MODULE-1

NEED FOR PLANED WATER SUPPLIES

- Water is a chemical compound and may occur in a liquid from in a solid form as in a gaseous form. All these three forms of water are extremely use full to mass providing the luxuries and imports, in addition to full filing his basic necessities of life
- Every one of us knows how important and precious the water is whenever thus no water in our taps is, we become helpless. No life is existing without water, since water is as essential for Life as air.
- It has been estimated that two –third of human body is constituted of water. Water is absolutely essential not only for survival of human beings, but also for animals, Plants and all other living beings.
- Further it is necessary that the water required for their need must be good, and it should not contain unwanted impurities are harmful chemical compounds as bacteria in it .Therefore, in order to insure the availability of sufficient quantity of good quality water, it becomes Almost impact in modern society, to plan and built suitable water supply schemes which may Provide potable water to the various section of community in accordance with their demands And requirements.
- The provision of such a scheme shell insure a constant and reliable water supply to that section of people for supplying safe whole some water to the people for drinking ,cooking, bathing ,washing etc.
- So as to keep the diseases away and there by promoting better health, but would also help in supplying water for fountains, gardens etc. and thus helping in maintaining better sanitation and beautification of surroundings. There by reducing environmental pollution besides promoting overall hygiene and public health, it shall insure safety against fire by supplying sufficient quantity water to extinguish it.
- The Existence of such a water supply scheme shall further help in attracting industries and thereby helping industrialization and modernization of the society, consequently reducing unemployment and ensuring better living standards such scheme shall therefore, help in promoting wealth and welfare of the entire humanity as a whole.

Following are the requirements for a protected Water supply scheme

- To provide potable water to the various section of a community in accordance with their demand and requirements.
- To ensure a constant and reliable water supply to the section of the peoples for which it has been designed.
- To supply water for fountain, garden etc.
- To ensure safety against fire by supplying sufficient quality of water to extinguish it.
- Tropes water supply scheme is essential for modernization and industrialization.

DEMAND FOR WATER

Which planning a water supply scheme, it is necessary to find out not only the total yearly water demand but also to assess the required average rates of flow (or draft) and the variations in these rates. The following quantities are therefore, generally assessed and recorded.

i) Total annual volume (V) in liters or million liters.
ii) Annual average rate of draft in liters per day, i.e. V/365
iii) Annual average rate of draft in liters per day per person i.e., liters per capita per day or lpcd called PER CAPITA DEMAND (q) or RATE OF DEMAND.
iv) Average rate of draft in liters per day per service i.e. $\frac{V}{365} \times \frac{1}{\text{No of services}}$
v) Fluctuations in flows expressed in terms of percentage ratios of maximum or minimum yearly, monthly, daily or hourly rates to their corresponding average values.

It is difficult to precisely assess the quantity of water demanded by the public, since there are many variable factors affecting water consumption certain thumb rules and empirical formulas are therefore generally used to assess this quantity, which may give fairly accurate results. The use of a particular method or a formula for a particular case has therefore, to be decided by the intelligence and fore sightedness of the designer. The various types of water demands, which a city may have, may be divided into the following classes.

i) Domestic water demand
ii) Industrial and commercial water demand
iii) Demand for public uses
iv) Fire demand
v) Water required compensating losses in wastes and thefts.

As correctly as possible the total water demand of a particular section of the community, all these demands must be considered and suitable provision made depending upon the needs of those people for whom the water supply scheme is to be designed.

DOMESTIC WATER DEMAND: This includes the water required in private buildings for drinking, cooking, bathing, lawn sprinkling, Gardening, sanitary purposes etc….
This amount varies according to the living conditions of the consumers on an average this domestic consumption under normal conditions in an Indian city is expected to be around 135 liters /day/person as per Id: 1172, 1971. The total domestic consumption generally amounts to 50- 60% of the total water consumption.

AVERAGE DOMESTIC WATER CONSUMPTION IN A INDIAN CITY

USE	**CONSUMPTION IN LPCD**
Drinking	5
cooking	5
Bathing	55
Washing of clothes	20
Washing of utensils	10
Washing and clearing of houses and residences	10
	30
TOTAL	135 lpcd

INDUSTRIAL AND COMMERCIAL WATER DEMAND

This includes the quantity of water required to be supplied to offices, Factories, different industries, hospitals, hostels, etc…. This will vary considerably with the nature of the city and with the number and types of industries and commercial establishment there is no direct relation of this consumption with the population and hence the actual requirements for all industries should be estimated. The water requirements for buildings other than residences as per isstandards are as follows.

Type of building	Age consumption in lpcd
1.Factories a) where bathrooms are required tobe provided b) where no bathrooms are requiredto be provided	45 30
2.Hospitals (including laundry) per bed a) Number of beds < 100 b) Number of beds > 100	 340 450
3.Nurse homes and medical quarters	135
4.Hostels	135
5.Hotels (Per bed)	180
6.Restauvants (Per seat)	70
7.offices	45
8.Cinemas, Auditoriums and theatres (per seat)	15

9. Schools a) Day schools b) Residential school	 45 135

DEMAND FOR PUBLIC USES (MUNICIPAL CONSUMPTION)

This includes the quantity of water required for public parks, gardening, washing and sprinkling on roads, use in public fountains etc…..

A nominal amount not exceeding 5% of the total consumption may be added to meet this demand on an arbitrary basis or else the consumption of water for municipal purposes as given below may be considered.

PURPOSE	**WATER CONSUMPTION**
Public parks	1.4 liters/m^2/day
Road watering	1-1.5 liters/m^2/day
Sewer cleaning	4.5 liters/head/day

Fire demand

In modern public water supply scheme a provision is made for firefighting.

- The quantity of water requirement for extinguishing the fire should be easilyavailable and should be always stored in the storage reservoir.
- The minimum water pressure available at the fire hydraulic should be of theorder of 100 to 150kn/m^2 or 10 to 15m of water head.
- Generally in fire breakout 3 jet streams are simultaneously thrown from eachhydrant, one to the property; which is burning, one to the left side of the property and one to the right side of the property.
- The discharge of each stream should be about 1100lt/min.
- E.g.: A city having a population say 50 lakh, if 6 fires breakout in a day and each fire stands for 3 hours, then calculate the total requirement of water and per capita demand.

Sol: total quantity of water required = 6*30*60*1100*3

= 35, 64,000 L

Per capita demand =35, 64,000/50, 00,000

= 0.7168lpcd < 1lpcd

- From above example, it is very clear that even though the total requirement isvery high and per capita fire demand is less and hence this demand is usually ignored. Which designing public water supply schemes the rate of fire demand is sometimes treated as a function of population and is worked out on basis of certain empirical formulas which are as follows.

WATER REQUIRED TO COMPENSATE IN THEFTS, WASTES, etc..

- It includes the water lost in leakage due to bad plumbing or damaged meters, stolen water due to unauthorized water connections and other losses and wastes, etc…. These losses should betaken into account while estimating the total requirement. These losses can be reduced by careful maintenance and universal metering. Even in the best managed water works this amount is usually taken as 15% of the total consumption.

For an Average. Indian town. As per I.S recommendations the per capita demand may be taken as given in table below.

USE	CONSUMPTION (LPCD)
Domestic use	135
Industrial use	50
Commercial use	20
Civic or public use	10
Waste, theft. Etc…	55
Total	270 lpcd

The above figure or 270 lpcd when multiplied by the population at the end of the design period shall give the total annual average water requirement of the city/day. When multiplied by 365 will give the volume of the yearly water requirement in litres.

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

PER CAPITA DEMAND (RATE OF DEMAND) (Q)

It is the annual average amount of daily water required by one person and includes the domestic use, industrial and commercial use, public use, wastes, thefts, etc…

$$= \text{Per capita Demand in litres/day/head} = \frac{\text{Total yearly water requirement of the city in liters}}{365 \times \text{Design population}}$$

$$q = \frac{V}{365P}$$

FACTORS AFFECTING PER CAPITA DEMAND

1. Size and type of city
2. Climatic conditions

3. Class of consumers
4. Quality of water
5. Pressure in the distribution system
6. Sewerage Facilities
7. System of supply
8. Policy of metering system
9. Cost of water

1. Size of the city :

- The per capita demand of the big city is generally largewhen campare to a small town . this may be due to thefact that big cities huge amount of water is required forclean and health environment
- A sewered house requires, 4 to 5 time the water requiredby un sewered houses
- some times even a smaller town may have high water consumption it is fully industrialized and hence it can be concluded that , the effect of the population on size of thecity is an indirect one

2. Climatic conditions :

- At hotter and dry places the consumption of water is generally more . this is because more water is consumedfor air conditions, air coolers etc...
- But in extremely cool country more water may be consumed , people may keep their tap open to avoidfreezin

3. Habit of the people :

- Rich and upper class comunities generally consumesmore water due to their high living standard
- The amount of water consumed is directly proportional to the economic standards of the consumers

4. Industrial and commercial activities :

- The presence of industrial and commercial activityincrease the water consumption in large amount.

5. Quality of the water supplied :

- If the quality and taste of the supplied water is good ,more water will be consumed because in such cases people will not use other sources of water like private wells and hand pumps etc....
- simillarly certain industries such as bailer feed which required high quality water will not develope there ownsupplies and use only the public water supply.

6. Pressure in the distribution system :

- If the pressure in the distribution system is high and sufficent to make the water to reach at the 3rd or 4th storey then water consumption shall definately will bemore
- The losses due to the leakage will be considerablyincreases if distribution pressure is high
- eg : If the pressure is increased from 20 to 30m (200- 300kN/M^2)then the losses may be increased by 20to30%

7. <u>Developement of sewerage facilities</u> :

- Water consumption will be more if a city provided witha flushing system .

8. <u>System of supply</u> :

- The water may be supplied either continuously or may besupplied during peak hours.(morning and during evening)
- For intermittent supply the losses are less and theconsumers uses water more effectively .

9. <u>System of supply</u> :

- If the water rate are high lesser quantity may be consumed by the people , but rich people are less affectedby the policies.

10. <u>Policy of metering and method of charging</u> :

- Water taxes are generally charged in two different ways(a). on the basis of meter reading .
  (b). on the basis of certain monthly fixed flat rates
- When the supplies are metered people use water morewisely.

ASSESSMENT OF NORMAL VARIATIONS

The maximum demands (monthly, daily or hourly) are generally expressed as ratios of their means. The following figures are generally adopted.

1. **MAXIMUM DAILY CONSUMPTION** is generally taken as 180% of the average, therefore
   Maximum daily demand(MDD) = 1.8 × Average daily demand (^DD)
   = 1.8 q
2. **MAXIMUM HOURLY CONSUMPTION** is generally taken as 150% of its average hooray consumption of maximum day, there fore
   Maximum hourly consumption = 1.5 (150%) × Average hourly consumption of the maximum day orof the maxm. Day.(Litres/day)peak demand

$$= \frac{2.7\,q}{24}$$

Therefore, (Litres/hr.)

Maximum hourly consumption of the maximum day

= (2.7 × Annual Average hourly demand)

The formula given by GOODRICH is also used for finding out the rather of peak demandrates to their corresponding average values.

GOODRICH FORMULA $P=180\ t^{0.10}$

Where,P = % of annual average draft for the time

t = 1/24 to 365 days

DESIGN PERIOD

1. A water supply scheme includes huge and costly structures such as dams,reservoirs, pen stock pipes which cannot be replaced or modified easily. E.g.: distribution pipes which are laid underground and cannot be replaced or added easily without digging the roads or dissupting the traffic.
2. In order to avoid these future complaints of expansions, the various components of a water supply scheme are purposely made larger, so as tosatisfy the community needs for a reasonable number of years to come.
3. This future period or the number of years for which a provision is made in designing the capacities of the various components of the water supplyscheme is known as „Design Period‟.
4. The design period should neither be too long nor should it to be too short.
5. The design period cannot exceed the useful life of the componentsstructure and is guided by the following consideration,

FACTORS GOVERNING THE DESIGN PERIOD

- Useful life of component structre and the chances of theirbecoming old and absolute.
- Ease and difficulty that is likely tobe faced in expansion, ifundertaken at future date.
- Amount and availibility of additional investment likely to incurred for additional provisional.eg: if funds are not available has tokeep smaller desingperiod
- The rate of interest on the borrowing and the additionalmoney invested. eg: if the interested rate is small, higher the value of desingperiod is adopted
- Anticipated rate of population growth, including possible shift in communites, industries & commercial establishment. eg: if rateof increase of population is less, a higher figure forthe design peroid may be choosen.

Design period for different components of a water supplyscheme.

SL NO	ITEM	DESIGN PERIOD IN YEARS
1	Storage by dams	50
2	Infiltration work	30
3	Pumping 1. Pump house. 2. electric motors & pumps	30 15
4	Raw & clear water conveying units.	30
5	Distribution system.	30

POPULATION FORECASTING

When the design period is fixed, the next step is to determine population in various periods because the population of the towns generally goes on increasing. The population is increased bybirths, decreased by deaths, increased or decreased migration. The correct present and past population can be obtained from census office. The WSS are not designed for the present population the future population expected by the end of the design period may be estimated by various methods. The method to be adopted to a particular town or city depends on the factors discussed in these methods.

The various methods of forecasting the population are

1. Arithmetical increase method
2. Biometrical increase method
3. Incremental increase method
4. Decreasing rate of increase method or decreasing rate method
5. Simple graphical method
6. Comparative graphical method
7. Master plan method or Zoning method
8. Ration method or Apportionment method
9. Logistics curve method.

ARITHMETICAL INCREASE METHOD

- This method is based upon the assumption that the population is increasing at a constant rate,ie. The rate of change of population with time is constant.
- If the present population of a particular town is „P‟ and the average increase in population forpast decade „Ia‟ the future population „Pn‟ at the end of „n‟ decades will be
- Pn = P+ nIa
- This method gives low results for developing areas, which develop faster than the post thismethod of limited value may be useful for smaller design periods or for old and very large cities with no industries and which have practically reached their maximum development or approaching saturation

GEOMETRICAL INCREASE METHOD

- This method assures that the percentage increase in population from decade to decade is constant. This method gives high results for young cities expanding at faster rates and useful for old developed cities. If the present population of the city is „P‟ and the Average percentage increase/ decade „Ig‟ then the population „Pn‟ at the end of "n‟ future decades will be

$$Pn = P\left[1+\frac{Ig}{100}\right]^n$$

INCREMENTAL INCREASE METHOD

- This method is a combination of the above two methods and therefore gives the advantages of both arithmetic and Geometric increase methods and hence gives satisfactory results. In this method the Average increase is first determined by the arithmetical increase method and to this added the average of the net incremental increase once for every future decade.
- $p_n = p_o + nI_a + \frac{n(n+1)}{2} L_i$

PROBLEMS

1. Estimate the population by 2001 by Arithmetic and geometric progression method using the following census, which method is ideal and why?

YEAR	1951	1961	1971	1981
POPULATION	19800	42000	75000	110000

Soln:

YEAR	POPULATION	INCREASE PER DELADE	% INCREASE PER DECADE
1951	19800	22200	112.12
1961	42000	33000	78.57
1971	75000	35000	46.67
1981	110000		
		Ia = 30067	Ig = 79.116

I) By ARITHMETICAL INCREASE METHODThe population by 2001 is given by

$P_{2001} = P_{1981} + nIa$

n= 2 decades

Ia = 30067

$P_{2001} =$ 100000 + 2(30067)

P_{2001} = 170134

II) By GEOMETRICAL INCREASE METHODThe population by 2001 is given by

$$P_{2001} = P_{1981}\left[1+\frac{Ig}{100}\right]^n$$

110000 1

P_{2001} = 352908

In this case AIM is ideal because, GIM gives very high results.

2. The census record of a town is as follows

YEAR	1940	1950	1960	1970	1980
POPULATION	81420	125000	170000	220000	230000

Workout the population after three decades by AIM, GIM, and IIM

Solution:

YEAR	POPULATION	INCREASE/DECADE	%INCREASE/DECADE	INCREMENTAL INCREASE
1940	81420	43580	53.52	1420
1950	125000	45000	36	5000
1960	170000	50000	29.41	40000
1970	220000	10000	4.55	
1980	230000			
		Ia = 37145	Ig = 30.87	Ii = -11194

I) By ARITHMETICAL INCREASE METHOD Population after 3 decades i.e. by the year 2010 $P_{2010} = P_{1980} + nI_a$

$= 230000+3(37145)$

$P_{2010} = 341435$

II) By GEOMETRICAL INCREASE METHOD population after 3 decades

$$P_{2010} = P_{1980}\left[1+\frac{Ig}{100}\right]^n$$

$$230000\left[1+\frac{30.87}{100}\right]^3$$

P_{2010} =515523

III) By INCREMENTAL INCREASE METHODPopulation by 1990

P_{1990} = P_{1980} +Ia +Ii

= 230000+37145-11194

P_{1990} = 255951

P_{2000} = P_{1990} +Ia +Ii

= 255951+37145-11194

P_{2000} = 281902

P_{2010} = P_{2000} +Ia +Ii

= 281902+37145-11194

P_{2010} =307853

3) The census data of population of a town are as follows Estimate the population by the year2011 by AIM GIM and IIM. Which method is ideal and why?

YEAR	1961	1971	1981	1991
POPULATION	80	120	145	160

Solution:

YEAR	**POPULATION (in thousand)**	**INCREASE/DECADE**	**%INCREASE/DECADE**	**INCREMENTAL INCREASE**
1961	80	40	50	
1971	120	25	20.83	-15
1981	145	15	10.34	-10
1991	160			
		Ia = 26.667	Ig = 27.056	Ii = 12.5

I) By ARITHMETICAL INCREASE METHOD

P2011 = P1991 + nIa

= 160+2(26.667)

P_{2011} =213.334(In Thousands)

II) By GEOMETRICAL INCREASE METHOD

$$P2011 = P1991\left[1+\frac{Ig}{100}\right]^n$$

P_{2011} =258.29(In Thousands)

III) By INCREMENTAL INCREASE METHOD

P_{2001} = P_{1991} +Ia +Ii

= 160+26.667-12.5

P_{2001} =174.167(In Thousands)

P_{2011} = P_{2001} +Ia +Ii

= 174.167+26.667-12.5

P_{2011} =198.334 (In Thousands)

IIM is IDEAL because GIM and AIM have lorger values. IIM gives a constant value.

SURFACE SOURCE OF WATER SUPPLIES

Surface sources are those sources of water in the which the water the water flows surface of the earth, and is thus directly available for water supplies. Theimportant of those sources are;

- Natural ponds and lakes;
- Streams and Rivers;
- Important reservoirs.

Ponds and Lakes as Surface Sources of Supplies

- A natural large sized depression formed within the surface of the earth, when gets filled up with water, is known as a pond or lake. The difference between a pond and a lake is only that of size. If the size of the depression iscomparatively small , it may be turned as a pond, and when the size is largerit may be termed as a lake.
- Sometimes, the underground water through some springs, also enters natural depression and get collected there, forming ponds and lakes.
- The quickly of water in a lake is generally good and does not need much purification . Larger and other lakes, however, provide comparatively purer water than smaller and newer lakes. Self –purification of water due to sedimentation of suspended matter , bleaching of colour, removal of bacteria, etc. makes the lake water purer and better , On the other hand , in still waters of lakes and pond , the algae, weed and vegetable growth take place freely , imparting bad smells, tastes and colours to their warter.
- The quantity of water available from lake is, however, generally small. Itdepends upon the catchment area of the lake basin, annual rainfall, useful for the only small towns and hilly areas.

Streams and Rivers as Surface Sources of supplies

- Small streams are, therefore, generally not suitable for water supply schemes, because the quantity of water available in them is generally verysmall, and they may even sometimes go dry.
- They are, therefore useful as sources of water only for small villages, especialy in hilly regions. Larger and perennial streams may, however beused as source of water , by

providing storage reservoirs, barrages, etc. across them.

- Rivers are the most important sources of water for public water supplyscheme. It is a well-known fact that most of the cities are settled near therivers , and it is generally -easy to find a river for supplying water to the city. Rivers may be perennial or non-perennial.
- The quality of water obtained from rivers is generally not reliable as it contains large amount of silt, sand and a lot of suspended matter. The disposal of the untreated or treated sewage into the rivers is further liableto contaminate their waters. The river waters must, therefore, be properlyanalysed and well treated before supplying to the public.
- Storage Reservoirs as Surface Source of Supplies
- Pool or artificial lake formed on the upstream side of the dam is known asthe storage reservoirs. The quality of this reservoirs 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. Generally, multipurpose reservoirs are planned these days operated so as to get optimum benefits.

Underground Sources

The underground water is generally available in the following forms;

- Infiltration Galleries
- Infiltration wells
- Springs
- Wells including tube wells.

Infiltration Galleries

- Infiltration galleries are the horizontal or nearly horizontal tunnels constructedat shallow depths (3 to 5 meters) along the banks of the rivers through the water bearing strata, as shown in fig 4.8.They are sometimes called horizontal wells.
- These galleries are generally constructed of masonry walls with roof slabs, and extract water from the aquifer by various porous lateral drain pipes located at suitable intervals in the gallery.
- These pipes are generally covered with gravel, so as to prevent the entry of the fine sand particles into the pipes.
- These tunnels and galleries are generally laid at a slope, and the water collected in them taken is taken to a sump well, from where it is pumped, treated and distributed to the consumers.

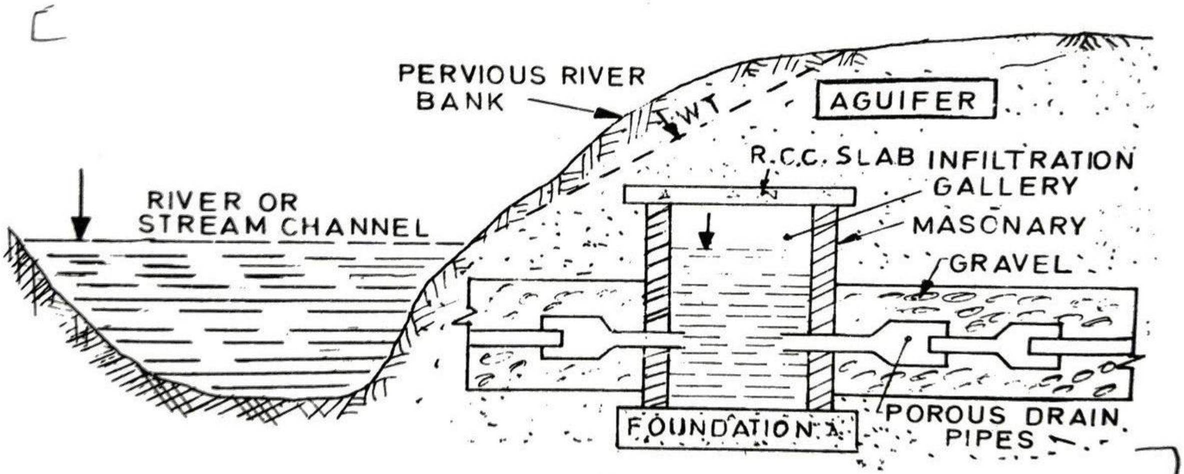


Fig. 4.8. Section of an Infiltration gallery.

Infiltration Wells

- Infiltration wells are the shallow wells constructed in series along the banks of ariver. In order to collect the river water seeping through their bottom as shown in fig 4.10.
- These wells are generally constructed of brick masonry with open joints. They are generally covered at top and kept open at bottom.

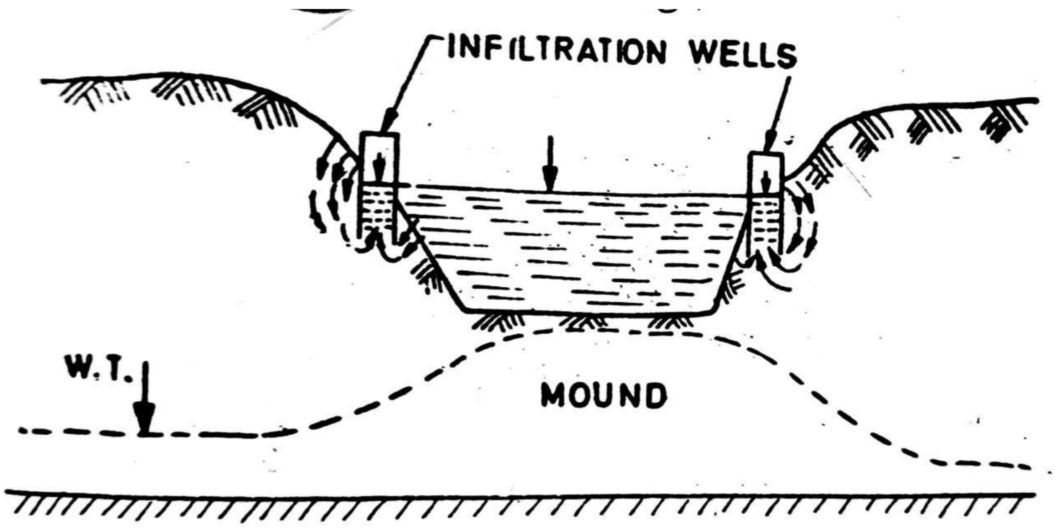


Fig. 4.10. Locations of Infiltration wells.

Springs

The natural outflow of ground water at the earth‟s surface is said to beform a spring. A pervious layer sandwiched between two impervious layers,give rise to a natural spring.

a) Gravity Springs

When the ground water table rises high and high water overflows throughthe sides of a natural valley or a depression (as shown in fig 4.13), the spring formed is known as the

Gravity spring.

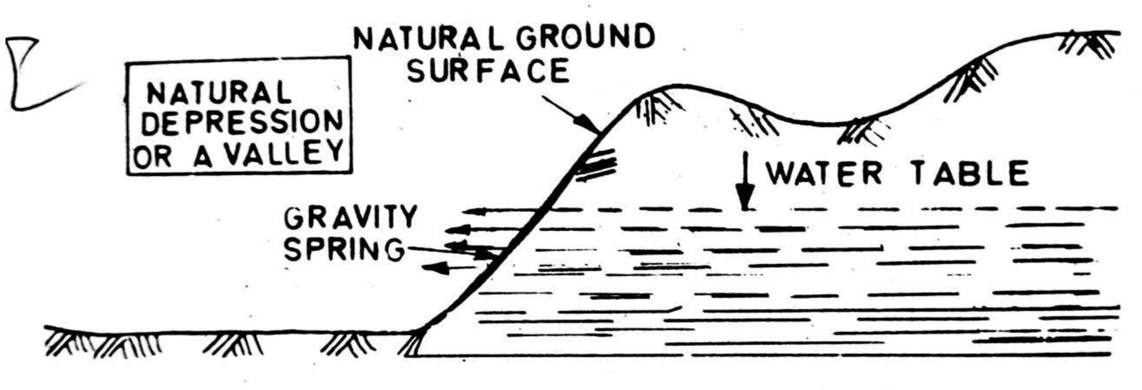


Fig. 4.13. Gravity spring.

B) Surface Springs

Sometimes, an impervious obstruction or stratum supporting the underground storage becomes inclined (such as shown in fig4.14).

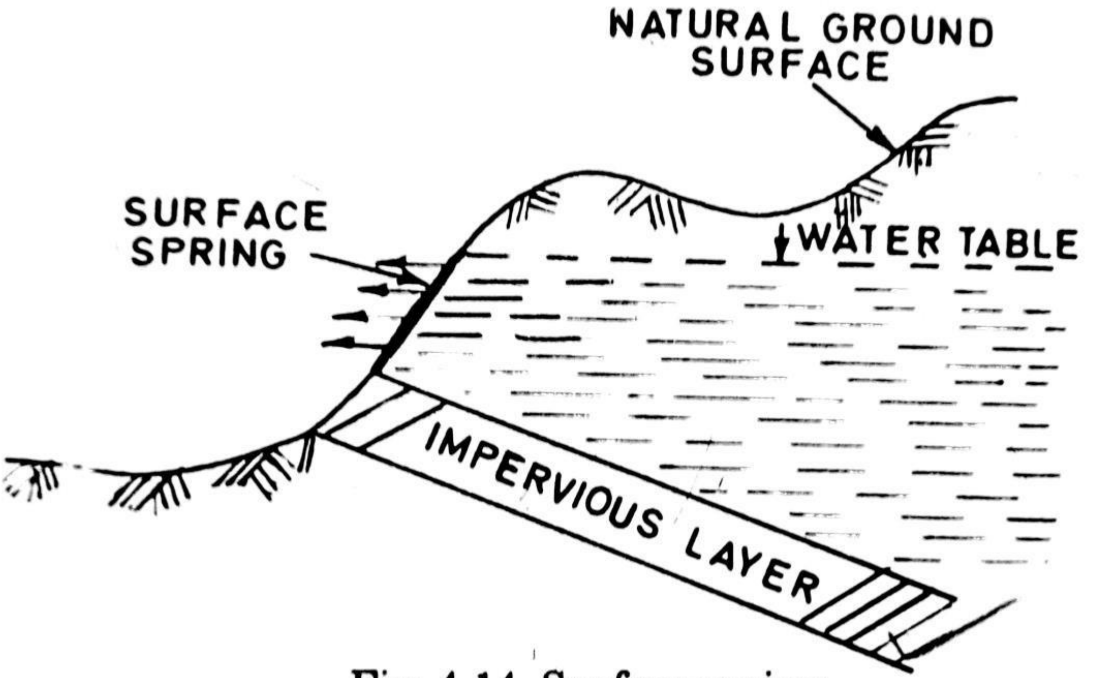


Fig. 4.14. Surface spring.

b) Artesian Springs

When the above storage is under pressure (ice. the water is flowing through some confined aquifer), such as shown in fig 4.15, the springformed is known as artesian spring.

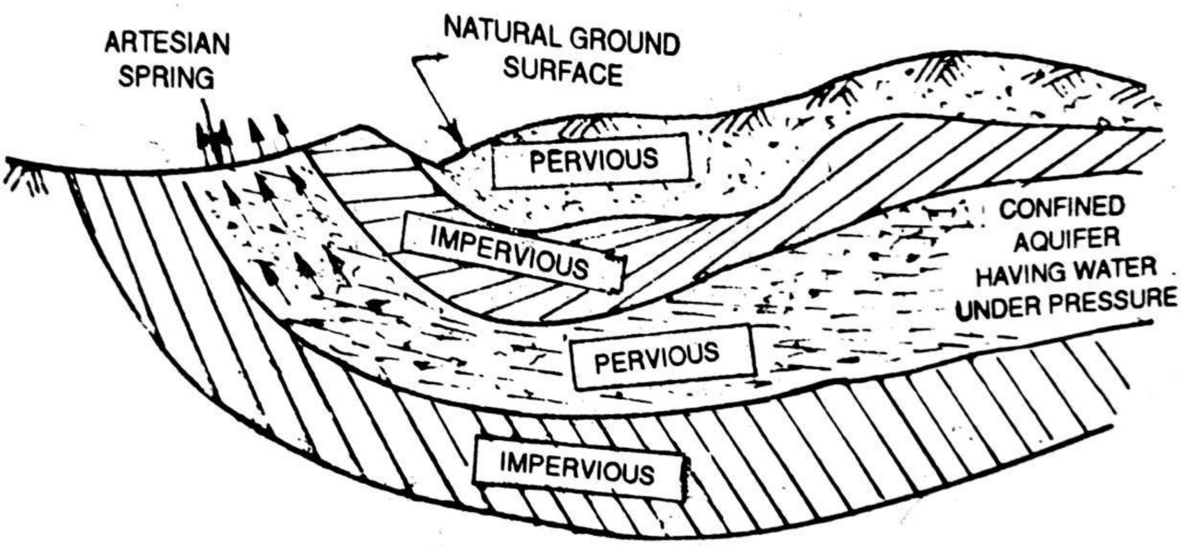


Fig. 4.15. Artesian spring.

WELLS

Water well is a hole usually vertical, excavated in the earth for bringingground water to the surface.

The wells may be classified into two types;

a) Open wells
b) Tube wells

Open wells or Drug wells

- Open wells are generally open masonry wells having comparatively bigger diameters, and are suitable for low discharges of the order of 18cubic meters per hours (i.e. about 0.005cumec).
- The diameters of the open well s generally vary from 2 to 9 m , and they are generally less than 20 m in depth. The walls of an open well may be built of precast ringer or in brick or stone masonry; the thickness generally varies from 0.5 to 0.75 m, according to the depth of the well fig4.16.

The open well may be classified into two types;

a) Shallow wells
b) Deep wells
  - A Shallow well is the one which rests in a previous strata and draw itssupplies from the surrounding materials.
  - A deep well is the one which rests on an impervious „mota" layer and draws its supply from the previous formation lying below the moat layer.

INTAKE STRUCTURES

Intakes or intake structure are masonry or construct slr whose function is toprovide calm and still water, free form flcatin matter for water supply.
Intake consists of the opening, stainess or grating through a which waterenters and the conduit for conveying the water, usually by gravity to a sumpwell. From the well the water pumped to the treatment plant.

Factors governing the location of an intake

The site for locating the intake should be selected carefully, keeping thefollowing point in mind.

- As far as possible, the site should be near the treatment plant so that the costof conveying water to the city is less.
- The intake must be located in the purer zone of the source so that the best possible quality of water is withdrawn from the source, thereby reducing the load on the treatment plant.
- The intake must river be loacated at the downstram or in the vinicinity of thethe point of disposal of waste water.
- The intake should never be loacted near the navigation channels, asotherwise, there are chances of intake water getting polluted due todischarge of refuse and waste from ships and boats
- The site should be such as to permit grates withdrawns of water, if requriedat a future date thus these should be sufficent scope for future additions andexpansions.
- The intake must be located at a place from where it can draw water evenduring the driest period the year.
- The intake site should remain easily accessible during floods and should not be concentrated in the vicinty of the intake.
- The mendring river, the intakes should not be located on caves at least onsharp curves.

Types of intake

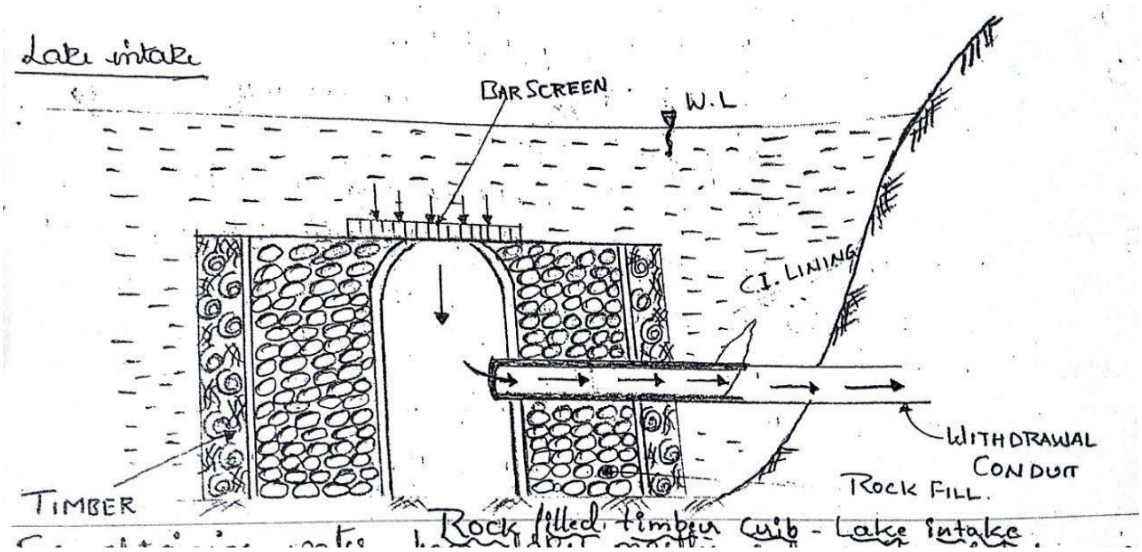
Depending on the source of water, the intake works are classified as follows,

i. Lake intake
ii. River intake
iii. Reservior intake
iv. Canal intake

Lake Intake

For obtaining water from lakes mostly submerseble intake are used

- These intake are constructed in the bed of the lake below the slow waterlevel so as to draw water in the dry seasons also.

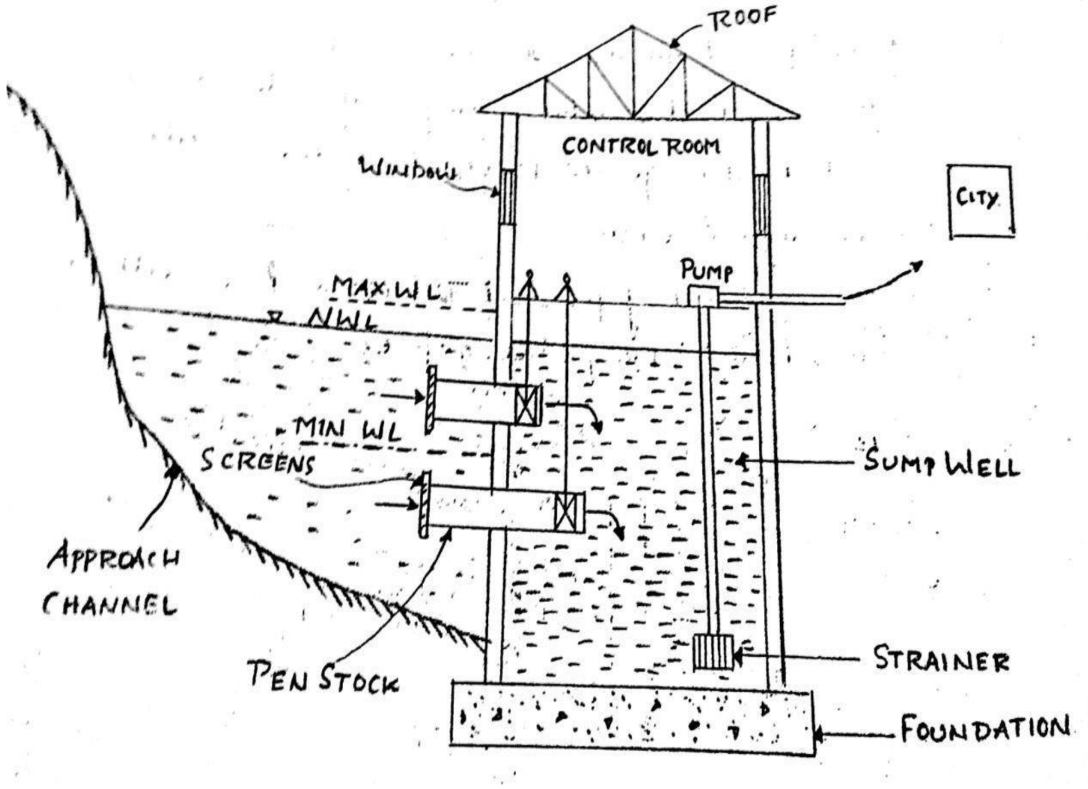


- In consists of a pipe laid in the bed of the river, on end of which is providedwith bell mouth opening with fire seruns.
- The water entris through, the bell mouth opening and flows water gravity.

<u>Advantages</u>

1. No changes from floating bodies
2. No trouble dur to ice
3. No obstruction to navigation

- **River intake**



It is circular masonry of tower of 4×7m in diameter constructed along thebank of river @ each place from where required

The water level in the lower position of the intake known as pump well,from penstake.

- The pensttakes are fitted with screens to check the entity of floating solidsand are placed on the downstream side to that water free from most of thesuspended solids may only enter the jack well
- Number of penstock opening are provided in the intake tower to admit water@ different river
- The opening and closing of penstock water is done with the help of whealsprovided @ the pump hence floor.

Reservior intake

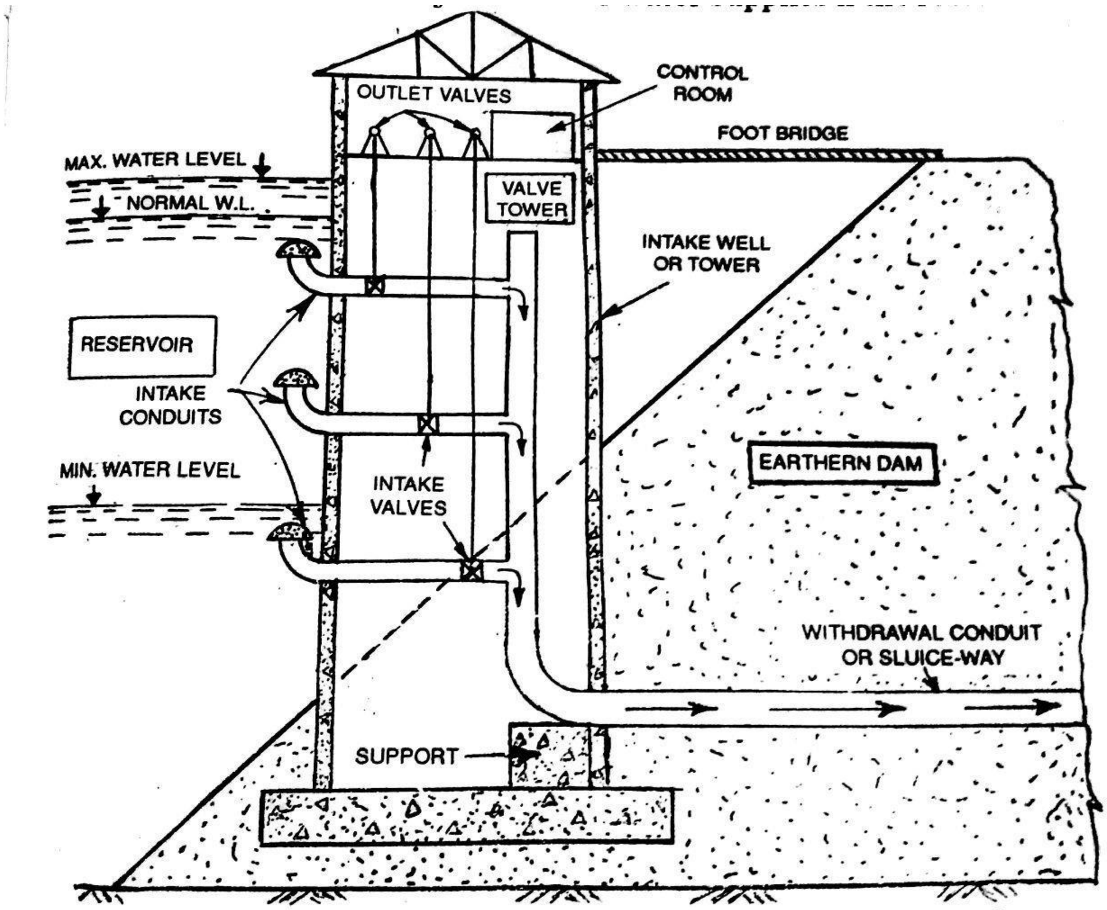
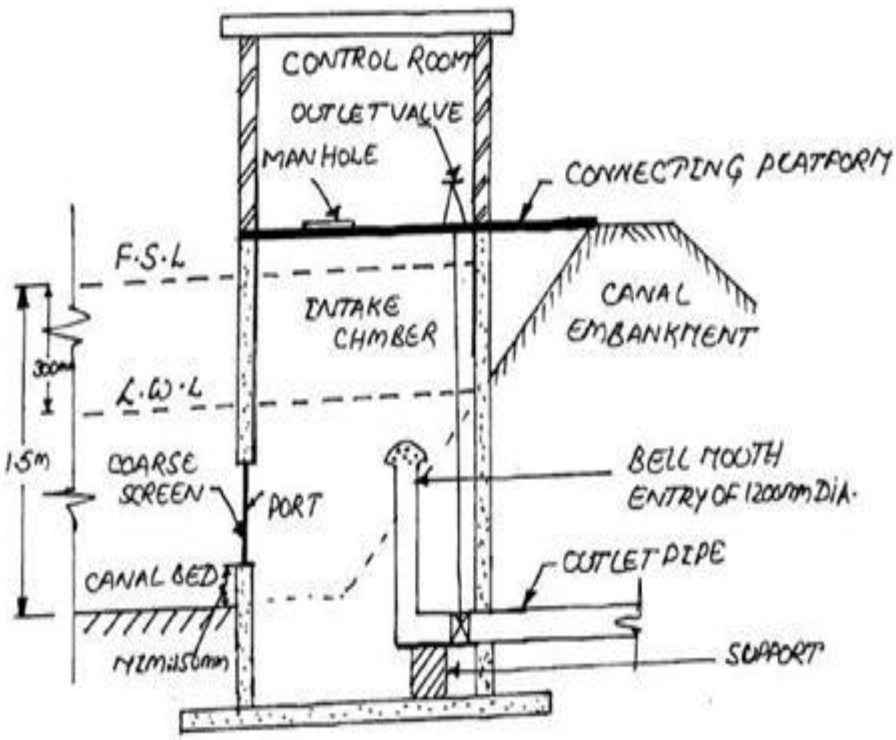


Fig. 5.11. Valve tower situated at the upstream toe of an earthen dam.

- It is mostly used to draw the water from carthen dam reservior. It consists ofan intake tower constructed on the slops of the dams @ such place from where intake can draw sufficent quantites of water even in the driest period.
- Intake pipes are fixed @ different levels, so as to draw water near thesurface in all variation of water level.
- These all inlet pipes are connected to one vertical pipe inside the intakewell.
- Screens are provided at the mouth of all intake pipes to prevent theconstruction of floating and suspended matter in them
- The water enters the vertical pipe is taken to the either side of the dam bymeans of an outlet pipe.
- It the tap of the intake tower slice values are provided to control

Canal intake**:**

- The entry of water in the intake chamber takes through coarse screen and the top of outlet pipe is provided with fine screen. The inlet to outlet pipe is of bell-mouth shape with perforations of the fine screen on its surface. The outlet valve is operated from the top and it controls the entry of water into the outlet pipe from where it is taken to the treatment plant.
- Fig shows the details of canal intake. A intake chamber is constructed in the canal section. This results in the reduction of water way which increases the velocity of flow. It therefore becomes necessary to provide pitching on the downstream and upstream portion of canal intake.



PUMPS

- The function of pump is to leave the water or any fluid to higher elevation or at higher pressure. Pumps are driven by electricity, diesiel or steam power.
- They are helpfulin pumping water from the sources that is from intake to the treatment plant and from treatment plant to the distribution system or service reservoir.
- In homes also pumps are used to pump water to upper floors or to store water in tanks over the buildings.

Types of pumps and their suitability

Based on the mechanical principle of water lifting pumps are classified as the following

Sl.No	Type of Pump	Examples	Suitability
1.	Displacement pumps	Reciprocating pumps. Rotary, chain, gear wheel, pump and wind mills.	This type of pumps are suitable for moderate heads and small discharges suitable for fire protection, water supply of individual houses.
2.	Velocity pumps	Centrifugal pumps, deep well, turbine pumps, jet pumps	This type of pumps are used widely in water supply schemes containing sand, silt etc.
3.	Boyancy pumps	Airlifting pumps	Airlifting pumps are generally adopted for pumping of water from deep wells to a lift of about 60m containing mud, silt, debries etc.
4.	Impulse pumps	Hydraulic Ram	Used for Small water supply projects to left the water for a height of about 30m or so.

Centrifugal pumps – components

- Centrifugal force is made use of in lifting water . Electrical energy is converted topotential or pressure energy of water
- Centrifugal pump consists of the following parts as shown in figCentrifugal Pump
- **Casing**: The impellor is enclosed in the casing, which is so designed that kinetic energyof the liquid is converted into pressure energy before it leaves the casing.
    - Delivery pipe
    - Delivery valve
    - Impeller
    - .Prime mover 6.Suction pipe
    - Strainer and foot valve

Description

- The pump consists of a Impeller is enclosed in a water tight casing. Water atlower level is sucked into the impellor through a suction pipe. Suction pipe should be air tight and bends in this pipe should be avoided.
- A strainer foot valve is connected at the bottom of the suction pipe to prevent entry of foreign matter and to hold water during pumping .Suction pipe is kept larger in diameter than delivery pipe to reduce cavitations and losses due to friction. An electric motor is coupled to the central shaft to impart energy.

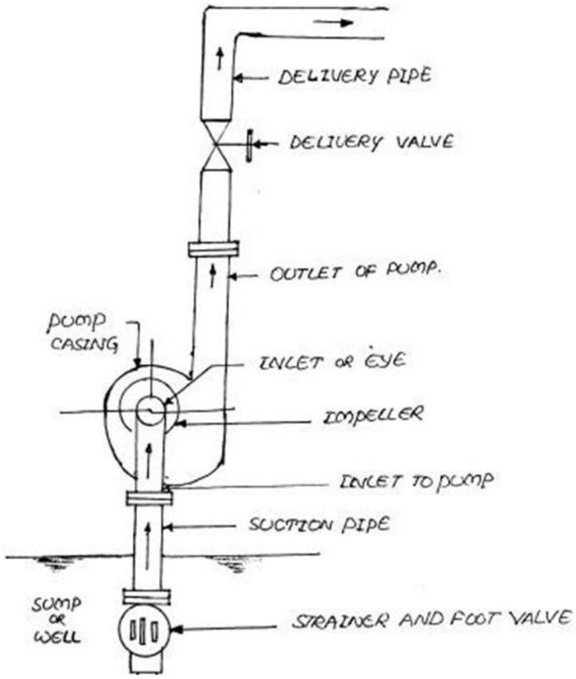
Working principle

- When the impellor starts rotating it creates reduction of pressure at the eye of the impellor, which sucks in water through the suction pipe. Water on entering the eye is caught between the vanes of the impeller.
- Rapid rotation of the impellor sets up acentrifugal force and forces the water at high velocity outwards against the causing convert the velocity energy into pressure energy which is utilized to overcome thedelivery head

Operation and maintenance

- Priming – Priming means filling up of the suction and casing completely with water. Pressure and suction developed by the impellor is proportional to the density of thefluid and the speed of rotation.
- Impellor running in air will produce only negligible negative pressure on the head. Hence it is required that is the casing and impellor is filled with water through a funnel and cock. Trapped air is released through pet cock. Initially the delivery valve is closed and the pump started.
- The rotation impellor pushes the water in the casing into the delivery pipe and the water in the casing into the delivery pipe and the resulting vacuum is filled by water raising through the suction pipe. The pass valve is opened while closing the bypass valve , while stopping the

pump delivery valve is closed first and the pump switched off.



- Maintenance may be 1) preventive maintenance 2) Break down maintenance. Preventive maintenance
  - Locates the sources of trouble and keep the equipment in good operating condition. It involves oiling, greasing of stuffing boxes, observing the temperature of the motor and the pump bearings, checking the valves, strainer, electrical contacts, earthings etc.

  Break down maintenance
  - Involves replacement of wornout components and testing . Sufficient amount of spares of impellors, bearings, slip-ring brushes, stater-contacts, gland packing, greases,oils, jointing materials, valves are to be kept instock to attend to the emergencies. It is usual to have one stand by pump in addition to the required number of pumps.
- Selection of pump horse power

  Basic data regarding the water availability like diameter, depth of the well, depth of the water table, seasonal variations of water table, drawdown duration of pumping and safe yield are to be collected accurately before selecting a pump. There are many varieties of specifications and choices available in the market and it is a tricky problem facing an engineer to select the best suited for his requirement.

Point to be observed in selecting a pump

1. Capacity and efficiency - The pump should have the capacity required and optimum efficiency.
2. Lift - Suction head from the water level to the pump level

3. Head – It is also called delivery head. Generally the total head (suction and deliveryhead)

should meet all possible situations with respect to the head.

4. Reliability – A reputed manufacture or similar make pump already in use may give thefailure rate and types of troubles.

5. Initial cost: The cost of the pump and its installation cost should be minimum.6.Power – Power requirements should be less for operation

7. Maintenance – Maintenance cost should be minimum. Availability of spares and cost ofspares are to be ascertained.

Horse-power of pump

The horse-power (H.P.) of a pump can be determined by calculated the work done by apump in raising the water upto H height.

Let the pump raise „W" kg of water to height „H" m

Then workdone by pump = W X H Kg m

= WQH mkg/sec

Where W → density of water in kg/m^3.Q → water discharge by pump in m^3/sec

Discharge x Total head The water horse power = ------------------------------------

75

W.H.P. = WHO/75

BHP= WHP/efficiency

BRAKE HORSE POWER of the pump is given byBHP(INPUT) + WQH/75n

HEAD POWER AND EFFICIENCY OF PUMPS

The total head against which a pump works is made up of

The suction Head (H_s)

The Delivery Head(H_d)

The Head loss due to friction entrance and exit in the rising main(H_f)

The suction HEAD is the difference in elevation between the low water level and center line of pump.

Delivery HEAD is the difference in elevation between the pump center line and point ofdischarge

Total HEAD (H) =H_s+H_d+H_f

The work done by the pump in lifting „Q" cumecs of water by a head(H) =WQH kg-m/sec.Where,

W = Specific weight of water, 1000 kg/m^3Q = discharge to be pumped, m^3/sec.

The water horse power of the pump is given byWHP(out put) = WQH/75

If „n" is the efficiency of the pump then

ECONOMICAL DIAMETER OF THE RISING (PUMPING) MAIN

- The economical diameter is a particular size of the pumping or rising main which while passing a given discharge of water gives the total annual expense to be minimum.
- If the diameter chosen is more than the economic dia, it will lead to higher cost of the pipe line on the other hand, if the dia of the pipe is less than the economical dia, the increased velocity will lead to higher friction headless and require more HP for the required pumping and the cost of pumping shall be much more than the resultant saving in the pipe cost.

LEA FORMULA

An empirical formula given by LEA Connecting the dia and discharge is given by

D = 0.97 to 1.22 $\sqrt{Q}$

Where

D = economical diaimeter

Q = Discharge to be pumped in „cusecs"

This relation gives optimum flow velocity varying between 0.8 to 1.35m/sec

FOR RIGOROUS ANALYSIS The total cost of pipe and pumping should be worked out at different assumed velocities (b/w 0.8 to 1.8m/sec) and a graph plotted between the annual cost and the size of the pipe. The economical size is one which gives the least annual cost.

DRINKING WATER STANDARDS

S.No.	CHARACTERICTICS	NORMALLY ACCEPTABLE VALUE	MAX. PERMISIBLE LIMIT
1.	Temperature	10°C – 15°C	-
2.	Turbidity (N.T.U)	2.5	10
3.	Colour (platinum cobalt scale)	5.0	25
4.	Taste and odour	Unobjectionable	
5.	pH	7.0-8.5	6.5-9.2
6.	Total dissolved solids(mg/litre)	500	1500
7.	Total hardness mg/l (as caco3)	200	600
8.	Chlorides (as Cl) mg/l	200	1000
9.	Sulphates (as So4) mg/l	200	400

10.	Nitrates (as No3) mg/l	45	45
11.	Fluorides (as F) mg/l	1.0	1.5
12.	Calcium (as Ca) mg/l	75	200
13.	Magnesium (as mg) mg/l	30-120	150
14.	Iron (as Fe) mg/l	0.1	1.0
15.	Manganese (As Mn) mg/l	0.05	0.5
16.	Phenolic compounds (as phenol) mg/l	0.001	0.002
17.	Arsenic (as mg) mg/l	0.05	0.05
18.	Chromium (as cr^{+6}) mg/l	0.05	0.05
19.	Cynamides (as CN) mg/l	0.05	0.05
20.	Coliform count per 100ml of water sample	Zero	-

MODULE 2

Water Treatment & Distribution

Introduction

- Water available in various sources contains various types of impurities and cannotbe directly used by the public for various purposes, before removing the impurities. For potability water should be free from unpleasant tastes, odours and must have sparkling appearance.
- The water must be free from disease-spreading germs. The amount and type of treatment process will depend on the quality of raw water and the standards of quality of raw water and the standards of quality to be required.
- The surface sources generally contains large amount of impurities therefore they requires sedimentation, filtration and chlorination as treatment. If the water containsalgae or other microorganisms, pre chlorination has to be done tastes and odours , dissolved gases like CO2, H2S are removed by aeration.
- During the flood season, the turbidity of the surface water may be high and flocculation may become necessary to remove turbidity.

Treatment unit flow diagram

- Water treatment includes many operations like Aeration, Flocculation, Sedimentation, Filtration, Softening, Chlorination and demineralization. Depending upon the quality of raw water and the quality of water desired. Several combinations of the above processes may be adopted as shown in the flow diagram.
- One complete water treatment plant requires the following process starting from thesource of water upto the distribution zone in order of sequence

Sl.No.	Name of the unit	Purpose
1.	Intake work including pumping plant	Raw water from the source for treatment
2.	Plain sedimentation	To remove suspended impurities such as silt, clay, sand etc.
3.	Sedimentation with coagualtion	To remove the suspended matter
4.	Filtration	To remove microorgans and colloidal matter
5.	Water softening plant	To remove hardness of water
6.	Miscellaneous treatment plants	To remove dissolved gases, tastes andodours.
7.	Disinfection	To remove pathogenic bacteria
8.	Clear water reservoir	To store the treated water
9.	Pumps for pumping the water in service	If town or city is situated at higher

	reservoirs	elevation then pumping is required.
10.	Elevated or underground service reservoir	For distribution of treated water.

General treatment processes adopted are,

- Screening: to remove all the floating matter from surface water, generally provided at intake
- Aeration: to remove objectionable taste and odour and also to remove the dissolved gases such as CO_2, H_2S etc. Iron and Manganese also removes some extent.
- Sedimentation with or without coagulation: to remove suspended impurities. By plain sedimentation silt, sand can be removed whereas sedimentation with coagulation removes very fine suspended particles and some bacteria can also remove.
- Filtration: to remove very fine suspended impurities and colloidal impurities that may have escaped the sedimentation tank. In addition to this, micro organisms present in water are largely removed.
- Disinfection: carried out to reduce to a safe minimum limit, the remaining micro organisms, and to prevent the contamination of water during transit from the treatment plant to the place of its consumption
- Miscellaneous processes: includes water softening, desalination, removal of iron, manganese and other harmful constituents.

Screens: Device which removes undesirable suspended and floating matters which may floating debris such as sticks, branches, leaves etc. Screens may be of 2 types

- Coarse screen
- Fine screen

AERATION:

- Process of bringing the water in contact with excess of air or oxygen. It is one of important unit operation of gas transfer.
- The aim of the aeration is to create extensive, new, and self-renewing interfaces between air and water, to keep interfacial films from building up in thickness.
- Objectives
    - It removes taste and odour caused by gases due to organic decomposition
    - It increase the DO content of the water
    - It remove H_2S and hence odour due to this is also removed
    - It decreases CO_2 and thereby reduces corrosiveness and raises its pH
    - It converts iron and manganese from their soluble states to their insoluble states, so that these can be precipitated and removed
    - Due to agitation of the water during aeration, bacteria may be killed to some extent

- It is also used for mixing chemicals with water, as in the 'Aeromix process' and in the use of diffused compressed air

Types of aerators

Aeration is done by the following main types of aerators:

1. Free fall aerators or gravity aerators
   a. Cascade aerators
   b. Inclined apron aerators
2. Stat tray aerators
3. Spray aerators
4. Air diffuser basins.

1. Free fall aerators

a. Cascade aerators

- Simplest of free fall aerator, weir and waterfalls of any kind are cascade aerators.
- A simple cascade consists of a series of 3 to 4 steps of concrete or metal.
- Water is allowed to fall through a height of 1 to 3 m thereby it comes to close contact with air
- Cascade can be either in open air, or may be in room which has plenty of louvered air inlet
- CO_2 reduction usually 50 to 60%

b. Inclined apron aerator with riffle plates

- In this type water is allowed to fall along an inclined plain or apron which is usually studded with riffle plates in herring bone fashion
- The breaking up of sheet of water will cause agitation & consequent aeration

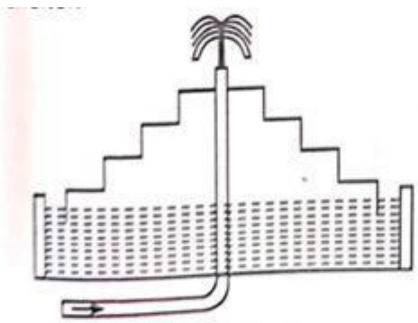


Fig. 6.5 : Circular type

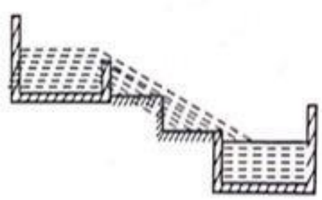


Fig. 6.6 : Straight steps

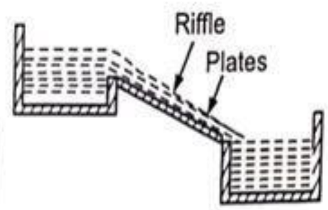
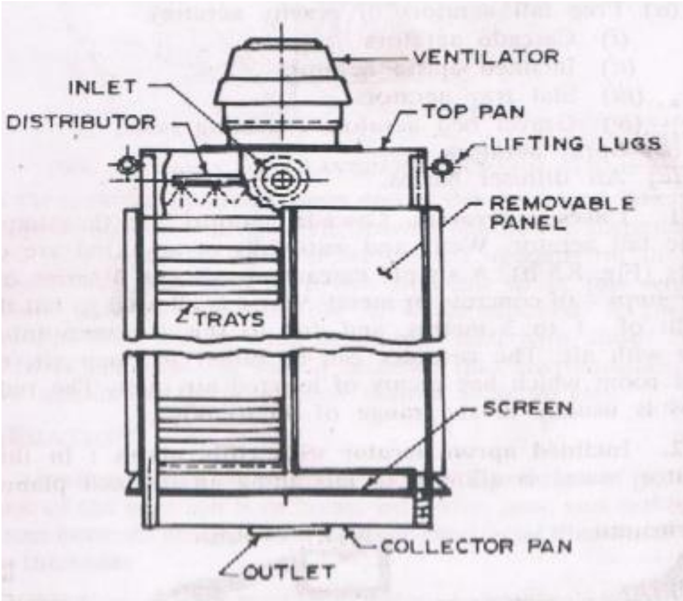


Fig. 6.7 : Inclined apron aerator

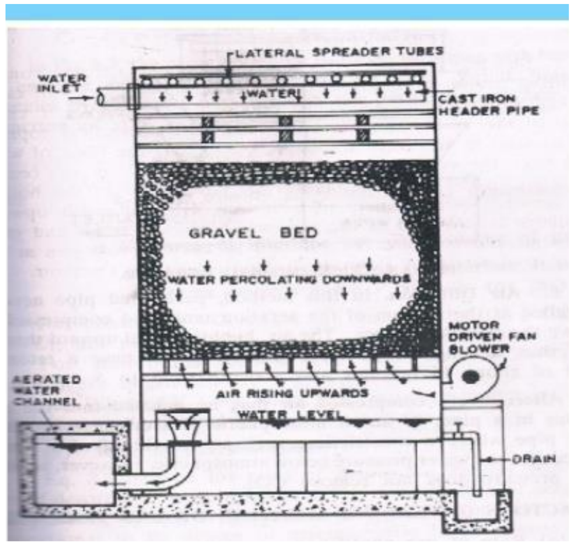
2. Slat tray aerators

- Most commonly used, consists of a closed round or square structure containing a series of closely stacked superimposed wood-slat trays.

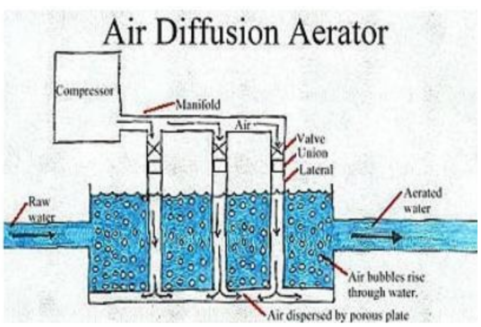
- Water enters the top of the aerator & is evenly distributed over top most trays. The slats in the trays are staggered so that the films of water raining over the edges of tray falls on the centers of the slats in the tray just below.
- Air is supplied to the bottom of the aerator with the help of a blower, which blows it upward.
- Ventilator which provide at the top discharges air & gases to the atmosphere.
- Water is collected in the collector pan at the bottom, then to reservoir

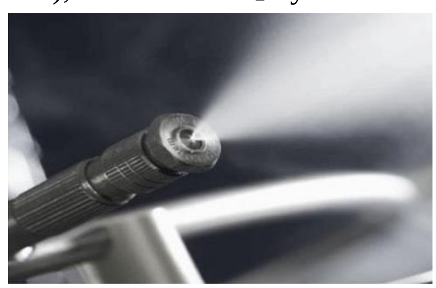


3. <u>Gravel bed aerator (Trickling beds)</u>
   - Cascading through beds of coke, lime stone or anthracite is believed to remove CO_2 more efficiently than other methods
   - Water is applied at the top and trickles down while air is blow upwards.
   - Thickness of gravel bed 1 to 1.5 m.
   - Other form i.e., trickling beds contains 3 to 4 trays filled with coke, slag or stone are used
   - Bed thickness 0.5 to 0.6 m, vertical distance b/n bed is 0.5 m.
   - Water is applied from top through a perforated distribution pipe, during tricking process aeration takes place.



4. Spray aerators
   - Divides water flow into fine streams and small droplets which come into intimate contact with air in their trajectory.
   - Water is sprinkled in fine jets through nozzles.
   - Requires considerable head (0.75 to 1.5 kg/ cm^2), it reduces CO_2 by 70 – 90%





5. Air diffusion
   - In this system perforated pipe network is installed at the bottom of the aeration tank & compressed air is blown through these pipes. Air bubbles travels upward through water causing aeration
   - Air diffuser basin retention period is about 15 min, depth 3 to 5 m.
   - Alternatively, compressed air may be injected into the flow of water in the pipe
   -

Limitations: of Aeration

- Not an efficient method of removal or reduction of taste and odour caused by relatively non-volatile substances such as oil of algae
- Odour removal is only 50% when symura was causative organism

- Taste & odours caused by chemicals due to industrial wastes discharges into receiving stream are not satisfactorily removed
- Fe & Mn can be precipitated by aeration only when organic matter is not present
- Possibility of air born contamination in water is there
- Additional lime may be required to neutralize CO_2 that would be removed by aeration
- Economical only in warmer month

SEDIMENTATION:

- **Sedimentation** is the removal of suspended particles by gravitational settling. Sedimentation tanks are designed to reduce the velocity of flow of water so as to permit suspended solids to settle out of the water by gravity.
- **Plain sedimentation**: when the impurities are separated from suspended fluid by the action of natural force alone, i.e., by the gravitational force and natural aggregation of the settling particles, the operation is called plain sedimentation
- **Sedimentation with coagulation (clarification):** when chemicals or other substances are added to induce or haste aggregation and settling of finely divided suspended matter, colloidal substances and large molecules, the operation is called sedimentation with coagulation or simply clarification
- **Chemical precipitation**: When chemicals are added to throw dissolved impurities out of the solution, the operation is called chemical precipitation
- **Discrete particles**: A particle that does not alter its shape, size and weight while settling or rising in water is known as discrete particle

TYPES OF SEDIMENTATION TANK:

1. Depending upon the method of operation
   a. Quiescent or Fill and draw type
   b. The continuous flow type

a) *Fill and draw type:*
   - Sedimentation tank is first filled with incoming water, and is allowed to rest for a certain time.
   - During the rest period suspended particles settles down at the bottom of the tank, at the end of the period, the clear water is drawn off through the outlet valve.
   - The tank is then cleaned of settled particles and filled again
   - Detention time: 24 hrs., cleaning time 6 -12 hrs., hence cycles of operation takes 30 – 36 hr.
   - Minimum 3 units are required to maintain constant supply

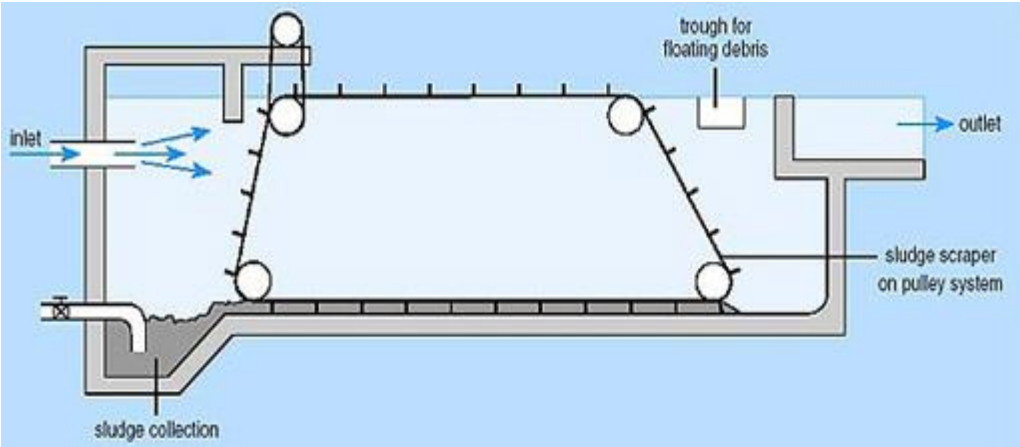
b) *Continuous flow type tank*

- Water is continuously keeps on moving in tank, through with a very with a very small velocity during which time the suspended particles settle at the bottom before they reach the outlet.
- Two types are
  - ✓ Horizontal flow tank
    - Generally rectangular in plan having L= 2W.
    - Water flows in horizontal direction, with a maximum permissible velocity 0.3 m/s
  - ✓ Vertical flow tank
    - Generally deep circular or rectangular basin with hopper bottom

2. Depending upon method of operation
   a. Rectangular tank with horizontal flow
   b. Circular tank with radial or spiral flow
   c. Hopper bottom tank with vertical flow

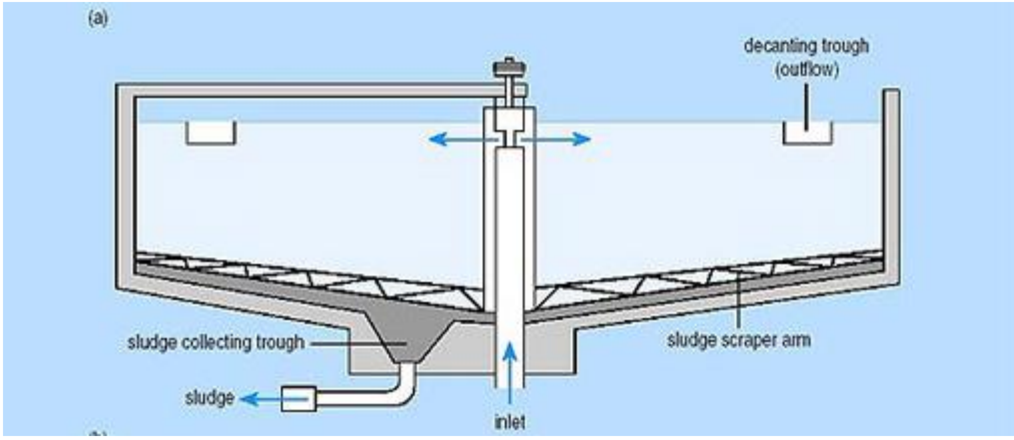
a) Rectangular tank with horizontal flow

- Baffles are provided to prevent short circuiting
- Figure sows rectangular tank, without baffles, but with a sludge hopper, & a sloping floor.
- Has high settling efficiency & provided with sludge removal equipment
- Sludge, scrapped by sludge scrapers & collected in hopper & removed through sludge withdrawal pipe

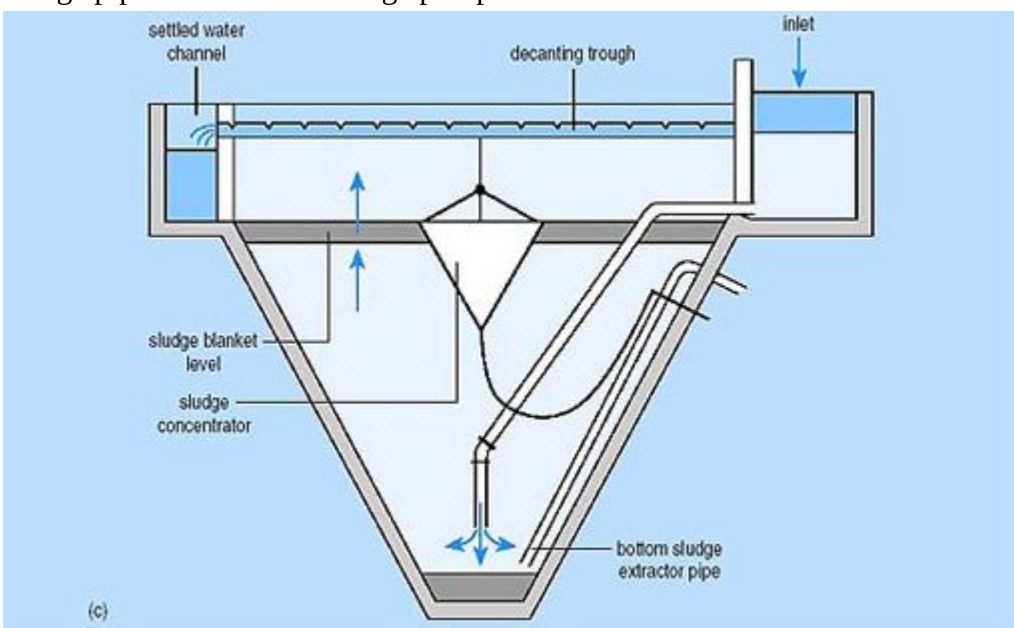


b) Circular tank with radial or spiral flow

- May have circular or spiral flow
- Influent enters through a central pipe & rises up to baffle ox or influent well & flows radialy (horizontally) towards the outlet provided circumferences
- Racking arms move slowly to scrap the sludge which is removed through sludge pipe connected to sludge pump



c) *Hopper bottom tank with vertical flow*

- Water enters through centrally placed inlet pipe & deflected downwards by deflector box
- Water travels vertically downwards & sludge settles at bottom of hopper removed by sludge pipe connected to sludge pump
- 

Design elements

In continuous flow following are important elements of design

1. Detention period & displacement efficiency
   - Detention time is the theoretical time taken by a particle of water to pass between entry and exit of settling tank. If V is the volume of the basin, Q is the discharge rate and t_o is the detention period

$t_o = V/Q = LBH/Q$

- D.T depends upon type of suspended impurities in water, vary from 4 to 8 hr.
- Flowing through period(t_o): it is average time required for a batch of water to pass through the settling tank
- It is always less than D.T due to short circuit effects
- Can be determine by placing NaCl in the influent & testing for Cl content
- Displacement Efficiency(η_d): it is ratio of flowing through period to detention period
- η_d= flowing through period/Detention period = t_d/t_o
- Varies from 0.25 to 0.5, well defined tank should provide flow through period at least 30% of detention period

2. Overflow rate and surface loading:
   - It is defined as quantity of water passing per hour (or per day) per unit horizontal area
   - Velocity of settlement (v_s) is also equal to rate of flow (Q) divided by the horizontal area (A) of the tank

     v_s = Q/A = surface loading,
3. Basin dimensions

- Surface area(A)=Volume of water in liters hr./ surface loading rate in liters/hr./m^2

SEDIMENTATION WITH COAGULATION- CLARIFICATION

- The efficiency of plain sedimentation is generally very low. Especially when water contains very fine suspended matter and colloidal matter.
- The chemicals added for this purpose converts the impurities which may be present in solution in colloidal suspension or in a finely divided form into particles which are of such a size that they may be readily removed b settlement.
- The coagulants neutralize the negative protective charge on the colloidal particles and allow them to coagulate.
- Ti is required to remove
  - Miscellaneous fragments of animals & vegetables, bacteria & virus
  - Plankton, finely divided mineral matter including clay
  - Organic colouring matter, complex mixture of colloidal & dissolved organic compounds derived from sewage & industrial effluent
- Factors affecting coagulation
  - Types and dose of coagulant
  - Time violence & method of mixing
  - Characteristics of water
    - Type & quantity of suspended matter
    - Temperature of water
    - pH of ware

CLARIFLOCCULATOR (COAGULATION SEDIMENTATION TANK)

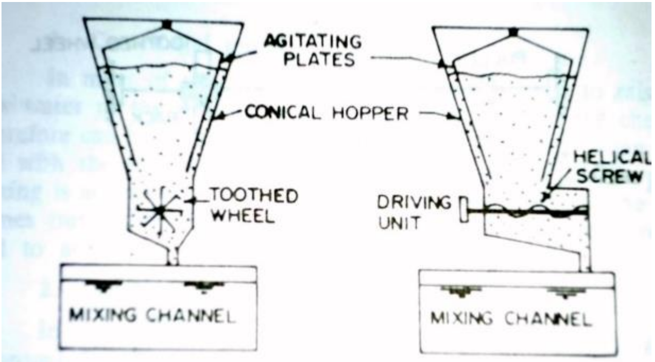
Coagulation sedimentation plant sometimes called as **Clariflocculator,** contains following 4 units

(1) Feeding device
(2) Mixing device or mixing basin
(3) Flocculation tank or flocculator
(4) Settling or sedimentation tank

- This process helps in removing turbidities up to as low as10 – 20 mg/l.
- helps in reducing bacteria from water & thus to reduce the B- Coli index about 70%

(1) Feeding device

- Chemical coagulant may fed into raw water either in a powder form(Dry feeding) or in a solution form (Wet feeding)
- *Dry feeding*
  - Common device use are shown in figure
  - They are in form of tank with hopper bottom with agitating plates inside the tank
  - Powder form of coagulant is filled in tank & allowed to fall in mixing basin
  - Dosage is regulated by speed of toothed wheel in turn controlled by connecting to a venture device installed in raw water pipes bring water to mixing basin
  - Thus quantity of coagulant released is controlled in proportion to quantity of raw water entering mixing tank



(a) BY TOOTHED WHEEL (b) BY HELICAL SCREW

- *Wet feeding:*
  - Solution of requires strength of coagulant is prepared & stored in tank & allowed to trickle down into mixing tank through outlet
  - Level of coagulant in feeding tank is maintained constant by means of float controlled valve, in order to ensure constant rate of discharge
  - In order to make discharge & coagulant flow in proportion ‘ a conical plug type arrangement’ as shown in figure are made

- In conical plug type arrangement,
  - ✓ Mixing basin & float chamber are interconnected together to maintain same water level
  - ✓ As the water level increases in mixing tank, correspondingly the water level in the float chamber increases, thereby float of the float chamber
  - ✓ As float rises, the *pinion & pulley* rotates in same direction, thereby lifting the conical plug & allowing more quantity of coagulant solution fall down into mixing basin
  - ✓ Also at the lower water flow rate, *pinion & pulley* arrangements automatically controls the dose of the coagulant as above.

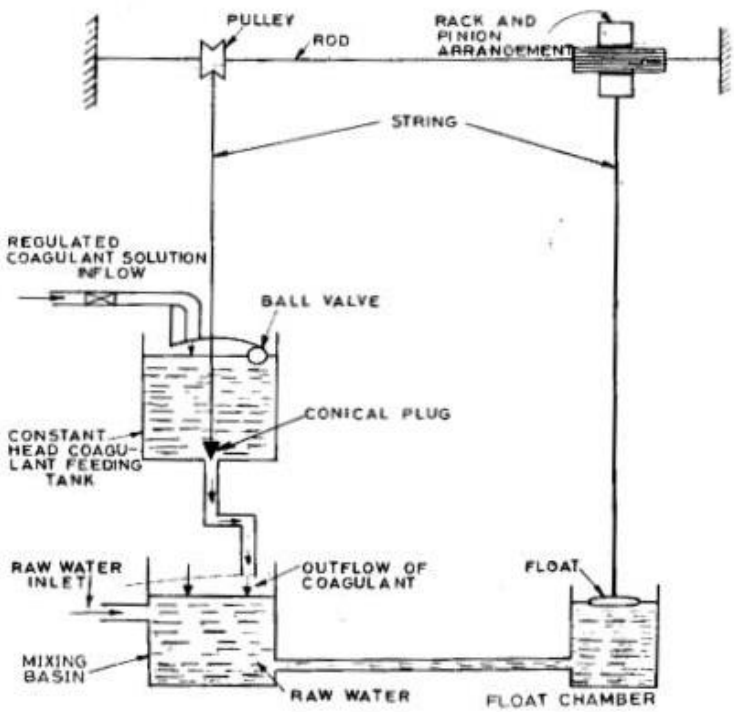
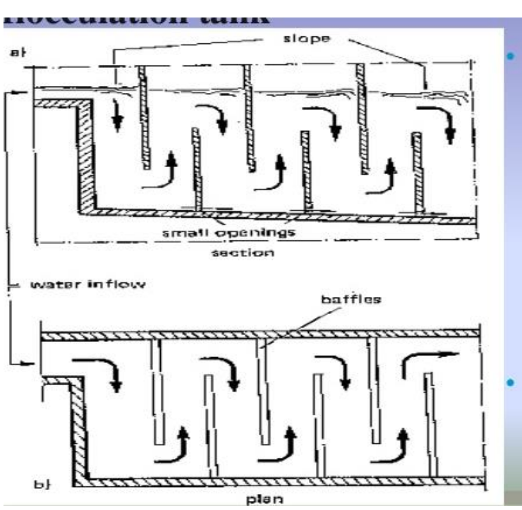
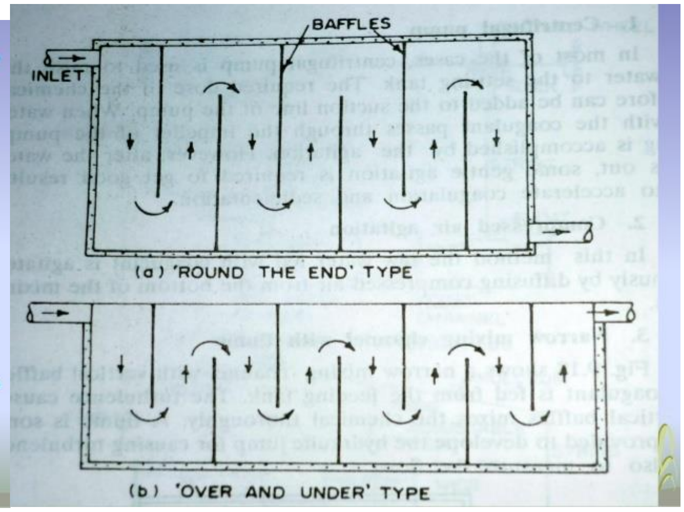


Fig. 9.19. Wet feeding by Conical plug and float arrangement.

(2) <u>Mixing device or mixing basin</u>

- After addition of coagulant, the mixture is thoroughly & vigorously mixed by mixing devices (such as mixing basins, centrifugal pumps, compressed air etc.,) to disperse the entire mass of coagulant into water
- Important mixing device is mixing basins which is of 2 types
  - *Mixing basins with baffle walls:* Rectangular tank divided by baffle walls provided in such a way as to allow *horizontal* water flow around their ends (Figure) or to make vertical water flow *over & under* baffles (Fig: 9.21)





Mixing basins equipped with mechanical devices: chemical added to raw water is vigorously mixed & agitated by a flash mixer (mechanically agitated mixing basin) for rapid dispersion of raw water, then transferred to flocculation tank provided with slow mixer

▪

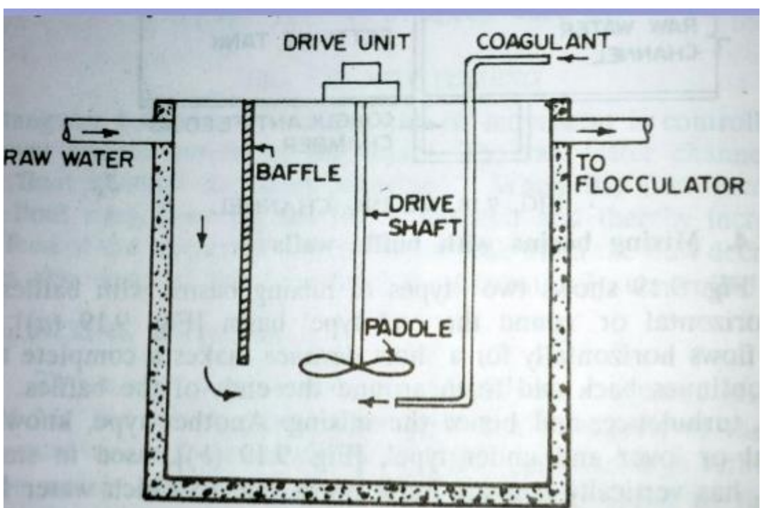


FIG. 9.20. FLASH MIXER

As shown in figure flash mixer consist of a rectangular tank provided with an impeller fixed to an impeller shaft

- ✓ Impeller is driven by an electric motor & revolves at high speed inside tank
- ✓ Coagulant is brought by coagulant pipe & discharged under rotating fan.
- ✓ Raw water is separately brought from inlet end, & deflected towards moving impeller by deflecting wall
- ✓ Thoroughly mixed water is taken out from outlet end.
- ✓ Drain valve is provided to remove the sludge from bottom of flash mixer

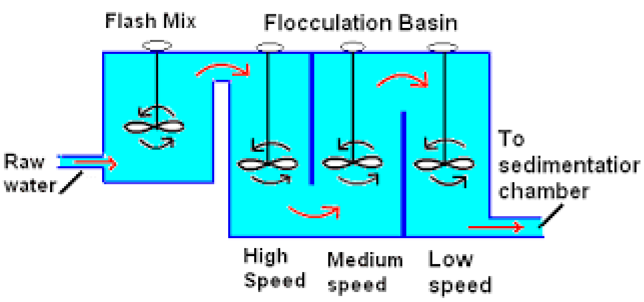
- ✓ Impeller speed generally 100 to 120 r.p.m, detention time 0.5 to 2 min
- ✓ For rapid mixing G'T = 30000 to 60000

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

- G' = velocity gradient in 1/s
- P= power dissipated in watts N m/s
- V = volume of raw water
- μ = absolute or dynamic viscosity of raw water in Ns/m^2

(3) Flocculation tank or flocculator

- From mixing basin, water is taken to a flocculation tank called flocculator, where it is given a slow stirring motion
- Rectangular tank is fitted with paddles operated by electric motors can serve best this purpose
- Paddle speed 2 -3 rpm, detention time 20 - 60 min (30 min), $G^1 = 20 – 80\ s^{-1}$
- Distance between paddles & wall is about 15 – 30 cm



(4) Settling or sedimentation tank

- Rectangular plain sedimentation tank with detention period 2–4 hr., surface loading varies between 1000–1250 liters/hr./m^2 of plan area is generally permitted for the settlement of the floc containing impurities

Common coagulants:

When coagulant is dissolved in water and thoroughly mixed in it a thick gelatinous precipitate, known as floc is formed. The Aluminum and ferric ions of the floc contain positive charge. Hence they attract the negatively charged colloidal particles of clay, turbidity and colour, thus helping in the removal of these impurities from water.

Following are common coagulants

1. Aluminum sulphate or Alum
2. Chlorinated copperas
3. Ferrous sulphate and lime
4. Magnesium carbonate

5. Polyelectrolyte
6. Sodium aluminates

1. Aluminium sulphate or alum: Chemical formula $Al_2(SO_4)_3.18H_2O$,

- optimum pH about 6.5 – 8.3, dosage varies from 5 to 85 mg/l, normal dose is about 17mg/l
- When added to water reacts with its bicarbonate alkalinities, to form gelatinous insoluble (floc) of aluminium hydroxides, which attracts other fine particles and suspended matter(colloids), thus grows in size and finally settles down to the bottom of the tank.
- $Al_2(SO_4)_3.18H_2O + 3Ca(HCO_3)_2 \rightarrow 3CaSO_4 + 2Al(OH)_3 + 6CO_2$
- If raw supplies contains not sufficient alkalinity, externals alkalies like lime [$Ca(OH)_2$] or soda ash [Na_2CO_3] are generally added
- $Al_2(SO_4)_3.18H_2O + 3Ca(OH)_2 \rightarrow 3CaSO_4 + Al(OH)_3 + 18H_2O$
- $Al_2(SO_4)_3.18H_2O + 3Na_2CO_3 \rightarrow 3Na_2SO_4 + Al(OH)_3 + 3CO_2 + 18H_2O$
- Also reduces taste and colour in addition to turbidity
- Drawback is sludge removal

2. Copperas

Ferrous sulphate reacts with natural alkalinity but the response is much slower than that between alum and natural alkalinity. Lime is generally added to raise the p^H to the point where ferrous lons are precipitated as ferric hydroxide by the caustic alkalinity

$$FeSo_4.7H_2O + Ca(HCu_3)_2 \rightarrow Fe(HCo_3)_2 + CaSo_4 + 7H_2O$$

$$Fe(HCo_3)_2 + 2Ca(OH)_2 \rightarrow Fe(OH)_2 + 2CaCo_3 + 2H_2O$$

$$4Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3$$

This treatment adds some hardness but no corrosiveness but dosing operation with two chemicals is more difficult.

3. ***Chlorinated copperas: $Fe_2(SO_4) + Fe_2Cl_3$***
   - When chlorine added to copperas(i.e., ferrous sulphate), chemically form Ferric sulphate and Ferric chloride known as chlorinated copperas, which is valuable coagulant for colour removal
   - Chlorinated copperas along with lime are effective coagulant
   - $Fe_2(SO_4) + 3Ca(OH_2) \rightarrow 3CaSO_4 + 2Fe(OH)_3$
   - $Fe_2Cl_3 + 3Ca(OH_2) \rightarrow 3CaCl_2 + 2Fe(OH)_3$
   - The resulting Ferric hydroxide forms floc and helps in sedimentation
   - Effective pH range for ferrous sulphate 4 to 7 & >9,
     For ferrous chloride 3.5 to 6.5 & >8.5
   - Very effective coagulant for treating low pH water
4. ***Sodum aluminates- $Na_2Al_2O_4$***
   - When dissolved & mixed with water, reacts with Ca & Mg salts present in raw water, results in Calcium & Aluminium aluminates precipitate
   - $Na_2Al_2O_4 + Ca(HCO_3)_2 \rightarrow Cl_2Al_2O_4 + Na_2CO_3 + CO_2 + H_2O$
   - $Na_2Al_2O_4 + CaCl_2 \rightarrow Ca_2Al_2O_4 + 2 NaCl$
   - $Na_2Al_2O_4 + CaSO_4 \rightarrow Ca_2Al_2O_4 + Na_2SO_4$
   - 1.5 times costlier than alum
   - Suitable for water which do not have natural desired alkalinity which cannot be treated with alum
   - Used for boiler feed waters, which permit very low value of hardness.

- The following are the mostly used Coagulants with normal dose and pH values required for best floc formation as shown in Table
-

Sl.No.	Coagulant	pH Range	Dosage mg/l
1.	Aluminium sulphate $Al_2(SO_4)_3$, 18 H_2O	5.5 – 8.0	5 – 85
2.	Sodium Aluminate, $Na_2Al_2O_4$	5.5 – 8.0	3.4 – 34
3.	Ferric Chloride ($Fecl_3$)	5.5 – 11.0	8.5 – 51
4.	Ferric Sulphate $Fe_2 (SO_4)_3$	5.5 – 11.0	8.5 – 51
5.	Ferric Sulphate $FeSO_47H_2O$	5.5 – 11.0	8.5 - 51

FILTRATION

The process of passing the water through beds of sand or other granular materials is known as filtration. For removing bacteria, colour, taste, odours and producing clear and sparkling water, filters are used by sand filtration 95 to 98% suspended impurities are removed.

Theory of filtration

The following are the mechanisms of filtration

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

Filtration is carries out in three types of filters

Gravity filters

1. Slow sand filter
2. Rapid sand filter
3. Pressure filter

Filter Materials

- **Sand**: Sand, either fine or coarse, is generally used as filter media. The size of the sand is measured and expressed by the term called effective size. The effective size, i.e. D10 maybe defined as the size of the sieve in mm through which ten percent of the sample of sand by weight will pass.
- The uniformity in size or degree of variations in sizes of particles is measured and expressed by the term called uniformity coefficient. The uniformity coefficient, i.e. (D60/D10) may be defined as the ratio of the sieve size in mm through which 60 percent of the sample of sand will pass, to the effective size of the sand.
- **Gravel:** The layers of sand may be supported on gravel, which permits the filtered water to move freely to the under drains, and allows the wash water to move uniformlyupwards.
- **Other materials**: Instead of using sand, sometimes, anthrafilt is used as filter media. Anthrafilt is made from anthracite, which is a type of coal-stone that burns without smoke or flames. It is cheaper and has been able to give a high rate of filtration.

Slow sand filter

- Slow sand filters are best suited for the filtration of water for small towns. Thesand used for the filtration is specified by the effective size and uniformity coefficient
- .The effective size, D10, which is the sieve in millimeters that permits 10% sand byweight to pass. The uniformity coefficient is calculated by the ratio of D60 and D10.

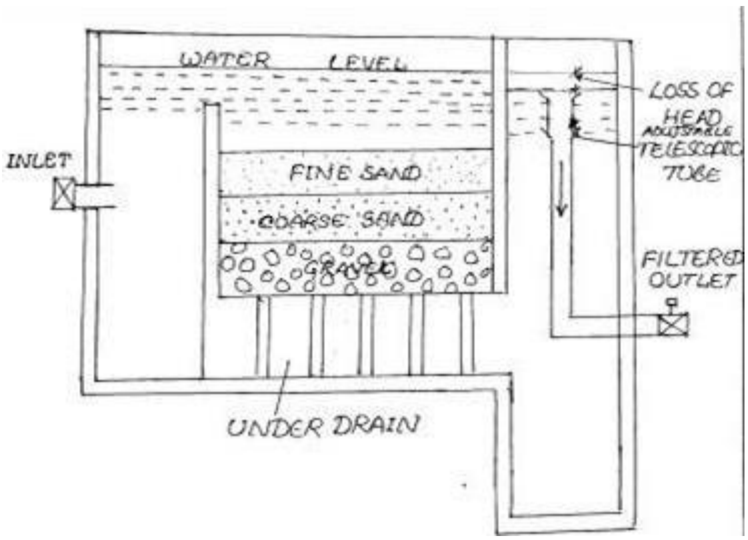
Principles of Slow Sand Filtration

- In a slow sand filter impurities in the water are removed by a combination of processes: sedimentation, straining, adsorption, and chemical and bacteriological action.
- During the first few days, water is purified mainly by mechanical and physical- chemical processes. The resulting accumulation of sediment and organic matter forms a thin layer on the sand surface, which remains permeable and retains particles even smaller than the spaces between the sand grains.

- As this layer (referred to as "Schmutzdecke") develops, it becomes living quartersof vast numbers of micro-organisms which break down organic material retained from the water, converting it into water, carbon dioxide and other oxides.
- Most impurities, including bacteria and viruses, are removed from the raw water as it passes through the filter skin and the layer of filter bed sand just below. The purification mechanisms extend from the filter skin to approx. 0.3-0.4 m belowthe surface of the filter bed, gradually decreasing in activity at lower levels as the water becomes purified and contains less organic material.
- When the micro-organisms become well established, the filter will work efficiently and produce high quality effluent which is virtually free of disease carrying organisms and biodegradable organic matter.
- They are suitable for treating waters with low colors, low turbidities and low bacterial contents.

Construction

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



Operation

- The water from sedimentation tanks enters the slow sand filter through a submersible inlet as shown in fig 5.3 This water is uniformily spread over a sand bed without causing any disturbances. The water passes through the filter media at an averagerate of 2.4 to 3.6 m3/m2/day.
- This rate of filtration is continued until the difference between the water level on the filter and in the inlet chamber is slightly less than the depth of water above the sand. The difference of water above the sand bed and in the outlet chamber is called the loss of head.

- During filtration as the filter media gets clogged due to the impurities, which stay in the pores, the resistance to the passage of water and loss of head also increases. When the loss of head reaches 60cm, filtration is stopped and about 2 to 3 cms from the top of bed is scrapped and replaced with clean sand before putting back into service to the filter.

- The scrapped sand is washed with the water, dried and stored for return to the filter at the time of the next washing . The filter can run for 6 to 8 weeks before it becomes necessaryto replace the sand layer.

Uses

- The slow sand filters are effective in removal of 98 to 99% of bacteria of raw water and completely all suspended impurities and turbidity is reduced to 1 N.T.U.
- Slow sand filters also removes odours, tastes and colours from the water but not pathogenic bacteria which requires disinfection to safeguard against water-borne diseases.
- The slow sand filters requires large area for their construction and high initial cost for establishment. The rate of filtration is also very slow.

Maintenance

- The algae growth on the overflow weir should be stopped. Rate of filtration should be maintained constant and free from fluctuation. Filter head indicator should bein good working condition. Trees around the plant should be controlled to avoid bird droppings on the filter bed, No coagulant should be used before slow sand filtration since the floc will clog the bed quickly.

Rapid sand filter

- Rapid sand filter are replacing the slow sand filters because of high rate offiltration ranging from 100 to 150m3/m2/day and small area of filter required. The main features of rapid sand filter are as follows.

Operation

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

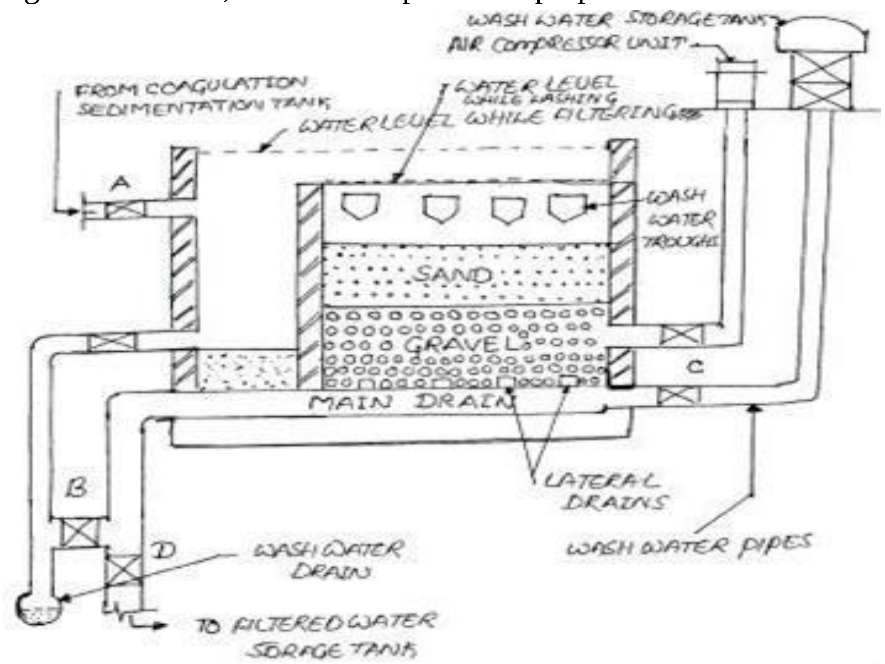
Depth of sand	-	60 to 75cm
Filter gravel	-	2 to 50mm size
		(Increase size towards bottom)
Depth of gravel	-	45cmDepth of water over sand
during filtration	-	1 to 2mOverall depth of filter
including 0.5m free board	-	2.6m
Area of single filter unit	-	$100m^2$ in two parts of each $50m^2$Loss of head -
	Max 1.8 to 2.0m	

Turbidity of filtered water - 1 NTU

Effective size of sand - 0.45 to 0.70mm Uniformity coefficient of sand - 1.3 to 1

Washing of filter

- Washing of filter done by the back flow of water through the sand bed as shownin Fig First the value „A" is closed and the water is drained out from the filter leaving a few centimeter depth of water on the top of sand bed.
- Keeping all values closed the compressed air is passed through the separate pipe system for 2-3 minutes, which agitatesthe sand bed and stirrer it well causing the loosening of dirt, clay etc. inside the sand bed. Now value „C" and „B" are opened gradually, the wash water tank, rises through the laterals, the strainers gravel and sand bed.
- Due to back flow of water the sand expandsand all the impurities are carried away with the wash water to the drains through the channels, which are kept for this purpose.



Construction

Rapid sand filter consists of the following five parts

1. Enclosure tank – A water tight tank is constructed either masony or concrete
2. Under drainage system – may be perforated pipe system or pipe and stracher system
3. Base material – gravel should free from clay, dust, silt and vegetable matter. Should bedurable, hard, round and strong and depth 40cm.
4. Filter media of sand – The depth of sand 60 to 75cm
5. Appurtenances – Air compressors useful for washing of filter and wash water troughs for collection of dirty water after washing of filter.

- Washing process is continued till the sand bed appears clearly. The washing of filter is done generally after 24 hours and it takes 10 minutes and during back washing the sand bed expands by about 50%.

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

Backwashing of Rapid Sand Filter

- For a filter to operate efficiently, it must be cleaned before the next filter run. If the water applied to a filter is of very good quality, the filter runs can be verylong. Some filters can operate longer than one week before needing to be backwashed. However, this is not recommended as long filter runs can cause the filter media to pack down so that it is difficult to expand the bed during the backwash
- Treated water from storage is used for the backwash cycle. This treated water is generally taken from elevated storage tanks or pumped in from the clear well.
- The filter backwash rate has to be great enough to expand and agitate the filter media and suspend the floc in the water for removal. However, if the filterbackwash rate is too high, media will be washed from the filter into the troughs and out of the filter.
- The filter should be backwashed when the following conditions have been met:
- The head loss is so high that the filter no longer produces water at the desired rate;and/or Floc starts to break through the filter and the turbidity in the filter effluent increases; and/or A filter run reaches a given hour of operation.

Comparison of slow sand filter and rapid sand filter

Sl.No.	ITEM	S.S.F	R.S.F
1.	Area	Need very large area	Needs small area
2.	Raw Water Turbidity	Not more than 30 NTU	Not more than 10NTU hence needs coagulation
3.	Sand Media	Effective size 0.2 to 0.3 mm uniformity coefficient2 to 3 single layer of uniform size	Effective size 0.45 to 0.7 mm uniformity coefficient 1.3 to 1.7 multiple graded layers of sand.
4.	Rate of Filtration	2.4 to 3.6m^3/m^2/day	100-150 m^3/m^2/day
5.	Loss of Head	0.6m to 0.7 m	1.8m to 2.0m
6.	Supervision	No skilled supervision is required	Skilled supervision is required

7.	Cleaning of Filter	Scraping of 21/2cm thick layer washing and replacing. Cleaning interval that is replacementof sand at 1 to 2 months.	Back wash with clean water under pressure to detach thedirt on the sand. Backwashing daily or on alternate days.
8.	Efficiency	Bacterial removal, taste, odour, colour and turbidity removal.	There is no removal of bacteria. Removal colour taste, odour and turbidity is good.

Operational Troubles in Rapid Gravity Filters

Air Binding

- When the filter is newly commissioned, the loss of head of water percolating through the filter is generally very small. However, the loss of head goes on increasing as more and more impurities get trapped into it.
- A stage is finally reached when the frictional resistance offered by the filter mediaexceeds the static head of water above the and bed. Most of this resistance is offered by the top 10 to 15 cm sand layer. The bottom sand acts like a vacuum, and water is sucked through the filter media rather than getting filtered through it.
- The negative pressure so developed, tends to release the dissolved air and other gases present in water. The formation of bubbles takes place which stick to the sand grains. This phenomenon is known as Air Binding as the air binds the filter and stops its functioning.
- To avoid such troubles, the filters are cleaned as soon as the head loss exceeds theoptimum allowable value.

Formation of Mud Balls:

- The mud from the atmosphere usually accumulates on the sand surface to form a dense mat. During inadequate washing this mud may sink down into the sand bed and stick to the sand grains and other arrested impurities, thereby forming mud balls.

Cracking of Filters:

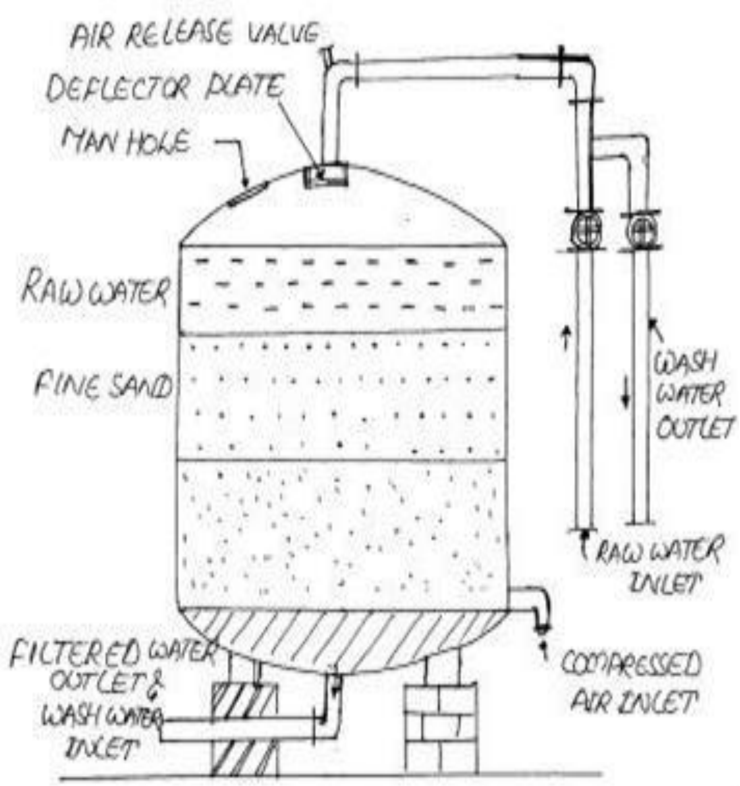
- The fine sand contained in the top layers of the filter bed shrinks and causes the development of shrinkage cracks in the sand bed. With the use of filter, the loss of head and, therefore, pressure on the sand bed goes on increasing, which further goes on widening these cracks.
- Remedial Measures to Prevent Cracking of Filters and Formation of Mud BallsBreaking the top fine mud layer with rakes and washing off the particles.
- Washing the filter with a solution of caustic soda. Removing, cleaning and replacing the damaged filter san

Pressure filter

- Pressure filter is type of rapid sand filter in a closed water tight cylinder through which the water passes through the sand bed under pressure. All the operations of the filter is similar to rapid gravity filter, expect that the coagulated water is directly applied to the filter without mixing and flocculation. These filters are used for industrial plantsbut these are not economical on large scale.
- Pressure filters may be vertical pressure filter and horizontal pressure filter. The Fig 5.5 shows vertical pressure filter. Backwash is carried by reversing the flow with values. The rate of flow is 120 to 300m3/m2/day.

Advantages

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



6. No sedimentation and coagulant tanks are required with these units.

Disadvantages

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

4. Change of filter media, gravel and repair of drainage system is difficult.

SOFTEHING

HARDNESS: Water is said to be 'hard' when it contains relatively large amounts of bicarbonates, carbonates, sulphates and chlorides of calcium and magnesium in it.

Troubles of hard water are,

1. Causes more consumption of soap in laundry work. A wastage of 25ppm of soap takes place per one ppm of hardness of water removed by it.
2. Leads to modification of some colours, and thus affects working of the dyeing system
3. Causes difficulties in manufacturing process (paper making, canning, rayon industry)
4. Causes choking and clogging troubles of house plumbing, formation of scales on boilers, other water heating system
5. Makes food tasteless, tough or rubbery.

Table: Classification of Hardness

Sl no	***Classification***	***Total hardness in mg/l as $CaCO_3$***
1	Soft	50
2	Moderately hard	50-150
3	Hard	150-300
4	Very hard	300

SOFTENING: Process of removing hardness causing salts from the water. In municipal water hardness is seldom reduced below 50 ppm and the carbonate or non-carbonate harness is generally not brought below 35 ppm

TYPES OF HARDNESS

Hardness is of two types:

i. Temporary hardness : which is deposited when water is boiled and is usually known as *carbonate hardness*
ii. Permanent hardness : produced by the sulphates, chliorides, and nitrates of calcium & magenesium, also known as *non-carbonate hardness*

Causing temporary hardness	*Causing permanent hardenss*

1. Calcium bicarbonate $Ca(HCO_3)_2$ 2. Magnesium bicarbonate $Mg(HCO_3)_2$	1. Calcium sulphate CaSO4 2. Magnesium sulphate $MgSO_4$ 3. Calcium chloride $CaCl_2$ 4. Magnesium chloride $MgCl_2$

(a) Removal of temporary hardness
   i. By boiling
   ii. By adding lim
(b) Removal of permanent hardness
   i. Lime – Soda process
   ii. Zeolite process
   iii. Demineralization or De – ionization process

LIME SODA PROCESS :

- In this process, lime [Ca(OH2)] and soda ash [Na2CO3] are added to hard water to react with Calcium and Magnesium salts to form insoluble precipitates of Calcium carbonate [$CaCO_3$] and Magnesium hydroxide[$Mg(OH)_2$] which can sediment out in sedimentation tank.
- The reactions are as follows,

$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3$ [1]

$Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow Ca(HCO_3)_2 + Mg(OH)_2$ [2]

$MgCO_3 + Ca(OH)_2 \rightarrow Mg(OH)_2 + CaCO_3$ [3]

$MgCl_2 + Ca(OH)_2 \rightarrow Mg(OH)_2 + CaCl_2$ [4]

$MgSO_4 + Ca(OH)_2 \rightarrow Mg(OH)_2 + CaSO_4$ [5]

$CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O$ [6]

$CaCl_2 + Na_2CO_3 \rightarrow CaCO_3 + NaCl$ [7]

$CaSO_4 + Na_2CO_3 \rightarrow CaCO_3 + Na_2SO_4$ [8]

- From the above reaction, it is clear that lime helps in removing entire carbonate hardness (carbonates and bicarbonates of calcium and magnesium) { [1] to [3] } and it reacts with non carbonate hardness of Mg { [4] & [5] }to convert the same into non carbonate hardness of Ca.
- Lime also reacts with carbon dioxide to form insoluble $CaCO_3$ which can remove.
- The non carbonate hardness of Calcium finally reacts with soda{ [7], [8] } to form insoluble precipitate $CaCO_3$ which can remove easily

Advantages

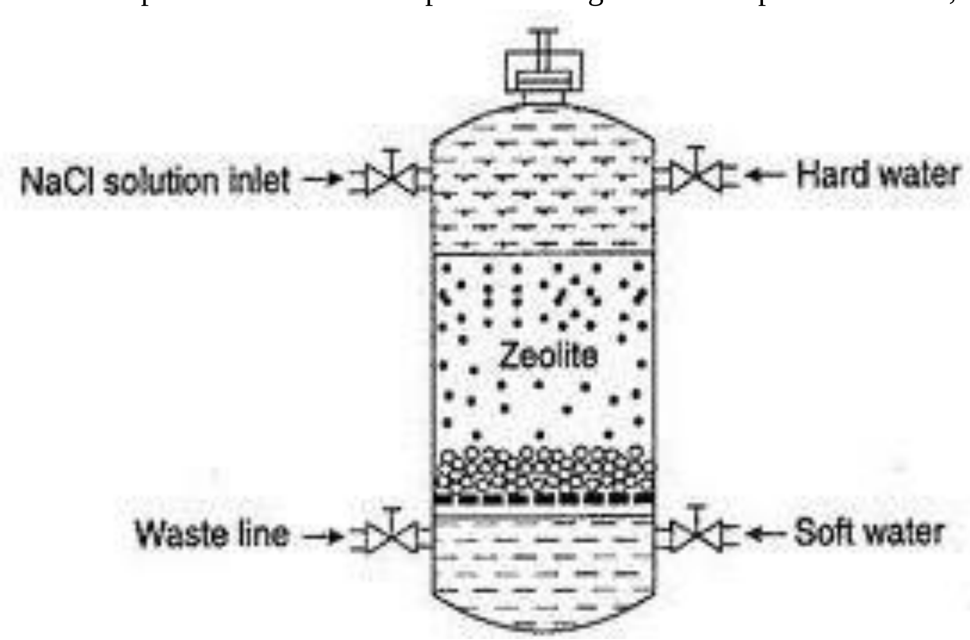
- Suitable for, less turbid, lower total solids and chalybeate free and acid free water
- Can reduce total mineral content, has bacterial effect
- Also removes iron & manganese and reduces total mineral content
- Increases pH & results in decrease in corrosion of distribution pipe

Disadvantages

- Large quantity of sludge formation, problem of handling and disposal of it
- Requires skilled supervision
- Recarbontaion is required to avoid incrustation of pipes
- Not possible to produce zero hardness.

ZEOLITE PROCESS (BASE EXCHANGE OR ION EXCHANGE PROCESS):

- In this process hard water is passed through a bed of special material, loosely called the Zeolite, which



  has the property of removing calcium and magnesium from the water and substituting sodium in their place by ion-exchange process phenomenon.
- Zeolite are complex compound of aluminium, silica and soda, some form of which are synthetic and others are naturally occurring.
  a. Natural zeolite - Glauconite (Green sand) – grren colour – exchange value 6500 to 9000g hardness/m^3 of zeolite
  b. Artificial zeolite - Permutit , $SiO_2Al_2O_3Na_2O$– white colour – exchange value 35000 to 40000 g hardness/m^3 of zeolite
- When hard water is passed through a bed of permutit, the following reactions takes places

Advantages

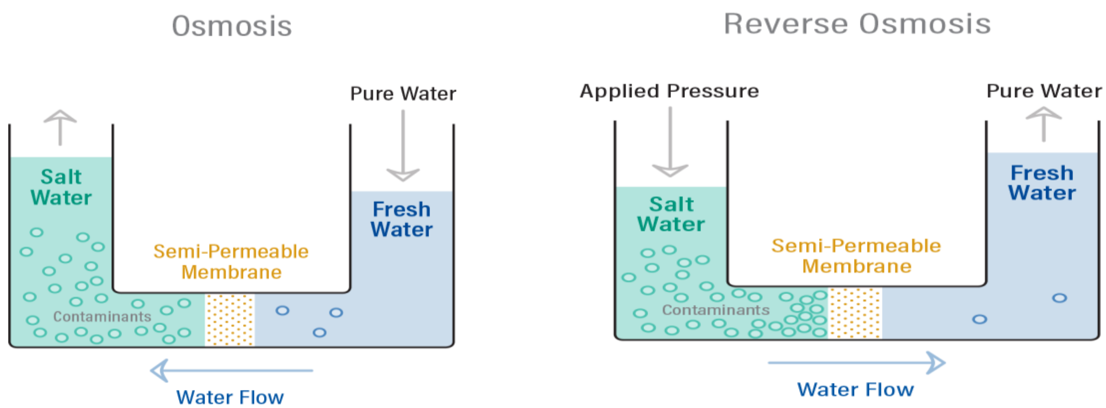
- Zeolite unit is compact, less space requirement
- Easy operation, no requirement of skilled labour
- Up to zero hardness can be achieved according to the requirement
- Almost automatic, less operating cost
- No incrustation of pipes, chemicals are cheap & easy to handle

Disadvantage

- Unsuitable for highly turbid water, water containing iron & manganese, acidic water
- careful operation to avoid zeolite injury, to equipment, to water quality
- likelihood of growth of bacteria on the bed of zeolite

REVERSE OSMOSIS

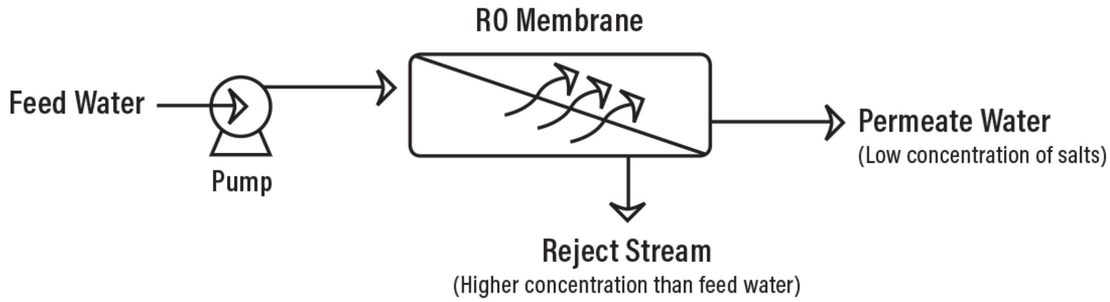
- Reverse Osmosis is a technology that is used to remove a large majority of contaminants from water by pushing the water under pressure through a semi-permeable membrane.
- Osmosis is a naturally occurring phenomenon and one of the most important processes in nature. It is a process where a weaker saline solution will tend to migrate to a strong saline solution. Examples of osmosis are when plant roots absorb water from the soil and our kidneys absorb water from our blood.
- Below is a diagram which shows how osmosis works. A solution that is less concentrated will have a natural tendency to migrate to a solution with a higher concentration. For example, if you had a container full of water with a low salt concentration and another container full of water with a high salt concentration and they were separated by a semi-permeable membrane, then the water with the lower salt concentration would begin to migrate towards the water container with the higher salt concentration.



- A semi-permeable membrane is a membrane that will allow some atoms or molecules to pass but not others. A simple example is a screen door. It allows air molecules to pass through but not pests or anything larger than the holes in the screen door

- Reverse Osmosis is the process of Osmosis in reverse. Whereas Osmosis occurs naturally without energy required, to reverse the process of osmosis you need to apply energy to the more saline solution. A reverse osmosis membrane is a semi-permeable membrane that allows the passage of water molecules but not the majority of dissolved salts, organics, bacteria and pyrogens. However, you need to 'push' the water through the reverse osmosis membrane by applying pressure that is greater than the naturally occurring osmotic pressure in order to desalinate (demineralize or deionize) water in the process, allowing pure water through while holding back a majority of contaminants.
- Below is a diagram outlining the process of Reverse Osmosis. When pressure is applied to the concentrated solution, the water molecules are forced through the semi-permeable membrane and the contaminants are not allowed through.

- Reverse Osmosis works by using a high pressure pump to increase the pressure on the salt side of the RO and force the water across the semi-permeable RO membrane, leaving almost all (around 95% to 99%) of dissolved salts behind in the reject stream. The amount of pressure required depends on the salt concentration of the feed water. The more concentrated the feed water, the more pressure is required to overcome the osmotic pressure.
- The desalinated water that is demineralized or deionized, is called permeate (or product) water. The water stream that carries the concentrated contaminants that did not pass through the RO membrane is called the reject (or concentrate) stream.



- As the feed water enters the RO membrane under pressure (enough pressure to overcome osmotic pressure) the water molecules pass through the semi-permeable membrane and the salts and other contaminants are not allowed to pass and are discharged through the reject stream (also known as the concentrate or brine stream), which goes to drain or can be fed back into the feed water supply in some circumstances to be recycled through the RO system to save water. The water that makes it through the RO membrane is called permeate or product water and usually has around 95% to 99% of the dissolved salts removed from it.

Nano filtration

- The nanofiltration membrane is not a complete barrier to dissolved salts. Depending on the type of salt and the type of membrane, the salt permeability may be low or high. If the salt permeability is low, the osmotic pressure difference between the two compartments may become almost as high as in reverse osmosis. On the other hand, a high salt permeability of the membrane would not allow the salt concentrations in the two compartments to remain very different. Therefore the osmotic pressure plays a minor role if the salt permeability is high.

The key terms used in the reverse osmosis / nanofiltration process are defined as follows.

- Recovery - the percentage of membrane system feed water that emerges from the system as product water or "permeate". Membrane system design is based on expected feed water quality and recovery is defined through initial adjustment of valves on the concentrate stream
- Rejection - the percentage of solute concentration removed from system feed water by the membrane. In reverse osmosis, a high rejection of total dissolved solids (TDS) is important, while in nanofiltration the solutes of interest are specific, e.g., low rejection for hardness and high rejection for organic matter.
- Passage - the opposite of "rejection", passage is the percentage of dissolved constituents (contaminants) in the feedwater allowed to pass through the membrane.
- Permeate - the purified product water produced by a membrane system.
- Flow - Feed flow is the rate of feedwater introduced to the membrane element or membrane system, usually measured in gallons per minute (gpm) or m3/h. Concentrate flow is the rate of flow of non-permeated feedwater that exits the membrane element or membrane system. This concentrate contains most of the dissolved constituents originally carried into the element or into the system from the feed source. It is usually measured in gpm or cubic meters per hour (m3/h).
- Flux - the rate of permeate transported per unit of membrane area, usually measured in gallons per square foot per day (gfd) or liters per square meter and hour (L/m2h)

Factors Affecting Reverse Osmosis and Nanofiltration Performance

- Permeate flux and salt rejection are the key performance parameters of a reverse osmosis or a nanofiltration process. Under specific reference conditions, flux and rejection are intrinsic properties of membrane performance. The flux and rejection of a membrane system are mainly influenced by variable parameters including:
    1. pressure
    2. temperature
    3. recovery
    4. feedwater salt concentration
- The following graphs show the impact of each of those parameters when the other three parameters are kept constant. In practice, there is normally an overlap of two or more effects.

Pressure

- With increasing effective feed pressure, the permeate TDS will decrease while the permeate flux will increase

Temperature

- If the temperature increases and all other parameters are kept constant, the permeate flux and the salt passage will increase

Recovery

- Recovery is the ratio of permeate flow to feed flow. In the case of increasing recovery, the permeate flux will decrease and stop if the salt concentration reaches a value where the osmotic pressure of the concentrate is as high as the applied feed pressure. The salt rejection will drop with increasing recovery

DISINFECTION

Disinfection is the process of the removal, deactivation or killing of pathogenic microorganisms.

Theory of Disinfection

- Microorganisms are destroyed or deactivated, resulting in termination of growth and reproduction such that they represent no significant risk of infection.
- When microorganisms are not removed from drinking water, drinking water usage will cause people to fall ill.
- When water leaves the filter plant, it is still found to contain some of the impurities.
- These impurities can be grouped as: -Bacteria, Viruses, Protozoa -dissolved inorganic salts, -colour, odour and taste, -iron and manganese.
- The substances or materials which are to be used for disinfection are called the

Mechanism

Disinfectant either destroys or inactivates the microorganisms, by following mechanisms

- Damage to cell wall of micro organisms
- Alteration of cell permeability
- Changing the nature of the call protoplasm
- Inactivation of critical enzyme systems responsible for metabolic activities

Method of Disinfection

Following are the minor methods of disinfection:

- Boiling of water
- Treatment with excess lime
- Treatment with ozone
- Treatment with iodine and bromine
- Treatment with Potassium permanganate
- Treatment with Ultra - violet rays
- Treatment with silver, called Electra – Katadyn process

1) <u>Boiling of water</u>

- Method of boiling water to destroy bacteria present in it.
- Not practically possible to boil huge amounts of public water supply.
- Kill the existing germs but cannot take care of future contamination.

2) <u>Treatment with excess lime,</u>

- Excesses lime addition (14ppm - 43 ppm) found to remove the bacterial load (99.3% – 100%) from highly polluted water.
- Excess lime addition raises pH value of water making it extremely alkaline. This extreme alkalinity (or acidity) has been found detrimental to the life of bacteria, thus killing them partially or completely.
- When the lime raises pH of the water is about 9.5, 99% to 100% bacteria are removed
- Removal of excess lime is done by re-carbonation process

3) <u>Treatment with Ozone</u>

- Ozone is faintly blue gas of pungent odour and an excellent disinfectant.
- Unstable allotropic form of oxygen atoms, produced by passing high tension electric current through a stream of air in a closed chamber, under the following chemical reaction.

$$\underset{\text{Oxygen}}{3O_2} \xrightarrow{\text{Under high voltage}} \underset{\text{Ozone}}{2O_3}$$

- Because of instability, Ozone breaks down into normal oxygen and releases nascent oxygen as below.

$$\underset{\text{Ozone}}{O_3} \longrightarrow \underset{\text{Oxygen Molecule}}{O_2} + \underset{\text{Nascent Oxygen}}{O}$$

- Nascent oxygen so produced is powerful oxidizing agent and removes the organic matter as well as bacteria from the water.
- Process: during the treatment, ozone gas is manufactured and bubbled through water contained in a sterilizing chamber having inlet and outlets for water as well as an inlet for ozone. The ozonized water comes out through the outlet at the top.
- Dosage is 2 to 3 ppm to obtain a residual chlorine of 0.1 ppm.
- Largely used in France and Russia & cities like Chandigarh in India
- Advantages
  - ➢ Ozone being unstable nothing being remain in water, by the time it reaches distribution system
  - ➢ Also removes odour, color, taste in addition to bacteria
  - ➢ Ozonized water becomes tasty & pleasant
- Disadvantages
  - ➢ Very costly; much costlier than chlorination
  - ➢ Must need Ozonize and electricity for manufacture
  - ➢ Since no residue can be maintained, does not ensure safety against future protection

4) <u>Treatment with Iodine & Bromine</u>

- Helps in killing pathogenic bacteria.
- Dosage limited to 8ppm and contact period of 5 min generally enough
- Available in the form of pills, very handy

- Used for small water supplies for army troops, private plants &swimming pools

5) Treatment with UV rays
   - Produced by passing electric current through mercury enclosed in quartz bulbs.
   - Highly effective disinfectant.
   - Colorless & less turbid (<15 mg/l) water is suitable for this treatment.
   - Process: water is passed several times around the quartz bulbs emitting such rays.
   - Depth of water over the bulb should be less than10 cm to achieve better penetration.
   - Used for treating Swimming pool waters in developed countries.

6) Treatment with $KMnO_4$ (Potassium permanganate)
   - Popular disinfectant for well water supply contaminated with lesser bacteria (up to 98%)
   - Also helps in oxidizing the taste producing organic matter.
   - Dosage: 1 to 2 mg/l with contact period of 4 to 6 hr., even added in small dose (0.05 to 0.10 mg/l) even after filtration and chlorination

7) Treatment with Silver or Electro- Katadyn Process
   - Metallic sliver ions are introduced into the water by passing it through by a tube containing solid silver electrodes which are connected to a DC supply of 1.5 volts.
   - This introduced silver ion has a strong germicidal action, thus acts as disinfectant.
   - Dose vary b/n 0.05 to 0.1 mg/l & required contact period 15min to 3 hrs.

8) Chlorination
   - Chlorine in its various forms is invariably & almost universally used for disinfecting public water supplies.
   - Cheap, reliable, easy to handle, easy measurable and capable of producing residual disinfecting effects for long periods.
   - Disadvantage is it imparts bitter and bad taste to the water in D/S

FLUORIDATION AND DEFLOURIDATION

- Fluoridation in drinking water must neither be totally absent nor should exceeds an optimum value of about 1 mg/L. to ensure this fluorides are added to waters found deficient in fluoride concentration under a process known as Fluoridation.
- When the fluoride concentration in given water exceeds the limiting values of 1 to 1.5mg/L the fluorides are removed from water under a process known as Deflouridation.

Fluoridation

- If fluoride is less than optimum value (1 mg/L) may result in dental caries in children
- May leads to the formation of weaker tooth enamel leading to early tooth decay & widely suggested that Fluoride proves beneficial to older people in reducing the hardening of the arteries and fluoride stimulates bone formation (helpful in treatment of Osteoporosis)
- Therefore optimum benefits of fluoride are obtained when fluoride concentration is achieves at about 1 mg/. Hence addition of external fluoride to public water supplies is known as fluoridation.

- Compounds which may be used for adding fluoride to the water are Sodium fluoride <Nave>, Sodium silico fluoride <Na_2SiF_6>, hydro-fluosilicic <H_2SiF_6>etc.,
- Fluoride dose should be carefully worked out because the presence of excess fluoride (about 1 to 1.5 mg/L) is also harmful, as to cause spotting and discolouration (mottling of teeth)
- Long term exposure may result in permanent grey to black discolouration of enamel (dental fluorosis)
- Drinking water with >5mg/l fluoride develop sever pitting of enamel
- >30 to 50 mg/L leads to gastroenteritis, skin irritation, deformation of bones and other skeletal abnormalities

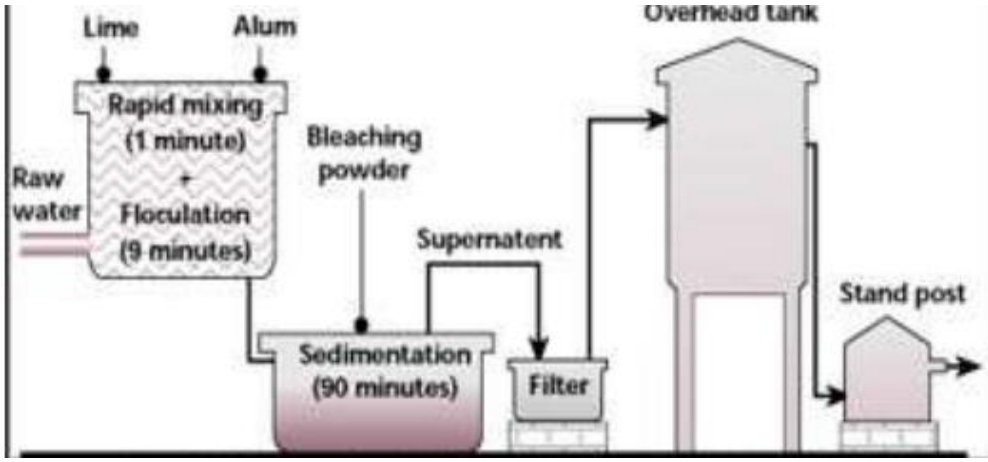
Deflouridation

- Excess intake of fluoride can leads to,
  - Dental fluorosis- discolored, blackened, mottled or chalky white teeth,
  - Skeletal fluorosis_ leads to severe and permanent bone and joint deformation
  - Nonskeletal fluorosis- leads to gastro-intestinal problem and neurological disorder.
  - Can cause fetus damage and affect IQ of children
  - Flourosis is in fact an irreversible diseases and it has no cure.

Method of defluoridation

1. Nalagonda Techniques
2. Adsorption by Activated carbon (Prasanti technology)
3. Iron exchange Adsorption method
4. Reverse Osmosis process

1. Nalagonda technique –
   - Particularly adopted in Rural area's ground water containing excess fluoride content
   - Uses aluminium slat (alum) for removing fluoride
   - Raw water is firstly mixed with adequate amount of lime (CaO) or sodium carbonate (Na_2CO_3) and thoroughly mixed which helps to ensure adequate alkalinity required for effective hydrolysis of aluminium salts (so that residual aluminium does not remains in treated water)

- 

Alum is added in solution form and water is stirred slowly for about 10 min and allowed to settle for nearly one hours
- Precipitated sludge is discarded and the clear supernatant containing permissible amount of Fluoride is withdrawn for use
- Bleaching powder is also generally added with lime prior to the addition of alum, to achieve simultaneous disinfection of treated water and also to keep the system free from undesirable biological growth
- The line diagram for the process is as shown in figure.

2. Prasanti Technology by Activated Alumina(AA)
   - In this method, the raw water containing high content of fluoride, is passed (percolated) through the insoluble granular beds of substance like Activated Alumina(AA) or Bone Char or Activated Carbon or Serpentines or Activated Bauxite which adsorb fluoride from percolating water, giving out defluoridated water
   - Except AA all other compounds founds to be uneconomical & AA is excellent medium for removal of excess fluoride
   - It is highly selective to fluoride in presence of sulphate and chloride when compared to synthetic ion exchange resins
   - The adsorption process is best carried out slightly acidic conditions (pH 5 to 7) lower order of pH is more effective for removal
   - AA after saturation can be cleaned and regenerated back washing with 1% Caustic soda solution (NaOH)
3. Ion Exchange Adsorption method
   - As water passes through the strong base anion exchange resin (Zeolite) in the chloride form contained in a pressure vessel, fluorides and other anions like arsenic, nitrates etc present in the water are exchanged with the chlorides ions of the resin, thus releasing chlorides into water and adsorbing fluoride ions into the resin.

- When resin gets saturated with anions like fluoride, nitrates, arsenic etc., as indicated by their increased concentration in the out flowing water, some can be cleaned and regenerated with 5 to 10% NaCl solution (Basins) and the bed is returned to services
- During regeneration, exchange process gets reversed as the anions adsorbed on the resins get replaced by chlorides ions and discharged to wastewater with excess chloride ions.
- Capacity of Plant- 500l/hr. to 5000l/hr.
- Although method is highly efficient, requires regular replacement of resin and large amount of salt(NaCl) for regeneration is hence costly

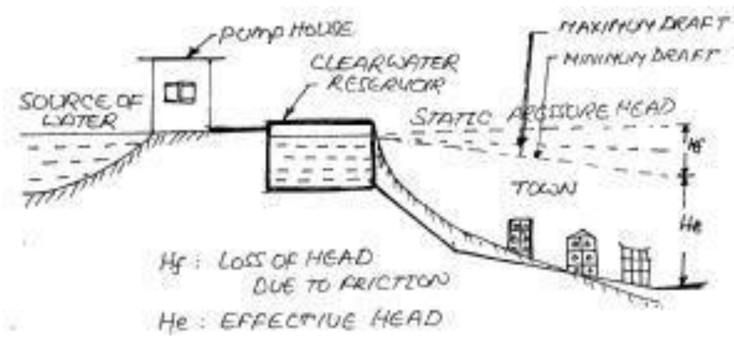
4. Reverse Osmosis process
    - Raw water is passed through a semi-permeable membrane barrier which permits the flow of salts including fluorides
    - Rarely used solely for defluoridation of village water supplies due to prohibitive high cost through it has high removed efficiently without use of any chemicals.

METHOD OF DISTRIBUTION:

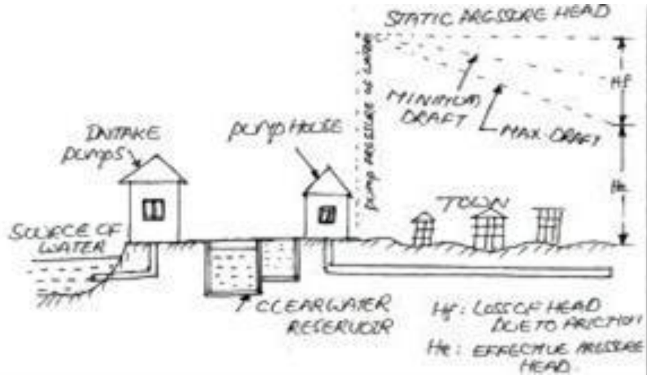
- For efficient distribution it is required that the water should reach to every consumer with required rate of flow. Therefore, some pressure in pipeline is necessary, which should force the water to reach at every place. Depending upon the methods of distribution, the distribution system is classified as the follows:

1. Gravity system
2. Pumping system
3. Dual system or combined gravity and pumping system

- Gravity system: When some ground sufficiently high above the city area is available, this can be best utilized for distribution system in maintaining pressure in water mains. This method is also much suitable when the source of supply such as lake, river or impounding reservoir is at sufficiently higher than city. The water flows in the mains due to gravitational forces. As no pumping is required therefore it is the most reliable system for the distribution of water as shown in fig



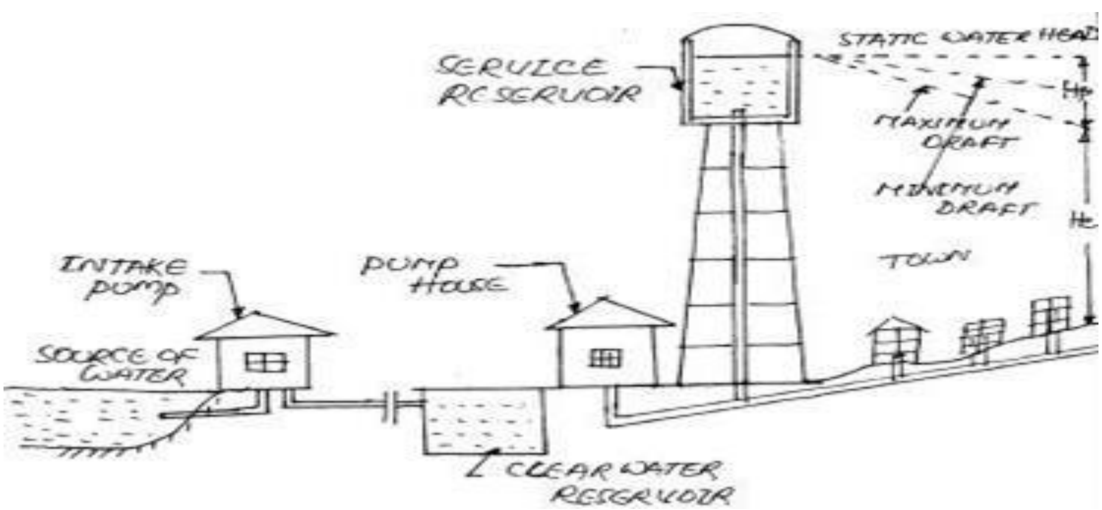
Pumping system:



- Constant pressure can be maintained in the system by direct pumping into mains. Rate of flow cannot be varied easily according to demand unless number of pumps are operated in addition to stand by ones.
- Supply can be effected during power failure and breakdown of pumps. Hence diesel pumps also in addition to electrical pumps as stand by to be maintained. During fires, the water can be pumped in required quantity by the stand by units.

Combined pumping and gravity system:

- This is also known as dual system. The pump is connected to the mains as well as elevated reservoir. In the beginning when demand is small the water is stored in the elevated reservoir, but when demand increases the rate of pumping, the flow in the distribution system comes from the both the pumping station as well as elevated reservoir.
- As in this system water comes from two sources one from reservoir and second from pumping station, it is called dual system. This system is more reliable and economical, because it requires uniform rate of pumping but meets low as well as maximum demand. The water stored in the elevated reservoir meets the requirements of demand during breakdown of pumps and for fire fighting



The water may be supplied to the consumers by either of the two systems.

1. Continuous system

   a. This is the best system and water is supplied for all 24 hours. This system is possible when there is adequate quantity of water for supply. In this system sample of water is always available for firefighting and due to continuous circulation water always remains fresh. In this system less diameter of pipes are required and rusting of pipes will be less. Losses will be more if there are leakages in the system.

2. Intermittent system

   a. If plenty of water is not available, the supply of water is divided into zones and each zone is supplied with water for fixed hours in a day or on alternate days. As the water is supplied after intervals, it is called intermittent system. The system has following disadvantages:

      i. Pipelines are likely to rust faster due to alternate wetting and drying. This increases the maintenance cost.
      ii. There is also pollution of water by ingress of polluted water through leaks during non-flow periods.
      iii. More wastage of water due to the tendency of the people to store more water than required quantity and to waste the excess to collect fresh water each time.
      iv. Inspire of number of disadvantages, this system is usually adopted in most of the cities and towns of India. In this system water can be supplied in the high level localities with adequate pressure by dividing the city in zones. The repair work can be easily done in the non-supply hours.

MODULE 3

Necessity for sanitation

- Every community produces both liquid and solid wastes .The liquid portion –waste water– is essentially the water supply of the community after it has been fouled by a variety of uses such as spent water from bathroom kitchen, lavatory basins, house and street washings, from various industrial processes semi solid wastes of human and animal excreta, dry refuse of house and street sweepings, broken furniture, wastes from industries etc are produced daily.
- If proper arrangements for the collection, treatment and disposal are not made, they will go on accumulating and create foul condition. If untreated water is accumulating, the decomposition of the organic materials it contains can lead to the production of large quantity of mal odorous gases. It also contains nutrients, which can stimulate the growth of aquatic plants and it may contain toxic compounds. Therefore in the interest of community of the city or town, it is most essential to collect,

treat and dispose of all the waste products of the city in such a way that it may not cause any hazardous effects on people residing in town and environment.

- Waste water engineering is defined as the branch of the environmental engineering where the basic principles of the science and engineering for the problems of the water pollution problems. The ultimate goal of the waste water management is the protection of the environmental in manner commensurate with the economic, social and political concerns.
- Although the collection of stream water and drainage dates from ancient times the collection of waste water can be treated only to the early 1800s. The systematic treatment of waste water followed in the 1800s and 1900s.

Definitions of some common terms used in the sanitary engineering. REFUSE:

This is the most general term to indicate the wastes which include all the rejects left as worthless, sewage, sullage – all these terms are included in this term.

Garbage:

It is a dry refuse which includes, waste papers, sweepings from streets and markets, vegetable peelings etc. The quantity of garbage per head per day amounts to be about .14 to .24 kg for Indian conditions. Garbage contains large amount of organic and putrefying matter and therefore should be removed as quickly as possible.

Rubbish:

It consists of sundry solid wastes from the residencies, offices and other buildings. Broken furniture, paper, rags etc are included in this term. It is generally dry and combustible.

Sullage:

It is the discharge from the bath rooms, kitchens, wash basins etc., it does not include discharge from the lavatories, hospitals, operation theaters, slaughter houses which has a high organic matter

Sewage:

It is a dilute mixture of the wastes of various types from the residential, public and industrial places. It includes sullage water and foul discharge from the water closets, urinals, hospitals, stables, etc.

Storm Water:

It is the surface runoff obtained during and after the rainfall which enters sewers through inlet. Storm water is not foul as sewage and hence it can be carried in the open drains and can be disposed off, in the natural rivers without any difficulty.

Sanitary Sewage:

It is the sewage obtained from the residential buildings & industrial effluents establishments‘. Being extremely foul it should be carried through underground conduits.

Domestic Sewage:

It is the sewage obtained from the lavatory basins, urinals &water closets of houses, offices & institutions. It is highly foul on account of night soil and urine contained in it. Night soil starts putrefying & gives offensive smell. It may contain large amount of bacteria due to the excremental wastes of patients. This sewage requires great handling &disposal.

Industrial Sewage:

It consists of spent water from industries and commercial areas. The degree of foulness depends on the nature of the industry concerned and processes involved.

Sewers:

Ewers are underground pipes which carry the sewage to a point of disposal.

Sewerage:

The entire system of collecting, carrying &disposal of sewage through sewers is known as sewerage.

Dry Weather Flow (DWF):

Domestic sewage and industrial sewage collectively, is called as DWF. It does not contain storm water. It indicates the normal flow during dry season.

Bacteria:

These are the microscopic organisms. The following are the groups of bacteria:

-Aerobic bacteria: they require oxygen &light for their survival.

-Anaerobic bacteria: they do not require free oxygen and light for survival.

Facultative bacteria: they can exist in the presence or absence of oxygen. They grow more in absence of air.

Invert:

It is the lowest point of the interior of the sewer at any c/s.

Sludge:

It is the organic matter deposited in the sedimentation tank during treatment.

Methods of domestic waste water disposal

After the waste water is treated it is disposed in the nature in the following two principal methods

- Disposal by Dilution where large receiving water bodies area available
- Land disposal where sufficient land is available

Sanitary Engg starts at the point where water supply Engg ends.

It can be classified as

- Collection works
- Treatment works
- Disposal works

- The collection consists of collecting tall types of waste products of town. Refuse is collected separately. The collection works should be such that waste matters can be transported quickly and steadily to the treatment works. The system employed should be self-cleaning and economical.
- Treatment is required to treat the sewage before disposal so that it may not pollute the atmosphere & the water body in which it will be disposed of .The type of treatment processes depend on the nature of the waste water characteristics and hygiene, aesthetics and economical aspects.
- The treated water is disposed of in various ways by irrigating fields or discharging in to natural water courses.

Different Methods of domestic waste water disposal include (Systems of Sanitation)

- Conservancy System
- Water Carriage System

Conservancy system

- Sometimes the system is also called as dry system. This is out of date system but is prevailing in small towns and villages. Various types of refuse and storm water are collected conveyed and disposed of separately. Garbage is collected in dustbins placed along the roads from where it is conveyed by trucks ones or twice a day to the point of disposal. all the non-combustible portion of garbage such as sand dust clay etc are used for filling the low level areas to reclaim land for the future development of the town. The combustible portion of the garbage is burnt. The decaying matters are dried and disposed of by burning or the manufacture of manure.
- Human excreta are collected separately in conservancy latrines. The liquid and semi liquid wastes are collected separately after removal of night soil it is taken outside the town in trucks and buried in trenches. After 2-3 years the buried night soil is converted into excellent manure. In conservancy system sullage and storm water are carried separately in closed drains to the point of disposal where they are allowed to mix with river water without treatment.

Water Carriage System

- With development and advancement of the cities urgent need was felt to replace conservancy system with some more improved type of system in which human agency should not be used for the collection and conveyance of sewage .After large number of experiments it was found that the water is the only cheapest substance which can be easily used for the collection and conveyance of sewage. As in this system water is the main substance therefore it is called as Water carriage system.

- In this system the excremental matter is mixed up in large quantity of water there are taken out from the city through properly designed sewerage systems, where they are disposed of after necessary treatment in a satisfactory manner.
- The sewages so formed in water carriage system consist of 99.9% of water and .1% solids .All these solids remain in suspension and do not changes the specific gravity of water therefore all the hydraulic formulae can be directly used in the design of sewerage system and treatment plants.

Conservancy System	Water Carriage System
1) Very cheap in initial cost.	1) It involves high initial cost.
2) Due to foul smells from the latrines, they are to be constructed away from living room so building cannot be constructed as compact Units.	2) As there is no foul smell latrines remain clean and neat and hence are constructed with rooms, therefore buildings may be compact.
3)The aesthetic appearance of the city cannot be improved	3) Good aesthetic appearance of city can be obtained.
4) For burial of excremental matter large area is required.	4) Less area is required as compared to conservancy system.
5) Excreta is not removed immediately hence its decomposition starts before removal, causing nuisance smell.	5) Excreta are removed immediately with water, no problem of foul smell or hygienic trouble.
6) This system is fully depended on human agency .In case of strike by the sweepers; there is danger of insanitary conditions in city.	6)As no human agency is involved in this system ,there is no such problem as in case of conservancy system

Sewerage systems:

1) Separate System Of Sewage
2) Combined System Of Sewage
3) Partially Combined Or Partially Separate System

1. Separate System Of Sewerage

In this system two sets of sewers are laid .The sanitary sewage is carried through sanitary sewers while the storm sewage is carried through storm sewers. The sewage is carried to the treatment plant and storm water is disposed of to the river.

<u>Suitable conditions for separate sewerage systems</u>:-

A separate system would be suitable for use under the following situations:

1. Where rainfall is uneven.
2. Where sanitary sewage is to be pumped.
3. The drainage area is steep, allowing to runoff quickly.

4. Sewers are to be constructed in rocky strata. The large combined sewers would be more expensive.

Advantages:

1) Size of the sewers are small
2) Sewage load on treatment unit is less
3) Rivers are not polluted
4) Storm water can be discharged to rivers without treatment.

Disadvantage

1) Sewerage being small, difficulty in cleaning them
2) Frequent choking problem will be their
3) System proves costly as it involves two sets of sewers
4) The use of storm sewer is only partial because in dry season the will be converted in to dumping places and may get clogged.

2. Combined System of Sewage

When only one set of sewers are used to carry both sanitary sewage and surface water. This system is called combined system.

Sewage and storm water both are carried to the treatment plant through combined sewers

<u>Suitable conditions for combined system</u>:-

1. Rainfall in even throughout the year.
2. Both the sanitary sewage and the storm water have to be pumped.
3. The area to be sewered is heavily built up and space for laying two sets of pipes is not enough.
4. Effective or quicker flows have to be provided.

After studying the advantages and disadvantages of both the systems, present day construction of sewers is largely confined to the separate systems except in those cities where combined system is already existing. In places where rainfall is confined to one season of the year, like India and even in temperate regions, separate system are most suitable.

Advantages:

1) Size of the sewers being large, chocking problems are less and easy to clean.
2) It proves economical as 1 set of sewers are laid.
3) Because of dilution of sanitary sewage with storm water nuisance potential is reduced

Disadvantages:

1) Size of the sewers being large, difficulty in handling and transportation.
2) Load on treatment plant is unnecessarily increased
3) It is uneconomical if pumping is needed because of large amount of combined flow.

4) Unnecessarily storm water is polluted.

3. Partially Combined or Partially Separate System

A portion of storm water during rain is allowed to enter sanitary sewer to treatment plants while the remaining storm water is carried through open drains to the point of disposal.

Advantages:-

1. The sizes of sewers are not very large as some portion of storm water is carried through open drains.
2. Combines the advantages of both the previous systems.
3. Silting problem I completely eliminated.

Disadvantages:-

1. During dry weather, the velocity of flow may be low.
2. The storm water is unnecessary put load on to the treatment plants to extend.
3. Pumping of storm water in unnecessary over-load on the pumps.

Table -2.2:- Comparison of Separate and Combined systems

Sl. no.	Separate system	Combined system
1.	The quantity of sewage to be treated is less, because no treatment of storm water is done.	As the treatments of both are done, the treatment is costly.
2.	In the cities of more rainfall this system is more suitable.	In the cities of less rainfall this system is suitable.
3.	As two sets of sewer lines are to laid, this system is cheaper because sewage is carried in underground sewers and storm water in open drains.	Overall construction cost is higher than separate system.
4.	In narrow streets, it is difficult to use this system.	It is more suitable in narrow streets.
5.	Less degree of sanitation is achieved in this system, as storm water is disposed without any treatment.	High degree of sanitation is achieved in this system.

Sources of Sewage:-

Sanitary sewage is produced from the following sources:

1. When the water is supplied by water works authorities or provided from private sources, it is used for various purposes like bathing, utensil cleaning, for flushing water closets and urinals or washing clothes or any other domestic use. The spent water for all the above needs forms the sewage.
2. Industries use the water for manufacturing various products and thus develop the sewage.
3. Water supplied to schools, cinemas, hotels, railway stations, etc., when gets used develops sewage.

4. Ground water infiltration into sewers through loose joints.
5. Unauthorized entrance of rain water in sewer lines. Nature of Sewage:-

Sewage is a dilute mixture of the various types of wastes from the residential, public and industrial places. The characteristics and composition i.e. the nature of sewage mainly depends on this source. Sewage contains organic and inorganic matters which may be dissolved, suspension and colloidal state. Sewage also contains various types of bacteria, virus, protozoa, etc. sewage may also contain toxic or other similar materials which might have got entry from industrial discharges. Before the design of any sewage treatment plant the knowledge of the nature of sewage is essential.

Quantity of Sanitary Sewage and Storm Water:-

The determination of sanitary sewage is necessary because of the following factors which depend on this:

1. To design the sewerage schemes as well as to dispose a treated sewage efficiently.
2. The size, shape and depth of sewers depend on quantity of sewage.
3. The size of pumping unit depends on the quantity of sewage. Estimate of Sanitary Sewage:-

Sanitary sewage is mostly the spent water of the community into sewer system with some groundwater and a fraction of the storm runoff from the area, draining into it. Before designing the sewerage system, it is essential to know the quantity of sewage that will flow through the sewer.

The sewage may be classified under two heads:

1. The sanitary sewage, and
2. Storm water

Sanitary sewage is also called as the Dry Weather Flow (D.W.F), which includes the domestic sewage obtained from residential and residential and industrials etc., and the industrial sewage or trade waste coming from manufacturing units and other concerns.

Storm water consists of runoff available from roots, yards and open spaces during rainfall.

Quantity of Sewage: - (Dry Weather Flow)

It is usual to assume that the rate of sewage flow, including a moderate allowance for infiltration equals to average rate of water consumption which is 135 liter/ head /day according to Indian Standards. It varies widely depending on size of the town etc. this quantity is known as Dry Weather Flow (D.W.F). It is the quantity of water that flows through sewer in dry weather when no storm water is in the sewer.

Rate of flow varies throughout 24 hours and is usually the greatest in the fore-noon and very small from midnight to early morning. For determining the size of sewer, the maximum flow should be taken as three times the D.W.F.

Design Discharge of Sanitary Sewage

The total quantity of sewage generated per day is estimated as product of forecasted population at the end of design period considering per capita sewage generation and appropriate peak factor. The per capita sewage generation can be considered as 75 to 80% of the per capita water supplied per day. The increase in population also result in increase in per capita water demand and hence, per capita production of sewage. This increase in water demand occurs due to increase in living standards, betterment in economic condition, changes in habit of people, and enhanced demand for public utilities.

Factors affecting the quantity of sewage flow:-

The quantity of sanitary sewage is mainly affected by the following factors:

1. Population
2. Type of area
3. Rate of water supply
4. Infiltration and exfiltration

In addition to above, it may also be affected by habits of people, number of industries and water pressure etc.

Population:-

- The quantity of sanitary sewage directly depends on the population. As the population increases the quantity of sanitary sewage also increases. The quantity of water supply is equal to the rate of water supply multiplied by the population. There are several methods used for forecasting the population of a community.

o Type of area covered:-

- The quantity of sanitary sewage also depends on the type of area as residential, industrial or commercial. The quantity of sewage developed from residential areas depend on the rate of water supply to that area, which is expressed a liters/ capita/ day and this quantity is obtained by multiplying the population with this factor.
- The quantity of sewage produced by various industries depends on their various industrial processes, which is different for each industry.
- Similarly the quantity of sewage obtained from commercial and public places can be determined by studying the development of other such places.

o Rate of water supply:-

- Truly speaking the quantity of used water discharged into a sewer system should be a little less than the amount of water originally supplied to the community. This is because of the fact that all the water supplied does not reach sewers owing to such losses as leakage in pipes or such deductions as lawn sprinkling, manufacturing processes etc. However, these losses may be largely be made up by such additions as surface drainage, groundwater infiltration, water supply from private wells etc. On an average, therefore, the quantity of sewage maybe considered to be nearly equal to the quantity of water supplied.

Groundwater infiltration and exfiltration:-

The quantity of sanitary sewage is also affected by groundwater infiltration through joints. The quantity will depend on, the nature of soil, materials of sewers, type of joints in sewer line, workmanship in laying sewers and position of underground water table.

- **Infiltration** causes increase to the ―legitimate‖ flows in urban sewerage systems. Infiltration represents a slow response process resulting in increased flows mainly due to seasonally-elevated groundwater entering the drainage system, and primarily occurring through defects in the pipe network.
- **Exfiltration** represents losses from the sewer pipe, resulting in reduced conveyance flows and is due to leaks from defects in the sewer pipe walls as well as overflow discharge into manholes, chambers and connecting surface water pipes. The physical defects are due to a combination of factors including poor construction and pipe joint fittings, root penetration, illicit connections, biochemical corrosion, soil conditions and traffic loadings as well as aggressive groundwater.
- It is clear that Infiltration and Exfiltration involve flows passing through physical defects in the sewer fabric and they will often occur concurrently during fluctuations in groundwater levels, and particularly in association with wet weather events; both of which can generate locally high hydraulic gradients.
- Exfiltration losses are much less obvious and modest than infiltration gains, and are therefore much more difficult to identify and quantify. However, being dispersed in terms of their spatial distribution in the sewer pipe, exfiltration losses can have potentially significant risks for groundwater quality.
- The episodic but persistent reverse ―pumping‖ effect of hydraulic gain and loss will inevitably lead to long term scouring of pipe surrounds and foundations resulting in pipe collapse and even surface subsidence.

Hydraulic constraints of the systems designed, and Life of the material and equipment.
Following design period can be considered for different components of sewerage scheme.
1.Laterals less than 15 cm diameter : Full development
2.Trunk or main sewers : 40 to 50 years
3.Treatment Units : 15 to 20 years
4.Pumping plant : 5 to 10 years

Rational method:-

Runoff from an area can be determined by the Rational Method. The method gives a reasonable estimate up to a maximum area of 50 ha (0.5 Km^2).

Assumptions and Limitations

Use of the rational method includes the following assumptions and limitations: Precipitation is uniform over the entire basin.

- Precipitation does not vary with time or space. Storm duration is equal to the time of concentration
- A design storm of a specified frequency produces a design flood of the same frequency.
- The basin area increases roughly in proportion to increases in length.
- The time of concentration is relatively short and independent of storm intensity. The runoff coefficient does not vary with storm intensity or antecedent soil moisture.
- Runoff is dominated by overland flow. Basin storage effects are negligible.
- The minimum duration to be used for computation of rainfall intensity is 10 minutes. If the time of concentration computed for the drainage area is less than 10 minutes, then 10 minutes should be adopted for rainfall intensity computations.

This method is mostly used in determining the quantity of storm water. The storm water quantity is determined by the rational formula:

$$\mathbf{Q=AIR/360\ m^3/s}$$

Where,

Q= quantity of storm water in m^3/sec C= coefficient of runoff
I= intensity of rainfall
A= area of drainage in hectare

Runoff coefficient:-

In rational method, the value of runoff coefficient, C is required. The whole quantity of rain water that fall over the ground does not reach the sewer line. A portion of it percolates in the ground, a portion evaporates, a portion is stored in ponds and ditches and only remaining portion of rainwater reaches the sewer line. The runoff coefficient depends mainly on characteristics of ground surface as porosity, wetness, ground cover etc., which varies from 0.01 for forest or wooded area to 0.95 for a water tight roof surfaces.

As every locality consists of different types of surface area, therefore for calculating the overall runoff coefficient the following formula is used:

Runoff coefficient (overall) I avg = $\frac{A1I1+A2I2+A3I3 \ldots\ldots\ldots AnIn}{A1+A2+A3+\cdots \ldots An}$

Where:

- A_1, A_2 ,A_3….are different types of area and
- C_1, C_2, C_3…..are their runoff coefficient respectively.

Quantity of storm water flow:-

When rain falls over the ground surface, a part of it percolates into the ground, a part is evaporated in the atmosphere and the remaining part overflows as storm water. This quantity of storm water is very large as compared with sanitary sewage.

Factors affecting storm water:-

The following are factors which affect the quantity of storm water:

1. Rainfall intensity and duration.
2. Area of the catchment.
3. Slope and shape of the catchment area.
4. Nature of the soil and the degree of porosity.
5. Initial state of the catchment.
   If rainfall intensity and duration is more, large will be the quantity of storm water available. If the rainfall takes place very slowly even though it continues for the whole day, the quantity of storm water available will be less.

Harder surface yield more runoff than soft, rough surfaces. Greater the catchment area greater will be the amount of storm water. Fan shaped and steep areas contribute more quantity of storm water. In addition to the above it also depends on the temperature, humidity, wind etc.

Estimate of quantity of storm water:-

Generally there are two methods by which the quantity of storm water is calculated:

1. Rational method
2. Empirical formulae method

In both the above methods, the quantity of storm water is a function of the area, the intensity of rainfall and the co-efficient of runoff.

Time of concentration:-

- The time taken for the maximum runoff rate to develop, is known as the time of concentration, and is equal to the time required for a drop of water to run from the farthest point of the watershed to the point for which the runoff is to be calculated.

- The time of concentration, t_c, of a watershed is often defined to be the time required for a parcel of runoff to travel from the most hydraulically distant part of a watershed to the outlet. It is not possible to point to a particular point on a watershed and say, ―The time of concentration is measured from this point.‖ Neither is it possible to measure the time of concentration. Instead, the concept of Tc is useful for describing the time response of a watershed to a driving impulse, namely that of watershed runoff.
- In the context of the rational method then, t_c represents the time at which all areas of the watershed that will contribute runoff are just contributing runoff to the outlet. That is, at t_c, the watershed is fully contributing. We choose to use this time to select the rainfall intensity for application of the rational method. If the chosen storm duration is larger than t_c, then the rainfall intensity will be less than that at t_c.
- Therefore, the peak discharge estimated using the rational method will be less than the optimal value. If the chosen storm duration is less than t_c, then the watershed is not fully contributing runoff to the outlet for that storm length, and the optimal value will not be realized. Therefore, we choose the storm length to be equal to t_c for use in estimating peak discharges using the rational method.
- The time of concentration refers to the time at which the whole area just contributes runoff to point.

$$\mathbf{T_c = T_e + T_f}$$

Where,
Tc = time of concentration
Te = time of entry to the inlet (usually taken as 5 – 10 min)
Tf = time of flow in the sewer

Time of concentration is made up of inlet time (over land flow) and channel flow time.

- *Time of entry (inlet time or overland flow):* is the time required for water to reach a defined channel such as a street gutter, plus the gutter flow time to the inlet.
- *Channel flow time*: is the **time of flow** through the sewers to the point at which rate of flow is being assessed. The channel flow time can be estimated with reasonable accuracy from the hydraulic characteristics of the sewer. The channel flow time is then determined as the flow length divided by the average velocity.
- The inlet time is affected by numerous factors, such as rainfall intensity, surface slope, surface roughness, flow distance, infiltration capacity, and depression storage. Because of this, accurate values are difficult to obtain. Design inlet flow times of from 5 to 30 min are used in practice.

Estimating Time of Concentration

- There are many methods for estimating t_c. In fact, just about every hydrologist or engineer has a favorite method. All methods for estimating t_c are empirical, that is, each is based on the analysis of one or more datasets. The methods are not, in general, based on theoretical fluid mechanics.
- For application of the rational method, Tx DOT recommends that Tc be less than 300 minutes (5 hours) and greater than 10 minutes. Other agencies require that Tc be greater than 5 minutes. The concept is that estimates of i become unacceptably large for durations less than 5 or 10 minutes. For long durations (such as longer than 300 minutes), the assumption of a relatively steady rainfall rate is less valid.

Materials of sewer/ Types of sewers

Different types of sewers are discussed

1. Asbestos Cement Sewers

- These are manufactured from a mixture of asbestos fibers, silica and cement. Asbestos fibers are thoroughly mixed with cement to act as reinforcement.
- These pipes are available in size 10 to 100 cm internal diameter and length up to 4.0 m.
- These pipes can be easily assembled without skilled labour with the help of special coupling, called ‗Ring Tie Coupling‘or Simplex joint.
- The pipe and joints are resistant to corrosion and the joints are flexible to permit 12^{o} deflection for curved laying.
- These pipes are used for vertical transport of water. For example, transport of rainwater from roofs in multistoried buildings, for transport of sewage to grounds, and for transport of less foul sullage i.e., wastewater from kitchen and bathroom.

Advantages

- These pipes are light in weight and hence, easy to carry and transport.
- Easy to cut and assemble without skilled labour.
- Interior is smooth (Manning n = 0.011) hence, can make excellent hydraulically efficient sewer.

Disadvantages

- These pipes are structurally not very strong.
- These are susceptible to corrosion by Sulphuric acid. When bacteria produce H2S, in presence of water, H2SO4 can be formed.

2. Bricks sewers: -

Brick sewers are made it site. They are used for construction of large size sewers. Now a day‘s brick sewers are replaced by concrete sewers because lot of labour is involved in the construction of brick sewers. This material is used for construction of large size combined sewer or particularly for storm water drains. The pipes are plastered from outside to avoid entry of tree roots and ground water through brick joints. These are lined from inside with stone ware or ceramic block to make them smooth and hydraulically efficient. Lining also make the pipe resistant to corrosion.

3. Cast Iron Sewers

These pipes are stronger and capable to withstand greater tensile, compressive, as well as bending stresses. However, these are costly. Cast iron pipes are used for outfall sewers, rising mains of pumping stations, and inverted siphons, where pipes are running under pressure. These are also suitable for sewers under heavy traffic load, such as sewers below railways and highways. They are used for carried over piers in case of low lying areas. They form 100% leak proof sewer line to avoid ground water contamination. They are less resistant to corrosion; hence, generally lined from inside with cement concrete, coal tar paint, epoxy, etc. These are joined together by bell and spigot joint. IS: 1536-1989 and IS: 1537-1976 provides the specifications for spun and vertically cast pipes, respectively.

4. Plain Cement Concrete or Reinforced Cement Concrete

Plain cement concrete (1: 1.5: 3) pipes are available up to 0.45 m diameter and reinforcement cement pipes are available up to 1.8 m diameter. These pipes can be cast in situ or precast pipes. Precast pipes are better in quality than the cast in situ pipes. The reinforcement in these pipes can be different such as single cage reinforced pipes, used for internal pressure less than 0.8 m; double cage reinforced pipes used for both internal and external pressure greater than 0.8 m; elliptical cage reinforced pipes used for larger diameter sewers subjected to external pressure; and hume pipes with steel shells coated with concrete from inside and outside. Nominal longitudinal reinforcement of 0.25% is provided in these pipes.

Advantages of concrete pipes

- Strong in tension as well as compression.
- Resistant to erosion and abrasion.
- They can be made of any desired strength.
- Easily molded, and can be in situ or precast pipes.
- Economical for medium and large sizes.
- These pipes are available in wide range of size and the trench can be opened and backfilled rapidly during maintenance of sewers.

Disadvantages

- These pipes can get corroded and pitted by the action of H_2SO_4.
- The carrying capacity of the pipe reduces with time because of corrosion.
- The pipes are susceptible to erosion by sewage containing silt and grit.

The concrete sewers can be protected internally by vitrified clay linings. With protection lining they are used for almost all the branch and main sewers. Only high alumina cement concrete should be used when pipes are exposed to corrosive liquid like sewage.

5. Corrugated iron sewers: - Corrugated iron sewers are used for storm sewers. The sewers should be protected from the effects of corrosion by galvanization or by bituminous coatings. They are made in varying metal thickness and in diameters up to 450cm.

6. Plastic sewers: - (PVC pipes) Plastic is recent material used for sewer pipes. These are used for internal drainage works in house. These are available in sizes 75 to 315 mm external diameter and used in drainage works. They offer smooth internal surface. The additional advantages they offer are resistant to corrosion, light weight of pipe, economical in laying, jointing and maintenance, the pipe is tough and rigid, and ease in fabrication and transport of these pipes.

7. High Density Polyethylene (HDPE) Pipes

Use of these pipes for sewers is recent development. They are not brittle like AC pipes and other pipes and hence hard fall during loading, unloading and handling do not cause any damage to the pipes. They can be joined by welding or can be jointed with detachable joints up to 630 mm diameter (IS: 4984-1987). These are commonly used for conveyance of industrial wastewater.

They offer all the advantages offered by PVC pipes.

- Steel sewers: - There sewers are used where lightness, imperviousness and resistance to high pressure are the prime requirements. There sewers are flexible and can absorb vibrations and shocks efficiently.
- There are mainly used for trunk or outfall sewers. Riveting should, as far as possible be avoided. These are used under the situations such as pressure main sewers, under water crossing, bridge crossing, necessary connections for pumping stations, laying pipes over self-supporting spans, railway crossings, etc. They can withstand internal pressure, impact load and vibrations much better than CI pipes.
- They are more ductile and can withstand water hammer pressure better. These pipes cannot withstand high external load and these pipes may collapse when negative pressure is developed in pipes. They are susceptible to corrosion and are not generally used for partially flowing sewers. They are protected internally and externally against the action of corrosion.

8. Vitrified Clay or Stoneware Sewers

These pipes are used for house connections as well as lateral sewers. The size of the pipe available is 5 cm to 30 cm internal diameter with length 0.9 to 1.2 m. These pipes are rarely manufactured for diameter greater than 90 cm. These are joined by bell and spigot flexible compression joints.

Advantages

- Resistant to corrosion, hence fit for carrying polluted water such as sewage.
- Interior surface is smooth and is hydraulically efficient.
- The pipes are highly impervious.
- Strong in compression.
- These pipes are durable and economical for small diameters.
- The pipe material does not absorb water more than 5% of their own weight, when immersed in water for 24 h.

Disadvantages

- Heavy, bulky and brittle and hence, difficult to transport.
- These pipes cannot be used as pressure pipes, because they are weak in tension.
- These require large number of joints as the individual pipe length is small.

9. Wooden sewers: -In early stages these sewers were put into use. They are difficult to construct and maintain. The life of sewers is short and they are now rarely in use.

Shapes of Sewers

Sewers are generally circular pipes laid below ground level, slopping continuously towards the outfall. These are designed to flow under gravity. Mostly sewers of circular shape are used in all the sewerage schemes, because of the following facts:

(i) It affords least perimeter and hence construction material required is minimum.
(ii) They are easy to construct and handle.
(iii) Since it has no corners, there are less chances of deposition of organic matters.
(iv) They possess excellent hydraulic properties.

However, sewers of non-circular shapes are used for the following reasons.

(i) To develop self-cleansing velocity in the sewer, when the flow is minimum.
(ii) To effect economy in the construction.
(iii) To increase the headway so that a man can enter easily for repairs, and cleaning. Following are the non-circular shapes of sewers which are commonly used for sewers:

1. Box or rectangular sewers
2. Egg-shaped or avoid sewers
3. Basket-handle sections
4. Horse shoe sewers
5. Parabolic sewers
6. Semi-circular sewers
7. Semi-elliptical sewers
8. U-shaped sewers

Hazen –Williams's formula:

$V = 0.85\ C\ R^{0.6}\ S^{0.54}$

Notations are same as previous; the value of C varies depending upon the surfaces.

Table

Sl no.	Type of material	Value of C
1	Stoneware pipes in good condition	110
2	New cast iron	130

3	Very smooth surfaces	140

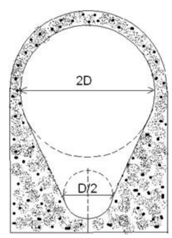
1. Box or rectangular type sewers

In olden days these sewers were constructed by laying concrete at bottom and constructing the sides with masonry. But now a day‘s masonry has been completely replaced by concrete. These are mainly used for out fall sewers. They have got relatively high hydraulic mean depth at large flows and therefore can have higher velocities when laid to the same slope as that of a circular or egg-shared sewer. They are therefore most suitable for large size storm sewers.

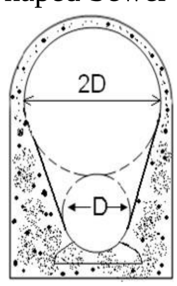
2. Egg-shared sewers

These are shown in figure. This share has got better hydraulic properties, but it is costly. Firstly due to longer perimeter more material for construction is required and secondly because of its odd shape it is difficult to construct. This sewer requires always a good foundation and proper reinforcement to make structurally stable. In India they are rarely used. They are most suitable in care of combined sewers.

The main advantage of this sewer is that it gives a slightly higher velocity during low flow, than a circular sewer of the same size.



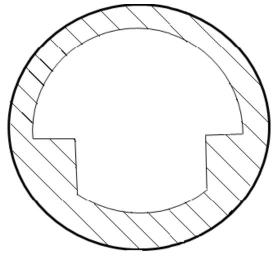
Standard Egg Shaped Sewer



New/ Modified Egg shaped Sewer

3. Basket-handle sewer.

The shape of this sewer resembles the shape of a basket handle. Small discharges flow through the bottom narrower portion. During rainy days, the combined sewage flows in the full section.



Basket-Handle Section

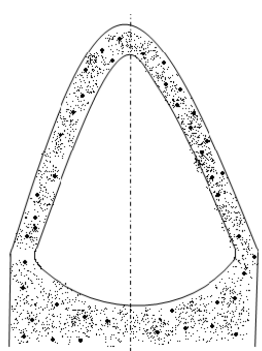
4. Horse-shoe sewers

This is as shown in fig. Its top is usually semi-circular with sides inclined or vertical. The bottom may be flat, circular or paraboloid. Its height is more than width. It is mostly used for sewers in tunnels. It is used for the construction of large sewers with heavy discharged such as trunk sewers. This shape gives increased head room.

Horse shoe sewer section

5. Parabolic sewers

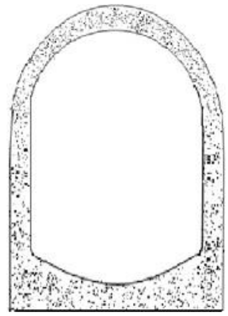
In this form of sewers, the upper arch takes the shape of parabola as shown in fig. The invert of the sewer may be flat, parabolic or elliptical. They are used for the disposal of relatively small quantities of sewage.



Parabolic section

6. Semi-circular sewers

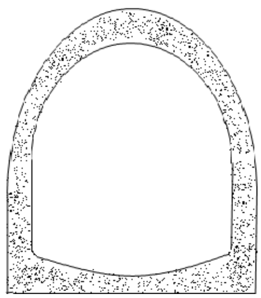
The semi-circular sewer gives a wider care at the bottom and hence, it becomes suitable for constructing large sewers with less available headroom. Now a day there are replaced by rectangular sewers.



Semi-circular Section

7. Semi-elliptical sewers

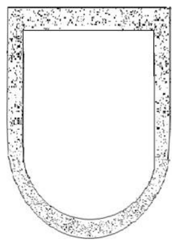
This shape of sewer is more suitable for soft soils as they are more stable. This shape is not suitable for carrying low discharges and it is normally adopted for sewers having diameter greater than 180cm or so.



Semi-elliptical section

8. U-shaped sewers

Two sections of U-shaped sewers are shown in fig. Trench provided at the bottom is called connote. These are easy to construct. Their invert may be flat or semi-circular. The sides are generally vertical and top may be flat or arched.



U-shaped section

Laying of sewers

After the sewer plan has been approved, the next step is to set out the work. The centre line of the trench is first staked out on the ground. The centre line pegs are driven at a distance of 7.5m or 15m.

The sight rail and boning rod system is the accepted method for laying the drains accurately to the gradients, indicated on the plans. Sight rails are set at all changes of gradients and at intermediate positions, if the distance for sighting is large. The sight rails are set in such a way that, the line sighted along the top edge of the rails represents, the true fall of the sewer, this gradient is shifted below the ground level by means of a Travellor of fixed length.

Sight rails are the horizontal cross rails placed on uprights. They are usually made up of a good straight piece of timber of 15cm width and 5cm thick and length to extend over the width of the trench. Travellor or boning rod contains of a rod and T-piece. It is most important that boning rod should be cut to the exact length required; otherwise the pipes may not be laid correctly to the required grade. The boning rod may be 8cm by 4cm timber piece of required length. A T-piece of 9cm by 45cm is securely fixed by nails at top (Fig 3.3).

Since the work of laying pipes is generally started from the lower end, the sight rails will therefore, be required to fix at this point. After fixing the first set of sight rails at the tower end, a second set of sight rails is similarly set at some distance upstream side. Knowing the reduced level of invert of the sewer at the lower end and the desired gradient of the sewer line, the reduced level of invert at second set of sight rail is calculated. The depth of invert below both the sight rails should be the same to obtain the desired correct gradient, because the top of sight rails are adjusted to the correct reduced levels according to the gradient required.

Testing of sewer line

It is necessary to test the sewer after its laying for water tightness before backfilling of the excaved earth.

Smoke test: - This test is performed for soil pipes, vent pipes laid above ground. The test is conducted under a pressure of 2.5m of water and maintained for 15 minutes after all trap real

have been filled with water. The smoke is produced by burning oil waste or tar paper in combustion chamber of a smoke machine.

Water test: - This test is performed for underground sewer pipes before back filling is done. The test should be carried out by suitably plugging the lower end of the drain and filling the system with water. A knuckle band shall be temporarily jointed at the top end and a sufficient length of vertical pipe is jointed so as to provide the required test head.

Subsidence of test water may be due to

(a) Absorption by pipes and joints

(b) Leakages at joints etc.

Any leakage if visible and defective part of work if any should be made good.

Test for straightness and obstruction: - For this test, a mirror is placed in front of one end of sewer and the image of the section is observed. If the sewer line is straight, the image should be circular. If it is not a complete circle, then it is not straight.

For testing for obstruction, by inserting a steel call at upper end and if there is no obstruction in the sewer line, the call will emerge out from the lower end.

Ventilation of Sewer

Sewage flowing in sewer has got lot of organic and inorganic matters present in it. Some of the matters decompose and produce gases. These gases are foul smelling, corrosive and explosive in nature. If these gases are not disposed of properly they may create a number of difficulties. They may cause air locks in sewers and affect the flow of sewage. They may prove to be dangerous for the maintenance squad working in sewers. They may also cause explosions and put the sewer line out of commission. For the disposal of these gases, ventilation of sewer line is a must.

Methods of Ventilation

Following are some of the means or fittings which help in the ventilation of sewers,

1.Laying sewer line at proper gradient.

2.Running the sewer at half full or 2/3 depth. 3.Providing manhole with gratings.

4.Proper house drainage.

5.Providing the ventilating columns or shafts.

SEWER APPURTENANCES

Sewage flowing in the sewer line contains a large number of impurities in the form of silt, fats, oils, rags etc. Under normal flows they are not likely to settle and choke the sewers, but during small flows self-cleansing velocity is not likely to develop and the chances of choking of the sewers are increased. Chokings have to be removed time to time, and facilities should be provided on the sewer lines for this purpose. Therefore, for proper functioning and to facilitate maintenance of the sewage system, various additional structures have to be constructed on the sewer lines. These structures are known as sewer appurtenances. Following are the important appurtenances,

1. Manholes
2. Inlets
3. Catch basins
4. Flushing devices
5. Regulators
6. Inverted siphons
7. Grease and oil traps
8. Lamp holes
9. Leaping weirs
10. Junction chambers

Manholes

The manholes are R.C.C or masonry chambers constructed on the sewer line to facilitate a man to enter the sewer line and make the necessary inspection and repairs. These are fitted with suitable cast iron covers. The manholes should be installed at every points where there is a change in direction, change in pipe size, or considerable change in gradient. As far as possible sewer line between two subsequent man holes should be straight. The centre distance between manholes is less for sewers of smaller size while it may behave such a size that man can easily enter in the working chamber. The minimum size is 50cm diameter.

Table gives the spacing of manholes as recommended by the Indian Standard IS: 1742–1960.

TABLE RECOMMENDED SPACING OF MANHOLES

Size of sewer	*Recommended spacing on straight reaches*
Dia. up to 0.3 m	45 m
Dia. up to 0.6 m	75 m
Dia. up to 0.9 m	90 m
Dia. up to 1.2 m	120 m
Dia. up to 1.5 m	250 m
Dia. greater than 1.5 m	300 m

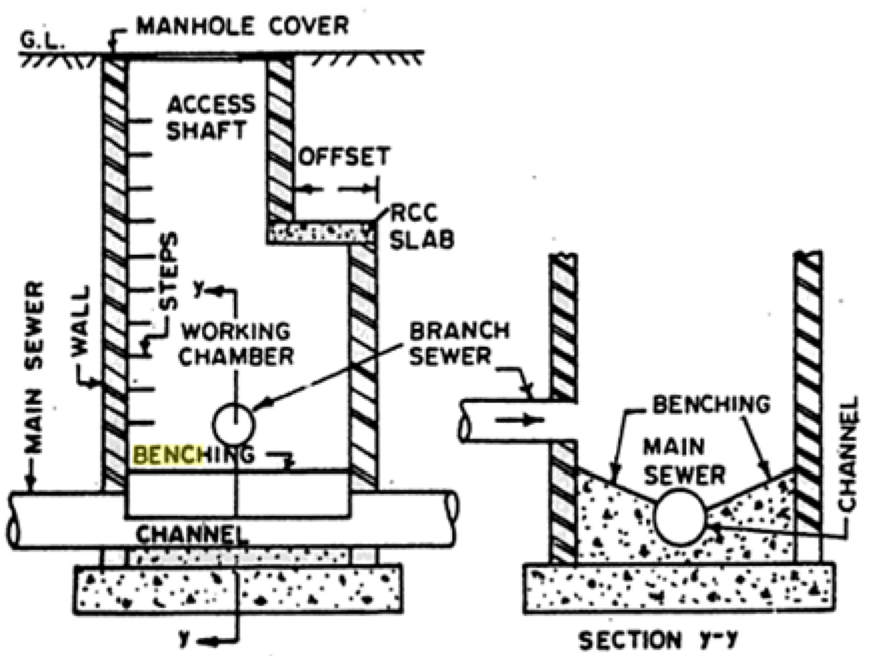


FIG. DEEP MANHOLE

The section of such manholes is not changed with depth. It is provided with heavy cover at its top.

Deep manholes are those which are deeper than 1.5 m. The size of such a manhole is larger at the bottom, which is reduced at the top to reduce the size of manhole cover. The reduction in size is achieved by providing an offset of RCC, as shown in Fig. 6.6. Steps are also provided on one vertical wall of the manholes to enable the workers to go upto the bottom. A heavy manholes cover is provided at the top, with suitable C.I. frame.

Dimensions of manholes : Table gives minimum internal dimensions for manhole chambers as recommended by Indian Standard IS 1742–1960.

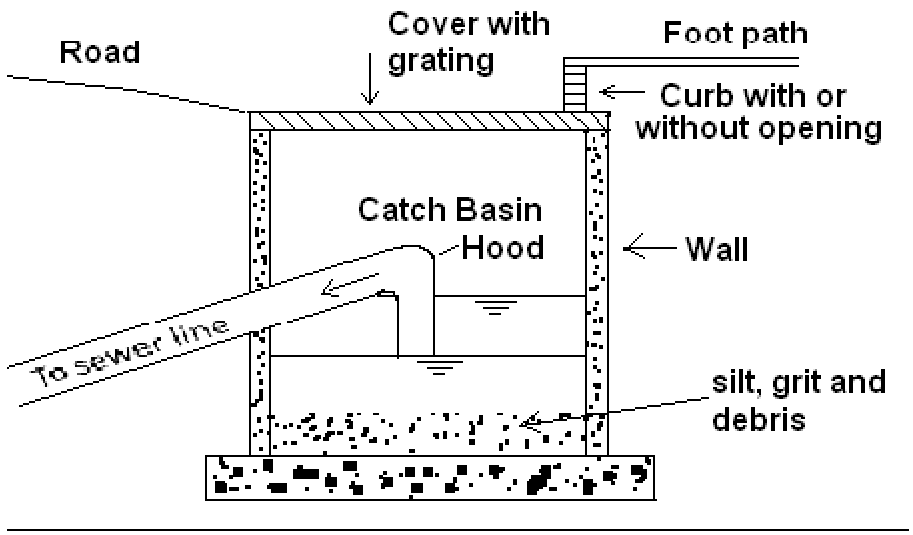
TABLE

S.N.	*Depth*	*Min. size specified*
1.	0.8 m or less	0.75 m × 0.75 m
2.	0.8 m and 2.1 m	1.2 m × 0.9 m
3.	Greater than 2.1 m	Circular chambers with min. dia. of 1.4 m ; or retanglar chamber with min. dimension of 1.2 × 0.9 m
	Min. wall thickness	
	(a) 1.5 m depth	20 cm
	(b) >1.5 m depth	30 cm

in square (1 m × 1 m) or retangular (0.8 m × 1.2 m) in cross-section.

Catch Basins

Catch basins are the structures of pucca chamber and a stout cover. They are meant for the retention of suspended grit, sludge and other heavy debris and floating rubbish from rain water which otherwise might have entered and cause choking problems. The outlet pipe from the catch basin may be submerged in order to prevent the escape of odours from the sewer and provision that also causes retention of floating matter. Their use is not recommended since they are more of a nuisance and a source of mosquito breeding apart from posing substantial maintenance problems.

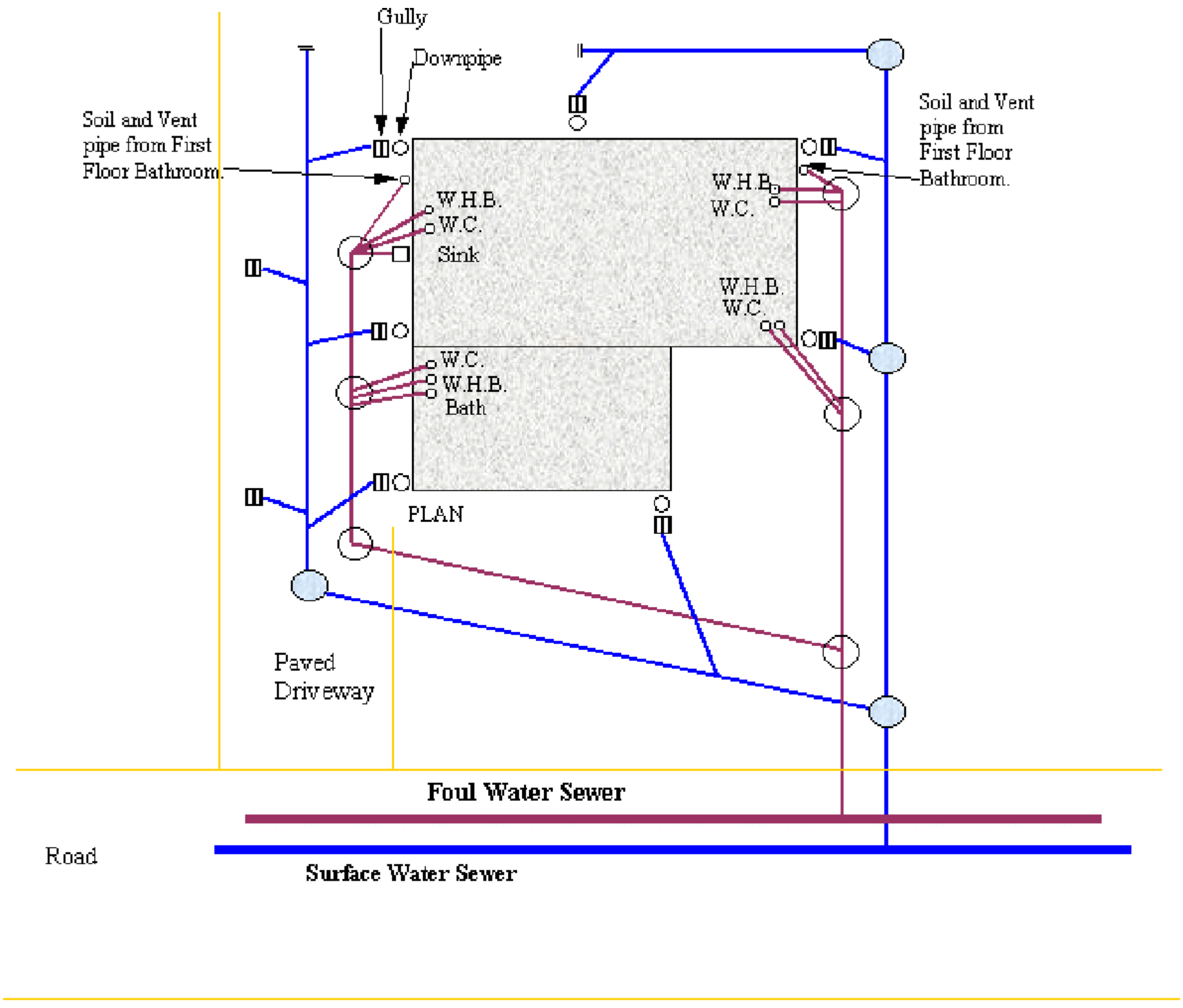


Catch Basins

Basic principles of good drainage system

1.) Material have adequate strength and durability. 2.) Diameter of drain to be as small as possible.
3.) Accessibility- every part of drain within reach for inspection and maintenance. 4.) Laid in straight runs as far as possible.
5.) Drains laid to a gradient.
6.) Inlet be trapped. To prevent entry of foul air into building.
7.) Access fittings - Inspection chamber, rodding eye, manhole - placed at changes of direction and gradient.
8.) Inspection chambers placed at junctions.
9.) Junctions between drains arranged so that incoming drain joints at oblique angle in direction of flow.
10.) Avoid drain under buildings if possible.

<u>Typical layout plan showing house drainage connections</u>



DRAINAGF LAYOUT

Maintenance of house drainage

For efficient working of the house drainage system it should be properly maintained and cleaned at regular intervals. Following points should be carefully looked at:

1. Entry of undesired elements - should take extreme precautions to avoid entry of undesired elements in the system such as grit, sand, decayed fruits, pieces of cloths, leaves, etc.
2. Flushing - advisable to flush the system once or twice in a day in order to maintain it in proper working order.
3. Inspection - various unit should be inspected at regular intervals and the obstructions if any should be removed. Damaged pipes should also be replaced.
4. Quality of materials - Better quality materials should be used

5. Use of disinfectants - Disinfectants should be freely used in the lavatory blocks, bathrooms, etc., to maintain good sanitary conditions in the building.
6. Workmanship - Laying of drains and fixing of pipes should be carried out by licensed or authorized plumbers only.

MODULE 4

Design of sewers

- After the determination of the quantity of sewage, variation in the quantity, the next step is to design the sewer section, which will be economical as well as can take the required discharge at self-cleaning velocity.

Estimate of sanitary sewage

- Sanitary sewage is mostly the spent water of the community draining into the sewer system with some ground water and a fraction into the sewer system with some ground water and a fraction of the storm runoff from the area, draining into it. The sewers should be capable of receiving the expected discharge at the end of design period. The provision however should not be much in excess of the actual discharge in the early years of its use to avoid depositions in sewers. The estimate of flow therefore requires a very careful consideration and is based upon the contributory population and the per-capita flow of sewage, both the factors being guided by the design period.

Design period

- Since it is both difficult and uneconomical to augment the capacity of the system at a later date, sewers are usually designed for the maximum expected discharge to meet the requirements of the ultimate development of the area.
  A design period of 30 years for all types of sewers is recommended.

- The future period for which the provision is made in designing the capacities of the various components of the sewerage scheme is known as the design period. The design period depends upon the following:
  - Ease and difficulty in expansion, Amount and availability of investment,
  - Anticipated rate of population growth, including shifts in communities, industries and commercial investments,
  - Hydraulic constraints of the systems designed, and
  - Life of the material and equipment.

Following design period can be considered for different components of sewerage scheme.
1. Laterals less than 15 cm diameter : Full development
2. Trunk or main sewers : 40 to 50 years
3. Treatment Units : 15 to 20 years
4. Pumping plant : 5 to 10 years

Population estimate

There are several methods for forecasting the population of a community. The most suitable approach is to base the estimation on anticipated ultimate density of population.

In case the desired information on population is not available the following densities are suggested for adoption.

Table Details of size of the town and density of population

Size of the town	Density of population per hectare
up to 5000	75-150
5000-20000	150-250
20000-50000	250-300
50000-100000	300-350
more than 100000	350-1000

Area

The tributary area for any section under consideration need to be marked on key plan. The topography, layout of buildings, legal limitations etc., determine the tributary area draining to a sewer section. The area is to be measured from the map.

Per capita sewage flow

Although the entire spent water of a community should contribute to the total flow in a sanitary sewer, it has been observed that a small portion is lost in evaporation, seepage in ground, leakage etc. Generally 80% of the water supply may be expected to reach the sewers. The sewers should be designed for a minimum of 150 lpcd.

Ground water infiltration

Estimate of flow in sanitary sewers may include certain flows due to infiltration of ground water through joints. The quantity will depend on the workmanship in lying of sewers and the height of ground water table, the material of sewer, nature of soil etc. However the following values may be assumed.

1. 5000-50000 liters/day/hectare.

2. 500-5000 liter/day/km of sewers/cm of diameter. Self-cleansing velocity

It is necessary to maintain a minimum velocity in a sewer line to ensure that suspended Solids do not deposit and cause choking troubles. Such a minimum velocity is called as self-cleansing velocity. Self-cleansing velocity is determined by considering the particle size and specific weight of the suspended solids in sewage.

The velocity which can cause automatic self-cleansing can be found out by the following formula given by Shield:

$$V = \sqrt{[(8K/f)\,((S_S-S)/S)\,g\,d]}$$

Where: f = Darcy‘s co-efficient of friction, 0.03
K = characteristics of solid particles
= 0.06 for organic and
= 0.04 for inorganic solids
S_S = specific gravity of particles
= 2.65 for inorganic and
= 1.2 for organic solids

s = specific gravity of sewage, 1.0

G = acceleration due to gravity
D = diameter of particle

As per **Badmin Lathom's** recommendations following values of self-cleansing velocities may be adopted for different sizes of sewers.

Table

Dia. of sewers in mm	Self-cleansing velocity in m/sec
150-300	1.0
300-600	0.75
>600	0.6

Non-scouring velocity

- The smooth interior surface of a sewer pipe gets scoured due to continuous abrasion caused by the suspended solids present in sewage. The velocity of flow in sewer should not be too high, as the suspended solids will cause wear to contact surface of the pipe and erode the pipe material of sewer.
- This will reduce the life of the sewer. It is, therefore, necessary to limit the maximum velocity in the sewer pipe. This limiting or non-scouring velocity will mainly depend upon the material of the sewer. The permissible maximum velocity to prevent eroding is termed as non- scouring velocity and it should be limited to 3.0 m/s.

Hydraulic formula for velocity
Empirical formulae used in design of sewers

1. Chezy‘s formula:

$$V = C\sqrt{R.S}$$

Where,

V = velocity of flow in m/sec R = hydraulic mean depth in m
S = slope of the sewer = Fall of sewer/length

C = Chezy‘s contact.

Kutter‘s formula for Chezy‘s contact is given by:

$$C = [23 + (0.00155/S) + (1/n)] / [1 + (23 + (0.00155/S)) (n/\sqrt{R})]$$

Where,

n = coefficient of roughness or rugosity factor

Table 3.3

Sl no.	Material of sewer	Value of n
1.	Brick sewers	
	a) Flush pointed	0.015
	b) Plastered smooth	0.013
2.	Stoneware sewers	0.014
3.	Cast iron	0.013
4.	Smooth earthen channel	0.020
5.	Rough channel	0.030

2. Manning's formula

$V = (1/n) R^{2/3} S^{1/2}$

3. Crimp's and Burge's formula:

$$V = 83.33 R^{2/3} S^{1/2}$$

Where V, R and S have same meaning and this formula is obtained by putting n = 0.012 in Manning's formula.

4. Bazin's formula:

$$V = 157.6 \frac{\sqrt{R.S}}{1.81 + (K/\sqrt{R})}$$

Where K is Bazin's constant

K = 3.17 for very rough channels

= 0.11 for very smooth surfaces

5. Hazen –Williams formula:

$$V = 0.85 C R^{0.6} S^{0.54}$$

Notations are same as previous; the value of C varies depending upon the surfaces.

Table

Sl no.	Type of material	Value of C
1	Stoneware pipes in good condition	110
2	New cast iron	130
3	Very smooth surfaces	140

Hydraulic Characteristics of Circular Sewer Running Full or Partially Full

Hydraulic Mean Depth, R = A/P Solving for half full sewer, R = D/4

a) Depth at Partial flow

$$d = \left[\frac{D}{2} - \frac{D}{2}\cos\left(\frac{\alpha}{2}\right)\right]$$

b) Therefore proportionate depth

$$\frac{d}{D} = \frac{1}{2}\left[1 - \cos\left(\frac{\alpha}{2}\right)\right]$$

c) Proportionate area

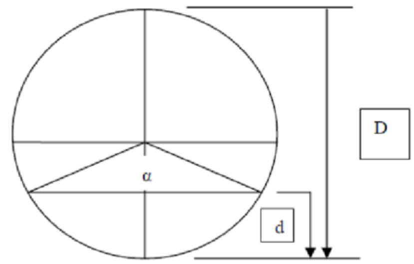
$$\frac{a}{A} = \left[\frac{\alpha}{360} - \frac{Sin\,\alpha}{2\pi}\right]$$

d) Proportionate perimeter: $\frac{p}{P} = \frac{\alpha}{360}$

f) Proportionate velocity = $\frac{v}{V} = \frac{N}{n}\frac{r^{2/3}}{R^{2/3}}$

e) Proportionate Hydraulic Mean Depth

$$\frac{r}{R} = \left[1 - \frac{360\,Sin\,\alpha}{2\pi\alpha}\right]$$



Section of a circular sewer running partially full

- In all above equations except ‗α‘everything is constant. Hence, for different values of ‗α‘, all the proportionate elements can be easily calculated. These values of the hydraulic elements can be obtained from the proportionate graph prepared for different values of d/D. The value of Manning‘s n can be considered constant for all depths. In reality it varies with the depth of flow and it may be considered variable with depth and accordingly the hydraulic elements values can be read from the graph for different depth ratio of flow.
- From the plot it is evident that the velocities in partially filled circular sewer sections can exceed those in full section and it is maximum at d/D of 0.8. Similarly, the discharge obtained is not maximum at flow full condition, but it is maximum when the depth is about 0.95 times the full depth.
- The sewers flowing with depths between 50% and 80% full need not to be placed on steeper gradients to be as self-cleansing as sewers flowing full. The reason is that velocity and discharge are function of tractive force intensity which depends upon friction coefficient as well as flow velocity generated by gradient of the sewer. Using subscript ‗s‘ denoting self-cleansing equivalent to that obtained in full section, the required ratios vs/V, qs/Q and ss/S can be computed.

DISPOSAL OF EFFLUENTS

OBJECTS OF SEWAGE DISPOSAL

- To eliminate or reduce danger to the public health by possible contamination of water supplies.
- To render the sewage inoffensive without causing odour or nuisance.
- To prevent the life of fish or other aquatic life by allowing raw sewage into bodies of water as such.
- The destruction of fish & other Aquatic life can be prevented by the swage disposal methods.
- With proper sewage disposal the environment or the areas does not
- become polluted. Sanitary conditions are maintained in the area.

There are two principal methods of sewage disposal by utilizing natural agencies i.e.,

1. Dilution i.e., disposal of sewage of water.

2. Land disposal or irrigation.

Both methods are very simple. But these may be regulated carefully so that the quantity of sewage put in water or applied to land is such that they are capable of receiving the organic load present in the effluent.

Dilution

Dilution is the disposal of sewage by discharging it into large bodies of water like sea, streams, rivers etc. This method is possible only when the natural water is available in large quantity near the town. Proper care should be taken while discharging sewage in water so that sewage may not pollute natural water and make it unfit for any other purposes like bathing, drinking, irrigation etc.

CONDITIONS FAVOURABLE FOR DILUTION

1. Where sewage is fresh.
2. Where favorable currents exits in a stream.
3. Where sewage is almost free from floating/ settleable solids.
4. Where thorough mixing is possible.
5. Where diluting water has high quantities of dissolved oxygen.
6. When the city is situated near river or sea.

Self purification phenomenon:

SELF PURIFICATION OF NATURAL STREAMS

The automatic purification of natural water is known as self purification. The self purification of natural water systems is a complex process that often involves physical, chemical, and biological processes working simultaneously. The amount of dissolved Oxygen (DO) in water is one of the most commonly used indicators of a river health. As DO drops below 4 or 5 mg/L the forms of life that can survive begin to be reduced. A minimum of about 2.0 mg/L of dissolved oxygen is required to maintain higher life forms. A number of factors affect the amount of DO available in a river. Oxygen demanding wastes remove DO; plants add DO during day but remove it at night; respiration of organisms removes oxygen. In summer, rising temperature reduces solubility of oxygen, while lower flows reduce the rate at which oxygen enters the water from atmosphere.

Factors Affecting Self Purification

1. Dilution: When sufficient dilution water is available in the receiving water body, where the waste water is discharged, the DO level in the receiving stream may not reach to zero or critical DO due to availability of sufficient DO initially in the river water before receiving discharge of wastewater.
2. Current: When strong water current is available, the discharged wastewater will be thoroughly mixed with stream water preventing deposition of solids. In small current, the solid matter from the wastewater will get deposited at the bed following decomposition and reduction in DO.
3. Temperature: The quantity of DO available in stream water is more in cold temperature than in hot temperature. Also, as the activity of microorganisms is more at the higher temperature, hence, the self-purification will take less time at hot temperature than in winter.
4. Sunlight: Algae produces oxygen in presence of sunlight due to photosynthesis. Therefore, sunlight helps in purification of stream by adding oxygen through photosynthesis.
5. Rate of Oxidation: Due to oxidation of organic matter discharged in the river DO depletion occurs. This rate is faster at higher temperature and low at lower temperature. The rate of oxidation of organic matter depends on the chemical composition of organic matter.

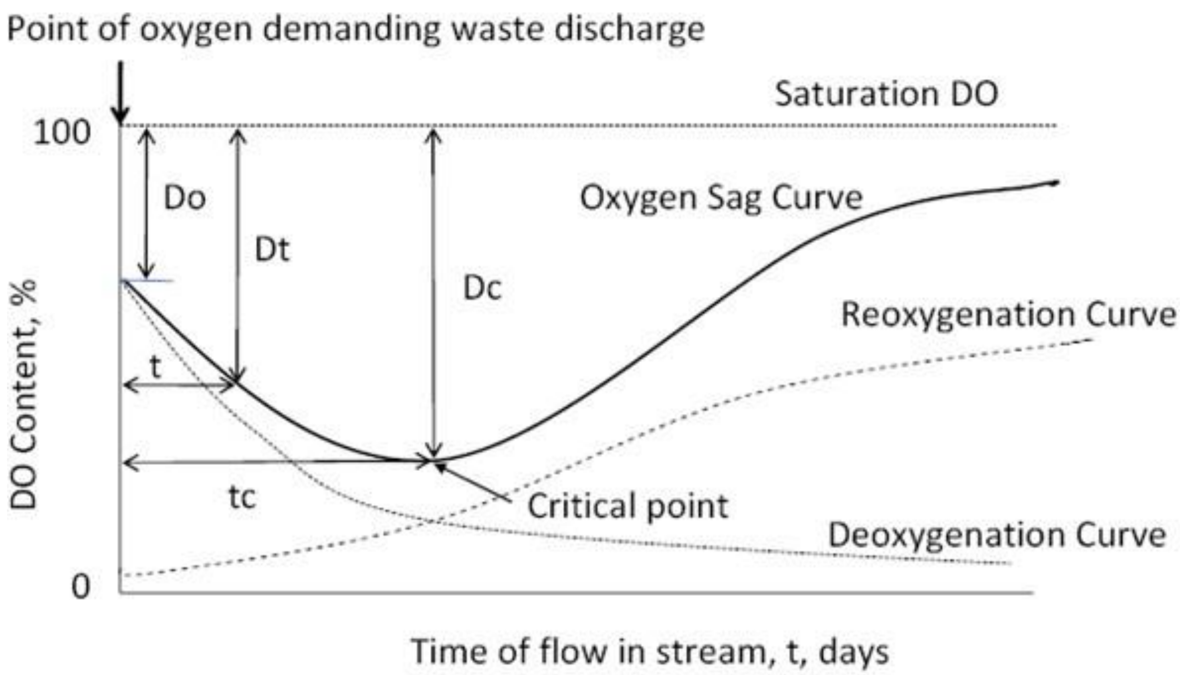
Oxygen Sag Curve

The oxygen sag or oxygen deficit in the stream at any point of time during self purification process is the difference between the saturation DO content and actual DO content at that time. The amount of resultant oxygen deflect can be obtained by algebraically adding the de - oxygenation and re - oxygenation curves. The resultant curve so obtained is called oxygen sag curve

Oxygen deficit, D = Saturation DO – Actual DO

The saturation DO value for fresh water depends upon the temperature and total dissolved salts present in it; and its value varies from 14.62 mg/L at 0oC to 7.63 mg/L at 30oC, and lower DO at higher temperatures.

The DO in the stream may not be at saturation level and there may be initial oxygen deficit ‗Do‘. At this stage, when the effluent with initial BOD load Lo, is discharged in to stream, the DO content of the stream starts depleting and the oxygen deficit (D) increases. The variation of oxygen deficit (D) with the distance along the stream, and hence with the time of flow from the point of pollution is depicted by the ‗Oxygen Sag Curve‘ . The major point in sag analysis is point of minimum DO, i.e., maximum deficit. The maximum or critical deficit (Dc) occurs at the inflexion points (as shown in fig) of the oxygen sag curve.



Deoxygenation, reoxygenation and oxygen sag curve

Deoxygenation and Reoxygenation Curves

De-oxygenation curve: The curve which represents (or) showing the depletion of D.O with time at the given temperature.

Re-oxygenation Curve: In order to counter balance the consumption of D.O due to the de – oxygenation, atmosphere supplies oxygen to the water and the process is called the re - oxygenation

When wastewater is discharged in to the stream, the DO level in the stream goes on depleting. This depletion of DO content is known as deoxygenation. The rate of deoxygenation depends upon the amount of organic matter remaining (Lt), to be oxidized at any time t, as well as temperature (T) at which reaction occurs. The variation of depletion of DO content of the stream with time is depicted by the deoxygenation curve in the absence of aeration. The ordinates below the deoxygenation curve (Figure 12.1) indicate the oxygen remaining in the natural stream after satisfying the bio-chemical demand of oxygen. When the DO content of the stream is gradually consumed due to BOD load, atmosphere supplies oxygen continuously to the water, through the process of re-aeration or reoxygenation, i.e., along with deoxygenation, re-aeration is continuous process.

The rate of reoxygenation depends upon:
i) Depth of water in the stream: more for shallow depth.
ii) Velocity of flow in the stream: less for stagnant water.
iii) Oxygen deficit below saturation DO: since solubility rate depends on difference between saturation concentration and existing concentration of DO.
iv) Temperature of water: solubility is lower at higher temperature and also saturation concentration is less at higher temperature.

Zones of Purification

When sewage is discharged into water, a succession of changes in water quality takes place. If the sewage is emptied into a lake in which currents about the outfall are sluggish and shift their direction with the wind, the changes occur in close proximity to each other and, as a result, the pattern of changes is not crisply distinguished. If, on the other hand, the water moves steadily away from the outfall, as in a stream, the successive changes occur in different river reaches and establish a profile of pollution which is well defined. However, in most streams, this pattern is by no means static. It shifts longitudinally along the stream and is modified in intensity with changes in season and hydrography.

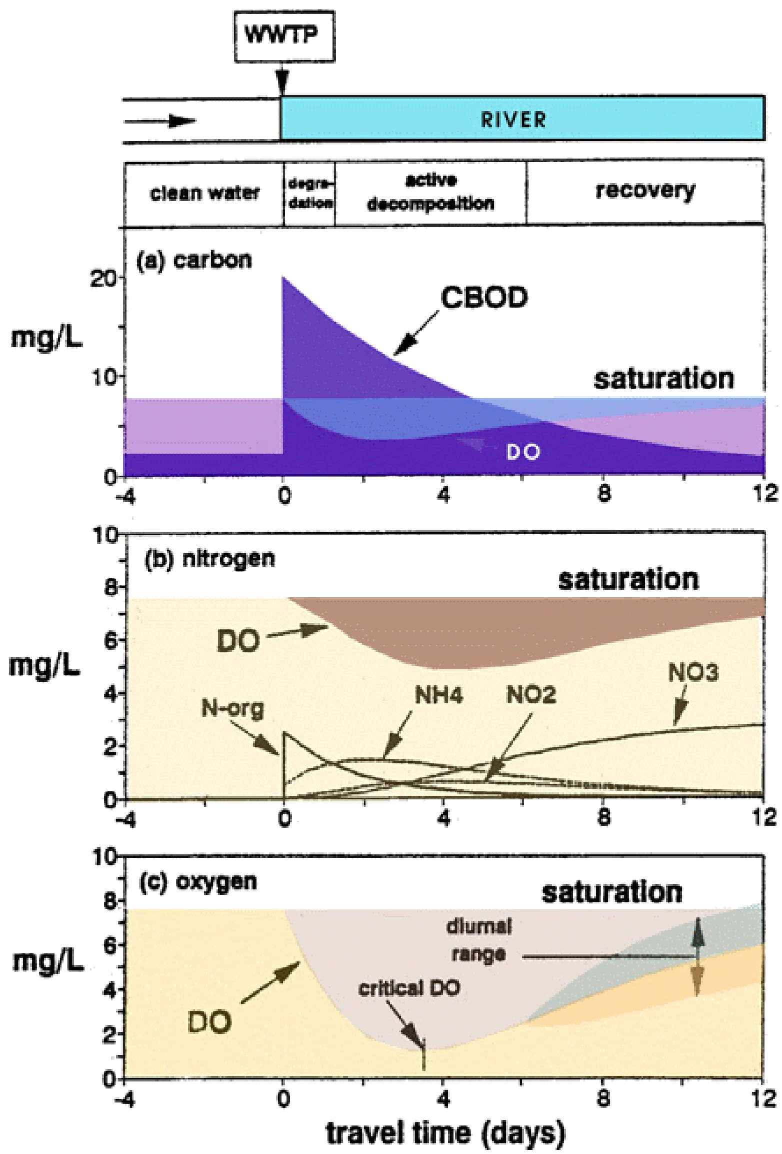
When a single large charge of sewage is poured into a clean stream, the water becomes turbid, sunlight is shut out of the depths, and green plants, which by photosynthesis remove carbon dioxide from the water and release oxygen to it, die off. Depending on the stream velocity, the water soon turns nearly black. Odorous sulfur compounds are formed and solids settle to the bottom, forming a sludge. The settled solids soon decompose, forming gases such as ammonia, carbon dioxide, and methane or marsh gas. Scavenging organisms increase in number until they match the food supply. The oxygen resources are drawn upon heavily and, when overloaded, become exhausted. Life in such waters is confined to anaerobic bacteria (which exist when no oxygen is available), larvae of certain insects such as mosquitoes, and a few worms. There are no fish; turtles are generally the only forms of higher life present. This condition is known as the **zone of degradation**.

In a second zone, or **zone of decomposition**, more solids settle out, the water becomes somewhat clearer, and sunlight penetrates the surface. Oxygen is absorbed from the atmosphere at the air-water interface permitting the establishment of aerobic (oxygen available) conditions. The aerobic bacteria continue the conversion of organic matter into nitrates, sulfates. and carbonates. These, together with the carbon dioxide produced by decomposition as well as by bacteria and plant life, are food sources. With sunlight now penetrating the water, and with abundant food, algae begin to flourish and form a green scum over the surface.

In the third zone, or **zone of recovery**, algae become more numerous and self-purification proceeds more rapidly. Green plants utilizing carbon dioxide and oxygen will liberate in the say time more oxygen than is consumed, thus hastening the recovery of the stream. Simultaneously, the fish that require little oxygen such as catfish and carp, are also found. As the dissolved oxygen increases, more types of fish appear. After recovery, in the **zone of cleaner water**, fish find the stream highly favorable, as the algae support various aquatic insects and other organisms on which fish feed. The water is clear or turbid according to concentration of algae, and may have odor for the same reason.

Throughout the stages of recovery of self-purification, disease organisms are greatly reduced in number because they lack proper food, and experience unfavorable temperatures and pH values of water. However, the water is still dangerous since all disease organisms have not perished,

RIVERS AND STREAMS



BOD of the resulting mixture:

At the outfall, BOD of the river/wastewater mixture (L_0) is given by:

$$L_0 = \frac{Q_r L_r + Q_w L_w}{Q_r + Q_w}$$

Where :

L_0 = Ultimate BOD at the point of waste discharge

Q_r = Flow in the river upstream of the discharge

L_r = Ultimate BOD of the river water

Q_w = Flow of wastewater from the discharge

L_w = Ultimate BOD in the discharged wastewater

Problem:

A wastewater effluent of 600 l/s with a BOD = 60 mg/l, DO = 2.5 mg/l and temperature of 25°C enters a river where the flow is 30 m³/sec and BOD = 3 mg/l, DO = 8.5 mg/l and temperature of 16°C. Deoxygenation constant for the waste is 0.10 per day at 20°C. The velocity of water in the river downstream is 0.15 m/s and depth of flow is 1.5 m. Determine the following after mixing of wastewater with the river water : (i) combined discharge. (ii) BOD (iii) DO, and (iv) Temperature

Wastewater discharge = 600 l/s = 0.6 m³/s.

(*i*) Combined discharge = $Q_R + Q_E = 30 + 0.6 = 30.6\ m^3/s$

(*ii*) $(BOD)_{mix} = \frac{(30 \times 3) + (0.6 \times 60)}{30 + 0.6} = 4.118$ mg/l

(*iii*) $(DO)_{mix} = \frac{(30 \times 8.5) + (0.6 \times 2.5)}{30 + 0.6} = 8.382$ mg/l

(*iv*) $(Temp.)_{mix} = \frac{(30 \times 16) + (0.6 \times 2.5)}{30 + 0.6} = 16.18°C$

Problem:

Sewage from a town is discharged into a river having a discharge of 250 lit/sec. If the quantity of sewage is 9 MLD and the BOD of sewage and river are 260 mg/l and 6 mg/l respectively, determine the BOD of the diluted water. If it is required to reduce the BOD of the diluted water to 20 mg/l, what should be the discharge in the river?

Answer:

I case

BOD of diluted water, C_{BOD} = 80.705 mg/l II case

Discharge in the river, Q_R = 154.28 MLD

Problem

Sewage from a town is discharged into a river having a discharge of 200 l/sec. If the quantity of sewage is 9 MLD and the BOD of sewage and river are 260 mg/l and 6 mg/l respectively, determine the BOD of the diluted water. If it is required to reduce the BOD of the diluted water to 20 mg/l, what should be the discharge in the river?

Answer:

River discharge, Q_R = 17.28 MLD

I. Concentration of BOD of the diluted water, C_{BOD} = 92.98mg/l

II. Discharge in the river, Q_R = 154.28 MLD

Problem:

Sewage from a town is discharged into a river having a discharge of 250 l/sec. If the quantity of sewage is 10 MLD and the BOD of sewage and river are 300 mg/l and 10 mg/l respectively, determine the BOD of the diluted water. If it is required to reduce the BOD of the diluted water to 20 mg/l, what should be the discharge in the river?

Answer:

I case

River discharge, Q_R = 21.6 MLD

BOD of diluted water, C_{BOD} = 101.77 mg/l

II case

Discharge in the river, Q_R = 280 MLD

Problem

A city discharges 100 cumecs of sewage into river, which is fully saturated with oxygen and flowing at the rate of 1500 cumecs during its lean days with a velocity of 0.1 m/s. The 5 day BOD of sewage at 20°C is 280 mg/L. Find when and where the critical DO deficit will occur in the downstream portion of river. Also find the value of critical DO deficit Assume self-purification constant of river as 4.0, coefficient of de-oxygenation as 0.1 per day at 20°C and Saturation DO = 9.2 mg/L.

Answer:

- i) DO mix = 8.625 mg/l
- ii) Initial DO deficit= 9.2 – 8.625 = 0.58 mg/l
- iii) 5 day BOD of the mix = 17.5 mg/l
- iv) Ultimate BOD of the mix = 25.59 mg/l
- v) Critical DO deficit = 4.12 mg/l
- vi) Time at which critical DO deficit occurs = 1.905 days
- vii) Distance downstream at which critical DO deficit occurs= 16.46 KM

Sewage farming

ADVANTAGES AND DISADVANTAGES OF LAND DISPOSAL

Advantages:-

1. Adds manure to land
2. Pollution of natural water courses is minimized.
3. Increase fertility of land.
4. Gives high calorific value to crops grown in sewage farms.
5. Does not require any installation of equipment involving high initial cost.
6. Crops could be grown and hence a return value is always possible to obtain.
7. Method specially suitable where large quantity of river water is not available at all times of the year.

DISADVANTAGES:-

1. Difficult to get land during rainy and harvest seasons.
2. Additional land is required for reserve.
3. Sanitary reasons may not permit growing of crops on sewage farms.
4. More land area is required is sewage volume is greater since land capacity is limited.
5. If all precautions are not taken, sewage farming results in sewage sickness to land and health to life.

METHOD OF LAND TREATMENT

Sewage mainly contains water which can be used for irrigation purposes . The fertilising value of sewage is more because it contains n, potash & phosphate .The sewage can be applied in the following forms

- BROAD IRRIGATION
- SEWAGE FARMING.

BROAD IRRIGATION

In this method, sewage is allowed to flow over cultivated lands, from which a part of the sewage evaporates, some percolates and the rest escape into surface drainage channels. Sewage waters the land and adds to its fertilizing value, due to the presence of nitrogen, phosphorus, potash etc. These fertilizing elements of sewage are consumed by the roofs of crops. Crops like cotton, potatoes, sugarcane, grass etc, can be profitably grown. This is also called sewage farming.

SEWAGE FARMING

The process in which sewage is used for growing crops is known as sewage farming .The fertilising elements of sewage i. e nitrates, sulphates, & phosphates are used by the roots of crops. The nutrients of sewage make the fields fertile .It is a profitable business & a good income can be generated by sewage farming.

APPLICATION OF SEWAGE METHODS

- FLOODING METHOD
- SURFACE IRRIGATION METHOD
- ZIG ZAG METHOD
- LAGOONING METHOD
- BASIN METHOD
- SUB-SOIL IRRIGATION METHOD
- RIDGE AND FERROW IRRIGATION METHOD.

FLOODING:- The area to be irrigated is divided into various parts surrounded by dykes. The sewage is filled like small ponds in between the dykes. The depth of dose varies from 3.0 cms. To 5.0 cms. Depending on the irrigation requirements.

SURFACE IRRIGATION:- This method is most suited in sloping area. Here, parallel drains are constructed in the fields. All these drains are connected to a distributaries drain with the help of regulating device so that sewage may flow in the require drain. Here when sewage flows over the fields, its large quantity is absorbed by the field and only excess quantity reaches another drain.

ZIG ZAG METHOD: In this method the ridges are arranged in a zag-zag method with corresponding furrows by their side

LAGOONING: These are used cheaply for sewage disposal. In this method the sewage is allowed to in a natural depression available or artificial constructed tanks. Detention period is about a month. During this period the sludge is stabilized and dried. The purified effluent passes way from an outlet placed at the other end. Lagoons should be shallow and must be constructed away from the town.

SUB SURFACE IRRIGATION: Here sewage is applied at the roofs of plants, through the open jointed agricultural drain-pipes. These pipes are laid about 1.0 m below the ground level. The sewage rises up due to capillary action. Here soil takes fewer loads but this is an economical method.

BASIN METHOD: In this method big trees are planted in an isolated manner, basins are formed around each tree. These basins are filled with sewage. This method is suitable for fruit gardens.

RIDGE AND FERROW IRRIGATION: In this method , sewage is supplied in furrows between crop rows. Sewage spreads laterally irrigating the area between two furrows. The width of furrow varies from 120-150 cm and the depth from 25-50 cm. The width of the ridge varies from 125-250 cm and length from 10-30 m. The percolated effluent is collected in underground drains flows towards natural drainage for disposal.

Comparison of dilution & land treatment method of disposal of sewage

S.No.	Disposal by Dilution	Disposal by Land Treatment
1.	Large volumes of water are required.	Large area of pervious soils are required.
2.	Income can't be generated.	Income can be generated by adopting sewage farming.
3.	It requires full or partial treatment.	It requires preliminary or primary treatment.
4.	It causes stream pollution and an unavoidable health hazard.	It saves the inland streams from getting polluted by sewage.
5.	The pollution travels down stream and thus the stream may lose its utility for various purposes such as bathing, recreation etc.	It returns to the land the fertilising element.
6.	It requires no or low head pumping as the stream passes through lowest contours and valleys.	It requires high head pumping therefore the cost of pumping is high.

FAVOURABLE CONDITIONS FOR SUB-SURFACE IRRIGATION

1. Subsoil water level is low.
2. Land is cheap.
3. Rainfall is less and irrigation water demand is heavy.
4. Large areas are available.
5. Where dilution water is not easily available.
6. Sub-surface strata is porous, favoring infiltration.
7. Climate is dry favoring drying up conditions.

SEWAGE SICKNESS

The phenomena of soil getting clogged and loses its capacity of receiving the sewage load when the sewage is applied continuously on a piece of land is called **sewage sickness**

PREVENTION OF SEWAGE SICKNESS:-

- Primary treatment like screening & sedimentation should be given to sewage before its application to land so that suspended solids are removed & the pores of soil will not be clogged.
- The sewage should be applied intermittently on land i.e by giving rest to the land for sometime .The land should be ploughed during non supply period of sewage so that soil gets aerated.
- Keeping some portion of land reserved in order to use the same in resting period .Enough area will be required for this purpose.
- By planting different crops on the same land by rotation system of crops .The soil will be aerated & will utilise the fertilizing elements of sewage.
- By providing sufficient under drainage system to collect the excessive sewage quantity.
- By frequent ploughing & rotation of soil.
- By not applying the sewage in excess quantity.

Problem:

. A town, having a population of 50000 and the rate of water supply as 160 l/day, disposes off its sewage successfully by land treatment. The area of land available is 180 hectares. If 80% of the water supplied is converted into sewage, find out the consuming capacity of soil.

Solution. Quantity of sewage $= 0.8 \times 50000 \times 160$

$= 6400 \times 10^3$ litres $= 6400\ m^3/day$

Total area of land $= 180$ hectares

Providing 50% exta area as reserve for rest and rotation,

Actual available area for land application $= \frac{180}{1.5} = 120$ hectares.

$\therefore$ Consuming capacity of soil $= \frac{6400}{120}$

$=$ **53.33** cubic metres/hectare/day.

Problem:

A town with a population of 50,000 is provided with 180 lpcd of water. If no ground water is likely to enter sewers and 80% of water supplied goes as sewage, how much land including 50% reserve will be required for continuous broad irrigation, if the permitted loading is 1,34,000 l/d hec.

Answer:

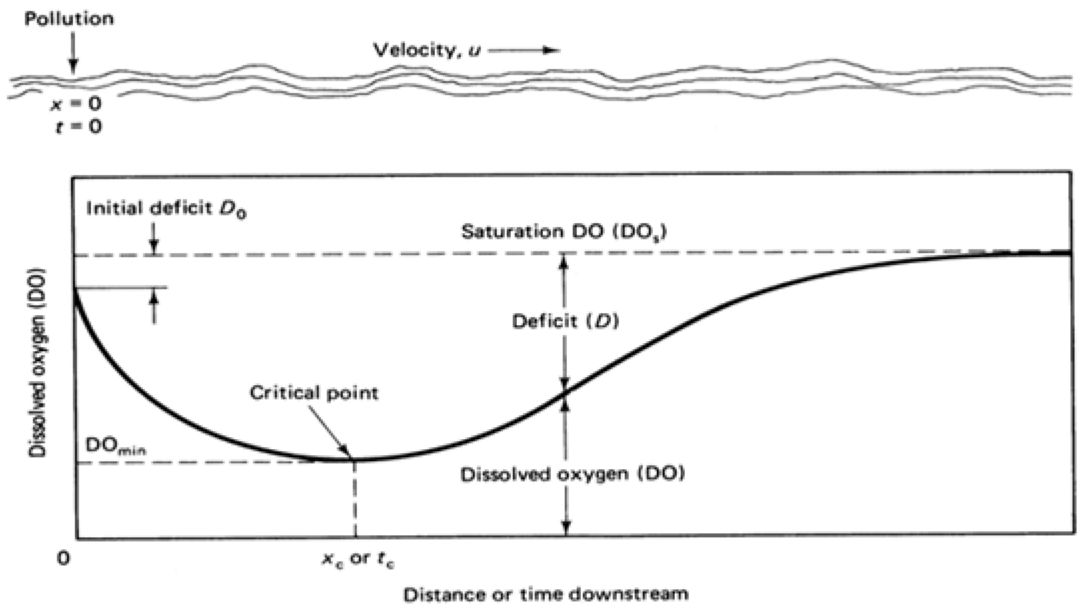
Sewage flow, $Q_s = 7.2$ MLD

Land area required = 53.73 hectares

50% reserve area = 26.86 hectares

Total land area required = 80.59 hectares

Streeter Phelps Equation



Streeter–Phelps oxygen-sag curve.

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1. DO content is one of the most widely used indicators of overall ecological health of a body of water
    - fish need 4 to 5 mg/L to survive
    - under anaerobic conditions, undesirable (smelly) microbes can take over
    - many factors affect the DO level
2. If a river was healthy before we began discharging wastewater, a significant factor in its continued health or illness is the BOD added to it by wastewater.
    a. At the outfall, BOD of the river/wastewater mixture (L_0) is given by:

$$L_0 = \frac{Q_r L_r + Q_w L_w}{Q_r + Q_w}$$

Where :

L_0 = Ultimate BOD at the point of waste discharge

Q_r = Flow in the river upstream of the discharge

L_r = Ultimate BOD of the river water

Q_w = Flow of wastewater from the discharge

L_w = Ultimate BOD in the discharged wastewater

b. BOD is comparable to what we have in our stoppered bottle at the beginning of our BOD test

- As time passes (ie, the water moves downstream) the oxygen content of the river water is consumed in just the same way oxygen is consumed in the test
- BOD (L_t) in a test bottle at time t is given by:

$$L_t = L_0 e^{-kDt}$$

- This formula holds in the river too (k_D is the deoxygenation constant; it can be adjusted for temperature using $k_T = k_{20}\theta^{T-20}$)
- Knowing an average velocity of flow, we can calculate the BOD for a given distance downstream

DO remaining depends both on the rate of deoxygenation and on the rate of reoxygenation or reaeration

The DO at any point downstream depends on these competing processes: rate of deficit increase = rate of deoxygenation - rate of reaeration

- This gives a differential equation with the solution:

$$D = \frac{k_D L_0}{k_R - k_D}\left(e^{-k_D t} - e^{-k_R t}\right) + D_0 e^{-k_R t}$$

- This is the Streeter-Phelps oxygen-sag curve formula
- Note that for a constant stream cross-section, t=x/u (with u=stream velocity); therefore:

$$D = \frac{k_D L_0}{k_R - k_D}\left(e^{-k_D x/u} - e^{-k_R x/u}\right) + D_0 e^{-k_R x/u}$$

to plot DO versus distance downstream we need to subtract D from D_s at each point

To start with, DO is being depleted faster than it can be replenished

- As long as this occurs, the DO of the stream will continue to drop
- Since the BOD is decreasing as time goes on, at some point, the rate of deoxygenation decreases to just the rate of reaearation
  - At this point (called the critical point) the DO reaches a minimum
  - Downstream of the critical point, reaeration occurs faster than deoxygenation, so the DO increases

- Using calculus and the

Streeter-Phelps equation we get: Critical Time at

$$t_c = \frac{1}{k_R - k_D} \ln\left(\frac{k_R}{k_D}\left[1 - \frac{D_0(k_R - k_D)}{k_D L_0}\right]\right)$$

which Max. DO deficit occurs , t_c

Problem:

100 cumece of sewage of a city is discharged in a perennial river which is fully saturated with oxygen and flows at a minimum rate of 1250 cumecs with a minimum velocity of 0.15 m/sec. If the 5-day BOD of the sewage is 260 mg/l, find out where the critical DO will occur in the river. Assume :

(i) The coefficient of purification of river as 4.0

(ii) Coefficient of DO as 0.11

and (iii) The ultimate BOD as 125% of the 5 day BOD of the mixture of sewage and river water.

Let us assume a temperature of 20° C, for which saturation DO is equal to 9.17 mg/l. Also, assume that DO of effluent is zero.

$$\therefore \quad (DO)_{mix} = \frac{(1250 \times 9.17) + (100 \times 0)}{1250 + 100} = 8.49 \text{ mg/l.}$$

Hence initial DO deficit $= D_0 = 9.17 - 8.49 = 0.68$ mg/l.

Also, 5-day BOD of sewage = 260 mg/l.

Let 5-day BOD of river = 0

$$\therefore \quad (y_5)_{mix} = \frac{(1250 \times 0) + (100 \times 260)}{1250 + 100} = 19.26 \text{ mg/l.}$$

It is given that ultimate BOD is equal to 125% of the 5-day BOD of the mixture of sewage and river water.

$$\therefore \quad L_0 = 1.25\,(y_5)_{mix} = 1.25 \times 19.26 = 24.07 \text{ mg/l.}$$

Thus, both D_0 and L_0 are known. Now, from Eq.

$$t_c = \frac{1}{K(f_s - 1)} \log_{10}\left[f_s\left\{1 - (f_s - 1)\frac{D_0}{L_0}\right\}\right]$$

Here, $K = 0.11$ (given) and $f_s = 4.0$ (given)

$$\therefore \quad t_c = \frac{1}{0.11(4-1)} \log_{10}\left[4\left\{1 - (4-1)\frac{0.68}{24.07}\right\}\right] \doteq 1.7078$$

$$\therefore \quad x_c = \text{velocity} \times \text{time} = 0.15\,(1.7078 \times 24 \times 3600) \times 10^{-3}$$

$$= 22.13 \text{ km}$$

Hence critical deficit will occur at 22.13 km downstream of the sewage disposal point.

Problem:

A town discharges 100 m^3/sec of sewage into a stream having a rate of flow of 1200 m^3/sec during lean days, at a 5 day BOD of sewage at the given temperature being 250 mg/l. Find the amount of critical D.O deficit and its location in the downstream portion of the stream. Assume de-oxygenation co-efficient K as 0.1 and co- efficient of self-purification as 3.5. Take saturation dissolved oxygen at given temperature as 9.2 mg/l.

MODULE 5

Wastewater sampling, significance and techniques

Two types of sampling techniques are used: Grab Sampling, Composite Sampling, Integrated Sampling.

A. Grab samples: Grab samples are single collected at a specific spot at a site over a short period of time (typically seconds or minutes). Thus, they represent a ―snapshot in both space and time of a sampling area. Discrete grab samples are taken at a selected location, depth, and time. Depth-integrated grab samples are collected over a predetermined part of the entire depth of a water column, at a selected location and time in a given body of water.

Grab samples consist of either a single discrete sample or individual samples collected over a period of time not to exceed 15 minutes. The grab sample should be representative of the wastewater conditions at the time of sample collection. The sample volume depends on the type and number of analyses to be performed.

A sample can represent only the composition of its source at the time and place of collection. However, when a source is known to be relatively constant in composition over an extended time or over substantial distances in all directions, then the sample may represent a longer time period and/or a larger volume than the specific time and place at which it was collected. In such circumstances, a source may be represented adequately by single grab samples. Examples are protected groundwater supplies, water supplies receiving conventional treatment, some well- mixed surface waters, but rarely, wastewater streams, rivers, large lakes, shorelines, estuaries, and groundwater plumes.

When a source is known to vary with time, grab samples collected at suitable intervals and analyzed separately can document the extent, frequency, and duration of these variations. Choose sampling intervals on the basis of the expected frequency of changes, which may vary from as little as 5 min to as long as 1h or more. Seasonal variations in natural systems may necessitate sampling over months. When the source composition varies in space (i.e. from location to location) rather than time, collect samples from appropriate locations that will meet the objectives of the study (for example, upstream and downstream from a point source, etc.).

B. Composite samples: Composite samples should provide a more representative sampling of heterogeneous matrices in which the concentration of the analytes of interest may vary over short periods of time and/or space. Composite samples can be obtained by combining portions of multiple grab samples or by using specially designed automatic sampling devices. Sequential (time) composite samples are collected by using continuous, constant sample pumping or by mixing equal water volumes collected at regular time intervals.

Flow-proportional composites are collected by continuous pumping at a rate proportional to the flow, by mixing equal volumes of water collected at time intervals that are inversely proportional to the volume of flow, or by mixing volumes of water proportional to the flow collected during or at regular time intervals.

Advantages of composite samples include reduced costs of analyzing a large number of samples, more representative samples of heterogeneous matrices, and larger sample sizes when amounts of test samples are limited. Disadvantages of composite samples include loss of analyte relationships in individual samples, potential dilution of analytes below detection levels, increased potential analytical interferences, and increased possibility of analyte interactions. In addition, use of composite samples may reduce the number of samples analyzed below the required statistical need for specified data quality objectives or project-specific objectives.

Do not use composite samples with components or characteristics subject to significant and unavoidable changes during storage. Analyze individual samples as soon as possible after collection and preferably at the sampling point. Examples are dissolved gases, residual chlorine, soluble sulfide, temperature, and pH. Changes in components such as dissolved oxygen or carbon dioxide, pH, or temperature may produce secondary changes in certain inorganic constituents such as iron, manganese, alkalinity, or hardness. Some organic analytes also may be changed by changes in the foregoing components. Use time-composite samples only for determining components that can be demonstrated to remain unchanged under the conditions of sample collection, preservation, and storage.

Collect individual portions in a wide-mouth bottle every hour (in some cases every half hour or even every 5 min) and mix at the end of the sampling period or combine in a single bottle as collected. If preservatives are used, add them to the sample bottle initially so that all portions of the composite are preserved as soon as collected.

c.Integrated (discharge-weighted) samples: For certain purposes, the information needed is best provided by analyzing mixtures of grab samples collected from different points simultaneously, or as nearly so as possible, using discharge-weighted methods such as equal-width increment (EWI) or equal discharge-increment (EDI) procedures and equipment. An example of the need for integrated sampling occurs in a river or stream that varies in composition across its width and depth. To evaluate average composition or total loading, use a mixture of samples representing various points in the cross-section, in proportion to their relative flows. The need for integrated samples also may exist if combined treatment is proposed for several separate wastewater streams, the interaction of which may have a significant effect on treatability or even on composition. Mathematical prediction of the interactions among chemical components may be inaccurate or impossible and testing a suitable integrated sample may provide useful information.

Both lakes and reservoirs show spatial variations of composition (depth and horizontal location). However, there are conditions under which neither total nor average results are especially useful, but local variations are more important. In such cases, examine samples separately (i.e., do not integrate them).

Preparation of integrated samples usually requires equipment designed to collect a sample water uniformly across the depth profile. Knowledge of the volume, movement, and composition of the various parts of the water being sampled usually is required. Collecting integrated samples is a complicated and specialized process that must be described in a sampling plan.

Effluent disposal standards for land, surface and ocean

GENERAL STANDARDS FOR DISCHARGE OF ENVIRONMENTAL POLLUTANTS PART-A : EFFLUENTS

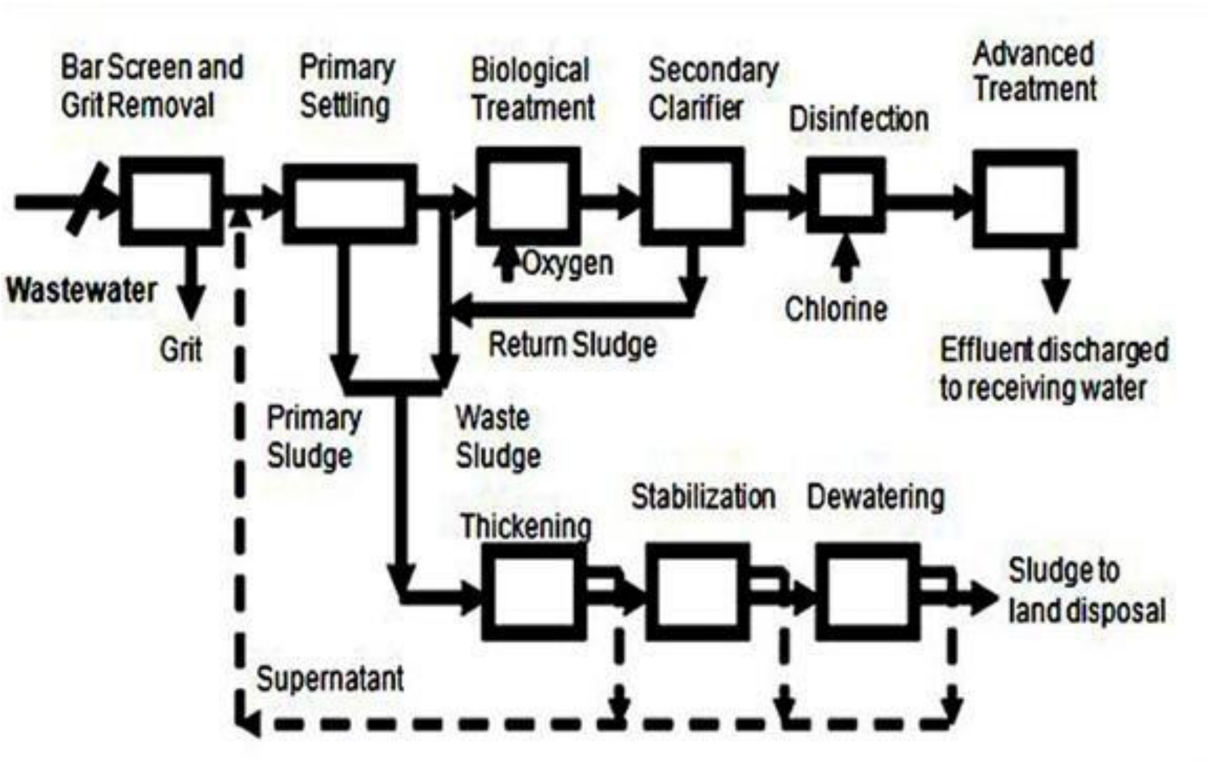
S. No.	Parameter	Standards			
		Inland surface water	Public Sewers	Land for irrigation	Marine coastal areas
1	2	3			
		(a)	(b)	(c)	(d)
1.	Colour and odour	See 6 of Annexure-I	--	See 6 of Annexure-I	See 6 of Annexure-I
2.	Suspended solids mg/l, Max.	100	600	200	(a) For process waste water- 100 (b) For cooling water effluent 10 percent above total suspended matter of influent.
3.	Particulate size of suspended solids	Shall pass 850 micron IS Sieve	--	--	(a) Floatable solids, max. 3 mm. (b) Settleable solids, max. 850 microns.
24.	***	*	--	***	--
5.	pH Value	5.5 to 9.0	5.5 to 9.0	5.5 to 9.0	5.5 to 9.0
6.	Temperature	shall not exceed 5oC above the receiving water temperature	--	--	shall not exceed 5oC above the receiving water temperature
7.	Oil and grease mg/l Max.	10	20	10	20
8.	Total residual chlorin mg/l Max.	1.0	--	--	1.0
9.	Ammonical nitrogen (as N), mg/l Max.	50	50	--	50

10.	Total Kjeldahl Nitrogen (as NH3) mg/l, Max.	100	--	--	100
11.	Free ammonia (as NH3) mg/l, Max.	5.0	--	--	5.0
12.	Biochemical Oxygen demand 1[3 days at 27oC] mg/l max.	30	350	100	100
13.	Chemical Oxygen Demand, mg/l, max.	250	--	--	250
14.	Arsenic (as As), mg/l, max.	0.2	0.2	0.2	0.2
15.	Mercury (as Hg), mg/l, Max.	0.01	0.01	--	0.01
16.	Lead (as Pb) mg/l, Max.	0.1	1.0	--	2.0
17.	Cadmium (as Cd) mg/l, Max.	2.0	1.0	--	2.0
18.	Hexavalent Chromium (as Cr+6), mg/l max.	0.1	2.0	--	1.0

Unit-V

TREATMENT OF WASTE WATER

Flow diagram of Municipal waste water treatment plant



- The principal objective of wastewater treatment is generally to allow human and industrial effluents to be disposed of without danger to human health or unacceptable damage to the natural environment. Irrigation with wastewater is both disposal and utilization and indeed is an effective form of wastewater disposal (as in slow-rate land treatment). However, some degree of treatment must normally be provided to raw municipal wastewater before it can be used for agricultural or landscape irrigation or for aquaculture.
- The quality of treated effluent used in agriculture has a great influence on the operation and performance of the wastewater-soil-plant or aquaculture system. In the case of irrigation, the required quality of effluent will depend on the crop or crops to be irrigated, the soil conditions and the system of effluent distribution adopted. Through crop restriction and selection of irrigation systems which minimize health risk, the degree of pre-application wastewater treatment can be reduced. A similar approach is not feasible in aquaculture systems and more reliance will have to be placed on control through wastewater treatment.
- The most appropriate wastewater treatment to be applied before effluent use in agriculture is that which will produce an effluent meeting the recommended microbiological and chemical quality guidelines both at low cost and with minimal operational and maintenance requirements. Adopting as low a level of treatment as possible is especially desirable in developing countries, not only from the point of view of cost but also in acknowledgement of the difficulty of operating complex systems reliably. In many locations it will be better to design the reuse system to accept a low-grade of effluent rather than to rely on advanced treatment processes producing a reclaimed effluent which continuously meets a stringent quality standard.

- Nevertheless, there are locations where a higher-grade effluent will be necessary and it is essential that information on the performance of a wide range of wastewater treatment technology should be available. The design of wastewater treatment plants is usually based on the need to reduce organic and suspended solids loads to limit pollution of the environment. Pathogen removal has very rarely been considered an objective but, for reuse of effluents in agriculture, this must now be of primary concern and processes should be selected and designed accordingly. Treatment to remove wastewater constituents that may be toxic or harmful to crops, aquatic plants (macrophytes) and fish is technically possible but is not normally economically feasible. Unfortunately, few performance data on wastewater treatment plants in developing countries are available and even then they do not normally include effluent quality parameters of importance in agricultural use.
- The short-term variations in wastewater flows observed at municipal wastewater treatment plants follow a diurnal pattern. Flow is typically low during the early morning hours, when water consumption is lowest and when the base flow consists of infiltration-inflow and small quantities of sanitary wastewater. A first peak of flow generally occurs in the late morning, when wastewater from the peak morning water use reaches the treatment plant, and a second peak flow usually occurs in the evening.
- The relative magnitude of the peaks and the times at which they occur vary from country to country and with the size of the community and the length of the sewers. Small communities with small sewer systems have a much higher ratio of peak flow to average flow than do large communities. Although the magnitude of peaks is attenuated as wastewater passes through a treatment plant, the daily variations in flow from a municipal treatment plant make it impracticable, in most cases, to irrigate with effluent directly from the treatment plant. Some form of flow equalization or short-term storage of treated effluent is necessary to provide a relatively constant supply of reclaimed water for efficient irrigation, although additional benefits result from storage.

Preliminary treatment

The objective of preliminary treatment is the removal of coarse solids and other large materials often found in raw wastewater. Removal of these materials is necessary to enhance the operation and maintenance of subsequent treatment units. Preliminary treatment operations typically include coarse screening, grit removal and, in some cases, comminution of large objects. In grit chambers, the velocity of the water through the chamber is maintained sufficiently high, or air is used, so as to prevent the settling of most organic solids. Grit removal is not included as a preliminary treatment step in most small wastewater treatment plants. Comminutors are sometimes adopted to supplement coarse screening and serve to reduce the size of large particles so that they will be removed in the form of a sludge in subsequent treatment processes. Flow measurement devices, often standing-wave flumes, are always included at the preliminary treatment stage.

The objective of primary treatment is the removal of settleable organic and inorganic solids by sedimentation, and the removal of materials that will float (scum) by skimming. Approximately 25 to 50% of the incoming biochemical oxygen demand (BOD_5), 50 to 70% of the total suspended solids (SS), and 65% of the oil and grease are removed during primary treatment. Some organic nitrogen, organic phosphorus, and heavy metals associated with solids are also removed during primary sedimentation but colloidal and dissolved constituents are not affected. The effluent from primary sedimentation units is referred to as primary effluent.

In many industrialized countries, primary treatment is the minimum level of pre application treatment required for wastewater irrigation. It may be considered sufficient treatment if the wastewater is used to irrigate crops that are not consumed by humans or to irrigate orchards, vineyards, and some processed food crops. However, to prevent potential nuisance conditions in storage or flow-equalizing reservoirs, some form of secondary treatment is normally required in these countries, even in the case of non-food crop irrigation. It may be possible to use at least a portion of primary effluent for irrigation if off-line storage is provided.

Primary sedimentation tanks or clarifiers may be round or rectangular basins, typically 3 to 5 m deep, with hydraulic retention time between 2 and 3 hours. Settled solids (primary sludge) are normally removed from the bottom of tanks by sludge rakes that scrape the sludge to a central well from which it is pumped to sludge processing units. Scum is swept across the tank surface by water jets or mechanical means from which it is also pumped to sludge processing units.

In large sewage treatment plants (> 7600 m^3/d in the US), primary sludge is most commonly processed biologically by anaerobic digestion. In the digestion process, anaerobic and facultative bacteria metabolize the organic material in sludge (see Example 3), thereby reducing the volume requiring ultimate disposal, making the sludge stable (non-putrescible) and improving its dewatering characteristics. Digestion is carried out in covered tanks (anaerobic digesters), typically 7 to 14 m deep. The residence time in a digester may vary from a minimum of about 10 days for high-rate digesters (well-mixed and heated) to 60 days or more in standard-rate digesters. Gas containing about 60 to 65% methane is produced during digestion and can be recovered as an energy source. In small sewage treatment plants, sludge is processed in a variety of ways including: aerobic digestion, storage in sludge lagoons, direct application to sludge drying beds, in-process storage (as in stabilization ponds), and land application.

Secondary treatment

The objective of secondary treatment is the further treatment of the effluent from primary treatment to remove the residual organics and suspended solids. In most cases, secondary treatment follows primary treatment and involves the removal of biodegradable dissolved and colloidal organic matter using aerobic biological treatment processes. Aerobic biological treatment (see Box) is performed in the presence of oxygen by aerobic microorganisms (principally bacteria) that metabolize the organic matter in the wastewater, thereby producing more microorganisms and inorganic end-products (principally CO_2, NH_3, and H_2O). Several aerobic biological processes are used for secondary treatment differing primarily in the manner in which oxygen is supplied to the microorganisms and in the rate at which organisms metabolize the organic matter.

High-rate biological processes are characterized by relatively small reactor volumes and high concentrations of microorganisms compared with low rate processes. Consequently, the growth rate of new organisms is much greater in high-rate systems because of the well controlled environment. The microorganisms must be separated from the treated wastewater by sedimentation to produce clarified secondary effluent. The sedimentation tanks used in secondary treatment, often referred to as secondary clarifiers, operate in the same basic manner as the primary clarifiers described previously. The biological solids removed during secondary sedimentation, called secondary or biological sludge, are normally combined with primary sludge for sludge processing.

Common high-rate processes include the activated sludge processes, trickling filters or bio-filters, oxidation ditches, and rotating biological contactors (RBC). A combination of two of these processes in series (e.g., bio-filter followed by activated sludge) is sometimes used to treat municipal wastewater containing a high concentration of organic material from industrial sources.

i. Activated Sludge

In the activated sludge process, the dispersed-growth reactor is an aeration tank or basin containing a suspension of the wastewater and microorganisms, the mixed liquor. The contents of the aeration tank are mixed vigorously by aeration devices which also supply oxygen to the biological suspension . Aeration devices commonly used include submerged diffusers that release compressed air and mechanical surface aerators that introduce air by agitating the liquid surface. Hydraulic retention time in the aeration tanks usually ranges from 3 to 8 hours but can be higher with high BOD_5 wastewaters. Following the aeration step, the microorganisms are separated from the liquid by sedimentation and the clarified liquid is secondary effluent. A portion of the biological sludge is recycled to the aeration basin to maintain a high mixed-liquor suspended solids (MLSS) level. The remainder is removed from the process and sent to sludge processing to maintain a relatively constant concentration of microorganisms in the system. Several variations of the basic activated sludge process, such as extended aeration and oxidation ditches, are in common use, but the principles are similar.

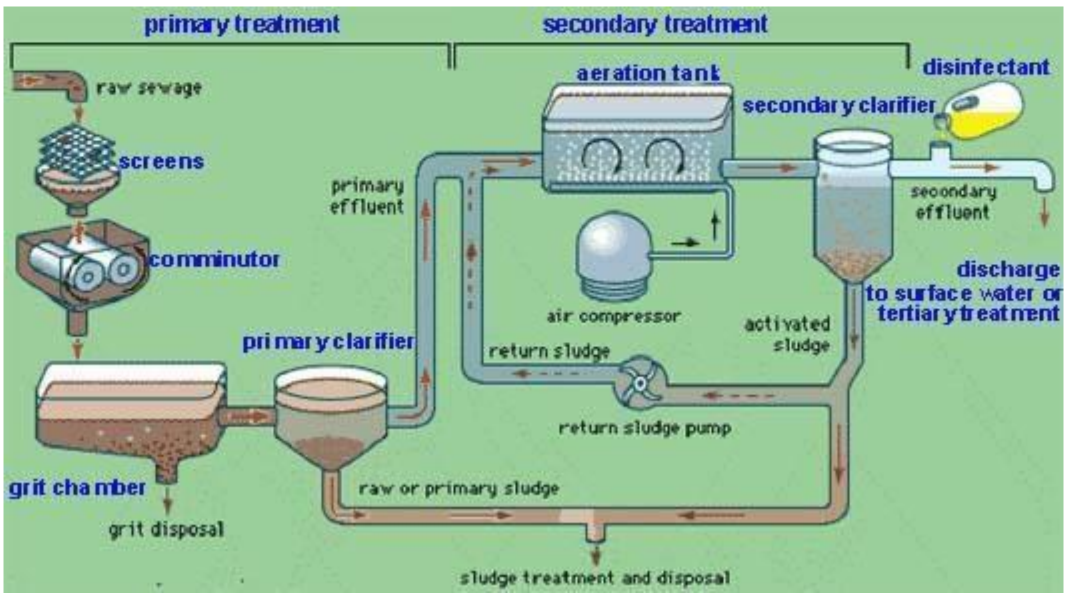
ii. Trickling Filters

A trickling filter or bio-filter consists of a basin or tower filled with support media such as stones, plastic shapes, or wooden slats. Wastewater is applied intermittently, or sometimes continuously, over the media. Microorganisms become attached to the media and form a biological layer or fixed film. Organic matter in the wastewater diffuses into the film, where it is metabolized. Oxygen is normally supplied to the film by the natural flow of air either up or down through the media, depending on the relative temperatures of the wastewater and ambient air. Forced air can also be supplied by blowers but this is rarely necessary. The thickness of the biofilm increases as new organisms grow. Periodically, portions of the film 'slough off the media. The sloughed material is separated from the liquid in a secondary clarifier and discharged to sludge processing. Clarified liquid from the secondary clarifier is the secondary effluent and a portion is often recycled to the bio-filter to improve hydraulic distribution of the wastewater over the filter.

Tertiary and/or advanced treatment

Tertiary and/or advanced wastewater treatment is employed when specific wastewater constituents which cannot be removed by secondary treatment must be removed. As shown in Figure , individual treatment processes are necessary to remove nitrogen, phosphorus, additional suspended solids, refractory organics, heavy metals and dissolved solids. Because advanced treatment usually follows high-rate secondary treatment, it is sometimes referred to as tertiary treatment. However, advanced treatment processes are sometimes combined with primary or secondary treatment (e.g., chemical addition to primary clarifiers or aeration basins to remove phosphorus) or used in place of secondary treatment (e.g., overland flow treatment of primary effluent).

Screening and Grit chambers

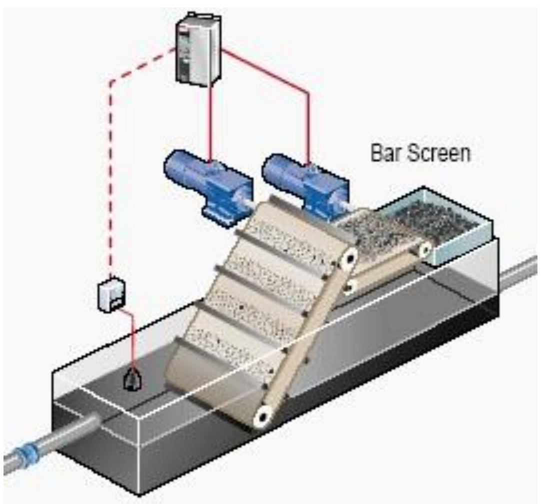


The removal of solids from the incoming wastewater flow is accomplished in steps. The first steps involve physical separation of the solids by screening and by gravity. The larger solids can be removed using screens and the heavy solids can be removed using settling processes. The dissolved organic material (and some of the lighter suspended solids) will remain in the sewage flow after primary clarification. The first step in the solids removal process is screening to remove the larger solids and "rags." After screening, a grit removal process is used to separate the heavier inorganic solids like sand and inert organics like coffee grounds from the flow. Rags can clog piping and pumps in downstream processes. Grit can also cause clogging problems and can damage pumps. Grit that isn't removed in the grit chamber will end up in the solids handling system where it will eventually collect in the digesters. This will reduce available digester capacity. These processes that remove inorganic solids are collectively referred to as pretreatment.

BAR SCREENS

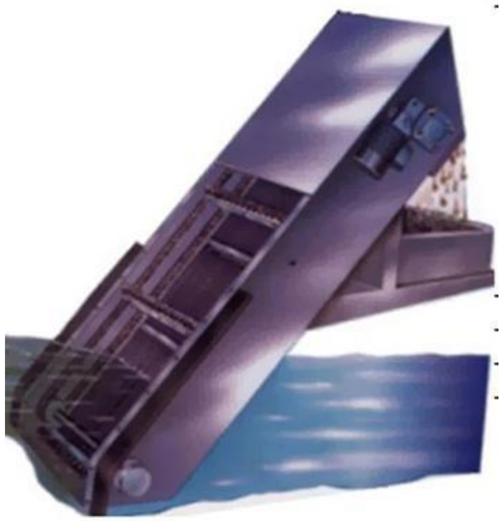
Bigger pieces of wood, plastic, metal, rubber, textile and other waste materials out of wastewater is being removed by **using bar screens**.

A bar screen consists of a series parallel of steel bars that are placed vertically in the influent flow channel. The bars are usually spaced about 1/2-3/4 inches apart. In some cases, two sets of screens are placed in the channel the upstream screen may have bars spaced 2-3 inches apart and the downstream screen will have the normal spacing. The front screen is sometimes referred to as a "trash screen." It is designed to catch large chunks of debris to avoid overloading the smaller screen. As the screen gets clogged with rags the water level upstream will rise. If the screen isn't cleaned regularly the upstream water level can back up and flood the structure.



Automatically cleaned bar screens

An operator must rake the collected debris from a manually cleaned bar screen. Manually cleaned bar screens are usually set at a 45-degree angle. This makes it easier to rake the debris from the



screen.

Manually cleaned bar screens

Automatically cleaned bar screens are designed with a set of rakes that are chain-driven. These units will operate periodically to remove the rags and deposit them in some type of container. The bar screen angle is usually between 60 and 90 degrees on an automatically cleaned screen system. The rags that are removed by the screen must be hauled to a landfill for disposal. However, some screen systems actually rake the screens, grind or shred the rags, and then return them to the waste flow.

Incoming septic sewage can cause corrosion problems with steel screens. Hydrogen sulfide formed by anaerobic decomposition will attack the metal bars. Bar screens should be inspected several times a year for corrosion and bent bars. Repair and replacement of the bars is the only maintenance issue for manually cleaned screens. Automatically cleaned screens need to have weekly inspections to check the conditions of the rake teeth and the chain drive.

Bar Screens are used to separate large debris such as rags and plastics.

Two types of bar screens:

1. Coarse sieve (***rake***) and
2. Fine sieve.

Coarse sieves have openings equal to or greater than 6 mm and generally protective role, while using a ***fine sieve***, with openings smaller than 6 mm, and can achieve significant removal of suspended solids from the wastewater.

The grid is usually made up of ***parallel rods***, while the fine screen usually used wire cloth or perforated metal plate.

Grit Chambers

Grit chambers are basin to remove the inorganic particles to prevent damage to the pumps, and to prevent their accumulation in sludge digestors.

A portion of the suspended solids load of municipal waste water consists of grit material. If not removed in preliminary treatment, grit in primary settling tank can cause abnormal abrasive wear on mechanical equipment and sludge pumps, can clog by deposition, and can accumulate in sludge holding tanks and digesters. Therefore grit removal is necessary to protect the moving mechanical equipments and pump elements from abrasion and accompanying wear and tear. Removal of grit also reduces the frequency of cleaning of digesters and settling tanks.

Grit removal devices rely upon the difference in specific gravity between organic and inorganic solids to effect their separation.

Composition of grit

Grit in sewage consists of coarse particles of sand, ash and clinkers, egg shells, bone chips and many inert materials inorganic in nature. Both the quantity and quality of grit varies depending upon

a) Types of street surfaces encountered
b) Relative areas served
c) Climatic conditions
d) Types of inlet and catch basins
e) Amount of storm water diverted
f) Sewer grades
g) Ground water characteristics, etc.,
h) Social habits

The specific gravity of grit is usually in the range of 2.4 to 2.65. Grit is non putrescible in nature and higher hydraulic subsidence than organic solids. Hence it is possible to separate the gritty material from organic solids by differential sedimentation in a grit chamber.

Types of grit chambers

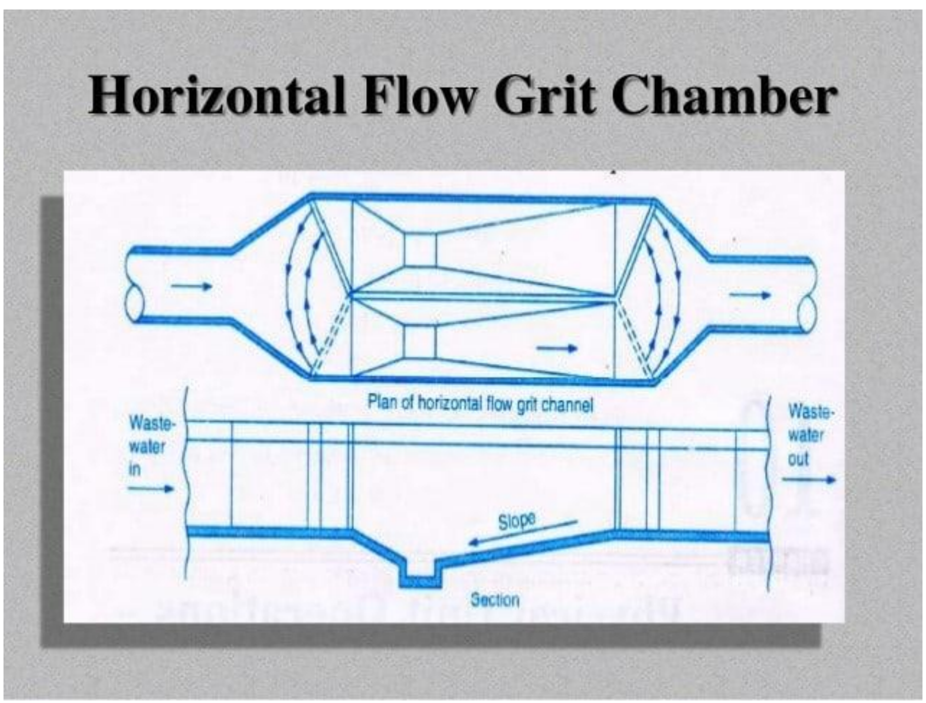
Grit chambers are of two types, mechanically cleaned and manually cleaned. The choice depends on several factors such as the quantity and quality of grit to be handled, head loss requirements, space requirements etc.,

Mechanically cleaned grit chambers are provided with mechanical equipment for collection, elevation and washing of grit which are operated either on a continuous or intermittent basis.

Manually cleaned grit chambers should have sufficient capacity for storage of grit between the intervals of cleansing. Atleast two tanks must be there, so that when one is under cleaning operation the other must be working. These tanks must be cleaned atleast once in every week. The simplest method of removal is by means of shovel and wheel barriers.

Aerated grit chambers

An aerated grit chamber is a special form of grit chamber consisting of a standard spiral flow aeration tank provided with air diffusion tubes placed on one side of the tank, 0.6 to 1m from the bottom. The heavier grit particles with their higher settling velocities drop down to the floor where as the lighter organic particles are remained in suspension and carried with the roll of the spiral motion, due to the diffused air and eventually out of the tank.



<u>Principle of Working of Grit Chamber</u>

Grit chambers are nothing but like sedimentation tanks, designed to separate the intended heavier inorganic materials (specific gravity about 2.65) and to pass forward the lighter organic materials. Hence, the flow velocity should neither be too low as to cause the settling of lighter organic matter, nor should it be too high as not to cause the settlement of the silt and grit present in the sewage. This velocity is called "differential sedimentation and differential scouring velocity". The scouring velocity determines the optimum ***flow through velocity***. This may be explained by the fact that the critical velocity of flow 'v_c' beyond which particles of a certain size and density once settled, may be again introduced into the stream of flow. It should always be less than the scouring velocity of grit particles. The critical velocity of scour is given by Schield's formula:

$$V = 3 \text{ to } 4.5 \, (g(S_s - 1)d)^{1/2}$$

A horizontal velocity of flow of 15 to 30 cm/sec is used at peak flows. This same velocity is to be maintained at all fluctuation of flow to ensure that only organic solids and not the grit is scoured from the bottom.

Types of Velocity Control Devices

1. A sutro weir in a channel of rectangular cross section, with free fall downstream of the channel.
2. A parabolic shaped channel with a rectangular weir.
3. A rectangular shaped channel with a parshall flume at the end which would also help easy flow measurement.

Design data

The basic data essential for a rational approach to the design of grit chambers are hourly variations of sewage flow and typical values for minimum, average and peak flows. Since the grit chamber is designed for peak flows and the flow through velocity is maintained constant within the range of flow, successful design and operation of grit chamber calls for a fairly accurate estimate of flow.

The quality and quantity of grit varies from sewage to sewage. Data relating to these two factors is very useful in proper design of grit collecting, elevating and washing mechanisms. In the absence of specific data, grit content may be taken as 0.025 to 0.075m^3 /million litres for domestic sewage and 0.06 to 0.12m^3/ million litres for combined sewage.

Design of grit chambers

i. Settling velocity or hydraulic subsidence value:

The settling velocity is governed by the size and specific gravity of grit particles and the viscosity of the sewage. The size of separation based on the minimum size and of grit to be removed is 0.2mm. The sp.gr of the grit may be as low as 2.4, but for design purposes a value of 2.65 considered as grit. The settling velocity is given by

$$V_S = 60.6(\delta_S - \delta \frac{3T+70}{100})$$

$$= 60.6(S_S - 1 \frac{3T+70}{100}) d$$

Where,

V_S= settling velocity in cm/sec

$S_S = \delta_S/\delta$= specific gravity

δ_S= mass density of particle in gm/cc δ= mass density of liquid in gm/cc d= particle size in cm

T= temperature of liquid in ^{o}c

For sp.gr of grit equal 2.65 and liquid equal to 1,

$$V_S = 60.6(2.65 - 1) \frac{3T+70}{100} d (3T + 70) d$$

For organic solids whose sp.gr equal to 1.2, V_S= 0.12 d $(3T \quad 70)$

Settling velocity is also called as Hydraulic subsidence rate (H.S.V)

ii. Overflow rate

A grit chamber designed for removal of 100% of grit particles of smallest size would also remove all grit particles larger than this. To obtain a 100% removal of the smallest size particles, it would be theoretically necessary for the detention period in the tank equal to the time required for the minimum sized particles to reach the tank bottom. In other words the conditions should be ideal for settling velocity particles. It can also be shown that the settling velocity V_S of the minimum sized particles is equal to surface loading rate (Q/A) or overflow rate in order to obtain a theoretical 100% removal of the particles.

The overflow rate of 1300 to 1700 m^3/m^2/day be taken for particles of 0.15 to 0.2mm dia and sp.gr 2.65 at 10^{o}c, which may be converted to any other temperature by the factor

$$\frac{3T + 70}{100}$$

iii. Flow through velocity

A horizontal velocity or flow through velocity of 15cm/sec to 30cm/sec is used at peak flows.

iv. Detention period

A period of 60 secs or one minute is usually adopted. Detritus tank

A detritus tank is similar to a grit chamber. The difference being only in the velocity of flow and the detention period. A detritus tank may be considered as a grit chamber in which the velocity of flow is such that an appreciable amount of organic matter settles down along matter by blowing the compressed air through the detritus tanks in order to lift the lighter organic solids or by washing in a grit washer.

Design aspects of detritus tank

Detritus tanks are normally rectangular in shape. The sides are vertical but tapered at bottom to form a through for the detritus collection. Overall depth of detritus tank may vary from 2.5 to 3.5m and detention period 3 to 4 minutes. Velocity of flow is kept between 20cm/sec to 40cm/sec.

Problem

Design a grit chamber for a town having a population of 1 lakh. Assume suitable data

necessary. Solution:

Assuming per capita sewage production as 120 litres/day Average daily flow = 1, 00,000 X

120 litres

$$= \frac{12X10^6}{1000X24X60X60}$$

= 0.139 m^3/sec Maximum flow = 3X 0.139 = 0.417 m^3/sec

Assuming a horizontal velocity of 30 cm/sec, and detention time as 60 seconds, Cross section

area of the tank = A = $\frac{FLOW}{VELOCITY}$ $\frac{0.417}{0.3}$ = = 1.39 m^2

Length of the tank required = L = horizontal velocity of flow X detention time

= 0.3 X 60 = 18 m.

There fore capacity of the tank = c/s area X length = 1.39 X 18

V = 25 m^3

Quantity of grit at the rate of 25 litres/million litres/day = $\frac{120X10^5X25}{10^6}$ =

300litres/day Assuming cleaning of grit chamber is once in a week,

Storage for one week = 7 X 300 = 2100 litres = 2.1 m^3 Total tank capacity = 25 + 2.1 = 27.1

m^3

Assuming the size of the particles as 0.2 mm and sp.gr 2.65, and temperature as 20 ^{O}c, Settling velocity given by Hazen formula,

V_S = d (3T + 70) = 0.02 (3 X 20 + 70) = 2.2 cm/sec.

Therefore depth of the tank required = settling velocity X detention time = 2.2 X 60 = 132 cm

Liquid depth = 1.32m

Breadth = $\frac{1.39 m^2}{1.32 m}$ = 1.05 m Provide 0.27 m free board.

Total depth of tank = 1.32 + 0.27 + 0.11 m sludge depth

Depth = 1.7 m; Breadth = 1.05 m; Length = 18 m

Provide two chambers of above dimensions so that when one is in cleaning operation, the other will be working.

Oil and Grease removal

Grease in sewage include fats, waxes, free fatty solids, calcium and magnesium soaps, mineral oils, etc., oil and grease find their way in sewage from restaurants, kitchens, garages, soap and candle factories, oil refineries, slaughter houses etc.,. If not removed, these substances may create the following difficulties:

1) If sewage is being discharged into the stream for disposal, unsightly scum, and foul odour may be developed at the surface of the stream. The scum retards reoxygenation and thus causes anaerobic condition.
2) They do not digest easily and therefore create problems in sludge digestion tanks.
3) They interfere with some of the treatment processes and also promote clogging of the trickling filters.
4) They affect the biological activities of the organisms and thus affect their smooth working.

They can be removed from sewage either by floatation or settling as scum or sludge. Formation of scum is promoted by diffusing air through the sewage. The tank in which scum formation is promoted by diffusion of air through the sewage is called skimming tank.

Skimming tanks

Skimming tanks are narrow rectangular tanks having at least two longitudinal baffle walls, interconnected. They are used to remove grease and fatty oils from the sewage. Air diffusers are provided at the bottom of the tank. Compressed air applied at the rate varying from 300 to 6000 m^3 per million litres of sewage agitates the sewage, which prevents settling of solids. Air tends to change the oil and grease to a soapy mixture. This mixture is carried to the surface by the air bubbles, some of which are entrained in it and may be skimmed off.

Design aspects

The ratio of length to depth of skimming tank should be about 2 or 1.5 to 1. Usual detention period is 3 minutes.

The surface area required for the tank can be found by the formula $A = 1110q / v_r$

Where,

A = surface area in sq.ft

q = rate of sewage flow in million gallons / day

V_r = minimum rising velocity of oily materials to be removed in inches/minute. Value of V_r in most of the cases is 25 cm/min.

PRIMARY TREATMENT – SEDIMENTATION

In preliminary treatment, removal of coarse natural solids, debris, grit, oil and grease etc., takes place. Finer suspended solids that cannot be removed from sewage from these processes can be removed by the process known as sedimentation. The process of sedimentation is carried out in tanks known as settling tanks or sedimentation tanks or clarifiers.

Usually at sewage treatment plants, this sedimentation is carried twice, once before the biological treatment and once after the biological treatment. Sedimentation before the biological treatment is called primary settling and that carried after the biological treatment is called secondary settling. If settleable solids are separated from sewage by gravitation and by natural flocculation alone, the process is termed as plain sedimentation. If chemicals are used to induce, or increase flocculation, the process is called chemical precipitation.

Sedimentation is the separation from water, by gravitation settling, of suspended particles that are heavier than water. It is one of the most widely used unit operations in the waste water treatment. The principles on which design of sedimentation tanks depends are that when a liquid containing solids in suspension is placed in a relatively quiescent state, those solids having a higher specific gravity than the liquid will tend to settle, and those with a lower specific gravity will tend to rise.

The well known Stokes‘s law on sedimentation expresses the relationship between the settling velocity size and density of particles settled, density and viscosity of liquid is given by

$$Vs = \frac{g\, d2\,(\delta s \quad \delta)}{18\mu} \quad \text{or} \quad \left(\frac{g}{18}\right)\left(\frac{Ss - 1}{\mu}\right) d$$

(For particles of size less than 0.1 mm) And Newton's for the same is

$V_S = 1.8 \, g \, (S_S – 1) \, d$

(For particles of size more than 1mm) Where,

V_S = settling velocity of discrete particular in cm/sec g = gravitational constant in cm/sec^2

μ = absolute or dynamic viscosity of the fluid in centi-poise

δ_S = mass density of particles in gm/cc

δ = mass density of liquid in gm/cc

$S_S = \delta_S / \delta$ = Sp.gr.

(1 centi poise = 10^{-2} poise or 10^{-2} dynes)

(1 centi stoke = 10^{-2} stokes or 10^{-2} cm^2/sec)

For the particular of size between 0.1mm, Hazen modification formula is

$Vs = d\,(3T + 70)$,

Where,

T = temperature of liquid, which applied for grit particles of sp.gr. 2.65 Characteristics of

settleable solids

The settleable solids to be removed in sedimentation tanks are mainly organic in nature, dispersed or flocculated. The specific gravity of these suspended solids may vary from 1.01 to 1.20. Generally raw sewage is a dilute heterogeneous suspension of low specific gravity ranging from fully dispersed to completely flocculated ones.

Process of sedimentation

In the process of sedimentation the velocity of flow is decreased to such a value that finer settleable will settle to the bottom of the tank. Such settling tanks in which sewage continuously keeps on moving with predetermined velocity are called continuous flow settling tanks. In olden days sewage was used to be filled in large tanks and allowed to remain quiescent for some time and then they were emptied. Such tanks are called fill and draw type tanks.

The liquid sewage coming out of settling tanks after the process of sedimentation is called effluent and the settled viscous sewage at the bottom of tank is called sewage sludge.

Design considerations

Several factors such as flow variations, density currents, solids concentrations, solids loading, area, detention time and overflow rate influence the design and performance of sedimentation tanks. Sedimentation tanks are designed for average flow condition.

Overflow rate or surface loading rate

The overflow rate represents the hydraulic loading per unit surface area of tank in unit time expressed as m^3/m^2/day. Overflow rates must be checked both at average and peak flows.

Detention period

The rate of removal of BOD and SS is maximum during the first 2 to 2.5 hours of settling and thereafter decreases appreciably. Longer detention beyond 4 hours may affect the tank performance adversely due to settling in septic conditions, particularly in tropical countries. Experience has shown that a detention period of 2 to 2.5 hours for primary settling tanks and 1.5 to 2 hours for secondary settling tanks would produce the optimum results.

Weir loading

Weir loading influences the removal of solids in particularly secondary settling tanks. It has been found that weir rates have less influence on the efficiency of settling tanks.

Depth

The depth sets the detention time in the settling tanks and also influences the sludge thickening in secondary settling tanks.

Sludge removal

Sludge can be removed manually, hydrostatically or mechanically from the sedimentation tanks. Mechanical cleaning of sludge should be preferred to manual cleaning even in small plants, where power is available for running the plant machinery. Even when power is not available or inadequate, hydrostatic removal should be adopted to avoid manual handling of sludge to prevent exposure of workers to health hazards.

Inlets and outlets

Performance of sedimentation tank is very much influenced by inlet devices which are intended to distribute and draw the flow evenly across the basin. All the inlets must be designed to keep the entrance velocity to prevent the formation of eddy or inertial currents in the tank to avoid short circuiting.

In horizontal flow rectangular tanks, inlets and outlets are placed opposite to each other separated by the length of tank with the inlet perpendicular to direction of flow. Inlet may be multiple pipe type, channel inlet with baffles or an inlet channel with submerged weir.

Outlet is generally an overflow weir located near the effluent end, preferably adjustable for maintaining the weir at a constant level. V- notches are provided on the weir to provide for uniform distribution of flow at low heads of discharge over the weir.

Types and shapes of sedimentation tanks

Circular tanks are more common than rectangular or square tanks. Upflow tanks have been used for sewage sedimentation but horizontal flow types are more popular. Rectangular tanks need less space than circular tanks and could be more economically designed where multiple units are required to be constructed.

For rectangular tanks, maximum length and widths may be 90m and 30m respectively with length to width ratios of 1.5 to 7.5 and length to depth ratios of 5 to 25 are recommended. A minimum of 2m in primary settling and 2.5m in case of secondary settling tanks are provided.

Diameters of circular tanks vary widely from 3 to 60m although the most common range is 12 to 30m. The water depth varies from 2m for primary to 3.5m for secondary settling tanks. Floors are sloped from periphery to centre at a rate of 7.5 to 10%. The inlet is generally at centre and outlet is a peripherical weir, the flow being radial and horizontal from centre to the periphery of the tank.

Performance

Primary sedimentation of domestic sewage may be expected to accomplish 30 to 45% removal of BOD and 45 to 60% removal of SS depending on the concentration and characteristics of solids in suspension. Secondary settling tanks if considered independently, remove a very high percentage of flocculated solids, even more than 99 %, under certain situations.

Table may be used for design of sedimentation tanks.

Sl. No	Item	Over flow rate $m^3/m^2/day$	Detention time hours	Weir loading $m^3/m/day$	Depth m	Dimensions m
1.	Primary settling only	30-60	2-2.5	100-150	3-3.5	Length=25-40 Width=6-10
2.	Primary settling followed by secondary treatment	30-100	1.5-2	150-300	3-3.5	Depth=3.6 Circular
3.	Primary with settlin g activated sludge return	30-60	1.5-2.5	150-300	3.5-4.0	dia.=12-45m Depth=4-5m

4.	Secondary for settling trickling filter	15-45	2-2.5	-	3.4	
5.	Secondary for settling activated sludge return	15-50	2-2.5	-	3.5-4.5	
6.	Secondary for settling extended aeration	10-40	3-4	-	3.5-5.0	

Problem

Design primary settling tanks required to treat the sewage from a town of population one lakh.

Answer:

Assuming per capita production of sewage as 120 litres/day Average daily sewage flow = 120 X 10^5 litres

= $\frac{120 X 10^6}{1000 X 24}$ = 500 m^3/hour

Assuming a detention period of 2 hours, Capacity of the tank = 500 X 2 = 1000 m^3

Assuming a depth of 3 m, the surface area required = 1000/3 = 333.33 m^2 Diameter of the tank:

Therefore d= $\frac{\sqrt{4 X 333.33}}{\pi}$

$\frac{\pi d^2}{4} = 333.33\ m^2$

d = 20.6 m say 21m

Check:

Overflow rate = Q/A = $\frac{12 X 10^6 X 4}{1000 X \pi X 21^2}$ = 34.64 m^3/m^2/day Which is in the range of 30-60 m^3/m^2/day, Therefore O.K

Weir loading = $\frac{Flow}{Weir\ length}$ $\frac{12 X 10^6}{1000 X \pi X 21}$

= 182 m^3/m/day

Which is 100-250 m^3/m/day range, therefore O.K

Allowing 0.6m width for effluent channel, overall diameter of the tank = 21+0.6+0.6 = 22.22 m

Allowing a free board of 0.6m, total depth of tank = 3+0.6 = 3.6m

Provide a bottom slope of 10% from periphery to centre.

However provide two tanks of the above dimensions, one being as standby unit.

Problem

Design a continuous flow type rectangular primary sedimentation tank fitted with mechanical sludge cleaning equipment for treating the sewage from a city having a population of 80000 persons which has an assured water supply rate of 100 lpcd.

Assume the maximum flow to be 1.4 times the average flow. The necessary design parameters may be assumed. Sketch the designed sedimentation tank

Answer:

Given:

Population = 80000

Rate of water supply = 100 lpcd Peak flow = 1.4 X average

Considering 80% of water supplied is coming out as sewage,, average sewage produced per day

$= \frac{80000\ X\ 100\ X\ 80}{1000\ X\ 100} = 6400\ m^3$

Hourly flow = 6400/24 = 266.67 m^3

Assuming a detention period of 2 hours, the capacity of the tank required C = 266.67 X 2 C =

533.33 m^3

Assuming the liquid depth as 3m, the surface area of the tank = 533.33 / 3 = 177.78 m^2 say 178 m^2

Assuming the ratio of length to width as 2, L X B = A or 2B X B = 178 m^2

Therefore Breadth = 178/2 = 9.43 m (say 9.5 m) Length of tank = 2 X 9.5 = 19 m

Provide a bottom slope of 1% since for cleaning of sludge mechanical devices are employed towards the inlet end, where it is collected in a hopper trough of 0.6 m width.

Provide a free board of 0.5 m and therefore total depth = 3.5 m Therefore the overall

dimensions of the tank are

Length = 19 m, Width = 9.5 m
Overall depth = 3.5 m Check:

i. Overflow rate:

V_o = flow / surface area = $\frac{6400 \frac{m^3}{day}}{19\ X\ 9.5\ m^2}$

= 35.45 m^3/m^2/day Which is in the range of 30-45 m^3 Therefore design is safe.

ii. Weir loading:

= Flow/perimeter = $\frac{6400}{2(19+9.5)}$ = 112.28 m^3/m/day

Therefore O.K

Problem

Design a set of two circular sedimentation tanks to treat 5 million litre of sewage per day. Assume depth of 2.5 m check for hydraulic and weir loading.

Solution

Sewage flow = 5 X 10^6 litres/day = 5000 m^3/day

Assuming a detention period of 2 hours, the total capacity of tank = $\frac{5000 X 2}{24}$ = 416.67

m^3 Providing two circular tanks, capacity of each tank = 416.67/2 = 208.335 m^3

Assuming a depth 2.5 m, the surface area of each tank = 208.335/2.5 = 83.334 m^2 i.e., $\pi d^2/4$ =

83.334 m^2

Therefore diameter of each tank, d = $\frac{\sqrt{4 X 83.334}}{\pi}$ = 10.3 m

Therefore provide two circular tanks of 10.5 m diameter and 2.5 m depth + 0.5 m free board and a bottom slope of 10% towards centre.

Check

Overflow rate = Flow/surface area = $\frac{5000}{\frac{2(\pi X 10.5^2)}{4}}$

V_o = 28.87 m^3/m^2/day

Which is less than 30 m^3/m^2/day

Weir loading rate = Flow/weir length = $\frac{5000}{2(\pi X 10.5)}$

= 75.79 m^3/m/day Which is less than 100 m^3/m of weir length/day Therefore design is O.K

Problem:

Design a circular settling tank for primary treatment of domestic sewage for a flow of 15 MLD. Assume suitable values of hydraulic retention time and surface loading rate.

Answer

Capacity = 1250 m^3 Surface area = 416.67 m^2 Diameter
= 23.0 m Depth
= 3m
Weir loading = 208 m^3/m/d

Problem:

Design a circular settling tank for primary treatment of domestic sewage for a flow of 12 MLD. Assume suitable values of hydraulic retention time and surface loading rate.

Answer:

Assumption of suitable hydraulic retention rate and surface loading rate Diameter of the tank, d = 20.6 m
Capacity of tank, C = 1000 m^3 Depth of tank, D = 3 m
Check for weir loading

Problem:

Design a set of primary clarifiers for a town having a population of one lakh. The flow is 130 lpcd. Check the hydraulic loading rate and weir loading rate.

Answer:

Discharge, Q = 13 MLD
Flow through each tank = 6500 m^3/d Assume, Detention time = 2 hrs Capacity, C = 541.66 m^3
Assume, Depth = 3 m Area, A = 180.55 m^2
Diameter of tank, d = 15.2 m Hydraulic loading rate = 36 m^3/m^2/d Weir loading rate = 136.19 m^3/m/d

Secondary Treatment

Biological treatment: biological treatment systems are living systems which relay on mixed biological cultures to break down waste organics and remove organic matter from the solution.

Biological treatment techniques

This may be classified as under following three heads:

1. Attached growth process (fixed film processes)
2. Suspended growth process
3. Combined process:

Attached growth process (fixed film processes)

These are the biological treatment process in which the micro organisms are responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissues are attached to some inert medium such as rock, slag or specially designed ceramic or plastic materials. Such processes include the following: i) Intermittent sand filter ii) Trickling filters iii) Rotating biological contractors iv) Packed bed towers v) Anaerobic lagoons (ponds) vi) Fixed film denitrification

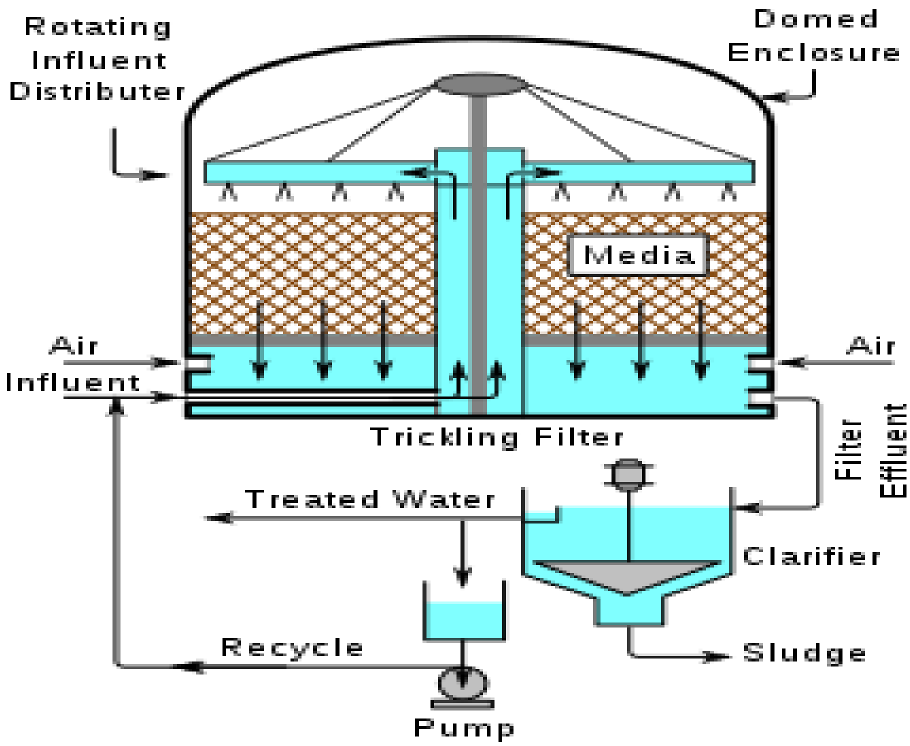
Suspended growth processes:

These are the biological treatment processes in which the micro organisms responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissues are maintained in suspension within the liquid in the reactor by employing either natural or mechanical mixing. In most processes, the required volume is reduced by returning bacteria from the secondary clarifier in order to maintain a high solids concentration. The suspended growth processes include the following: i) Activated sludge process ii) Aerated lagoons iii) Sludge digestion system iv)Suspended growth nitrification and suspended growth denitrification

Combined processes:

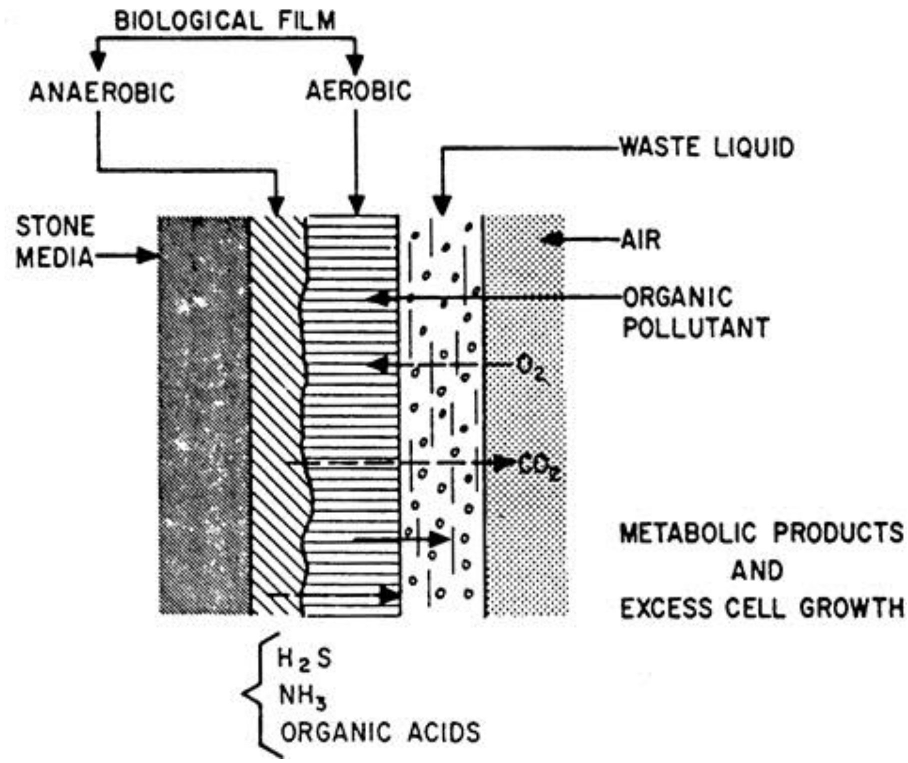
These consist of both attached growth processes as well as suspended growth processes. They include the following in sequence: i) Tricking filter, activated sludge: ii) Activated sludge, tricking filter, iii) Facultative lagoons

Tricking filters



- Also known as ***percolating filters*** or ***sprinkling filters***.
- Trickling filter is always preceded by primary sedimentation along with skimming devices to remove the scum which will prevents the clogging of settleable solids Using Spray nozzles or rotary distributors sewage is allowed trickle over a bed of coarse, rough, hard filter media, and it is then collected through the under drainage system filter media, and it is then collected through the under drainage system.
- The biological purification is brought mainly by aerobic bacteria which form a biological film known as ***bio film*** around the particles of the filtering media. the colour of this film is blackish, greenish and yellowish
- Apart from bacteria it also contains fungi, algae, lichens, protozoa etc.
- For the existence of bio film, sufficient oxygen is supplied by providing suitable ventilation facilities in the body of the filter.
- The staining due to mechanical action of filter bed is much less. Organic removal occurs by biosorption from the rapidly moving part of the flow and by progressive removal of soluble constituents from the more slowly moving portion.

Biological process in trickling filter



- Though, trickling filter is classified as an aerobic treatment device, the ***microbial film*** (or ***bio film***) or the ***slime layer*** formed on the filter medium is aerobic to a depth of only 0.1 to 0.2 mm and the remaining part of the film is anaerobic
- As the wastewater flows over the microbial film, the soluble organic material in the sewage is rapidly metabolized while the colloidal organics are adsorbed onto the surface. In the outer portion of the biological film, the organic matter is degraded by the aerobic micro organism.
- Since the food concentration is higher at the outer layer, the micro organisms near the outer layer are in rapid growth phase. As the micro organism at the outer surface grow, the thickness of the slime layer increases and the diffused oxygen is consumed before it can penetrate the full depth of the slime layer.
- Hence the lower zone of the film is in a state of starvation, due to which anaerobic media is established near the surface of the media.
- As a result of having no external organic source available for cell carbon, the micro organism near the media surface enter into the endogenous phase of growth and lose their ability to cling to the media surface.
- E

ventually there is scouring the slime layer due to the flowing liquid and fresh slime layer begins to grow on the media. This phenomenon of ***scouring*** of the slime is called sloughing or unloading of the filter

Construction of filter

Tank is either rectangular or square in shape if fixed nozzles are used and circular if rotator distributors are used. It consists of,

- ✓ A water tight holding tank
- ✓ Distribution system
- ✓ Filter medium
- ✓ Under drainage system

(For more detail refer page no 341-345 Environmental engineering II by Dr. B. C Punmia et al.,)

Loading, Efficiency and performance

1. Loading
a. Hydraulic loading :
i. as quantity of sewage per hectare of surface area per day – varies 25 to 40 million liters per day
ii. as quantity of sewage per unit volume of filter bed – 7.5 to 22.5 Ml/Ha m of filter/ day
b. organic loading
i. As Kg of 5 day of BOD per unit volume of filter bed – 1000 -2200 kg of 5 day BOD per ha m of filter volume per day, or as 15 – 30 kg of 5 day BOD / 1000 m^3 of filter material
2. Efficiency

Is expressed by the following equation evolved by National research Council (NRC) of USA

$$E(\%) = \frac{100}{1+0.0044\sqrt{u}}$$

$$E(\%) = \frac{100}{1+0.44\sqrt{U}}$$

$$E\ (\%)= \frac{Ci-Ce}{Ci} \times 100$$

Where,

E= Efficiency of the filter and its secondary clarifies, in terms of % of applied BOD removed
Ci = Influent BOD concentration
Ce= Effluent BOD concentration
U= organic loading in kg/Ha m/ day (called as unit hydraulic loading)
U=Unit hydraulic loading in kg/ m^3/day

Merits and demerits of Conventional trickling filter
Merits

- Stabilized effluents, can therefore disposed of in smaller quantity of dilution water
- It has good dependability to produce good effluent under widely varying weather and other conditions
- Can remove about 80% suspended solids and 75 – 80% of BOD
- High rate of filtration hence requires less space
- Working is simple and cheap and simple, does not required any skilled supervision
- Self cleaning, mechanical wear and tear is small
- Less electrical power requirement

Demerits

- High head loss making automatic siphonic dosing tanks necessary
- High cost of construction
- Requires preliminary treatment cannot treat raw sewage
- Humus tank is necessary
- May develops fly nuisance, odour nuisance and ponding nuisance

Classification of trickling filter

1. Conventional T.F / Low rate T.F
2. High rate T.F

a. Single stage filter
b. Multistage filter (two stage filter)

Advantages of high rate trickling filter

1. Since smaller filter volume is required, initial cost is less
2. Operating cost are also low
3. The trouble of bad smell or odour is much less
4. Absence of filter flies
5. Flexible working, hence efficiency is not seriously affected due to variations in the strength and character of sewage

Disadvantages

- Effluent is not highly nitrified hence requires more volume of dilution water
- Raw water cannot be treated and the process requires primary treatment of sewage

Table: Comparison between Conventional and High rate trickling filter

Sl.no	Characteristics	Conventional/low rate Filter	High rate Filter
1	Depth of media	1.8 to 2.4 m	1.2 to 1.8 m
2.	Hydraulic loading ($m^3/d/m^2$)	1 to 4	10 to 30 (including recirculation)

3.	Organic loading as 5 day BOD in g/d/ m^3	80 to 30	500 to 1000 (excluding recirculation)
4	Recirculation system	Usually not provided but can be provided if the hydraulic load does not exceed the limit	Always provided. Recirculation ration 1:1 to 4:1
5	Volume of the bed	5 times	1
6	Interval of dosing	Not more than 5 minutes. Sewage is applied in intervals	Not more than 15 seconds. Sewage is applied continuously.
7	Sloughing	Intermittent	Continuous
8	Cost of operation	More	Less
9	Land required	More	Less
10	Characteristics of final effluents	Contains BOD $\leq$ 20 mg/L; it is highly nitrified into nitrate stage	Contains BOD $\geq$ 30 mg/L; it is not fully nitrified
11	Secondary sewage	Highly oxidized, Black colour, having light, fine particles	Not fully oxidized; brownish black colour containing fine particles

Trouble and remedies in Trickling filter

Following are common troubles occurs at the site and operation

1. Fly nuisance

- Filters are often infested with small moth like, deceptively fragile flies called ‘Psychoda’. These flies do not bite, but may get into the eyes, nostrils and ears of men and animal.
- Remedial measures:

a. Flooding the filter for about 24 hrs, at weekly or biweekly intervals
b. Jetting down the inside walls of the fliter with high pressure hose
c. Chlorinating the filter influent(0.5 – 1.0 mg/L) for several hours at 1 to 2 week intervals
d. Application of larvicides such as DDT chordane, malathion etc., when the filter is not in use
e. Continuous hydraulic loading to hamper the flies and larvae

2. Odour nuisance

- Odours from filter are due to undesirable growth, sludging and anaerobic decomposition
- Remedial measures:

a. Maintaining a well ventilated filter, either by natural ventilation or by forced ventilation
b. Reduction of filter effluent or secondary clarifies effluent which will wash fine solids through the filter bed
c. Aeration or chlorination of sewage before primary settling of sewage

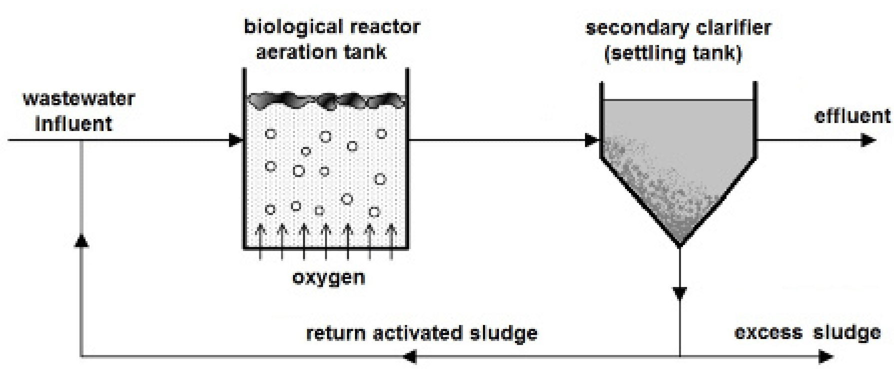
3. Ponding nuisance

- It is caused when all the voids of the trickling filter are filled up due to choking by heavy fungus or other suspended matters, due to which the sewage cannot pass through the filter and accumulate at the surface in the form of pond
- Remedial measures

a. Opening the clogged section by flushing with a fire hose and simultaneously loosening the aggregates by steel bar
b. Reducing the strength of filter influent by re-circulation
c. Flooding the filter once in a day and allowing of it to stand for 24 hours
d. Chlorinating the effluent the influent with a dose not exceeding 5 kg/100 m2 of filter area, once in a 4 to 7 days
e. Stopping the distributors over the ponded area

Activated sludge process

- Activated sludge process so named because the production of mass of micro organism capable of aerobically stabilizing a waste
- The activated sludge is the sludge which is obtained by settling sewage in presence of abundant oxygen so as to be supercharged with favourable aerobic micro organisms.
- The ASP of sewage process of sewage treatment is based on providing intimate contact between the sewage and biological active sludge
- Sludge is developed initially by prolonged aeration under conditions which favour the growth of organisms with special ability to oxidize organic matter



- The effluent from the primary settling tank is mixed with a dose of activated sludge and is aerated in an aeration tank for a period of some hours. During the aeration, the micro organisms in the sewage multiply by assimilating part of the influent organic matter.
- In this process, a part of organic matter is synthesized into new cells and part is oxidized to derive energy.
- The synthesis reaction, followed by subsequent separation of the resulting biological mass and the oxidation reaction are the main mechanisms of BOD removal in the activated sludge process.

- The biological mass generated in the aeration tank consists of zoogical bacteria, protozoa, rotifers etc. The biomass is generally is generally flocculent and quick settling. It is separated from activated sewage in a secondary settling tank and is recycled continuously to the aeration tank as an essential feature of the process.
- The mixture of recycled sludge and sewage in the aeration tank is referring to as mixed liquor. The recycling of sludge helps in the initial build up of a high concentration of active micro organisms in the mixed liquor which accelerates the BOD removal.
- Once the micro-organisms in the mixed liquor has been reached so as to maintain proper food/ micro-organism ration (F/M) for optimum operation, its further increase is prevented by regulating the quantity of sludge recycled and washing the excess from the system

F/M ratio,

Modification in ASP

As the use of ASP increased, many modifications were made both with respect to basin configuration, as well as aeration techniques. The more important processes are:

1.Conventional process
2.Tapered aeration process
3.Step aeration process
4.Contact bed stabilization process
5.Completely mixed process
6.Modified aeration process
7.Extended aeration process

Operational difficulties

There are some operational difficulties encountered in the operation of ASP

1. Rising sludge or floating sludge:

- May be due to denitrification in the settling tank releasing nitrogen bubbles which buoys up the sludge.
- This problem is associated with
  - High sludge age
  - Long solids retention time in the clarifier
- When sludge age exceeds 10 days and DO content is in excess of 2 mg/L, a degree of conversion of ammonium to nitrate will occur. Bacteria convert nitrates to nitrogen gas. The gas bubbles then buoy the sludge to the surface.
- Remedial measures are
  - Increasing the return sludge age
  - Increasing the speed of sludge scraper mechanism where possible
  - Decreasing the mean cell residence time by increasing the sludge washing rate.
  - Decreasing the rate of flow of aeration liquor into the offending tank.

2. Sludge bulking

- Sludge with poor settling characteristics is termed bulking sludge
- Occurs due to
  - inadequate air supply
  - low pH
  - prolonged detention of sludge in the settling tank
  - sludge accumulation on the bottom of the aeration tank
  - sudden discharge of septic solids from sewage system
- Controlled measures:
  - Reducing the sewage flow to the aeration tank for short period
  - Reducing the suspended solids in the mixed liquor
  - Prolonged aeration and reparation of returned sludge
  - Application of sewage either to the sewage or to the returned sludge, so as to control filamentous growth. Chlorine requirements are 0.2 to 1 % of dry solids weight in the return sludge
  - Application of hydrogen peroxide, to control the growth of filamentous organisms
  - Addition of hydrated lime to raise pH to 8 or more.

Advantages and disadvantages

Advantages:

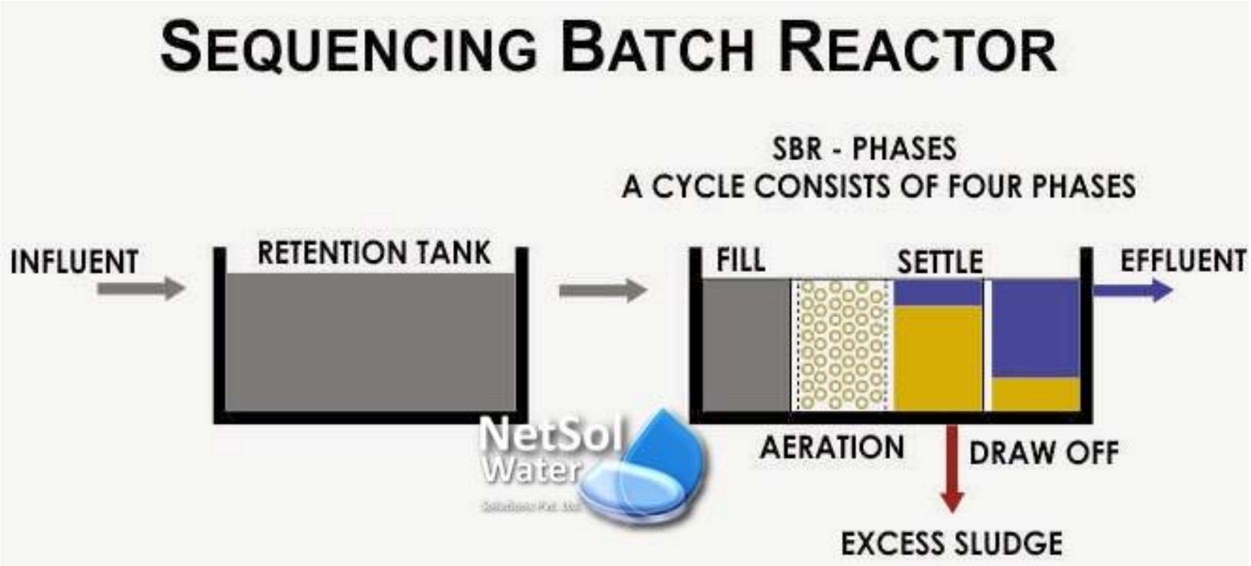
- Clear spacing and non-putrescible effluent possible
- No odour during the process as compared to other biological processes
- No fly nuisance.
- Highly efficient. Removal of suspended solids, BOD and bacteria are around 90% each.
- Degree of stabilization or nitrification is controllable between limits, so as to match with quality and character of receiving water.
- Relatively low cost of installation, as compared to the total cost of TF.
- Small area is required as compared to TF.
- The excess sludge has higher fertilizing values as compared to the sludge obtained from other treatment processes.
- Amount of hydraulic head consumed by the process is less.

Disadvantages

- Very sensitive to variations in the quality of sewage, particularly in respect of industrial waste which may cause sludge bulking.
- High cost of operation.
- Necessity of constant skilled attendance.
- Uncertainty of expected resulted under all conditions.
- Large quality of sludge is produced which is difficult to dewater, digest and dispose off.

Introduction to sequential batch reactors

What Is SBR?



SBR– in the process of wastewater treatment uses the activated sludge by the sequencing batch reactor plant. It is used for reducing the organic matters. SBR technology, separates the water and activated sludge through the oxygen bubbles.

Sequential Batch Reactor

Process in Brief: Biomass is developed in SBR tanks- generally operates in dual tank mode, where air is applied through blowers. Operation type is having four stages – Fill, aeration, settling and decantation. Wastewater is filled in tank, aeration is started for a specified period once filled to full level, then aeration stops and allow settling of biomass within tank itself. Once biomass is settled clear supernatant wastewater is decanted and discharged from the system.

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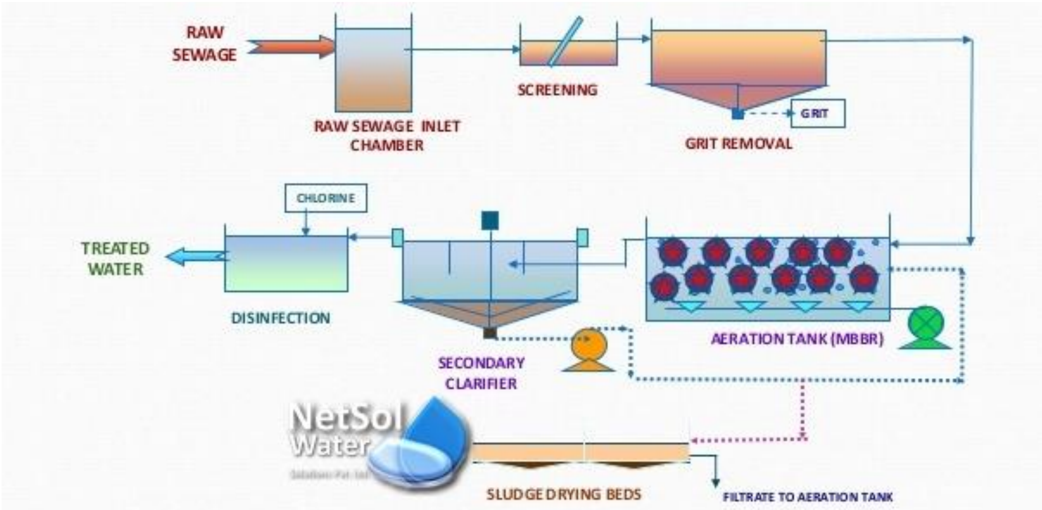
Major highlights: o Contains anoxic zone which provide control over denitrification

o Better operation control as operates in batch mode
o Better space management
o Require automation
o Cost is higher than conventional treatment process
o Require two tanks of same size to operate on continuous basis

Moving Bed Bio Reactor

MBBR- moving bed biofilm reactor sewage treatment plant is used in very less space, and that is known as a biofilm process.



Process in Brief: Biomass is developed on supporting media as biofilms. These media are continuously kept in suspended form by blowing air through blowers. The biofilms developed on supporting media, degrades the pollution load present in wastewater. Generally provided with a settler to separate biomass mixed with wastewater. Clear supernatant of the clarifier – settler is discharged from the system.

- Major highlights: o Provide good removal of pollution load.
o Need continuous monitoring of media is required.
o Particularly useful for packaged installation
o Installation Circular tanks are convenient
o More height can be kept for reactor
o Media replacement and checking is little cumbersome.

Sludge Digestion Process

- The process of stabilization of sludge withdrawn from the sedimentation basin.
- By decomposing the organic matter under controlled anaerobic conditions.
- 40% - 60% of organic solids converted into CO2 and CH4 gas.
- Remaining organic matter will be chemically stable and odourless with 90% - 95% of moisture content.
- This process reduces the sludge into three forms
- Digested sludge

1. Supernatant liquor
2. Gases of decomposition

- Supernatant liquor

1. Liquefied finely divided solid matter
2. BOD about 3000ppm
3. Contains 1500ppm to3000ppm of suspended solids, so re-treated with the raw sludge.

- Gases of decomposition

1. 65-70% CH4
2. 30% CO2

3. Traces of inert gases like N, H_2S
4. Collected and used as fuel

- Stages In Sludge Digestion Process

1. Acid fermentation/Acid production
2. Acid regression
3. Alkaline fermentation

- Stages In Sludge Digestion Process

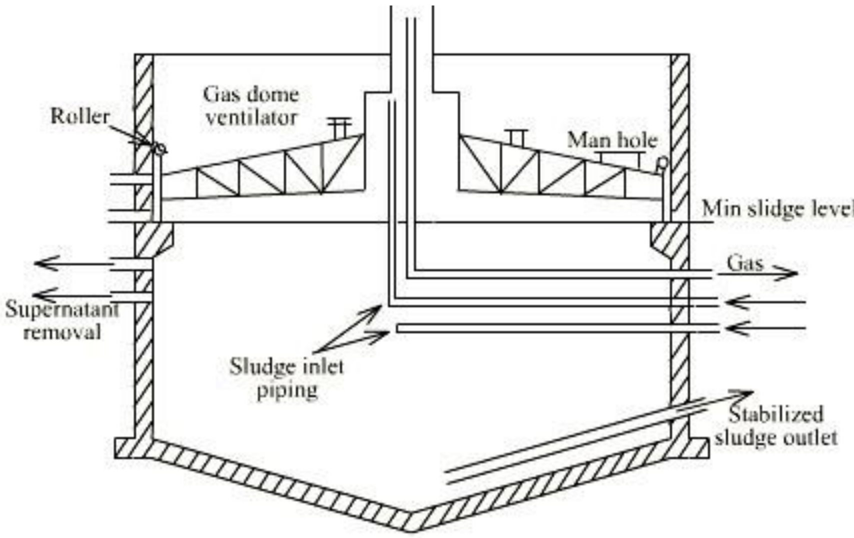
1. Acid fermentation/Acid production
2. Anaerobic and facultative bacteria (acid formers) acts.
3. Acid formers stabilize the organic solids through hydrolysis.
4. Soluble products fermented to volatile acids and organic alcohols of low molecular weight.
   1. Propionic acid, acetic acid etc
5. Evolution of CH_4 , CO_2 and H_2S gases.
6. Lowers pH value to less than 6 – highly acidic in nature.
7. Evolution of highly putrescible odours.
8. Continues for 15 days.

- Acid regression

1. volatile organic acids and nitrogenous compounds of the first stage acted upon by bacteria.
2. Forms acid carbonates and ammonia compounds.
3. Evolution of H_2S and CO_2 gases in small amount.
4. Offensive odour ϖpH value rises to 6.8
5. Entraps of gases of decomposition and forms formy scum layer.
6. Continues for 3 months. ϖBOD remains high.

- Alkaline fermentation

1. Proteins and organic acids attacked and broken up by anaerobic bacteria (methane formers).
2. Forms ammonia, organic acids and gases.
3. Liquid separates out from solids and the digestive sludge (or ripened sludge) is formed
4. Digestive sludge collected at the bottom of digestion tank.
5. Alkaline in nature – pH value rises little above 7.
6. Large amount of CH_4 and small amount of CO_2 and N gases are evolved.
7. Continues for 1 month.
8. BOD rapidly falls down.



Anaerobic sludge digester

Factors Affecting Sludge Digestion Process

- Temperature
- pH value
- Seeding with digested sludge
- Mixing and stirring of raw sludge with digested sludge

Temperature - Rate of digestion increases at higher temperature. ϖ
i. Zone of thermophilic digestion
♣ High temperature zone – 400C to 600C.
♣ Acted upon by heat loving thermophilic organisms.
♣ Optimum temperature is about 540C – digestion within 10 – 15 days.
ii. Zone of mesophilic digestion
♣ Moderate temperature zone – 250C to 400C.
♣ Acted upon by mesophilic organisms.
♣ Optimum temperature is about 290C – digestion in 30 days.

- pH value

1. Should not lower below 6.5
2. Optimum value – 7.2 to 7.4 (in the final stage)
3. Acidity increases due to
    1. Overdosing of raw sludge
    2. Over withdrawal of digested sludge
    3. Sudden admission of industrial wastes
    4. Remedy: Add hydrated lime – 2.3 to 4.5 kg per 1000 persons to the raw sludge.

5. Raw sludge of about 3- 5% of the weight of the digested sludge should only be added daily.

- Seeding with digested sludge

1. Seed with digested sludge from another tank
2. Speeds up the digestion process

- Mixing and stirring of raw sludge with digested sludge

1. Proper agitation methods mixes raw and digested sludge to form homogeneous mass of sludge.
2. Bacterial enzymes present in digested sludge gets thoroughly mixed up – helps in better decomposition