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LECTURES
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Contents

CHAPTER ONE

WATER SUPPLY ENGINEERING

1. Introduction

Water is the most important natural resource in the world since without it life cannot exist and most industries could not operate. Although human life can exist for many days without food, the absence of water for only a few days has fatal consequences. The presence of a safe and reliable source of water is thus an essential prerequisite for the establishment of a stable community. In the absence of such a source a nomadic life style becomes necessary and communities must move from one area to another as demands for water exceed its availability. Other conflicts in relation to water supplies can arise because of the effects which human and industrial wastes can have on the environment. This means that the importance of water as a natural resource which requires careful management and conservation must be universally recognized. Although nature often has great ability to recover from environmental damage, the growing demands on water resources necessitate the professional application of fundamental knowledge about the water cycle to ensure the maintenance of quality and quantity.

The chemical formula for water, H_2O, is widely recognized, but unfortunately it is somewhat of a simplification since water has several properties which cannot be explained by such a simple structure. Because of its molecular structure and its electrical properties of a very high dielectric constant and a low conductivity, water is capable of dissolving many substances, so that the chemistry of natural water is very complex. All natural waters contain varying amounts of other materials in concentrations ranging from minute traces at the ng/l level of trace organics in rain water, to around 35000mg/l in seawater. Wastewaters usually contain most of the dissolved constituents of the water supply to the area with additional impurities arising from the waste-producing processes.

2. Sustainable Development

In the developed world environmental matters now receive a great deal of public attention and the environment has taken on political implications. Although population growth is usually low in some countries, so that demands for water are not increasing greatly, there are a number of problems which focus attention on water quality control. Improved analytical techniques can now reveal the presence in water of hundreds of trace chemicals which arise from industrial processes and also as a consequence of some water and wastewater treatment processes.

Better understanding of food chains and the ability to understand complex biochemical and ecological reactions has resulted in water industries in many countries being subjected to more stringent restrictions on their operations and levels of service. The possibility of the greenhouse effect and changes in the ozone layer, producing far-reaching alterations in our environment, are further causes for public discussion and concern. In most developed countries there is an appreciation that environmental matters pose complex problems and there is a need for an overall view of the topic. In the less developed countries, although the need for environmental protection is recognized in some circles, the apparently more urgent pressures of population growth and economic survival usually control the situation.

The European Commission defines the objectives of a sustainable water policy as

- provision of a secure supply of safe drinking water in sufficient quantity.
- provision of water resources of sufficient quality and quantity to meet other economic requirements of industry and agriculture.
- quality and quantity of water resources sufficient to protect and sustain the good ecological state and functioning of the aquatic environment.
- management of water resources to prevent or reduce the adverse impact of floods and minimize the effects of droughts.

Sustainable development has been embraced as the leading philosophy that would on the one hand allow the world to develop its resources and on the other hand preserve unrenewable and finite resources and guarantee adequate living conditions for future generations. Presently the definition most often used of sustainable development is: the ability of the present generation to utilize its natural resources without putting at risk the ability of future generations to do likewise. Sustainable development is making efficient use of our natural resources for economic and social development while maintaining the resource base and environmental carrying capacity for coming generations. This resource base should be widely interpreted to contain besides natural resources: knowledge, infrastructure, technology, durables and human resources. In the process of development natural resources may be converted into other durable products and hence remain part of the overall resource base.

Water resources development that is not sustainable is ill-planned. In many parts of the world, fresh water resources are scarce and to a large extent finite. Although surface water may be considered a renewable resource, it only constitutes 1.5% of all terrestrial fresh water resources; the vast majority is groundwater (98.5%) part of which is virtually unrenewable. Consequently, there are numerous ways to endanger the future use of water either by over-exploitation (mining) of resources or by destroying resources for future use (e.g. pollution).

3. Water Resources

Water in our planet is available in the atmosphere, the oceans, on land and within the soil and fractured rock of the earth's crust Water molecules from one location to another are driven by the solar energy. Moisture circulates from the earth into the atmosphere through evaporation and then back into the earth as precipitation. In going through this process, called the Hydrologic Cycle. Water is a finite natural resource and in the context of a tripling of global water use since 1950 many parts of the world are facing growing pressures on their water resources. In Europe the demand for water has increased from 100 km^3 a year in 1950 to 550 km^3 a year in 1990 with a predicted rise to 650 km^3 a year by 2000. In such circumstances, over abstraction from surface and underground supplies may provide short-term solutions but they are not sustainable in the longer term. The science of hydrology is concerned with the assessment of water resources in the hydrological cycle figure (1) and their management for the optimum results. It will be appreciated that in any management plan for water resources it is vital to assess both the quality and quantity of the available supplies.

The origin of water resources is rainfall. As rainfall reaches the surface it meets the first separation point. At this point part of the rainwater returns directly to the atmosphere, which is called evaporation from interception ***I***. The remaining rainwater infiltrates into the soil until it reaches the capacity of infiltration. This is called infiltration ***F***. If there is enough rainfall to exceed the interception and the infiltration, then overland flow (also called surface runoff) ***Qs*** is generated. The overland flow is a fast runoff process, which generally carries soil particles. A river that carries a considerable portion of overland flow has a brown muddy colour and carries debris. The hydrologic cycle consists of four key components

1. Precipitation
2. Runoff
3. Storage
4. Evapotranspiration

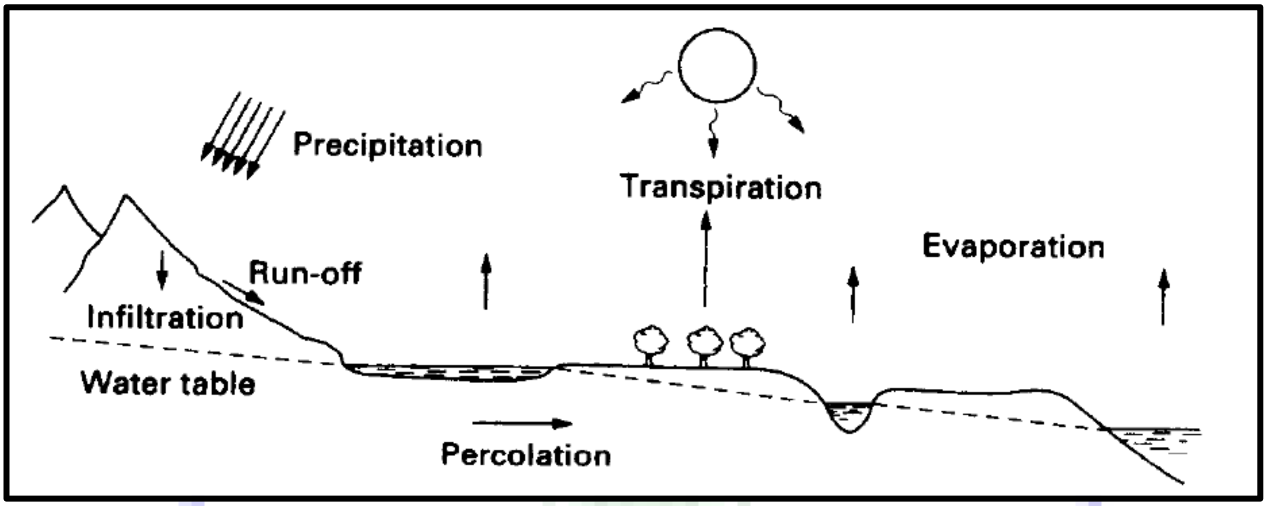

Figure (1) The hydrological cycle.

The various sources of water can be classified into two categories:

1. Surface sources, such as

a. Ponds and lakes;

b. Streams and rivers;

c. Storage reservoirs; and

d. Oceans, generally not used for water supplies, at present.

2. Sub-surface sources or underground sources, such as

a. Springs;

b. Infiltration wells; and

c. Wells and Tube-wells.

4. The role of engineers and scientists

Public works such as water supply and sewage disposal schemes have traditionally been seen as civil engineering activities and water engineering is probably the largest single branch of the civil engineering profession. The connection with civil engineering is due to the fact that most water engineering works involve large structures and require a good understanding of hydraulics.

Water science and technology is, however, an interdisciplinary subject involving the application of biological, chemical and physical principles in association with engineering techniques. Thus engineers and scientists who practice in water quality control must have a good appreciation of the interface between their individual disciplines and of the complex nature of many environmental reactions.

The increasing amount of information which is required for the efficient design and operation of water quality control systems means that practitioners must also be conversant with developments in information technology. Solutions to environmental problems are rarely cost-free and thus the choice between various options must be made with an understanding of basic economic principles. Major water quality control projects are undertaken by a team of specialists from many disciplines who can bring their own particular expertise to the project whilst appreciating the need for collaborative work between disciplines to produce a cost-effective, environmentally acceptable solution.

A major objective in water quality control work is to reduce the incidence of water-related diseases. This objective depends on the ability to develop water sources to provide an ample supply of water of wholesome quality, i.e. a water free from:

- ✓ visible suspended matter
- ✓ excessive color, taste and odor
- ✓ objectionable dissolved matter
- ✓ aggressive constituents
- ✓ bacteria indicative of human pollution.

Drinking water supplies must obviously be fit for human consumption, i.e. of potable quality, and they should also be palatable, i.e. aesthetically attractive. In addition, as far as is feasible, public water supplies should be suitable for other domestic uses such as clothes washing, and so on. Having provided a water of suitable quality and quantity by source protection and development and by the application of appropriate treatment processes, it becomes necessary to convey the supply to consumers via a distribution system comprising water mains, pumping stations and service reservoirs. Most domestic and industrial uses of water cause a deterioration in quality with the resultant production of a wastewater which must be collected and given suitable treatment before release to the environment. In many situations treated wastewaters provide a significant proportion of the water resources for other users.

5. *Water allocation principles*

An important purpose of water management is to match or balance the demand for water with its availability, through suitable water allocation arrangements. Water availability is dealt with in other courses (e.g. Hydrology). Water using activities aims to provide tools to estimate the demand for water for different types of use.

Water resource projects are constructed to develop or manage the available water resources for different purposes. According to the National Water Policy (2002), the water allocation priorities for planning and operation of water resource systems should broadly be as follows:

1. Domestic consumption

This includes water requirements primarily for drinking, cooking, bathing, washing of clothes and utensils and flushing of toilets.

2. Irrigation

Water required for growing crops in a systematic and scientific manner in areas even with deficit rainfall.

3. Hydropower

This is the generation of electricity by harnessing the power of flowing water.

4. Ecology / environment restoration

Water required for maintaining the environmental health of a region.

5. Industries

The industries require water for various purposes and that by thermal power stations is quite high.

6. Navigation

Navigation possibility in rivers may be enhanced by increasing the flow, thereby increasing the depth of water required to allow larger vessels to pass.

7. Other uses, Like entertainment of scenic natural view.

Demand for water is the amount of water required at a certain point. The *use* of water refers to the actual amount reached at that point.

We can distinguish *withdrawal* uses and *non-withdrawal* (such as navigation, recreation, waste water disposal by dilution) uses; as well as *consumptive* and *non-consumptive* uses.

Consumptive use is the portion of the water withdrawn that is no longer available for further use because of evaporation, transpiration, incorporation in manufactured products and crops, use by human beings and livestock, or pollution.

The terms "*consumption*", "use" and "*demand*" are often confused. The amount of water actually reaching the point where it is required will often differ from the amount required. Only a portion of the water used is actually consumed, i.e. lost from the water resource system.

6. Water demands

In the design of any waterworks project it is necessary to estimate the amount of water that is required. This involves determining the number of people who will be served and their per capita water consumption, together with an analysis of the factors that may operate to effect consumption. It is usual to express water consumption in liters or gallons per capita per day, obtaining this figure by dividing the total number of people in the city into the total daily water consumption. For many purposes the average daily consumption is convenient. It is obtained by dividing the population into the total daily consumption averaged over one year. It must be realized, however, that using the total population may, in some cases, result in serious inaccuracy, since a large proportion of the population may be served by privately owned wells. A more accurate figure would be the daily consumption per person served.

A fundamental prerequisite to begin the design of water supply facilities is a determination of the design capacity. This, in turn, is a function of water demand. The determination of water demand consists of four parts: (1) selection of a design period, (2) estimation of the population, commercial, and industrial growth, (3) estimation of the unit water use, and (4) estimation of the variability of the demand.

1.Population Determination

Determination of population is one of the most important factors in the planning, if the projects have to serve the community for a certain design period. Normally, a design period of 20 to 40 years is selected. What will be the population at the end of the design period, is the basic question. This can be achieved by using various methods used for population forecast.

2.Rate of Demand

The water consumption in a city may be conveniently divided into the following categories *(i)* domestic *(ii)* trade *(iii)* agricultural *(iv)*public and *(v)* losses. The total consumption of water depends upon several factors, such as climatic condition, cost of water, living standards of inhabitants, pressure in the pipelines, type of supply, …etc. The total quantity of water required divided by the total population gives per capita water demand. The accurate measurements of consumption is often very difficult because of standards of water supply and maintenance vary widely.

Following are the various types of water demands of a city or town:

i. Domestic water demand
ii. Industrial demand
iii. Institution and commercial demand
iv. Demand for public use

v. Fire demand
vi. Loses and wastes

6.1 Water Quantity Estimation

The quantity of water required for municipal uses for which the water supply scheme has to be designed requires following data:

1. Water consumption rate *(Per Capita Demand in litres per day)*
2. Population to be served.

Quantity= Per capita demand x Population

§ 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 (1) Water Consumption for Various Purposes

No.	Types of Consumption	Normal Range (lit/capita/day)	Average	%
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

1.Domestic water demand

The quantity of water required in the houses for drinking, bathing, cooking, washing etc is called domestic water demand and mainly depends upon the habits, social status, climatic conditions and customs of the people. As per IS: 1172-1963, under normal conditions, the domestic consumption of water in India is about 135 litres/day/capita. But in developed countries this figure may be 350 litres/day/capita because of use of air coolers, air conditioners, maintenance of lawns, automatic household appliances.

The details of the domestic consumption are

a) Drinking ------ 5 litres
b) Cooking ------ 5 litres
c) Bathing ------ 55 litres
d) Clothes washing ------ 20 litres
e) Utensils washing ------ 10 litres
f) House washing ------ 10 litres
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135 litres/day/capita

2.Industrial demand

The water required in the industries mainly depends on the type of industries, which are existing in the city. The water required by factories, paper mills, Cloth mills, Cotton mills, Cement, Sugar refineries etc. comes under industrial use. The quantity of water demand for industrial purpose is around 20 to 25% of the total demand of the city.

3.Institution and commercial demand

Universities, Institution, commercial buildings and commercial centers including office buildings, warehouses, stores, hotels, shopping centers, health centers, schools, mosques, cinema houses, railway and bus stations etc comes under this category. As per IS: 1172-1963, water supply requirements for the public buildings other than residences as follows.

Table (2) Institutional and Commercial Water Demand

Sl.No.	Type of Building	Construction per capita per day (litres)
1.	a) Factories where bathrooms are required to be provided.	45
	b) Factories where no bathrooms are required to be provided	30
2.	Hospitals per bed	
	a) No. of beds not exceeding 100 No.	340
	b) No. of beds exceeding 100 No.	450
3.	Nurses homes and medical quarters.	135
4.	Hostels	135
5.	Offices	45
6.	Restaurants (per seat)	70
7.	Hotel (per bed)	180
8.	Cinema concert halls and theatres (per seat)	15
9.	Schools	
	a) Day schools	45
	b) Boarding schools	135
10.	Garden, sports grounds	35 per sq.m
11.	Animal/vehicles	45

4.Demand for public use

Quantity of water required for public utility purposes such as for washing and sprinkling on roads, cleaning of sewers, watering of public parks, gardens, public fountains etc comes under public demand. To meet the water demand for public use, provision of 5% of the total consumption is made designing the water works for a city. The requirements of water for public utility shall be taken as given in *Table (3)*

Table (3) Public Use Water Demand

Sl.No.	Purpose	Water Requirements
1.	Public parks	1.4 litres/m^2/day
2.	Street washing	1.0-1.5 litres/m^2/day
3.	Sewer cleaning	4.5 litres/head/day

5.Fire demand

Fire may take place due to faulty electric wires by short circuiting, fire catching materials, explosions, bad intension of criminal people or any other unforeseen mis-happenings. If fires are not properly controlled and extinguished in minimum possible time, they lead to serious damage and may burn cities. All the big cities have full firefighting squads. As during the fire breakdown large quantity of water is required for throwing it over the fire to extinguish it, therefore provision is made in the water work to supply sufficient quantity of water or keep as reserve in the water mains for this purpose. In the cities fire hydrants are provided on the water mains at 100 to 150 m apart for fire demand. The quantity of water required for firefighting is generally calculated by using different empirical formulae. The rate of fire demand is sometimes treated as a function of population and is worked out from following empirical formulae:

No.	*Authority*	*Formulae (P in thousands)*
1	**American Insurance Association**	$Q, L/min. = 4637\sqrt{P}\left(1 - 0.01\sqrt{P}\right)$
2	* **Kuchling's Formula**	$Q, L/min. = 3182\sqrt{P}$
3	**Freeman's Formula**	$Q, L/min. = 1136.5\left(\frac{P}{5} + 10\right)$
4	**Ministry of Urban Development Manual Formula**	$Q, kilo\ Liters/min. = 100\sqrt{P}$ **For P > 50000**

Although the actual amount of water used in a year for fire fighting is small, the rate of use is large. The Insurance Services Office[5] uses the formula

$$F = 18C(A)^{0.5}$$

in which F is the required fire flow in gpm (l/min/3.78), C is a coefficient related to the type of construction, and A is the total floor area in ft^2 (m^2 × 10.76) excluding the basement of the building.

C ranges from a maximum of 1.5 for wood frame to a minimum of 0.6 for fire resistive construction. The fire flow calculated from the formula is not to exceed 8000 gpm (30,240 l/min) in general, nor 6000 gpm (22,680 l/min) for one story construction. The minimum fire flow is not to be less than 500 gpm (1890 l/min). Additional flow may be required to protect nearby buildings. The total for all purposes for a single fire is not to exceed 12,000 gpm (45,360 l/min) nor be less than 500 gpm (1890 l/min).

For groups of single and two-family residences the following table may be used to determine the required flow.

The fire flow must be maintained for a minimum of 4 hours as shown in Table -4. Most communities will require a duration of 10 hours.

In order to determine the maximum water demand during a fire, the fire flow must be added to the maximum daily consumption. If it is assumed that a community with a population of 22,000 has an average consumption of 600 l per capita/day and a fire flow dictated by a building of ordinary construction with a floor area of 1000 m^2 and a height of 6 stories, the calculation is as follows:

$$\text{Average domestic demand} = 22{,}000 \times 600 = 13.2 \times 10^6 \text{ l/day}$$

$$\text{Maximum daily demand} = 1.8 \times \text{avg} = 23.76 \times 10^6 \text{ l/day}$$

Table (4) Residential fire flow

Distance between adjacent units		Required fire flow	
ft	m	gpm	l/min
> 100	> 30.5	500	1890
31–100	9.5–30.5	750–1000	2835–3780
11–30	3.4–9.2	1000–1500	3780–5670
≤ 10	≤ 3.0	1500–2000†	5670–7560†

† For continuous construction use 2500 gpm (9450 l/min).

Table (5) fire flow duration

Required fire flow		Duration, h
gpm	l/min	
< 1000	< 3780	4
1000–1250	3780–4725	5
1250–1500	4725–5670	6
1500–1750	5670–6615	7
1750–2000	6615–7560	8
2000–2250	7560–8505	9
> 2250	> 8505	10

$$F = 18(1)(1000 \times 10.76 \times 6)^{0.5} = 4574 \text{ gpm}$$
$$= 17{,}288 \text{ l/min} = 24.89 \times 10^6 \text{ l/day}$$

$$\text{Maximum rate} = 23.76 \times 10^6 + 24.89 \times 10^6$$
$$= 48.65 \times 10^6 \text{ l/day}$$
$$= 2211 \text{ l per capita/day for 10 hours}$$

The total flow required during this day would be

$$23.76 + 24.89 \times 10/24 = 34.13 \times 10^6 \text{ l}$$
$$= 1551 \text{ l per capita/day}$$

The difference between the maximum domestic rate and the values above is frequently provided from elevated storage tanks.

6. Losses and Wastes

All the water, which goes in the distribution, pipes does not reach the consumers. The following are the reasons:

i. Losses due to defective pipe joints, cracked and broken pipes, faulty valves and fittings.

ii. Losses due to, consumers keep open their taps of public taps even when they are not using the water and allow the continuous wastage of water.

iii. Losses due to unauthorized and illegal connections

While estimating the total quantity of water of a town; allowance of 15% of total quantity of water is made to compensate for losses, thefts and wastage of water.

§ *Per Capita Demand*

If 'Q' is the total quantity of water required by various purposes by a town per year and 'P' is population of town, then per capita demand will be

Per capita demand = $\frac{Q}{P \times 365}$ litres/day

Per capita demand of the town depends on various factors like standard of living, no. and type of commercial places in a town etc. For an average Indian town, the requirement of water in various uses is as below

i. Domestic purpose	--------	135 litres/c/d
ii. Industrial use	--------	40 litres/c/d
iii. Public use	--------	25 litres/c/d
iv. Fire Demand	--------	15 litres/c/d
v. Losses, Wastage and thefts	--------	55 litres/c/d
	Total:	270 litres/capita/day

The total quantity of water required by the town per day shall be 270 multiplied with the total population in litres/day.

§ *Factors affecting per capita demand:*

Design Period

The design period (also called the design life) is not the same as the life expectancy. The design period is the length of time it is estimated that the facility will be able to meet the demand, that is, the design capacity. The life expectancy of a facility or piece of equipment is determined by wear and tear. Typical life expectancies for equipment range from 10 to 20 years. Buildings, other structures, and pipelines are assumed to have a useful life of 50 years or more. New water works are generally made large enough to meet the demand for the future. The number of years selected for the design period is based on the following:

- The rate of population growth.
- The useful life of the structures and equipment.
- The ease or difficulty of expansion.
- Performance in early years of life under minimum hydraulic load.

Design periods that are commonly employed in practice and commonly experienced life expectancies are shown in Table (6).

Table (6) Design periods for water works

Type of facility	Characteristics	Design period, y	Life expectancy, y
Large dams and pipelines	Difficult and expensive to enlarge	40–60	100+
Wells	Easy to refurbish/replace	15–25	25+
Treatment plants			
Fixed facilities	Difficult and expensive to enlarge/replace	20–25	50+
Equipment	Easy to refurbish/replace	10–15	10–20
Distribution systems			
Mains > 60 cm	Replacement is expensive and difficult	20–25	60+
Laterals and mains ≤ 30 cm	Easy to refurbish/replace	To full development[a]	40–50

Climatic conditions

The quantity of water required in hotter and dry places is more than cold countries because of the use of air coolers, air conditioners, sprinkling of water in lawns, gardens, courtyards, washing of rooms, more washing of clothes and bathing etc. But in very cold countries sometimes the quantity of water required may be more due to wastage, because at such places the people often keep their taps open and water continuously flows for fear of freezing of water in the taps and use of hot water for keeping the rooms warm.

Size of community

Water demand is more with increase of size of town because more water is required in street washing, running of sewers, maintenance of parks and gardens.

Living standard of the people

The per capita demand of the town increases with the standard of living of the people because of the use of air conditioners, room coolers, maintenance of lawns, use of flush, latrines and automatic home appliances etc.

Industrial and commercial activities

As the quantity of water required in certain industries is much more than domestic demand, their presence in the town will enormously increase per capita demand of the town. As a matter of the fact the water required by the industries has no direct link with the population of the town.

Pressure in the distribution system

The rate of water consumption increases in the pressure of the building and even with the required pressure at the farthest point, the consumption of water will automatically increase. This increase in the quantity is firstly due to use of water freely by the people as compared when they get it scarcely and more water loss due to leakage, wastage and thefts etc.

System of sanitation

Per capita demand of the towns having water carriage system will be more than the town where this system is not being used.

Cost of water

The cost of water directly affects its demand. If the cost of water is more, less quantity of water will be used by the people as compared when the cost is low.

Efficiency of water works administration

Leaks in water mains and services; and unauthorized use of water can be kept to a minimum by surveys.

Policy of metering and charging method

Water tax is charged in two different ways: on the basis of meter reading and on the basis of certain fixed monthly rate.

§Fluctuations in Rate of Demand

The per capita demand of town is the average consumption of water for a year. In practice it has been seen that this demand does not remain uniform throughout the year but it various from season to season, even hour to hour.

1. **Seasonal variations**

The water demand varies from season to season. In summer the water demand is maximum, because the people will use more water in bathing, cooling, lawn watering and street sprinkling. This demand will become minimum in winter because less water will be used in bathing and there will be no lawn watering. The variations may be up to 15% of the average demand of the year.

2. **Daily variations**

This variation depends on the general habits of people, climatic conditions and character of city as industrial, commercial or residential. More water demand will be on Sundays and holidays due to more comfortable bathing, washing etc as compared to other working days. The maximum daily consumption is usually taken as 180% of the average consumption.

3. Hourly variations

On Sundays and other holidays, the peak hours may be about 8 A.M. due to late awakening where as it may be 6 A.M. to 10 A.M. and 4 P.M. to 8 P.M. and minimum flow may be between 12P.M. to 4P.M. when most of the people are sleeping. But in highly industrial city where both day and night shifts are working, the consumption in night may be more. The maximum consumption may be rise up to 200% that of average daily demand.

The determination of this hourly variations is most necessary, because on its basis the rate of pumping will be adjusted to meet up the demand in all hours.

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

$$\textit{Per capita demand} = \frac{Q}{P \times 365} \quad \textit{litres/day}$$

Maximum daily demand = 1.8 x average daily demand

Maximum hourly demand of maximum day i.e. Peak demand

= 1.5 x average hourly 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

Important Notice

In the absence of water demand data, use the following equation:

$p = 180\,(t)^{-0.1}$ Where p= % of the annual average water daily demand (AAWD) for time (t) in days

Example

If the average annual water demand is 200gpcd , Find the maximum demand of water.

Solution

Max daily demand = $1.8\,(t)^{-0.1}\,Q = 1.8\,(1)^{-0.1}\,(200) = 360$ gpcd

Max Weekly demand = $1.8\,(t)^{-0.1}\,Q = 1.8\,(7)^{-0.1}\,(200 * 7) = 2{,}074.4$ gpcd

Max Monthly demand = $1.8\,(t)^{-0.1}\,Q = 1.8\,(30)^{-0.1}\,(200 * 30) = 7{,}686.2$ gpcd

Maximum hourly demand = 1.8 * 1.5 * annual average hourly demand = 2.7 x 200= 540 gpcd

§ Additional measures to reduce demands on scarce water resources could include

- installation of low-volume taps and plumbing fittings
- use of 'grey' water for toilet flushing
- on-site collection of rainwater for toilet flushing and garden watering
- development of domestic appliances with low water consumption
- recycling of wastewater effluents, possibly in dual-supply networks

Although all of the above proposals are capable of reducing domestic water consumption to some extent they do have inherent disadvantages due to installation and operating costs and, for some, the potential for water pollution from cross connections between potable and secondary supply systems.

6.2Population forecast

When the design period is fixed the next step is to determine the population of a town or city population of a town depends upon the factors like births, deaths, migration and annexation. The future development of the town mostly depends upon trade expansion, development industries, and surrounding country, discoveries of mines, construction of railway stations etc may produce sharp rises, slow growth, stationary conditions or even decrease the population. For the prediction of population, it is better to study the development of other similar towns, which have developed under the same circumstances, because the development of the predicted town will be more or less on the same lines. The following are the standard methods by which the forecasting population is done.

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 Increase Method
2. Geometric Increase Method
3. Incremental Increase Method
4. Decreasing Rate of Growth Method
5. Simple Graphical Method
6. Comparative Graphical Method
7. Ratio Method
8. Logistic Curve Method

6.2.1 Arithmetic Increase Method

This method should be used for forecasting population of those large cities which has reached their saturation population. This method is the simplest method of population forecasting though it gives lower results. In this method the increase in population from decade to decade is assumed constant. From the census data of past 3 or 4 decades, the increase in population for each decade is found, and from that, an average increment is found. For each successive decade this average increment is added. The population after 'n' decades can be determined by the formula.

$P_n = P + nI$ where

P_n: Future population at the end of n decades.

P: population at present   **n**: No. of decades

I: The average of increase of a decade

Example:

Predict the population for the years 2021, 2031, and 2041 from the following census figures of a town by Arithmetic Increase Method.

Year	1941	1951	1961	1971	1981	1991	2001	2011
Population (Thousands)	60	65	63	72	79	89	97	120

Year	Population (Thousands)	Increment per Decade
1941	60	-
1951	65	+5
1961	63	-2
1971	72	+9
1981	79	+7
1991	89	+10
2001	97	+8
2011	120	+23
Net values		+60
Averages		8.57

+ =increase ; - = decrease

$P_n = P + nI$

Average increases per decade = I = 8.57

Population for the years,

P_{2021}= population 2011 + nI   here n=1 decade

= 120 + 8.57 = 128.57

P_{2031}= population 2011 + nI here n=2 decade

=120 + 2 x 8.57 = 137.14

P_{2041}= population 2011 + nI here n=3 decade

= 120 + 3 x 8.57 = 145.71

6.2.2 Geometric Increase Method

This method is based on the assumption that the percentage increase in population from decade to decade remains constant. In this method the average percentage of growth of last few decades is determined, the population forecasting is done on the basis that percentage increase per decade will be the same. The population at the end of 'n' decades is calculated by

$$P_n = P\left(1 + \frac{I_g}{100}\right)^n$$

where

P_n: Future population at the end of n decades. **P**: population at present

I_g : average percentage of growth of 'n' decades **n**: No. of decades

Solving the example in *Page 19* using Geometric Increase Method

Year	Population (Thousands)	Increment per Decade	Percentage increase in population
1941	60	-	-
1951	65	+5	(5/60) x 100% = +8.33
1961	63	-2	-3.07
1971	72	+9	14.28
1981	79	+7	+9.72
1991	89	+10	+12.66
2001	97	+8	+8.98
2011	120	+23	+23.71
Net values		+60	+74.61
Averages		8.57	10.66

average percentage increase per decade, I_g= 10.66

$$P_n = P\left(1 + \frac{I_g}{100}\right)^n$$

The population at the end of various decades shall be as follows:

Year	Expected Population	
2021	I_g= 10.66, n=1	120 x (1+10.66/100) = 132.8
2031	I_g= 10.66, n=2	120 x $(1+10.66/100)^2$ = 146.95
2041	I_g= 10.66, n=3	120 x $(1+10.66/100)^3$ = 162.60

6.2.3 Incremental Increase Method

This method is improvement over the above two methods. The average increase in the population is determined by the arithmetical method and to this is added the average of the net incremental increase once for each future decade. The population in the next decade is found by adding to the present population the average increase plus the *average incremental increase* per decade. The process is repeated for the second future decade, and so on. Thus, the future population at the end of the *n* decades is given by:

$$P_n = P + nI + \frac{n(n+1)}{2}\, r$$

P_n: Future population at the end of n decades. **P**: population at present.

I: average increase per decade. **n**: No. of decades

r: Average incremental increase

Solving the example in *Page 19* using Incremental Increase Method

Year	Population (Thousands)	Increment per Decade	Incremental Increase
1941	60	-	-
1951	65	+5	-
1961	63	-2	-3
1971	72	+9	+7
1981	79	+7	-2
1991	89	+10	+3
2001	97	+8	-2
2011	120	+23	+15
Net values		+60	+18
Averages		8.57	3

Year	Expected Population	
2021	I= 8.57; n=1 ; r = 3	120 + 8.57 + 3 = 131.57
2031	I= 8.57; n=2 ; r = 3	120 + 2 x 8.57 + 9 = 146.14
2041	I= 8.57; n=3 ; r = 3	120 + 3 x 8.57 + 18 = 163.71

CHAPTER TWO

WATER POLLUTION

1. Introduction

The life and activities of plants and animals, including humans, contribute to the pollution of the earth, assuming that pollution is defined as the deterioration of the existing state. The purpose of this chapter is to review the various sources of water pollution in order to recognize the opportunities for eliminating, minimizing, reusing or treating these sources so that their negative effect on the environment will be minimized. When pollution control is considered, these questions should be asked and answered:

1. Can the pollution source be eliminated?

- Is it absolutely necessary?

- Can it be substituted by another source that accomplishes the same purpose but is less polluting to the environment?

2. Can the pollution source be minimized?

- Can the source be operated more efficiently to lower pollution?

- Can the pollutants be converted to another state (gaseous, liquid or solid) which is less polluting to the environment?

3. Can the pollutants be reused?

- Can the pollutants be purified and reused as raw materials?

- Can relatively pure water be separated from the pollutants and reused?

- Can the pollutant be recycled to a different source?

4. Can the pollutant be treated?

- Is the effect on the environment minimized by altering, destroying or concentrating the pollutant?

- Can the treated pollutant be reused or recycled?

Water becomes polluted when foreign substances enter the environment and are transported into the water cycle. These substances, known as pollutants, contaminate the water and are sometimes harmful to people and the environment. Therefore, water pollution is any change in water that is harmful to living organisms.

Sources of water pollution are divided into two main categories: point source and non-point source.

Point source pollution occurs when a pollutant is discharged at a specific source. In other words, the source of the pollutant can be easily identified. Examples of point-source pollution include a leaking pipe or a holding tank with a hole in it, polluted water leaving a factory, or garbage being dumped into a river. These sources of pollution are easy to identify because the cause of the pollution can be observed.

Non-point source pollution is more common, and contributes more pollution to surface water than does point source pollution. This type of pollution is difficult to identify and may come from pesticides, fertilizers, or automobile fluids washed off the ground by a storm. Non-point source pollution comes from three main areas: urban-industrial, agricultural, and atmospheric sources.

2. Sources of Water Pollution

2.1 Industrial Sources of Water Pollution

Any industry, in which water obtained from a water treatment system or a well comes in contact with a process or product can add pollutants to the water. The resulting water is then classified as a wastewater.

In the United States, the EPA has classified industries into Standard Industrial Classifications (SIC). Industries in any of these classifications can contribute to water pollution as their water supply is used in a process.

The following are examples of industrial water pollution sources:

Non-Contact Water

- Boiler feed water
- Cooling water
- Heating water
- Cooling condensate

Contact Water

- Water used to transport products, materials or chemicals
- Washing and rinsing water (product, equipment, floors)
- Solubilizing water
- Diluting water
- Direct contact cooling or heating water
- Sewage

• Shower and sink water

The wastewater can contain physical, chemical and/or biological pollutants in any form or quantity and cannot adequately be quantified without actual measuring and testing. The wastewater will typically either be discharged directly into a receiving body of water or into the sewerage system of a municipality, or it will be reused or recycled. Normally, a municipality will restrict industrial water pollutants to those listed below for municipalities, using a Pre-Treatment Ordinance.

A municipality require to limit by Pre-treatment Ordinance, industrial water pollutants to levels which will not 1) harm the municipal wastewater treatment system, 2) pass through the municipal system at levels not meeting the municipal discharge permit, or 3) be deposited in municipal sludge at an illegal concentration.

2.2 Municipal Sources of Water Pollution

The non-industrial municipal sources of water are typically as follows:

- Dwellings
- Commercial establishments
- Institutions (schools, hospitals, prisons, etc)
- Governmental operations

Table 2.1 lists municipal sources of water in terms of average flows per day and biological strength in BOD_5, the total amount of oxygen used by microorganisms during the first five days of biodegradation.

It is assumed that a non-industrial municipal wastewater source will contain no pollutants except for the following:

- Feces
- Urine
- Paper
- Food waste
- Laundry wastewater
- Sink, shower, and bath water

TABLE 2.1 Municipal Sources of Wastewater

Classification	Remarks	Average flow/ person/day	BOD_5 person per day
Municipality	Residential	100 gallons	0.20 lb.
Subdivision	Residential	100 gallons	0.20 lb.
Colleges		100 gallons	0.20 lb.
Hospitals	Per bed	200 gallons	0.40 lb.
Nursing homes		100 gallons	0.20 lb.
Schools, high	With cafeteria & showers	25 gallons	0.06 lb.*
Schools, elementary	With cafeteria & showers	20 gallons	0.06 lb.*
Factory or office bldg.	With showers/shift	35 gallons	0.06 lb.
Factory or office bldg.	Without showers/shift	25 gallons	0.06 lb.
Motels	Per unit	100 gallons	0.12 lb.
Restaurants			
Ordinary rest. (not 24 hours)	Per seat	35 gallons	0.20 lb.
24 hour rest.	Per seat	50 gallons	0.28 lb.
24 hour rest. on interstate	Per seat	70 gallons	0.40 lb.
Tavern	Per seat	20 gallons	0.12 lb.
Curb service	Per car space	50 gallons	0.28 lb.
Trailer park	2–1/2 Persons per trailer	50 gallons	0.20 lb.
Country clubs	Per member	50 gallons	0.20 lb.
Shopping center	Without food service or laundry	0.1 gal/sq. ft. of floor space based on flow	200 ppm

*When garbage grinders are used, the BOD_5 loading shall be increased to 0.07 lb. BOD_5 / person.

These pollutants are all biological and as such can be readily biodegraded. Any extraneous nonindustrial pollutants other than those listed above can be physical or chemical in nature, and ideally should be prevented from entering a municipal system with a Pre-treatment Ordinance, or removed from the municipal wastewater using some method of pre-treatment.

2.3 Agricultural Sources of Water Pollution

Normally, agricultural water pollutants are transported to an aboveground or underground receiving stream by periodic stormwater. Agricultural wastewater can be from a nutrient, fertilizer, pesticide or herbicide source.

Agricultural activities can also allow the runoff of soil into receiving streams. In such cases, pollutants can be any organic or inorganic constituent of the soil.

2.4 Natural Sources of Water Pollution

Areas unaffected by human activity can still pollute receiving steams due to stormwater runoff, which can be classified into animal, vegetable and soil sources. Again, animal and vegetable water pollution sources should be readily biodegradable. Soil sources will consist of any organic and inorganic material in the soil.

2.5 Stormwater Sources of Water Pollution

Stormwater has been mentioned above under agricultural and natural sources of water pollution, but will also transport industrial and municipal water pollutants to a receiving stream or underground water supply.

2.6 Landfill Sources of Water Pollution

Public, private, and industrial landfills can be a source of stormwater pollution because of runoff from the surface and underground leachate. Landfill regulations require daily cover, but during the day, rainfall can cause pollution from surface runoff. When stormwater leaches through the surface cap and downward through the landfill, the horizontally or vertically migrating discharge from below the landfill is known as leachate and can pollute surface or underground water. Because of the bacteria present in the dirt and in landfill material, there will always be aerobic and anaerobic biological activity occurring in a landfill.

Landfills are normally required to provide leachate, and in some cases, runoff collection and treatment or disposal to prevent contamination of the environment.

3. Types of pollutant

Contaminants behave in different ways when added to water. Non-conservative materials including most organics, some inorganics and many microorganisms are degraded by natural self-purification processes so that their concentrations reduce with time. The rate of decay of these materials is a function of the particular pollutant, the receiving water quality, temperature and other environmental factors. Many inorganic substances are not affected by natural processes so that these conservative pollutants can only have their concentrations reduced by dilution. Conservative pollutants are often unaffected by normal water and wastewater treatment processes so that their presence in a particular water source may limit its use. As well as the classification into conservative or non-conservative characteristics the following constituents of pollutants are of importance.

1. Toxic compounds which result in the inhibition or destruction of biological activity in the water. Most of these materials originate from industrial discharges and would include heavy metals from metal finishing and plating operations, moth repellents from textile manufacture, herbicides and pesticides, etc. Some species of algae can release potent toxins and cases have been recorded where cattle have died after drinking water containing algal toxins.

2. Anything which may affect the oxygen balance of the water, including

(a) substances which consume oxygen: these may be organic materials which are biochemically oxidized or inorganic reducing agents;

(b) substances which hinder oxygen transfer across the air-water interface. Oils and detergents can form protective films at the interface which reduce the rate of oxygen transfer and may thus amplify the effects of oxygen-consuming substances;

(c) thermal pollution, which can upset the oxygen balance because the saturation dissolved oxygen concentration reduces with increasing temperature.

3. Inert suspended or dissolved solids in high concentrations can cause problems, e.g. china-clay washings can blanket the bed of a stream preventing the growth of fish food and removing fish from the vicinity as effectively as a direct poison. The discharge of saline mine drainage water may render a river unsuitable for water-supply purposes.

It is obviously important to be able to assess the effect of a particular polluting discharge on a receiving water in quantitative terms and a first step is to utilize a mass balance approach. Figure 2.2 shows a river receiving a pollutant discharge

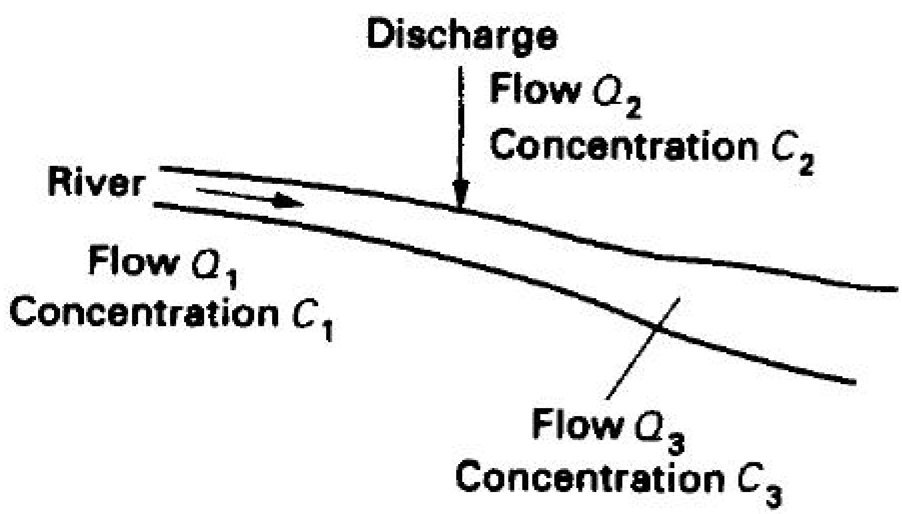


Figure 2.2 The mass balance concept in river pollution.

and it is possible to determine the downstream concentration of the pollutant, assuming instantaneous mixing with conservation of mass

$$Q_1 \times C_1 + Q_2 \times C_2 = Q_3 \times C_3$$

Since the sum of the flows arriving and leaving the discharge point must be equal (i.e. $Q_3 = Q_1 + Q_2$) the downstream concentration C3 is easily calculated.
Depending upon the nature of the pollutant it will then be possible to calculate the concentrations at points further downstream from the discharge, knowing the velocity of flow and hence the time of travel between the points.

Worked example on mass balance

A stream with flow of 0.1 m^3/s and chloride concentration of 52 mg/1 receives a discharge of mine drainage water with a flow of 0.025 m^3/s and a chloride concentration of 1250 mg/1.
By mass balance, 0.1 x 52 + 0.025 x 1250 = (0.1 + 0.025) X downstream concentration; hence, downstream concentration = (5.2 + 31.25)/0.125 =291.6 mg/l.

4. Classification of Water Pollutants

The various types of water pollutants can be classified in to following major categories:
1) Organic pollutants, 2) Pathogens, 3) Nutrients and agriculture runoff, 4) Suspended solids and sediments, 5) Inorganic pollutants (salts and metals), 6) Thermal Pollution 7) Radioactive pollutants.

4.1 Organic Pollutants

Organic pollutants can be further divided in to following categories:

a) Oxygen Demanding wastes: The wastewaters such as, domestic and municipal sewage, wastewater from food processing industries, canning industries, slaughter houses, paper and pulp mills, tanneries, breweries, distilleries, etc. have considerable concentration of biodegradable organic compounds either in suspended, colloidal or dissolved form. These wastes undergo degradation and decomposition by bacterial activity. The dissolved oxygen available in the water body will be consumed for aerobic oxidation of organic matter present in the wastewater. Hence, depletion of the DO will be a serious problem adversely affecting aquatic life, if the DO falls below 4.0 mg/L. This decrease of DO is an index of pollution.

b) Synthetic Organic Compounds

- Synthetic organic compounds are also likely to enter the ecosystem through various manmade activities such as production of these compounds, spillage during transportation, and their uses in different applications.
- These include synthetic pesticides, synthetic detergents, food additives, pharmaceuticals, insecticides, paints, synthetic fibers, plastics, solvents and volatile organic compounds (VOCs).

- Most of these compounds are toxic and bio-refractory organics i.e., they are resistant to microbial degradation.
- Even concentration of some of these in traces may make water unfit for different uses.
- The detergents can form foams and volatile substances may cause explosion in sewers.
- Polychlorinated biphenyls (PCBs) are used in the industries since 1930s which are complex mixtures of chloro-biphenyls. Being a fat soluble they move readily through the environment and within the tissues or cells. Once introduced into environment, these compounds are exceedingly persistent and their stability to chemical reagents is also high.

c) Oil

- Oil is a natural product which results from the plant remains fossilized over millions of years, under marine conditions. It is a complex mixture of hydrocarbons and degradable under bacterial action, the biodegradation rate is different for different oils, tars being one of the slowest. Oil enters in to water through oil spills, leak from oil pipes, and wastewater from production and refineries.
- Being lighter than water it spreads over the surface of water, separating the contact of water with air, hence resulting in reduction of DO. This pollutant is also responsible for endangering water birds and coastal plants due to coating of oils and adversely affecting the normal activities.
- It also results in reduction of light transmission through surface waters, thereby reducing the photosynthetic activity of the aquatic plants.
- Oil includes polycyclic aromatic hydrocarbons (PAH), some of which are known to be carcinogenic.

4.2 Pathogens

The pathogenic microorganisms enter in to water body through sewage discharge as a major source or through the wastewater from industries like slaughterhouses. Viruses and bacteria can cause water borne diseases, such as cholera, typhoid, dysentery, polio and infectious hepatitis in human.

4.3 Nutrients

The agriculture run-off, wastewater from fertilizer industry and sewage contains substantial concentration of nutrients like nitrogen and phosphorous. These waters supply nutrients to the plants and may stimulate the growth of algae and other aquatic weeds in receiving waters.

- Thus, the value of the water body is degraded.
- In long run, water body reduces DO, leads to eutrophication and ends up as a dead pool of water.
- People swimming in eutrophic waters containing blue-green algae can have skin and eye irritation, gastroenteritis and vomiting.
- High nitrogen levels in the water supply, causes a potential risk, especially to infants under six months. This is when the methaemoglobin results in a decrease in the oxygen carrying capacity of the blood (blue baby disease) as nitrate ions in the blood readily oxidize ferrous ions in the hemoglobin.

4.4 Suspended Solids and Sediments

These comprise of silt, sand and minerals eroded from land. These appear in the water through the surface runoff during rainy season and through municipal sewers. This can lead to the siltation, reduces storage capacities of reservoirs.

- Presence of suspended solids can block the sunlight penetration in the water, which is required for the photosynthesis by bottom vegetation.
- Deposition of the solids in the quiescent stretches of the stream or ocean bottom can impair the normal aquatic life and affect the diversity of the aquatic ecosystem.
- If the deposited solids are organic in nature, they will undergo decomposition leading to development of anaerobic conditions.
- Finer suspended solids such as silt and coal dust may injure the gills of fishes and cause asphyxiation.

4.5 Inorganic Pollutants

- ✓ Apart from the organic matter discharged in the water body through sewage and industrial wastes, high concentration of heavy metals and other inorganic pollutants contaminate the water. These compounds are non-biodegradable and persist in the environment. These pollutants include mineral acids, inorganic salts, trace elements, metals, metals compounds, complexes of metals with organic compounds, cyanides, sulphates, etc.
- ✓ The accumulation of heavy metals may have adverse effect on aquatic flora and fauna and may constitute a public health problem where contaminated organisms are used for food.
- ✓ Algal growth due to nitrogen and phosphorous compounds can be observed.
- ✓ Metals in high concentration can be toxic to biota e.g. Hg, Cu, Cd, Pb, As, and Se. Copper greater than 0.1 mg/L is toxic to microbes.

4.6 Thermal Pollution

Considerable thermal pollution results due to discharge of hot water from thermal power plants, nuclear power plants, and industries where water is used as coolant.

- As a result of hot water discharge, the temperature of water body increases, which reduces the DO content of the water adversely, affecting the aquatic life.
- This alters the spectrum of organisms, which can adopt to live at that temperature and DO level.
- When organic matter is also present, the bacterial action increases due to rise in temperature; hence, resulting in rapid decrease of DO.
- The discharge of hot water leads to the thermal stratification in the water body, where hot water will remain on the top.

4.7 Radioactive Pollutants

Radioactive materials originate from the following:

- Mining and processing of ores,
- Use in research, agriculture, medical and industrial activities, such as I^{131}, P^{32}, Co^{60}, Ca^{45}, S^{35}, C^{14}, etc.
- Radioactive discharge from nuclear power plants and nuclear reactors, e.g., Sr90, Cesium Cs^{137}, Plutonium Pu^{248}, Uranium^{-238}, Uranium^{-235},
- Uses and testing of nuclear weapons.

➢ These isotopes are toxic to the life forms; they accumulate in the bones, teeth and can cause serious disorders.

➢ The safe concentration for lifetime consumption is 1×10^{-7} microcuries per ml.

CHAPTER THREE

WATER TREATMENT

Introduction

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 finished water quality standards, state design criteria, constituents treated, topography and geology, hydraulic requirement, 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
8) Storage water with high pump station

Figure 3.1 shows the typical water treatment processes train. Also, table 3.1 indicates the capability of the various treatment techniques for removing contaminants.

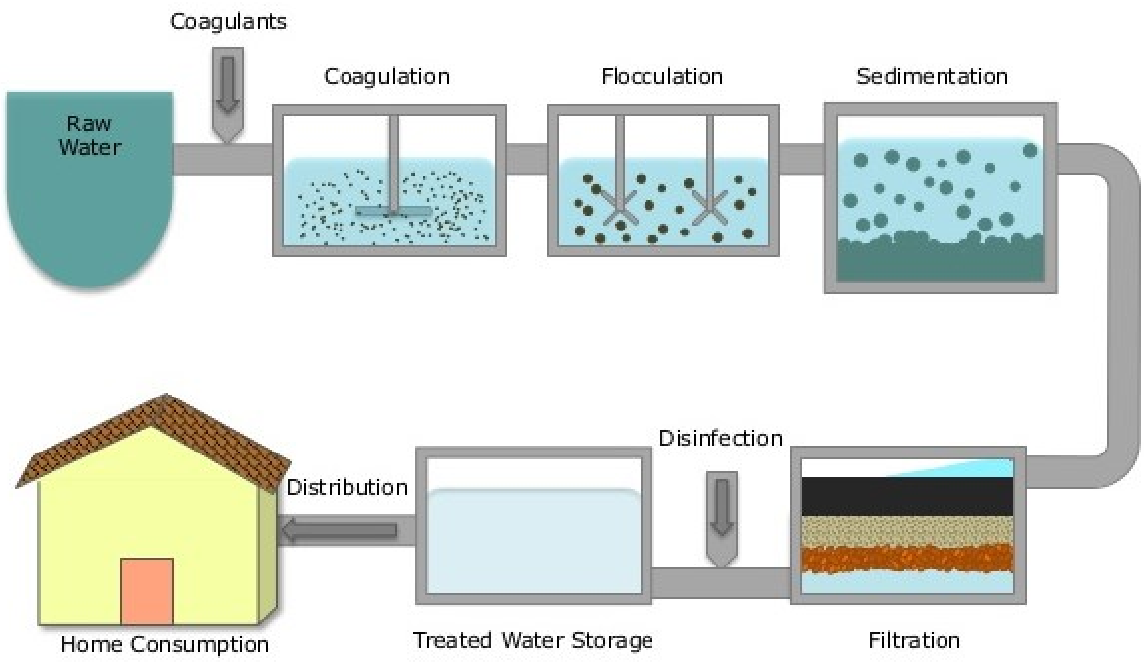


Figure 3.1 The typical water treatment processes

	Bacteria	Cysts	Viruses	Algae	Coarse Particles	Turbidity	Colour	Aluminium	Ammonia	Arsenic	Iron & Manganese	Nitrate	Pesticides	Solvents	Taste & Odour
Coagulation/flocculation[1]	+	+	+	+	++	++	++	++		+	++				
Sedimentation					++	+		+			+				
Gravel filter/screen				+	++	+		+			+				
Rapid sand filtration	+	+	+	+	++	+		+			+				
Slow sand filtration	++	++	++	++	++	++		+			+				
Chlorination	++		++	+			+		++						
Ozonation	++	+	++	++			+						++		++
UV	++	+	++	+											
Activated carbon							+						+	+	++
Activated alumina										++					
Ceramic filter	++	++		++	++	++									
Ion exchange									+	+		++			
Membranes	++	++	++	++	++	++	++	++		+	++	++	++		++

Table 3.1 the capability of the various treatment techniques for removing contaminants

1. Water Intakes

1.1 Definition of Intakes

Intakes are structures constructed in or adjacent to lakes, reservoirs, or rivers for the purpose of withdrawing water. In general, they consist of an opening with a screen or strainer through which the water enters, and a conduit (an open channel or a pipe line) to conduct the water a low-lift pumping station. The water is pumped from the low-lift pumping station to the water treatment plant. 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.

1.2 General factors governing location of intakes

1. As far as possible, 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 noy get flooded. Moreover, the flood waters should not be concentrated in the vicinity of the intake.

1.3 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 upthrust 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.
4) Selection of suitable screen to provide around the intake pipe not to permit entry of large and small objects such as logs, stones, aquatic lives, and vegetation.

1.4 Types of Intakes

Intake structures may be classified into two categories; exposed intakes and submerged intakes. Many varieties of these types have been used. The selection of the intake type is highly dependent on water source type. Intake structures may be classified into two categories; *exposed intakes* and *submerged intakes*.

Exposed intakes:

- Tower in lake or impounding reservoir (applicable to large systems and expensive)
- Floating or movable (good access for operation and maintenance)
- Siphon well (applicable to small systems, flexible, and easy to expand)

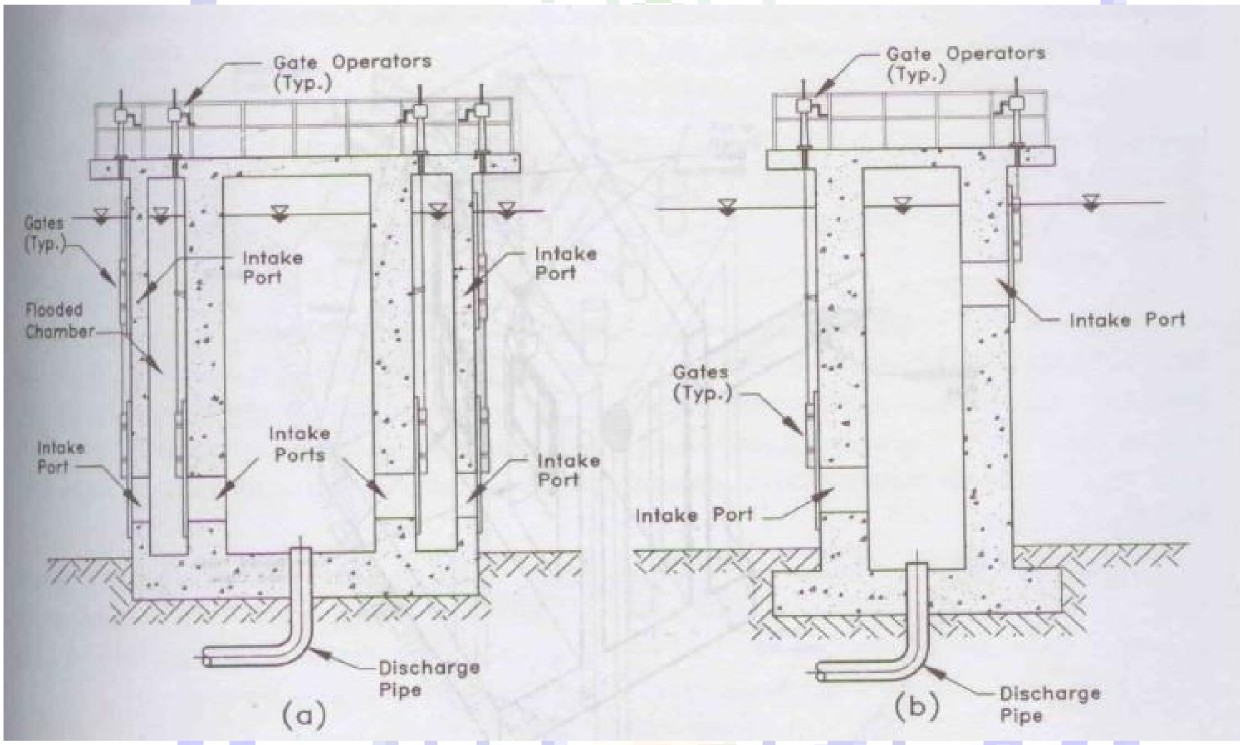


Figure 3.2 Exposed or Tower intakes

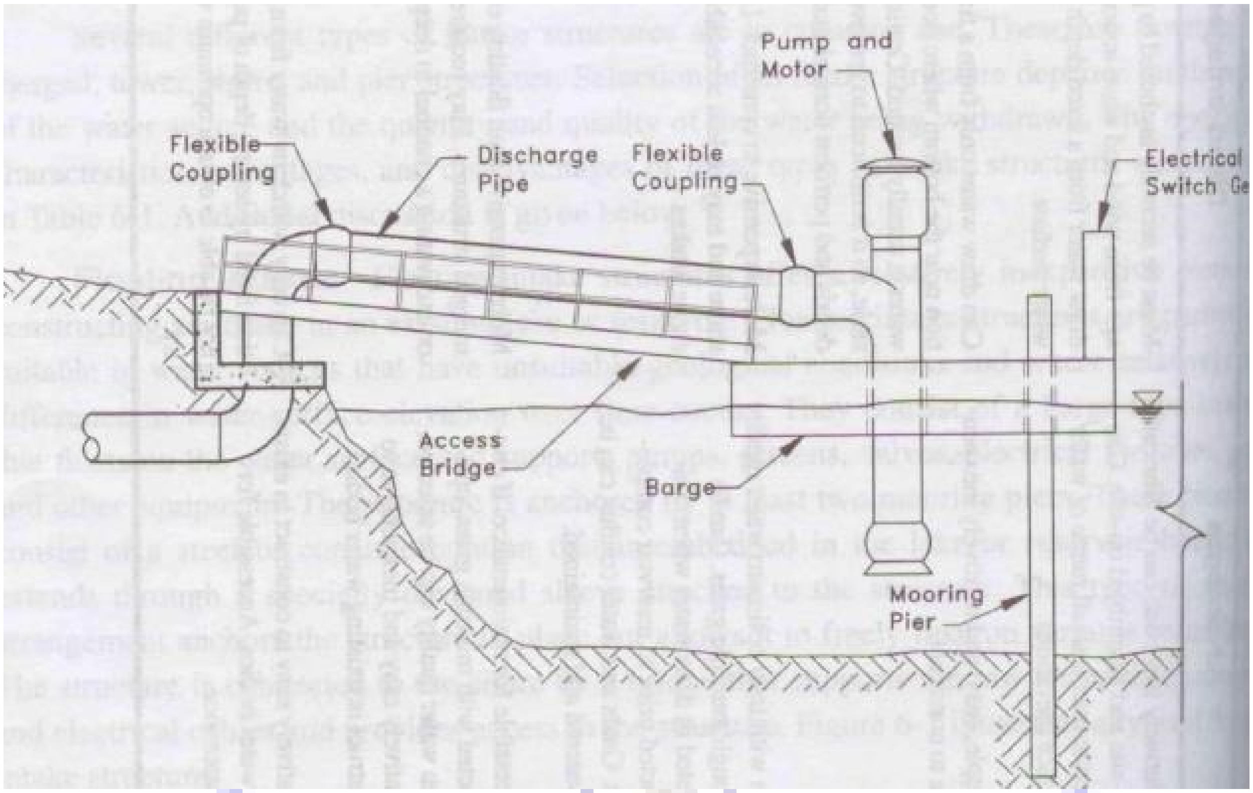


Figure 3.3 Floating intakes

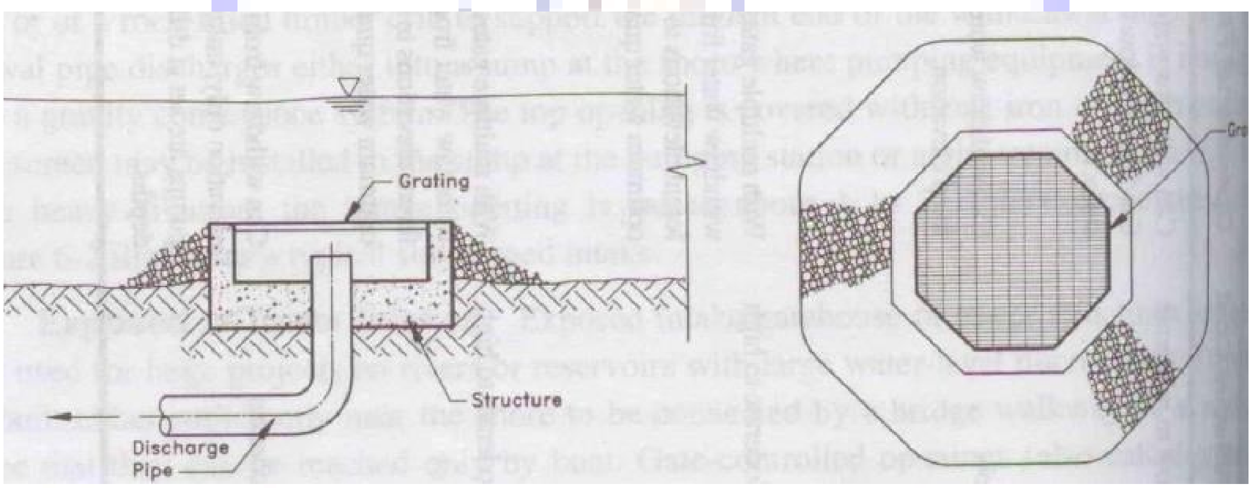


Figure 3.4 Submerged intakes

River intake:

They must be sufficient stable, and the water deep enough to allow a submergence of at least (1m) at all times with a clear opening beneath the pipe so that any tendency to form a bar is overcome. River intakes are especially likely to need screens to exclude large floating matter which might injure pumps.

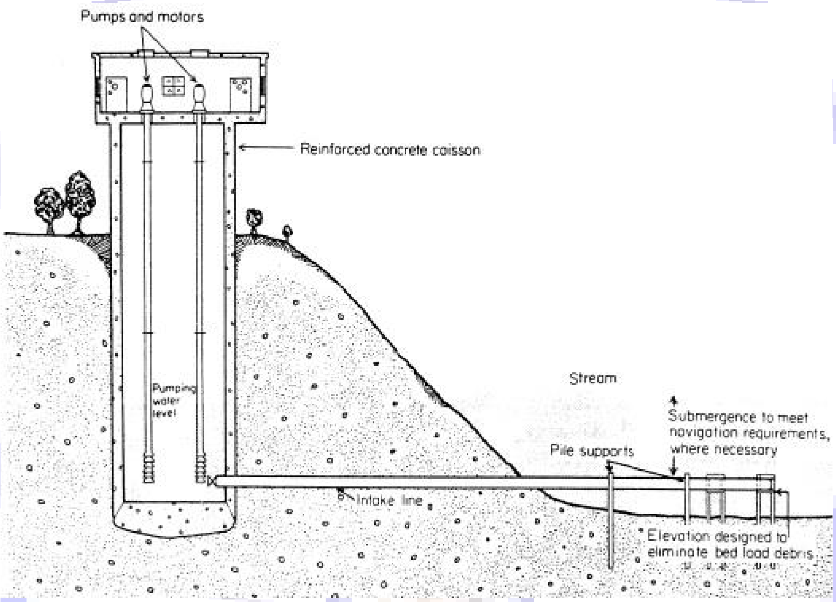


Figure 3.4 Screened pipe intake.

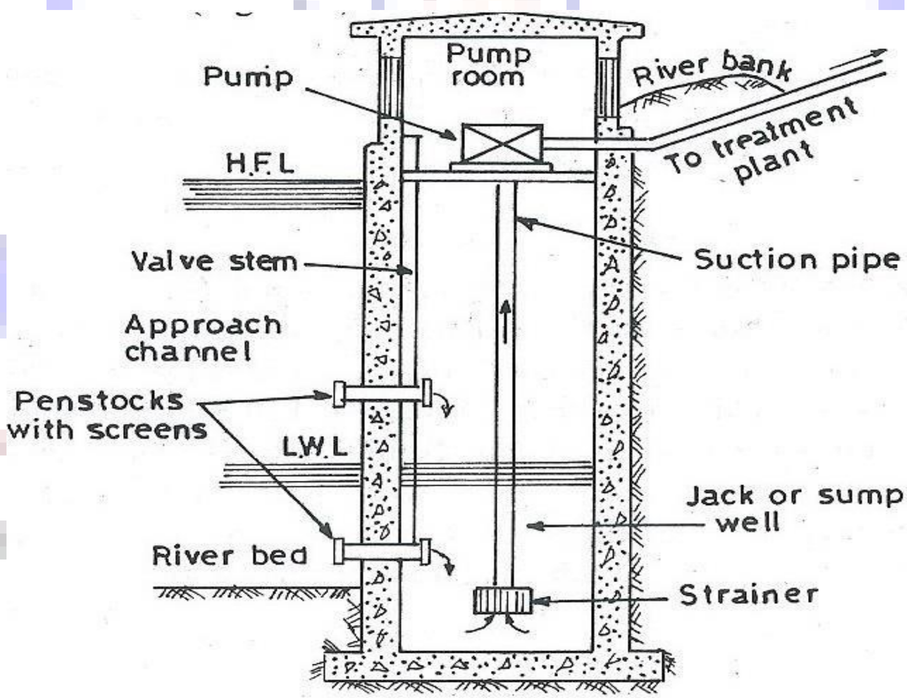


Figure 3.5 River intake.

§§ Both exposed and submerged inlet structures have been used in rivers. In large rivers that are controlled by locks and dams, the variation in flow and consequent variation in water surface elevation are of less concern than in unregulated waterways. For most water supplies, unlike lakes and reservoirs, special consideration must be given to the impact of floods and droughts on river intakes. In the first instance, structural stability, availability of power, and access must be considered in the design. In the second instance, provision must be made for alternative access to water when drought conditions lower the water level below the lowest intake port. While a reservoir or lake will have suspended matter during high wind events, it will seldom have the quantity or quality of the grit produced during flood events on rivers. The river intake structure must be designed to protect the pumps and valves in the transmission system from wear by grit.

§§ ***Intake Design***

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:

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

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

2.Screening

Definition:

Screening is a unit operation that removes floating and large suspended matter from water. Screens may be classified as coarse, fine, microstrainer, depending on the size of material removed. Screens may be located at the intake structure, raw water pump station, or water treatment plant. The screen is a device with openings uniform in size. It is placed across the flow to retain floating particles. It serves as a protective unit.

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.

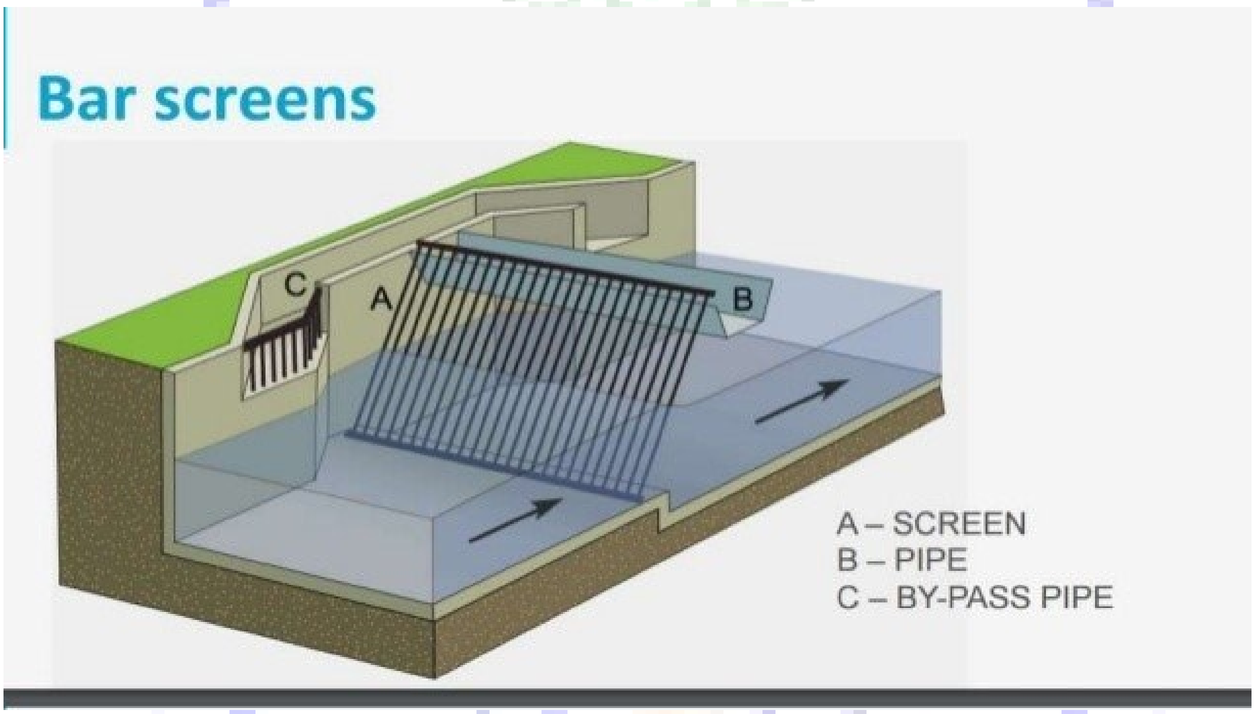


Figure 3.6 Screening

2.1 Types of Screens

1) According to its shape

1- racks 2- mesh

2) According to its size:

a) Course Screens, openings > 25 mm:

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 50 to 80 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° and 60°. The velocity through the coarse screen is generally less than 8 cm/s. Screens should be installed outside (on the water side) of any sluice gate or stop log slot, to prevent debris from interfering with their operation.

b) Fine Screens, openings < 6 mm

Fine screen 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. These screens consist of heavy wire mesh with 0.5 cm square openings or circular passive screens with similar opening widths. The screen area efficiency factor (0.5-0.6) and the typical velocity through the effective area is in the range of (0.4-0.8) m/s.

3) According to its workability, manual and mechanical.

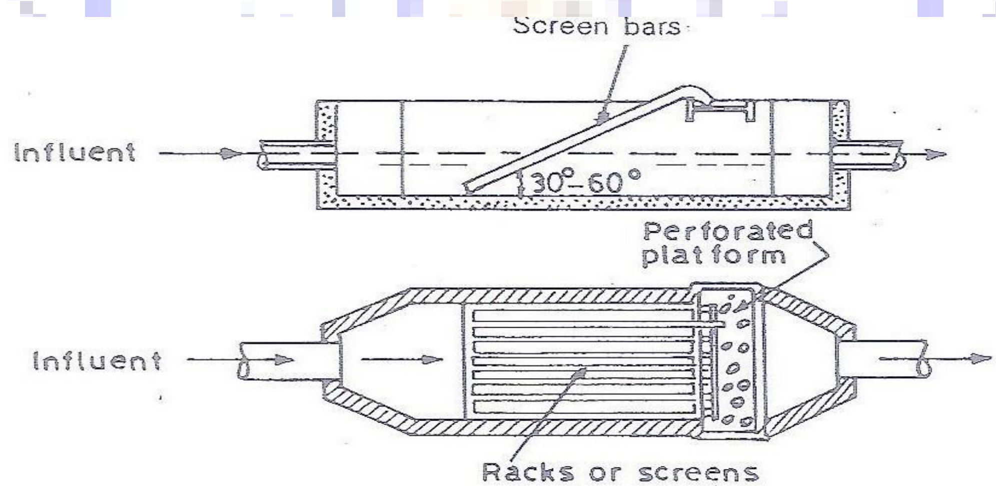
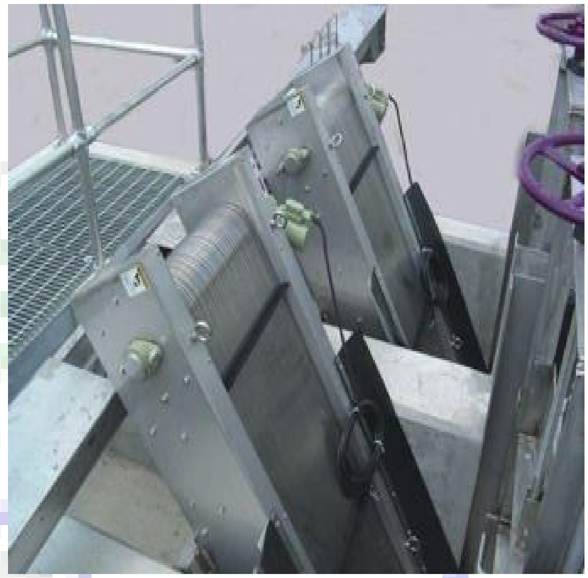


Figure 3.7 Screens Configuration

a) Coarse screen



b) Fine screen

c) Manual cleaning screen

d) Automatically cleaning screen

Figure 3.8 Types of screen

2.2 Requirements and Specifications for Design of Bar Screen

1- The velocity of flow ahead of and through a screen varies materially and affects its operation. Lower the velocity through the screen, the greater is the amount of screening that would be removed. However, at lower velocity greater number of solids would be deposited at the bottom of the screen channel.

2- Approach velocity of wastewater in the screening channel shall not fall below a self-cleansing velocity of 0.42 m/sec or rise to a magnitude at which screenings will be dislodged from the bars.

- The suggested approach velocity is 0.6 to 0.75 m/sec for the grit bearing waters. Accordingly, the bed slope of the channel should be adjusted to develop this velocity.
- The suggested maximum velocity through the screen is 0.3 m/sec at average flow for hand cleaned bar screens and 0.75 m/sec at the normal maximum flow for mechanically cleaned bar screen (*Rao and Dutta, 2007*). Velocity of 0.6 to 1.2 m/sec through the screen opening for the peak flow gives satisfactory result.

3. Head losses due to installation of screens must be controlled so that back water will not cause the entrant sewer to operate under pressure.

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:

$$h_L = \frac{v_b^2 - v_a^2}{2g} \times \frac{1}{0.7} \quad or \quad h_L = 0.0729\,(v_b^2 - v_a^2)$$

where,

h_L = head loss in m,

v_b = velocity through bar opening in m/s,

v_a = 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:

$$h_L = \beta \left(\frac{W}{b}\right)^{\frac{4}{3}} \frac{v^2}{2g} \sin\theta$$

where,

h_L= head losses, m

W = Width of bars facing the flow, m

β = Bar shape factor

= 2.42 for sharp edge rectangular bars

= 1.83 for rectangular bars with semicircular upstream

= 1.79 for circular bars

= 1.67 for rectangular bars with both u/s and d/s faces as semicircular.

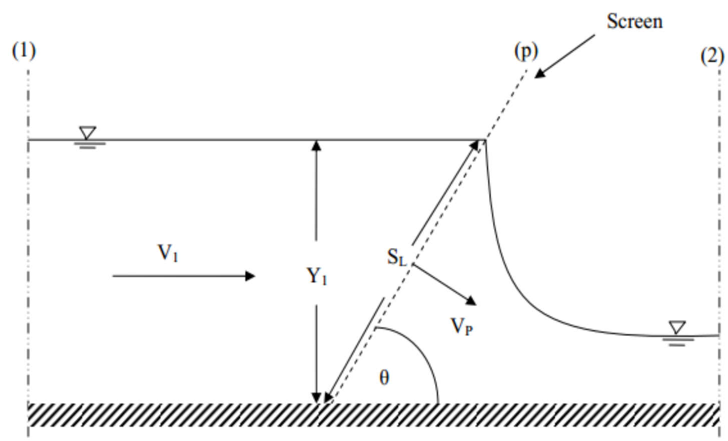
b = Clear spacing between the bars, m

v = geometric mean of the approach velocity, m/sec

θ = Angle of inclination of the bars with horizontal (30-60°).

§§ Usually accepted practice is to provide loss of head of 0.15 m but the maximum loss of head with the clogged hand cleaned screen should not exceed 0.3 m. For mechanically cleaned screen, the head loss is specified by the manufacturer, and it can be between 150 to 600 mm.

4. The submerged area of the surface of the screen, including bars and opening should be about 200% of the cross-sectional area of the incoming sewer for separate system, and 300% for the combined system.



§§ 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.

§§ As a design criterion the following might be followed also,

1) Flowing velocity through the openings = 0.3 – 1.0 m/sec

2) Head loss (h_L) = 150 -300 mm

Example

Design a coarse screen (rack) for a flow of 0.15 m^3/sec. The screen is placed in a canal 0.6 m in width and 0.4 m in height. Use rectangular bars 10 X 30 mm in cross section, assume size of the openings = 25 mm? assume size of the openings c/c = 35 mm

Solution

Assume n = number of bars

Width of the channel = n × Size of the bar + (n + 1) × Size of the opening

600 mm = n × 10 mm + (n + 1) × 25 ⇛ 575 = 35n ⇛ n = 16.4 ≈ 16

Number of spacing = n+1= 16 +1 = 17

Check the velocity:

Area of Spacing = Area of Opening = Height X width of opening

Width = 600 − 16 × 10 = 440 mm = 0.44m

Velocity through the rack$=\frac{0.15\ m^3/_{sec}}{0.4\times0.44}$ =0.852 m/sec within the acceptable velocity O. K

Assume the velocity through the channel $v_c = \frac{Q}{A_{channel}} = \frac{01.5}{0.6\times0.4}$ = 0.625 m/sec

Head loss through the openings $h_L = \frac{v_b^2 - v_a^2}{2g} \times \frac{1}{0.7} = \frac{0.852^2 - 0.625^2}{2g} \times \frac{1}{0.7}$ = 0.0244m

Or using 10 mm bars 35 mm c/c

Number of spaces $= \frac{\text{width}}{\text{spaces c/c}} = \frac{0.6}{0.35} \approx 17$

Number of bars = Number of spaces − 1 = 17 − 1 = 16

Size of openings = 35 − (5 + 5) = 25 *mm*

3. Pumps and Pumping Stations

A pump is a device which converts mechanical energy into hydraulic energy. It lifts water from a lower to a higher level and delivers it at high pressure. Pumps are employed in water supply projects at various stages for following purposes:

1. To lift raw water from wells.
2. To deliver treated water to the consumer at desired pressure.
3. To supply pressured water for fire hydrants.
4. To boost up pressure in water mains.
5. To fill elevated overhead water tanks.
6. To back-wash filters.
7. To pump chemical solutions, needed for water treatment.

3.1Pumps Classifications

Based on principle of operation, pumps may be classified as follows:

1. Displacement pumps (reciprocating, rotary)
2. Velocity pumps (centrifugal, turbine and jet pumps)
3. Buoyancy pumps (air lift pumps)
4. Impulse pumps (hydraulic rams)

3.2 Work and efficiency of pumps

The work done by a pump is equal to the product of the mass flow and the total head against which the flow is moved. Head, hydraulic energy, kinetic or potential energy is defining as equivalent to the potential energy of a column of water of specified height.

Total dynamic head (TDH) of a pump is the sum of the static suction head (SSH), the static discharge head (SDH), the total friction head (T.H.L), and velocity head (DH)

$$TSH = SSH + SDH$$

Pump capacity (Q) is a term used to describe the maximum flow rate through a pump at its designed conditions. It is a measurement usually given in gallons per minute (gpm) or cubic meters per hour (m^3/h).

Head (H) is expressed in units of height such as meters or feet. The static head of a pump is the maximum height (pressure) it can deliver. The capability of the pump at a certain RPM can be read from its Q-H curve (flow vs. height).

$$TotalHead(TH) = Static\ head(SH) + Dynamic\ head\ (DH) + Total\ head\ loss\ (THL) + Pressure\ required\ (Pr)$$

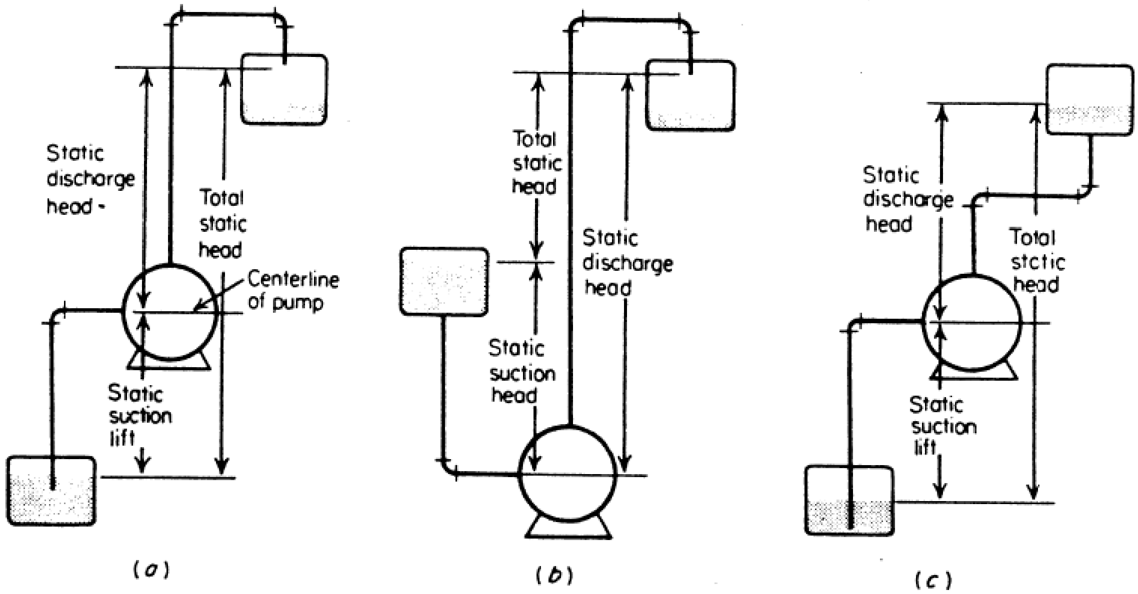


Figure 3.9 Head terms used in pumps

Pump efficiency (η) is defined as the ratio of water horsepower output from the pump to the shaft horsepower input for the pump. The efficiency of a particular pumps estimated by determining two values. These values are pump flow rate and total head.

$$\eta = \frac{P_{out}}{P_{in}}$$

Pump water power (Pw), In the hydraulic field, the load of a pump is expressed in theory in height of water. It is the hydraulic power communicated to the liquid of its passage through the pump (Watt or kW).

$$P_W = \gamma \times Q \times H$$

P_w: water power (Kw), : water weight density (9.81 kN/m^3), Q: pump capacity (m^3/sec), and H: total head (m).

Pump Power (P) is a metric in fluid dynamics that quantifies the energy-efficiency of pump systems. It is a measure of the electric power that is needed to operate a pump (or collection of pumps), relative to the volume flow rate. ... It is commonly used when measuring the energy efficiency of buildings.

$$P = \frac{P_w}{\eta} = \frac{\gamma \times Q \times H}{\eta}$$

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

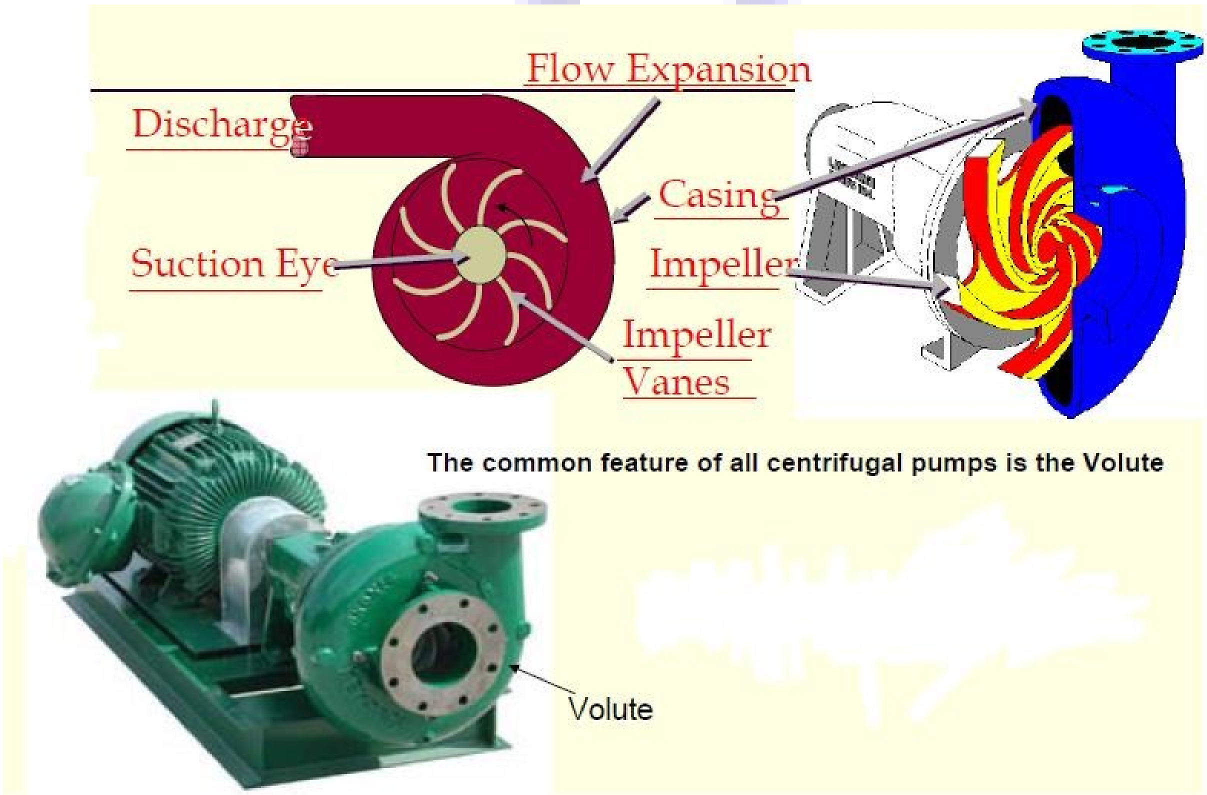


Figure 3.10 Centrifugal Pump

§§ ***Pump Characteristic Curves***. The performance of a centrifugal pump can be shown graphically on a characteristic curve. A typical characteristic curve shows the total dynamic head, brake horsepower, efficiency, and net positive Suction head all plotted over the capacity range of the pump.

3.3 Characteristic curves for centrifugal pump

3.3.1 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. (3.11) where the discharge is controlled by a valve.

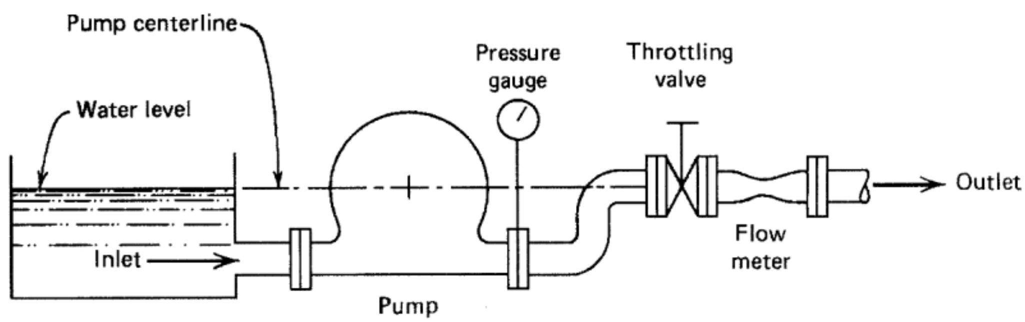
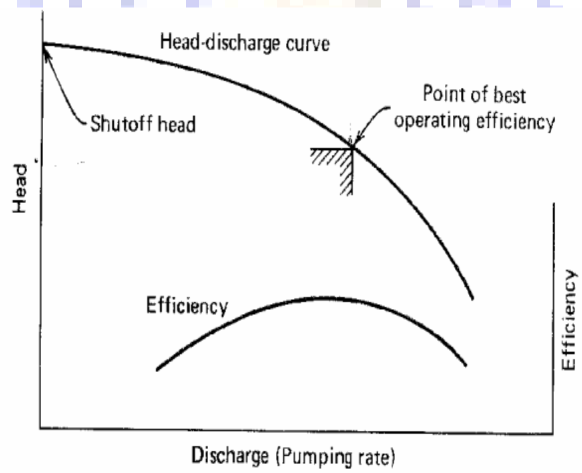


Figure 3.11 Schematic of a pump head discharge curve

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 drawn in Fig. (3.12). 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.



Effects of speed and diameter of impeller on centrifugal pump

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

$$\frac{Q_1}{\omega_1} = \frac{Q_2}{\omega_2} \quad , \quad \frac{H_1}{H_2} = \frac{\omega_1^2}{\omega_2^2} \quad , \quad \frac{P_1}{P_2} = \frac{\omega_1^3}{\omega_2^3}$$

Where

ω_1, ω_2: rotational speed of the pump conditions, rpm

Q_1, Q_2: discharge corresponding ω_1, ω_2

H_1, H_2: total dynamic head corresponding ω_1, ω_2

P: Pump Power or Water Power'

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

$$\frac{Q_1}{Q_2} = \frac{D_1}{D_2} \quad , \quad \frac{H_1}{H_2} = \frac{D_1^2}{D_2^2} \quad , \quad \frac{P_1}{P_2} = \frac{D_1^3}{D_2^3}$$

<u>*Example:*</u>

Determine the water power, pump power and motor load for a pump system designed to deliver 1.89 m^3/min (500gpm) against a total system head of 50m (164ft). Assume the efficiency of both pump and motor is 80%.

<u>*Solution:*</u>

$P_w = \gamma \times Q \times H = 9.81 * 1.89 * 50 = 927\text{kW}$

$$P = \frac{P_w}{\eta} = \frac{927}{0.8} = 1159kW$$

<u>Note: -</u> *The efficiency of pumps Ranges from as little as 40 prevent to as 90 percent depending upon the pump design, the fluid Pumped and nicety with which pump and application are matched.*

Example:

A Centrifugal pump operates at a speed of 1150 rpm and discharge of 2.3 m^3/min against a head of 120kpa. The power required is 5.5 kw compute:

1-The efficiency of the pump?

2-The discharge head and power if the pump speed was changed to 1750 rpm?

Solution:

1) $P_w = \gamma \times Q \times H = 9.81 * 2.3 * \frac{120}{9.81} = 276\text{kW}$

$$\eta = \frac{P_w}{P} = \frac{276}{5.5} \approx 50\ \%$$

2) $Q \propto \omega \qquad \therefore Q_1 \propto \omega_1 \quad , \quad Q_2 \propto \omega_2$

$$\frac{Q_1}{\omega_1} = \frac{Q_2}{\omega_2} \qquad \therefore \frac{2.3}{Q_2} = \frac{1150}{1750} \qquad \therefore Q_2 = 3.5 \text{ m}^3\text{/min}$$

$$H \propto \omega^2 \quad \frac{H_1}{H_2} = \frac{\omega_1^2}{\omega_2^2} \qquad \frac{12.23}{H_2} = \left(\frac{1150}{1750}\right)^2 \rightarrow\rightarrow H_2 = 28.31m$$

$$H \propto \omega^3 \quad \frac{P_1}{P_2} = \frac{\omega_1^3}{\omega_2^3} \qquad \frac{5.5}{P_2} = \left(\frac{1150}{1750}\right)^3 \rightarrow\rightarrow P_2 = 19.4kW$$

3.3.2 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. NPSHav 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:

$$NPSH_{av} = h_{abs} - h_s - h_l - h_{vp}$$

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:

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

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

Table (3.1) altitude-barometer pressure & temperature-vapor pressure

Barometric pressure vs. altitude

Altitude		Pressure	
m	ft	kPa	ft H_2O
0	0	101	33.9
305	1,000	98	32.8
457	1,500	96	32.1
610	2,000	94	31.5
1220	4,000	88	29.2
1830	6,000	81	27.2
2439	8,000	75	25.2
3049	10,000	70	23.4
4573	15,000	57	19.2

Vapor pressure of water vs. temperature

Temperature		Pressure	
°C	°F	kPa	ft H_2O
0	32	0.61	0.204
4.4	40	0.84	0.281
10.0	50	1.23	0.411
15.6	60	1.76	0.591
21.1	70	2.50	0.838
26.7	80	3.50	1.17
32.2	90	4.81	1.61
37.8	100	6.54	2.19
43.3	110	8.81	2.95
48.9	120	11.70	3.91
54.4	130	15.30	5.13
60.0	140	19.90	6.67

Example:

what is the maximum permissible difference in elevation between the water surface in intake structure and the pump under the following condition: -

altitude = 1000m

Max. water temp. = 25 º C

Intake pipe diameter = 150mm

Flow = $2m^3/min$

Minor losses $v^2/2g$, (k=1)

NPSH (required) = 40 kpa

Solution:

From altitude-barometer pressure table, for 1000m altitude, the barometer pressure is 90.16kPa.

From temperature-vapor pressure table, for 25 º C , the vapor pressure is 3.2kPa.

For losses, calculate the velocity $v = \frac{Q}{A} = \frac{2}{60\times(0.15)^2\times\pi} = 1.88$m/sec

Then losses $= \frac{v^2}{2g} = 0.18$m ------ 0.18* 9.81 = 1.77kPa

$NPSHav = habs - hs - hl - hvp$

$40 = (90.16 - 3.5) - hs - 1.77 - 3.2 \qquad \rightarrow\rightarrow \quad hs = 41.69kPa = 4.25m$

3.3.3 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. Figure (3.12 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. Figure (3.12 b)

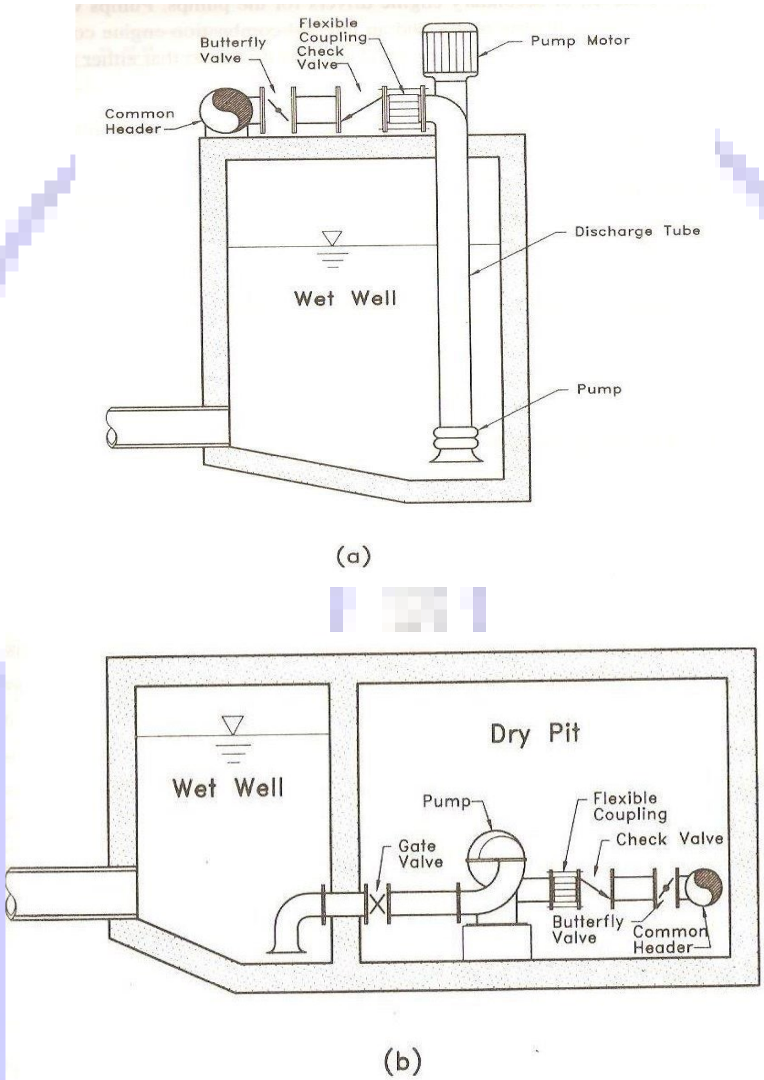


Fig (3.12) Types of pumping stations.

Design consideration for pump stations:

1) Location: the location of pump stations has an important bearing on the pressure maintained in the distribution system.

2) Architecture: the building should be adequate in size and have space for placing additional units as needed. It should be fireproof, pleasing in appearance, and have well-kept grounds. Its architecture can and should be in harmony with surroundings.

3) Capacity and operation: the pumps must have sufficient capacity to care for peak load, and the designer and operator must furnish and the pumping plant may operate 8-12 hours.

4. Conventional Water Treatment

Water treatment is said to be conventional if the water source is river which is the most preferable water source. The flowsheet of a conventional treatment plant is shown in Figure 4.1.

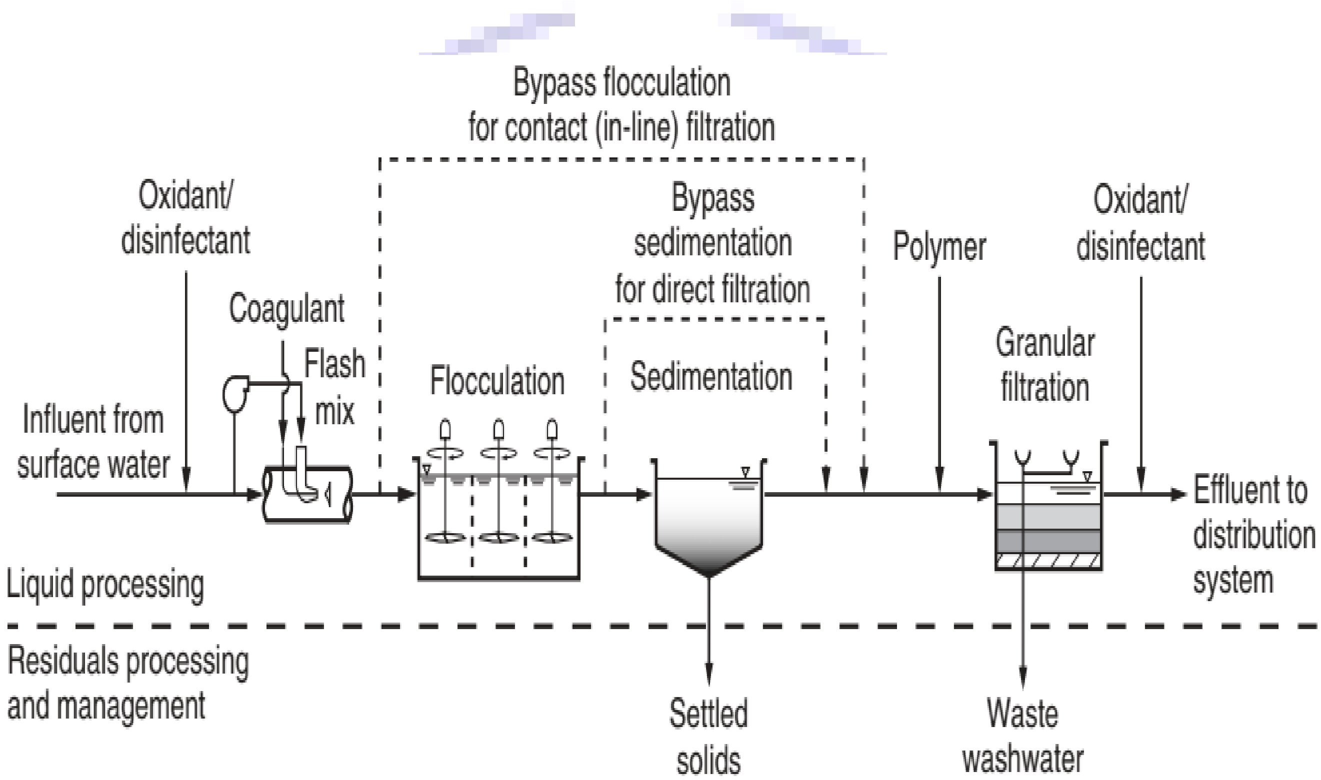


Fig.4.1 Typical water treatment process flow diagram employing coagulation (chemical mixing) with conventional treatment, direct filtration, or contact filtration.

4.1Coagulation Unit

Many impurities in water and wastewater are present as colloidal solids which will not readily settle. Their removal can, however, often be achieved by promoting agglomeration of such particles by flocculation, with or without the use of a coagulant followed by sedimentation or flotation.

4.1.1 Colloidal suspensions

Sedimentation can be used to remove suspended particles down to a size of about 50 µm depending on their density, but smaller particles have very low settling velocities so that removal by sedimentation is not feasible. Table (3.2) gives calculated settling velocities for particles with relative density 2.65 in water at 10 ºC. It can be seen that in practical terms the smaller particles have virtually nonexistent settling velocities. If these colloidal particles can be persuaded to agglomerate, they may eventually increase in size to such a point that removal by sedimentation becomes possible.

Table (3.2) Settling velocities for discrete particles of relative density 2.65 in water at 10~

Particle size (μm)	Settling velocity (m/h)
1000	6×10^{2}
100	2×10^{1}
10	3×10^{-1}
1	3×10^{-3}
0.1	1×10^{-5}
0.01	2×10^{-7}

- Coagulation has been defined as the addition of a chemical to a colloidal dispersion that result in particle destabilization by the reduction in forces that tend to keep the particles apart. Coagulation involves the reduction of surface charges and the formation of complex hydrous oxides that form flocculent suspensions.
- Collisions between particles can be improved by gentle agitation, the process of flocculation, which may be sufficient to produce settleable solids from a high concentration of colloidal particles. With low concentrations of colloids, a coagulant is added to produce bulky floc particles which enmesh the colloidal solids.

Coagulants

In water treatment plants, chemical coagulation is usually accomplished by the addition of metallic salts (Coagulants), common chemical used in coagulation include alum (alum sulfate) $AL_2(SO)_3.14.3\ H_2O$ or $18\ H_2O$, Ferrous sulfate $Fe_2(SO_4)_3$ sodium aluminates, ferrous sulfate $Fe\ SO_4$ and lime $Ca\ (OH)_2$.
Alum is used (mostly) for water containing appreciable amoral of organic matter. Coagulant such as alum and iron salt reacts with alkalinity of water, and hydrolysis in it, e.g., alum reacts to form aluminum hydroxide floc, a gelatinous precipitate.

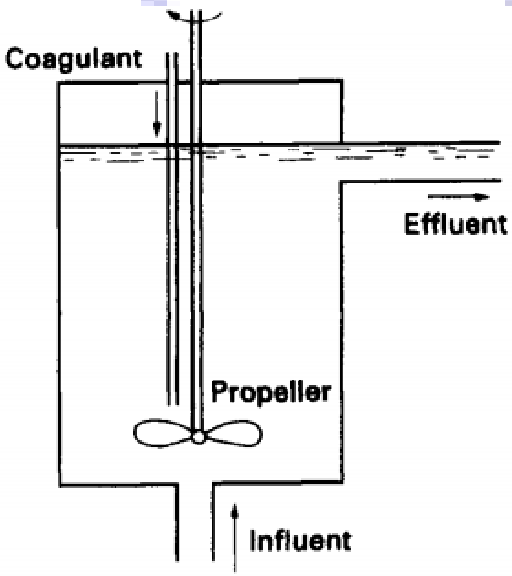
$$Al^{3+} + 3H_2O \rightarrow Al(OH)_3 + 3H^+$$

§§ The optimal dose will vary depending upon many factors such as, coagulant type, salt concentration, pH, temperature, nature of the colloids, size of turbidity particles, mixing and alum concentration. The latter two factors can be easily adjusted to an optimal condition in the design stage.

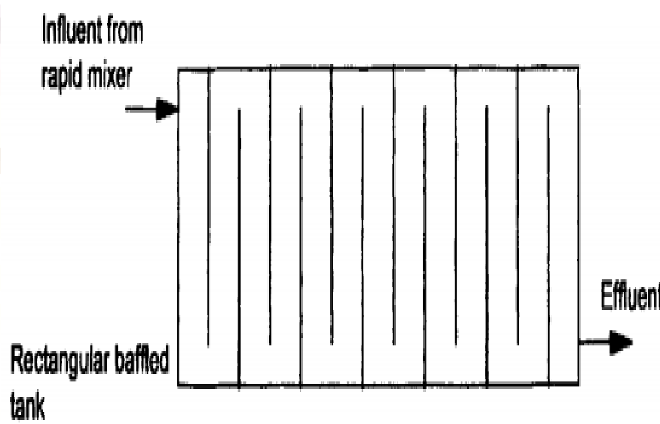
4.1.2 Rapid Mixing

§§ The function of rapid mixing is to ensure a homogeneous coagulation caused by a completely uniform dispersion of the coagulant throughout the raw water. The intent is for the coagulant to make contact with maximum number of colloidal particles before hydrolysis and adsorption or bridging action take place. the essential features of good mixing device as to produce (1) Adequate turbulence (2) Adequate mixing, and (3) Rapid dispersion of chemicals.

§§ The mixing has to be rapid because the hydrolysis of the coagulant is almost instantaneous (within a few seconds). The destabilization of colloids also takes very little time. Many techniques are used to provide rapid mixing for the dispersed of chemical in water. Basically, there are two groups: (1) Hydraulic Rapid Mixing, channels or chambers with baffles producing turbulent flow conditions (2) Mechanical Rapid Mixing, the power required for agitation of the water is imported by impellers, propellers or turbines. Generally, mechanical rapid mixers are more suitable for large water treatment plants than hydraulic ones.



Mechanical Mixing



Hydraulic mixing

Figure (4.2) Mechanical and Hydraulic mixing

- Treatment of water by coagulation involves -

  (1) Determination of optimum dose of coagulant by jar test.

  (2) Determination of power input for the flocculator.

- 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 figure (4.3).

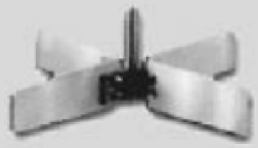
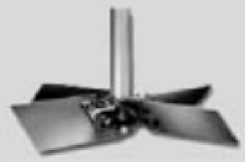
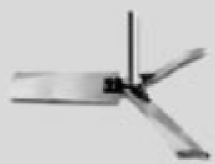
Impeller Type	Photograph	Power Number	Pumping Number	Application
Flat-bladed turbine (FBT)		3.6	0.9	Blending, maintaining suspensions, flocculation
Pitched-blade turbine (45° PBT)		1.26	0.75	Blending, maintaining suspensions, flocculation
Pitched-blade turbine with camber (hydrofoil, 3 blades)		0.2–0.3	0.45–0.55	Blending, maintaining suspensions, flocculation
Cast foil with proplets		0.23	0.59	Blending viscous liquids
Rushton turbine (6 blades)		4.5–5.5	0.72	Gas–liquid dispersion, solids suspension, flocculation
Propeller (pitch of 1:1)		0.32–0.36	0.4	Blending viscous liquids

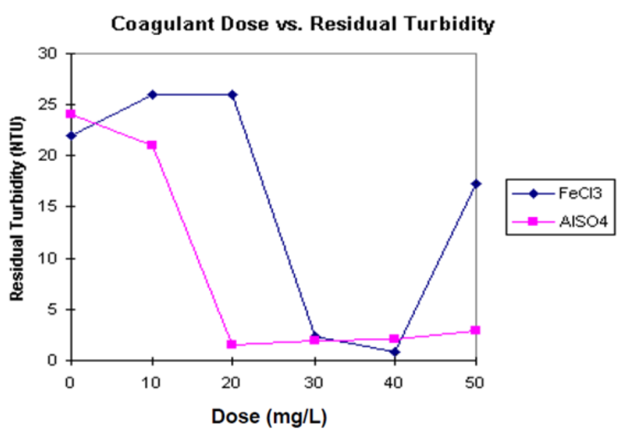
Figure (4.3) Types of impeller

4.1.3Coagulant Dosage and Jar Test

The jar test is a common laboratory procedure used to determine the optimum operating conditions for water or wastewater treatment. This method allows adjustments in pH, variations in coagulant or polymer dose, alternating mixing speeds, or testing of different coagulant or polymer types, on a small scale in order to predict the functioning of a large-scale treatment operation.

In the class jar test to determine optimum coagulant dose will be carried out, it is important to determine the optimum dose to avoid charge reversal and resuspension colloids. Optimum coagulant dose is considered as the amount of coagulant which produces water with lowest turbidity with economics considerations. The procedure for carrying out the test is explained in the lab manual in details. 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.



Jar Test apparatus and the expected results

4.1.4 Rapid Mixing Tank Design

Rapid mix unit is usually composed of circular or rectangular tanks. The unit design includes determination of tanks number, dimensions and power of mixing devices.

Detention time (D.T), sec.

D.T is the time required for a small amount of water to pass through a tank at a given flow rate. Mathematically, detention time is given by the following formula:

$$\text{D.T.} = \frac{V}{Q} \;\ldots\ldots\ldots(1)$$

Where: t = detention time, sec V = water volume, m^3 Q = water flowrate, m^3/sec

The Velocity Gradient (G), sec^{-1}

G is a measurement of the intensity of mixing in the tank. The velocity gradient determines how much the water is agitated in the tank, and also determines how much energy is used to operate the flash mixer.

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:

$$G = \sqrt{\left(\frac{P}{\mu V}\right)} \;\ldots\ldots\ldots(2)$$

Where:

P: Power input in (Watt)

μ: Dynamic viscosity (N.S/m^2)

V: Volume of water (m^3)

For turbulent conditions, power requirements can be completed from the mathematical relationship below:

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

Where:

P: Power consumption, (Watt)

K: factor demands on the number of blades in the propeller

ρ: Mass density of the fluid to be mixed, (kg/m^3)

D: Diameter of the propeller, (m)

N: Rotational speed of the propeller, (revolution/second)

The values of (K) range from (1.0) for a three bladed propeller to (6.3) for a turbine with six flat blades.

§§ Design criteria

1. Detention time (DT): 60 ~ 120 sec.
2. Velocity gradient (G): 700 ~ 1000 sec^{-1}
3. GT (Camp number): 30000 to 60000 (D.T = GT / G)
4. *Other design considerations include:*

- Design water depth = 0.5 to 1.1 times the basin diameter or width. For rectangular basins, it is preferred to use square area.
- Maximum Tank Volume= 8 m3 (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 vertexing.
- Chemicals should be added below the impeller, point of most mixing.
- 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.
- Water 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.
- The free board must not be less than 30cm.

Example

A city is planning for the installation of a water treatment plant (WTP) with a design flow of ($2m^3/sec$). Assuming rapid detention time and velocity gradient of 10 sec and 1000 sec^{-1} respectively and the power of mixer of (11.19kW) with 80% efficiency and 110 rpm . Design a rapid mix basin and size the mixing equipment. Use dynamic viscosity of $1.053*10^{-3}$ Pa.sec and assume a six flat blades propeller will be used.

Solution

The volume of the rapid mix tank is V

V=D.T. × Q = 10 × 2 = 20 m^3

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

P = 0.8 * 11.19 = 8.95kW

$$G = \sqrt{\left(\frac{P}{\mu V}\right)} \quad \text{or} \quad V = \frac{P}{\mu\, G^2} = \frac{8.95\times10^3}{1.053\times10^{-3}\times1000^2} = 8.5m^3$$

$$\text{No. of tanks} = \frac{Total\ tank\ volume\ required}{volume\ requirement\ per\ tank\ based\ on\ mixing} = \frac{20m^3}{8.5m^3/tank} = 2.35$$

Use 3 rapid mix tanks. Then volume of each tank is 6.67 m^3

The impeller diameter can be estimated using $P = K\,\rho\, N^3\, D^5$

$$D = \left[\frac{P}{K\rho N^3}\right]^{1/5} = \left[\frac{8.95\times10^3}{6.3\times1000\times1.83^3}\right]^{1/5} = 0.75m$$

Assume Impeller diameter to tank diameter ratio of 0.33, then the tank diameter would be equal to

$$\text{tank diameter} = \frac{Impeller\ diameter}{Impeller\ diameter\ to\ tank\ diameter\ ratio} = \frac{0.75}{0.33} = 2.27m$$

The surface area of the tank would be equal to $A_s = \pi/4\,(2.27)^2 = 4.05m^2$

$$\text{tank depth} = \frac{Tank\ volume}{Tank\ area} = \frac{6.67}{4.05} = 1.65m$$

Need to check liquid-depth to tank diameter ratio ; $\frac{Liquid\ depth}{Tank\ diameter} = \frac{1.65}{2.27} = 0.73$

Then 0.73 is within the guideline of 0.5 ~ 1.1

4.2Flocculation Unit

While rapid mix is the most important physical factor affecting coagulant efficiency, flocculation is the most important factor affecting particle-removal efficiency. The objective of flocculation is to bring the particles into contact so that they will collide, stick together, and grow to a size that will readily settle or filter out.

After destabilization (i.e. Coagulation), particles will be ready to a tract and agglomerate and form flocs. But this agglomeration is slow and they need help to accelerate this agglomeration. This help is called Flocculation "*which is the slow stirring or gentle agitation to aggregate the destabilized particles and form a rapid settling floc*". This gentle mixing increases the collisions between the particles and help them to agglomerate. Notice that rapid mixing will destroy the flocs, that's why we need gentle mixing. "Too much mixing will shear the floc particles so that the floc is small and finely dispersed". It is thus the conditioning of water to form flocs that can be readily removed by settling, dissolved air flotation or filtration, Fig (4.4)

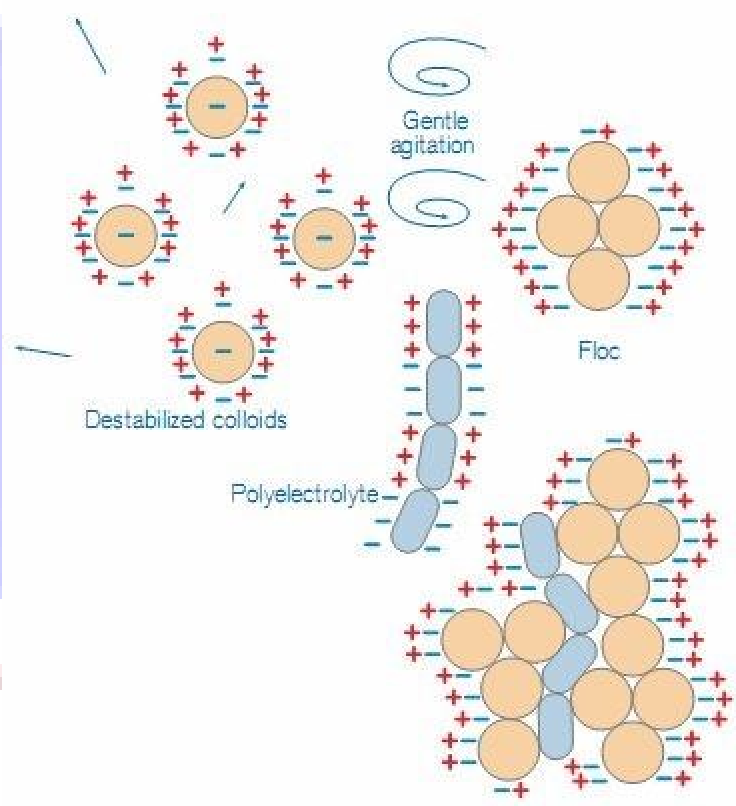


Fig (4.4) Schematic representation of flocculation process

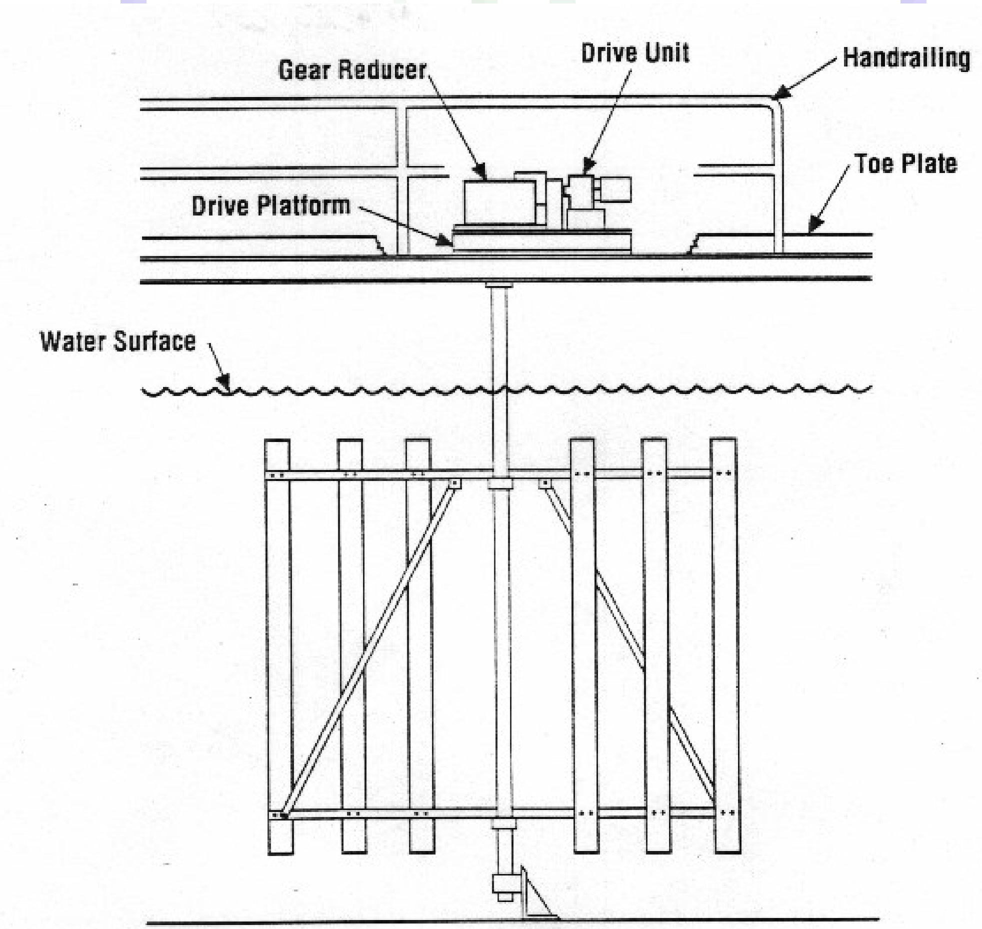
Flocculation occurs in a tank called a Flocculator or Flocculation Basin equipped with a method for slow mixing. The most common types of Flocculators are shown in the following.

Types of Flocculators

Flocculators are classified into two types;

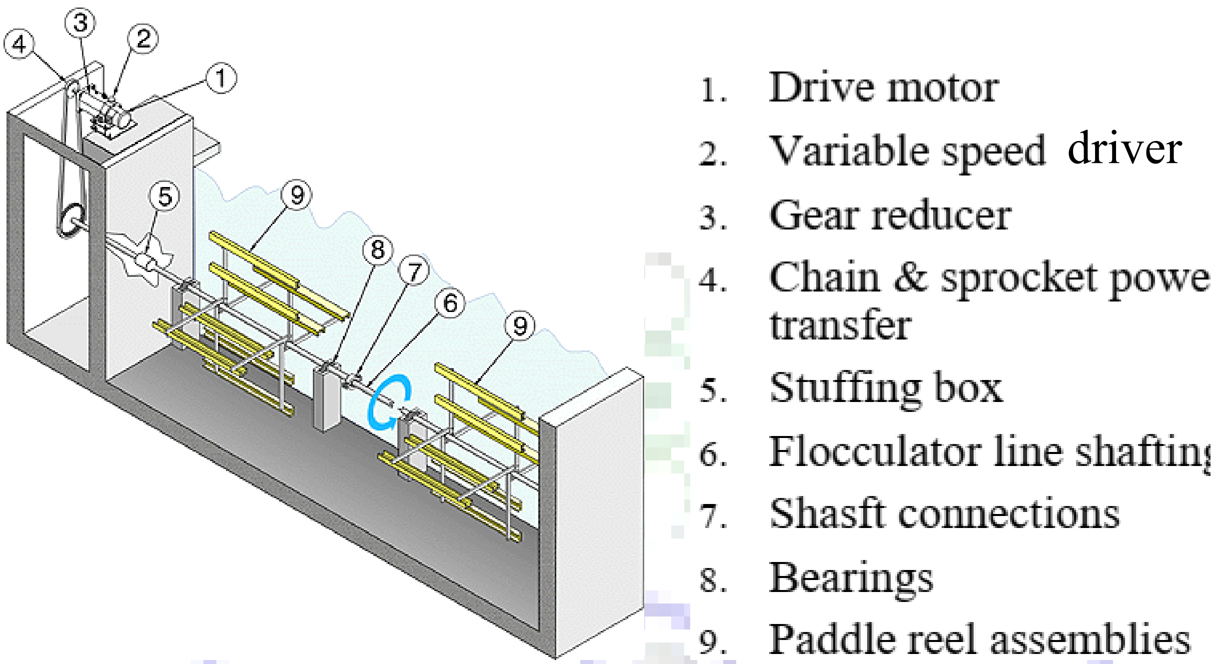
1 – Mechanical Flocculators

Mechanical (or paddle) flocculators in which slow mixing is mainly achieved using revolving paddles.



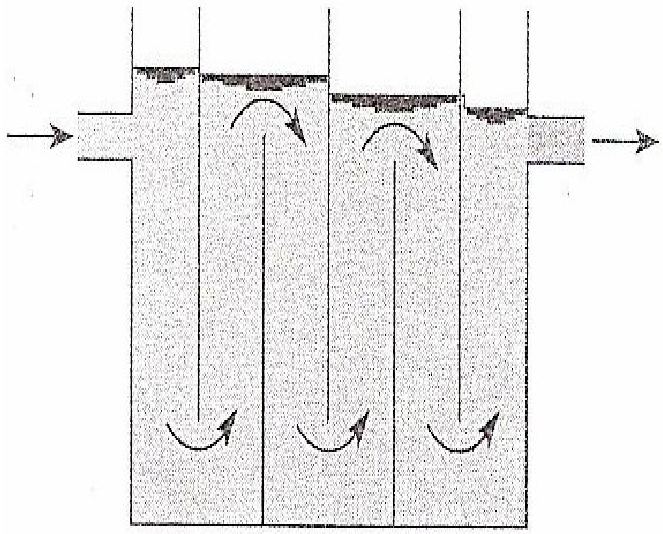
Two basic types of mechanical flocculators are used in flocculation facilities, horizontal-shaft with paddles and vertical-shaft, high energy flocculators. In recent years, vertical-shaft flocculators have been used with increasing frequency. The selection of one of the two types is mostly dependent on the type of filtration system used.

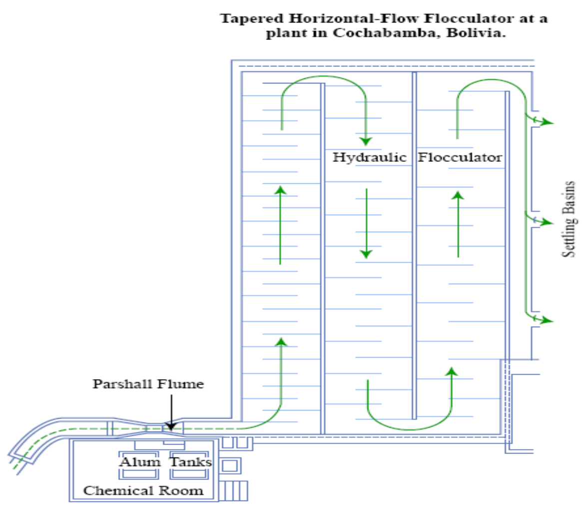
The principal element of mechanical flocculator systems are agitator impellers, drive motors, speed controllers and reducers, transmission systems, shafts and bearings.



2 – Hydraulic Flocculators

A more practical approach is to use hydraulic flocculators that do not require mechanical equipment, nor a continuous power supply if gravity flow is available, and which can be built primarily from concrete, brick, wood, or masonry.





Flocculation is directly proportional to the velocity gradient established in the water by a stirring action (G). The mean velocity gradient is given by:

$$G = \sqrt{\left(\frac{P}{\mu V}\right)}$$

Where:

G= velocity gradient, sec^{-1}

P: Power input in (Watt)

μ: Dynamic viscosity ($N.S/m^2$)

V: Volume of water (m^3)

4.1.1 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 above equation.

For paddle flocculators, the useful power input is directly related to the drag of the paddles. The power input to the water by horizontal paddles may be estimated as

$$P = \frac{1}{2}\, C_d \cdot A \cdot \rho \cdot v_p^3$$

Where

P: Power imparted to water, Watt

C_d: Drag coefficient, it varies with the length to width ratio of the paddle blades ranges from (1.2-1.9).

A: The cross-sectional area of the paddle (m^2),

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

ρ : Water density, 1000kg/m^3

The velocity of the paddles v may be estimated as

$$v = 2\pi \cdot r \cdot n$$

$$v_p = 2\pi \cdot k \cdot r \cdot n$$

where k = constant = 0.75

r = radius to centerline of paddle, m

n = rotational speed, rps

Notice1: The paddles impart a velocity to the water, so the velocity of the paddle must exceed the relative velocity. Experience has shown that the relative velocity of the water is 75% of the rotational velocity of blades. This represented by the coefficient (k) in above equation, i.e. the velocity of the paddle relative to the water is 75% of the absolute peripheral velocity of the paddle.

Notice2: The total area of the paddle (m^2),A

$$A = N_p \times N_b \times A_b$$

N_p= number of paddles

N_b= number of blades in one paddle

A_b= blade area, m^2

$$A_b = L_b \times W_b$$

L_b = blade length, m

W_b = blade width, m

§§ Design criteria

1- Design of Flocculation Unit

1. Detention time (t)=20 to 30min
2. Velocity gradient (G)= 30 to 50 sec^{-1}
3. G.t=10^4 to 10^5
4. Water depth= 3 to 5m

2- Design of paddle flocculator

For paddle flocculator, there are other design requirements;

a- Paddles area should not exceed 15 to 20% of the cross-sectional area of the flow.

b- Blade width (W_b)=10 to 15cm.

c- The tank is divided into two or more compartments using baffles provided with orifices uniformly distributed over the vertical surface of baffle.

d- Baffles are designed to provide orifice ratio of 3% to 6% of a velocity of 0.27m/sec.

e- The top of baffles is slightly submerged (1 to 2 cm) and the bottom should have a space of 2 to 3cm above the tank floor to allow for tank drainage.

f- Water depth is 1m greater than wheel diameter.

Parameter	Recommendation
G	$< 50\ s^{-1}$
Basin	
Depth	1 m > wheel diameter
Clearance between wheel and walls	0.3–0.7 m
Wheel	
Diameter	3–4 m
Spacing between wheels on same shaft	1 m
Spacing between wheel "rims" on adjacent shafts	1 m
Paddle board	
Width	10–15 cm
Length	2–3.5 m
Area of paddles/tank cross section	0.10–0.25
Number per arm	3
Spacing	at 1/3 points on arm
Tip speed	0.15–1 m/s
C_D $L/W = 5$	1.20
$L/W = 20$	1.50
$L/W >> 20$	1.90
Motor	
Power	1.5–2 × water power
Turn down ratio	1:4

Assignment:

Design a paddle flocculator by determining the basin dimensions, the paddle configuration, the power requirement, and rotational speeds for the following parameters:

Design flow rate = $50 * 10^3\ m^3/d$

t = 22 min

Three flocculator compartments with G = 40, 30, 20 sec^{-1}

Water temperature = 15 ºC

Example:

A water treatment plant is being designed to produce 50000m^3/d of water. Jar testing and plot plant analyses indicate that an alum dosage of 40 mg/l with flocculation at a GT value of 4.0×10^4 produces optimal results at the expected water temperature of 15 ºC. Determine:

1. The monthly alum requirements.
2. The flocculation basin dimensions if three cross-flow horizontal paddles are to be used. The flocculator should be a maximum of 15m wide and 5m deep in order to connect appropriately with the settling basin,
3- The power requirement.
4- The paddle configuration.

Solution:

1. The monthly alum requirements.

40 mg/l = 0.04 kg/m^3

0.04 kg/m^3 × 50000m^3/d × 30 month/day = 60 ton / month.

2. Basin Dimensions

a- Assume an average G value of 30 sec^{-1}

GT = 4.0×10^4 , then $t = \frac{GT}{G} = \frac{40 \times 10^4}{30} \; \frac{1\,min}{60\,sec} = 22.22$min.

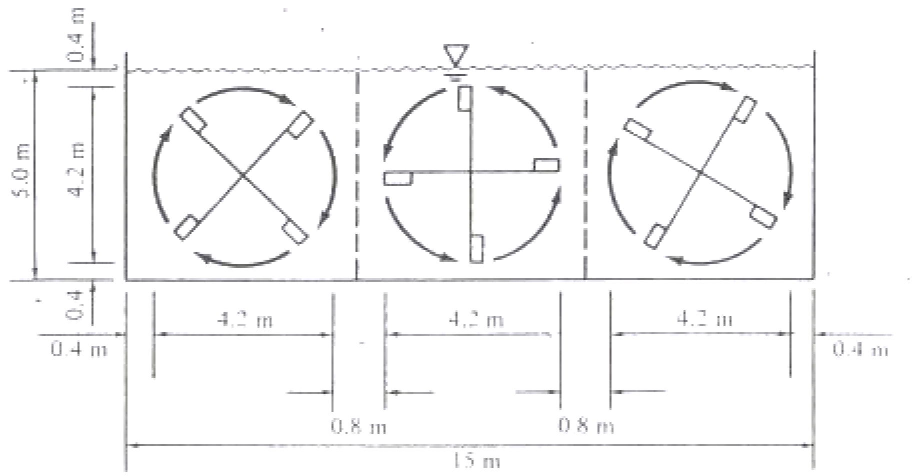
b- Volume of the tank is

V = Q t = 50000m^3/d × 22.22min. × 1 day/1440 = 771.5 m^3

c- The tank will contain three cross-flow horizontal paddles, so its length will be divided into three compartments. For equal distribution velocity gradient, the end area of each compartment should be square, (depth = 1/3 length), with max depth of 5m then,

the length is 15m and width is 15 × 5 × w = 771.5 width = w = 10.3m

d- The configuration of the tanks and paddles should be as follows



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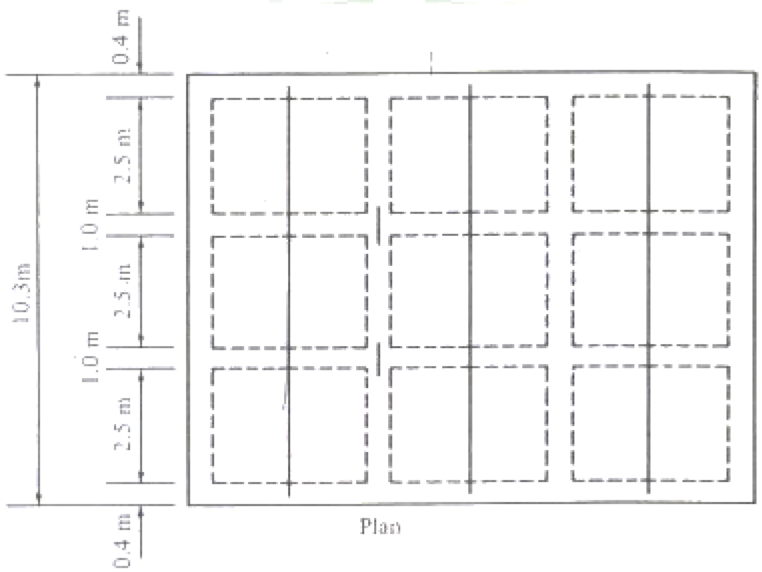


Plan

3- Power requirement

a- Assume G value tapered as follows

First compartment, G= 40 sec^{-1}

Second compartment, G= 30 sec^{-1}

Third compartment, G= 20 sec^{-1}

Calculate power requirement for compartments 1,2, and 3

$$P = G^2 V \mu$$

$$V = 771.5 / 3 = 257.2 m^3$$

From table $\mu = 1.139 * 10^{-3}$ Pa.sec at 15 ºC

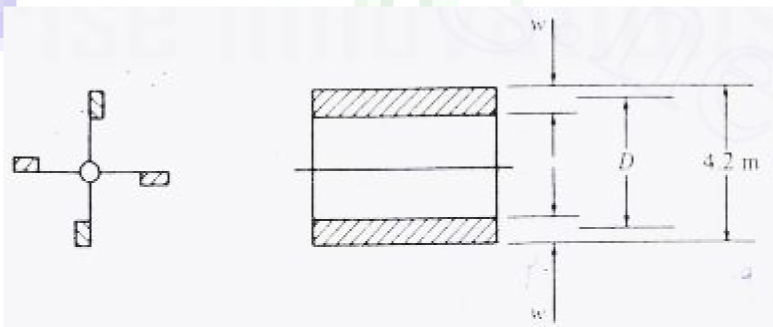
$P_1 = 40^2 * 257.2 * 1.139 * 10^{-3} = 468.7$ W = 0.47kW

$P_1 = 30^2 * 257.2 * 1.139 * 10^{-3} = 0.26$kW

$P_1 = 20^2 * 257.2 * 1.139 * 10^{-3} = 0.12$kW

4- Paddle configuration

a- Assume paddle design as shown below



Each paddle wheel has four boards 2.5 long and w wide - three paddle wheels per compartment

b- Calculate w from power input and paddle velocity

$$P = \frac{1}{2} C_d \cdot A \cdot \rho \cdot v_p^3$$

From table $\rho = 999.1$ kg/m^3 at 15 ºC

Assume v_p= 0.67m/sec * 0.75 = 0.5m/sec and $C_d = 1.8$

$$A = Length\ of\ boards\ \times w \times number\ of\ boards$$

3 paddles at 4 boards per paddle = 12 boards

12 * w * 2.5 = 30w = A

P_1= 468.7 = 0.5 * 1.8 * 30w * 999.1 * $(0.5)^3$

w= 0.14m

c- Calculate rotational speed of paddles

First Compartment

$v_p = 2\pi \cdot k \cdot r \cdot n$

0.67 * (60) = $2\pi \cdot 0.75 \cdot 2.1 \cdot n$ Then n = 4 rpm

Second Compartment

$$P = \frac{1}{2} C_d \cdot A \cdot \rho \cdot v_p^3$$

$$260 = \frac{1}{2} \times 1.8 \times (30 \times 0.14) \times 999.1 \cdot v_p^3$$

$$v_p = 0.41 m/sec$$

i.e Actual speed = 0.41/0.75 = 0.55m/sec

Then n= (0.55 * 60) / $2\pi \cdot 0.75 \cdot 2.1$ = 3.3 rpm

Third Compartment

The same procedure applied in second compartment using P= 120W we can find

$$v_p = 0.32 m/sec$$

i.e Actual speed = 0.32/0.75 = 0.42m/sec

Then n= (0.42 * 60) / $2\pi \cdot 0.75 \cdot 2.1$ = 2.55 rpm

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 six 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 CD = 1.8. All paddles remain submerged all the time.

Find:

a) The water velocity

b) The value of G and c) The Camp number.

Solution:

$v_p = 2\pi \cdot k \cdot r \cdot n = 2\pi \times 0.70 \times 2 \times 1.5 = 0.22 m/s$

$$P = \frac{1}{2} C_d \cdot A \cdot \rho \cdot v_p^3$$

A, paddle area = 0.2 x 15 x 4 x 1 = 12 m^2

$P = \frac{1}{2} \times 1.8 \times 12 \times 1000 \times (0.22)^3 = 115 W$

$$G = \sqrt{\left(\frac{P}{\mu V}\right)}$$

tank volume = 30 x 15 x 5 = 2250 m^3

one compartment volume = 2250/6 = 375 m^3

From table $\mu = 1 * 10^{-3}$ Pa.sec

P = 115 W

$G = \sqrt{\left(\frac{115}{1\times10^{-3}\times375}\right)} = 17.5\ sec^{-1}$

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

$$T = \frac{V}{Q} = \frac{2250 \times 24 \times 60}{100 \times 1000} = 32.4 min$$

Camp no. = Gt = 17.5 x 32.4 x 60 = 34000

4.3 Sedimentation Unit

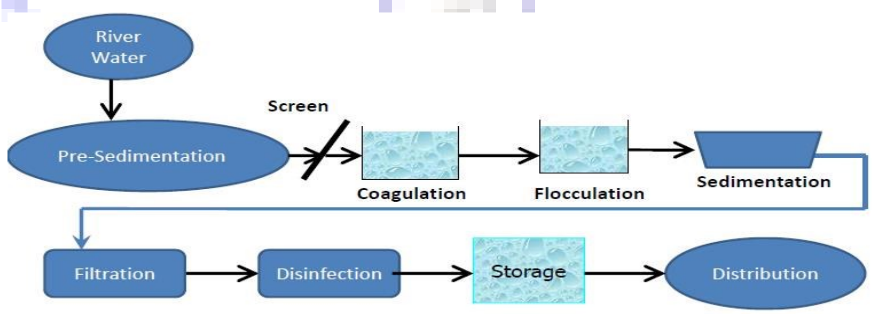
4.3.1 Definition of Sedimentation

Sedimentation is the separation of impurities known as discrete particles from the flowing fluid (flow) by the action of natural forces (gravity). This process takes place in a basin known as the sedimentation tank or the clarifier. The aim of sedimentation unit is to remove suspended solid particles from a suspension by settling under gravity. Particles that will settle within a reasonable period of time can be removed using a sedimentation tank (also called clarifiers).

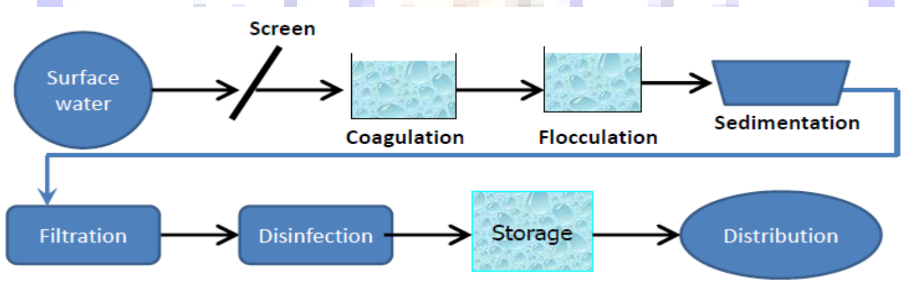
Sedimentation is the next step following coagulation and flocculation. The purpose of sedimentation is to prepare water for effective filtration. The efficiency of particle removal is a function of particle size and density and water temperature. Increasing the size or density of the particles increases the settling velocity and thereby facilitates removal.

4.3.2 Applications of sedimentation in water treatment:

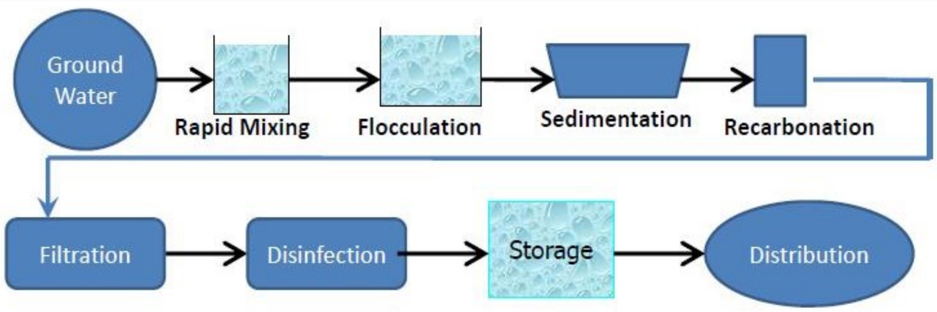
1. Plain settling (or pre - sedimentation) of river surface water.



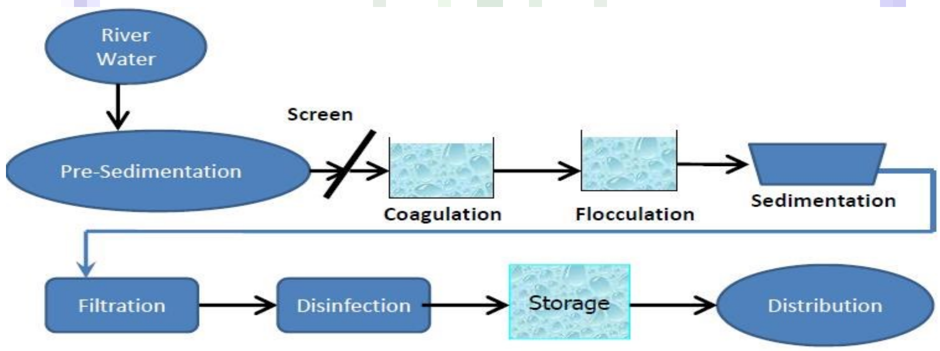
2. In filtration treatment plants treating surface water to removes flocculated solids. The sedimentation tank comes after the flocculation tank.



3. In Softening treatment plants treating hard water to removes flocculated solids. The sedimentation tank comes after the flocculation tank.

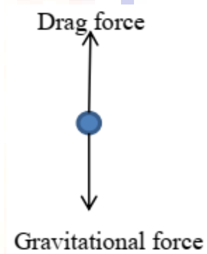


4. In aeration treatment plant removing iron and manganese from ground water.



4.3.3 Sedimentation Theory

When a particle settles in a fluid, it accelerates until the drag force due to its motion is equal to the submerged weight of the particles. At this point the particle will reach its terminal settling velocity, v_s.



$$Gravitational\ force = (\rho_s - \rho)gV$$

where;

ρ_s= density of solid particle.

ρ = fluid density (Water).

V= particle volume.

$$Drag\ force = C_D\ A_c\ \rho\ \frac{v_s^2}{2}$$

where;

C_D= Newton's drag coefficient.

A_c= cross sectional area of particle.

v_s= settling velocity of particle.

Equating gravitational and drag forces;

$$(\rho_s - \rho)gV = C_D\, A_c\, \rho\, \frac{v_s^2}{2}$$

Then;

$$v_s = \sqrt{\frac{2g\ V\ (\rho_s - \rho)}{C_D\, A_c\, \rho}}$$

For spherical particles;

$V = \frac{\pi d^3}{6}$ and $A_c = \frac{\pi d^2}{4}$ where; d is particle diameter.

Then;

$v_s = \sqrt{\frac{4g\ d\ (\rho_s - \rho)}{3C_D\ \rho}}$ Or put $\frac{\rho_s}{\rho} = G_s$ where G_s is the specific gravity of the solid particles.

$v_s = \sqrt{\frac{4g\ d\ (G_s - 1)}{3C_D}}$ (Newton's law for settling velocity)

The value of C_D is dependent on Reynolds number

$$Re = \frac{\rho\ .\ v_s\ d}{\mu}$$

For Re <0.5	$C_D = \frac{24}{Re}$
For 0.5 <Re<10^4	$C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$
For 10^3<Re<10^5	$C_D \cong 0.4$

When

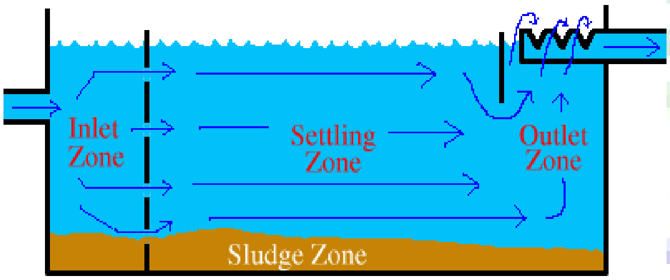
$C_D = \frac{24}{Re}$ then $v_s = \frac{g\ \rho\ .d^2\ .(G_s - 1)}{18\mu}$ (Stoke's law for settling velocity)

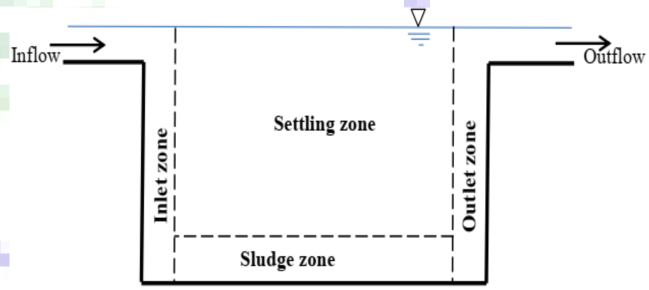
For turbulent flow conditions; $CD = 0.4$

$$v_s = \sqrt{3.3\ g\ d\ (G_s - 1)}$$

4.3.4 Ideal Settling Tank (Plain Sedimentation)

Plain sedimentation tank can be considered as an ideal settling tank. Plain sedimentation tank is not preceded by coagulation and flocculation units. Figure below illustrates an ideal rectangular clarifier (settling tank) with an *inlet zone* for transition of influent flow to uniform horizontal flow, a *settling zone* where the particles settle out of suspension by gravity, an *outlet zone* for transition of uniform flow in the sedimentation zone to rising flow for discharge, and a sludge zone where the settled particles collect.





Assumptions of ideal settling tank

1. Quiescent condition in settling one.
2. Uniform flow across the settling zone.
3. Uniform solid concentration as flow enters the settling zone.
4. Solids entering the sludge zone are not resuspended.

An ideal sedimentation basin is divided into the four zones:

1) Inlet zone: Region in which the flow is uniformly distributed over the cross section such that the flow through settling zone follows horizontal path.

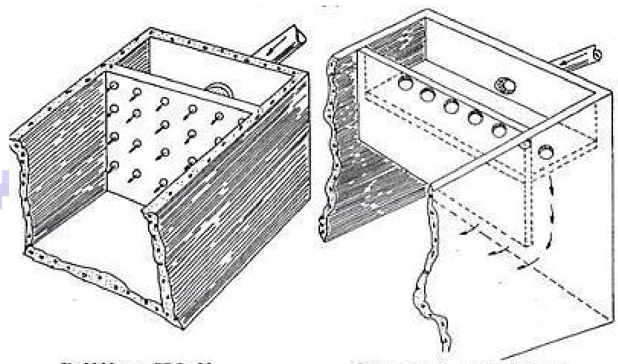
2) Settling zone: Settling occurs under quiescent conditions.

3) Sludge zone: For collection of sludge below settling zone.

4) Outlet zone: Clarified effluent is collected and discharge through outlet weir.

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. Inlets are shown below, the figure below. *The stilling wall*, also known as *a perforated baffle wall or diffusion wall*



Stilling Wall Channel or Flume

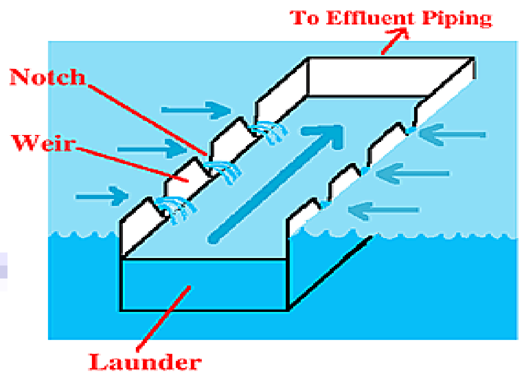
2.Settling zone

After passing through the inlet zone, water enters the settling zone where water velocity is greatly reduced. This is where the bulk of floc 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, and in some cases, tube settlers and lamella plates are included in the settling zone.

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 floc 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 figure shown.



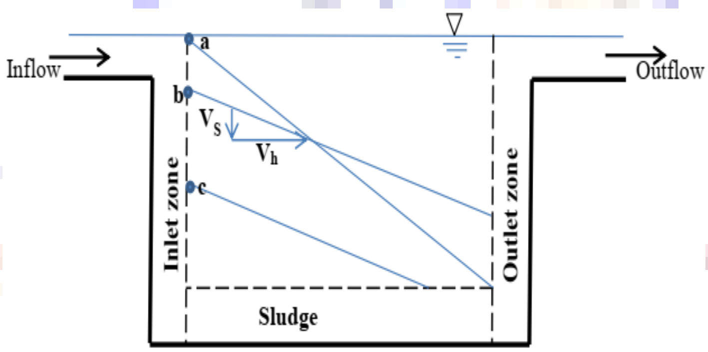
4.Sludge zone

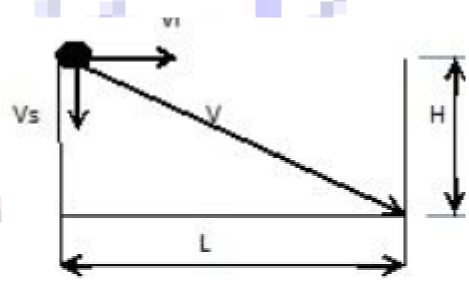
The sludge zone is found across the bottom of the sedimentation basin where the sludge collects temporarily. Velocity in this zone should be very slow to prevent resuspension of sludge. A drain at the bottom of the tank allows the sludge to be easily removed from the tank. The tank bottom should slope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

֍When a particle enters the settling zone, it will have horizontal velocity (flow velocity) component (v_h) equals that of water; and a vertical velocity component equals to its terminal settling velocity, v_s.

$$v_h = \frac{Q}{W \times H}$$; where w is the tank width and h is the water depth

The particle shall be removed from water if the resultant velocity takes it to the bottom of the tank (the sludge zone) before the outlet zone is reached.





Considering the velocity vector triangle and the dimensions of the tank (L= length, H = height and W = width).

If particle is to be removed, it's settling velocity and horizontal velocity (v_h) must be such that their resultant, (v), will carry it to the bottom of the tank before the outlet zone is reached, all particles with the same settling velocity will be removed.

$\frac{v_h}{v_s} = \frac{L}{H}$ then settling velocity $v_s = v_h \times \frac{L}{H} = \frac{Q}{W \times H} \times \frac{L}{H} = \frac{Q}{W \times L} = \frac{Q}{A_s}$

Where:

Q: The discharge entering the tank;

Ac: The cross-sectional area of the tank perpendicular to the flow = $W \times H$

As: The surface area= $W \times L$

⁂ Surface overflow rate (SOR)

Surface overflow rate (SOR) is numerically equal to the flowrate divided by the plan or surface area of the basin (SOR=Q/A_s). Physically, it represents the settling velocity of slowest settling particles (smallest particle) to be removed from the flow or to be settled at 100%. The particles which have settling velocity (v_s) greater than or equal to SOR will be entirely removed, while, those have settling velocity less than SOR will be removed at a ratio equals to their settling velocity (v_s) to SOR.

4.3.5 Types of Settling

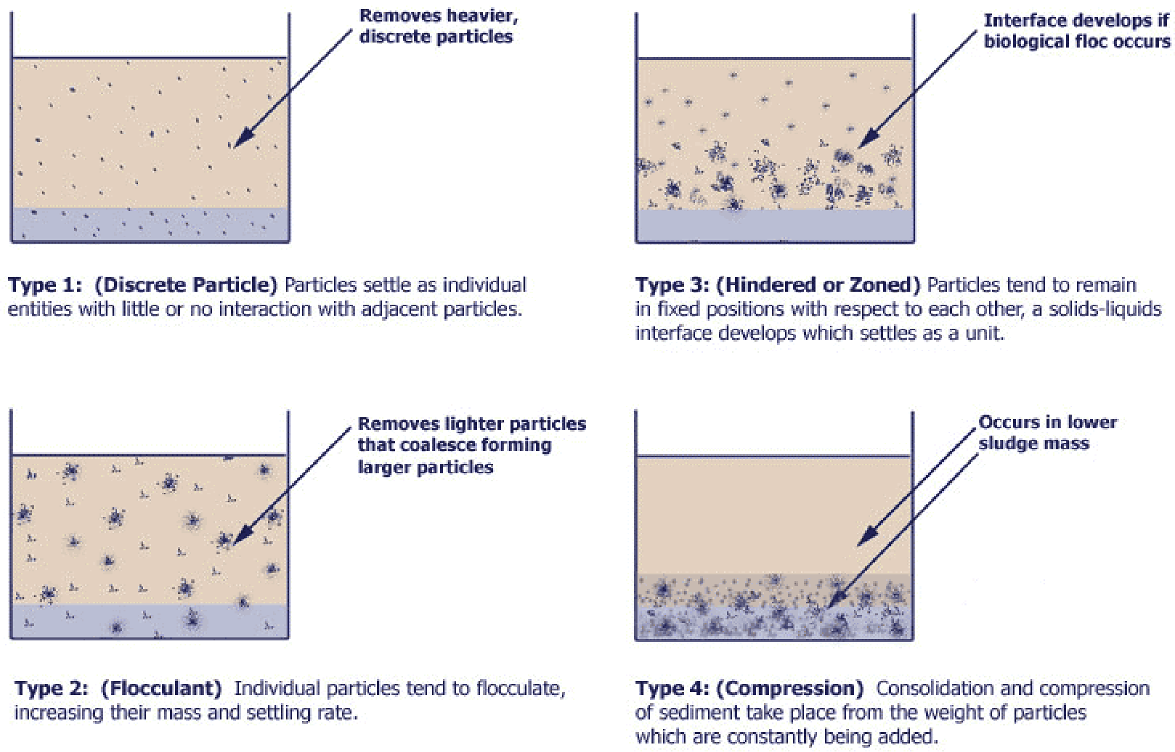
With such heterogeneous wastewaters/water and variable flows during the settling process in a sedimentation tank it is possible that four types of settling may occur. In general, four types of settling phenomena have been defined. The four types of settling are described below, and shown graphically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

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 sediments. Occurs in the lower sludge mass.



Type 1: (Discrete Particle) Particles settle as individual entities with little or no interaction with adjacent particles.

Type 3: (Hindered or Zoned) Particles tend to remain in fixed positions with respect to each other, a solids-liquids interface develops which settles as a unit.

Type 2: (Flocculant) Individual particles tend to flocculate, increasing their mass and settling rate.

Type 4: (Compression) Consolidation and compression of sediment take place from the weight of particles which are constantly being added.

⁂ Purpose of Settling:

1) To remove coarse dispersed phase.
2) To remove coagulated and flocculated impurities.
3) To remove precipitated impurities after chemical treatment.
4) To settle the sludge (biomass) after activated sludge process / tricking filters.

⁂ Principle of Settling:

1. Suspended solids present in water having a specific gravity greater than that of water tend to settle down by gravity as soon as the turbulence is retarded by offering storage.
2. Basin in which the flow is retarded is called settling tank.
3. Theoretical average time for which the water is detained in the settling tank is called the detention period.

⁂Sedimentation tanks have two functions:

The removal of settleable solids to produce an acceptable output and the concentration of the removed solids into a smaller volume. The design of a tank must consider both of these functions and the tank should be sized on whichever of the requirements is limiting. The sludge thickening function of a tank is likely to be important when dealing with relatively high concentrations of homogeneous solids.

4.3.6 Types of Sedimentation Tanks

Sedimentation tanks may have rectangular, circular or square shapes.

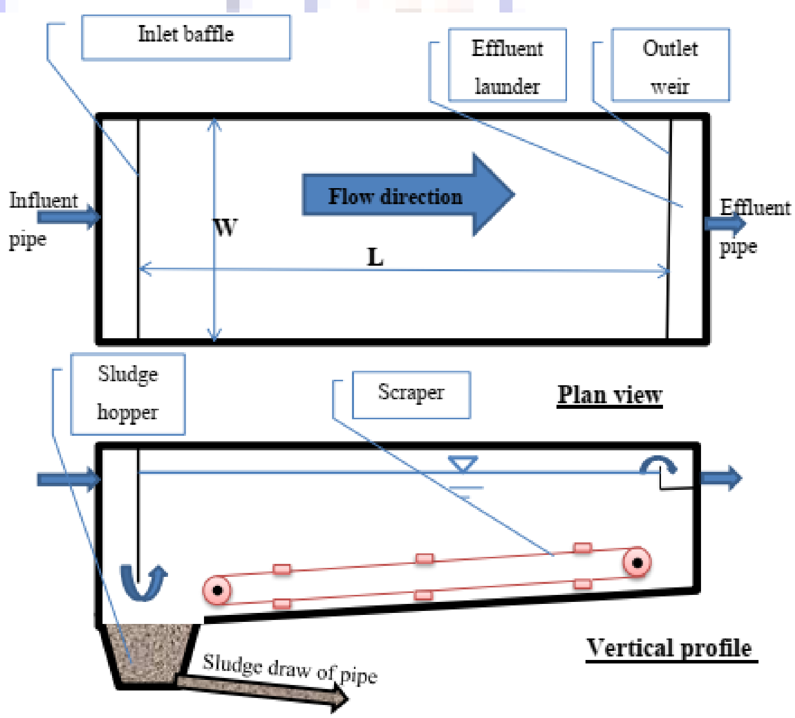
Rectangular sedimentation tanks

Figure shown shows plan view and a vertical profile of a rectangular sedimentation tank. Rectangular sedimentation tanks are usually designed to be long and narrow with the following characteristics;

- The flow is along the long axis.
- Length to width ratio (L/W) =3 to 6
- Tanks dimensions are selected to match the requirements of the chosen sludge collection equipment (scraper).

Generally;

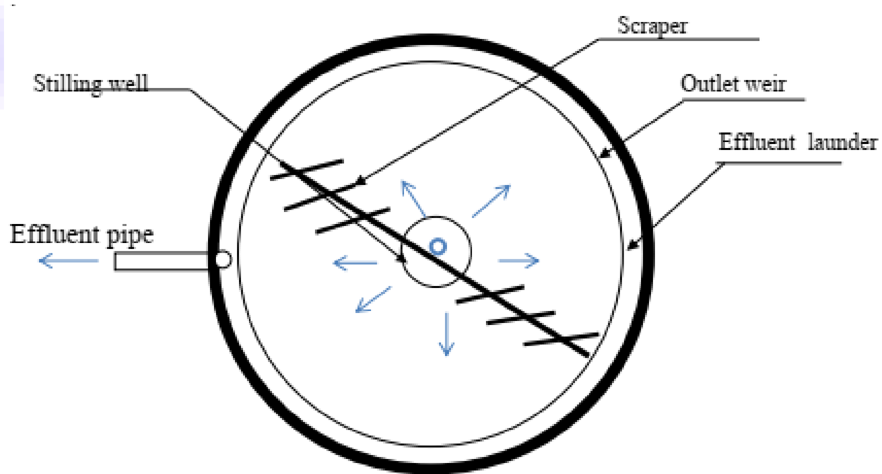
- Maximum tank length=100m
- Maximum tank width= 13.5m
- If scraper is used, bottom slope=1:24 to 1:12 (V:H).
- If scraper is not use (in case of small sedimentation tanks), bottom slope of 1:1 is usually adopted.
- The influent is discharged behind a baffle (inlet baffle).
- The overflow is flowing over an outlet weir into the effluent launder.



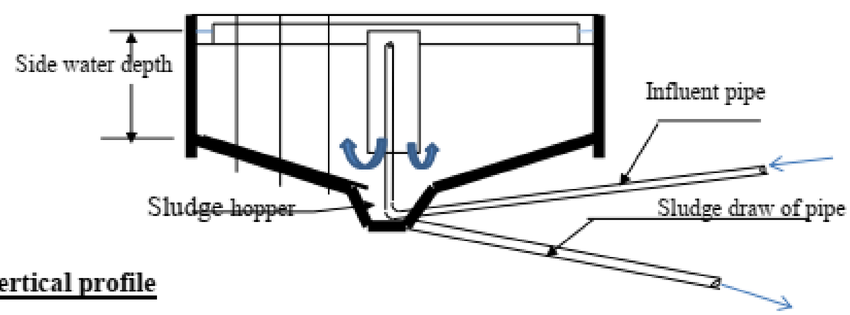
Circular sedimentation tanks

Figure below shows a plan view and a vertical profile in a circular sedimentation tank. The influent in discharged at the center of the tank in a cylinder called stilling well. The main characteristics of circular tanks are:

- Maximum diameter= 40m.
- Stilling well diameter = 0.1 to 0.2 of tank diameter and extends 1 to 2 m below the water surface.
- The overflow is flowing over an outlet weir and collected into the effluent launder at the tank periphery.
- Bottom slope as that of rectangular tanks



Plan view



Vertical profile

⁂ Why is it hard to reach ideal settling in sedimentation tanks?

1) Variable inlet flowing velocity.

2) Variable temperature inside the tank that will give different ρ_w and μ_w.

3) Un-similar particle distribution.

4) Short circuiting due to the shape of the tank.

4.3.7 Design of Sedimentation Unit

Design criteria

For sedimentation unit preceded by coagulation and flocculation processes, the following criteria are adopted:

- Surface overflow rate (SOR)= 20 to 40 m^3/m^2.day
- Detention time: Average 2 – 4 hrs.
- Depth or height (H): Average 3 – 6 m.
- Inlet velocity (v_h) 0.15 – 0.5 m/min.
- In rectangular tanks L/W = (2/1 – 5/1), L max less than 100 m.
- In circular tanks Diameter (D) max less than 50m, inlet diameter less than 10%D.
- Weir loading rate (the flow divided by the weir length): 120 – 250 $m^3/m/d$.

$$\text{Weir loading rate} = \frac{Q_{one}}{\text{weir length}}$$

Where;

Q_{one} = water flowrate received by one tank, m3/day

For circular tanks; weir length = π D (D= tank diameter)

For rectangular tanks;

If one effluent launder is used; weir length=W

If n effluent launders are used; weir length= (2n-1) W

** launder is a trough for holding or conveying water

- Scouring velocity (v_h): The horizontal velocity that will re-suspend the settled particles

$$v_h = \sqrt{\frac{8\beta g d\,(G_s - 1)}{f}} \qquad \beta = 0.04 - 0.06, \quad f = 0.02 - 0.03$$

Example:

Design a rectangular sedimentation tank (H, W and L of the tank) to treat 393 m^3/hr flows. Assume the smallest particle to be 100% removed is 0.03 mm in diameter, detention time = 3hr, inlet flowing velocity = 0.15m/min, temperature= 20 °C and G_s= 1.65. Find:

Solution:

From tables, we can find $\rho_w = 0.9982 \times 10^3$ kg/m^3 and $\mu = 1.002 \times 10^{-3}$ kg/m.sec

$$v_s = \frac{g\,\rho\,.d^2\,.(G_s - 1)}{18\mu} \quad \text{(Stoke's law for settling velocity)}$$

$$v_s = \frac{9.81 \times 0.9982 \times 10^3 \times (0.03 \times 10^{-3})^2 \times (1.65 - 1)}{18 \times 1.002 \times 10^{-3}} = 3.176 \times 10^{-4}\ m/sec = 27.39\text{m/day} = \text{SOR}$$

$$SOR = \frac{Q}{A_s} \leftrightarrow A_s = \frac{Q}{SOR} = \frac{393 \times 24}{27.39} = 344.36\ \text{m}^2$$

$$Q = \frac{V}{T} \quad V = A_S \times H = Q \times T\ ,$$

$$V = 344.36 \times H = 393 \times 3 \qquad \therefore H = 3.42\ \text{m O. K}$$

$$v_h = \frac{L}{T} \leftrightarrow L = 0.15 \times 3 \times 60 = 27m$$

$$A_S = L \times W \quad 344.36 = 27 \times \text{W} \rightarrow\rightarrow \text{W} = 12.75\text{m}$$

Check: $\frac{L}{W} = \frac{27}{12.75} = 2.12\ \text{O. K}$

Check: Weir loading rate (WLR) $= \frac{Q}{\text{weir length}}$

Assume effluent length = W = 12.75 m

$$\text{WLR} = \frac{Q}{\text{weir length}} = \frac{393 \times 24}{12.75} = 739.76\ \text{m}^3/\text{m/day} > 250\ \text{m}^3/\text{m/day NOT O. K}$$

Assume weir loading rate =250 m^3/m/day

$$250\,\frac{\frac{\text{m}^3}{\text{m}}}{\text{day}} = \frac{393 \times 24}{\text{weir length}} \qquad \therefore L = 37.7m \approx 38m$$

Check the scouring velocity:

Assume β =0.04 , f=0.02 i.e $\frac{\beta}{f} = 2$

$$v_h = \sqrt{\frac{8\beta g d\,(G_s - 1)}{f}} = \sqrt{8 \times 2 \times 9.81 \times (0.03 \times 10^{-3})\,(1.65 - 1)} = 0.055\ \text{m/sec} = 3.319\ \text{m/min}$$

$v_h = 0.15m/\min$ *and* $V_s > v_f$ *O. K*

4.4 Filtration Unit

4.4.1 Definition

The resultant water after sedimentation will not be pure, and may contain some very fine suspended particles and bacteria in it. To remove or to reduce the remaining impurities still further, the water is filtered through the beds of fine granular material, such as sand, etc. The process of passing the water through the beds of such granular materials is known as Filtration.

Filtration is the separation of non-settleable solids from water by passing it through a porous media. Filtration also aims to remove color, taste, odor, iron, manganese and microorganisms such as bacteria. In water treatment plants this is done in a unit known as the filter.

4.4.2Theory of Filtration يجب تحديثها

The following are the mechanisms of filtration

1. Mechanical straining – Mechanical straining of suspended particles in the sand pores.
2. Sedimentation – Absorption of colloidal and dissolved inorganic matter in the surface of sand grains in a thin film
3. Electrolytic action – The electrolytic charges on the surface of the sand particles, which opposite to that of charges of the impurities are responsible for binding them to sand particles.
4. Biological Action – Biological action due to the development of a film of microorganisms layer on the top of filter media, which absorb organic impurities.

4.4.3Types of filter

A number of classification systems are used to describe granular filters including filtration rate, media type, flow mechanism, washing technique, filtration rate control. Based on filtration rate filters are classified into rapid sand filter and slow sand filter.

Filters are classified also according to process workability:

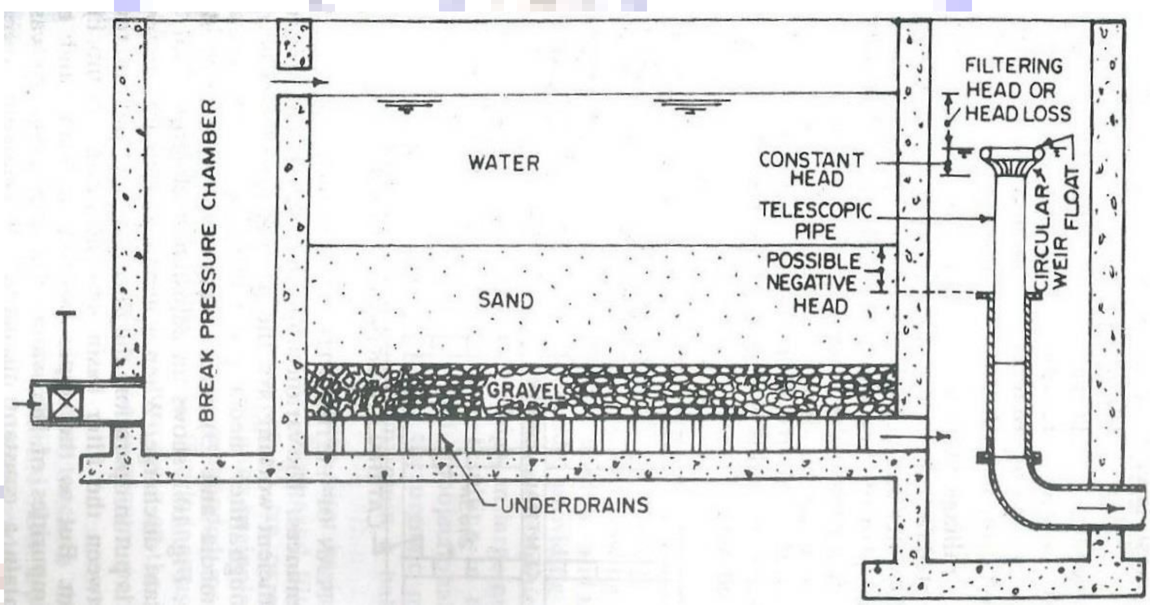
1) Gravity filters:

a) *Slow sand filter*: They consist of fine sand, supported by gravel. They capture particles near the surface of the bed and are usually cleaned by scraping away the top layer of sand that contains the particles.

Slow sand filters are best suited for the filtration of water for small towns. The sand used for the filtration is specified by the effective size and uniformity coefficient. The effective size, D_{10}, which is the sieve in millimeters that permits 10% sand by weight to pass. The uniformity coefficient is calculated by the ratio of D_{60} and D_{10}.

Construction of Slow Sand Filters

Slow sand filter is made up of a top layer of fine sand of effective size 0.2 to 0.3mm and uniformity coefficient 2 to 3. The thickness of the layer may be 75 to 90 cm. Below the fine sand layer, a layer of coarse sand of such size whose voids do not permit the fine sand to pass through it. The thickness of this layer may be 30cm. The lowermost layer is a graded gravel of size 2 to 45mm and thickness is about 20 to 30cm. The gravel is laid in layers such that the smallest sizes are at the top. The gravel layer is the retains for the coarse sand layer and is laid over the network of open jointed clay pipe or concrete pipes called under drainage. Water collected by the under drainage is passed into the out chamber.



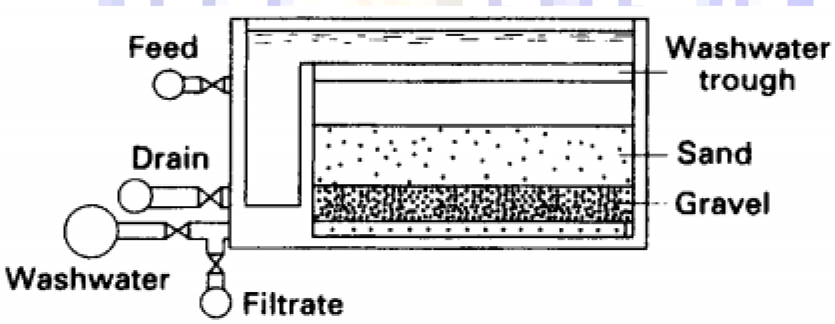
b) Rapid sand filter: They consist of larger sand grains supported by gravel and capture particles throughout the bed. They are cleaned by backwashing water through the bed to 'lift out' the particles.

Rapid sand filter is replacing the slow sand filters because of high rate of filtration ranging from 100 to 240 m^3/m^2/day and small area of filter required. The main features of rapid sand filter are as follows.

- Effective size of sand 0.45 to 0.55mm
- Uniformity coefficient of sand 1.2 ~ 1.7
- Depth of sand 60 ~ 70 cm
- Filter gravel 2 to 50 mm size (Increase size towards bottom)
- Depth of gravel 45cm
- Depth of water over sand during filtration 1 ~ 2m
- Overall depth of filter including 0.5m
- Area of single filter unit $100m^2$ in two parts of each $50m^2$
- Loss of head Max 1.8 ~ 2m
- Turbidity of filtered water 1NTU
- Percentage removal of coliforms 95%

Operation

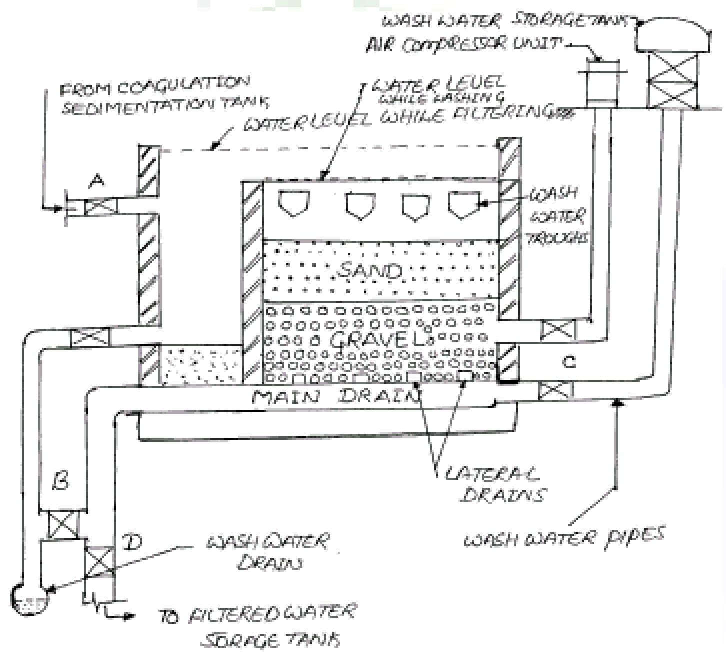
The water from coagulation sedimentation tank enters the filter unit through inlet pipe and uniformly distributed on the whole sand bed. Water after passing through the sand bed is collected through the under-drainage system in the filtered water well. The outlet chamber in this filter is also equipped with filter rate controller. In the beginning the loss of head is very small. But as the bed gets clogged, the loss of head increases and the rate of filtration becomes very low. Therefore, the filter bed requires its washing.



Washing Process

Washing of filter done by the back flow of water through the sand bed as shown:

First the valve 'A' is closed and the water is drained out from the filter leaving a few centimeter depth of water on the top of sand bed. Keeping all valves closed the compressed air is passed through the separate pipe system for 2-3 minutes, which agitates the sand bed and stirrer it well causing the loosening of dirt and clay inside the sand bed. Now valve 'C' and 'B' are opened gradually, the wash water tank, rises through the laterals, the strainers gravel and sand bed. Due to back flow of water the sand expands and all the impurities are carried away with the wash water to the drains through the channels, which are kept for this purpose.



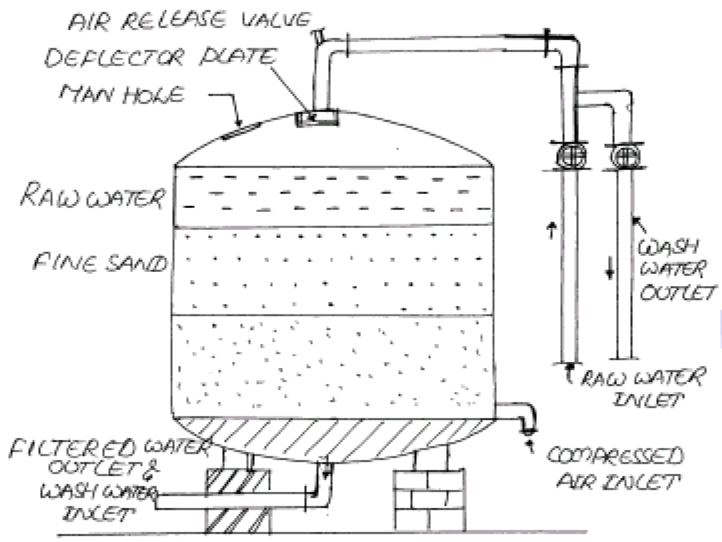
Washing process is continued till the sand bed appears clearly. The washing of filter is done generally after 24 hours and it takes 5 - 15 minutes and during back washing the sand bed expands by about 50%.

Rapid sand filter brings down the turbidity of water to 1 N.T.U. This filter needs constant and skilled supervision to maintain the filter gauge, expansion gauge and rate of flow controller and periodical backwash.

No.	Item	S.S. F	R.S. F
1	Area	Need very large area	Needs small area
2	Raw Water Turbidity	Not more than 30 NTU	Not more than 10NTU hence needs coagulation
3	Sand Media	Effective size 0.2 to 0.3 mm uniformity coefficient 2 to 3 single layers of uniform size	Effective size 0.45 to 0.55 mm uniformity coefficient 1.2 to 1.7 multiple graded layers of sand.
4	Rate of Filtration	2.4 to 3.6m^3/m^2/day	100-240 m^3/m^2/day
5	Loss of Head	0.6m to 0.7 m	1.8m to 2.0m
6	Supervision	No skilled supervision is required	Skilled supervision is required
7	Cleaning of Filter	By the surface of the filter bed and replacing it. Cleaning interval that is replacement of sand at 1 to 2 months.	Back wash with clean water under pressure to detach the dirt on the sand. Backwashing daily or on alternate days.
8	Efficiency	Bacterial removal, taste, odour, colour and turbidity removal.	There is no removal of bacteria. Removal colour taste, odour and turbidity is good.

2) Pressure filter:

Pressure filter is type of rapid sand filter in a closed water tight cylinder through which the water passes through the sand bed under pressure. All the operations of the filter is similar to rapid gravity filter, expect that the coagulated water is directly applied to the filter without mixing and flocculation. These filters are used for industrial plants but these are not economical on large scale. Pressure filters may be vertical pressure filter and horizontal pressure filter. The Figure below shows vertical pressure filter. Backwash is carried by reversing the flow with values. The rate of flow is 120 to 300m^3/m^2/day.



Advantages

1. It is a compact and automatic operation
2. These are ideal for small estates and small water works
3. These filters require small area for installation
4. Small number of fittings are required in these filters
5. Filtered water comes out under pressure no further pumping is required.
6. No sedimentation and coagulant tanks are required with these units.

Disadvantages

1. Due to heavy cost on treatment, they cannot be used for treatment large quantity of water at water works
2. Proper quality control and inspection is not possible because of closed tank
3. The efficiency of removal of bacteria & turbidity is poor.
4. Change of filter media, gravel and repair of drainage system is difficult.

4.4.4 Rapid sand filters

4.4.4.1 Filter media

Filters may be of single medium, dual-media or triple-media. In single medium, sand or anthracite is used alone and

The specifications of sand are;

Effective size (D_{10})=0.45 to 0.55mm

Uniformity coefficient (D_{60}/D_{10}) = 1.2 to 1.7

Sand depth=600 to 750mm.

The specifications of anthracite are;

Effective size (D_{10}) ≥ 0.7mm

Uniformity coefficient (D_{60}/D_{10}) ≤ 1.75

Anthracite depth= water depth.

⁂ Dual-media beds normally contain 0.15 to 0.3m of sand has effective size of 0.45 to 0.55mm overlaid by 0.46 to 0.76m of anthracite has effective size of 0.8 to 1.2 mm. A triple-media contains 5 to 10cm garnet has effective size of 0.15 to 0.35mm, 0.15 to 0.3m of sand has effective size of 0.35 to 0.5mm and 0.5 to 0.6m anthracite size of 0.8 to 1.2.

Gravel

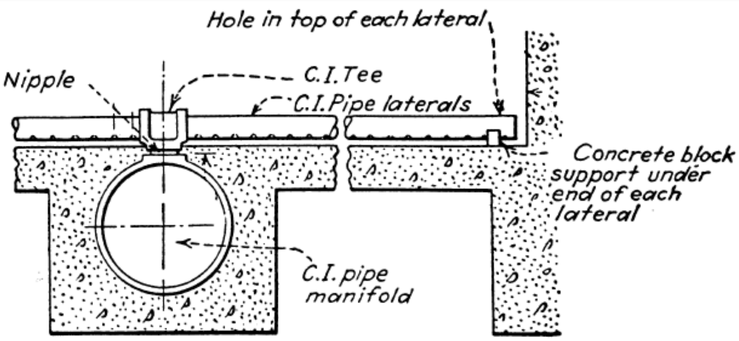
The sand is underlain by 400 to 600mm of gravel which serves to;

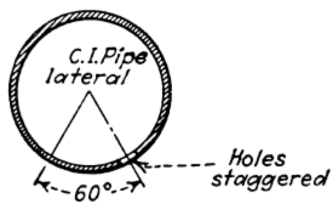
- Support the sand
- Allow the wash-water to move more uniformly upward to the sand.

Gravel is placed into four to six layers with the finest size on top. The arrangement of gravel layer is as follows;

4.4.4.2 The Underdrain System

The underdrain collects the filtered water from the gravel and distributes the wash-water during the washing process. A widely used type is the perforated-pipe system. It consists of a central ductile iron manifold or header into a number of laterals that can be attached. The perforations are placed alternately on the underside but 30° off-center.





Detail of Lateral Drilling

4.4.4.3The Washing Process

Washing consists of passing filtered water upward through the bed at such velocity that causes the sand bed expansion. The cleaning of a granular bed during backwash is a result of the shear produced by the rising water and of the scraping resulting from contacts between particles in the fluidized bed. The backwash velocity (V_b) is;

$$0.3\ m/min < v_b < v_t$$

$v_t = 10D_{60}$ for Sand where v_t in m/min and D_{60} in mm.

$v_t = 4.7D_{60}$ for Anthracite

It was found that the maximum scraping occurs when the bed is 10 percent expanded or;

$$v_b = 0.1v_t$$

Thus, for sand;

$$\boldsymbol{v_b = D_{60}}$$

And for anthracite;

$$\boldsymbol{v_b = 0.47D_{60}}$$

The rates above are for a temperature of 20°C but can be corrected for other temperature by;

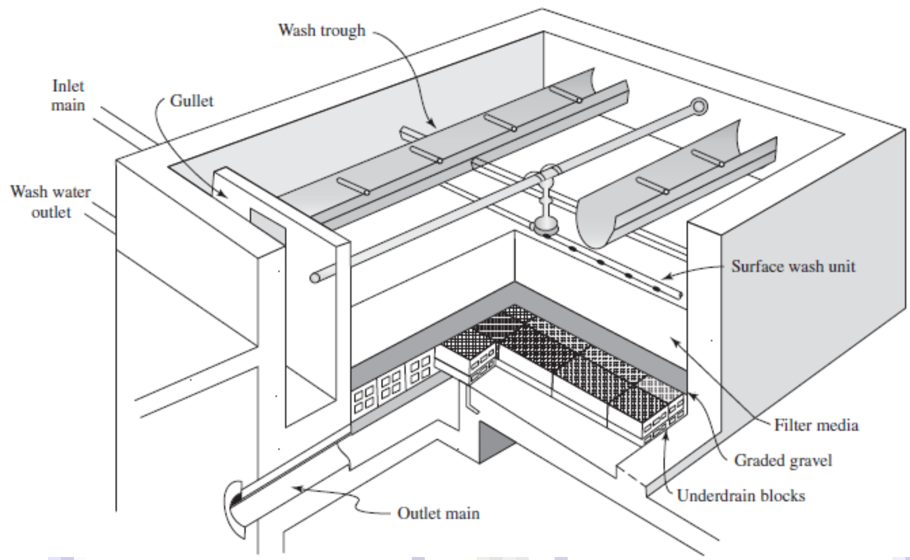
$$\boldsymbol{v_{b(T)} = v_{b(20)} \times \mu_T^{-1/3}}$$

Where; $\boldsymbol{v_{b(T)}}$ is the backwash velocity at temperature of T

μ_T is the dynamic viscosity at temperature of T

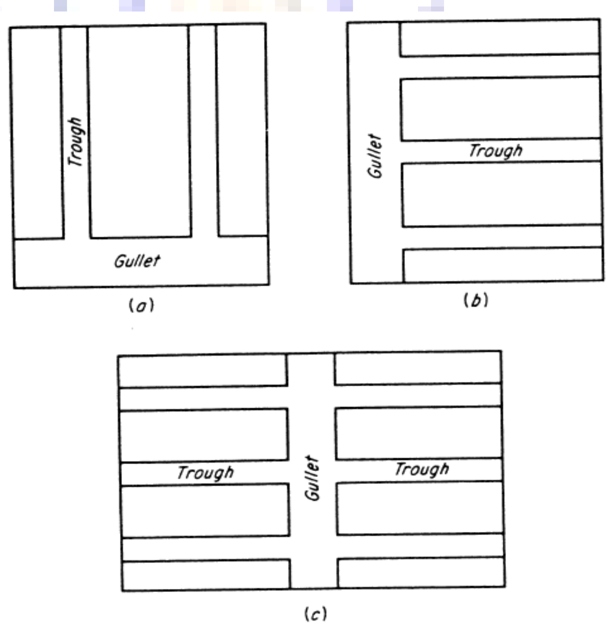
4.4.4.4Filter Unit and Wash water Troughs

A filter consists of two or more units of sizes depend on plant capacity. Usually, all the units are of the same capacity. The units may be placed in one or two rows. A unit of gravity filter (*See figure below*) is concrete box open at the top with a depth of 3m or more.



Total Filter Depth = *Thickness of sand + Thickness of gravel + water depth + diameter of lateral + free board*

The rising wash-water, after passing through the media, flows into wash-water troughs. The top edges of the troughs are horizontal and are placed at the same height, usually at a distance of 600 to 900mm above the sand level. The spacing of troughs (from edge to edge) is ≤ 2m. The troughs can be arranged as shown below:



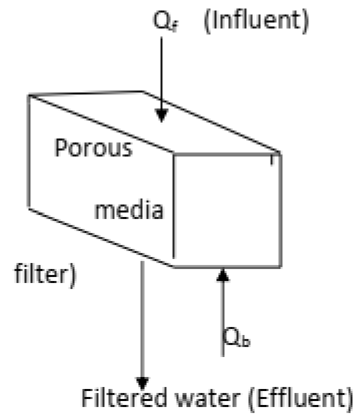
Gullet: The objectives of this main channel to collect the backwash water from the troughs.

The troughs may have rectangular section. For rectangular troughs, the dimensions are obtained as:

$$y = 1.73\sqrt[3]{\frac{q^2}{gb^2}}$$

Where y is the water depth (m) and b is the trough width (m) and q is the wash-water received by one trough (m^3/sec). A free board of 50 to 100mm is usually added to y.

Definitions



Q_f = Filtration discharge (flow through one filter)

$Q_f = A_s \times v_f$

A_s =surface area of the filter.

v_f = Filtration rate (m/day). Or m^3/m^2/day

Q_b= Backwash discharge (Washing water for one filter)

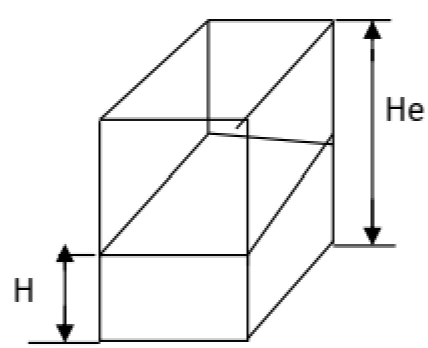
$Q_b = A_s \times v_b$

v_b =The backwash velocity

Also $Q_b = \frac{Volume\ of\ washing\ waer}{Washing\ time}$

⁂ An expansion space for the filter media to move when backwashed

$$\frac{H_e}{H} = \frac{(1-n)}{(1-n_e)}$$



H = Height of the filter bed.

H_e = Height of the expanded filter bed when backwashed

n = Porosity of the clean filter bed.

n_e = Porosity of the expanded filter bed,

$$n_e = \left(\frac{v_b}{v_s}\right)^{0.22}$$

v_b = Backwash velocity rate.

v_s= Settling velocity of the filter media (Stoke 's settling velocity)

q = Backwash discharge reaching one trough = $\frac{Q_b}{No.of\ troughs}$

4.4.4.5Design Criteria

- Effective size of sand 0.45 to 0.6mm
- Filtration rate (v_f) 100 -240 m/day.
- Uniformity coefficient of sand 1.2 ~ 1.7
- Depth of sand 60 ~ 70 cm
- Filter gravel 2 to 50 mm size (Increase size towards bottom)
- Depth of gravel 45-60cm
- Depth of water over sand during filtration 1 ~ 2m
- Overall depth of filter including 0.5m
- Area of single filter unit As 5 – 200 m^2
- Loss of head Max 1.8 ~ 2m
- Washing time 5 -15 min.
- Turbidity of filtered water 1NTU
- Percentage removal of coliforms 95%
- Filtration run = 12-72 hr (if the turbidity of the influent is: a. Low–long run, b. High–short run).

<u>Example:</u>

Design filtration unit for water treatment plant has a design capacity of 120000m^3/day. Use gravity rapid sand filters and assume water temperature is 20°C.

<u>Solution:</u>

Assume filtration rate=180m/day

$$Q_f = A_s \times v_f \rightarrow\rightarrow\rightarrow\rightarrow v_f = \frac{Q_f}{A_s} \quad \therefore A_{total} = \frac{120000}{180} = 666.67\text{m}^2$$

Let number of units = 2

$$\therefore A_{unit} = \frac{666.7}{2} = 333.33 \text{ m}^2$$

Check filtration rate during backwash process;

$$\text{filtration rate} = \frac{120000}{(2-1)\times 333.33} = \frac{360m}{day} > 240m/day \text{ NOT OK.}$$

Let number of units=n

$$A_{unit} = \frac{666.7}{n}$$

Put filtration rate during washing=240m/day

$$\therefore 240 = \frac{120000}{(n-1)\times\frac{666.7}{n}}$$

n= 4

use 4 units ⟶ $A_{unit} = \frac{666.7}{4} = 166.67\text{m}^2$

Let the unit dimensions of L * W and consider that L = 2W i.e L * W= 2W * W = 166.67

Then W= 9.13m and L= 18.26m

Let thickness of sand layer = 600mm

Let effective size of sand (D_{10}) =0.5mm

Let uniformity coef. = 1.4 = D_{60}/D_{10} That means (D_{60}) =0.7mm

Design of wash water troughs

For sand medium; $v_b = D_{60}$ Then v_b =0.7m/min

Let number of washwater troughs =2

Let width of trough=0.6m and Check spacing of troughs

$$spacing\ of\ troughs = \frac{9.13 - 2 \times 0.6}{2} = 3.97m \ > 2m \quad \therefore NOT\ OK$$

Let number of troughs=4

$$spacing\ of\ troughs = \frac{9.13 - 4 \times 0.6}{2} = 1.68m \ < 2m \quad \therefore OK$$

Use rectangular troughs

$$y = 1.73\sqrt[3]{\frac{q^2}{gb^2}}$$

$$Q_b = A_s \times v_b = 166.67 \times 0.7 = 116.67\ {m^3}/{min.} = 1.944\ {m^3}/{sec.}$$

The flowrate received by one trough=q = 1.944/4 = 0.486 ${m^3}/{sec.}$

$$y = 1.73\sqrt[3]{\frac{(0.486)^2}{9.81 \times (0.6)^2}} = 0.7\text{m}$$

Use free board of 50mm

Total depth of trough= 0.7+0.05=0.75m

Determination of unit depth

Unit depth = (sand thickness + gravel thickness +water depth +lateral dia.)

Let thickness of gravel layer = Let thickness of sand layer = 600mm = 0.6m

4.5 Disinfection Unit

4.5.1 Definition

Water used for drinking and cooking should be free of pathogenic (disease-causing) microorganisms that cause such illnesses as typhoid fever, dysentery, and cholera. Purification of drinking water containing pathogenic microorganisms requires a specific treatment called disinfection.

Disinfection reduces pathogenic microorganisms in water to levels designated safe by public health standards. This prevents the transmission of disease. An effective disinfection system kills or neutralizes all pathogens in the water. It is automatic, simply maintained, safe, and inexpensive. An ideal system treats all the water and provides residual (long term) disinfection.

The purpose of disinfecting water supplies is to prevent the spread of waterborne disease by destroying pathogenic organisms. Most of the physical and chemical treatment processes described previously will remove most of the microorganisms to some extent. However, very small numbers of microorganisms that are viable and pathogenic are all that are required to bring about disastrous epidemic.

4.5.2 Methods of disinfection

Water can be disinfected using;

1. Treatment with chlorine (Chlorination).
2. Treatment with Ozone (Ozonation).
3. Treatment with Ultraviolet irradiation.
7. Treatment with oxidizing agents
6. Treatment with ultrasonic waves.

4.5.2.1. Chlorination of Water

Chlorination is the most commonly used method for water disinfection. It is effective against many pathogenic bacteria, but at normal dosage rates it does not kill all viruses and worms. Chlorine is most often available commercially as chlorine gas cylinders, as sodium hypochlorite (household bleach) and as calcium hypochlorite. Chlorine gas is most often employed in large water treatment plants because of its lower cost; however, chlorine gas is difficult to handle since it is toxic, heavy, corrosive, and an irritant. At high concentrations, chlorine gas can even be fatal.

Chlorine readily combines with chemicals dissolved in water, microorganisms, plant material, tastes, odors, and colors. These components "use up" chlorine and comprise the *chlorine demand* of the treatment system. It is important to add sufficient chlorine to the water to meet the chlorine demand and provide residual disinfection.

1- Reactions of Chlorine

At the same time that chlorine is being used up by compounds in the water, some of the chlorine reacts with the water itself. The reaction depends on what type of chlorine is added to the water as well as on the pH of the water itself. When chlorine gas enters the water, it reacts with water and breaks down into *hypochlorous acid* (HOCl) and hydrochloric acid (HCl).

$$Cl_2 + H_2O \leftrightarrow HOCl + HCl$$

Hypochlorous acid may further break down, depending on pH:

$$HOCl \leftrightarrow H^+ + OCl^-$$

Hypochlorous acid may break down into a hydrogen ion and a hypochlorite ion , or a hydrogen ion and a hypochlorite ion may join together to form hypochlorous acid. The concentration of hypochlorous acid and hypochlorite ions in chlorinated water will depend on the water's pH. A higher pH facilitates the formation of more hypochlorite ions and results in less hypochlorous acid in the water. This is an important reaction to understand because hypochlorous acid is the most effective form of ***free chlorine residual (free available chlorine)***, meaning that it is chlorine available to kill microorganisms in the water. Hypochlorite ions are much less efficient disinfectants. So, disinfection is more efficient at a low pH (with large quantities of hypochlorous acid in the water) than at a high pH (with large quantities of hypochlorite ions in the water.)

free available chlorine will react with nitrogenous compounds like ammonia to form chloramines. The reaction of ammonia with HOCl is shown below:

$NH_3 + HOCl \rightarrow H_2O + NH_2Cl$ (monochloramine) at pH over 7.5

$NH_2Cl + HOCl \rightarrow H_2O + NHCl_2$ (dichloramine) at pH 5 – 6.5

$NHCl_2 + HOCl \rightarrow H_2O + NCl_3$ (nitrogen trichloride) at pH 4.4

Chlorine in water reacting with ammonia, nitrogenous compounds and organic matter is defined as ***combined available chlorine***. The above compounds are less active than free available chlorine as disinfecting agents, but may maintain residuals for longer time than free chlorine.

2- Chlorine Demand

Chlorine Demand = Chlorine added – Residual chlorine

It is the difference between the mount of chlorine added and the mount present as residual either Free or Combined after a contact time.

Chlorine demand to kill microorganism = C × t

Where: C = Concentration of the disinfect (chlorine) and t = Contact time between the microorganism and the disinfect (chlorine).

Residual chlorine is the remaining chlorine after disinfection in water. This may be necessary in the distribution system and storage tanks to protect water in the pipes from any pollution.

3- What is used in chlorination?

a) Chlorine gas, chlorine is obtained in pressurized cylinders ranging in weight from 45 to 1000 kg.

b) Hypochlorination, chemical compounds such as Sodium hypochlorite NaOCl (laundry bleach) and Calcium Hypochlorite $Ca(OCl)_2 . 4H_2O$.

Hypochlorites are useful in disinfecting waters of reservoirs and swimming pools also in controlling algae growth. Hypochlorites in water will give free available chlorine:

$Ca(OCl) \rightarrow Ca^{+2} + 2OCl^-$

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

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

4- Chlorination is classified according to its point application as:

a) **Plain Chlorination** is used with no other treatment. Chlorine is added to the water from the source. The dose added depends on the degree of contamination. About 0.5 mg/L or more is used for disinfection and obtain residual chlorine free or combined.

b) **Pre-chlorination**, applying chlorine before any other treatment. It may be added to the suction pipe of the raw water pumps or in the flash mixer.

Its advantages:

i. Improve the coagulation process.

ii. Reduce taste and odor caused by organic sludge in sedimentation tanks.

iii. Killing algae and other microorganisms, so filters may remain clean to obtain long filtration runs.

A dose of 5 – 10 mg/L is used to obtain 0.1 – 0.5 mg/L residual chlorine free or combined.

c) **Post-chlorination**, applying chlorine at the end of the treatment process. A contact time for about 30 minutes is required for disinfection when adding 0.25 – 0.5 mg/L chlorine dose to obtain 0.1 – 0.2 mg/L residual chlorine. In Iraq a dose of 1 -2 mg/L is added to obtain 0.1 – 0.5 mg/L residual.

The combination of pre-and post-chlorination may be needed if the raw water is highly polluted. Also, in some cases chlorine is injected into the distribution system to maintain the desired residual.

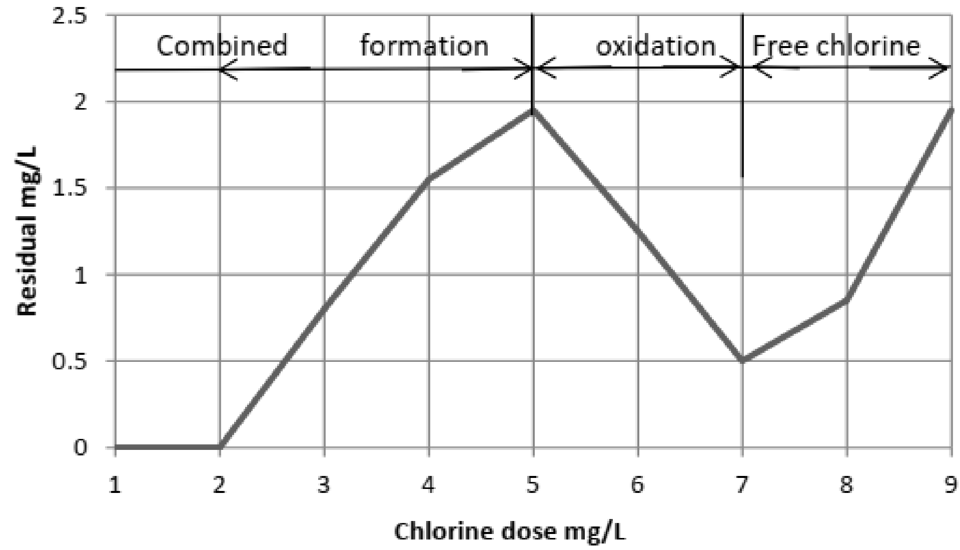
d) **Breakpoint chlorination**, chlorine is consumed in oxidizing many compounds present in water. No chlorine can be measured until the initial chlorine demand is satisfied. Then chlorine reacts and may form combined chlorine residual (some are odorous and undesirable like chlorophenols). These combines increase with the addition of chlorine dose until a maximum combined residual is reached. Further addition of chlorine will cause a decrease in the combined residual. This is the Breakpoint chlorination, at this point the combined are oxidized to oxides. After this point, free chlorine residual is present. A chlorine dose of 7 – 10 mg/L in order to obtain free chlorine about 0.5 mg/L.

The breakpoint is the point at which the chlorine demand has been totally satisfied; the chlorine has reacted with all reducing agents, organics, and ammonia in the water. When more chlorine is added past the breakpoint, the chlorine reacts with water and forms hypochlorous acid in direct proportion to the amount of chlorine added. This process, known as breakpoint chlorination, is the most common form of chlorination, in which enough chlorine is added to the water to bring it past the breakpoint and to create some free chlorine residual.

The dosage can produce breakpoint chlorination is determined by drawing chlorine residual curve (or chlorine demand curve). This curve represents the relationship between chlorine dosage applied and chlorine residual, see figure below.

The breakpoint indicates complete oxidation of the chloramines and other chlorine combinations, and the residual above the breakpoint is mostly free available chlorine.

e) **Super chlorination/Dechlorination**, is adding a high chlorine dose for quick disinfection, this will produce high residues, so dechlorination should be applied afterward. For dechlorination: aeration, adding chemicals, using activated carbon (adsorbent) may be applied.



Break point Chlorination

f) **Contact time**

The contact (retention) time (Table below) in chlorination is that period between introduction of the disinfectant and when the water is used. A long interaction between chlorine and the microorganisms results in an effective disinfection process. Contact time varies with chlorine concentration, the type of pathogens present, pH, and temperature of the water.

Contact time must increase under conditions of low water temperature or high pH (alkalinity). Complete mixing of chlorine and water is necessary, and often a holding tank is needed to achieve appropriate contact time. An alternative to the holding tank is a long length of a pipe to increase contact between water and chlorine.

Table shown below is used for calculating the contact time using the highest pH and lowest water temperature expected for the treated water. For example, if the highest pH anticipated is 7.5 and the lowest water temperature is 42 °F, the "K" value (from Table 1) to use in the formula is 15. Therefore, a chlorine residual of 0.5 mg/l necessitates 30 minutes contact time. A residual of 0.3 mg/l requires 50 minutes contact time for adequate disinfection.

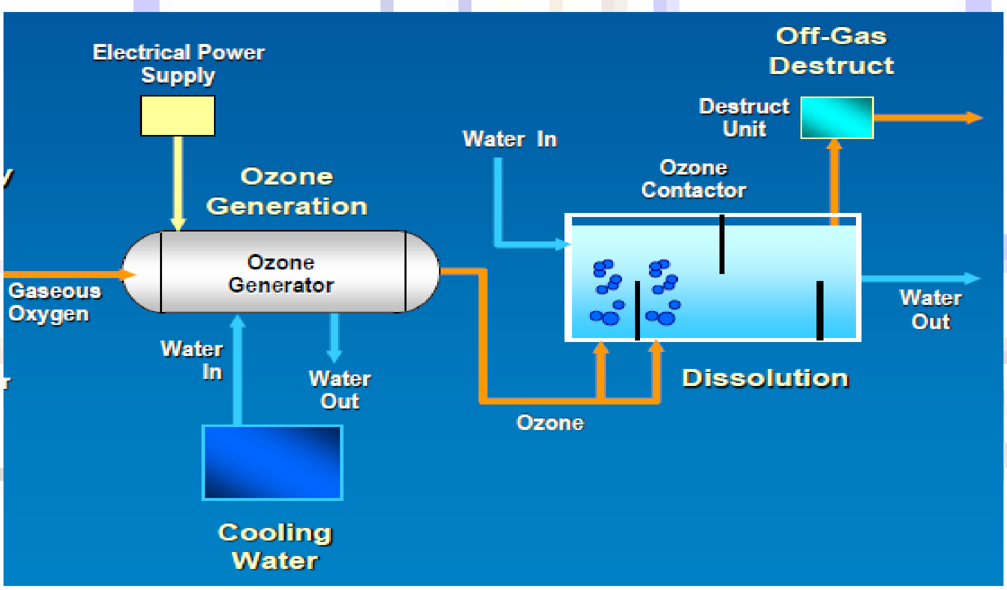
Table: Calculating Contact Time

minutes required = K / chlorine residual (mg/l)			
Highest	Lowest Water Temperature (degrees F)		
pH	≥ 50	45	≤ 40
6.5	4	5	6
7.0	8	10	12
7.5	12	15	18
8.0	16	20	24
8.5	20	25	30
9.0	24	30	36

4.5.2.2. Ozone

It's manufactured by electrical discharge into cooled dried air. Approximately 1% of the atmospheric oxygen is converted to O_3 at an energy consumption of 0.025 kWh per gram O_3. The mixture of air and ozone is transferred into water either by bubbling it through the bulk solution or by permitting droplets of water to fall through a rising column of gas.

After generation, ozone is fed into a contact tank containing the water to be disinfected. From the bottom of the contact tank, ozone is diffused into fine bubbles that mix with the water.



Advantages of ozone

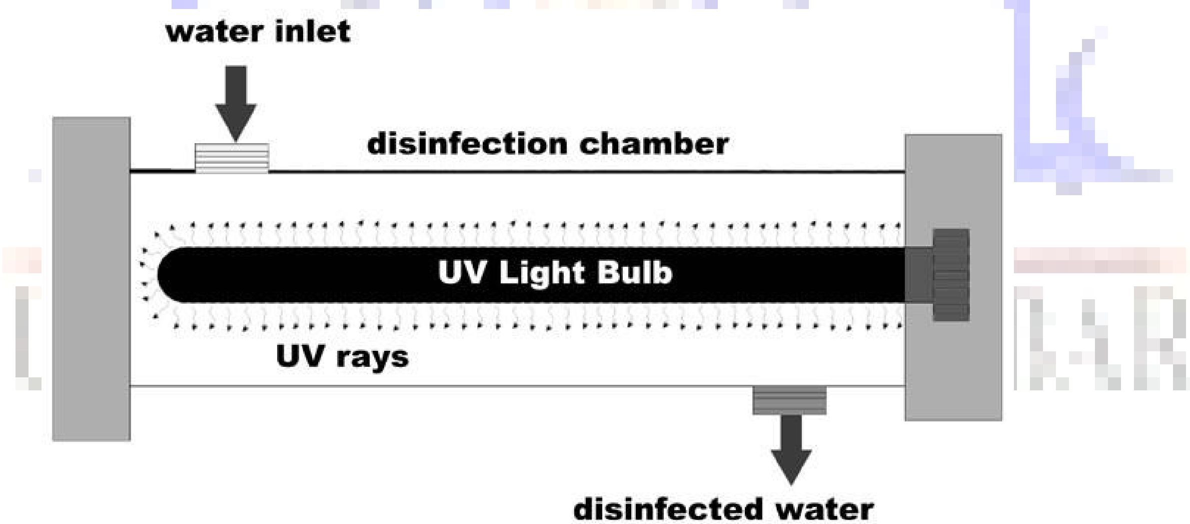
- Ozone is more effective than chlorine in destroying viruses and bacteria.
- The water needs to be in contact with ozone for just a short time (approximately 10 to 30 min.).
- Ozone More rapid in action with small dose (0.25-1.5) mg/L at low contact time (45 sec-2 min). than other oxidizing agents, and therefore, it leaves no harmful residual that would need to be removed from the water after treatment.
- Ozone is generated onsite, and thus, there are fewer safety problems associated with shipping and handling.

Disadvantages of ozone

- Low dosages may not effectively inactivate some viruses, spores, and cysts.
- Ozone is very reactive and corrosive, thus requiring corrosion-resistant material, such as stainless steel.
- Ozone is restricted to use.
- The cost of treatment is relatively high, being both capital- and power intensive
- There is no measurable residual to indicate the efficacy of ozone disinfection.

4.5.2.3. UV Disinfection

UV is invisible light radiation with a wavelength between 200 ~ 300 nanometers, see the figure shown below. Unlike chemical approaches to water disinfection, UV provides rapid, effective inactivation of microorganisms through a physical process. When bacteria, viruses and protozoa are exposed to the germicidal wavelengths of UV light, they became incapable of reproducing and infecting. UV light has demonstrated efficacy against pathogenic organisms, including those responsible for cholera, typhoid, hepatitis and other bacterial, viral and parasitic diseases.



Advantages of UV

- Cheap and effective disinfection.
- Chemical free.
- Relatively simple to install, operate and maintain.
- Inactivates protozoa.
- Minimal concerns over by-products.

Disadvantages of UV

- No disinfectant residual.
- It requires water to have low levels of colour and turbidity.
- It is ineffective if the dose and contact time are not correct.

Example:

Chlorine usage in the treatment of 25000 m^3/day is 9 kg/day. The residual chlorine after 10 minutes' contact is 0.2 mg/L. Calculate the dosage in milligrams per liter and chlorine demand of the water?

Solution

$Water\ treated\ per\ day = 25 \times 10^6\ L/day$

$Chlorine\ consumed\ per\ day = 9 \times 10^6\ mg/day$

$$Chlorine\ used\ per\ liter\ of\ water = \frac{9 \times 10^6}{25 \times 10^6\ \mathrm{L/day}} = 0.36\ mg/L$$

$Chlorine\ demand = 0.36 - 0.2 = 0.16\ mg/L$

Example:

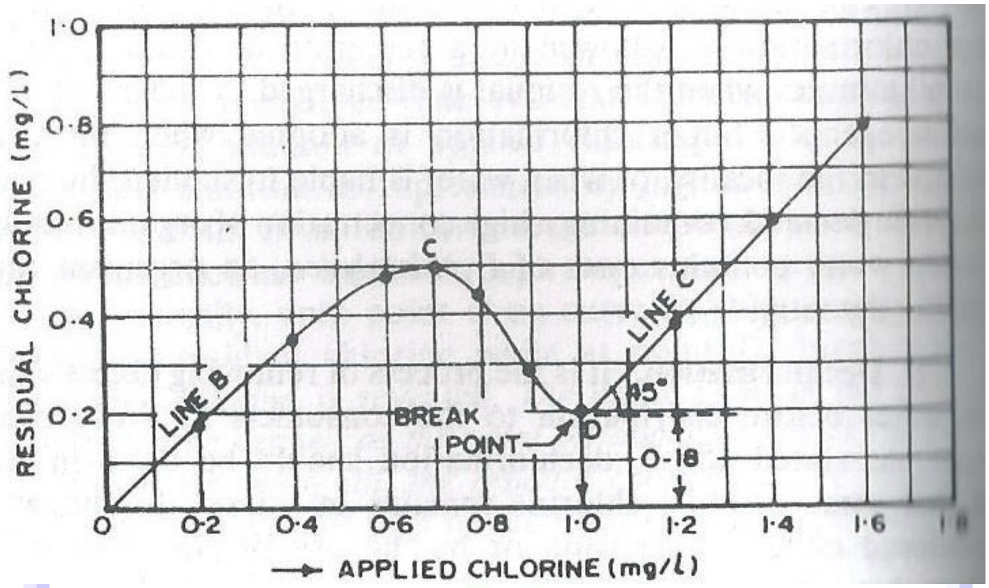
Results of chlorine demand test on a raw water are given below. Determine the break-point dosage and the chlorine demand?

Sample No.	Chlorine Dosage, mg/l	Residual Chlorine after 10min., mg/l
1	0.2	0.18
2	0.4	0.34
3	0.6	0.48
4	0.8	0.46
5	0.9	0.27
6	1	0.18
7	1.2	0.38
8	1.4	0.58
9	1.6	0.78

Solution

Sample No.	Chlorine Dosage, mg/l	Residual Chlorine after 10min., mg/l	Chlorine Demand, mg/l
1	0.2	0.18	0.02
2	0.4	0.34	0.06
3	0.6	0.48	0.12
4	0.8	0.46	0.34
5	0.9	0.27	0.63
6	1	0.18	0.82
7	1.2	0.38	0.82
8	1.4	0.58	0.82
9	1.6	0.78	0.82

Figure bellows the curve plotted the basis of the above data.



From the curve, we find that break point occurs at point D, at which the applied chlorine = 1 mg/L.

So, the break point dosage = 1.0 mg/L

Chlorine demand at break point = 1.0 - 0.18 = 0.82 mg/L

It is observed that since the slope of curve C is 45º, the chlorine demand (=0.82 mg/L) remains constant after break point, since all additional chlorine added after point D appears as free chlorine.

4.5 Storage Tanks

Types of storage used in water supply works:

1 - Ground Storage.

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

2 - Elevated Storage.

(Appears in different positions according to its function)

§ Ground Storage Tank or (Clear Water Tank)

Purpose

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

V_1 (m^3) = (0.5 – 1) hr * Q_{mm} (m^3/hr), Q_{mm} is the maximum monthly flow through one day

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

V_2 (m^3) = (0.25 – 0.4) * Q_{mm} (m^3/d) ***or*** (6 – 8 hr) * Q_{mm} (m^3/hr)

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

V_3 (m^3) = [Q_{md} (m^3/d) – Q_{mm} (m^3/d)] * 1 day , Q_{md} is the maximum daily flow

4. Saves 80% of fire Storage

V_4 (m^3) = 0.8 * Fire requirements

Note:

Specification recommended 60 m^3/hr for one fire on assumption of 2 hours fire for each 10,000 capita

$$\text{Fire demand} = \frac{Population}{10000} \times 60\ m^3/hr \times (1-2)hr\ \ or\ \ = \frac{Population}{10000} \times 120$$

Design Capacity of Ground Reservoir

V (m^3) = take Maximum value of [V_1 or V_2 or V_3] + V_4

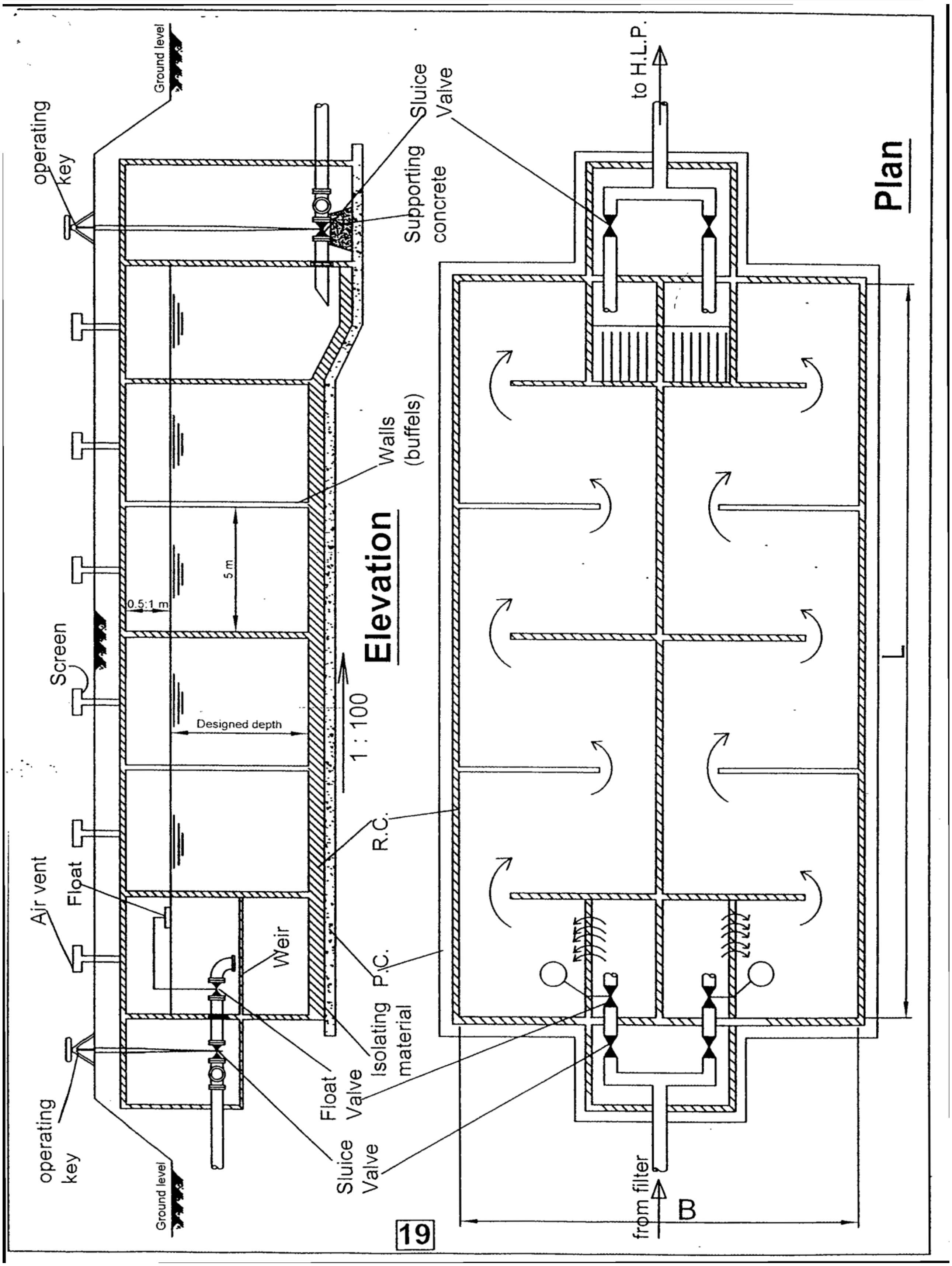
Almost, the ground storage tank is rectangular in shape, where:

L ≤ 50 m, It must be divisible by 10 m

L = 1.2 – 1.5 B

d = 3 – 5 m

Number of tanks n ≥ 2 tanks


operating key
Ground level
Sluice Valve
Supporting concrete
Walls (buffels)
5 m
0.5:1 m
Elevation
Screen
Designed depth
1 : 100
R.C.
Air vent
Float
Weir
P.C.
Isolating material
Float Valve
Sluice Valve
operating key
Ground level
to H.L.P.
Plan
L
B
from filter
19

Example:

It's required to design the ground reservoir of a WTP serves 80,000 capita with average water consumption of 200 l/c/d.

Solution

Calculations of flows

$$Q_{avg.} = \frac{80000\times200}{1000} = 16000\, m^3/day$$

Q_{mm} = 1.4 Qavg = 1.4 x 16000 = 22400 m^3/day

Q_{md} = 1.8 Qavg = 1.8 x 16000 = 28800 m^3/day

Design Capacity

$\mathbf{V_1}$ (m^3) = (0.5 – 1) hr * $\mathbf{Q_{mm}}$ (m^3/hr) = $0.5 \times \frac{22400}{24}$ = 466.67 m^3

$\mathbf{V_2}$ (m^3) = (6 – 8 hr) * $\mathbf{Q_{mm}}$ (m^3/hr) = $6 \times \frac{22400}{24}$ = 5600 m^3

$\mathbf{V_3}$ (m^3) = [$\mathbf{Q_{md}}$ (m^3/d) – $\mathbf{Q_{mm}}$ (m^3/d)] * 1 day = (28800 – 22400) × 1 = <u>6400 m^3</u>

$$V_4 = 0.8 \times \text{Fire demand} = 0.8 \times \frac{Population}{10000} \times 120 = 0.8 \times \frac{80000}{10000} \times 120 = 768\ m^3$$

V (m^3) = take Maximum value of [V_1 or V_2 or V_3] + V_4

Then V= 6400 + 768 = 7168 m^3

Assume d = 5 m ,

n = 2 ,

L = 40 m

V = n x L x w x d   7168= 2 * 40 * W * 5

Then W=18m