DESIGN AND OPERATING OF

WATER TREATMENT PLANTS

Basic studies for water supply system

For the proper design of water supply system, these basic studies must be done:

1.The design period.

2. Population present and future.

3.Water consumption.

4..Water source.

Design period

In water works the design period ranges from 30 to 50 years.

Factors affecting the design period:

1. Virtual age of different components of the WTP.
2. Cost (capital, operation and maintenance).
3. Population growth.
4. Water consumption.
5. Water source.
6. Labors skills.

1. Virtual age

Reinforced concrete constructions subject to water (30 – 50 years)

Steel constructions (20 – 30 years)

Pipes (50 – 80 years)

Mechanical equipments (10 – 15 years)

POPULATION PREDICTION

Water supply systems must be designed to serve the future population as well as the present population.

Population prediction methods

1.Arithmatical method.
2.Geometrical method.
3.Annual rate of increasing method.
4.Graphical extension method.
5.Graphical comparison method.

METHODS OF POPULATION PREDICTION

1. Arithmetic method

$$P_n = P_o + Ka_{(av)} * \Delta t$$

Where:

$Ka_{(av)} = \Sigma(\Delta P/ \Delta t)/\text{no. of } (Ka)$

P_n : Future Population at t_n

P_o : Present Population at t_o

$\Delta t = t_n - t_o$

t_o : Base year

t_n : Goal year

METHODS OF POPULATION PREDICTION

2.Geometrical method

$$\mathbf{Ln\ P_n = Ln\ Po + Kg_{(av)} * \Delta t}$$

Where:

$Kg_{(av)} = \Sigma(\Delta\ LnP/\Delta t\ )/\text{no. of }(Kg)$

P_n : Future Population at t_n

P_o : Present Population at t_o

$\Delta t = t_n - t_o$

t_o : Base year

t_n : Goal year

METHODS OF POPULATION PREDICTION

3.Annual rate of increase method

$$P_n = P_o * [1 + (\% \ r_p / 100)]^{\Delta t}$$

Where:

r_p : Annual rate of increasing in population

P_n : Future Population at t_n

P_o : Present Population at t_o

$\Delta t = t_n - t_o$

t_o : Base year

t_n : Goal year

METHODS OF POPULATION PREDICTION

4.Graphical extension method

This method assume that the population-time curve of the past records may extended into future by following the trend of increase in the past.

METHODS OF POPULATION PREDICTION

5.Graphical comparison method

It based on the assumption that the future population will increase in the same way as similar larger cities have increased in the past after reaching the present population of the investigated city.

WATER CONSUMPTION

In a normal city the ratio of consumption of water for these different uses may be as follow:

WATER USES	PERSENT OF TOTAL
Domestic	35
Commercial	20
Industrial	20
Public	25
Total	100

Factors affecting the rate of water consumption:

1. Size of community.
2. Climate.
3. Standard of living.
4. Pressure of water in distribution networks.
5. Water quality.
6. Water cost.
7. Sewage facilities.

FLUCTUATION OF DEMAND

Average rates of demand for various periods of time

Period	Maximum rates as % of average
Average	100
Maximum monthly	125 - 150
Maximum daily	150 - 180
Maximum hourly	200 - 300

FIRE DEMAND

Egyptian specification recommended 60 m^3/hr for one fire on assumption of 2 hours fire for each 10,000 capita or as indicated in the following table.

No.	Population (capita)	Required fire discharge
1	10,000	20
2	25,000	25
3	50,000	30
4	100,000	40

WATER SOURCES

1. Rain water

2. Surface water (fresh – salt)

(Rivers – Lakes – Seas – Oceans)

3. Ground water (fresh – salt)

(Wells)

WATER QUALITY

- **Proposes of studying water quality:**

1. Determine the degree of pollution.
2. Determine of design steps for water treatment process, (drinking water – industrial water – swimming ponds).
3. Assessment of treatment units.
4. Check the effluent of WTP with environmental.

Characteristics of water

1. Physical characteristics.

Parameter	Ranges for Drinking Water
1. Temperature	(15 – 20^{o}C)
2. Turbidity	$\leq$ 1 NTU
3. Color	Colorless'
4. Odor	No odor

Characteristics of water

2. Chemical characteristics.

2.1 pH : ranged from (6.5 – 9.5)

2.2 Dissolved Solids:

Manganese ≤ 0.1 mg/l,	Iron and Manganese ≤ 0.3 mg/l
Iron ≤ 0.3 mg/l,	Sodium ≤ 200 mg/l
Calcium ≤ 200 mg/l,	Magnesium ≤ 150 mg/l
Sulphate ≤ 400 mg/l,	Chloride ≤ 500 mg/l
Copper ≤ 1.0 mg/l,	Nitrate ≤ 10 mg/l
Nitrate ≤ 0.005 mg/l,	Lead ≤ 0.05 mg/l
Cadmium ≤ 0.005 mg/l,	Chloride ≤ 500 mg/l
Mercury ≤ 0.001 mg/l,	Toxic matters = Zero mg/l

Characteristics of water

3. Biological characteristics.

- Source, (Micro-organisms, bacteria, virus, protozoa…etc)
- Pathogens = (Harmful bacteria)
- Indicator = Used to indicate the present of pathogens.

- **Properties of an ideal indicator:**

1. Applicable for all types of water.
2. Always present when pathogens are present.
3. Non-pathogen for the lab. Personal.
4. Have a longer survival time outside the human body (24 hrs)

MUNICIPAL WATER SUPPLY WORKS

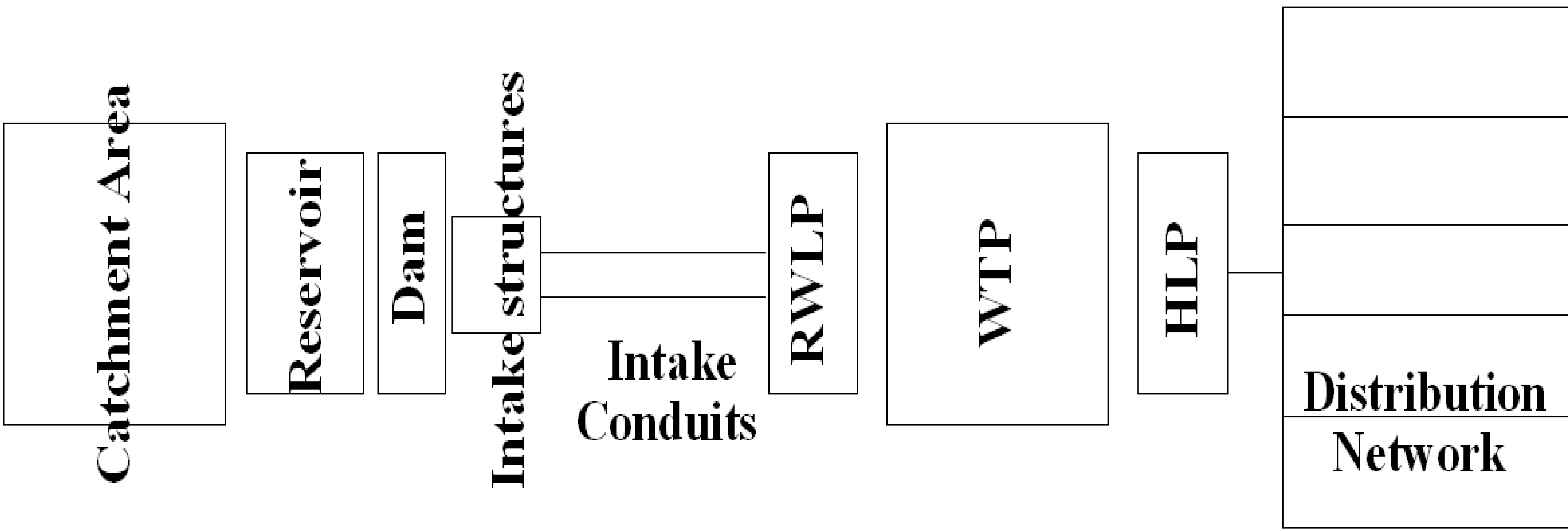
1. COLLECTION WORKS

2. PURIFICATION WORKS

3. DISTRIBUTION WORKS

Supply Works for Rain Water (RW)

This figure shows flow line in rain water treatment works



Rain Water Flow Rate (Q)

The rain water flow rate can be computed from this equation:

$$[\mathbf{Q = C * I * A}]\ \mathrm{m^3/d}$$

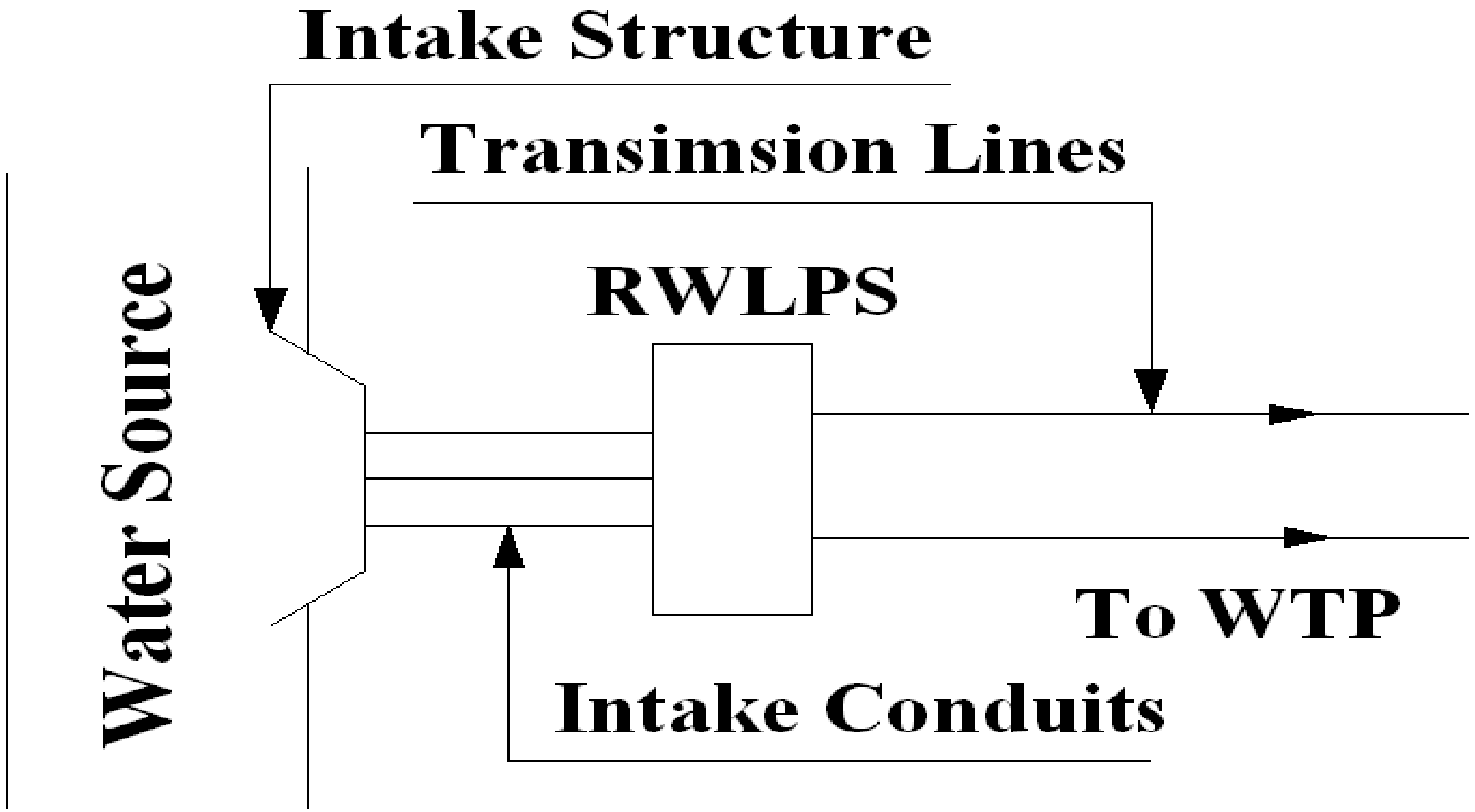
Where:

Q: Rain Water Flow Rate (m^3/d)

I: Intensity of Rain Fall (m/d)

A: Catchments' Basin Area (m^2)

Collection Works for Surface Water



Flow diagram in collection works

Collection Works for Surface Water

1. Intake structure.

2. Intake conduits.

3. Raw water lift pump station.

4. Transmission lines.

INTAKE STRUCTURE

- **Purpose of intake structure**

1. **Collect the water from the source.**
2. **Protect the embankment sides' slopes from failure.**
3. **Prevent clogging of intake conduits (because the intake structure consists of screen prevent the entrance of the undesired matters).**

•Factors affecting the choice of intake structure type

1. **Water source dimensions (width – depth).**
2. **Character of bottom.**
3. **Effect of currents, floods and storms upon the structure.**
4. **Water source pollution (on shore – on surface).**
5. **Navigation requirements.**
6. **Fluctuation in water level.**

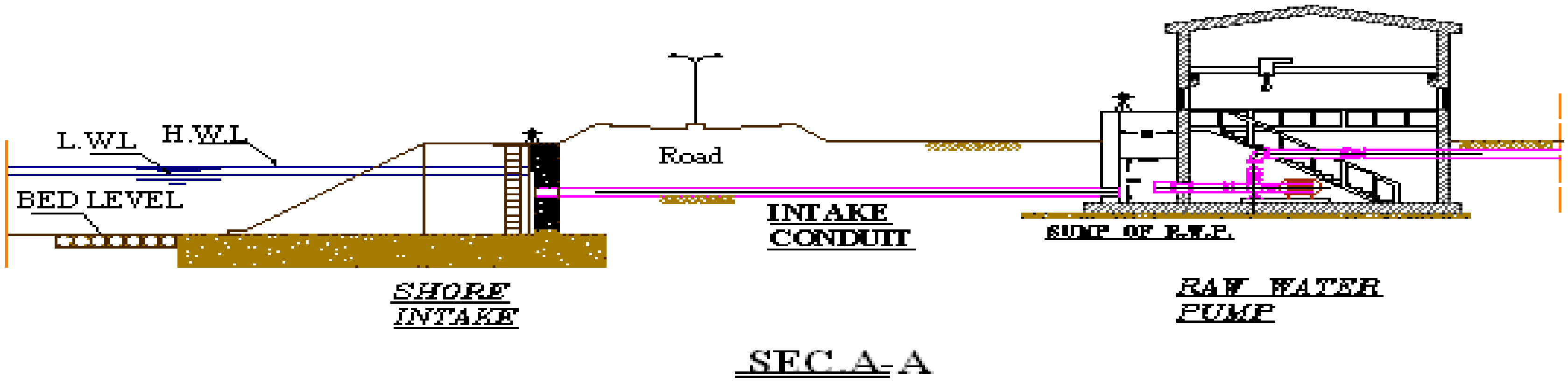
- **Factors affecting the choice of intake structure location**

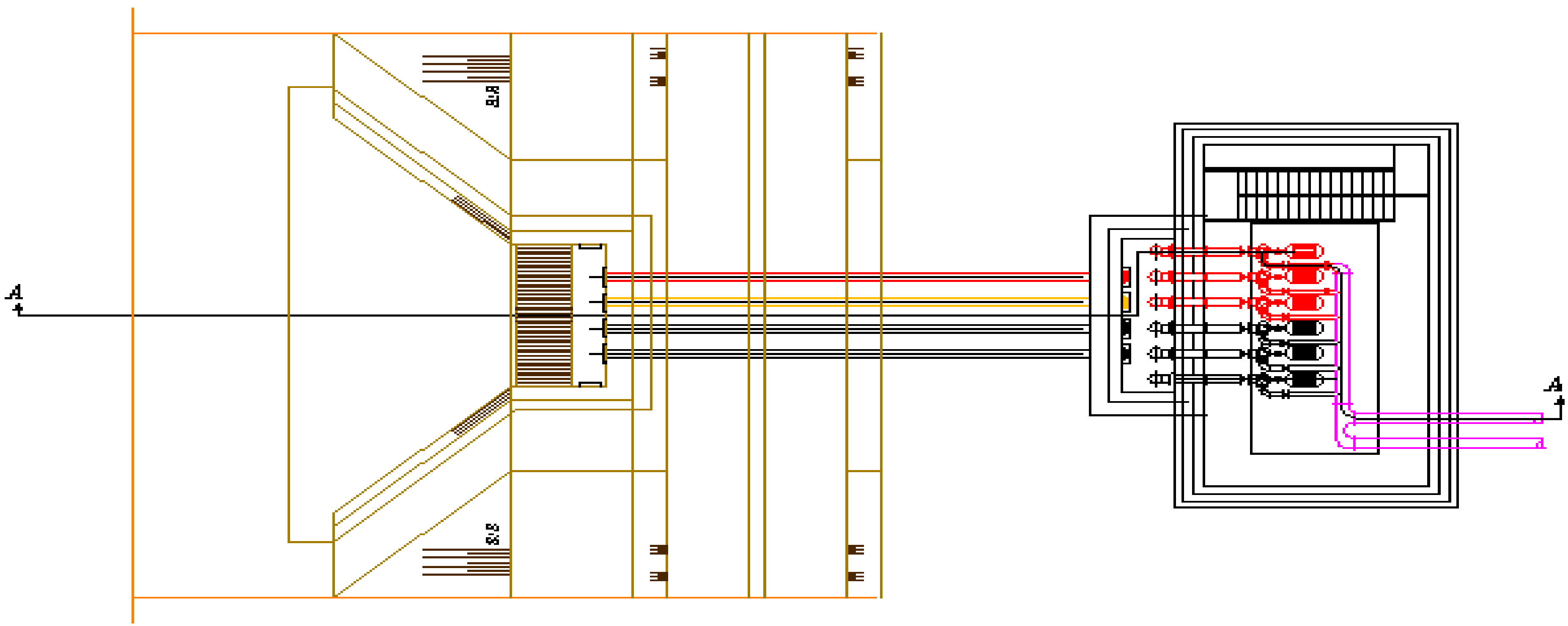
1. **Upstream the served city to prevent the direct pollution.**
2. **On straight part of the water source to prevent settling and scoring.**
3. **Restricted area taken around the intake structure (150 m upstream and 50 m downstream).**

TYPES OF INTAKE STRUCTURE

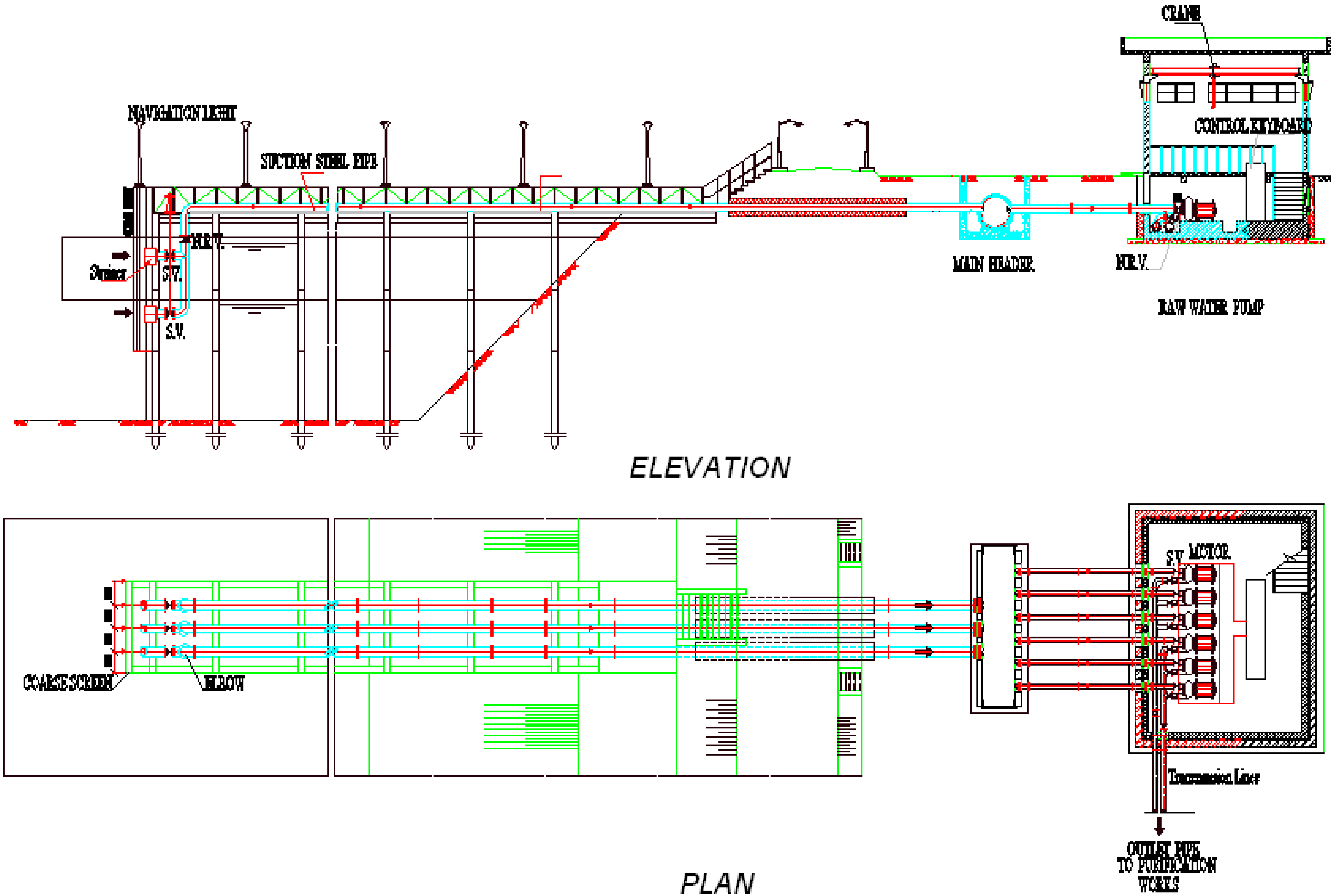
Type	Uses
Shore intake	None polluted shore – navigable canal – narrow and shallow canals.
Pipe intake	Wide rivers (150 – 300 m) – polluted shore – high fluctuation in water level – navigable. Material of pipe intake is (Ductile iron or steel). [Suction head $H_s \leq 7m$ = Atmospheric pressure H_a (=10.33m) – friction losses H_f – secondary losses H_{se} – vapors pressure H_v – velocity head H_{ve}].
Submerged intake	Narrow canals – polluted shore – deep canals.
Tower intake	Insufficient wide for pipe intake to allow navigation – wide fluctuation in water level – common in lakes.

SHORE INTAKE





PIPE INTAKE


CRANE
NAVIGATION LIGHT
SUCTION STEEL PIPE
CONTROL KEYBOARD
N.R.V.
Strainer
S.V.
S.V.
MAIN HEADER
N.R.V.
RAW WATER PUMP
ELEVATION
S.V. MOTOR
COARSE SCREEN
ELBOW
Transmission Line
OUTLET PIPE
TO PURIFICATION
WORKS
PLAN

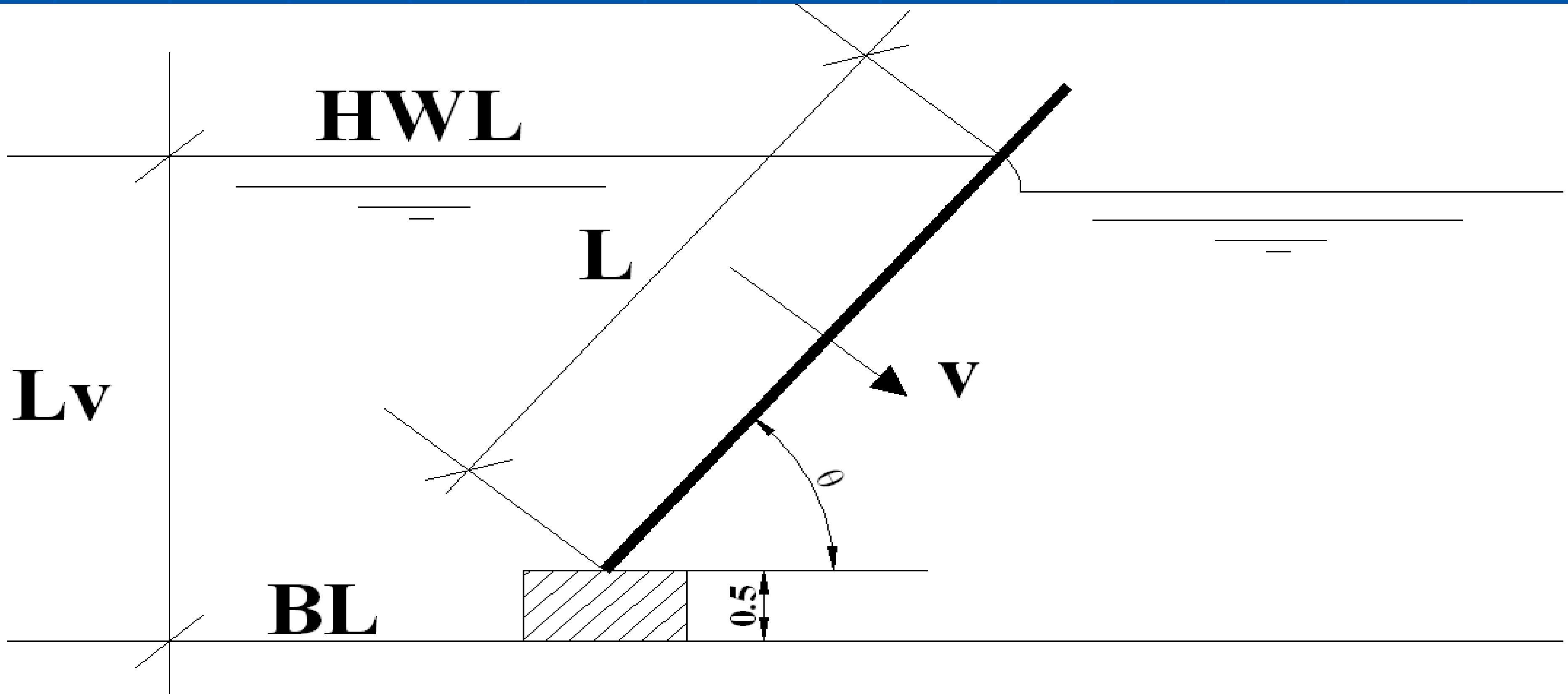
DESIGN OF SHORE INTAKE BAR SCREEN (L,S,B,Φ)

DESIGN CRITERIA

- Perpendicular velocity on screen projection ≤ 0.15 m/sec
- The maximum permissible head loss through the screen is (10 – 15 cm)
- The bar diameter (Φ) = 1.3 – 1.9 cm (or) 0.5 – 0.75 inches.
- The spacing between two bars (S) = 2 – 5 cm.
- The inclined angle of the bar screen (θ) = 30 – 60o
- The vertical bar screen length (L_v) = (HWL – BL) - 0.5 m
- The inclined bar screen length (L) = (L_v) / sin θ
- The inclined net area of the screen = n*S*L
- B = (n+1)*Φ + n*S
- n = number of spacing = (number of bars – 1)

Example

Design the bar screen of a shore intake serves a city of population 50,000 capita with average water consumption 250 l/c/d, if (HWL – BL = 4m) .



Solution

- L_v = 4 m, Pop = 50,000 capita, q_{av} = 250 l/c/d
- $Q_d = Q_{mm} = 1.4\ Q_{av} = 1.4 * Pop * q_{av} = 17500\ m^3/d$
- Assume wp = 24 h/d
- $Q_d = 0.203\ m^3/s$
- Assume v = 0.15 m/s
- $n*S*L = Q_d / v = 0.203 / 0.15 = 1.35\ m^2$
- Assume $\theta = 45^o$, S = 0.03 m
- $L = L_v / \sin\theta = 5.7$ m
- n = 1.35 / (0.03*5.7) = 8 opens
- Assume Φ = 1.6 cm
- $B = (n+1)*\Phi + n*S = 0.384$ m
- Take B = 0.4 m
- S_{act} = 0.032 m

INTAKE CONDUITS

- **Purpose of intake conduits**

Transmit the raw water from the water source to raw or low water lift pump station

DESIGN OF INTAKE CONDUITS

DESIGN CRITERIA

- Number (n) ≥ 2
- Diameter (Φ_{mm}) = 25 – 50 - … – 250 – 300 - … – 500 – 600 - … – 1000 mm.(It's preferred to not be exceed than 1000m)
- Design flow = Q_d = Q_{mm} (m^3/sec), minimum flow (Q_{min}) = 0.5 Qd
- Q_{mm} = 1.25 – 1.5 Q_{av}
- Ordinary velocity (V_{ord}) = 0.6 – 1.5 m/s
- Maximum maximum velocity at one pipe is broken (V_{mm}) $\leq$ 2.5 m/s (If not increase number of pipes at the same cross section area by decreasing the pipes diameter)
- Minimum velocity at minimum flow (V_{min}) $\geq$ 0.6 m/s (If not close pipe or more, taking into consideration that may be one pipe only works at minimum flow if we have to that)

IMPORTANT CHECKS

1. $V_{ord} = Q_{mm} / n*\pi/4*\Phi^2$ (0.6 – 1.5 m/s)
2. $V_{mm} = Q_{mm} / (n-1)*\pi/4*\Phi^2$ ($\leq$ 2.5 m/s)
3. $V_{min} = Q_{min} / n*\pi/4*\Phi^2$ ($\geq$ 0.6 m/s) if not close pipes.

IN CASE OF DESIGN IN PRESENT AND FUTURE

1. Number (n) $\geq$ 2 in the present.
2. Assume the ordinary velocity at future is 1.4 to 1.5 m/s, and get the present ordinary velocity = (V_{ord}.future) * [($Q_d$present) / ($Q_d$future)] $\geq$ 0.6 m/s. (If not, we have to decrease the pipes in the present and redesign)

CASE OF EXISTING INTAKE CONDUITS (GIVEN Φ, N) - REQUIRED THE MAXIMUM FLOW CAN BE CARRIED BY THESE PIPES

1. Assume V_{ord} = max. = 1.5 m/s, Get $Q_{d1} = 1.5*n*\pi/4*\Phi^2$

2. Assume V_{mm} = max. = 2.5 m/s, Get $Q_{d2} = 2.5*(n-1)*\pi/4*\Phi^2$

3. Q_{dmax} = the least from Q_{d1}, Q_{d2}

Example

It is required to design the intake conduits of the water collection works for a city of population 160,000 capita and average water consumption of 180 l/c/d (constant with project period), if the annual population rate of increase is 2% and design period is 40 years.

Solution (2)

In present

P = 300,000 capita

av.wc = 200 l/c/d

Q_{av} = 200,000 * 200 = 40,000,000 l/d = 40,000 m^3/d

Q_{mm} = 1.4 Q_{av} = 0.694 m^3/s

Q_{min} = 0.7 Q_{av} = 0.486 m^3/s

In future

P = 300,000*(1+0.01)40 = 446,660 capita

Total population increase ratio% = [(446,660-300,000)/(300,000)] * 100 = 48.9 %

wc increase rate% = 0.1 * 0.489 = 0.0489

av.wc = 200*(1+0.0489) = 209.8 l/c/d

Q_{av} = 446,660 * 209.8 = 9371 m^3/d

Q_{mm} = 1.4 Q_{av} = 1.52 m^3/s

Q_{min} = 0.7 Q_{av} = 0.759 m^3/s

Design in future

Assume wp = 24 hr/d

For economic design, assume v_{ord} = 1.5 m/s

So, min. required TXA = 1.52/1.5 = 1.013 m^2

Assume Φ = 600 mm = 0.6 m

Take n = 4 pipes

Important checks in future

$V_{ord} = Q_{mm}/4*\pi/4*\Phi^2$ = 1.345 m/s (ok) at (0.6 – 1.5 m/s)

$V_{mm} = Q_{mm}/3*\pi/4*\Phi^2$= 1.793 m/s (ok) at ($\leq$ 2.5 m/s)

$V_{min} = Q_{min}/4*\pi/4*\Phi^2$= 0.673 m/s (ok) at ($\geq$ 0.6 m/s)

<u>Important checks in present</u>

$V_{ord} = Q_{mm}/4*\pi/4*\Phi^2 = 0.86$ m/s (ok) at (0.6 – 1.5 m/s)
$V_{mm} = Q_{mm}/3*\pi/4*\Phi^2 = 1.14$ m/s (ok) at ($\leq$ 2.5 m/s)
$V_{min} = Q_{min}/4*\pi/4*\Phi^2 = 0.43$ m/s (not ok) not at ($\geq$ 0.6 m/s), close 2 pipes, V_{min} become = 0.86 m/s (ok) at ($\geq$ 0.6 m/s)

<u>Conclusion</u>

"Take <u>four pipes</u> **Φ (600)** mm (present and future) & close <u>two pipes</u> at minimum flow in present"

DESIGN OF RAW (LOW) LIFT PUMP STATION

FLOW CALCULATIONS

Total number of pumps = working + standby

Standby pumps = 50% to 150% working pumps

Working pumps ≥ 2

	Q_{min}	Q_{mm}
Value	… l/s	… l/s
n_w	≥ 2	≥ 2
n_{sb}	50% to 150% n_w	50% to 150% n_w
n_T		$n_w + n_{sb}$

In case of connect more than one pump in parallel, the result discharge will be as shown in the table:

X : discharge of one pump

Number	Total Discharge
1	X
2	1.6 X
3	2.2 X
4	3 X
5	3.5 X
6	4 X

HEAD CALCULATIONS

- **The total head** = the dynamic head + the static head
- **The dynamic head** = the friction losses through the intake conduits and through the rising main + the secondary losses
- **The static head** = level of water in the first unit in the plant – low water level of the water source
- **The level of the first unit of the plant** = water level in the ground tank (ground level) + the losses (main and secondary) through the plant between the units

IF NOT MENTION,

- Take the water level in the first unit in the plant = ground level + 5 meter.
- Take the secondary losses 20 % of the main losses.
- It can be used the **Darcy's law** to calculate the friction losses through the pipes:

$$[h_f = 4\ FLV^2/2g\Phi]\ \ (m)$$

Where:

- F = friction coefficient = 0.008 for concrete.
- L = pipe length by meter, if not mention take (30 – 70 m for shore intake), (100 – 200 m for pipe intake).
- V = water velocity through the pipe by m/sec.
- g = 9.81 m/s^2
- Φ = pipe diameter by meter.

HORSE POWER CALCULATIONS

Total HP = Water density (1000 kg/m^3) * n_w * $q_{onepump}$ (m^3/sec) * Total head (meter) / 75 * η_1 * η_2

Take η_1 * η_2 = 0.6 if not mention.

The life time of the pumps from 10 to 15 years, so we can use different pumps types through different phases.

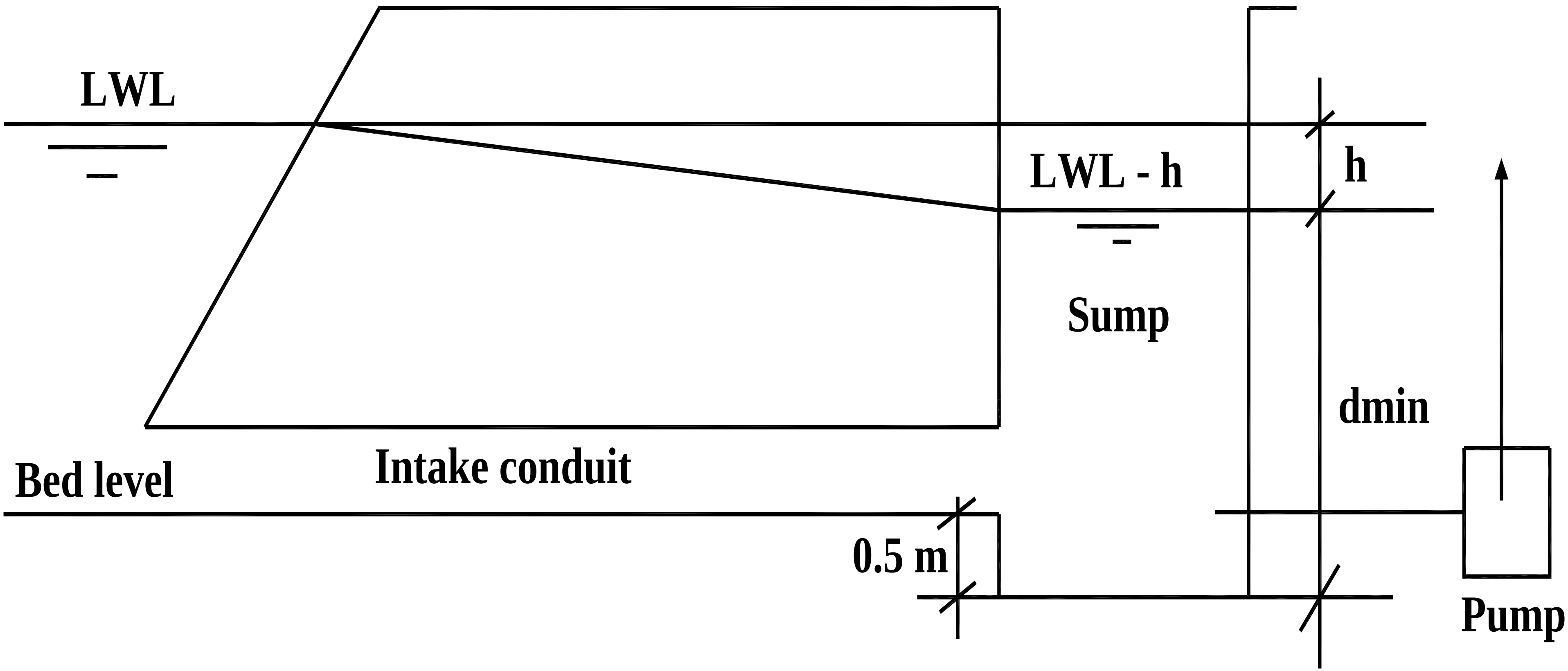
DESIGN OF SUMP

- Volume (m^3) = L * S * d
- Volume = bigger of (Q_{mm} * 2min) or (Q_{min} * 5min)
- S = 1 – 3 m
- L = total number of pumps * (1.5 – 3 m)
- d = Volume / (L*S) $\geq d_{min}$

where:

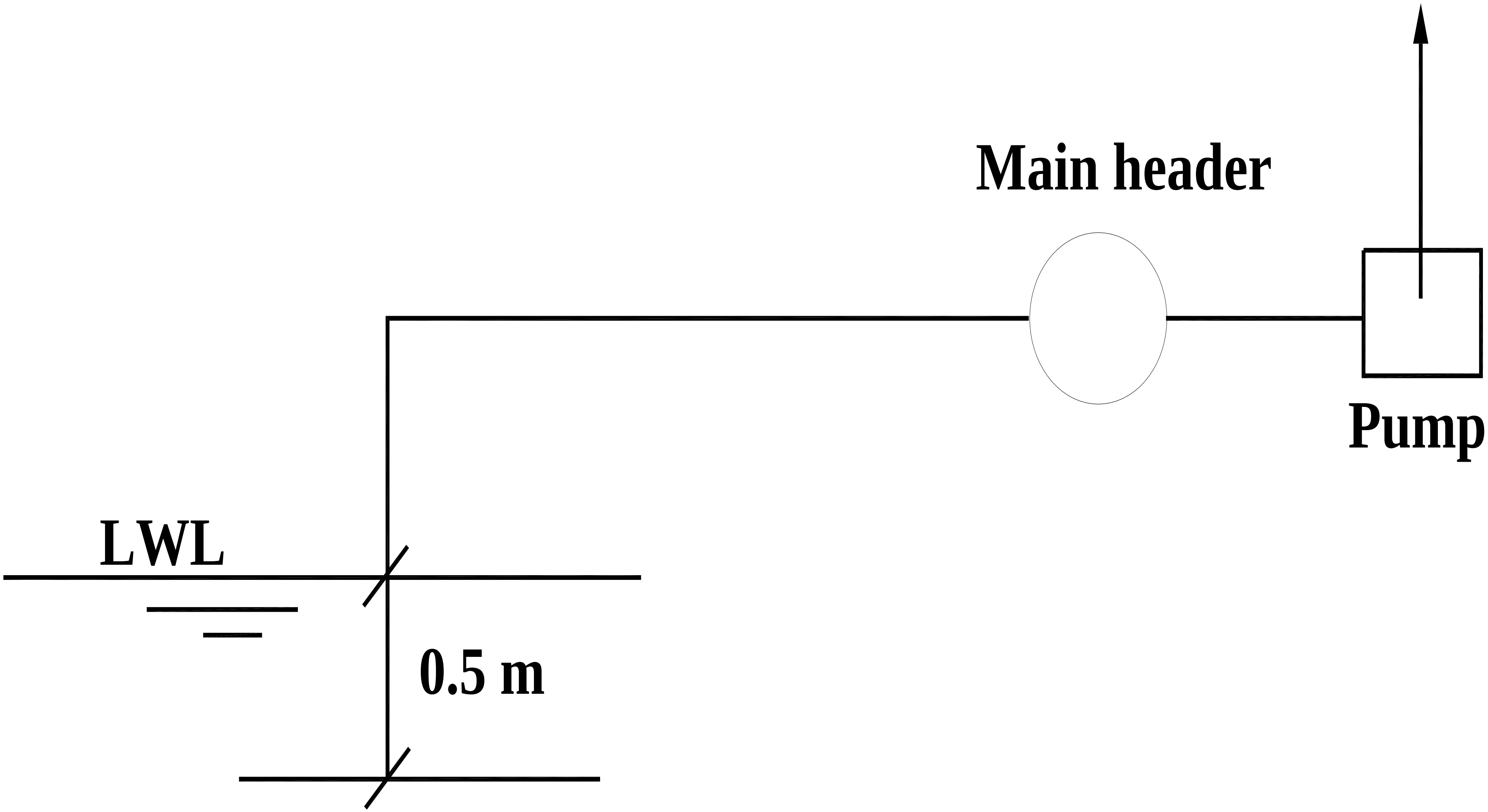
- d_{min} = (Low water level in the water source – main and minor losses through the intake conduit) – (Bed level in the source – 0.5m), taken into consideration that the intake conduits rested horizontally.

- Sump is designed for future only, and in present it is divided by separators from metal.


LWL
LWL - h
h
Sump
dmin
Bed level
Intake conduit
0.5 m
Pump

DESIGN OF MAIN HEADER

- Volume (m^3) = $\pi/4 * \Phi^2 * L$
- Volume = bigger of (Q_{mm} * 1min) or (Q_{min} * 3min)
- L = total number of pumps * (1.5 – 3 m)
- Get Φ.
- Main header is designed for future only.


Main header
Pump
LWL
0.5 m

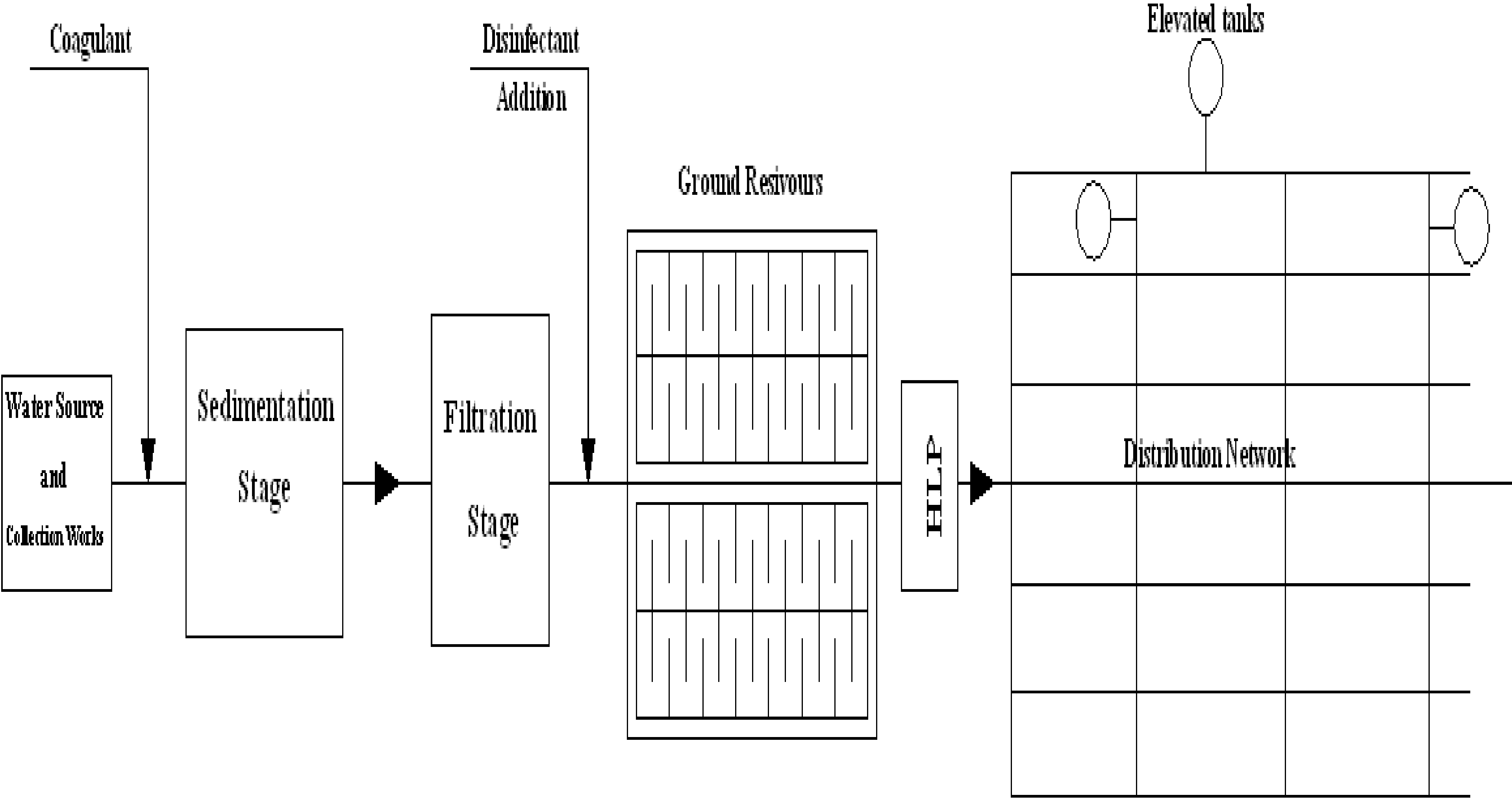
WATER PURIFICATION WORKS

Water treatment plants typically purify water by passing it through several processes starting by coagulation, sedimentation or flotation, filtration and disinfection.

The presence of these processes depended on the water source as soon as the types of impurities appear.

PURPOSES OF WATER PURIFICATION WORKS

1. Improve the physical characteristics of water, by removing turbidity, color and taste.
2. Destroy any contained bacteria, special pathogenic bacteria.
3. Removal of hardness, iron and manganese salts and excessive amount of gases and salts soluble in water.



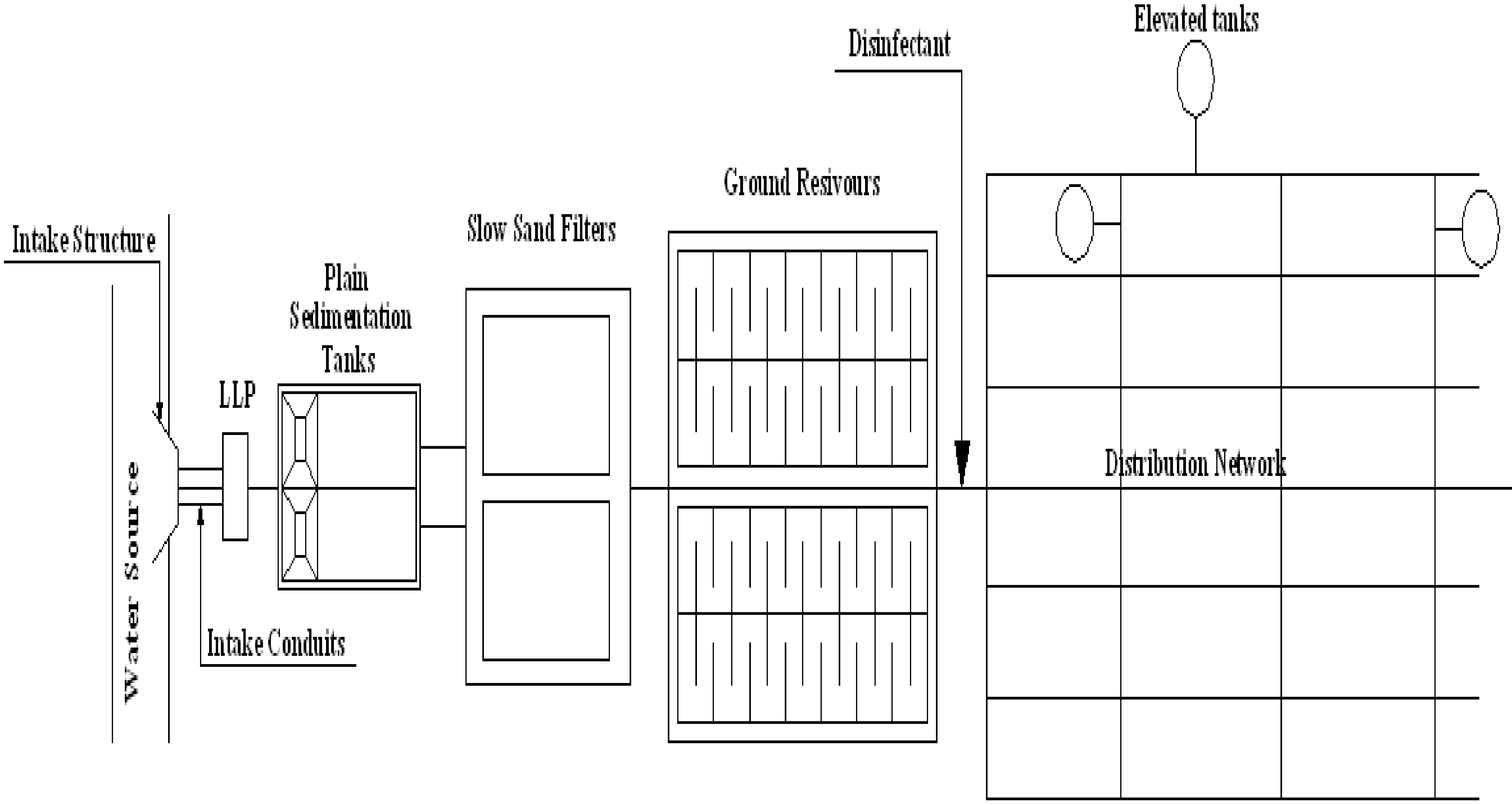
Flow Diagram in Typical Water Supply Works

In most surface water, two systems of water purification are in common use:

1.The slow sand filter plant – which consists of plain sedimentation followed by slow sand filtration and disinfection.

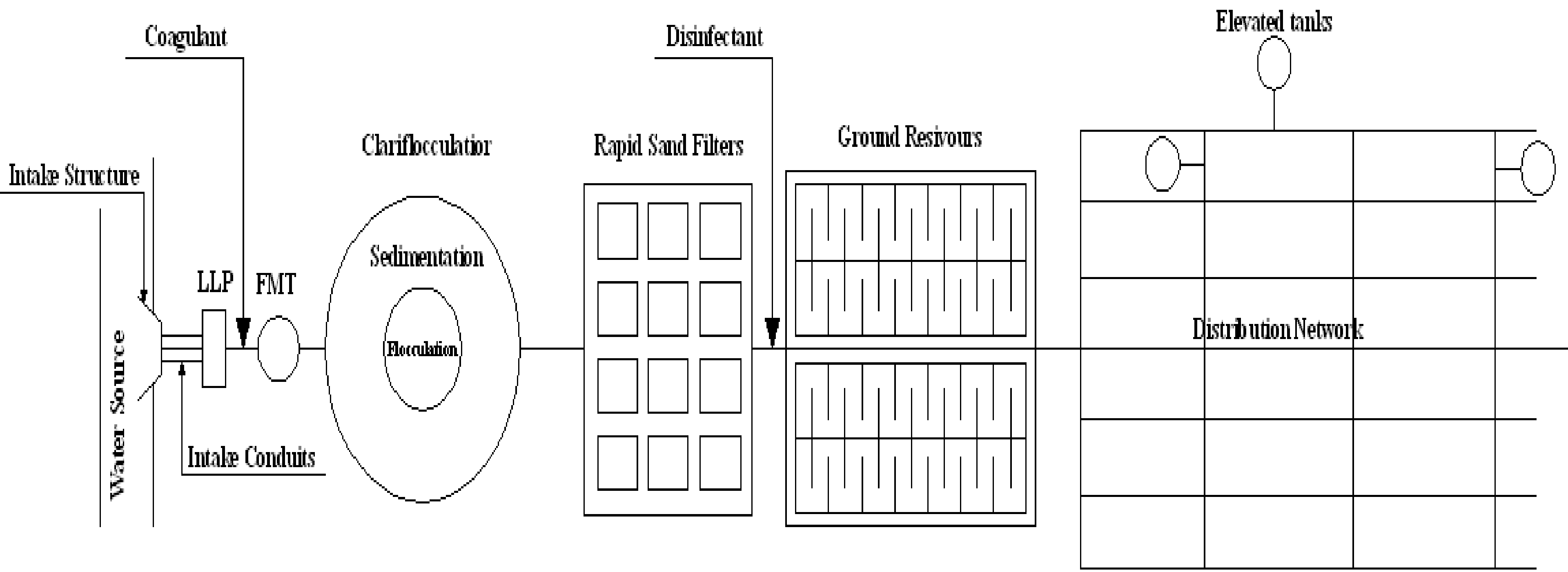
2. **The rapid sand filter plant** – which consists of sedimentation with coagulation followed by rapid sand filtration and disinfection.

Ground water sometimes need soften, manganese and iron removal and rarely disinfection.



Flow Diagram in SSF WTP

Flow diagram in RSF plant uses Clariflocculator as a chemical precipitation unit



Flow Diagram in RSF WTP

1.SEDIMENTAION STAGE

Different types of sedimentation tank according to operation technique:

1.Fill and draw (Batch System)

In this type, the raw water stays a sedimentation period inside a sedimentation basin.

2.Continues flow

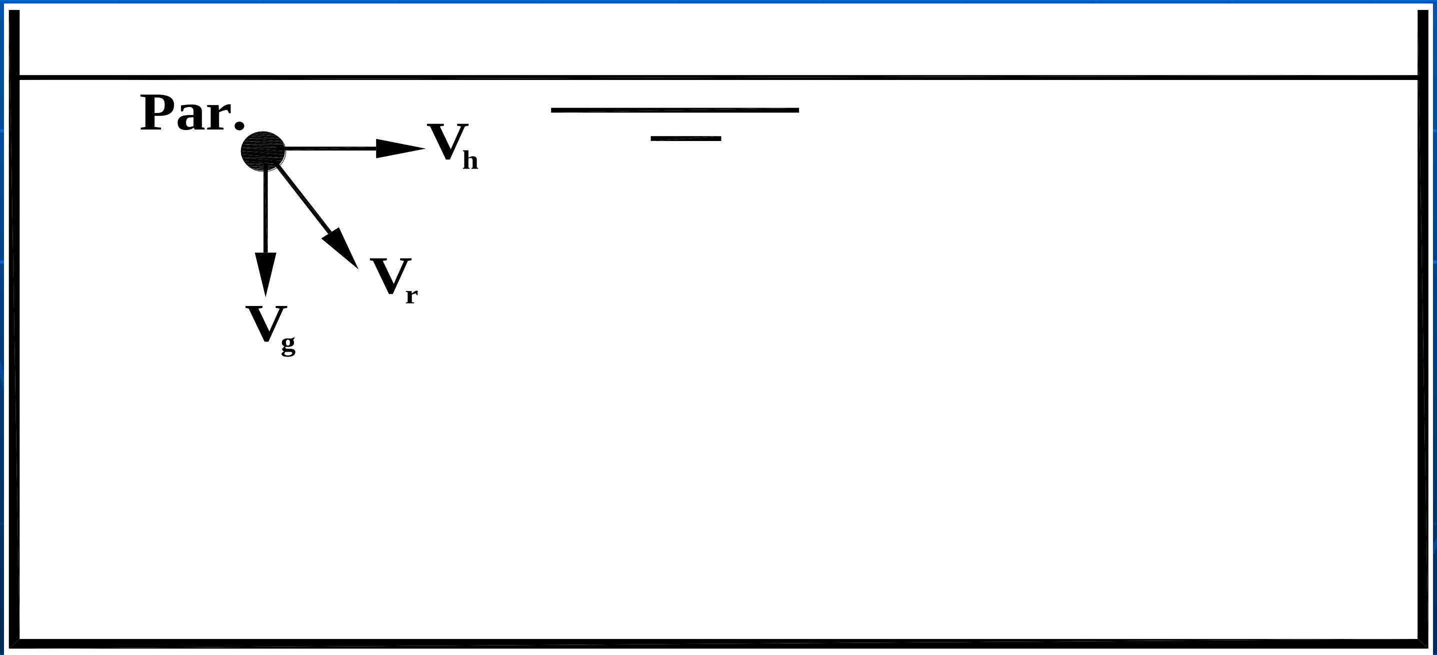
The flow inter the sedimentation basin from inlet arrangement, and in the same time exit from outlet arrangement, the retention time in the basin is the required sedimentation time.

Theory of sedimentation in ideal sedimentation tanks

The conditions must be done to be an ideal sedimentation tank:

1.The flow is laminar flow.
2.There are no dead zones.
3.The horizontal velocity is constant.
4.Good arrangement of inlet and outlet weirs.

The solids that have specific gravity more than water will be settled in a sedimentation zone if the flow velocity is reduced. The ideal sedimentation tank consists of (inlet zone – outlet zone – settlement zone – sludge zone).



Ideal Sedimentation Tank

Factors affect the sedimentation efficiency:

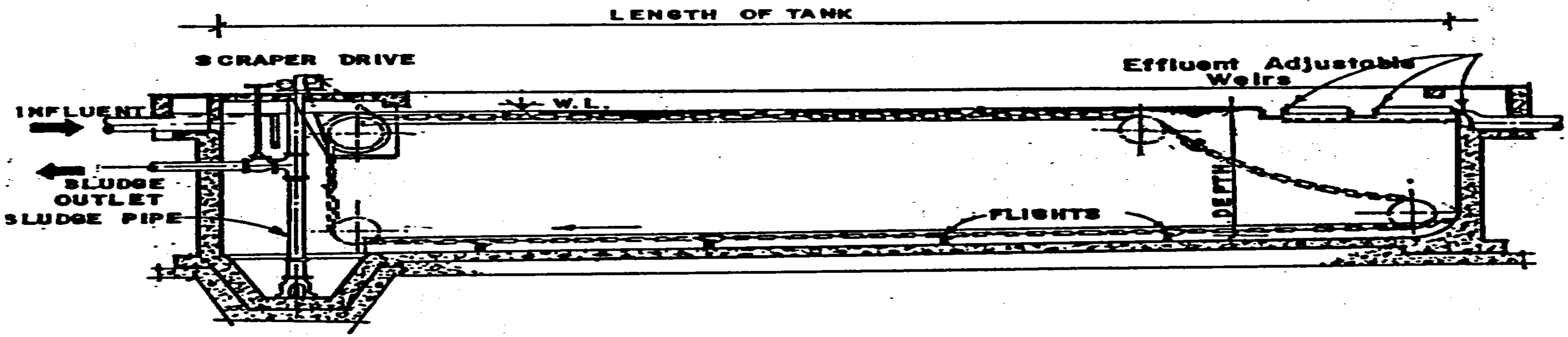
1. Shape and size of solids.
2. Specific gravity of suspended solids.
3. Suspended solids concentration in water to be treated.
4. Temperature of water to be treated.
5. Velocity of water to be treated.
6. Specific gravity of water to be treated.
7. Viscosity of water to be treated.
8. Retention time.
9. Relationship between tank dimensions.
10.Efficiency of pervious treatment.
11.Sludge removal method.
12.Surface loading rate.
13.Hydraulic load on out let weir.
14.Inlet and outlet arrangement.

PLAIN SEDIMENTATION PROCESS

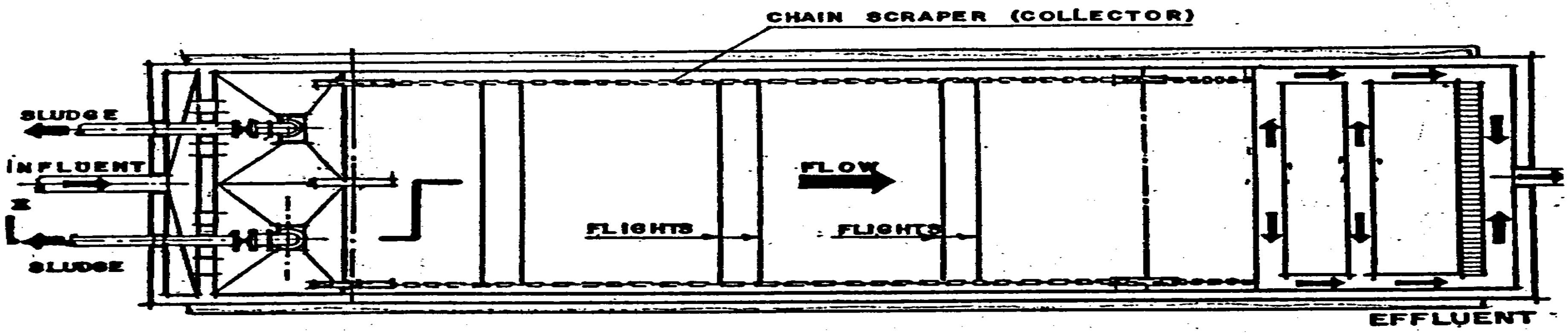
Design of Plain Sedimentation Tanks

- Removal Ratio = 60 -85 %
- $Q_d = Q_{mm} = 1.4\ Q_{av}\ (m^3/d)$,
- Get $Q_d\ (m^3/hr) = Q_d\ (m^3/d)/w.p\ (hr/d)$
- Retention Time = 4 – 6 hrs
- SLR = 25 – 40 $m^3/m^2/d = Q_d / S.A$
- HLOW = 150 – 300 $m^3/m/d = Q_d / L_w$
- For rectangular only $V_f \leq 0.3$ m/min $= Q_d / X.A$
- $n \geq 2$

Rectangular Sed. Tanks	*Circular Sed. Tanks (Clarifiers)*
$d = 3 – 5$ m	$d = 3 – 5$ m
$B = 2 – 4\ d$	$Ø \leq 35$ m
$L = 4 – 5\ B \leq 50$ m	$n \geq 2$
$n \geq 2$	Volume $= n\ \square/4\ Ø^2 * d$
Volume $= nLBd$	$S.A = n\ \square/4\ Ø^2$
$S.A = nLB$	$L_w = n\Pi Ø$
$X.A = nBd$	
$L_w = nB$	
$= n\ (B + 2\ L/7 – 2m)$	
$= n\ (2B + 2\ L/7 – 4m)$	
$= n\ (3B + 2\ L/7 – 4m)$	
$= n\ (4B + 2\ L/7 – 7m)$	
$= n\ (5B + 2\ L/7 – 7m)$	

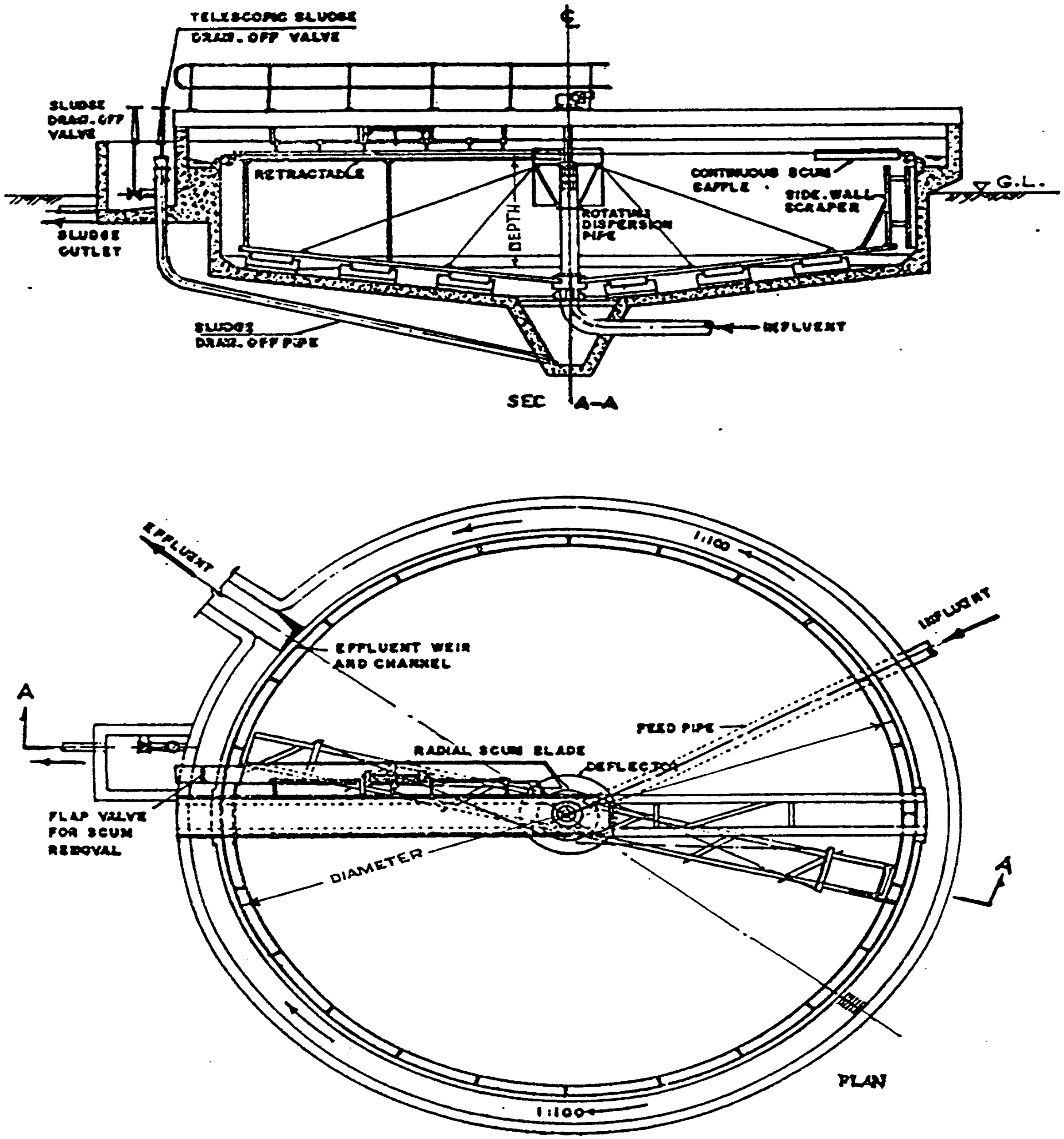


SECTION ELEVATION X — X



PLAN

Fig.2/4 RECTANGULAR SEDIMENTATION TANK



PRIMARY SEDIMENTATION TANK (CIRCULAR TANK)

Sludge Amount

Sludge = Water + [2 to 5 % Solids]

- Assume SS inlet the sedimentation tank = X mg/l
- RR is the Removal Ratio in the sedimentation process = 60 – 85 %
- Amount of SS in sludge per day = Y (t/d) = Q_d (m^3/d)*[X (g/m^3) / 10^6 (g/t)]*RR
- Assume concentration of SS in Sludge = C % = 2 – 5 %
- Assume Specific Gravity of Sludge = SG = 1.05 – 2.1
- Amount of Sludge per day (t/d) = Z (t/d) = Y (t/d) / C %
- Volume of Sludge per day (m^3/d) = V (m^3/d) = [Z (t/d) / SG * 1(t/m^3)]

Example

Design the plain rectangular sedimentation tanks for a water purification plant of an hourly output 5000 m^3. Then get the amount of sludge if the SS in raw water is 80 ppm and sludge specific gravity is 1.1.

Solution

(First) Design of plain sedimentation tanks

Assume that wp is 24 hr/day

Q_d = 5000 m3/hr

Assume RT = 4 hrs (from 4 – 6 hrs)

Minimum Total Volume = 4* 5000 = 20,000 m^3 = nLBd

Assume SLR = 30 $m^3/m^2/d$ (from 25 – 40 $m^3/m^2/d$) = 1.25 $m^3/m^2/hr$

TSA = 5000/1.25 = 4000 m^2 = n LB

d = Volume/TSA = 20,000/4000 = 5.0 m (from 3 – 5 m)

Assume B = 2d = 10 m

Assume L = 5B = 50 m

n = 8 tanks

Actual SA = 50 * 10 = 500 m^2

Actual Volume = 50 * 10 * 5 = 2500 m^3

Actual TSA = 8 * 500 = 4,000 m^2

Actual Total Volume = 8 * 2500 = 20,000 m^3

The important checks

Actual RT = 20,000 / 5000 = 4 hr (ok)

Actual SLR = (5000 / 4000) * 24 = 30 $m^3/m^2/d$ (ok)

V_f = (5000/ 400*60) = 0.21 m/min (ok)

HLOW = (5000/80) * 24 = 1500 $m^3/m/d$ (unsafe)

Take L_w = 178 m, HLOW = 690 $m^3/m/d$ (unsafe)

Take L_w = 242 m, HLOW = 495.7 $m^3/m/d$ (unsafe)

Take L_w = 322 m, HLOW = 372 $m^3/m/d$ (unsafe)

Take L_w = 378 m, HLOW = 317 $m^3/m/d$ (unsafe)

Take L_w = 458 m, HLOW = 262 $m^3/m/d$ (ok)

Sludge Amount

SS = 80 mg/l

Assume RR is Removal Ratio in plain sedimentation process = 80 %

Amount of SS in sludge per day = Y (t/d) = 120,000 (m^3/d)*[80 (g/m^3) / 106 (g/t)] * 0.8 = 7.68 ton/day

Assume concentration of SS in Sludge = C % = 3 %

Amount of Sludge (Water + SS) per day (t/d) = Z (t/d) = 7.68 (t/d) / 0.03 = 256 t/d

Specific Gravity of Sludge = SG = 1.1

Volume of Sludge per day (m^3/d) = V (m^3/d) = [256 (t/d) / 1.1 * 1(t/m^3)] = 232.73 (m^3/d)

COAGULATION PROCESS

- **Purpose**

Removal of most quantity of solids present in the raw water by chemical action.

- **Process**

Addition of a chemical matter (coagulant) to raw water that reacts with water alkalinity and produce a gelatinous forming (flocs.) that carries a positive charge at its surface, in the other side, suspended solids carry a negative charge at their surface. Attraction force appears between them, the suspended solids attaches to the flocs surface that causes increasing of flocs weight. Faster settling appears, sedimentation efficiency will increase.

•Factors affect the coagulation efficiency

1. pH of raw water.
2. Raw water temperature.
3. Mixing.
4. Coagulant type.
5. Feeding method (dry – wet).

- **Types of Coagulants**

1. Alum or [Aluminum sulphate ($AL_2(SO_4)_3 + 18H_2O$)].
2. Sodium Aluminates.
3. Ammonia alum.
4. Ferric chloride
5. Ferrous chloride.

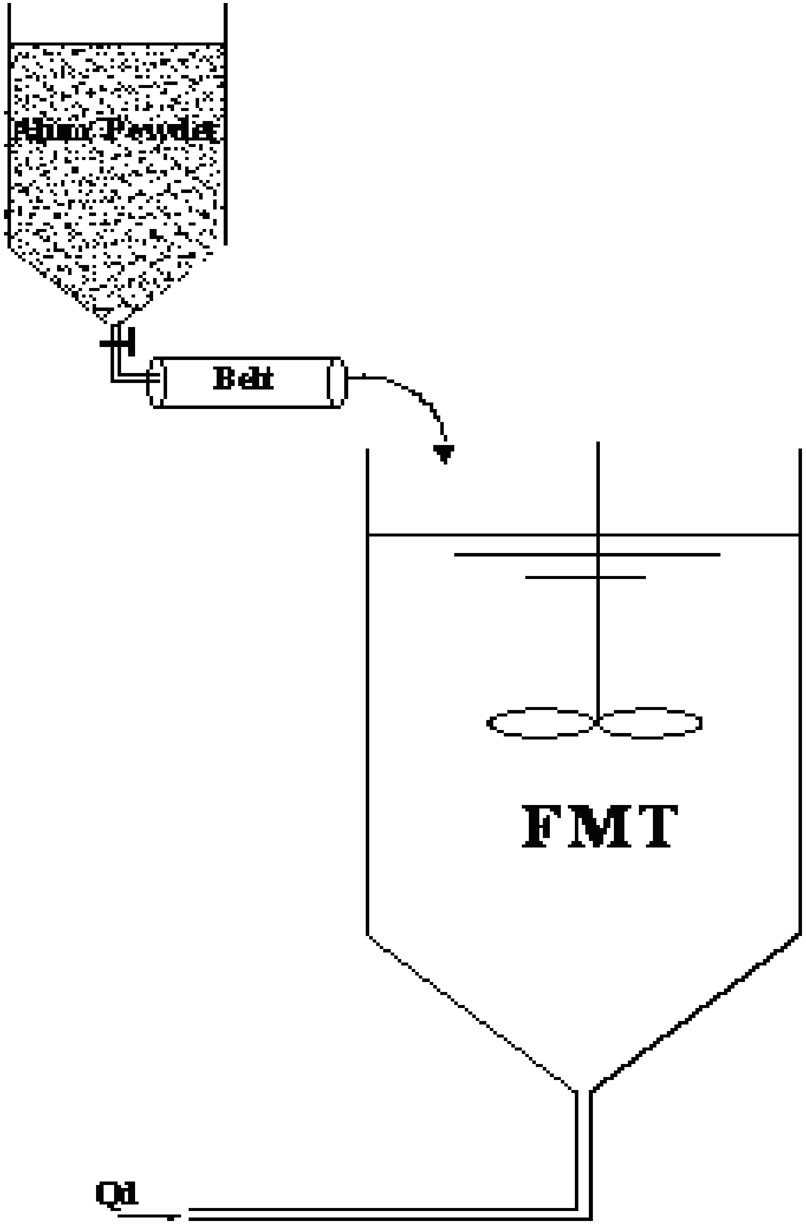
•Methods of mixing alum with raw water

1.Dry feeding

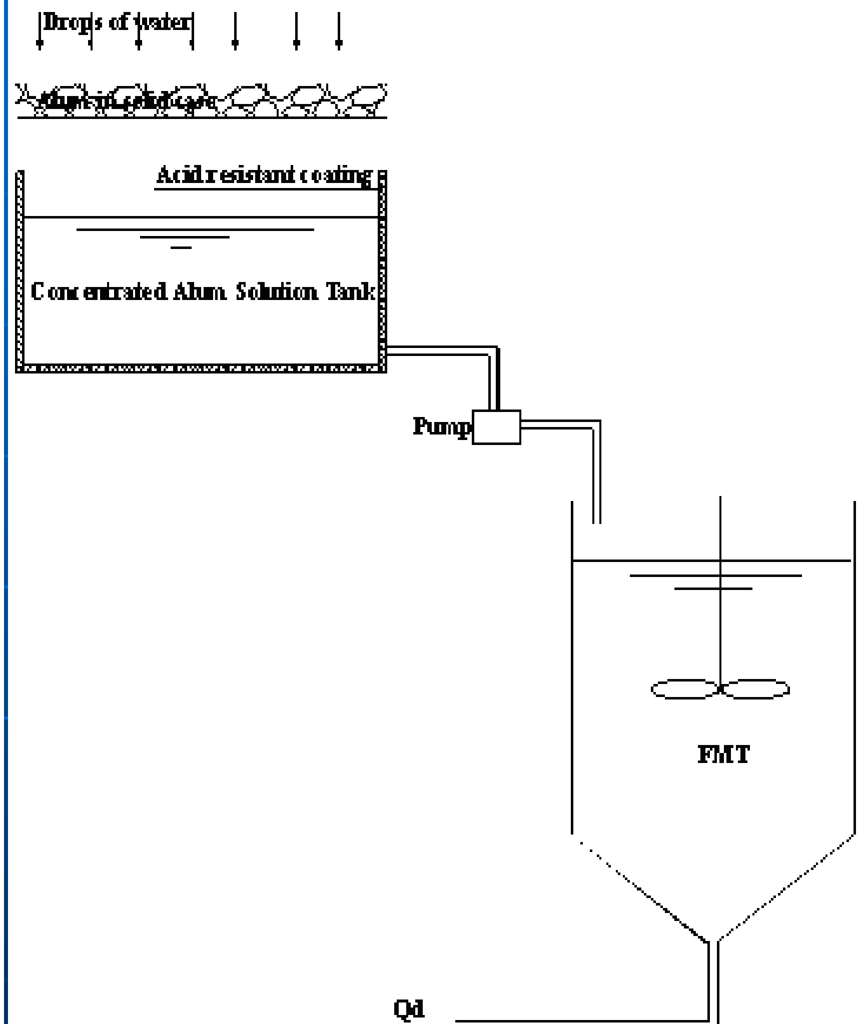
Use the alum as a powder in case of insoluble materials.

2. Wet feeding

Use the alum in liquid form (solution), better than dry feeding, need concentrated alum solution tank to prepare the alum solution.



Dry Feeding of Alum



Wet Feeding of Alum

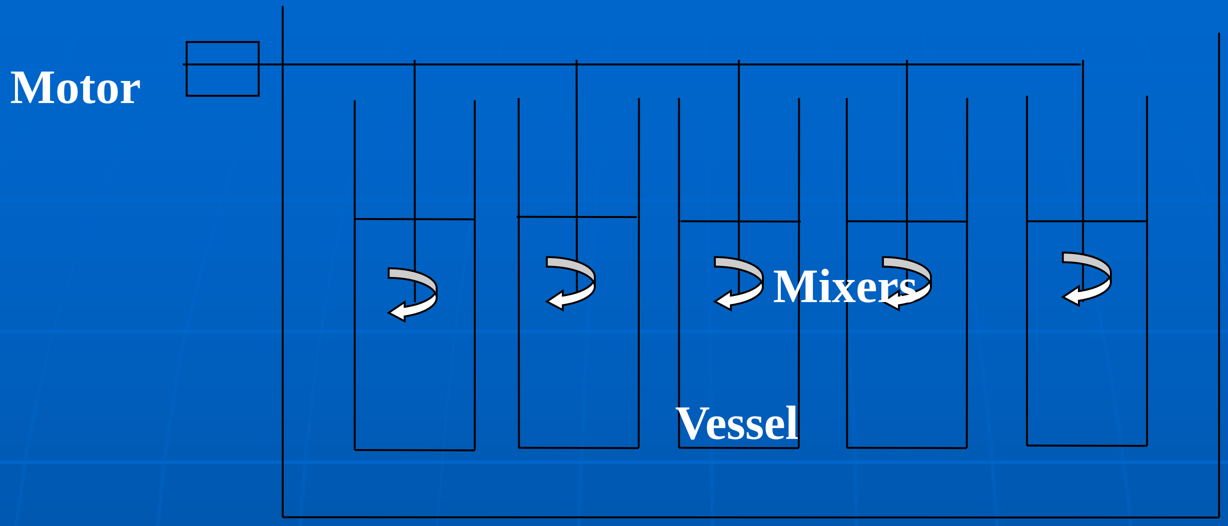
Jar test

Jar test is used to determine the daily coagulant dose.

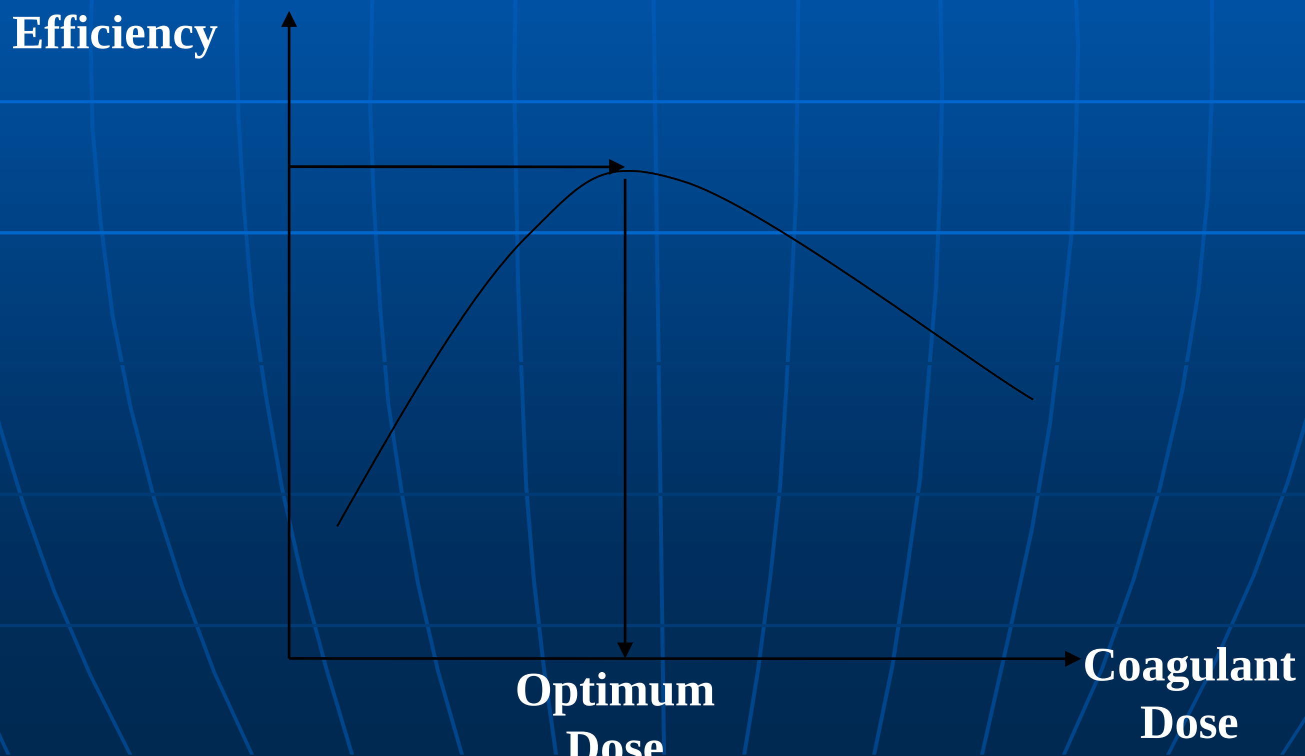
Steps of the test

1. 5 vessels each 1 liter put in them different coagulant doses.
2. Flash mixing for 30 sec. (100 - 300 rpm)
3. Gentle mixing for 10 min. (1 - 3 rpm)
4. Sedimentation for 30 min.

Get removal efficiency for each vessel, draw relationship between coagulant dose and efficiency, and then get from the curve the optimum efficiency.


Motor
Mixers
Vessel

Jar test


Efficiency
Optimum Dose
Coagulant Dose

CHMICAL SEDIMENTATION PROCESS

Design of Alum Solution Tanks

Alum Dose = X mg/l = 20 – 70 ppm
Concentration of alum solution = Z % = 5 – 10 %
n = 3 tanks (Working – under preparing – standby)
May be use 2 tank working at the same time or more
Square in shape
L = B = 1 - 5 m
d = 1 – 3 m
$Q_d = Q_{mm} = 1.4\ Q_{av}\ (m^3/d)$
Amount of alum per day (t/d) = Y (t/d) =$Xg/m^3 * Q_d\ (m^3/d)/10^6 (g/t)$
Amount of alum solution = W (t/d) = Y (t/d) / Z%
Alum solution specific gravity = SG = 1.01 – 1.07
Alum Solution Density = SG * Water density ($1\ t/m^3$) = D ($t/\ m^3$)

Volume of alum solution = W (t/d) / D (t/ m^3) = C (m^3/d)
Volume of alum solution tanks = C (m^3)
Volume of one tank = V (m^3) = C (m^3) / 3
Rate of pumping the alum solution to FMT (liter/hr) = V (m^3)*1000 (l/m^3)/(w.p/3)

Design of Flash Mixing Tank

$Q_d = Q_{mm} = 1.4\ Q_{av}$ (m^3/d)
Retention Time = 5 $\rightarrow$ 60 sec
d = 1.5 – 5 m
Circular
$n \geq 1$
$Ø \leq 35$ m
100 – 300 rpm

Example

Design concentrated alum solution and the flash mixing tanks for a water treatment plant of a daily output 20,000 m^3, if the alum dose is 40 mg/l. What would be the rate of dosing of concentrated alum solution?

Solution

Design of alum solution tanks

Q_d = 20,000 m^3/d
Daily Amount of alum = 20,000 * 40 = 0.8 t/d
Assume alum solution concentration is 10 %
Daily Amount of alum solution = 0.8* 100/10 = 25.2 t/d
Assume specific gravity of alum solution is (1.05)
Daily volume of alum solution = 8 / 1.05 = 7.62 m^3

Total alum solution tanks = 7.62 m^3

Assume alum solution tanks number = 3 tanks

Alum solution tank volume = 7.62 / 3 = 2.5 m^3

Assume d = 1 m

SA = 2.5 m^2

Assume alum solution tank is square in shape (B*B*d)

B = (2.5)0..5 = 1.6 m

Rate of pumping the alum solution to FMT (liter/min) = 7.62 m^3/d = 7,620 l/d = 5.3 l/min.

Design of flash mixing tank

Q_d = 20,000 / 3600 * 24 = 0.23 m^3/sec

Assume RT = 60 sec

V = 60 * 0.23 = 13.89 m^3

Assume d = 2.25m

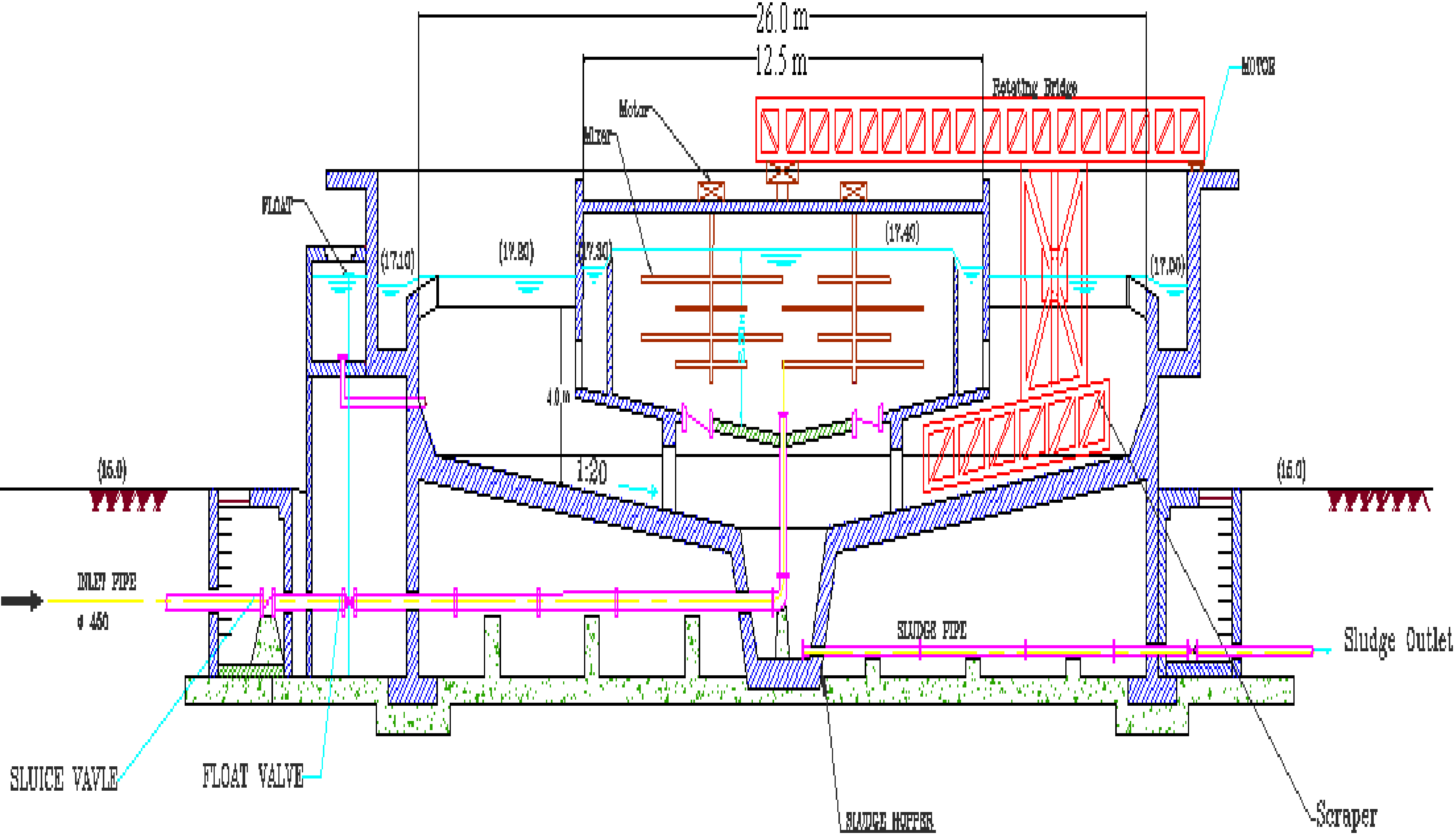
Assume n = 1

SA = 6.25 m^2

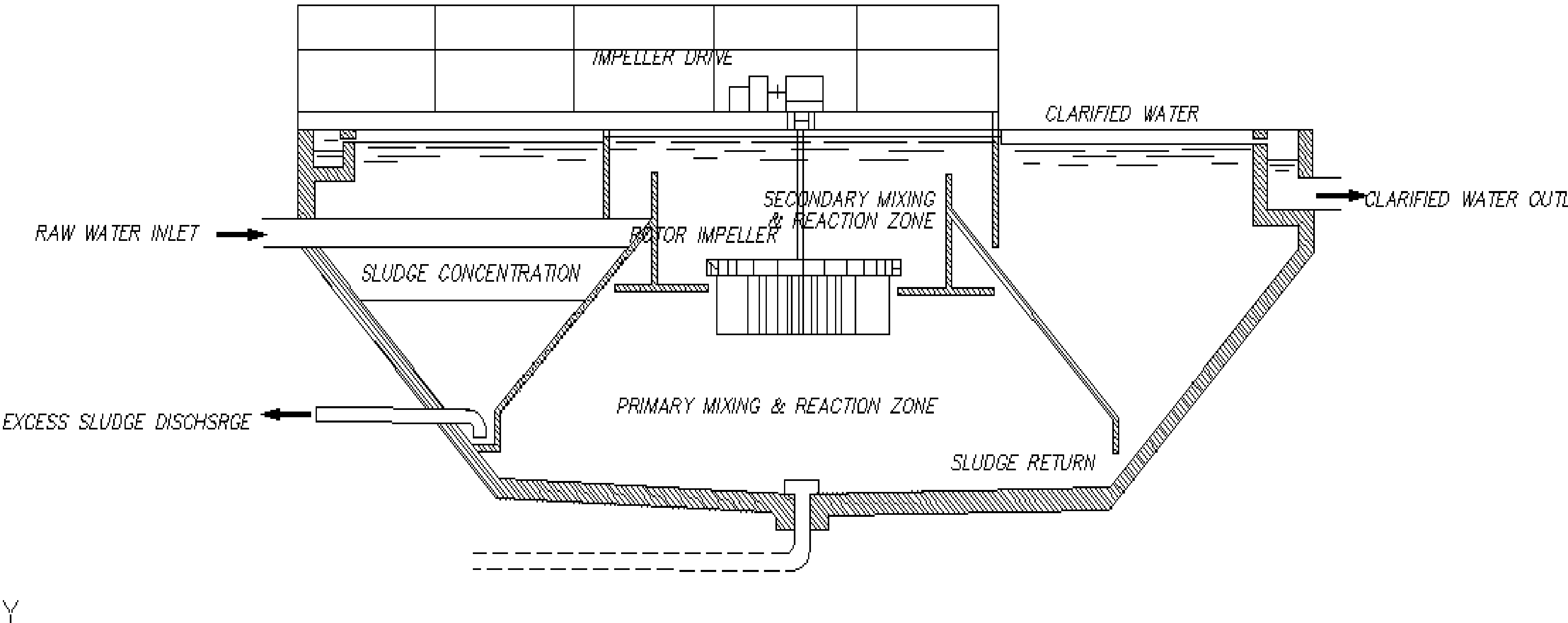
Assume circular Ø = 2.5 m

TYPES OF CHEMICAL SEDIMENTATION UNITS

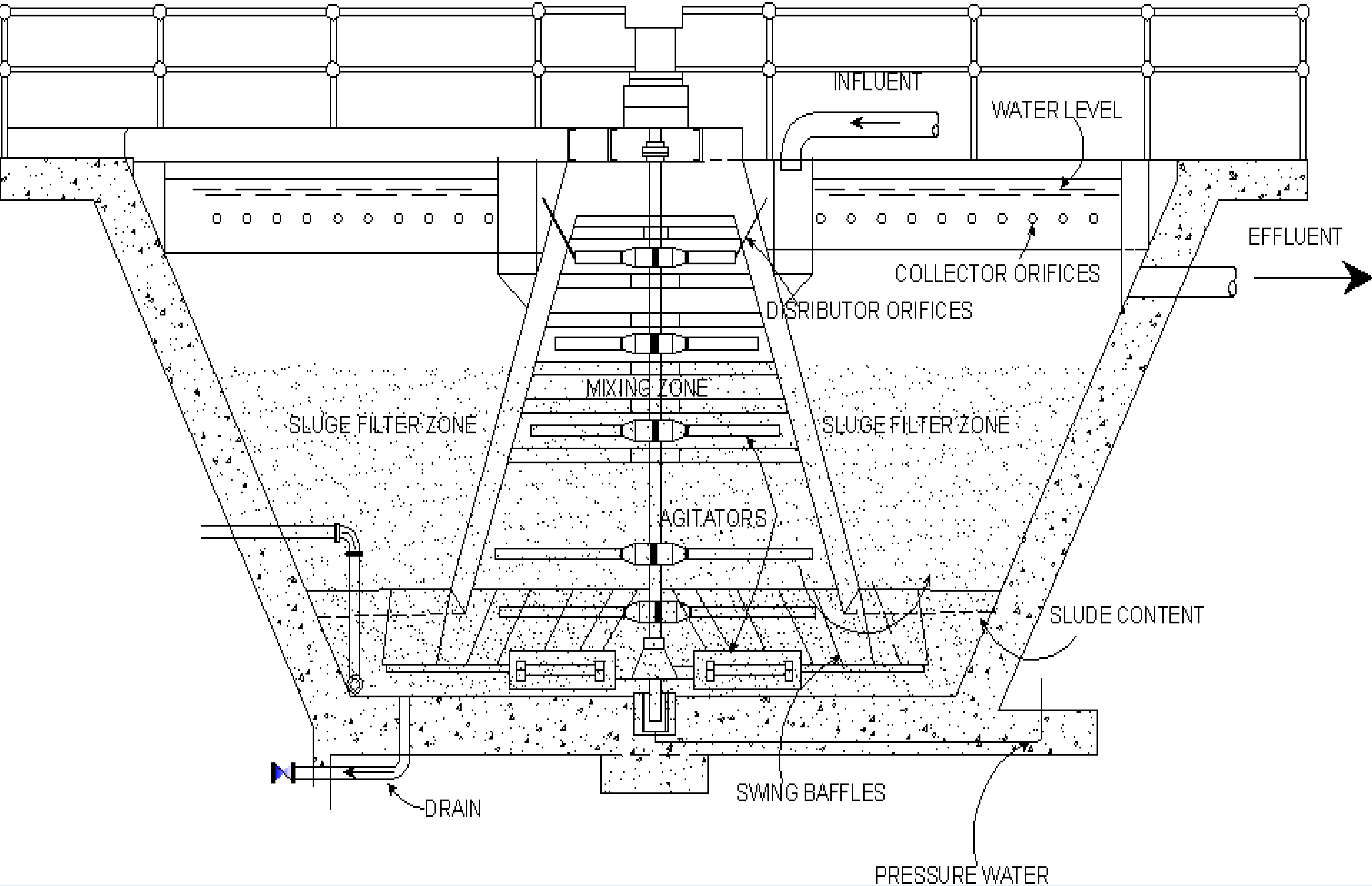
1. Clariflocculator
2. Accelator
3. Precipitator
4. Pulsator
5. Super Pulsator
6. Plate Settler
7. Tube Settler



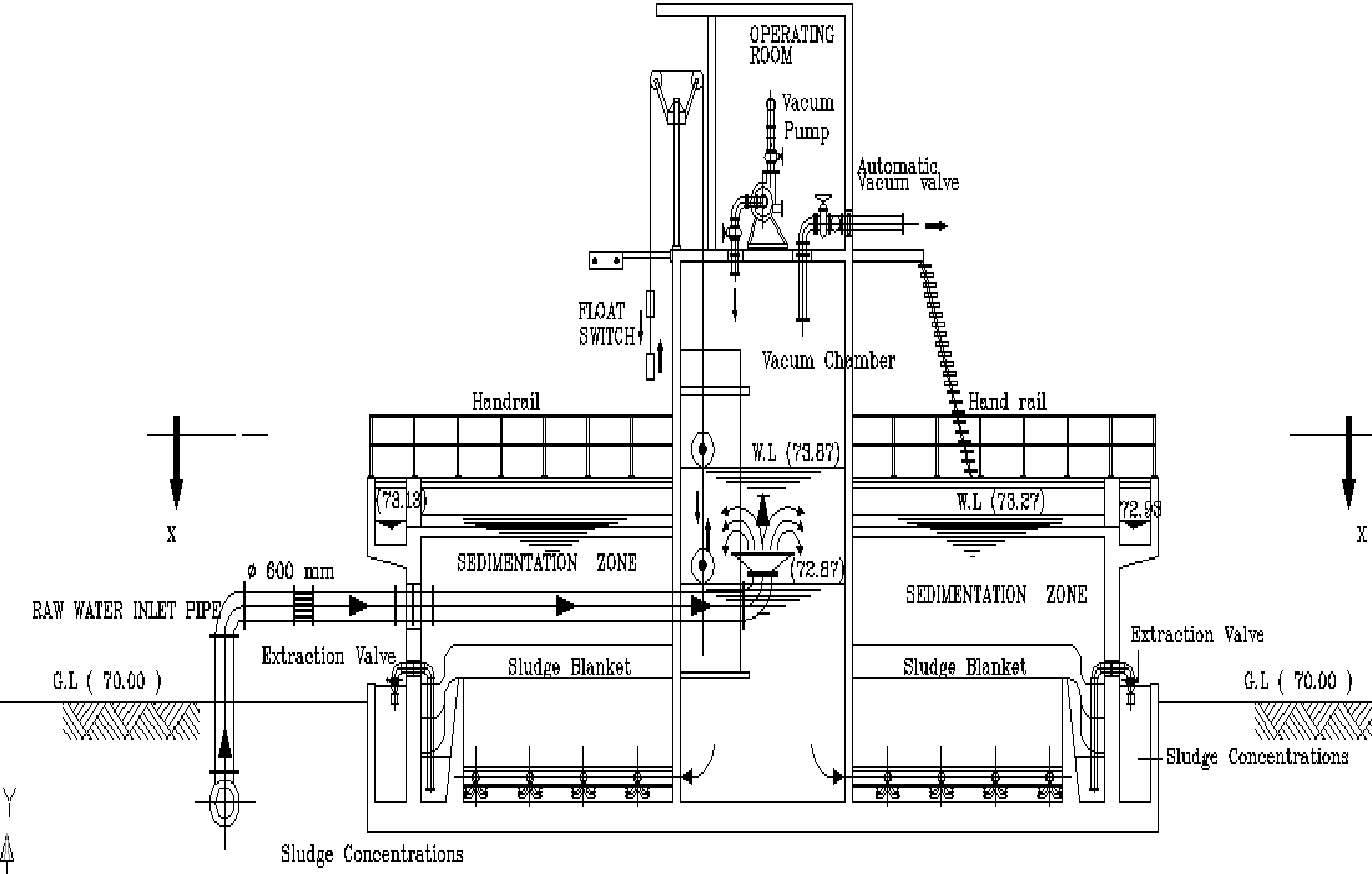
CLARIFLOCCULATOR


IMPELLER DRIVE
CLARIFIED WATER
SECONDARY MIXING
& REACTION ZONE
CLARIFIED WATER OUTL
RAW WATER INLET
ROTOR IMPELLER
SLUDGE CONCENTRATION
PRIMARY MIXING & REACTION ZONE
EXCESS SLUDGE DISCHSRGE
SLUDGE RETURN

Accelator

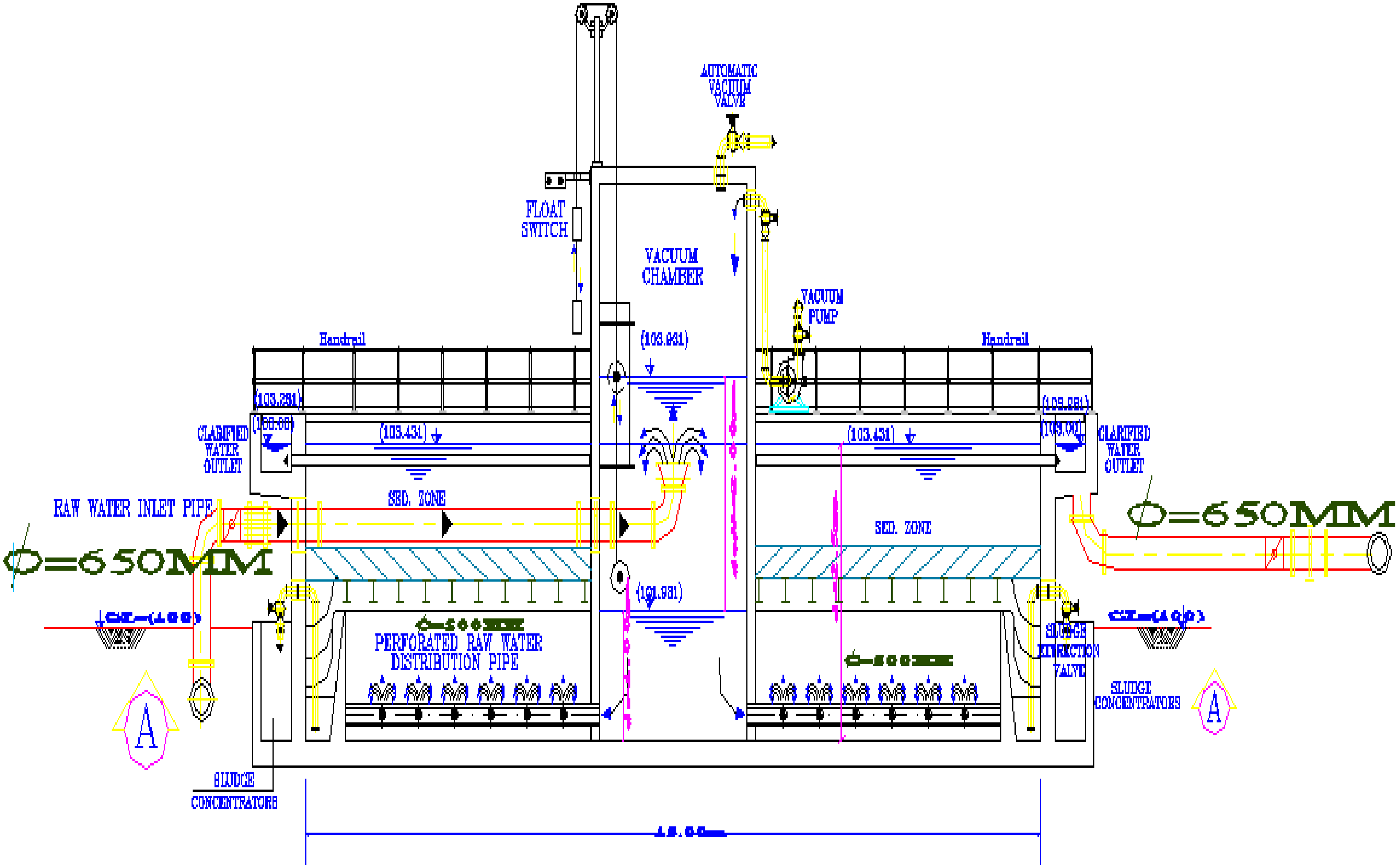


PRECIPITATOR



OPERATING ROOM
Vacum Pump
Automatic Vacum valve
FLOAT SWITCH
Vacum Chamber
Handrail
Hand rail
W.L (73.87)
(73.13)
W.L (73.27)
72.93
X
X
SEDIMENTATION ZONE
SEDIMENTATION ZONE
ø 600 mm
(72.87)
RAW WATER INLET PIPE
Extraction Valve
Extraction Valve
Sludge Blanket
Sludge Blanket
G.L (70.00)
G.L (70.00)
Sludge Concentrations
Sludge Concentrations
Y

PULSATOR



SUPER PULSATOR

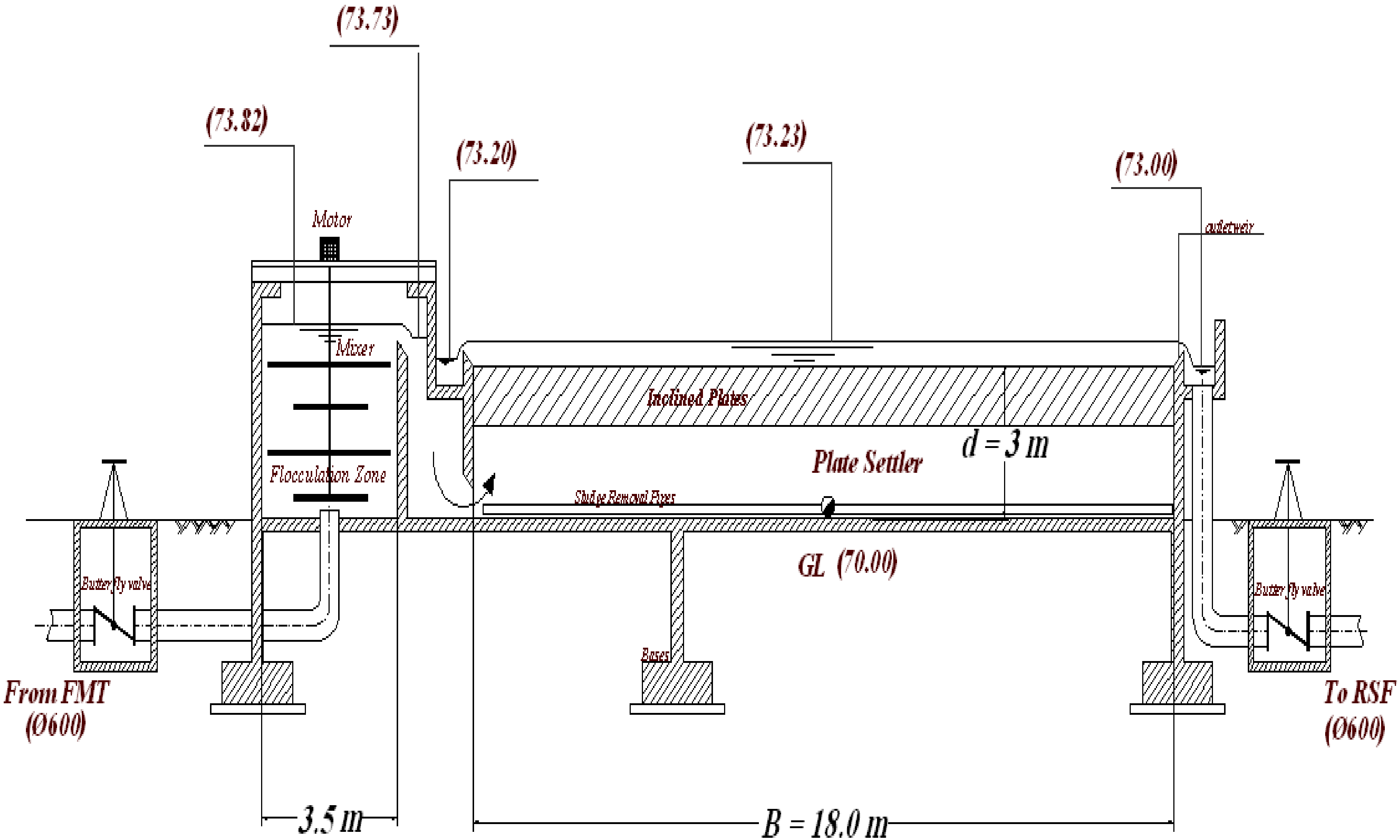


PLATE SETTLER

Design criteria of chemical precipitation units

Unit	T_i	T_o	d_i	d_o	SLR ($m^3/m^2/d$)
Clariflocculator	(1/3 to 1/2) hr	(2 to 4) hr	do – (0.5 to 1) m	(3 to 5)m	(25to40)
Accelator	(10 to 20) min	(1 to 2) hr	(0.5 to 1) m	(2 to 4)m	(40to60)
Precipitator	(10 to 25) min	(1 to 2.5) hr	do – (0.5 -1) m	(2 to 4)m	(60to100)
Pulsator	(2 to 5) min	(45 to 75) min	(0.5 to 1) m	(3 to 6)m	(60 to 90)
Plate settler	(25 to 45) min		(3 to 5) m		(100to150)
Tube settler	(10 to 30) min		(3 to 5) m		(120to180)

Example

The daily design flow of a WTP is 500,000 m^3; design the chemical precipitation units once as Clariflocculator or Accelator or Precipitator or Pulsator or Plate settler or Tube settler.

Solution

Assume wp is 24 hr/d

Q_d = (500,000 / 24) = 20,833.33 m^3/hr

1. Design of Clariflocculator

Inner Chamber	Outer Chamber
$T_i = (1/2)$ hr	$T_o = (1/2)$hr + 3 hr = 3.5 hrs
$V_i = T_i * Q_d = n (\pi/4) * Ø_i^2$ di = 10416.67 m^3	$V_o = T_o * Q_d = n (\pi/4) * Ø_o^2 d_o$ = 72916.66 m^3
$d_i = d_o -$ (0.5 m)= 2.5 m	d_o = (3.0) m
I.SA = $V_i / d_i = n (\pi/4) * Ø_i^2$ = 4166.67 m^2	T.SA = $V_o / d_o = n (\pi/4) * Ø_o^2$ = 24305.55 m^2
From n and I.S.A. Get $Ø_i$ = 14.5 m	Assume $Ø_o$ = Max. = 35 m
	Get min. n = 25
	Get Actual $Ø_o$ = 35 m

The Important checks

1.$Ø_i / Ø_o$ = 0.41

2.SLR = [Q_d (m^3/hr) / Outer S.A] * 24 = 25 to 40 m^3/m^2/day, where outer SA = n (π/4) ($Ø_o^2 - Ø_i^2$) = 19914.167 m^2, SLR = 25.1 m^3/m^2/d (ok)

3.HLOW = [Q_d (m^3/hr) / n π $Ø_o$] * 24 = 182 m^3/m/d (ok)

2 .Design of Accelator

Inner Chamber	**Outer Chamber**
$T_i = (10/60)$ hr	$T_o = (10/60)$hr + 1.5 hr = 1.67 hrs
$V_i = T_i * Q_d = n (\pi/4) * Ø_i^2$ di = 3472.22 m^3	$V_o = T_o * Q_d = n (\pi/4) * Ø_o^2 d_o$ = 34,791.66 m^3
$d_i = 1$ m	$d_o = (3.0)$ m
I.S.A = $V_i / d_i = n (\pi/4) * Ø_i^2$ = 3472.22 m^2	T.SA = $V_o / d_o = n (\pi/4) * Ø_o^2$ = 11,597.22 m^2
From n and I.SA. Get $Ø_i = 19$ m	Assume $Ø_o$ = Max. = 35 m
	Get min. n = 12
	Get Actual $Ø_o = 35$ m

The Important checks

1.SLR = [Q_d (m^3/hr) / Outer S.A] * 24 = 40 to 60 m^3/m^2/day, where outer SA = n (π/4) ($Ø_o^2$ - $Ø_i^2$) = 8138.88 m^2, SLR = 60 m^3/m^2/d (ok)

3.Design of Precipitator

Inner Chamber	**Outer Chamber**
T_i = (10/60) hr	T_o = (10/60)hr + (90/60) hr = 1.66 hrs
$V_i = T_i * Q_d = n\,(\pi/4) * Ø_i^2$ di = 3,472.22 m^3	$V_o = T_o * Q_d = n\,(\pi/4) * Ø_o^2\, d_o$ = 34,583.33 m^3
$d_i = d_o$ – (0.5 m)= 2.5 m	d_o = (3.0) m
I.S.A = $V_i / d_i = n\,(\pi/4) * Ø_i^2$ = 1388.89 m^2	T.SA = $V_o / d_o = n\,(\pi/4) * Ø_o^2$ = 11,527.77 m^2
From n and I.SA Get $Ø_i$ = 12 m	Assume $Ø_o$ = Max. = 35 m
	Get min. n = 12
	Get Actual $Ø_o$ = 35 m

The Important checks

1.SLR = [Q_d (m^3/hr) / outer S.A] * 24 = 60 to 100 m^3/m^2/day, where outer S.A = n (π/4) ($Ø_o^2$ - $Ø_i^2$) = 10183.02 m^2, SLR = 50 m^3/m^2/d (safe but waste)

4. Design of Pulsator

Inner Chamber	**Outer Chamber**
T_i = (2/60) hr	T_o = (2/60)hr + (60/60) hr = 1.033 hrs
$V_i = T_i * Q_d = n\ b^2\ di$ = 694.44 m^3	$V_o = T_o * Q_d = n\ B^2 d_o$ = 21520.83 m^3
d_i = 1 m	d_o = (3.0) m
I.S.A = $V_i / d_i = n\ b^2$ = 694.44 m^2	T.S.A = $V_o / d_o = n\ B^2$ = 7173.6 m^2
From n and I.S.A. Get b = 15.25 m	Assume B = Max. = 50 m
	Get min. n = 3
	Get Actual B = 49 m

The Important checks

1.SLR = [Q_d (m^3/hr) / Outer S.A] * 24 = 60 to 90 m^3/m^2/day, where outer SA = n (B^2 - b^2) = 6505.3 m^2, SLR = 76.9 m^3/m^2/d

5 .Design of Super Pulsator

Inner Chamber	**Outer Chamber**
T_i = (1/60) hr	T_o = (2/60)hr + (20/60) hr = 0.366 hrs
$V_i = T_i * Q_d = n\, b^2$ di = 347.22 m^3	$V_o = T_o * Q_d = n\, B^2 d_o$ = 7639 m^3
d_i = 1 m	d_o = (3.0) m
I.S.A = $V_i / d_i = n\, b^2$ = 347.22 m^2	T.S.A = $V_o / d_o = n\, B^2$ = 2546 m^2
From n and I.S.A. Get b = 13 m	Assume B = Max. = 50 m
	Get min. n = 2
	Get Actual B = 36 m

The Important checks

1.SLR = [Q_d (m^3/hr) / Outer S.A] * 24 = 120 to 220 m^3/m^2/day, where outer SA = n (B^2 - b^2) = 2254 m^2, SLR = 220 m^3/m^2/d

6. Design of Plate settler (Rectangular)

Vol. = (90/60) hr * 20833.33 m^3/hr = 18,750 m^3

n LBd = 18,750 m^3

Assume d = 4 m

n LB = 4,687.5 m^2

Assume L = 4 B

Assume L = max. = 50 m

$((50)^2 / 4)$ n = 4,687.5 m^2

n = 8

((L)2 / 4) * 8 = 4,687.5 m^2

L = 48.5 m

B = 12 m

SLR = (20833.33 * 24)/(8*48.5*12) = 107.4 $m^3/m^2/d$ (ok)

7. Design of Tube settler

Vol. = (20/60) hr * 20833.33 m3/hr = 6945 m^3

n LBd = 6945m^3

Assume d = 3 m

n LB = 2315 m^2

Assume L = 4 B

Assume L = max. = 50 m

$((50)^2 / 4)$ n = 2315m^2

n = 4

((L)2 / 4) * 4= 2315m^2

L = 48 m

B = 12 m

SLR = (20833.33 * 24)/(4*48*12) = 217 $m^3/m^2/d$ (ok)

2. FILTRATION STAGE

DEFINITION

Water filtration can be defined as a physical-chemical process for separating suspended and colloidal impurities from water by passing it through a bed of granular material. Water fills the pores of the filter media, and impurities are absorbed on the surface of the grain or trapped in the openings.

PURPOSE

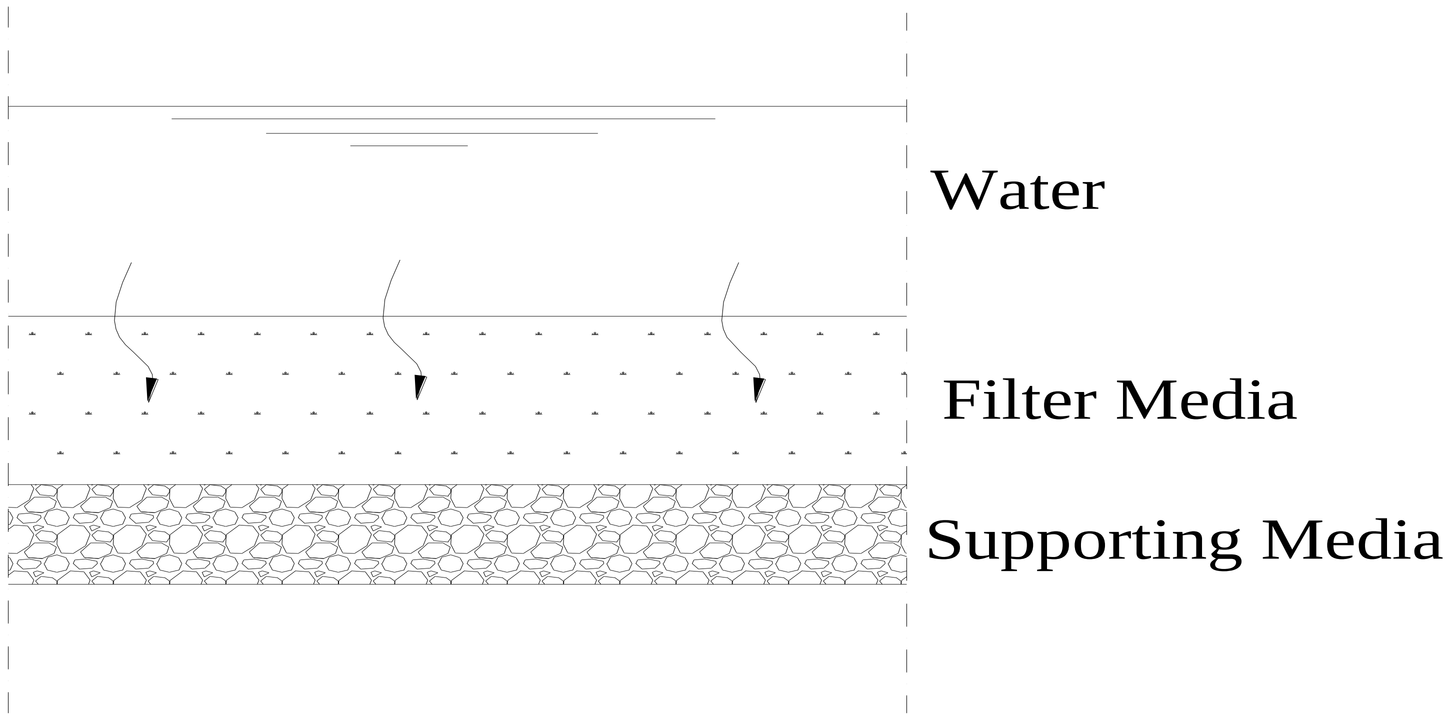
The purposes of filtration in water purification are:

- Removal of the remaining suspended solids and turbidity.
- Removal of iron and manganese salts.
- Removal of taste, odor and color.
- Removal of at least 90% of bacteria.
- Removal of algae.

Also some organic and microorganisms if available in raw water could by removed by filtration.

THEORY OF FILTRATION

Filtration theory depends on passing water through a porous material that removes the undesirable impurities from it.



MECHANSIM OF FILTRATION

The filtration mechanism can be done by the following:

1.Mechanical Straining

Impurities solids bigger size than voids between filter bed particles are arrested on it and removed from water. The major removal takes place in the upper few centimeters of the filter bed. The impurities which deposited on the filter bed surface help in straining the small particles also.

2. Sedimentation Action

Removal of suspended particles between the filter bed particles whose act as sedimentation basins. The suspended particles settle on the sides of filter bed particles.

3. Adsorption Action

Adsorb the colloidal matters on the filter bed particles as a result to coat it by a gelatinous layer from bacteria and microorganisms.

4. Electrolytic Action

The filter bed particles are electrically charged by negative charge opposite to the charged of impurities present in water to be filtrate. Due to that the filter bed particles attract the impurities.

When their charges get neutralized, the washing of filter bed renews the charges.

5. Biological Action

The organic impurities in water like algae, plankton…etc deposit on the filter bed capturing different microorganisms into them. The microorganisms find the source of food on the water particles, this leads to some important biological and chemical change in water quality.

APPLICATIONS OF FILTRATION

In case of high load of suspended solids (more than 50 ppm) in raw water, sedimentation process must be applied to raw water before filtration process to remove most of solids and prevent fast filter clogging.

In the other side, in case of low load of suspended solids (less than 50 ppm) there is no need to make any treatment before filtration and can use filtration as the preliminary treatment step (Direct Filtration).

Direct Filtration may be done with chemicals or without according to the load of turbidity.

CLASSIFICATION OF FILTRATION

The filtration can be classified according to:

1. Rate of filtration

1.1 Slow sand filter which operates at rates 5 to 8 $m^3/m^2/d$

1.2 Rapid sand filter which operate at rates 120 to 200 $m^3/m^2/d$

1.3 High rate filters operate at rates 300 to 900 $m^3/m^2/d$

2. Type of filter media

2.1 Sand

2.2 Carbon

2.3 Activated carbon

3. Number of layers in filter bed

3.1 Single media
3.2 Dual media
3.3 Multi-media

4. Direction of flow

4.1 Down flow
4.2 Up flow
4.3 Horizontal flow

5. Characteristic of flow

5.1 Gravity flow
5.2 Pressure flow

FACTORS AFFECT FILTRATION EFFICIENCY

Filtration is a complex technique which involves a certain number of factors which may have an effect on filtration efficiency

1.Depth of filter media

Increase depth of filter media provide water to take a long pass through it, which improve water quality, but it increases the head loss through the media which reduce operation period (time between two washes) and increase the back washing time.

2. Rate of filtration

The rate of filtration has low effect in quality of filtration, but it greatly affects the operation period (T) that proportion inversely with the rate of filtration (V) as follows:- $(T) \alpha (1/V^{1.5})$

3. The grain size of the sand

For a good filter, sand shouldn't have a coefficient of uniformity greater than 2.0 and preferably to be 1.5. The fine sand is suitable when the pre-treatment is poor, high bacteria and high turbidity removal is required. The coarse sand is suitable when the pre-treatment is good and water to be treated is not highly polluted.

4. Depth of water over filter media

There is inverse relation ship between the depth of water over filter media and filtration efficiency. In case of big water depth over filter media, the water subject during passing through filter media to high pressure which causes escaping of particles through the media voids, and get low filtration efficiency.

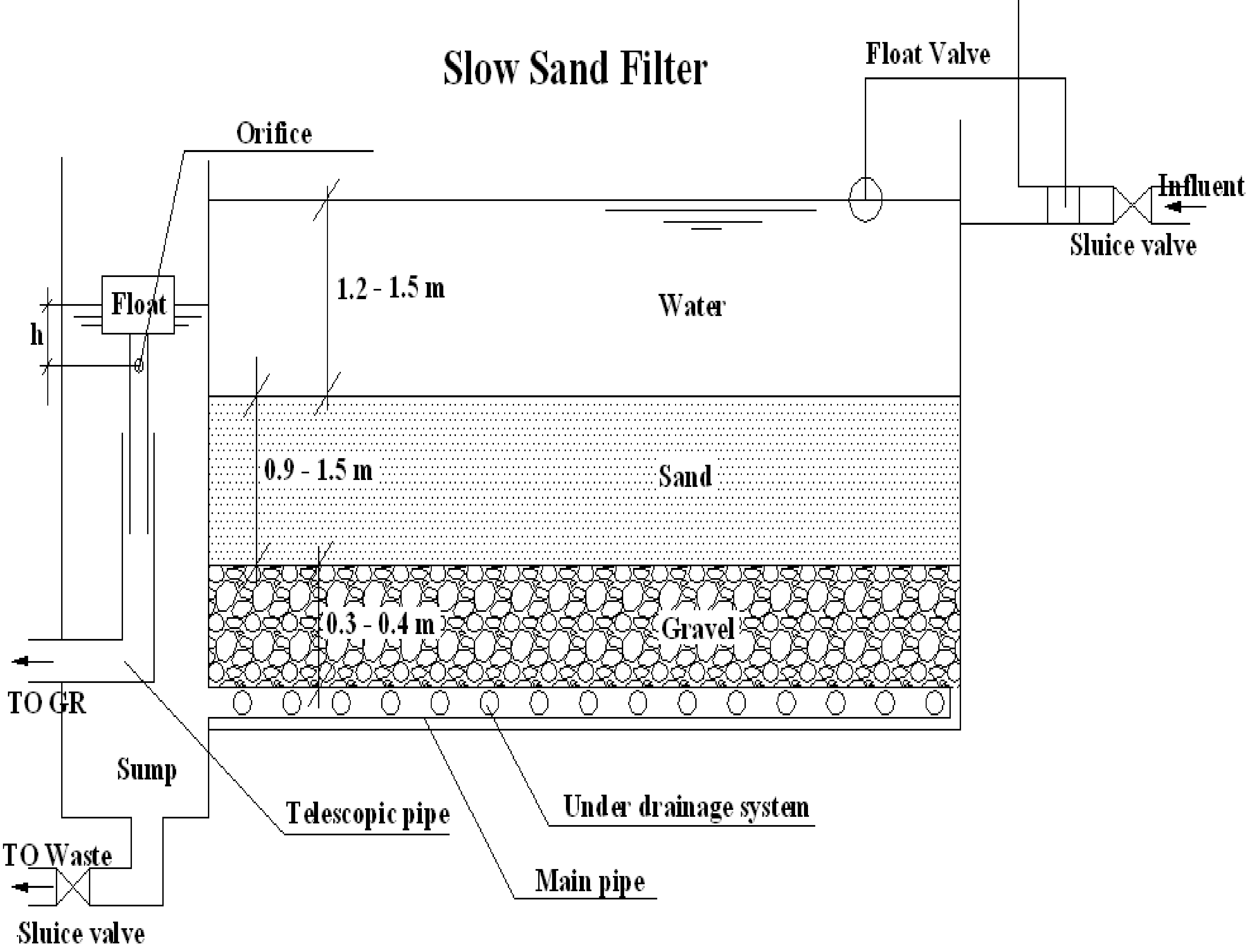
5. The maximum permissible head loss of the filter

When water passes through the filter media and under drainage system, it experiences frictional resistance and therefore head loss occurs, this loss of head can be measured as the difference in level between the inlet and outlet water levels. In case of big head loss occurs, most of the media voids beginning to be locked and the filter are needed to wash the filter

6. The characteristics of water to be filtrated

In case of high load of impurities in the influent water, it must expect that the operating period will be reduced as a result to increase rate of voids clogging.

Slow Sand Filter



Design Criteria of Slow Sand Filter

Water depth above the filter bed = 0.3 – 1.5 m
Filter bed depth (sand depth) = 0.9 – 1.5 m
Gravel bed depth = 0.3 – 0.6 m
Effective size of the sand = 0.25 – 0.35 mm
Dirty skin layer = 1 – 3 inch
Washing time (removal time) = 1 – 15 days (1 day if mechanical & 15 day if manual)
Repairing the filter is taking = 7 – 15 days
The whole cleaning process is taking = 8 – 30 days
The operation time (between two washes) is = 2 – 6 month.
Rate of filtration (ROF) = 3 – 8 m3/m2/d
Area of filter = 1000 – 2500 m2
The filter is Rectangular in surface area (L*B)
L & B $\leq$ 50 m
n $\geq$ 2
L/B = 1 – 1.25

Cleaning procedure in slow sand filters

• يتم كشط الطبقة العلوية (Dirty skin) مع جزء من طبقة الرمل حوالى (2 – 5 سم) عند انسداد المرشح (2 – 6 شهرا)، تستمر فترة التنظيف (1 – 2 أسبوع).

Example

Design the SSF for a WTP working 16 hr/d, if the design flow is 32,000 m^3/d

Solution

$Q_d = 32000\ m^3/d = 32000/16 = 2000\ m^3/h$

Assume that ROF = $6\ m^3/m^2/d = 6/24\ m^3/m^2/hr$

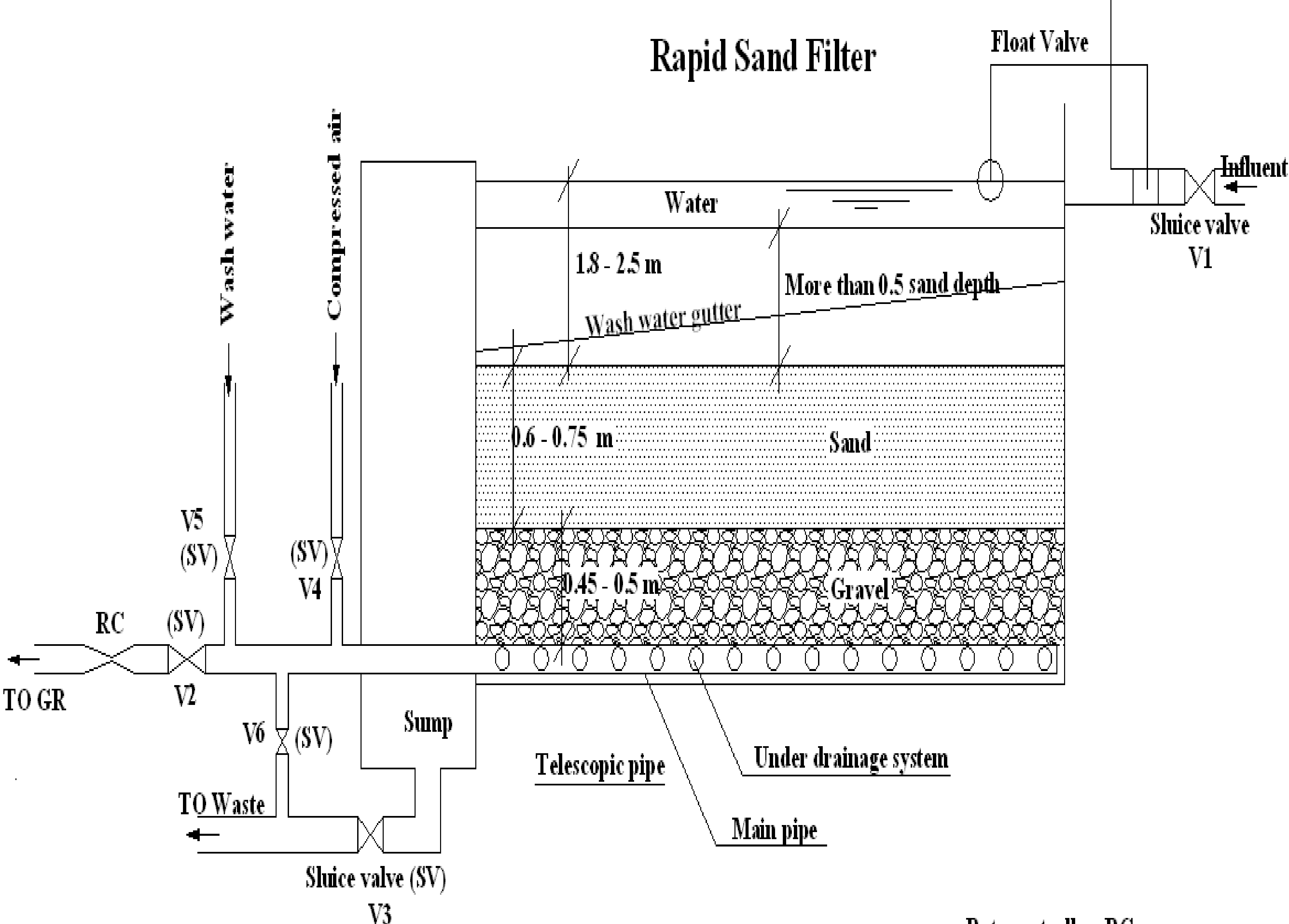
TSA = 2000/(6/24) = 8000 m2

Assume L = 50 m, B = L/1.25 = 40

SA = 50 * 40 = 2000 m^2

n = 8000/2000 = 4 filters (ok)

Rapid Sand Filter



Rate controlle = RC.

Cleaning procedure in rapid sand filters

Step	Open valves	Closed valves
1.	*V3 (empty the filter)*	*V1 & the rest still closed*
2.	*V4 (compressed air)*	*V3 & the rest still closed*
3.	*V5 & V3 (wash water)*	*V4 & the rest still closed*
4.	*V1 & V6 (starting)*	*The rest still closed*
5.	*V1 & V2 (Normal Operation)*	*The rest still closed*

Design Criteria of Rapid Sand Filter

Water depth above the filter bed = 1.5 – 2 m
Filter bed depth (sand depth) = 0.6 – 0.75 m
Gravel bed depth = 0.3 – 0.6 m
Effective size of the sand = 0.6 – 1.5 mm
Sand uniformity coefficient = 1.35 – 1.5
Sand specific gravity = 2.55 – 2.65
Effective size of the gravel = 1 – 50 mm
Wash water speed = 2.5 – 3.5 m/s
Cleaning period = 25 – 35 min
Repairing the filter is taking = 15 – 20 min
Washing by compressed air = 2 -5 min
Washed by pressured water = 10 min & 15 -20 min if no air
The operation time (between two washes) is = 12 – 36 hrs
Rate of filtration (ROF) = 100 – 200 $m^3/m^2/d$
Area of filter = 40 – 64 m^2

The filter is Rectangular in surface area (L*B)

L & B $\leq$ 8 m

B : L = 1 : 1.25 up to 1 : 2

Rate of washing (ROW) = 5 – 6 ROF

Empirical equation to determine minimum number of filters in the WTP = 0.044 * [Q_{mm} (m^3/d)]0.5

The filters numbers:

If $n_w \leq 5$ take n_T = any no. + 1 for wash

If $n_w > 5$ take n_T = even no. + 2 for wash

If $n_w \geq 30$ take n_T = no. divisible by 4 + 4 for wash

Amount of wash water (m3/d) = no. of washing by day * time of washing (10 min) * n_T * ROW (m^3/m^2/d)/(24*60 min/d) * SA (m^2)

The washing: 1. every 12 hrs. 2. every 24 hrs. 3. every 36 hrs.

فى محطات المياه الحديثة نظام تجميع المياه من قاع المرشح عبارة عن بلاطات خرسانية بها فوانى من البولي بروبين، وان كانت الفوانى فى المحطات القديمة تصنع من النحاس.

Example

Design the RSF for a WTP working 16 hr/d, if the design flow is 32000 m^3/d

Solution

Q_d = 32000 m^3/d = 32000/16 = 2000 m^3/h

Assume ROF = 200 $m^3/m^2/d$ = 200/24 $m^3/m^2/hr$ = 5 $m^3/m^2/hr$

TSA = 2000/5 = 400 m^2

Assume L = 8 m, B = L/1.25 = 6.25

SA = 8*6.25 = 50 m^2

n_w = 400/50 = 8 filters (ok)

n_T = 8 + 2 = 10 filters

Assume that ROW = 5 ROF = 25 $m^3/m^2/hr$

Amount of wash water (m^3/d) = no. of washing by day (1) * time of washing (10 min)* n_T (10) * ROW (25) ($m^3/m^2/hr$)/(60 min/hr) * SA (50) (m2) = 2083 m^3/d

% WW = (2083/32000) *100 = 6.5 %

3. DISINFECTION STAGE

- **PURPOSE**

A consumer cannot tell if drinking water is free of pathogens by normal inspection. There for, the main purpose of disinfection is to reduce the potential health risk associated of drinking water by inactivating pathogens. This prevents the possible spread of water-born diseases.

•FACTORS AFFECTING DISINFECTION

1. Contact time

The longer the contact time the greater the kill is.

2. Temperature

As temperature increase the rate of kill increase.

3. Characteristics of water

Suspended solids may shield bacteria from the action of the disinfectant.

Some compounds may adsorb the disinfectant.

Viruses, cysts and ova obstruct the disinfection process as they are more resistant to disinfectants than are bacteria.

4. Kind and concentration of disinfectant

Type and dose of disinfectant are important to achieve the desired goal.

5. Kind and concentration of organisms

The larger number of organisms, the longer is the time required for a given kill. Beside the need to high concentration of the used disinfectant as in case of presence of viruses, cysts and ova.

•Requirements of good disinfectant

1. Effective in destroying all kinds of pathogenic bacteria.
2. Do its task within a reasonable contact time at normal temperature.
3. Economical and easily available.
4. Give residual concentration to safe guard against re-contamination in water supply system.
5. Not toxic and objectionable to user after the water treatment.
6. Adaptability of practical, quick and accurate assay ‘techniques for determining disinfection concentration for operation control and as a measure of disinfecting efficiency.

METHODS OF DISINFECTION

Method	Chemical methods	Physical methods
Examples	1. Chlorination (Chlorine gas, Bleaching powder, Hypochlorite, Chloramines) 2. Permanganate 3. Iodine 4. Bromine 5. Acid sodium sulphate 6. Ozonization	1. Heating. 2. Ultraviolet radiation

COMPARISON BETWEEN SOME TYPES OF DISINFECTANTS

	Chlorine Gas	Chloramines	Ozone	Ultra-Violet
Dose	Total = 0.5 – 1.5 mg/l, Demand = 0.3 -0.5 mg/l, Residual = 0.1 -0.3 mg/l.	CL_2 (0.1 – 0.3)mg/l, Ammonia (0.2 - 0.8)mg/l.	1.5 mg/l	-------
Contact Time	10 – 60 min	45 – 75 min	10 - 20 min	1 – 2 sec
Advantages	1. Cheap 2. Residual for network 3. Available 4. Easy to store, and stored for along time. 5. Simple equipment required. 6. Easy to use.	1. Decrease CL_2 Dose 2. Doesn't effect pipes 3. No taste or odor is produced in water. 4. Cheaper than CL_2.	1. No chemical residual. 2. Doesn't effect in the pipe 3. No taste or odor. 4. Short contact time.	1. No chemical used. 2. Short contact time.
Disadvantages	1. High chlorine dose may cause change in the water color and taste due to damage of pipes or it self.	1. Can't store for along time. 2. Need along contact time. 3. Having chemical residual.	1. Expensive. 2. Can't be stored. 3. No residual. 4. Expensive equipment. 5. Complicated equipment.	1. Very expensive. 2. Used only when the cost of electricity is low. 3. Need training workers.

The advantages and disadvantages of chlorination

- **Advantages**

    1. Cheap

1. Residual for network
2. Available
3. Easy to store, and stored for along time.
4. Simple equipment required.
5. Easy to use.

- **Disadvantages**

1. High chlorine dose may cause change in the water colour and taste due to damage of pipes or it self.
2. Chlorine reacts with organic compound that appears in water and the results are cancer compounds.

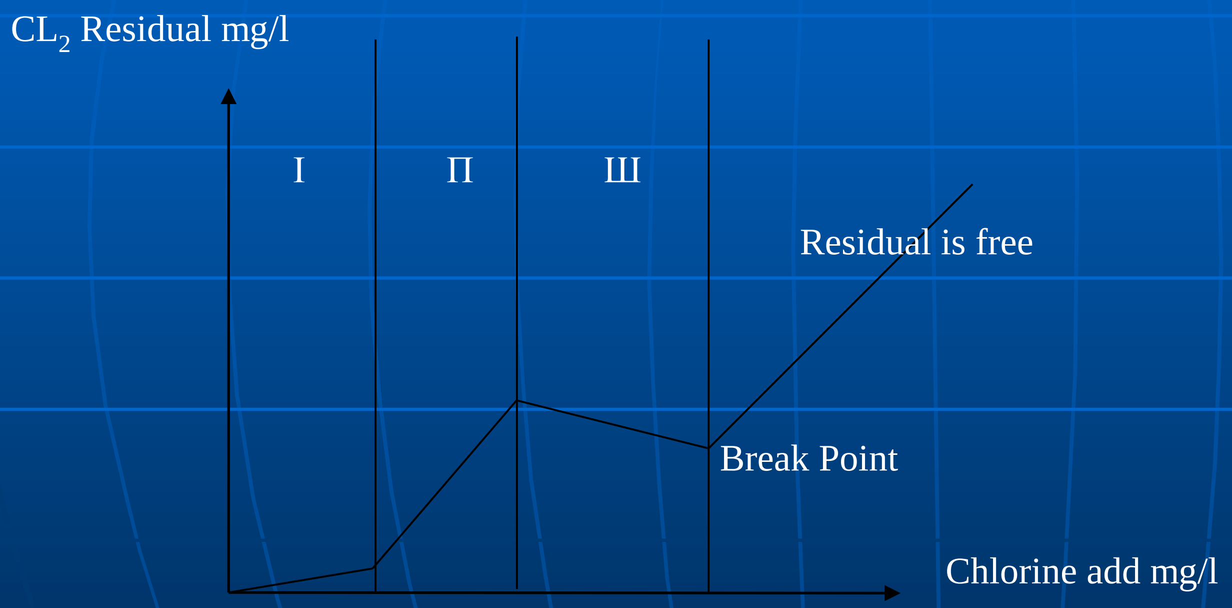
METHODS OF CHLORINATION

	Position	Dose	Purpose
Simple Chlorination	After filtration	0.5 -1.5 mg/l	Kill bacteria, safety
Pre-simple Chlorination	Before coagulation and after LLP	5 – 10 mg/l	Prevents algae growth in sedimentation tank and decreases bacterial load in the filter
Double Chlorination	Simple + Pre-simple Chlorination		
Super chlorination	After filtration	2 – 3 mg/l	Providing safety against harmful bacteria
Break point Chlorination	After filtration	From the curve shown	To ensure the residual chlorine is free

I - Destruction of CL_2 by reducing agents.

П - Formation of chloro-organic of chloramines

Ш - Destruction of components



DETERMINING OF CHLORINE DOSE

The required amount of Chlorine per day (Kg/d) = [Q_{mm} (m^3/d) * (A mg/l + B mg/l)] / 1000 gm/kg,

Where:

A is the chlorine dose required to satisfy disinfection.
B is the required residual chlorine after 30 minutes.

STORAGE WORKS

Types of storage used in water supply works:

1. Ground Storage.

(Appears in water treatment plant after disinfection stage and before high lift pump station)

2. Elevated Storage.

(Appears in different positions according to its function)

GROUND STORAGE

Ground Storage Tank or (Clear Water Tank)

Purpose

1. Produce contact time for disinfection = (0.5 – 1) hr

$\mathbf{C_1}$ (m^3) = (0.5 – 1)hr * $Q_{\mathbf{mm}}$ (m^3/hr), $Q_{\mathbf{mm}}$ (m^3/hr) = $Q_{\mathbf{mm}}$ (m^3/d)/wp

2. Saves Emergency Storage = (25 % - 40 %) of daily production

$\mathbf{C_2}$ (m^3) = (0.15 – 0.4) * $Q_{\mathbf{mm}}$ (m^3/d) ***or*** (4 – 10 hr) * $Q_{\mathbf{mm}}$ (m^3/hr)

3. Balancing difference between maximum daily and maximum monthly flow through one day

$\mathbf{C_3}$ (m^3) = [$Q_{\mathbf{md}}$ (m^3/d) – $Q_{\mathbf{mm}}$ (m^3/d)] * 1 day

4. Saves 80% of fire Storage

$\mathbf{C_4}$ (m^3) = 0.8 * Fire requirements

Fire requirements (m^3) = [Pop. (capita) / 10,000 (capita)] * (60 m^3/hr) * (1 to 2 hr)]

وذلك بفرض ان معدل الحريق (حريقة واحدة) لكل 10.000 نسمة – نحتاج 60 م3 ماء فى الساعة للاطفاء– زمن الحريق من ساعة لساعتين – الحريقة الواحدة تحتاج من 2 لـ 3 حنفيات حريق.

Code Fire Requirements

Egyptian specification recommended 60 m^3/hr for one fire on assumption of 2 hours fire for each 10,000 capita or as indicated in the following table.

No.	*Population (capita)*	*Required fire discharge*
1	10,000	20
2	25,000	25
3	50,000	30
4	100,000	40
5	More than 200,000	50 l/s

Design Capacity of Ground Reservoir

$\mathbf{C_d}$ (m^3) = take bigger of [C_1 or C_2 or C_3] + C_4

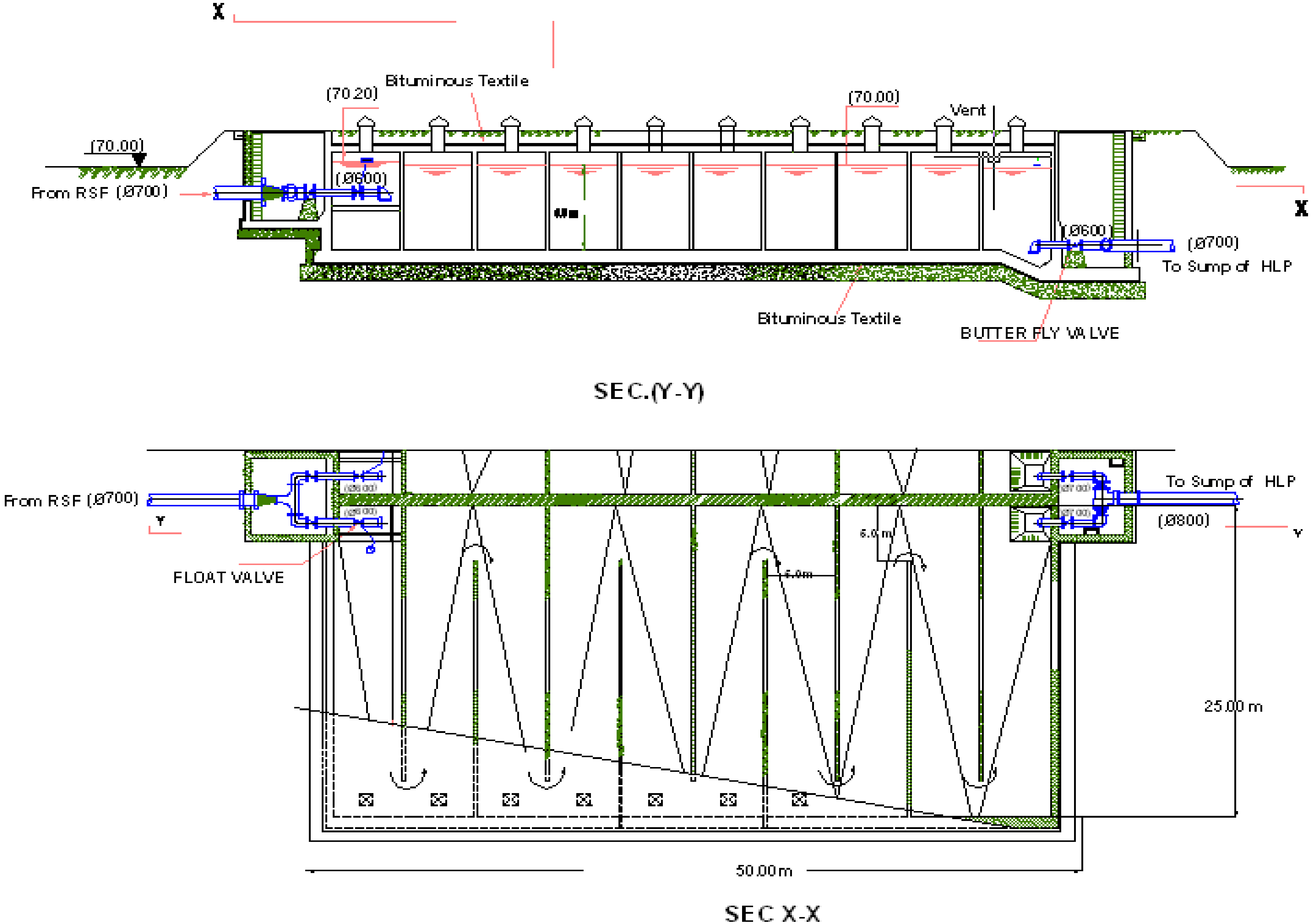
Almost, the ground storage tank is rectangular in shape, where:

$L \leq 50$ m, ويجب ان تقبل القسمة على 10 م

$L = 1.2 – 1.5$ B

$d = 3 – 5$ m

$n \geq 2$ tanks



Ground Reservoir

Example

It's required to design the ground storage of a WTP serves 300,000 capita with average summer water consumption of 420 l/c/d.

Solution

Assume that the wp = 24 hr/d

Calculations of flows

q_{mm} = 420 l/c/d

q_{av} = 420/1.4 l/c/d = 300 l/c/d

Q_{av} = 300 * 300,000 = 90,000,000 l/d = 90,000 m^3/d = 3,750 m^3/hr

Q_{mm} = 1.4* Q_{av} = 5,250 m^3/hr = 126,000 m^3/d

Q_{md} = 1.8* Q_{av} = 6,750 m^3/hr = 162,000 m^3/d

Design Capacity

C_1 = 1 hr * 5250 m^3/hr = 5,250 m^3
C_2 = 162,000 – 126,000 = 36,000 m^3
C_3 = 6 hrs * 5250 = 31,000 m^3
C_4 = 0.8 * [(300,000/10,000) * 120] = 2,880 m^3

C_d = 36,000 + 2,880 = 38,880 m^3

Take [4 tanks each (50m*40m*5m)]

That design volume will be 40,000 m^3 that saves about 7 hrs emergency (ok)

ELEVATED STORAGE

Elevated Storage Tank

Purpose

First: with respect to quantity

1. Cover the fluctuation in water consumption through day.
2. Cover the difference between the maximum consumption and maximum production through one day (maximum day) $= Q_{mh} - Q_{md}$
3. Save 20 % of fire demand.

Second: with respect to pressure

The locations of elevated tank:

1.Just after high lift pump to:

•Fix the head on pumps, then the pumps work at maximum efficiency.

•Prevent the effect of water hammer action on the high lift pumps.

•And at this case the elevated tank is called (Surge Tank).

2. At middle of city (at higher points) to:

•Improve water pressure in the network.

3. At extreme points to:

•Improve the water pressure in the network near to the city boundaries.

•Give ability to city extended in the future.

Design of Elevated Tanks

The capacity

Capacity will be determined from cumulative curve, where:

Capacity (m^3) = (a_{max} + b_{max}) l/c * Pop. (capita) * 0.001 (l/m^3) * Adjusted Factor + 0.2 fire demand (m^3/d)

Where:

Adjusted Factor = (1.5 to 1.8) * (Σ Reading / q_{av})

Determine the dimensions (cylinder in shape)

- Capacity per tank $\leq$ 2000 m^3
- $n \geq 2$ tanks
- $\Phi_i = (2 - 4)$ m
- d = about 10 m preferred to be from 1/2 to 2/3 Φ_o structurally.
- **Capacity** = $n * \pi/4 * (\Phi_o^2 - \Phi_i^2) * d$

Types of elevated tanks according to its function

1. Balance elevated tank. خزان موازنة(Pipe fill and draw works to networks & pipe to waste during empty to be washed).
2. Storage elevated tank. خزان تخزين (Pipe to fill and pipe to draw & pipe to waste during empty to be wash).