Lecture One

2019-2020

PROPERTIES AND CONTAMINANTS OF WATER

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Environmental Engineering Department- Third Stage

References:

1. Water Works Engineering: Planning, Design And Operation, bySyed R. Qasim, Edward M. Motley, Guang Zhu.
2. Water supply and sewarge, fifth edition, E.W. Steel

PROPERTIES AND CONTAMINANTS OF WATER

1.1 Introduction

Methods used for the treatment of a raw water **will depend on the properties of the water and the presence and concentrations of any contaminants.**

Groundwater usually have low levels of colour and turbidity and consistent microbiological quality, although water from shallow wells and some springs may be more variable. Particular problems may include high or low pH value and alkalinity and high concentrations of iron, manganese, nitrate, chlorinated solvents or pesticides.

Surface water may have high levels of colour and turbidity and exhibit poor microbiological quality. Quality may be variable and deteriorate following periods of heavy rainfall. Other problems may include low pH value and alkalinity and high concentrations of aluminum, iron, manganese, nitrate or pesticides.

1.2 Physical Parameters

Physical, chemical and microbiological characteristics relate to the quality of water for domestic and usually associated with the general appearance of the water.

Odor and Taste

It is the most common observation in water quality. The sources would be from organic compounds such as degradation of organic matters or petroleum, and inorganic compounds such as minerals, metals, salts which gives the taste. In addition, certain types of algae, especially the blue-green algae, can also impact foul tastes and odors. The significant effect of this would be upon our health and feeling aesthetic. Drinking water should be free from any objectionable taste or odour at point of use.

Colour

A water may appear coloured because of material in suspension and true colour can only be determined after filtration. The removal of colour from water is necessary not only for aesthetic reasons but also because chlorination of highly coloured waters can give rise to high concentrations of trihalomethanes which is formed as a result of reactions between chlorine and some organic substances present in raw waters. High colour also reduces the efficiency of disinfection by chlorination and ozonation. Drinking water limit is less than 5 colour units.

Turbidity

Turbidity is a measure of the amount of particulate matter that is suspended in water. It normally measured in Nephelometric Turbidity Units (NTU). The sources would be from organic compounds such as plant fiber, human waste, etc. and inorganic compounds such as clay or sand. Beside of feeling aesthetic and effect upon health, the chemical and biological processes are also effected.

Temperature

Water temperature affects the ability of water to hold oxygen, the rate of photosynthesis by aquatic plants and the metabolic rates of aquatic organisms. The sources would normally be the effect from changes of weather or heat and industrial activities such as cooling system. The temperature effects would be also upon the chemical properties such as the degree of gas solubility, density and viscosity.

Total Solid

Total solids (TS) is a measure of the amount of sediment moving along in a stream. The sources would be from organic compounds such as plant fiber, human waste, etc. and inorganic compounds such as clay or sand. Beside of feeling aesthetic and effect upon health, the chemical and biological processes would also effected. (It measured in mg/L).

Total Dissolved Solid

The total dissolved solids (TDS) in water consist of inorganic salts and dissolved materials. In other words, solid left in water after it was filtered and dried. The sources would be from organic compounds such as product from degradation of organic matters and inorganic compounds such as minerals and metals. The effect would cause the taste, color, and odor problems as well as our health, plus water would become corrosive. (It measured in mg/L).

Electrical conductivity

The EC in water is related to the TDS. It is a measure of ability of the water to conduct an electrical current. The EC increases when TDS increases and visa versa.

1.3 Chemical parameters

Dissolved oxygen (DO)

In nature, clean waters are saturated with DO or nearly. If organic wastes are discharged into the natural waters, microorganisms decompose these wastes and utilize DO. Then the level of DO in natural water is an indicator to the pollution.

Alkalinity

Alkalinity in natural water is due to the presence of bicarbonate and carbonate ions, and also it may be due to salts of silicate, phosphate, organic acids and hydroxides.

Hardness

Hard water is associated with formation of scales in boilers, heaters and hot water pipes. Hardness in natural water is due to calcium and magnesium ions. As a function of hardness, the water is classified as the following table.

Hardness description	Hardness as $CaCO_3$ mg/l
Soft	0–50
Moderately soft	50–100
Slightly hard	100–150
Moderately hard	150–200
Hard	>200
Very hard	>300

Aluminium; Al

Aluminium is a natural constituent of many raw waters. Aluminium compounds may also be introduced into treated water as the result of its use as a coagulant to remove colour and turbidity. The UK drinking water quality regulations include a national standard for aluminium of 200 µg/L. Aluminium can deposit within the distribution system and give rise to aesthetic problems. Aluminium in raw water can be removed by coagulation and filtration or membrane techniques. The use of aluminium sulphate as a coagulant in water treatment should normally result in a residual concentration of no more than 50 to 100 µg/L Al.

Iron and manganese Fe, Mn

Iron and manganese derived from minerals and sediments can be present in particulate or dissolved form in groundwaters and surface waters. Iron and manganese concentrations in surface waters are usually less than 1 mg/l but much higher concentrations (up to 50 mg/L Fe and 30 mg/L Mn) can be encountered in groundwaters. Iron and manganese suspensions cause aesthetic problems including metallic taste and discoloration of water fittings and laundry. High dissolved iron and manganese concentrations can also increase chlorine demand and thus reduce the efficiency of chlorine disinfection. The UK drinking water quality regulations include national standards for iron and manganese of 200 µg/L and 50 µg/L, respectively. Iron and manganese can be removed by filtration although oxidation, coagulation and sedimentation may be required for high concentrations particularly if the metals are in dissolved form.

Lead

The concentration of lead in raw waters rarely exceeds 20 µg/L but higher concentrations do occur in water drawn from strata containing galena or other lead ores. High levels of lead in drinking waters are usually caused by the dissolution of lead from lead pipework, tank linings or use of leaded water fittings. Traces of lead may also be derived from lead solder and from PVC pipes containing lead-based stabilizers. The UK drinking water quality regulations specify a standard for lead of 10 µg/l.

Nitrate

Nitrate (NO_3^-) occurs naturally in water as a result of the oxidation of ammonia, which is released during mineralization of organic nitrogen. In some areas, agriculture is the major source of nitrate in surface waters and groundwaters. The discharge of nitrate-containing effluents from sewage treatment works contributes to the concentration of nitrate in some surface waters. Ion-exchange, biological de-nitrification and certain membrane processes can reduce nitrate concentrations. Of these, only ion-exchange and membrane processes are likely to be practicable for small water supplies.

Pesticides

The use of pesticides for agricultural and non-agricultural purposes is widespread and there are approximately 450 different active ingredients in pesticides licensed. The UK drinking water quality regulations specify standards of 0.1 µg/L for individual pesticides and 0.5 µg/L for total pesticides.

Biochemical oxygen demand (BOD)

The BOD test gives an indication of the oxygen required to degrade biologically any organic matter present in a water, as well as the oxygen needed to oxidize inorganic materials such as sulphides. The BOD test is conducted for wastewater (not for drinking water) as an indicator for organic pollution.

1.2 Microbiological parameters

The microbiological quality of drinking water has traditionally been assessed by monitoring for bacteria called faecal indicator organisms (coliforms, E. coli, and enterococci). The presence of these organisms is indicative of faecal contamination and hence the possible presence of pathogens. Although indicator organisms are generally adequate for monitoring purposes they cannot completely be relied on to indicate the absence of pathogens. This is especially true where a pathogen is environmentally more robust, or can survive treatment better than the indicators. In these circumstances the indicator may be absent even though low numbers of pathogens still remain.

Table (1.1) **Water Quality Standards of Drinking Water**

No	Item	Standard Value
1	Standard Plate Count	100 in 1mL or less
2	Coliform Group Bacterial	Not to be detected
3	Cadmium	0.01 mg/L or less
4	Mercury	0.0005 mg/L or less
5	Selenium	0.01 mg/L or less
6	Lead	0.01 mg/L or less
7	Arsenic	0.01 mg/L or less
8	Chromium (VI)	0.05 mg/L or less
9	Cyanide	0.01 mg/L or less
10	Nitrates/Nitrites	10 mg/L as nitrogen or less
11	Fluoide	0.8 mg/L or less
12	Boron	1.0 mg/L or less
13	Carbon Tetrachloride	0.002 mg/L or less
14	1,4-dioxane	0.05 mg/L or less
15	1,1-dichloroethylene	0.02 mg/L or less
16	cis-1,2-dichloroethylene	0.04 mg/L or less
17	Dichloromethane	0.02 mg/L or less
18	Tetrachloroethylene	0.01 mg/L or less
19	Trichloroethylene	0.03 mg/L or less
20	Benzene	0.01 mg/L or less
21	Chloroacetic Acid	0.02 mg/L or less
22	Chloroform	0.06 mg/L or less
23	Dichloroacetic Acid	0.04 mg/L or less
24	Dibromochloromethane	0.1 mg/L or less
25	Bromate	0.01 mg/L or less
26	Total Trihalomethanes	0.1 mg/L or less
27	Trichloroacetic Acid	0.02 mg/L or less
28	Bromodichloromethane	0.03 mg/L or less
29	Bromoform	0.09 mg/L or less
30	Formaldehyde	0.08 mg/L or less
31	Zinc	1.0 mg/L or less
32	Aluminium	0.2 mg/L or less
33	Iron	0.3 mg/L or less
34	Copper	1.0 mg/L or less
35	Sodium	200 mg/L or less
36	Manganese	0.05 mg/L or less
37	Chloride	200 mg/L or less
38	Calcium, Magnesium, etc. (Hardness)	300 mg/L or less
39	Total Residue	500 mg/L or less
40	Methylene Blue Activated Substance	0.2 mg/L or less
41	Geosmin	0.00001 mg/L or less 1)
42	2-methylisoborneol (MIB)	0.00001 mg/L or less 2)
43	Nonionic Surfactant	0.02 mg/L or less
44	Phenols	0.005 mg/L as phenol or less 3)
45	Organic Compound (as concentration of TOC)	5 mg/L or less
46	pH Value	5.8-8.6
47	Taste	Not abnormal
48	Odor	Not abnormal
49	Color	5 degree or less
50	Turbidity	2 degree or less

Note 1), Note 2) : The standard value is 0.00002mg/L by 31 March 2007.

Note 3) : The standard value of organic compound etc.as the potassium permanganate consumption is 10mg/L by 31 March 2005.

Type of sample: Hospital wastewater
Source: Ibn Al-Nafees Hospital, Baghdad city
Results of physical analysis:

Turbidity	516 NTU
TSS	1450 ppm
TDS	900 ppm
Electrical Conductivity, EC	1780 µS/cm

Results of chemical analysis:

pH	8.12
BOD_5	325 ppm
COD	500 ppm
TOC	200 ppm
Alkalinity, as $CaCO_3$	552 ppm
Cations, ppm	
Lead, Pb	0.02
Cadmium, Cd	0.26
Iron, Fe	0.73
Nickel, Ni	0.01
Mercury, Hg	0.30
Copper, Cu	0.62
Silver, Ag	0.20
Zinc, Zn	0.02
Chromium, Cr	0.40
Manganese, Mn	0.06
Calcium, Ca	44
Magnesium, Mg	19.7
Sodium, Na	150
Potassium, K	60
Anions, ppm	
Total phosphorus, PO_4	10
Nitrate, NO_3	4.054
Nitrite, NO_2	0.214
Chlorides, Cl	189.94
Sulfate, SO_4	516

Results of microbiological analysis:

Bacteria	
Fecal coliform FC, CFU/100ml	
Total coliform count TC, CFU/100ml	
Total heterotrophic bacterial count TPC, CFU/ml	8.3×10^{12}
Other microorganisms	
Viruses	+
Parasites	+
Worms	+

Lecture two

2019-2020

Population forcasting and water consumption

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Environmental Eng

Third stage

2.1 Design Periods

A water treatment plant is generally designed and constructed to serve the needs of a community for a number of years in the future. The initial year is the year when the construction is completed and the initial operation begins. The design or planning year is the year when the facility is expected to reach its full designed capacity. Period between initial and design year is known as the ***design period***.

Design period is estimated based on the following:

- Useful life of the component.
- Expandability aspect.
- Anticipated rate of growth of population, including industrial, commercial developments & migration-immigration.
- Available resources.
- Performance of the system during initial period.

The design periods usually adopted are as

following:

- Dams: 25-100 years
- Treatment plant: 10-25 years
- Pumps: 10 years
- Distribution system: 35-45 years

2.2 Population Forecasting Methods

The various methods adopted for estimating future populations are given below. The particular method to be adopted for a particular case or for a particular city depends largely on the factors discussed in the methods, and the selection is left to the discretion and intelligence of the designer.

1. *Arithmetic Method*
2. *Geometric or uniform percentage Method*
3. Geometric increase method
4. *Declining growth Method*
5. *Logistic Curve Method*
6. *Ratio Method*
7. *Curvilinear Method*

2.2.1 Arithmetic Method

This method is based on the assumption that the population increases at a constant rate. This method is most applicable to large and established cities.

$$\frac{dP}{dt} = K_a$$

P=Population

t = time

K_a= arithmetic growth constant $\quad K_a = \frac{(P_2 - P_1)}{(t_2 - t_1)}$

$P = P_o + K_a \Delta t$

Example 1

The population number of a city at 1990 was 6200 capital and became in 2000, 8000 capital. Estimate the population at 2010.

$K_a = \frac{8000-6200}{2000-1990} = 180 cap.$

$P_{2010} = 8000 + 180(2000 - 1990) = 9800 cap.$

2.2.2 Geometric or uniform percentage method

This method assumes that the rate of growth is proportional to the population.

$$\frac{dP}{dt} = K_g P$$

$\ln P = \ln P_o + K_g \Delta t$

P=Population

t = time

K_g= rate constant $\quad K_g = \frac{\ln P - \ln P_o}{\Delta t}$

Example 2

The following table shows the number of the population as a function of time. Estimate the population number at 1970.

Year	1930	1940	1950	1960
Pop. number	62000	74000	85000	100000

Question: Can we apply Arithmetic method to solve the above question?

2.2.3 Geometric increase method

The average percentage of the last few decades/years is determined, and the forecasting is done on the basis that percentage increase per decade/year will be same. Thus, the population at the end of n years or decades is given as:

$$P_T = P_o(1 + AGR)^n$$

Where, AGR = annual growth rate of the population

- P_T = population at time n in the future
- P_o = present population
- n = periods of projection

Notices

1. AGR is 3% for developing countries and 5% for advanced countries.
2. This is the most common method in establishing population required in designing of water treatment and networks.

Example 3

The Annual Growth Rate of a town in Ethiopia is 3.5%. Assuming the present population of the town (in 2010) is 4500, what would be the population in 2025?

Solution

AGR = 3.5%; P_o = 4500

n = 2025-2010 = 15

$P_n = P_o(1+AGR/100)^n$

$P_{15} = 4500(1+3.5/100)^{15}=$**7540**

2.2.4 Declining growth method (Decreasing rate of increase)

Population is assumed to reach some limiting value or saturation point.

$$\boxed{\frac{dp}{dt} = k_2(p_{sat} - p)}$$

p_t : population at some time in the future

p_o: base population

p_{sat}: population at saturation level

p , p_o : are populations recorded n years apart

Δt : no. of years after base year

k_2 may be determined from successive censuses and the equation:

$$\boxed{k_2 = -\frac{1}{n}\ln\frac{p_{sat} - p}{p_{sat} - p_o}}$$

then,

$$\boxed{p_t = p_o + (p_{sat} - p_o)(1 - e^{k_2 \Delta t})}$$

2.2.5 Logistic Curve Method (Mathematical method)

The three factors responsible for changes in population are :

(i) Births, (ii) Deaths and (iii) Migrations. Logistic curve method is based on the hypothesis that when these varying influences do not produce extraordinary changes, the population would probably follow the growth curve characteristics of living things within limited space and with limited economic opportunity. The curve is S-shaped and is known as logistic curve.

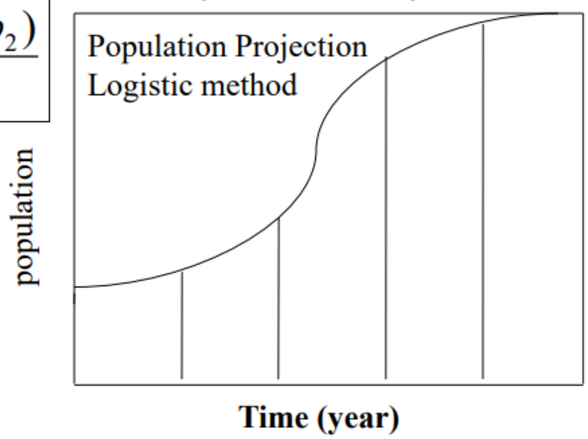
p_t : population at some time in the future
p_o: base population
p_{sat}: population at saturation level
p_1 , p_2 : population at two time periods
n : time interval between succeeding censuses
Δt : no. of years after base year

$$\boxed{p_t = \frac{p_{sat}}{1+e^{a+b\Delta t}}}$$

$$\boxed{\left|p_{sat}\right| = \frac{2p_o p_1 p_2 - p_1^2(p_o + p_2)}{p_o p_2 - p_1^2}}$$

$$\boxed{a = \ln\frac{p_{sat} - p_2}{p_2}}$$

$$\boxed{b = \frac{1}{n}\ln\frac{p_o(p_{sat} - p_1)}{p_1(p_{sat} - p_o)}}$$



Example 4

Estimate the population at 2000 using the following data by Declining growth and Curve fitting methods.

Year	**1970**	**1980**	**1990**	**2000**
Pop. number	10,000	15,000	18,000	??,???

2.2.6 Ratio Method (Ratio and Correlation)

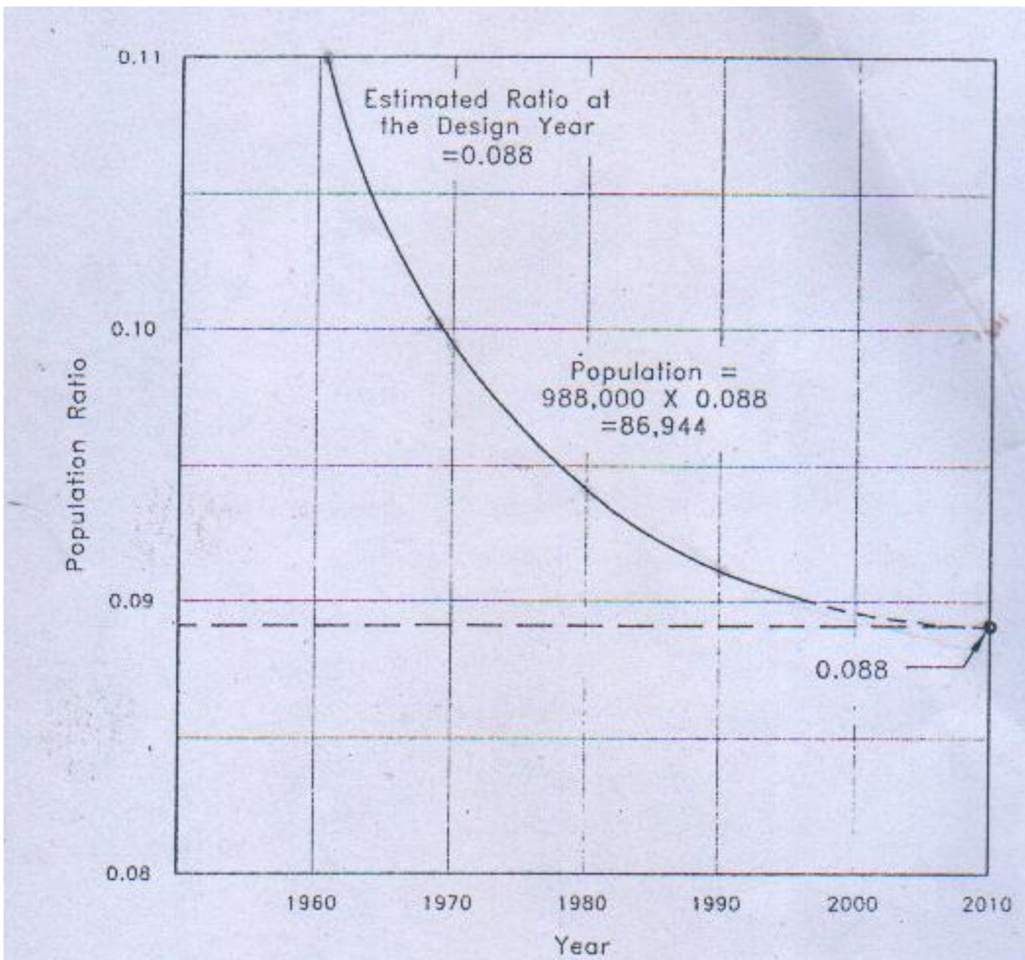
In this method, the local population and the country's population for the last four to five decades is obtained from the census records. The ratios of the local population to national population are then worked out for these decades. A graph is then plotted between time and these ratios, and extended up to the design period to extrapolate the ratio corresponding to future design year. This ratio is then multiplied by the expected national population at the end of the design period, so as to obtain the required city's future population.

Example 5

Estimate the population of a city using the ratio method. The design year is 2010. The estimated population in the year 2010 is 988,000.

Year	Population in 1000_s		Ratio
	City	Region or State or Country	
1960	50	455	0.11
1970	61	623	0.098
1980	72	766	0.094
1990	77	850	0.091

Solution



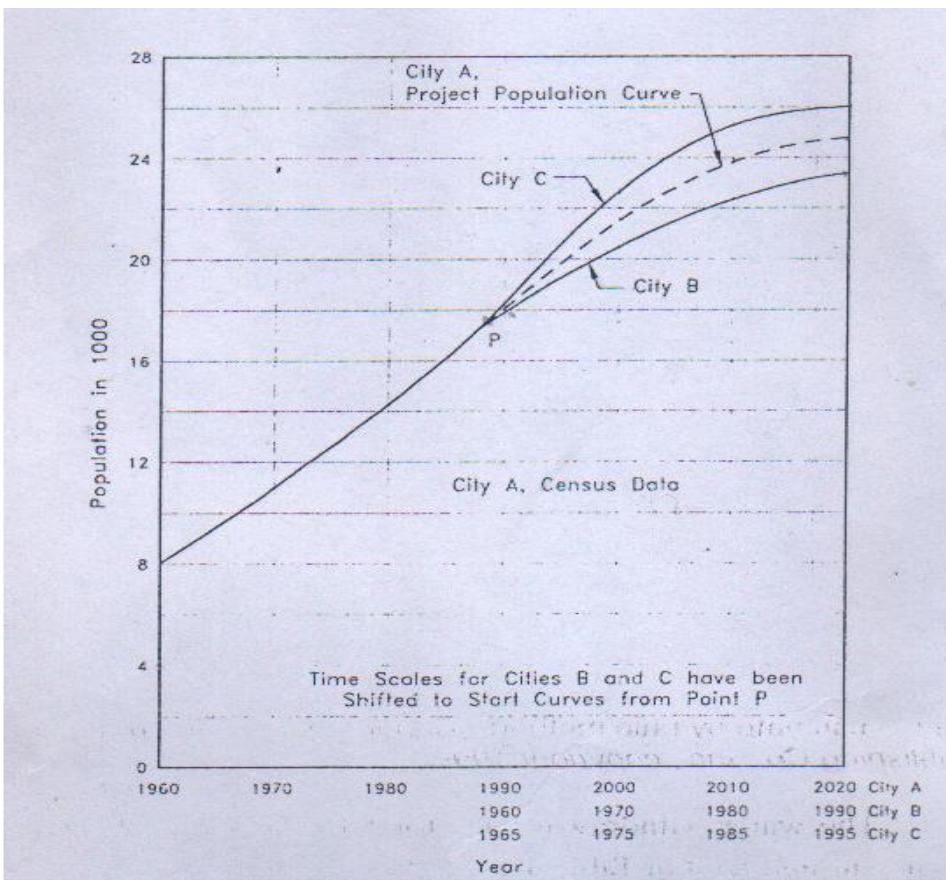
2.2.7 Curvilinear Method (Graphical method)

The procedures involves the graphical projection of the past population data for city being studied. The population data of other similar but large cities are also plotted in a such manner that all the curves are coincident at the present population value of the city being studied. These curves are used as a guides in future projection.

Example 6

Estimate the population of city A by using graphical comparison with cities B and C. The design year is 2020.

Year	Population in 1000s		
	City A	City B	City C
1960	8	18	16
1970	11	20.3	20
1980	14	22	25
1990	18	23.2	25.6



2.3 Water Consumption Rate

It is very difficult to precisely assess the quantity of water demanded by the public, since there are many variable factors affecting water consumption. The various types of water demands, which a city may have, may be broken into following classes:

Table (2.1) Water Consumption for Various Purposes:

	Types of Consumption	Normal Range (lit/capita/day)	Average	% of total
1	Domestic Consumption	65-300	160	35
2	Industrial and Commercial Demand	45-450	135	30
3	Public Uses including Fire Demand	20-90	45	10
4	Losses and Waste	45-150	62	25

Factors affecting per capita demand:

1) Size of the city: Per capita demand for big cities is generally large as compared to that for smaller towns as big cities have sewered houses.

2) Presence of industries.

3) Climatic conditions.

4) Habits of people and their economic status.

5) Quality of water: If water is aesthetically $ medically safe, the consumption will increase as people will not resort to private wells, etc.

6) Pressure in the distribution system.

7) Efficiency of water works administration: Leaks in water mains and services; and unauthorized use of water can be kept to a minimum by surveys.

8) Cost of water.

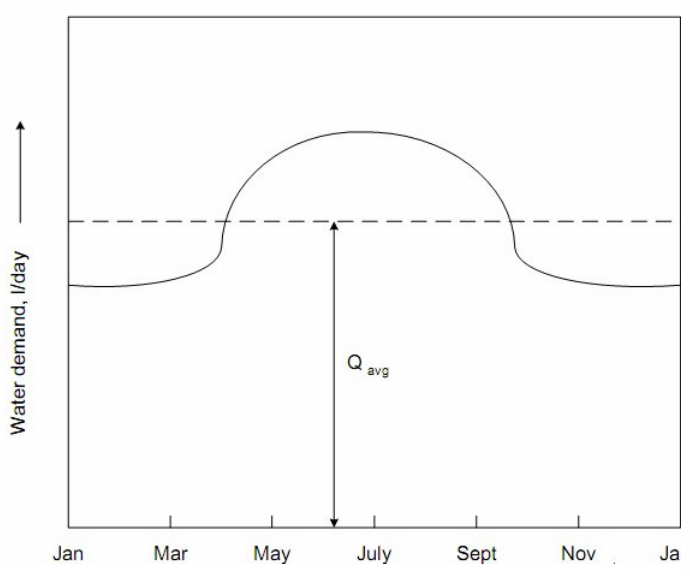
2.3.1 Fluctuations in Rate of Demand

Average Daily Per Capita Demand = Quantity Required in 12 Months/ (365 x Population).

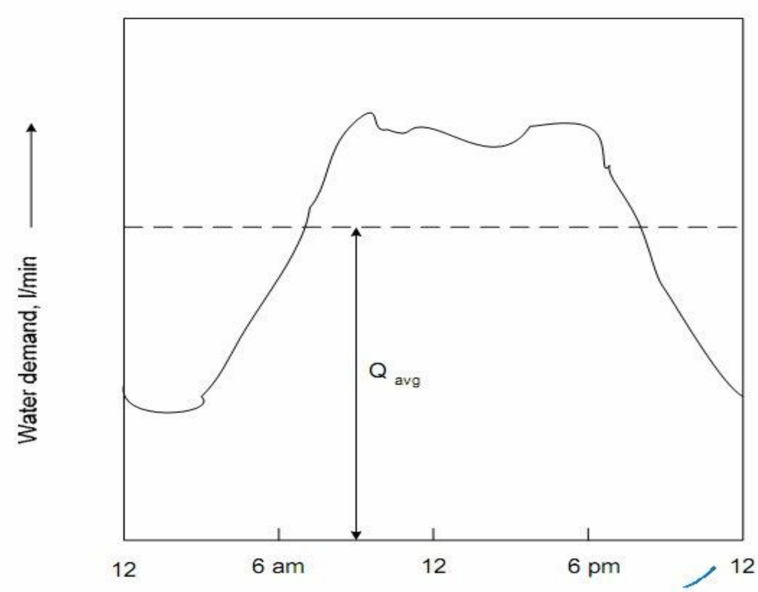
If this average demand is supplied at all the times, it will not be sufficient to meet the fluctuations.

- ***Seasonal variation*:** The demand peaks during summer. Fire breakouts are generally more in summer, increasing demand. So, there is seasonal variation .
- ***Daily variation****:* depends on the activity. People draw out more water on Sundays and Festival days, thus increasing demand on these days.
- ***Hourly variations:*** are very important as they have a wide range. During active household working hours i.e. from six to ten in the morning and four to eight in the evening, the bulk of the daily requirement is taken. During other hours the requirement is negligible. Moreover, if a fire breaks out, a huge quantity of water is required to be supplied during short duration, necessitating the need for a maximum rate of hourly supply.

Seasonal Variations



Hourly Variations



Maximum daily demand = 1.8 x average daily demand
Maximum hourly demand of maximum day i.e. Peak demand
= 1.5 x Maximum daily demand/24
= 1.5 x (1.8 x average daily demand)/24
= 2.7 x average daily demand/24
= 2.7 x annual average hourly demand

Example 7

Population of a city after 10 years is 35000 cap, and now is 28000 cap. The rate of consumption now is $1.6*10^6$ l/day and the design capacity of the treatment plant is $5*10^6$ l/day. If the rate of growth of the population is Arithmetic increase, find the year which the treatment plant reaches to the designed capacity?

2.3.2 Fire Fighting Demand

The fire demand per capita is very less on an average basis but the rate at which the water is required is very large. The rate of fire demand is sometimes treated as a function of population and is worked out from following empirical formula:

$$F = 320C\sqrt{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.2$$

	Authority	Formulae (P in thousand)	Q for 1 lakh Population)
1	American Insurance Association	Q (L/min)=4637 √P (1-0.01 √P)	41760
2	Kuchling's Formula	Q (L/min)=3182 √P	31800
3	Freeman's Formula	Q (L/min)= 1136.5(P/5+10)	35050
4	Ministry of Urban Development Manual Formula	Q (kilo liters/d)=100 √P for P>50000	31623

Note: 1 lakh = 100000 capita

Where
F= fire flow required m^3/day
A= total area m^2
C= coefficient related to the type of construction:
1.5 for wood frame construction
1 for ordinary construction (i.e., brick, or masonry).
0.8 for noncombustible construction
0.6 for fire resistive construction

Note

If a building consists from many materials, for example 2 materials. In this case we must calculate the total area A=A1+A2, and we consider that all building consist from the first material and we find F1. In the next step we consider that all building consist from the second material and we find F2. Finally, fire flow required equal:

F=F1*(A1/A)+F2*(A2/A) …………..2.3

The following table shows the duration for fire flow:

Table (2.2) Fire flow duration

Flow rate m^3/min	**Duration, h**
7.6	2
11.3	3
15.1	4
18.9	5
22.7	6
26.5	7
30.2	8
34	9
37.8	10

Example 8

Building with normal material consists of 2 stories each one with are 100 m^2. Find the flow rate fire demand and its period (duration)?

Example 9

A 4 stories wooden frame building has each floor of area 509 m^2. This building is adjacent to 5 story building of ordinary construction with 900 m^2 per floor. Determine the fire flow and duration for each building and that for both building (assuming that they are connected).

Lecture three

2019-2020

Raw water intake and screeing

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Environmental Eng

3.1 Process selection and the treatment process train

Water treatment plant utilize a number of treatment processes to achieve the desired degree of treatment. Each stage of this processes has a function to remove a contaminant. Then, the design engineer must evaluate numerous important factors in the selection of the treatment processes. These factors include: 1-finished water quality standards, 2- state design criteria, 3- constituents treated, 4- topography and geology, 5- hydraulic requirement, 6- energy requirement and plant economics.

The collective arrangement of various treatment processes is called *flow schema, processes diagram, or processes train.*

In the most water treatment plant, the processes include:

1) Water intake
2) Coarse and fine screen
3) Pumps and pumping station
4) Coagulation and flocculation tanks
5) Sedimentation tanks
6) Filtration
7) Disinfection

Figure 3.1 shows the typical water treatment processes train.

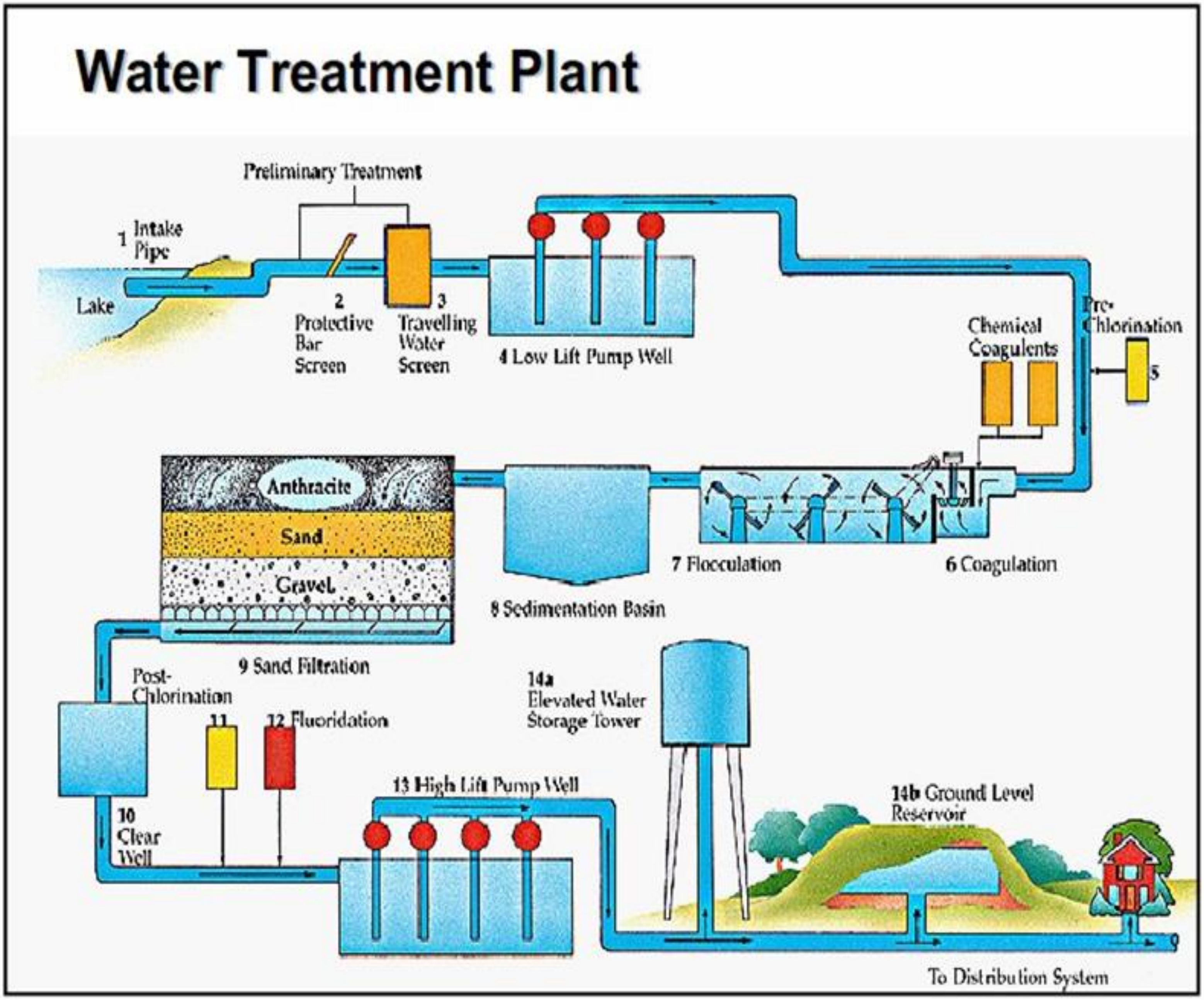


Fig (3.1) processes train of the water treatment plant

3.2 Intake Structure

The basic function of the intake structure, is to help in safely withdrawing water from the source over predetermined pool levels and then to discharge this water into the withdrawal conduit (normally called intake conduit), through which it flows up to water treatment plant. Some types of intakes structure are shown in Figs. (3.2-3.7).

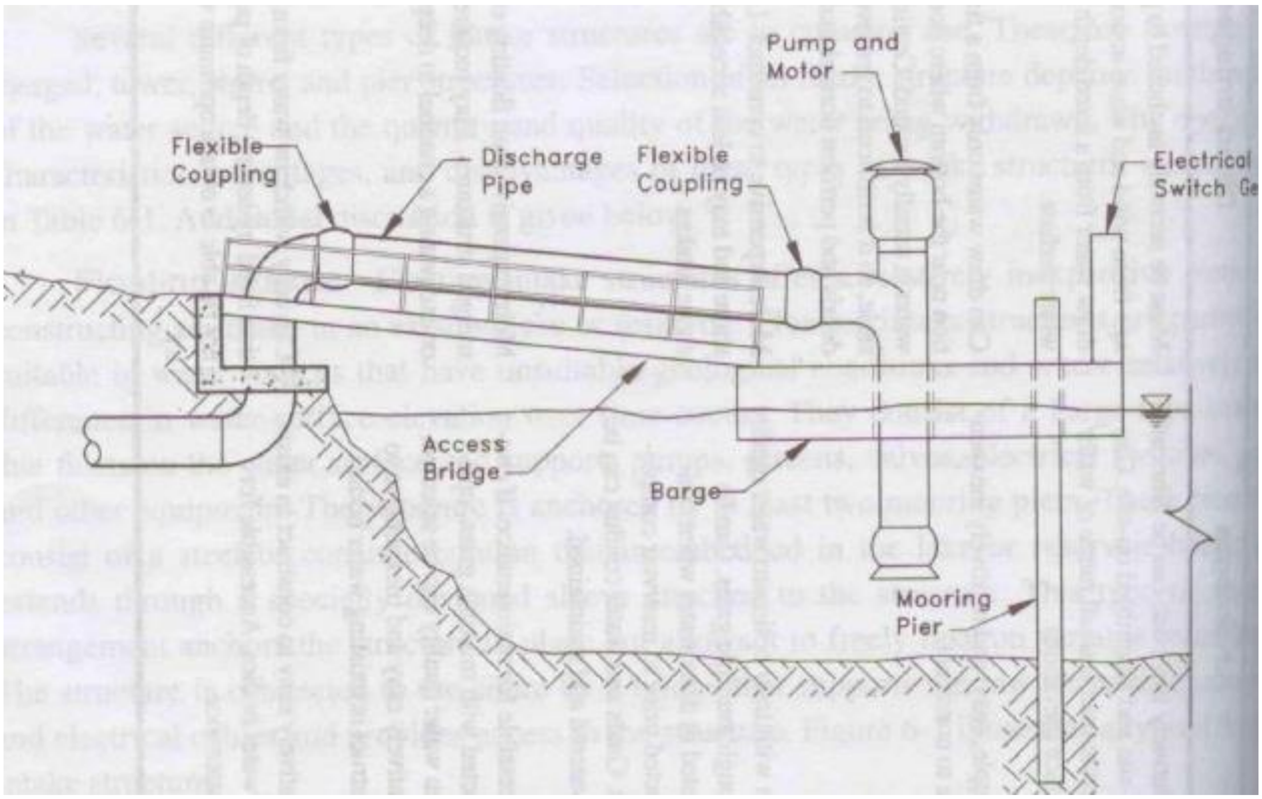


Fig. 3.2 Floating intakes

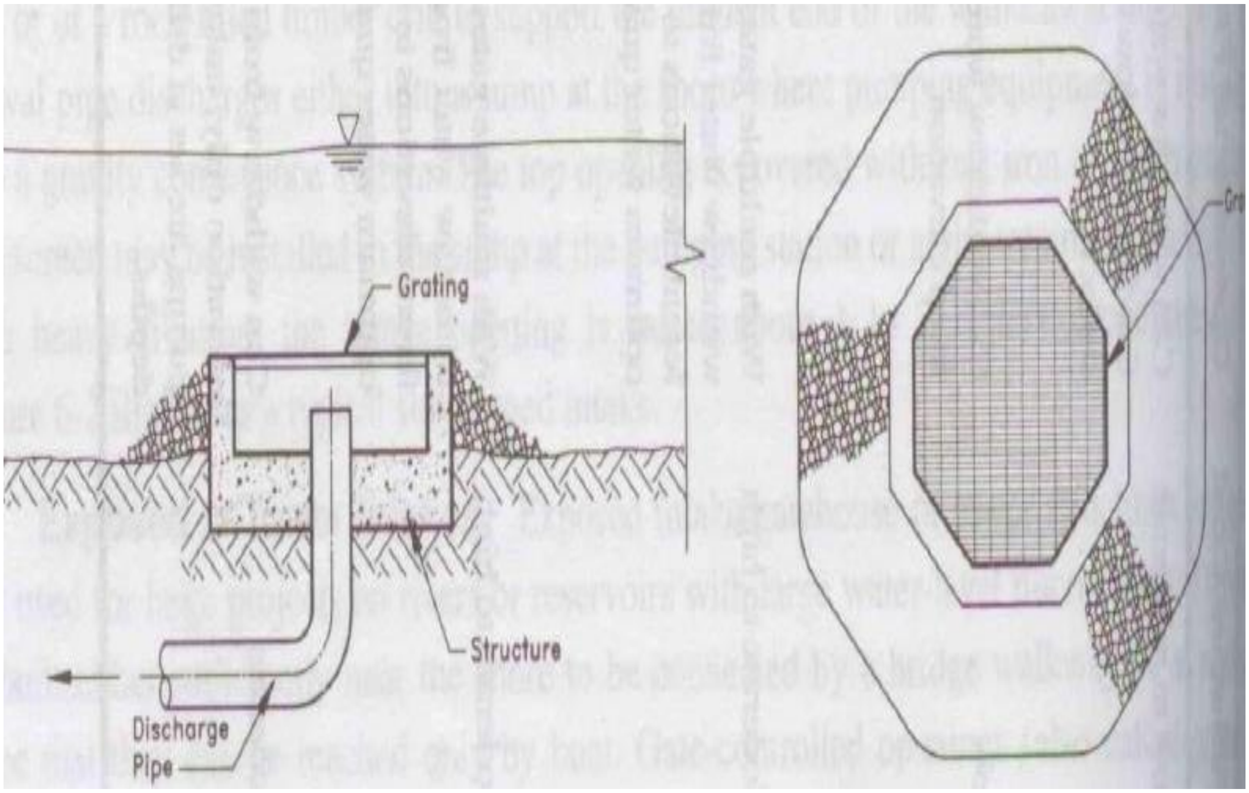


Fig. 3.3 Submerged intakes

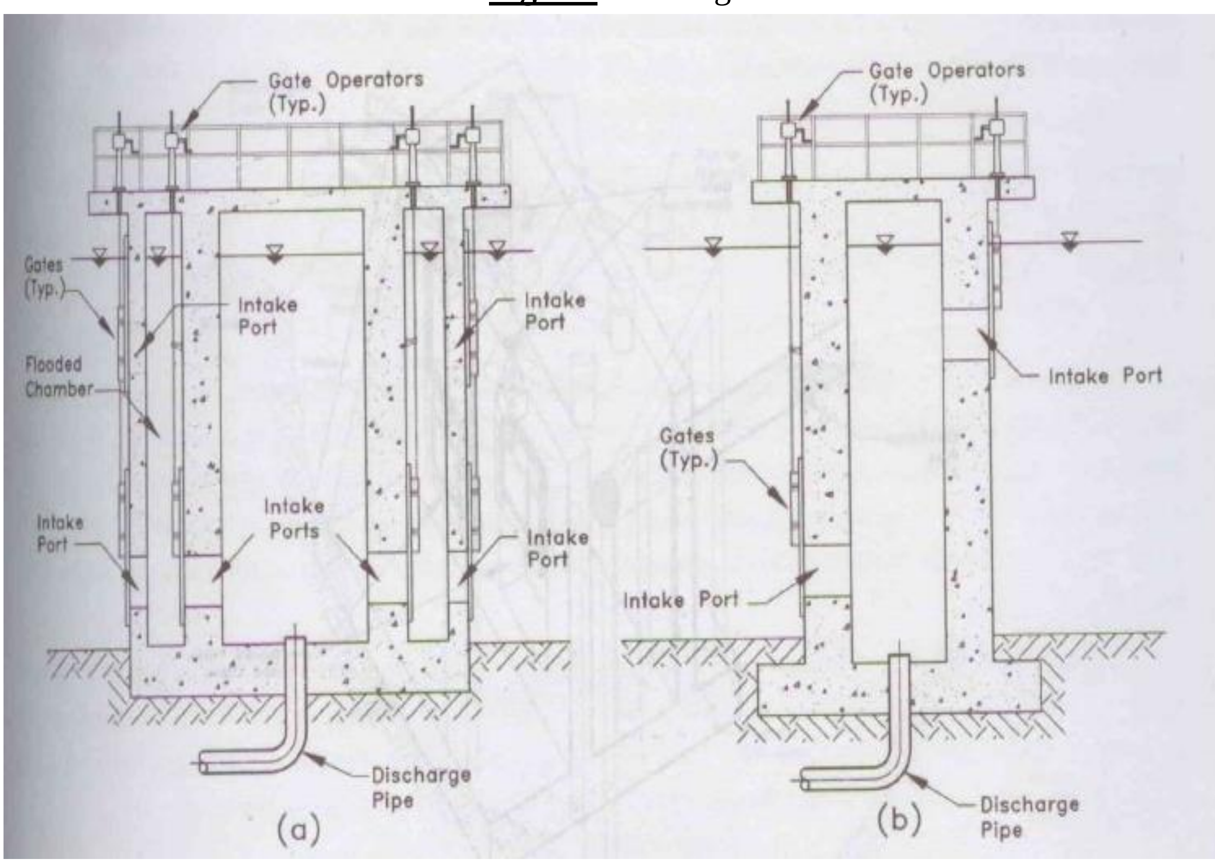


Fig. 3.4 Exposed or Tower intakes

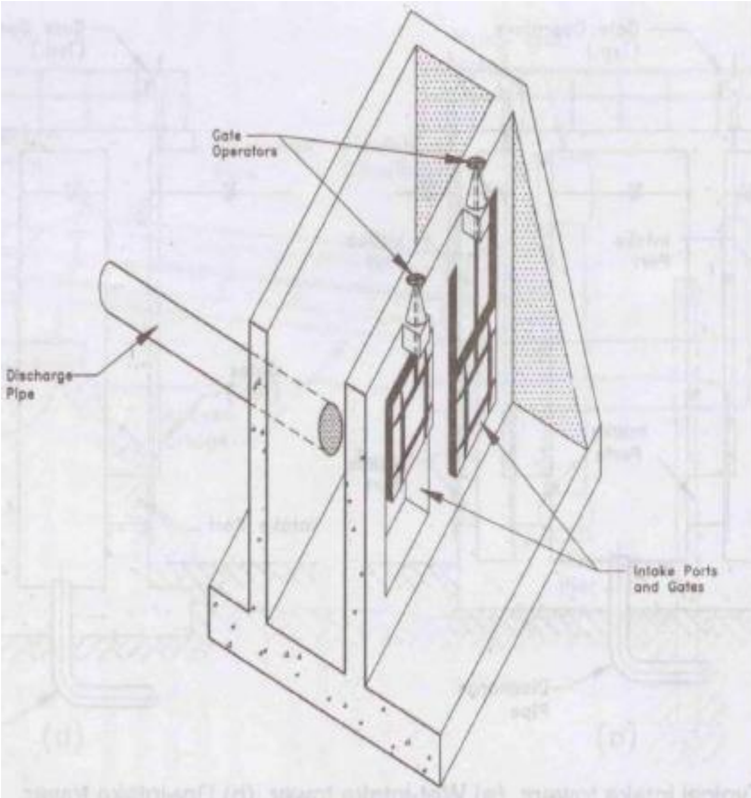


Fig. 3.5 Shore-intake structure

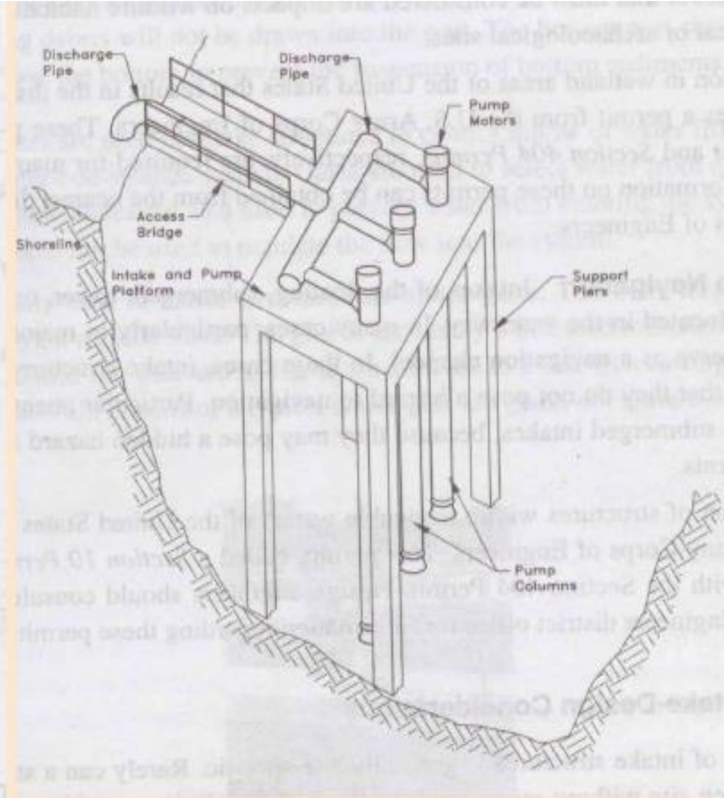


Fig. 3.6 Pier structure

3.2.1 Factors Governing Location of Intake

1) The site should be near the treatment plant so that the cost of conveying water to the city is less.

2) The intake must be located in the purer zone of the source to draw best quality water from the source, thereby reducing load on the treatment plant.

3) The intake must never be located at the downstream or in the vicinity of the point of disposal of wastewater.

4) The site should be such as to permit greater withdrawal of water, if required at a future date.

5) The intake must be located at a place from where it can draw water even during the driest period of the year.

6) The intake site should remain easily accessible during floods and should not get flooded. Moreover, the flood waters should not be concentrated in the vicinity of the intake.

3.2.2 Design Considerations

1) Sufficient factor of safety against external forces such as heavy currents, floating materials, submerged bodies, ice pressure, etc.

2) Should have sufficient self-weight so that it does not float by up thrust of water.

3) Intake velocity plays an important role if the intake is a gate. High intake velocity increase head losses, and low intake velocity require the intake port to be larger and so add to the cost of the structure.

3.2.3 Design of intakes

The hydraulic consideration in intake structure is energy losses due to the acceleration. The losses through the intake port can be calculated by using the orifice equation:

$$h_L = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2 \quad \ldots\ldots\ldots 3.1$$

where,

h_L = head loss, m
Q = discharge, m^3/s
C = coefficient of discharge (0.6-0.9)
A = effective submerged open area, m^2

3.3 Screening

A screen is a device with openings for removing bigger suspended or floating matter in water which would otherwise damage equipment or interfere with satisfactory operation of treatment units.

3.3.1 Types of Screens

Coarse Screens**:** Coarse screens also called racks, are usually bar screens, composed of vertical or inclined bars spaced at equal intervals across a channel through which raw water flows. Clear space between bars ranges from 20 to 50 mm. Bar screens are usually hand cleaned and sometimes provided with mechanical devices. These cleaning devices are rakes which periodically sweep the entire screen removing the solids for further processing or disposal. Hand cleaned racks are set usually at an angle of 45° to the horizontal to increase the effective cleaning surface and also facilitate the raking operations. Mechanical cleaned racks are generally erected almost vertically. The angle of inclination of rack with horizontal is between 30^o and 60^o.

Fine screen: Fine screen (< 2cm) is used to remove smaller objects such as leaves, twigs and fish. that may damage pumps or other equipment. They may be located either at the intake structure or at the raw water pump station.

Figure 3.2 shows types of screen.

a) Coarse screen

b) Fine screen

c) Manual cleaning screen

d) Automatically cleaning screen

Fig (3.2) Types of screen

3.3.2 Design of screen

The design velocity should be such as to permit 100% removal of material of certain size without undue depositions. Velocities of 0.6 to 1.2 m/s through the open area for the peak flows have been used satisfactorily. Further, the velocity at low flows in the approach channel should not be less than 0.3 m/s to avoid deposition of solids. Head loss varies with the nature of screenings (open area, blocked area, shape of the screen) and with hydraulic parameters at the upstream of the screen. The head loss through a **vertical bar screens** is calculated from the following formula:

$$h_L = \frac{V_b^2 - V_a^2}{2g} \times \frac{1}{0.7} \quad \text{.........3.2}$$

where, h_L = head loss v_b = velocity through bar opening in m/s, v_a = in m, approach velocity in m/s
velocity in channel.

Another formula often used to determine the head loss through an **inclined bar** rack is Kirschmer's equation:

$h_L = \beta\,(W/b)^{4/3}\,(v^2/2g)\sin\theta$ ……………..3.3

where h_L = head loss, m

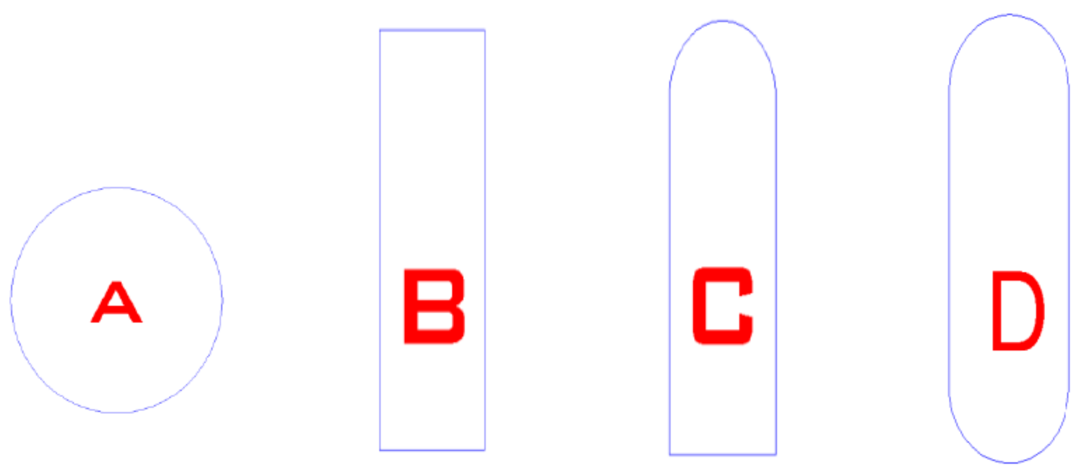
β = bar shape factor (1.79 for circular bar, 2.42 for sharp edge rectangular bar, 1.83 for rectangular bar with semicircle upstream, and 1.67 for rectangular bar with both u/s and d/s face as semicircular).

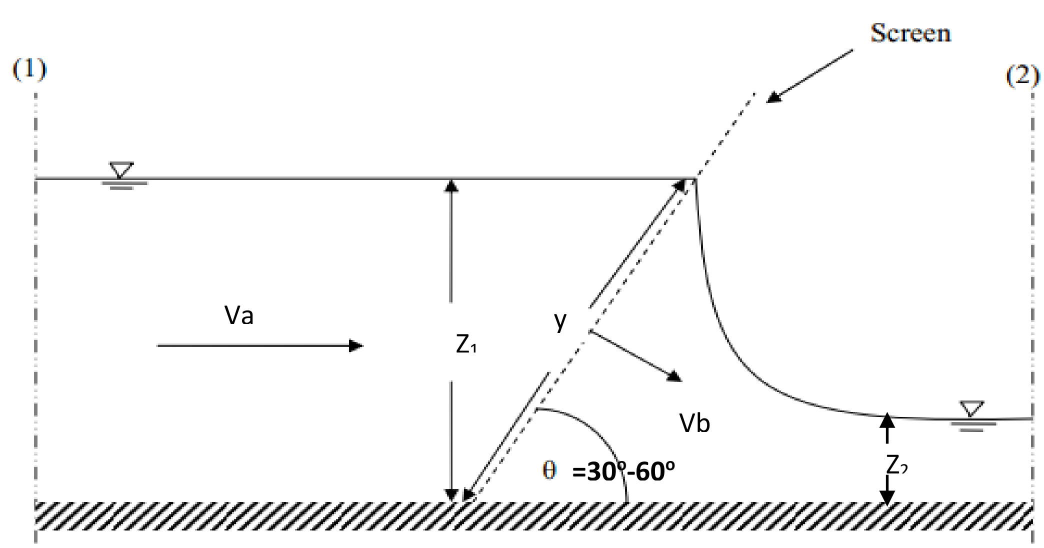
W = maximum width of bar u/s of flow, m

b = minimum clear spacing between bars, m

v = approach velocity, m/s

θ = angle of inclination of rack with horizontal (30-60^o)





r by

orifice equation (3.1). Orifice formula can be used when the screen is located at the pumping station.

Intake design example

Design an intake tower with gates meet the following requirement:

- Minimum reservoir elevation = 70 m msl
- Maximum reservoir elevation = 90 m msl
- Normal water surface elevation = 85 m msl
- Bottom elevation= 60 m msl
- Flow rate = 113500 m^3/day
- Velocity = 0.08 m/s

Solution

Q = 113500 m^3/day = 1.31 m^3/s

A = 1.32 / 0.08 = 16.38 m^2

This is too large for a single gate, so select two equal size square gates

Width = $(16.38 / 2)^{0.5}$ = 2.86 m, then use width and height = 3 m

Set the highest gate with its top two meters below the normal water surface elevation of 85 m, then a centerline elevation = 81.5 m (85-2-1.5). Now, set the lowest gate at a centerline elevation = 65 m. Provide additional gates at two levels equally spaced over 16.5 m range (81.5-65).

Spacing = 16.5 / 3 space = 5.5 m/space

Gates will be provided at centerline elevations of 81.5, 76, 70.5 and 65 m, as shown in the figure (1).

The head loss through the intake can be calculated from the orifice formula (eq 3.1). Two gates is used and others are standby. Therefore the flow rate = 1.31 / 2 = 0.66 m^3/s

$$h_l = \frac{1}{2 \times 9.81}\left(\frac{0.66}{\underbrace{0.6 \times 3 \times 3}_{\text{Area of the gate}}}\right)^2$$

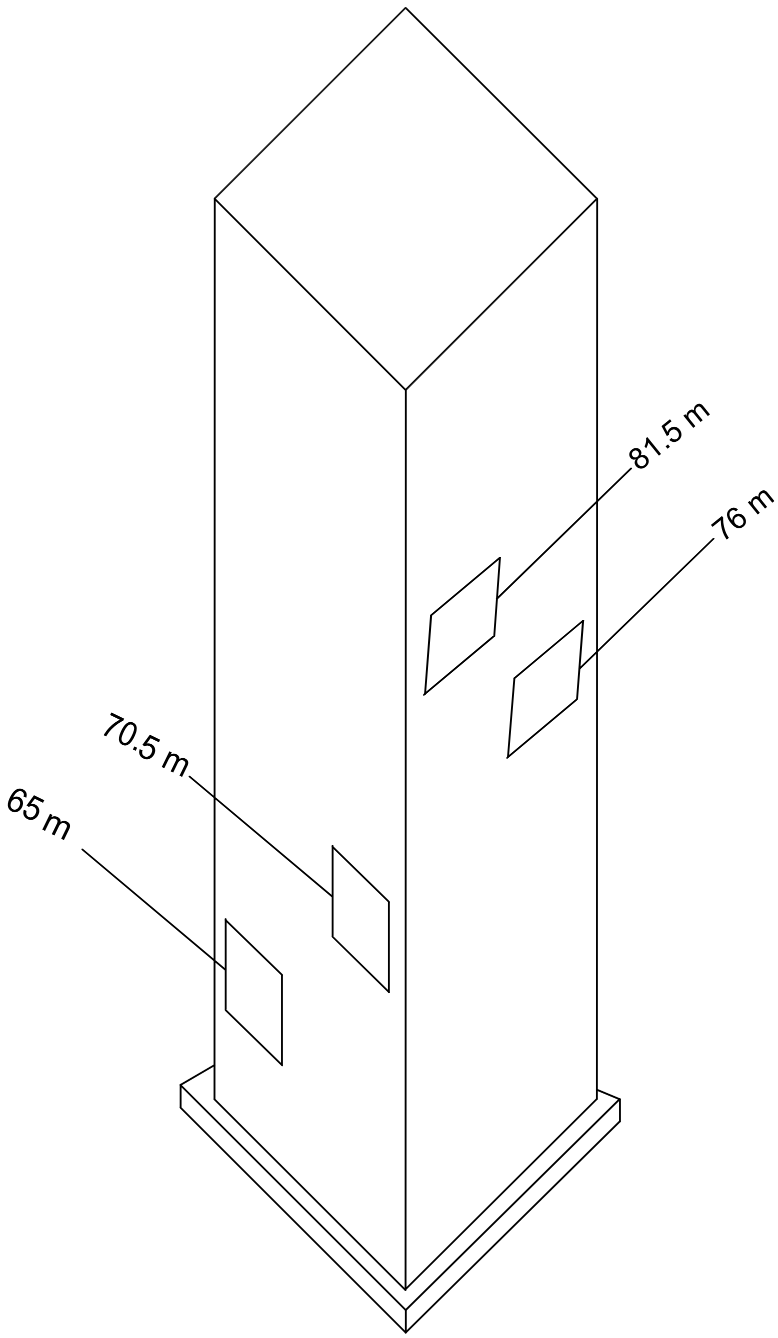
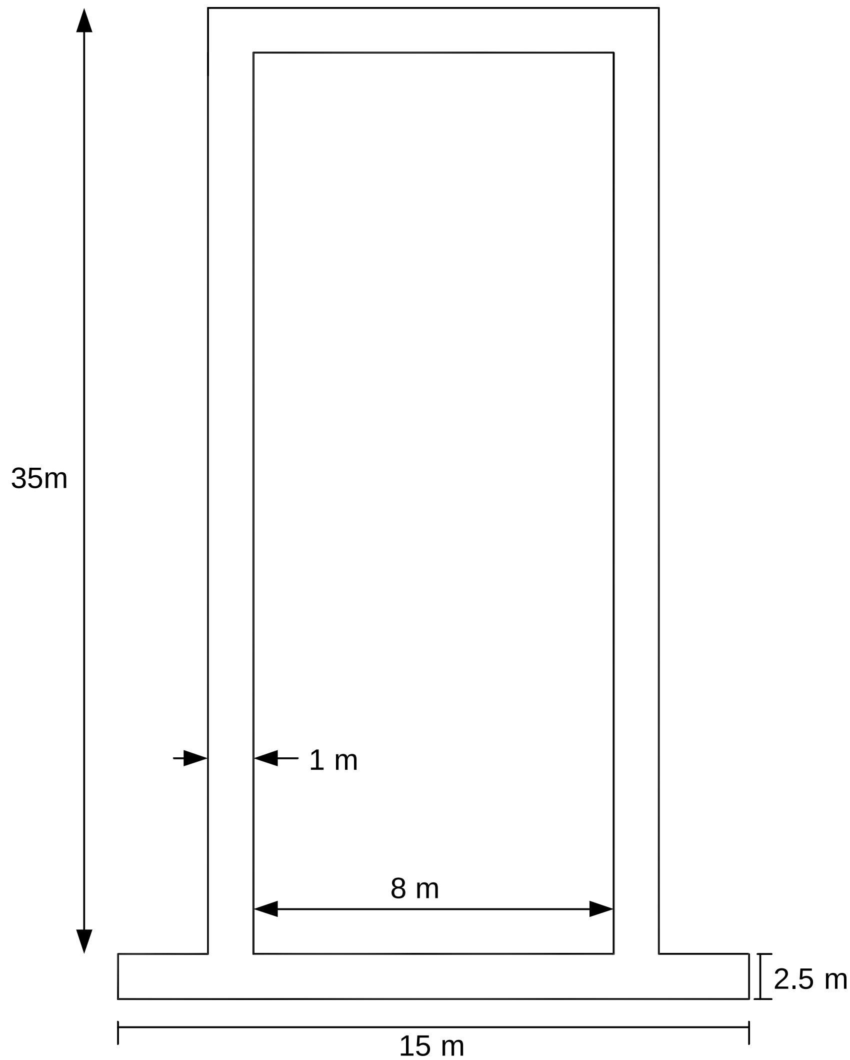


Fig (1) Intake structure

Now, to calculate the stability of the structure we must compare the weight of water displaced at the maximum elevation with the weight of tower when it is empty (worst condition).



Volume of water displaced by tower and base slab = 10m*10m*30m + 15m*15m*2.5m = 3563 m^3
The weight of water displaced = 3563 m^3 / 1000 kg/m^3 = $3.56*10^6$ kg

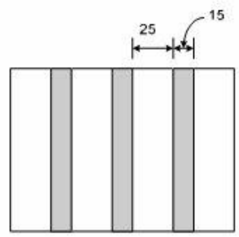
The weight of structure equal the weight of side walls plus the weight of the foundation slab.
Weight of side walls = (10m*10m*35m - 8m*8m*35m) * 2308 kg/m^3 = $2.91*10^6$ kg
Weight of the foundation slab = 15m*15m*2.5m* 2308 kg/m^3 =
$1.3*10^6$ kg
Total weight of structure = $2.91*10^6$ kg + $1.3*10^6$ kg = $4.21*10^6$ kg> $3.56*10^6$ kg OK
Safty factor = $4.21*10^6$ / $3.56*10^6$ = 1.2 Very GOOD

Design example

A mechanical bar screen is to be used in an approach channel with a maximum velocity of 1 m/s. The bars are 15mm thick, and the opening are 25mm wide. Determine the velocity between bars and the head losses.

Solution

- Assume the channel has a width (W) and depth (D)
- Net area of screen = WD [25 / (25+15) = (5/8)WD
- Area of channel = WD
- Use continuity equation



- $V_a A_a = V_b A_b$ OR $V_b = \frac{V_a A_a}{A_b}$

$$V_b = \frac{1 \times WD}{(5/8)WD} = 1.6 \text{ m/s}$$

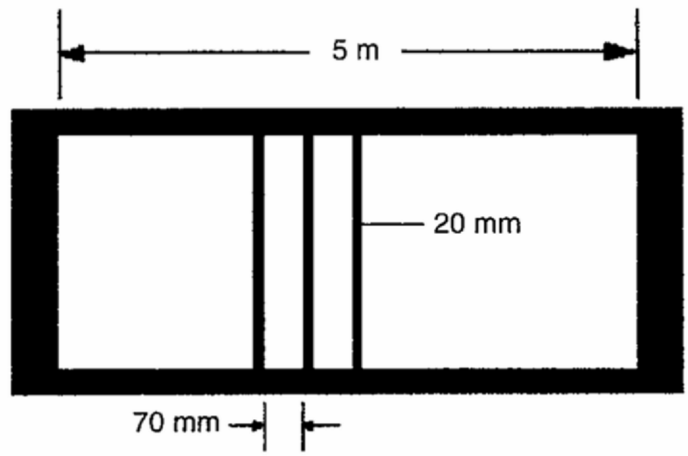
$$h_L = \frac{(V_b^2 - V_a^2)}{2g} \times \frac{1}{0.7}$$

$$h_L = \frac{(1.6)^2 - 1^2}{2g} \times \frac{1}{0.7} = 0.114m$$

Example

A bar screen measuring 2 m by 5 m of surface flow area is used to protect the pump in a shoreline intake of a water treatment plant. The plant is drawing raw water from the river at a rate of 8 m^3/sec . The bar width is 20 mm and the bar spacing is 70 mm. If the screen is 30% clogged, calculate the head loss through the screen. Assume Cd = 0.60

Solution



For screens used in shoreline intakes, the velocity of approach is practically zero. Thus, from the previous figure, the number of spacings is equal to one more than the number of bars. Let x number of bars,

20x + 70(x+1) = 5000
Then x = 54.7 = 55
Area of clear opening = 70(55+1)*2000 = 7.48 m^2

$$h_l = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2$$

$$h_l = \frac{(8)^2}{2\times 9.8(0.6\times 7.48\times 0.7)^2} = 0.33m$$

In this example we choose the orifice equation as a result to the exit of pump in a shoreline intake.

Design examples

Use Kirschmer's equation to find the head loss through a rectangular bar screen used in treatment plant, if the bar width is 15 mm and spacing is 25 mm. Assume that the width of channel (w) equal two times of water depth. Use Q = 0.6 m^3/s, approach velocity = 0.6 m/s and theta = 60^o.

Lecture four

2019-2020

Pumps and pumping station

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4.1 Types of pumps

Pump is needed to raise the water from a place to other one or more points in the system. For public water supply, pumps may be divided into three types:

1) Reciprocating pumps: Consists of a piston or plunger which alternatively draws water into a cylinder on the intake struck and then forces it out on the discharge struck. (Fig. 4.1)

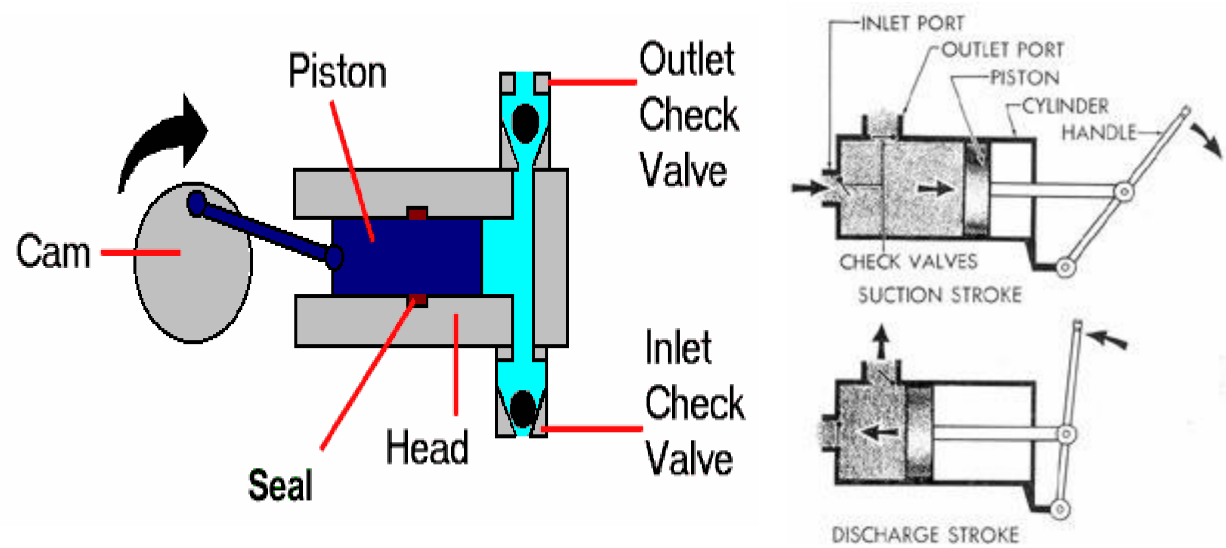
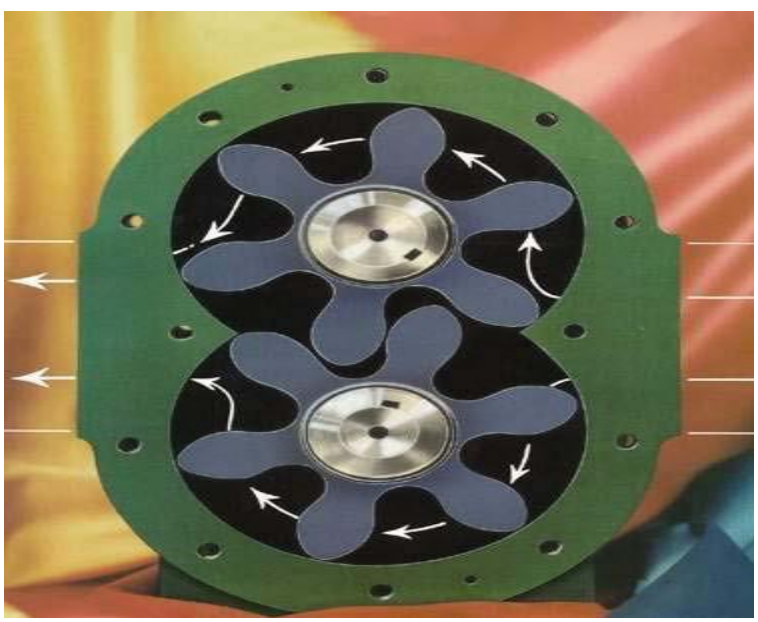


Fig. (4.1): Reciprocating Pump

2) Rotary pumps: Contains two rotary piston or gears which interlock and draw water into the chamber and force it continuously into the discharge pipe. (Fig. 4.2)



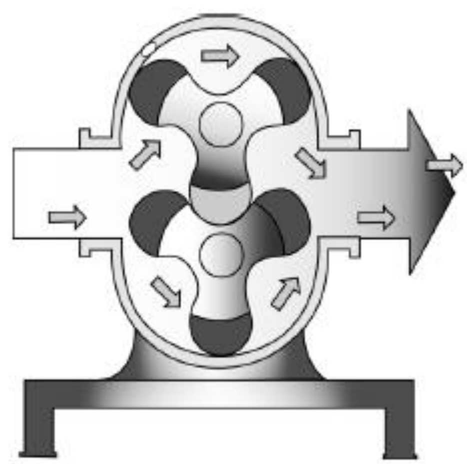


Fig. (4.2): Rotary Pump

3) Centrifugal pumps: It has an impeller with radial vanes rotating swiftly to draws water into the center and discharge it by centrifugal force. The are most commonly used pumps in water supply system. (Fig. 4.3)

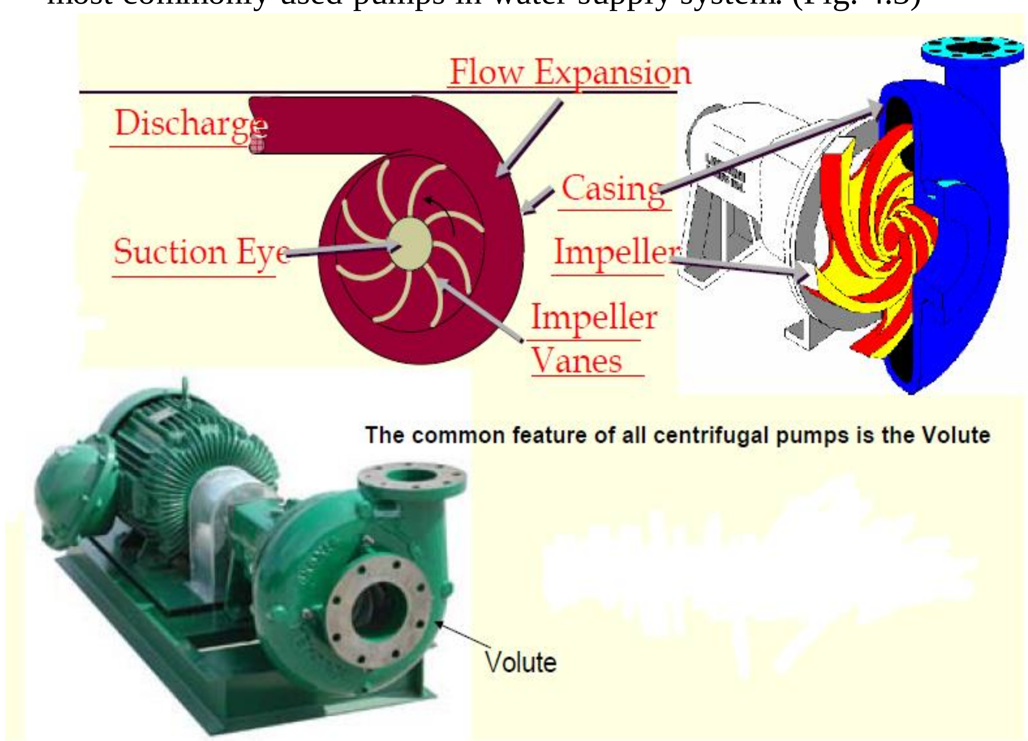


Fig. (4.3): Centrifugal Pump

4.2 Power and efficiency

The water power required is the net output of the pump and equal to:

$$P_w = K \times Q \times TDH \quad \text{.........................4.1}$$

Where

P_w: Power output of the pump, Kw
K: Constant, K=0.163 (Kw, m^3/min, m)
TDH: Total dynamic head, m
Q: The flow rate, m^3/min

Power input to the pump equal to:

$$P_P = \frac{P_w}{E_p} \quad \text{....................................4.2}$$

Where E_p is the Pump efficiency

4.3 Total dynamic head (TDH)

Total dynamic head can be calculated by:

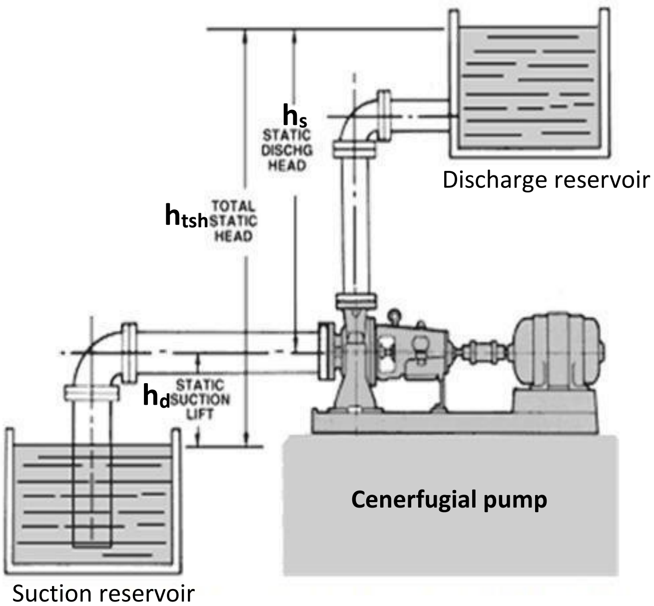
$$TDH = h_{tsh} + h_l + h_m + h_v \quad \text{................4.3}$$

4.3.1 Total Static head (h_{tsh})

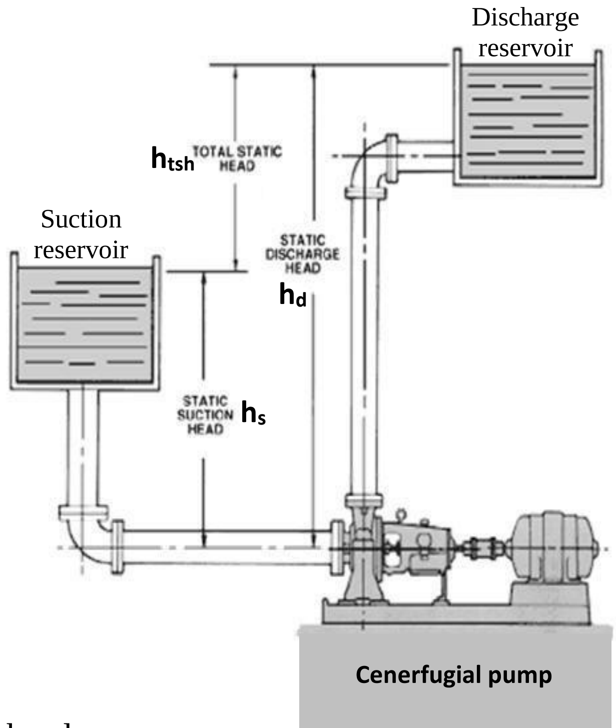
It is a difference in elevations of free water surface at discharge and suction reservoirs of the pumps. Total static head (h_{tsh}) equal to:

$h_{tsh} = h_d + h_s$ (for lift suction head, Fig. (4.4 a)

$h_{tsh} = h_d - h_s$ (for flooded suction head, Fig. (4.4 b)



a) Lift suction head



b) Flooded suction head

Fig. (4.4) Determine of pump total static head
a) Lift suction head
b) Flooded suction head

<u>Where</u>
h_d: Static discharge head (difference in elevation between discharge liquid level and the centerline of the pump impeller).
h_s: Static suction head (difference in elevation between the suction liquid level and the centerline of the pump impeller).

4.3.2 Head losses in pipes (h_L)

Head losses equal to the summation of friction head loss in discharge and suction pipes. It can be calculated from Darcy-Weisbach formula:

$$h_L = f \times \frac{v^2}{2g} \times \frac{L}{D} \quad ....4.4$$

<u>Where</u>
f: Friction factor, it can be found by moody diagram (Fig. 4.5) by using k/D and Reynolds number. k is a constant depends on the pipe type.
v: Velocity in the pipe, m/s
D: Pipe diameter, m.
L: Pipe length, m.

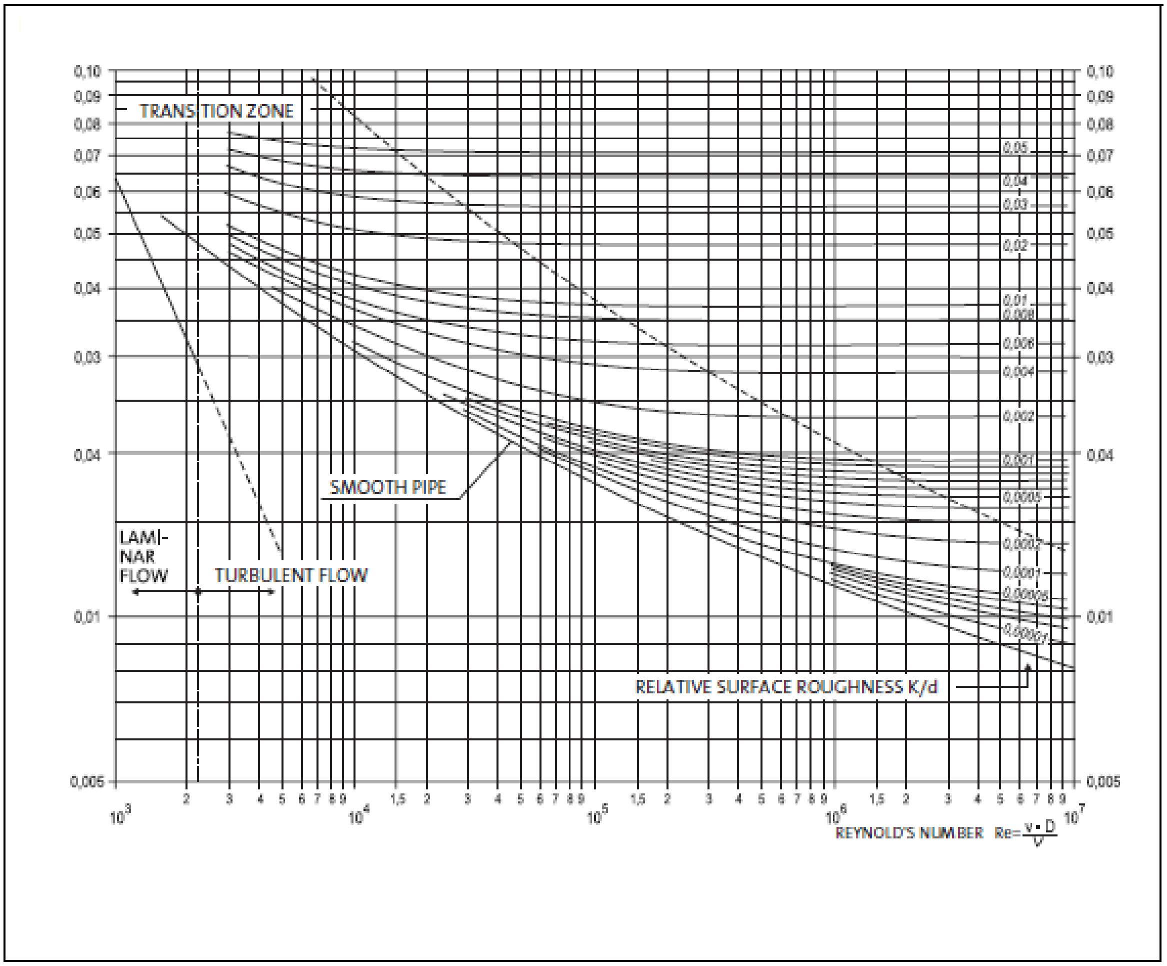


Fig.(4.5): Moody diagram for establishing the friction factor f. The value of f is obtained using Reynolds number and the relative roughness number k/D as parameters, where D is pipe internal diameter in mm and k equivalent surface roughness in mm. Completely turbulent flow can be assumed in wastewater application.

Table (4.1): Surface roughness values (mm).

Pipe material	k new	k old
Plastic	0,01	0,25
Drawn steel	0,05	1,0
Welded steel	0,10	1,0
Drawn stainless steel	0,05	0,25
Welded stainless steel	0,1	0,25
Cast iron	0,25	1,0
Bituminized cast iron	0,12	
Asbestos cement	0,025	0,25
Concrete	0,3...2,0	

Note: The surface of an old pipe material becomes rougher from erosion. Corrosion and sediment layers forming on the pipe surface may decrease the pipe diameter, also leading to higher flow losses.

The kinematic viscosity for water is dependent on temperature:

t °C	0	20	40	60	100
$\nu\ 10^{-6}\ m^2/s$	1,78	1,00	0,66	0,48	0,30

4.3.3 Minor losses (h_m)

Due to the entrance, exit, change in direction of discharge and suction pipes.

$$h_m = k \times \frac{v^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.5$$

Where

k: Constant depends on valves, fitting, etc.

4.3.4 Velocity head (h_v)

Can be calculated from the following equation:

$$h_v = \frac{v^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.6$$

4.4 Characteristic curves for centrifugal pump

4.4.1 Pump head-discharge curve

The head developed by a particular pump at various rates of discharge at a constant impeller speed is established by pump tests conducted by the manufacturer. The head gives the discharge pressure with the inlet static water level at the elevation of the pump centerline and excluding losses in suction and discharge pipes. Consider the test arrangement illustrated schematically in Fig. (4.6) where the discharge is controlled by a valve.

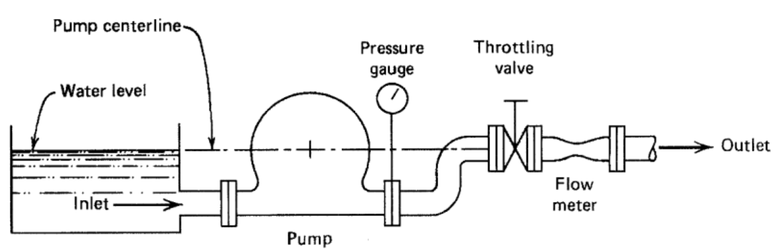


Fig. (4.6) Schematic of a pump head discharge curve

The discharge pressure is measured by a gauge and the rate of discharge is recorded by flow meter. The power input is measured and efficiency determine. With the valve in the discharge pipe is closed, the rotating impeller simply churns in the water causing the pressure at the outlet of the pump to rise to a value referred to the *shut off head*. As the valve is gradually opened allowing increasing water flow, the pump head decreases as drown in Fig. (4.7). The pump efficiency rises with increasing rate of discharge to an optimum value and then decreases. The flow rate at peak efficiency is determined by pump design and the rotational speed of the impeller.

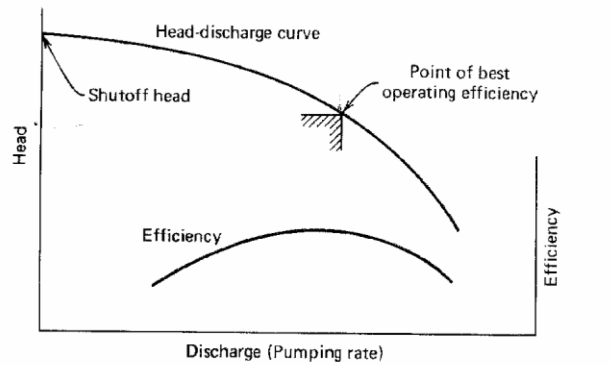


Fig (4.7) Characteristic curves for centrifugal pump

4.4.2 System head curve

When a pump lifts water from a reservoir into a piping system, the resistance to flow at various rates of discharge is described by a system head curve, fig (4.8). The two components of discharge resistance are the static head and the friction head loss that increases with pumping rate.

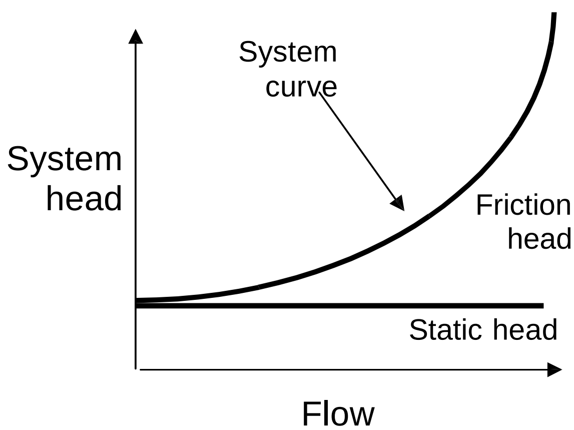


Fig (4.8) System head curve

4.5 Net positive suction head (NPSH)

It is the force available to drive the flow into the pump. Two values of NPSH are important in pump selection, these are NPSH available ($NPSH_{av}$) and NPSH required ($NPSH_{req}$). $NPSH_{av.}$is the absolute pressure at the suction port (inlet) of the pump. $NPSH_{av}$ is a function of the system. $NPSH_{av.}$ is a function of everything in the system on the suction side of the pump up to the suction nozzle of the pump. This includes the pressure on the surface of the liquid in the supply tank (h_{abs}), the difference between the liquid level and the centerline of the pump suction nozzle (h_s), the line losses, velocity head (h_l), and vapor pressure (h_{vp}). $NPSH_{req}$ is the minimum pressure required at the suction port (inlet) of the pump to keep the pump from cavitating. $NPSH_{req}$ is based on everything from the pump suction nozzle to the point in the pump where the pressure starts to increase. This includes the entrance losses and the friction losses or pressure drops getting into the pumping elements. $NPSH_{req}$ is a function of the pump design and varies with flow, speed and pump details. $NPSH_{av.}$ Is calculated from:

$$NPSH_{av} = h_{abs} \pm h_s - h_l - h_{vp} \quad \ldots\ldots\ldots\ldots..4.7$$

h_{abs}: Absolute pressure on the surface of the water in the suction reservoir (usually atmospheric pressure), m or Kpa

h_s: Suction head at the pump suction. It is positive under flooded suction condition and negative under suction lift condition. m or Kpa

h_l: Head loss due to friction, entrance, valve, etc, m or Kpa

h_{vp}: Vapor pressure of fluid at the operating temperature, m or Kpa. Vapor pressure is the pressure required to boil a liquid at a given temperature.

Absolute (atmospheric) pressure can be calculated from the following equations:

$$h_{abs} = h_b - 3.5 \text{ (Kpa)}$$

$$h_{abs} = h_b - 0.357 \text{ (m)}$$

The barometric pressure (h_b) is a function to the altitude of the pump. It can be calculated from the following table:

Table (4.2) Standard barometric pressure

Elevation (ft)	Elevation (m)	Barometric Pressure (ft)	Barometric Pressure (m)
0	0	33.9	10.3
1,000	305	32.7	9.97
2,000	610	31.6	9.63
3,000	914	30.5	9.30
4,000	1,220	29.3	8.93
5,000	1,524	28.2	8.59
6,000	1,829	27.1	8.26
7,000	2,134	26.1	7.95
8,000	2,440	25.1	7.65

And the following table used to calculate the vapor pressure. Table (4.3) Standard vapor pressure of water

Temperature (°F)	Temperature (°C)	Vapor Pressure (ft)	Vapor Pressure (m)
32	0	0.20	0.061
40	4.4	0.28	0.085
50	10.0	0.41	0.12
60	15.6	0.59	0.18
70	21.1	0.84	0.26
80	26.7	1.17	0.36
90	32.2	1.61	0.49
100	37.8	2.19	0.67

4.6 Cavitation

Since $NPSH_{av.}$ is the absolute pressure available less the vapor pressure of the liquid, the $NPSH_{av.}$ should always be greater than the $NPSH_{req}$. If this were not the case, the pressure at some point in the pump suction area will be less than the vapor pressure of the liquid, and cavitation will occur. Cavitation is the formation of pockets of vapor, or bubbles, at a point inside the pump where the liquid pressure drops below its vapor pressure. These vapor bubbles are carried along to the higher pressure area of the pump, where they collapse. It is the violent collapse of the bubbles that cause the damaging effects of cavitation; noise, erosion, and short service life. Cavitation also reduces capacity and efficiency, as well as causes pulsations in the discharge pressure. If the $NPSH_{av}$ drops below that of required by the pumps design (i.e., $NPSH_{av.} < NPSH_{req}$), the pressure within the impeller may be reduced to the vapor pressure of the fluid, Fig (4.9). If this occurs, the water will vaporize and a mixture of vapor and water will enter the pump.

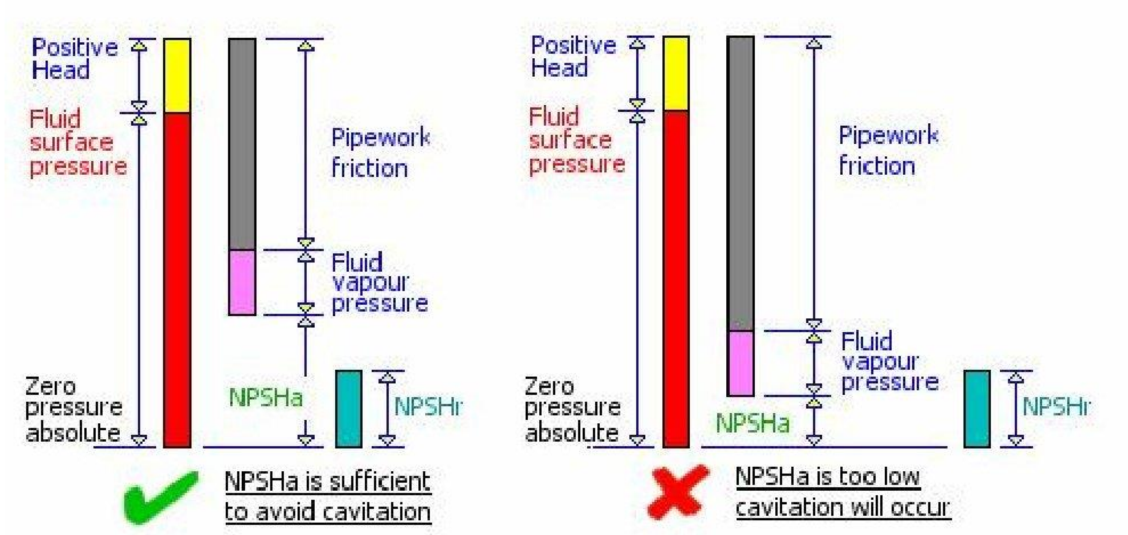


Fig (4.9) Understanding of the cavitation

Cavitation can be corrected by:

1) Increase the diameter of the pump suction pipe.
2) Decrease the pump speed.
3) Increase the static head on suction side.
4) Decrease of flow rate.

4.7 Effects of speed and diameter of impeller on centrifugal pump

The rotational speed of an impeller affects the operating characteristics of the pump. Equations 4.8 - 4.10 give the relationship of pump discharge, head and power output with rotational speed:

$$\frac{Q_1}{Q_2} = \frac{N_1}{N_2} \quad \text{……………..4.8}$$

$$\frac{TDH_1}{TDH_2} = \frac{N_1^2}{N_2^2} \quad \text{………..4.9}$$

$$\frac{P_{w1}}{P_{w2}} = \frac{N_1^3}{N_2^3} \quad \text{…………..4.10}$$

Where

N_1, N_2: rotational speed of the pump conditions, rpm

Q_1, Q_2: discharge corresponding N_1, N_2

TDH_1, TDH_2: total dynamic head corresponding N_1, N_2

Equations 4.11 – 4.13 give the relationship of pump discharge, head and power output with impeller diameter:

$$\frac{Q_1}{Q_2} = \frac{D_1}{D_2} \quad \text{…………….4.11}$$

$$\frac{TDH_1}{TDH_2} = \frac{D_1^2}{D_2^2} \quad \text{……….4.12}$$

$$\frac{P_{w1}}{P_{w2}} = \frac{D_1^3}{D_2^3} \quad \text{...................4.1}$$

4.8 Pumping stations

In general, pumping station can be classified as wet-pit or dry-pit. These classifications are based on the location of the pumps relative to the wet well or dry pit:

Wet-Pit Stations - In the wet-pit station, the pumps are submerged in a wet well involving the use of submersible pumps. The submersible pumps handle storm water very well and they allow for convenient maintenance in wet-pit stations because of easy pump removal. Submersible pumps are available in large sizes and should be considered for use in all station designs. Fig. (4.10 a)

Dry-Pit Stations - Dry pit stations consist of two separate elements: the storage box or wet well and the dry well. Storm water is stored in the wet well, which is connected to the dry well by horizontal suction piping. Dry-pit stations are more expensive than the wet-pit stations. At dry-pit stations, centrifugal pumps are usually used. The main advantage of the dry-pit station is the availability of a dry area for personnel to perform routine and emergency pump and pipe maintenance. Fig. (4.10 b).

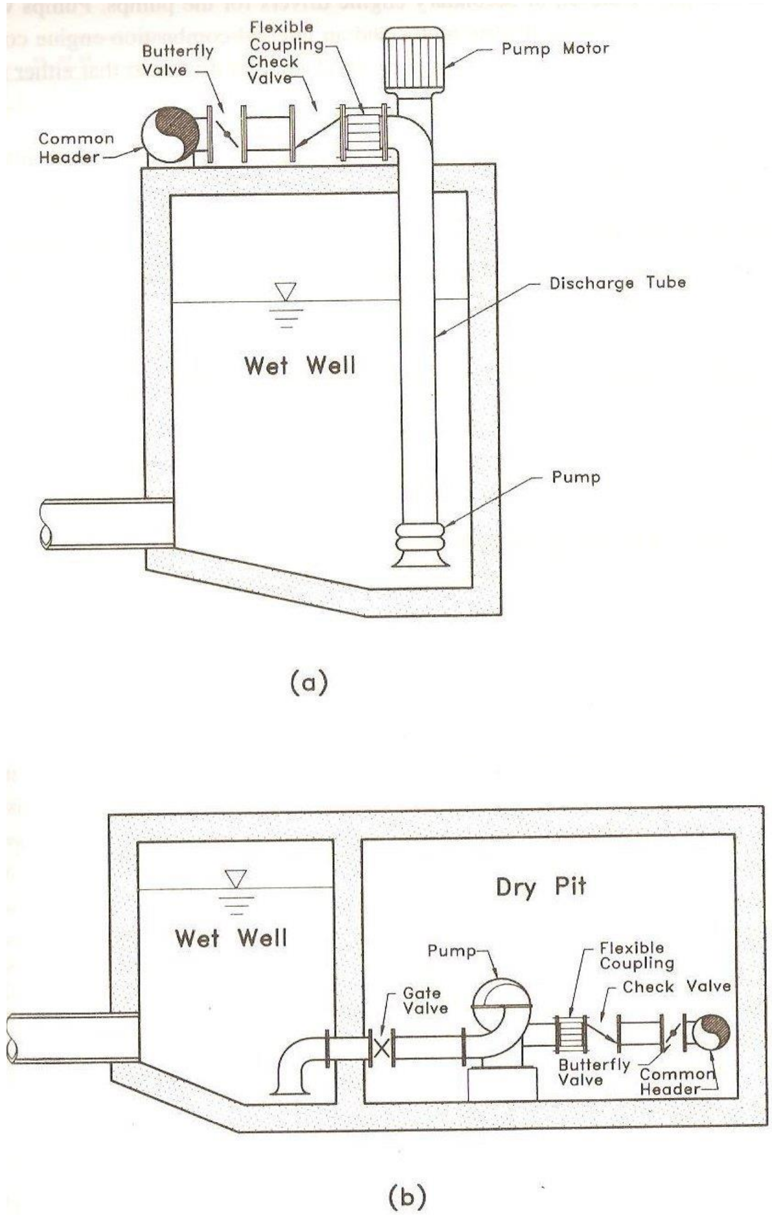


Fig (4.10) Types of pumping stations.

Exampl1

Determine the water power, pump power, and motor laod for a pump system designed to deliver 1.89 m^3/min against a total system head of 50 m. Assume the efficiency of both pump and motor is 80 percent.

$$P_W = 0.163 \times 1.89 \times 50 = 15.4 Kw$$

$$P_P = \frac{15.4}{0.8} = 19.25 Kw$$

$$P_m = \frac{19.25}{0.8} = 24.06 Kw$$

Example 2

A centrifugal pump works with two rotational speed. Draw the head discharge curves and connect between the (bep). Determine the values of head-discharge for a velocity of 1450 rpm and draw the curve . Finally, draw the working space between 60 and 120% of (bep).

	1150 rpm			1750 rpm	
Discharge (gpm)	Head (ft)	Efficiency (%)	Discharge (gpm)	Head (ft)	Efficiency (%)
0	96		0	220	
1000	93	65	1500	216	63
1500	89	77	2500	203	81
2000	82	83	3000	192	85
2200	77	84	3300	182	86
2500	70	83	3500	176	85
3000	49	71	4500	120	72

Solution:

$$Q_2 = Q_1\left(\frac{N_2}{N1}\right) = 1500\left(\frac{1450}{1750}\right) = 1240\, gpm$$

$$H_2 = H_1\left(\frac{N_2}{N_1}\right)^2 = 216\left(\frac{1450}{1750}\right) = 148\, ft$$

The other values are determined as above and listed in table below:

1450 rpm	
Discharge (gpm)	Head (ft)
1240	148
2040	140
2450	132
2730	125
2860	121
3730	82

Determine the working enclosure at 1750 rpm as 60 to 120% at bep (i.e., 3300 rpm.

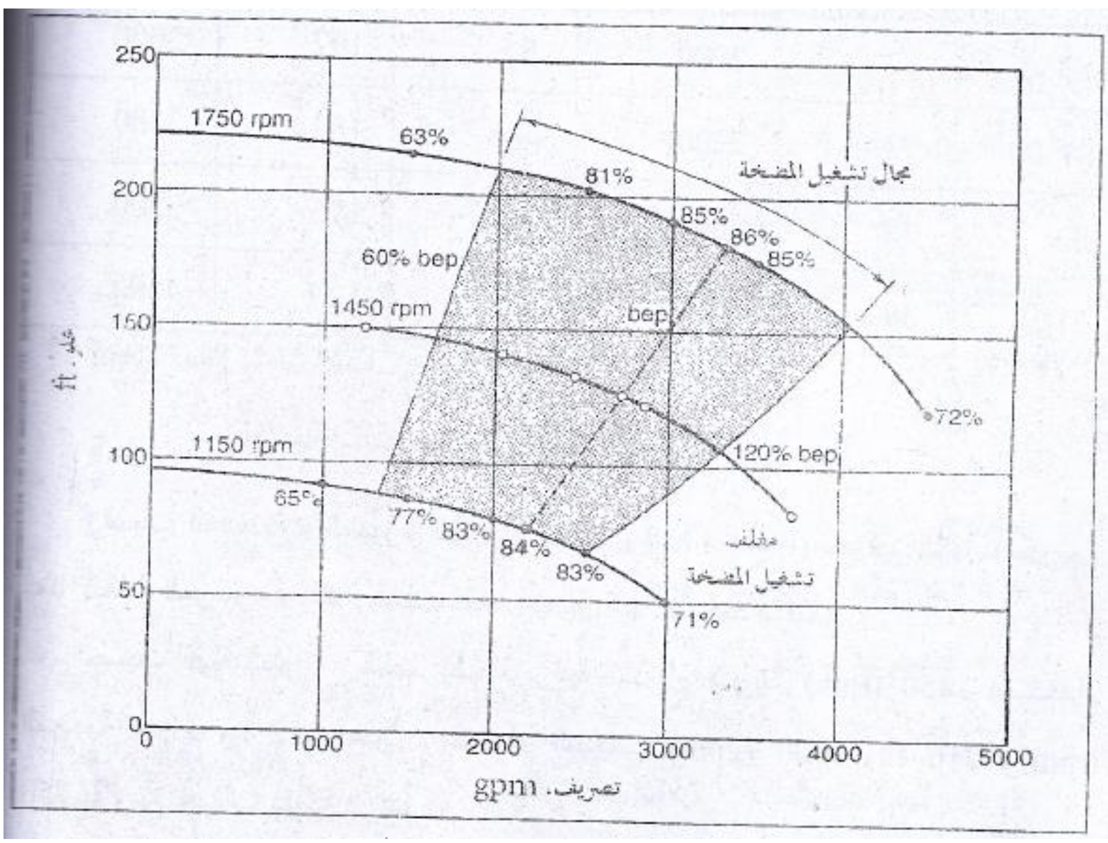
0.6 ×3300= 2000 gpm

1.20 ×3300= 4000 gpm

And at 1150 rpm, the (bep) is at 2200 gpm:

0.6 ×2200= 1300 gpm

1.2 ×2200= 2600 gpm



Example3

Assume that a water pumping station at 500 m elevation uses pumps which require 30 kpa positive suction head (NPSH) when delivering water at $30^{o}C$. What is the allowable suction lift of these pumps if the entrance and friction losses are 15 kpa?

Solution:

The barometric pressure from the table is 9.72m

$P_{atm.}$= 9.72-0.357 = 9.363m

The vapor pressure of water at $30^{0}C$ is 0.43m

$NPSH_{ava.}$=9.363-1.53-0.43-Hs=7.403-H_s

NPSH=3=7.403-H_s

Hs=42.6 Kpa

Example4

A centrifugal pump is used to raise the water from the river to the reservoir, find:

1) total dynamic head.

2) water power, pump power and motor power.

3) monthly cost pump operation, if power cost is 4 cents and pump operates 20 hr/day.

Unit Five

2019-2020

Coagulation and flocculation

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Environmental Eng

5.1 Suspended solids

Suspended solids in water include sand, soil, organic materials, bacteria, viruses and other. Typical size variations of particulates found in surface water are listed in Table (5.1). A suspension of particles that will not settled is known as a stable suspension. The particles that make up these suspension are known as colloids.

Table (5.1): Particle size found in water treatment

Material	Particle diameter (Micrometer)
Viruses	0.005 – 0.01
Bacteria	0.3 – 3
Small colloids	0.001 – 0.1
Large colloids	0.1 – 1
Soil	1 – 100
Sand	500
Floc particle	100 – 2000

The colloidal suspension may contain:

- organic materials
- metal oxides
- insoluble toxic compounds
- stable emulsions
- material producing turbidity

Colloids are commonly classified as:

· hydrophilic : Hydrophilic colloids are typically formed by large organic molecules that become hydrated (solvated) when they are present in water (e.g., proteins)

· hydrophobic : Hydrophobic colloids are made of small colloidal particles having little or no affinity for water (the solvent) (e.g., clays, metal oxides).

5.2 Characteristics of Colloids

The principal phenomena that control the behavior of the colloids are zeta potential (electrostatic force), Vander Walls forces and Brownian motion. The amount of coagulant to be added to the water will depend on the zeta potential, a measurement of the magnitude of electrical charge surrounding the colloidal particles. The *zeta potential* is the amount of repulsive force or electric charge, which keeps the particles in the water. If the zeta potential is large, then more coagulants will be needed. *Vander Wall's forces* refer to the tendency of particles in nature to attract each other weakly if they have no charge. Once the particles in water are not repelling each other, vander Wall’ s forces make the particles drift toward each other and join together into a group. Colloids have a sufficiently small mass that collusions with molecular size particles in water will cause constant movement of the colloids. The phenomenon of constant random movement of colloids is known as *Brownian* *motion*. The combination of positive and negative charge, results in a neutral, or lack of charge. As a result, the particles no longer repel each other. When enough particles have joined together, they become floc and will settle out of the water.

5.3 Theory of coagulation

Coagulation is a water treatment process that causes very small suspended particles to attract to one another and form larger particles. Then the coagulation process transfers the stable colloids particles to destabilized colloids particles by adding coagulants, as shown in Fig. (5.1).

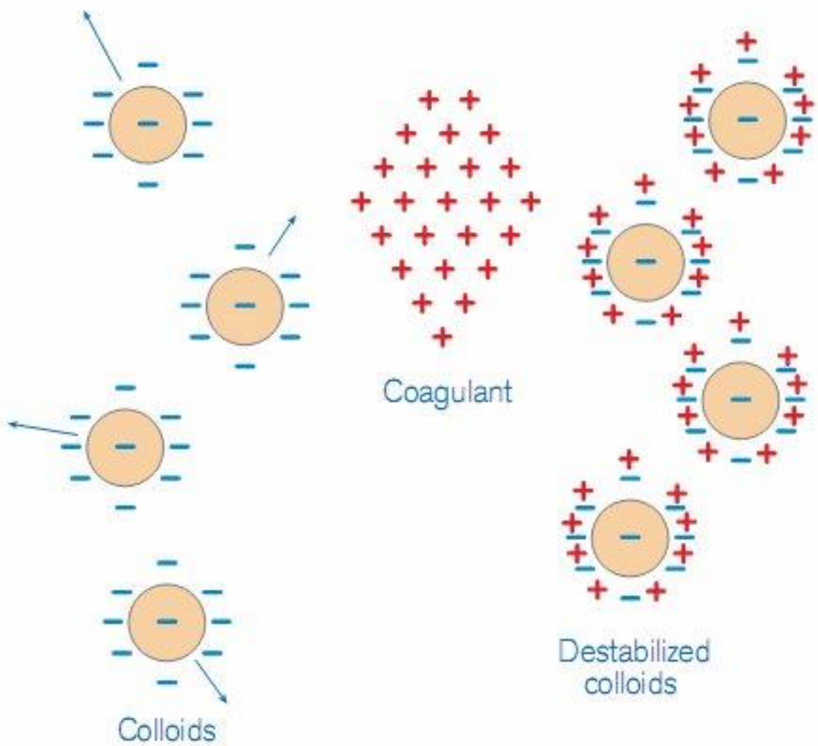


Fig (5.1) Schematic representation of destabilization of colloids

5.4 Effects of Coagulation Treatment on Wastewater

Primary Effect:

· Agglomeration and eventual removal of colloids (primarily responsible for wastewater turbidity and color)

Secondary Effects:

· Precipitation of some chemical species in solution such as phosphates, that can be present in the wastewater.

5.5 Chemicals Used for Coagulation

Coagulant reactions are carried out by the addition of coagulant, usually a metal salt to water. Commonly used coagulants are aluminum sulfate (alum), ferric sulfate $Fe_2(SO_4)_3$ and ferric chloride $FeCl_3$.

5.5.1 Use of Aluminum Sulfate (Alum)

Aluminum Sulfate is obtainable either as liquid alum or in the form of lumps. Also, it is the most widely used in drinking water. It is easily handled and not expensive.

When aluminum sulfate is added to water that contains calcium bicarbonate, the following reaction takes place:

$$\begin{array}{ccccccccccc} Al_2(SO4)_3.18H_2O & + & 3Ca(HCO_3)_2 & \rightleftarrows & 3CaSO_4 & + & 2Al(OH)_3 & + & 6CO_2 & + & 18H_2O \\ 666 & & 3*100 & & 3*136 & & 2*78 & & 6*44 & & 18*18 \\ & & \text{(as } CaCO_3\text{)} & & & & & & & & \end{array}$$

1 mg alum will produce 0.23 mg of insoluble $2Al(OH)_3$ precipitates and will consume 0.45 mg of alkalinity as $CaCO_3$.

5.5.2 Use of Ferric Sulfate

Ferric sulfate is applied with relative ease, and it produces a good flocs over a rather wide range of pH values. It has a particular advantage where manganese is present in sufficient amounts to require removal. When ferric sulfate is used, the following reaction occurs:

$$\underset{400}{Fe_2(SO_4)_3} + \underset{\substack{3*100 \\ (\text{as } CaCO_3)}}{3Ca(HCO_3)_2} \rightleftharpoons \underset{2*107}{2Fe(OH)_3} + \underset{3*136}{3CaSO_4} + \underset{6*44}{6CO_2}$$

1 mg of ferric sulfate will produce 0.54 mg of insoluble $Fe(OH)_3$ precipitates and will consume 0.75 mg of alkalinity as $CaCO_3$

5.5.3 Use of Ferric Chloride

$$\underset{2*162.5}{2FeCl_3} + \underset{\substack{3*100 \\ (\text{as } CaCO_3)}}{3Ca(HCO_3)_2} \rightleftharpoons \underset{2*107}{2Fe(OH)_3} + \underset{3*111}{3CaCl_2} + \underset{6*44}{6CO_2}$$

1 mg of ferric chloride will produce 0.66 mg of insoluble $Fe(OH)_3$ precipitates and will consume 0.92 mg of alkalinity as $CaCO_3$

5.5.4 Use of Polyelectrolytes

The term polyelectrolytes refers to all water-soluble organic polymers used for clarification, whether they function as coagulants or flocculants.

Water-soluble polymers may be classified as follows:

- anionic-ionize in water solution to form negatively charged sites along the polymer chain
- cationic-ionize in water solution to form positively charged sites along the polymer chain
- nonionic-ionize in water solution to form very slight negatively charged sites along the polymer chain

There are many coagulant aid as lime, soda ash, activated silica. Other chemical used in coagulation as sulfuric Acid, H_2SO_4, is used infrequently to acidify waters prior to treatment. Sodium Hydroxide, NaOH, which is also known as caustic soda, is an effective alkali. Also, when used to produce alkalinity, it does not increase the hardness of the water.

Sodium silicates, consisting of sodium oxide, Na_2O, and silica, SiO_2, in various proportions, are use in conjunction with aluminum sulfate to produce large floc particles. In the so-called high-rate upward-flow clarifiers, such large particles are especially desirable after being compacted and agglomerated by slow agitation.

5.6 Coagulant dosage

The selection of a coagulant requires the use of laboratory or pilot plant coagulation studies. Usually laboratory studies using the jar test are adequate for selecting a coagulant for a water treatment plant. A jar test is usually used to determine the proper coagulant and coagulant aid, if needed, and the chemical dosages required for the coagulation of a particular water. Samples of the water are poured into a series of containers. Various dosages of the coagulant and coagulant aid are added. The contents are rapidly stirred to simulate rapid mixing. Then the contents are gently stirred to simulate flocculation. After a given time, the stirring is ceased and the floc formed is allowed to settle.

The most important aspects to note are:

•The time for floc formation,
•The floc size,
•Its settling characteristics,
•The percent turbidity and color removed, and
•The final pH of the coagulated and settled water.
•The chemical dosage determined from the procedure gives an estimate of the dosage required for the treatment plant.

The jar test simulate the coagulation/flocculation process in a batch mode.A series of batch tests are run in which pH, coagulant type and dosage and coagulant aid are varied to get the optimal dosage (lowest residual turbidity). An economic analysis is performed to select these parameters. Jar tests generally are performed using 6 one-liter samples of the water to be treated. To these samples a range of coagulant (and possibly coagulant aid) dose is added (one sample is usually a blank). Immediately after the coagulant is added the samples are "flash mixed" for approximately one minute. The stirrer speed is then reduced to simulate a flocculation basin. Flocculation mode is generally maintained for about 20 minutes. At the end of the flocculation period the stirrers are turned off and the floc is allowed to settle for one-half hour. After this settling period supernatant samples are drawn off from each sample and analyzed for turbidity and sometimes alkalinity and pH.

The jar test apparatus and typical results from a jar test series might look like as shown in Figs. (5.2 a and b):

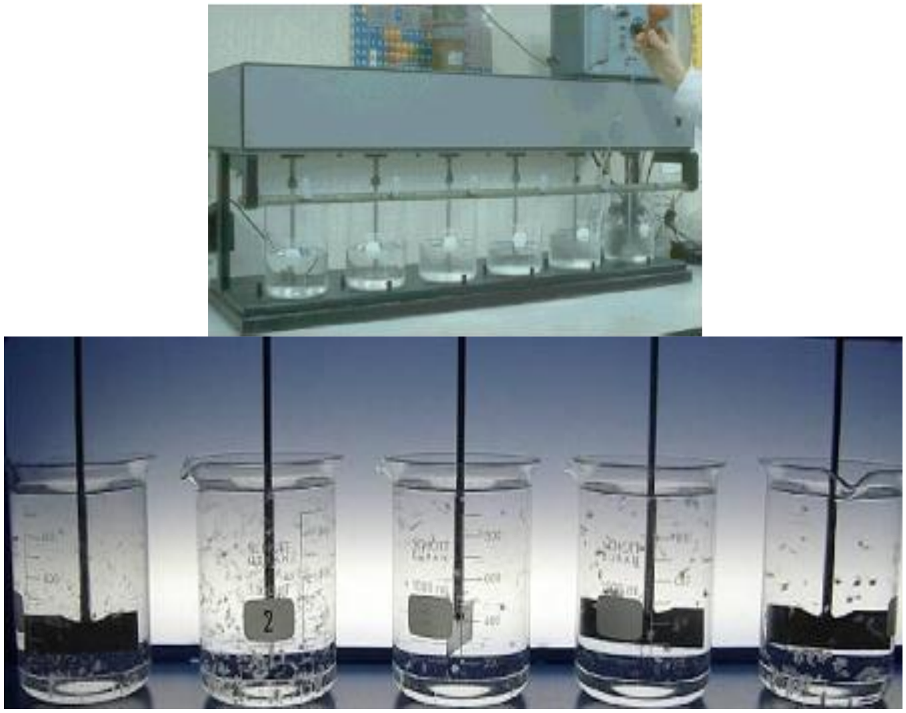


Fig.(5.2a): Jar test apparatus

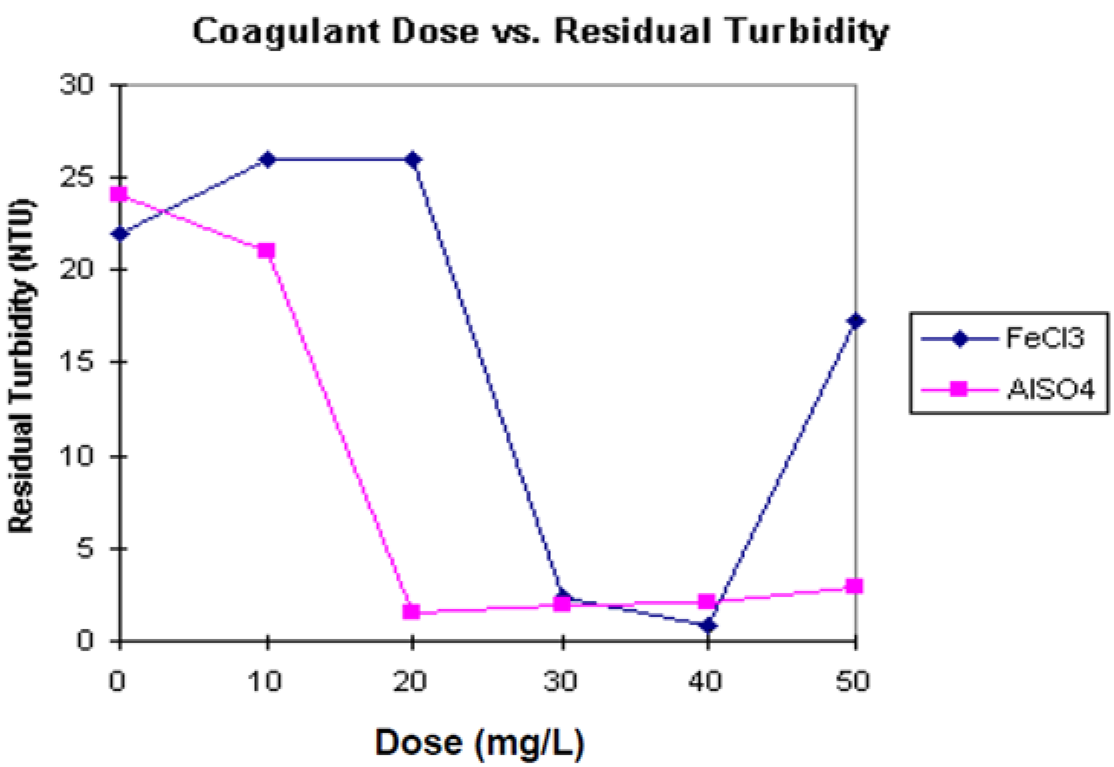
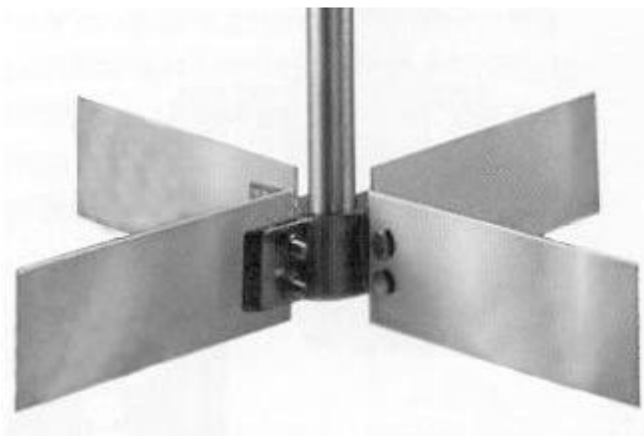


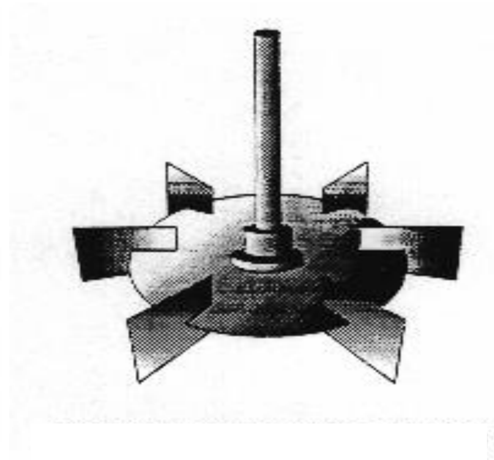
Fig.(5.2 b): Typical results from a jar test

5.6 Rapid mix

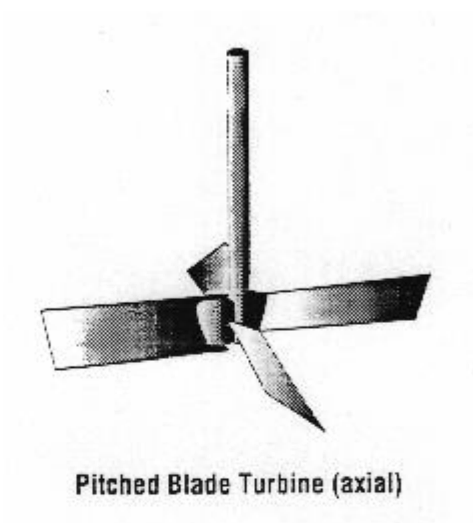
Rapid mixers should provide sufficient agitation to disperse the coagulant in raw water. Rapid mixing units can be classified according to the method of agitation *mechanical* or *static*. A mechanically agitated rapid mixer utilizes a mechanical mixer with an impeller or propeller to create turbulence in the mixing chamber. Examples of impellers and propellers used in water treatment are shown in fig (5.3). Fig (5.4) shows the mechanical radial flow.



Straight blade turbine



Disc turbine



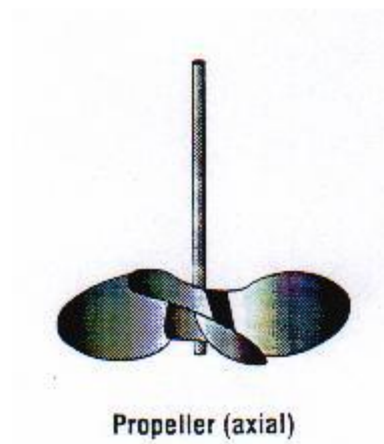
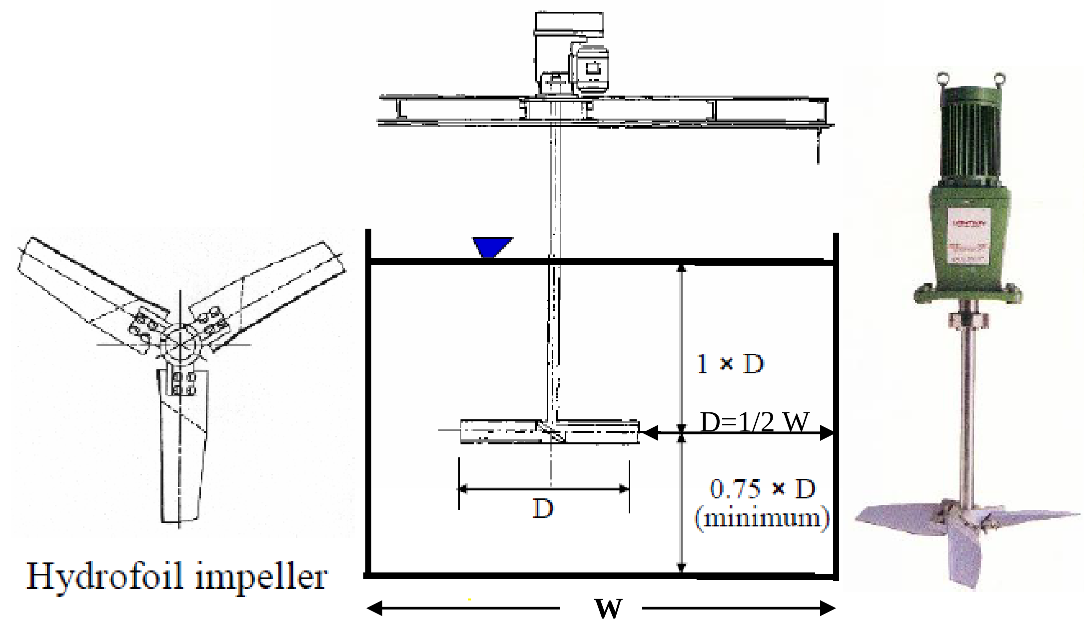


Fig (5.3) Types of impeller

Normally, the ratio between turbine diameter and the width of mixing tank range from 1/2 to 1/3, also the ratio between turbine diameter and the distance between the radial impeller and the bottom of the mixing tank equal to 1.



For static rapid mixing, arrangements are used such as channels or chambers with baffles producing turbulent flow conditions, overflow weirs, and hydraulic jumps (fig 5.5, 5.6, 5.7).

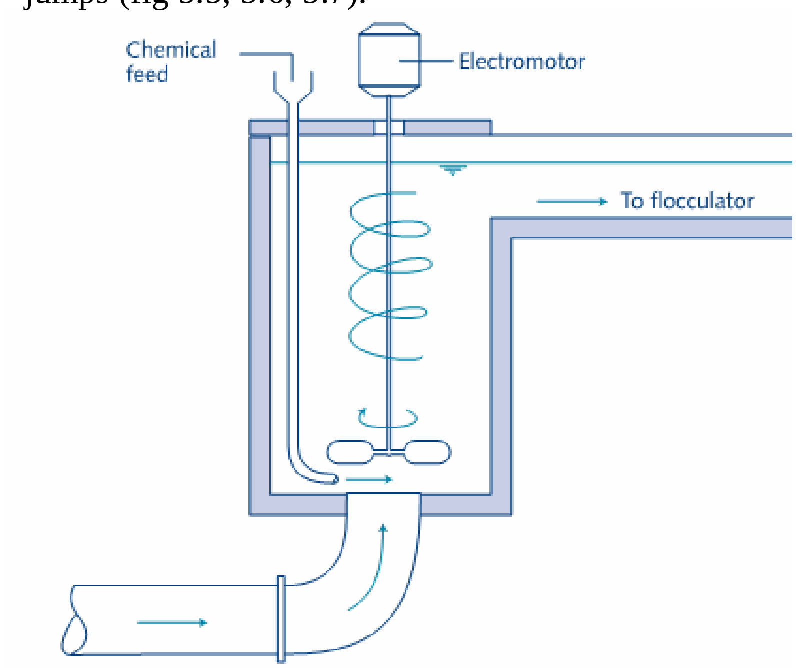


Fig (5.4) Mechanical rapid mixer

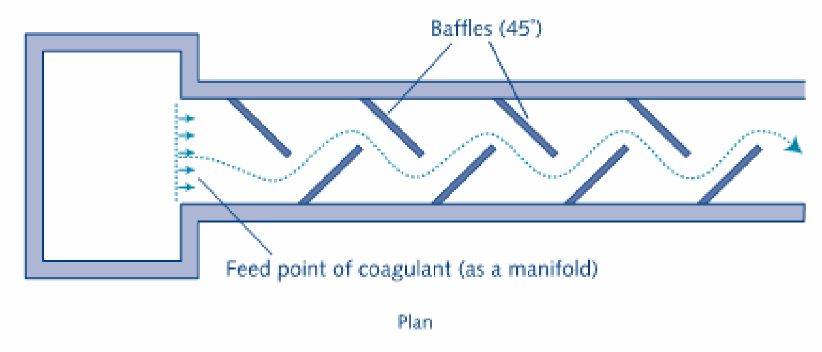


Fig (5.5) Baffled channel for rapid mixing (not very effective)

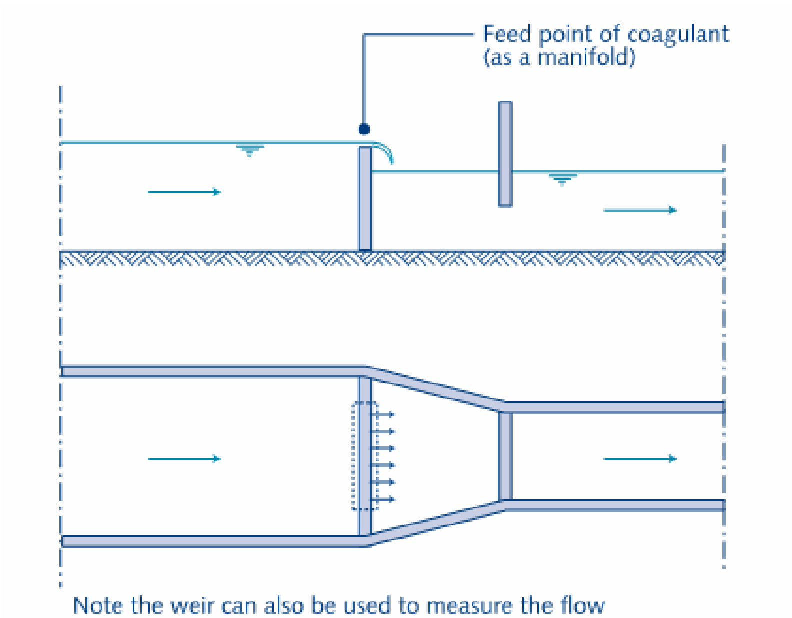


Fig (5.6) Overflow weir

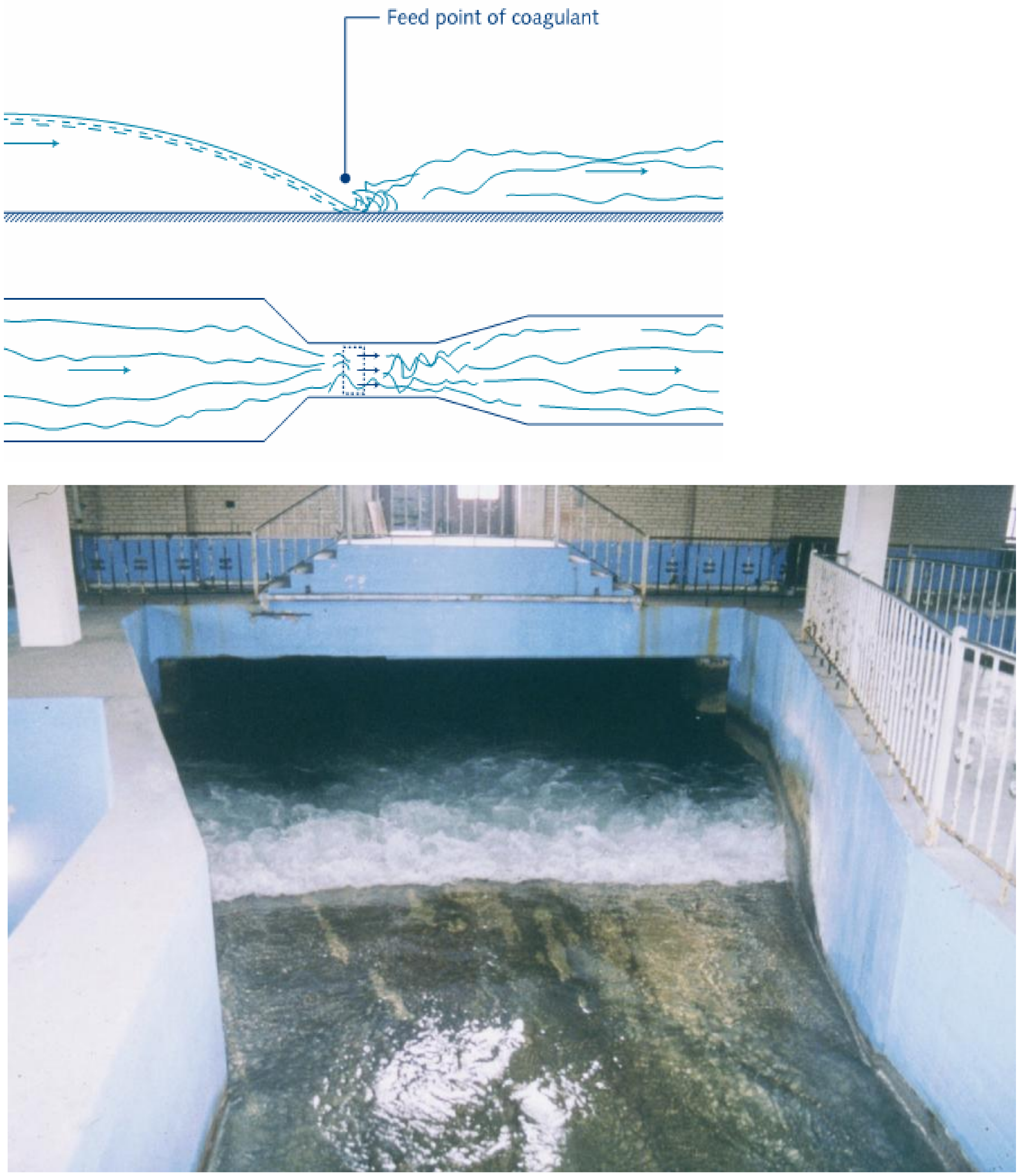


Fig (5.7) Hydraulic jump

5.7 Rapid Mix Tank Design

Traditionally, in water treatment plants, the degree of agitation in a mixing unit is measured by velocity gradient. The value of velocity gradient is given by:

$$\mathbf{G} = \sqrt{\frac{P}{\mu V}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.1$$

Where

G = velocity gradient, s^{-1}

P = power input, Watt. V = volume of water in mixing tank, m^3.μ = dynamic viscosity, Pa.s.

Some design criteria for rapid mix shown below:

G:700 <G <1000 s^{-1}

D.T: 60 to 120 s

GT (Camp number): 30000 to 60000 (D.T = GT / G)

The power imparted to the water by a mixer is calculated from:

$$p = 2\pi nT \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.2$$

Where n: impeller speed, rps T: impeller shaft torque, N.m

Other expressions for the power imparted to the water are given by:

$$p = N_p \mu n^2 d^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.3$$

$$p = N_p \rho n^3 d^5 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.4$$

Where

N_p: power number of the impeller d: impeller diameter, m

Radial flow

Straight blade turbine

4 blade (w/d=0.15)→ N_p=2.6 w/d: blade width to diameter ratio

4 blade (w/d=0.2) → N_p=3.3

Disc turbine

4 blade (w/d=0.25)→ N_p=5.1

6 blade (w/d=0.25)→ N_p=6.2

Axial flow

Propeller 1:1 pitch → N_p=0.3

Propeller 1.5:1 pitch → N_p=0.7

45^o pitched blade

4 blade (w/d=0.15)→ N_p=1.36 4 blade (w/d=0.25)→ N_p=1.94

Equation 5.3 is used for the laminar flow and eq (5.4) is used for turbulent.

$$R_n = \frac{d^2 n \rho}{\mu}$$

5.7.1 Rapid Mix Design Considerations

- Maximum Tank Volume= 8 m^3 (due to mixing equipment and geometry constraints).
- Mixing Equipment: Electric Motor, gear-type speed reducer, turbine of axial shaft impeller.
- Usually the turbine impeller provides more turbulence and is preferred in rapid mix tanks.
- The tanks are usually, baffled horizontally into two or three compartments in-order to provide sufficient residence time.
- Tanks should also be vertically baffled to minimize vortexing.
- Chemicals should be added below the impeller, point of most mixing.
- Design Liquid depth = 0.5 to 1.1 times the basin diameter or width.
- Impeller diameter is between 0.3 and 0.50 times the tank diameter or width.
- Vertical baffles extend into the tank about 10 % of the tank diameter or width.
- Impellers typically do not exceed 1.0 meter in diameter.
- Liquid depth may be increased to between 1.1 and 1.6 times the tank diameter if dual impellers on the shaft are employed. When dual impellers are employed, they are spaced about two impeller diameters apart.
- Transfer efficiency of motor power to water power is about 0.8 for a single impeller.

EXAMPLE OF RAPID MIX TANK DESIGN

A city is planning for the installation of a water treatment plant to remove iron. A low-turbidity iron coagulation plant has been proposed with the following design parameters:

$Q = 2\ m^3/s$

Rapid mix detention time, $t = 10\ s$

Rapid mix $G = 1{,}000\ s^{-1}$

Design a rapid-mix basin and size the mixing equipment.

Solution:

The volume of the rapid-mix tank by is:

$$V = Q \times t = (2\ m^3/s)(10\ s) = 20 m^3$$

Since the minimum tank volume is $8\ m^3$ is a guideline, tanks in parallel will have to be provided.

The design is also constrained by the availability of mixers and those limitations need to be evaluated.

Assume the following mixers are available:

Model	Rotational speeds, rpm	Power, kW	Model	Rotational speeds, rpm	Power, kW
JTQ25	30,45	0.18	JTQ300	110,175	2.24
JTQ50	30,45	0.37	JTQ500	110,175	3.74
JTQ75	45,70	0.56	JTQ750	110,175	5.59
JTQ100	45,110	0.75	JTQ1000	110,175	7.46
JTQ150	45,110	1.12	JTQ1500	110,175	11.19
JTQ200	70,110	1.5			

rpm = revolutions per minute
JTQ models have variable speeds from 1-45 rpm

The largest available mixer can achieve a water power of:

$$(11.19\,\text{kW})(0.8) = 8.95\ \text{kW}$$

The 0.8 is the assumed efficiency of transfer of motor power to water power. Given a G of 1,000 s^{-1}, and a viscosity at 18°C as 1.053×10^{-3} Pa·s, the required mixing can be calculated as:

$$\boxed{G = \sqrt{\frac{P}{\mu V}}}$$

$$V = \frac{P}{G^2\mu} = \frac{8.95\times10^3\ \text{W}}{\left(1{,}000\ \text{s}^{-1}\right)^2\left(1.053\times10^{-3}\ \text{Pa}\cdot\text{s}\right)} = 8.50\ \text{m}^3$$

This means that using the largest available mixer from the manufacturer, to supply 20 m^3 volume requirement the number of tanks needed are:

$$\begin{Bmatrix}\text{Number}\\ \text{of}\\ \text{Tanks}\end{Bmatrix} = \frac{\text{total tank volume required}}{\text{volume requirement per tank based on mixing}}$$

$$= \frac{20\,m^3}{8.50\,m^3/\text{tank}} = 2.35 \text{ or } 3 \text{ rapid mix tanks}$$

The volume for each tank is 6.67 m^3. For the JTQ1500 the rotational speed is 110 rpm (1.83 rps) and a turbine with six flat blades (vaned disc), the impeller diameter can be estimated as:

$$\boxed{P = K_T\,(n)^3\,(D_i)^5\,\rho}$$

$$D_i = \left(\frac{(P)}{(K_T)(n)^3(\rho)}\right)^{1/5} = \left(\frac{(8.95\times10^3\text{ W})}{(6.30)(1.83\text{rps})(1.053\times10^{-3}\text{ Pa}\cdot\text{s})}\right)$$

$$= (0.23)^{1/5} = 0.75 \text{ m}$$

Using a ratio of impeller diameter to tank diameter of 0.33, the tank diameter would be equal to:

$$\begin{Bmatrix}\text{Tank}\\ \text{diameter}\end{Bmatrix} = \frac{\text{impeller diameter}}{\text{ratio of impeller diameter to tank diameter}}$$

$$= \frac{0.75\text{m}}{0.33} = 2.27 \text{ m}$$

The surface area of the tank would be equal to:

$$\begin{Bmatrix}\text{Tank}\\ \text{surface}\\ \text{area}\end{Bmatrix} = \frac{\pi}{4}(2.27\text{ m})^2 = 4.05\text{ m}^2$$

With three 6.67 m^3 tanks, the depth of each tank would be:

$$\begin{Bmatrix}\text{Tank}\\ \text{depth}\end{Bmatrix} = \frac{\text{tank volume}}{\text{tank area}} = \frac{6.67\text{ m}^3}{4.05\text{ m}^2} = 1.65\text{ m}$$

Need to check the liquid-depth to tank ratio:

$$\left\{\frac{\text{liquid depth}}{\text{tank diameter}}\right\} = \frac{1.65 \text{ m}}{2.27 \text{ m}} = 0.73$$

This is within the guideline of 0.5 to 1.1.

Example:

A square rapid mixing basin with a depth of water equal to 1.25 times the width is to be designed for a flow of 7570 m^3/d. The velocity gradient is to be 790 s^{-1}, the detention time is 40 s, the operating temperature is 10^o C and the turbine shaft speed is 100 rpm. Determine:

1. The basin dimensions
2. Power required. Select the motor needed for the impeller if the efficiency of the motor and gearbox is 70%. Assume motors come in sizes of 1, 2.5, 5 and 7.5 hp.
   1 hp = 745.7 W or J/s.
3. Impeller diameter if a vane-disk impeller with six flat blades is employed and the tank has four vertical baffles (one on each tank)

1. Volume

= 7570 m^3/d x 1/(60x 24) (d/min) (min/60s) x 40s
= 3.5 m^3

Dimensions w x w x 1.25 w = 3.5 m^3
w = 1.41 m use w = 1.45 m
H = 1.25 x 1.45 = 1.81 m use H = 1.80 m
Total Height = 1.8 m + free board (assume 0.6 m) = 2.4 m

New volume = 1.45 x 1.45 x 1.8 = 3.785 m3
Detention time = 3.785/ (7570 x 60 x 60 x 24) = 43 s. ok

2. Power required

$P = G^2V\mu$ = $(790)^2$ x 0.00131 x 3.785 $(1/s)^2$ x (Ns/m^2) x (m^3)
= 3087 Nm/s (W)

Power of motor = 3087/(746 x 0.70) = 5.91 hp use a 7.5 hp motor

(1 hp = 746 watts)

3. Impeller diameter

$$P = \phi \rho N^3 D^5$$

$$D = \left(\frac{P}{\phi \rho N^3} \right)^{1/5}$$

$$= \left(\frac{3087}{6.30 x 999.7 x (100/60)^3} \right)^{1/5}$$

$$= 0.64 \text{ m}$$

D/w = 0.64/1.45 = 0.44 or 44% within 30 to 50% of the width (ok)

Head loss through influent pipe:

Let pipe diameter= 60 cm= 0.6 m

Velocity = Q/A= 0.087/(π/4*0.6^2)= 0.087/0.282=0.310 m/s

Kentance= 0.5, Kelbow 90=0.3, Kexit= 1

h_m= K*v^2/2g= (0.5+0.3+0.1)* 0.096/19.62= 0.004 m

Head loss over effluent weir:

$$Q = \frac{2}{3} C_d L \sqrt{2gh^3}$$

0.087=(2/3)*0.6*1.45*(19.62*h3)^0.5

h= 0.104 m

5.7 Flocculation

Flocculation is the process of gentle and continuous stirring of coagulated water for the purpose of forming flocs through the aggregation of the minute particles present in the water. It is thus the conditioning of water to form flocs that can be readily removed by settling, dissolved air flotation or filtration, Fig (5.8). The efficiency of the flocculation process is largely determined by the number of collisions between the minute coagulated particles per unit of time.

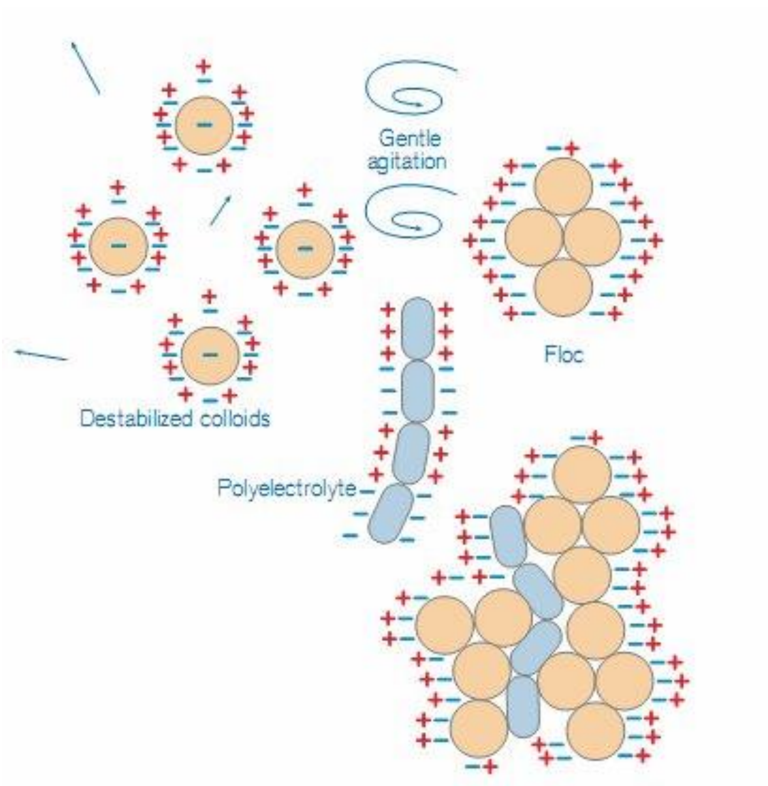
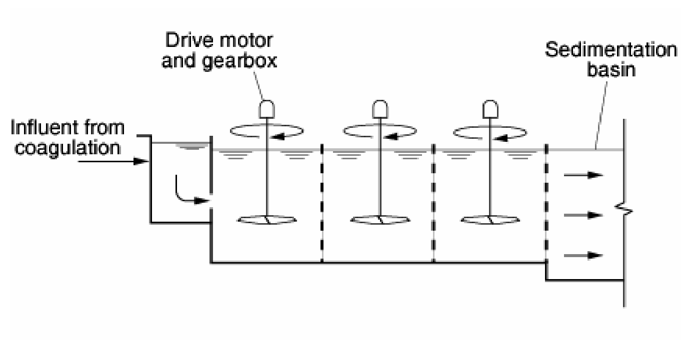
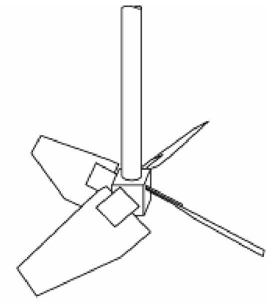


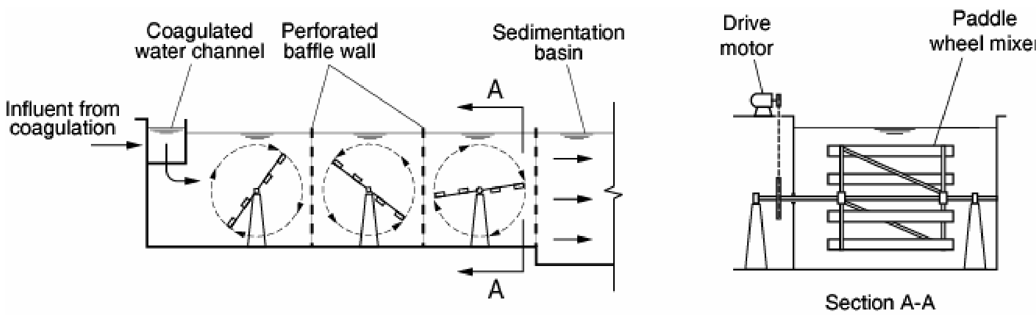
Fig (5.8) Schematic representation of flocculation process

There are mechanical and hydraulic flocculators. In mechanical flocculators the stirring of the water is achieved with devices such as paddles, paddle reels or rakes, as shown in Fig. (5.9).





Foil-Type Mixing Blade



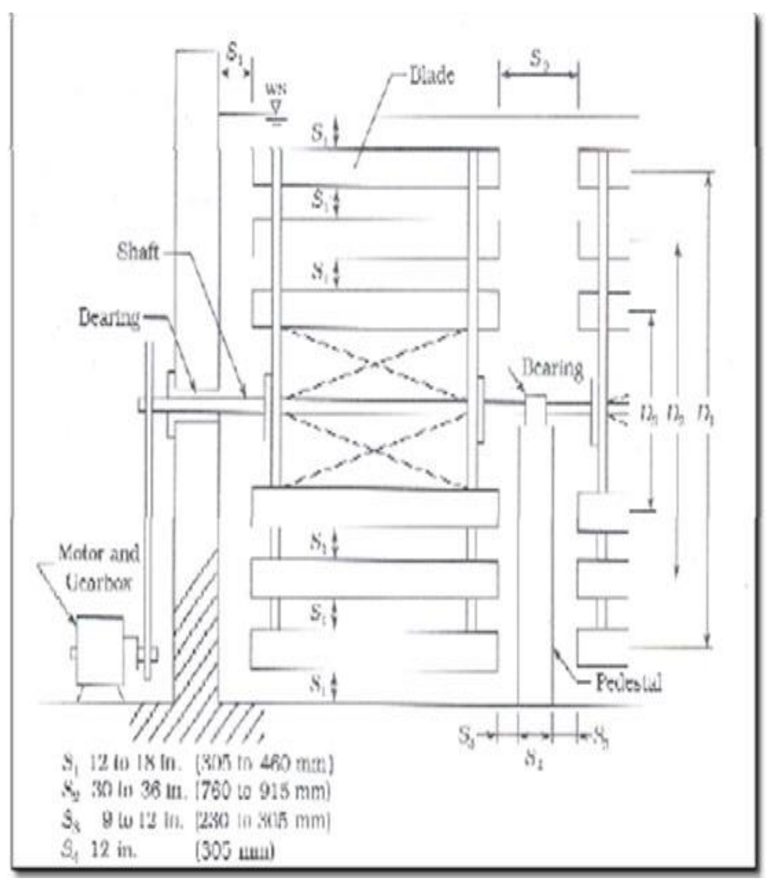
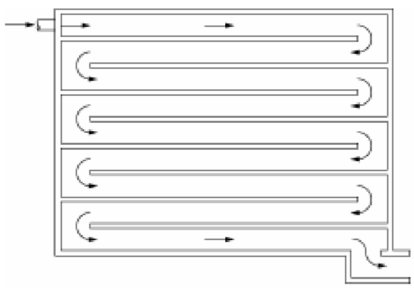
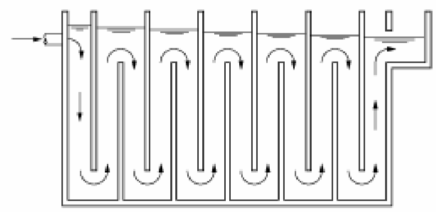


Fig (5.9) Mechanical flocculators

These devices can be fitted to a vertical or horizontal shaft. Vertical shaft flocculators are usually placed in a square tank with several chambers (four or more). With horizontal shaft flocculators having a traverse flow, one should provide at least four rows of shafts, with partitions of baffles, so as to avoid short-circuiting.
The hydraulic flocculators utilize horizontal or vertical hydraulic baffled channel as shown in fig (5.10). They are rarely used in large size of water treatment plant, because of their sensitivity to flow changes.



Horizontal Channel Hydraulic Flocculator (plan)



Vertical Channel Hydraulic Flocculator (profile)

Fig (5.10) Hydraulic flocculators

5.7.1 Agitation requirement

The detention time in the flocculation tank is higher than that in rapid mixing tank. Detention time from 20 to 30 min. Typical velocity gradient G for flocculators range from 25 to 65 s^{-1}. The velocity gradient can be obtained by using eq (5.1). In the case of paddle wheel mixers (horizontal), the water power is given by:

$$p = \frac{C_d A_p v_{prel}^3}{2} \text{......5.2}$$

Where

p: power imparted to water, watt

C_d: drag coefficient, which varies with the length to width ratio of the paddle blades. (L/W = 5)→C_d = 1.2

(L/W = 20)→C_d = 1.5

A_p: area of the paddles, m^2

v_p: velocity of the paddle relative to the water, m/s

The velocity of the paddle relative to the water is 75% of the absolute peripheral velocity of the paddle.

$$v_{pabs} = \pi n d$$
$$v_{prel} = 0.75 v_{pabs} \text{ ………………..5.3}$$

Notice: The paddles impart a velocity to the water, so the velocity of the paddle must exceed the the relative velocity. Experience has shown that the relative velocity of the water is 75% of the rotational velocity of blades.

Where

n: rotational speed of the blades, rps

d: diameter from center to center of each paddle blade, m

Design Criteria:

- The general design criteria for a basic rectangular flocculation tank are as follows:
- Energy input: Gt=10,000 to 100,000, t =5x10^4 s average, G=30 s^{-1} average, 10-70 range
- DT: 20-30 minutes at Qmax.
- Depth: 10-15’
- Stages: 3-4 common, 2-6 range
- Among the first considerations are the selection of the mode of mixing and the physical relationship between the flocculators and clarifiers. Subsequent decisions include: the number of tanks, number of mixing stages and their energy level and baffling type
- Design usually based on:

- DT
- mixing energy level (G).

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

Find:

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

Solution

a) Rotational speed, $V_p = \frac{2\pi\rho\nu}{60}$

where V_p = velocity of paddle blades, m/s
n = rpm
r = distance from shaft to centre of paddle, m

$$V_p = \frac{2\pi \text{ x } 2 \text{ x } 1.5}{60} = 0.31 \text{ m/s}$$

Speed differential = 70% x 0.31 m/s = 0.22 m/s

b) $P = \frac{C_D A \rho \upsilon^3}{2}$

where A, paddle area = 0.2 x 15 x 4 x 4 = 48 m^2

$$P = \frac{1.8x\,48x1000x(0.22)^3}{2} = 460W = 460 \text{ W}$$

(i.e. m^2 x kg x m^{-3} x m^3s^{-3} = m^2kgs^{-3} = W (kg ms^{-2} ms^{-1})

$$G = \left(\frac{P}{\forall\mu}\right)^{0.5}$$

$\forall$ = tank volume = 30 x 15 x 5 = 2250 m^3
μ = viscosity = 1cP = 1 x 10^{-3} $kgm^{-1}s^{-1}$
P = 460 W

$$G = \left(\frac{460}{2250\,x10^{-3}}\right)^{0.5} = 14.3s^{-1}$$

(i.e. m^2 kgs^{-3} x m^{-3} x kg^{-1} $ms]^{0.5}$ = $[s^{-2}]^{0.5}$ = s^{-1})

c) The retention time of the flocculator is found by dividing the tank volume by the flow rate:

$$\frac{2250\,m^3}{100\,x\,10^3\;x\,(24\,x\,60)^{-1}} \quad = \quad 32.4 \text{ min}$$

Camp no. = Gt = 14.3 x 32.4 x 60 = 27800

This value is within the normal range of 20,000 and 200,000.

Effluent structure (diffusion wall)

The diffusion wall separating flocculation and sedimentation tanks is of concrete, with circular ports. Diffusion wall is used to distribute the flow into the sedimentation tank and consequently to prevent floc breakup.

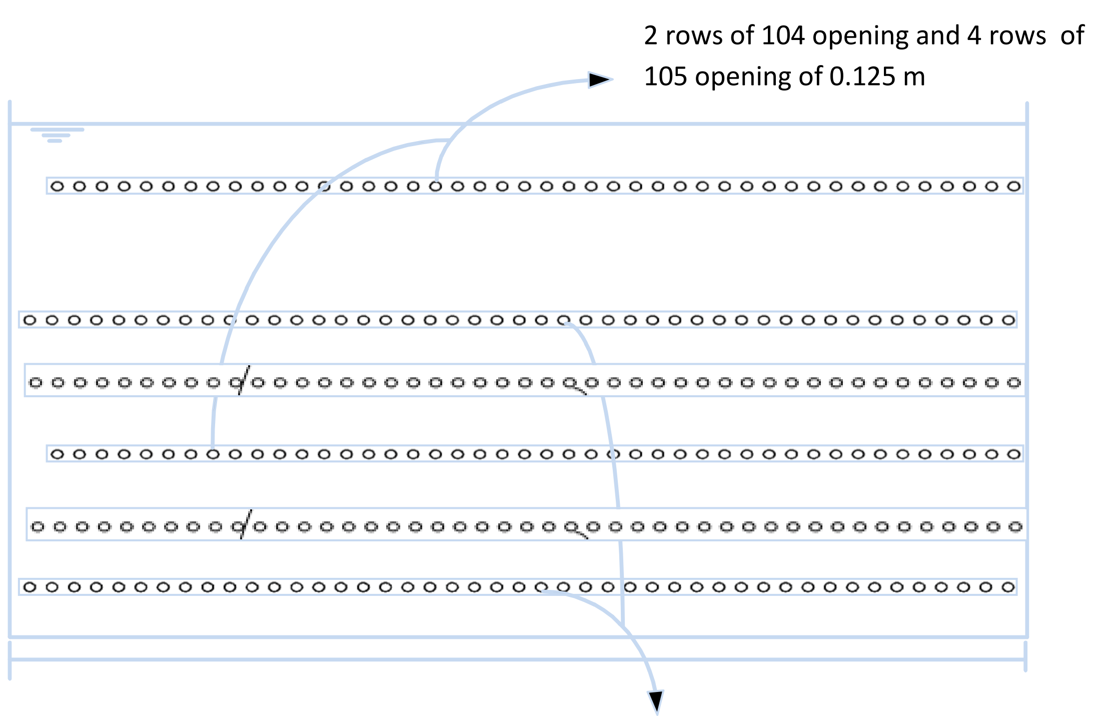
Let velocity through the port = 0.15 m/sec (to prevent floc breakup)

$\therefore$ A = Q/v = 1.157 / 0.15 = 7.176 m^2

If diameter of port = 12.5 cm

$\therefore$ area of each port = $\frac{\pi}{4}(0.125)^2 = 0.0123\,\text{m}^2$

Number of ports = 7.176 / 0.0123 = 628 provide in six rows



Example: A flocculation basin is to be designed and the design flow for the basin is 13.0 MGD. The basin is to be a cross–flow horizontal- shaft, paddle-wheel type with a mean velocity gradient of 26.7 sec^{-1} (at 50 F), a detention time of 45 min, and a GT value from 50,000 to 100,000. Tapered flocculation is to be provided in three compartments of equal depth. The G values are to be 50, 20, and 10 sec^{-1}. The basin is to have a width of 90 ft to adjoin an existing basin. The paddle wheels are to have blades with a 6-in width and length of 10 ft. The outside blades should clear the floor by 1 ft and be 1 ft below the water surface. There are to be six blades per paddle wheel, and the blades should have a clear spacing of 12 in. Adjacent paddle wheels should have a clear spacing of 30 to 36 in. between blades. The wall clearance is 12-18 in. Determine:

a. The basin dimensions (1 in. increments).
b. The paddle–wheel design.
c. The power to be imparted to the water in each compartment.

Solution

a. The basin dimensions.

$$V = \frac{13000000\,gal}{1440\,\text{min}} \times 45\,\text{min} \times \frac{ft^3}{7.48\,gal} = 54{,}311\ ft^3$$

Area of the profile section = 54311 ft^3/ 90 ft = 603.46 ft^2

Let X = Compartment length and depth

(3X)*X = 603.46 ft^2

X =14.183 ft = 14 ft- 3 in

3X = (3) (14 ft-3 in) = 42 ft-9 in

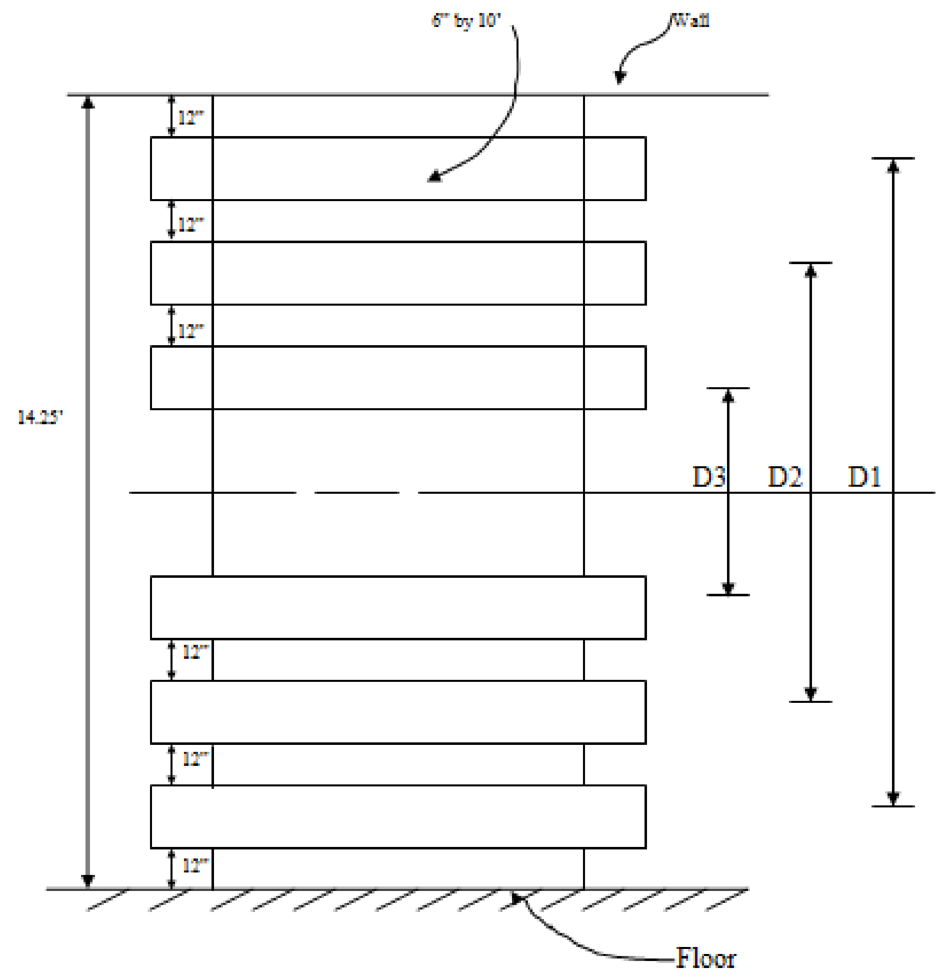
V= (14.25) (42.75) (90.0) = 54, 827 ft^3

b. Assume 7 wheels with 6 in $\times$10 ft blades.

Let the spacing between wheels be s and between wheels and wall be ½ s

7s+ 7(10 ft) = 90 ft

s = 2.86 ft or 34.3 in which is between 30 to 36 in.
Try 6 blades per wheel as shown in the drawing



D1 = 14.25' – (2)(1') – 2(3/12') = **11.75'**

D2 = 11.75'-(2)(1')- 2(3/12') –2(3/12') = **8.75'**

D3 = 8.75' – (2)(1')- 2(3/12') –2(3/12') = **6.25'**

Area of blades per shaft = (0.5'*10')(6)(7) = **210 ft^3**

Cross section of basin = (14.25') (90') = **1282.5 ft^3**

% Area = (210/1282.5)*100% = **16.4%** (between 15 to 20 %)

c. The power to be imparted to the water in each compartment

1st compartment

$P = \mu V G^2$

$P = (2.73*10^{-5}$ lb-sec/ft^2) * (50/sec)^2 * (54827 ft^3/ 3)

= 1.2473 ft-lb/sec = **2.27 HP**

2nd compartment

$P = \mu VG^2$

$P = (2.73*10^{-5}$ lb-sec/ft^2) * (20/sec)^2 * (54827 ft^3/ 3)

= 199.57 ft-lb/sec = **0.363 HP**

3rd compartment

$P = \mu VG^2$

$P = (2.73*10^{-5}$ lb-sec/ft^2) * (10/sec)^2 * (54827 ft^3/ 3)

= 49.89 ft-lb/sec = **0.091 HP**

Total HP = **2.72 HP**

Example

Given: A flocculation basin. Q=12MGD, horizontal shaft, paddle wheel. The mean G= 25s^{-1} @ 50°F. t=45min. The Gt must be between 50,000-100,000. Use 3 stages of equal depth in which the G's decrease: 45, 20, and 10. L=0.5W, L=3H. The paddles are to be made of redwood, 10'x6". The outside blade is to be 1.5' from the floor of the tank as well as from the top of the water surface. Use 6 blades/wheel and maintain a clear spacing of 12" between blades. Adjacent wheels are to maintain a clear spacing of 24-36" between blades. The wall clearance is to be between 12-18". C_D=1.50 for the paddles. Use the power equations $P=.97C_DAv^3$ and $P=\mu \cancel{V}G^2$.

Example

Treatment plant with capacity 50*10^6 l/day. This treatment plant required 20 mg/l alum. If the water alkalinity as $CaCO_3$ is 6 mg/l. Find the yearly alum quantity and the quantity of CaO which is used as coagulant aid? Atomic weights: Al=27, O=16, H=1, C=12, Ca=40

Example

Design a coagulation tank if Q=3000 m^3/day, G=700s^{-1}, D.T=60s, viscosity= 1.03*10^{-3} N.s/m^2

Unit Six

Sedimentation tank

Asst. Prof. Dr. Ahmed Hassoon Ali+ Lecturer Zainab

Environmental Engineering Department- Third Stage

Environmental Engineering Department- Third Stage

2019-2020

6.1 Definitions of Sedimentation:

It is the process of removing solid particles heavier than water by gravity force.

- Particles that will settle within a reasonable period of time can be removed using a sedimentation tank (also called clarifiers).
- Sedimentation is used in water treatment at the locations indicated in Figures 1 through 4.

6.2 Applications of sedimentation in water treatment:

1. Plain settling (or pre-sedimentation) of river surface water .
2. In filtration treatment plants treating surface water to removes flocculated solids. The sedimentation tank comes after the flocculation tank.
3. In Softening treatment plants treating hard water to removes flocculated solids. The sedimentation tank comes after the flocculation tank.
4. In aeration treatment plant removing iron and manganese from ground water.

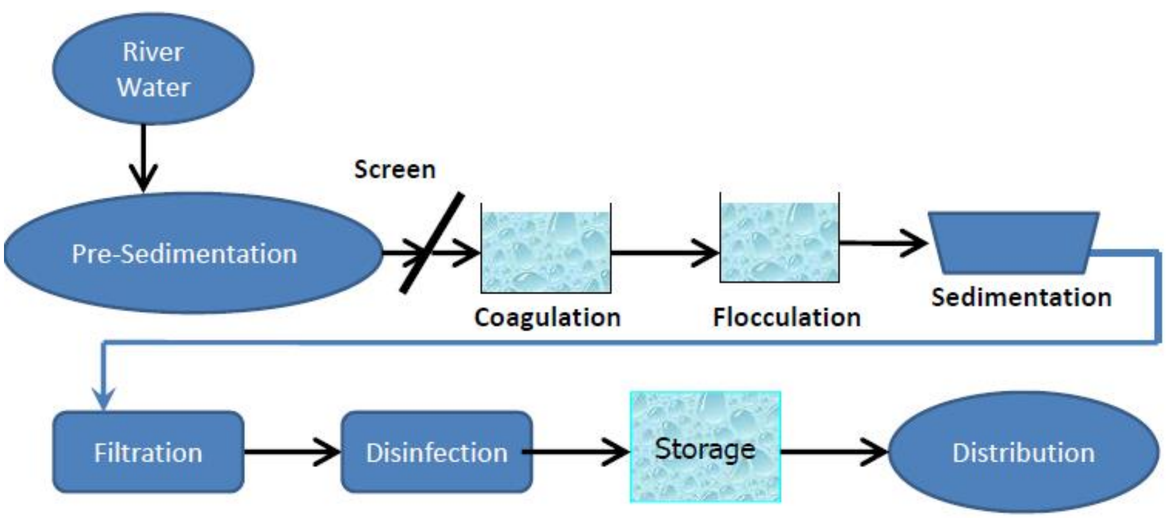


Figure 1: Filtration Treatment Plant (River Water)

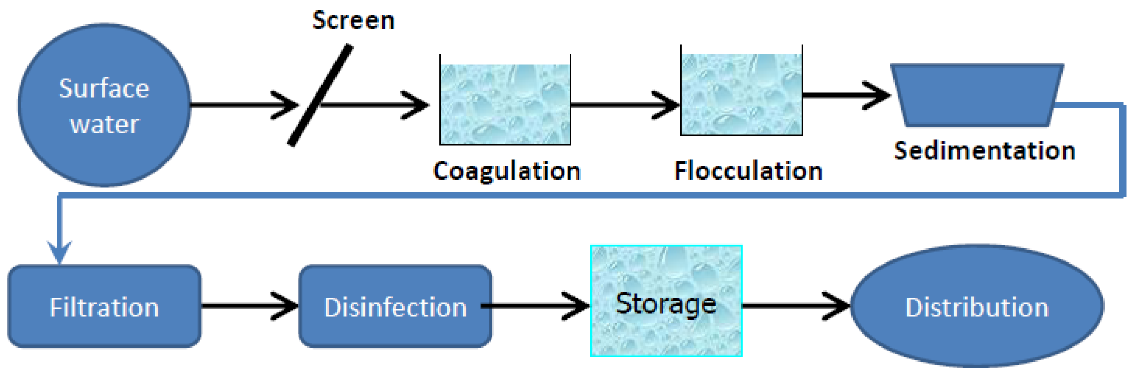


Figure 2 : Filtration Treatment Plant

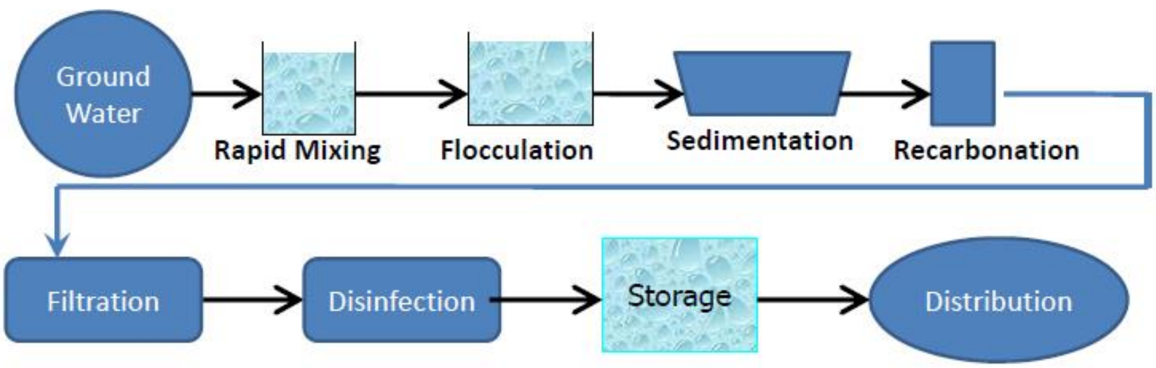


Figure 3: Softening Treatment Plant Single stage softening

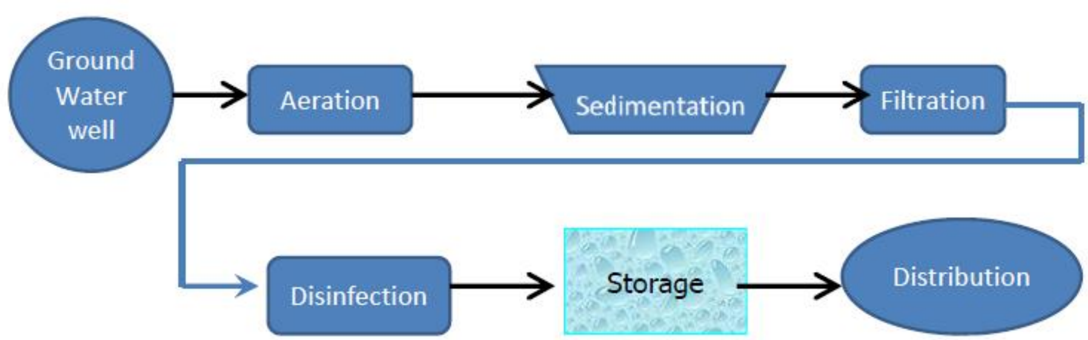


Figure 4 : Aeration Treatment Plant (iron and manganese removal plant)

6.3 Geometry of sedimentation tanks:

Sedimentation tanks are either rectangular or circular tanks:

- Rectangular basins are the simplest design, allowing water to flow horizontally through a long tank. This type of basin is usually found in large-scale water treatment plants. Rectangular basins have a variety of advantages - predictability, cost-effectiveness, and low maintenance. In addition, rectangular basins are the least likely to short-circuit, especially if the length is at least twice the width. A disadvantage of rectangular basins is the large amount of land area required.

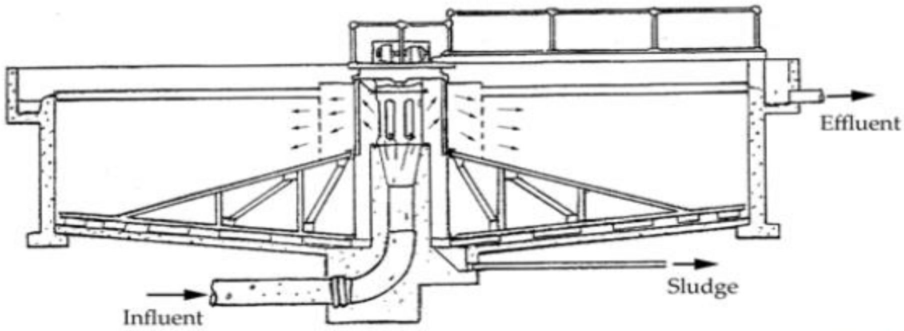
- Square or circular sedimentation basins with horizontal flow are often known as **clarifiers**. This type of basin is likely to have short-circuiting problems.

Figure 5 : Rectangular sedimentation Tank

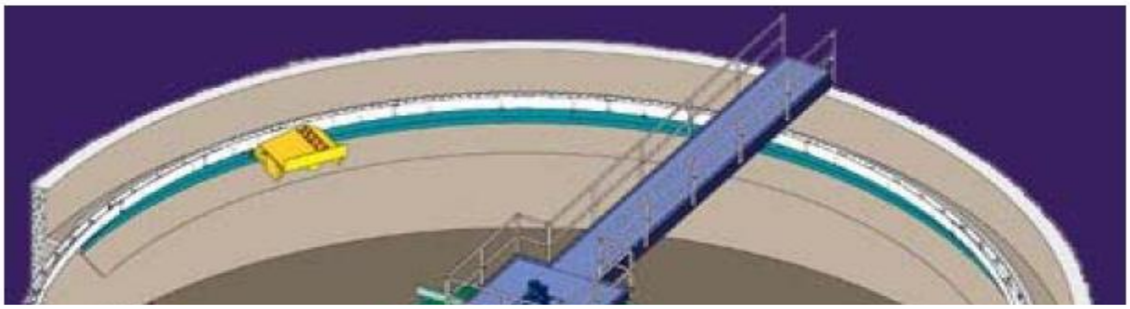
Figure 2 : Filtration Treatment Plant

Figure 6 : Rectangular sedimentation Tank

Figure 7 : Circular sedimentation Tank



A circular sedimentation basin (Taken from D.S. Sarai, 2006)



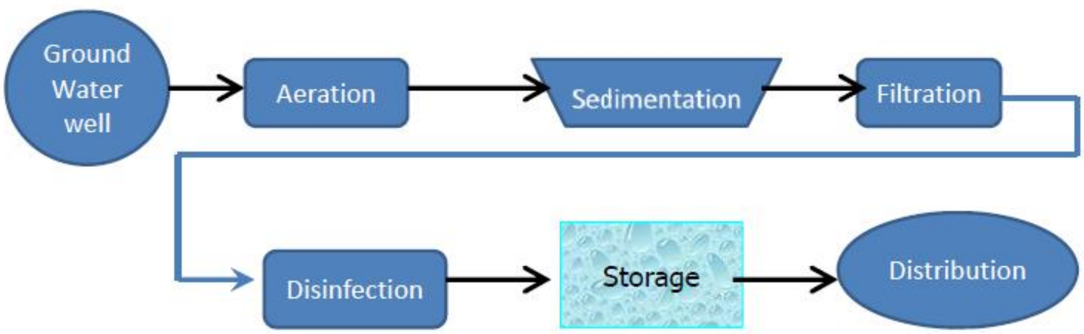


Figure 4 : Aeration Treatment Plant (iron and manganese removal plant)

6.4 Zones of sedimentationtank

All sedimentation basins have four zones - the inlet zone, the settling zone, the sludge zone, and the outlet zone, as shown in fig (6.1).

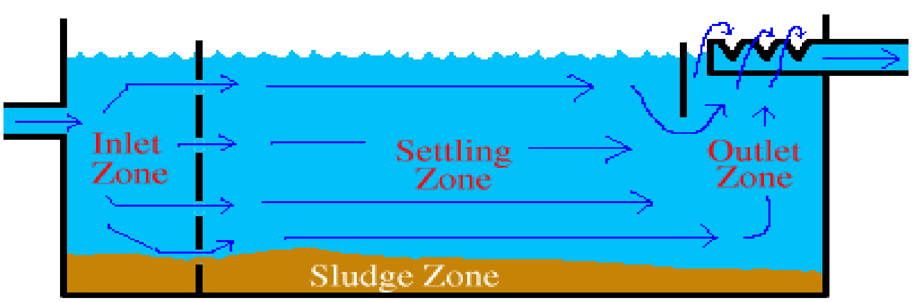


Fig (9) Zones of sedimentation tank

6.4.1 Inlet zone

The two primary purposes of the inlet zone of a sedimentation basin are to distribute the water and to control the water velocity as it enters the tank. In addition, inlet devices act to prevent turbulence of the water. The incoming flow in a sedimentation basin must be evenly distributed across the width of the basin to prevent short-circuiting. **Short-circuiting** is a problematic circumstance in which water by passes the normal flow path through the tank and reaches the outlet in less than the normal detention time.

Two types of inlets are shown below, Fig (10). The **stilling wall**, also known as a **perforated baffle wall or diffusion wall**, spans the entire tank from top to bottom and from side to side. Water leaves the inlet and enters the settling zone of the sedimentation basin by flowing through the holes evenly spaced across the stilling wall.

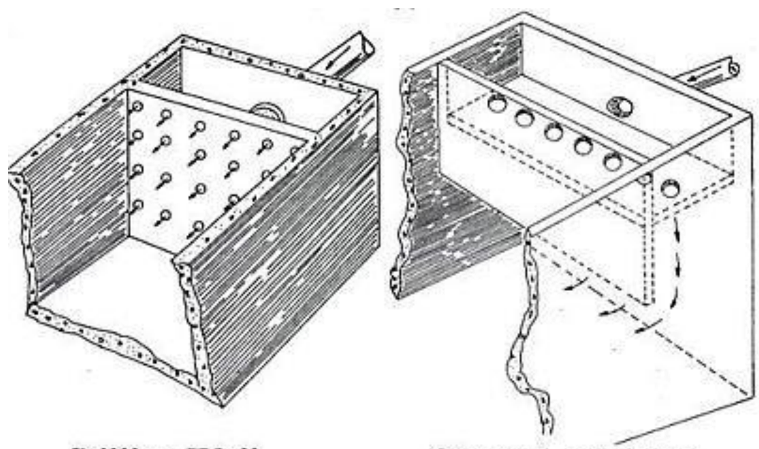


Fig (10): Types of inlet zone

The second type of inlet allows water to enter the tank by first flowing through the holes evenly spaced across the bottom of the channel and then by flowing under the baffle in front of the channel. The combination of channel and baffle serves to evenly distribute the incoming water.

6.4.2 Settling zone

After passing through the inlet zone, water enters the settling zone where water velocity is greatly reduced. This is where the bulk of flock settling occurs and this zone will make up the largest volume of the sedimentation tank. For optimal performance, the settling zone requires a slow, even flow of water. The settling zone may be simply a large expanse of open water. But in some cases, tube settlers and lamella plates, such as those shown in Fig. (11), are included in the settling zone.

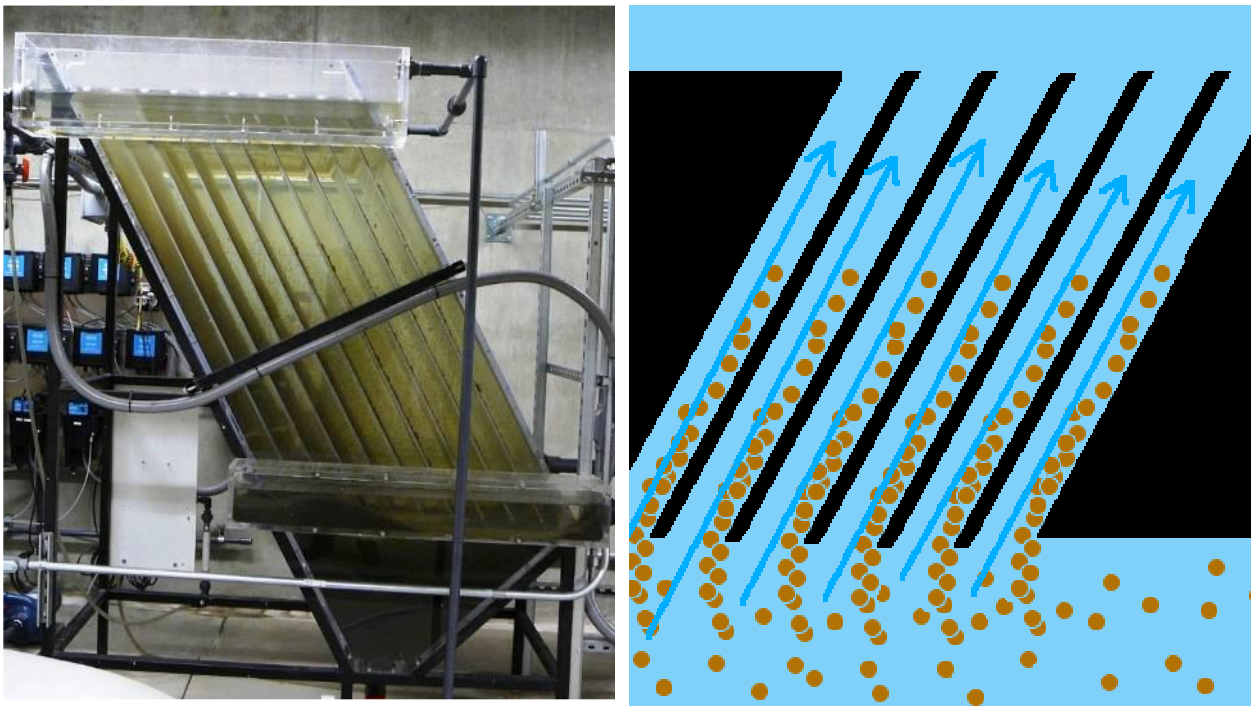


Fig (11) Tube settlers and lamella plates

6.4.3 Outlet zone

The outlet zone controls the water flow out of the sedimentation tank. Like the inlet zone, the outlet zone is designed to prevent short-circuiting of water in the basin. In addition, a good outlet will ensure that only well settled water leaves the tank. The outlet can also be used to control the water level in the basin. Outlets are designed to ensure that the water flow out of the sedimentation tank has the minimum amount of flock suspended in it. The best quality water is usually found at the very top of the sedimentation basin, so outlets are usually designed to skim this water off the sedimentation tank. A typical outlet zone begins with a baffle in front of the effluent. This baffle prevents floating material from escaping the sedimentation tank. After the baffle comes the effluent structure, which usually consists of a launder, weirs, and effluent piping. A typical effluent structure is shown in Fig. (12)

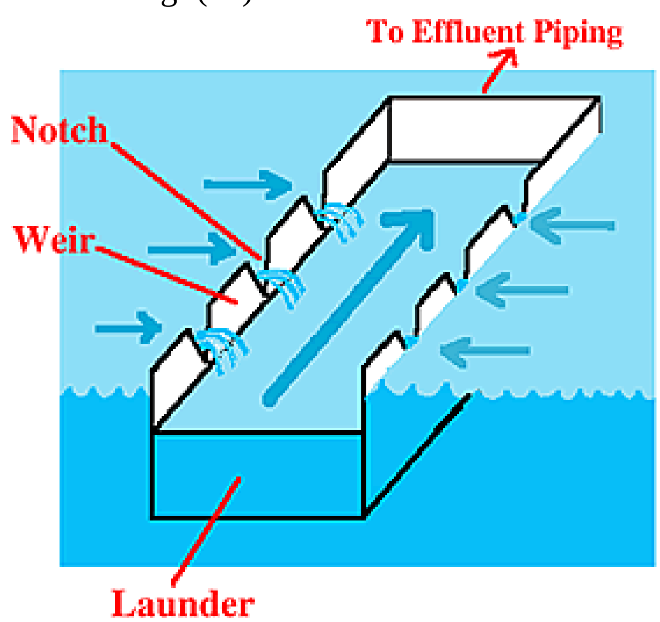


Fig (12) typical effluent structure

The primary component of the effluent structure is the **effluent launder**, a trough which collects the water flowing out of the sedimentation tank and directs it to the effluent piping. The sides of a launder typically have weirs attached. **Weirs** are walls preventing water from flowing uncontrolled into the launder. Weirs serve to skim the water evenly off the tank. A weir usually has notches, holes, or slits along its length. These holes allow water to flow into the weir. The most common type of hole is the V-shaped notch shown on the figure above which allows only the top inch or so of water to flow out of the sedimentation basin. Conversely, the weir may have slits cut vertically along its length, an arrangement which allows for more variation of operational water level in the sedimentation basin. Water flows over or through the holes in the weirs and into the launder. Then the launder channels the water to the **outlet**, or **effluent pipe**. This pipe carries water away from the sedimentation tank and to the next step in the treatment process. The effluent structure may be located at the end of a rectangular sedimentation tank or around the edges of a circular clarifier. Alternatively, the effluent may consist of **finger weirs**, an arrangement of launders which extend out into the settling tank as shown in Fig. (13).

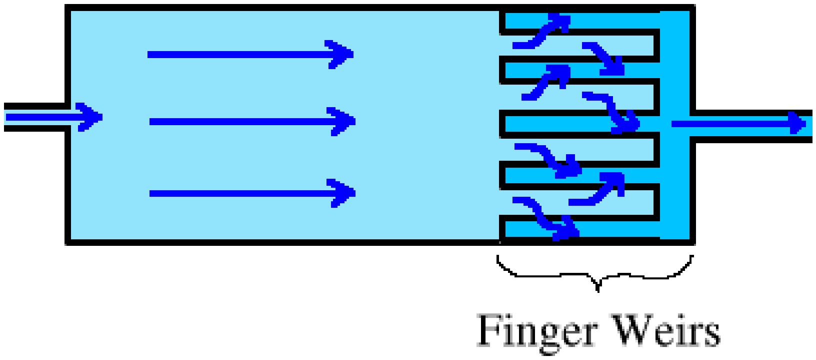
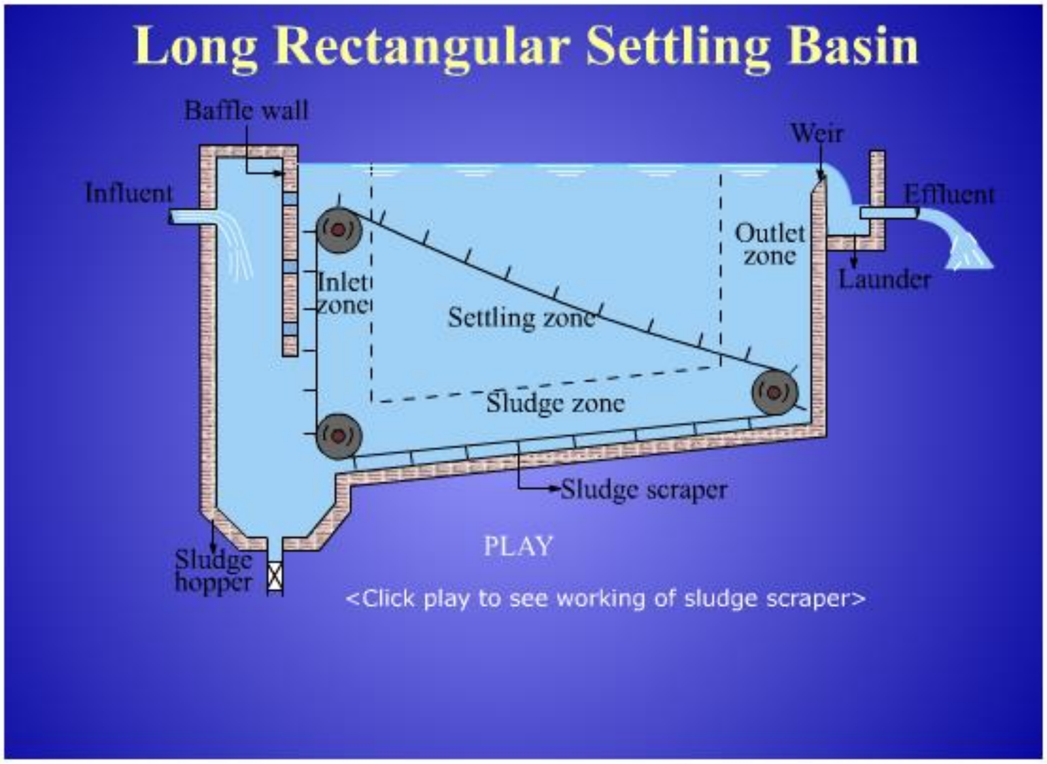
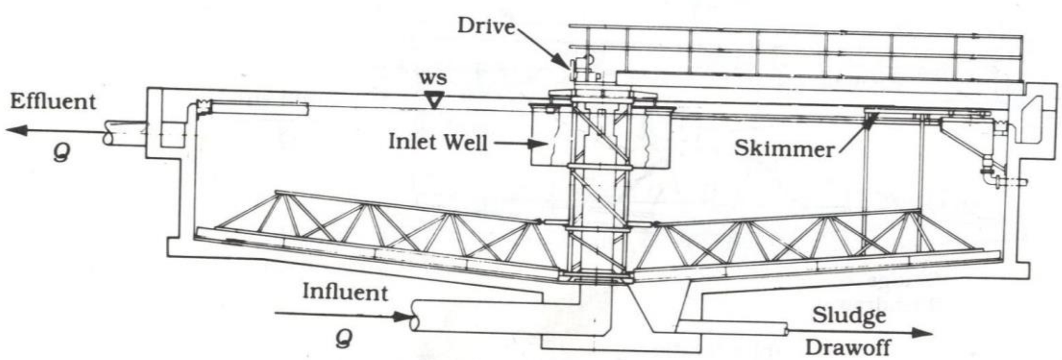


Fig. (13) Finger weirs

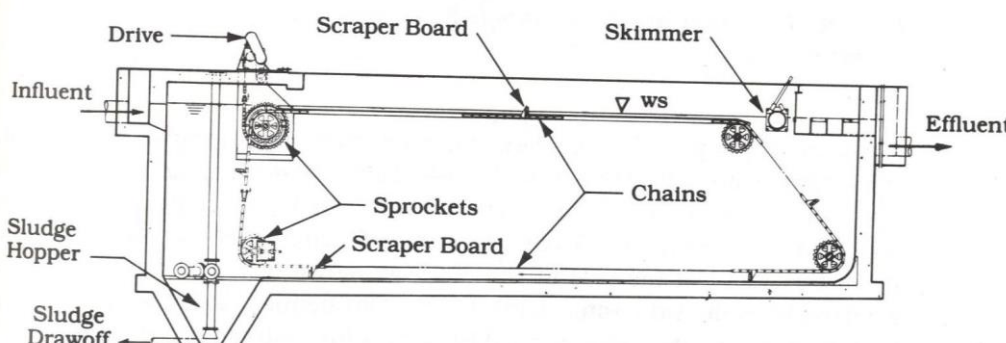
6.4.4 Sludge zone

The sludge zone is found across the bottom of the sedimentation basin where the sludge collects temporarily. Velocity in this zone should be very slow to prevent resuspension of sludge. A drain at the bottom of the tank allows the sludge to be easily removed from the tank. The tank bottom should slope toward the drains to further facilitate sludge removal. Slopes: Rectangular 1% towards inlet and circular 8%.





(b) Elevation



(b) Longitudinal Section

Fig. (14): Sludge zone

6.5 Types of Settling

With such heterogeneous wastewaters/water and variable flows during the settling process in a sedimentation tank it is possible that four types of settling may occur. In general, four types of settling phenomena have been defined . The four types of settling are described below, and shown graphically in Fig. 15.:

Type I: Discrete particle settling - Settling of particles in a suspension of low solids concentration, particles settle as individual entities, with little or no interaction with adjacent particles.

Type II: Flocculent Particles – Individual particles tend to coalesce, or flocculate, increasing their mass and settling rate.

Type III: Hindered or Zone settling – The particles tend to remain in fixed positions with respect to each other, a solids-liquid interface develops at the top of the settling mass, which settles as a unit. Occurs if biological floc develops.

Type VI: Compression settling- The concentration of particles is so high that sedimentation can only occur through compaction of the structure. Occurs in the lower sludge mass.

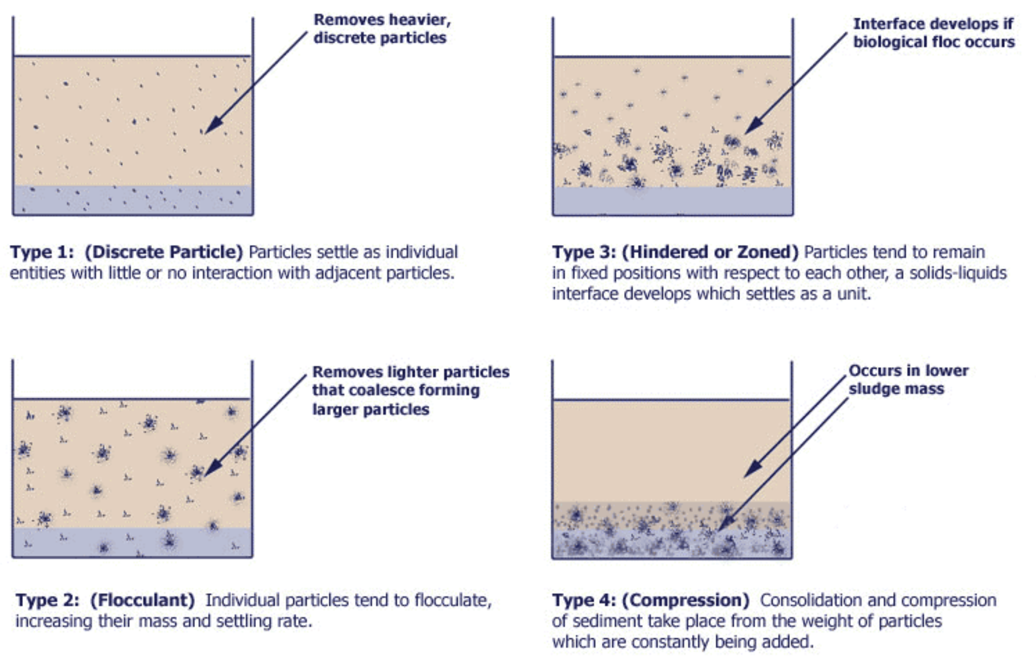


Fig. (15): Types of settling.

6.6 Sedimentation of discrete particles

To show the steps of the solution concerning the sedimentation tank, we will take a rectangular tank, as shown in Fig. (16). From this figure it can be shown that we have two phases. The first is the top view (surface view) and his area equal to:

$$A_h = L \times W$$

The second phase is the side view and his area equal to:

$$A_1 = W \times h$$

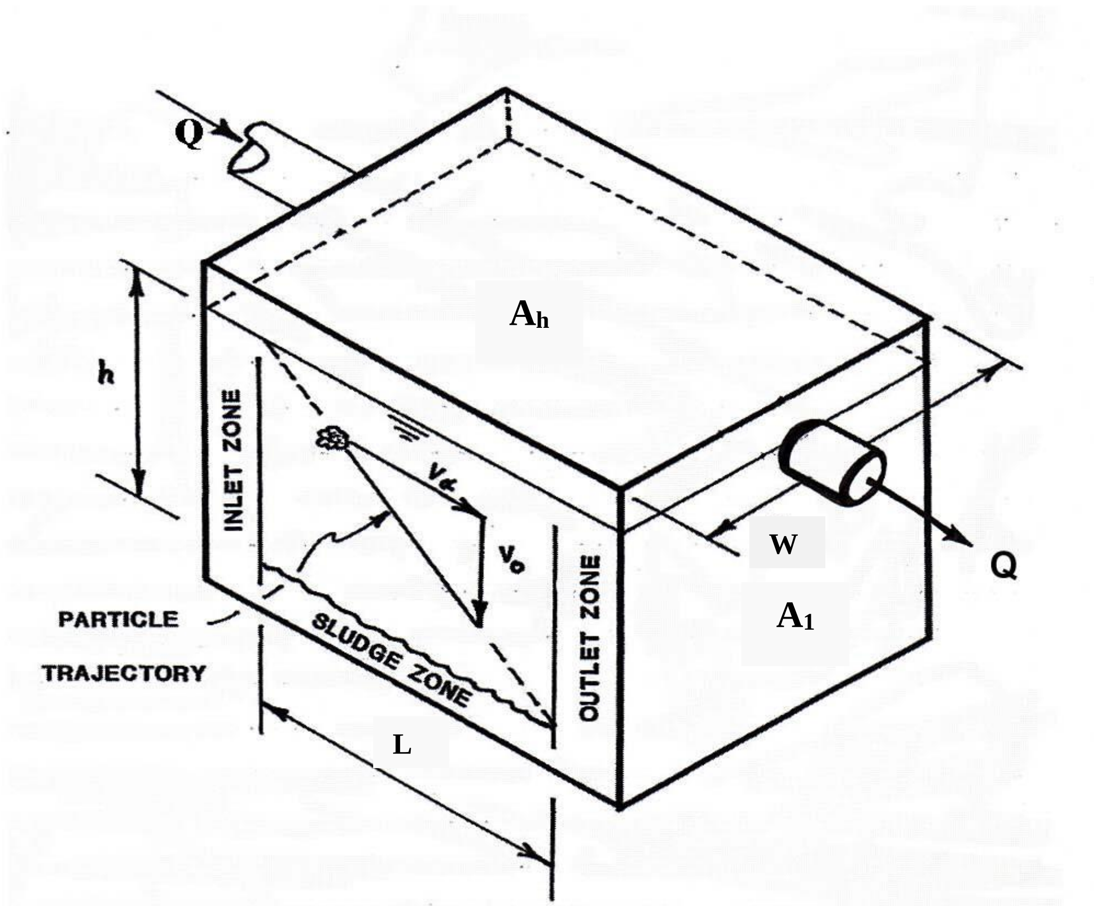


Fig (16) Rectangular sedimentation tank

6.7 Terminologyof sedimentationtank

In this section we will discuss the important parameters control the design of a sedimentation tank.

6.7.1 Surface over flow rate (SOR)

When the water contained colloidal particles enters the tank, the tank will be closed from 2 to 4 hr and then the water start to move up. This velocity is called SOR:

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

6.7.2 Horizontal velocity

From Fig (6.8), we find that the side view is responsible to calculate the horizontal velocity:

$$V_h = \frac{Q}{h \times W}$$

6.7.3 Settling velocity

This velocity refers to the velocity of particles towards the bottom of the tank. Settling velocity can be calculated from Newton's law:

$$V_s = \sqrt{\frac{4}{3}\frac{(\rho_p - \rho_w)}{\rho_w}\frac{gd_p}{C_D}}$$

In practice, it is found that C_D is a function of the Reynolds Number, R_n, and, for spherical particles, it can be represented by the following expressions

$R_n < 0.5$ (laminar flow), $C_D = \frac{24}{R_n}$

$0.5 < R_n < 10^3$, $C_D = \frac{24}{R_n} + \frac{3}{R_n} + 0.34$

$10^3 < R_n$, $C_D \approx 0.44$

Where $R_n = \rho_w v_s d_p / \mu$

For laminar flow ($R_n < 0.5$) Newton's law yields another equation which called Stoke’s Law:

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

Where

ρ_p : Density of particle

ρ_w : Density of water

d_p : Particle diameter

6.7.4 Scour velocity

It is a horizontal velocity that will cause the resuspend of the settled particles. The horizontal velocity just sufficient to cause scour has been defined as:

$$V_{scour} = \frac{8\beta(s-1)gd_p}{f}$$

Where

V_{scour}: Scour velocity

S: Specific gravity of the particles

d_p: Particle diameter

β : Constant ranges 0.04 - 0.06

f: Darcy-Weisbach friction factor, 0.02 – 0.03

$V_h < V_{scour}$

6.7.5 Detention time

It is the time required for the particle to settle. D.T is very important to find the volume of the tank. It is equal to:

$$D.T = \frac{V}{Q} \quad \text{or} \quad D.T = \frac{h}{V_s}$$

6.7.6 Percentage removal

As we know, SOR is the water velocity towards up (water surface in the tank), and the settling velocity is the velocity towards dawn (the bottom of the tank).

SOR $\uparrow$

V_s $\downarrow$

Now, if SOR>V_s the water will exit from the tank before arrival of all particles to the bottom of the tank (sludge zone), here the partial removal is occurred and his value is called percentage removal.

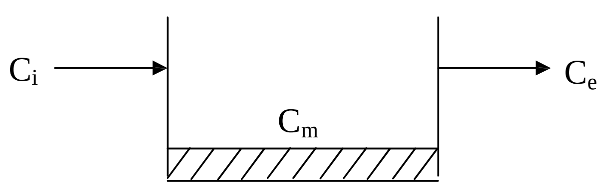
Percentage removal = $(V_s/SOR) \times 100$

If $V_s \geq SOR$, all suspended particles will be removed before the water exits from the tank, here complete percentage removal is occurred (100%).

When we want to design the sedimentation tank, we carried out that **V_s= SOR**

6.7.7 Mass of sludge

Sludge is the particles collected in the bottom of the tank.
Sludge concentration = influent Concentration $\times$ percentage removal



$C_m = C_i \times$ percentage removal
$C_e = C_i - C_m$

Mass of dry sludge = $C_m \times$ flow rate (Q)
For example, if Q = 480 m^3/day
C_m=150 mg/l

Then the mass dry sludge = (150×1000) mg/m^3 $\times$ 480 m^3/day = 72 kg/day

Mass of wet sludge = Mass of dry sludge / percentage of solid
Wet sludge volume = $W \times L \times h_s$ = Mass of wet sludge / sludge density
Where h_s is height of sludge

6.7.8 Efficiency of the tank

$$\% E = \frac{C_i - C_e}{C_i} \times 100$$

$$\% E = \frac{C_m}{C_i} \times 100$$

An ideal sedimentation tank exhibits the following characteristics which are commonly used to describe the settling behavior of discrete particles:

1) The flow through the tank is distributed across the cross section of the tank.
2) The particles are dispersed in water.
3) The settling of the particles is predominantly of type I
4) Sedimentation tank is divided into four distinct zones: inlet, settling, sludge and outlet zones. Table 6.1 lists typical values.

Table 6.1: Design parameters and operating standards

Parameter	Rectangular	Circular, Radial flow
Settling Velocity (mm/ sec)	0.1- 0.5	-----
Horizontal velocity (m/hr)	14-15	-----
Surface loading (m^3/ m^2. day)	10- 50	10-45
Retention time (hrs)	1.5- 4	1.5-4
Outflow weir loading (m^3/m. day)	100-450	100-450
Average depth (m)	1.75 – 3.0	1.5 – 2.5
Plan dimensions (m)	Up to 100 long; length: width from 4:1 to 5:1	3.3 – 30 diameter.
Base slope	1:25 to 1:100; 22.3 deg to 0.6 deg.	1:6 to 1:8. 7.5 deg to 10 deg.

Example

Water contains particles with 0.5 mm diameter and specific density of 2650 kg/m^3. If water density = 998.2 kg/m^3 and viscosity = $1.002*10^{-3}$ N.s/m^2. Find the settling velocity and the diameter of the circular tank, if the flow rate = 0.5 m^3/s, and D.T = 2 hr, use two tanks.

Example

Raw water ($Q=5450\ m^3/d$) pass through the sedimentation rectangular tank with the width equal to 6 m. The particles density equal to 2000 kg/ m^3 and kinetic viscosity of water equal to $1.14*10^{-6}\ m^2/s$. Drive the equation between the diameter of particles and the length of the tank required, and draw the curve?

Example

A rectangular sedimentation tank with W=25m, L=50m and h=3m. The flow rate = 1500 m^3/h, and the influent particles concentration = 2000mg/l. The following table shows the percentage of particles as a function of his diameter:

Volume of particle (mm)	%
0.001	10
0.005	20
0.01	15
0.015	15
0.02	20
0.025	20

Find:

1) Settling velocity, if particles density = 2650 kg/ m^3 and viscosity of water = $1.002*10^{-3} N.s/m^2$
2) Percentage removal for each particle and total percentage removal
3) Sludge depth, if the tank is cleaned one time per month. If percentage of solide = 8% and sludge density = 1150 kg/ m^3
4) Effluent concentration particles

Example

$Q=1000 m^3/d$, D.T= 2hr

D_p= 0.06 mm

$\rho_p=1200\ kg/m^3$, $\mu=1.027\times10^{-3}\ N.s/m^2$

$V_{scour}=V_h/70\%$, find L,W,h if L=4W

6.8 Fraction of particles removal

The sedimentation tank is designed so that all particles having a settling velocity greater or equal SOR are fully removal. Particles having a settling velocity (v_i) smaller than the SOR are partial removal, Fig. (17). The velocity v_i is expressed by:

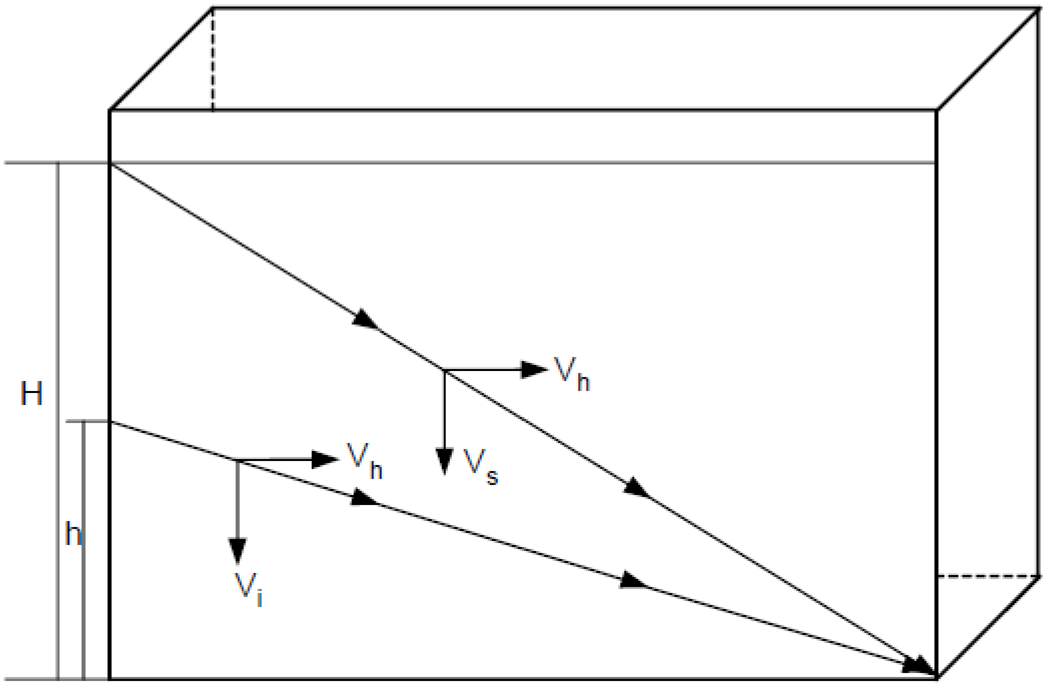


Fig (17) Settling behavior of discrete particles

$$V_s = \frac{H}{D.T}$$

$$V_i = \frac{h}{D.T}$$

In a typical river water suspension sample, a large gradation of particles size occurs. To determine the removal efficiency at a given settling time, it is necessary to consider the entire range of particle settling velocities and the fractions that are removed. Therefore the total removal efficiency of a settling tank can be determined by *1) a batch settling test or 2) a sieve analysis*. The batch settling test employs a settling column, as shown in Fig. (18). The test sample is placed in the column and samples are taken at timed intervals, usually each 30 to 60 sec for 5 min and then every 1 to 2 min for the remainder of the test. The test is continue from 30 min to 2 hr. The weight of suspended solids is measured for each sample and expressed as a fraction of solids remaining.

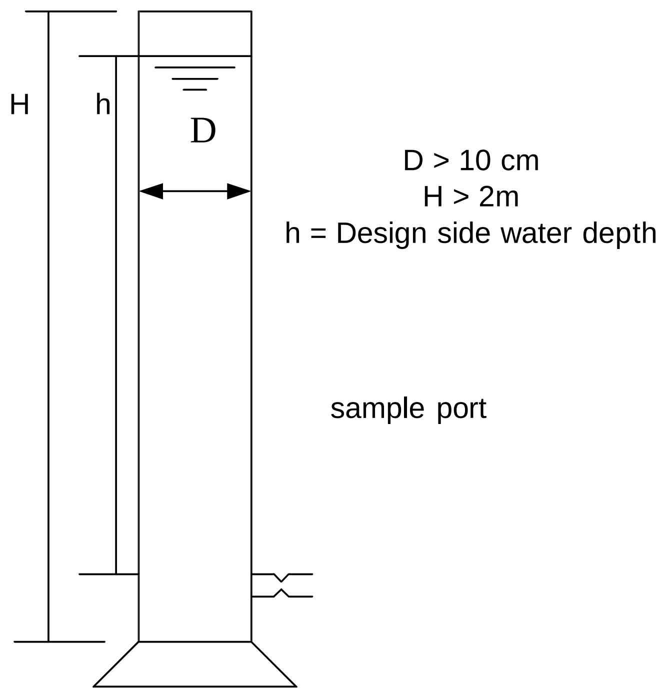


Fig (18) Standard settling column used for discrete settling test

The sieve analysis technique requires a sample of dry solids that constitute the suspension to be removed in the tank. With either method, a settling velocity analysis curve is developed as shown in Fig (19). The total solids removal efficiency is then determined by graphically integrating the area under the curve. The fraction of all particles removed will be:

$$f = (1 - X_s) + \frac{1}{V_s}\int_0^{X_s} V_i$$

$$f = (1 - X_s) + \frac{1}{V_s}\Sigma V\Delta X$$

Where: f: fraction of particles removal
X_s: fraction of particles with $V<V_s$
$(1-X_s)$: fraction of particles with $V \geq V_s$

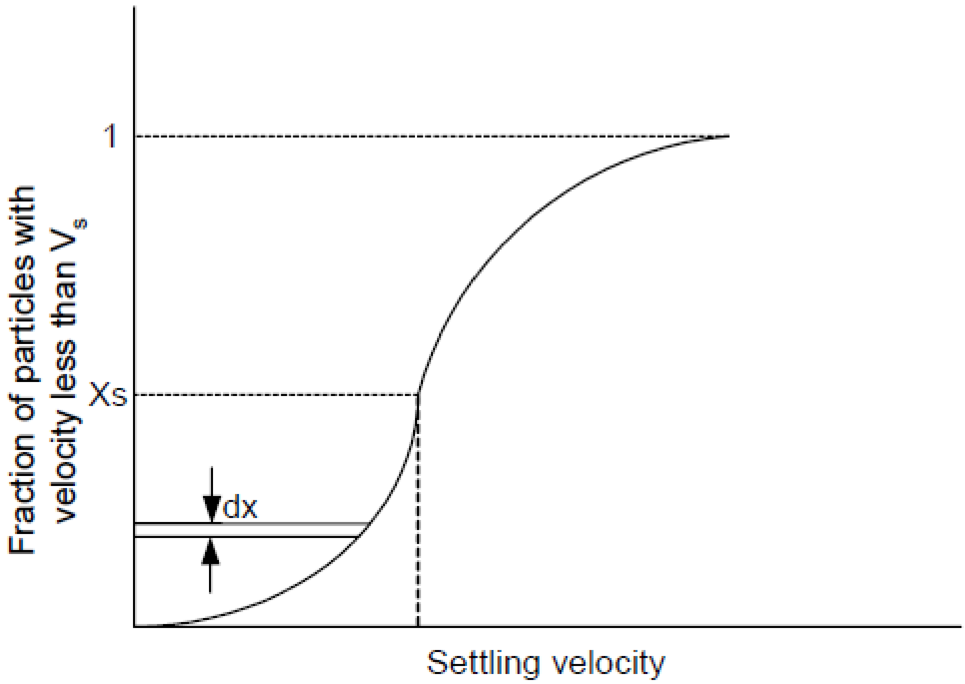


Fig (19) Cumulative distribution of particle settling velocity

Example:

Given: A settling basin is designed to have a surface overflow rate of 32.6 m/day = 0.37mm/s (800gpd/ft^2). Find the overall removal obtained for a suspension with the size distribution given below. The specific gravity of the particles is 1.2 and T=20°C. μ=1.027, ρ=0.9997

Particle size, mm	0.10	0.08	0.07	0.06	0.04	0.02	0.01
Weight fraction greater than size, percent	10	15	40 40% of the particles > .07	70	93	99	100 100% of the particles > .01
Weight fraction less than size, percent	90 90% of the particles pass the .10 sieve	85	60	30	7 7% of the particles pass the .04 sieve	1	0

Sample calculations for the table below:

v, Stokes Law:

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

v = 107.62d^2

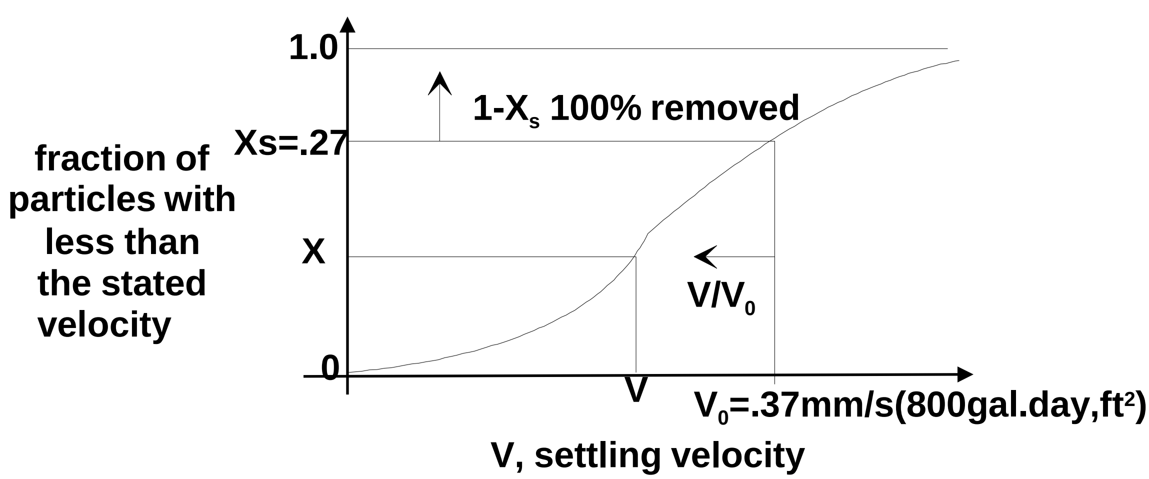
for d=.10mm

v = 107.62(.10) ^{2}

v=1.076 say 1.08
for d=.04
$v = 107.62(.04)^2$
v=0.172mm/s
Reynolds number, if the $R_e < .5$, Stokes Law applies.
$R_e = \rho_w v_s d_p/\mu$, = (0.10mm x 1.08mm/s) / $1.011\text{x}10^{-6}$m/s x $(1000\text{mm/m})^2$
R_e =0.10

Weight fraction, %	10.0	15.0	40.0	70.0	93.0	99.0	100
v, mm/s, from above calc.	**1.08**	**0.689**	**0.527**	**0.387**	**0.172**	**0.043**	**0.011**
Nr,	0.1	0.05	0.04	0.02	0.01	0.001	0.0001
Weight fraction remaining %	**90.0**	**85.0**	**60.0**	**30.0**	**7.0**	**1.0**	**0**

Plot the above vs. weight fraction remaining , e.g. 1.08, 90.0; 0.689,85 etc.



All particles with a settling velocity greater than .37mm/s will be 100% removed. From the graph, the fraction $(1\text{-}X_s)$ is equal to 0.73 or 73%; a portion of the remaining 27% will be removed, graphically this is the area above the settling curve, but below the Xs line. One way to obtain this desire area is to assume increments of Δx, say 0.04, and pick off the corresponding v, velocity, from the graph. The resulting product Δx(v) is the area for that increment. The increments are then summed to obtain the total area.

Δx	0.04	0.04	0.04	0.04	0.04	0.04	.027
v	0.06	0.16	0.22	0.26	0.30	0.34	0.37
Δx(v)	0.0024	0.0064	0.0088	0.0104	0.0120	0.0136	.0099

Total Δx(v) = .0635

The overall removal is:

fraction removed = $(1-X_s) + \dfrac{1}{SOR}\int_0^{X_s} v\,dx$

fraction removed = 0.73 + 1/.37(0.0635)

fraction removed = 0.898 = 89.9%

6.9 Flocculent Suspension

Settling initially starts as Type I (discrete), but the particles coalesce (flocculate) during settling.

Flocculation leads to a change in size, shape and weight as they settle $\longrightarrow$ Vs changes with time.

Paths of the particles are curved (not linear as in discrete settling).

We can perform batch tests to obtain the data required to size primary sedimentation basins where flocculation is occurring.

Use a batch settling column with a height equal to the depth of the clarifier that is to be designed.

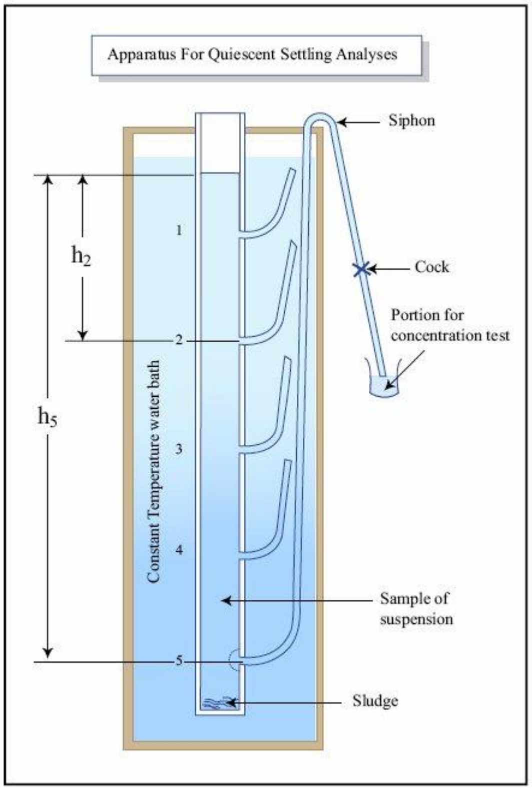


Fig (20) Flocculent column used for flocculent suspension

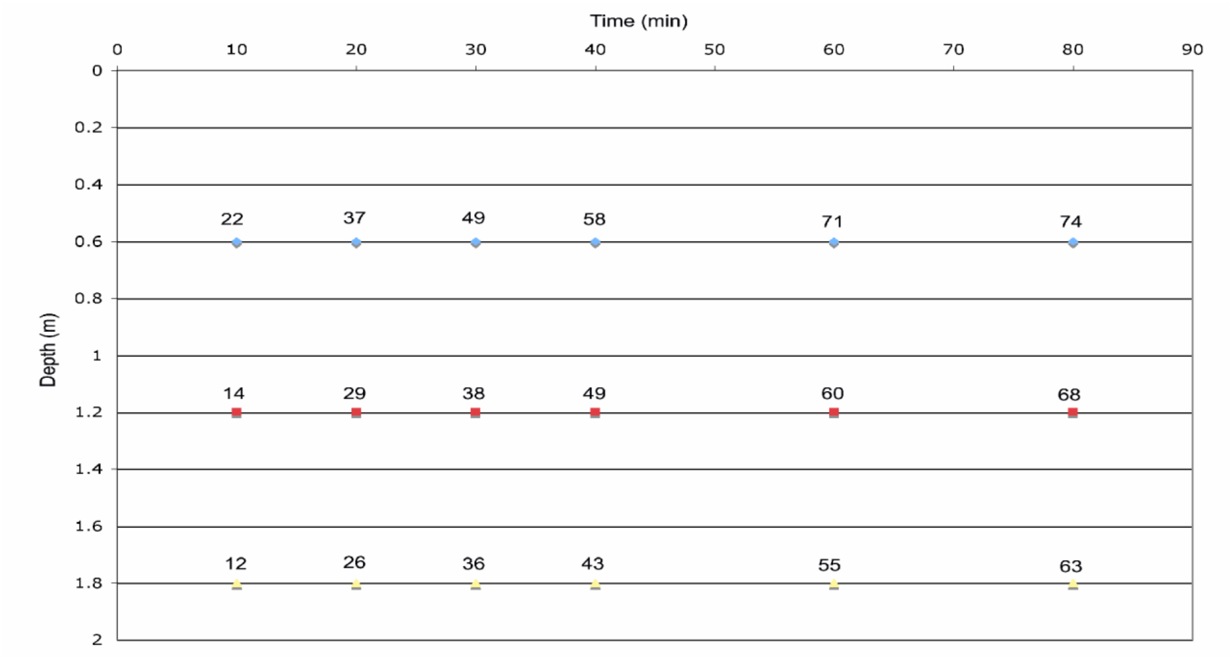
Example

Using the sample data provided, estimate the percentage removal of solids in an ideal basin with a depth of 1.8 m and a residence time of 50 min.

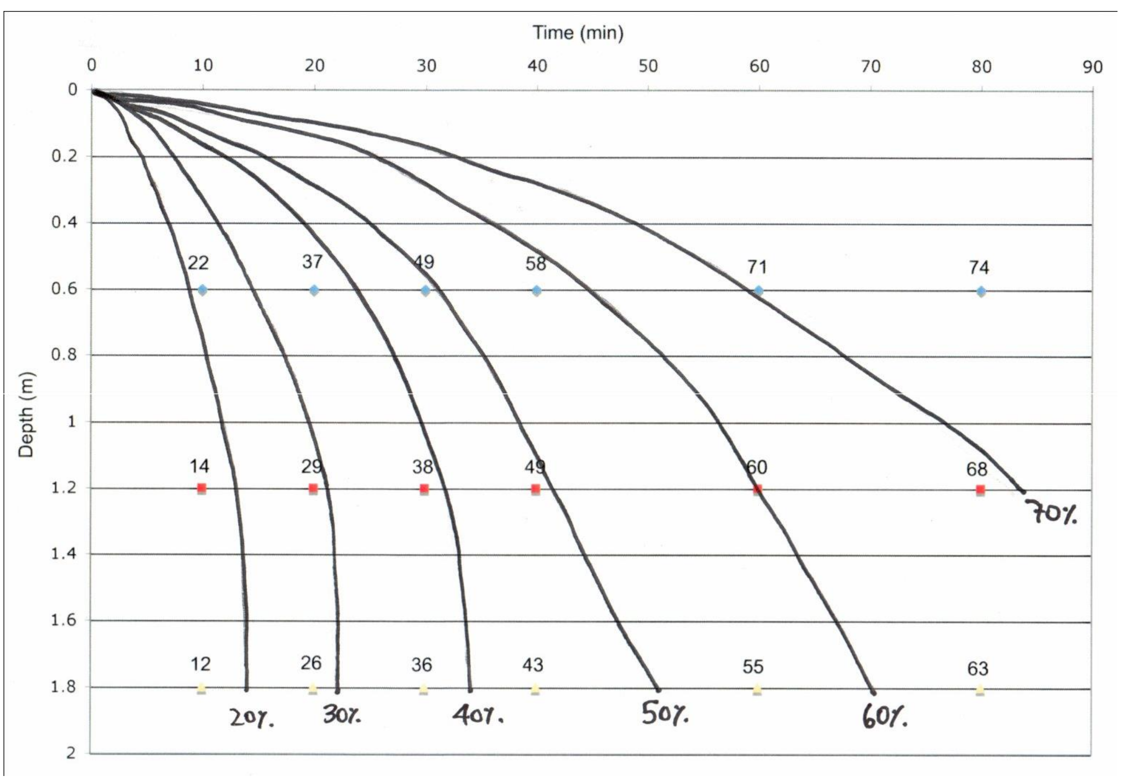
Percentage Removal %			Settling Time
1.8 m	1.2 m	0.6 m	min
12	14	22	10
26	29	37	20
36	38	49	30
43	49	58	40
55	60	71	60
63	68	74	80

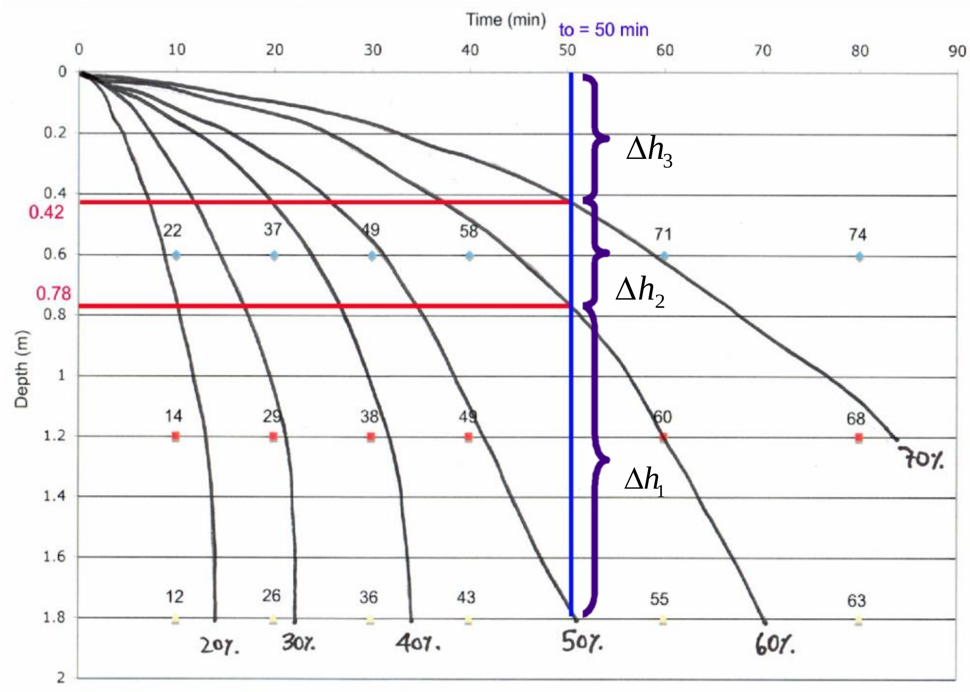
Solution

Step 1: Plot the data



Step 2: Draw the Constant Percent Removal Lines





- Vo = (1.8 m)/(50 min) = 0.036 m/min.
- From the curve, at to= 50 min, 50% of the particles have vs ≥ vo and are totally removed.
- Need to determine the number of particles with vs ≤ vo that are removed.
- Fraction is partially removed in a ratio of h/H.

Step 3: Calculate the Total Percent Removal
Particles with a settling velocity less than SOR will be removed in a ratio of (h/H). The total percent solids removal (R) for flocculating particles in a column of height H at a specified SOR.

$$\%R = \sum_{h=1}^{n} \left(\frac{\Delta h_n}{H} \right) \left(\frac{R_n + R_{n+1}}{2} \right) \times 100\%$$

$$\Delta h_1 = \frac{(1.8 - 0.78)m}{1.8m} \times \frac{(60+50)\%}{2} = 31\%$$

$$\Delta h_2 = \frac{(0.78 - 0.42)m}{1.8m} \times \frac{(70+60)\%}{2} = 13\%$$

$$\Delta h_3 = \frac{(0.42 - 0)m}{1.8m} \times \frac{(100+70)\%}{2} = 19.8\%$$

Total removed = 31 + 13 + 19.8 = 64%

6.10 Hindered or settling Zone

Hindered settling is a term used in sedimentation to describe settling of large numbers of particles that are settling as "a mass," often interfering with each others settling, rather than settling "unhindered" as individual, discrete particles. This phenomenon may also manifest itself when all of the settled particles come together at the bottom of the tank prior to being incorporated into the sludge zone (may also be called the compression zone). While this condition appears to be more prevalent in wastewater treatment secondary sedimentation, it is worthy to note in potable water treatment where large amounts of flocculated algae, etc., may be involved.

6.11 Design consideration

Surface loading rates are used to calculate surface area requirements of a sedimentation tank.
Detention time is used in conjunction with the surface loading rate to calculate the volume and side water depth of the sedimentation tank.
Inlet zone or influent structure the inlet zone distributes the flow across the sedimentation tank and dissipates incoming velocity. To achieve these two function perforated wall is provided as shown in Fig. (21). Rectangular tank is constructed to be integral with flocculation tank. A diffusion wall separates the two tanks and serves as the sedimentation tank inlet. The influent structure in circular tank is provided as a part of the sludge collection equipment and generally are designed by the equipment supplier.
Outlet zone or effluent structure traditionally overflow weirs and launder troughs have been used for outlet control in a sedimentation tank. Either V-notch or submerged orifice weir plates are commonly used. The length of weir required is determined by the weir overflow rate or weir loading rate. Typical launder trough for rectangular, square and circular sedimentation tank are illustrated in Fig. (22).

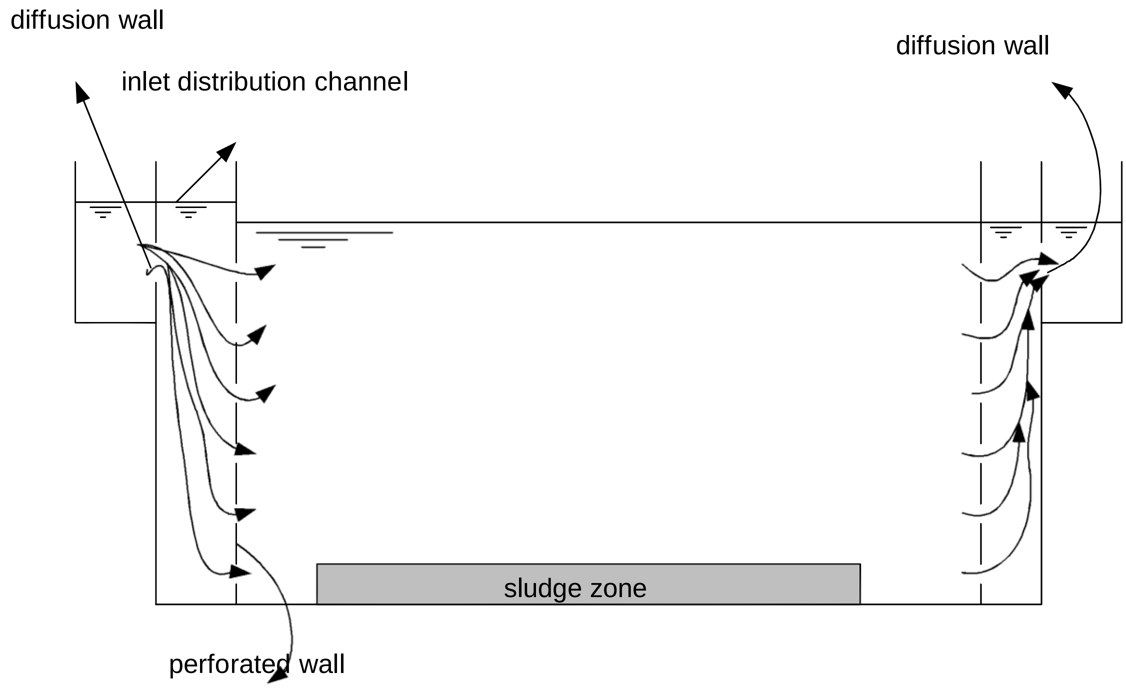


Fig (21) Inlet and outlet structure

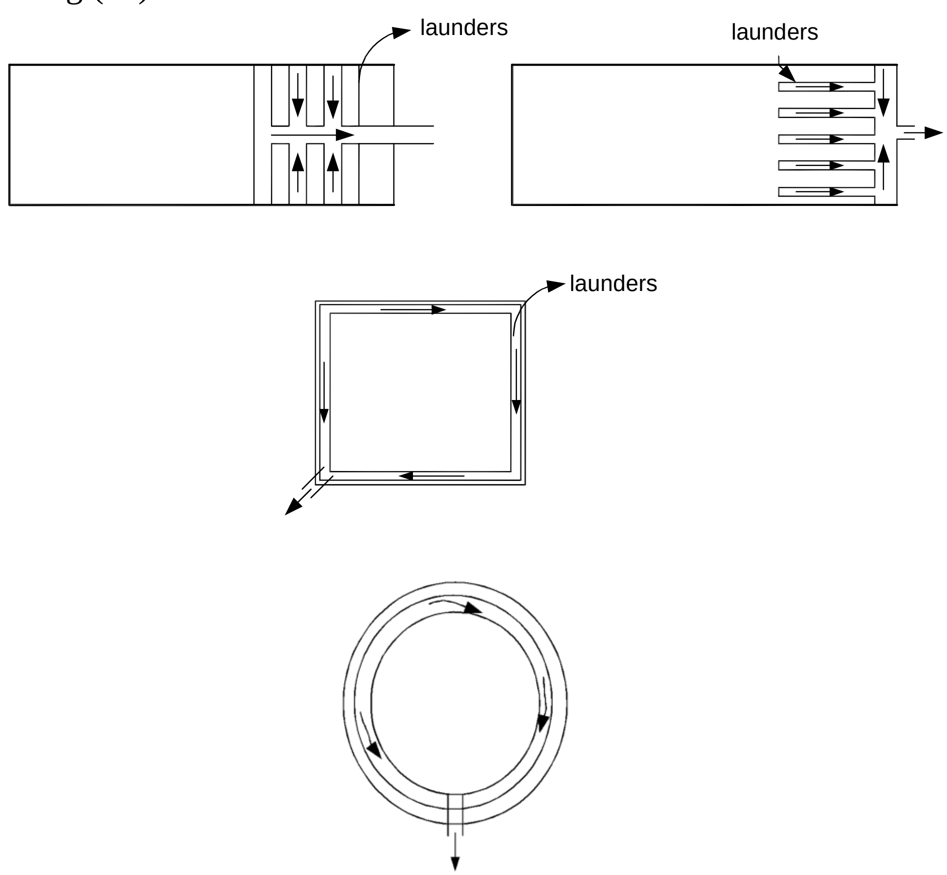
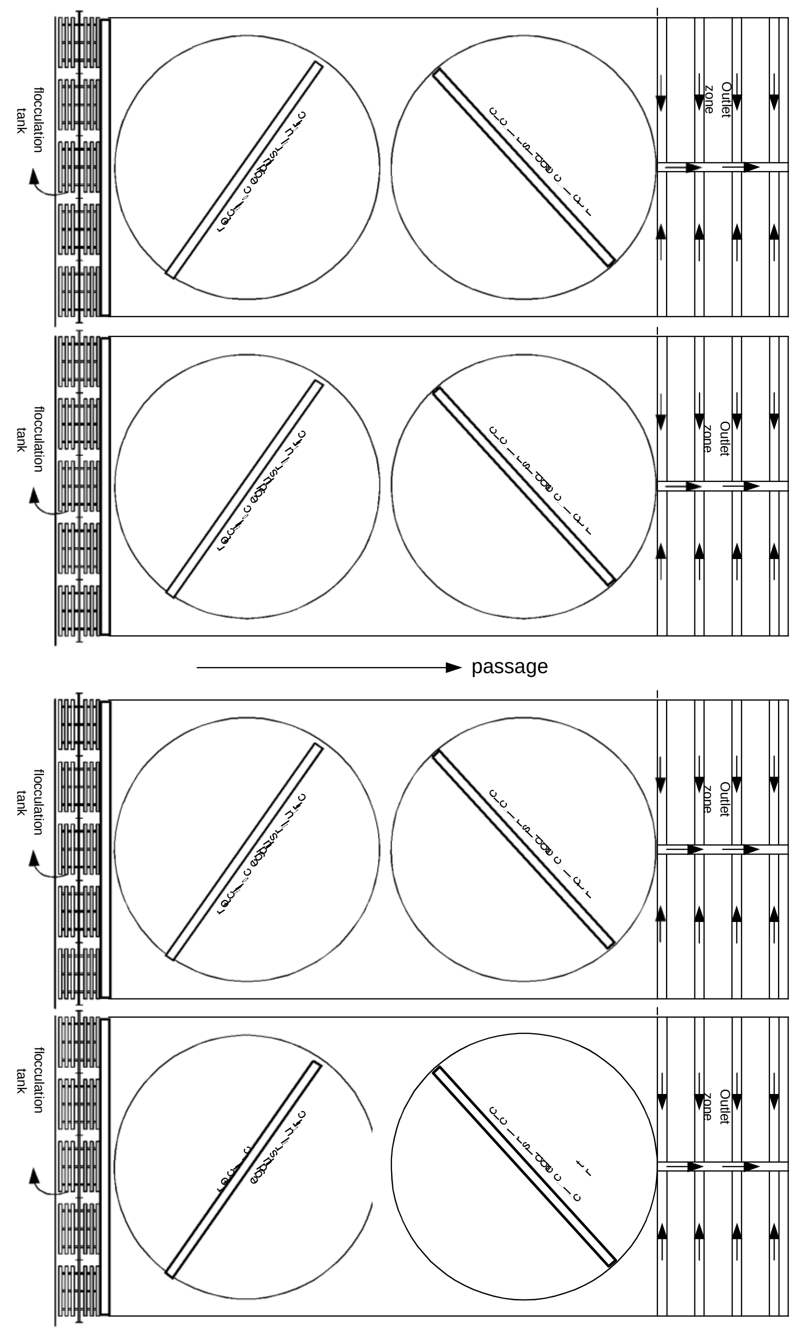


Fig (22) Typical launder trough

Sedimentation tank design

Exemple:
Number of tank = 4
Maximum discharge = 113500 m^3/day
Length to width ratio = 2 to 3
Surface loading rate = 35 m^3/m^2.day
Weir loading rate = 250 m^3/m.day D.T= 4hr



Solution

1- Dimensions of the tank

$Q = 113500/4 = 28375\ m^3/day = 0.328\ m^3/s$ for each tank
$A = Q/SOR = 28375/\ 35 = 810\ m^2$
If, W = 18.5 m then L = 810/18.5 = 43.7 = 44 m
L/W ration = 44/18.5 = 2.3 this ratio within the range
Depth (h) = SOR* D.T = 35m/day*1/24 day/hr*4 hr = 5.8 m
Then the sedimentation tank dimension is $44*18.5*5.8\ m^3$
Fig (1) shows the sedimentation tank with effluent and settling zones

2- Influent structure design

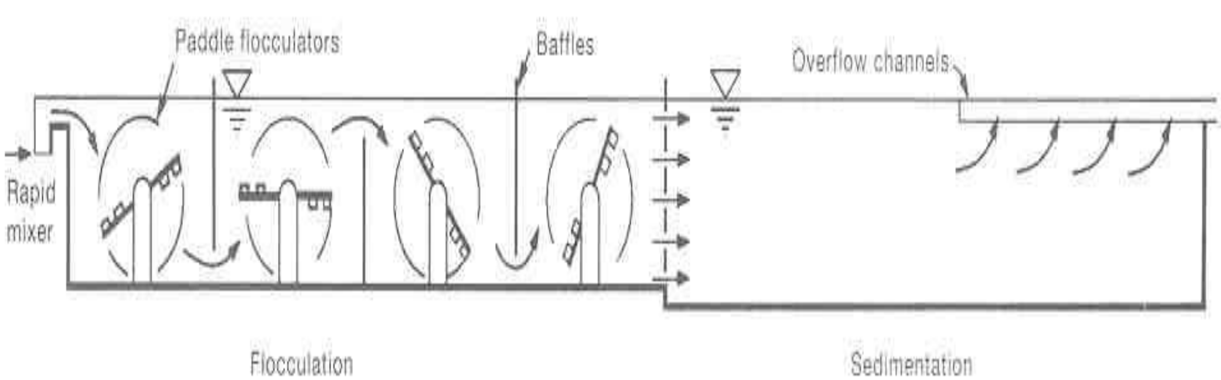
The detail influent structure design is provided in lecture five as effluent flocculation tank. The head loss at the diffusion wall between the flocculation and sedimentation tanks is very small and can be neglected.

3- Effluent structure design

A) Weir length
Weir length required = Q/wier loading = 28375/250 = 113.5 m
Take 8 launder troughs (width = 0.5m) with a center effluent collection channel.
Each trough is 8.5m (depends on the width of the tank).
Actual weir length = 2 troughs *8.5m* 1 side per trough + 6 troughs *8.5m* 2 side per trough = 119m >113.5 m

يكون ذو أتجاه واحد لان الاتجاه ألأخر هو ملاصق لحائط الخزان ولايحسب من ضمن طول weir

Fig (2) shows the effluent collection system



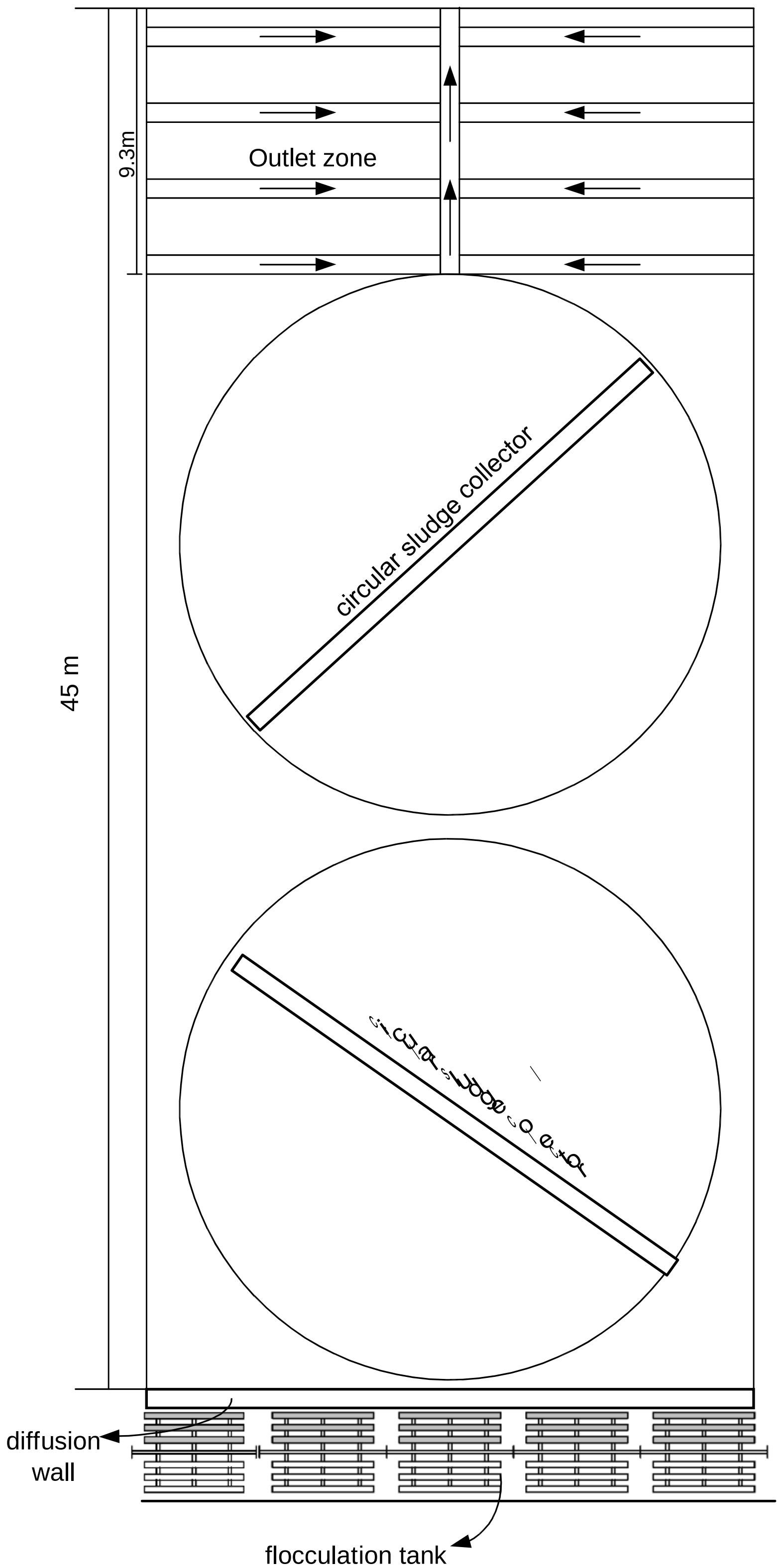


Fig (23) Sedimentation tank

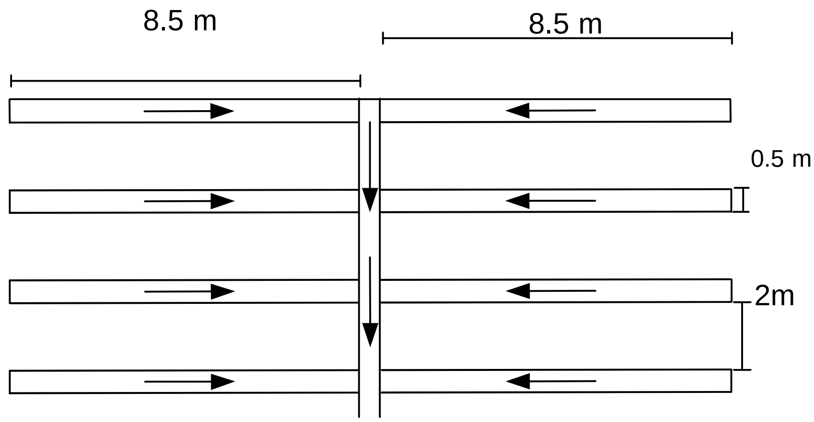


Fig (24) Effluent collection system

B) Total number of V- notches

Use 90^{o} V-notches weirs.

Provide 9 notches per 2 m long weir plate and 4 plates on each side of each 8.5 m trough, as shown in fig (3).

Notch shall be 7.5 cm deep and spaced at 20 cm.

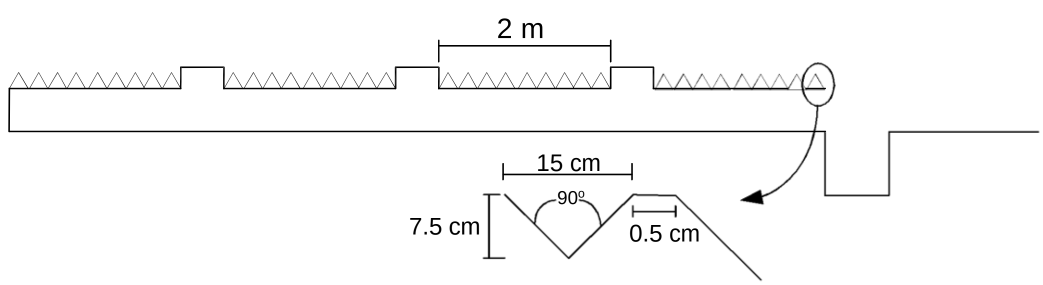


Fig (25) effluent launder with weir

Total number of notches in each tank = 2 troughs *1 side per trough * 4 plates pers ide * 9 notches per plate + 6 troughs *2 side per trough * 4 plates per side * 9 notches per plate = 504 notches

C) Head over the V- notch weirs

Flow rate per notch (q) = 0.328 m^3/s / 504 notch = $6.52*10^{-4}$ m^3/s per notch

The head over each notch can be calculated from the following formula:

$$q = \frac{8}{15} C_d \sqrt{2\,g}\ \tan\frac{\theta}{2} H^{2.5}$$

Where

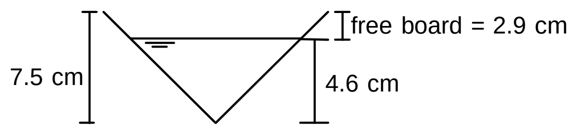
C_d: coefficient of discharge = 0.6

H: head over notch, m

θ : angle of V-notch = 90

$6.52*10^{-4}$ m^3/s = (8/15) * 0.6 * $(2 * 9.81)^{0.5}$ tan (90/2) * $H^{2.5}$

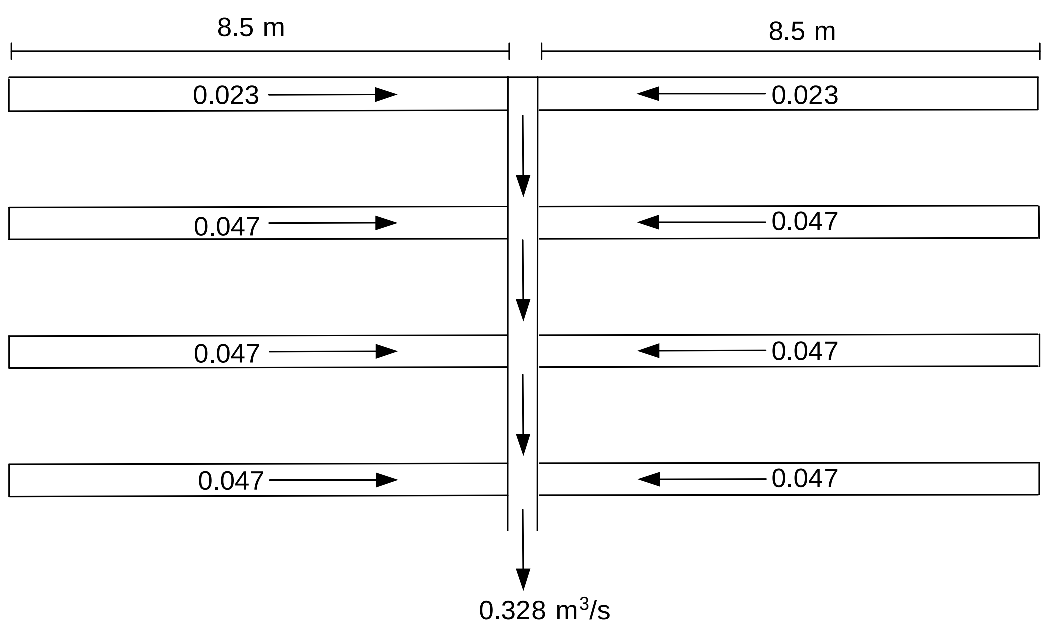
H = 0.046 m
Then the free board for V-notch = 7.5 – 4.6 = 2.9 cm



D) Head loss through the effluent launder trough

Here we must calculate the water flow rate in each launder trough and calculate water depth at the downstream end of the launder trough and water depth at the upstream end of the launder trough.

The flow rate at the exit point of each effluent launder trough is calculated as:
$Q' = q * 2$ side per trough * 4 plates pers ide * 9 notches per plate = $6.52*10^{-4} * 2 *4 *9 = 0.047\ m^3/s$



- Water depth at the downstream end of the launder trough (y_2)

$$y_2 = \left(\frac{(Q')^2}{gb^2}\right)^{1/3}$$, b : launder width

$y_2 = ((0.047)^2/9.81 *(0.5)^2)^{1/3} = 0.096$ m

- Water depth at the upstream end of the launder trough (y_1)

$$y_1 = \sqrt{y_2^2 + \frac{2\times(Q')^2}{gb^2 y_2} + \frac{F\times L\times(Q')^2}{12gb^2\times r\times d}}$$

Where
F: Darcy friction factor (0.03-0.12)
r: mean hydraulic radius, m
d: mean depth of channel, m
L: length of launder trough, m

Assume the mean depth in the effluent launder trough (d) = 0.14 m

r = b*d /(2d+b) = 0.14*0.5 / (2*0.14+0.5) = 0.09 m

$$y_1 = \sqrt{(0.096)^2 + \frac{2\times(0.0469)^2}{9.81\times(0.5)^2\times 0.096} + \frac{0.1\times 8.5\times(0.0469)^2}{12\times 9.81\times(0.5)^2\times 0.09\times 0.14}} = 0.18\text{m}$$

Then loss through the trough = y_1 - y_2 = 0.18 - 0.098 = 0.082 m

4- Sludge quantity

The quantity of sludge is estimated by totaling (1) the suspended solids and other constituents removed, (2) the amount of metal hydroxide, (3) the calcium carbonate produced, (4) the amount of coagulant aid.
In this design example, we assume that the following data are applied in coagulation and flocculation processes. All coagulants used will be solid in the sedimentation tank and can be removed.

Max. turbidity	17 NTU
Max. seasonal iron concentration	0.7 mg/l
Max. seasonal manganese concentration	0.4 mg/l
Optimum coagulant, ferric sulfate	25 mg/l
Optimum coagulant aid, cationic polymer	0.05 mg/l
Hydrated lime ($Ca(OH)_2$) for pH adjusment	15 mg/l
Seasonal potassium permanganate ($KMnO_4$)	4 mg/l

Production of solids can be calculated as following:

1) *Solids from raw water:*
   Quantity = 17 NTU * 1 TSS/NTU * 10^{-6} kg/mg * 10^3 L/m^3 * 113500 m^3/d = 1930 kg/d
2) *Solids due to precipitation of iron content:*
   Quantity = (106.9 g/mol $Fe(OH)_3$/55.9 g/mol Fe) * 0.7 mg Fe/L * 10^{-6} kg/mg * 10^3 L/m^3 * 113500 m^3/d = 152 kg/d
3) *Solids due to precipitation of manganese content*:

Quantity = (86.9 g/mol MnOO/54.9 g/mol Mn) * 0.4 mg Mn/L * 10^{-6} kg/mg * 10^3 L/m^3 * 113500 m^3/d = 72 kg/d

4) Solids due to precipitation of ferric sulfate:
Quantity = (0.54 kg $Fe(OH)_3$/kg $Fe_2(SO_4)_3$ * 25 g/m^3 $Fe_2(SO_4)_3$ * 10^{-3} kg/g * 113500 m^3/d = 1532 kg/d

5) Solids from the polymer:
Quantity = 1 * 0.05 mg polymer/L * 10^{-6} kg/mg * 10^3 L/m^3 * 113500 m^3/d = 5.7 kg/d

6) Lime solids during pH adjustment (Assume 20% of $Ca(OH)_2$ precipitates as $CaCO_3$) :
Quantity = 0.2 * (100 g/mol $CaCO_3$/74 g/mol $Ca(OH)_2$) * 15 mg $Ca(OH)_2$/L * 10^{-6} kg/mg * 10^3 L/m^3 * 113500 m^3/d = 460 kg/d

7)Solids due to precipitation of potassium permanganate content:
Quantity = (86.9 g/mol MnOO/158 g/mol $KMnO_4$) * 4 mg $KMnO_4$/L * 10^{-6} kg/mg * 10^3 L/m^3 * 113500 m^3/d = 250 kg/d

Now, we can calculate the total solids produced:
Total quantity of solids = 1930+152+72+1532+5.7+460+250 = 4402 kg/day
Note Because of many uncertainties associated with raw water quality, chemical dosages, this value is increased by 20-60%.

Assume that 90% of solids produced are removal from sedimentation tank:
∴ quantity of solids removal = 4402 * 0.9 = 3962 kg/day
Assume that the sludge with 2% solids (then 98% water):
∴ Mass of wet sludge = 3962 / 0.02 = 198100 kg/day
Now, we can used the following equation to find the density of wet sludge:

$$\frac{M_{ws}}{\rho_{ws}} = \frac{M_{ds}}{\rho_{ds}} + \frac{M_{wt}}{\rho_{wt}}$$

Where
M_{ws}: Mass of wet sludge
M_{ds}: Mass of dry sludge
M_{wt}: Mass of water

Let density of solids = 2400 kg/m^3 and water density = 1000 kg/m^3, sub in the previous equation.

$$\frac{198100}{\rho_{ws}} = \frac{3962}{2400} + \frac{\overbrace{194138}^{198100 * 0.98}}{1000} \Rightarrow \rho_{ws} = 1012 \text{ kg/m}^3$$

Volume of sludge withdrawn (V_{ws})= 3962kg/d/0.02×1012kg/m^3=195.7 m^3/d

Sludge pipe design:

Use 2 pipes D= 20cm, velocity=0.6m/s

Q=0.6×π/4 $(0.2)^2$ =0.0188m^3/s=1628.6m^3/d(for each pipe)

Volume of sludge with drawn for each pipe=196m^3/d/2=98m^3/d

Required opening time period=98m3/d/1628.6m3/d=6%

0.06×24×60=86.4min per day=3.6 min per hr

2019-2020

Unit Seven

Filtration

Environmental Engineering Department

Third Stage

Asst. Prof.Dr. Ahmed Hassoon Ali+ Lect. Zainab Nasir

7.1 Purposes of Filtration:

The purposes of filtration is to remove suspended particles from water by passing the water through a medium such as sand. As the water passes through the filter, floc and impurities get stuck in the sand and the clean water goes through. The filtered water collects in the clear well, where it is disinfected and the sent to the customers.

Filtration is usually the final step in the solid removal process which began with coagulation and advanced through flocculation and sedimentation. in filter, up to 99.5% of the suspended solids in the water can be removed, including minerals, floc, and microorganisms.

7.2 Requirements:

Filtration is now required for most water treatment systems. Filters must reduce turbidity to less than 0.5 NTU in 95% of each month's measurements and the finished water turbidity must never exceed 5 NTU in any sample.

Turbidity alone does not have health implications. Although turbidity is not harmful on its own, turbid water is difficult to disinfect for a variety of reasons. Microorganisms growing on the suspended particles may be hard to kill using disinfection while the particles themselves may chemically react with chlorine, making it difficult to maintain a chlorine residual in the distribution system. Turbidity can also cause deposits in the distribution system that create tastes, odors, and bacterial growths. However, turbid drinking water has other troublesome implications as well. Sand filtration removes some cyst-forming microorganisms, such as *Giardia* which cannot be killed by traditional chlorination. **Cysts** are resistant covers which protect the microorganism while it goes into an inactive state.

Regulations require that at least 99.9% of *Giardia* cysts and 99.99% of viruses be removed from drinking water. Since it is difficult to test directly for these microorganisms, turbidity in water can be used as an indicator for their presence. By requiring a low turbidity in the finished water, treatment plants are ensuring that few or no *Giardia* are present in finished drinking water.

7.3 Location in the Treatment Process:

In the typical treatment process, filtration follows sedimentation (if present) and precedes disinfection. Depending on the presence of flocculation and sedimentation, treatment processes are divided into three groups - conventional filtration, direct filtration, and in-line filtration.

The most common method of filtration is **conventional filtration**, where filtration follows coagulation/flocculation and sedimentation. This type of filtration results in flexible and reliable performance, especially when treating variable or very turbid source water. Some treatment plants operate without some or all of the sediment removal processes which precede filtration. If filtration follows coagulation and flocculation, without sedimentation, it is known as **direct filtration**. This method can be used when raw water has low turbidity. Another type of filtration, known as **in-line filtration**, involves operating the filters without flocculation or sedimentation. A coagulant chemical is added to the water just before filtration and coagulation occurs in the filter. In-line filtration is often used with pressure filters, but is not as efficient with variable turbidity and bacteria levels as conventional filtration is.

7.4 Mechanisms of Filtration:

The filter used in the filtration process can be compared to a sieve or microstrainer that traps suspended material between the grains of filter media. However, since most suspended particles can easily pass through the spaces between the grains of the filter media, ***straining*** is the least important process in filtration. Filtration primarily depends on a combination of complex physical and chemical mechanisms, the most important being ***adsorption***. Adsorption is the process of particles sticking onto the surface of the individual filter grains or onto the previously deposited materials. The forces that attract and hold the particles to the grains are the same as those that work in coagulation and flocculation. In fact, some coagulation and flocculation may occur in the filter bed, especially if coagulation and flocculation of the water before filtration was not properly controlled. Incomplete coagulation can cause serious problems in filter operation.

The third mechanism of filtration is biological action, which involves any sort of breakdown of the particles in water by biological processes. This may involve decomposition of organic particles by algae, plankton, diatoms, and bacteria or it may involve microorganisms eating each other. Although biological action is an important part of filtration **in slow sand filters**, in most other filters the water passes through the filter too quickly for much biological action to occur.

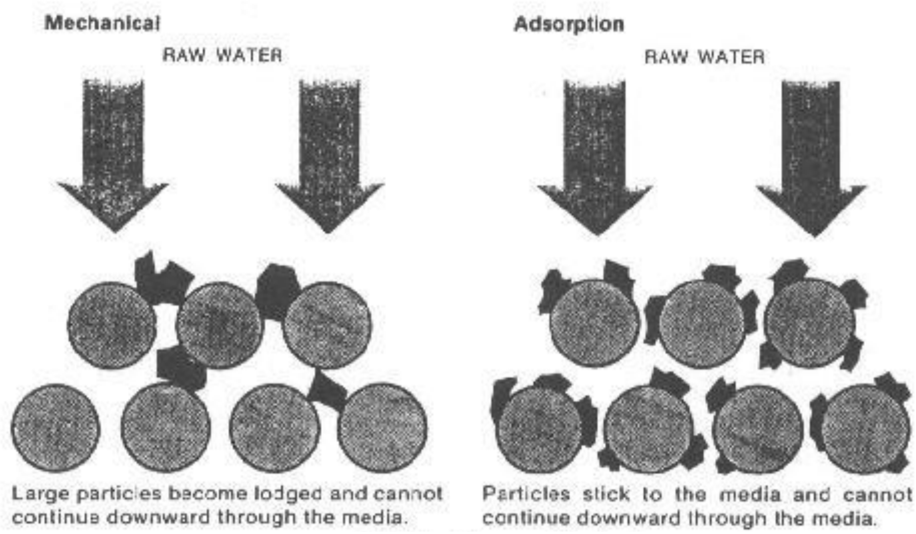


Figure 7.1 : The two removal mechanisms

7.5 Classification of filters:

1) According to type of granular medium used:

- **Single medium (sand or anthracite)**
- **Dual media (anthracite and sand)**
- **Multi-media (anthracite, sand, garnet**)

Dual media filters: better longer filtration run . Available pore volume is maximum at the top of filter and gradually decreases to a minimum at the bottom of filter.

2) According to flow through medium

- **Gravity filters:** are open to the atmosphere. Flow through the medium is achieved by gravity
- **Pressure filters:** Filter medium is contained in pressure vessel. Water is delivered to the vessel under pressure

3) According to rate of filtration

- **Rapid sand filters**
- **Slow sand filters**

4) According to filter flow control scheme

- **Constant rate** (constant head or variable head).
- **Declining rate** (constant head or variable head).

The table below shows the characteristics of four types of filters which can be used in water treatment.

Table (7.1): Types of filters.

Type of filter	**Slow Sand Filter (SSF)**	**Rapid Sand Filter (RSF)**	**Pressure Filter**	**Diatomaceous earth filter (Diatomite filter)**
Filtration rate (gpm/ft^2)	0.015-0.15	2-3	2-3	1-2
Pros	Reliable. Minimum operation and maintenance requirements. Usually does not require chemical pretreatment.	Relatively small and compact.	Lower installation and operation costs in small filtration plants.	Small size. Efficiency. Ease of operation. Relatively low cost. Produces high clarity water. Usually does not require chemical pretreatment.
Cons	Large land area required. Need to manually clean filters.	Requires chemical pretreatment. Doesn't remove pathogens as well as slow sand filters.	Less reliable than gravity filters. Filter bed cannot be observed during operation.	Sludge disposal problems. High head loss. Potential decreased reliability. High maintenance and repair costs.
Filter Media	Sand.	Sand. Or sand and anthracite coal. Or sand and anthracite coal and garnet.	Sand. Or sand and anthracite coal. Or sand and anthracite coal and garnet.	Diatomaceous earth. Diatomaceous earth consists of fossilized remains of diatoms, a type of hard-shelled algae.
Gravity or Pressure?	Gravity.	Gravity.	Pressure.	Pressure, gravity, or vacuum.
Filtration Mechanism	Biological action, straining, and adsorption.	Primarily adsorption. Also some straining.	Primarily adsorption. Also some straining.	Primarily straining.

Cleaning Method	Manually removing the top 2 inches of sand.	Backwashing.	Backwashing.	Backwashing.
Common Applications	Small groundwater systems.	Most commonly used type of filter for surface water treatment.	Iron and manganese removal in small groundwater systems.	Beverage and food industries and swimming pools. Smaller systems.

7.5.1 History:

The history of water treatment dates back to approximately the thirteenth century B.C. in Egypt. However, modern filtration began much later. John Gibb's **slow sand filter**, built in 1804 in Scotland, was the first filter used for treating potable water in large quantities. Slow sand filters spread rapidly, with the first one in the United States built in Richmond, in 1832. A set of slow sand filters adapted from English designs was built in 1870 in Poughkeepsie, and is still in operation. A few decades after the first slow sand filters were built in the U.S., the first rapid sand filters were installed. The advent of **rapid sand filtration** is linked to the discovery of coagulation. By adding certain chemicals (coagulants) to turbid water, the material in the water could be made to clump together and quickly settle out. Using coagulation, clear water for filtration could be produced from turbid, polluted streams. By the end of the nineteenth century, there were ten times as many rapid sand filters in service as the slow sand type. Currently, slow sand filtration is only considered economical in unusual cases. **The diatomaceous earth filter** was developed by the U.S. Army during WWII. They needed a filter that was easily transportable, lightweight, and able to produce pure drinking water. The diatomaceous earth filter is used in smaller systems, but is not commonly part of water treatment plants.

We will discuss two types of filters below - the slow sand filter and the rapid sand filter. The pressure sand filter is essentially a rapid sand filter placed inside a pressurized chamber while the diatomaceous earth filter is not commonly used in treatment of drinking water.

7.5.2 Slow Sand Filter:

The slow sand filter is the oldest type of large-scale filter. In the slow sand filter, water passes first through about 36 inches of sand (91.4 cm), then through a layer of gravel, before entering the underdrain. The sand removes particles from the water through adsorption and straining. Typical slow sand filtration velocities are only about 0.04-0.4m/hr.

Unlike other filters, slow sand filters also remove a great deal of turbidity from water using biological action. A layer of dirt, debris, and microorganisms builds up on the top of the sand. This layer is known as **schmutzdecke**, which is German for "dirty skin." The schmutzdecke breaks down organic particles in the water biologically, and is also very effective in straining out even very small inorganic particles from water.

Maintenance of a slow sand filter consists of raking the sand periodically and cleaning the filter by removing the top two inches of sand from the filter surface. After a few cleanings, new sand must be added to replace the removed sand.

Cleaning the filter removes the schmutzdecke layer, without which the filter does not produce potable water. After a cleaning, the filter must be operated for two weeks, with the filtered water sent to waste, to allow the schmutzdecke layer to rebuild. As a result, a treatment plant must have two slow sand filters for continuous operation. Slow sand filters are very reliable filters which do not usually require coagulation/flocculation before filtration. However, water passes through the slow sand filter very slowly, and the rate is slowed yet further by the schmutzdecke layer. As a result, large land areas must be devoted to filters when slow sand filters are part of a treatment plant. Only a few slow sand filters are operating in the United States although this type of filter is more widely used in Europe.

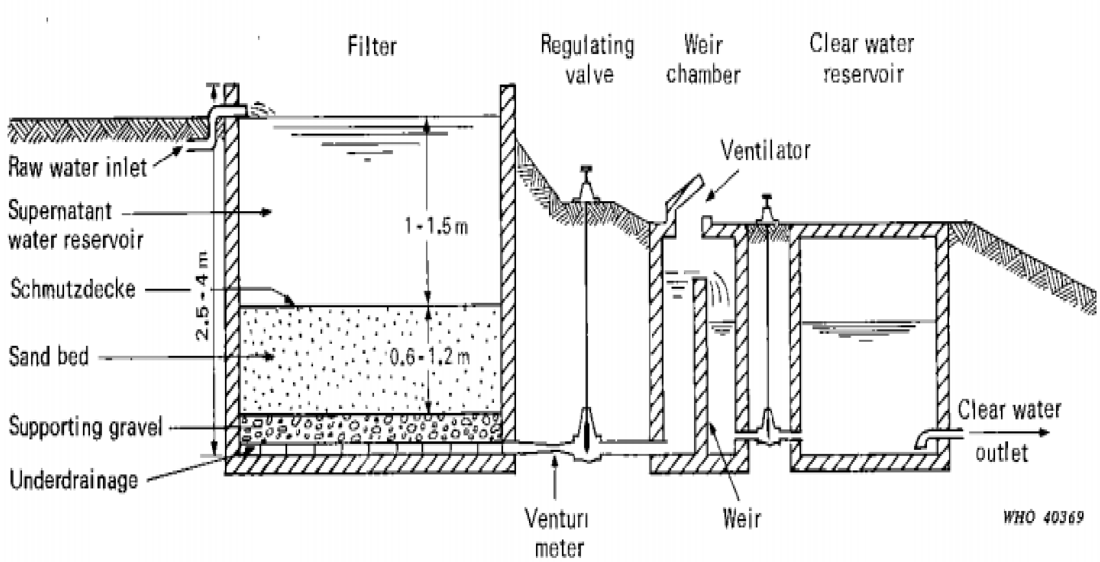


Figure (7.2): Diagram of slow sand filter.

7.5.3 Rapid Sand Filter:

The rapid sand filter differs from the slow sand filter in a variety of ways, the most important of which are the much greater filtration rate and the ability to clean automatically using backwashing. The mechanism of particle removal also differs in the two types of filters - rapid sand filters do not use biological filtration and depend primarily on adsorption and some straining.

Since rapid sand filters are the primary filtration type used in water treatment , we will discuss this filter in more detail. Typical rapid sand filtration velocities are only about 0.4-3.1 m/hr.

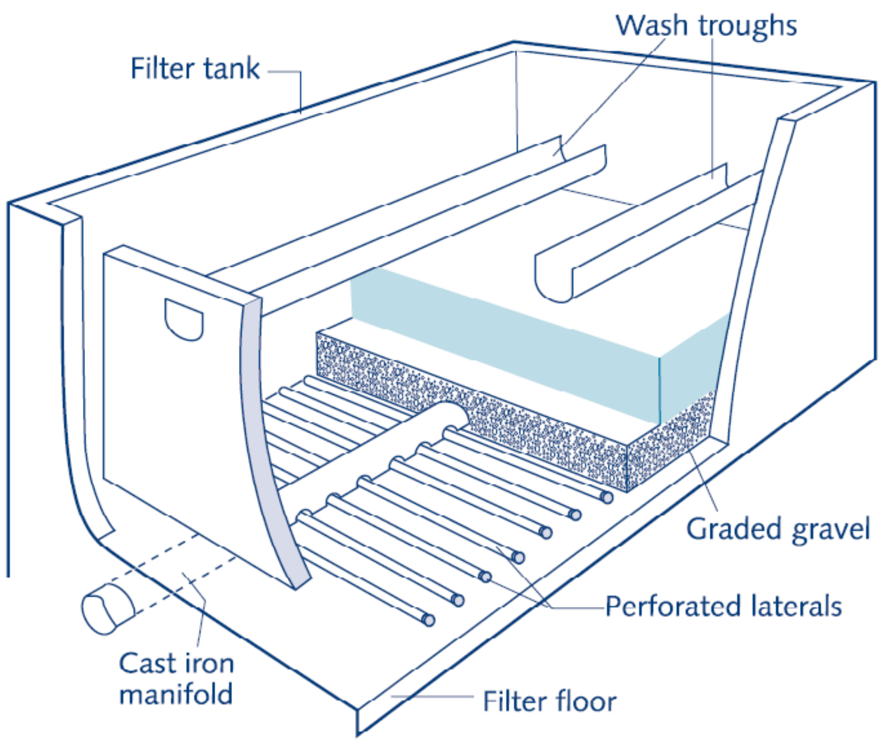


Figure (7.3): Rapid filter

A diagram of a typical rapid sand filter is shown above (Fig. 7.3). The filter is contained within a **filter box**, usually made of concrete. Inside the filter box are layers of **filter media** (sand, anthracite, etc.) and gravel. Below the gravel, a network of pipes makes up the **underdrain** which collects the filtered water and evenly distributes the backwash water. **Backwash troughs** help distribute the influent water and are also used in backwashing (which will be discussed in a later section.). In addition to the parts mentioned above, most rapid sand filters contain a **controller**, or **filter control system**, which regulates flow rates of water through the filter. Other parts, such as valves, a loss of head gauge, surface washers, and a backwash pump, are used while cleaning the filter.

7.6 Filter media:

Filter media commonly are specified by effective size and uniformity coefficient. The effective size (d_{10}) is the size of the standard sieve opening that will pass ten percent by weight of the media. The uniformity coefficient (d_{60}/d_{10}) is the ratio of the standard sieve opening that will pass sixty percent by weight of the media to its effective size. Graphical representation of a standard sieve analysis and determination of d_{10},d_{60} and uniform coefficient are illustrated in Fig (7.4).

7.6.1 Single medium filters:

Single medium filters utilize a single material, most commonly well grad sand. Typical effective size, uniformity coefficient and bed depths for these filters are listed in Table (7.2). In these filters, after backwashing larger grains settle faster than smaller grains, in a phenomena called stratification or reverse gradation. This phenomena is shown in Fig. (7.5). Reverse gradation is the major advantages of the single medium filters. The smaller the top of the filter trap most of the particles, therefore only the top 4 or 5 cm of the filter bed is utilized for filtration. Particles that pass through this Additionally, because only the top 4 to 5 cm of the bed are used, the solids holding capacity of the bed is small, and so filter runs are shortened.

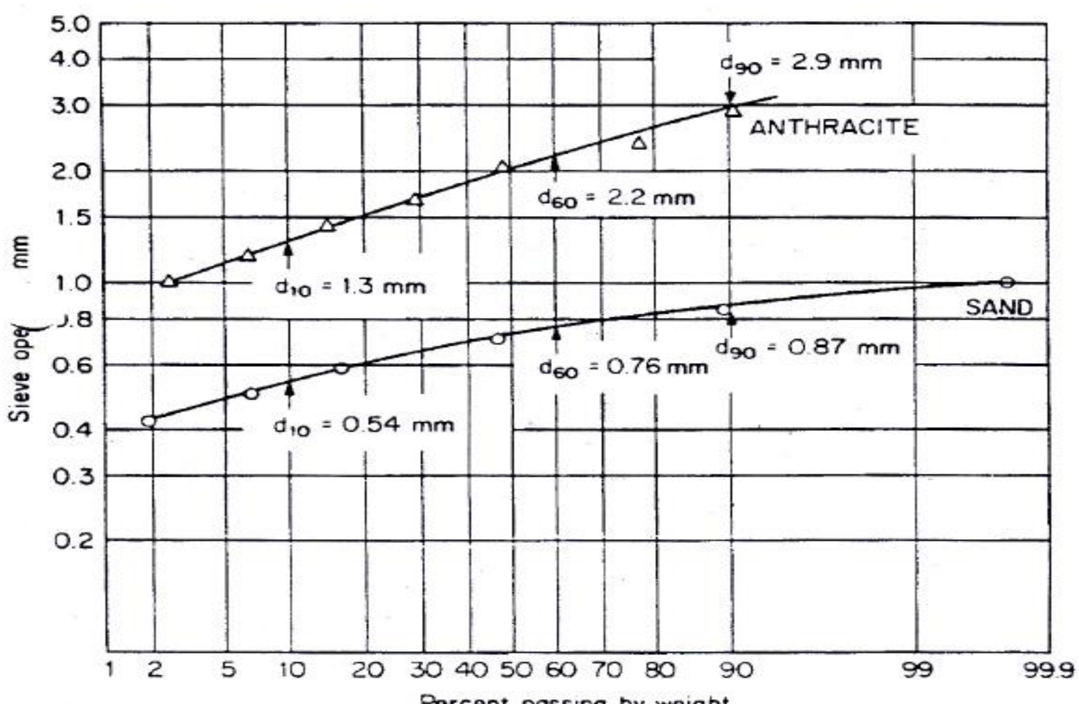


Figure (7.4): Typical sieve analysis of two filter media.

7.6.2 Dual medium filters:

Another solution to the problem of reverse gradation is the dual media filter. Typical dual media filters utilize anthracite coal and quartiz sand as filter media. The anthracite with specific gravity of 1.55 is lighter than the sand which has a specific gravity of 2.65. Therefore, a larger anthracite grain has the same settling velocity as a much as smaller sand grain. This characteristics allows coal grain to be placed on top of smaller sand grains to create a gradation as shown in fig. (7.5). Typical design values for dual media filters are listed in Table (7.2). It can be used a mixed media filters in which we can use garnet (SG=4) in additional to sand and anthracite.

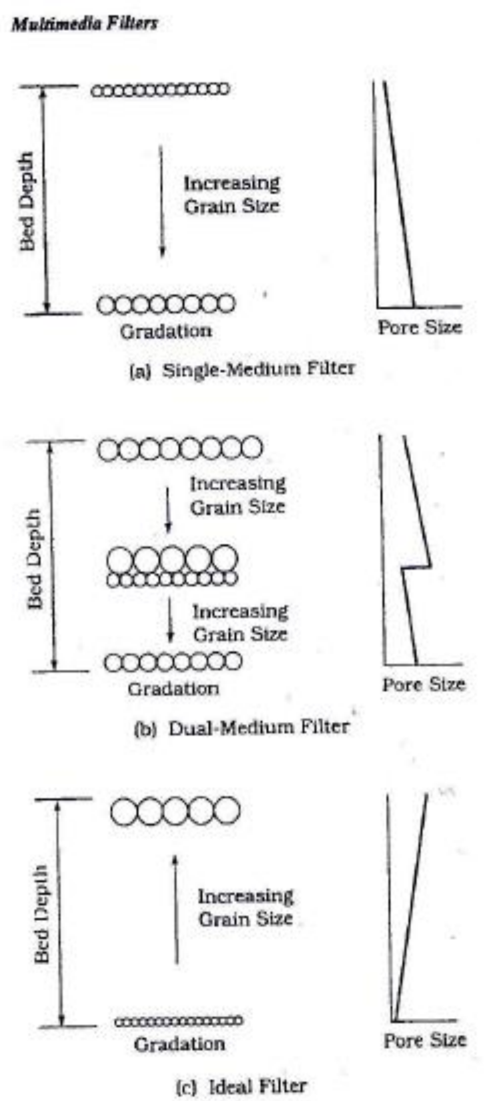


Figure (7.5): Gradation and pore size in various filters.

Table (7.2) : Typical media design values for various filters.

Parameters	Single media filters	Dual media filters	Multi media filters
Anthracite layer			
Effective size, mm	0.5-1.5	0.7-2	1-2
Uniformity coefficient	1.2-1.7	1.3-1.8	1.4-1.8
Depth, cm	50-150	30-60	50-130
Sand layer			
Effective size, mm	0.45-1	0.45-0.6	0.4-0.8
Uniformity coefficient	1.2-1.7	1.2-1.7	1.2-1.7
Depth, cm	50-150	20-40	20-40
Garnet			
Effective size, mm			0.2-0.8
Uniformity coefficient			1.5-1.8
Depth, cm			5-15

The following equation can be used to calculate the size of the media grains with different specific density and equal settling velocity:

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

Example:

Find the particle size of anthracite and ilmenite which have settling velocity equal to that of sand 0.5 mm in diameter.

Solution:

For the anthracite:

$$\frac{d_1}{0.5} = \left\{\frac{2.6-1}{1.5-1}\right\}^{2/3} = 1.1mm$$

The following equation can be used to calculate the size of the media grains with different specific density and equal settling velocity:-

$d_1/d_2 = [Sg_2 – 1/Sg_1-1]^{2/3}$

Example:-

Find the particle size of anthracite and ilmenite which have settling velocities equal to that sand 0.5 mm in diameter.

$S.g_{sand} = 2.6$, $S.g_{anth.} = 1.55$, $S.g_{il} = 4.2$

Solution:-

For anthracite $d_1/0.5= [2.6-1/1.5-1]^{2/3}= 1.1$ mm

For the ilmenite:

$$\frac{d_1}{0.5} = \left\{\frac{2.6-1}{4.2-1}\right\}^{2/3} = 0.3mm$$

0.5   1.1   0.3   at the same settling velocity

$\downarrow$ $\downarrow$ $\downarrow$

Anthracite smaller than 1.1 mm would remain above 0.5 mm sand, and grains of ilmenite larger than 0.3 mm would remain below it.

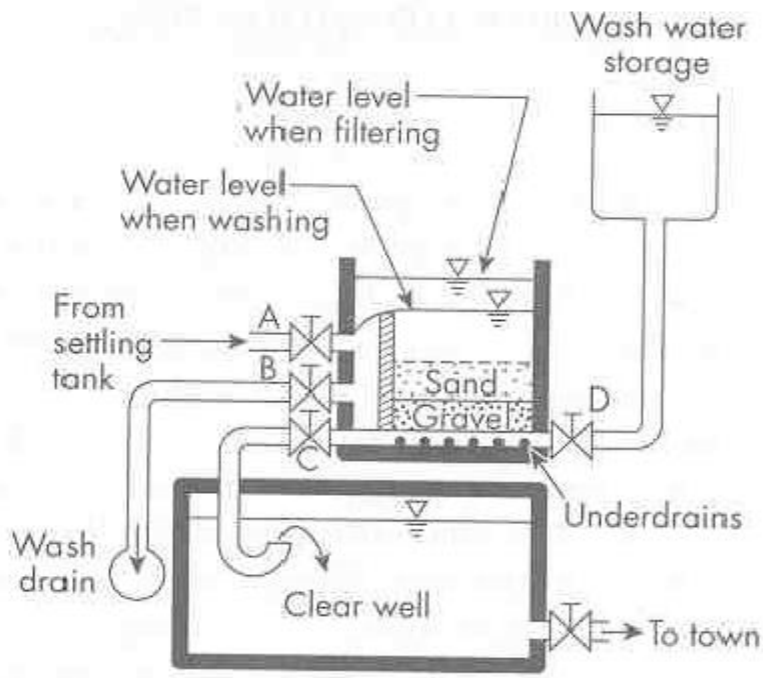
7.7 Filter components:

Filter components are shown in Fig. (7.6). They are:

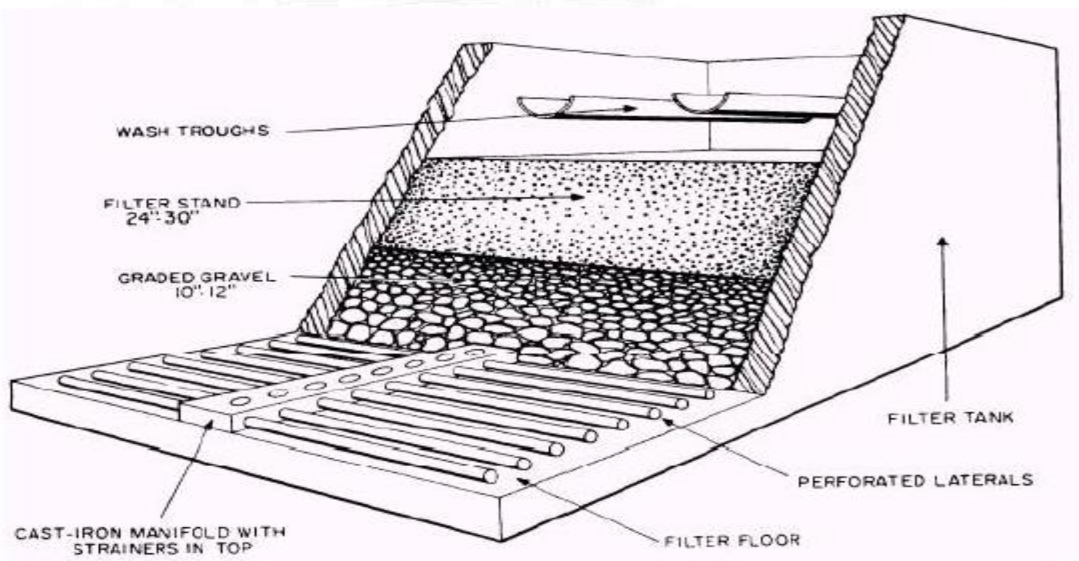
1) Influent pipe 2) Effluent pipe 3) Wash water pipe 4) Filter box

5) Filter media 6) Gravel support 7) Underdrain system 8) Washwater trough.

The influent pipe transports the water from sedimentation tank to the filter. Effluent pipe transports the filtered water to the next step of the water treatment. Washwater pipe transports clean water to the bottom of the filter for the backwash process.



Wash water
storage
Water level
when filtering
Water level
when washing
From
settling
tank
A
B
C
D
Sand
Gravel
Underdrains
Wash
drain
Clear well
To town



WASH TROUGHS
FILTER STAND
24"-30"
GRADED GRAVEL
10"-12"
FILTER TANK
PERFORATED LATERALS
CAST-IRON MANIFOLD WITH
STRAINERS IN TOP
FILTER FLOOR

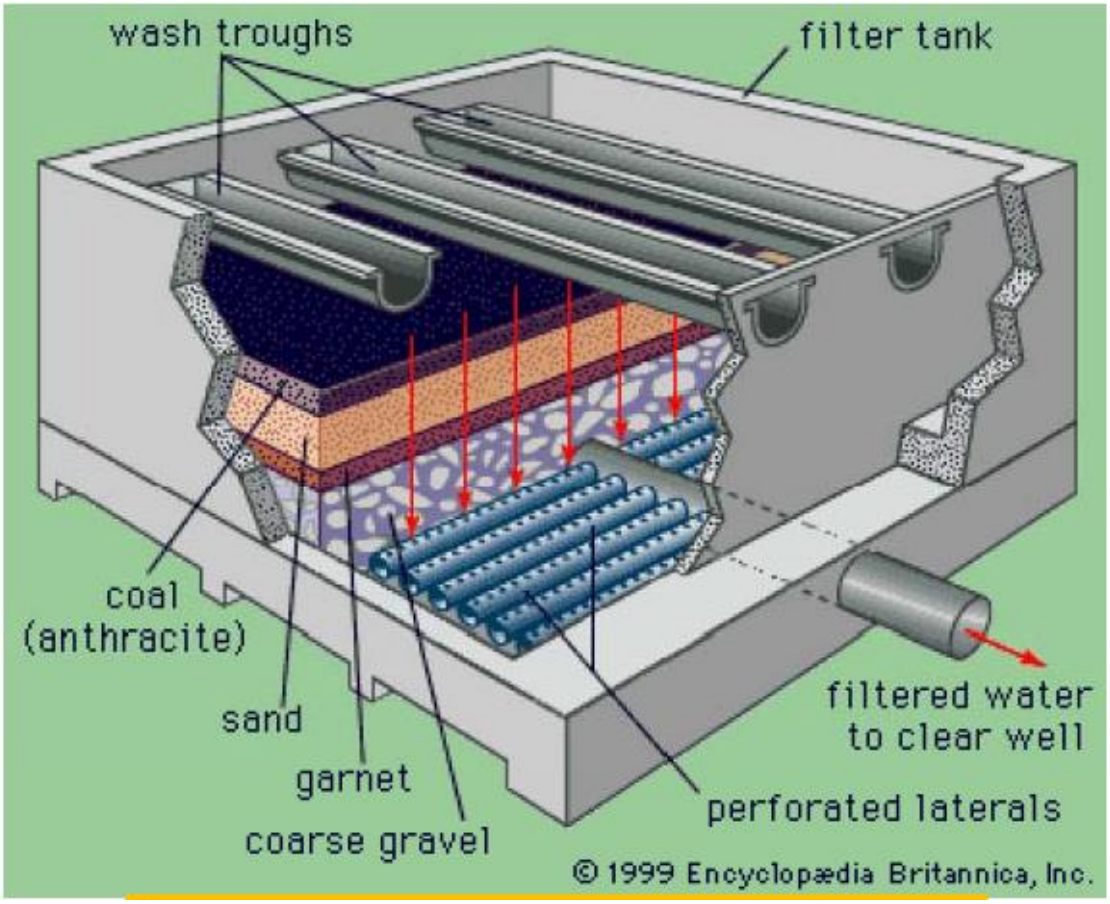


Figure 7.6: Rapid sand filter components

7.7.1 Filter box:

The filter tank is generally constructed from concrete and is most often rectangular. Filters in large plants are usually constructed next to each other in a row, allowing the piping from the sedimentation basins to feed the filters from a central pipe gallery or from a inlet channel. The sizes of filters vary according to the quantity to be treated. The number of filters is selected to minimize the effect of removing the filter from service washing on remaining filters. Filter bed sizes vary from 25 to 100 m^2 with length in the range of 2.5 to 8 m and length to breadth ratio of 1,25 to 1.33.

7.7.2 Underdrain system:

The underdrain serves to support the filter medium and gravel, to collect filtered water evenly from the bottom of the filter, and to distribute air and water evenly across the bottom of the filter during backwashing. Key to these functions is the evenness of filtration and the distribution of backwash air and water. The underdrain system can be one of the following types, connected to main drain: pipe laterals, Concrete block, False floor and porous plates or strainer nozzles.

One common type of uderdrain consist of a manifold and perforated laterals installed below the gravel bed. Most new designs using a plastic piping system for filter underdrains. Piping materials must be certified for contact with potable water. Typical lateral size ranging from 4 to 8 inch (100 – 200 mm) with the underdrain system header in the range of 8 to 16 inch (200 to 400 mm).

Table 7.3 gives the typical design parameters of an underdrain system (lateral manifold system) for a small rapid sand filter. Typical arrangements of perforated pipe under drains are shown in Fig. (7.7). Figure (7.8) is a view of an installation in progress of perforated PVC underdrain system.

Table 7.3: Design Criteria for Underdrains

Criterion	Value
Minimum diameter of underdrains	20 cm
Diameter of the perforations vertical	6-12 mm (suggested at a slight angle to the axis of the pipe)
Spacing of perforations along laterals	7.5 cm for 6 mm perforations
Ratio of total area of perforation to total cross-sectional area of laterals	0.25 for 6 mm perforations 0.50 for 12 mm perforations
Ratio of total area of perforations to the entire filter area	0.003
Length to diameter ratio of the lateral	60:1
Maximum spacing of lateral	30 cm
Cross-sectional area of the manifold	1.5-2.0 times the total area of laterals
Velocity of the filtered water outlet	1.0-1.8 m/s

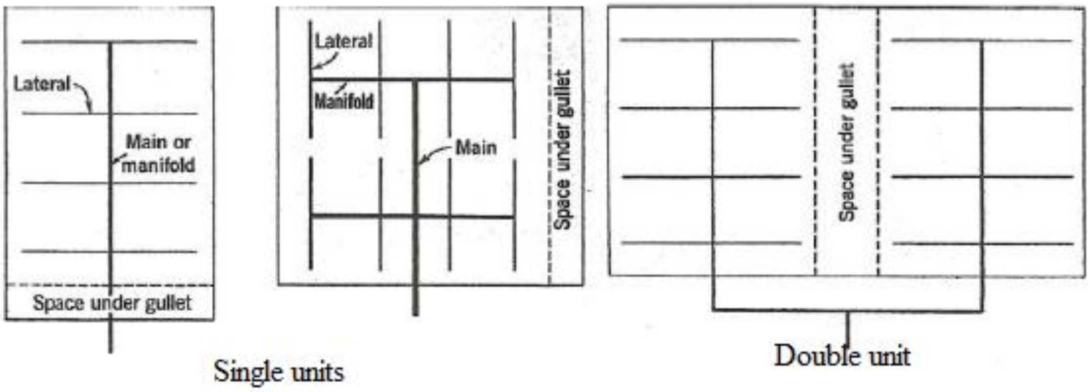


Figure (7.7): Perforated Pipe Underdrains

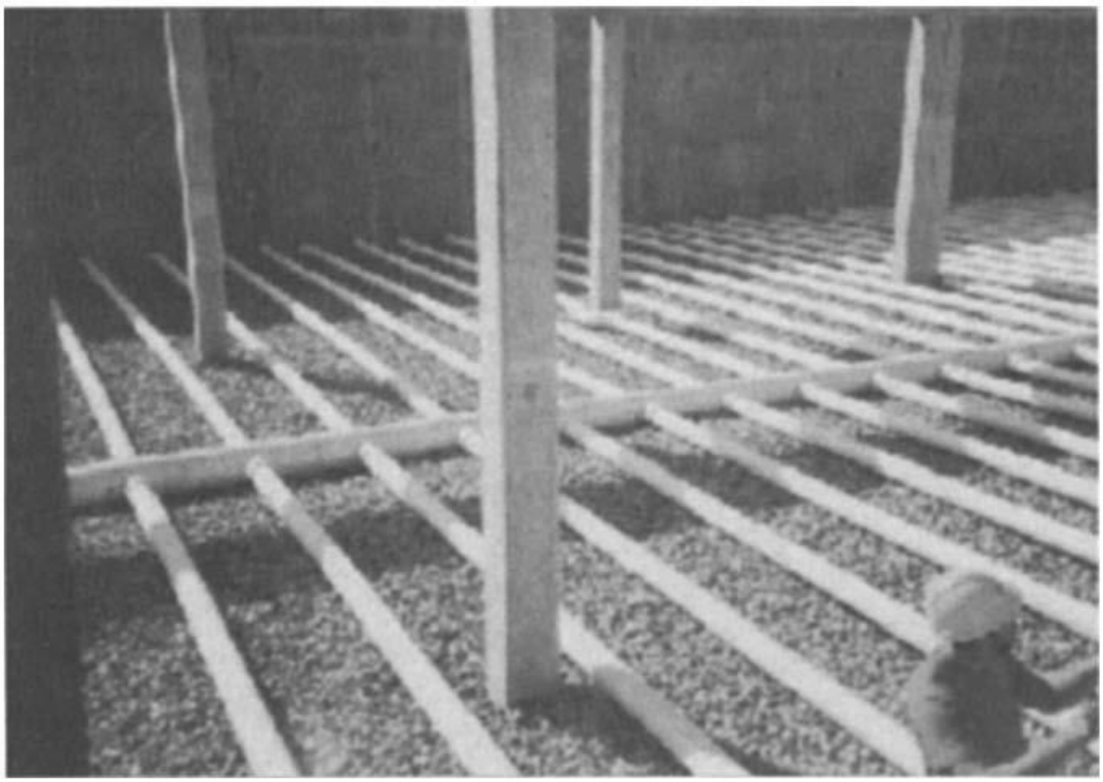


Figure (7.8): Underdrain installation

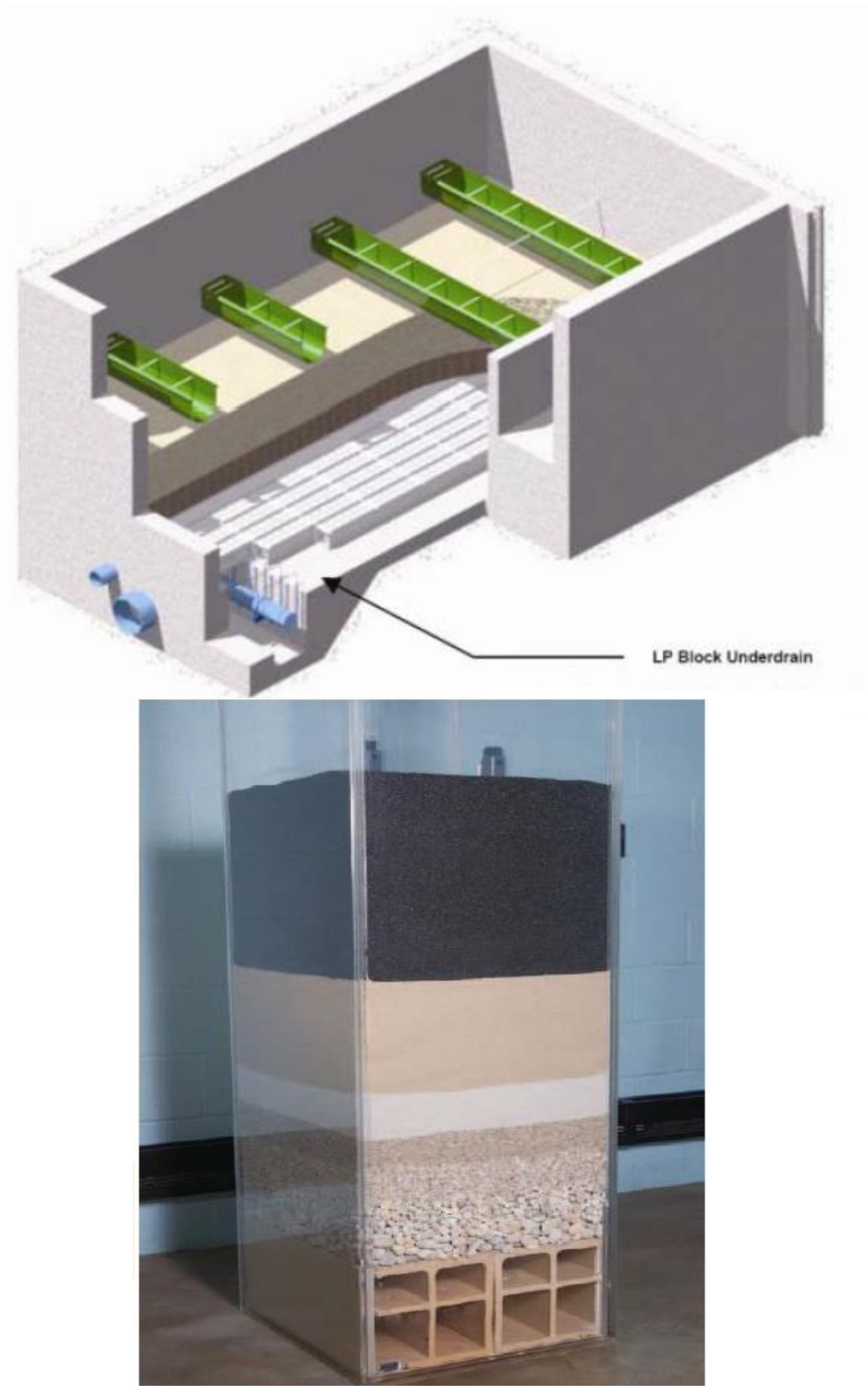
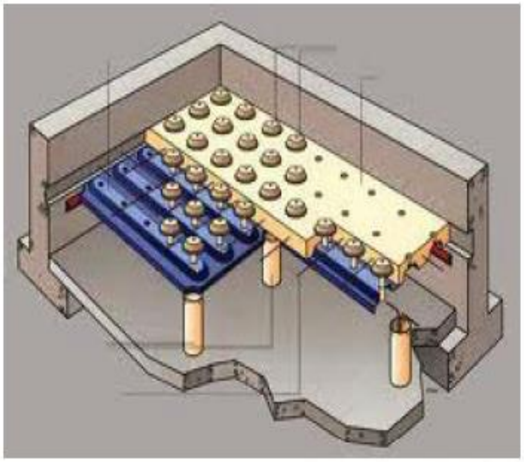
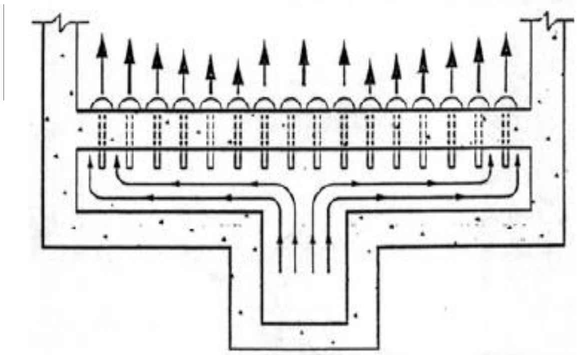
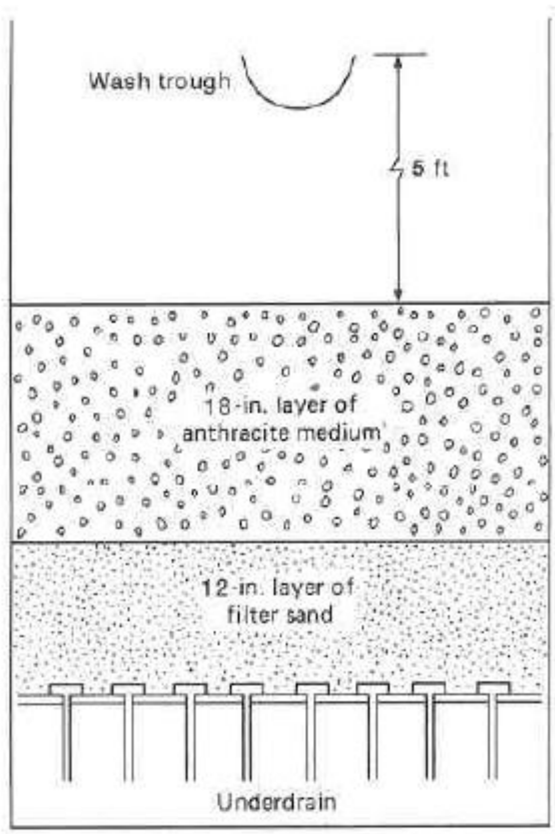


Figure (7.9): Rapid sand filter components: with ducts under- drain system







(a)

<u>Figure (7.10):</u> Rapid sand filter perforated slab and nozzle under-drain

7.7.3 Washwater troughs:

Washwater troughs placed above the filter media collect the backwash water and carry it to the drain system. The bottom of the trough should be above the top of the expanded sand to prevent possible loss of sand during backwashing. The clear horizontal distance between troughs is usually 5 to 7 ft (1.5- 2 m). The trough usually has a semicircle bottom. Troughs may be made of concrete, fiberglass-reinforced plastic, or other structurally adequate and corrosion resistant materials. The dimensions of a filter trough may be determined by use of the following equation:

$Q = 2.49\ bh^{3/2}$

Where Q = rate of discharge, ft^3/sec

b = width of trough, ft.

h = maximum water depth in trough, ft.

Some free board should be allowed in the wash water troughs.

The use of the above formula is illustrated in the following example:

Example:

Rectangular troughs 24 ft long, 18 in wide and 7 ft on centers are to serve a filter that is washed at a rate of 30 in. per minute. Determine: (a) the depth of the troughs if their invert is to be kept level and they are to discharge freely into the gutter, and (b) the height of the top of the trough above the sand if a 30-in. bed is to be expanded by 50 percent.

Solution:

Rate of discharge Q = 24x7x[30/(12x60)]

= 7 ft3/sec

b = 18/12 = 1.5 ft

Applying the above data in Equation,

$7 = 2.49x1.5xh^{3/2}$

$h^{3/2} = 7/(2.49x1.5)$

$h = [7/(2.49x1.5)]^{2/3}$

= 1.52 ft………………….(a)

Expansion of sand = 0.5x30 = 0.5x(30/12) =1.25 ft

Height of trough top above sand = 1.52 + 1.25

= 2.77 ft plus some free board…………(b)

Typical arrangement of wash water troughs is shown in Fig. (7.11) below.

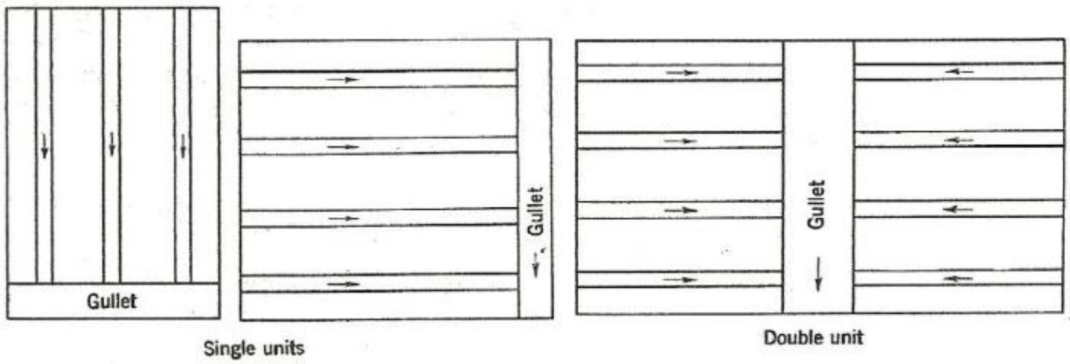


Figure (7.11): Typical arrangement of wash water troughs.

7.8 Filter Cleaning:

7.8.1 When to Backwash:

Rapid sand filters, pressure filters, and diatomaceous earth filters can all be backwashed. During **backwashing**, the flow of water through the filter is reversed, cleaning out trapped particles.

Three factors can be used to assess when a filter needs backwashing. Some plants use the length of the **filter run**, arbitrarily scheduling backwashing after 72 hours or some other length of filter operation. Other plants monitor **turbidity of the effluent** water and **head loss** within the filter to determine when the filter is clogged enough to need cleaning.

Head loss is a loss of pressure (also known as head) by water flowing through the filter. When water flows through a clogged filter, friction causes the water to lose energy, so that the water leaving the filter is under less pressure than the water entering the filter. Head loss is displayed on a head loss gauge. Once the head loss within the filter has reached between six and ten hours (1.5-2.5 m) , a filter should be backwashed.

7.8.2 The Process of Backwashing:

The first operation is to close valves 1 and 4 (Fig. 12) and allow the filter to drain until the water lies a few centimeters above the top of the bed. The valve 5 is opened, and air is blown back through a compressed air unit at a rate of about 1-1.5 m^3 free air/min.m^2 of bed area for about 2-3 minutes at a pressure of

20-35 kN/m^2. The water over the bed quickly becomes very dirty as the air-agitated sand breaks up surface dirt. Following this, valves 2 and 6 are opened, and an upward flow of water is sent through the bed at a carefully designed high velocity. This should be sufficient to expand the bed (20-50%) and cause the sand grain to be agitated so that deposits are washed off them, but not so high that the sand grains are carried away in the rising upward flush of water. After the washing of the filters has been completed, valves 2 and 6 will be closed, and valves 1 and 3 opened. This restores the inlet supplied through valve 1. The filtered water is wasted to the gutter for a few minutes after this, until the required quality is achieved. Ultimately, valve 3 is closed and 4 is opened to get the filtered water a gain. The entire process of backwashing the filters and restarting the supplies takes about 15 minutes. The specified minimum backwash time for a rapid filters is 5 minutes, The amount of water required to wash a rapid filter may vary from 3-6% of the total amount of water filtered. Upward wash rates are usually of the order of 0.3-1.0 m/min.

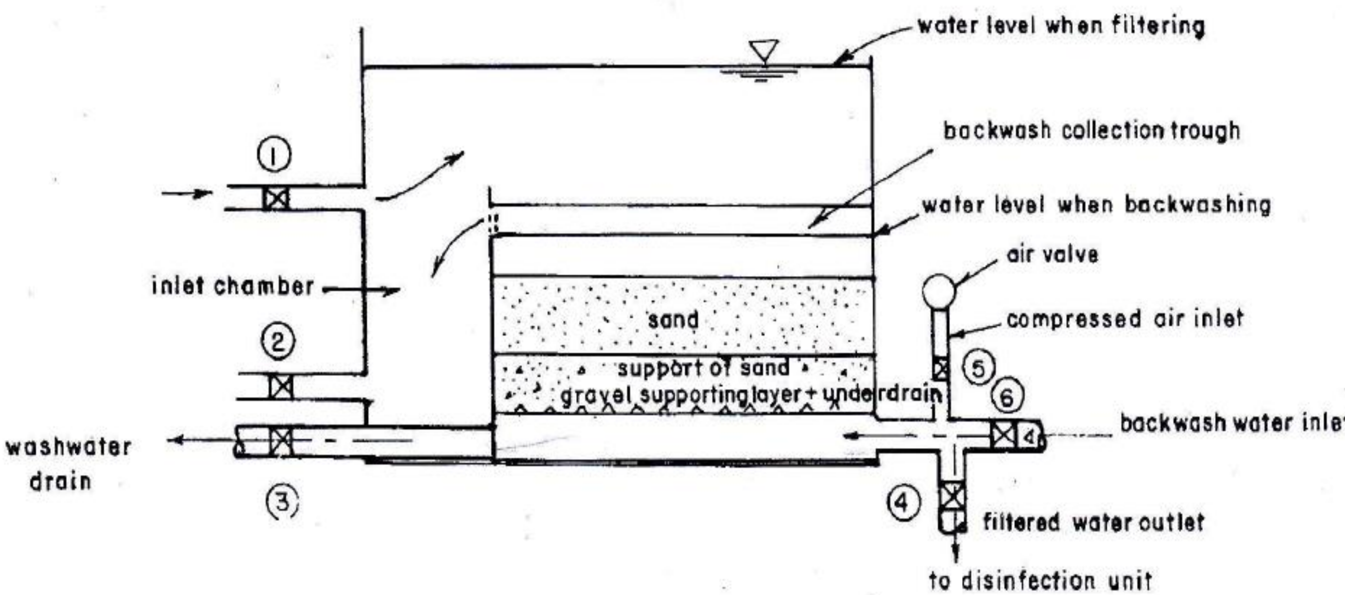


Figure (7.12): Diagrammatic section of rapid sand filter.

The design of wash water troughs is similar to that in the pervious lecture. The back wash velocity for the sand and anthracite are calculated from the following equations:

$U_b = d_{60}$ (for sand)

$U_b = 0.47d_{60}$ (for anthracite)

Where U_b is the backwash velocity, m/min. The terminal settling velocity for the sand and anthracite is calculated from:

$U_t = 10d_{60} = 10d_{10}$* uniformity coefficient (for sand).

$U_{t=}$ 4.7d60=4.7d_{10}* uniformity coefficient (for anthracite).

If the porosity of the sand or anthracite is specified, the :

$U_b = U_t \times f^{0.45}$

7.8.3 Surface Washing:

At the same time as backwashing is occurring, the surface of the filter should be additionally scoured using **surface washers**. Surface washers spray water over the sand at the top of the filter breaking down mud balls.

7.9 Estimation of head losses:

7.9.1 Clean water head loss:

Several equations have been developed to describe the flow of clean water through a porous medium. Carmen-Kozeny equation used to calculate head loss is as follow :

$$h_L = f\frac{(1-\alpha)LV_s^2}{\phi\alpha^3 dg}$$

$$f = 150\frac{(1-\alpha)}{R_n} + 1.75$$

$$R_n = \frac{\phi d V_s \rho}{\mu}$$

Where, h= head loss, m

f= friction factor

α= porosity

ϕ= particle shape factor (1.0 for spheres, 0.82 for rounded sand, 0.75 for average sand, 0.73 for crushed coal and angular sand).

L= depth of filter bed or layer, m

d= grain size diameter, m

V_s= superficial (approach) filtration velocity, m/s.

g= acceleration due to gravity, 9.81 m^2/s.

d_g=geometric mean diameter between sieve sizes, d_1 and d_2.

R_n= Reynolds number

μ=viscosity, N.s/m.

7.9.2 Back wash water head loss:

The head loss through back wash is the summation of the heads loss in the expanded bed, the gravel, the underdrains, the pipe, and valves.

$H=h_f+h_g+h_u+h_p$

h_f is the head loss in the expanded bed:

$h_f=L(1-\alpha)(S_g-1)$

where L is the depth of unexpanded bed, S_g is the specific density of the medium.

h_g is the head loss in the gravel:

$$h_g = 200L_g \frac{U_b\mu}{\rho g\phi^2 d_{60}^2} \times \frac{(1-\alpha)}{\alpha^3}$$

where L_g is the depth of the gravel.

Hu is the head loss in the underdrain system:

$$h_u = \frac{1}{2g}\left(\frac{U_b}{\gamma\beta}\right)^2$$

Where γ is an orifice coefficient and β is the ratio of the orifice area to the filter bed area (normally 0.2 to 0.7 percent).

h_p is the head loss in the pipe and valves. If these are replaced by an equivalent pipe the head loss is calculate from:

$$h_p = F\frac{L}{d}\left(\frac{4AU_b}{\pi d^2}\right)^2$$

where L is the depth, d is the diameter and F is the friction factor of the equivalent pipe and A is the filter bed area.

7. 10 Design of rapid sand filter:

We will design a rapid sand filter and a clear well chamber.
For the rapid sand filter, the most important dimension is the surface area. Filters must be designed so that the water flowing through is spread out over enough surface area that the filtration rate is within the recommended range.

The **clear well** is a reservoir for storage of filter effluent water. In this lesson, we will design a clear well with sufficient volume to backwash the rapid sand filter we design. However, clear wells have other purposes, most important of which is to allow sufficient contact time for chlorination. We will discuss chlorination in the next lesson.

Example:

A water treatment plant will typically have several filters. Each filter in our calculations will be assumed to have the following specifications.

- Square tank
- Basin depth: 10 ft
- Media depth: 2-3 ft
- Surface area: <2,100 ft^2
- Filtration rate: 2-10 gal/min-ft^2
- Flow through filter: 350-3,500 gpm
- Backwash frequency: every 24 hours
- Backwash period: 5-10 minutes
- Backwash water: 1-5% of filtered water

- Backwash rate: 8-20 gal/min-ft^2
- Filter rise rate: 12-36 in/min
- Bed expansion: 50%
- Backwash trough 3 ft above media
- Backwash water piped to raw water intake

As you can see, backwashing is a very important part of filter calculations. We will briefly identify some of the backwash characteristics below.

The **backwash frequency** is the same as filter run time. Either term can be used to signify the number of hours between backwashing.

The **backwash period** is the length of time which backwashing lasts.

The **backwash water** is the water used to backwash the filter. For the filters we're considering, backwash water should be 1-5% of the water filtered during the filter run.

The **backwash rate** is the rate at which water is forced backwards through the filter during backwashing. This rate is homologous to the filtration rate, only with water moving in the other direction through the filter. The backwash rate is typically much greater than the filtration rate.

The **filter rise rate** is the speed at which water rises up through the filter during backwashing. This is another way of measuring the backwash rate.

During backwashing, the water pushes the media up until it is suspended in the water. The height to which the media rises during backwashing is known as the **bed expansion**. For example, if the filter media is 2 feet deep, it may rise up to 3 feet deep during backwashing. This is a 50% bed expansion:

$$\text{Bed expansion} = \frac{(\text{New depth} - \text{Old depth}) \times 100\%}{\text{Old depth}}$$

$$\text{Bed expansion} = \frac{(3\text{ ft} - 2\text{ ft}) \times 100\%}{2\text{ ft}}$$

Bed expansion = 50%

Most of these backwash specifications merely describe the type of filter we will be considering and are not used in calculations. However, two factors - the filter rise rate and the backwash period - will be used when calculating the volume of the clear well chamber.

Overview of Calculations:

1. Calculate the approximate number of filters required.
2. Calculate the flow through one filter.
3. Calculate the surface area of one filter.
4. Calculate the length of the tank.
5. Calculate the clearwell volume.

1. Number of Filters:

The treatment plant's flow should be divided into at least three filters. You can estimate the number of filters required using the following formula:

$$\text{Number of filters} = 2.7\sqrt{Q}$$

Where:

Q = Flow, MGD

So, for a plant with a flow of 1.5 MGD, then the approximate number of filters would be:

$$\text{Number of filters} = 2.7\sqrt{1.5}$$

$$\text{Number of filters} = 3$$

2. Flow:

Next, the flow through one filter is calculated just as it was for one tank of the sedimentation basin:

$Q_{one} = Q_{total} / n$

$Q_{one} = (1.5\ \text{MGD}) / 3$

$Q_c = 0.5\ \text{MGD}$

So the flow through each of our three filters will be 0.5 MGD.

3. Surface Area:

The required filter surface area is calculated using the formula below:

$A = Q_{one} / F.R.$

Where:
A = filter surface area, ft^2
Q_c = flow into one filter, gpm
F.R. = filtration rate, gal/min-ft^2

We will use a filtration rate of 4 gal/min-ft.^{2} We will also have to convert from gpm to MGD. The calculations for our example are shown below:

$A = 500{,}000$ gal/day × (1 day / 1440 minutes) / 4 gal/min-ft^2

$A = 87\ ft^2$

4. Tank Length:

Since the filter tank is a square, the length of the tank can be calculated with the following simple formula:

$L = \sqrt{A}$

Where:

L = Length, ft
A = Surface area, ft^2
In the case of our example, the length of one tank is calculated as follows:

$L = \sqrt{87}$

$L = 9.3$ ft
This is the final calculation required for the design of the filter.

5. Clearwell Volume (Tank of backwash):

The volume of the clear-well must be sufficient to provide backwash water for each filter. First we calculate the total filter area:
Total filter area = A × (Number of filters)
For our example, the total filter area is:
Total filter area = 87 ft^2 × 3
Total filter area = 261 ft^2
Then we calculate the volume of the clearwell as follows:
V = (Backwash period) (Total filter area) (Filter rise rate)

We will assume a 5 minute backwash period and filter rise rate of 30 in/min. So, for our example, the volume of the clearwell would be calculated as follows:

$V = (5 \text{ min})\ (261 \text{ ft}^2)\ (30 \text{ in/min})\ (1 \text{ ft} / 12 \text{ in})$

$V = 3,\ 263 \text{ ft}^3$

You will notice that we translated from inches to feet.

So, for our plant, we need three filters, each with a surface area of 87 ft^2 and a length of 9.3 ft. In order to accommodate backwashing all three filters at once, the clearwell volume should be 3,263 $ft.^3$

Example:

Filters of water treatment plant have the following data:

-Total flow is 15000 m^3/day. – Filtration rate is 125 m/day.

- Dimension of one filter is 4 m width and 6 m length.

- Backwash of filter every 42 hrs.-Backwash rate is 14.4 m/hr for 0.2 hr.

-Influent suspended solid concentration to filter is 0.0018%.

Effluent suspended solid concentration out filter is 5% of influent suspended solid concentration.

-Specific gravity of S.S is 1.2 ton/m^3.

-Use two trough for each filter, n=0.013

Find the following: (1) No. of filters required. (2) Volume of voids of filter.

(3) Volume of tank of backwash water of filter. (4) Dimensions of $\sqcup$ shape trough.

Solution:

1- Q= 15000 m^3/day, Filtration rate, F.R= 125 m/day

$$Total area = \frac{Q}{F.R} = \frac{15000}{125} = 120 m^2$$

$$Area of one filter = 4 \times 6 = 24 m^2$$

$$No. of filters required = \frac{120}{24} = 5\, filters (use one stan dby)$$

2- Influent S.S= 0.0018%=18 ppm= 18 mg/L
Effluent S.S= 0.05 × Influent S.S= 0.05 × 18= 0.9 mg/L
S.S removed= 18-0.9=17.1 mg/L=17100 mg/m^3
Weight of S.S removed=17100 × 125×(42/24)×(4×6)=0.089775 Ton
Volume of voids of filter= 0.089775/1.2=0.0745 m^3

3- Volume of backwash water (Volume of Clearwell tank)= Area of filter ×backwash rate × time for washing.
=(4×6)×14.4×0.2=69.12 m^3.

4- Assume b=y

$$y = 1.73\left[\frac{Q^2}{gy^2}\right]^{1/3}$$

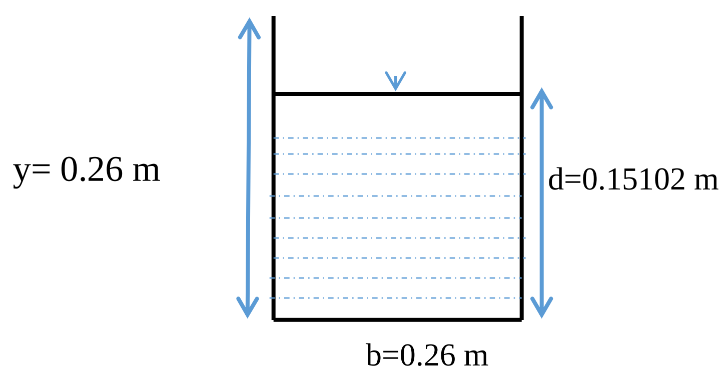
Q backwash= $V_{backwash}$×A of half filter
= 14.4 ×(4×3)= 172.8 m^3/hr= 0.048 m^3/sec

$$y^5 = 1.73^3 \frac{Q^2}{g}$$

$$y^5 = 1.217 \times 10^{-3}$$

$$y = 0.26126 m$$

$$b = 0.26126 m$$



$$y_X = y(1 - 0.422\frac{X^2}{L^2})$$

Where y_X is the water depth at any distance, m.

X is the distance, m

y is the trough depth, m.

$$y_X = y(1-0.422\frac{X^2}{L^2})$$

$$y_L = y(1-0.422\frac{L^2}{L^2})$$

$$y_L = 0.578y$$

$$d = \frac{y}{1.73} = \frac{0.26126}{1.73} = 0.15102m$$

5- Slop of trough $S = (\frac{nv}{r^{2/3}})^2$

$$r = \frac{A}{P} = \frac{d \times b}{2d+b} = \frac{0.15 \times 0.26}{2 \times 0.15 + 0.26} = 0.0696$$

$$v = \frac{Q}{A} = \frac{Q}{d \times b} = \frac{0.048}{0.15 \times 0.26} = 1.231m/s$$

$$S = (\frac{0.013 \times 1.231}{0.0696^{2/3}})^2 = 0.0089m/m$$

Total head loss in trough=S× length of trough (width of filter)= 0.0089×4=0.035m

6- Head loss in filter bed

Applied Carmen-Kozney equation:

$$h_f = \frac{fL(1-e)v_s^2}{e^3 g d_p}$$

$$v_s = 125\frac{m}{day}\times\frac{1day}{86400\sec} = 0.0014m/\sec$$

$$e = \frac{Volumeofvoids}{volumeoffilterbed}$$

$$Volumeoffilterbed = 0.6\times4\times6 = 14.4m$$

$$e = \frac{0.0745}{14.4} = 0.00517$$

$$h_f = \frac{2.6\times10^{-4}\times0.6(1-0.00517)(0.0014)^2}{0.00517^3\times9.81\times\frac{0.5}{1000}} = 0.224m$$

2019-2020

Unit Eight

Disinfection

Env. Eng. Dep. – Third Stage

8.1 Disinfection:

The filtered water may normally contain some harmful disease's producing bacteria in it. These bacteria must be killed in order to make the water safe for drinking. The process of killing these bacteria is known as ***Disinfection.***

Elemental chlorine is commonly employed in municipal treatment applications. Water disinfection is also practiced by means of storage or by the application of heat, irradiation by ultraviolet rays, applying metal ions such as copper and silver, and oxidants such as halogens, and ozone etc.

8.2 Points of Chlorine Application:

Use of chlorine in various stages of treatment, and even in raw water collection and potable water distribution system, is common practice. Multiple or split chlorination schemes frequently enhance the efficiency of many other unit water treatment processes, such as flocculation. In practice, the terms *plain-chlorination (simple chlorination), pre- chlorination, post- chlorination*, and *re-chlorination* are commonly used to specify the location at which chlorine is applied.

Plain or simple chlorination represents the sole public health safeguard and more than half of all existing water treatment facilities in the United States fall into this category. It involves the application of chlorine to water that receives no other treatment.

Pre-chlorination is useful in the following applications:

1. To improve coagulation and to suppress the decomposition of organic matter in sludge deposited in flocculation basins,
2. To control algae and other microorganisms,
3. To destroy taste-, odor- and color-producing materials by oxidation,
4. To retard decomposition in settling basins,
5. To improve filter operation by reduction and equalization of the bacterial and algae load,
6. To control slime and mud ball formation in filters, and
7. To provide a safety factor in disinfecting heavily contaminated waters.

Post chlorination follows filtration for disinfection and for provision of free or combined residual chlorine in a part or the entire distribution system. Chlorine should be applied either to the filter effluent or to the filter clear well influent.

Re-chlorination is common where the distribution system is long and complex and where the plant effluent residual is insufficient to control bacterial and algal re-growth and red water troubles. Table (8.1) shows the addition of chlorine at different locations.

Table (8.1): Locations of chlorine addition.

الموقع	التركيز (ملغ/ل)	ملاحظات
أ- شبكة الصرف الصحي:		
الإقلال من نمو الطحالب في المصارف	١-١٠	التحكم بالفطريات والبكتريا المنتجة للطحالب
التحكم بالتآكل (H_2s)	(٢-٩)لكل ملغ/ل H_2s	تهديم كبريت الهيدروجين H_2s في المصارف
التحكم بالروائح	(٢-٩) لكل ملغ/ ل H_2s	في المصارف قليلة الميل الطولي وفي المحطات الضخ والرفع
ب- محطة المعالجة:		
إزالة الشحوم	٢-١٠	يضاف قبل التهوية المسبقة
انقاص الـ(BOD_5)	(٠.٥-٢) لكل ملغ /ل BOD_5 مزال	أكسدة المواد العضوية
أكسدة كبريتات الحديدوز		انتاج كبريتات الحديديك وكلورايد الحديديك
التحكم بانسداد المرشح البيولوجي	١-١٠	كلور متبقي عند الفراغات
التحكم بالذباب والبعوض عند المرشح الحجري	٠.١-٠.٥	يطبق عادة حين موسم التكاثر
التحكم بانتفاخ الحماة	١-١٠	إجراء مؤقت
أكسدة مياه أحواض الهضم	٢٠-١٤٠	
التحكم بالرغوة من أحواض الهضم	٢-١٥	
انقاص النترات		تحويل النترات إلى أمونيا
جـ- المياه الصادرة عن المعالجة		
انقاص البكتريا	٢-٢٠	للمياه الصادرة عن المحطة
د- التعقيم:		
بعد ترسيب أولي في المحطة	٥-٢٠	
بعد ترسيب كيميائي	٢-٦	
بعد المرشحات البيولوجية	٣-١٥	
بعد الحماة المنشطة	٢-٨	
بعد الحماة المنشطة والمرشحات	١-٥	

8.3 Chlorination:

One of the most methods used to disinfect the water is chlorination. Many properties of chlorine make it an ideal disinfectant. It is highly soluble in water, so it is easy to apply and it is inexpensive. When chlorine gas is dissolved in water, hypochlorite acid is formed (HOCL). In this form chorine exists as free chlorine residual:

$$Cl_2 + H_2O \rightarrow HOCl + H^+ + Cl^-$$

Hypochlorous acid is a week acid that dissociate to form hypochlorite ion (OCl^-):

$$HOCl \rightarrow OCl + H^+$$

The relative concentration of HOCl and OCl^- vary with pH, water temperature and concentration of chlorine in solution. HOCl is many stronger an oxidant than OCl^- . The predominate concentration of HOCl and OCl^- are below pH 6 and above pH 7.5 respectively.

Free chlorine is also added in water from hypochlorite salt (calcium and sodium):

$$Ca(OCl)_2 + 2H_2O \rightarrow 2HOCl + Ca(OH)_2$$

$$NaOCl + H_2O \rightarrow HOCl + NaOH$$

Factors that affect the disinfection efficiency of chlorine are:

1) Nature of the disinfectant.
2) Concentration of the disinfectant.
3) Length of contact time with the disinfectant.
4) Temperature.
5) Type and concentration of organisms.
6) pH.

8.3.1 Chloramines:

Some plants use chloramines rather than hypochlorous acid to disinfectant the water. To produce chloramines, first chlorine gas or hypochlorite is added to the water to produce hypochlorous acid. Then ammonia is added to the water to react with the hypochloruos acid and produce a chloramine. Three types of chloamines can be formed in water – monochloramine, dichloramine and trichloramine.

Monochloramine is formed from the reaction of hypochlorous acid with ammonia:

$\mathbf{NH_3 + HOCl \rightarrow NH_2Cl + H_2O}$ (monochloramine)

Monocloromine may then react with more hypochlorous acid to form a dichloramine:

$\mathbf{NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O}$ (dichloramine)

Finally, the dichloromine may react with hypochlorous acid to form a trichloramine :

$\mathbf{NH_2Cl + HOCl \rightarrow NCl_3 + H_2O}$ (trichloramine or nitrogen trichloride)

8.3.2 Breakpoint chlorination:

When chlorine is added to water, it is consumed in oxidizing a wide variety of compound in water. No chlorine can be measured until the initial chlorine demand is satisfied. Then chlorine reacts with ammonia to produce combined chlorine residual (chloromines). The combined chlorine residual increases with additional dosage until a maximum combined residual is reached. Further addition of chlorine causes a decrease in combine residual. This is called *breakpoint chlorination*. After breakpoint chlorination is reached, free chlorine residual develops at the same rates as that of the applied dose.

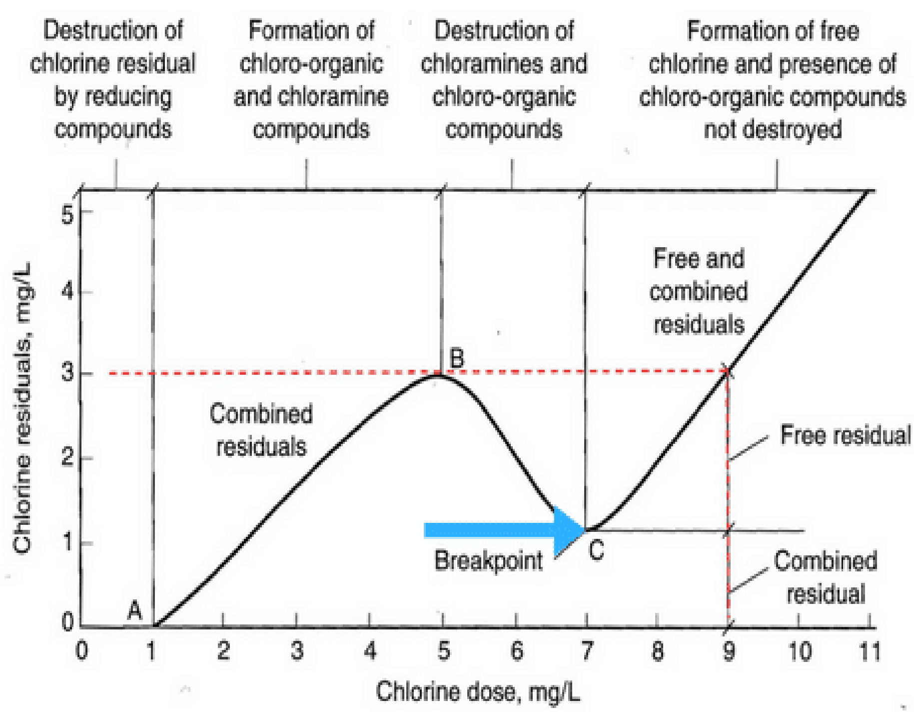


Figure 8.2: Chlorine Breakpoint curve

8.3.3 Chlorine dosage residual:

The chlorine dosage is the amount of chlorine added to the water to produce a specified residual. Chlorine dosage required for any of the application points in the treatment plant are the best determined by bench scale or pilot plant testing. Chlorine dosage may vary with the raw water quality, temperature, and other climatic conditions. Typically, the dosage may range from 0.2 to 4 mg/L.

Chlorine residual will vary according to operating experience. Typically, the combined chlorine residual may range from 0.5- 1 mg/L.

Example 1:

Determine the breakpoint dosage and the dosage required to provide a free residual of 1mg/L from the following table:

Dosage, mg/L	1	2	3	4	5	6	7
Residual, mg/L	0.8	1.55	1.95	1.24	0.5	0.85	1.95

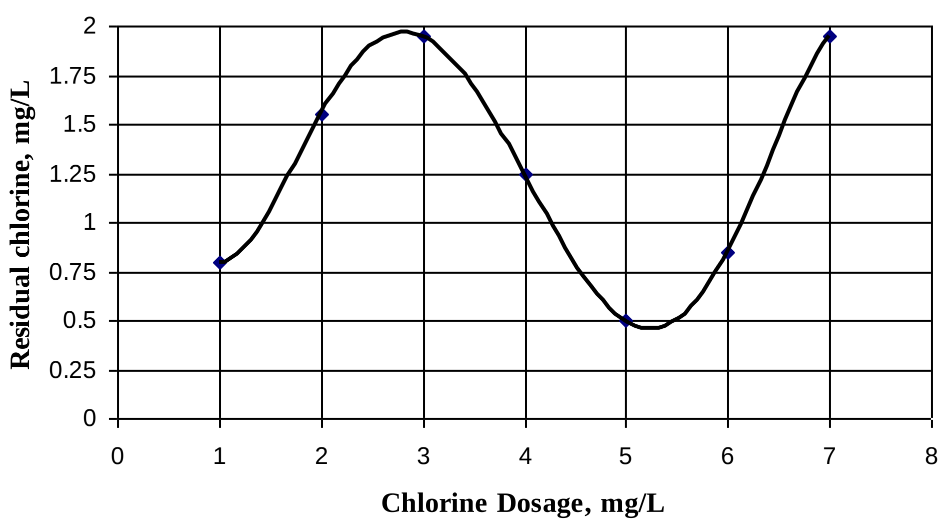
Solution:

At first, plot the chlorine demand curve.

1. The breakpoint dosage is 5.1 mg/L.
2. The combined residual after the breakpoint is approximately equal 0.475 mg/L.

The total residual (free + combined)= 0.475+1= 1.475 mg/L.

Therefore, the chlorine dosage to reach 1mg/L free residual chlorine will be 6.7 mg/L.



Example 2:

For the data in the table below, find:

1. The chlorine dosage at the breakpoint.
2. The daily quantity of NaOCl to be applied to this water if the free residual is 0.4 mg/L and the flow is 6.34 MGD.
3. The chlorine dosage at combined residual=0.3 mg/L

Atomic weights are: Na=23, O=16, Cl=35.5

Dosage, mg/L	0.2	0.4	0.6	0.8	1	1.2	1.4	1.6
Residual, mg/L	0.19	0.37	0.51	0.5	0.2	0.4	0.6	0.8

Solution:

At first, plot the chlorine demand curve. From the curve

1. The chlorine dosage at breakpoint= 1mg/L.
2. The residual combined after breakpoint is 0.2 mg/L and the free residual is 0.4 mg/L.

Therefore, the total residual is 0.6 mg/L and the chlorine dosage for this residual is 1.4 mg/L

$$NaOCl \longrightarrow Na^+ + OCl^-$$

74.5 51.5

One mole of OCl^- is equivalent to 1 mole of Cl_2

$$Cl_2 + H_2O \longrightarrow (H^+ + OCl^-) + HCl$$

71 51.5

Concentration of NaOCl used to achieve free residual=0.4 mg/L is:

74.5/x = 71/1.4

X= 1.47 mg/L

The daily quantity of NaOCl= 1.47 mg/L × Kg/10^6 mg × (6.34× 10^6) gallon/day × 3.784 L/gallon = 35.24 kg.

3. The chlorine dosage at combined chlorine= 0.3mg/L is 0.32 mg/L.