what is the dilution factor?

Mathematical Model for the BOD Curve

- The rate of oxygen utilization, at a given incubation time decreases as concentration of organic matter remaining un-oxidized becomes gradually smaller.

The rate of de-oxygenation depends on temperature and amount and nature of organic matter

Thus, at a certain temperature, the rate of de-oxygenation is assumed to be directly proportional to the amount of organic matter at that time

i.e., $\frac{dL_t}{dt} = -kL_t$ ....(2.2)

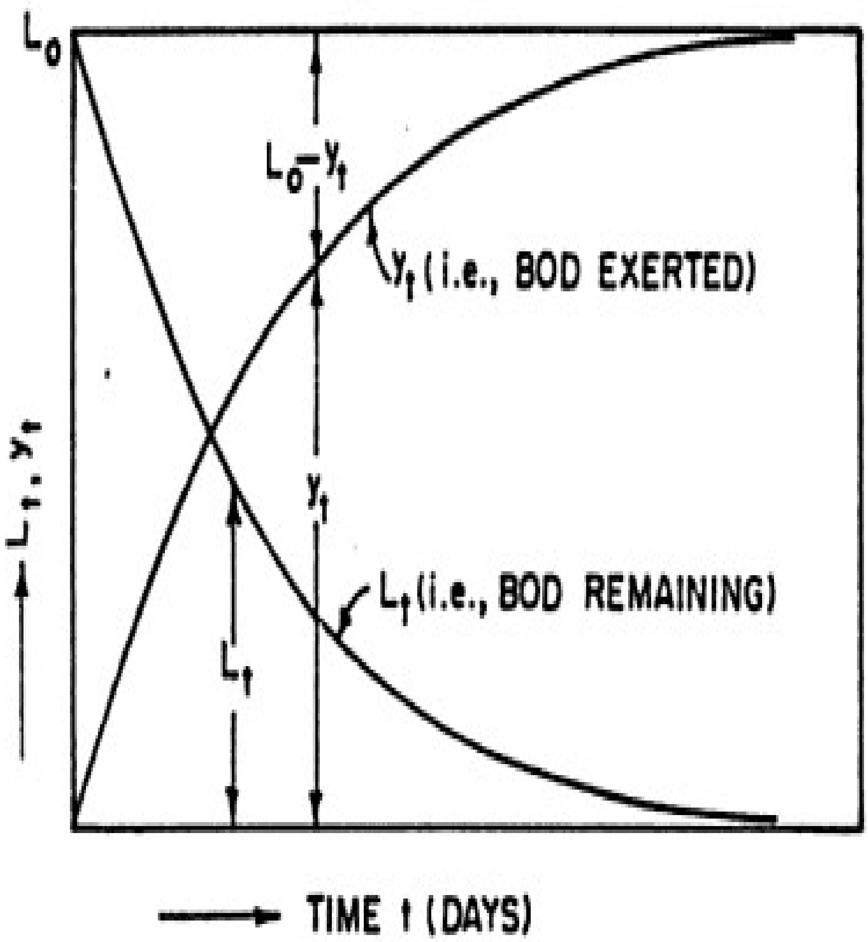


Fig. First stage BOD curve

Mathematical Model for the BOD Curve

Where,

- Lt = oxygen equivalent of carbonaceous oxidizable organic matter after t days from the start of oxidation (mg/l).
- $\frac{dL_t}{dt}$ = rate of disappearance of organic matter by aerobic biological oxidation
- t = time in days.
- K = rate constant (in per day)
- Minus sign indicates that with the passage of time (i.e., increase in t) the value of Lt decreases

Mathematical Model for the BOD Curve

☺Integrating the above equation (2.2), we get

$$\int \frac{dL_t}{dt} = \int -kL_t$$

$$log_e L_t = -kt + C \;\;\ldots\ldots\ldots(2.3)$$

Where, C is a constant of integration,

When t = zero (0), i.e. at start Lt = L. Substituting in the equation (2.3), we have

$$log_e L \quad = -k * 0 + C$$

$$C = log_e\, L$$

$$log_e L_t = -kt + log_e L$$

$$log_e L_t - log_e L \quad = -kt$$

Mathematical Model for the BOD Curve

$$log\frac{L_t}{L} = 2.3log_{10}\frac{L_t}{L} = -kt$$

$$log_{10}\frac{L_t}{L} = \frac{-kt}{2.3}$$

$$log_{10}\frac{L_t}{L} = -0.434kt \text{ ....(2.4)}$$

Using 0.434k = k_D is the De-oxygenation constant or the BOD rate constant,

$$log_{10}\frac{L_t}{L} = -k_D t$$

$$\frac{L_t}{L} = 10^{-k_D t} \text{ ....(2.5)}$$

$$L_t = L10^{-k_D t} \text{ ....(2.6)}$$

Mathematical Model for the BOD Curve

- Now, L is the initial organic matter (expressed as oxygen equivalent)
- Lt is the organic matter left after t days; which means that during t days,

the quantity of organic matter oxidized = L - Lt…..(2.6)

Let say the quantity of organic matter oxidized is Y_t

$$Y_t = L - L_t$$

$$Y_t = L\left(1 - \frac{L_t}{L}\right)$$

$$\frac{L_t}{L} = 1 - \frac{Y_t}{L}$$

But, also $\frac{L_t}{L} = 10^{-k_D t}$, from equation (2.5)

Mathematical Model for the BOD Curve

Therefore, $1 - \frac{Y_t}{L} = 10^{-k_D t}$

$\boldsymbol{Y_t = L(1 - 10^{-k_D t})}$**.......(2.7)**

$\boldsymbol{Y_t}$ is the oxygen absorbed in t days, i.e. BOD of t days.

e.g. $\boldsymbol{BOD_5 = Y_5 = L(1 - 10^{-5k_D})}$

The ultimate first stage BOD (Yu) would be obtained from the equation (2.7), substituting t =∞days in it.

$$Y_u = L(1 - 10^{-k_D \infty})$$

$\boldsymbol{Y_u = BOD_u = L}$ **......(2.8)**

Hence, the ultimate first state $\boldsymbol{BOD_u}$ (Yu) of a given sewage is equal to the initial oxygen equivalent of the organic matter present in this sewage (L)

Mathematical Model for the BOD Curv

- **K_D value**

The value of KD determines the speed of the BOD reaction, without influencing the ultimate BOD

The coefficient of de-oxygenation is different at different temperatures, but finally, Yu is constant.

$$K_{DT} = K_{D(20)} * 1.047^{T-20}$$

KD(20°) = De-oxygenation constant at 20°c. varies b/n (0.05 to 0.2) per day depending upon the nature of the organic matter.

KD(T) = De-oxygenation constant at température T°c.

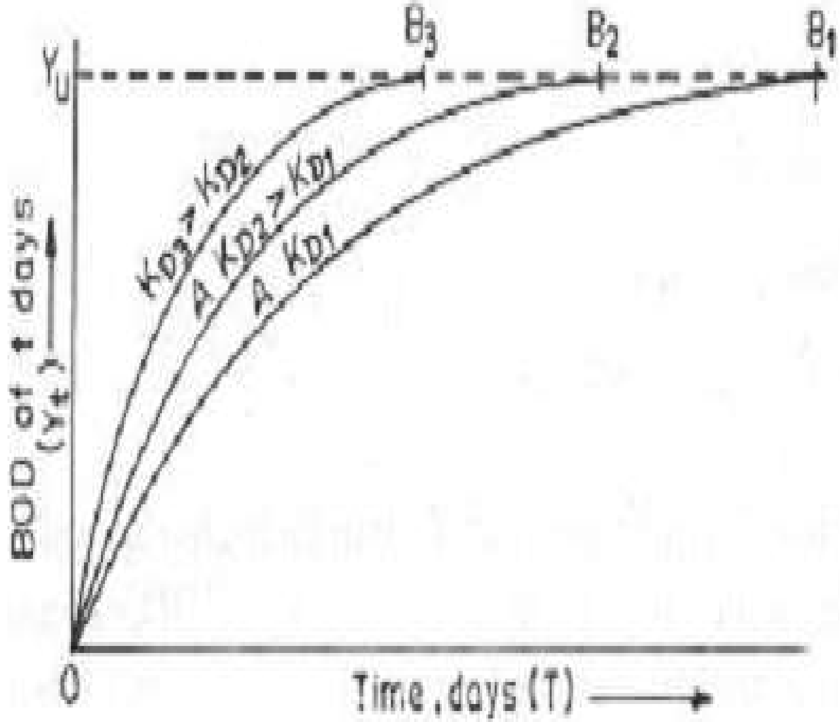


Fig. of BOD exertion as a function of K_D

Mathematical Model for the BOD Curve

- Table: Typical values of K_D at 20°c for various types of waters and WW

Water type	K_D value per day
Tap waters	< 0.05
Surface waters	0.05 - 0.1
Municipal wastewaters	0.1 - 0.15
Treated sewage effluents	0.05 - 0.1

Table: Sewage type depending on BOD value

Nature of sewage	5 day BOD at 20°C (ppm or mg/l)
1. Strong sewage	450 to 550
2. Average sewage	350
3. Weak sewage	250
4. Standard filter sewage effluent	20
5. Very good filter sawage effluent	5 to 10

Mathematical Model for the BOD Curve

Problems

1. In order to conduct a 5-day BOD test the sample of WW was diluted with specially prepared dilution water, with a dilution factor of 150. the contents of DO in the beginning and end of test were 11ppm and 7ppm respectively. Compute the 5-day BOD. What is the nature of the WW?

Solution:

$$\mathbf{BOD_5} = (\text{oxygen consumed}) * \text{dilution factor}$$

$$\mathbf{BOD_5} = (11\ - 7) * 150$$

$$\mathbf{BOD_5} = 600\text{ppm}$$

This indicates the WW is strong, need proper treatment before disposal.

Mathematical Model for the BOD Curve

2. Determine the ultimate BOD for a sewage have 5-day BOD at 20°c as 160ppm. Assume the de-oxygenation constant as 0.2per day.

Solution:

- we have, BOD_u = Yu = L
- From, $BOD_5 = Y_5 = L(1 - 10^{-5k_D})$
- $160 = L(1 - 10^{-5*0.2})$
- L = 177.78ppm

Mathematical Model for the BOD Curve

3. The BOD_5 of a wastewater is 150mg/l at 20°c. The k value is known to be 0.23 per day. What would BOD8 be, if the test was run at 15°c?

Solution: $BOD_8 = Y_8 = L(1 - 10^{-8k_D})$

$But,\ BOD_5 = Y_5 = L(1 - 10^{-5k_D})$ at 20°c

$$150mg/l = L(1 - 10^{-5*0.23})$$

L = 161mg/l

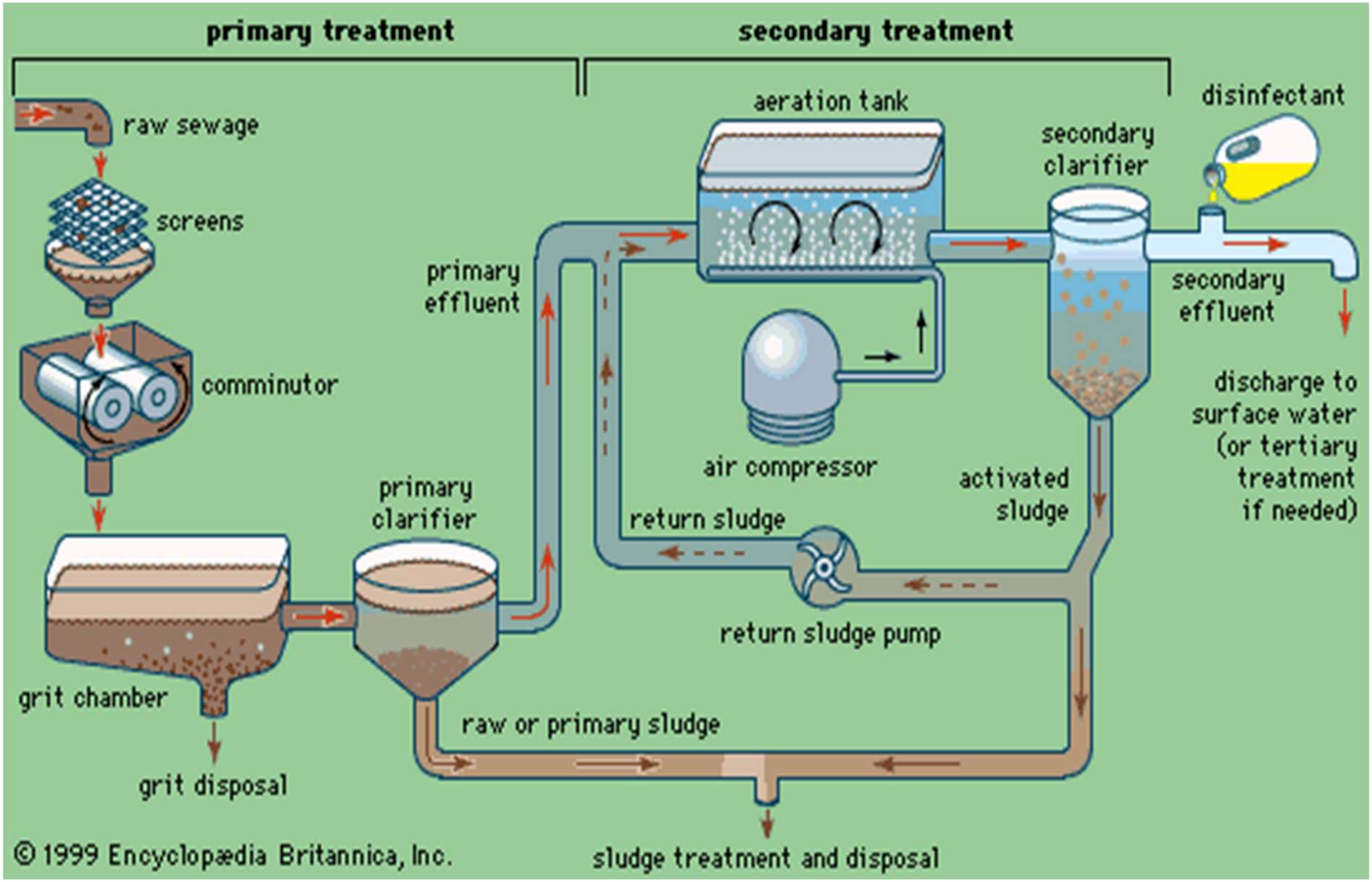
And $\mathbf{K_{D15} = K_{D(20)} * 1.047^{15-20}}$

$\mathbf{K_{D15} = 0.23 * 1.047^{15-20}}$ **= 0.18**

So, $BOD_8 = Y_8 = 161(1 - 10^{-8*0.18})$ = 156mg/l

PRELIMINARY AND PRIMARY WASTEWATER TREATMENT METHODS

Preliminary Treatment



Preliminary Treatment

- consists solely in separating the floating materials and heavy settleable inorganic solids.
- It also helps in removing the oils and greases, etc.
- reduces the BOD of the WW, by about 15 to 30%.

- The processes used are:
  1. Screening
  2. communitors
  3. Grit chambers or Detritus tanks
  4. Skimming tanks

Preliminary Treatment

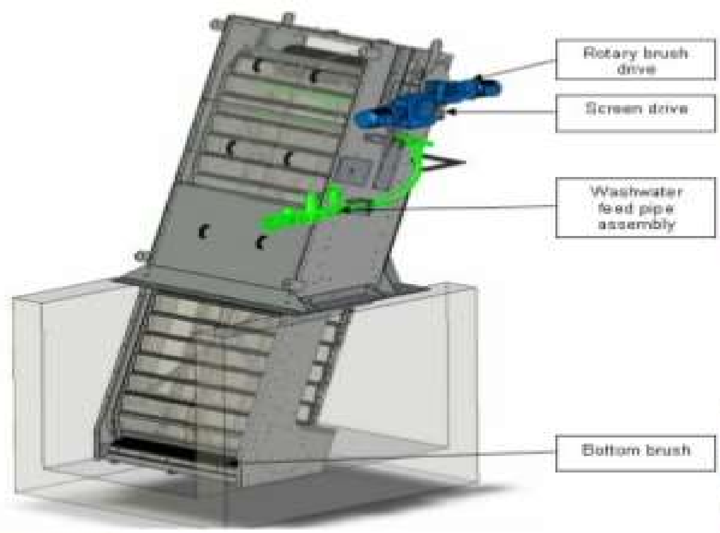
1) Screening

- ✓remove the floating matter, such as pieces of cloth, paper, wood, cork, hair, fiber, kitchen refuse, fecal solids, etc.
- ✓What if floating materials not removed?
  - will choke the pipes, or adversely affect the working of the sewage pumps.

- ✓Depending upon the size of the openings, screens may be classified as
  - coarse screens,
  - medium screens, and
  - fine screens.

Preliminary Treatment

- ✓**Coarse screens (Racks)**: the spacing between the bars (i.e. opening size) is about 50 mm or more.
- ✓These screens help in removing large floating objects.
- ✓collects about 6 liters of solids per million liter of WW



Preliminary Treatment

Medium screens: the spacing b/n bars is about 6 to 40 mm.

- These screens will ordinarily collect 30 to 90 liters of material per million liter of sewage.
- The screenings usually contain some quantity of organic material, which may putrefy and become offensive,

Fine Screens: have perforations of 1.5 mm to 3 mm in size.

- The installation of these screens proves very effective, and remove 20% of the suspended solids.
- get clogged very often, and need frequent cleaning.

Preliminary Treatment

Disposal of Screenings

- screenings is material separated by screens.
- It contains 85 to 90% of moisture and other floating matter.
- It may also contain some organic load which may putrefy, causing bad smells and nuisance.
- To avoid such possibilities, the screenings are disposed of either by:
  - burning,
  - burial, or
  - dumping.
- The dumping is avoided when screenings are from medium and fine screens,

Preliminary Treatment

2. Comminutors/shredder

- are the patented devices, which break the larger sewage solids to about 6mm in size,

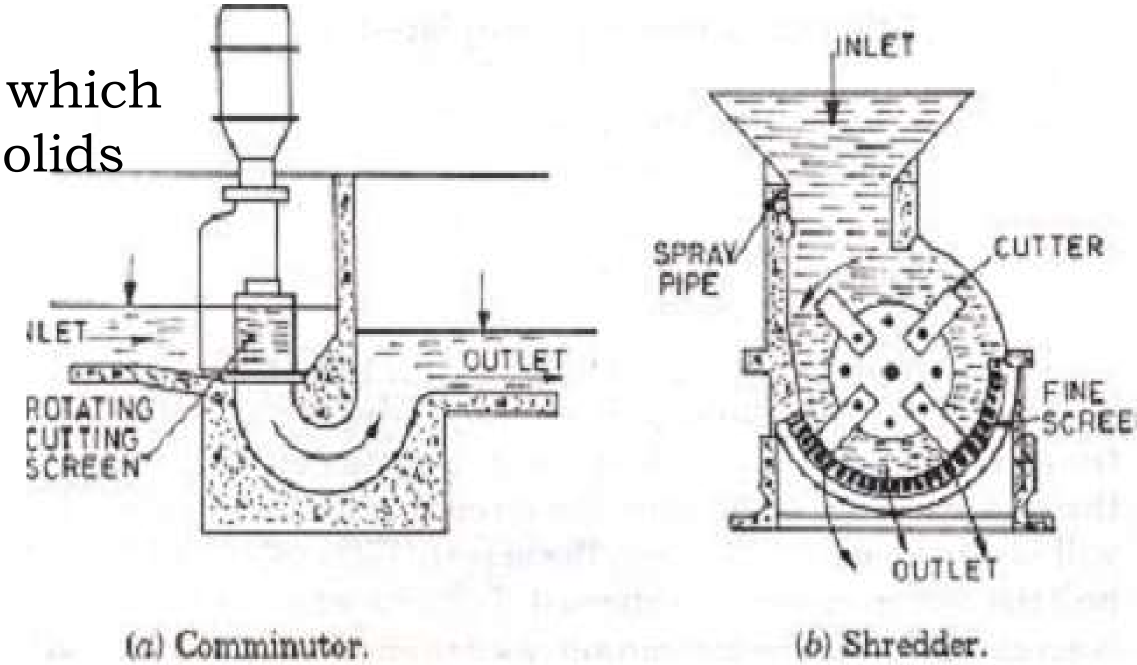


Figure of Comminutor and shredder

- eliminate the problem of disposal of screenings, by reducing the solids to a size which can be processed elsewhere in the plant.

Preliminary Treatment

3) Grit Removal Basins (grit chamber, detritus tank,..)

- are the **sedimentation basins** placed in front of the wastewater treatment plant.
- The grit chamber remove the inorganic particles (specific gravity about 2.65 and nominal diameter of 0.15 to 0.20mm or larger) such as sand, gravel, egg shells, bones, and other non-putresible materials
- Grit may clog channels or damage pumps due to abrasion, and to prevent their accumulation in sludge digesters.
- These inorganic materials is removed by the process of sedimentation due to gravitational forces.
- What is sedimentation process?
- The organic material is not allowed to settle in this process, as they cause septicity of sewage.

Preliminary Treatment

Constant Velocity Horizontal Flow Grit Chambers

- is an enlarged channel or a long basin, in which the cross-section is increased, so as to reduce the flow velocity of sewage.
- why flow velocity reduction is required?
  - to cause the settlement of the entire silt and grit present in sewage.
- The important point in the design of the grit basins is that the flow velocity should neither be too low nor should it be so high. Why?
  - if too slow lighter organic matter will settle down, septicity!!
  - if too high, even silt and grit not settle down.

Preliminary Treatment

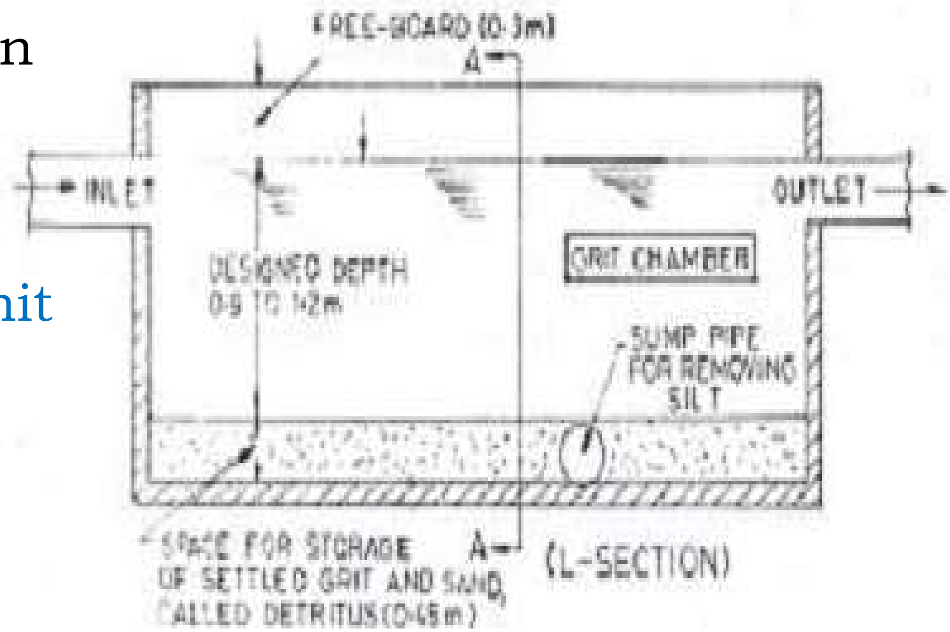
Design of a Rectangular Grit Chamber provided with a Proportioning Weir at Effluent End

- Two design parameters, detention time (d_t) and settling velocity (v_s).
- What is detention time?
  - Is the time required by WW to reach outlet from inlet
- What is settling velocity?
  - The velocity of grit by which it reaches the bottom of the chamber.
- After fixing the depth and the detention time, we can easily design the dimensions of a rectangular chamber,

$$length\ (l) = flow\ velocity\ (v_h) * d_t$$

Preliminary Treatment

- ✓ two to three separate chambers in parallel should provided.
  - One for low flow, other for peak flow
  - in manual cleaning as one unit can work, the other is shut down for cleaning.



The grit chambers can be cleaned periodically at about 3 weeks interval, either manually, mechanically or hydraulically.

proportional flow weir is provided for controlling velocity of flow.

Preliminary Treatment

Example:

1. A grit chamber is designed to remove particles with a diameter of 0.2mm, specific gravity 2.65. Settling velocity for these particles has been found to range from 0.016 to 0.022m/sec, depending on their shape factor. A flow through velocity of 0.3m/sec will be maintained by proportioning weir. Determine the channel dimensions for a maximum wastewater flow of 10,000cu m/day.

Solution:

Horizontal velocity of flow = V_h = 0.3m/sec.

Settling velocity is between 0.016 to 0.022 m/sec, and hence let it be 0.02m/sec.

Q = 10,000m3/d = 0.116m3/s

Q = velocity $*$ cross-section area = $V_h * A$

Preliminary Treatment

Therefore, $0.116 = 0.3 * A$

$$A = 0.385\ m^2$$

Assuming a depth of 1m, we have

$$A = depth\ (d)\ * width\ (B)$$

$1 * B = 0.385$

$B = 0.385m \cong 0.4m$

Settling velocity

$$Vs = 0.02m/sec$$

Detention time = $\frac{depth\ of\ the\ basin}{settling\ velocity} = \frac{1m}{0.02m/s} = 50sec.$

$L_{tank} = V_h *$ Detention time = $0.3m/s * 50s = 15m$

Hence, use a rectangular tank, with dimensions:

Length (L = 15m) Width (B = 0.4m) and Depth (D = 1.0m)

Preliminary Treatment

2. Design a suitable grit chamber cum Detritus tank for a sewage treatment plant getting a dry weather flow from a separate sewerage system @4001/s. Assume the flow velocity through the tank as 0.2m/sec and detention period of 2 minutes. The maximum flow may be assumed to be 3 times of dry weather flow.

Solution

Detritus tanks are nothing but rectangular grit chambers, designed to flow with a smaller flow velocity (of about 0.09m/sec) and longer detention periods (3 to 4 minutes)

The length of the tank

= Velocity ∗ Detention time = 0.2 ∗ (2 ∗ 60) = 24m

The discharge passing through each tank

= 400l/s = 0.4m3/sec

Preliminary Treatment

Therefore, Cross-sectional area required

$$A = \frac{discharge}{velocity} = \frac{0.4}{0.2} = 2m$$

Assuming the water depth in the tank to be 1.2m,

$$\text{The width of the tank} = \frac{cross-sectional\ area}{depth}$$

$$= \frac{2}{1.2} = 1.7m$$

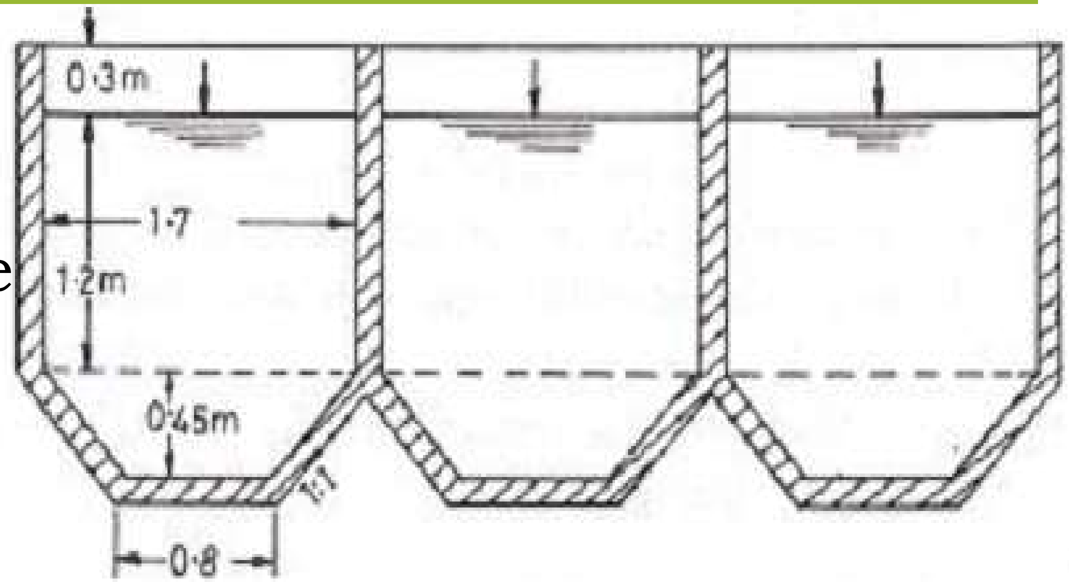
Hence, use a Detritus tank with 24m*1.7m*1.2m size.

At the top, a free-board of 0.3m may be provided; and at the bottom, a dead space depth of 0.45m for collection of detritus may be provided.

Thus, the overall depth of the tank = 1.2 + 0.3 + 0.45 = 1.95m.

Preliminary Treatment

The tank will be 1.7m wide up to 1.5m depth, and then the sides will slope down to form an elongated trough of 24m length and 0.8m width at the bottom with rounded corners, as shown in figure



Preliminary Treatment

4) Skimming Tanks

- ✓employed for **removing oils and grease** and placed before the sedimentation-tanks.
- ✓These oil and greasy materials may be removed in a skimming tank, in which
  - ✓**air is blown** by an aerating device through the bottom.
  - ✓The rising air tends to coagulate and congeal (solidify) the grease, and cause it to rise to the surface from where it is removed.
- ✓A detention period of about 3 - 5 min. is sufficient,

Preliminary Treatment

The surface area required for the tank can be found out by using the formula:

$$A = 0.00622 * \frac{q}{Vr}$$

Where, q = rate of flow in m3/day

Vr = minimum rising velocity of greasy material to be removed in m/minute

= 0.25m/minute in most cases

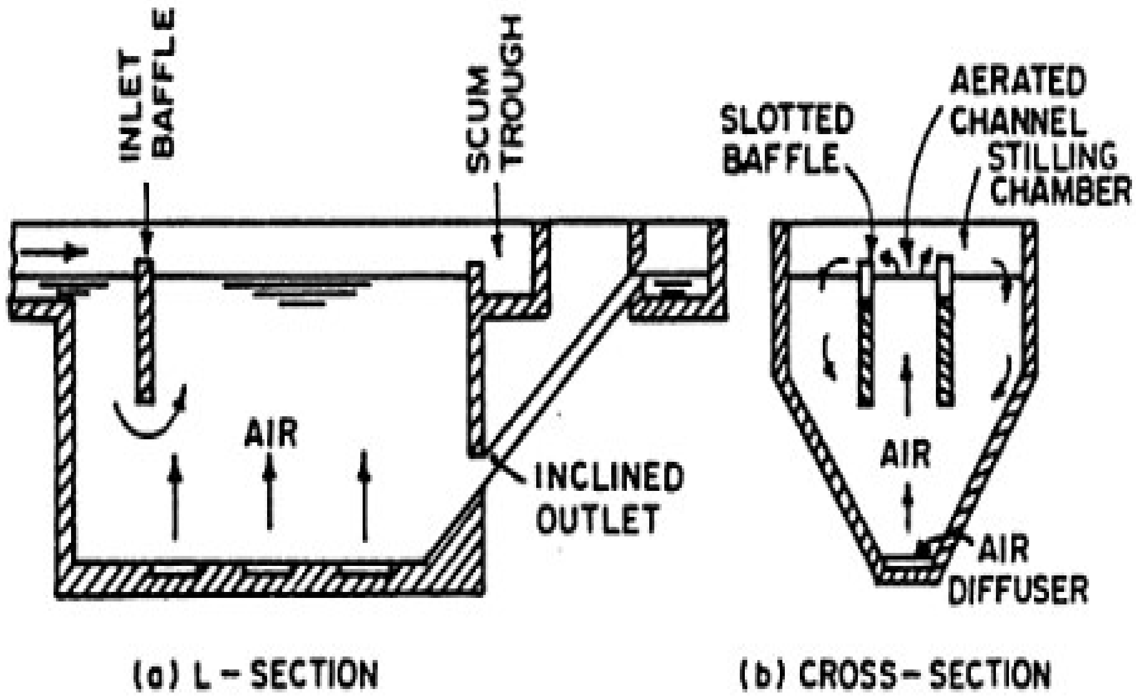


Figure of Skimming tank

Primary Wastewater Treatment

- consists in removing large suspended organic solids.
- This is usually accomplished by sedimentation in settling basins.
- What is sedimentation?

Primary Wastewater Treatment

Sedimentation

- is the **physical separation of suspended material** from water or wastewater by the action of gravity.
- SSs are separated by settling to the bottom of sedimentation tanks by gravitational force.
- **Sedimentation tanks** are tank designed to remove this organic matter from the sewage effluent coming out from the grit chambers.

Primary Wastewater Treatment

The principle behind sedimentation.

The very fundamental principle underlying the process of sedimentation is that the organic matter present in sewage is having specific gravity greater than that of water (i.e. 1.0).

- In still sewage, these particles will, tend to settle down by gravity;
- in flowing sewage, particles are kept in suspension, because of the turbulence in water.
- Hence, as soon as the turbulence is retarded by offering storage to sewage, these impurities tend to settle down at the bottom of the tank offering such storage.

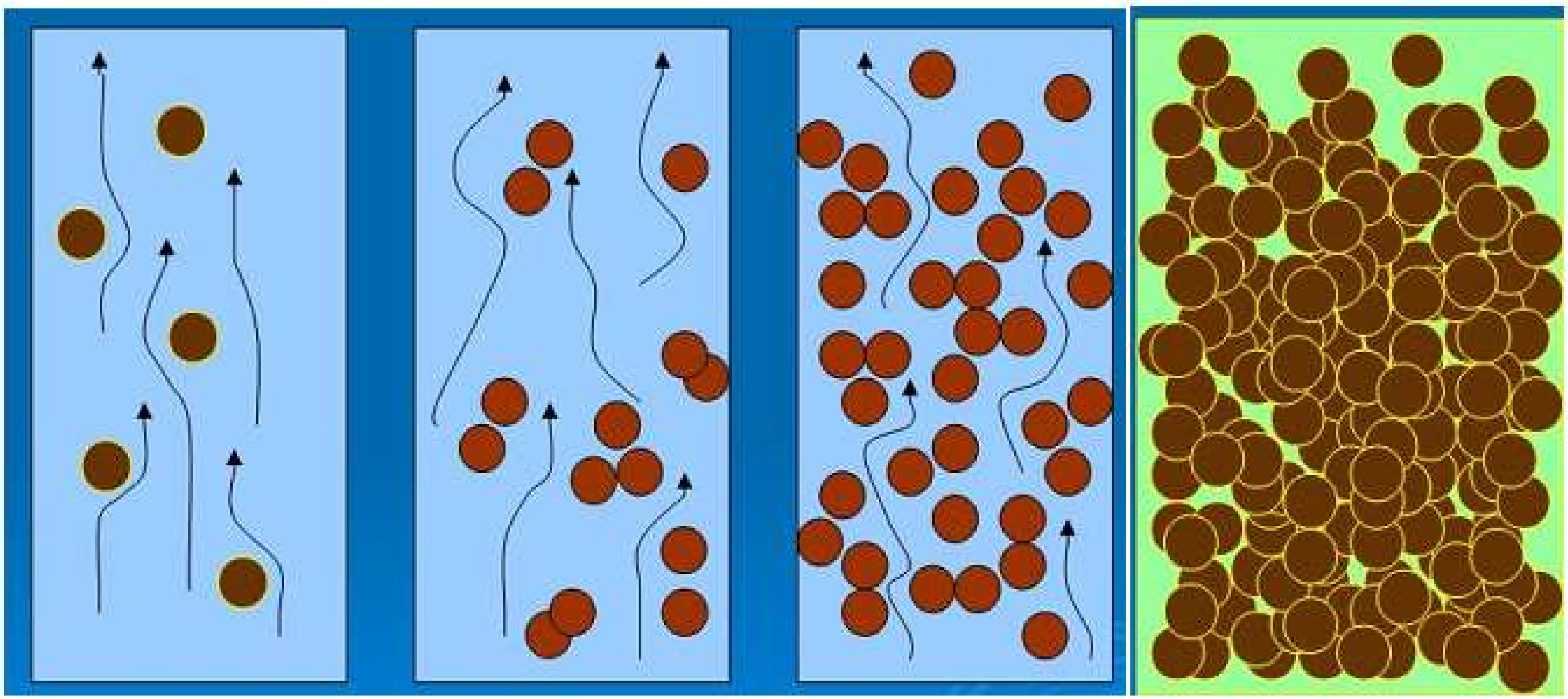
Primary Wastewater Treatment

Types of Settling

Depending on the particles concentration and the interaction between particles, 4 types of settling can occur: Discrete (type I), Flocculent (type II), Hindered (type III), Compression (type IV)

1. **Discrete particle settling**: The particles settle without interaction and occur under low solids concentration.
2. **Flocculent settling:** particles initially settle independently, but flocculate in the depth of the clarification unit.
   - The velocity of settling particles is usually increasing as the particles aggregates.

Primary Wastewater Treatment



Discrete Flocculent Hide hindered Compression

3. **Hindered/zone settling :** Inter-particle forces are sufficient to hinder the settling of neighboring particles. The particles tend to remain in fixed positions with respect to each others.

Primary Wastewater Treatment

4. **Compression settling:** occurs when the particle concentration is so high that so that particles at one level are mechanically influenced by particles on lower levels. The settling velocity then drastically reduces.

Primary Wastewater Treatment

Settling of Discrete Particles (Type I Settling)

- when discrete particles is placed in quiescent fluid, it will accelerate until the friction resistance (drag force, FD) is equal to the impelling force (driving force) acting on the particles.
- At this stage, the particles attain a uniform or terminal velocity and settles down with this constant velocity called settling velocity, V_s.
- The settling velocity of this particle is expressed by Stoke's law,

$$V_s = \frac{g}{18}(G-1)\frac{d^2}{v}$$

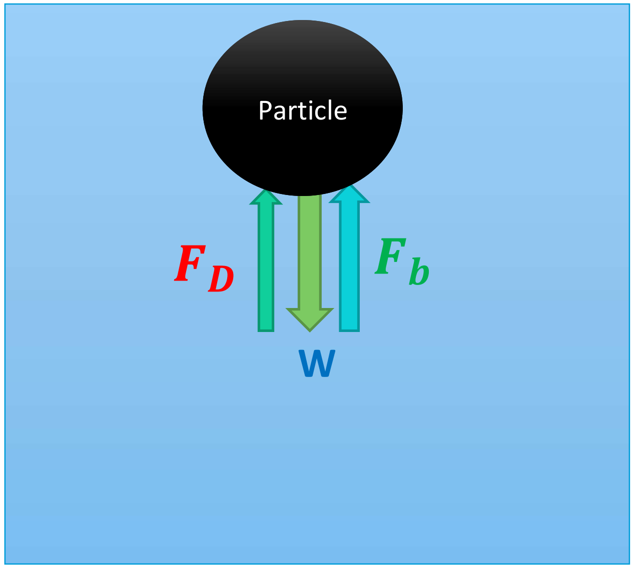
Primary Wastewater Treatment

Derivation of Stoke's Law: 3 forces on settling particle

Drag force, particle weight, **buoyancy force**

1. **The drag force:** is given by Newton's law, as

$$\text{Drag force}(F_D) = C_D A \rho_w \frac{v^2}{2}$$



Where, C_D = Coefficient of drag

A = Area of particle

ρ_w = Density of water

v = velocity of fall

Primary Wastewater Treatment

2. The weight of the particle (W) $= mg = v_p \rho_p g$

3. Force of buoyance (F_b) = $v_p \rho_w g$

Where, Vp= volume of particle

ρ_w= density of water

ρ_p= density of particles

upward and downward force will become equal when v (in drag force equation) becomes equal to v_s.

$$C_D A\rho_w \frac{v_s^2}{2} + v_p \rho_w g = v_p \rho_p g$$

$$C_D A\rho_w \frac{v_s^2}{2} = v_p \rho_p g - v_p \rho_w g$$

Primary Wastewater Treatment

$$C_D\, A * \rho_w * \frac{{v_s}^2}{2} = v_p g(\rho_p - \rho_w)$$

$$v_s^2 = \frac{2\, v_p g(\rho_p - \rho_w)}{C_D\, A * \rho_w}$$

For spherical particles

$$v_p = \frac{4}{3}\ \pi r^3 \text{ and } A = \pi r^2$$

$v_s^2 = \frac{4dg(\rho_p - \rho_w)}{3C_D\rho_w}$, d = particle diameter

$v_s^2 = \frac{4dg(G-1)}{3C_D}$ ... general equation to calculate settling velocity of particles

Primary Wastewater Treatment

The coefficient of drag (C_D) is a function of Reynolds number, R_e

$C_D = \frac{24}{R_e}$ (laminar flow or streamline flow)

Where, R_e is the particle Reynolds number = $\frac{v_s d}{\upsilon}$

Therefore, the above equation then becomes,

$$v_s^2 = \frac{4}{3}\frac{g(G-1)d}{\frac{24}{R_e}}$$

$$v_s^2 = \frac{4}{3} * g(G-1)d * \frac{R_e}{24}$$

$$v_s^{\cancel{2}} = \frac{g}{18} * (G-1)d * \frac{\cancel{v_s}d}{\upsilon}$$

$where, \upsilon = kinematic\ viscosity\ of\ liquid$ (m²/s)

Primary Wastewater Treatment

$$v_s = \frac{g}{18} * (G - 1) * \frac{d^2}{\upsilon}$$

The above Stoke's equation is valid for particles of size less than 0.1mm; in which case, the viscous force predominates over the inertial force, leading to what is known as streamline settling

a) For transition settling (d between 0.1mm and 1.0mm)

Here $1 < R_e < 10^3$

$$C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34$$

b) For turbulent settling (d > 1.0mm)

Here $R_e < 10^3$ $C_D = 0.34 \text{ to } 0.4$

For turbulent settling, the v_s reduces to:

$$v_s = 1.8\sqrt{gd * (G - 1)}$$

Primary Wastewater Treatment

Sedimentation Tanks

- are designed for effecting settlement of particles by reducing the flow velocity or by detaining the sewage in them.
- They are generally made of reinforced concrete and may be rectangular or circular in plan.
- Long narrow rectangular tanks with horizontal flow are generally preferred to the circular tanks with radial or spiral flow

Sedimentation basin has four district zones: inlet, settling, outlet and sludge zone

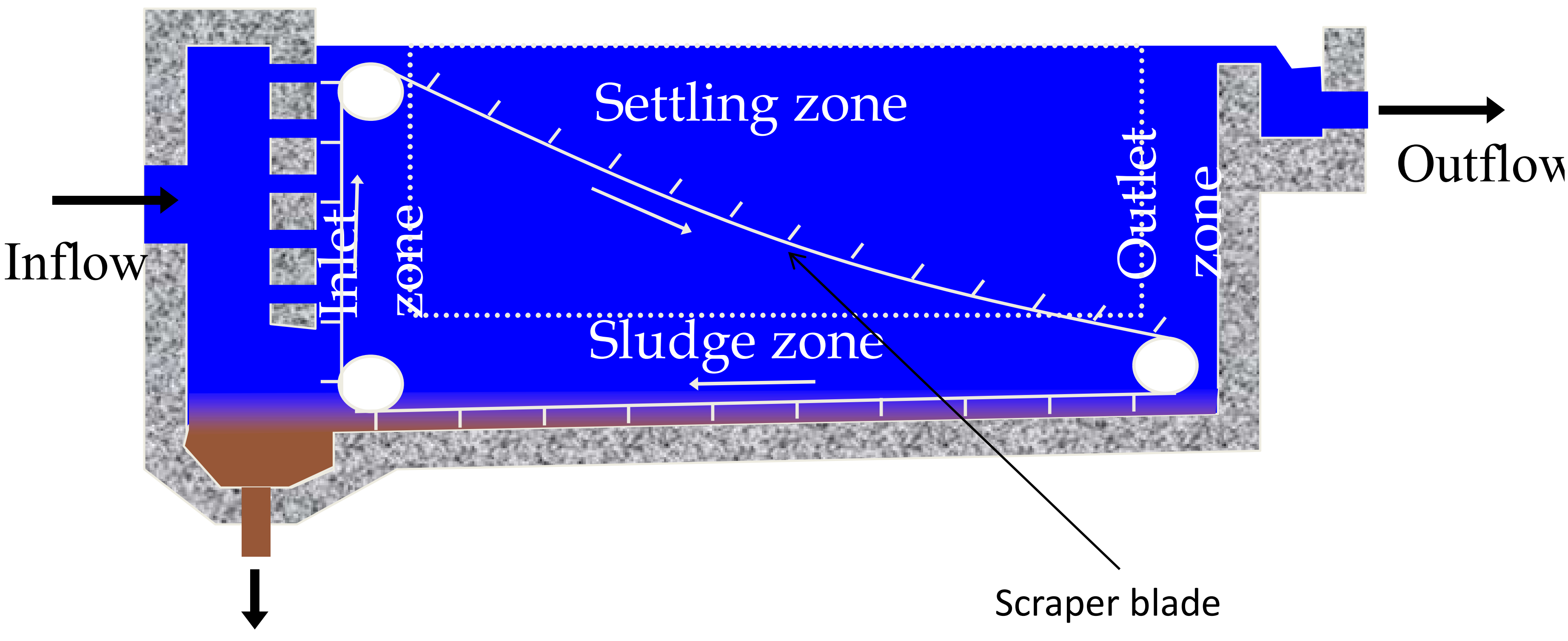


Fig. of rectangular sedimentation tank

Primary Wastewater Treatment

- Sedimentation tanks may function either intermittently or continuously.
- The **Intermittent settling** tanks are simple settling tanks which store sewage for a certain period and keep it in complete rest.
- In a **continuous flow** the flow velocity is only reduced, and the sewage is not brought to complete rest, as is done in an intermittent type.

Primary Wastewater Treatment

Design of a Continuous Flow Sedimentation Tank

Depends on the following assumptions:

1. Particles settles in settling zone of the tank
2. The flow is horizontal and steady and the velocity is uniform in all parts of settling zone for time equal to $\mathbf{t_d}$.
3. The concentration of suspended solid of each size is the same at all points of the vertical cross-section at the inlet end.
4. A particle is removed when it reach the bottom of settling zone.

Primary Wastewater Treatment

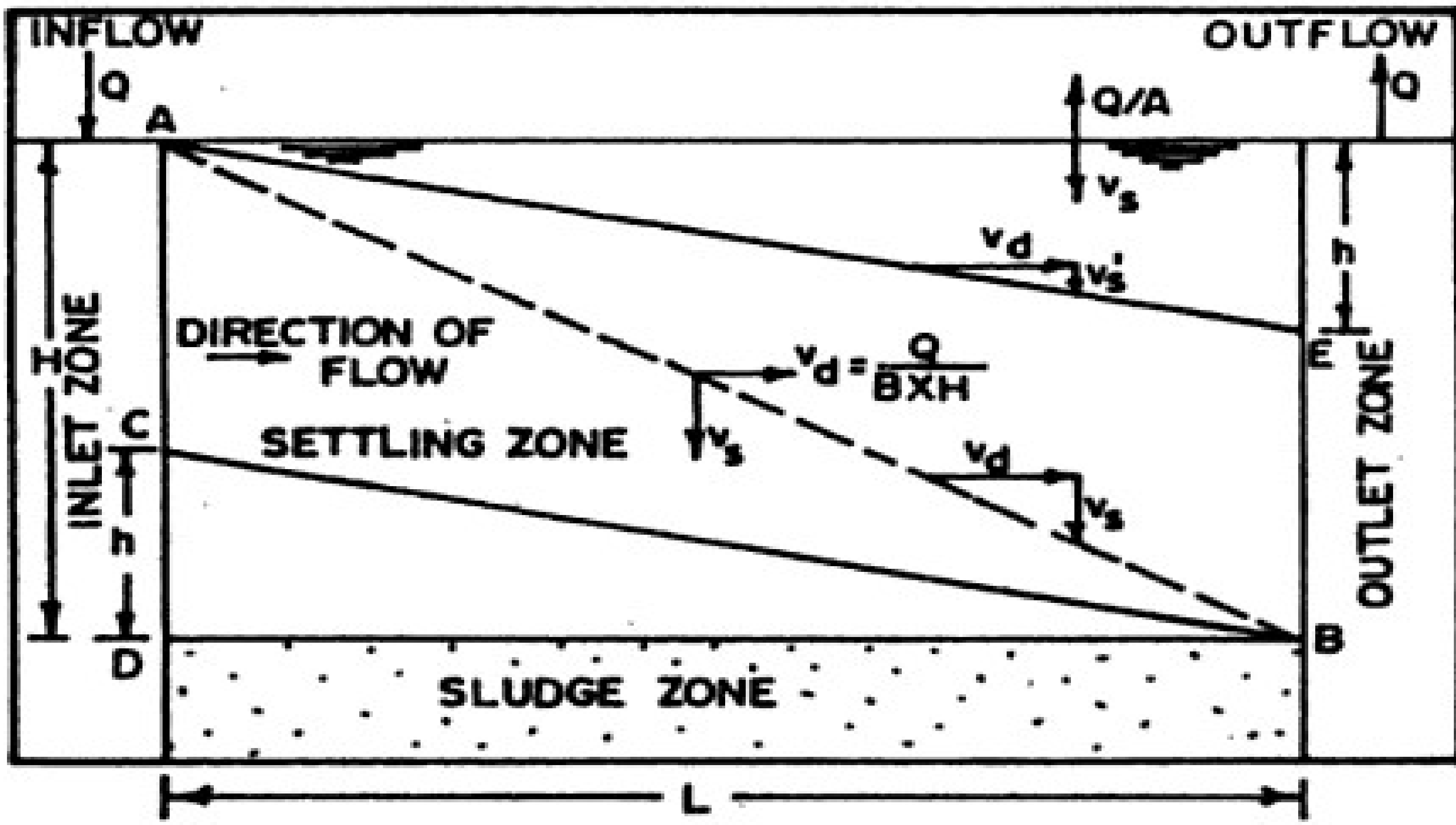


Fig. of vertical cross-section of the tank

Primary Wastewater Treatment

- Let L and H be the length and depth, respectively of the settling zone. Let B be the widht of the tank and Q be the discharge rate. Then, the horizontal velocity is:

$$\mathrm{V_h} = \frac{\text{discahrge}}{\text{cross\_scetional area}} = \frac{\mathrm{Q}}{\mathrm{BH}} \ldots\ldots\ldots\ldots 3.1$$

- And the time of horizontal flow is:

$$t_d = \frac{L}{V_h} = \frac{LBH}{Q} = \frac{V}{Q} \ldots\ldots\ldots\ldots\ldots 3.2$$

- The time for falling through distance H will be:

$$t_d = \frac{H}{V_s} \ldots\ldots\ldots\ldots\ldots\ldots .3.3$$

Equating equation 3.2 and 3.3:

$$\frac{H}{V_s} = \frac{LBH}{Q}$$

Primary Wastewater Treatment

$V_s = \frac{Q}{LB} = \frac{Q}{A_s} \ldots\ldots\ldots\ldots\ldots 3.4$, A_s is surface/plan area

Equation 3.4 defines the overflow rate or overflow velocity

And suggests all particle with $V_s > \frac{Q}{A_s}$ will reach the bottom before the end outlet of the tank.

- If a smaller particle having $V_s' < \frac{Q}{A_s}$ enters the tank at point C it will settle through only at height h. then,

$$t_d = \frac{h}{V_s'} = \frac{LBH}{Q}$$

Primary Wastewater Treatment

$$h = V_s' \frac{LBH}{Q} = \frac{V_s'}{V_s} H \ldots\ldots\ldots\ldots 3.5$$

Therefore, these particles will not settle if they enter the basin above point C.

Primary Wastewater Treatment

Table. Design criteria for primary sedimentation tank

parameter	value	
	range	typical
Detention time, hr	1.5-2.5	2
Overflow rate,$m^3/m^2/d$		
Average flow	32-48	
Peak flow	80-120	100
Weir loading, m3/m/d	125-500	250
Dimensions, m		
Rectangular		
Depth	3-5	3.6
Length	15-90	25-40
Width*	3-24	6-10
Sludge scrapper speed, m/min	0.6-1.4	1
Circular		
Depth	3-5	4.5
Diameter	3.6-60	12-45
Bottom slope, mm/m	60-160	80
Sludge scrapper speed, m/min	0.02-0.05	0.03

*must divide into bays of not greater than 6m wide

Source: Howard S. Peavy

Primary Wastewater Treatment

Constructional Details of the Sedimentation Tanks

a. Inlet and Outlet Arrangement

- In order to reduce short circuiting and to distribute the flow uniformly proper arrangement must be made for smooth entry of water.
- A most suitable type of an inlet for a rectangular settling tank is in the form of a channel extending to full width of the tank, with a submerged weir type baffle wall.
- A similar type of outlet arrangement is also used these days. It consists of an outlet channel extending for full width of the tank

Primary Wastewater Treatment

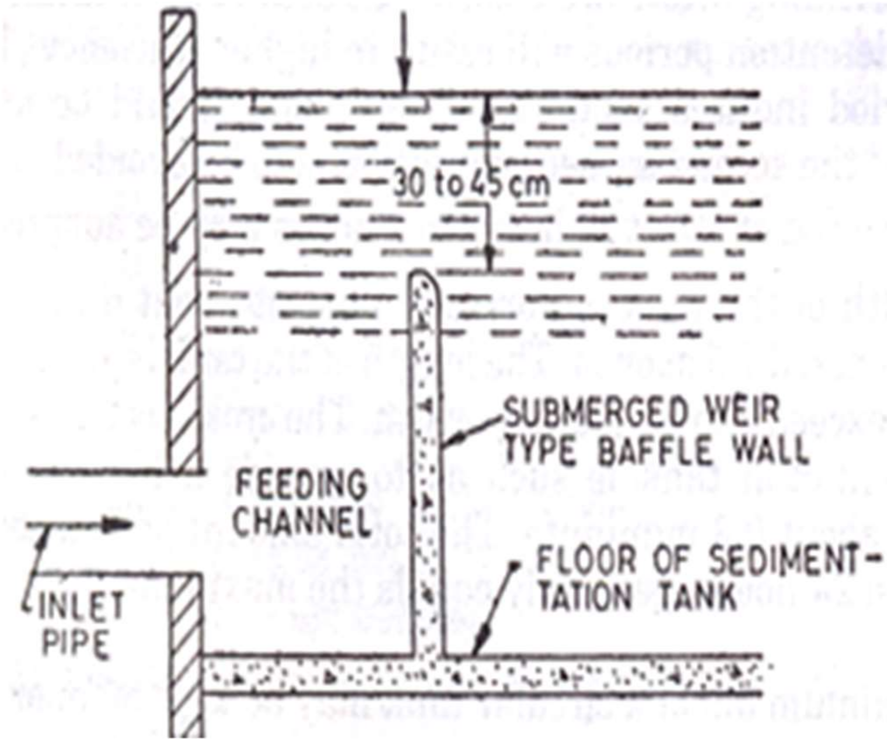


Fig. Section of a submerged type or a weir type inlet

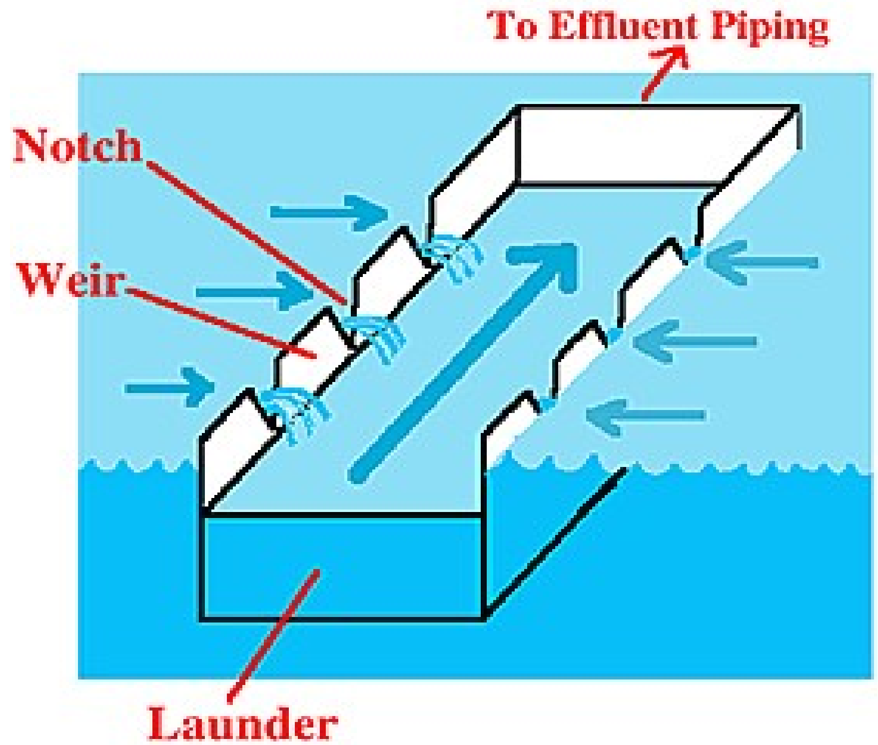


Figure of Weir type outlet

Primary Wastewater Treatment

b. Baffles

- Baffles are required to distribute the sewage uniformly through the cross-section of the tank, and thus to avoid short circuiting.
- Both inlets and outlets are, therefore, protected by hanging baffles,

c. Skimming Troughs

- When the amount of oils and greasy matter is small, a skim trough is generally provided in the sedimentation tank itself, near its outlet end,

Primary Wastewater Treatment

d. Cleaning and Sludge Removal

- the deposited sludge should have to removed before it becomes stale and septic.
- It also reduces the capacity of the tank and its detention period,
- it leads to the evolution of foul gases formed due to the anaerobic decomposition.
- Modern sedimentation tanks, however, are generally provided with mechanical cleaning devices (scraped by scrapers).

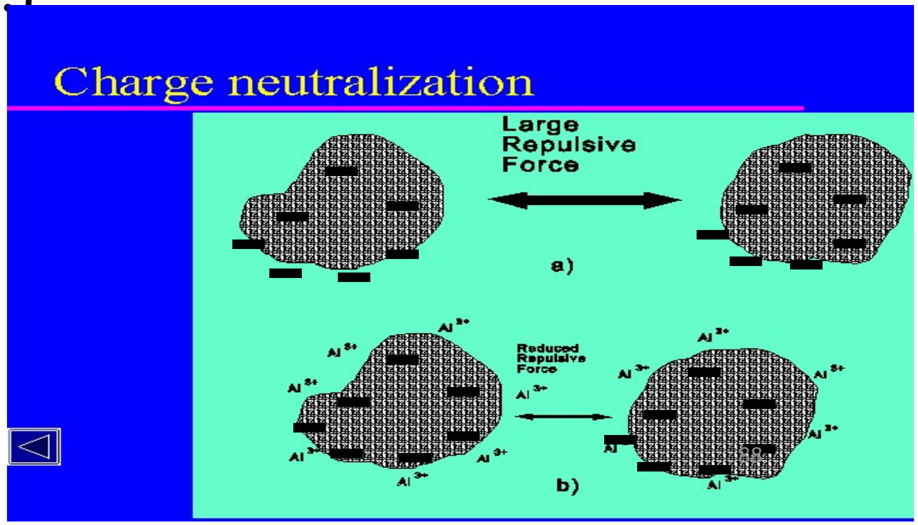
Primary Wastewater Treatment

Sedimentation Aided with Coagulation

What is Coagulation?

- **Coagulation** is the destabilization of colloids (very fine SS) by addition of chemicals that neutralize the negative charges
- Colloids have a net negative surface charge
- The chemicals are known as coagulants, usually higher valence cationic salts (Al^{3+}, Fe^{3+} etc.)

• Coagulation is essentially a chemical processes



Primary Wastewater Treatment

- Then, the destabilized particles agglomerate into a large size particles known as flocs which can be effectively removed by sedimentation or flotation.
- The process of forming flocs is called flocculation.

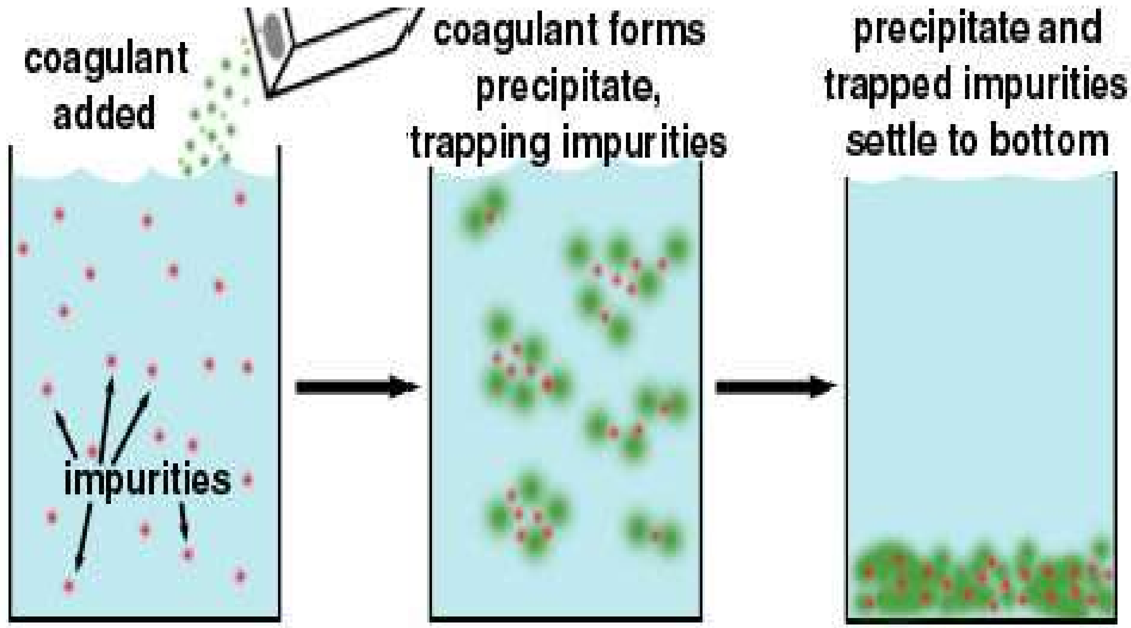


Fig. of sedimentation by coagulation

Primary Wastewater Treatment

Why coagulation and flocculation?

Various sizes of particles in raw water

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

GravIty settlIng

Colloids – so small: gravity settling not possible

Primary Wastewater Treatment

Table of Properties of the important coagulants used in sewage treatment

S. No.	Name of coagulant	B.O.D. removed as %age of total present	SS removed as %age of total Present	Dosage required in ppm	pH value required for proper functioning	Remarks
I.	Ferric chloride	80 - 90	90 - 95	25 - 35	5.5 to 7.0	This coagulant is widely used for sewage treatment, wherever, coagulation is adopted.
2.	Ferric sulphate with lime	60	80	35 - 40	8.0 to 8.5	Ferric sulphate has been found to be more effective than chlorinated copperas, if used in conjunction with lime. Hence ferric chloride and ferric sulphate are mainly used, as coagulants in sewage.
3.	Alum	60	80	40 - 90	6 to 8.5	It is generally not used in sewage although used for treating water supplies on a large scale.
4.	Chlorinated copperas	70 - 80	80 - 90	35 - 80	5.5 to 7.0 and 9.0 to 9.5	This coagulant is effective for producing sludge for activated sludge process.

Primary Wastewater Treatment

Problems

1. Find the terminal settling velocity of a spherical particle with diameter 0.5mm and specific gravity of 2.65 settling through water at 20°C

Solution:

Step 1: Assume laminar flow, then ρ_w = 998.2kg/m^3 and dynamic viscosity= 1.002*$10^{-3}N.s/m^2$ at 20°C

$$\mathrm{v_s} = \frac{\mathrm{g}}{18} * (\mathrm{G} - 1) * \frac{d^2}{\upsilon} = \frac{g}{18} * (\rho_p - \rho_w) * \frac{d^2}{\mu}$$

$$v_s = \frac{9.81m/s(2650 - 998.2)kg/m^3 * (5 * 10^{-4})^2 m}{18 * 1.002 * 10^{-3} N.s/m^2}$$

$$v_s = 0.22m/s$$

Step 2: Check Reynolds number

Primary Wastewater Treatment

$$\text{Re}= \frac{0.22m/s*\left(5*10^{-4}\right)\ \ m*998.2kg/m^{3}}{1.002*10^{-3}N.s/m^{2}}$$

Re = 112 which is transition flow

Re < 1…laminar flow

Re >10^4 … $turbulent\ flow$

1< Re < 10^4…transition flow

Step 3: $C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34 = \frac{24}{112} + \frac{3}{\sqrt{112}} + 0.34 = 0.84$

Step 4: using general eqn.$v_s^2 = \frac{4}{3} * 9.81 * \frac{2650-998.2}{0.84*998.2} * 5 * 10^{-4}$

$$v_s \ \ = 0.11m/s$$

Primary Wastewater Treatment

Step 5: With v_s =0.11 repeat step 2, 3 and 4

Re = 55

C_D= *1.18*

$v_s = 0.10$m/s

Primary Wastewater Treatment

2. A municipal wastewater treatment plant processes an average flow of 5000m3/d, with peak flow as high as 12,500m3/d. design a primary clarifier to remove approximately 60 percent of the suspended solids at average flow.

Solution:

Assume overflow rate of 35m3/m2.d

$$A_s = \frac{Q}{q_o} = \frac{5000m3/d}{35m3/m2.d} = 143m2$$

Assume tank shape as circular,

$$A_s = \frac{\pi d^2}{4}$$

Primary Wastewater Treatment

$$143 = \frac{\pi d^2}{4}$$

d=13.5m

Assume depth 3m

$$volume = 143 * 3m = 429m3$$

And detention time

$$t_d = \frac{volume}{discharge} = \frac{429m3}{5000m3/d} = 0.09d = 2.06hr$$

Exercise: redesign using peak flow and compare the result.

Primary Wastewater Treatment

1. A city must treat about 15,000m3/d of water. Flocculating particles are produced by coagulation and column analysis indicates that an overflow rate of 20m/d will produce satisfactory removal at a depth of 3.5m. Determine the size of the required settling tank. And check detention time, check horizontal velocity.

   Take length to width ratio as 3/1.

Primary Wastewater Treatment

Solution:

I. Compute surface area (provide two tanks at 7500m3/d each)

$$Q = q_o A_s$$

$$7500m3/d = 20m/d * A_s$$

$$A_s = 374m2$$

II. length-to-width ratio of 3/1, calculate surface dimensions.

w*3W = 375m2

Width= 11m

Length=34

Primary Wastewater Treatment

III. Check retention time.

$$t = \frac{volume}{flow\ rate} = \frac{11m * 34 * 3.5}{\frac{7500m}{d} * \frac{1d}{24hr}}$$

$$t = 4.19hr$$

IV. check horizontal velocity

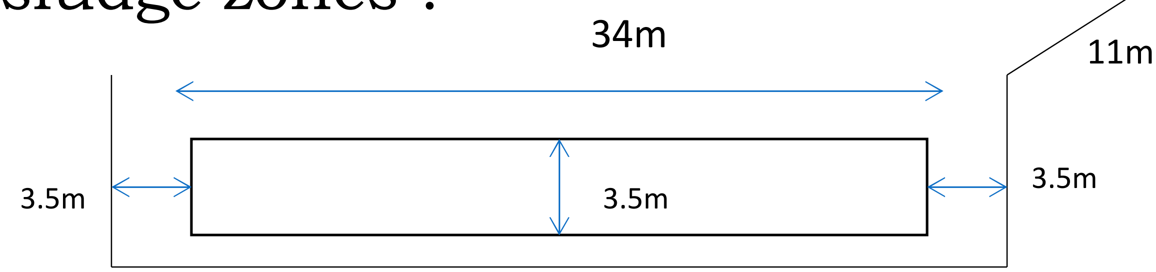
$$v_h = \frac{Q}{A_x} = \frac{\frac{7500m^3}{d} * \frac{1d}{24hr}}{11m * 3.5m} = 8.1m/hr$$

Primary Wastewater Treatment

V. check weir overflow rate. If simple weir is placed across end of the tank, overflow length will be 11m and overflow rate will be

$$\frac{7500m3}{d} * \frac{1d}{24hr} * \frac{1}{11m} = 28.4\frac{m^3}{hr.m}$$

VI. Add inlet and outlet zones equal to depth of tank and sludge zones .



Depth settling zone plus 0.5 freeboard plus 0.5 sludge zone.

Primary Wastewater Treatment

Problems

1. Design a suitable rectangular sedimentation tank (provided with mechanical cleaning equipment) for treating the sewage from a city provided with an assured public water supply system, with a maximum daily demand of 12 million liters per day. Assume suitable values of detention period and velocity of flow in the tank. Make any other assumptions, wherever needed.

Take detention period 2.5 hrs.

CHAPTER 4

SECONDARY/BIOLOGICAL AND TERTIARY WASTEWATER TREATMENT

The Role of Microorganisms in Wastewater Treatment

- Micro-organisms are important in the treatment of WW.
- They (bacteria, fungi, protozoa, and crustaceans) play an essential role in the conversion of organic waste to more stable or less polluting substances.

How?

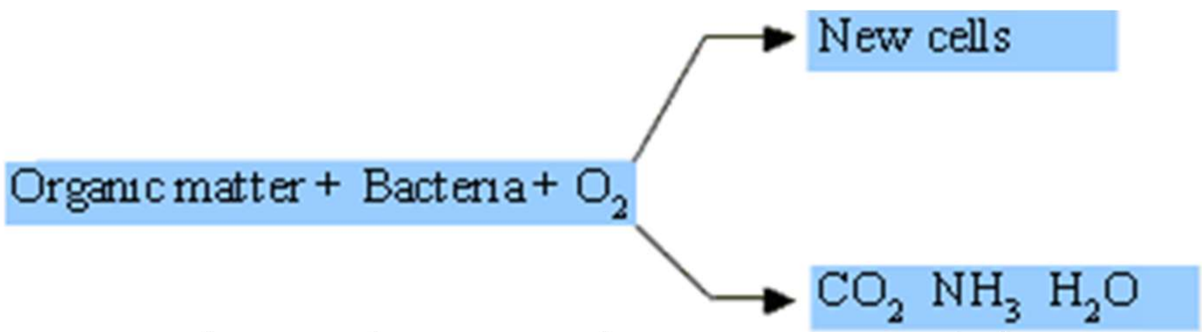
- Waste from humans is a useful food substrate for the micro-organisms.
- And they require cellular building blocks, such as (carbon) C, (hydrogen) H, (oxygen) O, (nitrogen) N, (phosphorus) P, and minerals for growth.
- These can be obtained through consuming organic substances containing these elements, or from inorganic materials, such as carbon dioxide, water, nitrate and phosphate.

The Role of Microorganisms in Wastewater Treatment

- Micro-organisms also require energy.
- They obtain this through respiration. In this process organic carbon is oxidized to release its energy.
- Oxygen or other hydrogen acceptors is needed for the respiration process.
- Algae and photosynthetic bacteria utilize energy from sunlight, while certain types of bacteria can utilize energy from chemical reactions not involving respiration.
- These building blocks and energy are used to synthesize more cells for growth and also for reproduction.
- Finally new cells die-off and settle down/removed by other means.

The Role of Microorganisms in Wastewater Treatment

- Three types of processes to represent the conversion of organic wastes by micro-organisms.
    1. Aerobic oxidation : utilize oxygen to oxidize organic substances to obtain energy for maintenance, mobility and the synthesis of cellular material.



    2. anaerobic oxidation: utilize nitrates, sulphates and other hydrogen acceptors to obtain energy for the synthesis of cellular material from organic substances.

The Role of Microorganisms in Wastewater Treatment

- **Methane (CH_4)** is a source of heat but, if released to the atmosphere without being combusted, it contributes to the greenhouse gas effect.
- **hydrogen sulphide (H2S)** contributes to WW odour.

3. **photosynthetic.**

$$aCO_2 + rH_2O + tNH_3 \xrightarrow{\text{Sunlight}} C_w H_x O_y N_z + bO_2$$

Microbial Growth Kinetics

- Growth of a microbial population is defined as an increase in numbers or an increase in microbial mass.
- Growth rate is the increase in microbial cell numbers or mass per unit time.
- Microbial populations can grow as:
    1. batch cultures (closed systems) or
    2. continuous cultures (open systems).

Microbial Growth Kinetics

1. Batch Cultures

- When a suitable medium is inoculated with cells, the growth of the microbial population shows four distinct phases.

The lag phase is a period of cell adjustment to the new environment

At **log/exponential phase** the number of cells increases exponentially

At **stationary phase** microorganisms cannot grow indefinitely, mainly because of lack of nutrients and electron acceptors, and the production and the accumulation of toxic metabolites.

During **death phase**, the death (decay) rate of the microbial population is higher than the growth rate.

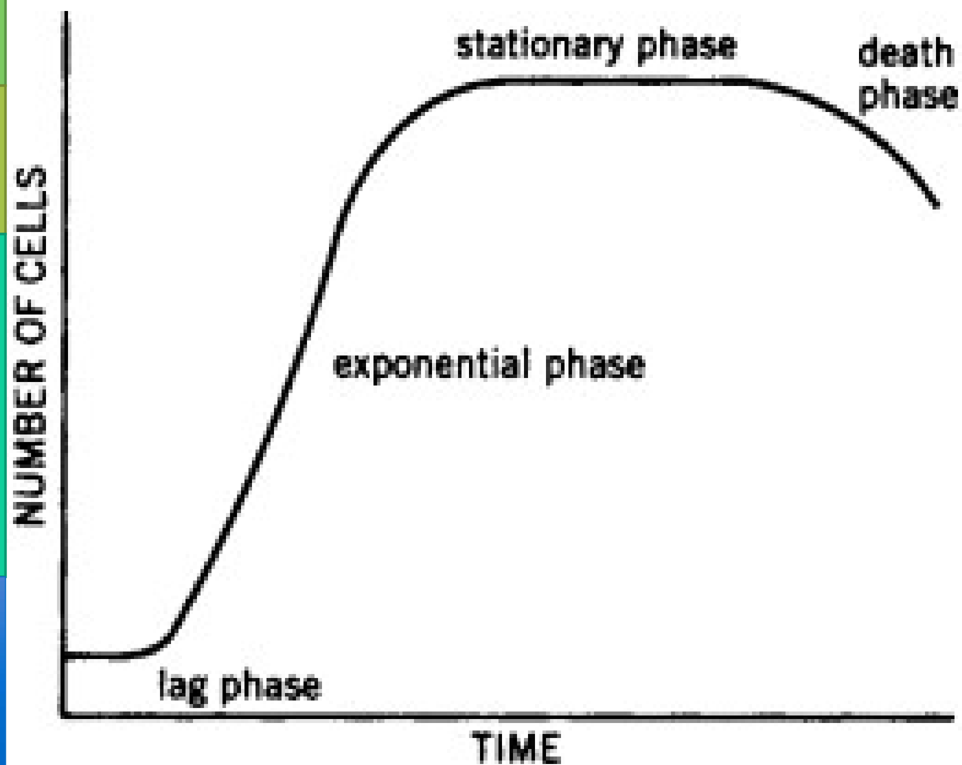
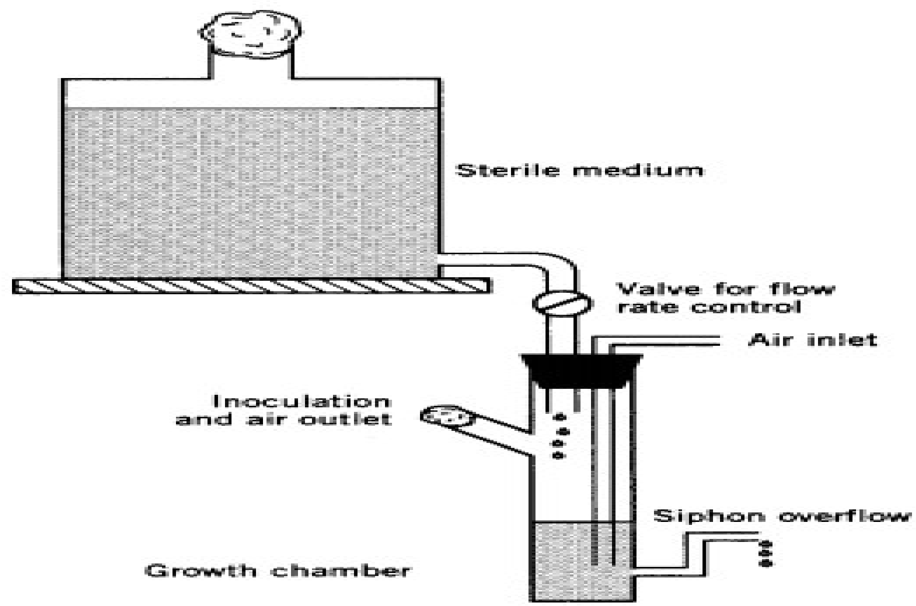


Figure of Microbial growth curve

Microbial Growth Kinetics

2. Continuous Culture of Microorganisms

- the exponential growth phase over a long period of time can be achieved by growing continuously the cells in a completely mixed reactor in which a constant volume is maintained.
- The most commonly used device is the chemostat



Microbial Growth Kinetics

Physical and Chemical Factors Affecting Microbial Growth

1. Substrate Concentration
2. Temperature
3. pH
4. Oxygen Level

Biological Wastewater Treatment

- to introduce WW contact with bacteria (cells) which feed organic matter in WW.
- the purpose of biological treatment is BOD reduction.

Principle:

→ Simple bacteria (cells) eat the organic material.

→ Through their metabolism, the organic material is transformed into cellular mass,

→ This cellular mass can be precipitated at the bottom of a settling tank or retained as slime on solid surfaces or vegetation in the system.

→ Then, the WW exiting the system is much clearer than it entered.

→ Cells need oxygen to breath, so adequate supply of O2 should there for the operation biological WWT.

Biological Wastewater Treatment

The common methods of biological wastewater treatment are:

a) Aerobic processes such as **trickling filters**, rotating biological contactors, **activated sludge process, oxidation ponds and lagoons**, oxidation ditches, **constructed wetland**

b) Anaerobic processes such as anaerobic digestion, and

c) Anoxic processes such as denitrification.

Biological Wastewater Treatment

1. TRICKLING FILTERS

- ✓ consist of tanks of coarser filtering media(see figure-next page).

I. Principles of operation

- ✓ over tanks of coarser filtering media the WW is allowed to sprinkle or trickle down, by means of spray nozzles or rotary distributors.
- ✓ The percolating sewage is collected at the bottom of the tank through a well designed under-drainage system.
- ✓ sufficient quantity of oxygen is supplied by providing suitable ventilation facilities in the body of the filter
- ✓ The purification is done mainly by the aerobic bacteria, which form a bacterial film around the particles of the filtering media.
- ✓ The effluent must be taken to the secondary sedimentation tank for settling out the organic matter oxidized while passing down the filter.

Biological Wastewater Treatment

Figure of Photographic view of trickling filter with its rotary distributors

Biological Wastewater Treatment

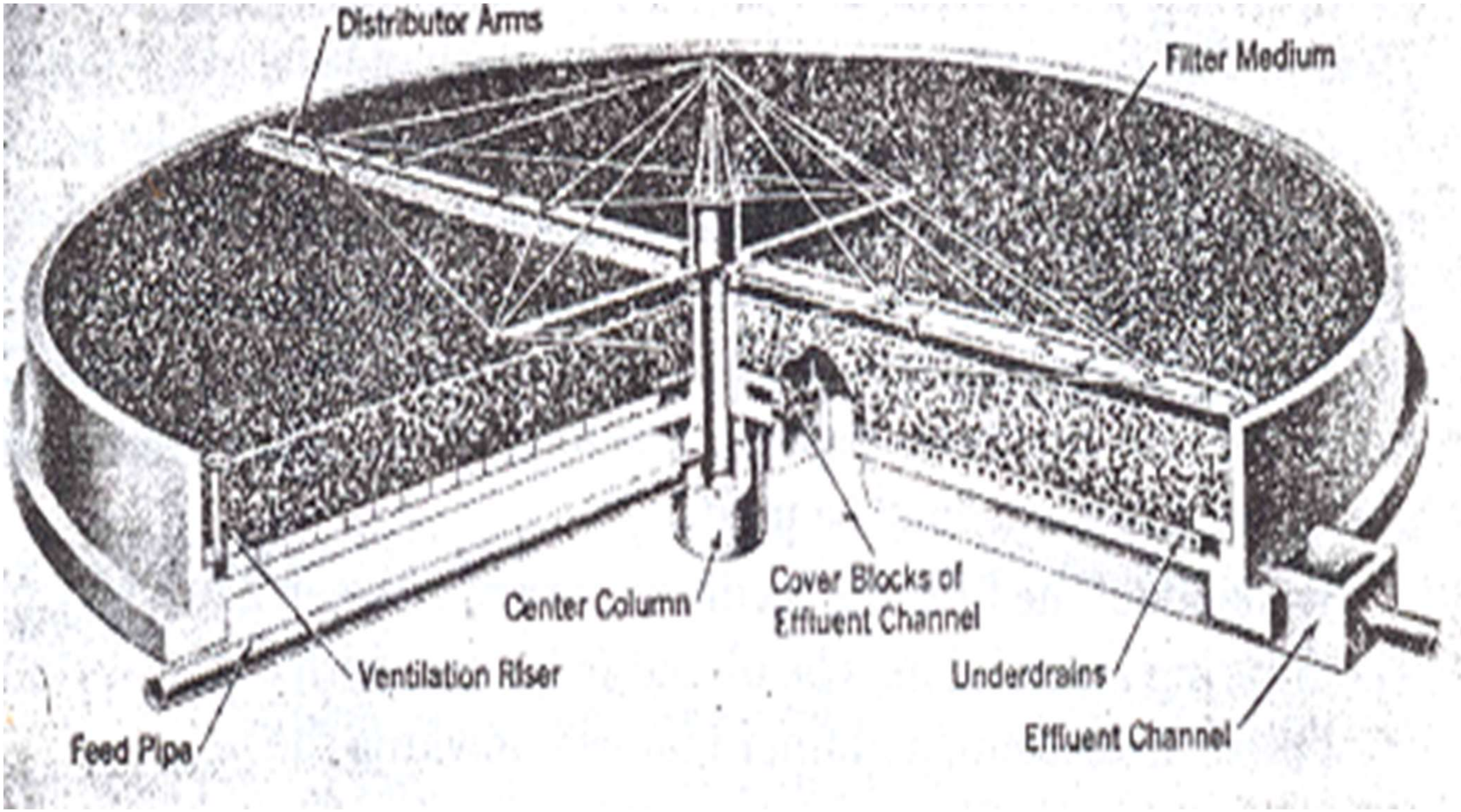


Figure Photographic view of a conventional circular trickling filter with rotary distributors

Biological Wastewater Treatment

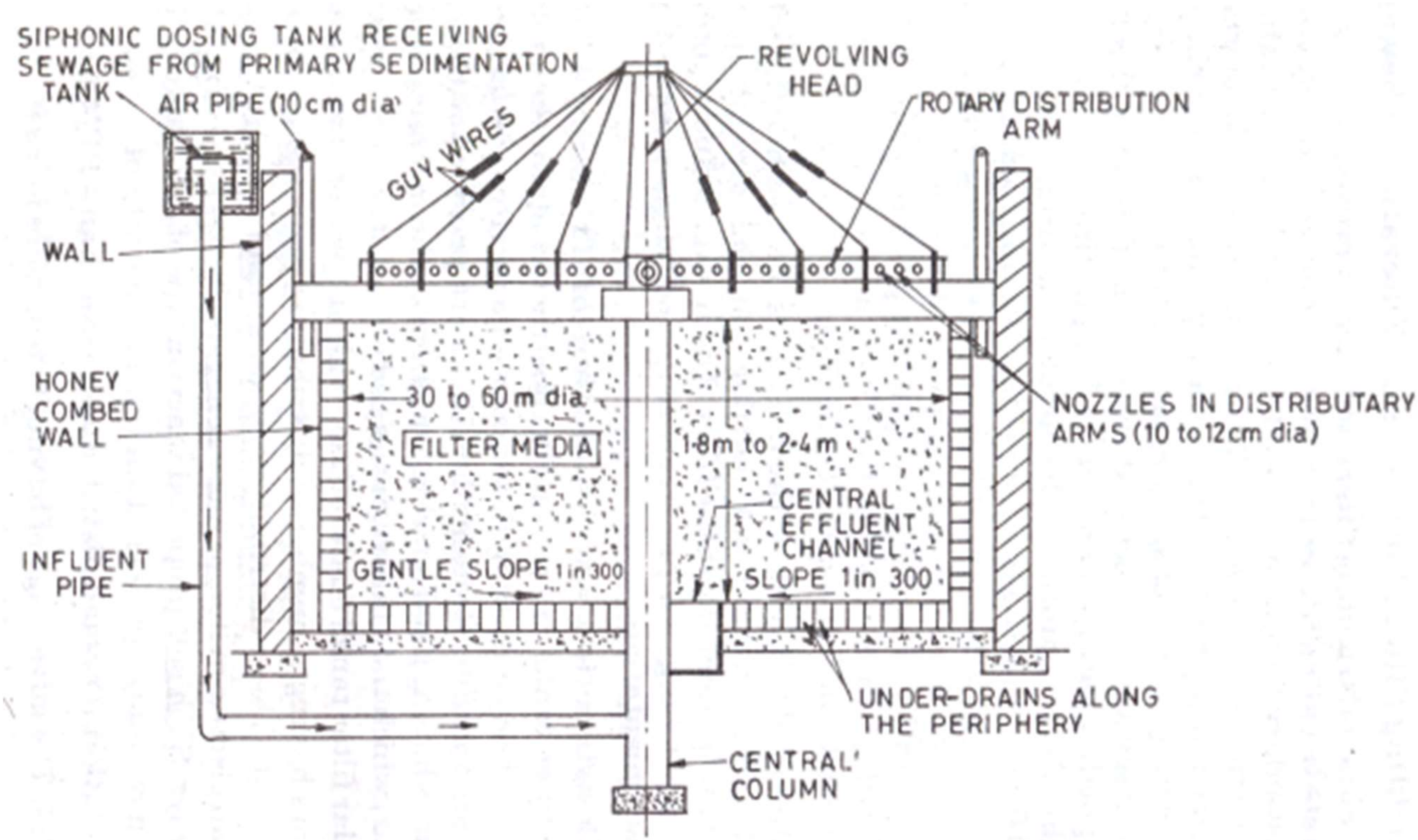


Figure of Typical section of a conventional circular trickling filter

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II. Sewage distributors over filters: they are two

rotary distributors and spray nozzles

Difference between the rotary distributors and that of spray nozzles

- With a rotary distributor, **the application of sewage to the filter is practically continuous**;
- whereas with spray nozzles, **the filter is dosed for 3 to 5 minutes, and then rested** for 5 to 10 minutes before the next application.

III. Filtering medias (stone used)

- consists of coarser materials like cubically broken stones or slag
- free from dust and small pieces.
- The size of the material used may vary between 25 to 75mm.
- should be washed before it is placed in position.
- should not be easily affected by acidic WW, and should be sufficiently hard.
- Its resistance to freezing and thawing is another important property,

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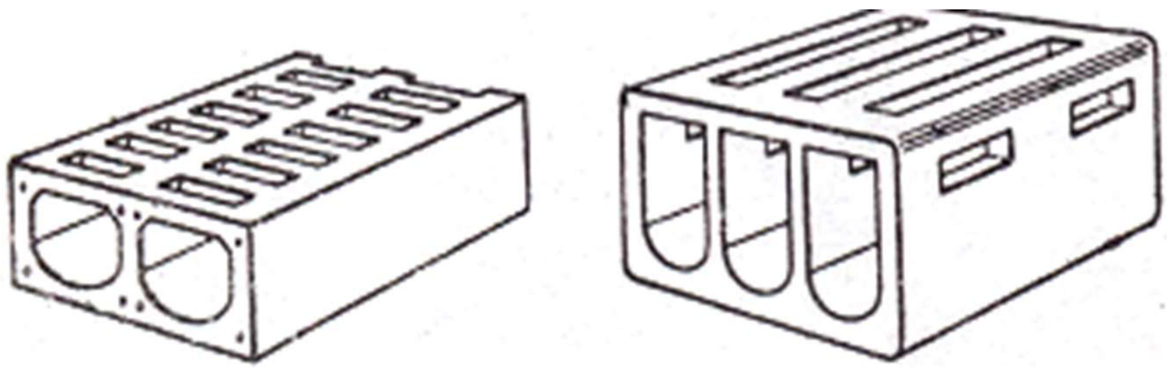
- The depth of the filtering media may vary between 2 to 3 meters.
- The filtering material may be placed in layers; with coarsest stone used near the bottom, and. finer material towards the top.

under drains

- ensures satisfactory drainage
- also ensure satisfactory ventilation and aeration of the filter bed
- Vitrified clay blocks are generally used as under-drains.



(a) Under-drain block for standard trickling filters.

(b) Under-drain block for high rate trickling filters with heavy hydraulic loading

Types of Trickling Filters

(1)Conventional trickling filters or standard rate or low rate trickling filters

(2)High rate filters or High rate trickling filters

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Design of Trickling Filters

involves the design of :

- the diameter of the circular filter tank and its depth and also design of the rotary distributors and under-drainage system.
- The design of the filter size is based upon the values of the filter-loadings adopted for the design.
- This loading on a filter can be expressed in two ways:

(i) Hydraulic-loading rate: the quantity of sewage applied per unit of surface area of the filter per day

- For conventional filters may vary between 22 and 44 (normally 28) million liters per hectare per day.
- For the high rate trickling filters it is about 110 to 330 (normally 220) M.L/ha/day

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(i) organic loading rate:

- the mass of BOD per unit volume of the filtering media per day.
- for conventional filters it vary between 900 to 2200 kg of BOD_5 per ha-m.
- about 6000 - 18000 kg of BOD_5 per ha-m in high rate trickling filters.

With an assumed value of organic loading (900 to 2200kg/ha-m)and hydraulic loading (22 and 44 Ml/ha/day).

$$total\ volume\ of\ filter = \frac{total\ BOD5}{organic\ loading}$$

$$surface\ area\ of\ the\ filter = \frac{total\ volume\ of\ filter}{hydraulic\ loading}$$

- Knowing the volume and area of the cylindrical filter, we can easily find out its diameter and depth.

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- the filter diameter and depth is designed for average value of sewage flow.
- The rotary distributors, under-drainage system, and other connected pipe lines etc. are, however, designed for peak flow and of course checked for the average flow.
- Moreover, since the rotary distributors are available indigenously only up to 60m in length, it is desirable to keep the diameter of the filter tank up to a maximum of 60m.
- If the required filter diameter is more than 60m, then it is better to use more units of lesser diameter.

Examples

a. Design suitable dimensions of circular trickling filter units for treating 5 million liters of sewage per day. The BOD of sewage is 150mg/l.

b. Also design suitable dimensions for its rotary distribution system and under drainage system.

solution

Total BOD present in sewage to be treated per day

$$= (5 * 10^6) l/d * 150\text{mg/l} = 750\text{kg}$$

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Assuming the value of organic loading as 1500kg/ha.m/day

$$\text{volume of filtering-media} = \frac{\text{total BOD}}{\text{oranic loading}}$$

$$= \frac{750kg}{1500\,\frac{kg}{ha.m}} = 0.5\text{ha. m} = 5000\text{m}^3$$

Assuming the effective depth of filter as 2m,

$$\text{surface area of the filter } (A_s) = \frac{5000\text{m3}}{2m} = 2500\text{m}^2$$

- Using a circular trickling filter of diameter 40m,
- The number of units required

$$= \frac{\text{Total area required}}{\text{Area of one unit}} = \frac{2500}{\frac{\pi}{4} * 40^2} \approx 2 \text{ units}$$

Check for Hydraulic loading

assume the value of hydraulic loading as, 25 million l/ha.day.

$$surface\ area\ of\ the\ filter = \frac{total\ volume\ of\ filter}{hydraulic\ loading}$$

$$A_s = \frac{5000m3}{25 * 10^3 m3/10^4 m2} = 2000m2$$

- The A_s chosen is 2500m^2, which is greater than 2000 m^2, and hence safe.
- Hence, 2 units each 40m diameter and 2m effective depth (i.e. 2.6m overall depth), can be adopted. An extra third unit as stand-by may also be constructed.

Design of Rotary Distributors

design for peak flow, assume peak flow as 2.25 times the average flow.

$$Q_p = 2.25 * 5\text{ML/day} = 11.25\text{ML/day} = 0.13\text{m}^3/\text{sec}$$

- This flow is divided into two filter units; and, therefore,

$$Q_{one\ filter} = 0.065\text{m}^3/\text{sec}$$

- Assuming that the velocity at peak flow is 2m/sec through the central column of the distributor,

$$D_{central\ column} = \sqrt{\frac{0.065}{2} * \frac{1}{\frac{\pi}{4}}} = 0.2\text{m}$$

Note: check the velocity through the column at average flow, as it should not be less than 1m/sec.

Check for velocity at average flow

- Discharge through each unit at average flow

$$= \frac{5}{2}\,\mathrm{ML/day} = 2.5\mathrm{ML/day} \;= 0.029\mathrm{m^3/sec}$$

Velocity at average flow $= \frac{0.029}{\frac{\pi}{4} * 0.2^2} = 0.92\mathrm{m/sec} < 1\mathrm{m/sec}$,

we should reduce the adopted diameter, Let us use 0.19m diameter, then the velocity at average flow

$$V_{at\ avg.} = \frac{0.029}{\frac{\pi}{4} * 0.19^2} = 1.023\mathrm{m/sec}\ > 1m/sec\ ;\ permissible$$

Then, $V_{at\ peak} = \frac{0.065}{\frac{\pi}{4} * 0.19^2} = 2.29\mathrm{m/sec}$

- Hence, we may use a central column of 0.19m in diameter.

Design of Arms

Let us use rotary reaction spray type distributor with 4 arms.

Then, the discharge per arm, $Q_{arm} = \frac{0.065}{4}\,\mathrm{m^3/sec} = 0.016\mathrm{m^3/s}$

Diameter of filter used = 40m (assumed previously).

Arm length $= \frac{\text{filter diam}eter}{2} - 1 = \frac{40-2}{2} = 19\mathrm{m}$

We can use each arm of 19m length with its size reducing from near the central column towards the end.

Let say, the first two sections, each of 6m length, and the third section 7m length

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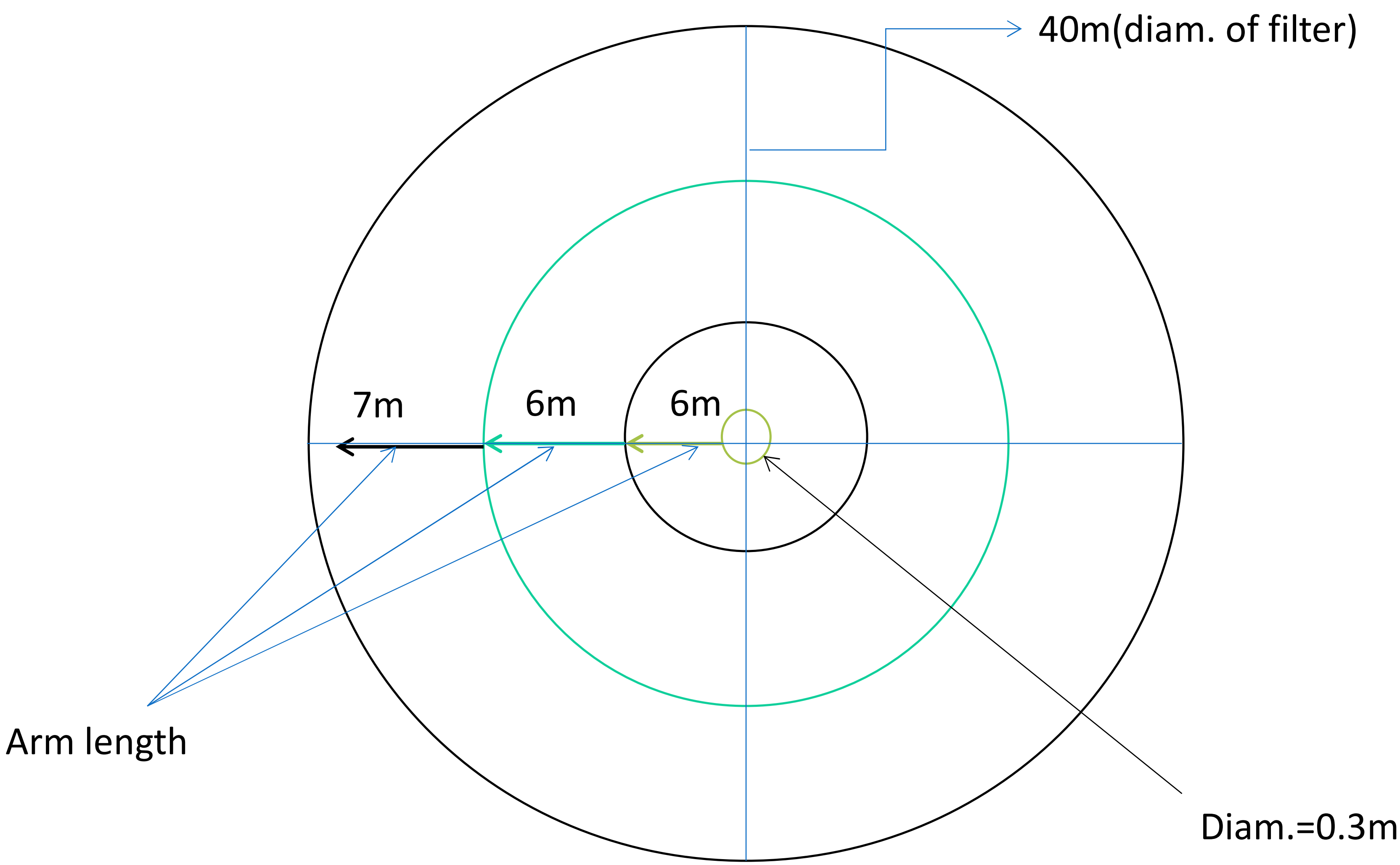

Plan of filter tank

- ✓ The flow in the arms has to be adjusted in the proportion of the filter area covered by these lengths of arm.
- ✓ Let A_1, A_2, and A_3 be the circular filter areas covered by each length of arm, starting from the central column. Allowing for 0.3m diameter in centre to be used for central column, etc., these areas would be:

$$A_1 = \pi(r_2^2 - r_1^2) = \pi(6.15^2 - 0.15^2) = 118.69\text{m}^2$$

$$A_2 = \pi(12.15^2 - 6.15^2) = 344.77\text{m}^2$$

$$A_3 = \pi(20^2 - 12.15^2) = 792.5\text{m}^2$$

$$\text{Total area of filter (A)} = \pi(20^2 - 0.15^2) = 1256\text{m}^2$$

- Proportionate areas served by each section of arm

$$1^{st} = \frac{A_1}{A} = \frac{118.69}{1256} * 100\% = 9.45\%$$

$$2^{nd} = \frac{A_2}{A} = \frac{344.77}{1256} * 100\% = 27.45\%$$

$$3^{rd} = \frac{A_3}{A} = \frac{792.5}{1256} * 100\% = 63.1\%$$

- Full discharge through an arm, i.e., $0.016 m^3/sec$, will flow through the first section, and this will go on reducing through the second and third sections.

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(i) Design of 1st section

Q_{arm1} = 0.016m^3/s, and Assuming the velocity through the arm as 1.2 m/s

$$\text{The area of arm required, } A_{arm1} = \frac{0.016}{1.2} = 0.0133\text{m}^2$$

$$\text{Arm Diameter require, } D_{arm1} = \sqrt{\frac{0.0133}{\frac{\pi}{4}}} = 0.13\text{m} = 130\text{mm}$$

(i) Design of second section

Discharge through the 2^{nd} section

$$Q_{arm2} = (100 - 9.45)\% * 0.016 = 0.0145\text{m}^3/s$$

$$A_{arm2} = \frac{Q}{V} = \frac{0.0145}{1.2} = 0.012\text{m}^2$$

$$D_{arm2} = \sqrt{\frac{0.012}{\frac{\pi}{4}}} = 0.124\text{m} = 124\text{mm}$$

(i) Design of third section

$$Q_{arm3} = (100 - 9.45 - 27.45)\% * 0.016 = 0.010\text{m}^3/\text{sec}$$

$$D_{arm3} = \sqrt{\frac{0.010}{1.2 * \frac{\pi}{4}}} = 0.103\text{m} = 103\text{mm}$$

Each arm length can thus be made of three sections, i.e. first 6m from center to be 130mm diameter, next 6m of 124mm diameter, and the last 7m of 103mm diameter.

exercise

Redesign trickling filter with the first arm is 12m and the last arm is 7m in length.

Design of Orifices

Each arm section will be provided with different no. of orifices, depending upon the Q to be passed through each section.

Total discharge through each arm = 0.016m3/sec.

$$Q_{orifice} = \mathrm{C_d} * \mathrm{A} * \sqrt{2\mathrm{gh}}$$

Assuming that 10mm diameter orifices are provided with coefficient of discharge (Cd) being 0.65, with an assumed water head (h), causing flow, as 1.5m,

$$Q_{orifice} = 0.65 * \frac{\pi}{4} * (0.01)^2 * \sqrt{2 * 9.81 * 1.5}$$

$$= 2.768 * 10^{-4}\ \mathrm{m^3/s}$$

$$No_{orifices} = \frac{\text{Total discharge through each arm}}{\text{Discharge through each orifice}} = \frac{0.016}{2.768 * 10^{-4}}$$

$\approx$ 58

Number of orifices through the first section

$$= \frac{9.45}{100} * 58 \approx 5$$

Number of orifices through the second section

$$= \frac{27.45}{100} * 58 \approx 16$$

Number of orifices through the third section

$$= \frac{63.1}{100} * 58 \approx 37$$

Spacing of orifices can be

- In the first section, 5 number in 6m length, i.e. $\frac{6}{5} = 1.2\text{m}$
- In the second section, 16 numbers in 6m length, i.e. $\frac{6}{16} = 0.375\text{m}$
- In the third section, 37 numbers in 7m length, i.e. $\frac{7}{37} = 0.189\text{m}$

Design of Under-drainage System

$$Q_{peak\ in\ each\ filter\ unit} = 0.065m^3/s$$

- ✓ Let us design the under-drainage system with a central rectangular channel, fed by radial laterals discharging into the channel.
- ✓ Assume velocity through effluent channel as 1m/s (min. V = 0.9m/s).

$$A_{channel} = \frac{\text{Discharge}}{\text{velocity}} = \frac{0.065}{1} = 0.065\text{m}^2$$

Assume 0.225m width,

$$Depth_{channel} = \frac{0.065}{0.225} = 0.288\text{m} \approx 0.3\text{m}$$

Calculate the slope of the bed of the channel S, is given by:

$$Q = \frac{1}{N} * A * R^{2/3} * S^{1/2}$$

Where, N = Manning's Coefficient = 0.018 (assumed)

$$A = 0.225m * 0.3m = 0.0675m^2$$

$$R = \frac{A}{P} = \frac{0.0675}{(0.225 + 0.3 + 0.3)} = 0.082$$

$$0.065 = \frac{1}{0.018} * 0.0675 * (0.082)^{2/3} * S^{1/2}$$

$$S^{1/2} = \frac{0.065}{0.706}$$

$$S = \frac{1}{117.9}$$

2. ACTIVATED SLUDGE PROCESS

- provides an excellent method of treating sewage.
- BOD removal is up to 80 - 95%, and bacteria removal is up to 90 - 95%.

Principle of operation:

- The WW effluent from primary sedimentation tank mixed with 20 to 30% of volume of activated sludge (from ASP unit).
  - This activate sludge contains a large concentration of highly active aerobic microorganisms.
- the sewage are intimately mixed together with a large quantity of air for about 4 to 8 hours.
- the moving organisms will oxidize the organic matter and the suspended and colloidal matter tends to coagulate and form a precipitate which settles down readily in the secondary settling tank.

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- The settled sludge (containing microorganisms) called activated sludge is then recycled to the head of the aeration tank to be mixed again with the sewage being treated.

- The mixture of WW and activated sludge is called mixed liquor.
- The biological mass (biomass) in the mixed liquor is called the mixed liquor suspended solids (MLSS) or mixed liquor volatile suspended solids (MLVSS).
- The MLSS consists mostly of microorganisms, non-biodegradable suspended organic matter, and other inert suspended matter.

Units of an Activated Sludge Plant

1. **Aeration Tanks** 2. **secondary clarifier**

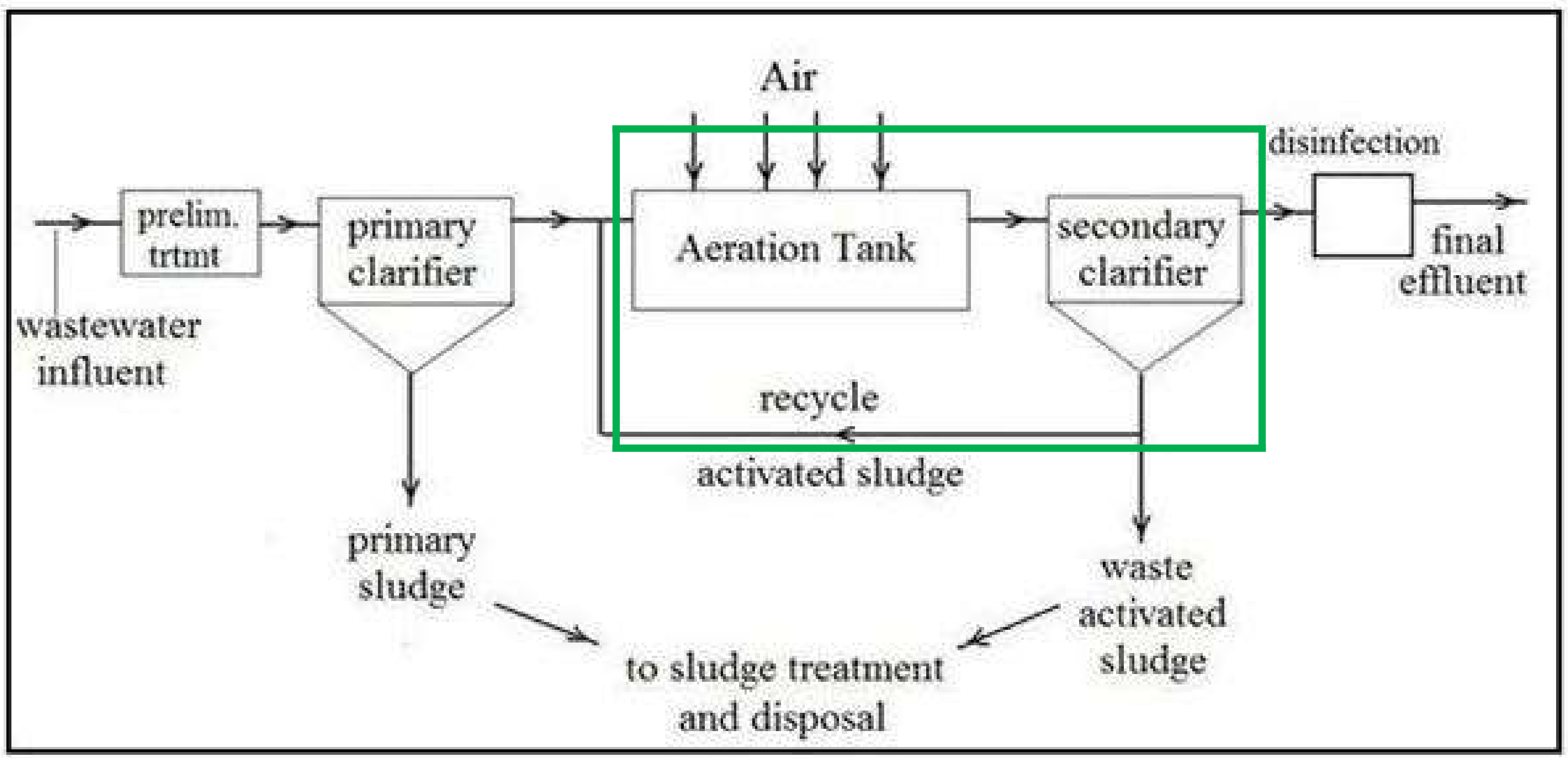

Figure of a conventional AS plant giving high degree of treatment

1. Aeration tanks

- Is tank in which mixed liquor get mixed with air.
- are normally rectangular tanks 3 to 4.5m deep and about 4 to 6m wide.
- The length may range between 20 to 200m and the detention period between 4 to 8 hours for municipal sewages.
- Air is continuously introduced into these tanks using one of the following method:
  - I. **Air diffusion**: compressed air under a pressure of 35-70kN/m2 is introduced into the aeration chamber by diffuser.
  - II. **Mechanical aeration**: atmospheric air is brought in contact with the sewage in this method.

2. Secondary Sedimentation Tank

- From the aeration tank, the WW flows to the final sedimentation tank.
- The detention period for such a sedimentation tank may be kept between 1.5-2 hours, as the same is usually found to give optimum results.
- The length to depth ratio may be kept at about 7 for rectangular ones.
- The depth may be kept in the range of 3.5 to 4.5m.

Design considerations in an activated sludge plant

1. Aeration Tank Loadings 2. Sludge Volume Index (SVI)

1. Aeration Tank Loadings

The important terms which define the loading rates of an activated sludge plant, include:

i. Aeration Period (i.e. Hydraulic Retention Time - HRT)
ii. BOD loading per unit volume of aeration tank (i.e. volumetric loading)
iii. Food to Micro-organism Ratio (F/M Ratio)
iv. Sludge age

i. The Aeration Period or HRT

- For continuous flow aeration tank,

$$\text{Detention period (t)} = \frac{\text{Volume of the tank}}{\text{Rate of sewage flow in the tank}} = \frac{V(m^3)}{Q(m^3/hr}$$

ii. Volumetric BOD Loading

$$= \frac{\text{Mass of BOD5 applied per day to the aeration tank through influent sewage in gm}}{\text{Volume of the aeration tank in } m^3}$$

$$= \frac{Q * Y_o\ (gm)}{V\ (m^3)}$$

Where, Q = Sewage flow into the aeration tank in m3

Y_o = BOD5 in mg/l (or gm/m3) of the influent sewage

V = Aeration tank volume in m3

(iii)Food (F) to Micro-organisms (M) Ratio

- The BOD load applied to the system in kg or gm is represented as food (F), and the total microbial suspended solid in the mixed liquor of the aeration tank is represented by M.

$$\text{F/M ratio} = \frac{\text{Daily BOD5 load applied to the aerator system in gm}}{\text{Total microbial mass in the system in gm}}$$

Daily BOD load applied to the aerator system

$$\text{F} = \text{Q}^* \; BOD_5 \;\; \text{gm/day}$$

$$\text{F} = \text{Q}^* \; Y_o \;\; \text{gm/day} \;\ldots\ldots\ldots\ldots\ldots\ldots(4.1)$$

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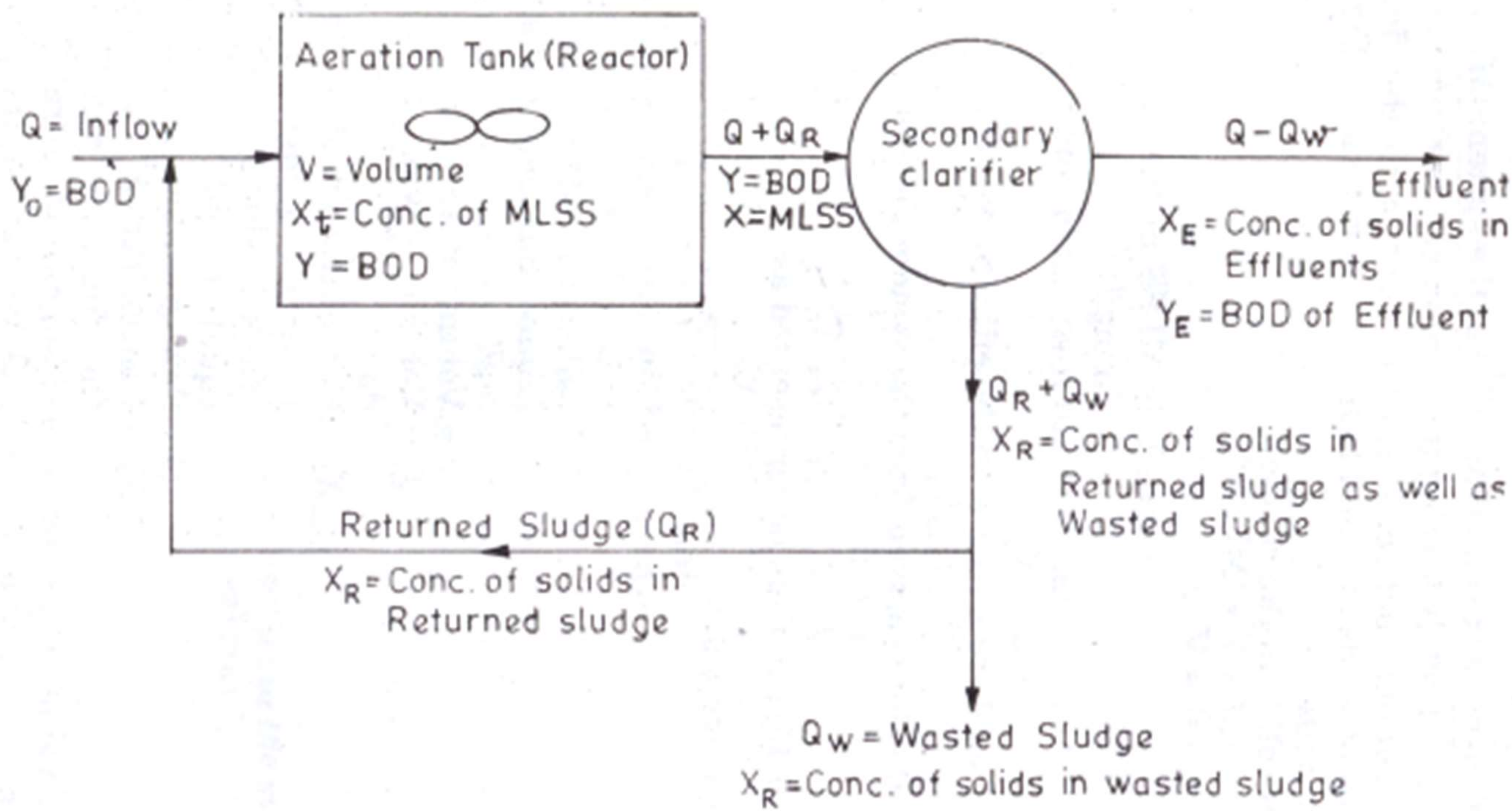

Figure of Flow chart of conventional activated sludge plant

The total microbial mass in the aeration system (M): Mixed Liquor Suspended Solids (MLSS) times the volume of the aeration tank (V).

$$M = MLSS * V$$

$$= X_t * V \dots\dots\dots\dots\dots\dots\dots.(4.2)$$

Where, X_t is MLSS in mg/l

Dividing (4.1) by (4.2), we get

$$\frac{F}{M}\text{ratio} = \frac{F}{M} = \frac{Q}{V} * \frac{Y_o}{X_t}$$

iv. Sludge Age

- average time for which particles of suspended solids remain under aeration.

$$\text{Sludge age } (\theta_c) = \frac{\text{Mass of suspended solids (MLSS) in the system (M)}}{\text{Mass of solids leaving the system per day}}$$

$$\text{Mass of solids in the reactor (M)} = V * (\text{MLSS})$$
$$= V * X_t$$

Mass of solids leaving the system per day is equal to Mass of solids removed with the wasted sludge per day plus Mass of solids removed with the effluent per day

Mass of solids removed with the wasted sludge per day

$$= Q_w * X_R \;\ldots\ldots\ldots 4.3$$

Mass of solids removed with the effluent per day

$$= (Q - Q_w) * X_E \ldots\ldots\ldots 4.4$$

Therefore, Total solid removed from the system per day is the summation equation 4.3 and 4.4.

$$= Q_w * X_R + (Q - Q_w) * X_E$$

Thus:

$$\text{Sludge age} = \theta_c = \frac{V * X_t}{Q_w * X_R + (Q - Q_w) * X_E}$$

When the value of X_E (suspended solids concentration in the effluent of activated sludge plant) is very small, then the term ($(Q - Q_w) * X_E$ in the above equation can be ignored, leading to:

$$\theta_c = \frac{V * X_t}{Q_w * X_R}$$

- another rational loading parameter is the specific substrate utilization rate (q) per day, and is defined as:

$$q = Q * \left(\frac{Y_o - Y_E}{V * X_t}\right)$$

- Under steady state operation, the mass of wasted activated sludge is further given by:

$$Q_w * X_R = y * Q(Y_o - Y_E) - K_e * X_t * V \ldots\ldots 4.5$$

Where, y = maximum yield coefficient

$$= \frac{\text{microbial mass synthesized}}{\text{mass of substrate utilized}}$$

K_e = Endogenous respiration rate constant (per day)

- The values of y and K_e are found to be constant for municipal waste waters, their typical values being:

y = 1.0 with respect to TSS (i.e. MLSS)

= 0.6 with respect to VSS (i.e. MLVSS)

K_e = 0.06 (per day)

From equations (4.5), dividing all terms for $X_t * V$ we can also work out as:

$$= \frac{1}{\theta_c} = yq - K_e$$

2. Sludge Volume Index (SVI)

- is used to indicate the physical state of the sludge produced in a biological aeration system.
- It represents the degree of concentration of the sludge in the system,
- Done in lab.

$$\text{SVI} = \frac{\text{Vo (ml/l)}}{\text{Xo (mg/l)}} = \frac{V_o}{X_o}\,\text{ml/mg}$$

Where, Vo: settled sludge volume in ml in liter of mixed liquor.

Xo: concentration of settled suspended solids in a liter of mixed liquor in mg.

- The usual adopted range of SVI is between 50 - 150 ml/gm and such a value indicates good settling sludge.

Sludge Recycle and Rate of Return Sludge

- The relationship b/n sludge recirculation ratio $\frac{Q_R}{Q}$ with X_t (MLSS in tank) and X_R (MLSS in returned or wasted sludge) is given as:

$\frac{Q_R}{Q} = \frac{X_t}{X_R - X_t}$, Where, QR = Sludge recirculation rate in m3/d

Xt = MLSS in the aeration tank in mg/l

XR = MLSS in the returned or wasted sludge in mg/l

The settleability of sludge is determined by sludge volume index (SVI), which is determined in the laboratory.

- If it is assumed that the sedimentation of suspended solids in the laboratory is similar to that in the sedimentation tank, then:

$$X_R = \frac{10^6}{SVI}$$

where, SVI value in mg/l

Then,

$$\frac{Q_R}{Q} = \frac{X_t}{\frac{10^6}{SVI} - X_t}$$

- The return sludge has always to be pumped and the pump capacity should be designed for a minimum return sludge ratio of 0.50 to 0.75 for large plants and 1.0 to 1.5 for smaller plants irrespective of the theoretical requirement.

Size and Volume of the Aeration Tank

✓ Using equation (a)

$$\theta_c = \frac{V * X_t}{Q_w * X_R} \ldots\ldots\ldots\ldots\ldots a$$

Using equation (b)

$$Q_w * X_R = y * Q(Y_o - Y_E) - K_e * X_t * V \ldots\ldots\ldots b$$

✓ Using combination of Equations (a) and (b:

$$V * X_t = \frac{y * Q(Y_o - Y_E)\theta_c}{1 + K_e\theta_c}$$

✓Alternatively, the tank volume can be determined for an assumed value of F/M ratio and tank MLSS (Xt).

$$\frac{F}{M} = \frac{Q}{V} * \frac{Y_o}{X_t}$$

Oxygen Requirement of the Aeration Tanks

- The total oxygen requirement may be computed by using the equation

$$O_2 = \left(\frac{Q(Y_o - Y_E)}{f} - 1.42 Q_w * X_R\right) \text{gm/day}$$

Where,

$$f = \frac{BOD_5}{BOD_u} = \frac{\text{5 day BOD}}{\text{Ultimate BOD}} \cong 0.68$$

1.42 = oxygen demand of biomass in gm/gm

- The above formula represents the oxygen demand for carbonaceous BOD removal and does not account for nitrification.

Table 4-4 Characteristics and design parameters of different activated sludge systems

Process type	Flow regime	MLSS mg/l	$\frac{\text{MLVSS}}{\text{MLSS}}$	$\frac{\text{F}}{\text{M}}$	HRT hrs	Volumetric Loading kg BOD_5 per m^3	SRT (days) θ_c	$\frac{Q_R}{Q}$ Return Sludge ratio	BOD removal percent	kg O_2 reqd. per kg BOD_5 removed	Air requirement in m^3 per kg of BOD_5 removed
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Conventional	Plug	1500 to 3000	0.8	0.4 to 0.3	4 to 6	0.3 to 0.7	5 to8	0.25 to 0.5	85 to 92	0.8 to 1.0	40 to 100
Taperd aeration	Plug	1500 to 3000	0.8	0.4 to 0.3	4 to 6	0.3 to 0.8	5 to 8	0.25 to 0.5	85 to 92	0.7 to 1.0	50 to 75
Step aeration	Plug	2000 to 3000	0.8	0.4 to 0.3	3 to 6	0.7 to 1.0	5 to 8	0.25 to 0.75	85 to 92	0.7 to 1.0	50 to 75^{+}
Contact stabilization	Plug	1000 to 3000* 3000 to 6000**	0.8	0.5 to 0.3	0.5 to 1.5* 3 to 6**	1.0 to 1.2	5 to 8	0.25 to 1.0	85 to 92	0.7 to 1.0	50 to 75
Complete mix	Complete mix	3000 to 4000	0.8	0.5 to 0.3	4 to 5	0.8 to 2.0	5 to 8	0.25 to 0.8	85 to 92	0.8 to 1.0	50 to 75
Modified aeration	Plug	300 to 800	0.8	3.0 to 1.5	1.5 to 3	1.2 to 2.4	0.2 to 0.5	0.05 to 0.15	60 to 75	0.4 to 0.6	25 to 50
Extended aeration	Complete mix	3000 to 5000	0.5 to 0.6	0.18 to 0.1	12 to 24	0.2 to 0.4	10 to 25	0.5 to 1.0	95 to 98	1.0 to 1.2	100 to 135

Example

1. Design a conventional activated sludge plant to treat domestic sewage with diffused air aeration system, given the following data:

   Population = 35000

   Average sewage flow = 180 l/c/d

   BOD of sewage = 220mg/l

   BOD removed in primary treatment = 30%

   Overall BOD reduction desired = 85%

Solution:

Requirements: dimension of aeration tank, dimension of secondary clarifier,

Daily sewage flow

$$= \mathrm{Q} = 180 * 35000\,\mathrm{l/day} = 6300\mathrm{m}^3/\mathrm{day}$$

BOD of sewage coming to aeration

$$= Y_o = 70\% * 220 mg/l = 154 mg/l$$

BOD removed in activated plant

$$= 0.85 * 154 = 130.9 mg/l$$

Check for Efficiency in Activated plant $= \frac{130.9}{154} = 0.85$ =85%

From Table 4 4, for efficiency of 85 - 92%, we use F/M ratio as 0.4 to 0.3 and MLSS between 1500 and 3000.

So let us adopt F/M = 0.30, and MLSS (Xt) as = 3000 mg/l

Using $\frac{F}{M} = \frac{Q * Y_o}{V * X_t}$, calculate the volume of aeration tank.

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$$0.30 = \frac{6300m^3/d * 154\text{mg/l}}{\text{V} * 3000\text{mg/l}}$$

V = volume of aeration tank

$$= \frac{6300 * 154}{3000 * 0.30} = 1078\text{m}^3$$

Check for Aeration period or HRT (t)

$$\text{Using, t} = \frac{\text{V}}{\text{Q}} = \frac{1078m^3}{6300m^3/d} * 24\ \text{h/d}$$

$$= 4.11\ \text{h (within the limits of 4 to 6 h)} \dots \dots\ \text{ok}$$

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Check for sludge age(θ_c)

From equation

$$V * X_t = \frac{y * Q(Y_o - Y_E)\theta_c}{1 + K_e\theta_c}$$

Where, V = 1078m3

Xt = 3000mg/l

y = yield coefficient = 1.0 with respect to MLSS

Q = 6300m3/d

Ke = Endogenous respiration constant = 0.06d-1

Yo = BOD of influent in aeration tank = 154mg/l

YE = BOD of effluent = (154-130.9)mg/l = 23.1mg/l

Substituting the values, we get

$$1078m^3 * 3000\text{mg/l} = \frac{1 * 6300m^3/d(154 - 23.1)\, mg/l * \theta_c}{1 + 0.06d^{-1} * \theta_c}$$

$\theta_c = 5.13days \ldots \ldots$..ok! as it lies between 5 and 8 days

Check for volumetric loading

Volumetric loading = $\frac{Q * Y_o}{V}$ gm of BOD/m^3 of tank volume

$$= \frac{6300m^3/d * 154\, mg/l}{1078m^3} = 900 \text{ mg}/l = 0.9 \text{ kg/m}^3$$

(not within the permissible range of 0.3 - 0.7kg/m^3)

So increase the volume of aeration tank!!!

(i) Check for Return sludge ratio (for SVI ranging between 50 - 150ml/gm

Let us take 100ml/gm.

Using equation

$$\frac{QR}{Q} = \frac{X_t}{\frac{10^6}{SVI} - X_t}, \quad X_t \text{ is MLSS}$$

Where, SVI = 100ml/gm

X_t = 3000mg/l

$$\frac{QR}{Q} = \frac{3000}{\frac{10^6}{100} - 3000} = 0.43 = 43\%$$

(i.e. within the prescribed range of 25 to 50%) …ok!

Biological Wastewater Treatment

Tank Dimensions

Adopt aeration tank of depth (D) 3m and width (B) 4.0m. The total length of the aeration channel required

$$= \frac{\text{Total volume required}}{\text{B} * \text{D}} = \frac{1078m^3}{4.0\text{m} * 3\text{m}} = 90\text{m}$$

- ✓ Provide a continuous channel, with 3 aeration chambers, each of 30m length.
- ✓ Total width of the unit, including 2 baffles each of 0.25m thickness = 3 * 4.0m + 2 * 0.25 = 12.5m.
- ✓ Total depth provided including free-board of 0.5m will be 3 + 0.5 = 3.5m.

Overall dimensions of the Aeration tank will be 30m * 12.5m * 3.5m.

Rate of Air Supply Required

Assuming the air requirement of the aeration tank to be $100m^3$ of air per kg of BOD removed,

Air required i.e. blower capacity

$$= 100 * \frac{130.9\mathrm{mg/l} * 6300m^3/d}{1000} = 53\mathrm{m}^3/\mathrm{min}$$

Design of Secondary sedimentation Tank

Adopting a surface loading rate of $20m^3/day/m^2$ at average flow of $6300m^3/day$,

(i) Surface area required

$$= \frac{6300m^3/d}{20m^3/d/m^2} = 315\mathrm{m}^2$$

Adopting a solids loading of $125kg/day/m^2$ for MLSS of 3000mg/l (3kg/m3),

(i) the surface area required

$$= \frac{6300m^3/d * 3\mathrm{kg}/m^3}{125\mathrm{kg/day/m}^2} = 151.2\mathrm{m}^2$$

The higher surface area of $315m^2$ is adopted.

Adopting a circular tank,

$$\text{diameter of tank} = \sqrt{\frac{315 * 4}{\pi}} = 20\text{m}$$

Weir loading for a circular weir placed along the periphery of the tank having length 20πm will be:

$$= \frac{6300}{20\pi}\text{m}^3/\text{day}/\text{m} = 100.3 < 150;\ \text{ok!}$$

Note: lf weir loading exceeds the permissible value; we may provide a trough instead of a single weir at the periphery.

Hence, provide 20 m diameter secondary settling tank.

Design of Sludge Drying Beds

In order to design sludge drying beds, the quantity of excess wasted sludge will be calculated by using the equation below:

$$\theta_c = \frac{V * X_t}{Q_w * X_R}$$

$$5.13\ \mathrm{d} = \frac{1078m3 * 3\mathrm{kg/m^3}}{Q_w * X_R}$$

$$Q_w * X_R = \frac{1078m3 * 3\mathrm{kg/m^3}}{5.13\mathrm{d}} = 630\mathrm{kg/d}$$

If density of sludge is known, it is possible to calculate the required volume of sludge. For e.g. 10 kg/m^3 SS concentrating in secondary sludge, sludge volume

$$= \frac{630\mathrm{kg/d}}{10\mathrm{kg/m^3}} = 63\mathrm{m^3/d}$$

Note: This secondary sludge volume of 63m^3/d shall be taken to sludge drying beds, along with the primary sludge.

3. Waste Stabilization Pond

- Is large shallow basins enclosed by earthen embankments in which wastewater is biologically treated by natural processes involving pond algae and bacteria.

Types of Pond

- Anaerobic Ponds
- Facultative and
- Maturation/aerobic pond

- WSP comprise a single series of these anaerobic, facultative and maturation ponds or several of such series in parallel.
- A long hydraulic retention time is necessary because of the slow rate at which the organic waste is oxidized.
- Typical hydraulic retention times range from 10 -100 days depending on the temperature of a particular region.

Biological Wastewater Treatment

Figure of Waste Stabilization Pond

I. Anaerobic Ponds

- Anaerobic ponds are unmixed basins designed to enhance the settling and biodegradation of particulate organic solids by anaerobic digestion.
- Pond depth is usually between 3 to 5 meters and
- HRT for municipal sewage is b/n 1 - 3 days and for industrial WW may increase to 20 days.
- In cold climates, anaerobic ponds mainly act as settling ponds, whereas higher sewage temperatures enhance the anaerobic degradation process.
- At higher temperatures BOD is therefore more effectively removed.
- Typical TSS removal percentages range between 50 and 70%.
- BOD removal rate is increase with temperature and range b/n 30 and 75%.

Treatment Mechanisms

- BOD removal is the combined effect of sedimentation and biological degradation.
- Biological degradation is due to the anaerobic degradation of complex organic material.
- Anaerobic ponds require some preliminary treatment of municipal WW.
  - Usually coarse screening and grit chamber is applied

II. Facultative Ponds

- are the second treatment step in a pond system.
- In facultative ponds the anaerobic pond effluent is further treated, aimed at further BOD, nutrient and pathogen removal.
- Facultative ponds are usually 1.5 - 2.5m deep.
- The HRT of this ponds is varies b/n 5 and 30 days.

Processes In facultative ponds

- the top layer of facultative ponds is aerobic due to oxygen production by algae and the bottom layer is anaerobic due to the absence of algae activity.
- The three main mechanisms for BOD removal are aerobic digestion, sedimentation and anaerobic digestion.
- Sedimentation results only in temporary storage of BOD in the sludge layer.
- This BOD (in sludge) is removed while the pond is de-sludged. Part of the sludge BOD is however anaerobically transformed into methane gas.

Biological Wastewater Treatment

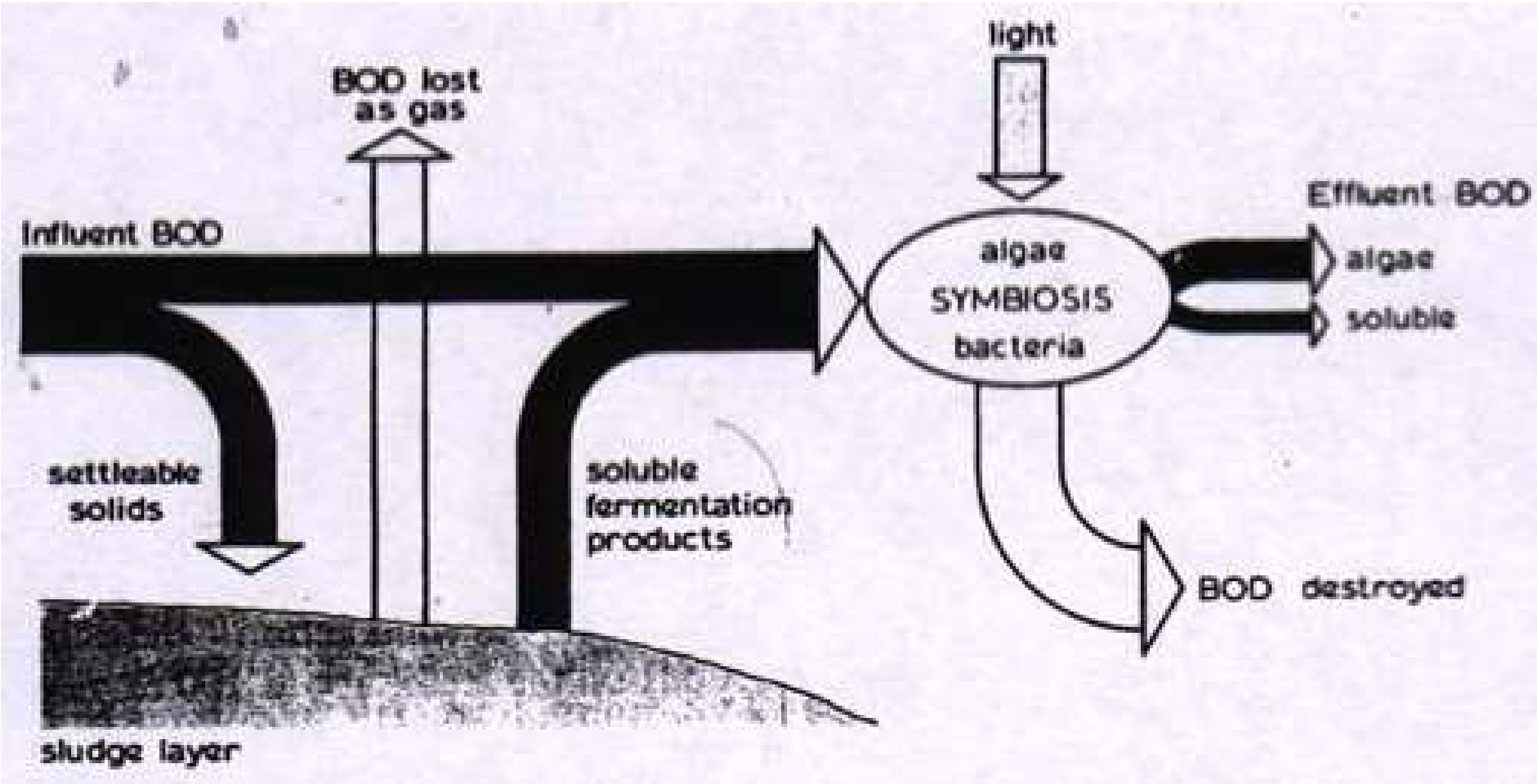


Figure of BOD removal mechanisms in a facultative pond

III. Maturation Ponds

- Maturation ponds are shallow ponds (1 - 1.5m deep).
- An active algal biomass is maintained throughout the entire depth of the system
  - so that during daytime large amounts of oxygen are produced.
- BOD removal is much slower than in facultative ponds, since the most easily degradable substances consumed already.
- The major application for maturation ponds is to polish or upgrade facultative pond effluents and achieve substantial microbial reductions to allow safe use of the effluents in agriculture or aquaculture

Biological Wastewater Treatment

Photo of Kaliti (A.A)WSP- effluent diversion weir for irrigation

Removal of Pathogenic Microorganisms in maturation pond

- Pathogen removal occurs in anaerobic, facultative and maturation ponds, but only maturation ponds are designed on the basis of required removal rates for pathogens.
- Four groups of pathogenic micro-organism can be distinguished in WW: bacteria, viruses, protozoa and helminthes.
- Both helminth eggs and protozoan cysts are removed by sedimentation.
- Removal bacteria (fecal coliform) and virus is due to a combination of several processes:
    - Adsorption to particles and subsequent sedimentation
    - Grazing by other micro-organisms (protozoa)
    - Natural decay

Biological Wastewater Treatment

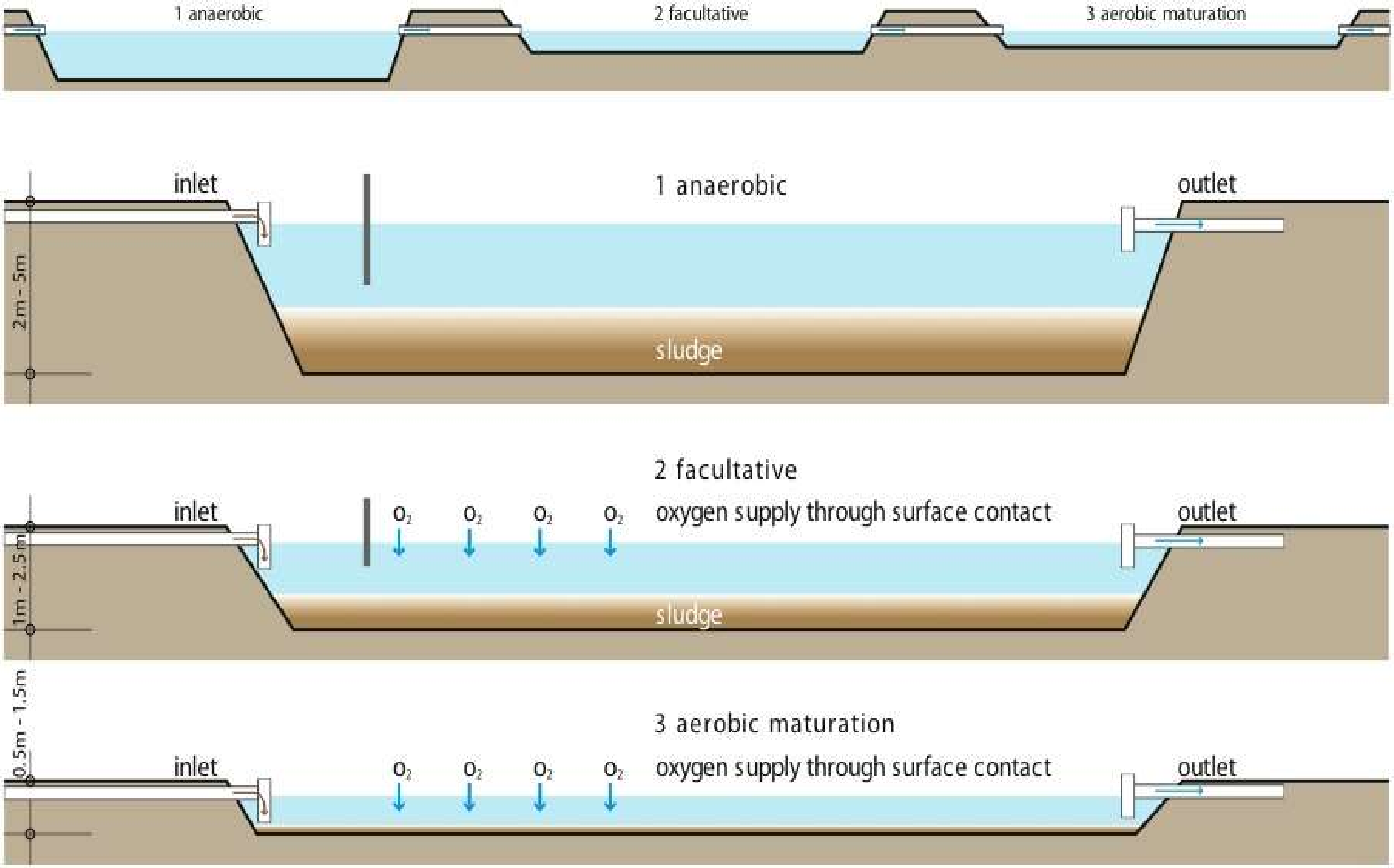


Figure of Typical scheme of a waste stabilization system

Biological Wastewater Treatment

Physical Design of WSP

i. **Pond Location:** Ponds should be located at least 200m downwind from the community they serve and away from any Likely area of future expansion.

ii. **Preliminary Treatment: a**dequate screening and grit removal facilities must be installed.

iii. **Pond Geometry:** pond geometry includes not only the shape of the pond but also the relative positions of its inlet and outlet.

iv. **Pond Configurations:** Configurations can includes either series or parallel operations
   - the advantages of series operation is improved treatment because of reduced short circuiting and
   - the advantages of parallel configuration is that the loading can be distributed more uniformly over a large area
   - combinations of parallel & series operation can be accomplished

Process Design of WSP

i. Effluent Quality Requirements

- The general WHO guideline standards for the discharge of treated wastewaters into inland surface waters :

Parameter	Effluent limit
BOD	30 mg/l
Suspended solids	100 mg/l
Total N	100 mg N/l
Total ammonia	50 mg N/l
Free ammonia	5 mg N/l
Sulphide	2mg/l
pH	5.5 – 9.0

Biological Wastewater Treatment

ii. Design Parameters

- The four most important parameters for WSP design are temperature, net evaporation, flow and BOD.
- Faecal coliform and helminth egg numbers are also important if the final effluent is to be used in agriculture or aquaculture.
- The usual design temperature is the mean air temperature in the coolest month (or quarter).
- Net evaporation has to be taken into account in the design of facultative and maturation ponds, but in anaerobic scum……
- The mean daily flow must be estimated since the size of the ponds, and hence their cost, is directly proportional to the flow.

Biological Wastewater Treatmen

- The BOD may be measured using 24-hour flow-weighted composite samples.
- If WW does not yet exist, it should be estimated from the following equation:

$$BOD\ g/l = \frac{B(g/d)}{Q(l/d)}$$

Where

B = BOD contribution, g/c/d

Q = wastewater flow, l/c/d

- Values of B vary between 30 and 70g/c/d, with affluent communities producing more BOD than poor communities.

A suitable design value for Ethiopia is 45g/c/d

(source: AA Water Sewerage Authority report 2003)

Biological Wastewater Treatment

- Faecal coliform Nos. are important if the pond effluent is to be used for unrestricted crop irrigation or for fishpond fertilization.
  - The usual range is 10^7- 10^8 faecal coliforms per 100 ml, and a suitable design value is $5*10^7$ per 100 ml.
- Helminth egg numbers are also important when pond effluents are used for restricted crop irrigation (irrigation of all crops except salads and vegetables eaten uncooked) or fishpond fertilization.
  - The usual range is 100 - 1000 eggs per liter.

Design of Anaerobic Ponds

Designed without risk of odour & nuisance on the basis of volumetric BOD loading (λv, g/m3/d), which is given by:

$$\lambda v = \frac{total\ BOD}{volume\ of\ pond} = \frac{y_o Q}{V} \ldots\ldots\ldots 4.8$$

Where y_o = influent BOD, mg/l

Q = flow, m3/d

V = anaerobic pond volume, m3

- Once a value of λv has been selected, the anaerobic pond volume is then calculated from above equation.
- The mean hydraulic retention time in the pond (θ, d) is determined from: $\theta = \frac{V}{Q}$

Biological Wastewater Treatment

Table of Volumetric loading (g/m3/d) Vs temperature

Temperature (^{0}C)	Volumetric loading (g/m^{3}d)	BOD removal (%)
< 10	100	40
10 - 20	20T - 100	2T + 20
20 - 25	10T + 100	2T + 20
> 25	350	70

Source: Mara and Pearson, 1986 and Mara et al., 1997

- Retention times in anaerobic ponds < 1 day should not be used.
- If the value of θ < 1 day, a value of 1 day should be used and the corresponding value of V recalculated.

Design Facultative Ponds

designed on the basis of surface BOD loading(λS, kg/ha/d), which is given by:

$$\lambda S = \frac{y_o Q}{A_s} \dots\dots\dots 4.9$$

Where A_s = facultative pond surface area, m^2

- Different investigators relate the permissible design value of λS to d/t parameters:
  - Arceivala et al., 1970 relates to latitude (L):
  $$\lambda S = 375 - 6.25L$$
  - McGarry and Pescod (1970), relates to temperature (T):
  $$\lambda S = 60(1.099)^T$$
  - Mara (1987):more appropriate global design equation, to temp.
  $$\lambda S = 350(1.107 - 0.002T)^{25-T}$$

Biological Wastewater Treatment

- Once a suitable value of λS has been selected, the pond area is calculated from equation (4.9) and its retention time (θ_s, d) from:

$$\theta_f = \frac{A_s D}{Q_m}$$

- Where: D = pond depth, m, Qm = mean flow, m3/day
- The mean flow is the mean of the influent and effluent flows (Qi and Qe), the Qe is less b/e, net evaporation and seepage.

$$\theta_f = \frac{A_s D}{\frac{1}{2} Q_i + Q_e}$$

- A minimum value of θ_s of 5days should be adopted for temperatures below 20^oC, and 4days for temperatures above 20°C.

Design of Maturation Ponds

- Designed for Faecal Coliform, Helminth Egg, and Nutrient Removal

Faecal Coliform Removal: using method of Marais (1974)

- This assumes that faecal coliform removal can be modeled by first order kinetics in a completely mixed reactor.
- The resulting equation for a single pond is thus:

$$N_e = N_i(1 + K_T * HRT)$$

Where Ne = number of FC per 100 ml of effluent

Ni = number of FC per 100 ml of influent

K_T = first order rate constant for FC removal, per day

HRT = retention time, d

Biological Wastewater Treatment

- For a series of anaerobic, facultative and maturation ponds,

$$N_e = \frac{N_i}{(1 + K_T * HRT_{anaerobic}) * \left(1 + K_T * HRT_{facultative}\right) * (1 + K_T * HRT_{aerobic})^n}$$

Where:

Ne = number of feacal coliform per 100 ml effluent

N_i = number of feacal coliform per 100 ml influent

K_T = first order temperature dependent rate (day-1)

n = No. of maturation ponds (each pond the same HRT)

- A series of n maturation ponds should have total HRT of 5 days.
- The value of K_T is highly temperature dependent. (Arthur, 1983)

Found that:

$$K_T = 2.6(1.19)^{T-20}$$

Helminth Egg Removal:

- Analysis of egg removal data from ponds has yielded the following relationship which is equally valid for anaerobic, facultative and maturation ponds:

$$R = 100[1 - 0.14\exp(-0.38\theta)]$$

Where R = percentage egg removal

θ = retention time, d

Biological Wastewater Treatment

Table of Design values of helminth egg removal (R %) for hydraulic retention times (θ)

θ	R	θ	R	θ	R	θ	R
1.0	74.67	3.2	90.68	6.0	97.06.	12	99.61
1.2	76.95	3.4	91.45	6.5	97.57	13	99.70
1.4	79.01	3.6	92.16	7.0	97.99	14	99.77
1.6	80.87	3.8	92.80	7.5	98.32	15	99.82
1.8	82.55	4.0	93.38	8.0	98.60	16	99.86
2.0	84.08	4.2	93.66	8.5	98.82	17	99.88
2.2	85.46	4.4	93.40	9.0	99.01	18	99.90
2.4	87.72	4.6	94.85	9.5	99.16	19	99.92
2.6	87.85	4.8	95.25	10	99.29	20	99.93
2.8	88.89	5.0	95.62	10.5	99.39		
3.0	89.82	5.5	96.42	11	99.38		

Source: Ayres et al.(1992)

Biological Wastewater Treatment

<u>Nutrient Removal:</u> Pano and Middlebrooks's (1982) equations

Their equation for temperatures below $20^{o}C$ is:

$$C_e = \frac{C_i}{\{1 + [(A/Q)(0.0038 + 0.000134T)\exp((1.041 + 0.044T)(pH - 6.6))]\}}$$

And for temperatures above $20^{o}C$:

$$C_e = \frac{C_i}{\left\{1 + \left[5.035 * 10 - 3 * \frac{A}{Q}\right][\exp(1.54 * (pH - 6.6))]\right\}}$$

Where:

Ce = ammonical nitrogen concentration in pond effluent (mg N/l)

C_i = ammonical nitrogen concentration in pond influent (mg N/l)

A = pond surface area (m2)

Q = wastewater flow rate (m3/day)

T = temperature (°C)

Hydraulic Balance

- To maintain the liquid level in the ponds, the inflow must be at least greater than net evaporation and seepage at all times.

$$Q_i \geq 0.001A * (e + s)$$

Where, Qi = inflow to first pond, m3/d

A = total area of pond series, m2

e = net evaporation (i.e. evaporation less rainfall), mm/d

s = seepage, mm/d

Biological Wastewater Treatment

Example

Using the following data design WSP

- Town of 20,000 population
- Consumption of drinking water of 150l/c/day and wastage of 85%
- No significant infiltration into sewer system
- Average BOD production of 45g BOD/c/day
- Measured influent concentration of $4*10^8$ FC/100ml
- Clay bottom (hydraulic conductivity 10^{-7}m/s)
- Climate of the area (Latitude = ± 16°S)

Maximum monthly temperature	33°c (September)
Minimum monthly temperature	27°c (June/July)
Total annual rainfall	1143mm
Maximum monthly rainfall	206mm (November)
Minimum monthly rainfall	15mm (August)

Requirements

- ✓Sludge remove in anaerobic ponds only once every two years
- ✓Design each pond with a freeboard of 0.5m
- ✓The treated effluent must have a BOD concentration below 20mg/l and should be reusable for agricultural purposes (use standards according to WHO)

Task:

- ➢Design a conventional WSP system (anaerobic + facultative + maturation).
- ➢Provide for each pond the dimensions (L, W and D), the volume, surface area and the residence time.
- ➢Calculate C_{in} and C_{out} from each pond.

Biological Wastewater Treatment

Solution:

- Influent flow Q:

$$Q = 20{,}000 * 0.15\text{m}^3/\text{day} * 0.85 = 2550\text{m}^3/\text{day}$$

- Influent BOD concentration Ci:

$$C_i = \frac{45 * 10^3 \text{ mg BOD/c/day}}{150\text{L/c/day} * 0.85} = 353\text{mg BOD/L}$$

- Influent BOD-load Li:

$$L_i = 0.045\text{kgBOD/capita.day} * 20{,}000 = 900\text{kgBOD/day}$$

- Clay bottom with low hydraulic conductivity
  - Limited infiltration (10-7m/s = 0.36mm/h = 8.64mm/day)
  - No lining is necessary
- Precipitation and evapotranspiration will influence the system but since no detailed data are available this aspect will not be considered.

Anaerobic pond

Calculations done at 27°c (coldest temperature) = worst case scenario

- Volumetric loading rate at 27°c = 0.35kg BOD /m3/day…from Table of Volumetric loading (g/m3/d) Vs temperature

$$V_{wastewater} = \frac{900\text{kg BOD/day}}{0.35\text{kg BOD/m}^3.\text{day}} = 2571\text{m}^3$$

From WHO (1997): sludge generation rate= 40 liter sludge/capita/year

de-sludging period as 2years (given)

$$V_{sludge} = 0.04\text{m}^3\text{sludge/capita/year} * 20{,}000 * 2\text{years}$$
$$= 1600\text{m}^3$$

$$\text{Total volume} = 2571 + 1600 = 4171\text{m}^3$$

Resulting hydraulic residence time (HRT):

$$\theta = \frac{V}{Q} = \frac{2571\text{m}^3}{2550\text{m}^3/\text{day}} = 1.01\text{day (after two years sludge accumulation)}$$

$$= \frac{4171\text{m}^3}{2550\text{m}^3/\text{day}} = 1.64 \text{ day (no sludge present in the pond)}$$

Resulting mid-depth surface area A_s for standard pond depth of 4m:

$$A_s = \frac{4171}{4} = 1043\text{m}^2$$

Assume Pond shape: normally square (equal length and width)

Slope: usually 33% for stability reasons (0.33m vertical rise per 1m of horizontal progress)

- ✓ Side of square at half depth $= \sqrt{\text{area}} = 32\text{m}$
- ✓ Side of square at the water surface

$$= \text{side at half depth} + (2 * (0.5 * \text{depth}) * \text{inverse slope})$$

$$= 32 + (2 * 2 * 3) = 44\text{m} \rightarrow \text{area} = 1936\text{m}^2$$

Biological Wastewater Treatment

- Side of square including 0.5m freeboard

$$= 44 + (2 * 0.5 * 3) = 47\text{m} \rightarrow \text{area} = 2209\text{m}^2$$

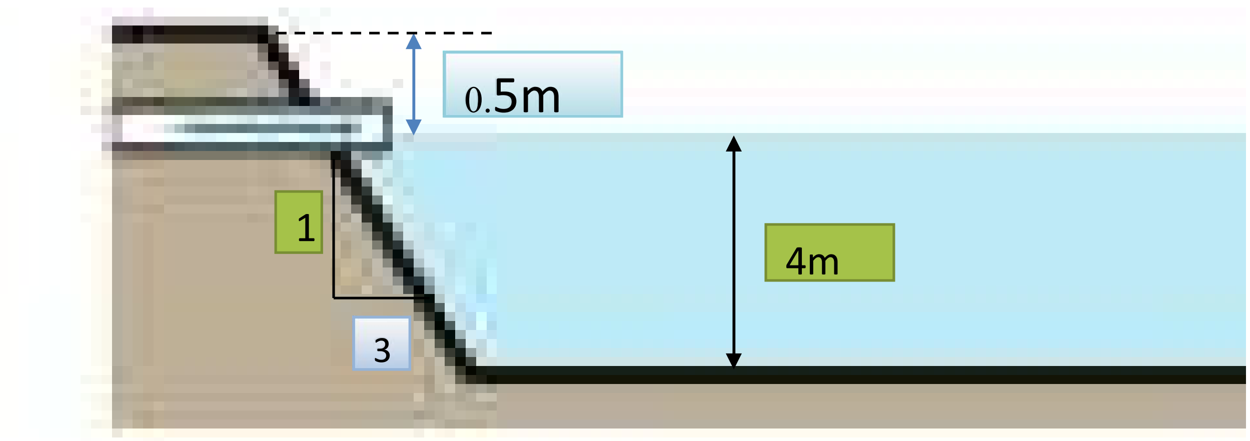
Expected BOD removed: **70%**

- Effluent BOD concentration:

$$C_e = 353 - (0.7 * 353) = \mathbf{106}\text{mg BOD/l}$$

- Effluent load:

$$L_e = 0.106 \text{ kg BOD/m}^3 * 2550\text{m}^3/\text{day} = \mathbf{270}\text{kg BOD/day}$$



Facultative pond

Calculations done at 27°c (coldest temperature) = worst case scenario

Design: based on surface loading rate λs (BOD/ha/day)

Use Arthur (1983) formula: $\lambda s = (20 * \mathrm{T}) - 60$

$= (20 * 27) - 60 = 480\mathrm{kg\ BOD/ha/day} \sim 500\mathrm{kg/ha/d}$

$$mid\_depth\ A_s = \frac{270\mathrm{kg\ BOD/day}}{500\mathrm{kg\ BOD/ha/day}} = 0.54\mathrm{ha} = 5400\mathrm{m}^2$$

Resulting volume if standard pond depth of 1.8m

$$V = A * D = 5400 * 1.8 = 9720m^3$$

$$\mathrm{HRT} = \frac{\mathrm{V}}{\mathrm{Q}} = \frac{9720}{2550} = 3.8\ \mathrm{days}$$

Biological Wastewater Treatment

Usually rectangular pond with L:W ratio 2:1, slope 33% is adopted.

$$5400\mathrm{m}^2 = \mathrm{L} * \mathrm{W} = (2 * \mathrm{W}) * \mathrm{W} = 2\mathrm{W}^2$$

$$\mathrm{W} = 52\mathrm{m} \text{ and } \mathrm{L} = 104\mathrm{m} \text{ (at half depth)}$$

$$\mathrm{W} = 57.5\mathrm{m} \text{ and } \mathrm{L} = 109.5\mathrm{m} \text{ (at water surface)}$$

$$\mathrm{W} = 60.5\mathrm{m} \text{ and } \mathrm{L} = 112.5\mathrm{m} \text{ (including free board)}$$

Expected BOD removal: 80%

Effluent BOD concentration: 106 – (0.8*106) = **21.2**mg/BOD/l

Effluent load: 0.0212kg BOD/m^3 * 2550 m^3/day = **54.1**kg BOD/day

Maturation pond:

Calculations done at 27°c (coldest temperature) = worst case scenario

$$N_e = \frac{N_i}{(1 + k_T * HRT_{anaerobic}) * \left(1 + k_T * HRT_{facultative}\right) * (1 + k_T * HRT_{maturation,n})^n}$$

Where:

N_e = 1000 FC/100ml (required by WHO for agricultural reuse)

N_i = $4*10^8$ FC/100ml (given concentration)

$k_T = k_{20°c} * \theta^{(T-20)} = 2.6 * (1.19)^{27-20} = 8.8\ day^{-1}$ (Arthur, 1983)

- ✓ HRT anaerobic = 1.01 day (worse case, if full of sludge)
- ✓ HRT facultative = 3.8 days

Calculate Ne for different numbers of maturation ponds in series and check whether or not Ne is below the standard.

n	HRT per pond	N_e (Arthur)	HRT per pond	N_e (WHO)
1	5	26,102 FC/100ml	5	422,483 FC/100ml
2	2.5	2,220 FC/100ml	3	63,921 FC/100ml
3	1.67	304 FC/100ml	3	6,030 FC/100ml

Retain Arthur solution, select three ponds, each with a HRT of 1.67 days.

$$N_e = \frac{4*10^8\ \text{FC/100ml}}{(1 + 8.8d^{-1} * 1.01d) * (1 + 8.8d^{-1} * 3.8d) * (1 + 8.8d^{-1} * 1.67d)^3}$$

$$= \frac{310FC}{100ml} \ldots almost\ equal\ to\ 304 \ldots\ldots safe!!$$

Volume per pond:

$$\text{V} = 1.67\ \text{days}\ * 2550\text{m}^3/\text{day}\ = 4258\text{m}^3$$

Take standard depth of 1.5m

Mid depth area

$$A = \frac{V}{D} = \frac{4258}{1.5} = 2839\text{m}^2$$

Normally L:W = 2:1

$$\text{W} = 38\text{m and L} = 76\text{m (at half depth)}$$
$$\text{W} = 42.5\text{m and L} = 80.5\text{m (at water surface)}$$
$$\text{W} = 45.5\text{m and L} = 83.5\text{m (including free board)}$$

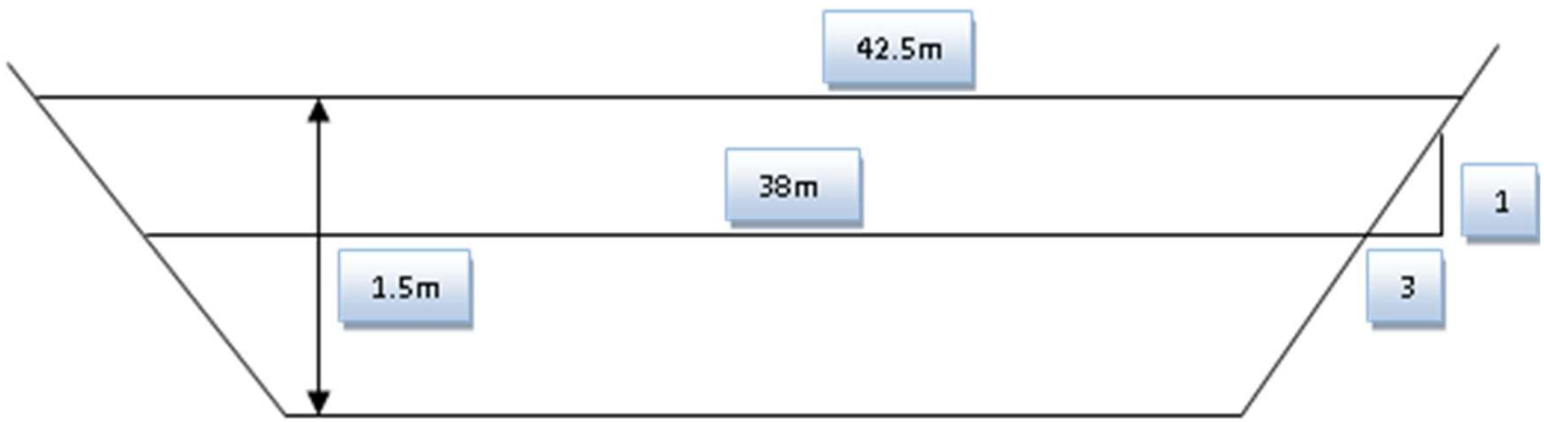


Figure of Sectional view of maturation pond

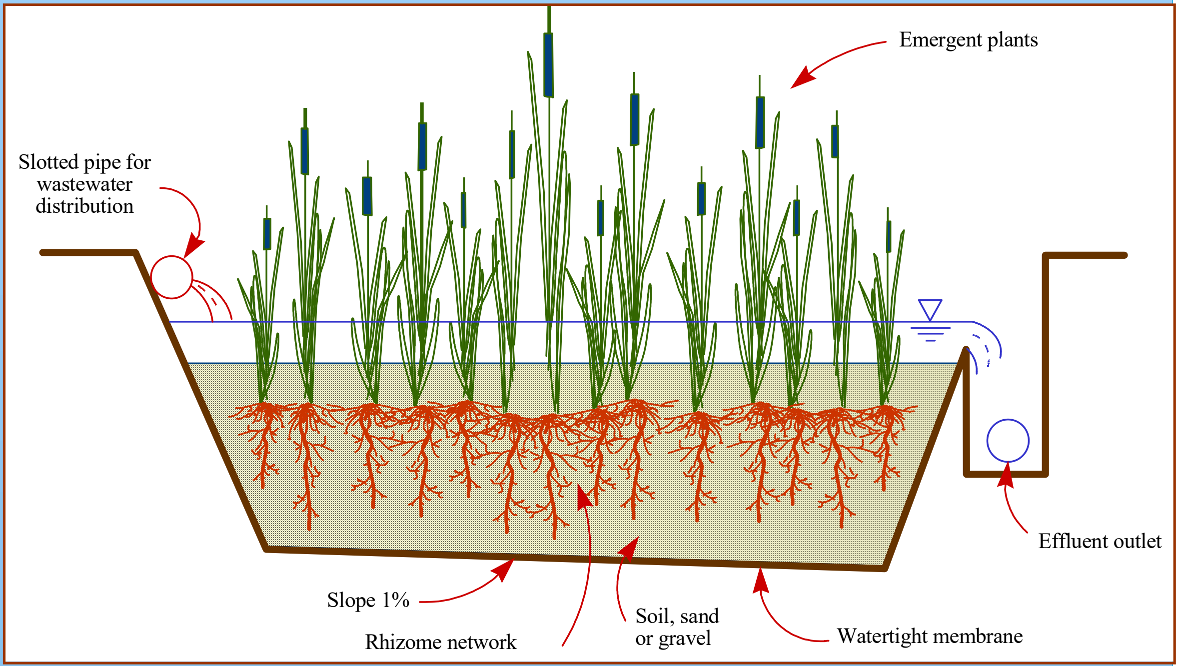
Constructed Wetlands

- a designed, manmade complex of saturated substrate, emergent and submerged vegetation, animal life, and water that simulate wetlands for human uses and benefits.
- CW's are practical alternatives to conventional treatment of domestic sewage, industrial and agricultural wastes, storm water runoff, and acid mining drainage.
- The pollutants removed by CW's include organic materials, suspended solids, nutrients, pathogens, heavy metals and other toxic or hazardous pollutants.

Biological Wastewater Treatment

Compartments in wetlands

- Sediment / gravel bed
- Root zone / pore water
- Litter / detritus
- Water
- Air
- Plants
- Roots
- Bacteria growing in biofilms



Treatment is the result of complex interactions between all these compartments

Types of Constructed Wetlands

- Free Water Surface (FWS) and Subsurface Flow (SSF) systems.

FWS

- consist of basins or channels, with some sort of subsurface barrier to prevent seepage, soil or another suitable medium, to support the emergent vegetation, and water at a relatively shallow depth.
- The shallow water depth, low flow velocity, and presence of the plant stalks and litter regulate water flow and, especially in long, narrow channels minimize short circuiting.
- In FWS systems, the flow of water is above the ground, and plants are rooted in the sediment layer at the base of water column.

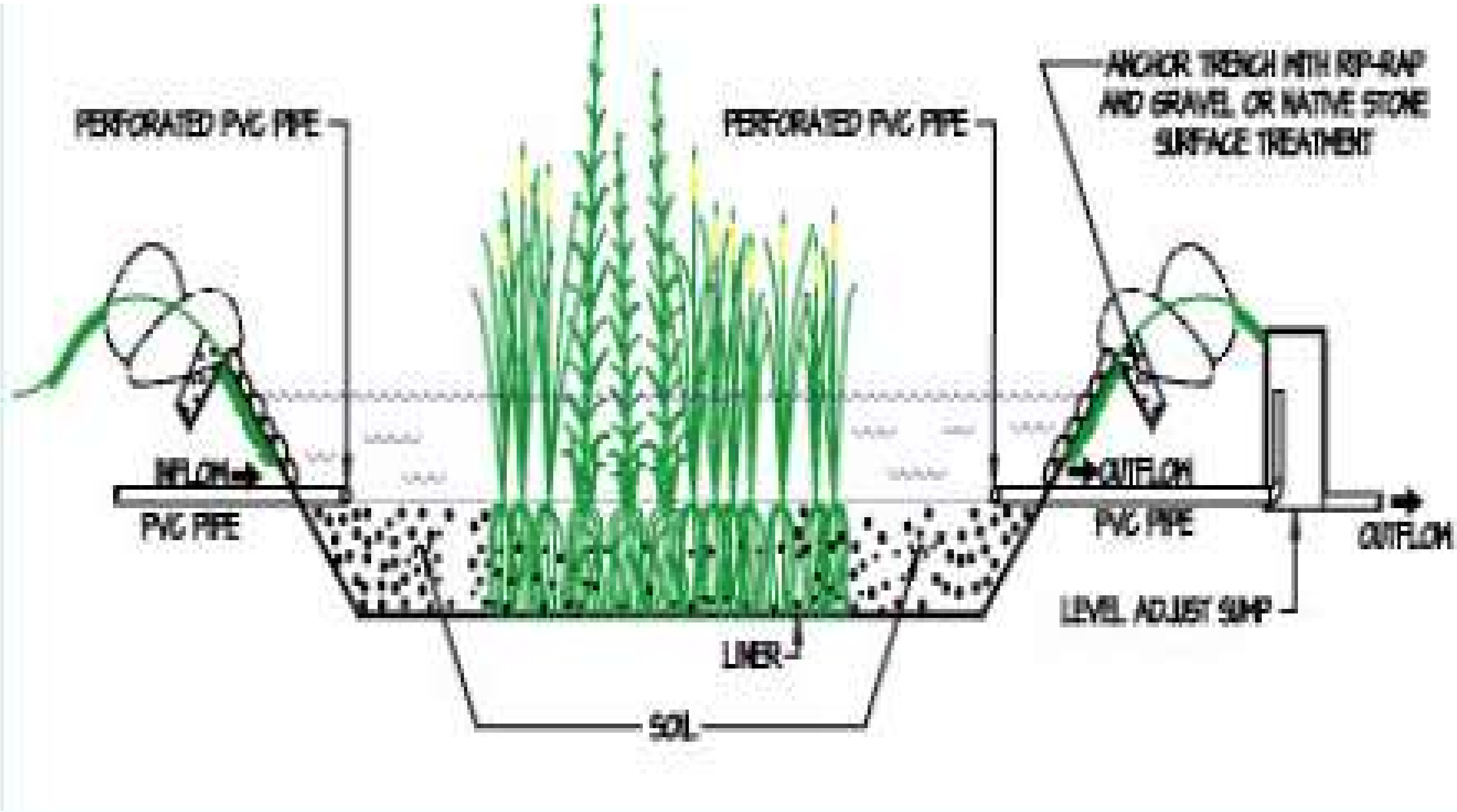


Figure of Emergent vegetation in FWS Constructed Wetlands

SSF

- These systems are essentially horizontal trickling filters when they use rock media.
- They have the added component of emergent plants with extensive root systems within the media.
- in SSF systems, water flows though a porous media such as gravels or aggregates, in which the plants are rooted.
- There are 2 types of SSF systems: horizontal SSF (HSSF) and vertical SSF (VSSF).

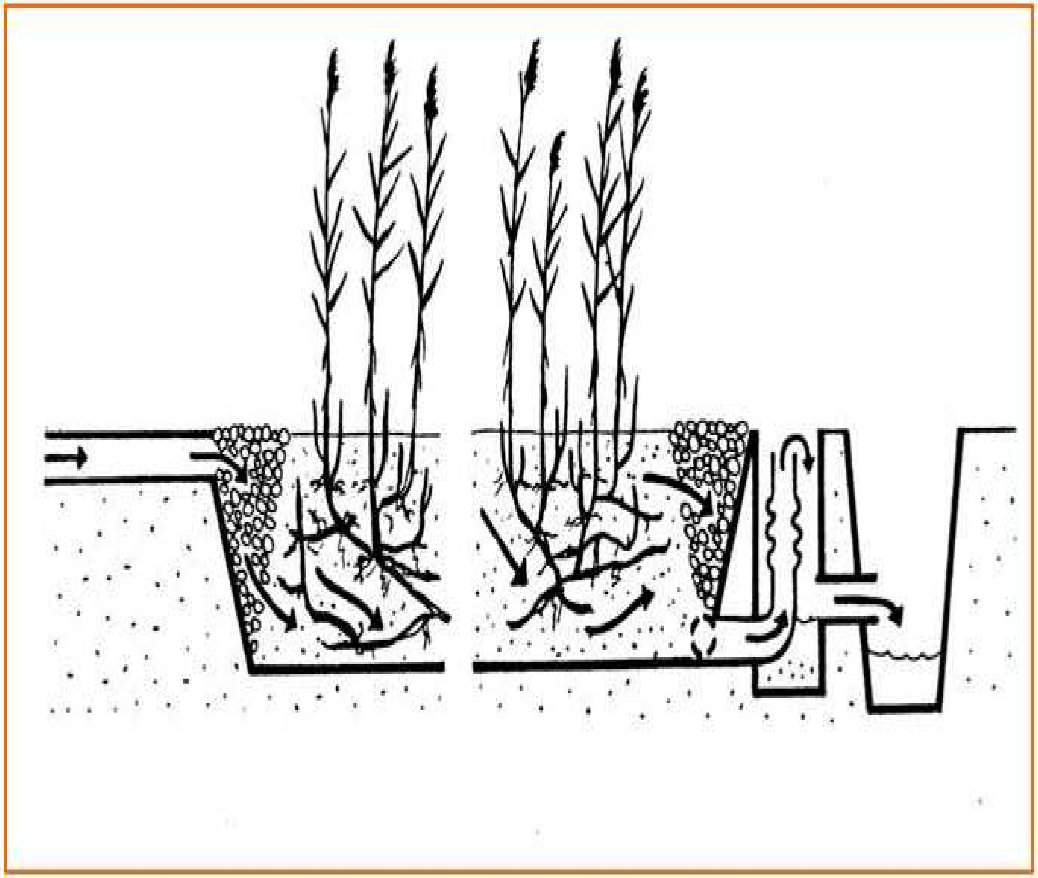


Figure of Emergent macrophyte treatment system with horizontal SSF

Site Selection for CW

- **Topography:**
  - Because grading and excavating represent a major cost factor, topography is an important consideration to select an appropriate site.
- Soil Permeability for Free Water Surface Systems
  - In selecting a site for FWS wetland the underlying soil permeability must be considered.
  - The most desirable soil permeability is 10^{-6} to 10^{-7} m/s.
  - Sandy clays and silty clay loams can be suitable when compacted.
  - Sandy soils are too permeable to support wetland vegetation.
  - Highly permeable soils needs to be lined (with clay or artificial liners).
- **Hydrological Factors**

Biological Wastewater Treatment

- Hydrological Factors such as, precipitation, infiltration, evapotranspiration (ET), hydraulic loading rate, and water depth can all affect the removal of pollutants:
  - **by altering the detention time,**
  - **by either concentrating or diluting the wastewater.**
- For a CW, the water balance can be expressed as follows:

$$Q_i - Q_o + P - ET = [\frac{dV}{dt}]$$

Where,

Q_i - influent wastewater flow, Q_o - effluent wastewater flow,
P - precipitation, volume/time ET - evapotranspiration, volume/time
V - volume of water, and t – time

- Ground-water inflow and infiltration are excluded from the above equation because of the impermeable barrier.
- if the system operates at a relatively constant water depth (dV/dt = 0), the effluent flow rate can be estimated using the above equation.

Vegetation

- The major benefit of plants is the transferring of oxygen to the root zone.
- the stalks, roots, and rhizomes penetrate the soil or support medium, and transport oxygen deeper than it would naturally travel by diffusion alone.
- The emergent plants most frequently found in WW wetlands include cattails, reeds, rushes, bulrushes and sedges.

Biological Wastewater Treatment

bulrushes (*Scirpus* spp.),

spikerush (*Eleocharis* spp.),

Cattail

Tertiary Treatment Processes

- Is a final treatment stage.
- The purpose of tertiary treatment is to raise the effluent quality before it is discharged to the receiving environment (sea, river, lake, ground, etc.).
- More than one tertiary treatment process may be used at any treatment plant.
- It is also called effluent polishing.

Tertiary Treatment Processes

Filtration

- Sand filtration removes much of the residual suspended matter.
- Filtration over activated carbon, also called carbon adsorption, removes residual toxins

Lagooning

- provides settlement and further biological improvement through storage in large man-made ponds or lagoons.
- These lagoons are highly aerobic

Nitrogen removal

Ammonia $\xrightarrow{\text{oxidation}}$ nitrate (nitrification),

Nitrate $\xrightarrow{\text{reduction}}$ nitrogen gas (de-nitrification)

Then, N2 gas is released to the atmosphere and thus removed from the water.

- Sand filters, lagooning and reed beds can all be used to reduce nitrogen,

Tertiary Treatment Processes

Phosphorus removal

- Phosphorus can be removed biologically
- Phosphorus removal can also be achieved by chemical precipitation, usually with salts of iron (e.g. ferric chloride), aluminum (e.g. alum), or lime.
- This may lead to excessive sludge production as hydroxides precipitates and the added chemicals can be expensive.

Disinfection

- Is a process of killing/deactivating the microorganisms in water.
- Common methods of disinfection include ozone, chlorine, ultraviolet light, or sodium hypochlorite.
- Chlorination remains the most common form of WW disinfection due to its low cost and long-term history of effectiveness.

Odour Control

- Odours emitted by sewage treatment are typically an indication of an anaerobic or "septic" condition.
- odours is treated with carbon reactors, a contact media with bio-slimes, small doses of chlorine, or circulating fluids to biologically capture and metabolize the obnoxious gases.
- Other methods of odour control exist, including addition of iron salts, hydrogen peroxide, calcium nitrate, etc. to manage hydrogen sulfide levels.

CHAPTER 5

SEWAGE EFFLUENT DISPOSAL TECHNIQUES

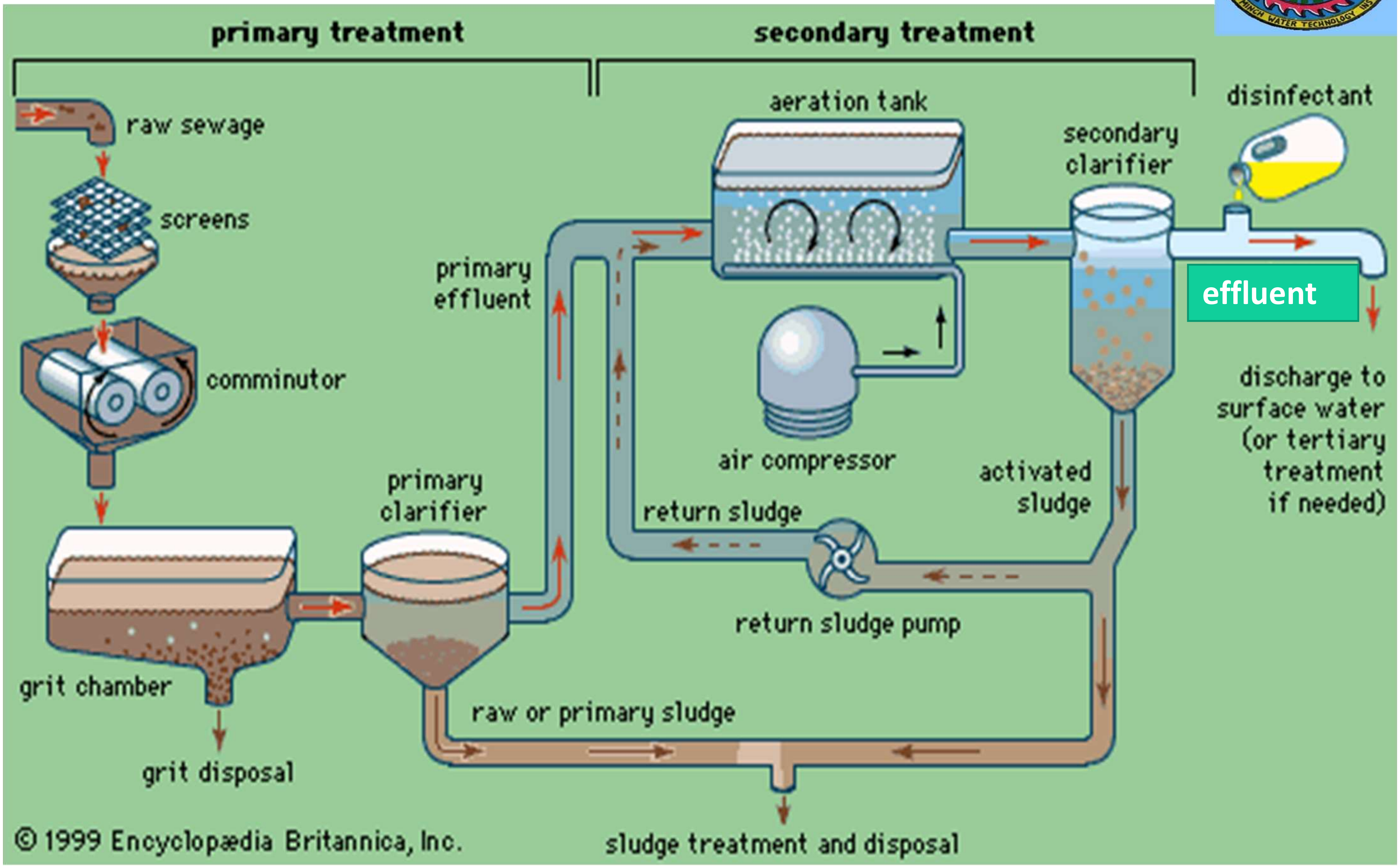


Fig. of WW treatment system and its effluent

There are two general methods of disposing of the sewage effluents:

1. Dilution i.e. disposal in water; and
2. Effluent Irrigation or Broad Irrigation or Sewage Farming, i.e. disposal on land

Disposal by Dilution and Oxygen Sag Curve

- the effluent from the sewage treatment plant is discharged into a river stream, or a large body of water, such as a lake or sea.
- The discharged sewage, in due course of time, is purified by what is known as self purification process of natural waters.
- Disposal by Dilution method is favourable:
  - (i) When sewage is comparatively fresh (4 to 5hr old), and free from floating and settleable solids,
  - (ii) When the diluting water has high dissolved oxygen content.
  - (iii) Where diluting waters are not used for the purpose of navigation or water supply,
  - (iv) Where the flow currents of the diluting waters are favorable, causing no deposition, nuisance or destruction of aquatic life.

Disposal by Dilution and Oxygen Sag Curve

Dilution in Rivers and Self Purification of Natural Streams

- the receiving water gets polluted due to waste products, present in sewage effluents.
- because the natural forces of purification, such as dilution & dispersion, sedimentation, oxidation-reduction in sun-light, etc., go on acting upon the pollution elements, and bring back the water into its original condition.
    - This automatic purification of polluted water, in due course, is called the self-purification phenomenon.

i. Dilution and Dispersion

When sewage of concentration Cs flows at a rate Qs in to a river stream with concentration CR flowing at a rate QR, the concentration C of the resulting mixture is given by:

Disposal by Dilution and Oxygen Sag Curve

$$C_{ww}Q_{ww} + C_R Q_R = C(Q_{ww} + Q_R)$$

$$C = \frac{C_{ww}Q_{ww} + C_R Q_R}{Q_{ww} + Q_R}$$

Sedimentation

The settle-able solids, if present in sewage effluents, will settle down into the bed of the river, near the outfall of sewage, thus, helping in the self purification process.

sunlight

- The sun light has a bleaching and stabilizing effect of bacteria.
- It also helps certain micro-organisms to derive energy from it

The Oxygen Deficit of a Polluted River-Stream.

- ✓The oxygen deficit D at any time in a polluted river-stream is the difference b/n the actual DO content of water at that time and the saturation DO content.

$$\text{Oxygen deficit (D)} = \text{Saturation DO} - \text{Actual DO}$$

- ✓In order to maintain clean conditions in a river-stream, the D must be nil, and this can be found out by knowing the rates of de-oxygenation and re-oxygenation.

Rate de-oxygenation

- ✓Is due to the amount of organic matter remaining to be oxidized at the given time (i.e. Lt).

Disposal by Dilution and Oxygen Sag Curve

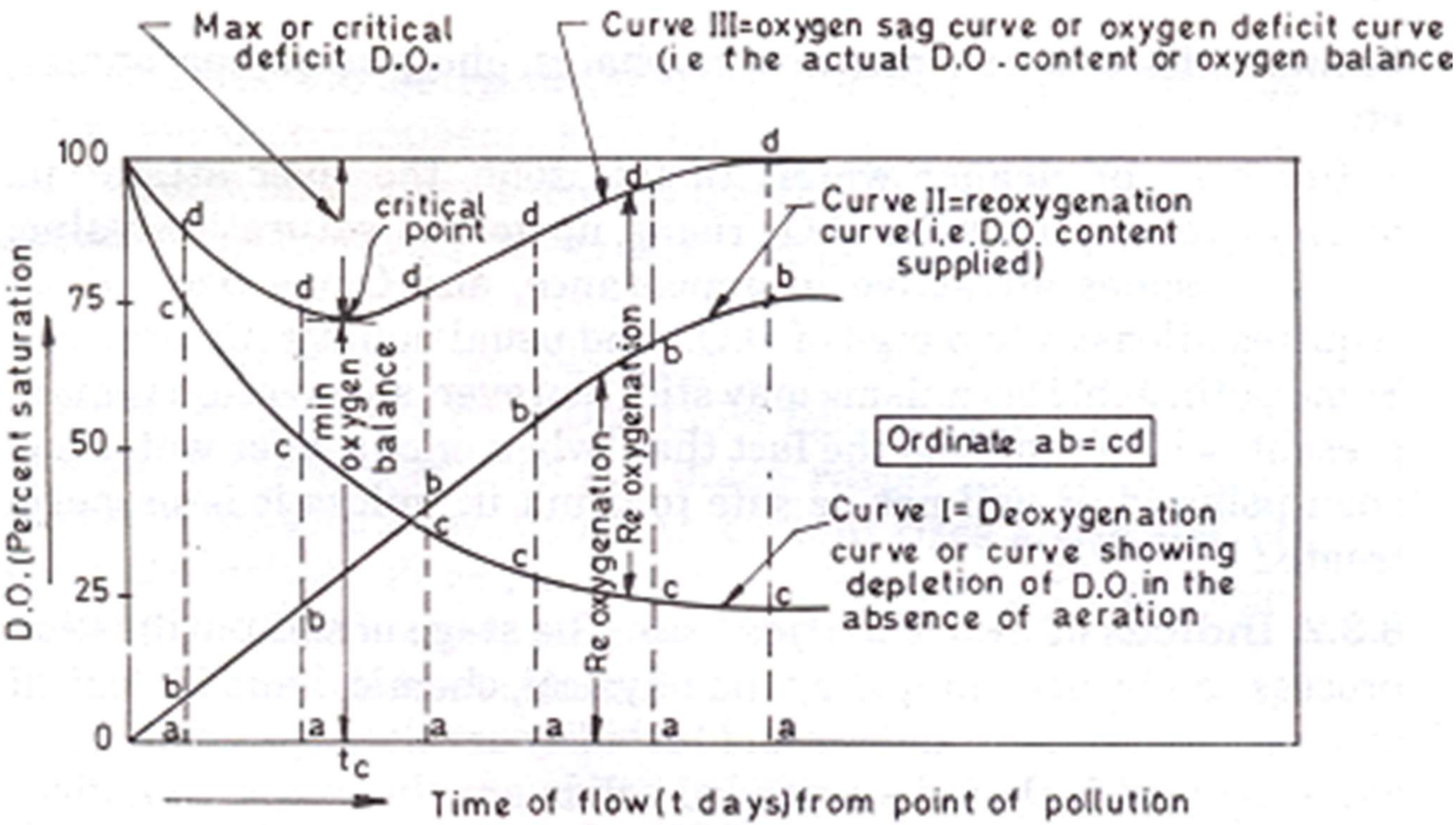


Figure of De-oxygenation curve (rate of oxygen consumption in river)

Disposal by Dilution and Oxygen Sag Curve

Rate Re-oxygenation

- Due to atmosphere supplies of oxygen to the water

Oxygen Deficit Curve

- In a running polluted stream exposed to the atmosphere, the de-oxygenation as well as the re-oxygenation go hand in hand.
- If de-oxygenation is more rapid than the re-oxygenation, an oxygen deficit will result.
- The amount of resultant oxygen deficit can be obtained by algebraically adding the de-oxygenation and re-oxygenation curves (see curve III-Figure above).
- The resultant curve so obtained is called the oxygen sag curve or the oxygen deficit curve.

Disposal by Dilution and Oxygen Sag Curve

- It can also be seen that when the de-oxygenation rate exceeds the re-oxygenation rate, the oxygen sag curve shows increasing deficit of oxygen;
- but when both the rates become equal, the critical point is reached, and then finally when the rate of de-oxygenation falls below that of re-oxygenation, the oxygen deficit goes on decreasing till becoming zero.
- The entire analysis of super-imposing the rates of de-oxygenation and re-oxygenation have been carried out mathematically, and the obtained results expressed in the form of famous Streeter-Phelps equation; i.e.,

$$D_t = \frac{K_D * L}{K_R - K_D} * [(10)^{-K_D*.t} - (10)^{-K_R*t}] + [D_0 * (10)^{-K_R*t}] \ldots 5.2$$

Disposal by Dilution and Oxygen Sag Curve

Where, D_t = the DO deficit in mg/l after t days.

L = Ultimate first stage BOD of the mix at the point of waste discharge

D_o = Initial oxygen deficit of the mix at the mixing point in mg/l.

K_D = De-oxygenation coefficient (BOD rate constant) for the WW, determined in the laboratory. K_D varies with temperature as:

$$K_{D(T)} = K_{D(20)} * (1.047)^{T-20°}$$

K_R = Re-oxygenation coefficient for the stream, varies with temperature as per the equation:

$$K_{R(T)} = K_{R(20)} * (1.016)^{T-20°}$$

t = x/u, x=distance at which D_t occur, u=velocity of river

✓The oxygen deficit curve can be plotted easily with the help of Eq. (5.2), by using different values of t in days:

Disposal by Dilution and Oxygen Sag Curve

The critical time (t_c) at which the minimum dissolved oxygen occurs can be found by differentiating Eq. (5.2) and equating it to zero; which on solving gives

$$t_c = \left[\frac{1}{K_R - K_D}\right] \log\left(\left\{\frac{K_D * L - K_R * Do + K_D * D_0}{K_D * L}\right\} * \frac{K_R}{K_D}\right) \ldots\ldots 5.3$$

and the critical or maximum oxygen deficit is given by

$$D_c = \frac{K_D * L}{K_R}(10)^{-K_D * t_c} \ldots\ldots\ldots 5.4$$

The constant K_R/K_D is sometimes represented by f, called self-purification constant

Then equation 5.3 become, $t_c = \frac{1}{K_D(f-1)} \log\left(\left\{1-(f-1)*\frac{D_0}{L}\right\}*f\right) \ldots 5.5$

$$D_c = \frac{L}{f}(10)^{-K_D * t_c} \ldots\ldots\ldots\ldots\ldots 5.6$$

Disposal by Dilution and Oxygen Sag Curve

Taking logarithm of equation 5.6, we get

$$\log D_c = \log\frac{L}{f} - K_D * t_c \quad \ldots\ldots 5.7$$

Substituting the value of t_c from Eq. (5.5) in Eq. (5.7), we get

$$\log D_c = \log\frac{L}{f} - \frac{K_D}{K_D(f-1)} \log\left(\left\{1-(f-1)*\frac{D_0}{L}\right\}*f\right)$$

$$\log D_c = \log\frac{L}{f} - \frac{1}{(f-1)} \log\left(\left\{1-(f-1)*\frac{D_0}{L}\right\}*f\right)$$

$$(f-1)\left(\log\frac{L}{f} - \log D_c\right) = \log\left(\left\{1-(f-1)*\frac{D_0}{L}\right\}*f\right)$$

$$\log\left(\frac{\frac{L}{f}}{D_c}\right)^{(f-1)} = \log\left(\left\{1-(f-1)*\frac{D_0}{L}\right\}*f\right)$$

$$\left(\frac{\mathbf{L}}{\boldsymbol{D_c * f}}\right)^{(\mathbf{f-1})} = \boldsymbol{f} * \left(\mathbf{1} - (\boldsymbol{f}-\mathbf{1}) * \frac{\boldsymbol{D_0}}{\boldsymbol{L}}\right)$$

Examples

1. The sewage of a town is to be discharged into a river stream. The quantity of sewage produced per day is 8 million liters, and its BOD is 250mg/l. If the discharge in the river is 200l/s and its BOD is 6mg/l, find out the BOD of the diluted water.

Solution:

Disposal by Dilution and Oxygen Sag Curve

2. A city discharges 100 cumecs of sewage into a river, which is fully saturated with oxygen and flowing at the rate of 1500 cumecs during its lean days with a velocity of 0.1m/sec. The 5-days BOD of sewage at the given temperature is 280mg/l. Find when and where the critical DO deficit will occur in the downstream portion of the river, and what is its amount? Assume coefficient of purification of the stream (f) as 4.0 and coefficient of de-oxygenation (KD as 0.1)

Solution:

$Q_{ww} = 100m^3/s$ $\qquad$ $Q_{river} = 1500m^3/s$

$BOD_{5ww} = 280mg/l$ $\qquad$ $BOD_{5river} = 0$

KD = 0.1/day $\qquad$ f = 4.0

Disposal by Dilution and Oxygen Sag Curve

Using, $C_sQ_s + C_RQ_R = C(Q_s + Q_R)$

$$BOD_sQ_s + BOD_RQ_R = BOD_{combined}\,(Q_s + Q_R)$$

$$100*280 + 0*1500 = BOD_{combined}\,(100+1500)$$

$$BOD_{combined} = 17.5mg/l$$

First critical time, $t_c = \frac{1}{K_D(f-1)} \log\left(\left\{1-(f-1)*\frac{D_0}{L}\right\}*f\right)$ in day

But L is unknown which mixed ultimate BOD

$$BOD_5 = L*\left(1-10^{-K_Dt}\right)$$

$$17.5 = L*(1-10^{-0.1*5})$$

L = 25.6mg/l

Therefore, $t_c = \frac{1}{0.1(4-1)} \log\left(\left\{1-(4-1)*\frac{0}{25.6}\right\}*4\right) =$

Disposal by Dilution and Oxygen Sag Curve

- WHAT ABOUT AMOUNT?

$$\left(\frac{L}{D_c * f}\right)^{(f-1)} = f * \left(1 - (f-1) * \frac{D_0}{L}\right)$$

$$\left(\frac{25.6}{D_c * 4}\right)^{(4-1)} = 4 * \left(1 - (4-1) * \frac{0}{25.6}\right)$$

- WHERE?

Distance = velocity of river * time (critical)

Exercise

1. A city discharges 1500 litres per second of sewage into a stream whose minimum rate of flow is 6000 litres per second. The temperature of sewage as well as water is 20°c. The 5 day BOD at 20°c for sewage is 200mg/l and that of river water is 1mg/l. The DO content of sewage is zero, and that of the stream is 90% of the saturation DO If the minimum DO to be maintained in the stream is 4.5 mg/l, find out the degree of sewage treatment, required. Assume the de-oxygenation coefficient as 0.1 and re-oxygenation coefficient as 0.3.

Disposal by Dilution and Oxygen Sag Curve

2) A waste water effluent of 560 l/s with a BOD = 50mg/l, DO = 3.0 mg/l and temperature of 23°c enters a river where the flow is 28m3/sec, and BOD = 4.0 mg/l, DO = 8.2mg/l, and temperature of 17°c. K1 of the waste is 0.10 per day at 20°c. The velocity of water in the river downstream is 0.18m/s and depth of 1.2m.

Determine the following after mixing of waste water with the river water:

- Combined discharge
- BOD
- DO
- Temperature

Disposal of Sewage Effluents on Land for Irrigation

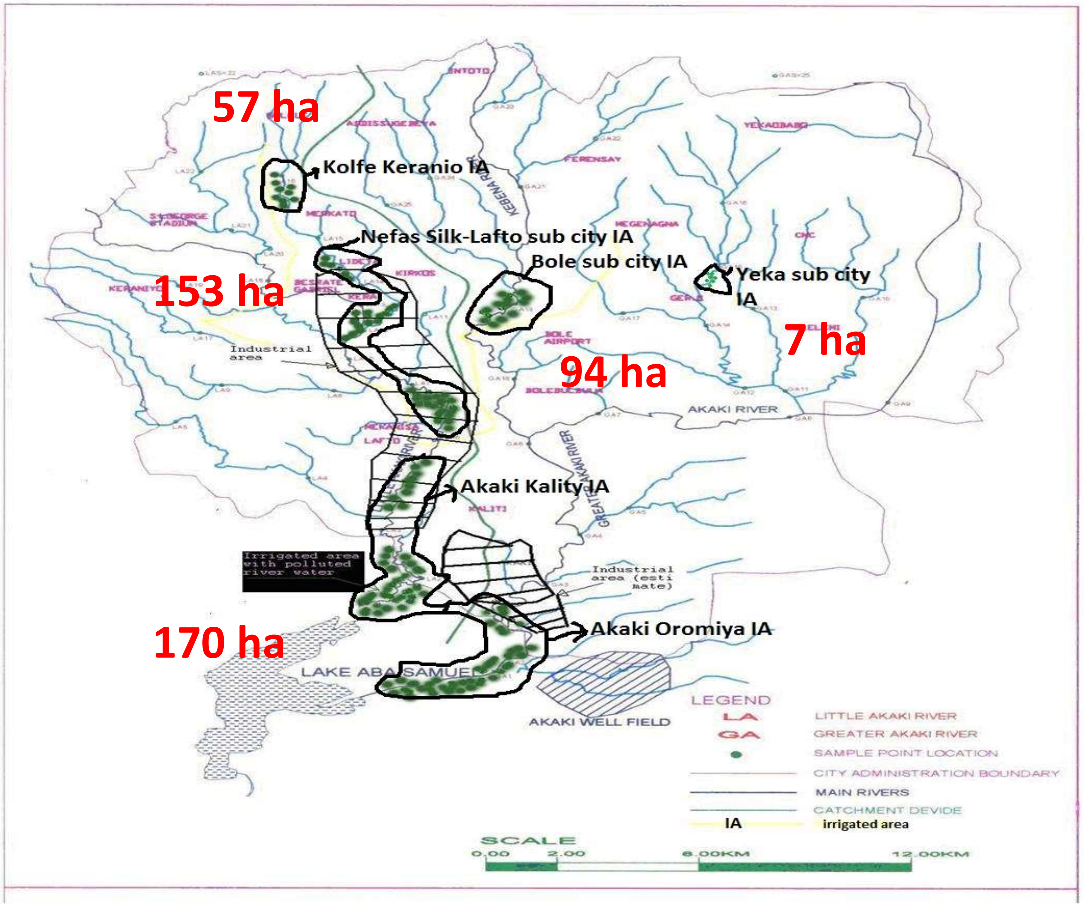
- In this method, the sewage effluent is generally disposed of by applying it on land.
- This method can then be used for irrigating crops.
- help in increasing crop yields (by 33% or so) (fig below).
- When raw or partly treated sewage is applied on to the land, a part of it evaporates, and the remaining portion percolates through the ground soil.
- While percolating through the soil, the suspended particles present in the sewage are caught in the soil voids.
- If proper aeration of these voids is maintained, the organic sewage solids caught in these voids get oxidized by aerobic process.

Land Disposal and Treatment

Fig. of WWT effluent for irrigation

Land Disposal and Treatment

Land Disposal and Treatment



Wastewater Irrigated Area (Total 481 ha)

Land Disposal and Treatment

- Application of too strong or too heavy load of sewage will also similarly result in the quick formation of anaerobic conditions.
  - soil to get clogged
- Hence, the sewage load should be reduced by diluting it or by pre-treating it.

Table of Recommended Doses for Sewage Farming

Types of soil	Doses of sewage in cubic meters per hectare per day	
	Raw sewage	Settled sewage
Sandy	120 – 150	220 – 280
Sandy loam	90 – 100	170 – 220
loam	60 – 80	110 – 170
Clayey loam	40 – 50	60 – 110
clayey	30 – 45	30 - 60

Examples

1. A town having population of 40,000 disposes sewage by land treatment. It gets a per capita assured water supply from waterworks at a rate of 130l/day. Assuming that the land used for sewage disposal can absorb 80m3 of sewage per hectare per day, determine the land area required, and its cost at the rate of $25,000 per hectare. Make suitable assumptions where needed.

Solution:

Population = 40,000

Rate of water supply = 130 l/day/person

Total water supplied per day

= 40,000*130l/d=5,200,000l/d=5,200 m^3/d

Land Disposal and Treatment

- Assuming that 80% of this water appears as sewage,
- The quantity of sewage produced per day
$$= 0.8 * 5200 = 4160 m^3/d.$$
- Therefore, area of land required for disposing sewage
$$= \frac{4160\, m^3/d}{80\, m^3/ha.d} = 52 \text{ hectares}$$
- Providing 50% extra land for rest and rotation,
- The total land area required
$$= 1.5 * 52 = 78 \text{ hectares}$$
- Cost of land involved
$$= 25{,}000 * 78 = \$1{,}950{,}000$$

Exercise

2. A town disposes sewage by land treatment. It has a sewage farm of area 150 hectares. The area included an extra provision of 50% for rest and rotation. The population of the town being 50,000 and rate of water supply 140 litres per capita per day. If 75% of the water is converted into sewage, determine the consuming capacity of the soil.

SLUDGE TREATMENT AND DISPOSAL

Introduction to sludge

WHAT IS SLUDGE?

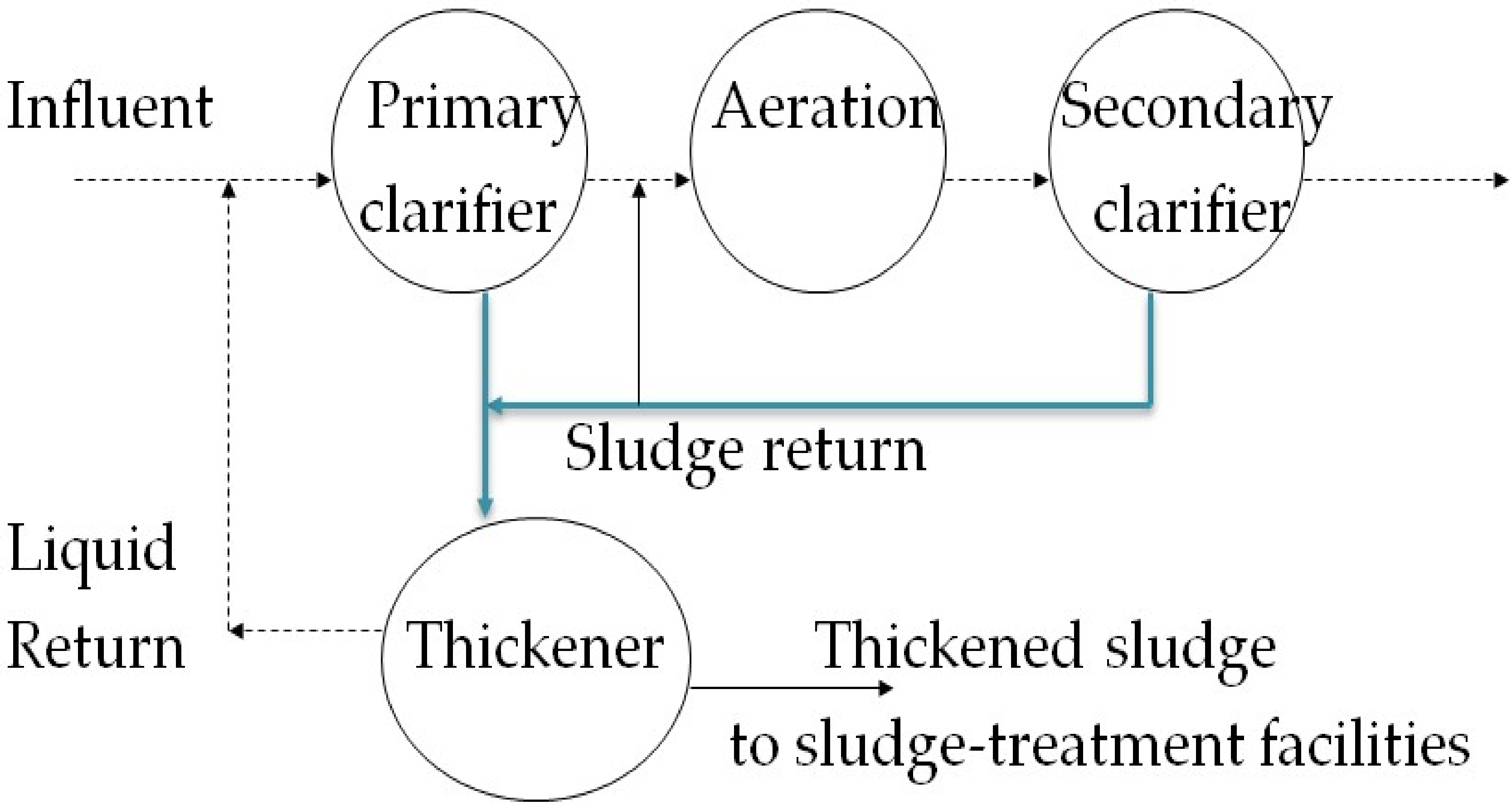
YorkshireWater

WHAT IS SLUDGE?

- Mainly water (up to 99%)
- Dissolved solids
- Settled and suspended solids
  - Faecal matter
  - Bacteria and other micro-organisms
  - Nutrients (N, P, K)
  - Metals
  - Energy

YorkshireWater

Source of sludge



Objective of sludge management

Reduce volume (removal of water)
Reduce/remove odour
Stabilise organic material (BOD removal)
Remove pathogens
Reclaim useful by-products (biogas, soil conditioners)
Safe/appropriate disposal & recycling

Main type of sludge treatment

- Thickening
- Digestion
- Advanced Digestion
- Dewatering
- Conditioning
- Phyto-conditioning
- Incineration

YorkshireWater

Main type of sludge treatmen

Sludge Thickening

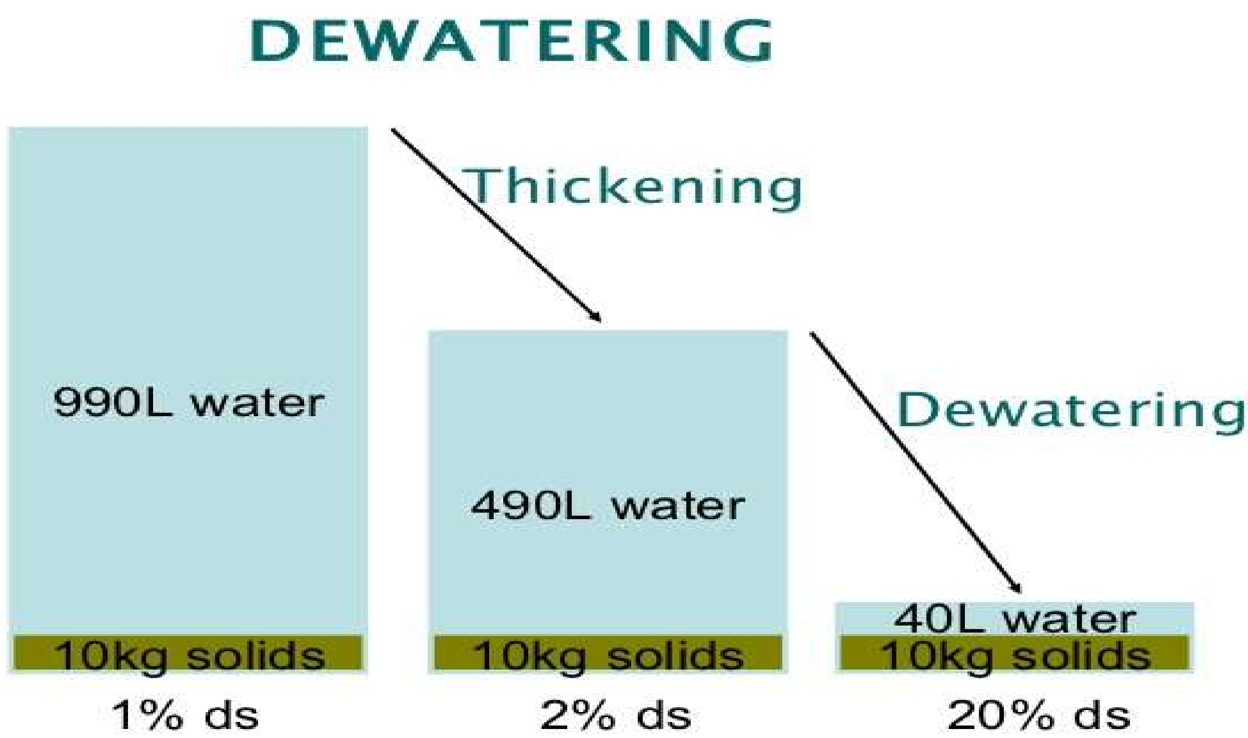
- is a procedure used to remove water and increase the solids content.

Sludge Digestion Process

- The sludge withdrawn from the sedimentation basins contains a lot of putrescible organic matter, and if disposed of without any treatment, the organic matter may decompose, producing foul gases and a lot of nuisance, pollution and health hazards.
- In order to avoid such pollutions, the sludge should stabilized by decomposing the organic matter under controlled anaerobic conditions, and then disposed of suitably after drying on drying beds, etc.

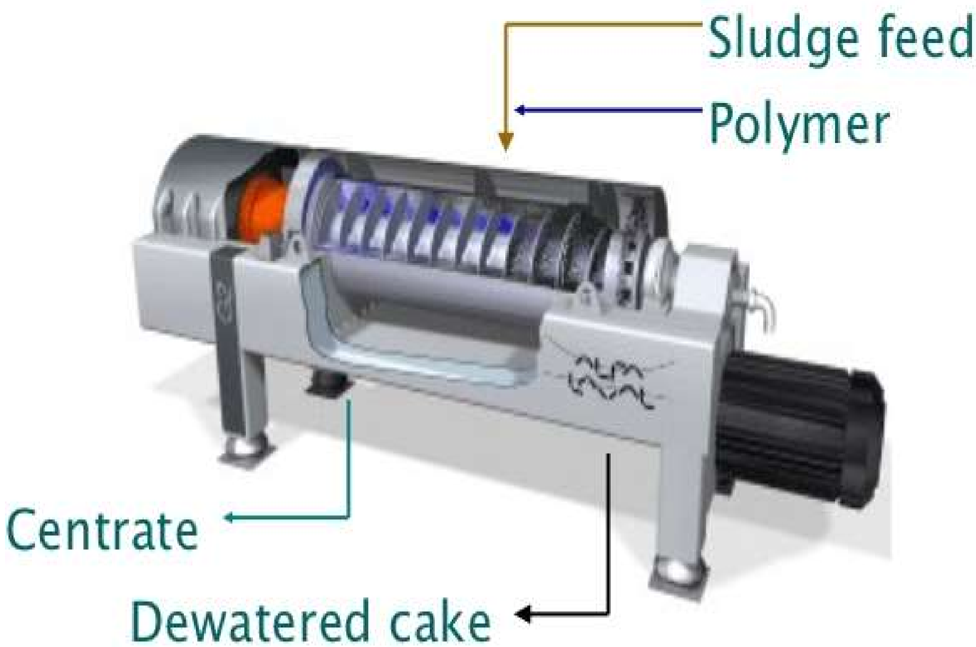
Main type of sludge treatment

- Factors Affecting Sludge Digestion and Their Control
  - Temperature, pH value, Seeding with digested sludge, Mixing and stirring of the raw sludge with digested sludge.



Main type of sludge treatment

DEWATERING



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DEWATERING

ireWater

Main type of sludge treatment

CONDITIONING

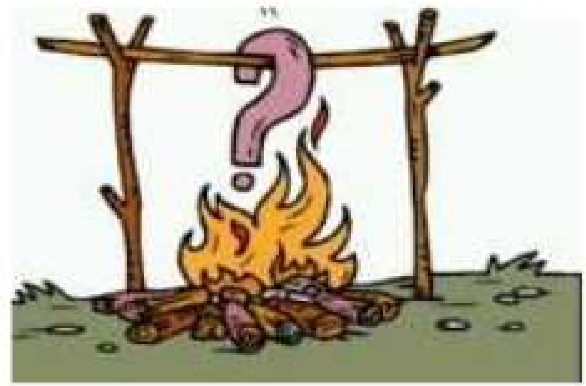
CONDITIONING – Aim

Provides required secondary retention
Allows drainage of free water
Forms a stable, friable product
Promotes aerobic conditions

Disposal of Dewatered Sludge

- ✓Dumping into the Sea
- ✓Burial into the Trenches
- ✓Incineration

-END-



ANY
QUESTIONS??

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