MODULE I

ENVIRONMENTAL ENGINEERING- I

ENVIRONMENTAL ENGINEERING

It involves waste water management, air pollution control, recycling, waste disposal, radiation protection, industrial hygiene, animal agriculture, environmental sustainability, public health and environmental engineering law.

THE ROLE OF THE ENVIRONMENTAL ENGINEER

As pollutants enter air, water, or soil, natural processes such as dilution, biological conversions, and chemical reactions convert waste material to more acceptable forms and disperse them through a large volume.

THE ROLE OF THE ENVIRONMENTAL ENGINEER

Engineers adapt the principles of natural mechanisms to engineered systems for pollution control when they construct tall stacks to disperse and dilute air pollutants, design biological treatment facilities for the removal of organics from wastewater, use chemicals to oxidize and precipitate out the iron and manganese in drinking-water supplies, or bury solid wastes in controlled landfill operations.

THE ROLE OF THE ENVIRONMENTAL ENGINEER

The role of environmental engineer is to build a bridge between biology and technology by applying all the techniques made available by modern engineering technology to the job of cleaning up the debris left in the wake of an indiscriminate use of that technology.

ENGINEERING, ETHICS AND THE ENVIRONMENT

The engineer recognizes that the greatest merit is the work and exercises his profession committed to serving society, attending to the welfare and progress of the majority

The engineer should reject any paper that is intended to harm the general interest, thus avoiding a situation that might be hazardous or threatening to the environment, life, health, or other rights of human beings.

ENGINEERING, ETHICS AND THE ENVIRONMENT

It is an inescapable duty of the engineer to uphold the prestige of the profession, to ensure its proper discharge, and to maintain a professional demeanor rooted in ability, honesty, temperance, honesty, and justice; with the consciousness of individual well-being subordinate to the social good.

SOURCES OF WATER SUPPLY

Water is one of the nature’s precious gifts to mankind. All living things consist mostly of water. Water is the most essential component of life and is vital for sustenance.

SOURCES OF WATER SUPPLY

Uses of water include agricultural, industrial, household, recreational and environmental activities. The majority of human uses fresh water. 97% of the water on the Earth is salt water and only three percent is fresh water; slightly over two thirds of this is frozen in glaciers and polar ice caps. The remaining unfrozen fresh water is found mainly as groundwater, with only a small fraction present above ground or in the air.

SELECTION OF SOURCES OF WATER

The following factors should be considered while selecting the site for the sources of water supply scheme:

1. Quantity of water

The quantity of water available at the source must be sufficient to meet various demands and requirements of the design population during the entire design period. Plans should be made to bring water from other sources if the available water is insufficient.

SELECTION OF SOURCES OF WATER

2. Quality of water

The water available at the source must not be toxic, poisonous or in any way injurious to health. The impurities should be as minimum as possible and such that, can be removed easily and economically.

SELECTION OF SOURCES OF WATER

3. Distance of water supply source

The source of supply must be situated as near to the city as possible. Hence, less length of pipes needs to be installed and thus economical transfer and supply of water. The source nearest to the city is usually selected.

SELECTION OF SOURCES OF WATER

4. Topography of city and its surroundings

The area or land between the source and the city should not be highly uneven i.e. it should not have steep slopes because cost of construction or laying or pipes is very high in these areas.

SELECTION OF SOURCES OF WATER

5. Elevation of source of water supply

The source of water must be on a high elevation than the city so as to provide sufficient pressure in the water for daily requirements. When the water is available at lower levels, then pumps are used to pressurise water. This requires an excess developmental and operational tasks and cost. It may also have breakdowns and need repairs.

SOURCES OF WATER

Rain water

"Rainwater" by definition is precipitation that is collected from relatively clean, above-ground surfaces – usually rooftops. Rainwater is free and mostly clean and requires less treatment than greywater, so it is an ideal source of water for harvesting. In commercial buildings, the large rooftop areas can often collect enough rainwater to meet all the non-potable uses like toilet flushing and irrigation. Rainwater harvested from roof top is stored in small underground tank or cistern, for small individual supplies.

SOURCES OF WATER

Surface water

Surface water is any water that collects on the surface of the Earth. Fresh surface water is maintained by rainfall or other precipitation, and it is lost through seepage through the ground, evaporation, or use by plants and animals.

The following are the different surface sources of water:

1. Ponds and lakes
2. Streams and rivers
3. Storage reservoirs

SOURCES OF WATER

Ponds and lake: The natural or artificial depressions where surface run off is collected in rainy season are known as ponds or lakes. Usually they contain shallow water with marsh and aquatic plants and animals. The catchment area of those sources is small and hence the quantity of water is not reliable. But the quality is reliable and it requires little treatment before use. This source is suitable for small water supply schemes.

SOURCES OF WATER

Streams and rivers: Rivers constitute the principal source of water supply. It is a well known fact that most of the cities are settled near the rivers, and it is generally easy to find a river for supplying water to the city. Rivers may be perennial or non perennial. Perennial rivers are those in which the water is available throughout the year. Perennial rivers can be used as sources of public supplies directly. In case of non perennial rivers, the weir or barrage or low dam may be constructed to form a storage reservoir from where water may be drawn by intake works.

SOURCES OF WATER

Storage reservoirs: A water supply scheme drawing water directly from a river or a stream may fail to satisfy the consumers demand during extremely law flows; while during high flows, it may again difficult to carry out its operations due to floods. A barrier in the form of a dam may, therefore, sometimes be constructed across the river, so as to form a pool of water on the upstream side of the barrier.

SOURCES OF WATER

Storage reservoirs Cont…: This artificial lake which is formed by constructing dam across a river valley is termed as storage reservoir. The quality of this reservoir water is not much different from that of a natural lake. The water stored in the reservoir can be used easily not only for water supplies but also for other purposes such as irrigation, hydro electric power generation, fishery, etc.

SOURCES OF WATER

Groundwater

The ground water is the only largest source of fresh water naturally available. It is percolated water through the ground and accumulated in an underground basin formed by the rock. The groundwater is more clear and colourless but harder than surface water. Groundwater is often cheaper, more convenient and less vulnerable to pollution than surface water. Therefore, it is commonly used for public water supplies.

SOURCES OF WATER

Groundwater Cont…

The following are the underground source of water:

1. Springs
2. Infiltration galleries
3. Infiltration wells and
4. Wells and tubes

SOURCES OF WATER

Springs: A spring is any natural situation where water flows from an aquifer to the Earth's surface. It is a component of the hydrosphere. The water of the spring may contain some type of salt or minerals. So, it should be tested before use. This source is suitable for water supply in hilly town.

SOURCES OF WATER

The following are the different types of springs:

a) Artesian Spring: Artesian Springs occur when the groundwater, under pressure, finds its way to the land surface (Fig. 2.1). The spring flows because the pressure in the aquifer, which is covered by a confining layer, is greater than atmospheric pressure at the land. Some artesian springs discharge hot water. Such springs are termed as ‘hot springs’

SOURCES OF WATER

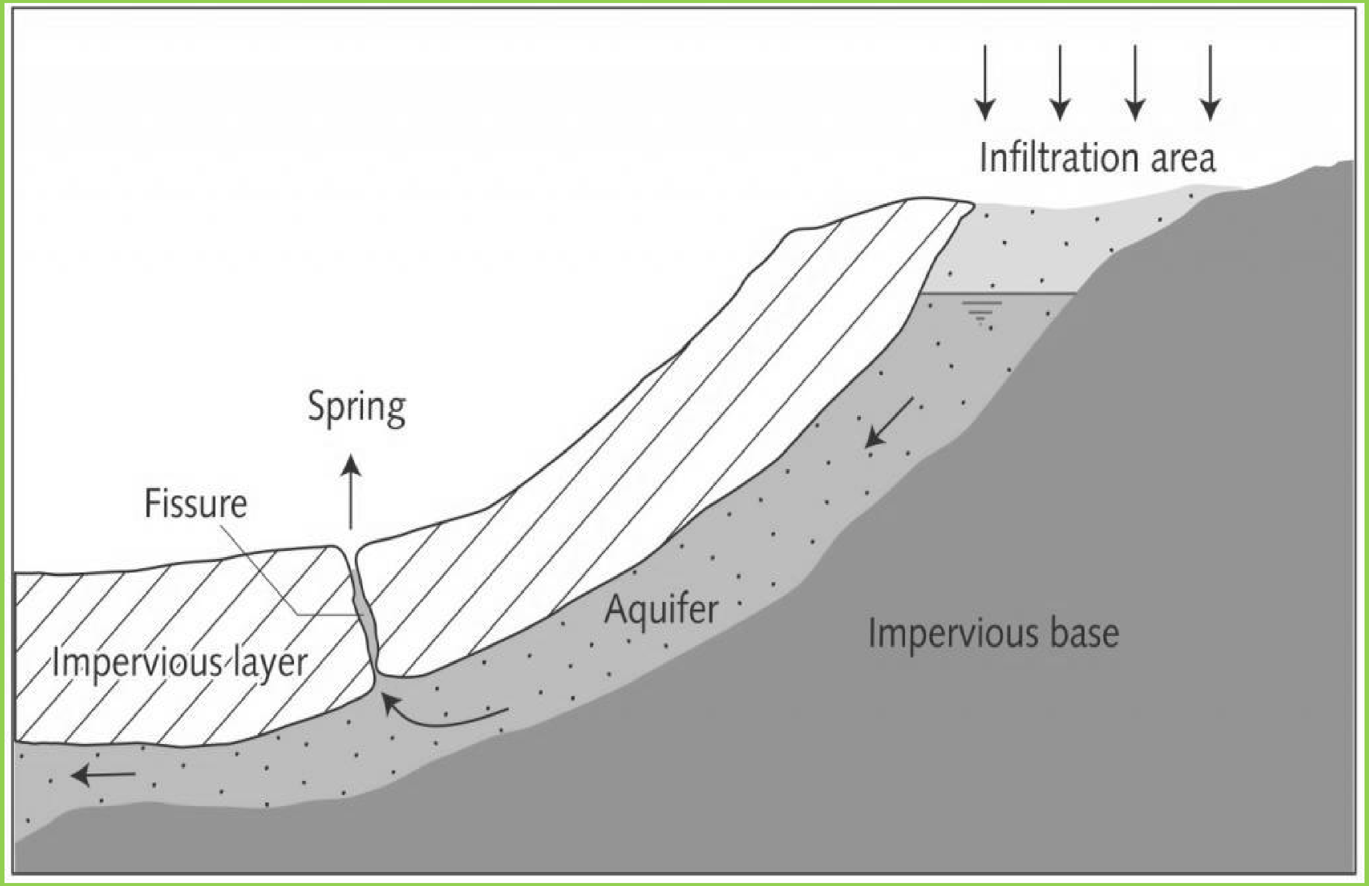


Fig. 2.1 Artesian spring

SOURCES OF WATER

b) Gravity Spring: The gravity spring comes into existence when the water table rises along the hill slope and the water finds a path on the slope through which it rushes out by gravity. The discharge of water from such spring is variable as the water table may rise or fall in different seasons.

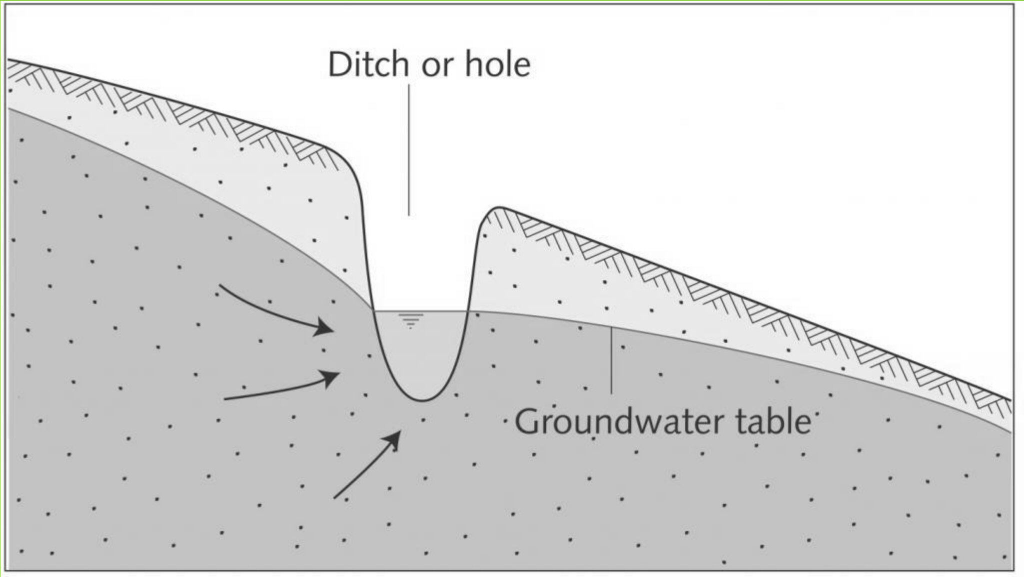


Fig. 2.2 Gravity spring

SOURCES OF WATER

(c)Surface Spring: When subsoil water forms storage due to the presence of impervious layer in the form of a valley, then the surface spring comes into existence Figure 2.3 shows a surface spring. A cut-off wall is constructed on the impervious layer to form a reservoir from where water is supplied to the consumers.

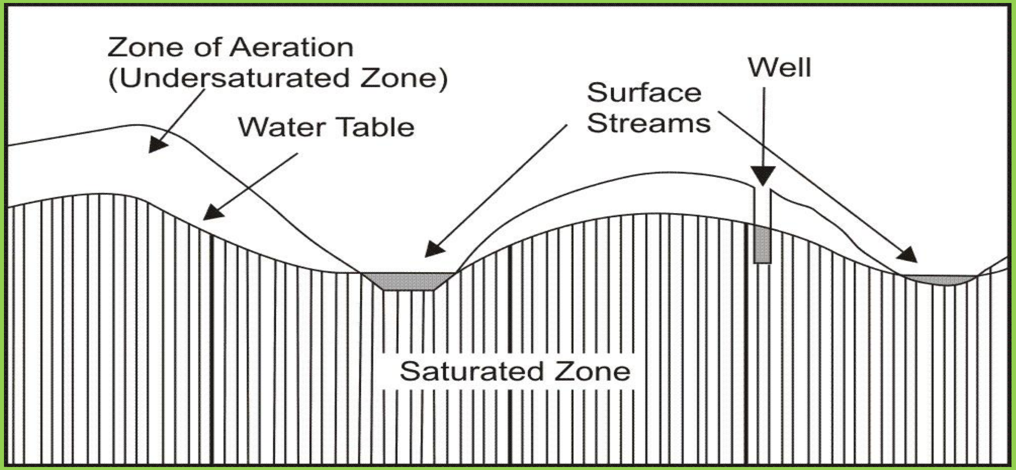


Fig. 2.3 Surface spring

SOURCES OF WATER

ii) Infiltration galleries

For tapping water from sandy river beds, the infiltration wells are sunk in series in sandy river beds. These are constructed or brick masonry with open joints. The water percolates through these joints and gets collected in the wells. The top of the wells are covered with R.C.C. slab having manhole for inspection. Again, the water from the infiltration wells gets collected in a jack well. Then the water from the jack well is pumped out and stored in a storage reservoir. The quality of water is good and it requires no treatment. The quantity of water from this source is suitable for small water supply schemes.

SOURCES OF WATER

Fig. 2.4 Infiltration gallery

ii) Infiltration galleries

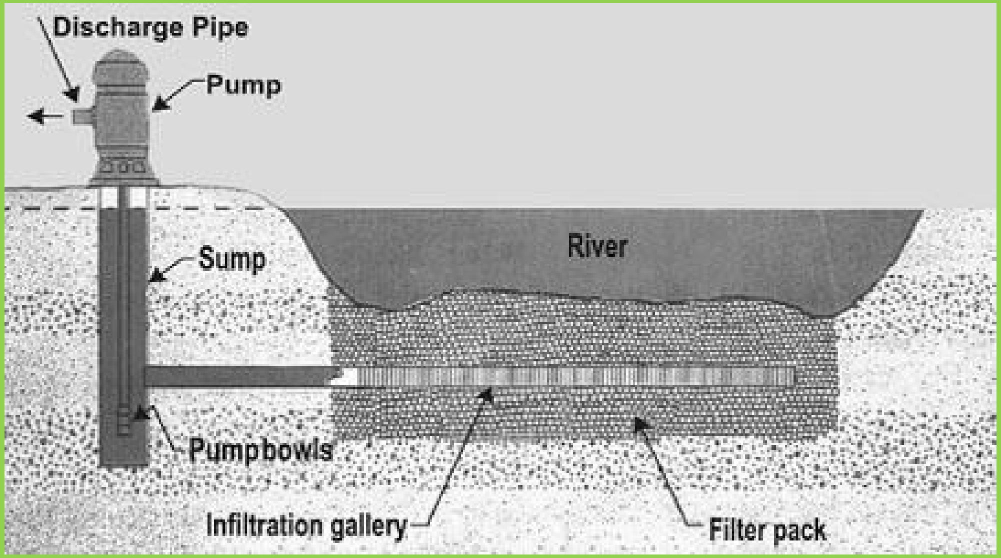
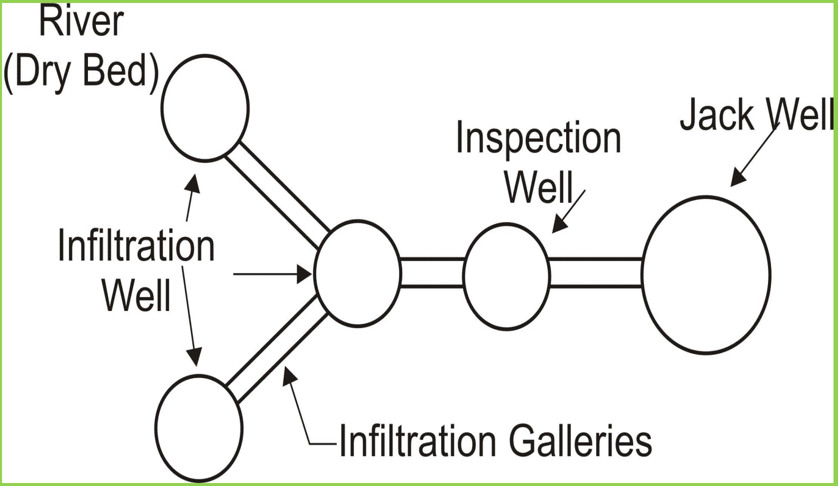


Fig. 2.4 Infiltration gallery

Fig. 2.5 Arrangements of wells and galleries



SOURCES OF WATER

iii) Infiltration wells

For tapping water from sandy river beds, the infilteration wells are sunk in series in sandy river beds. These are brick masonry structures with open joints. The water percolates through these joints and gets collected in the wells. The top of the wells are covered with RCC slab having manhole for inspection. Again, the water from the jack well is pumped out and stored in a storage reservoir. The quality of water is good and it requires no treatment. The quantity of water from this source is suitable for small water supply schemes. The figure 2.5 shows the arrangements of wells and galleries.

SOURCES OF WATER

Wells and tubes

Well is an artificial hole, made in the ground for the purpose of getting ground water. Well may be of two types – open well and tube well.

The open well draws water from the topmost pervious layer. The diameter of this well varies from this from 1m to 3m and the depth varies from 10 m to 20 m depending upon the nature of soil and depth of water table. This type of wells is suitable for meeting the requirements of individual homes.

SOURCES OF WATER

Tube Wells

The tube well draws water from the lower most pervious layer. The diameter and the depth of this well vary from 40 mm to 150 mm and 100 m to 300 m respectively, depending upon the nature of soil and suitable water bearing strata. The tube well is constructed by sinking G.I pipes. The deep tube well is considered as the best source of water for any water supply scheme.

SOURCES OF WATER

4. Water obtained from reclamation

Reclaimed or recycled water is the process of converting wastewater into water that can be reused for other purposes. Reuse may include irrigation of gardens and agricultural fields or replenishing surface water and groundwater (i.e., groundwater recharge). Reused water may also be directed toward fulfilling certain needs in residences (e.g. toilet flushing), businesses, and industry, and could even be treated to reach drinking water standards. This last option is called either "direct potable reuse" or "indirect potable" reuse, depending on the approach used.

WATER DEMAND AND QUANTITY

INTRODUCTION

Whenever an engineer is assigned to design a water supply scheme for a particular city, it becomes imperative upon him, to first of all, evaluate the amount of water available and the amount of water demanded by the public.

WATER DEMAND AND QUANTITY

The total quantity of water required by the community has to be assessed for the design of various components of any water supply scheme. The total requirement of water by the community depends on the following factors:

1. Population of the community.
2. Rate of water supply per capita per day.
3. Design Period.

POPULATION OF THE COMMUNITY

Determination of population is one of the most important factors in the planning, if the project has to serve the community for a certain design period. The design period of a project is normally 20 to 40 years. What will be the population at the end of the design period is the basic question. The probable population at the end of the design period can be forecasted by using various methods.

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1. Population of the community.
2. Rate of water supply per capita per day.
3. Design Period.

RATE OF WATER SUPPLY PER CAPITA PER DAY

It is the amount of water supplied per person per day to satisfy the various demands. It is expressed as liters per capita per day (lpcd).

$$\text{Per capita consumption per day in litres} = \frac{\text{Total consumption in litres in year}}{\text{Population X 365}}$$

Generally the per capita demand values ranges between 10-300 lpcd. These variations in total water consumption of different cities or towns depend upon various factors.

FACTORS AFFECTING PER CAPITA DEMAND

The per capita demand depends on many factors. These factors are to be studied carefully before arriving at the rate of demand for a particular locality. Following are the factors which influence the rate of demand:

Social and economic status of consumers

The people with higher social and economic status have better standards of living and consume more quantity of water for kitchen use, bath, sanitary use, cloth washing, gardening, floor washing, cleaning cars etc. Middle class community consumes moderate amounts, while the poor slum dweller consumes very less quantity. The consumption also depends on the unique habits of the people of the locality.

FACTORS AFFECTING PER CAPITA DEMAND

Climate conditions

The consumption of water is generally more during hot and dry climates because of increased use for bathing, air cooling, drinking, sprinkling lawns, gardens and roofs etc. Similarly, in extremely cold countries, more water may be consumed, because the people may keep their taps open to avoid freezing of pipes, and there may be more leakage from pipe joints, since metals contract with cold.

FACTORS AFFECTING PER CAPITA DEMAND

Industries and commercial activities

The consumption is more in the towns of increased industrial and commercial activities. Many industries require really huge amounts of water, and such, increase the water demand considerably. The industrial water demand is having no direct connection with the population or the size of the city, but more industries are generally situated in big cities, thereby increasing the per capita demand for big cities. For properly planned and zoned cities, the requirements can be assessed more accurately by estimating the industrial and commercial demands separately.

FACTORS AFFECTING PER CAPITA DEMAND

Quality of water supply

If the quality, taste and colour of the supplied water is good, consumption will be higher, as the people tend to use more water and also avoid other sources like private wells, hand pumps etc. Similarly, certain industries such as boiler feed, food, fisheries etc., which require standard quality waters and will use public supplies, provided the supplied water is up to their required standards.

FACTORS AFFECTING PER CAPITA DEMAND

System of supply

The water may be supplied either continuously for all the 24 hours of the day, or may be supplied only for peak periods during the morning and evening. The second system, i.e. the intermittent supplies, may lead to some saving in water consumption due to losses occurring for lesser time and more vigilant use of water by the consumers. But many places, the intermittent supplies may not give much saving over the continuous supplies, because the water stored in tanks, drums, etc. thrown away as soon as the fresh supply is restored and the people have a general tendency to keep the taps open during non-supply hours, so that water goes on flowing unattended even after the supply is restored, thus resulting in wastage of water.

FACTORS AFFECTING PER CAPITA DEMAND

Pressure in distribution system

Consumption increases with the increase in distribution pressure, causing high rate of flow and availability of water to the upper floors. The losses and waste due to leakage are considerably increases if this pressure is high.

FACTORS AFFECTING PER CAPITA DEMAND

Policy of metering

Water supply may be metered or unmetered. In the case of non-metered supply, consumers are charged at some flat rate. In the case of un-metered supply, every consumer thinks that he has to pay a fixed amount only, irrespective of amount water used by him. Therefore, he never cares to close the flowing taps and wastes large quantity of water. In the case of metered supply, because consumer has to pay as per quantity of water supplied to him; he will be more careful and will use only that much amount of water as actually required by him. This way the rate of demand is generally less in the case of metered supply.

FACTORS AFFECTING PER CAPITA DEMAND

Cost of water

The rate at which water is made available to the consumers may also affect the rate of demand. If the water rates are high, lesser quantity may be consumed by the people. This may not lead to large savings as the affluent and rich people are little affected by such policies.

FACTORS AFFECTING PER CAPITA DEMAND

System of sewage collection

The per capita demand is more for the towns provided with water carriage system due to increased use for flushing water closet and urinals etc. Consumption is less in conservancy system.

DESIGN PERIOD

A water supply scheme includes huge and costly structures which cannot be replace or increased in their capacities easily. In order to avoid the difficulties arising out of future expansion, the components of water supply scheme are purposely made larger. Thus the scheme should satisfy the needs of the community for reasonable number of years to come. This future period or number of years for which provision is made including the construction period of the scheme is called “design period.”

The design period should neither be too long or too short, as to make the scheme uneconomical.

VARIOUS TYPES OF WATER DEMAND

When designing the water supply project for a town or city, it is essential to determine the detailed quantity of water required for various purposes by the city. But as there are so many aspects involved in demand of water, it is impossible to precisely figure out the actual demand.

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

1. Domestic water demand;
2. Industrial water demand;
3. Institutional and commercial water demand;
4. Demand for public use;
5. Fire demand; and
6. Water required to compensating losses in wastes and thefts.

Domestic water demand

The amount of water necessary in the residences for drinking, bathing, cooking, washing, flushing of toilets, gardening etc is known as domestic water demand and primarily depends on the habits, social status, weather and traditions of the people. As per Indian Standard: 1172-1963, under normal conditions, the domestic consumption of water in India is about 135 litres/day/capita. But in developed countries this figure between 325 - 340 litres/day/capita because of use of air coolers, air conditioners, maintenance of lawns and automatic household appliances.

Industrial water demand

The water needed in the industries mostly relies on the kind of industries that are established within the town. The water needed by various industries like paper mills, cloth mills, cotton mills, breweries, sugar refineries etc. comes under industrial use. Large industries may use their own source of water. However, if water is required from the public water supply, then the water required depends on the type and size of industry. About 20 – 25% of total water demand is normally considered as industrial water demand.

Institutional and commercial water demand

This type of water demand includes the water requirement for the public buildings other than residences. Commercial buildings, malls, colleges, hotels, cinema houses, bus depots and other similar public buildings comes within this category. Institutional demand includes the needs of schools, offices, hostels etc.

Demand for public use

Volume of water necessary for public utility needs like for washing and sprinkling on roads, cleaning of sewers, watering of public parks, gardens, public fountains etc. comes under public demand. Usually 5 % of total water demand for city is considered for public use while designing water supply scheme.

Fire demand

Water requirement for firefighting purpose fall under this head. The volume of water necessary for firefighting is usually computed by making use of various empirical formulas. For cities having populations exceeding 50,000, the water required in kilo litres may be computed by using the relation.

$$\text{Kilo litres of water required} = 100\sqrt{P}$$

where,

P = population of city in thousands

Fire demand

Thus for a town of 100,000 population, the per capita demand would be

$$Q = 100 \times \sqrt{100} \text{ kilo litres per day}$$

$$= \frac{100 \times 10 \times 1000}{100000} \text{ litres /capita/ day}$$

$$= \mathbf{10} \text{ litres /capita/ day}$$

Water required to compensating losses in wastes and thefts.

There are always losses and wastage occurs in pipeline while water distribution. The main reasons for this are listed below:

1. Damage pipe line and or faulty accessories like valves, fittings etc.
2. Water taps kept open in public or residences causing water wastage.
3. Due to illegal and unauthorized connections.

The loss due to unauthorised connection may be reduced by surveying the areas and detecting them. Wastage can also be caused by the householder who throws away the stored water and collects fresh water when it is released. While calculating the total amount of water of a town; allowance of 12- 15% of total quantity of water is designed to make up for losses, thefts and wastage of water.

FORECASTING POPULATION

The data about the present population of a city under consideration can be obtained from the senses records of the municipality or civic body. Once decided the design period, the population to be served at the end of the period should be forecasted to calculate the water requirements. The population at a future date is determined from the present population and the population of the previous years. There are several methods for population forecast, but the judgment must be exercised by the engineer as to which method is most applicable for a particular location.

FORECASTING POPULATION

The following are some methods by which future population is forecasted:

1. Arithmetical increase method.
2. Geometrical increase method.
3. Incremental increase method.
4. Graphical extension method.
5. Graphical comparison method.
6. Zoning method or master plan method.
7. Ratio and correlation method.
8. Logistic curve method.

Arithmetical increase method.

This method is suitable for large and old city with considerable development. If it is used for small, average or comparatively new cities, it will give lower population estimate than actual value. In this method the average increase in population per decade is calculated from the past census records. This increase is added to the present population to find out the population of the next decade. Thus, it is assumed that the population is increasing at constant rate.

Arithmetical increase method.

Hence, $\frac{dp}{dt} = C$ i.e., rate of change of population with respect to time is constant.

Therefore, Population after n^{th} decade will be $Pn = P + n.C$

Where, Pn is the population after 'n' decades and 'P' is present population.

Arithmetical increase method.

Example: 1 Predict the population for the year 2021, 2031, and 2041 from the following population data.

Year	1961	1971	1981	1991	2001	2011
Population	8,58,550	10,15,685	12,01,593	16,91,582	20,77,840	25,85,895

Arithmetical increase method.

Solution:

Year	Population	Increment
1961	858550	
1971	1015685	157135
1981	1201593	185908
1991	1691582	489989
2001	2077840	386258
2011	2585895	508055
Total		**1727345**

Average increment C = $\frac{1727345}{5}$ = **345469**

Population forecast for year 2021 is,

$P_{2021} = 2585895 + 345469 \times 1 = \mathbf{2931364}$

Similarly, $P_{2031} = 2585895 + 345469 \times 2 = \mathbf{3276833}$

$P_{2041} = 2585895 + 345469 \times 3 = \mathbf{3622302}$

Geometrical increase method(Geometrical progression method)

In this method the percentage increase in population from decade to decade is assumed to remain constant. Geometric mean increase is used to find out the future increment in population. Since this method gives higher values and hence should be applied for a new industrial town at the beginning of development for only few decades.

The population at the end of n^{th} decade 'Pn' can be estimated as:

Geometrical increase method(Geometrical progression method)

The population at the end of n^{th} decade 'Pn' can be estimated as:

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

Where, I_G = geometric mean (%)

P = Present population

n = no. of decades.

Geometrical increase method(Geometrical progression method)

Example 2: Considering data given in example 1 predict the population for the year 2021, 2031, and 2041 using geometrical progression method.

Geometrical increase method(Geometrical progression method)

Solution

Year	Population	Increment	Geometrical increase Rate of growth
1961	858550		
1971	1015685	157135	(157135/858550) = 0.18
1981	1201593	185908	(185908/1015685) = 0.18
1991	1691582	489989	(489989/1201593) = 0.41
2001	2077840	386258	(386258/1691582) = 0.23
2011	2585895	508055	(508055/2077840) = 0.24
Total		**345469**	**1. 24**

Geometrical increase method(Geometrical progression method)

Geometric mean I_G = 1.25/5 = 0.25 i.e., 25%

Population in year 2021 is, $P_{2021} = 2585895 \text{ x } (1+0.25)^1 = \mathbf{3232369}$

Similarly for year 2031 and 2041 can be calculated by,

$$P_{2031} = 2585895 \text{ x } (1+0.25)^2 = \mathbf{4040461}$$

$$P_{2041} = 2585895 \text{ x } (1+0.25)^3 = \mathbf{5050576}$$

Incremental increase method

This method is modification of arithmetical increase method and it is suitable for an average size town under normal condition where the growth rate is found to be in increasing order. While adopting this method the increase in increment is considered for calculating future population. The incremental increase is determined for each decade from the past population and the average value is added to the present population along with the average rate of increase.

Incremental increase method

Hence, population after n^{th} decade is $Pn = P + n\,d + \frac{n(n+1)}{2}\,r$

Where, Pn = Population after n^{th} decade

d = Average increase

r = Incremental increase

Incremental increase method

Example 3: Considering data given in example 1 predict the population for the year 2021, 2031, and 2041 using Incremental increase method.

Year	Population	Increase (d)	Incremental increase (r)
1961	858550		
1971	1015685	157135	
1981	1201593	185908	+28773
1991	1691582	489989	+304081
2001	2077840	386258	-103731
2011	2585895	508055	+121797
Total		**1727345**	**350920**
Average		**345469**	**87730**

Incremental increase method

Population in year 2021 is,

$$P_{2021} = 2585895 + (345469 \text{ x } 1) + \frac{1(1+1)}{2} \text{x } 87730 = \mathbf{3019094}$$

For year 2031 $P_{2031} = 2585895 + (345469 \text{ x } 2) + \frac{2(2+1)}{2} \text{x } 87730$

$= \mathbf{3540023}$

For year 2031$P_{2041} = 2585895 + (345469 \text{ x } 3) + \frac{3(3+1)}{2} \text{x } 87730$

$= \mathbf{4148682}$

Example 4: The following is the population data of a city available from past census records. Determine the population of the city in 2021 by (a) arithmetical increase method (b) geometrical increase method (c) incremental increase method.

Year	1941	1951	1961	1971	1981	1991	2001
Population	12500	17000	27000	42000	58000	68000	74000

Solution:

The computations about increment, % increment and incremental increase per decade are arranged in the table given below:

Year	Population	Increment per decade	% increment per decade	Incremental increase
1941	12500			
1951	17000	4500	36.00	
1961	27000	10000	58.82	+5500
1971	42000	15000	55.56	+5000
1981	58000	16000	38.10	+1000
1991	68000	10000	17.24	-6000
2001	74000	6000	8.82	-4000
Total		**61500**	**214.54**	**+1500**
Average		**10250**	**35.76**	**300**

In the above table, percentage increase for the first decade (1941 to 1951)

$$= \frac{17000-12500}{12500} \text{ X } 100 = \frac{4500}{12500} \text{ X } 100 = 36\%$$

Similarly, % increment for other decades has been calculated.

(a) Arithmetical Increase Method

$$Pn = P + n.C$$

Where, P = Population in 2001 = 74000

$$n = \text{number of decades} = \frac{2021 - 2001}{10} = 2$$

C = average increase per decade = 10250

$$\therefore \quad P_{2021} = 74000 + 2 \text{ X } 10250 = \mathbf{94500}$$

(b) Geometrical Increase Method

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

Where, I_G = geometric mean (%) = 35.76%

P = Present population = 74000

$$n = \text{number of decades} = \frac{2021-2001}{10} = 2$$

$$\therefore \quad P_{2021} = 74000\,(1+35.76/100)^2 = \mathbf{1{,}36{,}388}$$

(c) Incremental Increase Method

Population after n^{th} decade is $P_n = P + n\,d + \frac{n(n+1)}{2}\,r$

Where, Pn = Population after n^{th} decade = P_{2021}

P = Present population = 74000

d = Average increase = 10250 (from table)

r = Incremental increase = 300 (from table)

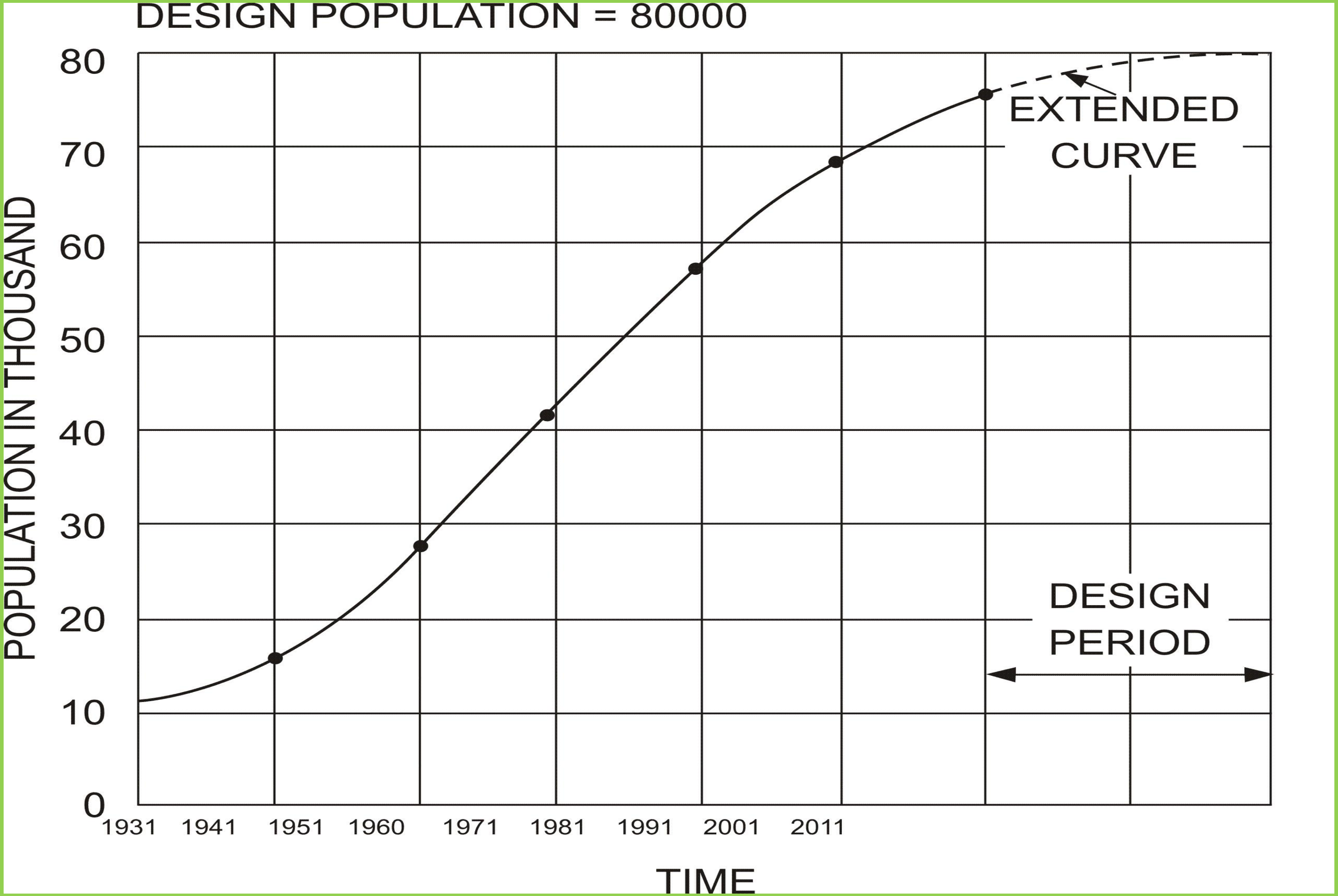
$\therefore \quad P_{2021} = 74000 + 2 \times 10250 + \frac{2(2+1)}{2} \times 300$

= 95,400

Graphical extension method

In this method, the populations of last few decades are correctly plotted to a suitable scale on graph (Figure 3.1). The population curve is smoothly extended for getting future population. From the extended part of the curve, the population at the end of any future decade is approximately determined. This extension should be done carefully and it requires proper experience and judgment.

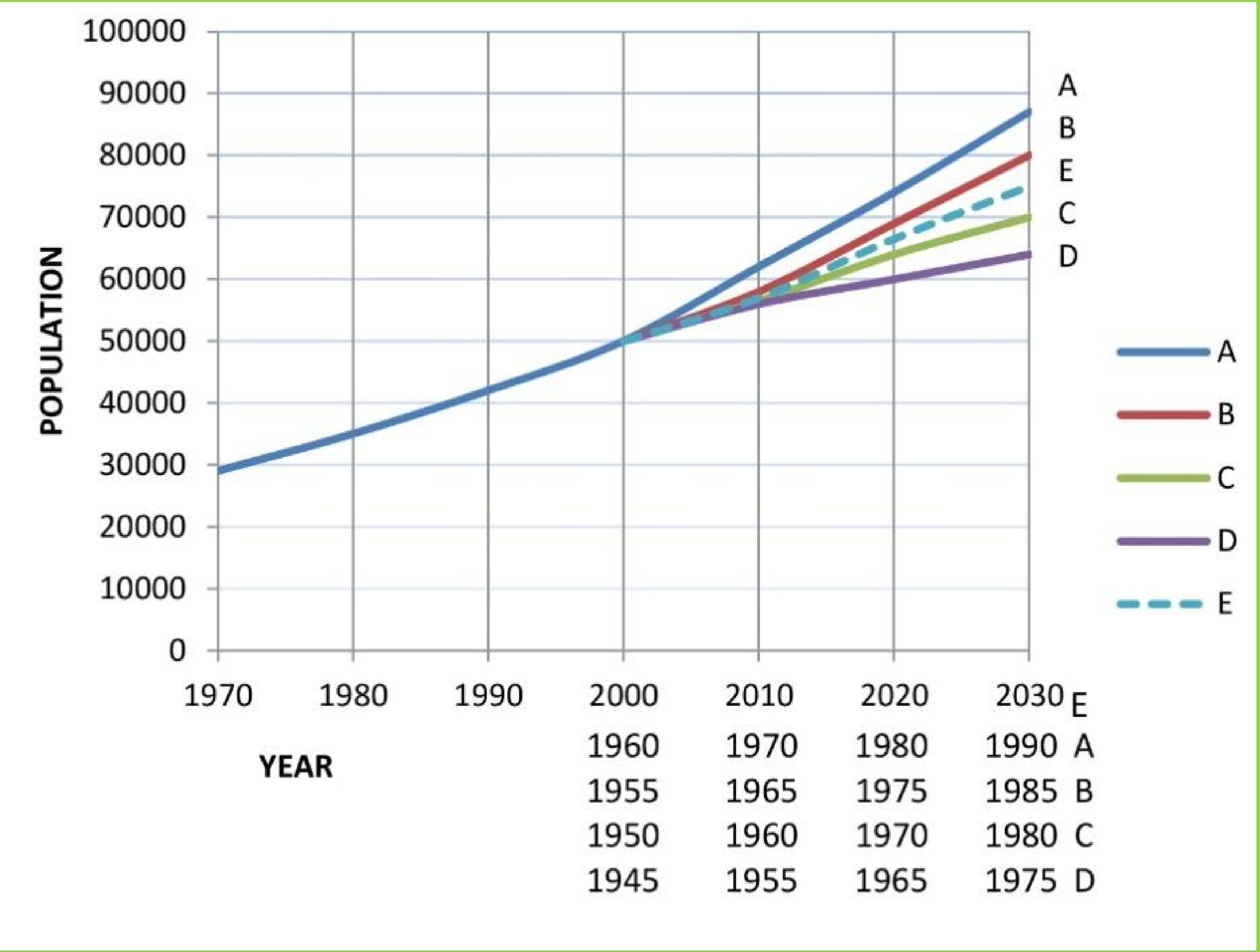
Graphical extension method



Graphical comparison method.

In this method the census populations of cities already developed under similar conditions are plotted. The curve of past population of the city under consideration is plotted on the same graph. The curve is extended carefully by comparing with the population curve of some similar cities having the similar condition of growth. The advantage of this method is that the future population can be predicted from the present population even in the absence of some of the past census report. The use of this method is explained by a suitable example given below.

Graphical comparison method.



Graphical comparison method.

Thus, as shown in Fig. 3.2, the population of city *E* under consideration is plotted up to 2000 at which the population is 50,000. The city *A* having similar conditions, reached the population of 50,000 in 1960 and its curve is plotted from 1960 onwards. Similar curves are plotted for other cities *B*, *C* and *D* which reached the population of 50,000 in 1955, 1950 and 1945 respectively. The curve of city *E* can be then continued (shown dotted line), allowing it to be influenced by the rate of growth of the larger cities. In practice however, it is difficult to find identical cities with respect to population growth.

Zoning method or master plan method.

The big and metropolitan cities are generally not developed in haphazard manner, but are planned and regulated by local bodies according to master plan. The master plan is prepared for next 25 to 30 years for the city. According to the master plan the city is divided into various zones such as residence, commerce and industry. The population densities are fixed for various zones in the master plan. From this population density total water demand and wastewater generation for that zone can be worked out. By this method it is very easy to assess precisely the design population.

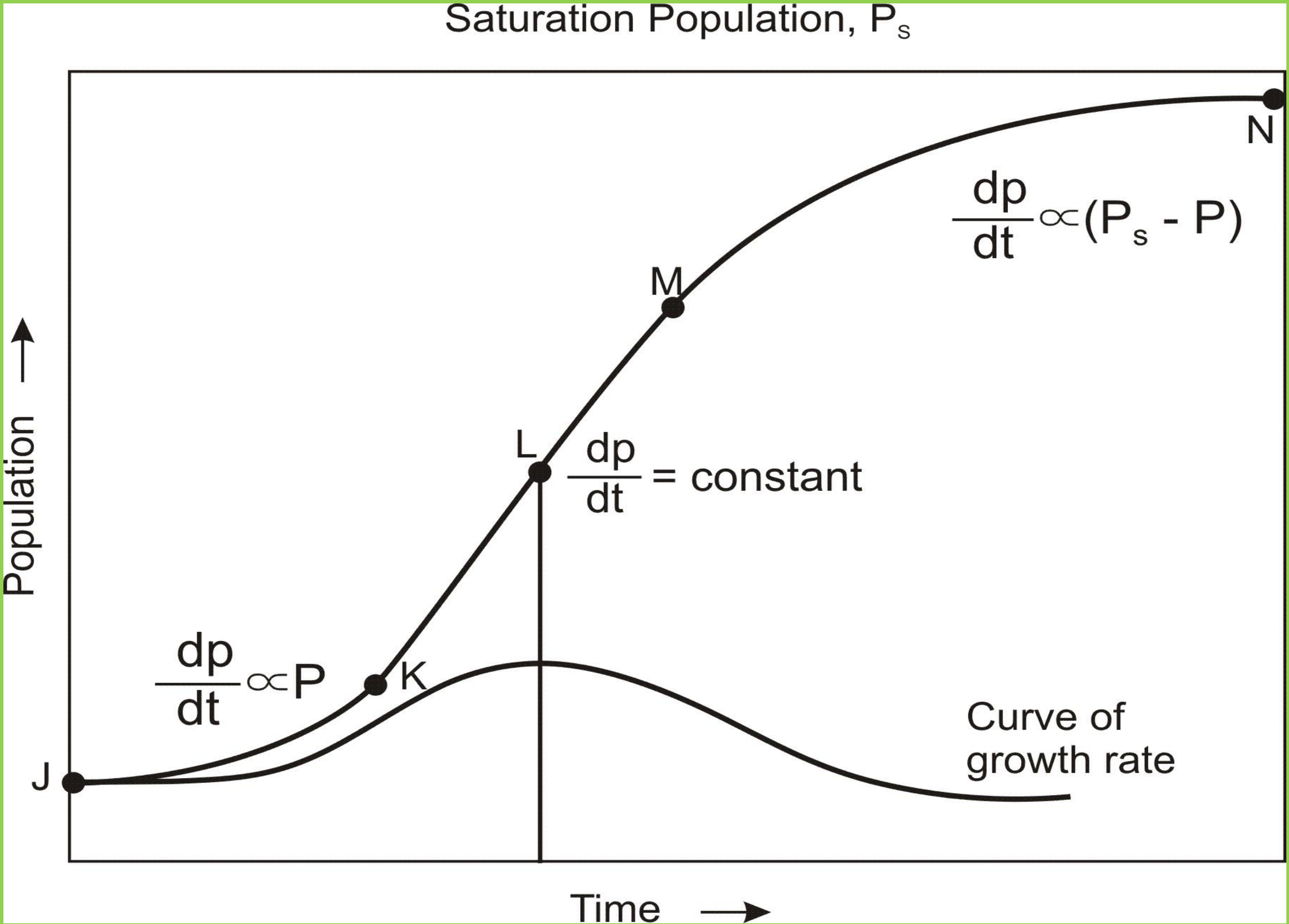
Ratio and correlation method.

The population growth of a small town or area is related to big towns or big areas. The increases in population of big cities bear a direct relationship to the population of the whole state or country. In this method, the local to national population ratio is determined in the previous two or decades. Depending upon conditions or other factors, even changing ratio may be adopted. These ratios may be used in predicting the future population. This method takes into account the regional and national factors affecting population growth. This method is useful for only those areas whose population growth in the past is fairly consistent with that of state or nation.

Logistic curve method.

This method is used when the growth rate of population due to births, deaths and migrations takes place under normal situation and it is not subjected to any extraordinary changes like epidemic, war, earth quake or any natural disaster, etc., and the population follows the growth curve characteristics of living things within limited space and economic opportunity. If the population of a city is plotted with respect to time, the curve so obtained under normal condition looks like S-shaped curve and is known as logistic curve (Figure 3.3).

Logistic curve method.



Logistic curve method.

In Figure 3.3, the curve shows an early growth JK at an increasing rate i.e. geometric growth or log growth, $\frac{dp}{dt}$ α P,the transitional middle curve KM follows arithmetic increase i.e. $\frac{dp}{dt}$ = constant. For later growth MN the rate of change of population is proportional to difference between saturation population and existing population, i.e. $\frac{dp}{dt} \propto (P_s - P)$. P.F. Verhulst has put forward a mathematical solution for this logistic curve JN, which can be represented by an autocatalytic first order equation, is given by

$$\log_e\left(\frac{Ps-P}{P}\right) - \log_e\left(\frac{P_s-P_0}{P_0}\right) = -K.P_s.t \qquad (4)$$

where,

P = Population at any time t from the origin J

P_s = Saturation population

P_0 = Population of the city at the start point J

K = Constant

t = Time in years

Logistic curve method.

From the above equation we get,

$$\log_e\left[\frac{Ps-p}{P}\right] \times \left[\frac{P_0}{P_s-P_0}\right] = -K.Ps.t$$

$$\left[\frac{Ps-P}{P}\right] \times \left[\frac{P_0}{P_s-P_0}\right] = \log_e{}^{-1} (-K.P_s.t)$$

$$\left[\frac{Ps-P}{P}\right] = \left[\frac{P_s-P_0}{P_0}\right] \log_e{}^{-1} (-K.P_s.t)$$

$$\frac{Ps}{P} - 1 = \left[\frac{P_s-P_0}{P_0}\right] \log_e{}^{-1} (-K.P_s.t)$$

$$\frac{Ps}{P} = 1+\left[\frac{P_s-P_0}{P_0}\right] \log_e{}^{-1} (-K.P_s.t)$$

$$P = \frac{Ps}{1+\left[\frac{P_s-P_0}{P_0}\right] \log_e{}^{-1} (-K.Ps.t)}$$

Substituting $\left[\frac{P_s-P_0}{P_0}\right] = \mathrm{m}$ and $-K.Ps = \mathrm{n}$, where m and n are constants, we get

$$P = \frac{Ps}{1+m \log_e{}^{-1} (n.t)}$$

Logistic curve method.

Mc. Lean further suggested that if three pairs of characteristic values P_0, P_1 and P_2 at time $t=t_0$, $t=t_1$ and $t=t_2=2t$, are selected from the useful range of census population data, the values of Ps, m and n can be found from the following simultaneous equations:

$$Ps = \frac{2P_0P_1P_2 - P_1^2(P_0+P_2)}{P_0P_2 - P_1^2}$$

$$m = \frac{P_s - P_0}{P_0}$$

$$n = \frac{1}{t_1}\log_e\left[\frac{P_0(P_s - P_1)}{P_1(P_s - P_0)}\right]$$

$$n = \frac{2.3}{t_1}\log_{10}\left[\frac{P_0(Ps - P_1)}{P_1(Ps - P_0)}\right]$$

Logistic curve method.

Example: 5

The population of a city in three consecutive years i.e. 1991, 2001 and 2011 is 80,000; 250,000 and 480,000, respectively. Determine (a) The saturation population, (b) The equation of logistic curve, (c) The expected population in 2021.

Solution

It is given that

$P_0 = 80{,}000 \quad t_0 = 0$

$P_1 = 250{,}000 \quad t_1 = 10$ years

$P_2 = 480{,}000 \quad t_2 = 20$ years

The saturation population can be calculated by using equation

$$Ps = \frac{2P_0P_1P_2 - P_1^{\,2}(P_0+P_2)}{P_0P_2 - P_1^{\,2}}$$

Logistic curve method.

$$Ps = \frac{2X80{,}000\ X250{,}000X480{,}000 - 250{,}000^2 X\ (80{,}000+480{,}000)}{80{,}000X480{,}000-250{,}000^2}$$

$$= \mathbf{655{,}602}$$

We have, $m = \frac{Ps-P_0}{P_0} = \frac{655{,}000-80{,}000}{80{,}000} = \mathbf{7.195}$

$$n = \frac{2.3}{t_1} log_{10}\left[\frac{P0(Ps-P_1)}{P1(Ps-P_0)}\right]$$

$$n = \frac{2.3}{10} log_{10}\left[\frac{80{,}000(655{,}602-250{,}000)}{250{,}000(655{,}602-80{,}000)}\right]$$

n= **-0.1488**

Hence, logistic curve equation is , $P = \frac{655{,}602}{1+7.195\ log_e{}^{-1}\ (-0.1488.t)}$

Time t =2021-1991=30 years

Logistic curve method.

Population in 2021,

$$P_{2021} = \frac{655{,}602}{1+7.195\ log_e{}^{-1}\ (-0.1488 X\ 30)}$$

$$= \frac{655{,}602}{1+7.195 \text{ X } 0.0117} = \mathbf{605{,}436}$$

Logistic curve method.

Example: 5

In two periods of each of 20 years, a city has grown from 30,000 to 170,000 and then to 300,000. Determine (a) The saturation population (b) The equation of the logistic curve (c) The expected population after the next 20 years.

Solution

It is given that

$P_0 = 30{,}000 \quad t_0 = 0$

$P_1 = 170{,}000 \quad t_1 = 20$ years

$P_2 = 300{,}000 \quad t_2 = 40$ years

Logistic curve method.

i. The saturation population can be calculated by using equation

$$Ps = \frac{2P_0P_1P_2 - P_1^2(P_0+P_2)}{P_0P_2-P_1^2}$$

$$Ps = \frac{2X30,000\ X170,000X300,000\ -170,000^2X\ (30,000+300,000)}{30,000X300,000-170,000^2}$$

$$= \mathbf{326,000}$$

$$\text{We have, } m = \frac{Ps-P_0}{P_0} = \frac{326,000-30,000}{30,000} = \mathbf{9.87}$$

$$n = \frac{2.3}{t_1}\log_{10}\left[\frac{P0(Ps-P_1)}{P1(Ps-P_0)}\right]$$

$$n = \frac{2.3}{20}\log_{10}\left[\frac{30,000(326,000-170,000)}{170,000(326000-30,000)}\right]$$

$n = \mathbf{-0.119}$

Logistic curve method.

b) Equation of logistic curve

$$\text{Logistic curve equation, } P = \frac{326{,}000}{1+9.87\,\log_e^{-1}(-0.119.t)}$$

$$\text{c) Population after 20 years, } P_{60} = \frac{326{,}000}{1+9.87\,\log_e^{-1}(-0.119X\,60)}$$

$$= \frac{326000}{1+9.87\text{ X }0.000794} = \mathbf{324{,}000}$$

FACTORS AFFECTING POPULATION GROWTH

Population growth is defined as increase in the number of people that reside in a country, state, or city. The population growth of a city depends up on following factors.

- **Economic factors:** Development of new industries, discovery of oil or other minerals in the vicinity of the city will affect growth of population.
- **Development programmes:** Development of projects of national importance such as river valley projects etc. will affect the population growth.

FACTORS AFFECTING POPULATION GROWTH

- **Social facilities:** Educational, medical, recreational and other social facilities.
- **Communication links:** Connection of the town with other big cities, and also to the mandies of agricultural products.
- **Tourism:** Tourist facilities, religious places or historical buildings etc.
- **Community life:** Living habits, social customs, and general education in the community.
- **Unforeseen factors:** Earthquakes, floods, epidemics, frequent famines etc. may affect growth of population.

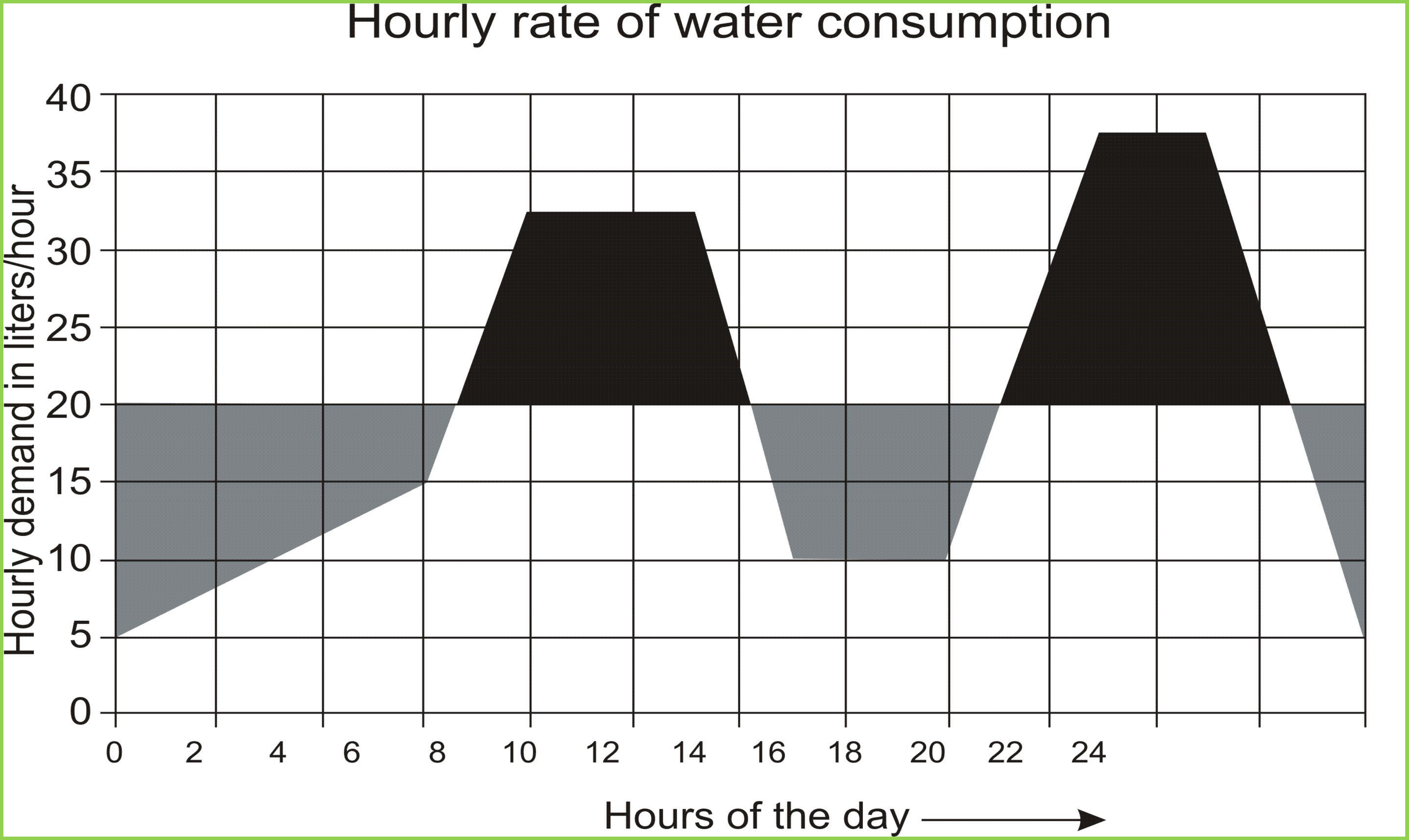
VARIATION IN RATE OF DEMAND

It is observed that the consumption per capita varies from season to season, month to month, day to day as well as hour to hour. The peak demand, ie the maximum consumption in an hour or in a day depends on the habits of the people, climate conditions, the presence of industry, type of industry, and the mode or hours of supply of water by the authorities. The peak seasonal consumption occurs in summer when more water will be used for bathing, watering lawns etc. The peak monthly consumption occurs in summer months.

VARIATION IN RATE OF DEMAND

The maximum daily consumption may occur on Sunday when more water for washing and bathing is used. A typical peak hourly demand graph is shown in figure 3.4. It may be noted that the peak demand is during 7 to 9 am on week days and steadily falls and the minimum is reached between 11 am to 1.00 pm and again reaches the peak between 7 to 9 pm. The minimum demand on the water system is from 9 pm to 5 am.

VARIATION IN RATE OF DEMAND



VARIATION IN RATE OF DEMAND

So, an adequate quantity of water must be available to meet the peak demand. To meet all the fluctuations, the supply pipes, service reservoirs and distribution pipes must be properly proportioned. The water is supplied by pumping directly and the pumps and distribution system must be designed to meet the peak demand.

VARIATION IN RATE OF DEMAND

The effect of monthly variation influences the design of storage reservoirs and the hourly variations influences the design of pumps and service reservoirs. As the population decreases, the fluctuation rate increases. The rate of demand of the peak hour of the peak day of the maximum demand season is called the absolute maximum hourly demand. The average consumption is found by dividing the annual consumption by the number of days in a year.

Maximum daily demand = 1.8 X average daily demand.

MODULE II

INTAKE AND CONVEYANCE

Intake structure constructed at the entrance of the conduit and thereby helping in protecting the conduit from being damaged or clogged by ice, trash, debris, etc., can vary from a simple concrete block supporting the end of the conduit pipe to huge concrete towers housing intake gates, screens, pumps, etc. and even sometimes, living quarters and shops for operating personnel.

INTAKE AND CONVEYANCE

Intake structures are used for collecting water from the surface sources such as river, lake, and reservoir and conveying it further to the water treatment plant. These structures are masonry or concrete structures and provides relatively clean water, free from pollution, sand and objectionable floating material.

SITE SELECTION OF INTAKE STRUCTURES

There are certain factors which affects the site selection of intakes. The following points should be considered while selecting the site for intake works:

1. The intake point should satisfy the condition for the availability water throughout the year.
2. The water at the site should be more or less clear so that excessive treatment is avoided.

SITE SELECTION OF INTAKE STRUCTURES

3. The site should not be selected at the zone of heavy current which may damage the structure of intake works.
4. The intake should be constructed in the upstream side.
5. The intake should never be located in the curves in river.
6. The intake should never be constructed near the navigation channel.
7. The intake should be constructed such that it is accessible during flood.

SITE SELECTION OF INTAKE STRUCTURES

8. The site must be well connected by good approach of roads.
9. The site should not be at the zone of the river where pollution of water is suspected.
10. The site should be selected as near as possible to the treatment plant to reduce the conveyance cost.

TYPES OF INTAKE STRUCTURES

Intake works may be classified under different categories depending upon the available source of surface water.

Category 1

Submerged intake: It is the one which is constructed entirely under water. It is commonly used to obtain supply from a lake.

Exposed intake: It is in the form of a well or tower constructed near the bank of a river, or in some cases even away from the river banks. Exposed intakes are more common due to ease in operation.

TYPES OF INTAKE STRUCTURES

Category 2

Wet intake: It is a type of intake tower in which the water level is practically the same as the level of the sources of supply. It is sometimes known as Jack well and is most commonly used.

Dry intake: In case of dry intake there is no water in the water tower. Water enters through entry port directly into the conveying pipes. It is simply used for the operation of valves etc.

TYPES OF INTAKE STRUCTURES

Category 3

In this category intake structures are classified as follows:

1. River intake
2. Reservoir intake
3. Lake intake
4. Canal intake

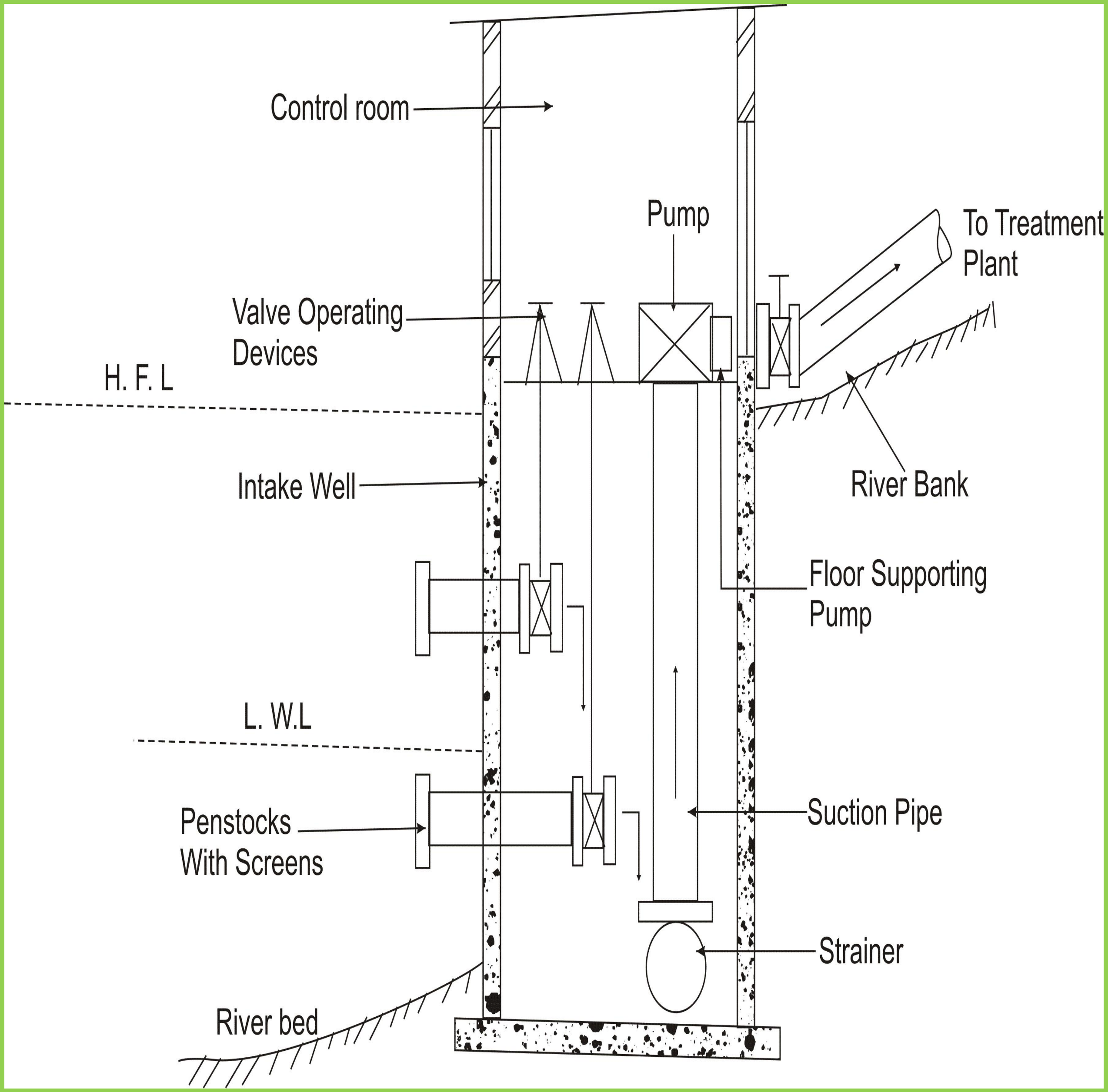
River intake

A circular or rectangular sump well is constructed with masonry work in the bank of the river in such a way that the water can enter the well in both the conditions such as high flood level (HFL) and lowest water level (LWL). It is provided with two or more large pipes called penstocks. These pipes are provided with valves by which the water entering the intake is regulated. The penstocks installed at different levels.

River intake

Screens are provided at the end of the pipes to eliminate suspended matters. The main suction pipe having strainer at bottom is inserted into the sump well as shown in Fig. 4.1. The water collected in the well is pumped out and sent to the treatment plant.

River intake



River intake

The site condition will guide the type of structure to be constructed. Sometimes intake works may be taken to protect the structure from silting and heavy current of the river.

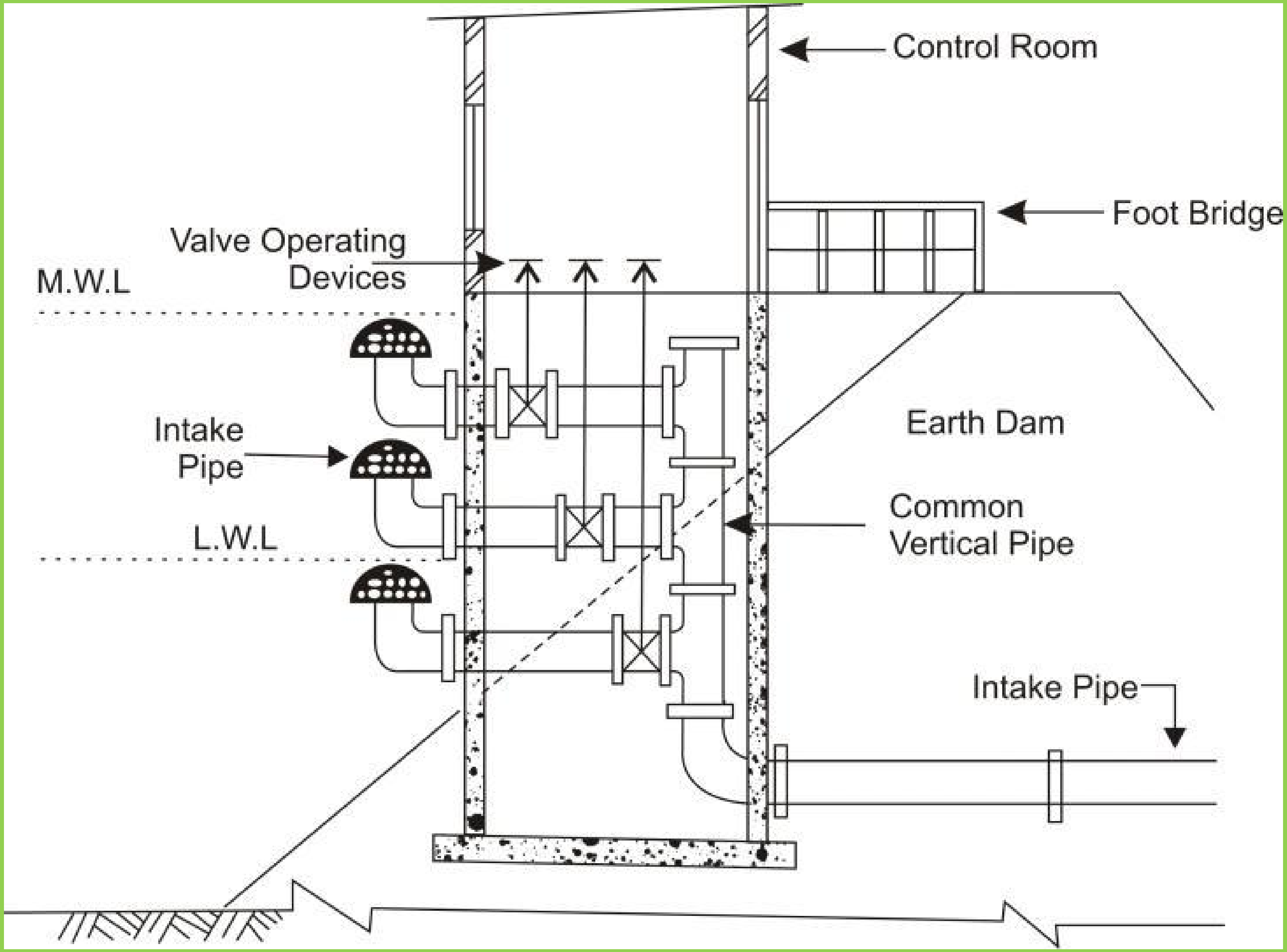
Reservoir intake

The reservoir intake consists of a circular or rectangular well in an earthen dam; the intake consists of a concrete tower located on the upstream side of the dam at the deep portion. In a masonry or concrete dam the intake located in the body of the dam itself.

Reservoir intake

Inlet pipes are inserted in different levels of the well for drawing water. Valves are provided to regulate the flow through the pipes. Screens are provided over the pipes to prevent floating matters entering the intake pipes as shown in Fig. 4.2. The intake pipe is extended below the lowest water level of the canal and it carries a hemispherical screen at the end. The floor of the intake is kept below the lowest water level of the reservoir so that water will be available even in dry season.

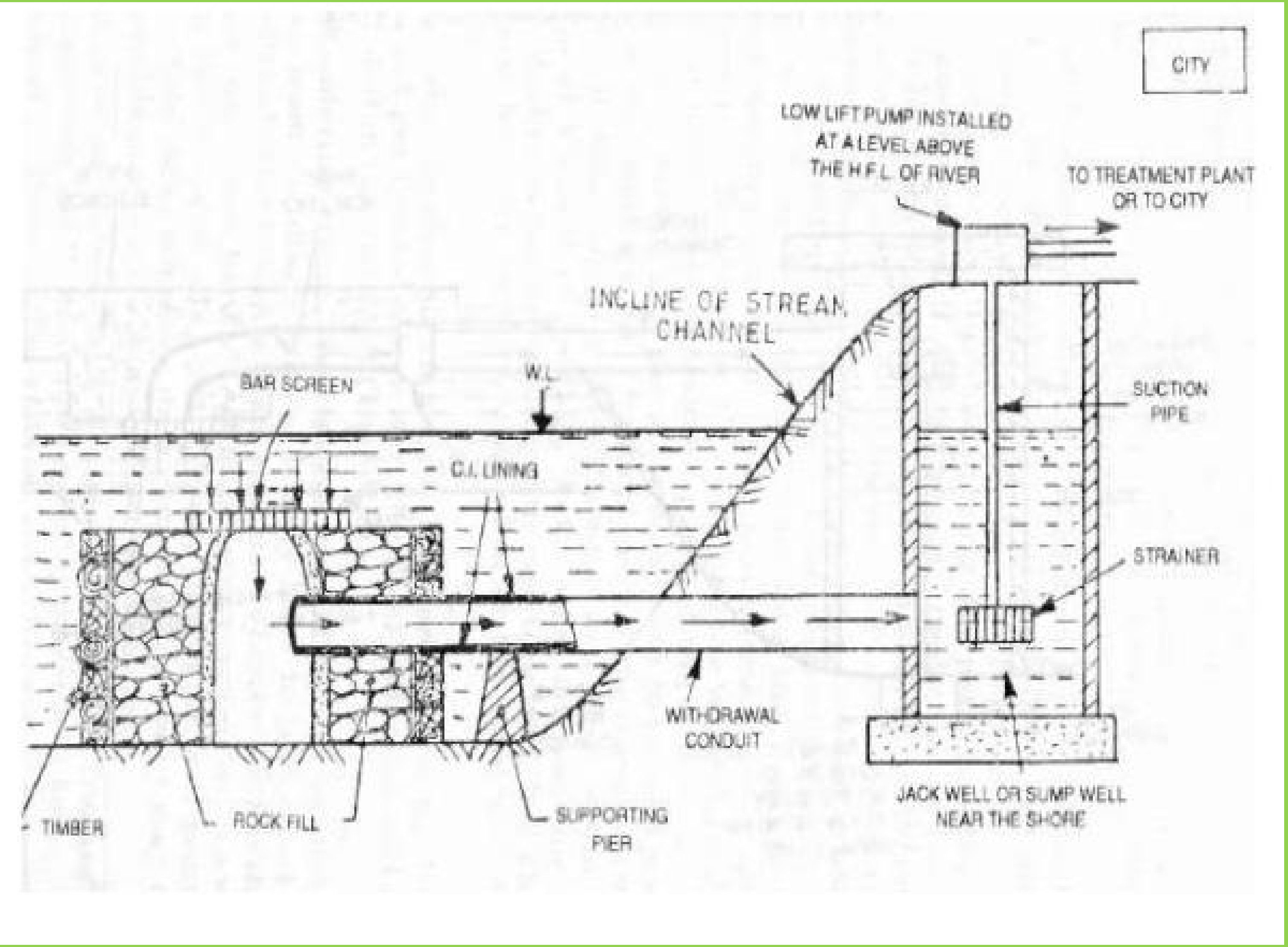
Reservoir intake



Lake intake

A submersible rectangular chamber is constructed at the bed of the lake from where water can be available throughout the year. The top cover of the chamber consists of several holes having gratings on it to prevent the entry of debris, weeds, aquatic lives, etc. into the chamber. A bell mouthed pipe is connected to the pumping unit through the suction pipe as shown in Fig. 4.3. The whole arrangement is protected by stone rip-rap.

Lake intake

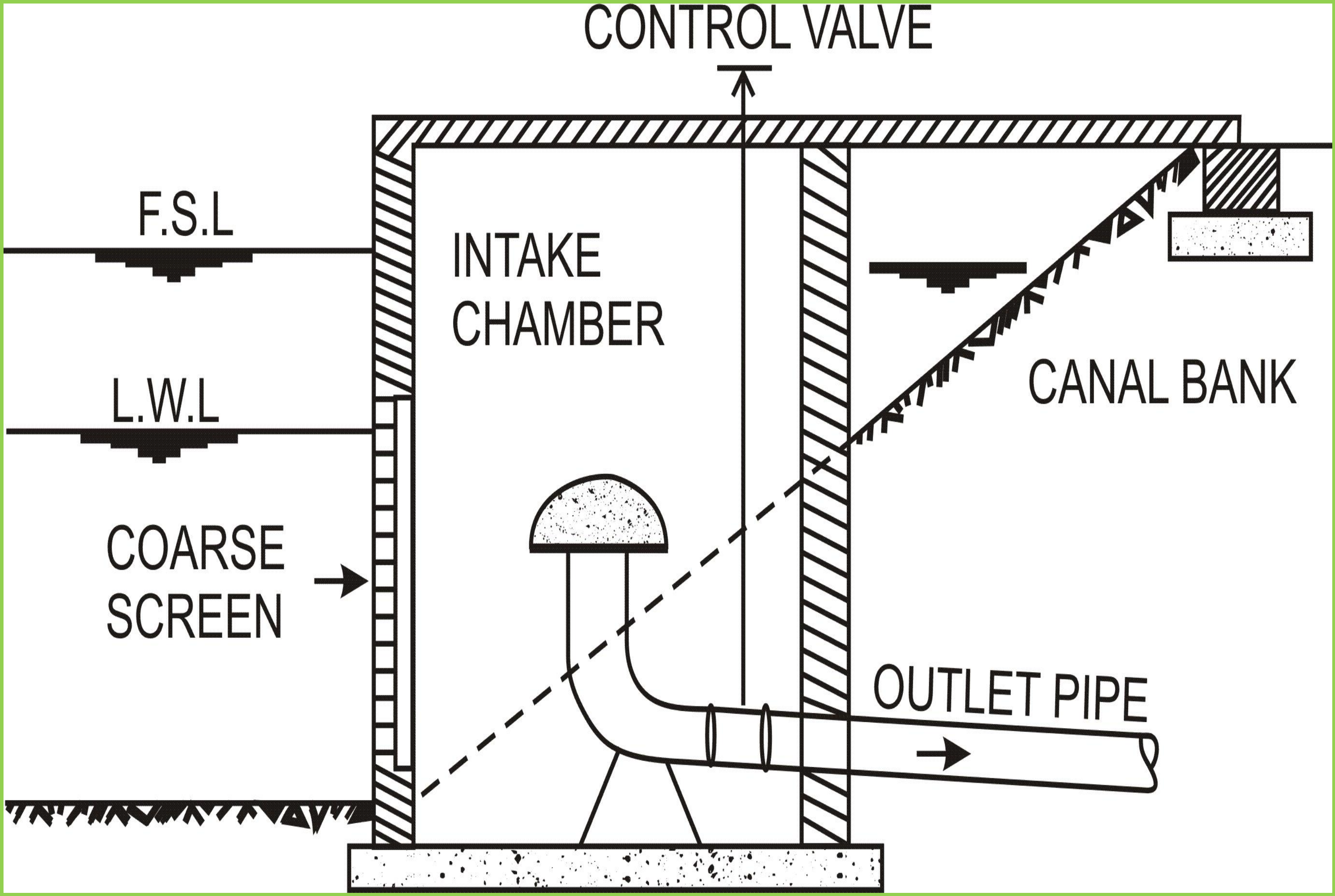


CITY
LOW LIFT PUMP INSTALLED
AT A LEVEL ABOVE
THE H.F.L. OF RIVER
TO TREATMENT PLANT
OR TO CITY
INCLINE OF STREAM
CHANNEL
BAR SCREEN
W.L.
SUCTION
PIPE
C.I. LINING
STRAINER
WITHDRAWAL
CONDUIT
JACK WELL OR SUMP WELL
NEAR THE SHORE
TIMBER
ROCK FILL
SUPPORTING
PIER

Canal intake

A canal intake is a circular or rectangular structure constructed with masonry or concrete by the bank of the canal. An intake pipe is inserted into the well drawing water. On the canal side, the well consists of an opening with screen as shown in Fig. 4.4. The intake pipe is extended below the lowest water level of the canal and it carries a bell mouth screen at the end. A manhole is provided on the well cap for inspection work. The intake pipe is connected to the pumping unit for sending water to the water treatment plant.

Canal intake



Canal intake

Example: 1 Design a bell mouth canal intake for a city of 80,000 persons drawing water from a canal which runs for 10 hours a day with a depth of 1.8 m. Also calculate the head loss in the intake conduit if the treatment works are 0.5 km away. Draw a neat sketch of the canal intake. Assume average consumption per person is 150 liters/day. Assume the velocity through the screens and bell mouth to be less than 16cm/Sec. and 32 cm/Sec. respectively.

Canal intake

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Data given

Population	=	80,000
Canal Running time	=	10 hours
Depth of water	=	1.8 m
Length of pipe	=	0.5 m
Water consumption per person	=	150 lpcd
Velocity through the screen	=	16 cm/sec = 0.16 m/sec
Velocity through the bell mouth	=	32 cm/sec = 0.32 m/sec

Canal intake

Solution

i) Discharge through intake

Daily discharge $= 150 \times 80{,}000$

$= 12 \times 10^6 = 12$ MLD

Since the canal runs only for 10 hours per day

$$\text{Intake load / hour} = \frac{12 X 10^6}{10 X 1000} = 1200 \text{ m}^3 \text{ /Hr}$$

$$\therefore Q = \frac{1200}{60 X 60} = 0.3333 \text{ m}^3\text{/Sec.}$$

ii) Area of course screen in front of intake

$$\text{Area of screen } A_s = \frac{Q}{V} = \frac{0.3333}{0.16} = 2.083 \text{ m}^2$$

Let the area occupied by solid bars be 30% of the total area

$$\text{Actual area of screen} = \frac{2.083}{(1-0.3)} = 2.98 \text{ m}^2$$

Let as assume minimum water level at 0.3 m below normal water level. Also, let as keep bottom of screen at 0.2 m above canal bed level.

Canal intake

$\therefore$Available height of screen = 1.8-0.3- 0.2 = 1.3 m

Required length = $\dfrac{2.98}{1.3}$ = 2.29 m

Hence provide length = 2.3 m

Hence provide screen of size 1.3 x 2.3 m

iii) Design of bell mouth entry

Area of bell mouth, $A_b = \dfrac{0.3333}{0.32} = 1.042 \text{ m}^2$

$$\text{Dia}D_b = \sqrt{\frac{1.042 \, X \, 4}{\pi}} = 1.15 \text{ m}$$

Hence provide bell mouth of 1.2 m diameter

iv) Design of intake conduit

Let as assume a velocity of 1.5 m/sec in the conduit.

Area of intake conduit $A_p = \frac{0.3333}{1.5} = 0.222\ m^2$

Dia. of intake conduit, $D = \sqrt{\frac{0.222 X 4}{\pi}} = 0.532\ m$

However, provide 0.5 m diameter conduit, so that actual flow velocity is

$$V = \frac{0.3333}{\frac{\pi X (0.5)^2}{4}} = 1.7\ m/sec.$$

For head loss through the conduit, consider Hazen – Williams formula

$$V = 0.85\ C_H\ R^{0.63}\ S^{0.54}$$

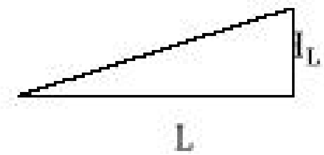
Take coefficient of hydraulic capacity for cast iron pipe $C_H = 130$

Hydraulic mean depth $R = \frac{A}{P} = \frac{D}{4} = \frac{0.5}{4} = 0.125\ m$

Hence slope of the energy line is given by,

$$1.7 = 0.85 \times 130 \times (0.125)^{0.63}\ S^{0.54}$$

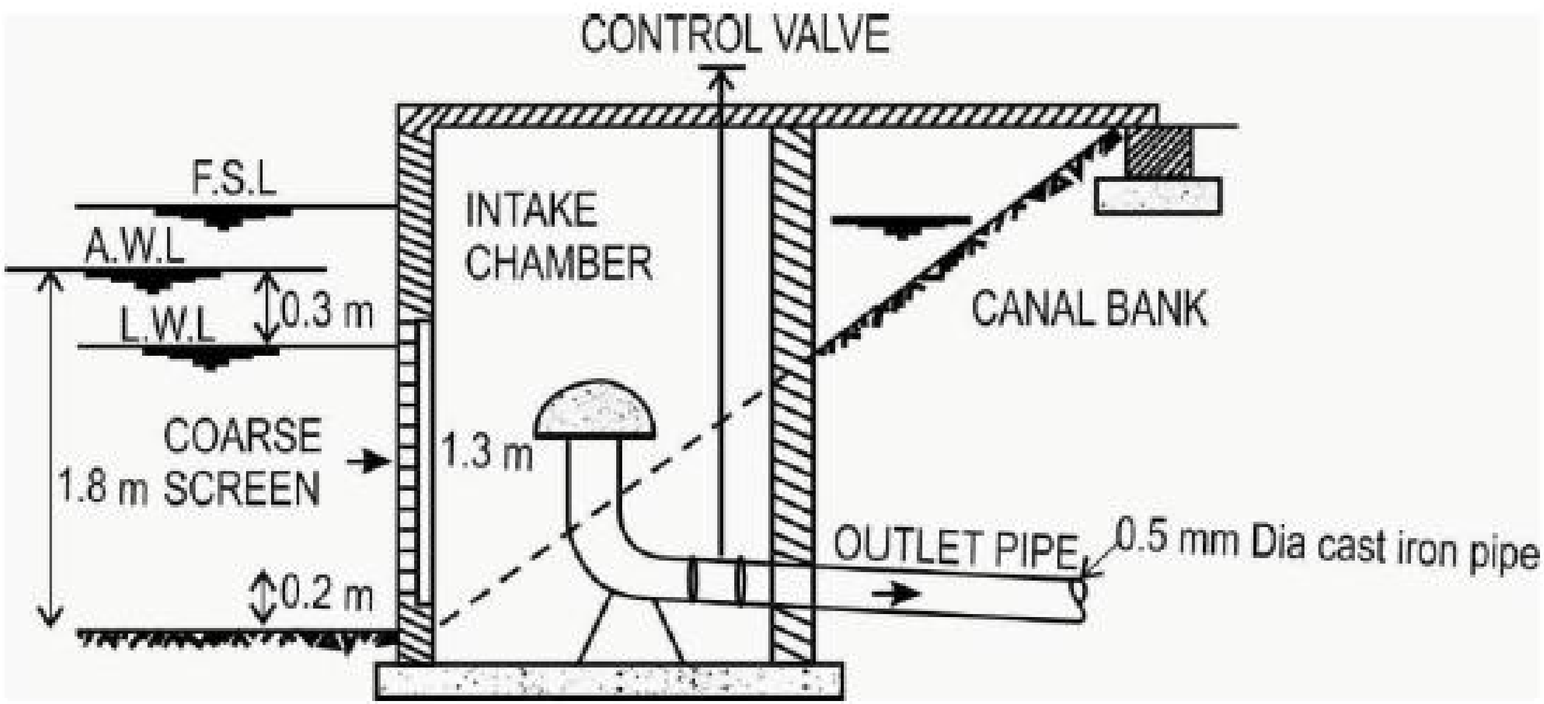
$$\therefore S = \left(\frac{1.7}{29.81}\right)^{\frac{1}{0.54}} = 4.97 \text{ X } 10^{-3}$$



$$\text{But, slope } S = \frac{H_L}{L}$$

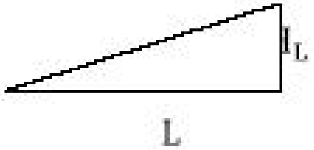
$$\text{There for } H_L = 4.97 \text{ X } 10^{-3} \text{X}[\,0.5 \text{ X } 1000]$$

$$= 2.49 \text{ m}$$

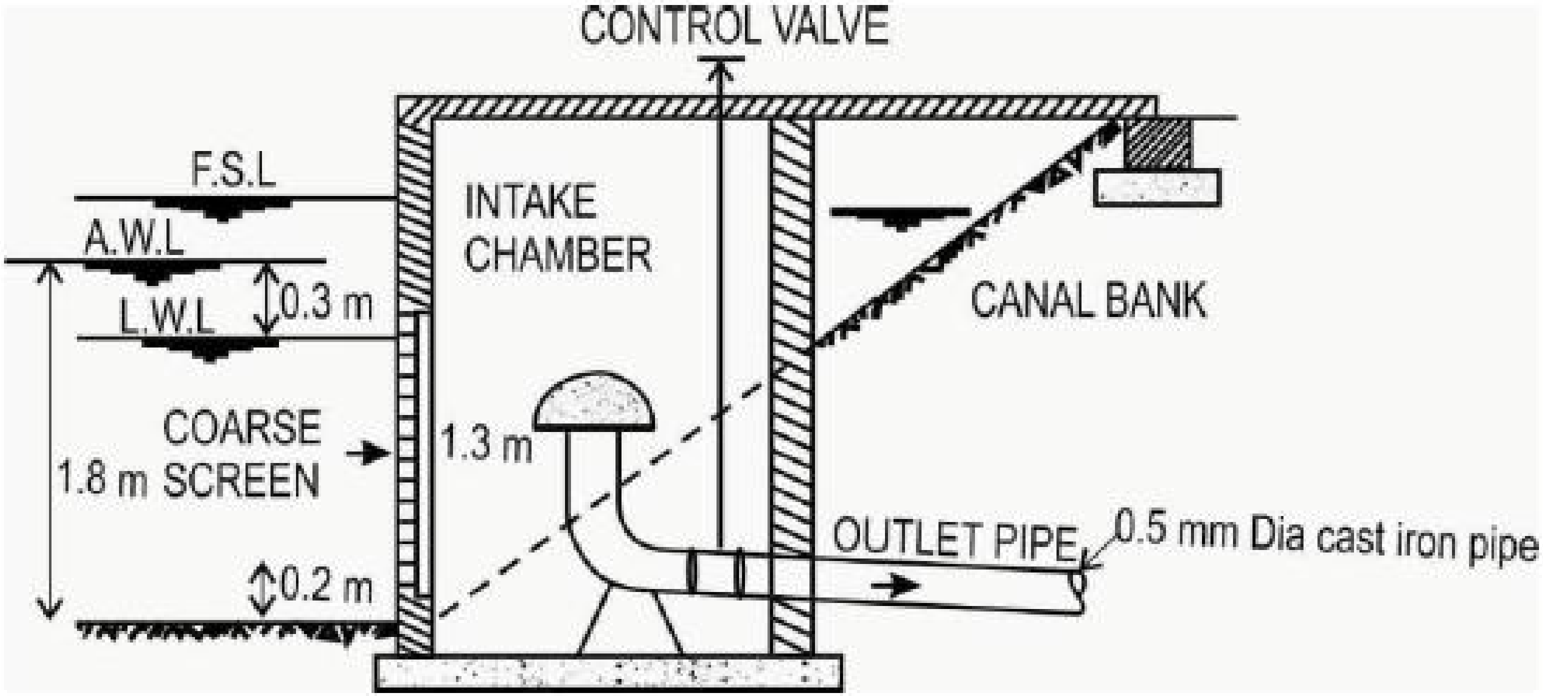


$$\therefore S = \left(\frac{1.7}{29.81}\right)^{\frac{1}{0.54}} = 4.97 \text{ X } 10^{-3}$$

$$\text{But, slope } S = \frac{H_L}{L}$$



$$\text{There for } H_L = 4.97 \text{ X } 10^{-3} \text{X[} 0.5 \text{ X } 1000]$$
$$= 2.49 \text{ m}$$



PUMPS AND PUMPING STATIONS

A pump is a device which converts mechanical energy into hydraulic energy. It lifts water from a lower level to a higher level and delivers it at high pressure.

In most of the cases, pumping is required to lift the water from a river, lake or reservoir to the treatment plant. After the treatment, again water may be lifted either to supply water directly into the mains or to the overhead storage tanks from where water may flow under gravity. In some cases, direct pumping may be necessary to increase pressure in the supply line. When wells are the source of supply, pumping may be necessary to lift the water from the wells.

NECESSITY OF PUMPING

The application of pumping unit becomes necessary under the following circumstances:

a) It is not always possible to carry water by gravity from surface source to the treatment plant. So, pumping unit becomes necessary at this position.

b) It becomes extremely necessary for drawing water from underground source.

c) It becomes necessary to lift the treated water to overhead tanks or reservoir.

d) It becomes necessary to boost the line pressure.

e) It becomes necessary to supply water under pressure for fire hydrants.

f) Pumping is necessary for the miscellaneous operations in the water treatment plants.

NECESSITY OF PUMPING

5.3 CLASSIFICATION OF PUMPS

Pumps can be classified on the basis of the following:

(a) Mechanical principle of operation
(b) Type of power required, and
(c) Type of service called for

NECESSITY OF PUMPING

(a)Classification of pumps based on mechanical principle of operation

Based on the mechanical principle of operation, pumps can be broadly classified into the following four types:

(i) Displacement pumps.
(ii) Centrifugal pumps.
(iii) Air lift pumps.
(iv) Miscellaneous pumps.

(b) Classification of pumps based on type of power required:

(i) Steam engine pumps.
(ii) Diesel engine pumps.
(iii) Electrically driven pumps.

NECESSITY OF PUMPING

(b) Classification of pumps based on type of service called for:

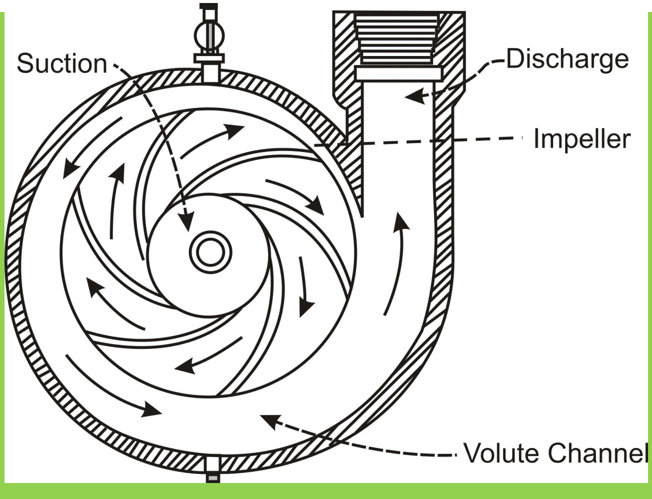
(i) Low lift pumps.
(ii) High lift pumps.
(iii) Deep well pumps.
(iv) Booster pumps.

CENTRIFUGAL PUMP

Centrifugal pumps are the most commonly used kinetic-energy pump. Centrifugal force pushes the liquid outward from the eye of the impeller where it enters the casing. When the water in the chamber of a pump is rotated vigorously by the impeller about the central point, a centrifugal force develops which moves the water towards the periphery of the chamber. Thus, the velocity head is converted to pressure head and these head forces the water through the delivery pipe.

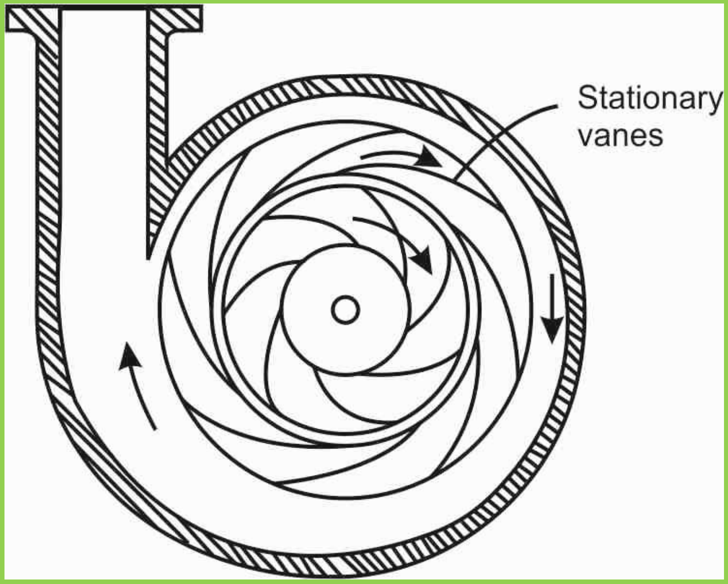
Volute Type

In this type, the chamber is spiral shaped and consists of impellers which are rotated by motor. The suction takes through the centre of the impeller ring. When the impellers rotate, the water from the centre is forced towards the periphery of the chamber as shown in Fig. 5.1. The velocity of flow in the chamber is remaining uniform. The velocity head is thus converted to pressure head which causes the water to flow through the delivery pipe.



Turbine Type

In this type, a diffusion ring is provided between the impellers and the casing. The ring carries fixed diffusers through which the water forces out towards the periphery. In this case also the velocity head which causes the water to flow through the delivery pipe.



Advantages of Centrifugal pump

1. Centrifugal pump speed is high, can be directly connected with the motor and steam turbine.
2. Transmission mechanism is simple and compact.
3. Easy to operate, easy to adjust and repair, and easy to automate and remote operation.
4. It requires minimum space for installation as it compact in design.
5. Simple design, few moving parts, long service life.
6. Flow rate easily adjustable via valve at the outlet of the pump or via rotational speed.
7. Centrifugal pump can handles all types of fluids.

Disadvantages of centrifugal pumps

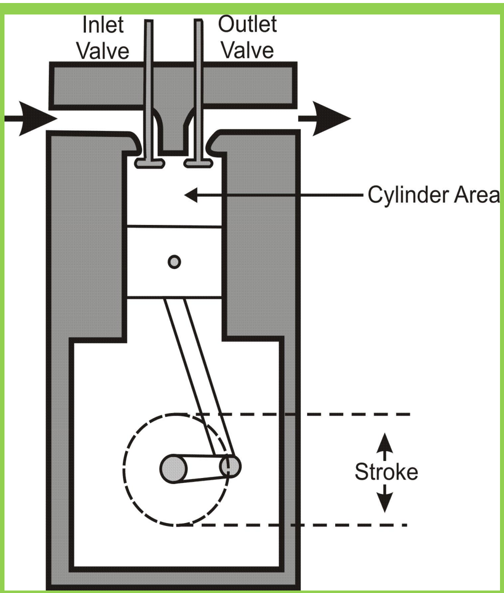
1. The pump will not work, if the chamber is not full of water. So, the priming should always be done before starting the pump.
2. Risk of cavitations with warm water or low intake pressures.
3. The pump will not work if there is any leakage in the suction side.

RECIPROCATING PUMP

The reciprocating pump is a positive plunger pump. It is also known as positive displacement pump or piston pump. It is often used where relatively small quantity is to be handled and the delivery pressure is quite large. The construction of these pumps is similar to the four stroke engine as shown in figure 5.3. The crank is driven by some external rotating motor. The piston of pump reciprocates due to crank rotation. The piston moves down in one half of crank rotation, the inlet valve opens and fluid enters into the cylinder.

RECIPROCATING PUMP

In second half crank rotation the piston moves up, the outlet valve opens and the fluid moves out from the outlet. At a time, only one valve is opened and another is closed so there is no fluid leakage. Depending on the area of cylinder the pump delivers constant volume of fluid in each cycle independent to the pressure at the output port.



RECIPROCATING PUMP

Advantages of reciprocating pump

1. Reciprocating pump having high efficiency.
2. No priming needed for reciprocating pump.
3. Reciprocating pump can deliver water at high pressure.
4. Reciprocating pump can work in wide pressure range.
5. It gives a constant discharge under variable heads.

RECIPROCATING PUMP

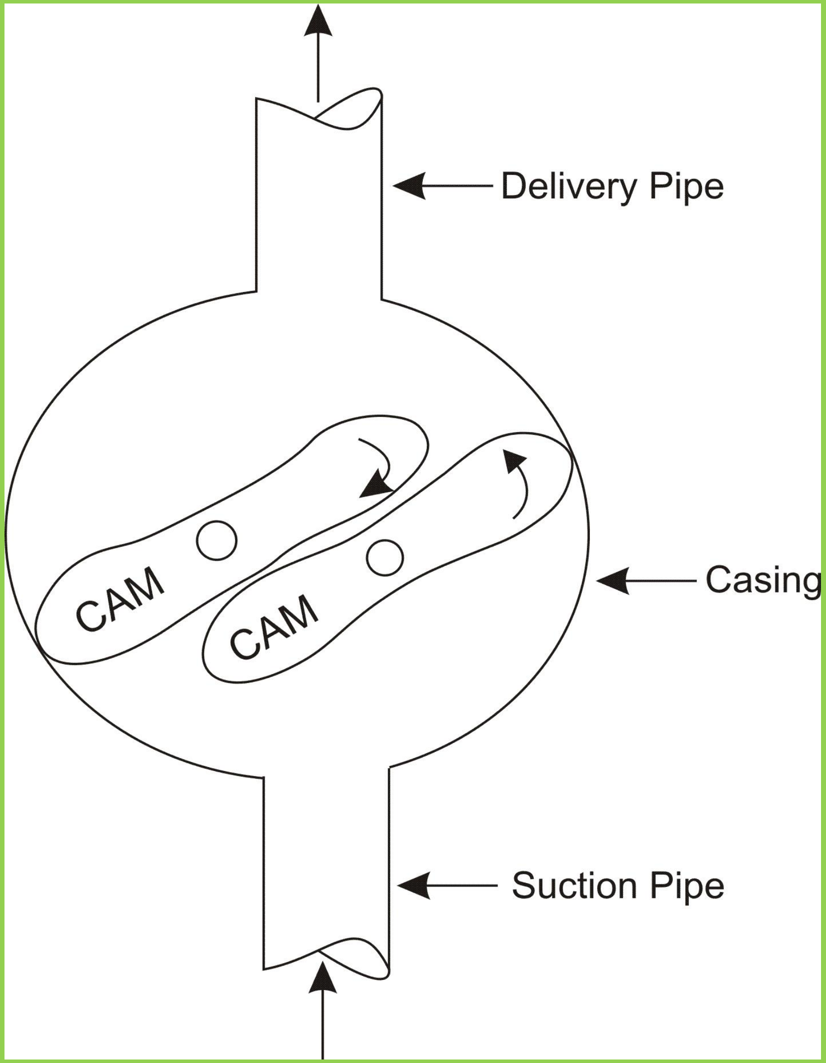
Disadvantages of centrifugal pumps

1. More parts mean high initial cost.
2. Maintenance cost is high, because parts like valves require constant attention.
3. No uniform torque.
4. Low discharging capacity.
5. Single acting reciprocating pumps produce pulsating flow.
6. Difficult to pump viscous fluid.
7. High wear in parts.
8. They are unsuitable for pumping waters containing sediments.

ROTARY PUMP

It consists of two cams that are pivoted in a casing. These cams mesh together and rotate in opposite directions and thereby the suction takes place through the suction pipe as shown in Fig. 5.4. It is then forced out through the discharge pipe, as the cams rotate. A definite quantity of water depending on the size and shape of the gears is thus raised with each revolution.

ROTARY PUMP



Advantages of Rotary Pumps

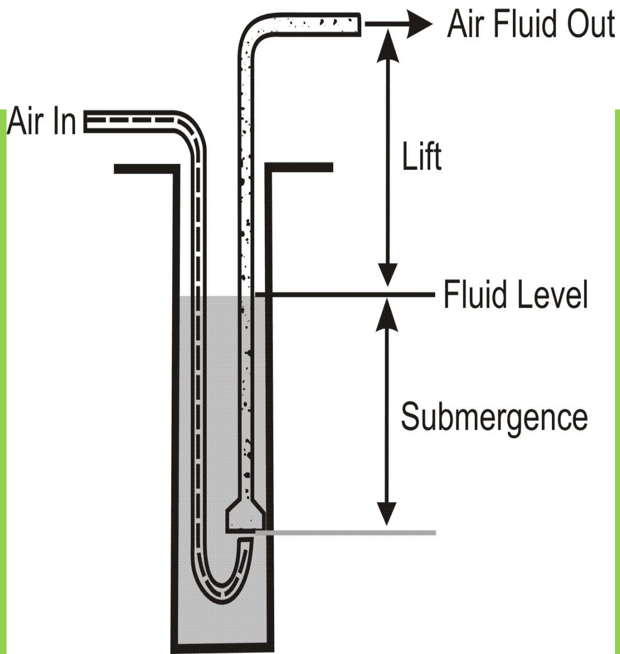
1. They can deliver liquid to high pressures.
2. No priming is required.
3. Give a relatively smooth output, (especially at high speed).
4. Can pump viscous liquids.
5. It requires no valve and its operation is simple.

Disadvantages of Rotary Pumps

1. More expensive than centrifugal pumps.
2. Should not be used for fluids containing suspended solids.
3. Excessive wear if not pumping viscous material.
4. Must never be used with the discharge closed.

AIR LIFT PUMPS

Air lift pumps are generally used for pumping water from deep wells. It consists of a casing pipe in which an educator pipe and an air pipe are introduced. The bottom end of the air pipe carries an air diffuser which is introduced into the educator pipe in upward direction.



AIR LIFT PUMPS

The pump injects compressed air at the bottom of the discharge pipe which is immersed in the liquid. The compressed air mixes with the liquid causing the air-water mixture to be less dense than the rest of the liquid around it and therefore is displaced upwards through the discharge pipe by the surrounding liquid of higher density. Solids may be entrained in the flow and if small enough to fit through the pipe, will be discharged with the rest of the flow at a shallower depth or above the surface. Airlift pumps are widely used in aquaculture to pump, circulate and aerate water in closed, recirculating systems and ponds. Other applications include dredging, underwater archaeology, salvage operations and collection of scientific specimens.

Advantages

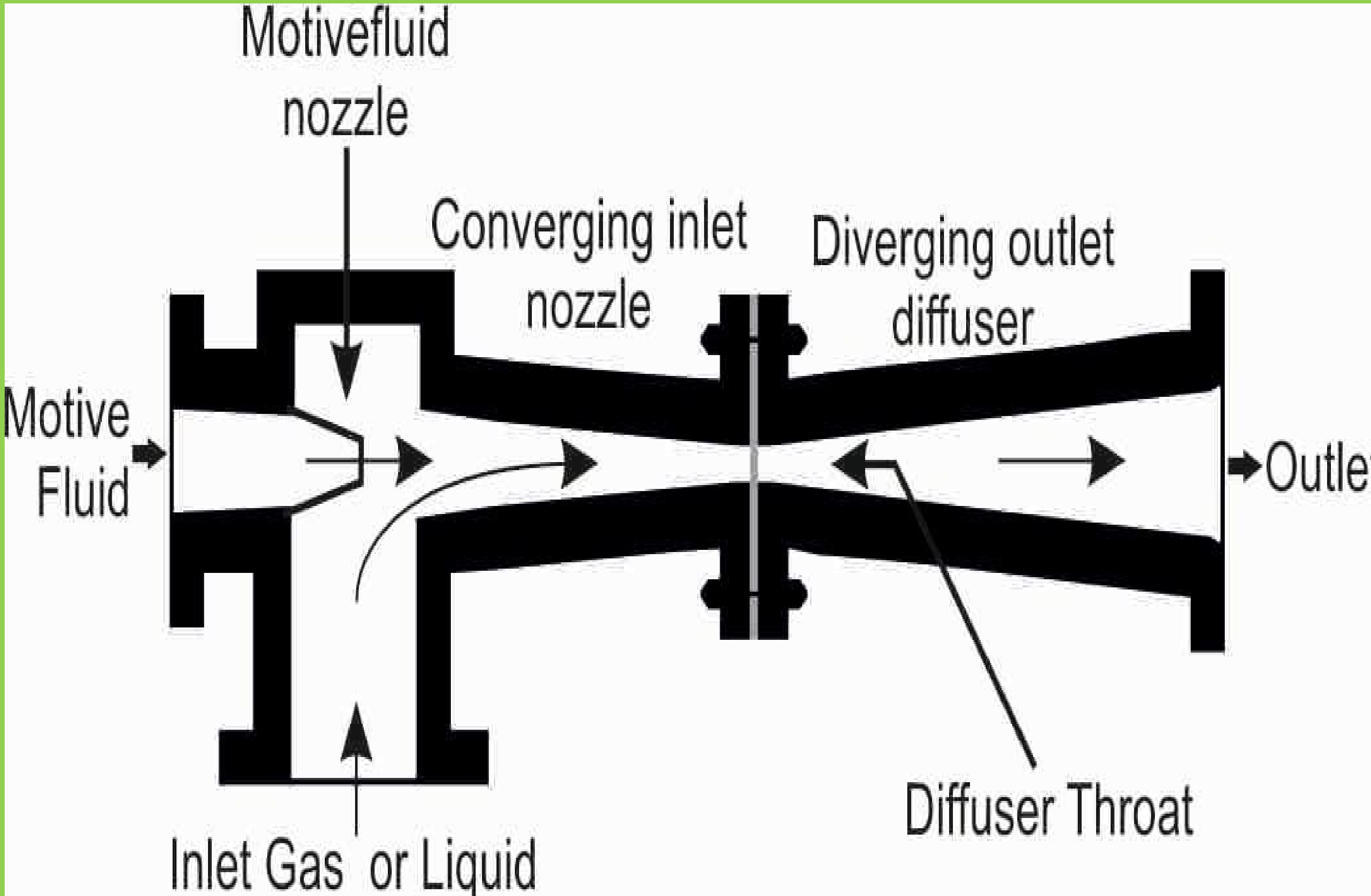
1. Air lift pumps are reliable, cheaper and simple in operation.
2. The liquid is not in contact with any mechanical elements. Therefore, the pump will not be abraded.
3. It can also be act as a water aerator in aqua culture activities.
4. Since there are no restrictive pump parts, solids up to 70% of the pipe diameter can be reliably pumped.
5. The pumping from a number of wells can be done by installing a common compressor unit.
6. The yield of a well, using such a pump, can be increased by using more amount of compressed air.

Disadvantages

1. Operational cost is more.
2. Conventional airlift pumps have a flow rate that is very limited.
3. The suction is limited.
4. This pumping system is suitable only if the head is relatively low. If you want to obtain a high head, you have to choose a conventional pumping system.
5. Because of the principle, a lot of air remains in the liquid. In certain case, this can be problematic.

JET PUMPS

Jet lift pump is a pump having no moving parts and utilizes fluids in motion under controlled conditions. Jet pumps are generally used for pumping water from small wells. They are potable and sometimes used in construction works for dewatering trenches, etc. A typical arrangement of a jet pump is shown in Fig. 5.6. In this pump, compressed air or steam or water is made to enter the pipe A, which is nozzled at its discharge end, thus forcing out through the nozzle. The jet of compressed air thus comes out of the nozzle at high velocity, and discharges into the throat of another pipe (B) as shown. This high velocity jet creates a suction which draws the water up the pipe B and finally discharges through the discharge end.


Motivefluid
nozzle
Converging inlet
nozzle
Diverging outlet
diffuser
Motive
Fluid
Outlet
Diffuser Throat
Inlet Gas or Liquid

1. **Advantages**
    1. They are compact and light in weight, thus making them portable and easy to handle.
    2. They can also handle sediment waters without much troubles.
    3. Reliable operation and low maintenance costs.
    4. Worn parts can be easily and inexpensively replaced.

Disadvantages

1. Efficiency of jet pump is generally low.
2. The pumps are to be tailor-made for different situations.
3. Air-jets or liquid-jets are required to form homogeneous slurry.
4. Cannot pump over longer distances unlike centrifugal pumps.

QUALITY OF WATER

COMMON IMPURITIES IN WATER

The impurities in water are either organic or inorganic in nature. They may be broadly classified as:

1. Physical impurities.
2. Chemical impurities.
3. Bacteriological impurities.

The above impurities may be present in the form of :

1. Suspended solids
2. Colloidal particles
3. Dissolved solids

WATER QUALITY ANALYSIS

Water quality refers to the chemical, physical, biological and radiological characteristics of water. It is a measure of the condition of water relative to the requirements of one or more biotic species and or to any human need or purpose.

WATER QUALITY ANALYSIS

The examination of water may be divided into three classes:

1. Physical Examination.
2. Chemical examination.
3. Bacteriological examination

PHYSICAL EXAMINATION

These consist of:

1. Temperature test
2. Colour test
3. Turbidity test
4. Taste and odour test

Temperature

The temperature of water is measured by means of ordinary thermometers. Density, viscosity, vapor pressure and surface tension of water are all dependent upon the temperature. The saturation values of solids and gases that can be dissolved in water and the rates of chemical, biochemical and biological activity are also determined on the basis of temperature. The temperature of surface water is generally same as the atmospheric temperature while that of ground water may be more or less than atmospheric temperature.

Color

The color of water is usually due to presence of organic matter in colloid condition, and due to the presence of mineral and dissolved organic and inorganic impurities. Transparent water with a low accumulation of dissolved materials appears blue. Dissolved organic matter such as humus, peat or decaying plant matter, etc. produce a yellow or brown color. Some algae or din flagellates produce reddish or deep yellow waters.

Color

the color the water is compared with standard color solution or color discs by using an instrument called Tintometer. The instrument has an eye-piece with two holes. The slide of standard colored water is seen through one hole, while the slide of the water to be tested is seen through the other hole. The intensity of colour in water is measured in terms of arbitrary unit of colour on the cobalt scale. The color produced by one milligram of platinum cobalt in a liter of distilled water has been fixed as the unit of color. For public water supply, the number on cobalt scale should not exceed 20, and should preferably less than 10.

Turbidity

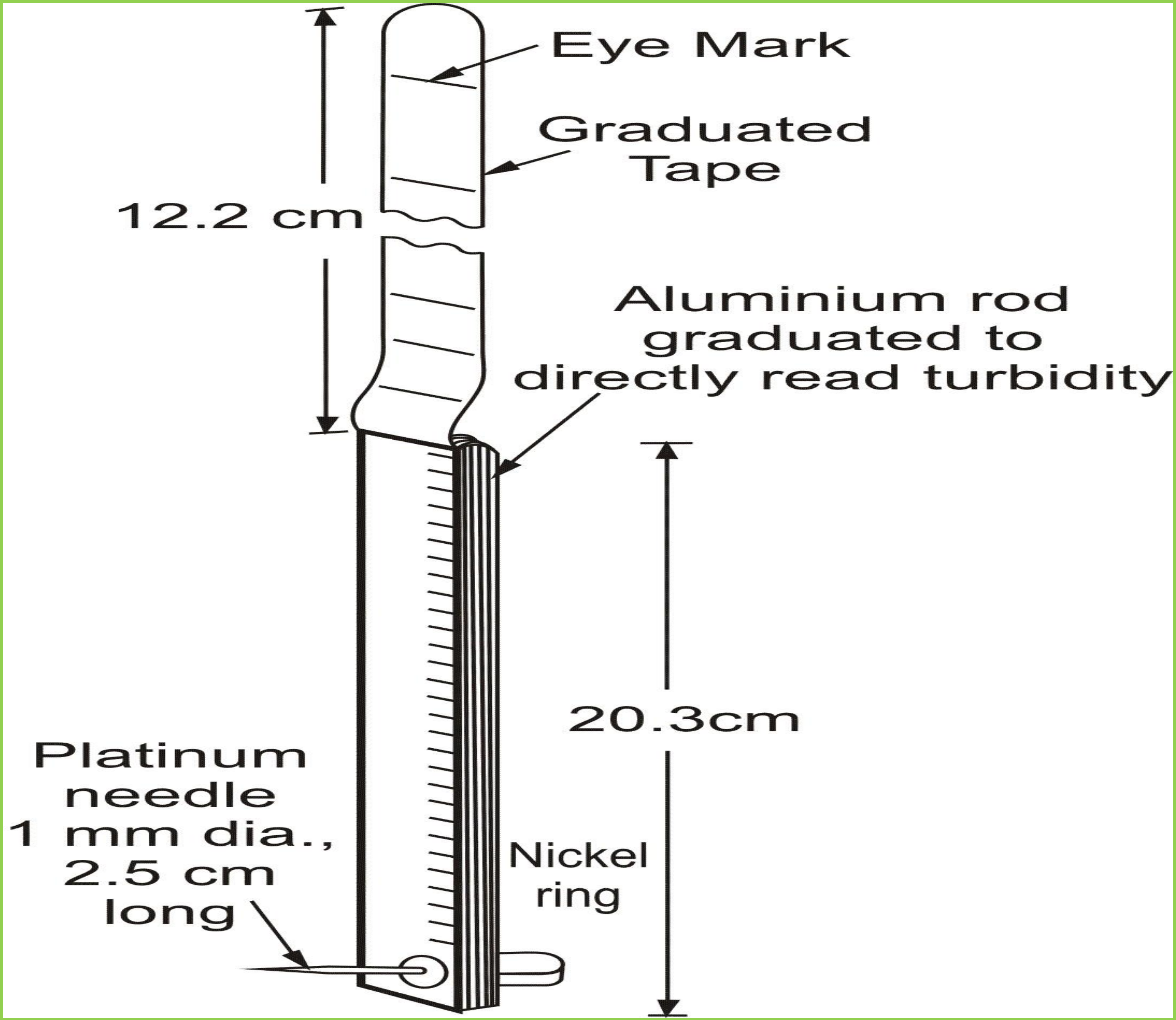
It is caused due to presence of suspended and colloidal matter in the water. Ground waters are generally less turbid than the surface water. The character and amount of turbidity depends on the type of soil over which the water has moved. Turbidity is a measure of the resistance of water to the passage of light through it. Turbidity is expressed in parts per million (ppm or milligrams per liter or mg/L).

Turbidity

Earlier, the turbidity produced by one milligram of silica in one liter of distilled water was considered as the unit of turbidity. Turbidity was previously determined by Jackson candle Turbidity units (JTU). This unit is now replaced by more appropriate unit called Nephelometric Turbidity Unit (NTU) which is the turbidity produced by one milligram of formazin polymer in one liter of distilled water. Nephelometry method has better sensitivity, precision and applicability over a wide range of particle size and concentrations as compared to older methods. The permissible turbidity of domestic water may be between 5 to 10 ppm.

The following are common methods of measuring turbidity of water:

1. By turbidity rod.
2. By Jackson’s turbidimeter.
3. By Nephlometers.

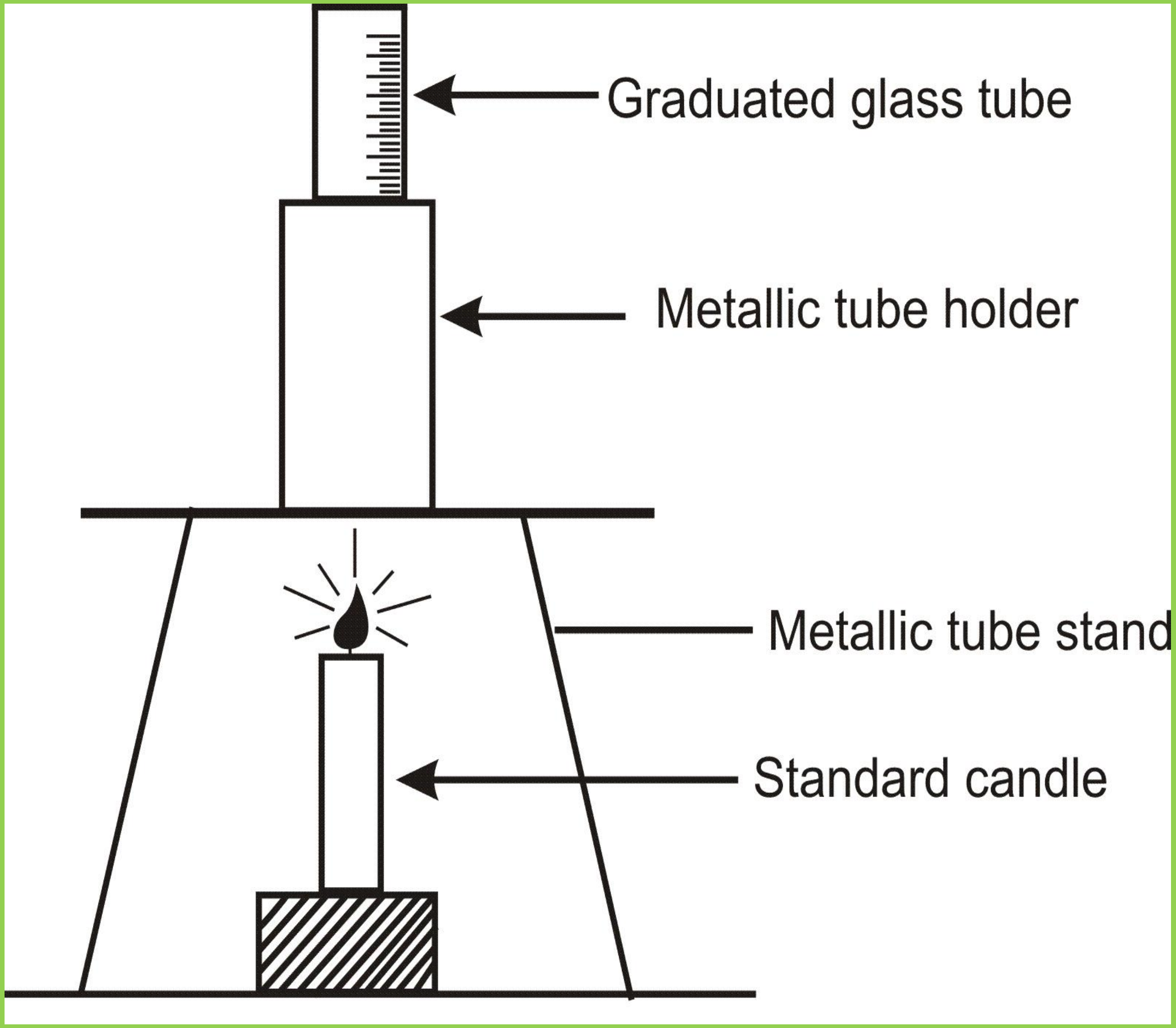


Eye Mark
Graduated
Tape
12.2 cm
Aluminium rod
graduated to
directly read turbidity
20.3cm
Platinum
needle
1 mm dia.,
2.5 cm
long
Nickel
ring

Turbidity rod is used for measuring turbidity of water in the field. It consists of a graduated aluminum rod, about 20.3 cm in length, at the upper end of which is attached a graduated non-stretchable tape of about 12.2 cm long. At the lower end of the aluminum rod, a screw containing a platinum needle and a nickel ring is inserted. The standard platinum needle is 1mm in diameter and 2.5 cm long. The graduated tape has a mark at its top end specifying the position of eye during the test.

In order to find the turbidity, the lower end of the rod is gradually immersed in water whose turbidity is to be determined. Eye is kept constantly at the marked position and the platinum needle is watched. The rod is moved slowly in water till the platinum needle just disappears from the vision due to turbidity of water. The reading of the graduated tape near the water surface directly gives turbidity in ppm.

By Jackson's turbidimeter



c) Nephelometer

A nephelometer is an instrument for measuring concentration of suspended particulates in a liquid or gas colloid. A nephelometer measures suspended particulates by employing a light beam (source beam) and a light detector set to one side (often 90°) of the source beam. Particle density is then a function of the light reflected into the detector from the particles. To some extent, how much light reflects for a given density of particles is dependent upon properties of the particles such as their shape, color, and reflectivity. Nephelometers are calibrated to a known particulate, then use environmental factors (k-factors) to compensate lighter or darker colored dusts accordingly. K-factor is determined by the user by running the nephelometer next to an air sampling pump and comparing results. There is a wide variety of research-grade nephelometers on the market.

Tastes and odors

Tastes and odors in water are due to the presence of:

1. Dead or living microorganisms.
2. Dissolved gases such as hydrogen sulphide, methane, carbon dioxide or oxygen combined with organic matter.
3. Excess chlorination during treatment.
4. Pollution by industrial wastes
5. Mineral substances such as sodium chloride, iron compounds; and Carbonates and sulphates

The intensity of odour is measured by osmoscope in terms of “Threshold number”. Threshold number is the dilution ratio at which odour is just detectable. For example if 10 cc of water sample has to be added to 100 cc of distilled water to produce detectable odour, then the threshold number of the sample is 10. For public water supply, it should not be greater than 3.

Specific conductivity of water

The total amount of dissolved salts present in water can be estimated by measuring the specific conductivity of water. The specific conductivity of water is determined by means of a portable ionic water tester and is expressed as micro-mho per cm at 25°C. ‘mho’ is the unit of conductivity and it equals to 1 Ampere per volt. The specific conductivity of water in micro mho per cm at 25°C is multiplied by a coefficient generally 0.65 so as to directly obtain the dissolved salt content in mg/L or ppm. The actual value of this coefficient depends upon the type of salt present in water.

CHEMICAL EXAMINATION

Chemical tests are carried out to determine the following:

1. Total Solids
2. pH
3. Acidity
4. Alkalinity
5. Hardness
6. Chlorides
7. Sulphates
8. Iron
9. Nitrates
10. Dissolved Oxygen

Solids

The total solids obtained by filtering and evaporating are kept in the platinum dish and held over a Bunsen flame till every part of solids is raised to a bright red heat. The organic matter is burnt off in this process and only inorganic matter is left behind. The dish is cooled, and weight of the matter remaining behind gives the amount of ‘fixed solids’. Solids that can settle by gravity are settleable solids. The others are non-settleable solids. IS acceptable limit for total solids is 500 mg/L and tolerable limit is 3000 mg/L of dissolved limits.

1.pH

pH value denotes the acidic or alkaline condition of water. It is expressed on a scale ranging from 0 to 14, which is the common logarithm of the reciprocal of the hydrogen ion concentration. The recommended pH range for treated drinking waters is **6.5 to 8.5.**

When a substance dissolves in water, it ionizes into positively charged Hydrogen (H^+) ions and negatively charged Hydroxyl (OH^-) ions. When OH^- ions are more than the H^+ ions, alkalinity is caused. If the concentration of both the types is equal the sample is said to be neutral. The p^H value is determined by the following methods.

i) Electrometric method

ii) Colorimetric method

i) Electrometric method

In this method a potentiometer is used to measure the electrical pressure exerted by the H^+ ions. A meter (p^H meter) connected to the circuit indicates the p^H value directly.

i) Colorimetric method

In this method, chemical reagents or indicators are added to the sample of water. The colour produced is compared with standard colour water kept in sealed tubes of known p^H values. This is a simple test and commonly adopted.

2. **Acidity**

The acidity of water is a measure of its capacity to neutralise bases. Acidity of water may be caused by the presence of uncombined carbon dioxide, mineral acids and salts of strong acids and weak bases. It is expressed as mg/L in terms of calcium carbonate. Acidity is nothing but representation of carbon dioxide or carbonic acids. Carbon dioxide causes corrosion in public water supply systems.

2. Alkalinity

The alkalinity of water is a measure of its capacity to neutralise acids. It is expressed as mg/L in terms of calcium carbonate. The various forms of alkalinity are (a) hydroxide alkalinity, (b) carbonate alkalinity, (c) hydroxide plus carbonate alkalinity, (d) carbonate plus bicarbonate alkalinity, and (e) bicarbonate alkalinity, which is useful mainly in water softening and boiler feed water processes. Alkalinity is an important parameter in evaluating the optimum coagulant dosage.

1.Hardness

If water consumes excessive soap to produce lather, it is said to be hard. Hardness is caused by divalent metallic cations. The principal hardness causing cations are calcium, magnesium, strontium, ferrous and manganese ions. The major anions associated with these cations are sulphates, carbonates, bicarbonates, chlorides and nitrates. The total hardness of water is defined as the sum of calcium and magnesium concentrations, both expressed as calcium carbonate, in mg/L. BIS value for drinking water is 300 mg/L as $CaCO_3$. Hardness is of two types, temporary or carbonate hardness and permanent or non carbonate hardness. Hard water consume more soap and hence not suitable for laundries. Hard water is also unfit for usage in boilers as it causes boiler scale.

Temporary hardness

Temporary hardness is caused by carbonates and bicarbonates of calcium and magnesium. It can be removed by simple boiling. It is also known as carbonated hardness.

Permanent Hardness

Permanent hardness, also called non-carbonate hardness and is caused by chlorides and sulphates of calcium and magnesium. Permanent hardness cannot be removed by boiling and require method of water softening. The three methods, which are commonly adopted for softening waters containing either permanent hardness, or permanent as well as temporary hardness both, are:

1) Lime soda process;
2) Zeolite process; and
3) Demineralisation process:

There are three methods of determining total hardness of water.

i) Clark's method

ii) Hehner's method

iii) Versenate method

iv) Clark's method

In this method standard soap solution is added to the water and vigorously shaken for about five minutes. The difference between the soap solution used and the lather factor gives the hardness.

i) Hehner's method

In this method the temporary hardness is determined by titration with a standard solution of sulphuric acid, using methyl orange as indicator. To determine permanent hardness, standard sodium carbonate solution is added to the water and evaporated. The amount of sodium carbonate in excess over that required to convert the sulphates and chlorides in to carbonates gives the permanent hardness.

6. Chlorides

Chloride ion may be present in combination with one or more of the cations of calcium, magnesium, iron and sodium. Chlorides of these minerals are present in water because of their high solubility in water. Each human being consumes about six to eight grams of sodium chloride per day, a part of which is discharged through urine and night soil. Thus, excessive presence of chloride in water indicates sewage pollution. IS value for drinking water is 250 to 1000 mg/L. Chlorides are estimated by titration with standard silver nitrate solution using potassium chromate as indicator.

7. Sulphates

Sulphates occur in water due to leaching from sulphate mineral and oxidation of sulphides. Sulphates are associated generally with calcium, magnesium and sodium ions. Sulphate in drinking water causes a laxative effect and leads to scale formation in boilers. It also causes odour and corrosion problems under aerobic conditions. If barium chloride solution is added to a sample of water containing sulfate ions, barium sulfate is formed. Barium sulfate is insoluble in water, and will be seen as a white precipitate. Sulphate should be less than 50 mg/L, for some industries. Desirable limit for drinking water is 150 mg/L. May be extended up to 400 mg/L.

8. Iron

Iron is found on earth mainly as insoluble ferric oxide. When it comes in contact with water, it dissolves to form ferrous bicarbonate under favorable conditions. This ferrous bicarbonate is oxidised into ferric hydroxide, which is a precipitate. Under anaerobic conditions, ferric ion is reduced to soluble ferrous ion. Iron can impart bad taste to the water, causes discoloration in clothes and incrustations in water mains. IS value for drinking water is 0.3 to 1.0 mg/L.

9. Nitrates

Nitrates in surface waters occur by the leaching of fertilizers from soil during surface run-off and also nitrification of organic matter. Presence of high concentration of nitrates is an indication of pollution. Concentrations of nitrates above 45 mg/L cause a disease methemoglobinemia. IS value is 45 mg/L. Nitrate is measured either by reduction to ammonia or by matching the colors produced with phenoldisulphonic acid.

10.. Dissolved Oxygen

Dissolved Oxygen is the amount of gaseous oxygen (O_2) dissolved in the water. Oxygen enters the water by direct absorption from the atmosphere, by rapid movement, or as a waste product of plant photosynthesis. Water temperature and the volume of moving water can affect dissolved oxygen levels. Oxygen dissolves easier in cooler water than warmer water. Adequate dissolved oxygen is important for good water quality and necessary to all forms of life. Dissolved oxygen levels that drops below 5.0 mg/L cause stress to aquatic life.

The dissolved oxygen can be determined by using various modifications of the basic Winkler method or by the polarographic method.

BACTERIOLOGICAL TEST

Bacterial examination of water is very important, since it indicates the degree of pollution. Water polluted by sewage contains one or more species of disease producing pathogenic bacteria. Pathogenic organisms cause water borne diseases, and many non pathogenic bacteria such as *E.Coli*, a member of coliform group, also live in the intestinal tract of human beings. *Coliform* itself is not a harmful group but it has more resistance to adverse condition than any other group.

BACTERIOLOGICAL TEST

So, if it is ensured to minimize the number of coliforms, the harmful species will be very less. So, coliform group serves as indicator of contamination of water with sewage and presence of pathogens.

The methods to estimate the bacterial quality of water are:

1. Standard Plate Count Test
2. E-coli test
3. Membrane Filter Technique

Standard Plate Count Test

In this test, the bacteria are made to grow as colonies, by inoculating a known volume of sample into a solidifiable nutrient medium (Nutrient Agar), which is poured in a petridish. After incubating (35°C) for a specified period (24 hours), the colonies of bacteria (as spots) are counted. The bacterial density is expressed as number of colonies per 100 ml of sample.

E coli Test

It is observed that at $37^{o}C$, Coliform in lactose medium ferment within 48 hours with formation of gas. The E-coli test is done in three stages.

1. Presumptive test
2. Confirmed test
3. Completed test

Presumptive test

Sample of water are placed in sterile tubes and lactose is added to them. The tubes are incubated for 24 hours at $37^{o}C$. If gas is formed, then the test is positive indicating the presence of bacteria. If no gas is formed, it is again examined at the end of 48 hours. If still no gas is formed, the test is negative.

Confirmed test

This test is carried out to see if E-coli group is present in water. The following two methods are adopted.

1. When a sample of water is positive in the presumptive test, a small portion of the lactose broth is transferred to another fermentation tube containing green lactose bile. After 48 hours if gas formation seen the test is positive showing that the water is unsafe for drinking.

2. In the second method a small sample is taken on plate containing "Endo Eosin Ethylene blue agar" and is incubated at $37^{o}C$ for 24 hours. If colonies are seen then the test is positive.

Completed test

The test involves inoculation of bacterial colonies of the previous test into tubes containing lactose broth medium and incubated for 24 to 48 hours at 37^{o}C. If gas is formed after this period, then the result is positive and water is not safe for drinking.

The final result of the E-coli test is expressed in terms of E-coli index or MPN.

E-coli Index

This is an index showing the approximate number of E-coli present in cc of the sample of water. Presumptive tests are conducted with different dilutions of the water with sterilized water. For each diluted sample a number of tests are conducted. The percentages of positive results are noted. For single sample with only a few portions, the index is taken as the reciprocal of the smallest portion showing positive results.

10ml	1ml	0.1ml	0.01ml	0.001ml
+ve	+ve	+ve	+ve	- ve

The coliform index in the above case will be the reciprocal of 0.01.

$$\text{ie.} \quad = \quad \frac{1}{0.01} = 100 \text{ per ml}$$

MPN (Most Probable Number)

The laws of statistical probability are applied to the results obtained in the E-coli test and finally expressed in terms of MPN. This statistical number represents the bacterial density which is mostly likely to be present in the sample. Most probable number is a number which represents the bacterial density which is most likely to be present. E-Coli is used as indicator of pollution.

MPN (Most Probable Number)

E-Coli ferment lactose with gas formation with 48 hours incubation at 35°C. Based on this E-Coli density in a sample is estimated by multiple tube fermentation procedure, which consists of identification of E-Coli in different dilution combination. For the given results of the test, MPN is noted from Maccardy's statistical tables. MPN is a better and more popular expression than E-coli index.

Membrane Filter Technique

In this test a known volume of water sample is filtered through a membrane with opening less than 0.5 microns. The bacteria present in the sample will be retained upon the filter paper. The filter paper is put in contact of a suitable nutrient medium and kept in an incubator for 24 hours at 35°C. The bacteria will grow upon the nutrient medium and visible colonies are counted. Each colony represents one bacterium of the original sample. The bacterial count is expressed as number of colonies per 100 ml of sample.

WATER-BORNE DISEASES

Waterborne diseases are caused by pathogenic microorganisms that most commonly are transmitted in contaminated fresh water. Infection commonly results during bathing, washing, drinking, in the preparation of food, or the consumption of food that is infected.

WATER-BORNE DISEASES

Among the many types of microorganisms that may cause waterborne diseases are:

- Protozoa
- Bacteria
- Intestinal parasites
- Viruses

WATER-BORNE DISEASES

The some of the important water-borne diseases are given below.

Amebiasis: caused by protozoa. Symptoms include fatigue, diarrhea, flatulence, abdominal discomfort and weight loss.

Campylobacteriosis: caused by bacteria. Symptoms include diarrhea, abdominal pain and fever.

Cholera: caused by bacteria. Symptoms include muscle cramps, vomiting and diarrhea.

Cryptosporidiosis: caused by protozoa. Symptoms include diarrhea and abdominal discomfort.

Giardiasis: caused by protozoa. Symptoms include diarrhea and abdominal discomfort.

WATER-BORNE DISEASES

Hepatitis: caused by a virus. Symptoms include fever, chills, jaundice, dark urine and abdominal discomfort.

Shigellosis: caused by bacteria. Symptoms include bloody stool, diarrhea and fever.

Typhoid fever: caused by bacteria. Symptoms include fever, headache, constipation, diarrhea, nausea, vomiting, loss of appetite and an abdominal rash.

Viral gastroenteritis: caused by a virus. Symptoms include gastrointestinal discomfort, diarrhea, vomiting, fever and headache.

WATER-BORNE DISEASES

Hepatitis: caused by a virus. Symptoms include fever, chills, jaundice, dark urine and abdominal discomfort.

Shigellosis: caused by bacteria. Symptoms include bloody stool, diarrhea and fever.

Typhoid fever: caused by bacteria. Symptoms include fever, headache, constipation, diarrhea, nausea, vomiting, loss of appetite and an abdominal rash.

Viral gastroenteritis: caused by a virus. Symptoms include gastrointestinal discomfort, diarrhea, vomiting, fever and headache.

MODULE III

TREATMENT OF WATER

Water treatment is any process that makes water more acceptable for a specific end-use. The end-use may be drinking, industrial water supply, irrigation, river flow maintenance, water recreation or many.

treatment removes contaminants and undesirable components, or reduces their concentration so that the water becomes fit for its desired end-use.

TREATMENT OF WATER

Substances that are removed during the process of drinking water treatment include

Suspended solids,

Bacteria,

Algae,

Viruses,

Fungi and

Minerals such as iron and manganese.

TREATMENT OF WATER

Substances that are removed during the process of drinking water treatment include

Suspended solids,

Bacteria,

Algae,

Viruses,

Fungi and

Minerals such as iron and manganese.

OBJECTIVE OF WATER TREATMENT PROCESSES

Following are the purpose of water treatment:

i. To remove colour, turbidity and dissolved gas of water.

ii. To remove objectionable taste and odour.

iii. To remove the disease producing micro-organisms so that water is safe for drinking purpose.

iv. To remove hardness of water.

v. To make water suitable for a wide variety of industrial purposes such as steam generation, brewing, dying etc.

WATER TREATMENT PROCESSES

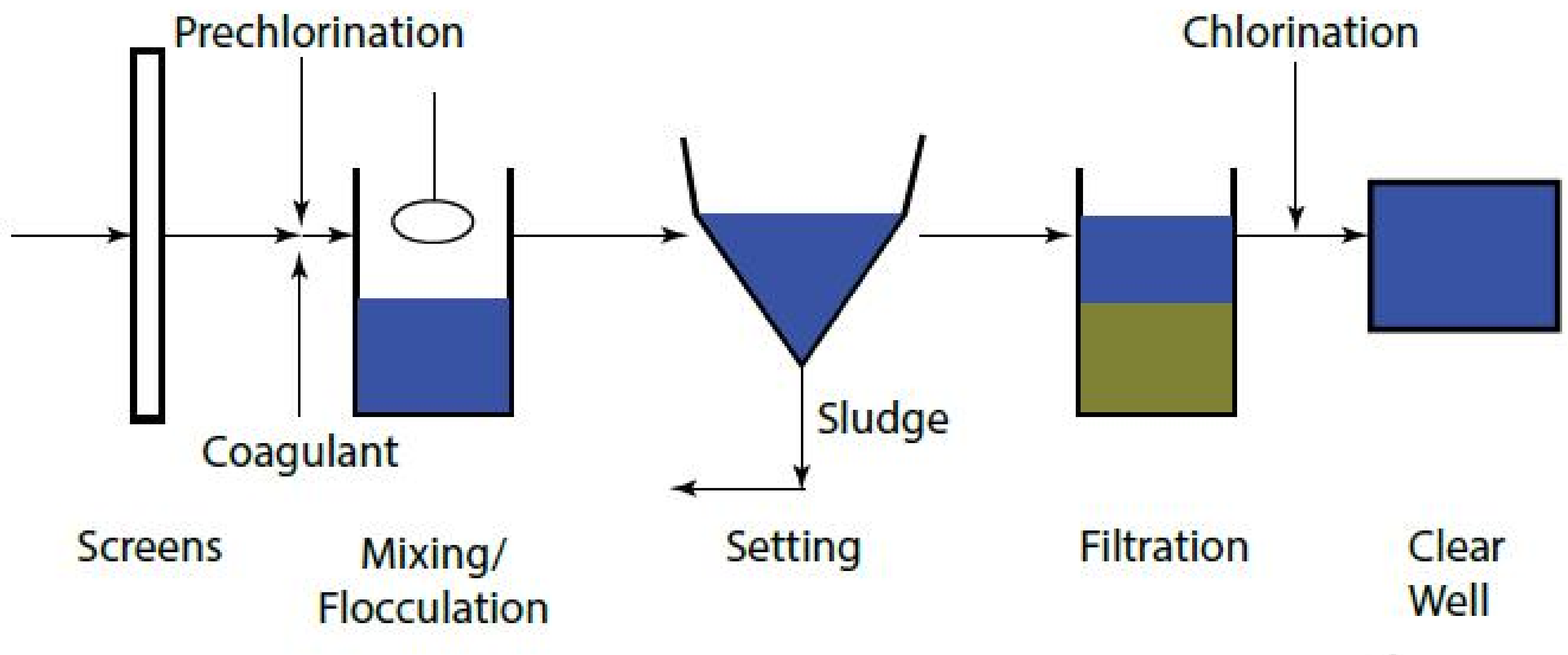


Fig. 7.1 Schematic layout of treatment plant

OBJECTIVE OF WATER TREATMENT PROCESSES

For surface waters, following are the treatment processes that are generally adopted.

1. Screening: This is adopted to remove all the floating matter from surface waters. It is generally provided at the intake point.

2. Aeration: Aeration is often the first major process at the treatment plant.

Aeration: Aeration is often the first major process at the treatment plant. This is adopted to remove objectionable tastes and odour and also to remove the dissolved gases such as carbon dioxide, hydrogen sulphide, volatile organic chemicals etc. The iron and manganese present in water is also oxidized to some extent. This process is optional and is not adopted in cases where water does not contain objectionable taste and odour.

3. Sedimentation with or without coagulants: Sedimentation is a physical water treatment process using gravity to remove suspended solids from water. With the help of plain sedimentation, silt, sand etc. can be removed. However, with the help of sedimentation with coagulants, very fine suspended particles and some amount of bacteria can be removed.

4. Filtration:

The process of filtration forms the most important stage in the purification of water. Filtration removes very fine suspended impurities and colloidal impurities that may have escaped from the sedimentation tank. In addition to this, the micro-organisms present in water are largely removed.

5. Disinfection:

Water disinfection means the removal, deactivation or killing of pathogenic microorganisms. It also prevents the contamination of water during its transit from the treatment plant to the place of its consumption. When microorganisms are not removed from drinking water, drinking water usage will cause people to fall ill.

6. Miscellaneous processes.

These include water softening, desalination, removal of iron, manganese and other harmful constituents.

SEDIMENTATION

The particles which do not change their shape, size and weight, while settling down in a fluid are known as discrete particles. The suspended impurities in water consists of discrete particles such as inorganic solids having specific gravity about 2.65 and organic solids having specific gravity 1.04. The particles having specific gravity more than 1.20 readily settle down at the bottom of the tank due to the force of gravity. This phenomenon of settlement is known as hydraulic subsidence. Every particle has its own hydraulic subsidence value. But the lighter particles cannot

SEDIMENTATION

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SEDIMENTATION

Every particle has its own hydraulic subsidence value. But the lighter particles cannot settle down due to force of gravity. Such particles are converted to settleable size by the application of some coagulant in water. The sedimentation tanks are designed to give complete rest to the flowing water or water is allowed to flow at a very low velocity. The heavier inorganic impurities settle at the bottom of tanks and lighter inorganic impurities float on the surface of liquid level. The former impurities are removed from bottom while latter impurities are removed from the top.

Plain sedimentation.

When the impurities are separated from suspending fluid by the action of natural forces alone; ie. by gravitation and natural aggregation of the settling particles, the operation is called plain sedimentation.

Sedimentation with coagulation

Very fine colloidal particles cannot come together and get settle down by the above treatment since they have negative charge associated with them, which impart stability to the colloidal system formed. Hence they are removed by addition of chemicals called coagulants to water before sedimentation. These on dissolution in water produce cations which can neutralize the negative charge of the particles and hence from an insoluble gelatinous flocculent precipitate, which adsorb and forming bigger floc and this get settledown easily under the force of gravity.

Theory of sedimentation

The process of settlement of a particle is obstructed or opposed by the following three forces.

i. The velocity of flow

The velocity of flow carries the particle horizontally. The greater the flow area, the lesser is the velocity and hence more easily the particle will settle down.

ii. The viscosity of water in which the particle is travelling

The viscosity varies inversely with temperature. Warm water is less viscous and, therefore, offers less resistance to settlement.

iii. The size, shape and specific gravity of the particle

The greater is the specific gravity, more readily the particle will settle. The size and shape of the particle also affect the settling rate.

Types of settlings

Particles may settle out of a suspension in the following four ways, depending upon the concentration of the suspension and the flocculating properties of particles (Fig.7.2).

1. Type I: Discrete particle settling
2. Type II: Flocculent particles settling
3. Type III: Hindered or zone settling
4. Type IV: Compression settling

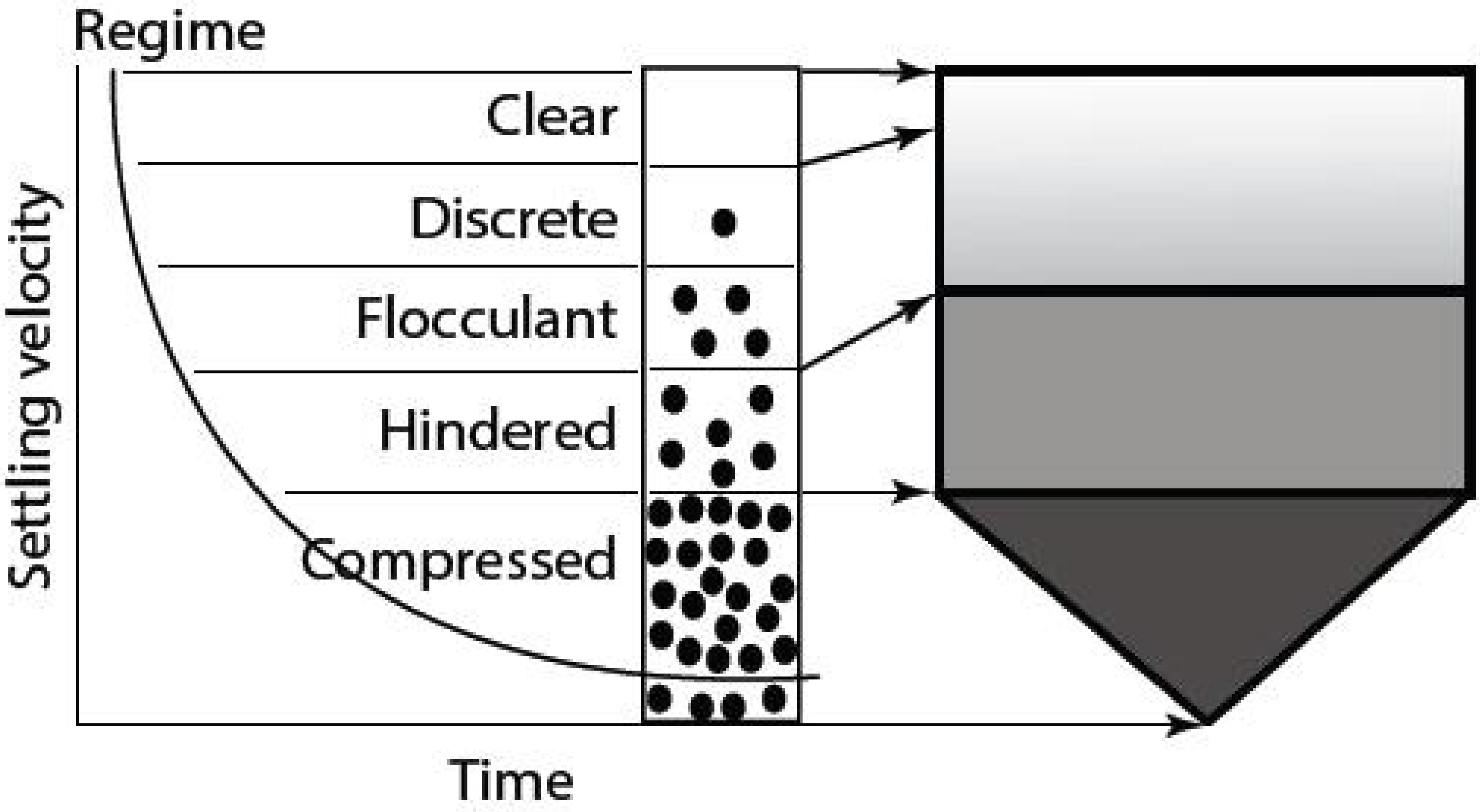


Fig. 7.2 Types of settling

Types of settlings

Particles may settle out of a suspension in the following four ways, depending upon the concentration of the suspension and the flocculating properties of particles (Fig.7.2).

1. Type I: Discrete particle settling
2. Type II: Flocculent particles settling
3. Type III: Hindered or zone settling
4. Type IV: Compression settling

1. Type I: Discrete particle settling - Particles settle individually without interaction with neighboring particles.

2. Type II: Flocculent Particles – Flocculation causes the particles to increase in mass and settle at a faster rate.

3. Type III: Hindered or Zone settling –The mass of particles tends to settle as a unit with individual particles remaining in fixed positions with respect to each other.

4. Type IV: Compression – The concentration of particles is so high that sedimentation can only occur through compaction of the structure.

In water treatment, only type I and type II settling are encountered

Type I Settling

Type I sedimentation is concerned with the settling/removal of non flocculating, discrete particles from water. In this type of settling the size, shape and specific gravity of the particles do not change with time and settling velocity remains constant. When a discrete particle is placed in a quiescent fluid, it will accelerate until the frictional resistance FD of the fluid equals the impelling force FI acting on the particle. At this stage, the particle attains a uniform or terminal velocity and settles down with this constant velocity known as settling velocity.

The impelling force (F_I) is evidently equal to the effective weight of the particle:

$$F_I = (\rho_p - \rho) g V \qquad \text{.....(7.1)}$$

Where,

ρ_p = Mass density of particle

ρ = Mass density of fluid

g = Acceleration due to gravity

V= Volume of Particle, $\frac{\pi}{6}d^3$ Where, d is the diameter of spherical particle

a. Newton's Law

The drag force F_D depends on

i. Dynamic viscosity

ii. Mass density ρ of the fluid and

iii. shape and size of the particle

The drag force is given by Newton's law for frictional drag in the following form:

$$F_D = C_D . A . \frac{\rho V_s^2}{2} \quad \text{....(7.2)}$$

Where, F_D = Drag force

C_D = drag coefficient

A = projected area of the particle $\frac{\pi}{4}d^2$.

Equating the two, we get an equation for the settling velocity in the form

$$V_s = \sqrt{\frac{4g(\rho_s - \rho)d}{3C_D\rho}} \qquad \text{.....(7.3)}$$

or closely,

$$V_s = \sqrt{\frac{4g(S_s - 1)d}{3C_D}} \qquad \text{.....(7.4)}$$

Where, S_s = specific gravity of the particle.

Above equations requires the determination of drag coefficient C_D which is related to Reynolds number R.

b. Stokes Law

Stoke's law for the drag of small settling spheres in a viscous fluid, neglecting the inertia force is given by

$$F_D = 3\pi\mu V_s.d$$

Equating this to Eq. 7.2 is given by

$$C_D = \frac{24}{R} \qquad \text{.....(7.5)}$$

Substituting this value of C_D in Eq. 7.3 we get

$$V_s = \frac{1}{18}\frac{g}{\mu}(\rho_s - \rho)d^2$$

$$V_s = \frac{1}{18}\frac{g}{V}(S_s - S)d^2$$

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

Where,

v = Kinematic viscosity of water in centistokes which varies with temperature of water.

Type II Settling

Type II settling is the settling of particles that flocculate as they settle. The process flocculation produces larger particles and settles at a faster rate. Settling of particles in coagulation cum sedimentation tank is an example for type II settling.

7.5.1 Classification of sedimentation tanks

Depending upon the nature of working, the sedimentation tanks are of the following two types.

i. Fill and draw types tanks

ii. Continuous flow type tanks

i. Fill and draw type tanks

These are also known as intermittent type sedimentation tanks. The water is filled in the tanks and it is then allowed to rest for a certain time. During the period of rest, the particle in suspension will settle down at the bottom of tank. The clear water is then drawn off and tank is cleaned of silt and filled again. Generally, a detention time of 24 hours is allowed. At the end of the period, the clear water drawn off through the outlet valve. The plan and section of fill and draw type tank is given below.

The plan and section of fill and draw type tank is given below.

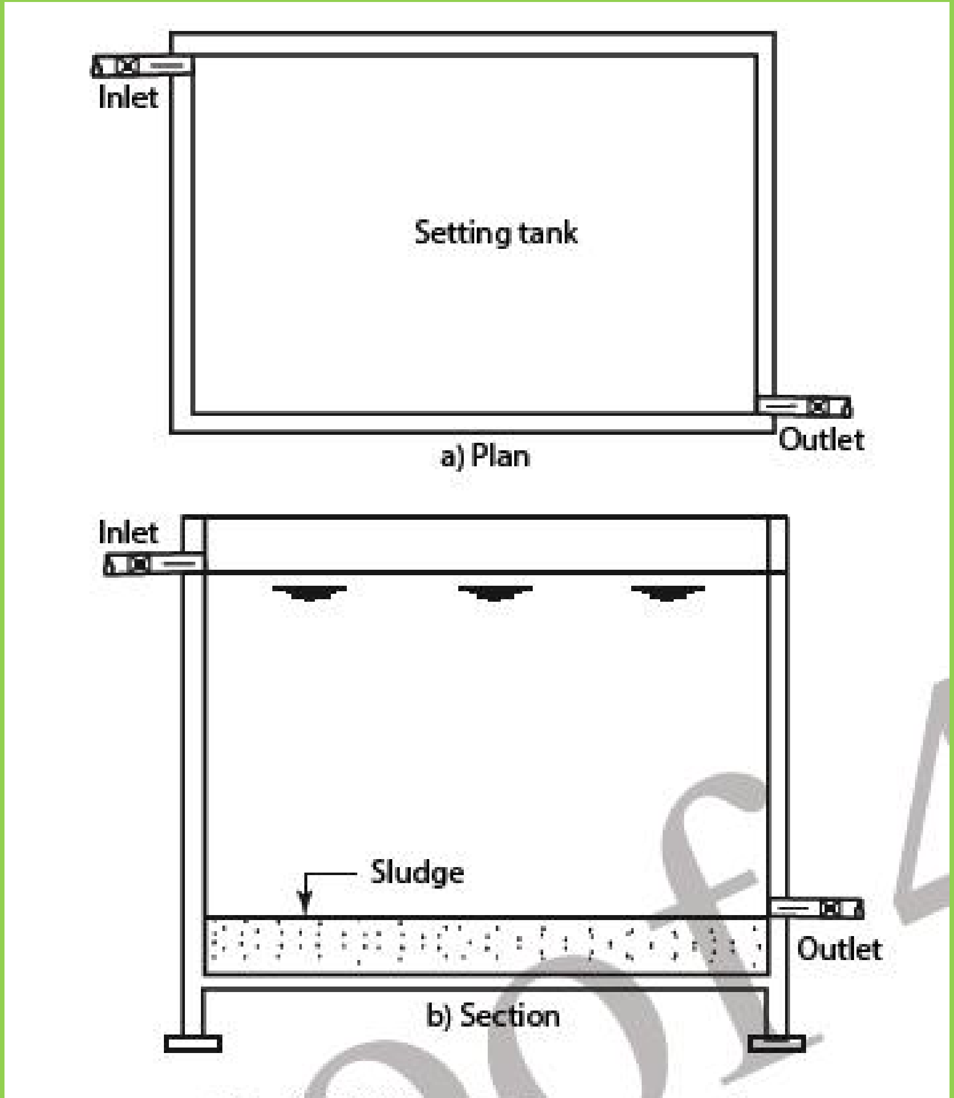


Fig. 7.3 Fill and draw type tank

ii. Continuous flow type tanks

In the continuous flow type, the water continuously keeps on moving in tank, though with a very low velocity during which time the suspended particles settle at the bottom before they reach the outlet. There are two types of continuous flow tanks.

a) Horizontal flow tanks

b) Vertical flow tanks

a. Horizontal flow tank.

In the horizontal flow type, the tank is generally rectangular in plan having length equal to at least twice the width. The water flows practically in the horizontal direction, with a maximum permissible velocity of 0.3 m/sec. These are further divided in to

i. Rectangular tanks

ii. Circular tanks

which are further classified as

- Radial flow tanks
- Circumferential flow tanks

b) Vertical flow tanks

Vertical flow type sedimentation tanks are generally circular in shape and flow takes place in vertical direction. Hoper bottom is provided at the bottom of the tank to dispose the collected sludge

PLAIN SEDIMENTATION TANKS

Plain sedimentation tanks are usually of the following three types:

a) Rectangular tanks

b) Circular tanks

c) Hopper Bottom tanks

a. Rectangular sedimentation tank

In this type of tank, its capacity depends upon the volume of water to be treated. The length depends on the velocity of flow and detention period. The detention period may vary from 4-6 hours. The width of the tank varies from 10m – 12m, and the depth of the tank varies from 2m to 4m. Here, the length of travel of the particles is increased by providing baffle walls. Thus, the velocity of flow is much reduced to maintain the designed detention period.

a. Rectangular sedimentation tank

Due to the low velocity of flow the heavier particles are settled down at the bottom of the tank as sludge. After some interval of time, the sludge is removed through the sludge removal pipe by opening the valve as shown in Fig. 7.4

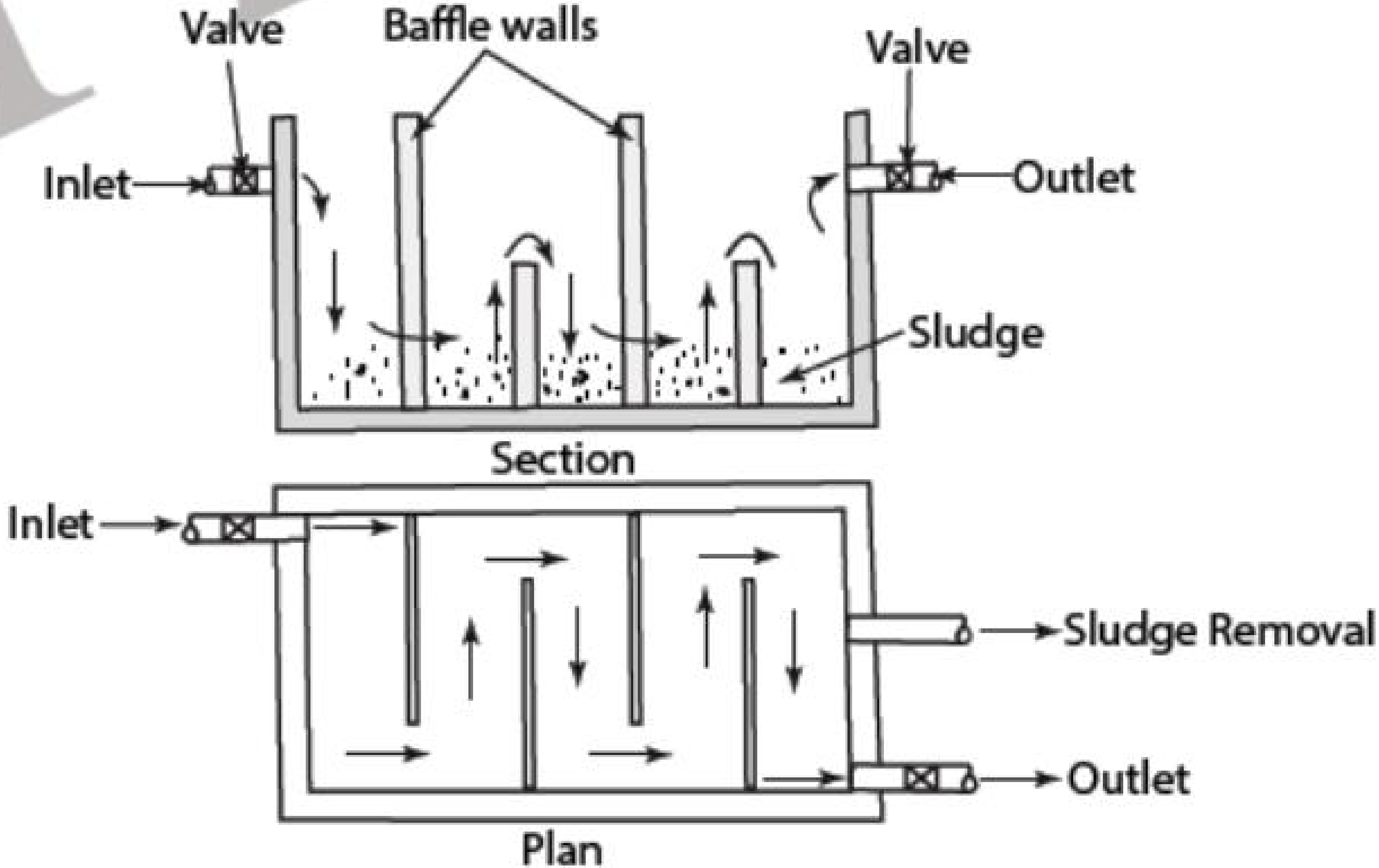


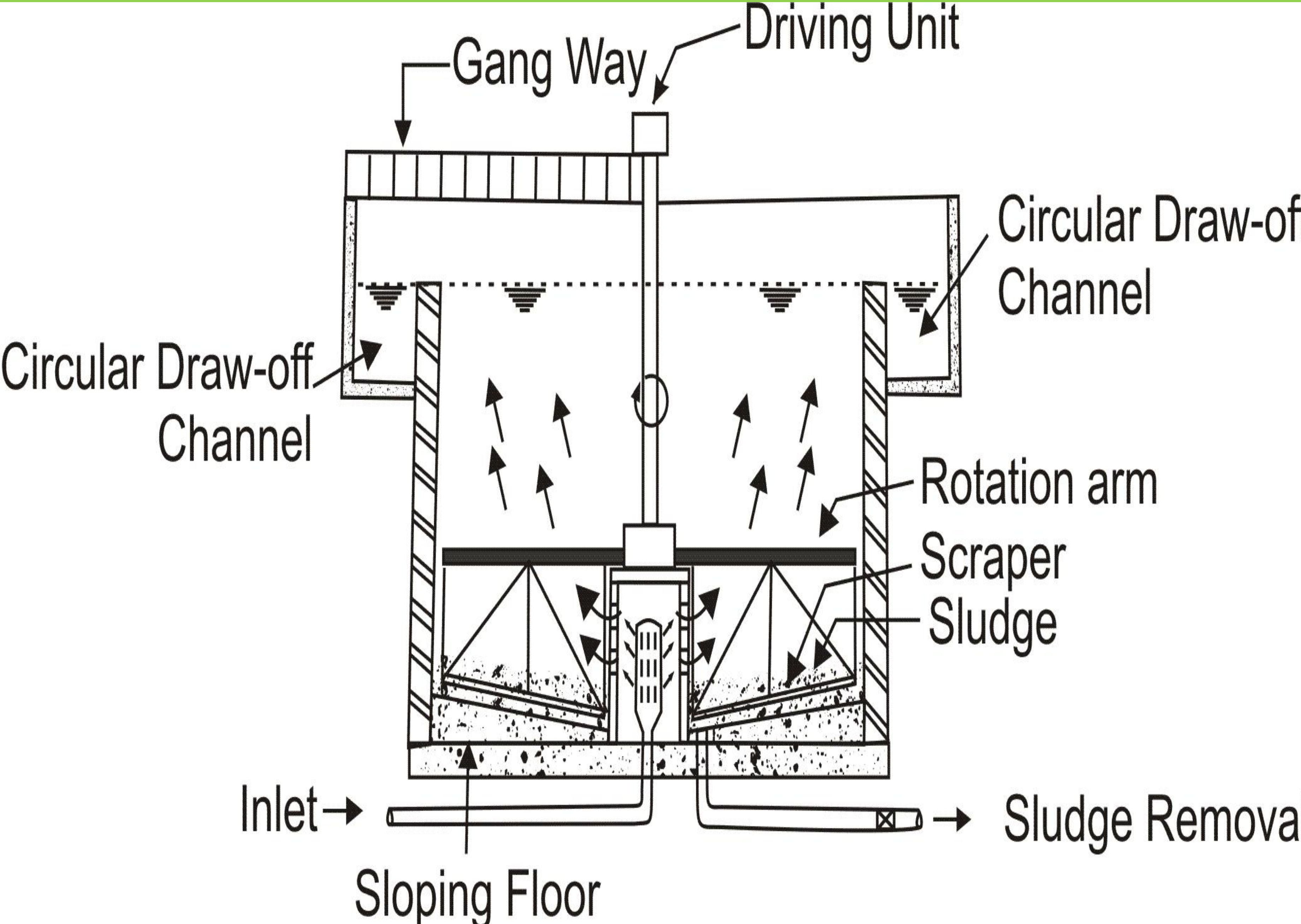
Fig 7.4 Rectangular sedimentation tank

Circular sedimentation tank

The circular sedimentation tank may be radial or spiral flow. But the tank with radial flow is commonly adopted. In this tank, the water is allowed to enter through the pipe which is provided at its centre. The water flows upwards gently through the openings. The water is collected at the circular draw-off channel from where it is taken to next unit through the outlet pipe as shown in Fig.7.5. The sediments or sludge are settled down at the bottom of the tank.

Circular sedimentation tank

A driving unit is provided for rotating an arm which consists of scrapper. The circular motion of the scrapper helps the sludge to discharge through the sludge removal pipe.


Driving Unit
Gang Way
Circular Draw-off Channel
Circular Draw-off Channel
Rotation arm
Scraper
Sludge
Inlet
Sludge Removal
Sloping Floor

Hopper bottom sedimentation tank

In this tank, the water is allowed to enter through the centrally placed inlet pipe and is deflected downwards by the action of a deflector box. The water flows downwards inside the box and then it rises in upward direction through the opening between the box and the wall of the tank. When the water rises in upward direction, the particles having specific gravity more than 1.0 cannot follow the path and ultimately settle down at the bottom of the tank due to the property of hydraulic subsidence.

Hopper bottom sedimentation tank

The sludge settles at the bottom of the hopper, from where it is removed with the help of a sludge pipe connected to a sludge pump.

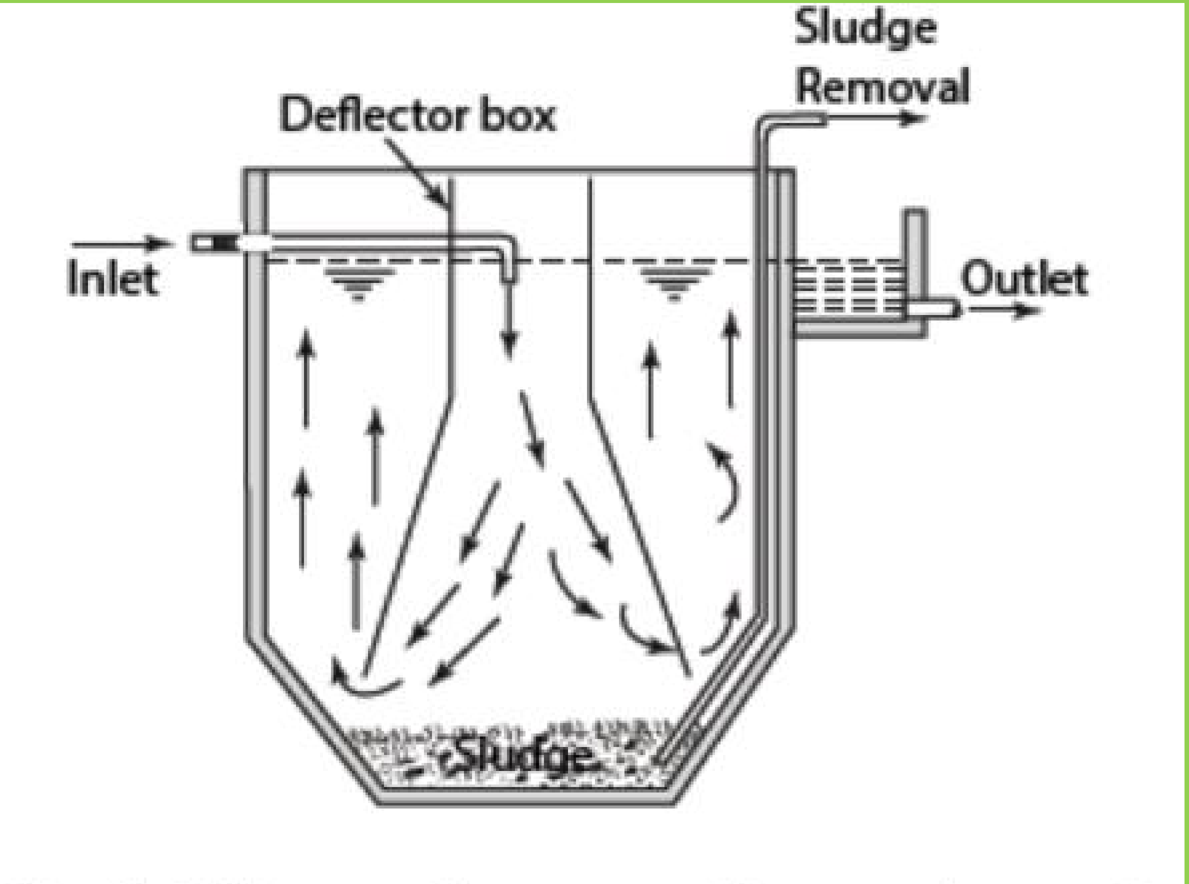


Fig. 7.6 Hopper bottom sedimentation tank

SEDIMENTATION WITH COAGULATION (CLARIFICATION)

In plain sedimentation the heavier particle settle down. However, fine particles take many hours or some days to settle down. Colloidal particles which are of size finer than 0.0001 mm carry electrical charges on them. These are continuous in motion and will never settle down under gravity. As it is not possible to have detention period in the sedimentation tank more than 3 to 4 hours, coagulation is adopted.

Clarification

The chemically assisted sedimentation comprises several separate processes of treatment which go to make up the complete system known as clarification. It is achieved in three stages:

i. Addition of coagulants
ii. Formation of floc
iii. Sedimentation

Clarification

In the first stage, it is usual to introduce chemical at some point of high turbulence in the water. This may be achieved by either passing the water over a weir or through an orifice plate or may be produced by mechanical stirring using a flash ‘mixer’. The term coagulation is used to mean the first stage in the formation of precipitate while flocculation consists of building up the particles of floc to larger size which can be removed by sedimentation in the third stage of clarification.

Principle of coagulation

The principle of coagulation has been explained by the following phenomenon.

a) Floc formation

b) Electric charges

a. Floc formation

When a coagulant is added to the water and mixed thoroughly, a thick gelatinous precipitate is formed which is insoluble in water. This precipitate is called floc. As the floc settle down, it attracts and arrests the colloidal particles and brings them down.

b. Electric charges

It is observed the ions of floc posses positive charge. Colloidal particles are negatively charged ions. So, the floc attracts the colloidal particles while it travels towards the bottom of tank.

Flocculation

Flocculation is essentially an operation designed to force agitation in the fluid and induce coagulation. Basically, flocculation is a slow mixing or agitating process in which destabilized colloidal particles are brought into intimate contact in order to promote their agglomeration.

Common Coagulants

The following are the usual chemicals which are commonly used for the coagulation.

a) Aluminum Sulphate or alum
b) Chlorinated copperas
c) Ferrous Sulphate and Lime
d) Magnesium Carbonate
e) Polyelectrolyte and
f) Sodium Aluminate

a. Aluminum Sulphate (Alum)

The chemical composition of Aluminum Sulphate is $Al_2(SO4)_3$ $18H_2O$. It is commonly known as alum. It is available in the form of a solid lump, but applied in a powder or liquid form. It is very effective if bicarbonate alkalinity is present in water. If the water possesses no alkalinity, some amount of lime is to be added to water. When alum is mixed with water, a chemical reaction takes place and aluminum hydroxide (Al(OH)3), calcium sulphate (CaSO4) and carbon Dioxide (CO2) are formed.

a. Aluminum Sulphate (Alum)

The aluminum hydroxide is insoluble in water and it forms the floc. It is effective between pH value 6.5 and 8.5. The dosage of this coagulant depends on various factors such as turbidity, colour, pH-value, etc. In practice the dosage of alum varies from 10 to 30 mg per liter. Alum is preferred over other coagulants because it reduces taste and odour in addition to turbidity in water.

b. Chlorinated Copperas

When chlorine is mixed with the solution of Ferrous sulphate, a chemical reaction takes place which forms ferric sulphate [$Fe_2(SO_4)_3$] and ferric chloride [$FeCl_3$]. The combination of these two compounds is known as chlorinated copperas. Both the compounds are effective for the formation of floc. Sometimes, ferric sulphate and ferric chloride may be applied independently with lime. In that case, ferric hydroxide [$Fe(OH_3)$] is formed which is also effective for the formation of floc. The ferric sulphate is effective for pH-value 4 to 9 and ferric chloride is effective for pH value 3.5 to 6.5.

c. Ferrous Sulphate and lime

The ferrous sulphate and lime when mixed with water, a chemical reaction takes place and ferrous hydroxide [Fe(OH)2] is formed. This compound is again oxidized by the dissolved oxygen in water and finally ferric hydroxide is formed. This ferric hydroxide forms the floc.

d. Magnesium carbonate and lime

When magnesium carbonate and lime are dissolved in water, magnesium hydroxide and calcium carbonate are formed. Both these are soluble in water, resulting in the formation of sludge which is in slurry form. Due to this, it is not commonly used.

e. Polyelectrolytes

Polyelectrolytes are high molecular weight water-soluble polymers. The amount of polyelectrolyte used is very small in reaction to the amount of primary coagulant. The usual dosage of polyelectrolytes is 1ppm.

f. Sodium Aluminate

Sodium Aluminate Na2Al2O4 is alkaline in reaction and is used very much less often than alum because of its cost. This coagulant removes both temporary and permanent hardness, and is effective for a pH range of 6 to 8.5 naturally available in water.

METHOD OF FEEDING COAGULANTS

The coagulant may be fed either in powder form (dry feeding) or in solution form (wet feeding). The way of feeding depends upon the characteristics of the coagulant and the convenience of its application, the dosage of coagulant and the size of the treatment plant

7.8 MIXING DEVICES

After the addition of the coagulant to raw water, the water needs to be thoroughly and vigorously mixed so that the coagulant gets fully dispersed into the entire mass of water. The various mixing devices indicated below are adopted.

1. Mixing basins with baffle walls
2. Mixing basins with mechanical means
3. Mixing channel
4. Hydraulic channel
5. Compressed air
6. Centrifugal pumps

1. Mixing basins with Baffle Walls

These are rectangular basins or tanks which are provided with baffle walls. The disturbance created by the presence of baffle walls in the path of the flowing water causes vigorous agitation of water which results in thorough mixing of water with the coagulant. Such basins are of two types as indicated below.

a) Horizontal or round the end type

b) Vertical or over and under type

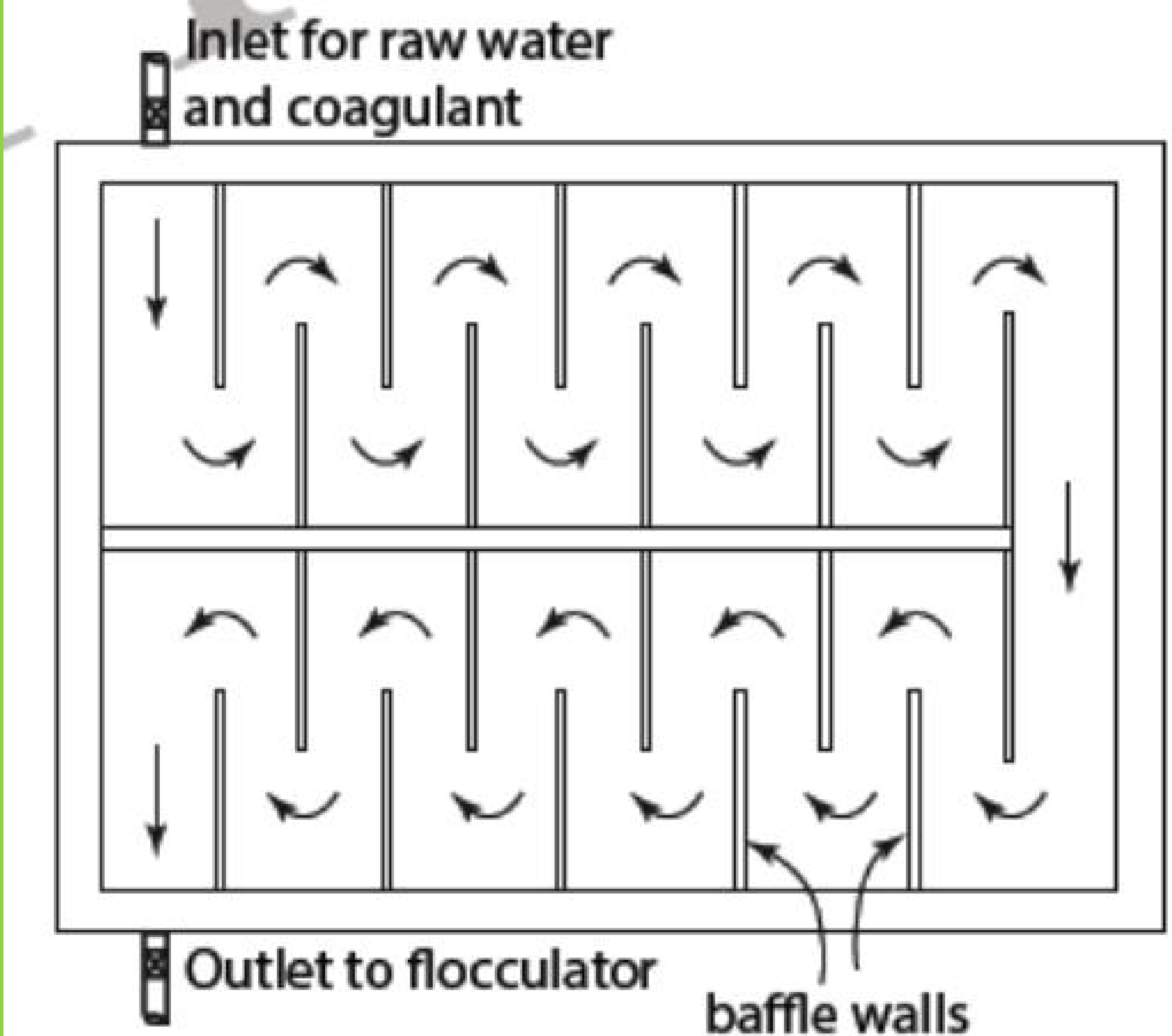


Fig. 7.7 Round the End type mixing basin

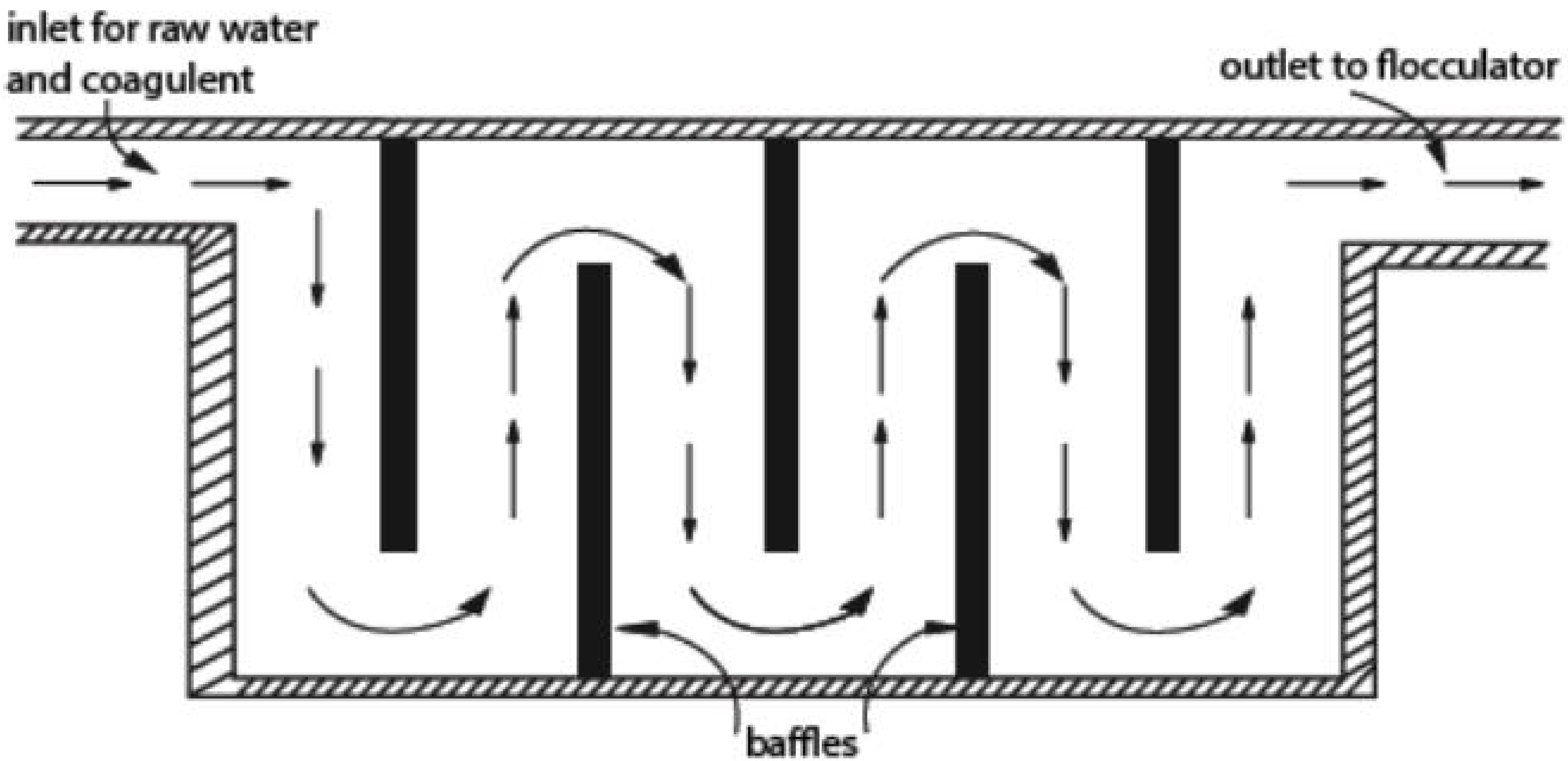


Fig. 7.8 Over and Under type mixing basin

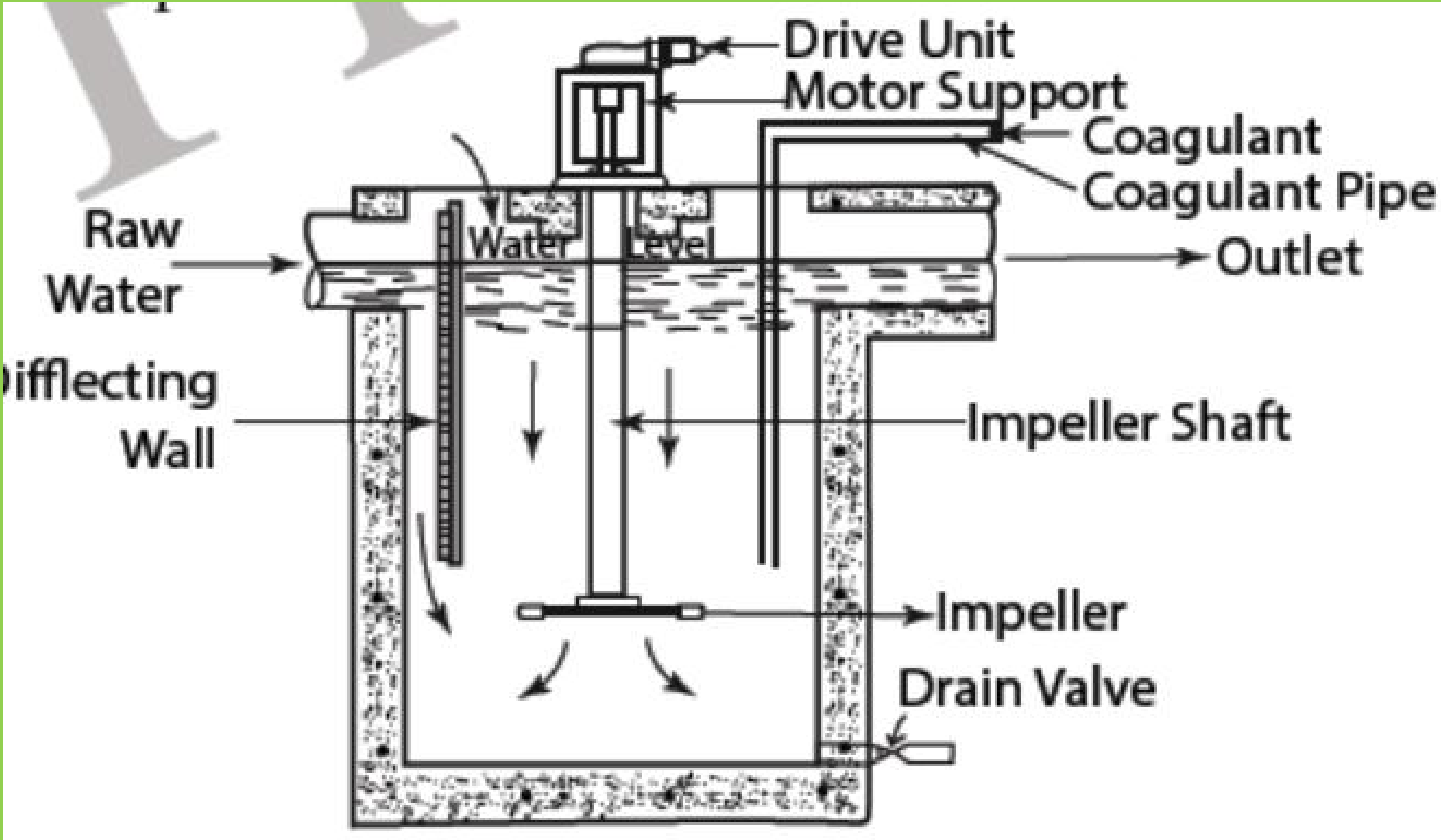


Fig. 7.9 Flash Mixer

2. Mixing basins with Mechanical Means

A flash mixer is a typical mixing basin with mechanical driven impeller or paddle. It contains a deep, circular or square tank which is provided with a paddle fixed at the lower end of a vertical shaft which is driven by an electric motor. The usual ratio of impeller diameter to tank diameter is 0.2 to 0.4. The impeller is rotated at a speed of more than 100 rpm imparting a tangential velocity greater than 3 m/s at the tip of the impeller blade. The ratio of the tank height to diameter is 1:1 to 3:1 and a detention period of ½ to 1 minute are provided.

3. Mixing channel

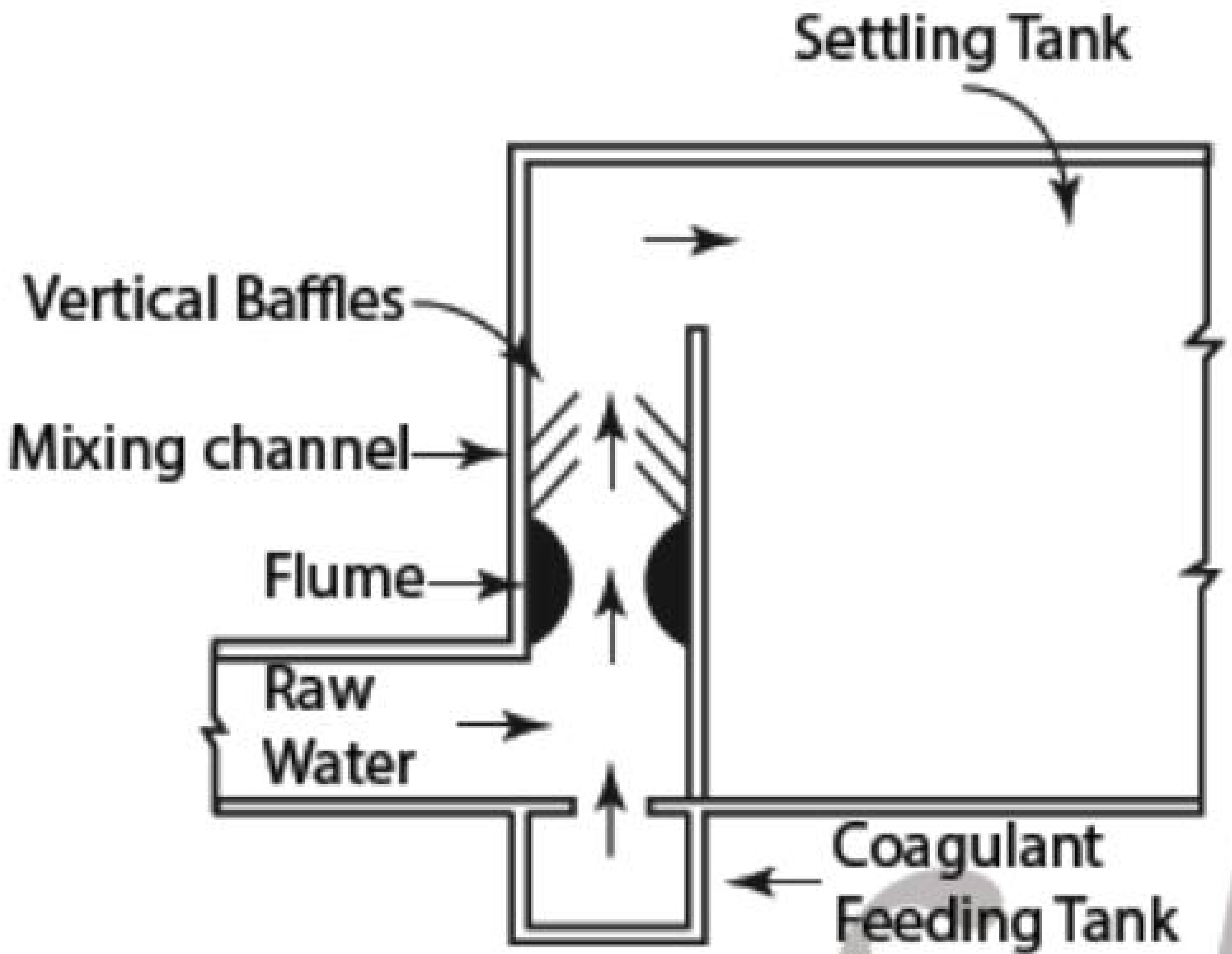


Fig.7.10 Mixing Channel

Mixing Channel

In a mixing channel, the coagulant is fed through the feeding tank which then strikes with the vertical baffles provided creating a violent agitation which causes thorough mixing of water with the coagulant. Flume is provided to create a hydraulic jump to create turbulence.

4. Hydraulic Jump Method

Flume is provided with the channel in which the coagulant and the raw water flow. On passing through the flume, hydraulic jump is created which causes thorough mixing of the coagulant and the raw water.

5. Compressed Air

Water is fed with coagulant in the mixing basin in this method. The compressed air is diffused from the bottom of the mixing basin which rises through the water leads to mixing of the coagulant with the water.

6. Centrifugal Pumps

In most of the cases, centrifugal pump is used to raise the raw water to the settling tank. The required dose of the chemical therefore canbe added to suction line of the pump. When water fed with the coagulant passes through the impeller of the pump, mixing is accomplished by agitation. However, after the water comes out, some gentle agitation is required to get good results, and to accelerate coagulation and sedimentation.

7.9 FLOCCULATION TANKS

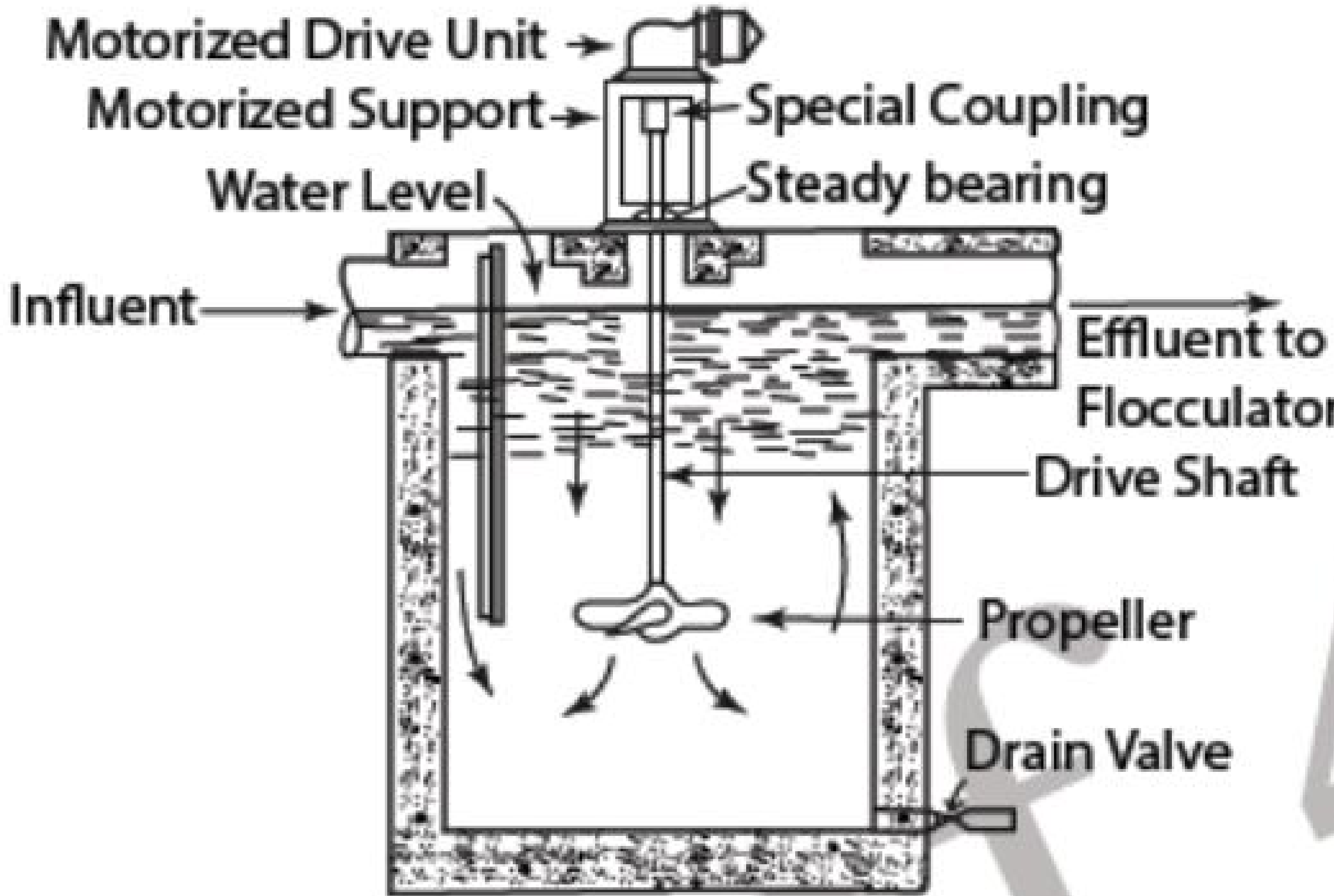


Fig. 7.11 Mechanical Flocculators

Mechanical Flocculators

Water is taken from the mixing basin to flocculators for flocculation. In a flocculator, slow stirring of water is brought about to permit the build up and agglomeration of the floc particles. Mechanical flocculators are provided with paddles for stirring of water. Thus, these are also known as paddle flocculators. Depending upon the direction of flow of water in the tanks the mechanical flocculators may be classified as

(i) longitudinal flow flocculators,and

(ii) vertical flow flocculators.

The design criteria for these flocculators are as follows:

- Velocity of flow in the flocculator : 0.2 to 0.8 m/minute; normally 0.4 m/minute.
- Detention period : 10 to 40 minutes; normally 30 minutes
- Depth of tank : 2 to 4.5 m.
- Total area of paddle : 10 to 25% of the cross-sectional area of the tank.
- Outlet flow velocity to settling tank : 0.15 to 0.25 m/s (to prevent settling or breaking of flocs).

CLARIFIER

The water from the flocullator is taken on to the clarifier whose design is same as those of plain sedimentation tank. Here the water is retained for sufficient period of time for the purpose of settlement of floc along with the suspended particles. 2-2.5 hrs is adopted as the detention period with Surface Overflow Rate(SOR) 30-40 m3/day/m2.

7.11 DORR CLARIFLOCCULATOR

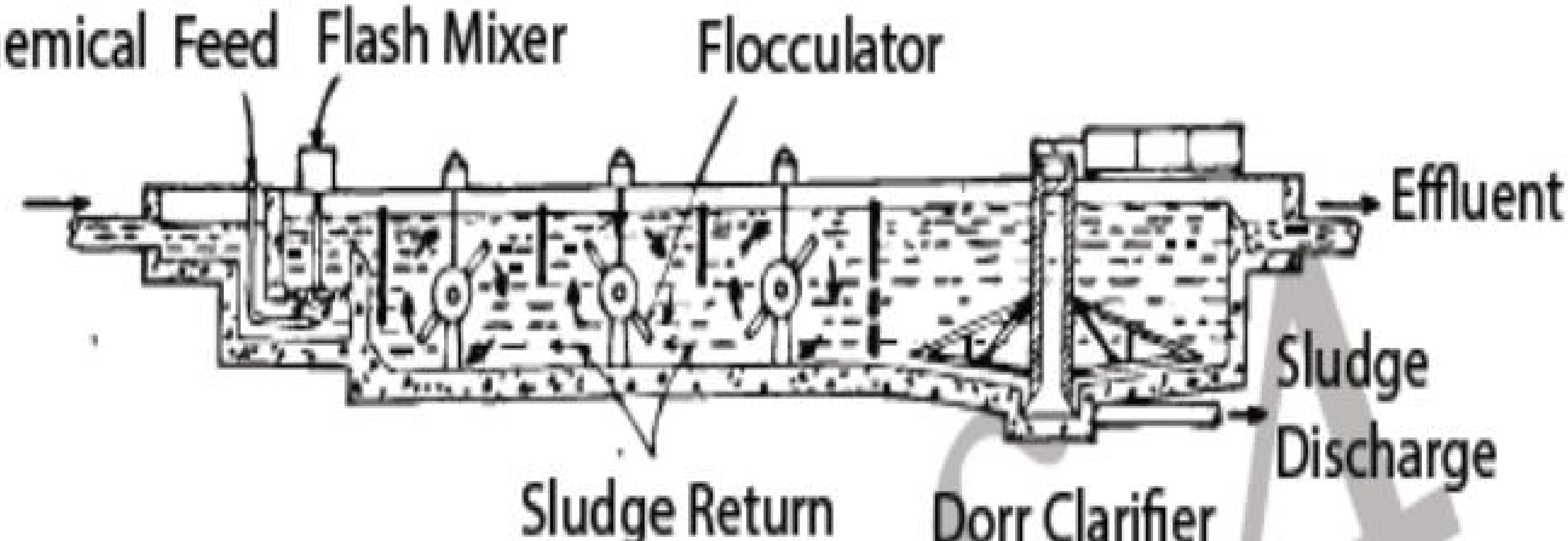


Fig. 7.12 Dorr Clariflocculator

DORR CLARIFLOCCULATOR

It consists of coagulant feed, flash mixer, flocculator and clarifier as a single unit. The coagulant is fed which is then mixed in the flash mixer, flocculation takes place in the flocculator and settling of particles take place in clarifier within a single unit called dorrclariflocculator.

7.12 JAR TEST

For the formation of good floc, the dose of coagulant added for the

7.12 JAR TEST

For the formation of good floc, the dose of coagulant added for the purpose of coagulation and sedimentation must be optimum. The optimum dose of coagulant is determined in the laboratory by the jar test taking alum as the coagulant. The jar test apparatus consist of the rotary device known as multiple stirrers with paddles, motor to drive those stirrers and beakers with the capacity of about 1000ml. An equal volume of water is fed in all the six beakers provided on which the jar test is to be performed.

The stirrers are then lowered so that they are immersed in the water. The dosage of alum with varying concentration as 5,10,15,20,25,30 mg/L are added in the different beakers and the paddles are rotated at the speed of 100rpm for 1 to 2 minutes which causes thorough mixing of coagulant with water. The speed is then lowered to 20rpm in order to form a good floc. The floc are allowed to settled down and residual turbidity of each jar is determined. The dosage of alum corresponding to lowest residual turbidity is defined as the optimum dosage.

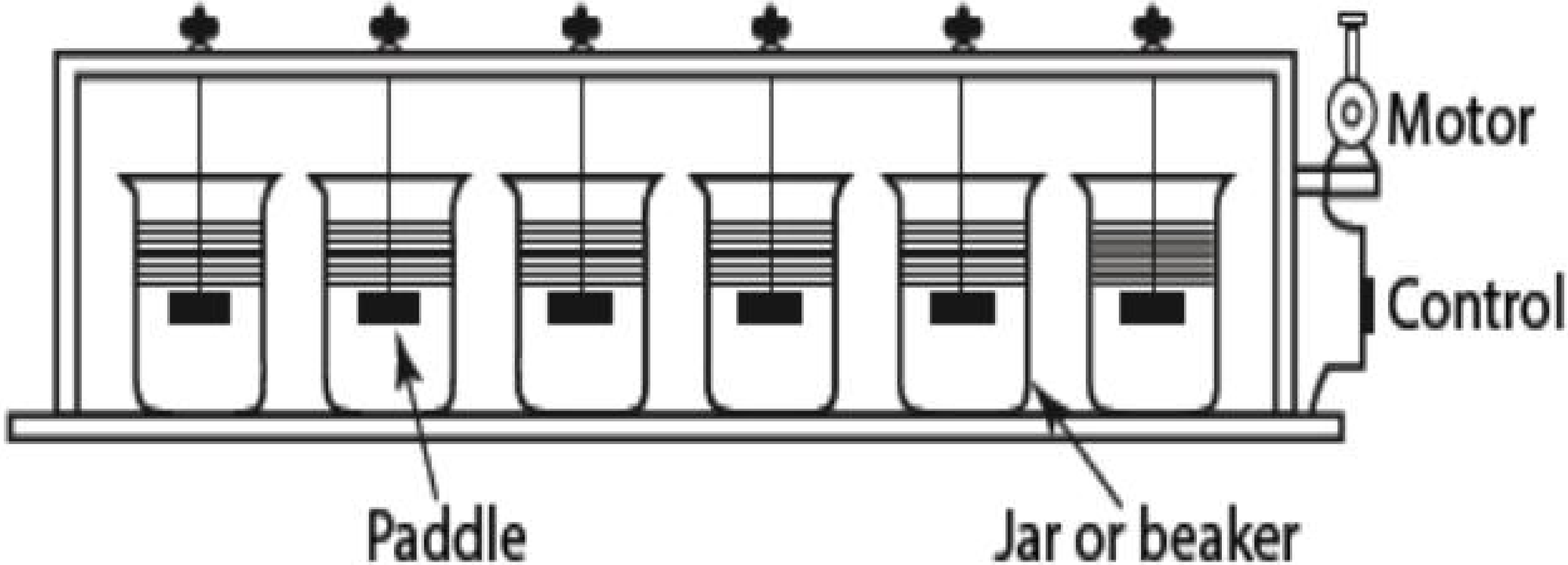


Fig.7.13 Jar Test Apparatus

Problem 7.1. A water-supply project has to supply water for a town whose daily peak demand is 15 million liters. Design a suitable sedimentation tank. Assuming the velocity of flow in the tank is 250mm/min. and the detention period of 4 hours. Assume depth of tank as 4 m and a free board of 0.5 m.

Solution:

Quantity of water to be treated in 24 hours = $15 \text{ X } 10^6$

Detention period = 4 Hours

$\therefore$ Quantity of water to be detained in 4 hours $\dfrac{15\times 10^6}{24}\times 4$

$= 2.5 \text{ X } 10^6$

$= 2.5 \text{ X } 10^3 \text{ M}^3$

$\therefore$ Capacity of tank= 2500 M^3

Velocity of flow = 250 mm/min

= 0.25 m/min

Detention period = 4 hours = 4X 60 min

Length of tank required = 0.25 X 4 X 60

= 60 mtr.

Area of cross-section of tank required $\frac{2500}{60} = \frac{250}{6} M^2$

Depth of water tank = 4 - 0.5 = 3.5 mtr.

Width of water tank $\frac{250}{6 \times 3.5} = 11.9\, mtr$

Say 12 mtr.

Add 20% of length for overcoming inlet and outlet conditions.

$\therefore$ Net length = 1.2 X 60 = 72 mtr.

$\therefore$ Dimension of tank required is 72m X 12m X 4m

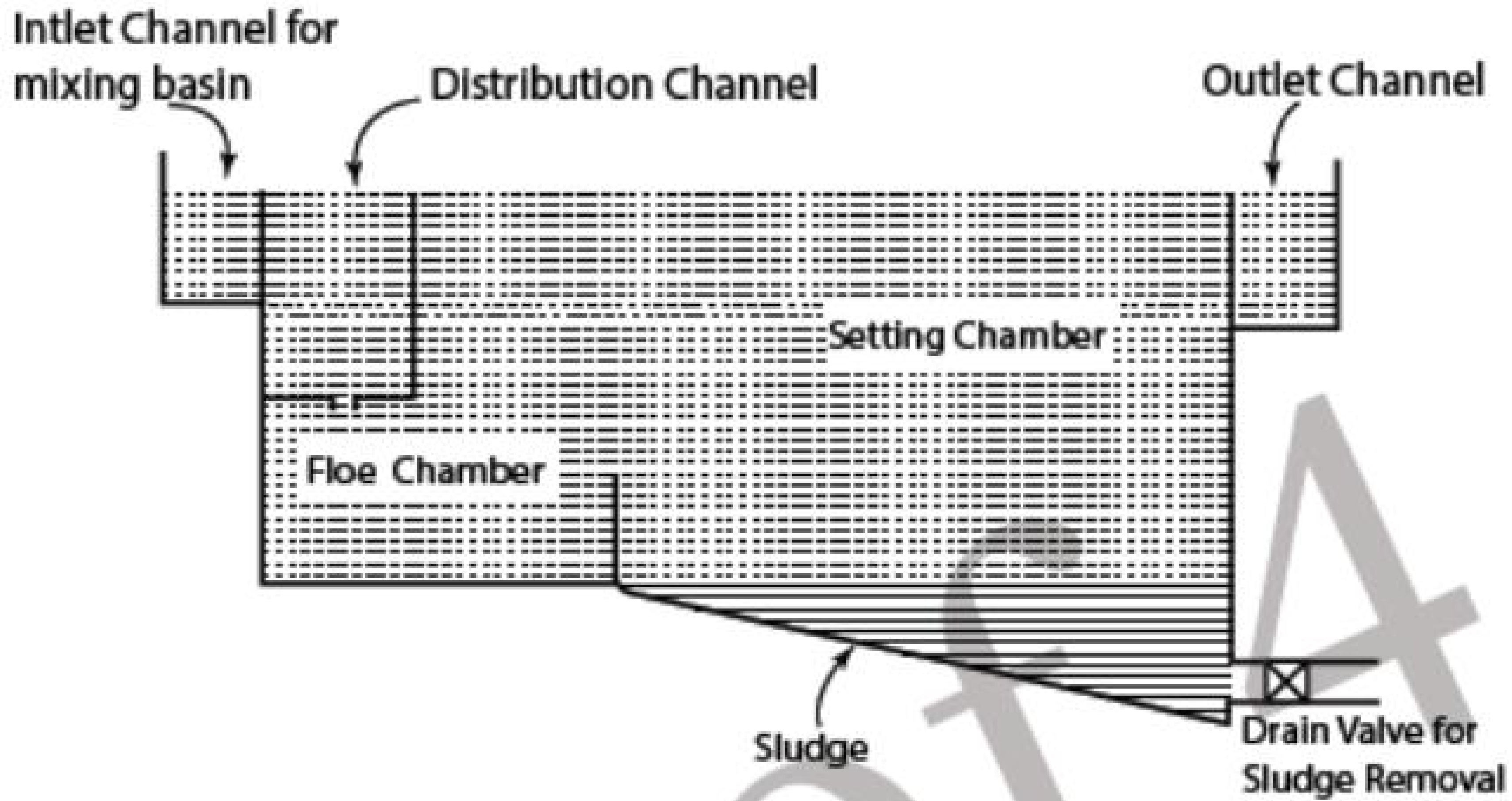


Fig.7.14 Coagulation cum sedimentation tank

It has been possible to combine the flocculation chamber along with the sedimentation tank as shown in figure 7.14 such a tank is known as coagulation-sedimentation tank. In such a tank, a plain floc-chamber without any mechanical devices is provided before the water enters the sedimentation chamber. The detention period for the floc chamber is kept about 15 to 40 minutes, and that for the settling tank as about 2 to 4 hours.

The depth in the floc chamber may be kept about half that of in the settling chamber. The water from the mixing basin enters this tank and clarified water comes out of the outlet end. The design principles for such a tank are the same as those applied to a plain sedimentation tank.

Problem 7.2: The population of a town is 100,000 and the average per capita demand is 135 lpcd. Design the coagulation cum sedimentation tank for supplying water to the town. The maximum demand may be taken as 1.5 times the average demand. Detention period is 5 hrs. for settling tank and 30 min for flocculation chamber. Flow rate is 900ltrs/hr/m^2 of plan area.

Solution:

First of all, we will design the settling tank and then the floc chamber.

(i) Design of settling tank

Average daily consumption = Population X Per capita demand

= 100000 X 135

= 13.5 X 10^6 litres

Maximum daily demand = 1.5 X 13.5 X 10^6

= 20.25 X 10^6 litres

Detention period = 5 hrs

$\therefore$ Quantity of water to betreated in 5 hrs = $\dfrac{20.25 \times 10^6}{24} \times 5$

= 4.22 X 10^6 ltrs.

= 4.22 X 10^3 m^3

Hence capacity of tank = 4.22 X 10^3 m^3

Over flow rate $= 900$ ltrs/Hr./m^2

ie $$\frac{Q}{B \times L} = 900$$

Where $$Q = \frac{4.22 \times 10^6}{5} = 844 \times 10^3 \, ltrs/Hr.$$

$$\therefore B \times L = \frac{844 \times 10^3}{900} = 937.8 \, m^2$$

Provide width of tank = 15 mtr. and depth = 4 mtr.

$\therefore$ Length of tank = 937.8/15 = 62.5 m

Say 65 mtr.

Hence, use a tank of 65m X 15m X 4m. Provide extra depth for sludge storage, say use 4.5m depth at the starting end and $4.5 + \frac{1}{60}$ x 65 = 5.8 m at downstream end (using 1 in 50 slope). Use a free board of 0.5m above the water level.

Detention period $= 30 \text{ min}$

$$\therefore \text{Capacity of chamber} \frac{20.25 \times 10^3}{24} \times \frac{30}{60}$$

$$= 421.88 \text{ cumecs}$$

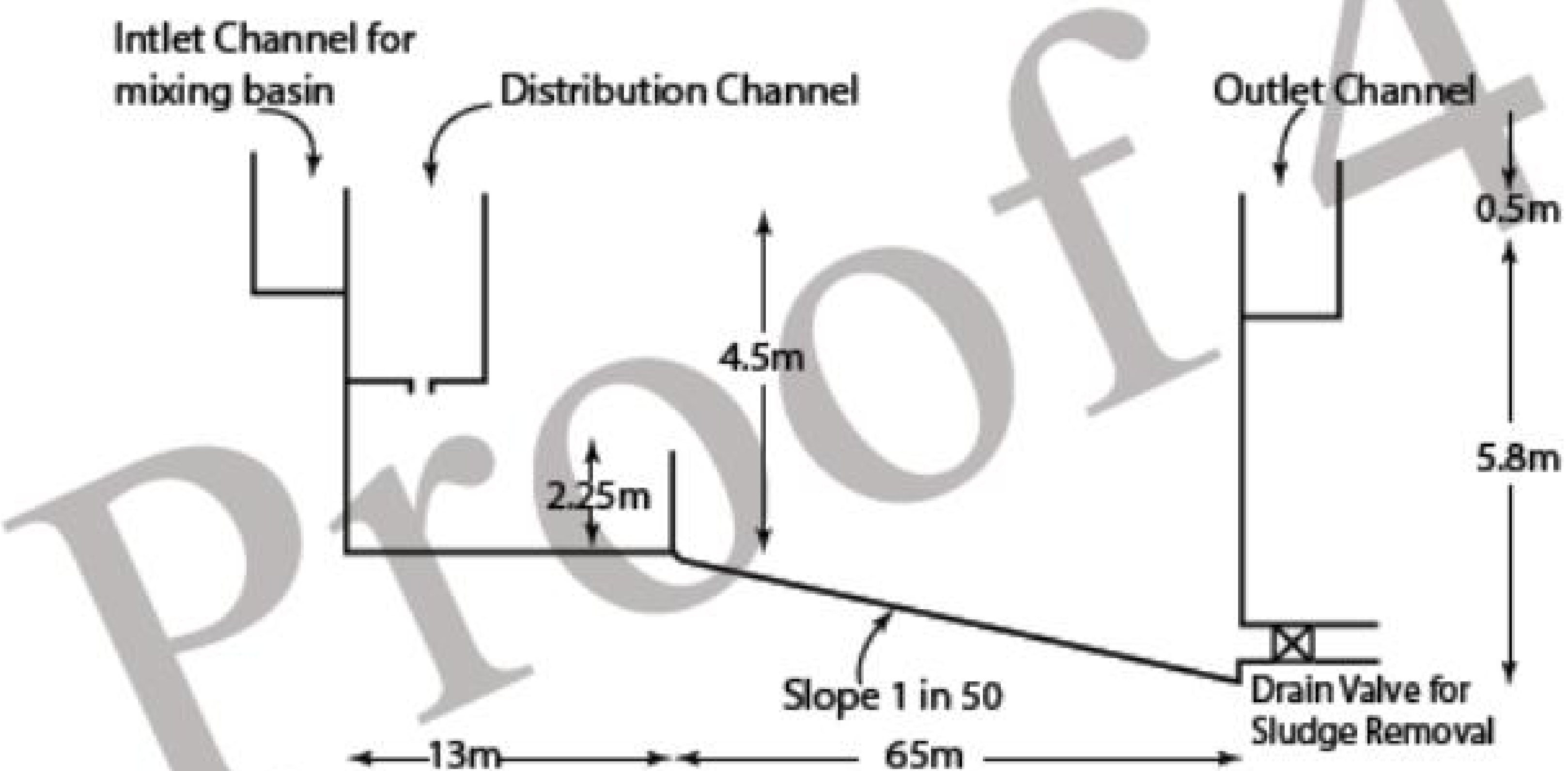
$$\text{Plan area required} = \frac{\text{Capacity}}{\text{Depth}}$$

$$\frac{421.88}{2.25} = 188m^2$$

Provide same width of 15 mtr.

$$\therefore \text{length of tank} = \frac{188}{15} = 13mtr..$$

The details and dimensions are shown in figure given below.



MODULE IV

FILTRATION OF WATER

Screening and sedimentation removes a large percentage of the suspended solids and organic matter present in raw water. But the resultant water will not be pure, and may contain some very fine suspended particles and bacteria present in it. To remove the remaining impurities, the water is filtered through the beds of fine granular material such as sands etc. The process of passing the water through beds of granular materials (called filters) is known as filtration. Filtration may help in removing colour, odour, turbidity, and pathogenic bacteria from water.

THEORY OF FILTRATION

The filters purify the water under four different processes. These actions are summarised below.

i) Mechanical straining

The suspended particles present in water, and which are of bigger size than the size of the voids in the sand layers of the filter, cannot pass through these voids and get arrested in them. The resultant water will, therefore, be free from them. Most of the particles are removed upper sand layers. The arrested particles including the coagulated flocs forms a mat on the top of the bed, which further helps in straining out the impurities.

ii) Flocculation and sedimentation

It has been found that the filters are able to remove even particles of size smaller than the size of the voids present in the filter. This fact may be explained by assuming that the void spaces act like tiny coagulation-sedimentation tanks. The colloidal matter arrested in these voids is a gelatinous mass and, therefore, attract other finer particles.

iii) Biological metabolism

Certain micro-organisms and bacteria are generally present in the voids of the filters. They may either reside initially as coatings over sand grains, or they may be caught during the initial process of filtration. Nevertheless, these organisms require organic impurities (such as algae, plankton etc.) as their food for their survival. These organisms therefore, utilise such organic impurities and convert them into harmless compounds by the process of biological metabolism.

iv) Electrolytic Charges

The purifying action of filter can also be explained by the theory of ionisation. According to this theory, a filter helps in purifying the water by changing the chemical characteristics of water. This may be explained by the fact that the sand grains of the filter media and the impurities in water carry electrical charges of opposite nature. When these oppositely charged particles and impurities come in contact with each other, there by changing the character of the water and making it purer. After a certain interval, the electric charges of sand grains get exhausted and have to be restored by cleaning the filter.

FILTER MATERIALS

Sand, either fine or course, is generally used as filter media. The layers of sand may be supported on gravel.

Uniformity coefficient of sand is the ratio of sieve size in mm through which 60% of the sample of sand by weight will pass to the effective size of sand.

$$\therefore \text{Uniformity coefficient} = \frac{D_{60}}{D_{10}}$$

CLASSIFICATION OF FILTERS

The filters are classified in two categories

i. Slow sand filter

ii. Rapid sand filters

The rapid sand filter further classified in to

a) Gravity type rapid filter

b) Pressure type rapid filter

The above classification is based on the rate of filtration.

On the basis of gravity and pressure the filters may be classified as;

On the basis of gravity and pressure the filters may be classified as;

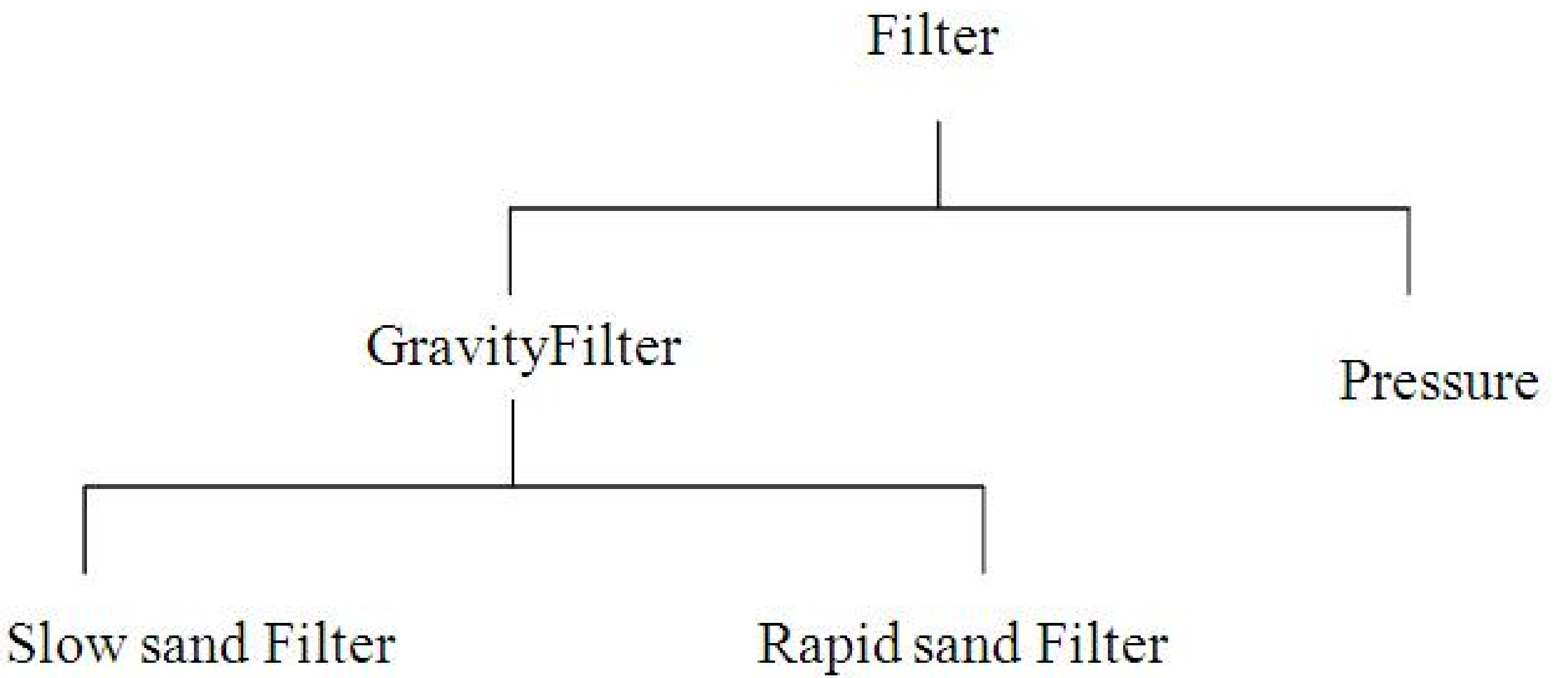


Fig.8.1 Classification of filters

Combining above two classifications we will study the following three types of filters.

i. Slow sand filter

ii. Rapid sand filter

iii. Pressure filter

Combining above two classifications we will study the following three types of filters.

i. Slow sand filter

ii. Rapid sand filter

iii. Pressure filter

SLOW SAND FILTER

The theory of slow sand filter is based on the principle that if water is allowed to percolate slowly through the filtering media, then the biological, chemical and physical characteristics of water are improved considerably. And it permits sufficient time for those improvements. That's why; the water is allowed to enter the filter bed slowly by suitable inlet arrangement. As the filtration takes much time, it is not suitable for large scale. It is suitable for drinking water only for small towns.

A slow sand filter consists of the following parts:

i. Enclosure tank
ii. Under drainage system
iii. Base material
iv. Filter Media or Sand
V. Appurtenance

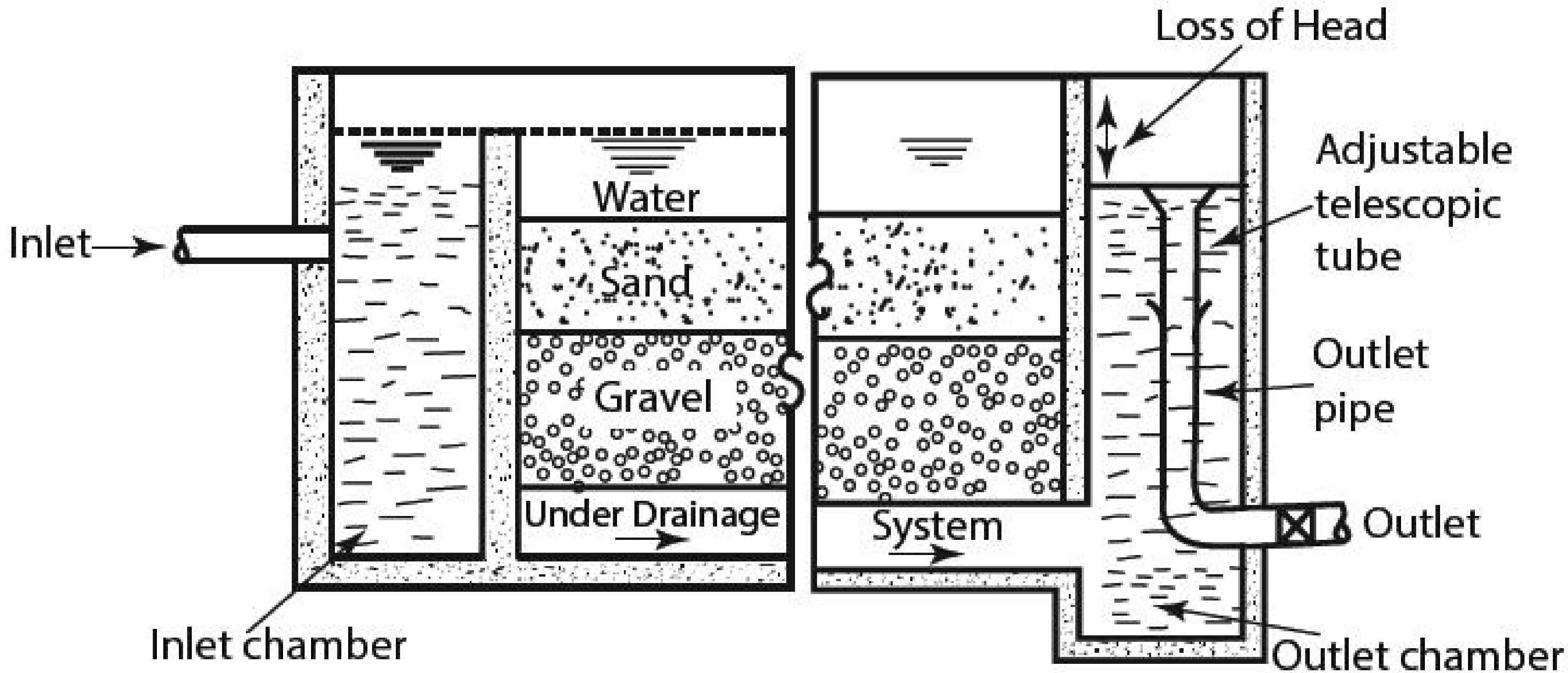


Fig. 8.2 Slow sand filter

i) Enclosure tank

It consists of an open water-tight rectangular tank, made of masonry or concrete. Bed slope is kept at about 1 in 100 towards the central drain. The depth of tank may vary from 2.5 to 3.5m. The plan area of the tank may vary from 100 to 2000 m^2.

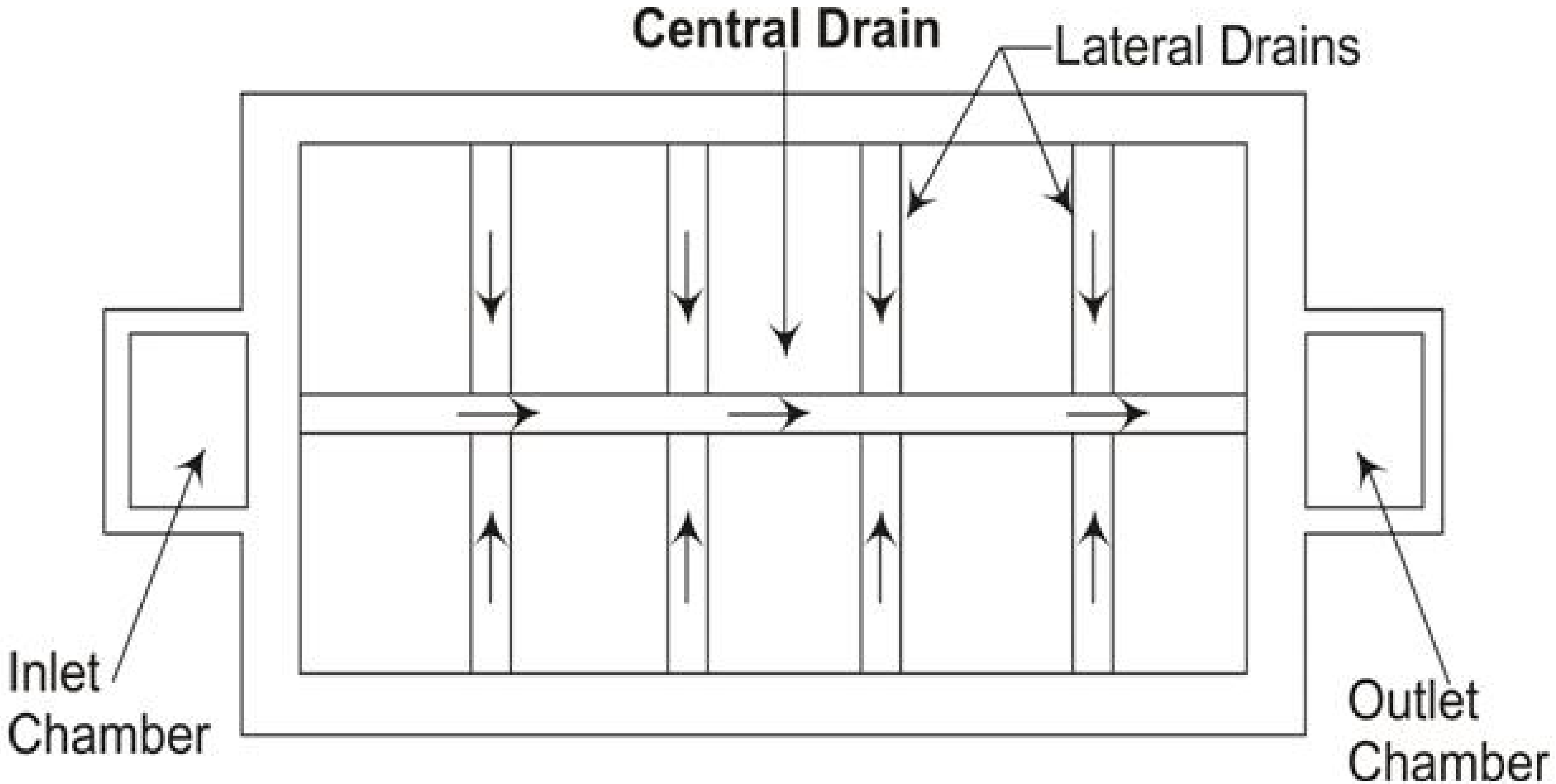


Fig. 8.3 Under drainage system

The under drainage system consists of a central drain and lateral drains. The laterals are open jointed pipe drains or some other kind of porous drains placed 3 to 5m apart and sloping towards a central drain. The laterals collect the filtered water and discharge it into the main drain, which leads the water to the filtered water well.

iii) Base material

The base material is gravel, and it supports sand. It consists of 30 to 75cm thick gravels of different sizes, placed in layers. The size of gravel in the bottom-most layer is generally kept 40 to 65mm.

iv) FilterMedia or Sand

The filtering media consists of sand layers about 90 to110cm. In depth, and placed over a gravel support. The effective size of sand varies from 0.20 to 0.30mm and uniformity coefficient of sand is about 2 to 3. The finer the sand, the better will be the efficiency of filter regarding removal of bacteria.

v) Appurtenance

Besides these arrangements, certain other appurtenances are provided for the efficient functioning of these filters. Vertical air pipe passing through layer of sand is for proper functioning of filtering layers. Devices used for measuring and controlling the ‘loss of head’ to be installed.

Operation

The water enters the filter through the inlet chamber. After filtration it is collected in outlet chamber. The outlet chamber is fitted with a telescopic tube. The filter is filled with water to a depth of 1-1.5m above the surface of sand. The water is passed through the layer at a rate of 100-150 lph/m2 which is same as the rate of feeding. The filter head which is the difference of level between the water-level in the filter tank and the outlet chamber is 100-150mm for a new or freshly cleaned filter.

Operation

But as the filter gets clogged, the head is increased by adjusting manually the telescopic tube up to a loss of head of 60mm. this gives a constant discharge through the filter. When the filter head exceeds the permissible value (up to 1.3 m) the filter needs cleaning. As the process of filtration goes on the filtering media gets clogged due to the impurities and the loss of head goes on increasing gradually. When the loss of head reaches a certain limit, the working of filter is stopped.

At that time the filter requires cleaning. For the purpose of cleaning, the layer of sand is scrapped to a depth of about 25mm and replaced by clean sand before the filter is stored again for service.

Advantages of slow sand filter

1. It can remove turbidity to the extent of 50 to 60 ppm
2. It can remove colour to the extent of 25 percent
3. It can remove bacteria to the extent of about 95 percent.

Disadvantages of slow sand filter

1. Slow rate of filtration
2. Large surface area required, hence costly.
3. Removal of colour and turbidity is less.

Problem 8.1

Design a slow sand filter from following data.

Population to be served = 50,000 persons

Per capita demand = 150 ltrs/head/day

Rate of filtration = 180 ltrs/hr./sq.m

Length of each bed = Twice the breadth

Assume maximum demand as 1.8 times the average daily demand. Also assume that one out of six will be kept as stand by.

Solution:

Average daily demand = Population X per capita demand

= 50000 X 150 ltrs/day

= 7.5×10^6

Maximum demand = $1.8 \times 7.5 \times 10^6$

= 13.5×10^6 litres/day

Rate of filtration = (180×24) ltrs/m^2/day

$$\text{Total surface area of filters required} = \frac{\text{Max. daily demand}}{\text{Rate of filtration}}$$

Solution:

Average daily demand = Population X per capitademand

$= 50000 \times 150$ ltrs/day

$= 7.5 \times 10^6$

Maximum demand $= 1.8 \times 7.5 \times 10^6$

$= 13.5 \times 10^6$ litres/day

Rate of filtration $= (180 \times 24)$ ltrs/m^2/day

$$\text{Total surface area of filters required} = \frac{\text{Max.daily demand}}{\text{Rate of filtration}}$$

$$= \frac{13.5 \times 10^6}{180 \times 24} = \underline{\underline{3125\ m^2}}$$

Area of each filter $= \frac{3125}{(6-1)} = \frac{3125}{5} = 625 m^2$

Since one out of six will be kept as standby.

$\therefore$ No. of units required = 6 -1=5

Now, if L is the length and B is the breadth of each unit.

$$2B \times B = 625$$

$$\therefore B = \sqrt{\frac{625}{2}} = 17.7m \text{ say } 18m$$

$$\therefore L = 2 \times 18 = 36m$$

Hence use six filter units with one unit as stand by, each unit of size 36m X 18m arranged in series with 3 units on either side.

Problem 8.1

Design a slow sand filter from following data.

Population to be served = 50,000 persons

Per capita demand = 150 ltrs/head/day

Rate of filtration = 180 ltrs/hr./sq.m

Length of each bed = Twice the breadth

Assume maximum demand as 1.8 times the average daily demand. Also assume that one out of six will be kept as stand by.

RAPID SAND FILTER

It is observed that the rate of infiltration is more in coarse sand than that in fine sand. So, the theory of rapid sand filter is based on the principle of increasing the rate of filtration by providing coarse sand as filter media. The filtration head is also increased to increase the pressure head and the rate of filtration.

We have seen in slow sand filter that the rate of filtration is very low and it requires more space for installation. To overcome these difficulties, the fine sand in slow sand filter is replaced by coarse sand to achieve greater percolation and to reduce surface area.

Rapid sand filters are of two types.

Gravity type

Pressure type

We have seen in slow sand filter that the rate of filtration is very low and it requires more space for installation. To overcome these difficulties, the fine sand in slow sand filter is replaced by coarse sand to achieve greater percolation and to reduce surface area.

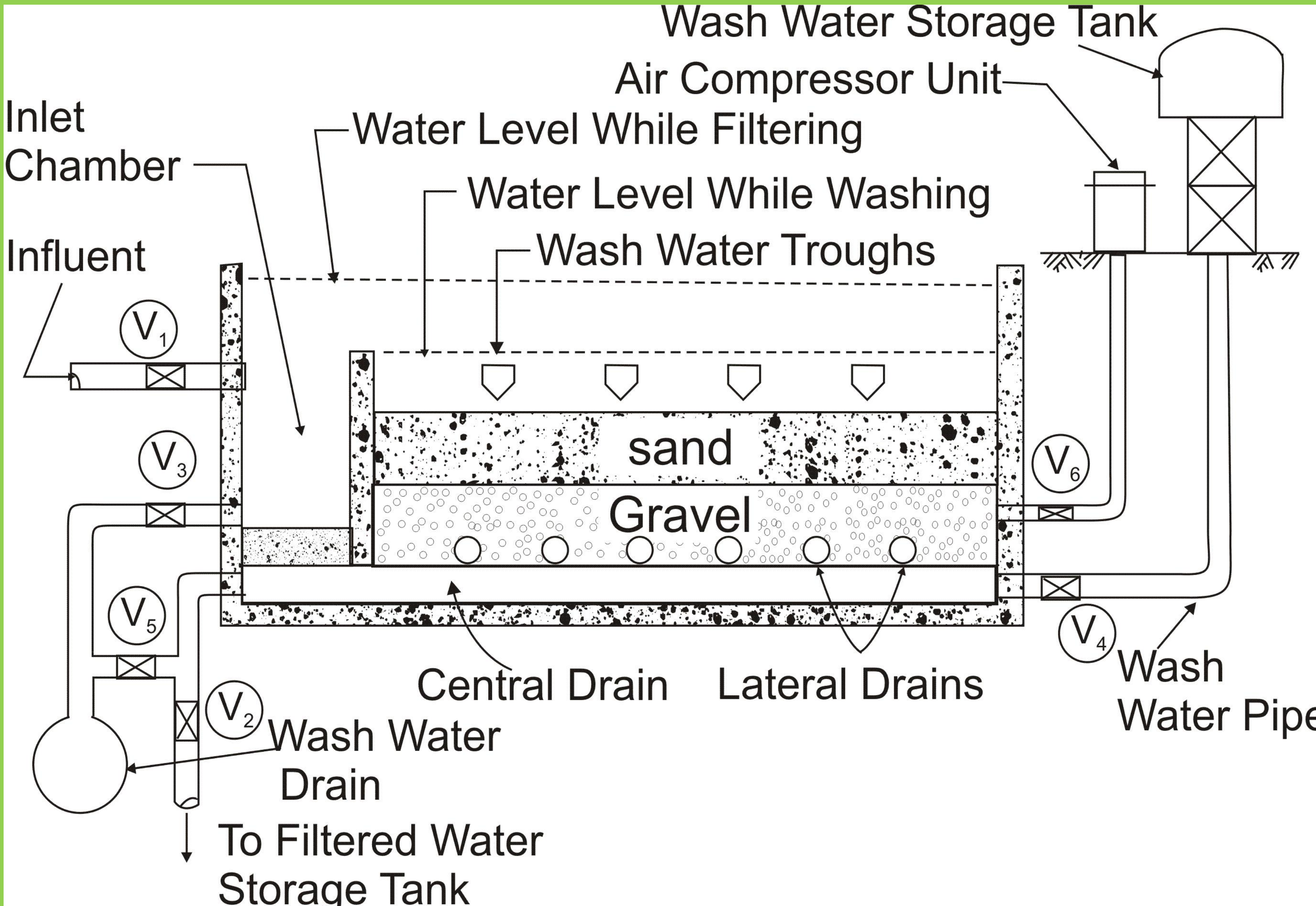
Rapid sand filters are of two types.

Gravity type

Pressure type

The gravity type rapid sand filters are most commonly used in water supply projects.

Usually, rapid sand filters are preceded by sedimentation with chemical coagulation.


Wash Water Storage Tank
Air Compressor Unit
Inlet
Chamber
Water Level While Filtering
Water Level While Washing
Wash Water Troughs
Influent
V1
V3
V6
sand
Gravel
V5
V4
V2
Central Drain
Lateral Drains
Wash
Water Pipe
Wash Water
Drain
To Filtered Water
Storage Tank

Essential features of rapid sand filters

- Enclosure tank
- Under drainage system
- Base material
- Filter media or sand
- Appurtenance

Enclosure tank: It consists of a water tight tank constructed with brick masonry. The inside surface is plastered with rich cement mortar (1:3) with water proof compound and finished with neat polish. The depth varies from 2 to 4m. The surface area depends on the volume of water to be filtered. However, the surface area varies from 30 to 60 m2.

Under Drainage System: The under drainage system consists of central drain and perforated lateral drains. The lateral drains are connected to the central drain from both sides and they are placed at a distance of 30cm centre to centre. Generally, G.I. pipes of required diameter are used in under drainage system.

Base Materials: Clean gravels of different size are used as base materials. These gravels are placed on the under drainage system in four layers, each layer being 15cm thick. The bottom layer is made of bigger size gravels of 20 to 40mm. Two intermediate layers are provided. In first layer, gravel size is 12 to 20mm and in second layer, it is 6 to 12mm. The top layer is made of small size gravels 3 to 6mm.

Filter Media of Sand: The coarse sand of effective size of 0.35 to 0.65mm and uniformity coefficient 1.2 to 1.8 is generally used as the filtering media. The depth of sand layer varies from 60 to 100cm.

Appurtenance: In addition to the above following appurtenance are also provided

Air compressor: It is provided for sending the compressed air through the under drainage system at the time of washing the filter.

Trough: These are provided on the top of sand layer for carrying dirty water at the time of washing the filter.

Rate Control: This is provided to control the rate of flow.

Working of Rapid Sand Filter

In normal working condition, the valves V1 and V2 are kept open and other valves are kept closed. The water from the coagulation tank enters the inlet chamber through the inlet pipe. Then the water uniformly spreads over the filter media. The filtered water is collected in the central drain through the filter media. Finally, the water is taken to the storage tank.

When loss of head exceeds some limit, then a negative head is formed. At this time the function of filter stopped. It requires washing to resume normal working condition.

Back Washing of Filter

- During washing period the valves V1 and V2 are kept closed.
- The valves V4 and V6 are opened. The wash water and compressed air are forced through the under drainage system.
- After some time, the valve V6 is closedand valve V3 is opened so that the dirty water can be removed through the wash water drain.
- When washing is over, the valves V3 and V4 are closed. But V1 and V5 are kept open for some time.
- Finally, the valve V5 is closed, and V1remains open. Now V2 is opened to start the normal working.

Problem 8.2: Design a rapid sand filter to treat 4 million litres of raw water per day allowing 4% of filtered water for backwashing. Half hour per day is used for backwashing. Assume necessary data.

Solution

Total quantity of filtered water required per day $= \dfrac{4\,ML}{0.96}$

$= 4.167$ ML/day

Now, given that 0.5 hour is lost every day in washing the filter, we have

Filtered water required per hour $\dfrac{4.167}{23.5}\ ML/hour$

$= 0.177$ ML/hr.

Now, assuming the rate of filtration to be 5000 litres/hr/sq.m, we have

The area of filter required $\dfrac{0.177 \times 10^6}{5000} m^2 = 35.46 m^2$

Now, assuming the length of the filter bed (L) as 1.5 times the width of the filter bed (B), and two beds, the total area provided

$$2 X (L X B) = 35.46$$

$$2 X (1.5B X B) = 35.46$$

$$B^2 = \frac{35.46}{3} = 11.82$$

$$B = 3.44m$$

$$\therefore L = 1.5 X 3.44 \quad = 5.16$$

Say 5.2m and

$$B = \frac{35.46}{2 \times 5.2} = 3.4$$

Hence, adopt 2 filter units, each of dimensions **5.2m X 3.4m**

Design of under-drainage system.

Let a “manifold and lateral system” be provided below the filter bed, for receiving the filtered water, and to allow back washing for cleaning the filter. This consists of central manifold pipe, with laterals having perforations at their bottom.

To design this system, let us assume that the total area of the perforations in all the laterals is 0.2% of the total filter area.

∴ The total area of the perforations = 0.2% X Filter area

$$= \frac{0.2}{100}(5.2 \times 3.4)m^2$$

$$= 0.035 \ m^2$$

Now, assuming the area of each lateral = 2 times the area of perforations in it

Let us assume 13 mm dia. perforations, we have

Total area of laterals = 2 X Total area of perforations in it

$$= 2 X 0.035 = 0.07 \ m^2$$

Now, assuming the area of the manifold to be about twice the of laterals, we have

The area of manifold = 2 X 0.070 = 0.14 m^2

∴ Dia. of manifold (d) is given by

$$\frac{\pi}{4} \times d^2 = 0.14$$

$$d = \sqrt{\frac{0.14 \times 4}{\pi}} = 0.42m$$

Hence, use at 45cm diameter manifold pipe laid length wise along the centre of the filter bottom. Laterals are running perpendicular to the manifold at 15 cm spacing.

The number of laterals is then given as

$$\frac{5.2 \times 100}{15} = 34.6$$

Provide 35 numbers on each side of the manifold. Hence, use 70 laterals in all, in each unit.

Now, length of each lateral $\frac{\text{Widthof filter}}{2} - \frac{\text{Dia of manifold}}{2}$

$$\frac{3.4}{2} - \frac{0.45}{2} = \frac{52.95}{2} = 1.475m.$$

Now, adopting 1.3 mm diameter perforations in the laterals,

We have total area of perforations = $0.035 m^2 = 350 cm^2$

$$350 cm^2 = n \frac{\pi}{4} (1.3)^2$$

Where, n = Total no. of perforations in all 70 laterals

$$\therefore n = 350 \times \frac{4}{\pi} \frac{1}{(1.3)^2} = 263.8;$$

∴ No of perforations in each lateral

$$= \frac{264}{70} = 3.8; \text{ Say } 4$$

∴ Area of perforations per lateral

$$= 4 \times \left[\frac{\pi}{4} \times (1.3)^2 \right] cm^2 = 5.30\, cm^2.$$

Now, area of each lateral

= 2 X Area of perforations per lateral

$= 2 \text{ X } 5.30 = 10.60 cm^2$

∴ Diameter of each lateral

$$= \sqrt{10.60 \times \frac{4}{\pi}} = 3.67 cm;.$$

Hence, use 70 laterals each of 3.7cm dia. @ 15cm c/c, each having 4 perforations of 1.3 mm size, with 45 cm diameter manifold.

Check:

$$\frac{\text{Length of each lateral}}{\text{Dia of 1 ateral}} = \frac{1.475m}{3.7cm} = \frac{147.5}{3.7} = 39.9$$

(Which is less than 60, and hence O.K.)

Now, let us assume that the rate of washing of the filter be 60c rise/minute.

∴The wash water discharge

$$= \frac{0.6\times(5.2 \times 3.4)}{60} m^3 / \text{sec} = 0.177 m^3 / \text{sec}$$

∴Velocity of flow in the laterals for wash water

$$= \frac{0.177}{70 \times \left[\frac{\pi}{4} \times \left(\frac{3.7}{100}\right)^2\right]} = 2.35 m / \text{sec}$$

The dimensions of a concrete V-bottom trough are now designed by using an empirical formula.

$$Q = 1.376\,b.\,y^{\frac{3}{2}}$$

Where, Q = discharge in m^3/sec.

b = width of trough in m = y(assume)

y = water depth in the trough in mtr.

Let,

$$0.059 = 1.376\,Y.\,y^{\frac{3}{2}}$$

$$y^{\frac{5}{2}} = \frac{0.059}{1.376} = 0.0429$$

$$\therefore\ y = (0.0429)^{\frac{2}{3}} = 0.28m = 28cm$$

Keeping 5 cm freeboard, adopt the depth of trough

28 + 5 = 33cm

Hence, 3 no wash water troughs of size 28cm X 33 cm may be used as shown in fig given below.

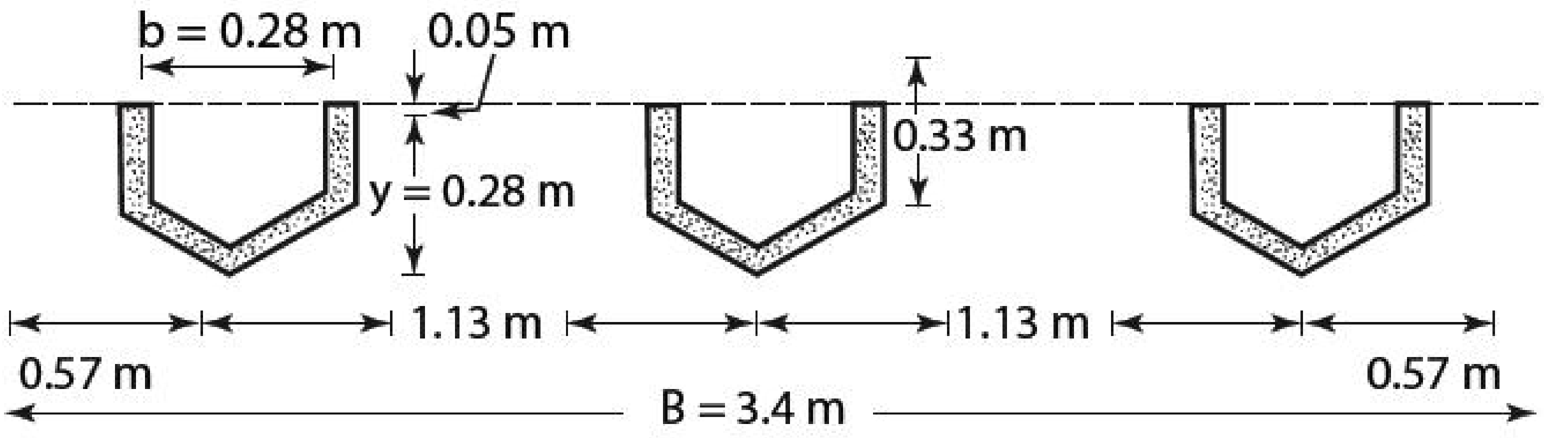
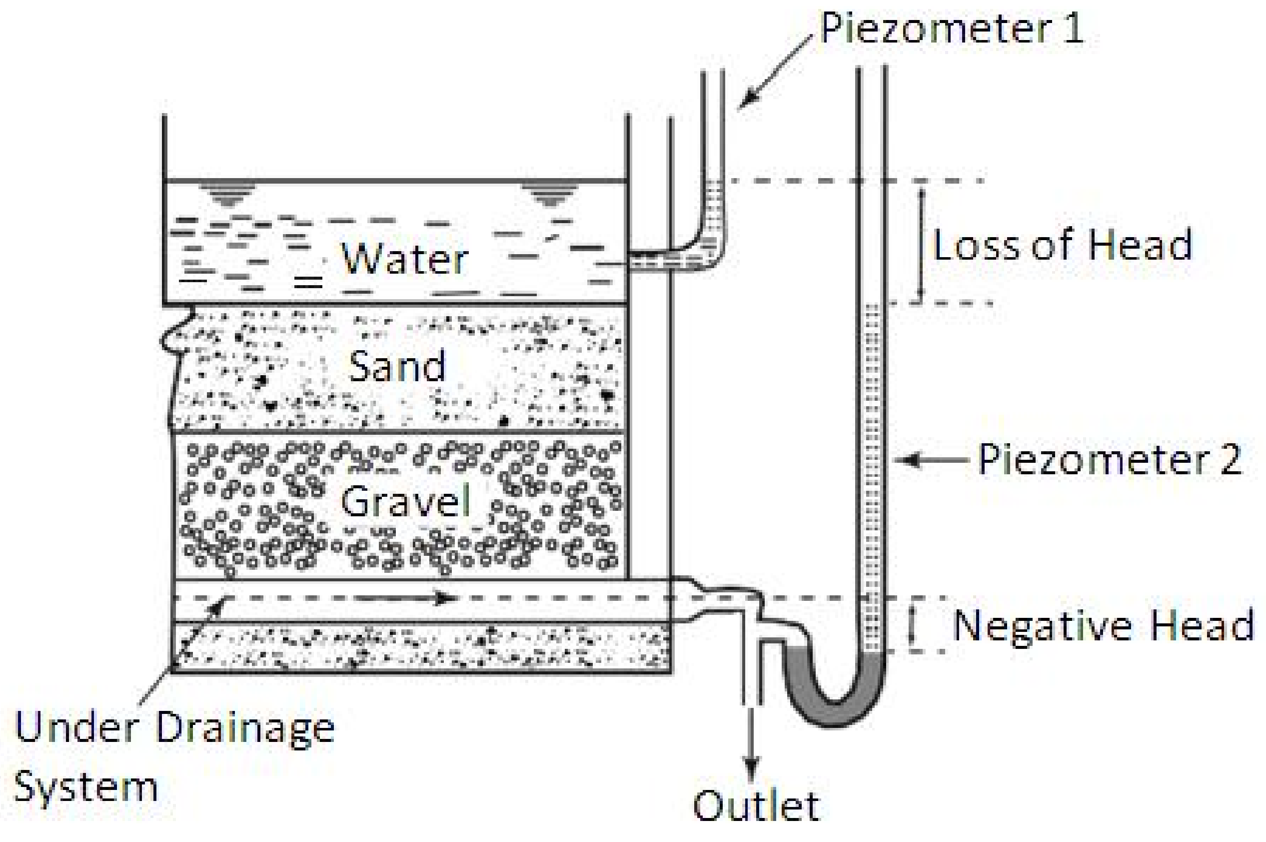


Fig. 8.5 Section of washwater trough

TROUBLES IN RAPID SAND FILTER

The following are the common troubles in rapid sand filter:

1. Loss of Head and Negative Head



Loss of Head: The water has to face a frictional resistance when it passes through the filtering media. So, it loses some of its head. If piezometer-1 is inserted in the water above the sand bed and piezometer-2 is fitted with outlet pipe as shown in Fig.8.6, the two piezometers will show a difference of water level. That difference is known as *loss of head*. Initially, the loss of head is small. But it increases gradually due to the deposition of suspended matters over the sand bed. Ultimately, a stage may come, when the water level in the piezometer-2 goes below the centre line of the under drainage system.

Negative Head: When the water level in the piezometer-2 goes below the centre line of under drainage, then the distance between the centre line and the water level in piezometer-2 is known as *Negative head* that is shown in Fig. 8.6. Due to the formation of negative head the lower portion of filter acts as a vacuum and the water is sucked in upward direction through the filter media.

Air Binding

When the negative head is formed, it tends to release the dissolved air and gases which may be present in water. Thus bubbles are formed. These bubbles adhere to the sand grains and the process of filtration is stopped. This phenomenon is known as *Air binding.*

Mud Balls

If the washing of filter is not done perfectly, then the fine colloidal sub-stances, (ie. mud) may exist in the sand bed. The existing mud forms nodules all over the sand bed. These nodules are known as mud balls. The mud balls may interfere with the working of filter.

Cracking of Filter

If the filter is not washed in proper time, then it is found that the sediments on the sand bed form cracks on the edges of the filter media. This is known as cracking of filter. This trouble is eliminated by breaking the mud balls and washing the filter properly.

COMPARISON BETWEEN SLOW SAND FILTER AND RAPID SAND FILTER

Table 8.1Comparison between slow sand filter and rapid sand filter

No	Item	Slow sand filter	Rapid Sand filter
1	Base material of gravel.	Varies from 3 to 56mm in size and 30 to 75cm in depth.	Varies from 3 to 40mm size and 60 to 90cm in depth.
2	Coagulation.	Not required.	Essential.
3	Compactness.	Requires large area for its installation.	Requires small area for its installation.
4	Construction.	Simple.	Complicated as under drainage system is to be properly designed and constructed.

5	Efficiency.	Very efficient in removal of bacteria and less efficient in the removal of colour and turbidity.	Less efficient in the removal of bacteria and more efficient in the removal of colour and turbidity.
6	Economy.	High initial cost of both land and material.	Cheap and quite economical.
7	Filter media of sand.	Effective size = 0.20- 0.30mm Uniformity coefficient= 2-3	Effective size = 0.35- 0.60mm Uniformity coefficient= 1.2- 1.70
8	Flexibility.	Not flexible for meeting variations in demand.	Quite flexible for reasonable fluctuations in demand.
9	Loss of head.	15cm to 75cm.	3m to 3.5m

10	Method of cleaning.	Scraping of top layer of 15mm to 25mm. It is a laborious method.	Agitation and back washing with or without the help of compressed air. Short, easy and speedy method.
11	Period of cleaning.	1 to 3 months.	2 to 3 days.
12	Rate of filtration.	100 to 200 litres per hour per m^2 of filter area.	3000 to 6000 litres per hour per m^2 filter area.
13	Skilled supervision.	Not essential.	Essential.
14	Suitability.	The filter can be constructed of local labor and material. It is suitable for small towns and villages where land is cheaply available.	It is suitable for big cities where land cost is high and variation in demand of water is considerable.

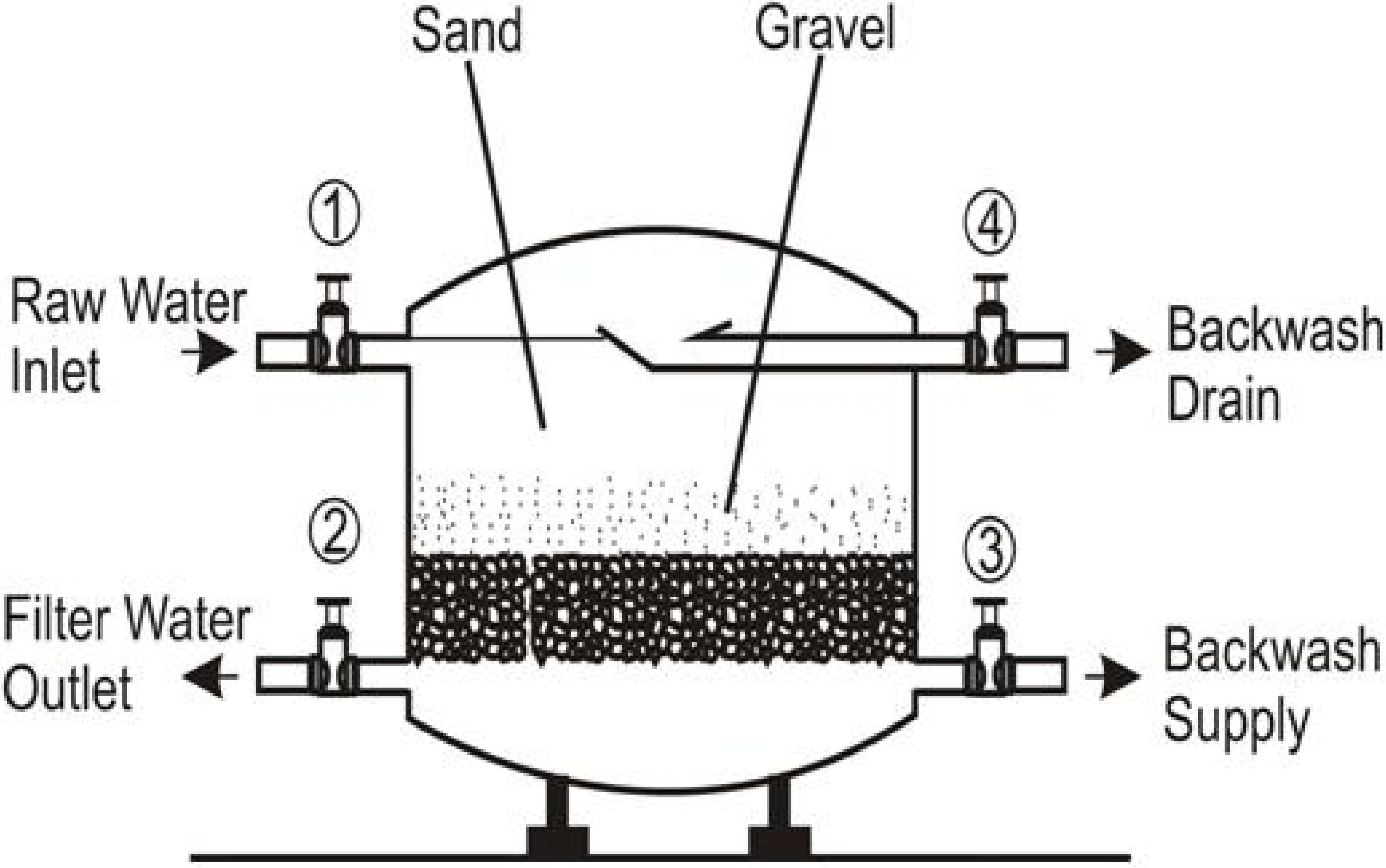


Fig. 8.7 Pressure filter

PRESSURE FILTER

Pressure filters are just like small rapid gravity filters placed in closed vessels, and through which water to be treated is passed under pressure. Since water is forced through such filters at a pressure greater than the atmospheric pressure, it is necessary that these filters are located in air tight vessels. The raw water is pumped into the vessels by means of pumps, the pressure so developed may normally vary between 30 to 70 metre head of water.

Construction

The filter vessel may be installed either in a horizontal or in a vertical position, depending upon which, they may be classified as horizontal pressure filters or vertical pressure filters. Typical cross- section of vertical pressure filter is shown in figure 8.7.

Working and operation of pressure filters

The pressure filter is operated like an ordinary rapid gravity filter except that the raw coagulated water is neither flocculated nor sedimented before it enters the filter. The flocculation takes place inside the pressure filter itself. Under normal working conditions, the coagulated water under pressure enters the filter vessel through the inlet valve 1, and the filtered water comes out of the outlet valve 2.

Hence, under this condition, only these two valves are kept open and all other valves are kept closed. The commonly used coagulant is alum and is kept in a pressure container connected to the influent line to the filter. Little time is thus available for this coagulant to get mixed properly or to form floc outside the filter vessel.

The cleaning of the filter may be carried out by back washing as is done in a normal rapid gravity filter. The compressed air may also be used, if designed, in order to agitate the sand gains. For cleaning the inlet and outlet valves (valves 1 and 2) are closed and the wash water valve 3 and wash water gutter valve 4 are opened. After the completion of cleaning, these valves may be closed, and raw supplies restored. However, the filtered supplies should not be collected for a little time and wasted through valve 4, as is done in a rapid gravity filter.

Rate of filtration of pressure filters

The pressure filters can yield filtered water at rates much higher than what can be obtained from rapid gravity filters. The rate of filtration normally ranges between 6000 to 15000 liters per hour per sq.m of filter area.

Efficiency and suitability of pressure filters.

The pressure filter is less efficient than rapid sand filter in removing turbidity, colour and bacterial load. The quality of their effluent is poorer and they are, generally, not used for public supplies. It is only suitable for industrial plants, private estate, small colonies, etc.

Advantages of pressure filters

The advantages of pressure filters over rapid gravity filters are given below.

1. Pressure filters are compact and can be handled easily
2. It requires lesser space and lesser filtering material for treating the same quantity of water.
3. Sedimentation and coagulation tanks are avoided.
4. They are more flexible, as the rate of filtration can be changed by changing the pumping pressure.

Advantages of pressure filters

5. It is economical for treating smaller quantities of water.
6. Since the water coming out of the filter possesses sufficient residual head, the pumping of the filtered water is not required.

Disadvantages of pressure filters

The disadvantages of pressure filters over rapid gravity filters are given below.

1. They are less efficient in removing bacteria and turbidities, and hence, the quantity of the filtered effluents is poorer.
2. They are costlier, particularly for treating large scale municipal supplies.
3. Since filtration takes place in a closed tank, proper inspection and quality control is not possible.
4. Since these filters are operated under pressure, the normal tendency is to pump the water at higher rates, and thus obtaining still poorer quality of effluents.

Assignment 2

1. Design a rapid sand filter to treat 4 million litres of raw water per day allowing 4% of filtered water for backwashing. Half hour per day is used for backwashing. Assume necessary data. ***7 marks***

2. Explain Newton's law and Stocks Low in connection with settling of discrete particles (Type I Settling) ***3 Marks***

MODULE V

DISINFECTION OF WATER

When water comes out of filter plants, it may contain bacteria and other micro-organisms. Some of which may be pathogenic. It is therefore necessary to disinfect water to kill bacteria and other micro-organisms, and thus prevent water borne diseases. The chemicals used for killing these bacteria are known as disinfectants, and the process is known as disinfection or sterilisation.

MINOR METHODS OF DISINFECTION

Following are the minor methods of disinfection.

1. Boiling method
2. Excess lime treatment
3. Iodine and bromine treatment
4. Ozone treatment
5. Potassium Permanganate
6. Silver treatment
7. Treatment with ultra-violet rays

1. **Boiling method**: When water is boiled above a certain temperature, bacteria are killed. The boiling of water is the most effective method of disinfection. But to boil water on a large scale is impossible. However, in case of epidemic, the consumers may be advised to boil the water before use for drinking and domestic purposes.

2. **Excess lime treatment:** The treatment of lime is given to water for the removal of dissolved salts. But it was found that excess lime is added to water, it will also work in addition as a disinfecting material. The action of excess lime is based on the fact that lime increases pH value of water. The extreme acidity or alkalinity is detrimental to bacteria. Hence, when pH value of water is about 9.50 or so, bacteria can be removed to the extent of 99.93 or 100 percent. But when this treatment is adopted for disinfection, the excess lime is to be removed by any suitable method of recarbonation after disinfection.

3. Iodine and bromine treatment: When water is treated with iodine or bromine, it is disinfected. The dosage of iodine or bromine is about 8ppm, and the contact period with water is about 5 minutes. The use of iodine and bromine is limited to small water supplies such as swimming pools, private plants etc.

4. Ozone treatment: The atmospheric oxygen is in molecular form containing two atoms of oxygen. But when a high-tension electric current is passed through a stream of air in a closed chamber triatomic molecules of oxygen are formed as shown by the following equation.

$$O_2 + O = O_3$$

Such oxygen is known as ozone. The third atom is very loosely bounded and the ozone easily breaks down into oxygen and releases nascent oxygen which is very powerful in killing bacteria. The equipment called Ozonizer is used in the ozone treatment.

5. Potassium permanganate: This disinfectant works as a powerful oxidising agent and is found to be effective in killing cholera bacteria. However, it is less effective for other water-disease producing organisms. The water treated with this disinfectant produces a dark brown coating on porcelain vessels and this coating is difficult to remove.

6. Silver treatment: It has been found that silver has the property of disinfecting water. The metallic silver is placed as filter media and water while passing through such a filter absorbs some portion of silver which disinfects the water. The dosage of silver varies from 0.05 to 1 ppm and the period of contact is about 15 min. to 3 hrs.

7. Treatment with ultra-violet rays: The ultra-violet ray offers an effective method for sterilization of water, since the light is effective in killing both the active bacteria as well as spores. U-V rays are generated by machines consisting of mercury-vapor lamp enclosed in a quartz globe.

This treatment does not develop any taste or colour in the water and there is no danger of over dose. It is suitable for water supply installation of private institutions, swimming pools etc.

CHLORINATION

In this treatment for disinfection, the chlorine is used as disinfecting material. For treatment on large scale, chlorination is invariably used as treatment for disinfection. This method is cheap, reliable, easy to handle, easily measurable, and above all, it is capable of providing residual disinfecting effects for long periods, thus affording complete protection against future recontamination of water in the distribution system. Its only disadvantage is that when used in greater amounts, it imparts bitter and bad taste to the water, which is not accepted by certain consumers.

Advantages of chlorination

1. It accomplishes greater bacterial purification in minutes.
2. It is cheap and avoids, wholly or in part, the necessity for raw water storage.
3. It provides extra security against water-borne diseases.
4. The processes is economical and cheap.
5. It is harmless to human beings.
6. It is reliable and effective.
7. Residual chlorine can be maintained in water.

4 CHLORINE DEMAND

Chlorine and its compounds are consumed by a variety of organic and inorganic materials present in water due to its oxidising power before disinfection is achieved. The amount of chlorine consumed in the oxidation of these impurities, before any disinfection is achieved, is known as chlorine demand of water. After the chlorine demand is fulfilled, chlorine will appear as free available residual chlorine. The free available residual chlorine will then serve as disinfectant to kill to the pathogens present in water.

4 CHLORINE DEMAND

Thus the difference between the amount of chlorine added to the water and the residual chlorine is called chlorine demand.

Generally, most waters are satisfactorily disinfected if the free available residual chlorine is about 0.2 mg/litre at the end of 10 minutes contact period.

FORMS OF APPLICATION OF CHLORINE

Chlorine may be applied to water in one of the following form:

(i) As free chlorine
(ii) As chloramines
(iii) As bleaching powder
(iv) As chlorine dioxide.

(i) Free chlorine

Free chlorine is available in gaseous or liqiud form. Chlorine is stored in cylinder, 80% of the contents being in liquid form and the rest in gaseous form. The chlorine is fed to a water-supply by means of a device called chlorinator. It regulates the flow of gas from the chlorine container at the desired rate.

Advantages

1. It can be stored for long time.
2. It is very powerful and effective disinfectant
3. It is available cheaply
4. Initial coast of chlorine plant is low.
5. There is no sludge formation.
6. Skilled supervision is not necessary.

(ii) Chloramines

Chlorine is not stable in water. Hence it is sometimes mixed with ammonia to form stable compounds called chloramines. In this treatment, ammonia as NH_3 is added to the water just before the chlorine is applied. The usual proportions are 1 part of ammonia to 4.5 parts of chlorine by weight. These have adequately good disinfecting properties and are specially useful for disinfecting swimming pools.

Advantages:

1. It is more effective than chlorine alone.
2. It prevents bad taste and odour, particularly those due to phenols.
3. The quantity of chlorine required becomes less, especially if organic matter is present in large amounts.
4. Water treated with this causes less irritation to skin, nose and eyes.
5. There is no danger of over dose.

(iii) Bleaching powder

Bleaching powder is a compound of chlorine and contains about 30% chlorine. When bleaching powder is used as disinfectant, it is also called hypo-chlorination. Bleaching powder is available in the form of powder. It is unstable and loses chlorine when exposed to atmosphere. Hence it has to be stored carefully. It is therefore used only on small installations or under emergency conditions. Hypochlorite is applied to water as a solution by means of hypochlorite feeding apparatus.

Advantages

1. They are available in small packets in powder form.
2. Their chlorine content does not decrease with storage.
3. They can be applied to water in dry condition or as solution.

(iv) Chlorine dioxide.

Chlorine dioxide is a very effective and powerful disinfectant. The chlorine dioxide gas is costly and very unstable and has to be used immediately after its production. For these reasons, it is generally not used for treating ordinary public supplies. However, because of its stronger disinfecting powers, it may be used for treating waters containing larger disinfecting powers, it may be used for treating waters containing larger amounts of organic impurities.

9.6 TYPES OF CHLORINATION

Depending upon the stage at which chlorine is applied to water; chlorination can be of the following types.

(1) Plain chlorination

(2) Pre-chlorination

(3) Post-chlorination

(4) Double chlorination

(5) Break point chlorination

(6) Super chlorination

(7) Dechlorination

(1) Plain Chlorination

This term is used to indicate that only chlorine is added to water and no other treatment has been given to raw water. Water from deep wells, lakes, reservoirs etc., is comparatively clear with turbidity less than 30ppm. In such cases no treatment such as sedimentation, coagulation etc. is necessary. When no other treatment except chlorination is given before supplying water to consumers, it is called plain chlorination.

(2) Pre-Chlorination

Pre-chlorination is the process of applying chlorine to the water before filtration or before sedimentation. It helps in improving coagulation, and reduces the loads on the filters. It also reduces the taste, odour, algae and other organisms. The normal doses required are as high as 5 to 10mg/L. Pre-chlorination is, however, always followed by post chlorination, so as to ensure safety of water.

(2) Pre-Chlorination

Advantages

1. It reduces the quantity of coagulants required.
2. It reduces the bacterial load on filters.
3. It helps in maintaining longer filter runs.
4. It controls the algae and planktons in basins and filters.
5. It prevents putrefaction of sludge in settling basins.
6. It eliminates tastes and odour.

(3) Post-chlorination

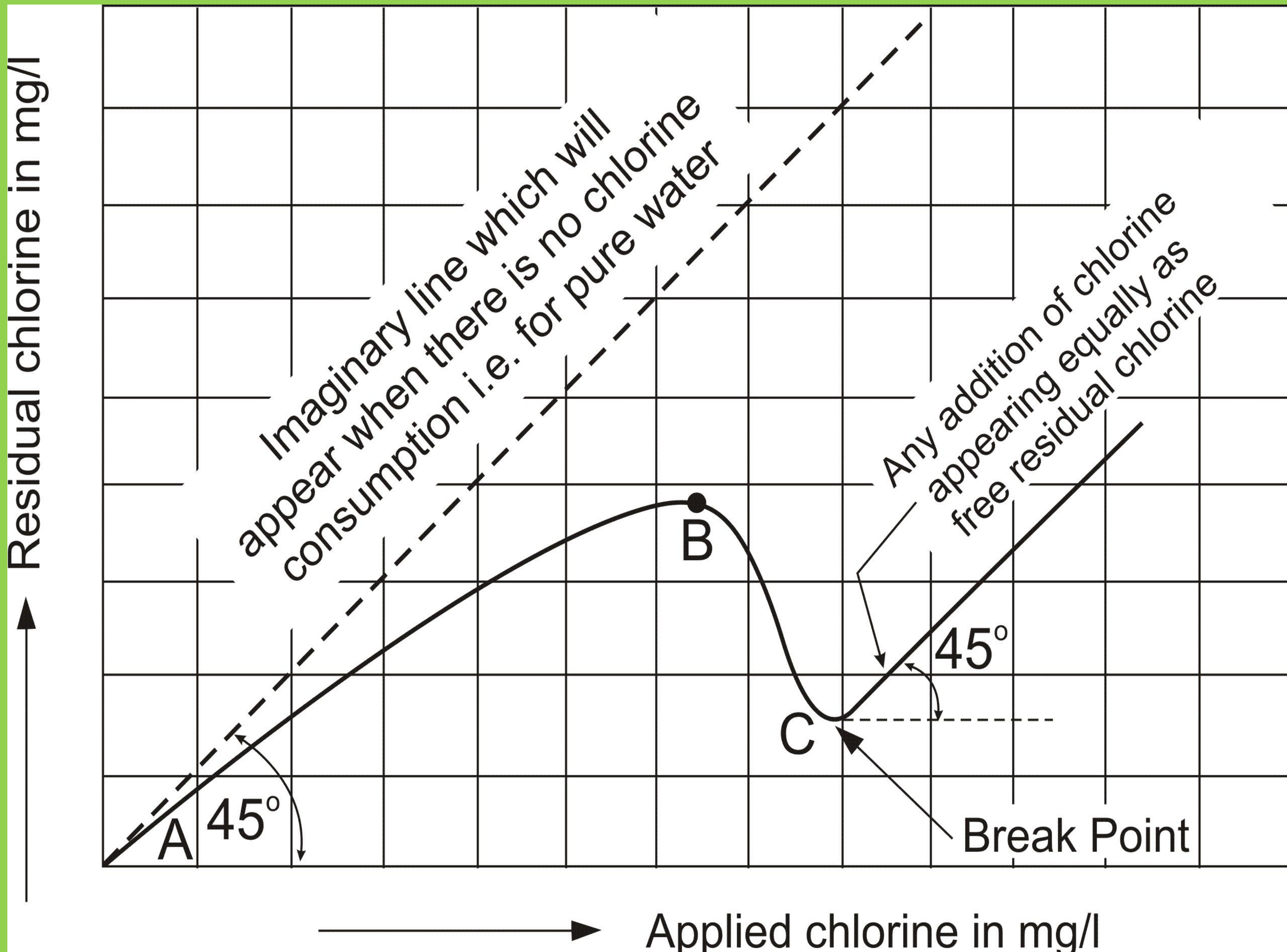
Post chlorination or sometimes simply called chlorination is the normal standard process of applying chlorine in the end, when all other treatments have been completed. While treating normal public supplies, the post chlorination is adopted after filtration and before the water enters the distribution system. The dose of the chlorine should be so adjusted that the residual chlorine is about 0.1 to 0.2 ppm before water enters the distribution system. It is useful for protection against contamination from cross-connections.

(4) Double chlorination

The term double chlorination is used to indicate that the water has been chlorinated twice. The prechlorination and post chlorination are generally used in double chlorination. Post chlorination, however, is generally always used. The pre-chlorination is used when the waters is highly turbid and contaminated. The advantage of double chlorination is similar to those of prechlorination. In addition, the maintenance of two chlorinating plants serves as a factor of safety.

(5) Break point chlorination

Break point chlorination is a term which gives us an idea of the extent of chlorine added to water. In fact it represents, that the much dose of chlorination, beyond which any further addition of chlorine will appear as free residual chlorine. When chlorine is added to the water it kills bacteria and disinfection is effected, and it oxidizes the organic matter.



Residual chlorine in mg/l
Imaginary line which will appear when there is no chlorine consumption i.e. for pure water
Any addition of chlorine appearing equally as free residual chlorine
B
45°
C
A
45°
Break Point
Applied chlorine in mg/l

When chlorine is added to the water, it first of all generally reacts with the ammonia present in the water, so as to form chloramines. If chlorine is slowly added to the water, the residual will go on increasing with the addition of chlorine. However, some chlorine is consumed for killing bacteria, and thus the amount of residual chlorine shall be slightly less than that added, as shown by the curve AB.

If the addition of chlorine continued beyond the point B, the organic matter present water gets oxidise, and, therefore, the residual chlorine content suddenly falls down, as shown by the curve BC. The point C is the point beyond which any further addition of chlorine will appear equally as free chlorine, since nothing of it shall be utilised. This point “C” is called the ***break point***, as any chlorine that is added to water beyond this point, breaks through the water, and appears as residual chlorine. The addition of chlorine beyond break point is called break point chlorination.

At the point B, when oxidation of organic matter starts, a bad smell and taste generally appears; this will disappears at the break point C, when the oxidation has been completed.

It is a general practice to add chlorine beyond break point, and thus to ensure a residual of 0.2 to 0.3 mg/l of free chlorine to prevent future contamination.

Break point chlorination has practical significance since application of chlorine at or slightly higher than the break point concentration will have the following advantages:

(i) It will remove taste and odour.
(ii) It will have adequate chlorine residual.
(iii) It will have desired chlorine residual.
(iv) It will complete the oxidation of ammonia and other compounds.
(v) It will remove colour due to organic matter.
(vi) It will remove manganese.

(6) Super chlorination

Super-chlorination is the application of chlorine beyond the stage of break point. The addition of chlorine sufficient to give a residual chlorine content of 1 to 3 ppm has proved useful to destroy odours and tastes resulting from chloroproducts formed between the decomposition products from vegetable matter and algae. Excess chlorine may be added at any point or points of chlorination, though it is usually applied after filtration Super chlorination is adopted when there is an epidemic in the locality, or when water is liable to sudden fluctuations in chlorine demand, containing a high concentration of organic impurities or when water containing pathogenic microorganisms causing amoebic dysentery.

(7) Dechlorination

It is the process of removing excess chlorine from water before distribution to the consumers to avoid chlorine tastes. This is generally required when super-chlorination has been practiced. The de-chlorination process may either be carried out to such an extent that sufficient residual chlorine of 0.1 to 0.2 mg/L only remains in water after de-chlorination. As stated above, dechlorination should be done in such a way that some residual chlorine remains in water. Dechlorination is achieved either by aeration or by the use of chemicals such as sodium thio-sulphate, sodium sulphate, activated carbon, potassium permanganate or sulphur dioxide. By filtering super-chlorinated water through beds of granular activated carbon, excess chlorine oxidises the carbon to carbon dioxide, due to which odour, taste, and colour are successfully removed.

9.7 FACTORS AFFECTING CHLORINATION

The destruction of pathogens by chlorination is dependent upon the following factors.

(i) Turbidity

(ii) Presence of metallic compounds

(iii) Ammonia compounds

(iv) pH of water

(v) Temperature

(vi) Time of contact

(vii) Number and concentration of bacteria.

1. **Turbidity:** The effect of turbidity in water is to make it difficult to obtain free residual chlorine. The penetration of chlorine and destruction of bacteria in turbid water is very uncertain. Due to this reason, chlorination is preferred after filtration.

2. **Presence of metallic compounds:** If metallic compounds like iron and manganese are present in water, utilizes large amounts of chlorine to convert these into their higher stages of oxidation which are insoluble in water. Hence it is essential to remove iron and manganese, to make chlorination is more effective.

3. Ammonia compounds: The presence of ammonia with or without organic matter may form combined available chlorine which is not so effective in killing bacteria as free available chlorine. Hence to add sufficient chlorine to react with the amount of natural ammonia present and a further dose of chlorine enough to create an excess of free chlorine for speedy disinfection.

4. pH of water: Increasing pH reduces effectiveness of chlorine. The effective sterilising compound, hypochlorous acid, is formed in greater quantities at low pH than at high pH values.

5. Temperature of water: Reduction in the temperature of water results in substantial decrease in the killing power of both free and combined chlorine. In order to have 100% bactericidal activity, the requirement of residual chlorine increases with decrease in temperature and increase in pH.

6. Time of contact: The bactericidal activity of chlorination is not instantaneous. The percentage kill of bacteria and viruses depends upon the time of contact between the chlorine and the pathogens, before the water is supplied to the consumers. The contact time required for disinfection by free chlorine acting in clear water is 20 minutes, but it is 60 minutes for combined chlorine, such as chloramines etc.

7. Number and concentration of bacteria: At a given pH value and temperature, the greater the numbers of bacteria, the longer will be the time required to reduce them below a given value.

MISCELLANEOUS WATER TREATMENT METHODS

Besides the normal treatment process, such as coagulation, sedimentation, filtration and disinfection; certain other special treatments are sometimes required in order to remove the special minerals, tastes, odours, colours etc. from water.

MISCELLANEOUS WATER TREATMENT METHODS

The following are the some of the miscellaneous treatment methods for making the water better in every way.

1. Removal of colour, odour and taste
2. Fluoridation
3. Defluoridation
4. Desalination

REMOVAL OF COLOUR, ODOUR AND TASTE

The presence of organic matter, algae, dissolved hydrogen Sulphide and contamination due to industrial wastes containing phenol, excessive chlorine, etc. and dissolved iron and manganese salts impart **color, odor and taste to the water.**

Following are some important water treatment methods employed for *removing color, taste and odor***.**

i) Aeration

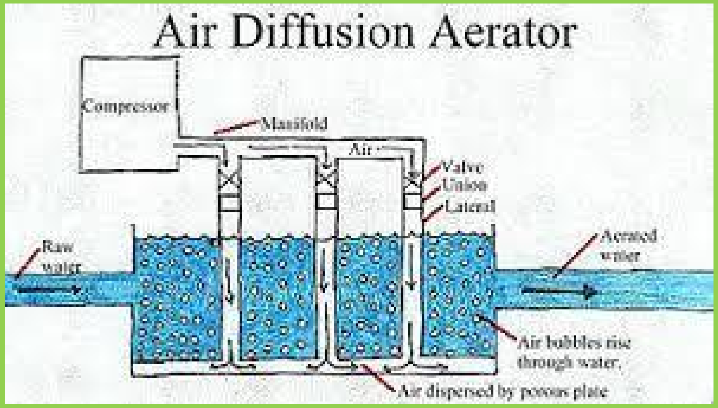
Aeration is the process of bringing water in intimate contact with air. During the process, water absorbs oxygen from the air. Aeration removes iron, manganese, CO_2, H_2S and oxidises certain organic impurities present in the water. Up to certain extent bacteria is also killed. However, it can remove CO_2 up to 10% only and residue to the extent of 3-5 mg/liter always remains in water.

The aeration can be done by the following methods

a. By using Air diffusion

In this method, compressed air is bubbled through the water, so as to thoroughly mix it with water. Perforated pipes are, therefore, installed at the bottom of the aeration tanks, and the compressed air is blown through them. The compressed air is thus bubbled up from the bottom of the tank. During its upward movement through the water body, it gets thoroughly mixed up with the water contained in the tank, thereby completing the aeration process.

By using Air diffusion



b. By using Trickling Beds

In this method, the water is allowed to trickling down the beds of coke, supported over the perforated bottom plate. The water is spread over the bed by the rotating arm. While flowing through the coke the water becomes free from the dissolved gases and thus the color, odour and taste are removed.

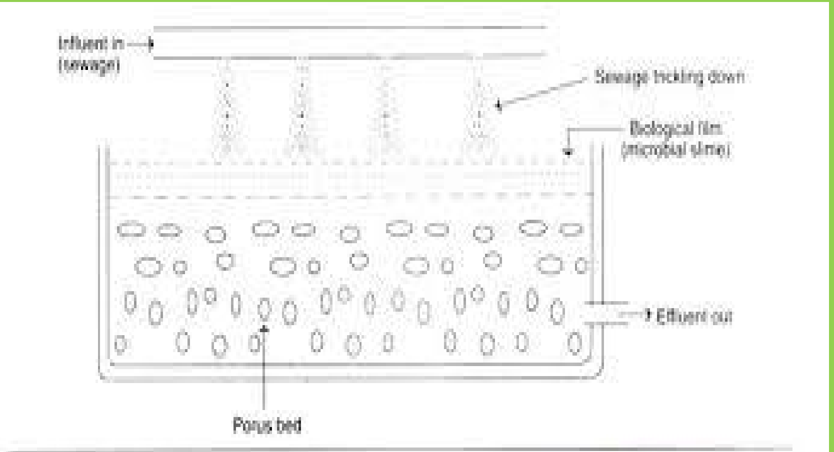
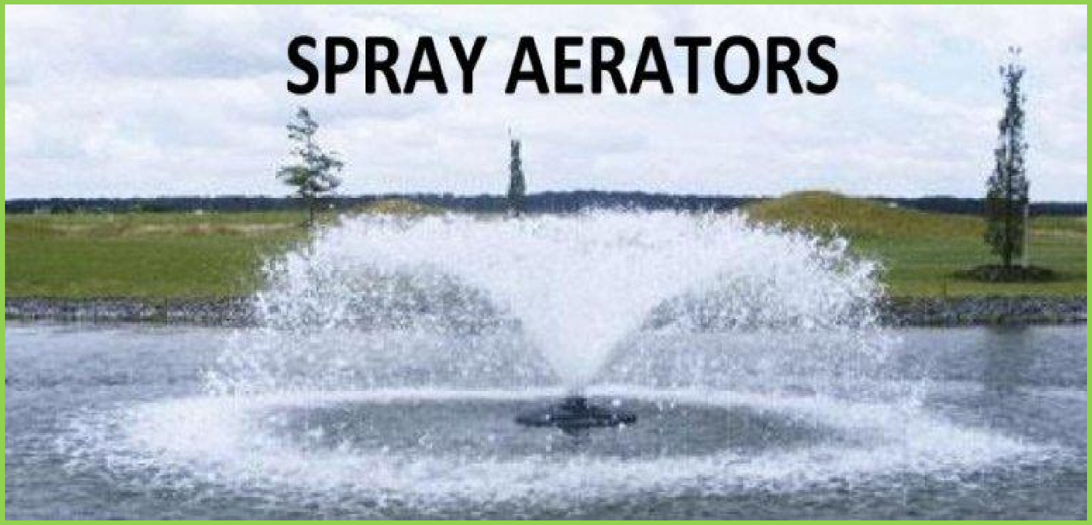


Fig. 57.6 : A diagrammatic representation of trickling filter.

c. Spray Nozzles

In this method, water is sprinkled in air or atmosphere through special nozzles which breaks the water into droplets, thus permitting the escape of dissolved gases. Carbon di-oxide gas is thus considerably removed in this method. However, considerable head of water is required for the working of these nozzles, which function efficiently at pressure of 10 to 14m head of water.



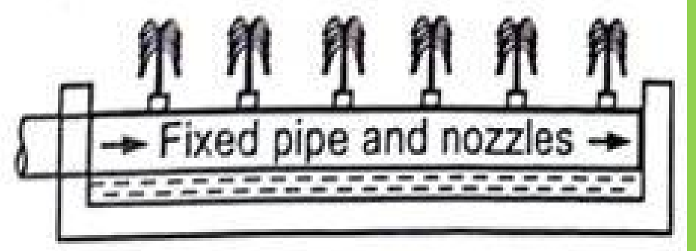
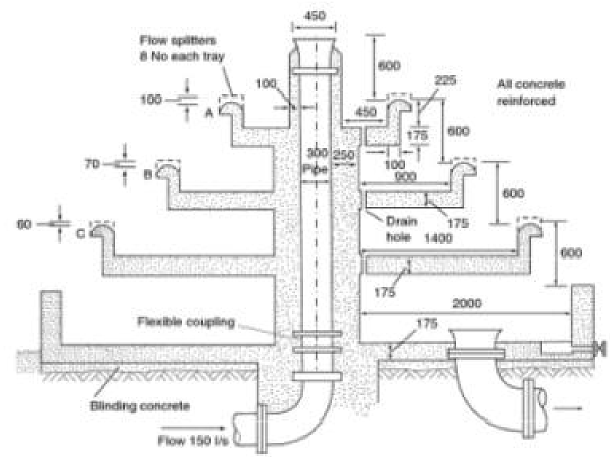


Fig. 6.4 : Spray nozzles (fixed type)

d. By using cascades

In this method, the water is made to fall through a certain height over a series of steps with a fall about 0.15 to 0.3m in each step. The water flows in thin film so that it comes in contact with air, and thus the colour, odour and taste of water are removed.



Cascade Aerator

(ii) Water Treatments with Activated carbon.

Activated carbon is obtained by charring wood or saw dust at 500°Cin a closed vessel, thereby hydrocarbons, which usually interfere with the absorption of organic matters, are expelled. It is available in granular or powdered from. As it is highly porous and possesses free valencies; both properties provide it the high adsorption capacity.

Activated carbon is most widely used for the removal of tastes and odors from the public water supplies, because it has excellent properties of attracting gases, finely divided solid particles and phenol type impurities.

(iii) Water Treatments with Copper Sulphate.

Copper sulphate ($CuSO_45H_2O$) solution usually applied with dose of 0.5 to 0.75 mg/liter to the treated water, just before it is allowed for distribution in the mains. It can also be added in the lakes or reservoirs. Cropper sulphate helps in removing odors, tastes and colors from the water. Its main advantage is that it checks the growth of algae, even before production and also kills some bacteria.

iv) By Oxidation

The Oxidation of organic matters is done by adding chlorine, potassium permanganate, ozone, etc. to water as oxidizing agents. Generally the excess chlorine is added beyond the break point for oxidizing the organic matters.

FLUORIDATION

Water fluoridation is the controlled addition of fluoride to a public water supply to reduce tooth decay. Fluoridated water contains fluoride at a level that is effective for preventing cavities; this can occur naturally or by adding fluoride. Fluoridated water operates on tooth surfaces: in the mouth, it creates low levels of fluoride in saliva, which reduces the rate at which tooth enamel demineralises and increases the rate at which it remineralises in the early stages of cavities.

FLUORIDATION

Water fluoridation is the controlled addition of fluoride to a public water supply to reduce tooth decay. Fluoridated water contains fluoride at a level that is effective for preventing cavities; this can occur naturally or by adding fluoride.

DEFLUORIDATION

Fluoride mainly enters the human body through drinking water. When fluoride enters in human body it combines with bones, since fluoride has affinity for calcium phosphate in the bones. Excessive concentration of fluoride can also cause dental fluorosis or mottled enamel when consumed during childhood years.

DEFLUORIDATION

Dental fluorosis is the condition of discoloured, blackend, mottled, or chalky white teeth. Skeletal fluorosis leads to severe and permanent bone and joint deformations. Defluoridation is the removal of excess fluoride from water.

Following are the principal methods of defluoridation:

Calcium phosphates. Bone has great affinity for fluorides and can be used in the filter for removal of fluorides. The bone is calcinated at 400^{o}C to 600^{o}C for 10 minutes followed by mineral acid treatment. It is then pulverized to pass 40 to 60 meshes and is used in the filter bed. The filter is regenerated with alkali and acid. One cubic meter of bone can treat 1 million litres of water containing 3.5 ppm. of fluoride.

Bone charcoal. It is essentially tri-calcium phosphate and carbon, and has been used successfully for the removal of fluorides.

Synthetic tri-calcium phosphate. It can be prepared from milk of lime and phosphoric acid when the reaction is carefully controlled. This material has been used in contact filters for removal of fluorides. The regeneration can be done by 1% caustic soda followed by a dilute hydro-chloric acid or carbon dioxide wash to neutralise the excess alkali.

Fluorx. Fluorx is a special mixture of tri-calcium phosphate and hydroxyapatite. It is used as filter medium. Fluorex can beregenerated by washing with 1.5% solution of caustic soda. It can be next rinsed with twice its volume of water and the excess soda can be neutralised with the solution of carbon dioxide at 0.15% strength.

Ion-exchanger. There are a number of ion-exchanger materials which can be used for removal of fluorides. Fluorides in water can be removed by successive passage through beds of cation-exchanger. Alum treated cation exchange resin from Avaram bark can be used as an effective material for removing fluorides from water.

Nalgonda Technique. In India and particularly in rural areas, ground water containing excess fluoride is treatd by Nalgondatechnique. Nalgonda technique uses aluminium salt (alum) for removing fluoride. The raw water is firstly mixed with adequate amount of lime (CaO) or sodium carbonate (Na_2CO_3) and thoroughly mixed. Alum solution is then added, and water is stirred slowly for about 10 minutes, and allowed to settle for nearly one hour. The precipitated sludge is discarded, and the clear supernatant containing permissible amount of fluoride is withdrawn for use.

Activate carbon. Removal of fluoride from water has also been effected by treatment with activated carbon, at pH 3.0. No removal takes place at pH 8 or above. The carbon when used as a contact bed can be regenerated with the weak acid and alkaline solution.

DESALINATION

Only about 0.5% of the earth's water is potable. The remaining 97% is ocean water and 2.5% is brakish water, both of which are impotable because of their dissolved salt content. The process of removing this salt from water is known as desalination, and the resultant water which is free from salt is known as fresh water.

DESALINATION

Following are common methods used for desalination:

1. Distillation.
2. Reverse osmosis
3. Electro dialysis.
4. Freezing.
5. Solar evaporation.

WATER SOFTENING

Water is said to be ‘hard’ when it contains relatively large amount of carbonates, bicarbonates, sulphates and chlorides of calcium and magnesium dissolved in it. These materials react with soap, causing a precipitation which appears as a scum or curd on the water surface. all be removed by anion exchange.

Hardness is of two types: (i) temporary hardness, and (ii) permanent hardness. The temporary hardness is the one which is deposited when water is boiled, and is usually known as carbonate hardness. The permanent hardness is also known as non-carbonate hardness, and is mainly due to the presence of sulphates, chlorides and nitrates of calcium and magnesium.

Removal of temporary hardness: Temporary hardness or bicarbonate hardness can be removed by boiling and by adding lime. Method of boiling has practical limitations on large scale.

Removal of permanent hardness:

Permanent hardness can be removed by one of the following methods.

1. Lime-soda process.
2. Zeolite process.
3. Ion exchange process.

Lime-soda process.

Lime soda process is one of the water softener systems. In this system calcium hydroxide and soda ash are used as reagents. By this process soluble magnesium and calcium salts are removed as calcium carbonate and magnesium hydroxide precipitated.

Lime-soda process.

Lime soda process is one of the water softener systems. In this system calcium hydroxide and soda ash are used as reagents. By this process soluble magnesium and calcium salts are removed as calcium carbonate and magnesium hydroxide precipitated. After removal of this precipitate, we obtain soft water.

Zeolite process

Zeolite is micro-porous mineral which is used as catalyst in many industrial purposes such as water purification and air purification. Zeolite process for water softening has become a commercial success for the reason that zeolite can be easily regenerated. When Ca^{2+} and Mg^{2+} ions containing hard water is passes through a bed of sodium zeolite, the sodium ions are replace by the calcium and magnesium ions.

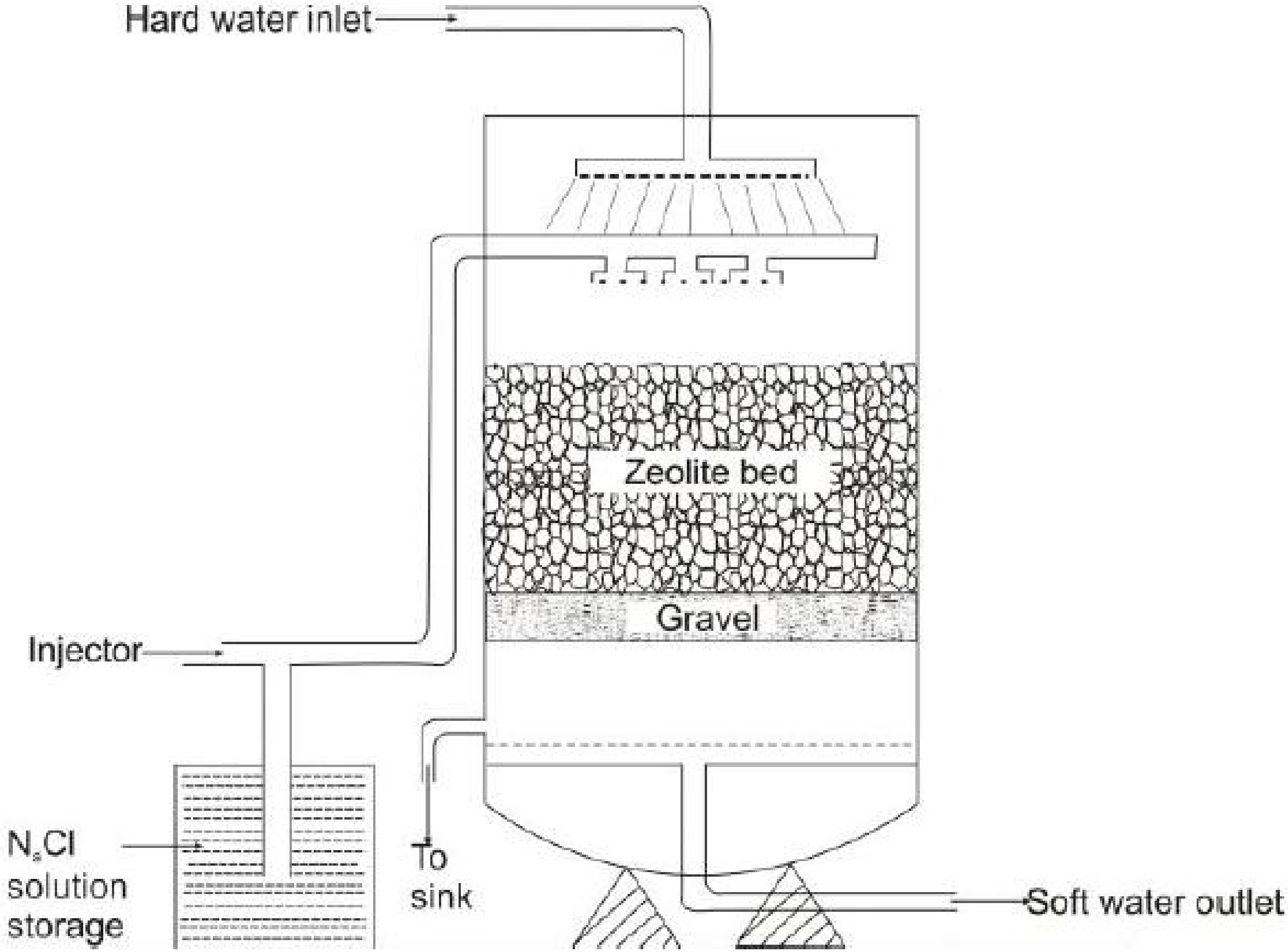


Fig.10.4 Zeolite process

3. Ion exchange process.

Ion exchange water treatment system is one of the most common processes and it is works on the basis on ion exchange principle. It removes the scale-forming calcium and magnesium and other metal ions from hard water. To remove these ions water are passes through ion exchange column filled with synthetic resin.

Electro Dialysis.

Electro Dialysis (ED) is used to transport salt ions from one solution through ion-exchange membranes to another solution under the influence of applied electric potential difference. This is done in a configuration called an electrodialysis cell. Two different membranes are used- one more selective to anions and the other more selective to cations.

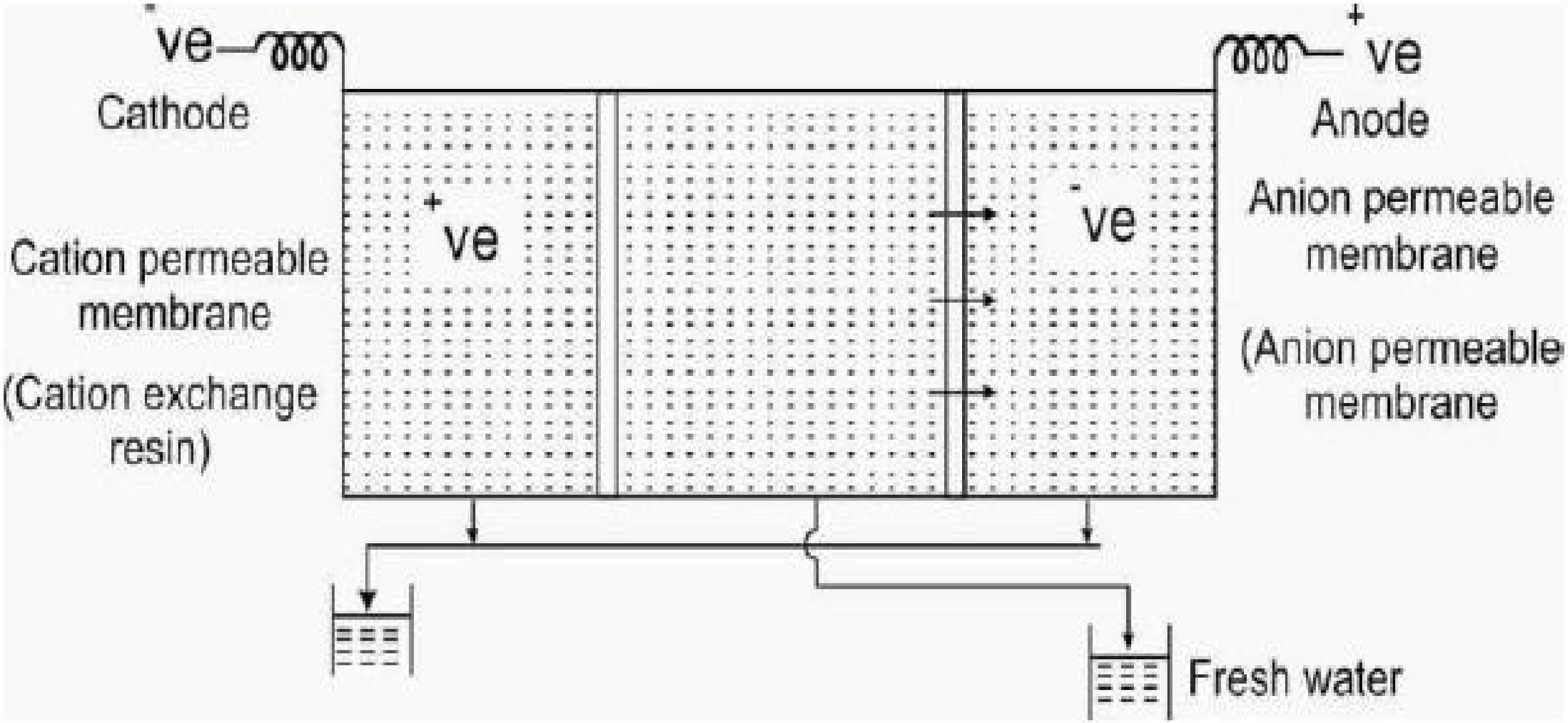


Figure 10.2 Electro Dialysis

Electro Dialysis.

Brackish water is introduced into a narrow channel, one of those walls is an anion permeable membrane, and another which is a cation-permeable membrane. The passage of electric current aids the diffusion of these ions, and the electric energy required is proportional to the concentration of salts in the saline water. The process is suitable especially for brackish waters with total concentration up to 5000mg/L.

MODULE VI

DISTRIBUTION OF WATER

After the water has been properly treated and made safe and wholesome, it has to be supplied to the consumers in their individual homes. The water has, therefore, to be taken from the treatment plant to the roads and streets in the city, and finally to the individual houses. This process of carrying the water from the treatment plant to the individual homes is accomplished through a well planned distribution system. A distribution system consists of pipes, valves, hydrants, meters, pumps, service reservoirs etc

REQUIREMENTS OF A GOOD DISTRIBUTION SYSTEM

The various requirements for proper functioning of a distribution system are:

1. Pressures should be great enough to adequately meet consumer needs.
2. Pressures should be great enough to adequately meet fire fighting needs.
3. Pressures should not be excessive because development of the pressure head brings important cost consideration and as pressure increases leakages increases too.

REQUIREMENTS OF A GOOD DISTRIBUTION SYSTEM

4. Purity of distributed water should be maintained. This requires distribution system to be completely water-tight.

5. Maintenance of the distribution system should be easy and economical.

6. Water should remain available during breakdown periods of pipeline.

7. During repairs, it should not cause any obstruction to traffic.

LAYOUTS OF DISTRIBUTION NETWORKS

The distribution pipes are generally laid below the road pavements, and as such, their layouts will generally follow the layouts of the roads. There are, in general, four different types of pipe networks; any one of which, either singly or in combinations, can be used at a particular place, depending upon the local conditions and orientation of roads. These systems are:

1. Dead end system.
2. Grid iron system.
3. Ring System and
4. Radial system.

1. Dead End System

It is also known are tree system. It is a layout which consists of one supply main from which sub mains are taken and branches are taken from the sub main which ends at service consumers. This type of layout may have to be adopted for old towns which have developed in a haphazard manner, without properly planned roads. The water supply mains have then to be taken along the main roads, and branches taken off wherever needed, thus resulting in the formation of a number of dead ends as shown in Fig.

1. Dead End System

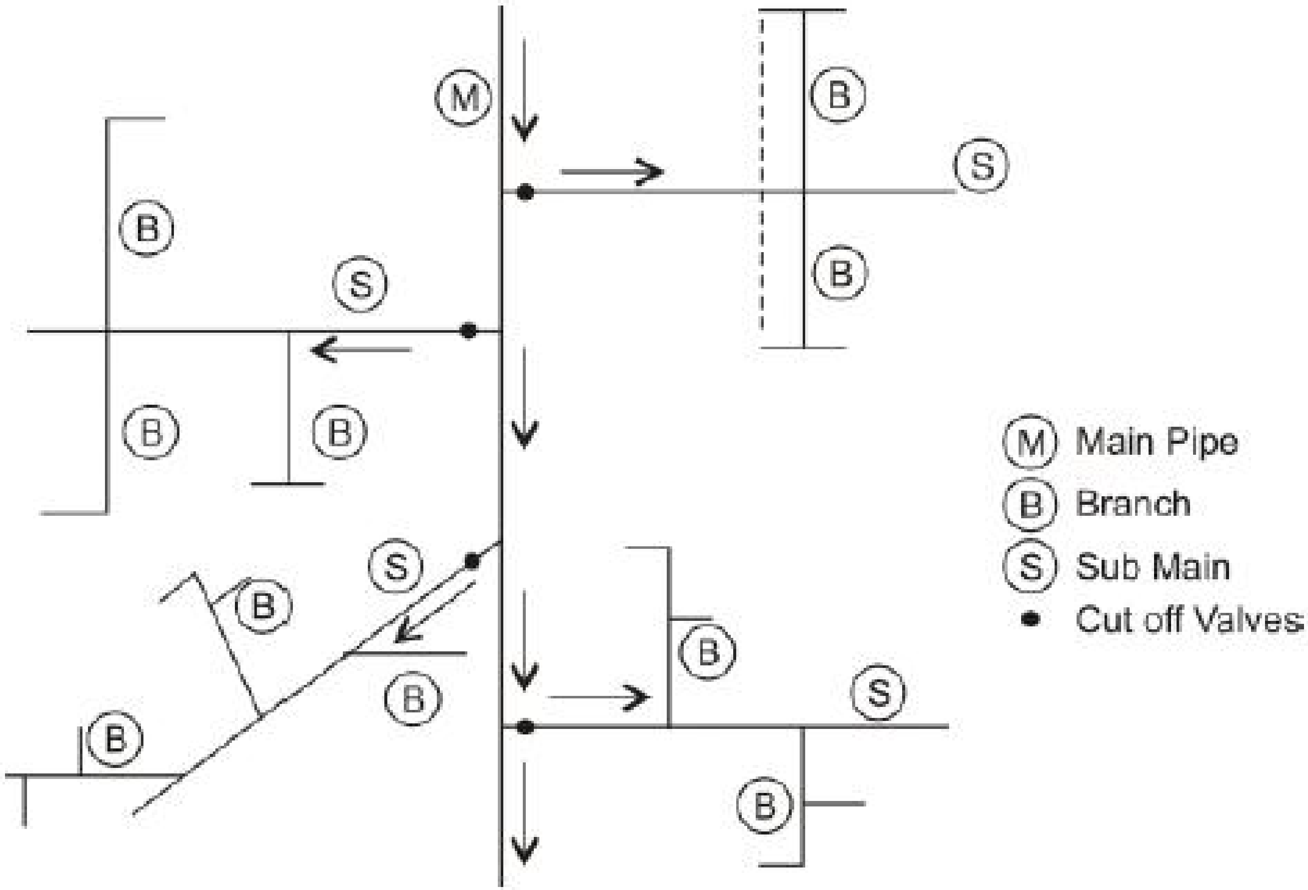


Figure 11.1 Dead end or Tree systems

Advantages:

The advantages of this system are:

1. It is a simple layout which is easy to be performed.
2. Analysis and calculations are easy.
3. Construction cost is less.
4. Pipe with lesser diameter can be used.
5. A number of valves required are less.
6. It is easy to operate and maintain.

Disadvantages:

The disadvantages of this system are:

1. Stagnation of water at the dead end.
2. Pressure remains unbalanced.
3. The problem occurs during firefighting as the discharge is low.
4. Suspended particle settles in the pipe.
5. The problem arises when repair need to be done as large part (area) gets affected.

2. Grid iron system.

It is also known as interlaced system or reticulation system. This system is an improvement over dead end system. In this type of system main, sub mains and branch are interconnected. Along the main roads, the main supply line is laid. Sub mains are then taken in both the directions as shown in the figure 11.2. Branches are taken from mains and sub-mains and are interconnected. In a well planned city or a town, the roads are generally developed in a grid-iron pattern, and the pipe lines in such places can follow them easily.

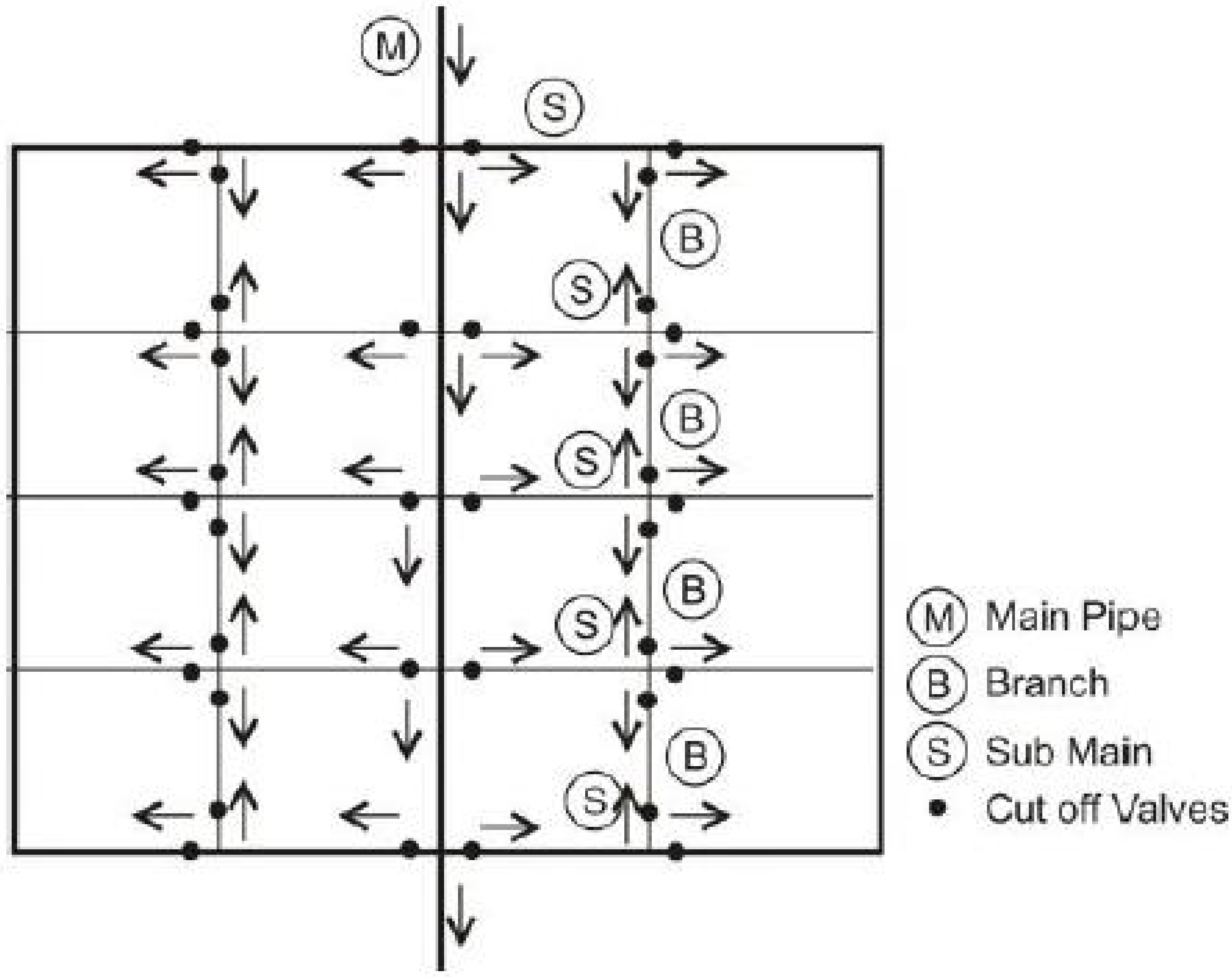


Figure 11.2 Grid-iron system.

Advantages:

The advantages of this system are:

1. No stagnation of water takes place as there are no dead ends present.
2. Water is available in adequate amount for firefighting.
3. The suspended particle does not settle in the pipe.
4. Only a small part (area) gets affected during repair.

Disadvantages:

1. The disadvantages of this system are:
2. Analysis and calculations are difficult.
3. A number of valves required are more.
4. The cost is more for laying pipes than in dead end layout.

3. Ring System

This system is also sometimes called circular system. An area to be served is fixed and then the main is laid around that area. The sub mains are taken from the mains and are run on inside the area to be served as shown in Fig.11.3. The distribution area is divided into rectangular or circular blocks and main water pipes are laid on the periphery of these blocks. This system is used for cities with planned roads. The advantages and disadvantages of this system are the same as that of the grid iron system.

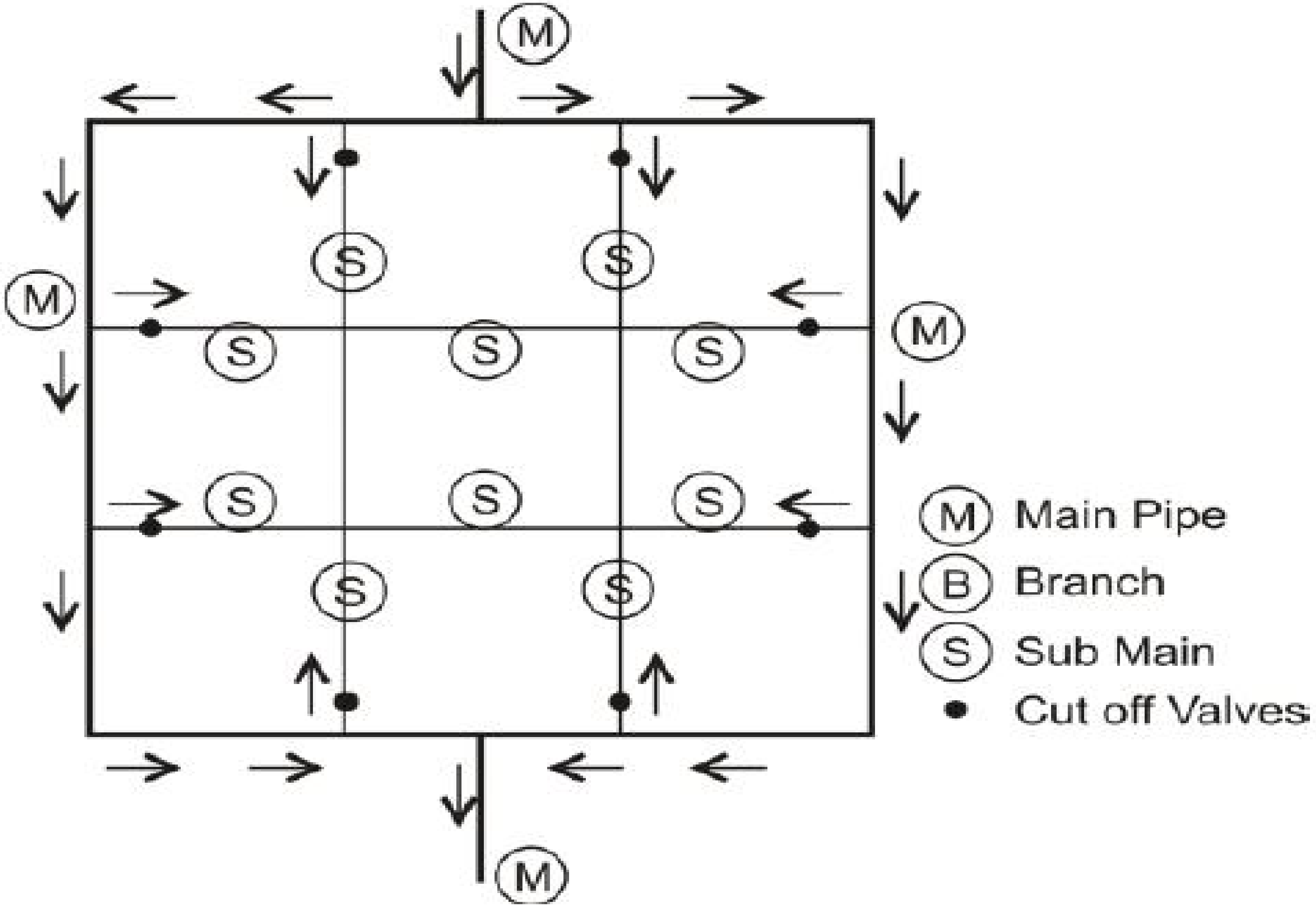


Figure 11.3 Ring system.

4. Radial system.

In this system the area to be served is dividing into smaller zones and each zone is supplied with the distribution reservoir and the water is supplied radially towards the distribution zone as shown in Fig. It supplies water with high pressure and low head loss. It is just the reverse of the ring or circular system. During actual practice, only a single way of the layout is not possible so a combination of layouts is used. The calculations for design of sizes are also simple.

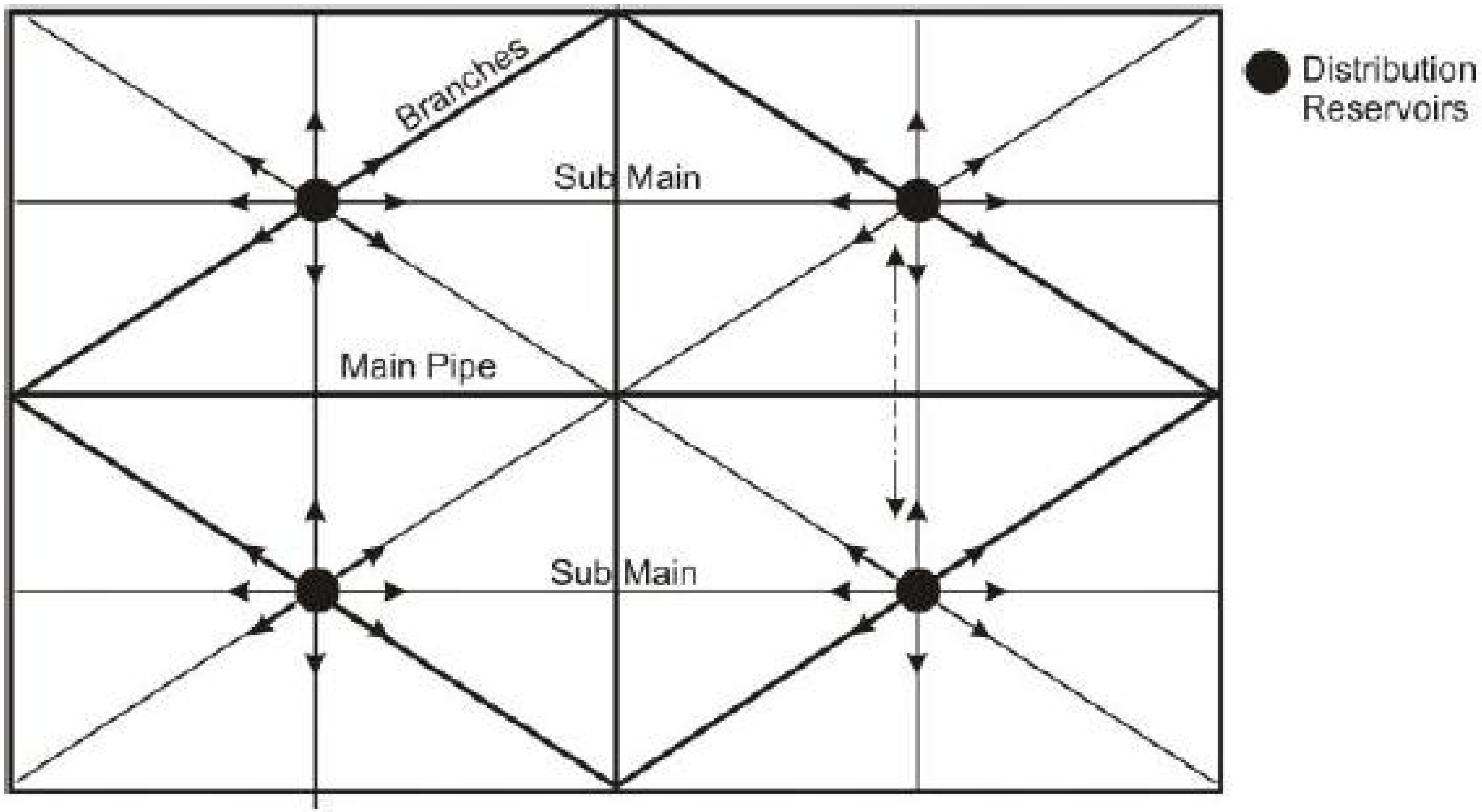


Figure 11.4 Radial system

11.4 METHODS OF DISTRIBUTION

The main object of a distribution system is to develop adequate water pressures at various points of consumer’s taps. Depending upon the level of the source of water and that of the city, topography of the area, and other local conditions and considerations, the water may be forced into the distribution system in the following three ways.

1. By gravitational system;
2. By pumping system; and
3. By combined gravity and pumping system.

Gravity system

In this system, the water flows under the force of gravity from the distribution reservoir to the distribution area. This system is suitable when the source of water treatment plant and the distribution reservoir are situated at a high level than the distribution area. This method is the most economical and reliable, since no pumping is involved at any stage. However, it needs a lake or a reservoir as a source of supply.

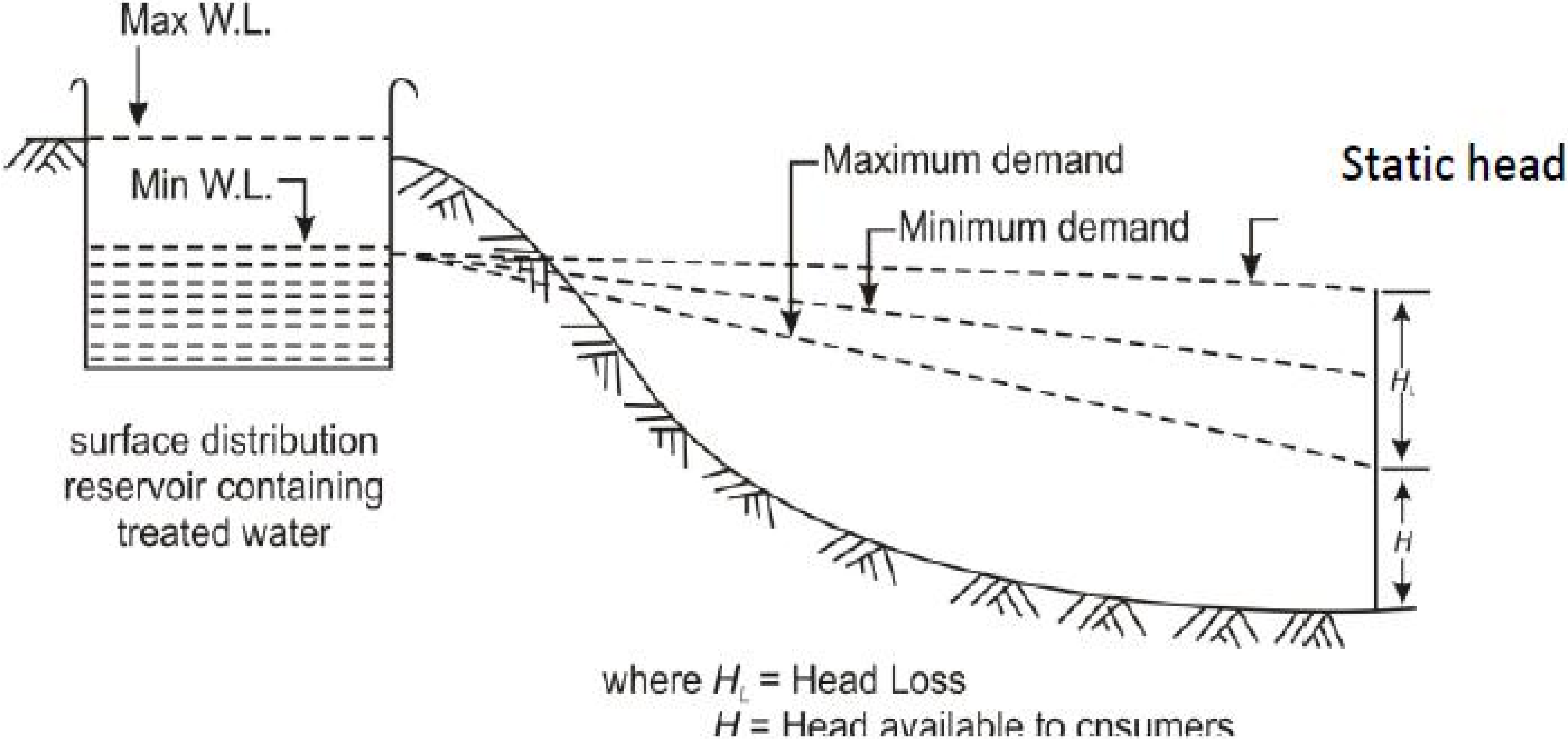


Figure 11.5Gravitational distribution system

Pumping system

As shown in Fig. 11.6, in this system, the water is lifted from the lower elevation and directly supplied to the consumers. Here, the treatment plant is not necessary. This system is adopted when suitable surface source is not available near the town or city. But this system is solely dependent on the mechanical and electrical power. So in case of any failure of the mechanism, the supply of water is highly disturbed. In this system, the water pressure on the consumer’s end is high.

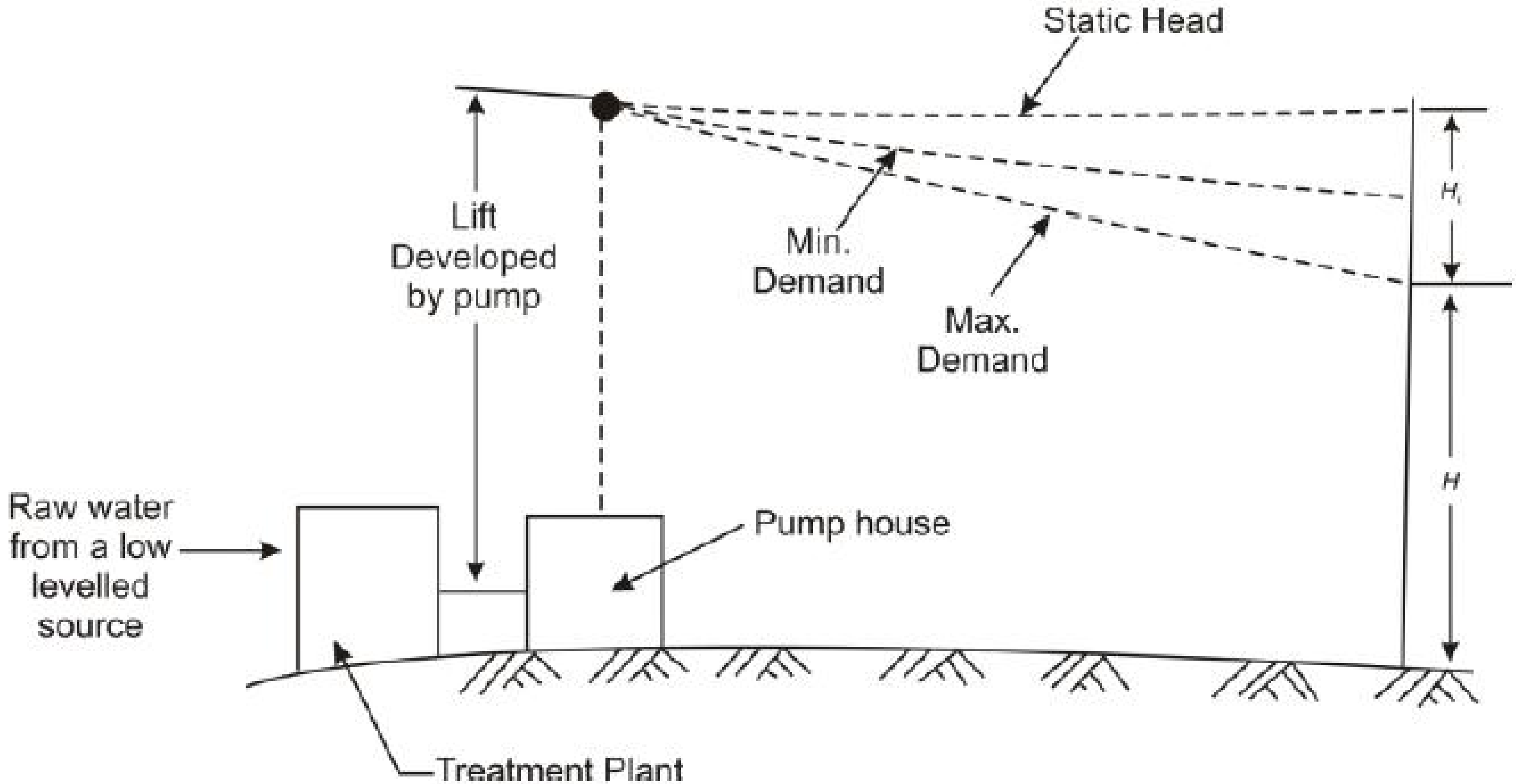


Figure 11.6 Pumping system for water distribution

Combined gravity and pumping system.

In this system, the treated water is pumped at a constant rate and stored into an elevated distribution reservoir, from where it is distributed to the consumers by the mere action of gravity. Sometimes, the entre water is first of all pumped into the distribution reservoir, and many a times, it is pumped into the distribution mains and reservoirs, simultaneously. This method thus, combines pumping as well as gravity flow, and is sometimes called pumping with storage system. Fig. 11.7 shows this system with the hydraulic gradient lines for minimum and maximum demand.

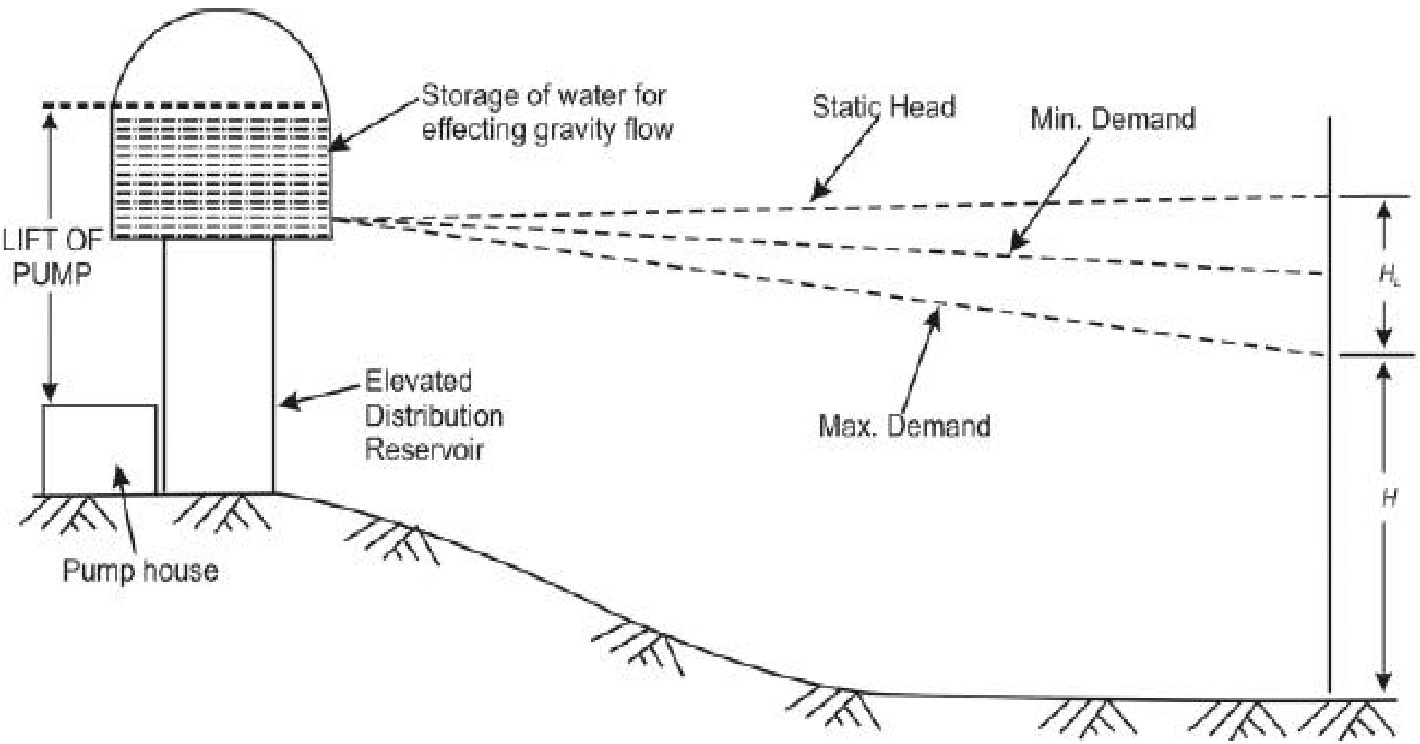


Figure 11.7 Combined gravity and pumping system for water distribution

11.5 SYSTEMS OF WATER SUPPLY

Water may be supplied to the consumers by the following two systems:

1. Continuous water supply
2. Intermittent water supply

Continuous Supply

In this case water is available for 24 hours. So the system is always under pressure. So there is no chance of infilteration i.e, negative pressure cannot occur and as a result the quality of water is better. If there are some minor leakages etc. in the system, large volume of water is wasted because of long duration of flow. In this system, water is not stagnant in the pipe at any instant, and hence fresh water is always available.

Intermittent supply

In this case, water is supplied to the consumers only during some fixed hours of the day. This is the most common system adopted in India. This method is adopted either sufficient pressure is not available or when sufficient quantity is not available. Under these circumstances, various distribution zones of the city are supplied water by turn.

PIPE APPURTENANCES

12.1 INTRODUCTION

In water works, the various types of pipe appurtenances such as valves, sluices, sockets, elbow etc. are needed to control the flow of water, to release the excessive pressure in the pipe line, to eliminate the accumulation of air in the sumits of the pipe line. Again, in house plumbing variuos typesof pipe fitting such as taps, sockets, elbows, nipples, stop cocks, gate valves, check valves, tees etc. are required.

The necessities of the various appurtenances in distribution system are as follows:

1. To control the rate of flow of water.

2. To release or admit air into pipeline according to the situation.

3. To prevent or detect leakages

4. To meet the demand during emergency and

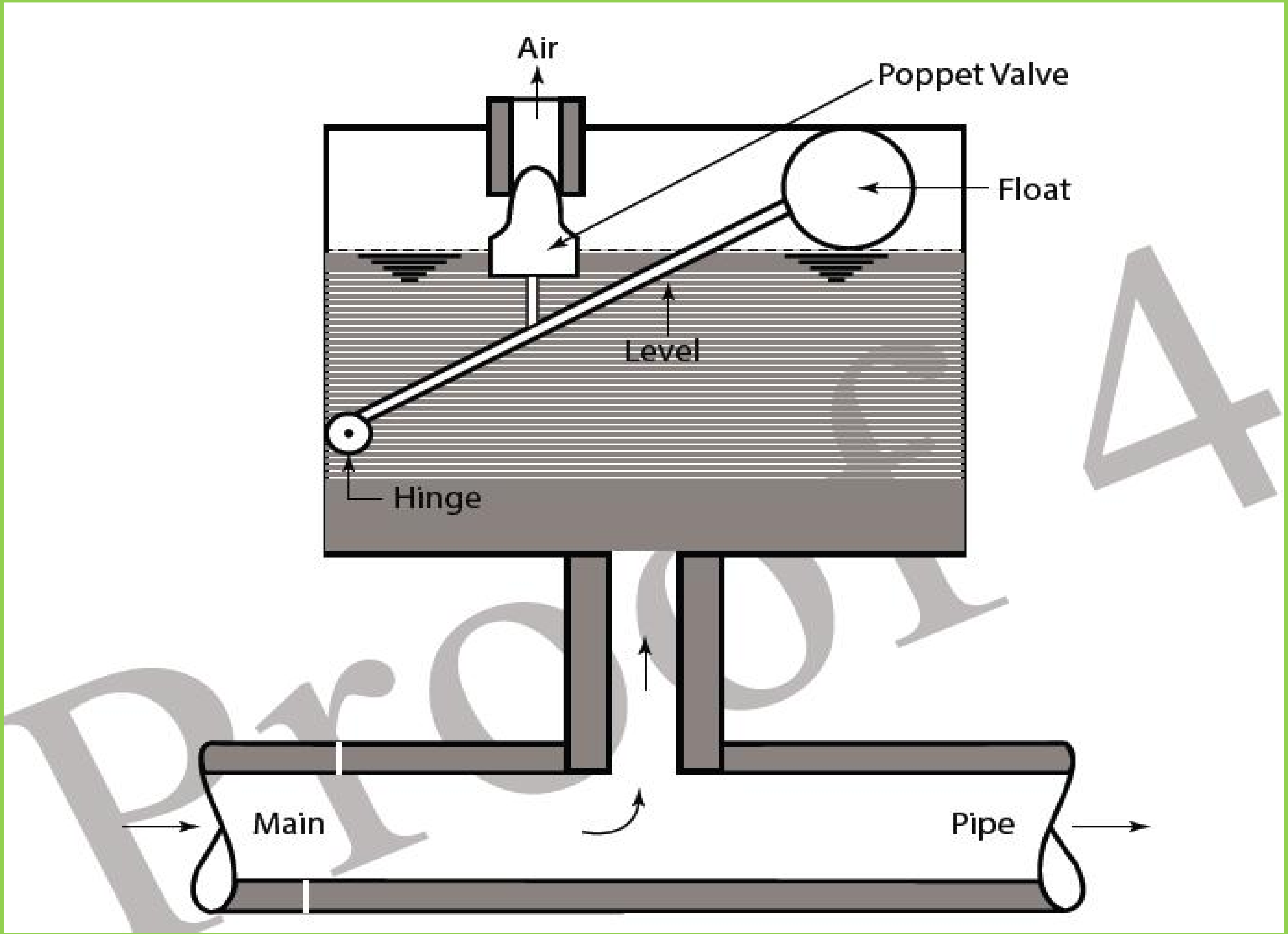
5. Ultimately to improve the efficiency of the distribution

The following are the important appuertenances in pipe lines:-

1. Air Valves
2. Reflux valves
3. Relief valves
4. Sluice valves or Gate valves
5. Scour valves
6. Stop valves
7. Bib cocks
8. Fire hydrants
9. Ferrule
10. Water-meter

12.2 AIR VALVES

Air valves are also known as air relief valves. The water following through the pipe line always carries some air with it. This air tends to accumulate at the summits of the pipe line. Due to the accumulation of air, a backward pressure is created which causes a blockage to the flow of water. Thus the discharge through the pipe is suddenly decreassed and ultimately it may be stopped. So the air relief valve is provided at the sumit to release the air pressure. The air valve consist of a cast-iron chamber in which a float, lever and a poppet valve are provided, as shown in Fig


Air
Poppet Valve
Float
Level
Hinge
Main
Pipe

REFLUX VALVES

Reflux valves are also known as check valves or non-return valves. These possess some automatic device which allows the water to flow in one direction only. These are made of brass or gun metel. As shown in Fig. 12.2, a valve is provided at one end and it can rest on a projection on the other end. This valve is provided in the pipe line which draws water from the pump. When the pump is operated, the valve is opened and the water flows through the pipe (as indicated by arrows). But, when the pump is suddenly stoped for it fails due to power failure, the valve is automatically closed and the water is prevented from returning to the pump.

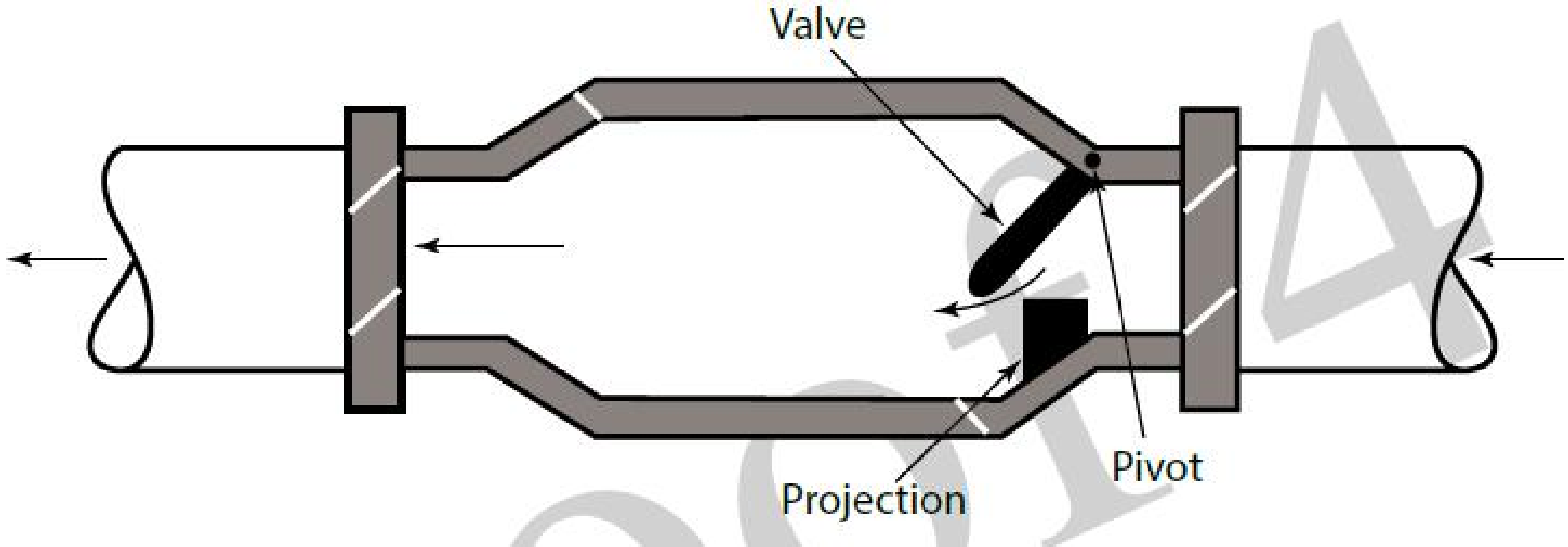


Fig. 12.2 Reflux Valve

RELIEF VALVES

Figure 12.3 shows a relief valve. The relief valves are also known as pressure relief valves or cutoff valves or safety valves. The power of the spiring of the valve is so adjusted that the valve always remains in closed position up to some premissible water pressure in the pipe line. When the pressure of the water suddenly exceeds the premissible pressure due to water hammer phenomenon, then the valve is opned automatically and the excess pressure is released instantaneously. Thus the pipe line is protected from burstin. These valves are provided along the pipe lines at some specific points where the pressure is likely to increase.

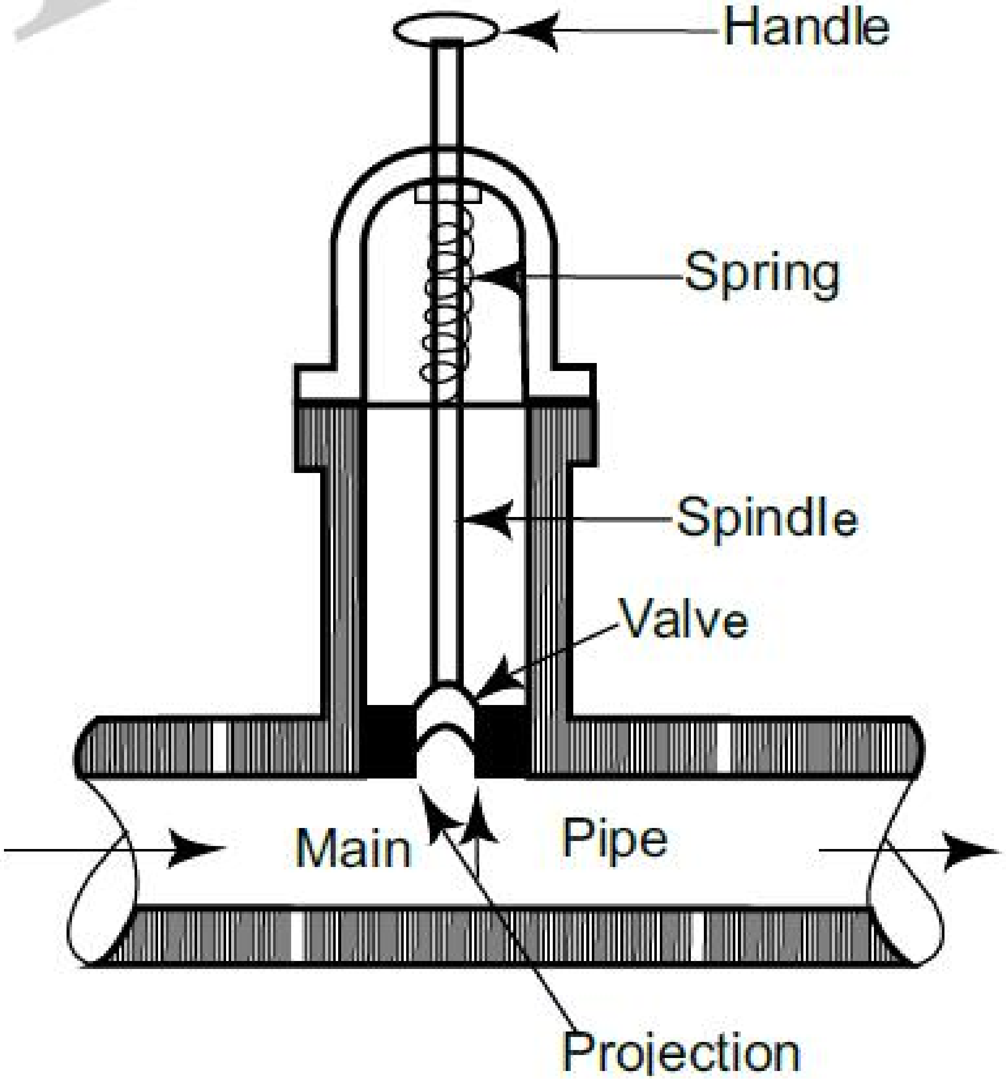


Fig. 12.3 Relief Valve

12.5 SLUICE VALVES

Sluice valves are also known as gate valve or shut off valve. These valves are provided to stop the flow of water through the pipe and are essential to divide the main line into several sections. Moreover, in branch lines or at some specific points on the distribution system these valves are provided to perform the repair works without disturbing the water supply in the other sections. As shown in Fig. 12.4, it consits of a spindle wghich carries a wedge at the button and a handle at the top. The spindle is threaded and can be moved up and down. When the pindle is rotated anti-clockwise, the wedge closes the opening and the flow of water is stopped.

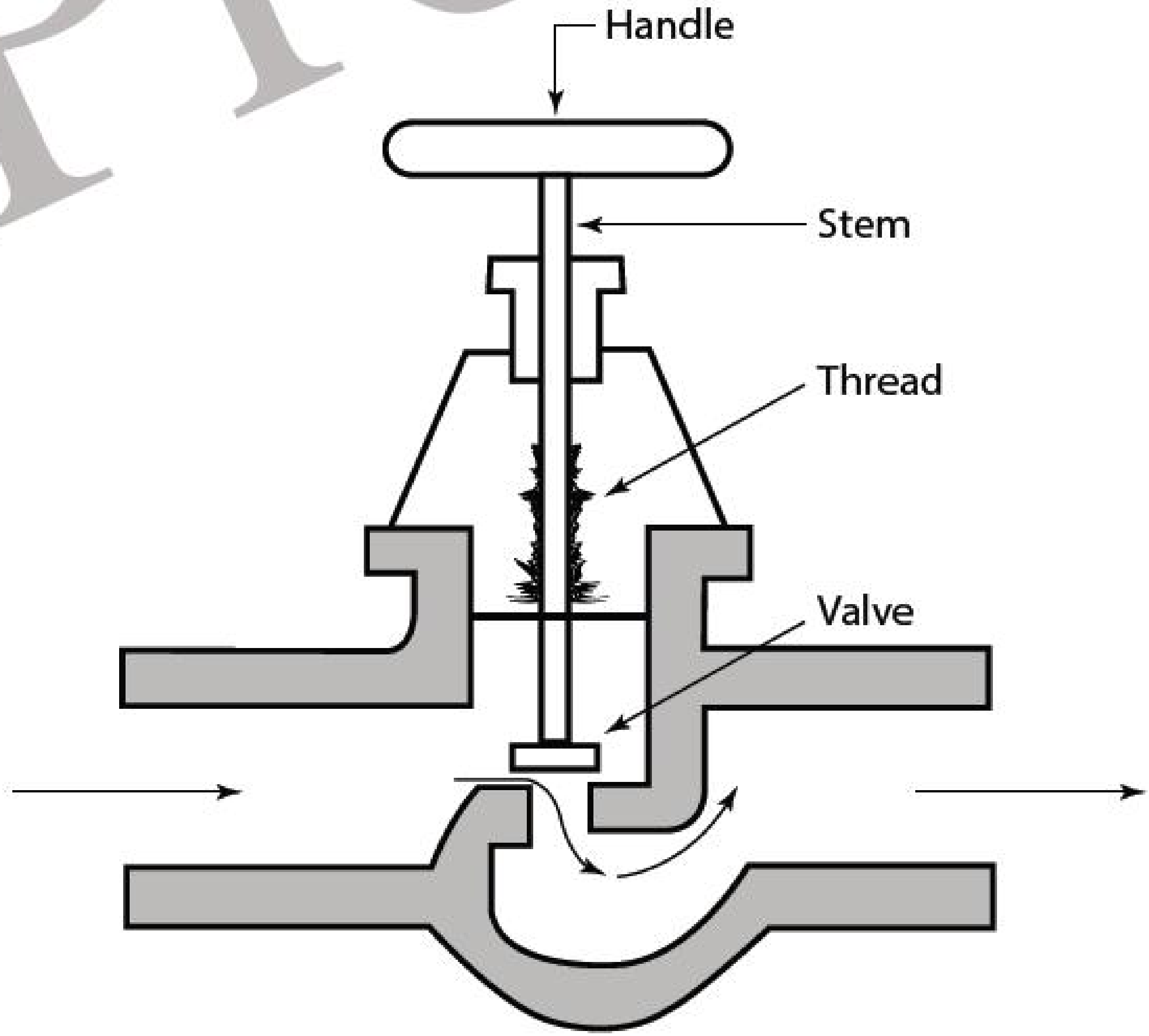


Fig. 12.4 Sluice Valve

14.6 SCOUR VALVES

Scour valves are also known as wash-out valves. These are similar to the sluice valves, but the fuction is different. The scour valves are provided at the dead-end of the pipe line. The function of this valve is to remove the sand, slit, etc. from the pipe line. The valve is opened by turning the spindle and the muddy water is allowed to flow out. When the washing is completed, the valve is closed by turning the spindle.

12.7 STOP COCKS

The stop cocks are practically sluice valve of small sizes. These are provided on the pipe line leading to wash basin, water tanks, flushing tanks, etc. to stop or open the flow of water when necessary. These are made of brass or gunmetel. As shown in Fig. 12.5 the stem of the stop cock is threaded. So, the valve can be moved up and down by turning the handle.

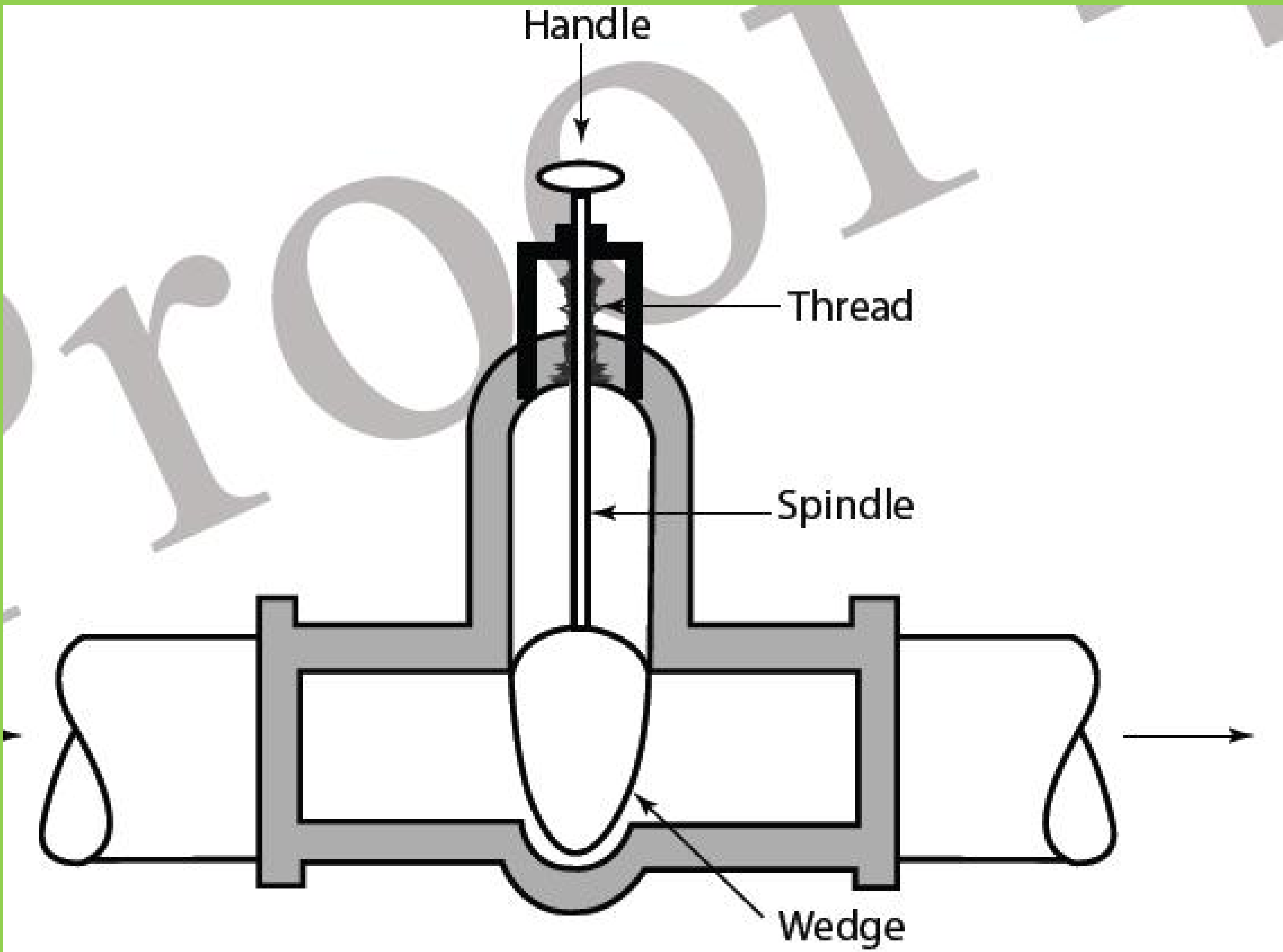


Fig. 12.5 Stop Cock

12.8 BIB COCKS

Bib cocks are small size water taps which are fixed on the pipe line in wash basins, bathrooms, kitchens, etc. from where the consumers obtain water. It is operated by a handl. Figure 12.6 shows a bib cock. The stem of the handle is threade. So, the valve can be moved up and down by turning the handle. The clockwise turning of handle stops the flow of water and anticlockwise turning opens the flow of water. These are generally made of brass or gun metal or plastic.

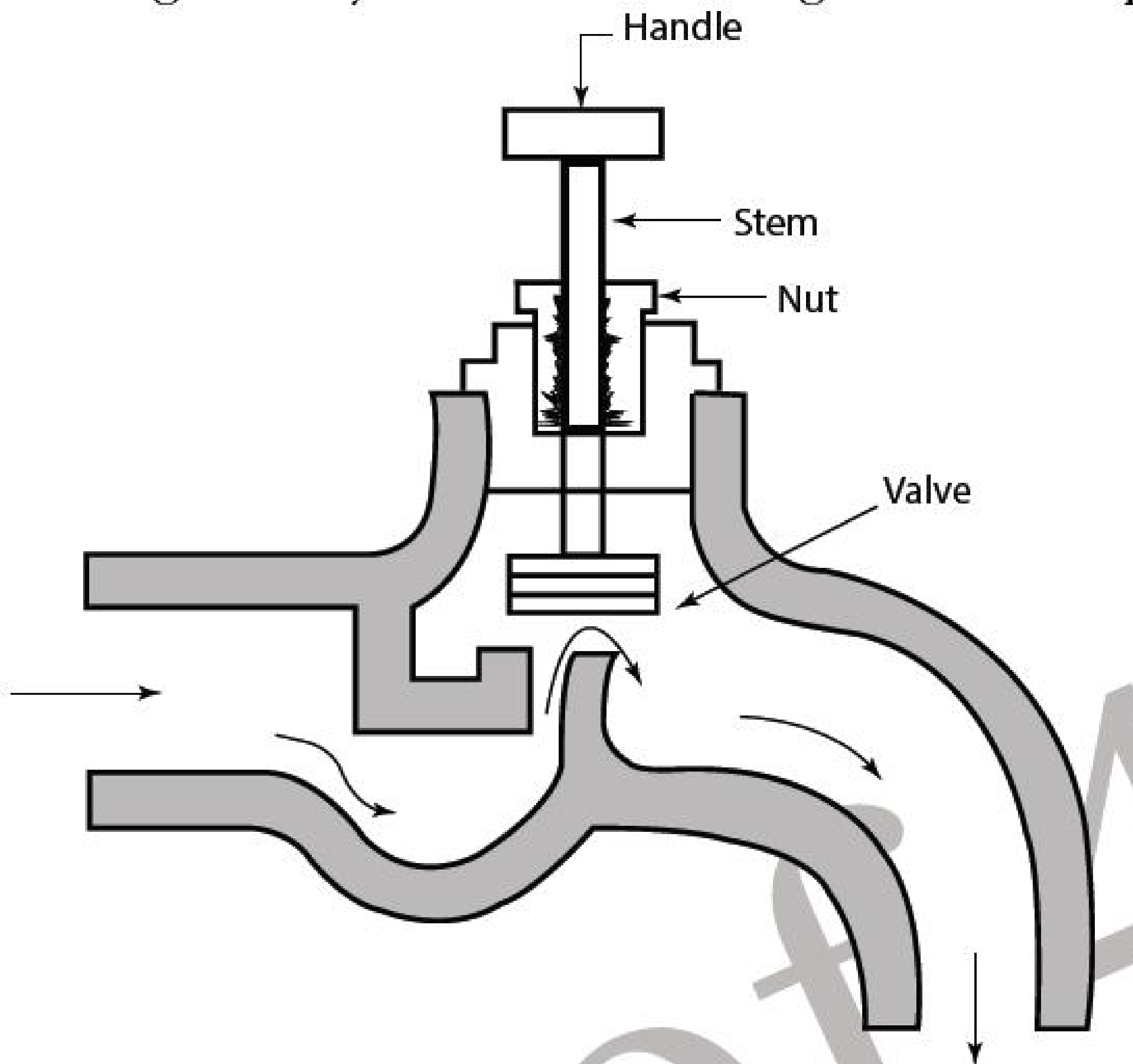
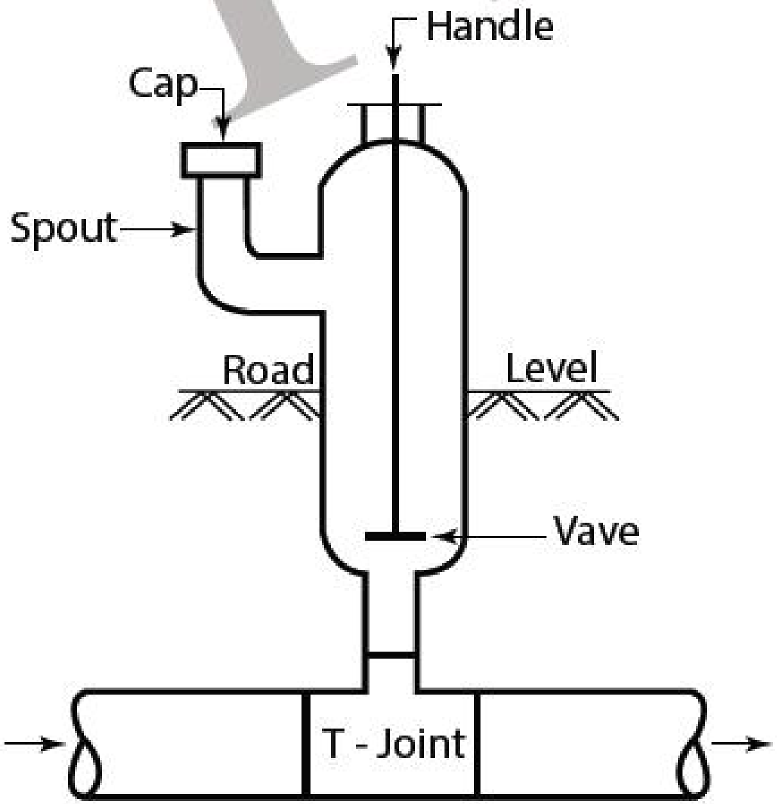


Fig. 12.6 Bib cocks

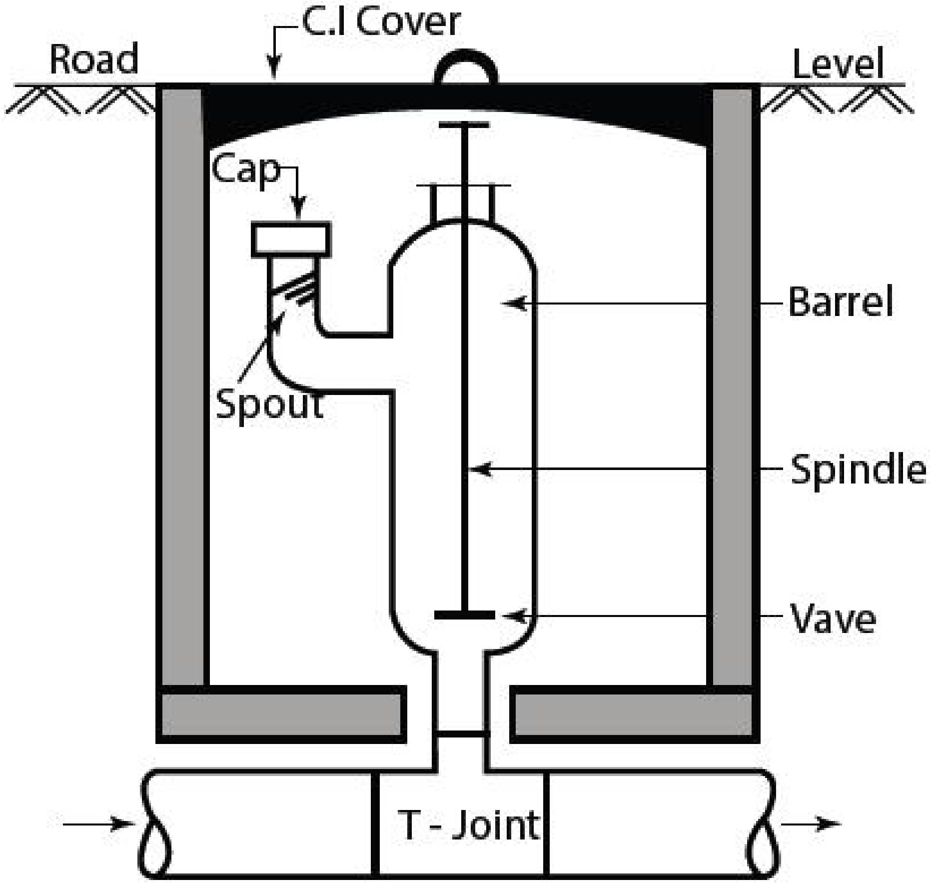
12.9 FIRE HYDRANT

Fire hydrant is an outlet provided in the main waterline for tapping water in case of fire. When it occurs in some places, the fire brigade vehicles run to the apot and connect the hos epipe to the spout by removing the cap. Then the valve is opened by turning the handle. After finishing the work, the cap is replaced and the valve is closed. The hydrants are provided on the main line at important points. The location of the fire hydrant should be marked on a map by the fire-brigade authority. The hydrant may be of two types:-

1. Post hydrant.
2. Flush hydrant



(b) Post hydrant



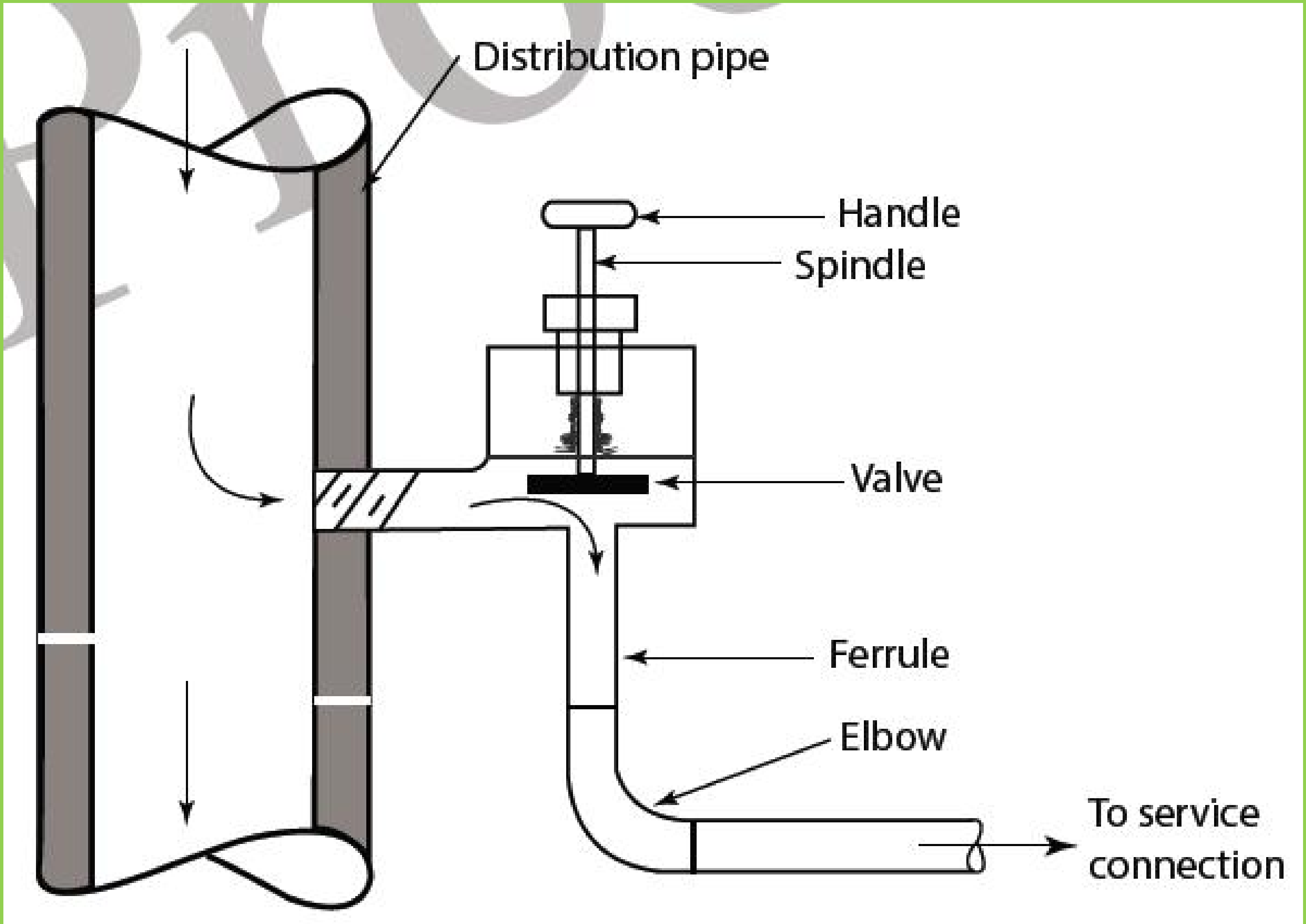
(b) Flush hydrant

The post hydrant (shown in Fig 12.7 (a)) is projected above the road level. This type of hydrant is prominent and can be located easily. But, it is liable to be damaged by the miscreant.

The flush hydrant is provided below the road level. It is provided by cast iron box or brick mansory. A cast iron cover is placed over the box (Fig. 12.7 (b)). But it is difficult to locate the position of the hydrant easily. However, some signal should be provided above the ground level to detect the hydrant easily.

12.10 FERRULE

Ferrule is a device by which water connection is given to the consumers. It is connected between the distribution pipe and service connection. It controls the quantity of water to be supplied to the consumers. In case of any dispute, the water supply to a house is disconnected by operating the ferrule. It is manufactured of brass or gunmetal in the shape of a 'T'. As shown in Fig 12.8, the open ends of the ferrule are threaded. One end is connected to the distribution pipe by making a hole and the other end is connected to service connection pipe with the help of an 'Elbow'. The valve is moved up and down by rotating the handle.



Distribution pipe
Handle
Spindle
Valve
Ferrule
Elbow
To service
connection

12.11 WATER METER

The device by which the quantity of water flowing through a particular point is measured is known as water meter. It helps directly to compute the volume of water used by a consumer from the reading on the meter. The water-tax is cahrged according to the volume of water consumed. Figure 12.9 shows a rotary water meter. The mete may be following types:-

1. Displacement type
2. Velocity type

The displacement type meter records, the number of times a container of known volume is filled up and emptied by the flowing water. From the reading, the volume of water can be worked out.

Fig. 12. 9 Rotary water meter

The velocity type meter gives a reading on the dial according to the velocity of flow of water. Form the reading, the volume of water can be worked out from the manufacturer’s rating table.

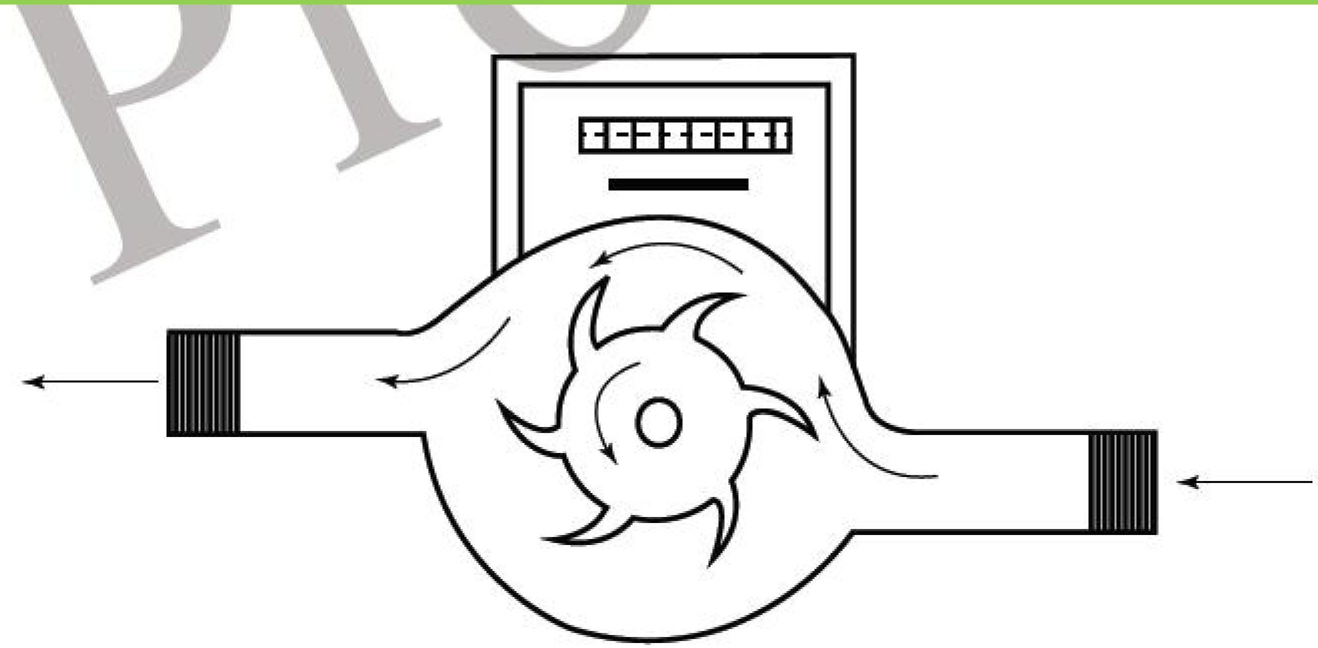


Fig. 12. 9 Rotary water meter

The displacement type meter records, the number of times a container of known volume is filled up and emptied by the flowing water. From the reading, the volume of water can be worked out.

ANALYSIS OF DISTRIBUTION NETWORKS

Pipe network analysis is the analysis of the fluid flow through a hydraulics network, containing several or many interconnected branches. The aim is to determine the flow rates and pressure drops in the individual sections of the network. This is a common problem in hydraulic design. The following are the two important methods are used for analysis of pressures in the distribution system.

1. Hardy cross method
2. Equivalent pipe method

Hardy-Cross Method

Analysis of water distribution system includes determining quantities of flow and head losses in the various pipe lines, and resulting residual pressures. In any pipe network, the following two conditions must be satisfied:

1. The algebraic sum of pressure drops around a closed loop must be zero, i.e. there can be no discontinuity in pressure.
2. The flow entering a junction must be equal to the flow leaving that junction; i.e. the law of continuity must be satisfied.

Hardy-Cross Method

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1. The algebraic sum of pressure drops around a closed loop must be zero, i.e. there can be no discontinuity in pressure.
2. The flow entering a junction must be equal to the flow leaving that junction; i.e. the law of continuity must be satisfied.

Based on these two basic principles, the pipe networks are generally solved by the methods of successive approximation. The widely used method of pipe network analysis is the Hardy-Cross method.

This method consists of assuming a distribution of flow in the network in such a way that the principle of continuity is satisfied at each junction. A correction to these assumed flows is then computed successively for each pipe loop in the network, until the correction is reduced to an acceptable magnitude.

If Q_a is the assumed flow and Q is the actual flow in the pipe, then the correction Δ is given by

$$\Delta = Q - Q_a; \text{ or } Q = Q_a + \Delta$$

Now, expressing the head loss (H_L) as

$$H_L = K.Q^n$$

we have, the head loss in a pipe

$$= K.(Q_a + \Delta)^n$$

$$= K.[Q_a^n + x.Q_a^{n-1}\Delta + \text{.........negligible terms}]$$

$$= K.[Q_a^n + x.Q_a^{n-1}\Delta]$$

Now, around a closed loop, the summation of head losses must be zero.

ie. $\sum K.[Q_a^n + x.Q_a^{n-1}\Delta] = 0$

or $\sum K.Q_a^n = - \sum Kx\, Q_a^{n-1}\Delta$

Since, Δ is the same for all the pipes of the considered loop, it can be taken out of the summation.

$$\sum K.Q_a^n = - \Delta. \sum K\, n\, Q_a^{n-1}$$

or

$$\Delta = \frac{-\Sigma K.Qa^n}{\Sigma n.KQ\, a^{n-1}}$$

Since Δ is given the same sign (direction) in all pipes of the loop, the denominator of the above equation is taken as the absolute sum of the individual items in the summation. Hence,

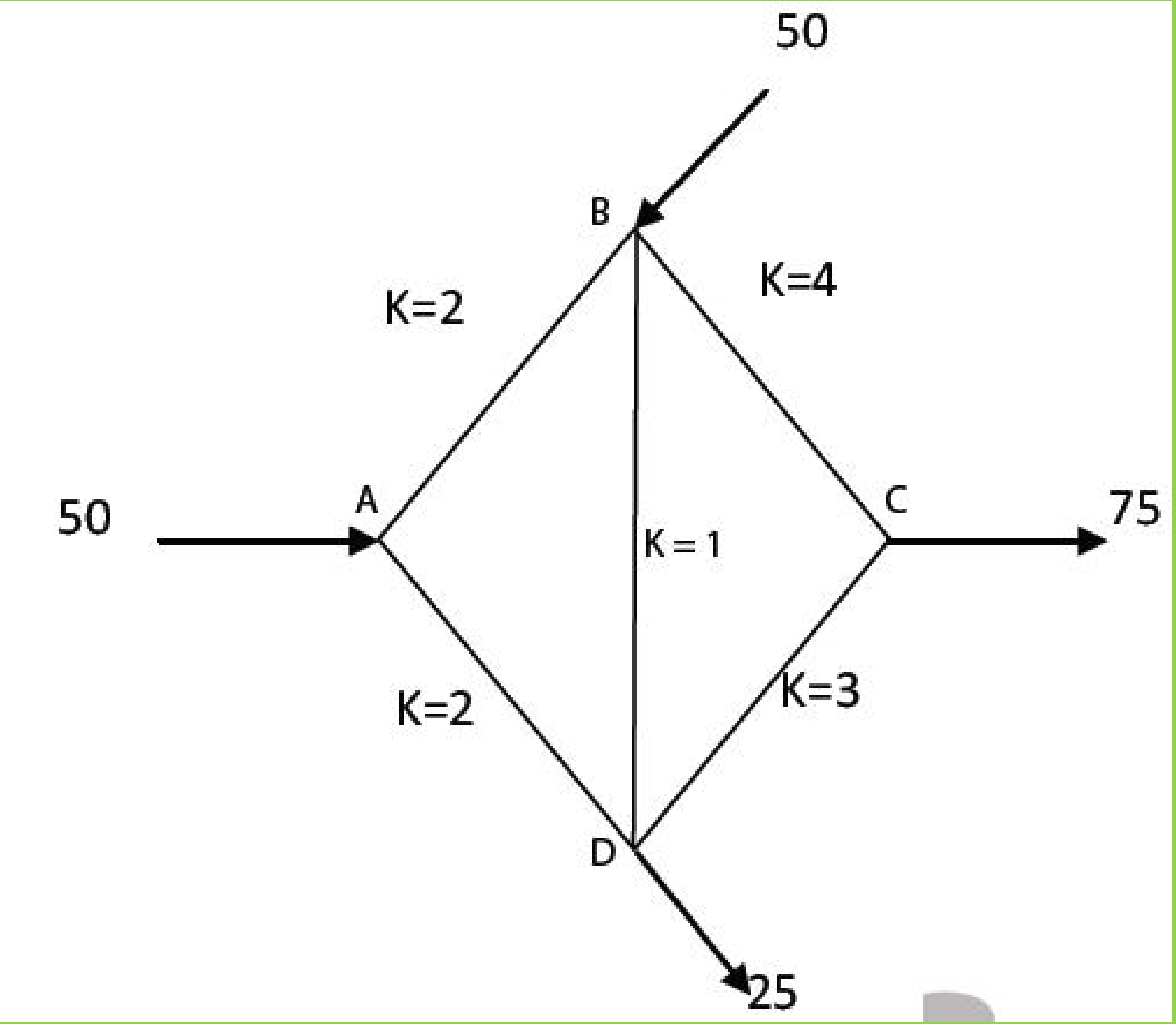
$$\Delta = \frac{-\Sigma K.Qa^{n}}{\Sigma\left|n.\mathrm{KQ}\,a^{n-1}\right|}$$

or

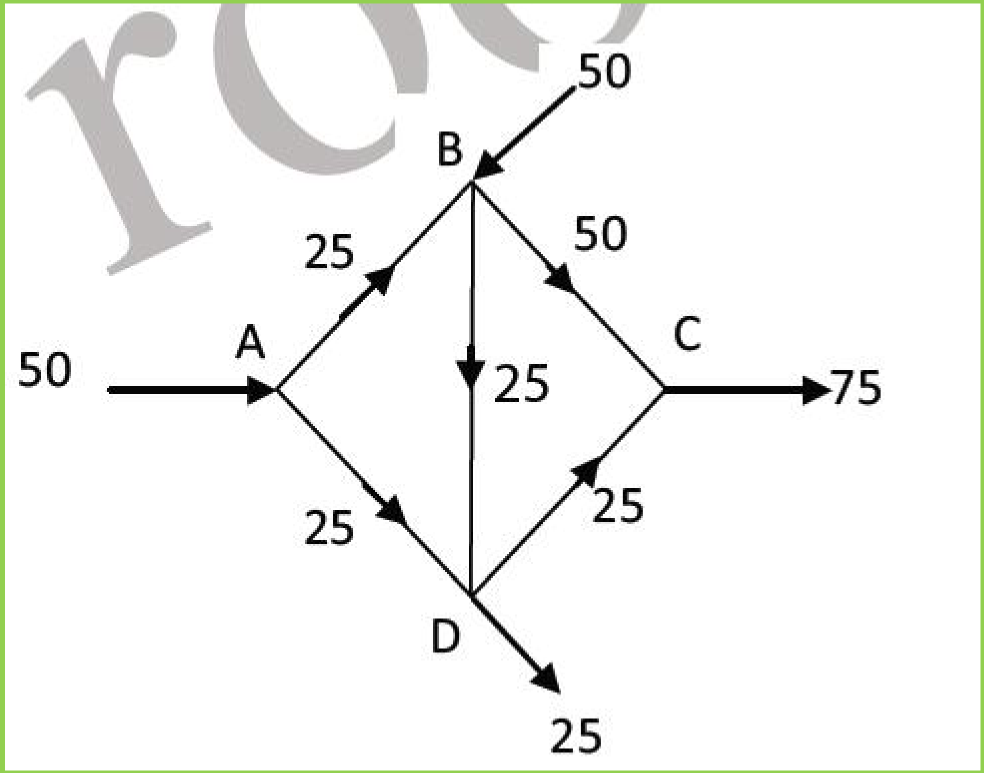
$$\Delta = \frac{-\Sigma H_L}{n.\Sigma\left|\dfrac{H_L}{Q_a}\right|}$$

Where, H_L is the head loss for assumed flow Q_a.

Problem11.1: Determine the distribution of flow in the pipe network shown in figure. The head loss h_L, may be assumed as KQ^n. The flow is turbulent and pipes are rough. The value of k for each pipes are rough. The value of k for each pipe is indicated in the figure. Use Hardy-Cross method.


50
B
K=2
K=4
50
A
K = 1
C
75
K=2
K=3
D
25

Solution: First of all, the magnitudes as well as the directions of the possible flows in each pipe are assumed keeping in consideration the law of continuity at each junction.



The head loss in each pipe is taken to KQ^n. Assume n=2 for using Manning's or Darcy's formula.

Hence,

$H_L = KQ^2$

Use given values of K for analyzing the two loops ABDA and BCDB by Hardy Cross procedure.

For First correction

Pipe	Assumed flow Q_a (l/s)	K (given)	$H_L = KQ_a^2$	$\left\lvert \frac{HL}{Qa} \right\rvert$	Corrected Q after I correction $Q_{a1} = Qa + \Delta_1$ l/s
<u>For loop ABDA</u>					
AB	25	2	1250	50	22.5
BD (Common)	25	1	625	25	25-2.5+12.5=35
DA	-25	2	-1250	50	-27.5
Sum Σ			625	125 $\Delta_1 = \frac{-\sum HL}{n \sum \left\lvert \frac{HL}{Qa} \right\rvert}$ $= \frac{-625}{2 \times 125} = -2.5$	
<u>For loop BCDB</u>					
BC	50	4	10000	200	37.5
CD	-25	3	-1875	75	-37.5
DB (Common)	-25	1	-625	25	-25+2.5-12.5= -35
Sum Σ			7500	300 $\Delta_1{}^1 = \frac{-\sum HL}{n \sum \left\lvert \frac{HL}{Qa} \right\rvert}$ $= \frac{-7500}{2 \times 300} = -12.5$	

Second correction

Pipe	Assumed flow Q_a (l/s)	K (given)	$H_L=KQ_a^2$	$\left\lvert\frac{HL}{Qa}\right\rvert$	Corrected Q after II correction $Q_{a1}=Qa+\Delta_1$ l/s
<u>For loop ABDA</u>					
AB	22.5	2	1012.5	45	19.8
BD (Common)	35	1	225.0	35	35-2.7+0.3=32.6
DA	-27.5	2	-1512.5	55	-30.2
Sum Σ			725	135 $\Delta_2=\frac{-\Sigma HL}{n.\Sigma\left\lvert\frac{HL}{Qa}\right\rvert}=\frac{-725}{2\times135}=-2.7$	
<u>For loop BCDB</u>					
BC	37.5	4	5625	150	37.2
CD	-37.5	3	-4218.75	112.5	-37.8
DB (Common)	-35	1	-1225	35	-35-0.3+2.7=-32.6
Sum Σ			181.25	297.5 $\Delta_2^1=\frac{-\Sigma HL}{n\Sigma\left\lvert\frac{HL}{Qa}\right\rvert}=\frac{-181.25}{2\times297.5}=-0.3$	

The corrected flow after second correction is plotted on pipe network in the figure given below.