EVEN -018 WASTEWATER TREATMENT PLANTS

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BASICS

- WHAT IS WASTEWATER?
- WHY IT IS ENVIRONM. UNFRIENDLY?
- WASTEWATER COMPOSITION?
- WHAT ARE WASTEWATER SOURCES?
- WHY WE NEED TO TREAT IT?
- WASTWATER CHARACHTERISTICS?
- WHAT ARE THE TWO BASIC EQUATIONS OF WASTEWATER TREATMENTS?

COMPOSITION OF WW

- 99.9 WATER + 0.1 SOLIDS
- TS = 100 THEN;
  - TSS = 30
    - SETLEABLES = 15 (ORGANIC = 10 AND INORGANIC = 5)
    - NON SETLEABLES = 15 (ORGANIC = 10 AND INORGANIC = 5)
  - TDS = 70
    - ORGANIC = 30
    - INORGANIC = 40
- TOTAL ORGANIC = 50
- TOTAL INORGANIC = 50

REMEMBER!

- ORGANICS + O_2 $\longrightarrow$ STABLE ORG. MATTER + CO_2 + H_2O
- ORGANICS (WITHOUT O_2) $\longrightarrow$ STABLE ORG. MATTER + CO_2 + H_2O

TERMINOLOGIES TO BE REMEMBERED!

- BOD 5, 20
- COD
- TS = TSS + TDS = TVS + TFS
- TSS = TVSS + TFSS
- TDS = TVDS + TFDS
- TVS = TVSS + TVDS
- TFS = TFSS + TFDS
- MLSS, MLVSS

WASTEWATER TREATMENT ENVIRONMENT… THINK ABOUT IT!!!!!

- TEMP., PH, NO TOXIC…
- WHAT BACTERIA CAN EAT!!!!!
- IS FOOD CONCENTRATION MATTERS!!!!! ….FOOD TO MICROORGANISMS RATIO.
- IS TIME IMPORTATNT? WHAT TIME? HRT OR SLUDGE TIME??? DO YOU KNOW SLUDGE AGE..?
- NUTRIENTS MATTERS? ORGANIC:N:P = 100 : 5 : 1
- AERATE AND MIX WITH NO SETTLING,….. SETTLE WITH NO AERATE AND MIX!!!! ***DON'T MIX PROCESSES!!!***

WASTEWATER TREATMENT ENVIRONMENT… THINK ABOUT IT!!!!!

- SOLIDS SEPARATE FIRST BY EASY MEANS..
- GIVE AEROBIC BACTERIA AIR AND PREVENT AIR FROM ANAEROBIC BACTERIA … REMEMBER THAT!!!
- HOW U GIVE AIR TO AEROBIC ANAEROBES (CASCADES, MECHANICAL (SA, DIFFUSERS), NATURALLY).

WASTEWATER TREATMENTS

- PRE-TREATMENT (SCREEN, GRIT REMOVAL)
- PRIMARY TREATMENT (PLAIN SETTLING OF DISSOLVED AIR FLOTATION)
- SECONDARY TREATMENT
- TERTIARY TREATMENT

WASTEWATER TREATMENTS

- PRETREATMENT
  - SCREEN; MECHANICAL AND MANUAL
  - GRIT REMOVAL
  - OIL AND GREASE
  - PRE-AERATION
  - EQUALIZATION
  - HEAVY METALS SEPARATION

WASTEWATER TREATMENTS

- PRIMARY
  - PHYSICAL; FLOTATION, SEDIMENTATION
  - CHEMICAL; NEUTRALIZATION, COAGULATION

WASTEWATER TREATMENTS

- SECONDARY
  - DISSOLVED ORGANIC REMOVAL
    - SUSPENDED GROWTH
      - ACTIVATED SLUDGE, OXIDATION DITCH, AERATED LAGOON, PONDS, UASB, ANAEROBIC POND
    - ATTACHED GROWTH
      - TRICKLING FILTER, RBC, FLUDIZED BED, UAFFRAERATED LAGOON
  - SUSPENDED SOLIDS REMOVAL
    - SETTLING; PLAIN OR CHEMICALLY

WASTEWATER TREATMENTS

- TERTIARY
  - FILTRATION
  - COAGULATION AND SEDIMENTATION
  - CARBON ADSORPTION
  - MEMBRANE
  - MICROSTRAINER
  - POLISHING PONDS
  - WEEDS

WASTE WATER TREATMENT

•Primary Treatment (Physical Treatment)

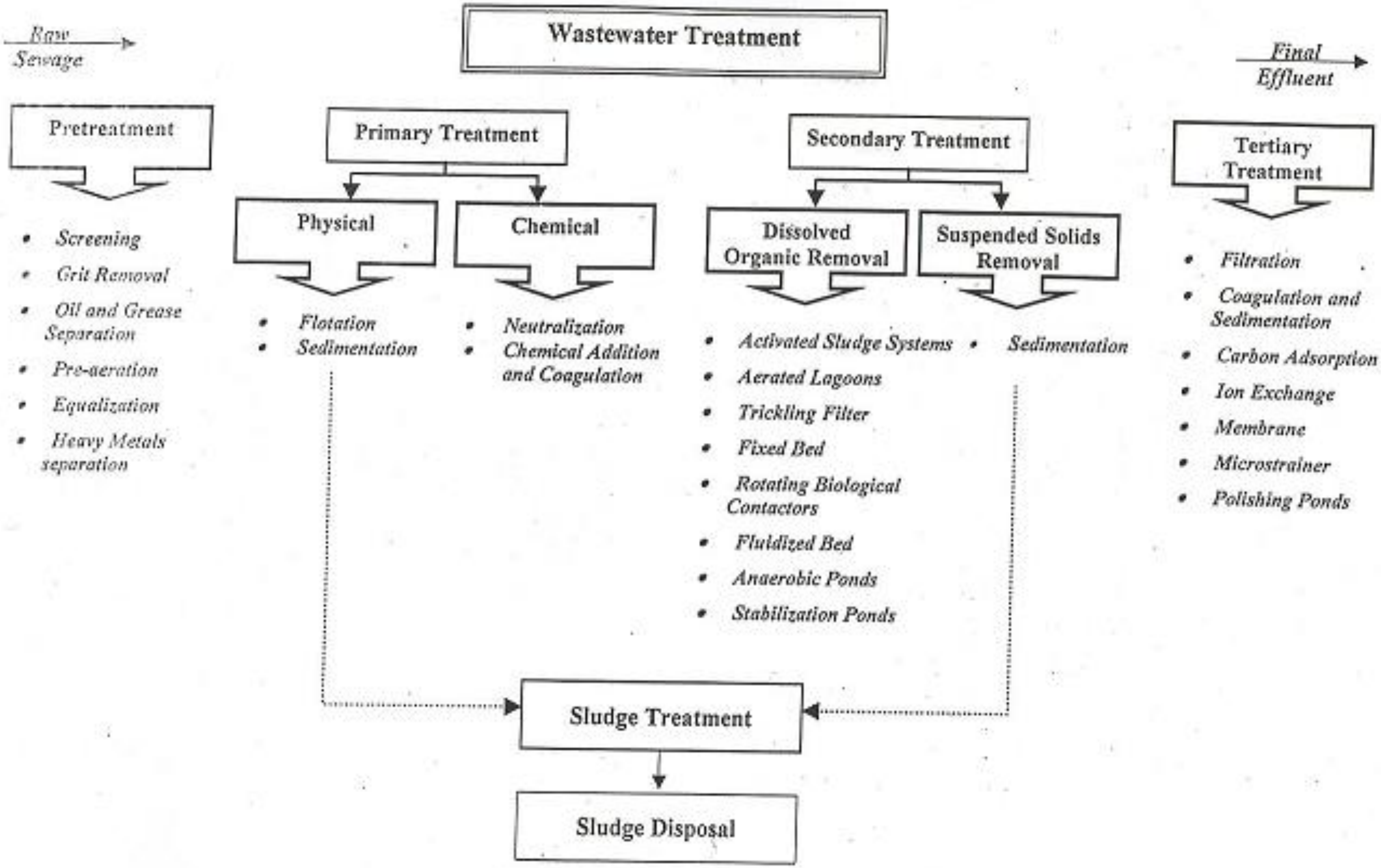
Physical treatment depends only on the physical characteristics of impurities that present in waste water to be treated. So, its efficiency is small.

•Secondary Treatment (Biological Treatment)

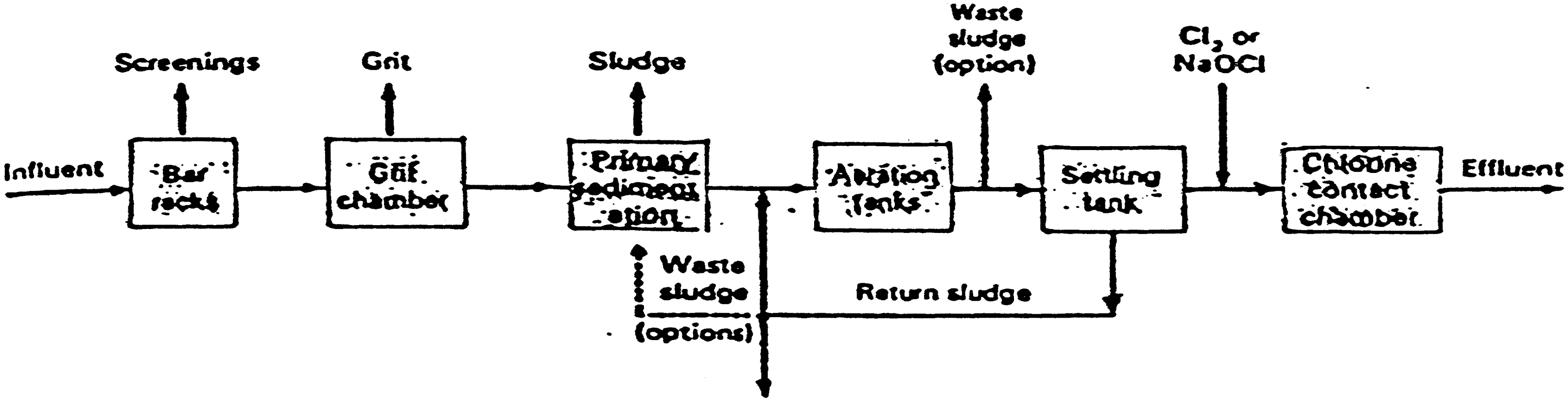
Secondary treatment depends on the biological characteristics of microorganisms that present in waste water to be treated such as bacteria that helps in the removal of organic matters.

•Tertiary Treatment (Polishing)

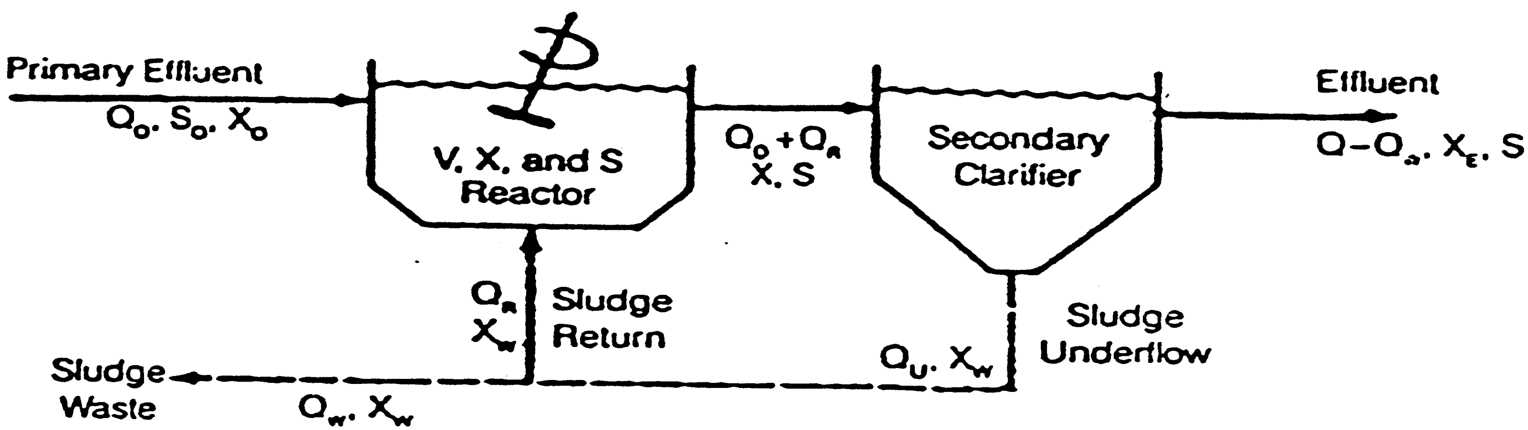
In case of required an effluent with very good characteristics, tertiary treatment has been done. It can remove salts, odor, taste and the rest of organic matter using for example (Activated Carbon Filter, Dynasand Filter.…etc).



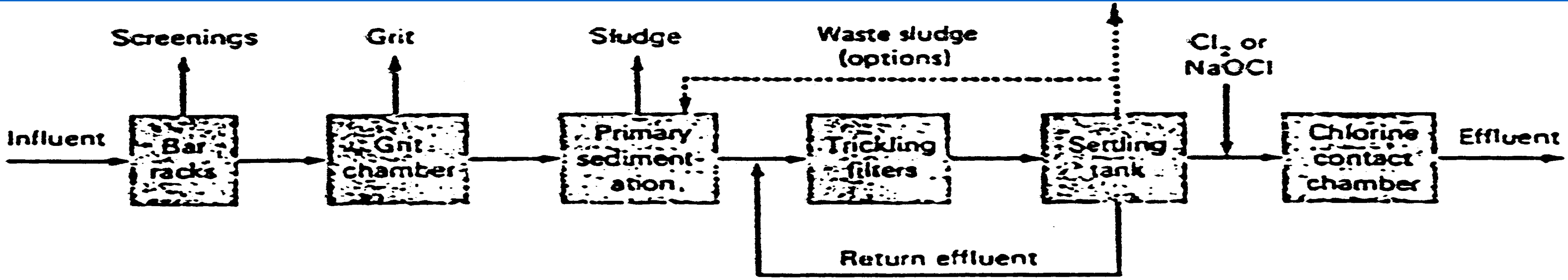
Main Wastewater Treatment Sequence and Substitution Diagram



Screenings
Grit
Sludge
Waste sludge (option)
Cl_2 or NaOCl
Influent
Bar racks
Grit chamber
Primary sedimentation
Aeration tanks
Settling tank
Chlorine contact chamber
Effluent
Waste sludge (options)
Return sludge



Primary Effluent
Q_o, S_o, X_o
V, X, and S
Reactor
Q_o+Q_R
X, S
Secondary
Clarifier
Effluent
$Q-Q_w, X_E, S$
Q_R
X_w
Sludge
Return
Sludge
Underflow
Q_U, X_w
Sludge
Waste
Q_w, X_w


Screenings
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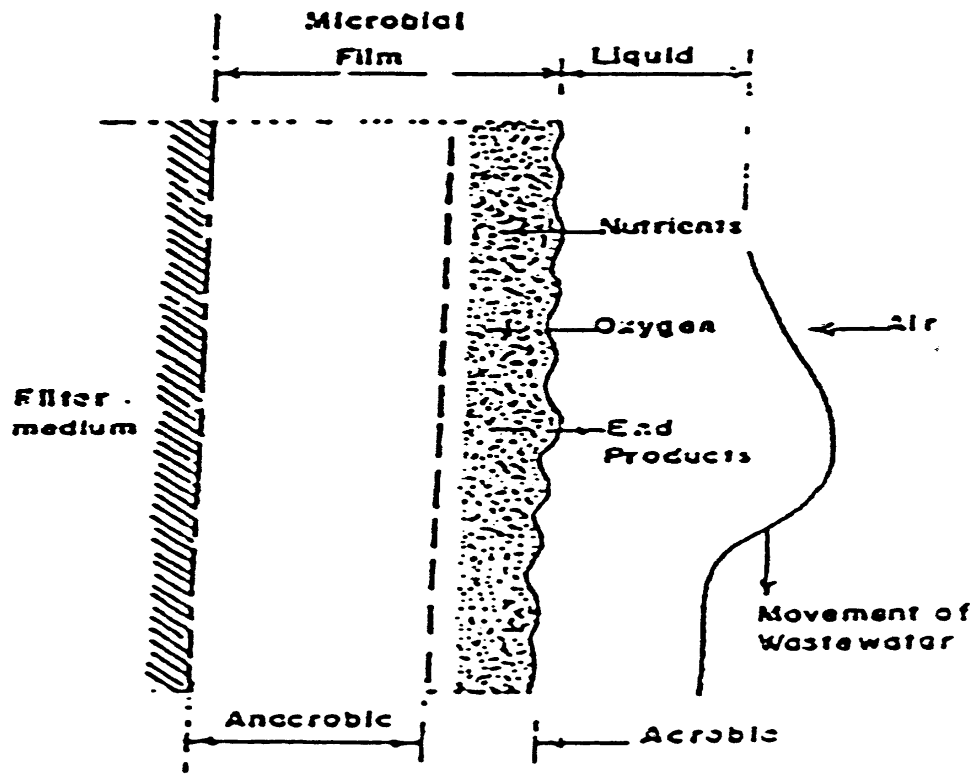
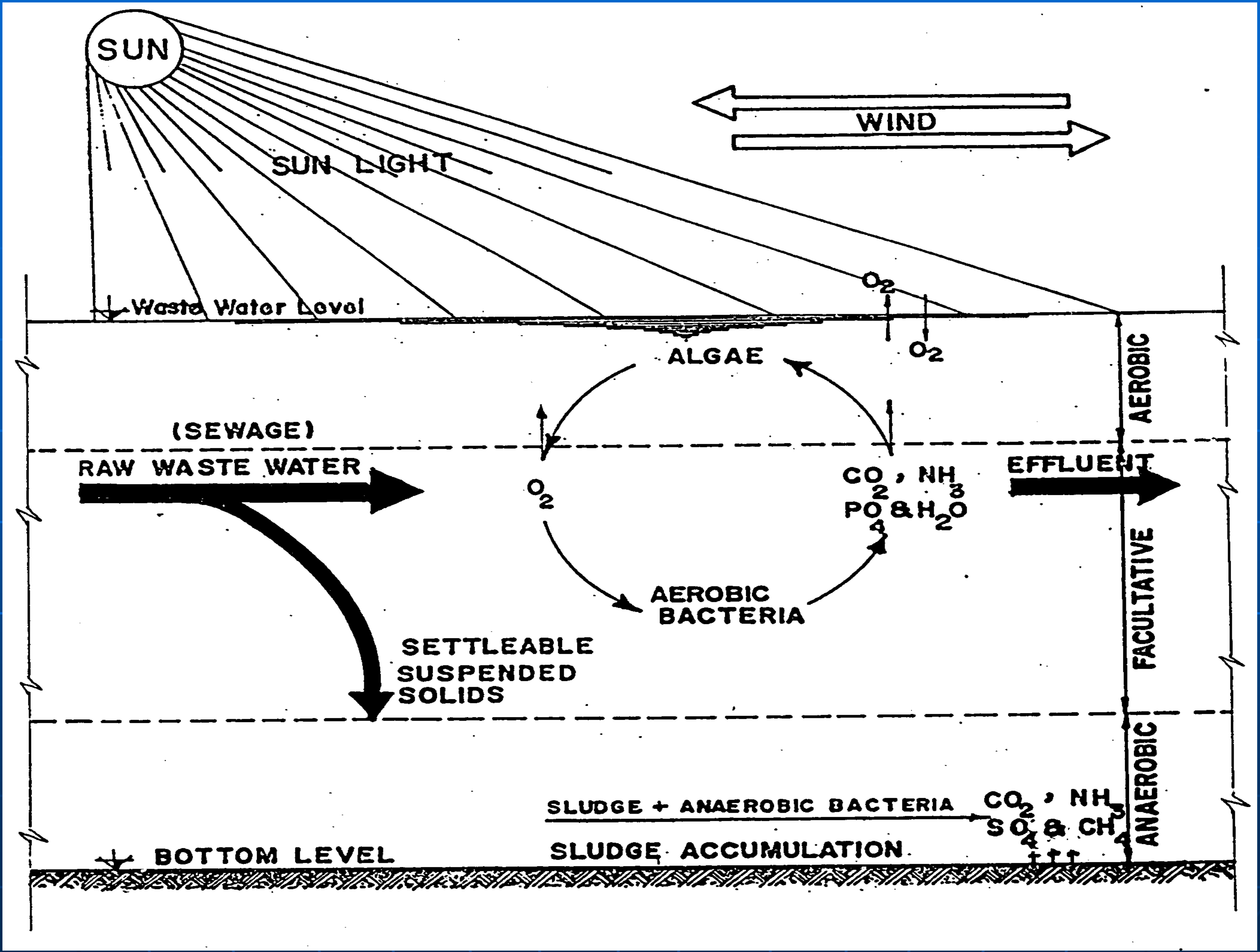
Effluent
Return effluent


Microbial Film
Liquid
Nutrients
Oxygen
Air
Filter medium
End Products
Movement of Wastewater
Anaerobic
Aerobic



SUN
WIND
SUN LIGHT
Waste Water Level
O_2
O_2
ALGAE
AEROBIC
(SEWAGE)
RAW WASTE WATER
O_2
CO_2, NH_3
PO_4 & H_2O
EFFLUENT
FACULTATIVE
AEROBIC
BACTERIA
SETTLEABLE
SUSPENDED
SOLIDS
ANAEROBIC
SLUDGE + ANAEROBIC BACTERIA
CO_2, NH_3
SO_4 & CH_4
BOTTOM LEVEL
SLUDGE ACCUMULATION

LAWS AND REGULATIONS

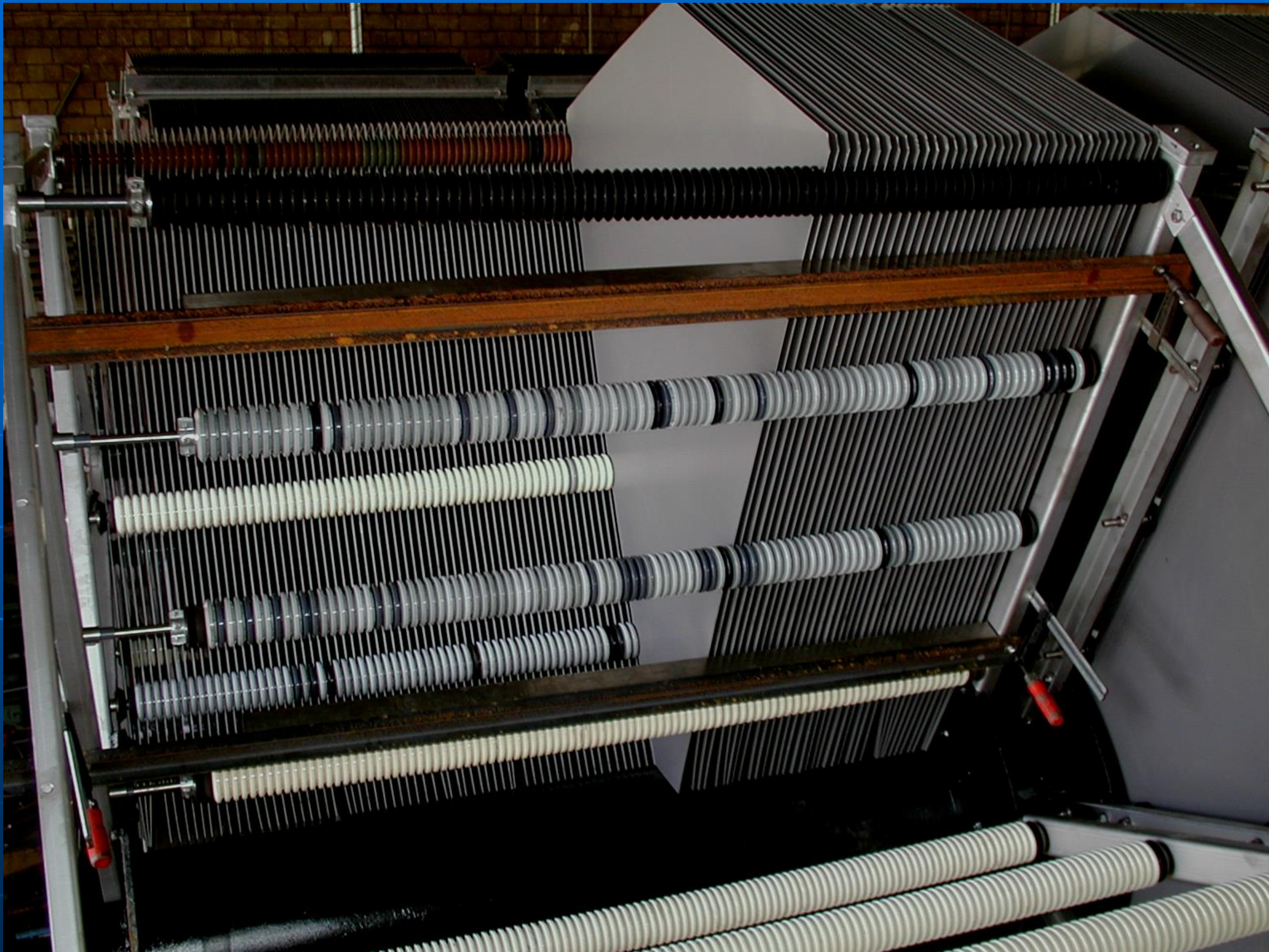
- 4/94
- 48/82: DISCHARGE TO WATER BODIES: DRAIN OR RIVER
- 44/2000, ECP : CONNECT TO MUNICIPALITY
- CODE OF PRACTICES FOR REUSE, SLUDGE HANDLING,…

جدول (٥-٢)
إعادة استخدام مياه الصرف الصحى فى الزراعة
أنواع النباتات والتربة وطرق الرى

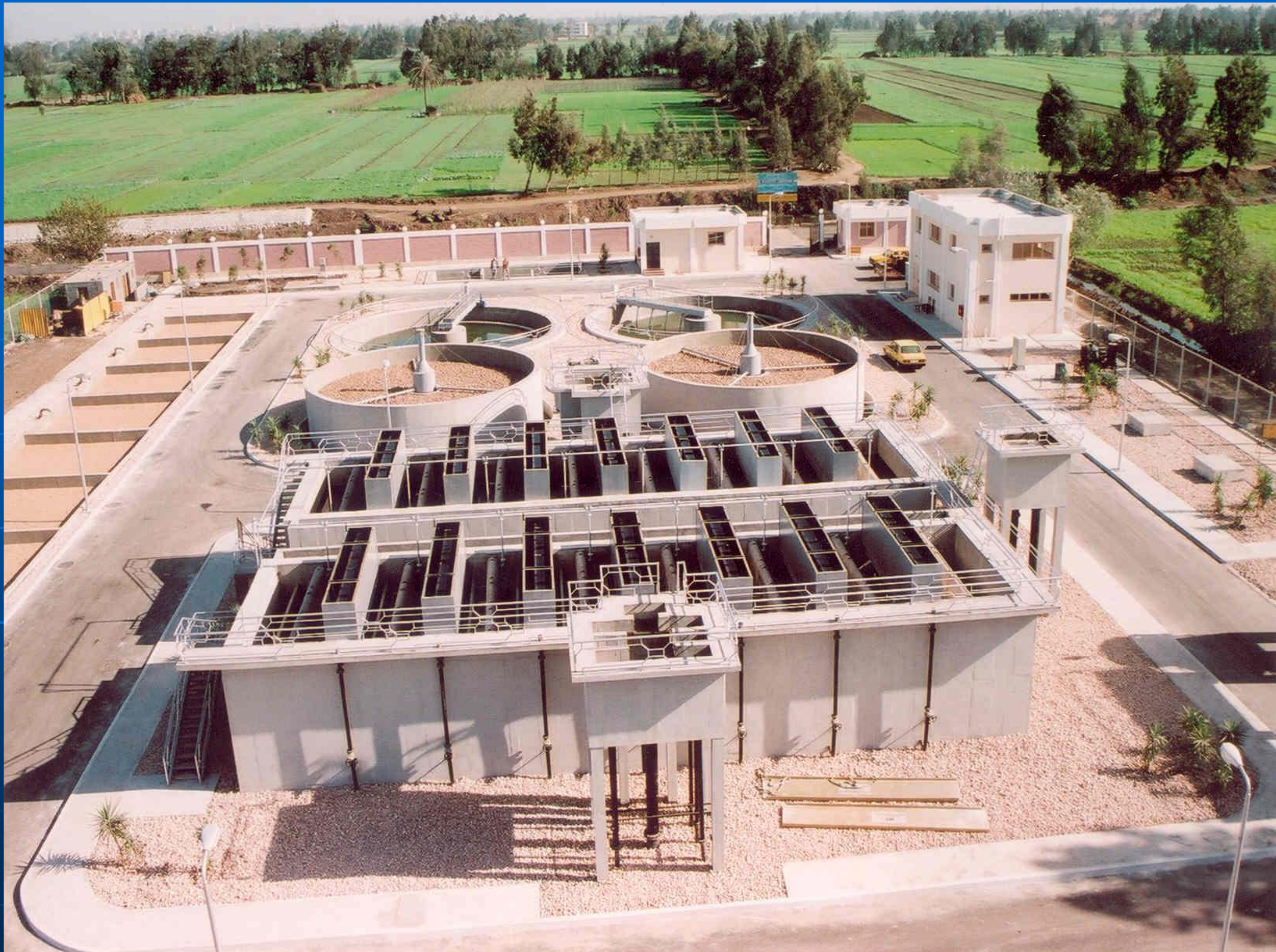
رقم المجموعة	درجة المعالجة	النباتات المسموح بزراعتها	الاحتياطات البيئية والصحية	طرق الرى	نوع التربة
الأولى	مياه صرف صحى	الأشجار الخشبية والنخيل	عمل سياج حول المزارع. عدم التلامس مع المياه مباشرة مع عدم دخول غير العاملين للمزارع. اتخاذ الإجراءات الصحية اللازمة للحماية من الإصابة بالكائنات الممرضة والعلاج.	بالخطوط	خفيفة القوام
الثانية	معالجة ابتدائية	القطن – الكتان – الزهور	مثل المجموعة السابقة	بالخطوط ، النقاطات مع استخدام المرشحات	خفيفة القوام
الثالثة	معالجة ثانوية	محاصيل الأعلاف والحبوب المجففة المحاصيل والفواكه القشرية. الخضراوات التى تطهى. الفواكه المصنعة بالحرارة.	يمكن تربية الماشية غير المدرة للبن أو المنتجة للحوم. يجب طهى الطعام قبل تناوله	بالخطوط – بالتنقيط	خفيفة متوسطة القوام
الرابعة	معالجة متقدمة	النباتات التى تؤكل نيئة. النباتات القشرية. جميع المحاصيل والبساتين. الأعلاف والمراعى الخضراء	لا توجد	جميع الطرق ماعدا الرش	جميع أنواع التربة

RESEARCH ON!!!

- COMPARATIVE STUDY OF LAWS AND REGULATIONS
- ENVIRONMENTAL CONCERNS OF RESIDUAL POLLUTION
- LOW COST TREATMENT OPTIONS
- CARBON CREDIT
- SMELL CONTROL
- IMPROVING WW TECHNOLOGIES
- TRADITIONAL TREATMENTS
- ROLE OF PATHOGENS AND ITS CONTROL
- INDUSTRIAL WW TREATMENTS; CASE STUDY:….
- ANY ????



RBC-Roller prepared for transport, diameter 3,0m



STANDARD BOD TEST

1. Prepare a volume of the waste sample = (Vs) ml.
2. Put it in a vessel of 300 ml capacity.
3. Add fresh water (diluted water) to the waste sample to complete the total volume to 300 ml.
4. Then measure initial DO concentration (DO_I).
5. Put the vessel in incubator at temperature of 20o C for 5 days.
6. Then measure final DO (DO_F).

Where:

- **DO_I (mg/l) is the measured concentration of dissolved oxygen for whole sample after dilution at the initial of the test (at time (t) = zero).**

- **DO_f (mg/l) is the measured concentration of dissolved oxygen for whole sample after dilution at the end of the test (at time (t) = 5 days)**

- **The waste water sample volume = Vs (ml)**

- **The total sample volume after dilution = 300 ml**
- **Decimal Friction (P) = [Vs ml / 300 ml]**

Standard BOD = $BOD_5(20)$ (mg/l) = $(DO_I - DO_f) / P$

PURPOSE OF PRIMARY TREATMENT

<u>Removal of</u>

- Large and big floating particles.
- (40 to 60 %) of SS and (25 to 35 %) of BOD.
- Sand, gravel, silt…..etc.
- Oil and grease.

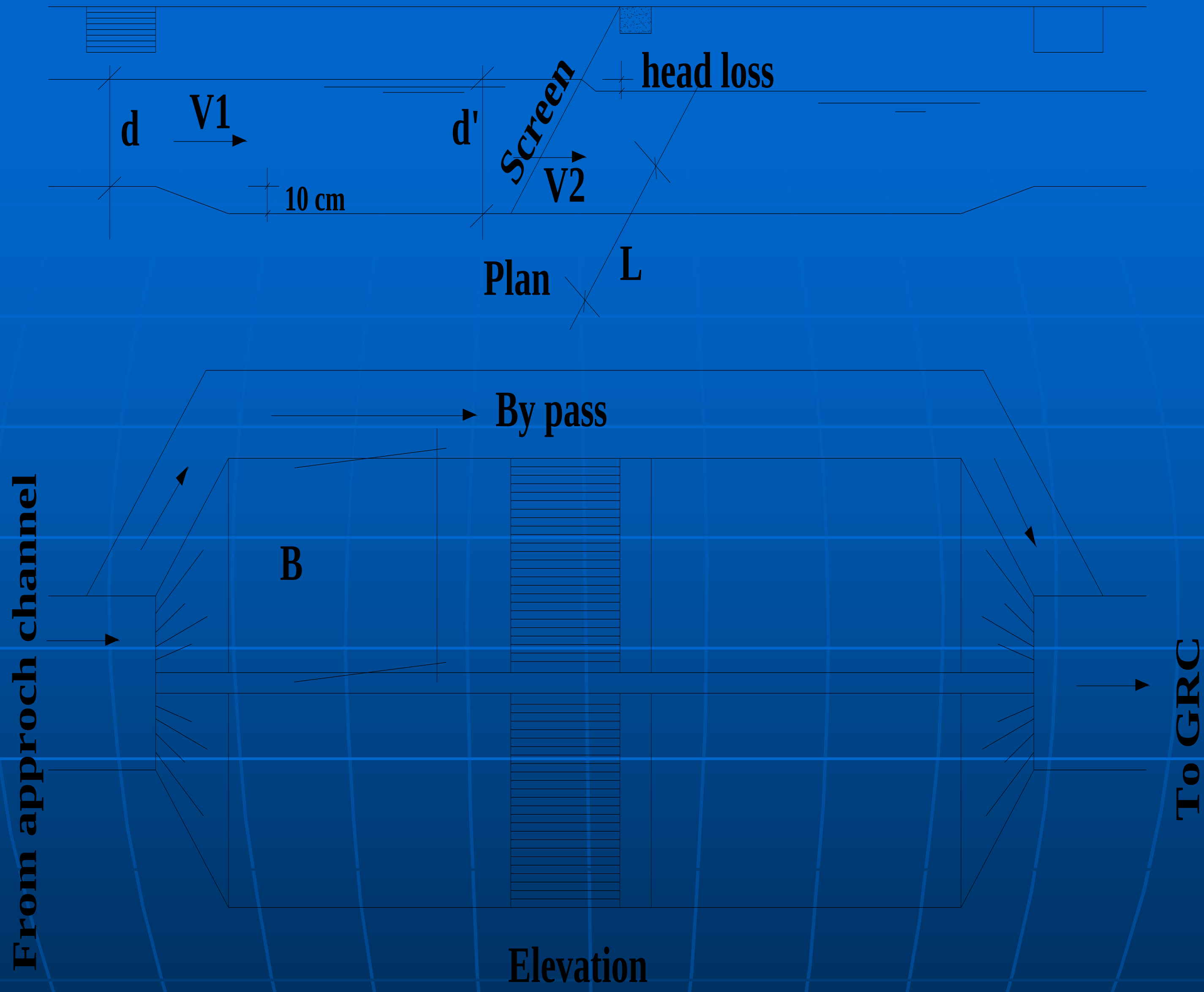
Purpose of
Preliminary Treatment Units

Deceleration Chamber

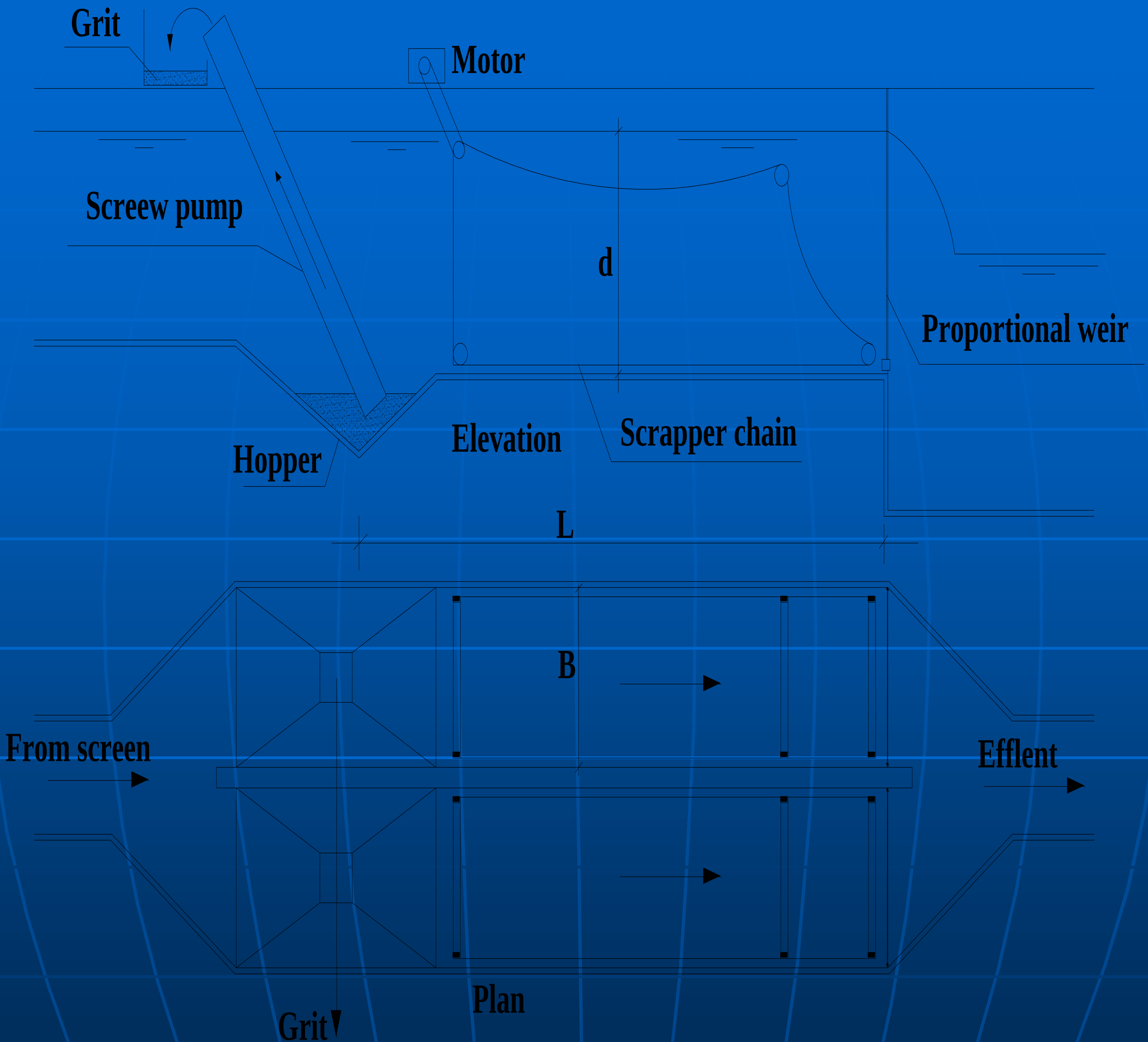
Decreasing velocity of flow before the screen to prevent the escaping of particles from the screening opens.

Approach channel

Transmission of waste water from deceleration chamber to screen at aerated condition and with suitable velocity that helps in avoid anaerobic reaction between units.



Coarse Screen



Grit Removal Chamber

Screen

It's used to remove big and strange impurities from waste water, such as (paper – wood – plastics - …..), to protect the down stream equipments against damage.

Skimming Tank

Removal of oil, grease and most of float matters.

Grit removal chamber (GRC)

Removal of particles has size more than 0.2 mm, such as (sand, gravel, grit …..etc), to:

Protect down stream mechanical equipments.

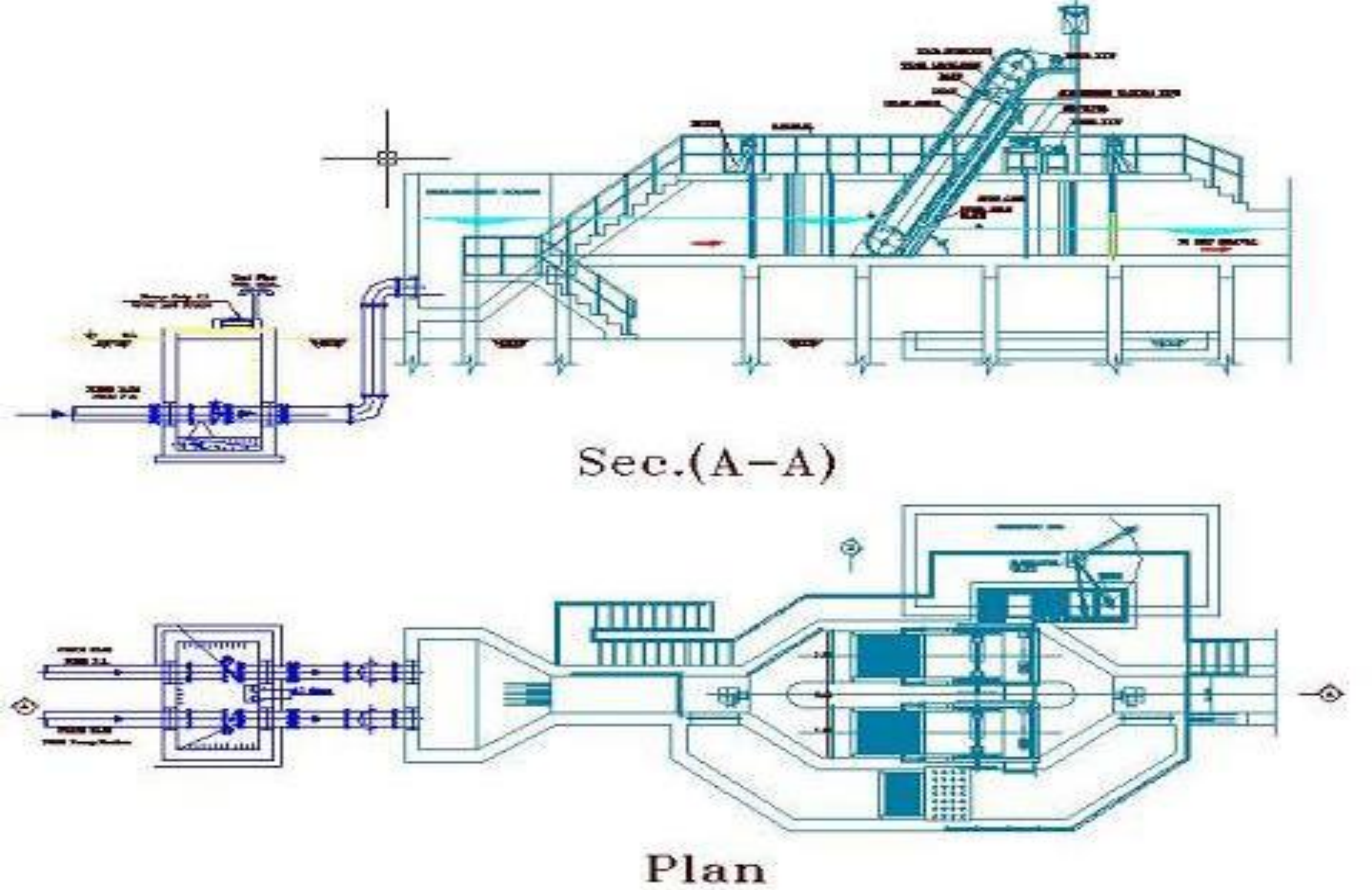
Reduce conduit clogging.

Prevent interface with other treatment.

Improve sludge quality.

Screw Pump

Preliminary Treatment



DECCELERATION CHAMBER AND SCREEN

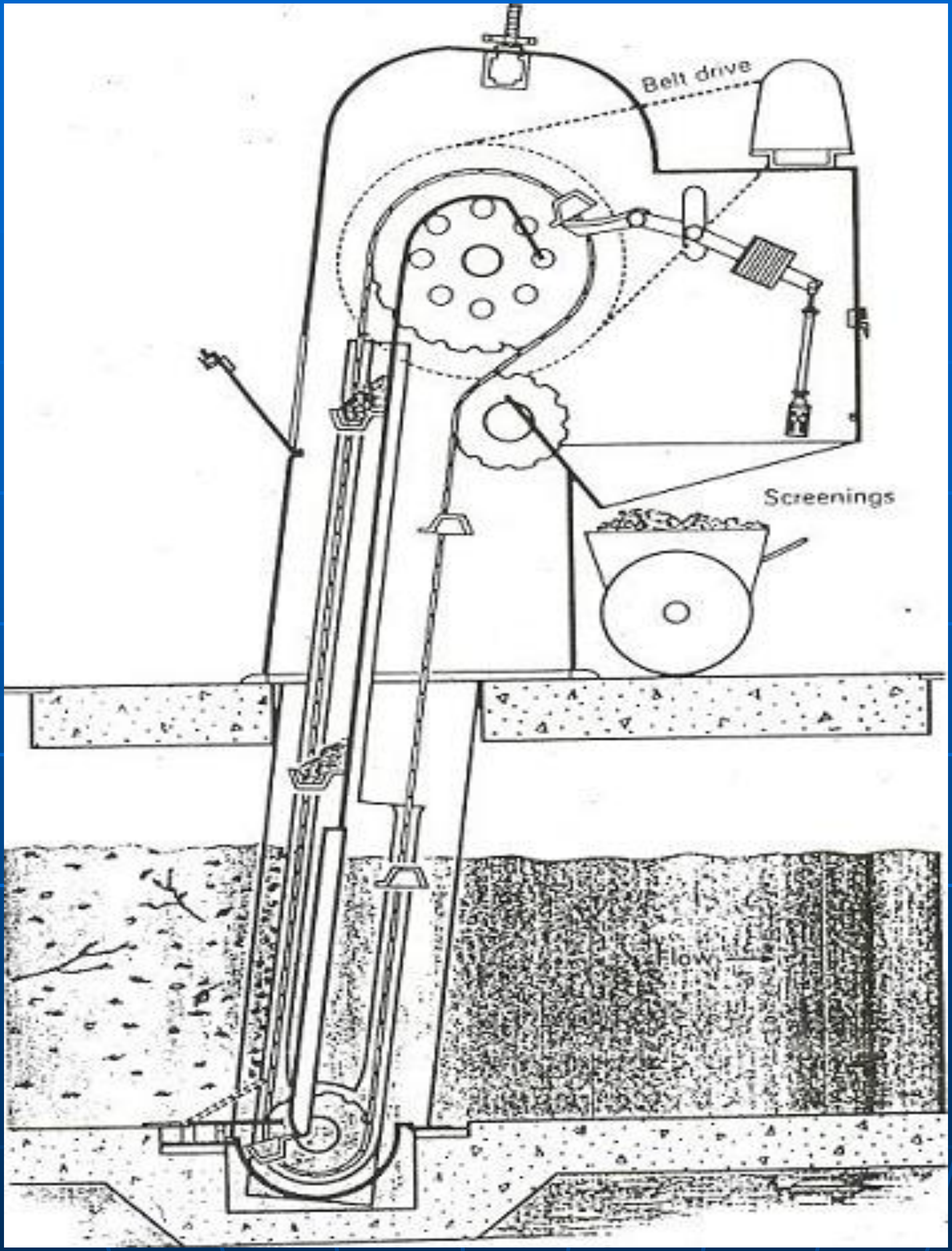
Force mains and deceleration chamber

Screens

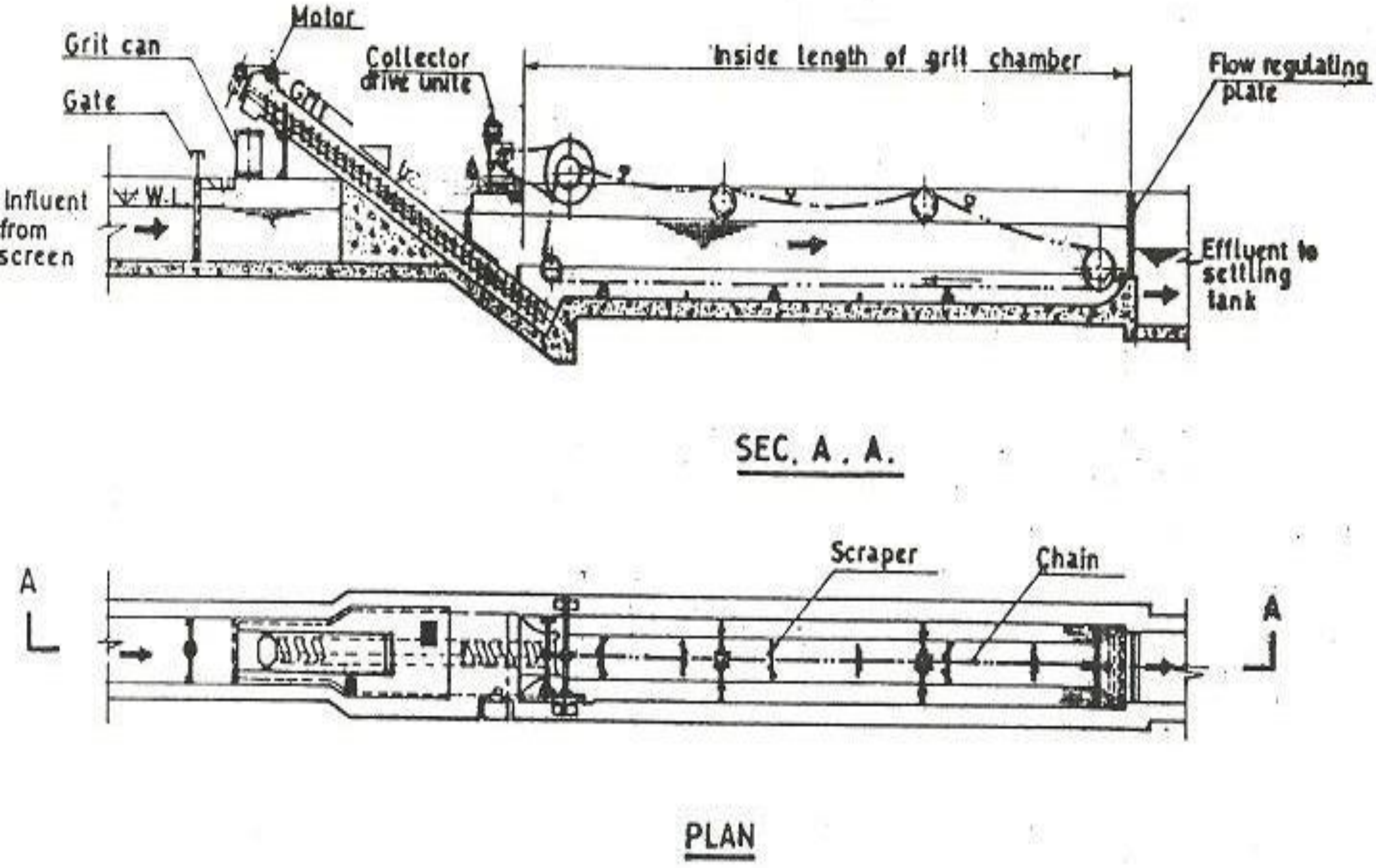
Mechanical Screens

Screening

Screening Disposal



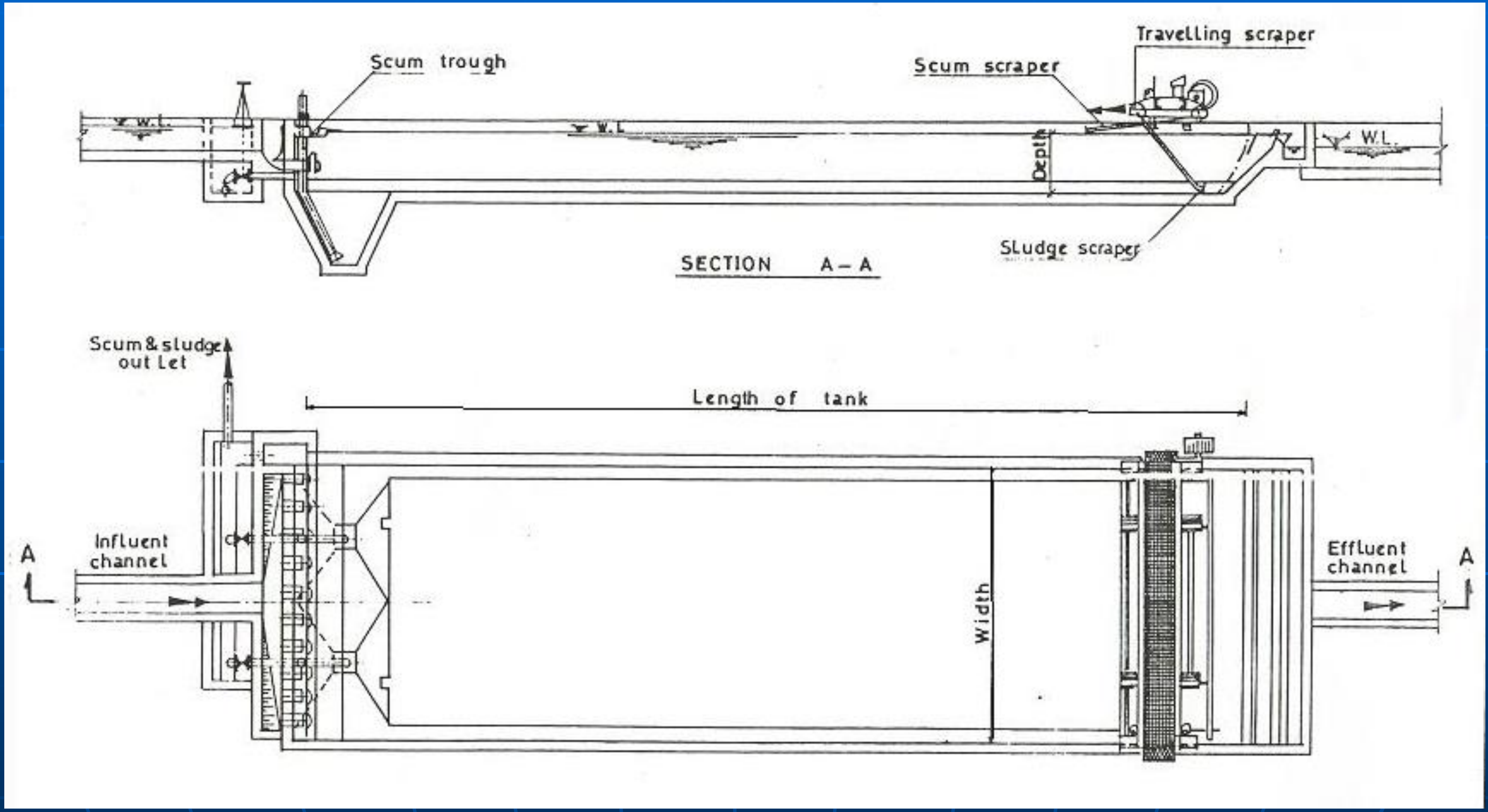
MECHANICAL SCREEN



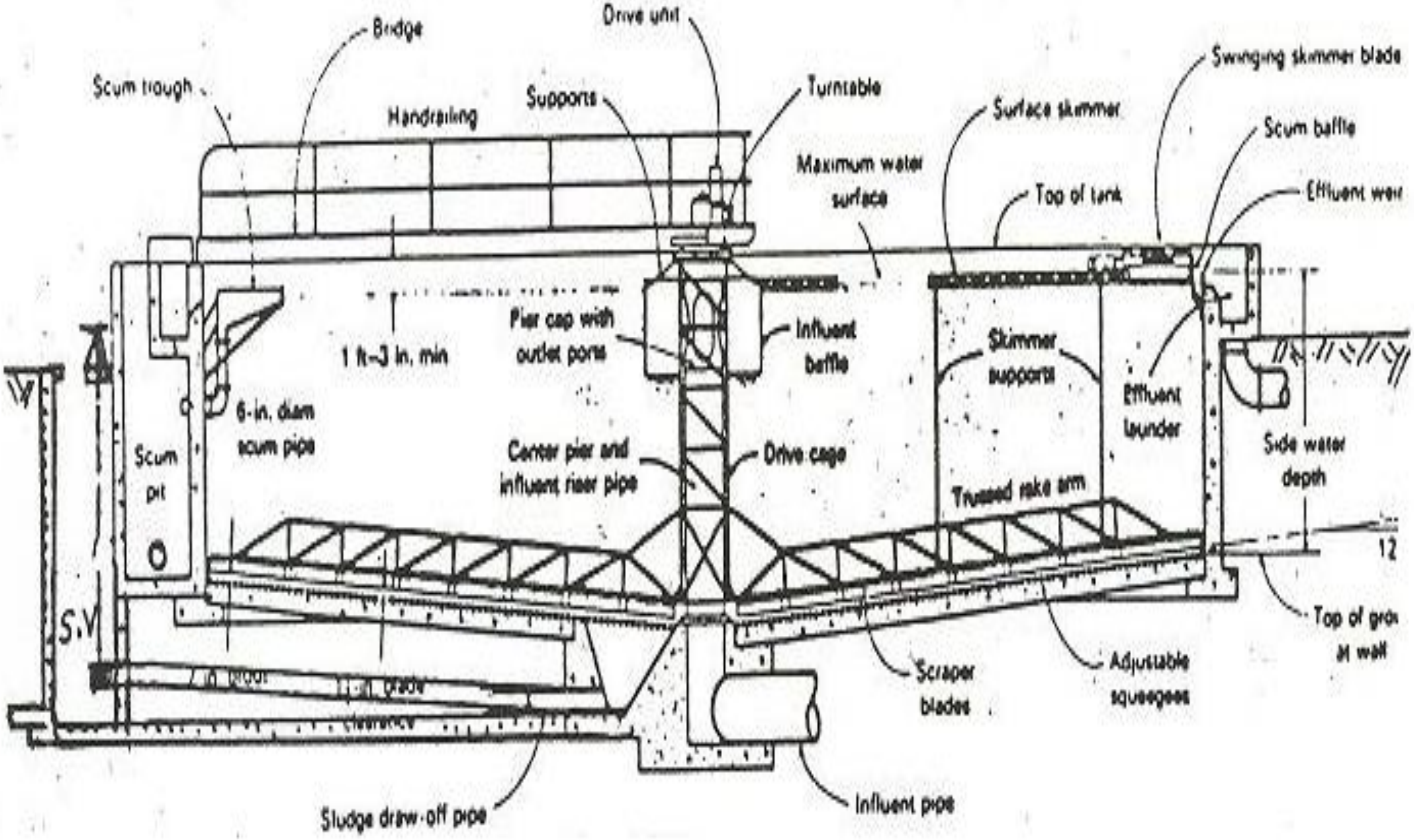
GRIT REMOVAL CHAMBER

Purpose of Primary Sedimentation Tank

- Removal of 40 to 60 % of SS
- Removal of 25 to 35 % of BOD
- Removal of Oil and Grease



PRIMARY SEDIMENTATION TANK (Rectangular)



PRIMARY SEDIMENTATION TANK (Circular)

Wastewater Treatment Calculation

Q_{av} = Pop * (av. SF)
Peak factor (PF) = $(18 + (Pop)^{0.5})/(4 + (Pop)^{0.5})$, (Pop in thousand).
Minimum factor (MF) = $0.2 * (Pop)^{1/6}$, (Pop in thousand).
SF = sewage flow = 0.8 to 0.9 from average water consumption.

Design Criteria of Preliminary Treatment Units

1.Deceleration chamber

$Q_d = Q_{max} = PF * Q_{av}$
RT = 1 to 3 min.
$n \geq 1$ (take 1 at first)
Vol = n LBd
SA = n LB
XA = n Bd
L = (2 to 4) * B
d = 1 – 2 m
Velocity = 0.6 to 1.5 m/s

2. Approach channel

Open channel flow (manning)

Q_{av} = Pop * (av. SF)

$Q_d = Q_{max}$ = PF * Q_{av}

$n \geq 1$ (take 1 at first)

XA = n Bd

V = 0.6 to 1.5 m/s

للحصول على قيمة مبدئية للعرض نفترض السرعة القصوى عند التصرف الاقصى ١.٥ م/ث

Assume V_{max} = 1.5 m/s

B = 2dmax (best section)

Get (B,d_{max}) at Q_{max}

للحصول على أقل ميل لقاعها نفترض السرعة عند التصرف الادنى ٠.٦ م/ث ونعوض فى معادلة الاستمرارية نحصل على مساحة المقطع المطلوبة.

من قيمة العرض المحددة مسبقا نعوض فى مساحة المقطع نحصل غلى العمق الادنى.

ونعوض والسرعة ٠.٦ م/ث نحصل على اقل ميل لها. 60 = (1/n)نعوض فىماننج بفرض

Assume V_{min} = 0.6 m/s

$Q_{min} = V_{min} * A_{min} = 0.6 * Bd_{min}$

Then get (d_{min})

From manning get S_{min}

$V_{min} = (1/n)\, R_{(2/3)} S_{min(1/2)}$

R = Area / wetted perimeter = $Bd_{min} / (B+2d_{min})$

Assume (1/n) = 60

After that you can get d_{max} at V_{max} from manning also. Fix the slope.

3. Screen

Types of screen

Coarse screen. Space between bars (5 – 10 cm)
Medium screen. Space between bars (2 – 5 cm)
Fine screen. width of opening = (0.08 – 0.24 cm), circular plate), length of opening = (0.64 – 5.1 cm)

Reasons for using fine screen

When the PST is over loaded.
With certain types of industrial waste water.
When the PST isn't used.
When there is no secondary treatment.

Disposal of the screening (screened material)

Sanitary land fills.
Incineration.
Grinding and return to the head works of the plant.

Design of coarse screen

Perpendicular velocity on screen projection ≤ 0.15 m/sec
The velocity before the bar screen (V_1) ≥ 0.6 m/s
The velocity between the bar screen (V_2) ≤ 1.5 m/sec
Where $V_1 = Q_{max} / (nBd')$
Where $V_2 = Q_{max} / (nNSd')$
h loss through the screen $= 1.4 (V_2^2 – V_1^2)/2g$
h loss ≤ 10 cm (at the beginning of working)
h loss ≤ 30 cm (before cleaning)
$Q_d = Q_{max} = PF * Q_{av}$
$Q_{av} = Pop * (av.\ SF)$
$n \geq 2$ (take 2 at first) + by pass
A_{net} in inclined plane = 2 to 3 (approach channel area)
A_{net} in inclined plane = n (N S L) m^2
Where,
N = no. of spacing in one screen = no of bars – 1
S = spacing between bars = according to screen type (coarse – medium)
$L = d' / \sin\theta$
d' = d (of approach channel + 5 to 10cm)
θ = 30 – 60o (manual screen)
Screen bar diameter (Ø) = 1.3 – 1.6 – 1.9 cm
The screen width = B = (N + 1)*Ø + N * S

4. Grit Removal Chamber

Methods used to control velocity of the chamber

- Use parabola cross section area.
- Use rectangular cross section with proportional weir at the end of the chamber.

Design data of rectangular type-

$Q_d = Q_{max.}$

Horizontal velocity = 0.25 to 0.35 m/s

R T = 45 to 90 sec (take 60)

SLR = 1000 to 2500 $m^3/m^2/d$ (take 1200)

$n \geq 2$

L = 15 to 20 m $B \leq d$

Diameter of sand particals mm	Surface loading rate 1000 m3/m2/day
0.79	3.50
0.36	2.30
0.28	1.70
0.17	1.10

. No. of chambers (n) $\geqslant$ 2

. To fix the velocity in the grit removal chamber, it must feet it by the proportional weir at outlet of it (for rectangular section chamber).

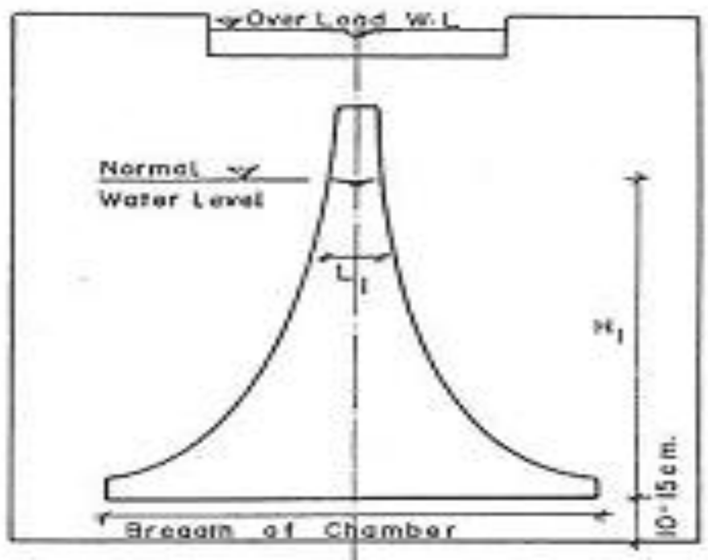
. Proportional weir equation:

$$Q = 4\ L\ H^{3/2}$$

$$,\ L_1\ H_1^{0.5} = L_2\ H_2^{0.5} = \ldots\ldots = \text{constant}$$

where L = width of weir

H = height of water at width L



Design Criteria of Primary Sedimentation Tank

Purpose

Removal of 40 to 60 % of SS
Removal of 25 to 35 % of BOD

Design data

Retention time = T = 1.5 to 2.5 hr (in case of flowed with secondary treatment), = 2 to 4 hr (in case of no secondary treatment appearance)
SLR = 30 to 45 m3/m2/d
HLOW = 150 to 600 m3/m2/d
d = 3 to 5 m
B = 2 to 3 d
L = 4 to 5 B
L $\leq$ 50 m
Ø $\leq$ 35 m

Factors affecting efficiency of sedimentation process

1. Shape and size of solids.
2. Specific gravity of suspended solids.
3. Temperature.
4. Velocity of flow.
5. Retention time.
6. Efficiency of pervious treatment.
7. Sludge removal method.
8. Scum removal method.
9. Surface loading rate.
10. Viscosity of sewage.
11. Specific gravity of sewage.
12. Inlet and outlet arrangement.

Example

Design the following treatment units for a sewage treatment plant:

- GRC (conventional with proportional weir).
- PST (rectangular)

Given the following data:

- Average sewage flow is 17,280 m^3/d
- SLR of GRC = 1200 $m^3/m^2/d$
- SLR of PST = 30 $m^3/m^2/d$

Solution

Assume PF = 1.5 So, Qmax = 1.5 * 17,280 = 25,920 m^3/d

Design of GRC

RT = 45 – 90 sec = 60 sec = Vol./Q

SLR = 1000 – 2500 = 1200 $m^3/m^2/d$ = 50 $m^3/m^2/hr$ = Q/SA

V = 0.25 – 0.35 = 0.3 m/s = 1080 m/hr = Q/XA

RT*V = L = 18 m

nBL = 36 B = SA = 25,920/1200 = 21.6 m

B = 0.6 m

Vol. = 18 m^3

d = 0.85 m

Design of PST

Q_d = 1080 m^3/hr

Assume R.T = ^{3} hrs

Volume = 3 * 1080 = 3240 m^3 = nLBd

Assume SLR = 30 m3/m2/d = 1.25 m^3/m^2/hr

SA = 1080/1.25 = 864 m2 = n LB

d = 3240/864 = 3.75 m

Assume B = 2d = 7.5 m

Assume L = 4B = 30 m

n = 4 tanks

The important checks

RT = 3375 / 1080 = 3.125 hr

SLR = (1080 / 900) *24 = 28.8 m^3/m^2/d (safe but waste)

V_f = (1080/ 112.5*60) = 0.16 m/min

HLOW = (1080/30) * 24 = 864 m^3/m/d

Take L_w = 55 m, HLOW = 471 m^3/m/d (ok)

BIOLOGICAL TREATMENT

PROBLEM

The effluent of primary treatment still contains relatively large quantities of organic matter in suspension or in solution

TARGET

Transformation of organic matter into stable non offensive compounds

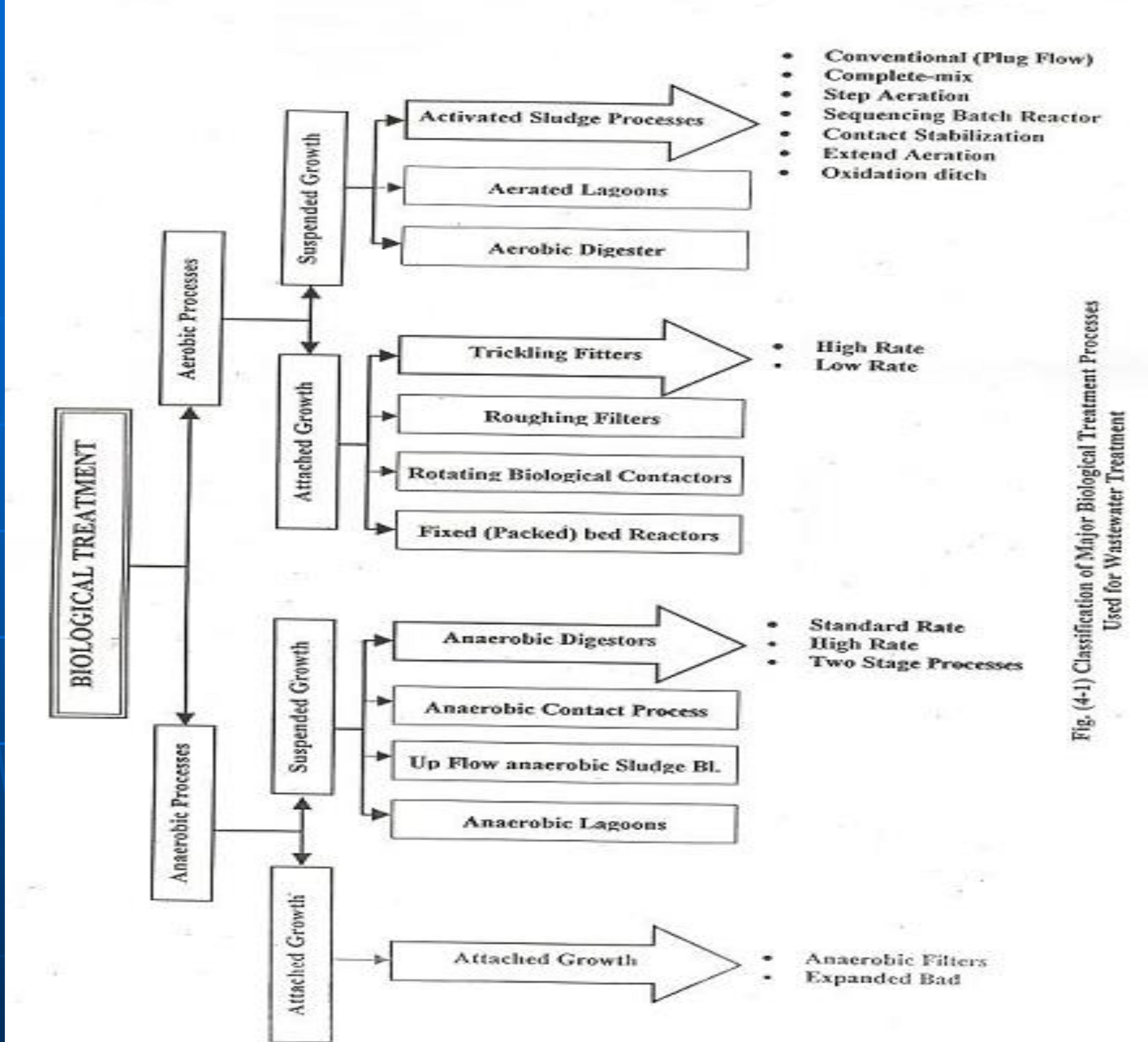


Fig. (4-1) Classification of Major Biological Treatment Processes Used for Wastewater Treatment

TYPES OF BIOLOGICAL TREATMENT

AERODIC PROCESS

ATTACHED GROWTH

TRICKLING FILTERS

Trickling-filter systems are commonly used for secondary treatment of municipal wastewater

Primary effluent is sprayed on bed of crushed rock or gravel coated with biological films

Removal of the organic matter is the result of an adsorption process which occurs at the surface of biological slimes covering the filter media

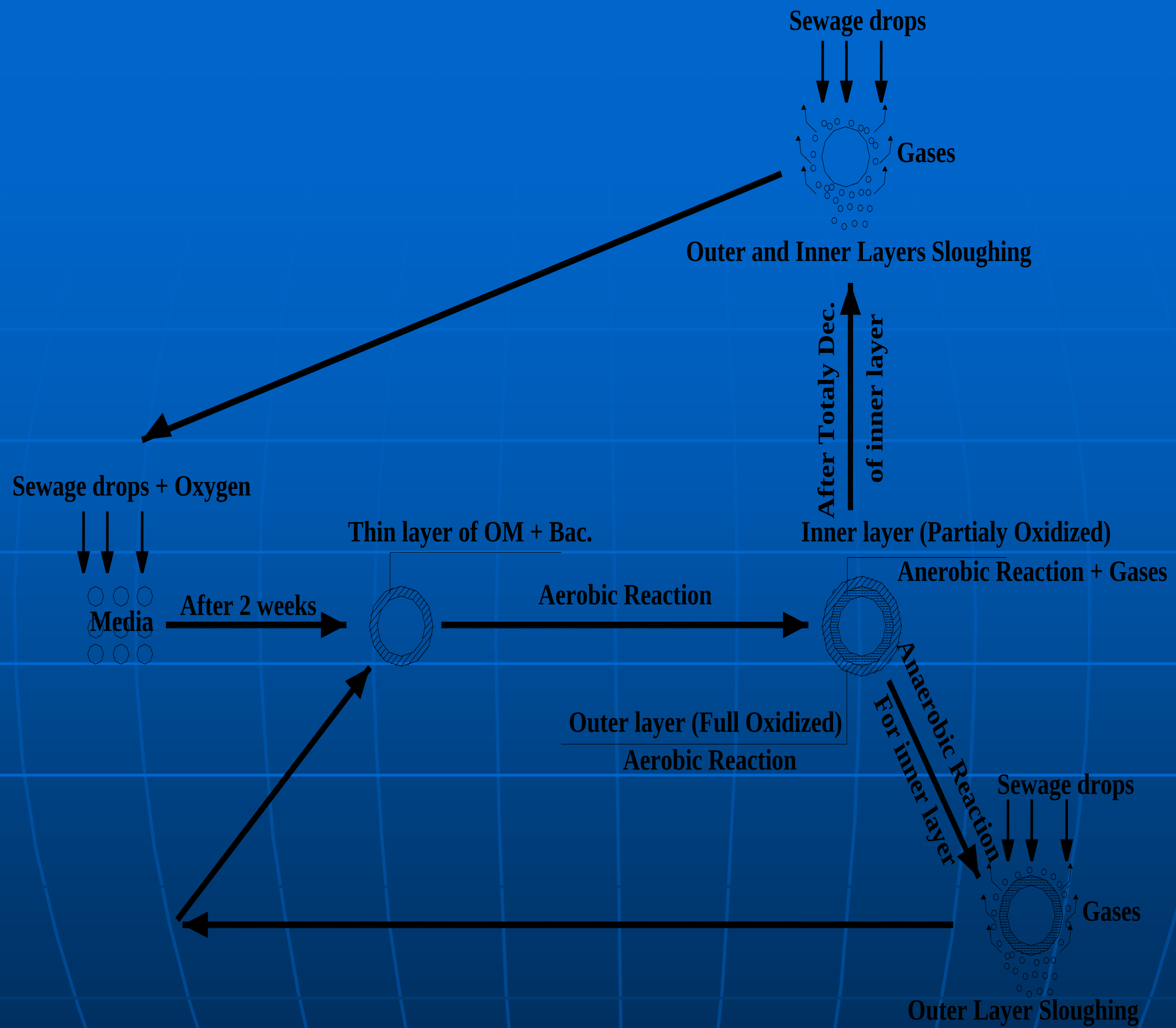


Figure (1/2). Mechansim of BOD Removal in Attached Growth Biological Treatment

TRICKLING FILTERS TYPES

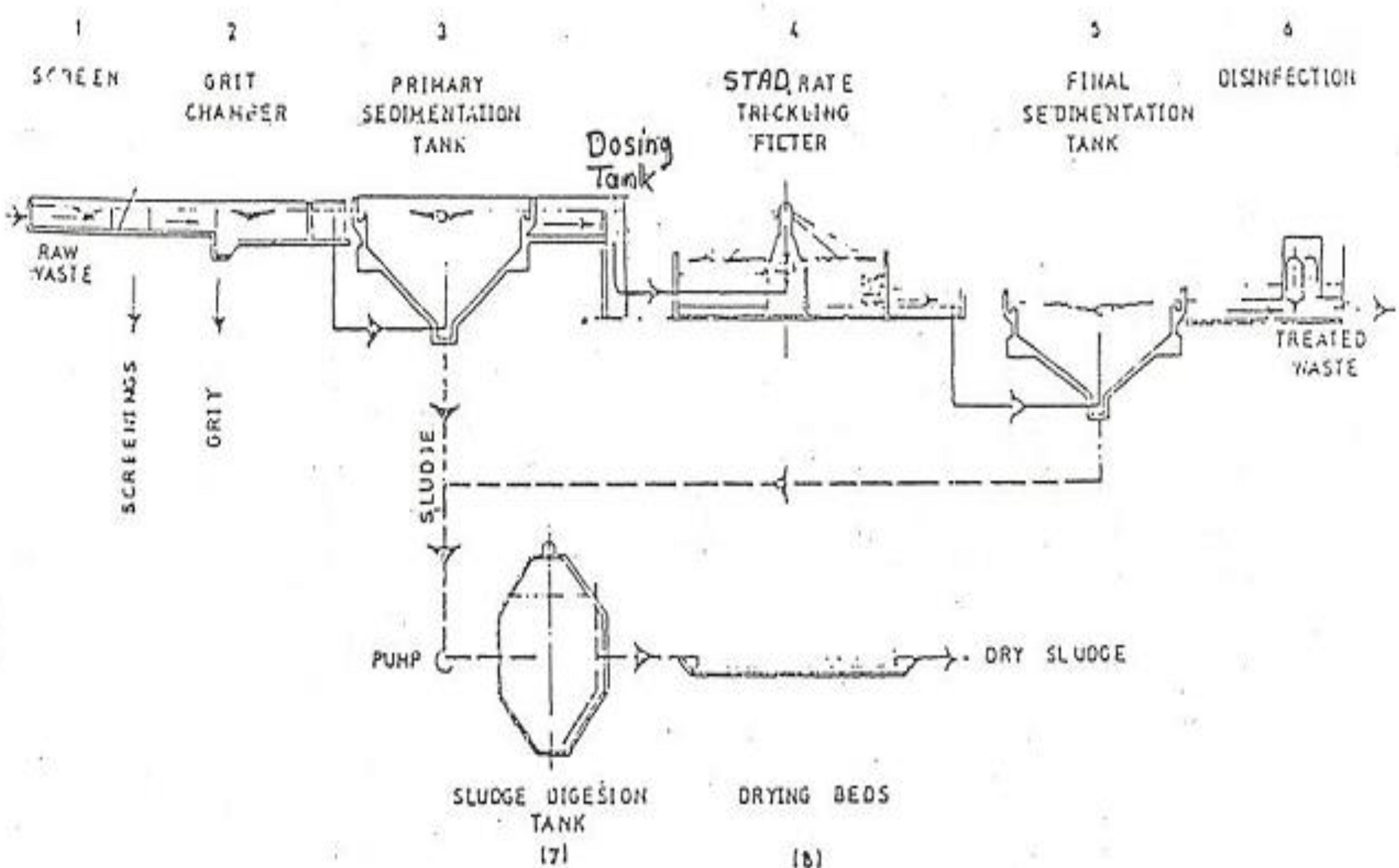
High Rate Trickling Filter

Sewage flow is continuously applied and uniformly distributed in the form of a spray over the filter media

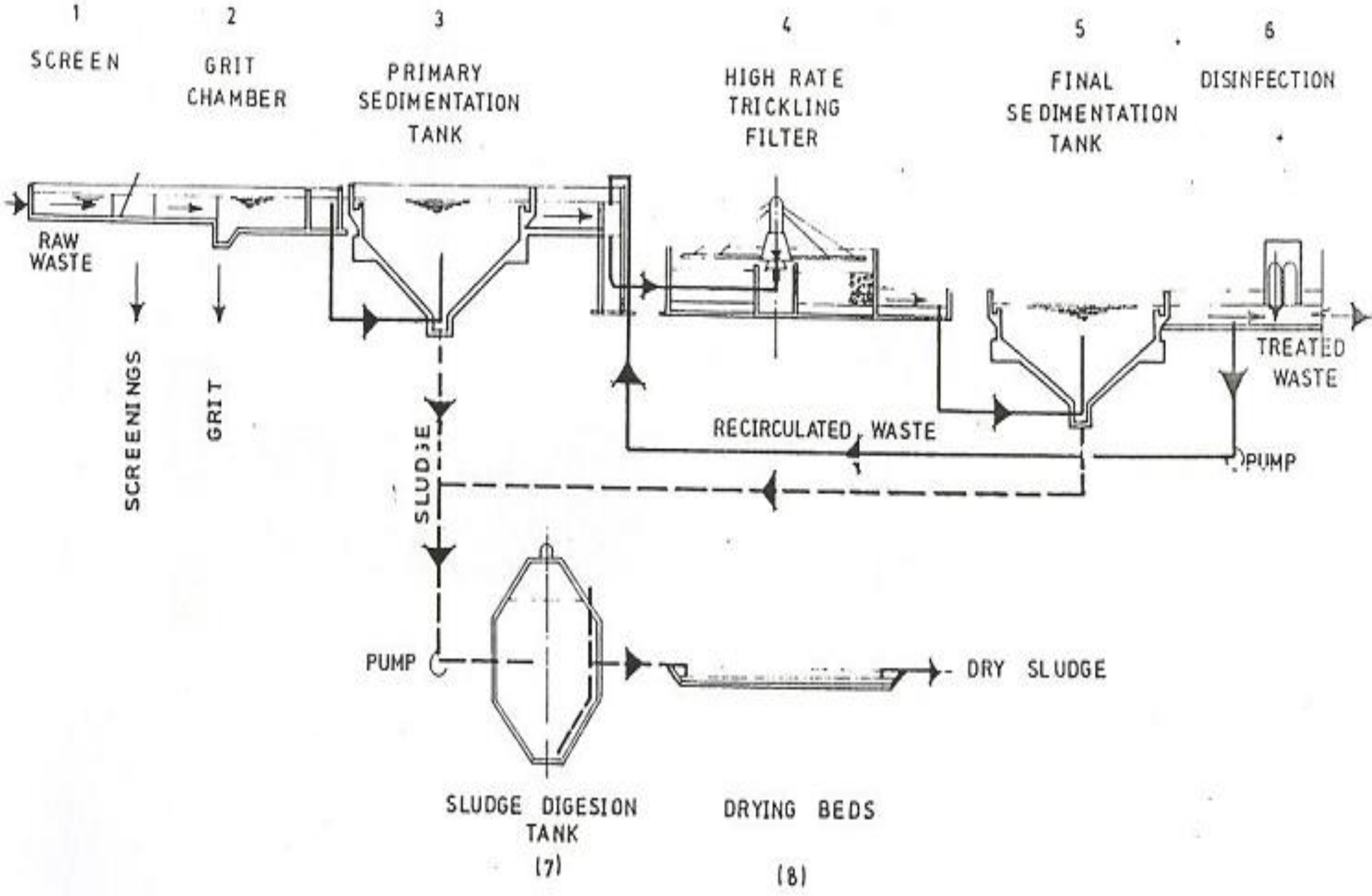
Part of effluent from the final settling tank is returned to the influent of the filter (Circulation)

Standard Rate Trickling Filter

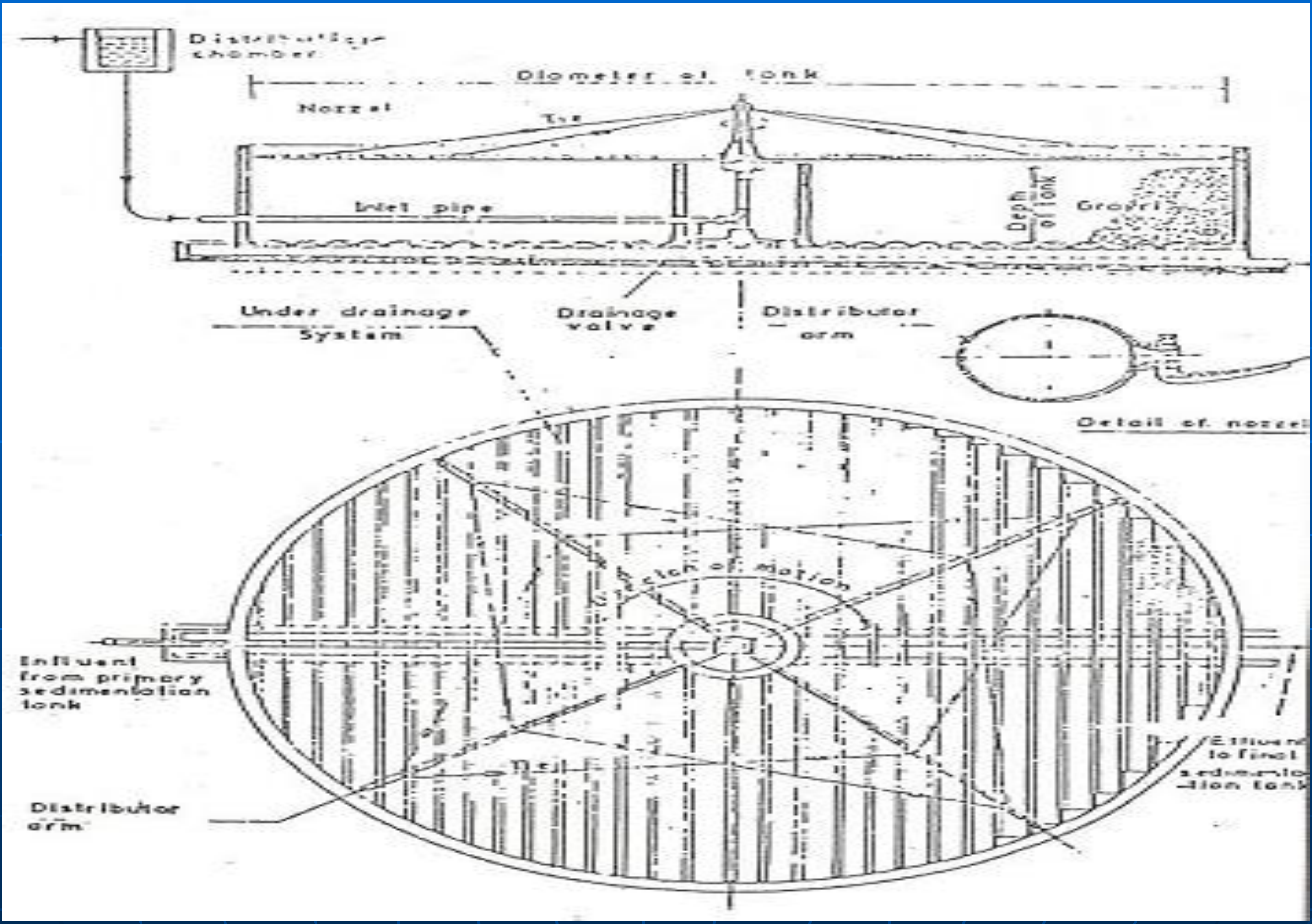
Sewage flow is intermittently applied but uniformly distributed in the form of a spray over the filter media



Flow Diagram in Standard Rate Trickling Filter WWTP



Flow Diagram in High Rate Trickling Filter WWTP



High Rate Trickling Filter

Trickling Filter

Trickling Filter

Comparison of Low Rate and High Rate Trickling Filter

Character		Low rate filter	High rate filter
Hydraulic loading	$m^3/m^2/d$	1 - 3	10 - 20
Organic loading	kg BOD/1000 m^3/d	70 - 200	450 - 2000
Depth	ms	1.8 - 3	0.9 - 2.5*
Recirculation		None	0.75:1 to 4.5:1
Rock volume		5-10 times	1
Power requirement		None	2-10 HP/1000 m^3
Filter flies		Many	Few
Sloughing.		Intermittent	Continuous
Operation.		Simple	Some skill
Dosing intervals		Intermittent (not more than 5 min.)	Continuous
Effluent		Fully nitrified	Nitrification at low loading

Effect of Recirculation in High Rate Trickling Filter

1 - Part of organic matter in sewage feed is brought into contact with growth on filter media more than once.

2 - Recirculated liquid contains active micro-organisms not found in such quantity in raw sewage, thus providing seed continuously.

3 - Diurnal organic load distributed more uniformly.

4 - When plant flow is low (night) operation is not shut-off.

5 - Maintenance of flow during low periods precludes long detention periods which may result in Septicity. Stale sewage is freshened.

6 - Increasing flow improves uniformity of distribution, increases sloughing, reduces clogging tendencies.

7 - Higher velocities and continual sloughing makes conditions less favorable for growth of filter flies.

8 - Continual seeding with active micro-organisms and enzymes stimulates hydrolysis and oxidation and increases rate of biochemical stabilization.

9 - Provides for more flexibility of operation.

Disadvantages of Recirculation in High Rate Trickling Filter

1 - Increased operating costs because of pumping.

2 - Larger settling tanks in some designs.

3 - Temperatures reduced as a result of number of passes of liquid.

In cold weather this results in decreased biochemical activity.

4 - Amount of sludge solids to digesters may be increased.

Design of Trickling Filters

BOD_1 (Raw water)

BOD_2 (Primary treated)

BOD_3 (Secondary treated)

% Primary treatment efficiency (PTE) = $[(BOD_1 - BOD_2)/ BOD_1] * 100$

% Combined efficiency (CE) = $[(BOD_2 – BOD_3)/ BOD_2] * 100$

% Overall efficiency (OE) = $[(BOD_1 – BOD_3)/ BOD_1] * 100$

Where:

- $\% \; CE = [1/(1+0.0085(2.72L_o/F)^{0.5})]*100$
- F = Recirculation Factor = $[(1+R)/(1+0.1R)^2]$
- Allowable Hydraulic Load (h_o) = $(Q_d + RQ_d)/(SA)$
- Allowable Organic Load (L_o)
- Total Organic Load (TOL) = $[Q_d * B_2 + RQ_d * B_3]$ gm BOD/d
- Volume = (TOL/L_o) m^3
- Surface Area = (Circular in shape), $Ø \leq 35$ m
- For all units $n \geq 2$ units

Allowable BOD Loading on High Rate and Low Rate Filter in Kg/1000 m^3 of Filter Media per Day

BOD of Raw Sewage Mg/l	Allowable BOD Loading Kg/1000^3/d.	
	Low-Rate Filter	High-Rate Filter
Less than 100	70	450
100 - 150	80	850
150 - 300	90	1100
300 - 450	120	1400
450 - 500	150	1700
600	200	2000

Recirculation Ratios in High Rate Filters

BOD of Raw Sewage (mg/l)	Recirculation Ratio (Qr)
< 150	0.75
150 - 300	1.50
300 - 450	2.25
450 - 600	3.00
600 - 750	3.75
750 - 900	4.50

AEROBIC PROCESS

SUSPENDED GROWTH

ACTIVATED SLUDGE PROCESS

Activated sludge process is advantaged with high removal percentage of organic and suspended matters as they can easily reach more than 90%

The conventional activated sludge process is suitable for large cities because of the much smaller area required by the plant in comparison with all other biological processes

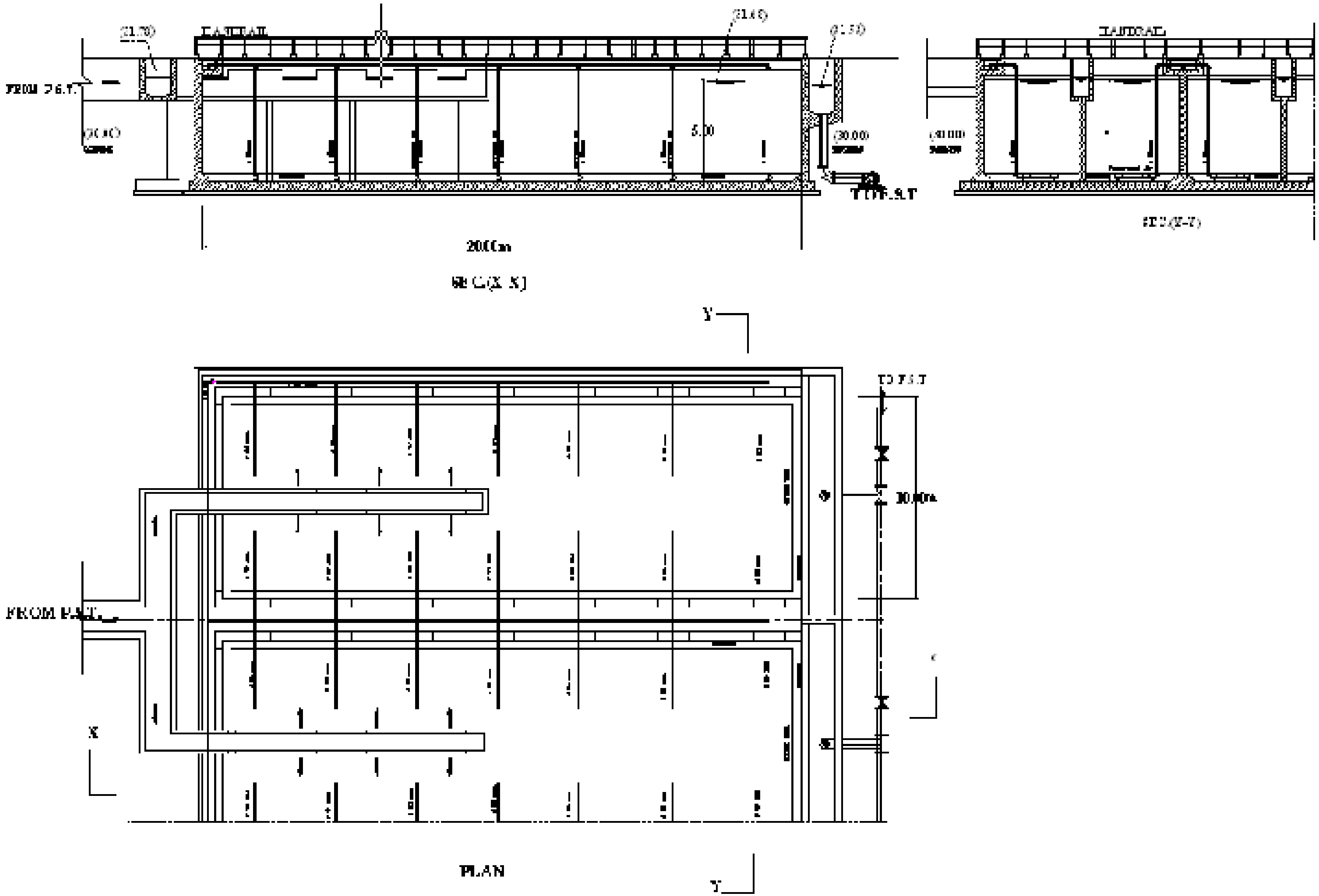
Many modifications of the conventional process for use in small towns such as extended aeration process has been developed . In this process, the primary sedimentation tanks and digestion tanks are eliminated for simplicity of operation and maintenance

Conventional activated sludge process

The effluent from the PST is aerated for a few hours in an aeration tank

During this period, flocculation, adsorption and various oxidation reactions take place

The biological process is accelerated by addition of substantial volume of settled sludge from FST that composes microorganisms in an active state of growth and acclimated of the waste being treated



Aeration tank

Aeration tank

Compressed Air Pipe

Design of Aeration Tank

Retention Time = 6 – 8 hrs

Organic Load not exceed 450 kg BOD/1000 m^3 of volume

BOD loading factor = 0.2 – 0.5 kg BOD/kg of VSS

Air requirements = 70 – 80 m^3 of air per day per kg of BOD removed

Depth = 3 – 5 m

Width/Depth = 1.5 – 2.0

Volume (m^3) = (Q_{max})*(BOD)/(S)*(MLSS)

Q_{max} (m^3/day)

BOD = BOD_5(20) mg/l

S = Sludge loading factor = 0.2 – 0.5 with an average value 0.35 is common

MLSS = concentration of mixed liquor SS mg/l (1200 – 3000 mg/l)

Sludge Recirculation Ratio = r = (100)*(SVI)*(%MLSS)/100 – (SVI)*(%MLSS)

SVI = Sludge Volume Index

%MLSS = MLSS*0.0001

Methods of Aeration in Aeration Basin

1. Direct aeration. (Diffused air – Pure oxygen)

2. Indirect aeration (Mechanical – Hydraulic)

2.1 Hydraulic (Fountain – Falls – Steps – Spraying – Venture)

2.2 Mechanical (Surface aerator – Inclined aerator – Jet aerator – Double)

Advantage of Activated Sludge Process

1. Complete freedom from odors and files.

2. Minimum area of land.

3. Comparative low initial cost for large projects giving the same results.

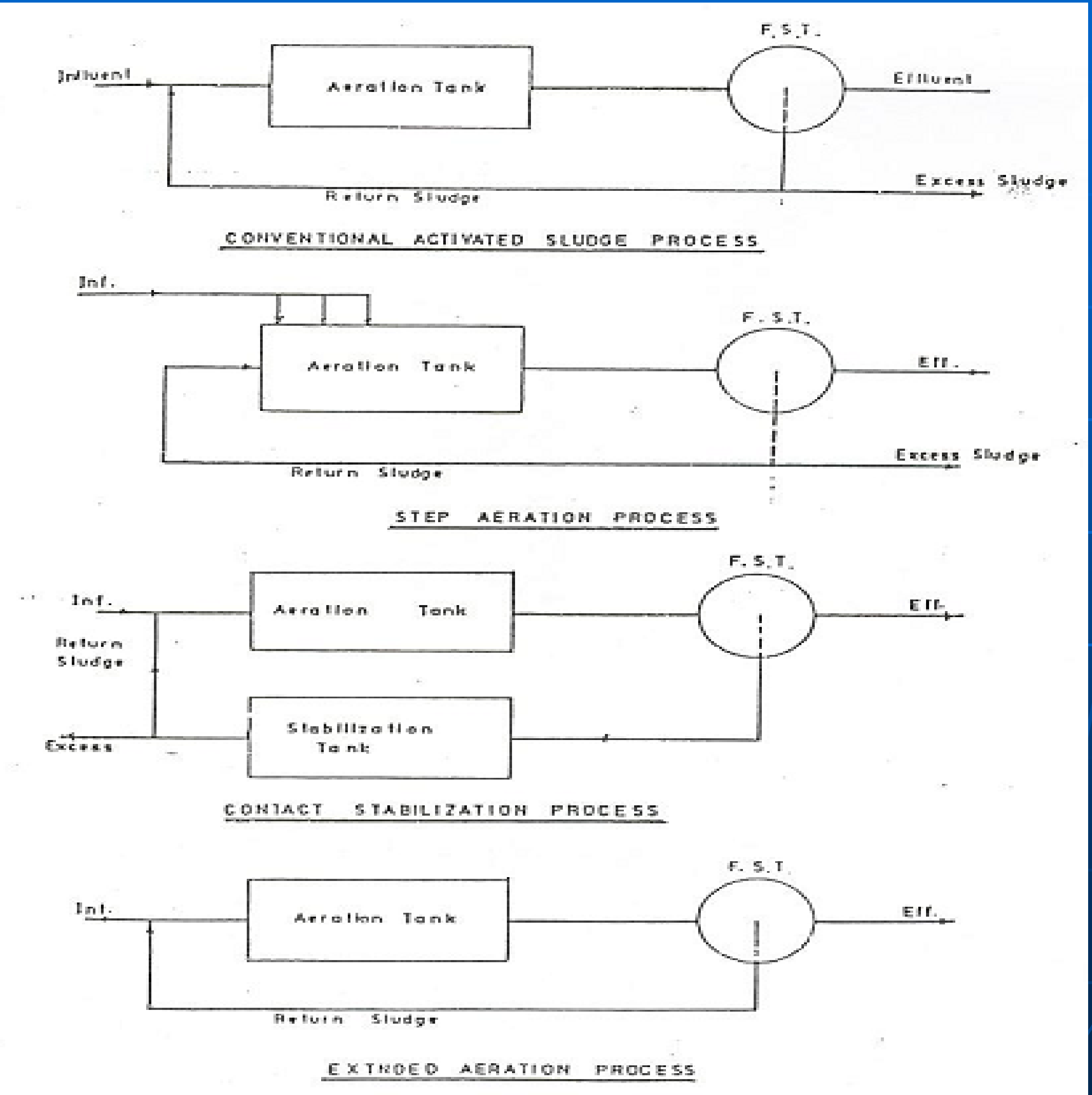
4. Flexibility of operation.

5. The adaptability of final sludge to the utilization as by produce.

Disadvantage of Activated Sludge Process

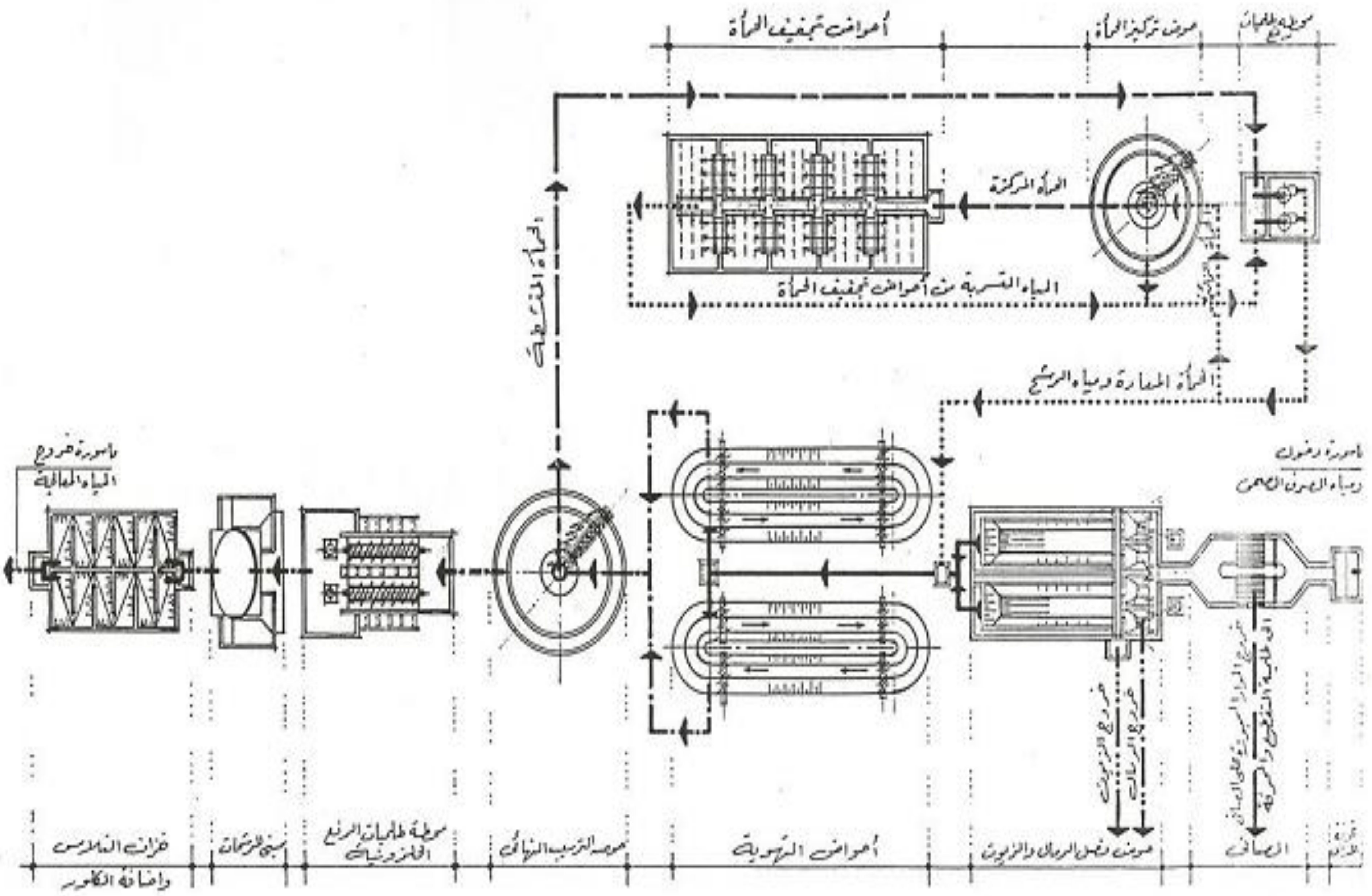
1. Relative high rate of operating cost.
2. Need of trained supervision.
3. The interference of the process caused by the presence of industrial wastes in the sewage treated.
4. The sludge is more difficult to handle, being more watery and requires more time to dry.
5. Sometimes without any clear reasons the effluent is not satisfactory.

Modification of the Conventional Activated Sludge

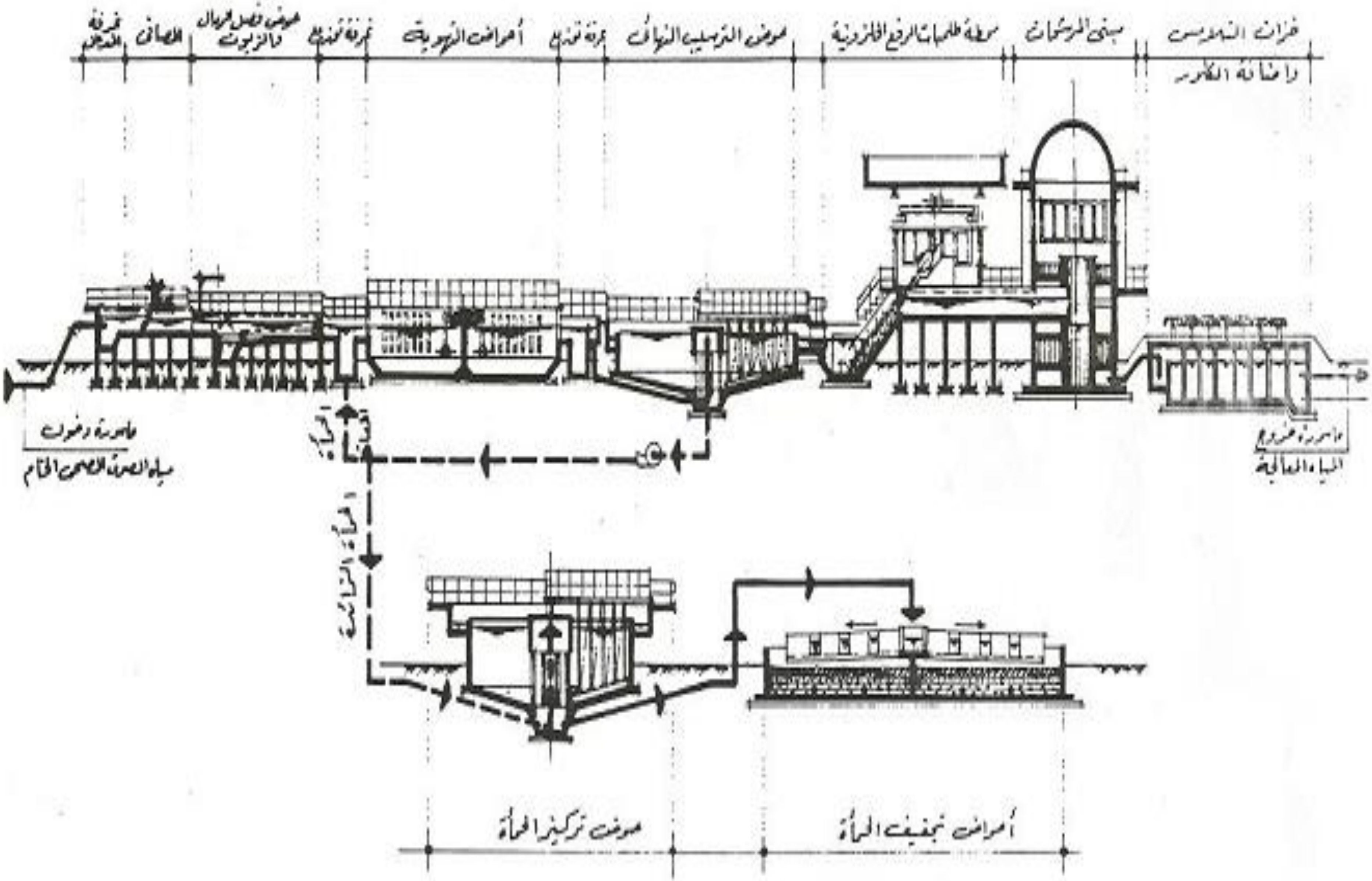


SUMMARY OF LOADING AND OPERTIONAL PARAMETERS FOR AERATION PROCESSES

Process	BOD Loading (gm BOD/m³/d)	F/M ratio (gm BOD/d/gm MLSS)	RT (hrs)	%Return Sludge	% BOD efficiency
Extended Aeration	150 - 500	0.05 – 0.2	20 - 30	100	85 - 95
Conventi onal	450 - 600	0.2 – 0.5	0.6 – 7.5	30	90 - 95
Step Aeration	500 - 800	0.2 – 0.5	5.0 – 7.0	50	85 – 95
Contact Stabilizat ion	500 - 800	0.2 – 0.5	6.0 – 9.0	100	85 - 90
High Rate	1300 - 1500	0.5 – 1.0	2.5 – 3.5	100	80 - 85
High Purity Oxygen	1900 - 2000	0.6 – 1.5	1.0 – 3.0	50	90 - 95



Flow line in activated sludge sewage treatment plant (Extended Aeration)



Hydraulic Profile in Wastewater Treatment plant

Final Settling Tanks

These tanks receive the wastewater effluent from the mechanical aeration tanks in order to settle the suspended solids materials after being fixed and oxidized at the aeration tanks

Circular type is recommended for the FST

Design Criteria for Final Settling Tank

Retention period = 1.5 – 2.5 hrs

Rate of surface loading for the tank = 30 – 45 $m^3/m^2/day$

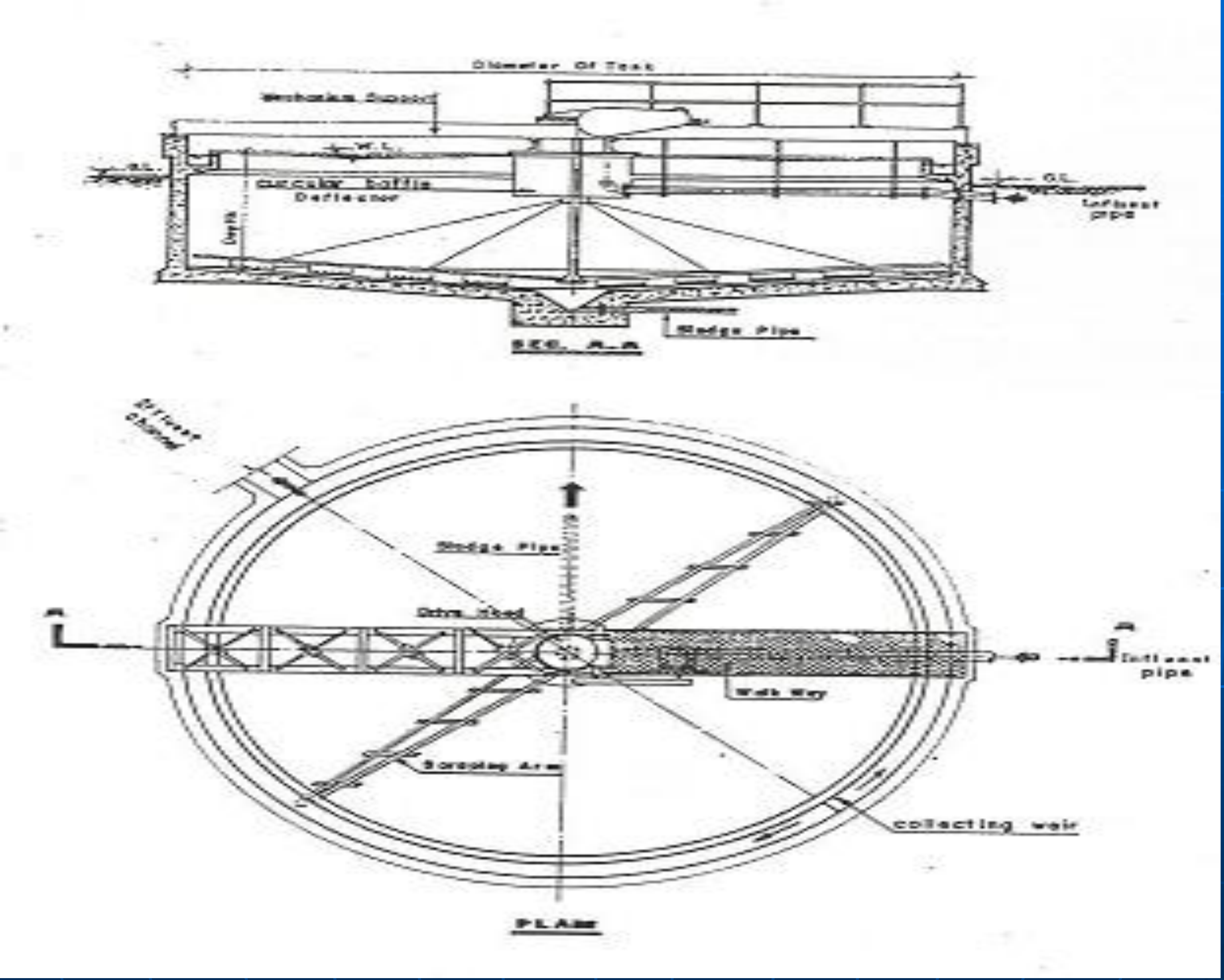
Rate of loading on the effluent weir not to exceed 600 $m^3/m/day$

Depth of tank = 2 – 5 m

Maximum diameter = 35 m

Minimum number of tanks = 2 tanks

Horizontal velocity of flow $\leq$ 0.3 m / min



Final Sedimentation Tank

Final Sedimentation Tank

Waste Stabilization Ponds

Waste stabilization ponds are large, relatively shallow basins enclosed by earthen embankments in which raw sewage is treated by entirely natural processes involving both algae and bacteria

Ponds are used extensively for the treatment of industrial wastes and mixtures of industrial wastes and domestic sewage that are amenable to biological treatment

Ponds have become very popular with small communities where sufficient land is available and where the temperature is favorable for their operation because of their low construction and operation costs and high efficiency of fecal bacteria removal over other treatment methods

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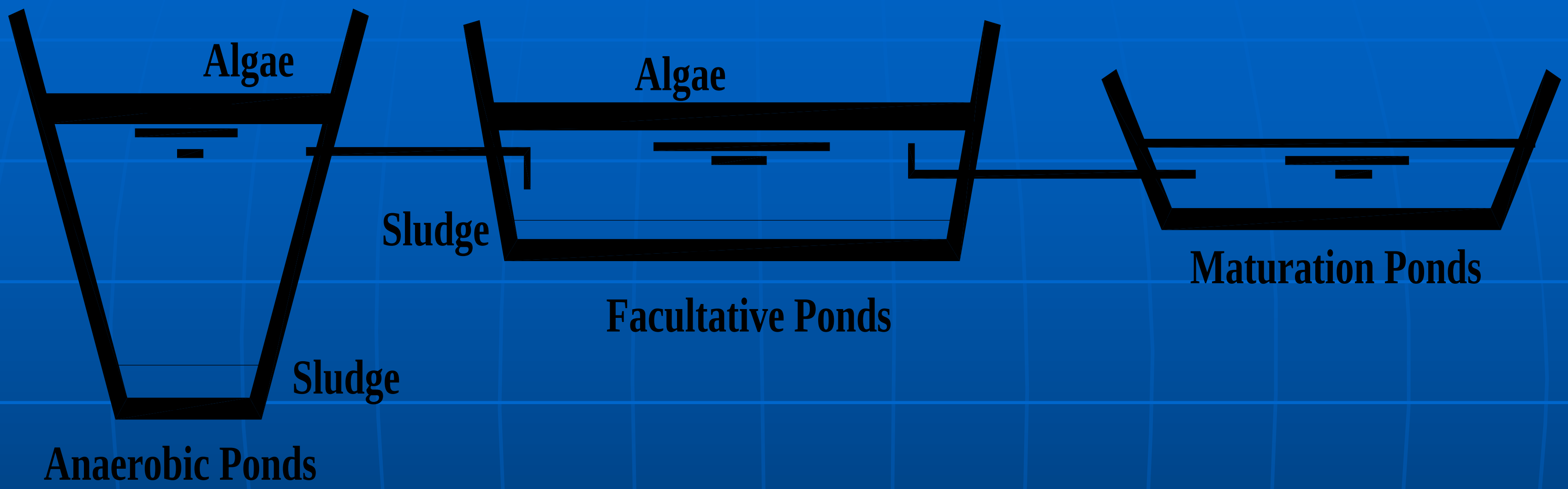
Ponds Classification

Ponds are classified according to the nature of biological activity takes place

Required oxygen is provided through atmospheric surface transfer, photosynthesis and mechanical surface aerators



Stabilization Ponds

Types of Stabilization Ponds

1. Anaerobic Ponds

Using deep ponds (covered by oil and grease) to get anaerobic decomposition for organic matters to decomposition their quantity. (Deep ponds of depth = 3 – 6 m)

Pretreatment strong raw wastewaters that have high solids content. Solids settle to the bottom and digested anaerobically.

Supernatant liquor is discharged into a facultative pond for further treatment.

Sludge desludging is required when the pond is half-full of sludge.

Require temperature above 15 – pH above 6 to maintain anaerobic conditions

Optimum retention time is 5 days

BOD removal up to 75%

BOD loading = 200 – 500 IB/acre/day

Sludge accumulation = 0.03 – 0.04 m^3/m^2/year

Anaerobic Ponds

2. Facultative Ponds

Mixture of aerobic and anaerobic conditions. Aerobic are maintained in the upper layers where anaerobic conditions exist towards the bottom. (Depth of 1.0 – 2.0 m)

3. Maturation Ponds

Wholly aerobic conditions .Destruction of pathogens. (Shallow ponds of depth = 1.0 – 1.5 m). BOD removal is small. Retention time = 7 days

Facultative Pond

Maturation Pond

Design of Facultative Ponds

II.2.1.1. FIRST ORDER KINETICS

The simplest approach to the ration design of facultative ponds is to assume that they are completely mixed reactors which BOD removal follows first order kinetics,

$$\frac{Le}{L_i} = \frac{1}{1+K_1 \cdot t^*}$$

$$OR\ t^* = \left(\frac{Li}{Le} - 1\right)\frac{1}{K_1} \qquad (1)$$

Where,

L_i and L_e are the influent and effluent BOD, respectively in mg/l.

K_1 is BOD rate constant day^{-1}.

$$V = Q.t^* \qquad (2)$$

V is the volume of pond = A*D
t* the retention period
Q is the discharge
From equations (1) & (2)

$$A = \frac{Q}{DK_1}\left(\frac{Li}{L_e} - 1\right) \qquad (3)$$

To maintain the pond contents predominantly aerobic rather than anaerobic, L_e should be in the range 50-70 mg/l for pond depth 1-1.5 m.

The rate constant K_1 is a gross measure of bacterial activity its value is strongly temperature dependent. Its variation with temperature is usually described by the equation:

$$K_T = K_{20}\ \theta^{T-20} \qquad (4)$$

Where K_T and K_{20} are the values of K_1 at T°C and 20°C, respectively, and θ is constant. The value of θ is about 1.05, and the value of K_1 is about 0.3 d^{-1} at 20°C selection of Le as 60 mg/l and combining equations (3) & (4) gives the following design equation for the surface area of the pond "A":

$$A = \frac{Q\,(L_i - .60)}{18\,D(1.05)^{T-20}} \qquad (5)$$

The temperature is mean value of the coldest month.

II.2.1.2. MCGARRY AND PESCOD EMPIRICAL PROCEDURE

McGarry and Pescod empirical formula correlates the maximum BOD_5 surface loading that can be applied to the faculative pond before it failed and the mean monthly ambient air temperature as follows:

$$L_s = 11.2\,(1.054)^T \quad (6)$$

Where, L_s = maximum BOD loading Kg/ha.d

T = Temperature in°F

The formula was obtained from the analysis of operational data from many facultative ponds all over the world.

For design purposes, a factor of safety was introduced in equation (6) which accordingly was modified to

$$L_d = 7.5\,(1.054)^T \quad (7)$$

Where L_d is the design loading in Kg/ha.d.

Design of Maturation Ponds

Two ponds in series each, with a retention period of 7 days are required to reduce the BOD_5 to less than 25 mg/l providing that the influent BOD to the ponds do not exceed 75 mg/l.

The reduction of faecal bacteria in a pond-anaerobic, facultative or maturation has been found to follow first order kinetics, accordingly:

$$N_e = \frac{Ni}{1+K_b\ t} \qquad (8)$$

Where, N_e = number of Fc/100 ml of effluent

N_i = number of FC/100 ml of influent

K_b = first order rate constant for FC removed, d^{-1}

t = retention time in the pond

For n ponds in series equation (8) becomes:

$$N_e = \frac{N_i}{(1+K_b t_1^*)(1+K_b t_2^*)\ldots(1+K_6 t_n^*)}$$

K_b is temperature dependant, its value is given by the equation.

$$K_b(T) = 2.6\ (1.19)^{T-20} \qquad (10)$$

Where, $K_b(T)$ is the value of K at T°C

A reasonable d ign value of N_i is 4×10^7 FC/100 ml.

Chlorine Equipments

Chlorine Storages

EXAMPLE ON EXTENDED AERATION SYSTEM

FLOW RATES

Q_{av} =25000 m^3/day=289 l/s =0.289 m^3/s
PF = 1.6
MF = 0.6
Q_{max} = 40000 m^3/day = 463 l/s =0.463 m^3/s
Q_{min} = 15000 m^3/day = 174 l/s =0.174 m^3/sec
BOD = 250 gm /m^3
SS = 250 gm /m^3
TOTAL AIR REQUIRED = 128 l/s = 460.8 m^3/h

SECONDARY TREATMENT

SELECTOR TANK

F/M ratio = 2.27
TANK VOLUME = Q*BOD / (F/M * MLSS) = 860.41 m^3
Take length of chamber = **25.0** m
Take breadth of chamber = **9.0** m
Water depth = **3.8** m
ACTUAL RETENTION TIME at peak flow = **21.8** min (OK)
ACTUAL RETENTION TIME at average flow =29.7 min (OK)

AERATION TANKS (EXTENDED AERATION)

STAGE (1)

The influent BOD of raw sewage $(BOD)_{in}$ = 250 ppm
% of removal = 0.96
V = $(Q*(L_i - L_e) / (F/M) * MLSS)$

WHERE

V = VOLUME OF TANKS
F/M = FOOD MICRO – ORGANISM RATIO = 0.07
Q = 24 HOUR DESIGN FLOW = 25000 m^3/d
L_i = INFLUENT BOD 250 ppm
L_e = EFFLUENT BOD = 10.0 ppm
MLSS = MIXED LIQUOR SS = 3200 mg/l

EXAMPLE ON EXTENDED AERATION SYSTEM

FLOW RATES

$Q_{av} = 25000\ m^3/day = 289\ l/s = 0.289\ m^3/s$

$PF = 1.6$

$MF = 0.6$

$Q_{max} = 40000\ m^3/day = 463\ l/s = 0.463\ m^3/s$

$Q_{min} = 15000\ m^3/day = 174\ l/s = 0.174\ m^3/sec$

$BOD = 250\ gm/m^3$

$SS = 250\ gm/m^3$

TOTAL AIR REQUIRED = 128 l/s = 460.8 m^3/h

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Q = 24 HOUR DESIGN FLOW = 25000 m^3/d
L_i = INFLUENT BOD 250 ppm
L_e = EFFLUENT BOD = 10.0 ppm
MLSS = MIXED LIQUOR SS = 3200 mg/l
V = 25000*240.0/208 = 28846 m^3
DETENTION TIME = V/Q
DETENTION TIME = (28846/25000) days = **27.7 Hours**

USE RECTANGULAR TANKS WITH SURFACE AERATORS

DEPTH OF TANK (D) = 4 m

AREA OF TANKS (A) = (V/D) = 28846/4 = 7211.54 m^2

ORGANIC LOADING RATE FOR AERATION TANKS = TOTAL BOD OF INFLUENT SEWAGE / VOLUME OF AERATION TANK = $(Q_{av}(OR)*(BOD)_{in}/(V)$ = 216.67 kg BOD / 1000 m^3 / day OK

NO. OF TANKS (NO) = 2

AREA OF TANK (A_1) = (A / NO) = 3605.77 m^2

WIDTH OF TANK = 10 m

TOTAL LENGTH OF TANK = 361 m

STRAIGHT LENGTH OF TANK = 90.25 = 90.0 m

RETURNED SLUDGE

R = C1/ (C2 - C1)

Where

R = RETURN SLUDGE RATE
C1 = MLSS mg/l IN AERATION TANK = 3200 mg/l
C2 (RASS) = EXPECTED CONCENTRATION OF MLSS IN THE RETURN SLUDGE = 8000mg/l
R = 3200/4800 = 0.67
AMOUNT OF RETURNED SLUDGE = R * Q = 0.67*25000 = 16750 = m^3/d = 194 l/sec
ACTUAL RETENTION TIME WITH RECYCLE FLOW =16.6 hr

RETURNED SLUDGE PUMP

(2+1) pump each with Q = 97 l/sec at head = 12 m
POWER FOR EACH PUMP = 25.5 Kw
SELLECTED POWER = 30.0 Kw
TOTAL POWER REQUIRED = 60.0Kw

OXYGEN REQUIRED

O_2 r./hr. SHOULD NOT LESS THAN 2kg OF OXYGEN PER kg OF BOD REMOVED
O_2 r./hr = (2/24)* 25000*240/ 1000 = 500.0 kg/hr = 12000 kg/day.
OR USE
$Oc = \{a(F/M)+b\}M$

Where

Oc = CARBONACCEOUS OXYGEN REQUIRED kg/day

F/M =kg BOD/day/kg MLSS IN AERATION TANK = 0.07

a = CONSTANT = 0.55 FOR DOMESTIC

b = CONSTANT = 0.15 FOR DOMESTIC

M = MLSS IN AERATION TANK = 92308 kg

Oc = 17146 kg/day

Oc = 714 kg/hr

On = 4.6(^NH3)* Qav(OR). /1000

ASSUME ^NH3 =35 ppm =5520kg/day =230kg/hr

TOTAL O2 = Oc + On

TOTAL O2 = 22666kg/day =944kg/hr.

POWER REQUIRED FOR AERATION

HP = { O2 (9.17) (1.024) ^(20 -T) } / CWTR [CDc (B) - Cr] z

Where

O2 =Oxygen required (kg / hr) = 944
T = Maximum temperature = 40 oC
CWTR =Clean water transfer rate = 3
CDc =O2 Saturation concentration at design conditions = 7.6
B =(kg O2 required /day) / (kg MLSS in aeration tank)= 0.95
Cr =Desired O2 residual (1.0 - 2.0 mg/ l) =2
Z =(kg O2 required / day) / (kg BOD5 removed / day) 0.9
HP =38286400, No. of aerators =2 * 3 = **6aerators**
Net power required for each aerator =63.7 HP
Total power for each aerator = 65.8 kw
Standard motor = **75.0 kw,** Power required at first stage = 450.0 kw

FINAL SEDIMENTATION DISTRIBUTION CHAMBER

Take detention time at maximum design flow = 40seconds
Capacity of distribution chamber = 26.3 m3
Take length of chamber = 4.0m
Take breadth of chamber = 3.0m
Water depth = 2.2m
No. of effluent weirs = 2
Length of each weir = 3.0

For rectangular effluent weir

$Q = 1.772*b*h^{3/2}$

Where

Q = design flow m3/sec.

b = length of weir m

h = water head over weir m

Flow for each weir at Q max. = 28375.0m^3/day = 0.3 m^3/sec.

$h^{3/2} = 0.062$

h = 0.156m

Flow for each weir at Q_{av} =20875m^3/day

= 0.2m^3/sec.

$h^{3/2} = 0.045$

h = 0.127m

FINAL SEDIMENTATION TANKS

DETENSTION TIME (T) = 1 - 4 HOURS.
DETENSTION TIME AT AVRAGE FLOW T = 3.5 HOURS
SURFACE LOADING RATE (SLR) = 10 – 24 $m^3/m^2/day$
FOR AVERAGE FLOW = 40 - 48 $m^3/m^2/day$

FOR PEAK FLOW

ASSUME (SLR) = 28 $m^3/m^2/day$
WEIR LOADING RATE = 124 - 220 $m^3/m/day$

DESIGN

SURFACE AREA OF TANKS =(Qav + (R) *Qav)/SLR = 1491.0714 m^2

CAPACITY OF TANKS = (Q_{av} + (R) Q_{av}) * T = 6088.5417 m^3

ASSUME NO. OF TANKS = 2 TANKS

THE TANK SURFACE AREA = 746 m^2

THE TANK DIAMETER (D) = 31 m

THE DEPTH OF WATER (P) = VOLUME / AREA =4m

ACTUAL TANK VOLUME = 3017.54 m^3

ACTUAL RETENTION TIME AT PEAK FLOW = 2.55 hr

ACTUAL SLR AT PEAK FLOW = 37.61 m^3 / m^2 / day

CHECK

WEIR LOADING AT PEAK FLOW = Q_{MAX} / PERIMETER = 205.5 $m^3/m/$ day < 220 OK

Tank Dimensions

NO = 2 TANK

D = 31m

WATER DEPTH =4m

Sludge hopper

The quantity of excess sludge produced from final sedimentation tank = QW1 = 412.50 m^3/day

Take the retention time of sludge inside the sludge hopper = 0.5 hr

Capacity of sludge hopper = 4.30 m^3

Take sludge hopper (upper diameter) = 2.5 m

Take sludge hopper (lower diameter) = 1.6 m

Sludge hopper depth = 1.24 m

Effluent channel

Max. flow rate = 0.33 m^3/sec.
Q = A*V
Wet cross section area of channel A =b*d

Where

b =width of channel = 1.00 m
d = maximum depth of water in channel
V = velocity of flow in channel = 0.8 m/sec
Maximum depth of water in channel = 0.411 m

Effluent channel

Max. flow rate = 0.33 m^3/sec.
Q = A*V
Wet cross section area of channel A =b*d

Where

b =width of channel = 1.00m
d = maximum depth of water in channel
V = velocity of flow in channel = 0.8m/sec
Maximum depth of water in channel = 0.411 m

Effluent weir

An adjustable 90 v-notch weir is proposed

Proposed weir dimension shall be as shown

No. of notches =perimeter /0.12 = 811

Q_{av}/notch = 0.000178 m^3/sec

Q_{max}/notch = 0.000285 m^3/sec

To find head over the notch using formula

q = 1.37*h^5/2

h_{av}^5/2 = 0.000130028

h_{av} = 0.028m

$h_{maximum}$^5/2 = 0.00021

$h_{maximum}$ = 0.034m

IWW Treatment BASICS

- WHAT IS INDUSTRIALWASTEWATER?
  - WHAT IS DOMESTIC?? CONSTITUENTS, PATTERN, COD/BOD, BOD:N:P, PH, …
  - FOR INDUSTRIAL:
    - Depend on industry and materials processed
    - Very strong, easily biodegradable, large inorganic, inhibitory, nutrient deficient, Ph, pattern depend on industry operations.

NATURE

- BIODEGRADABILITY
- STRENGTH
- VOLUME
- VARIATIONS
- SPECIAL CHARACTERISTICS

BIODEGRADABILITY

- Treat it by biological means!
  - Quantities of organics, no inhibition,
  - COD/BOD < 3 is biodegradable,
    - Poultry industry : COD 2000 – 4000
      BOD 1500 – 3000; C/B=1.5
    - Tobacco Process : COD 4500 – 11800
      BOD 760 – 3200; C/B > 3
    - Dyestuff industry : COD 4400
      BOD 55; C/B=80

STRENGTH

- Indus. wastewater have organic strength higher than domestic ww.
- Agro-industries are typical of very high strength organic ww.
  - Sugar mill : COD 50000, BOD 25000
  - Coconut cream: COD 12900, BOD 8900

- VOLUME
  - Vary from small volume to huge ; starch extraction of 3 m3.d up to paper industry is equiv. to 160 000 capita in term of hyd. Load and 1.7 million p.e. in terms of BOD load
- VARIATIONS
- SPECIAL CHARACTERISTICS
- MANUFACTURE PROCESS

MAJOR TECHNIQUES

- EQUALIZATION
- OIL AND GREASE, DAF
- PH ADJUSTMENT
- REMOVAL OF INHIBITORS
- NUTRIENT SUPPLIMENT

EQUALIZATION

- MAY FOR HYDRAULIC OR LOADING.
- MIXING IS NEEDED, BUT AERATION?
- TEMPERATURE CONTROL; FOOD INDUSTRIES;;;;DILUTION, THEN IT IS BLENDING TANK NOT EQUALIZATION

OIL AND GREASE

- GOS
- DAF; COAGULANTS USUALLY NEEDED. PH CONTROL IS NEEDED.
- LARGE METAL HYDROXIDE SLUDGE
- Fine Screen / Clarifier

PH ADJUSTEMENT

- VERY ACIDIC TO VERY ALKALINE
- Hold and Blend
- To adjust acidic; use sodium hydroxide or lime. Lime is cheaper; use in case of large plants. Handling with safety. Lime is slower in reaction; reaction tank to b increased.20 min. a minimum. Mix with air or stirrer.
- To adjust alkaline; use sulphuric acid. Cost wise. If anaerobic treatment is used and large quantity of acid is required use hydrochloric acid.

Removal of inhibitory

- Organics, metals, ammonia, flourides.
- A combination of PH, PRECIPITATION, AND COAGULATION IS FREQUENTLY USED. ESPECIALLY FOR METALS REMOVAL.
- CHEMICAL OR BIOLOGICAL SOMETIMES.

NUTRIENTS SUPPLEMENT

- IN DOMESTIC, NUTRIENTS ARE REMOVED, BUT IN IWWTP, USUALLY ADDED.
- SLAUGHTERHOUSE BLOOD CONTAINS HIGH NITROGEN TO BE REMOVED.
- Bcz; IWW has high org. load vs the N & P.
- USE UREA AND PHOSPHORIC ACID
- IN SMALL IWWTP: AMMONIUM DIHYDROGEN PHOSPHATE TO AVOID HANDLING OF PHOS. ACID.
- BLEND DOMESTIC WW WITH IWW.
- MINIORITY OF K, Mg, Ca, Fe, Mn,… supplement occasionally.
- Biocatalysts !! Mixture , dried bacterial mixture, by-products of bacterial fermentation. PUCCACCI AND EM