CHAPTER ONE

WATER QUANTITY REQUIREMENT

1.1 Introduction

While designing water supply scheme for a town or city it is necessary to determine the total quantity of water required for various purposes by the city; in which the first duty of the engineer is to determine this quantity of water, and then finding out the suitable water sources from where the demand can be satisfied.

Actually the determination of the quantity of water is dependent upon the size of the community and the purpose for which it is needed.

Based on this information three items need to be determined for calculating the total quantity of water required for a town or city, these are:

- Design population
- Rate of demand (water consumption for various purpose)
- Design period

1.2 Rate of demand

In order to arrive at a reasonable value of rate of demand for any particular town, the demand of water for various purposes is divided under the following categories.

1. Domestic water demand
2. Commercial water demand
3. Industrial water demand
4. Institutional water demand
5. Fire fighting water demand
6. Loss and waste

1.2.1 Domestic Water Demand

It includes the quantity of water required in the houses for drinking, bathing, washing hands and face, flushing toilets, washing cloths, floors, utensils, etc.

In developed countries the domestic water demand may be as high as 350l/cap/day. In many cases water demands are fixed by governmental agencies. Water demand data provided by ministry of water resources of Ethiopia are given in tables below.

Table 1 Estimation of per capita demand for piped water in l/c/d (1997) for population of > 30,000(urban and rural)

No.	Activity	House Connection	Yard Connection	Public fountains (Stand pipes)	Rural Schemes
1	Drinking	2.5	2.5	2.5	2.5
2	Cooking	7.5	5.5	4.5	3.5
3	Ablutions	17	12	7	5
4	Washing dishes	5	4	4	3
5	Laundry	15	8	7	4
6	House cleaning	7	3	2	2
7	Bath and shower	20	4	3	
8	Toilets	6	1		
	Total	**80**	**40**	**30**	**20**

Table 2 Estimate of per capita demand for piped water in l/c/d (1997) for population of < 30,000(for urban between 2500 and 30000).

No.	Activity	House Connection	Yard Connection	Public fountains (Stand pipes)	Rural Schemes
1	Drinking	1.5	1.5	1.5	1.5
2	Cooking	5.5	3.5	3.5	3.5
3	Ablutions	15	10	6	5
4	Washing dishes	5	2	2	2
5	Laundry	15	8	7	3
6	House cleaning				
7	Bath and shower	4	1		
8	Toilets	20	4		
	Total	**70**	**30**	**20**	**15**

Table 3 Estimate of per capita demand for rural schemes in l/c/d (1997)

No.	Activity	Minimum	Average	Maximum
1	Drinking	1.5	1.5	3.5
2	Cooking	2.5	3.5	4.5
3	Ablutions	4	5	5
4	Washing dishes	2	2	4
5	Laundry		3	3
6	House cleaning			
7	Bath and shower			
8	Toilets			
	Total	**10**	**15**	**20**

1.2.2 Commercial Water Demand

Commercial buildings & commercial centers include stores, hotels, shopping centers cinema houses, restaurants, bar airport, automobile service station, railway and bus stations, etc, (Refer table 4)

1.2.3 Institutional Water Demand

This is also known as public demand. It is the water required for public buildings and institution such as schools, hospitals, public parks, play grounds, gardening, sprinkling on rods, etc, (Refer table 4.)

Table 4: Commercial and Institutional demand

No.	Description	Unit	Water demand in liters
1	Clinics	No	500 per clinic
2	Health center	Beds	150 per bed
3	Hospitals	Beds	200 per bed
4	Students	No	10 per pupil
5	Offices	No	350per office or 5 per employee
6	Tej house hotels and pubs	No	1000
7	Bakeries and groceries	No	250

1.2.4 Industrial Water Demand

The water requirements for this purpose defend up on the type and size of the industry. (Refer table 5)

Table 5: Typical values of water use for various industries

Industry	Range of flow (*Gal/ ton Product)
Cannery	
Green beans	12000-17000
Peaches & pears	3600-4800
Other fruits & vegetables	960-8400
Chemical	
Ammonia	24000-72000
Carbon di oxide	14400-21600
Lactose	144000-192000
Sulfur	1920-2400
Food and beverage	
Beer	2400-3840
Bread	480-960
Meat packing	3600-4800
Milk products	2400-4800
Whisky	14400-19200
Pulp and paper	
Pulp	60000-190000
Paper	29000-38000
Textile	
Bleaching	48000-72000
Dyeing	7200-14400

***1gal. = 3.7854 lit.**

1.2.5 Fire fighting water demand (Fire demand)

Fires generally break in thickly populates localities abs the industrial area, and cause serious damages of properties and some time live of people are lost. Fire may take place sue to faulty electric wires by short circuiting, fire catching materials, explosions, bad iterations of criminal people or any other unforeseen happenings. If fires are not properly controlled and extinguished in minimum possible time, they lead to serious damages and may burn the cities.

In cities fire hydrants should be provided on the mains at d100 to 150 m apart Fire brigade men immediately connect these fire hydrants with their engines & start throwing water at very high rate on the fire.

Fire demand is treated as a function of population and some of the empirical formulae commonly used for calculating demand as follows;

a) John R.Freeman 's formula:

$$\mathbf{Q = 1136.50}\ (\frac{p}{5}+10)$$

Where **Q** = Quantity of water required in 1/min.

P = population in thousands

It also states that

$$\mathbf{F = 2.8}\ \sqrt{P}$$

Where **F**= period of occurrence of fire in years.

b) Knuckling 's formula

$$\mathbf{Q = 3182}\sqrt{p}$$

C) National Boarded of Fire Underwriter's formula (widely used in **USA**)

$$\mathbf{Q = 4637}\ \sqrt{p}\ \mathbf{(1\text{-}0.01}\sqrt{p}\ \mathbf{)}$$

Example 1

Workout fire demand for a population of 100,000. Use formulae of Freeman, Knuckling National Board of Fire Underwriter.

Name of Formula	Formula	Fire Demand in l/min.
Freeman	$Q = 1136.50\ (\frac{p}{5}+10)$	*34095* $F = 2.8\ \sqrt{100} = 28$ *year*
Kuichling	$Q = 3182\ \sqrt{p}$	*31820*
National Board of Fire Underwriter	$Q = 4637\ \sqrt{p(1-0.01\sqrt{p})}$	*4173*

Although the actual amount of water in a year for fire fighting is small than the rate of use, the insurance service office uses the formula

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

Where f=the required fire flow in g p m (lit/min/3.78)

C= a coefficient related to the type of construction

A= total floor area ff^2 (m^2x10-76) excluding the basement of the building

C= coefficient ranges from a max of 1.5 *for wood frame* to a minimum of 0.60 for *fire resistive construction*. The fire flow calculated from the formula is not to exceed 30, 240 lit /min in general, nor 22,680 lit/min for one story construction .The minim fire flow is not to be less than 1890 lit/min. Additional flow may be required to protect nearby buildings. The total for all purposes for a single fire is not to exceed 45,360 lit/min nor be less than 1990 lit/min.

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

The fire flow must be maintained for a minimum of 4 hours as shown in table –6. most communities will require duration of 10 hours.

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

Average domestic demand=22,000x600=18.2×10^6 lit/day

Maximum daily demand= $1.8\times13.2\times10^6=23.76\times10^6$lit/day

$F=18(1)\ (1000\times10.76\times6)^{0.5}=17{,}288$ lit/min $=24.89\times10^6$lit/day

Max rate= $23.76\times10^6+24.89\times10^6$

$=48.65\times10^6$ lit/day

=2211 lit per capita/day for to hours

The total flow required during this day would be

$23.76+24.89\times10/24=34.13\times10^6$liters

=1551 lit per capital/day

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

Table –6: Residential fire flows

Distance b/n adjacent units/m	Required fire flow/lit min
>30.5	1890
9.5-30.5	2835-3780
3.4-9.2	3780-5670
< 3.0	5670-7560

For continuous construction use 9450 lit/min

6) **Unaccounted for Water**

These include the Quantity of water due to wastage, losses, thefts, etc, i.e.

- Waste in the pipelines due to defective pipe joints, cracked and broken pipes, faulty valves and fittings
- Water that is lost when consumers keep open their taps or public taps even when they are not using water and allow continuous wastage of water.
- Water that is lost due to unauthorized and illegal connection

While estimating the total water demand of water for a town or city, allowance for these losses and wastage should be done. Generally, 15 – 40 % of the total quantity of water is made to compensate for lose, thefts and wastage of water.

1.3 Per capita demand

In community water is used for various purposes as described above. For the purposes of estimation of total requirement the water demand is expressed in liters/capita/day i.e. per capita demand.

The following are the factors affecting per capita demand

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

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

(iii) **Industries and commercial activities:** As the quantity of water required in certain industries is much more than domestic demand, their presence in the town will enormously increase per capita demand of the town. As a matter of the fact, the water required by the industries has no direct link with the population of the town.

(iv) **Quality of water:** If the quality of water is good, the people will consume more water. On the other hand, if the water has unpleasant taste or odor, the rate of consumption will down.

(v) **System of sanitation:** If a town is provided with water carriage system of sanitation, the per capita demand increases because the people will use more quantity of water for flushing sanitary fixtures like water closets.

(vi) **Cost of water:** The cost of water directly affects its demand. If the cost of water is more, the people will use less quantity of water.

(vii) **Use of water meters:** If metering is introduced for the purpose of charging, the consumer will be cautious in using water and there will be less wastage of water and thus per capita demand may lower down.

(viii) **System of supply:** The supply of water may be continuous or intermittent. In the former case, water is supplied for 24 hour and in the latter case water is supplied for certain duration of day only.

It is claimed that intermittent supply system will reduce per capita demand. But sometimes, the results are proved to be disappointing, mainly for the following reasons:

- During non-supply period, the water taps are kept open and hence, when the supply starts, water flowing through open taps is unattended and this results in waste of water.
- There is tendency of many people to through away water stored previously during non-supply hours to collect fresh water. This also results in waste of water and increase per capita demand.

Variation in rate of consumption

The per capita daily water consumption (demand) figures discussed above have been based upon annual and it indicates the average consumption. The annual average daily consumption, while useful, does not tell the full story.

In practice it has been seen that this demand does not remain uniform throughout the year. Climatic conditions, the working day, etc tends to cause wide variations in water use. The variation may be categorized into two broad classes

i. Seasonal fluctuation
ii. Daily and hourly fluctuation

Through the week Monday will usually have the highest consumption, and Sunday the lowest. Some months will have an average daily consumption higher than the annual average .In most cites the peak month will be July or august. Especially hot, dry weathers will produce a week of maximum consumption, and certain days will place still greater demand upon the water system. Peak demands also occur during the day, the hours of occurrence depending upon the characteristics of the city. There will usually be a peak in the morning as the day's activities start and a minimum about 4 A.M.A curve showing hourly variation in consumption for a limited area of city may show a characteristic shape .it but there will be a fairly high consumption thorough the working day. The night flow, excluding industries using much water at night, is a good indication of the magnitude of the loss and waste.

The important of keeping complete records of water pupate of city for each day and fluctuations of demand through out the day cannot be overemphasized. So far as possible the information shield be obtained for specific areas. These are the basic data required for planning of water works improvement .If obtained and analyzed, they will also Indicate trends in per capita consumptions and hourly demands for which further provision must be made.

In the absence of data it is some times necessary to estimate the maximum water consumption during a month, weekday, or hours. The maximum daily consumption is likely to be 180 % of the annual average and may rich 200 %. The formula suggested by R.O Goodrich is convenient for estimating consumption and is:

$$P=180t^{-0.10}$$

Where p=the percentage of the annual average consumption for the time t in days from 2/24 to 360.

The formula gives consumption for the maximum day as 180 percent of the average, the weekly consumption 148 percent, and the monthly as 128 percent. These fingers apply particularly to smaller residential cites. Other cites will generally have smaller peaks.

The maximum hourly consumption is likely be about 150 percent of the average for that day. Therefore the maximum hourly consumption for a city having an annual average consumption of 670 lit/day per capita would occur on the maximum day and would be 670x1.80x1.5,or 1800 lit/day.

The fire demand must also be added, according to the method indicated in the above articles. Maximum rate of consumption is of less importance than max flow but is required in connection with design use of water, and the proportion of peak demand provided from storage. Usually it will vary form 25 to 50 percent of the daily average.

Peaks of water consumption in certain areas will affect design of the distribution system. High peaks of hourly consumption can be expected in residential or predominantly residential sections because of heavy use of water for lawn watering especially where under ground system are used, air condition or in other water using appliance. Since use of such appliances is increasing peak hourly consumptions are also increasing.

1.4 Design period

Before designing and construction a water supply scheme, it is the engineer's duty to assure that the water works should have sufficient capacity to meet the future water demand of the town for number of years. The number of years for which the designs of the water works have been done is known as the design period.

The period should neither should neither be to short or too long, Mostly water works are designed for design period of 22-30 years which is fairly good period, The following are the normal design periods for various units of water supply system:

Sr.No.	Name of Unit	Design period in years
1	Pump house	30
2	Electric motors & pumps	15
3	Water treatment units	15
4	Distribution (pipe line)	30

Data required for fixing design period

The economic design period of a structure depends up on its, *fire cost, ease of expansion*, *and likely hood of obsolescence in connection with design, the water consumption at the end of the period must be estimated*. Over design is not conservative since it may burden a relatively.

Over design is not conservation since it may burden a relatively small community with the cost of extravagant works designed for a far large population. Very appropriately design different segment of the water testament and distribution systems for differing periods using differing capacity criteria.

1. **Development of source**: The design period will depend upon the source .for ground water, it is easy to drill additional wells, the design period will be short, perhaps 5 yeas .for surface waters requiring impoundments, the design period will be longer, perhaps as much as 50 years. The design capacity of the source should be adequate to provide the maximum daily demand anticipated during the design period, but not necessarily up on a continuous basis.
2. **pipe line from source:** The design period is generally long since the life of pipe is long and the cost of material is only a portion of the const of constriction .25 year or more would not be unusual. The design capacity of the pipe line should based up on average consumption of suitable velocities under all anticipated flow conditions
3. **Water treatment plant:** The design period is commonly 10 to 15 years since expansion is generally simple it is considered in the initial design. Most treatment units will be designed for average daily flow at the end of the design period since overloads do not result in major flow at the efficiency. Hydraulic design should be based up on max-anticipated flow.
4. **Pumping plant:** The design period is generally 10 years since modification and expansion are easy it initially considered. Pump selection requires knowledge of max flow including fire demand, average flow, and minimum flow during the design period.

5. **Amount of storage:** The design period may be influenced by cost factors peculiar to the construction of storage vessels, which dictate minimum unit cost for tank of specific size. Design requires knowledge of average consumption, fire demand, maximum hours, maximum week, and maximum month as well as the capacity of the source and pipelines from the source.

6. **Distribution system:** The design period is indefinite and the capacity of the system should be sized to accommodate the maximum anticipated development over factors

affecting per capital flow should be considered. Maximum hourly flow including fine demand is the basis for design.

Summary

In general the following points should kept in mined while fixing the design period for any water supply scheme.

- Funds available for the completion of the project(the higher the availability of the fund the higher will be the design period.)
- Lie of the pipe and other structural materials used in the water supply scheme. (Design period in no case should have more life than the components and materials used in the scheme. At least the design period should be nearly equal to the materials used in water supply works.)
- Rate of interest on the loans taken to complete the project.(if the interest rate is less, it will be good to keep design period more otherwise the design period should be small)
- Anticipated expansion rate of the town.

1.5. POPULATION FORECASTING

The knowledge of population forecasting is important for design of any water supply scheme. The design is done in the bases of projected population at the end of the design period.

The following are the common methods by which the forecasting of population is done.

1, Arithmetic increases method

2. Geometric increase method

3. Incremental increase method

4. Decrease rate method

5. Simple graphical method

6. Master plan curve method

7. Logistic curve method

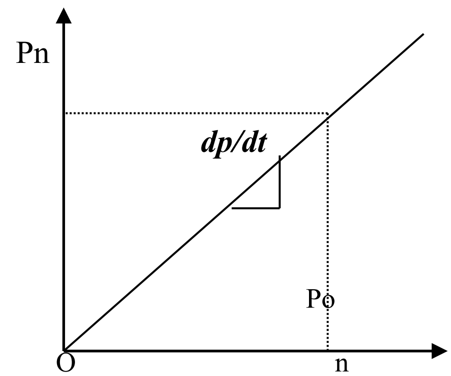
8. Ration & correlation

1. Arithmetic increase method

This method is based on the assumption that the population is increasing at a constant rate i.e. the rate of change of population with time is constant.

$$\frac{dp}{dt} = k$$

$$\int_{p_o}^{p_n} dp = K \int_{o}^{n} dt$$



$$Pn = Po + K_n$$

Where; Pn= population at n decades or years

n= decade or year

K= arithmetic increase

This method is generally applicable to large and old cities.

Example: The following data has been noted from the statistics authority for certain town.

Year	1940	1950	1960	1970
Population	8000	12000	17000	22500

Calculate the probable population in the year 1980, 1990, 2000, and 2006.

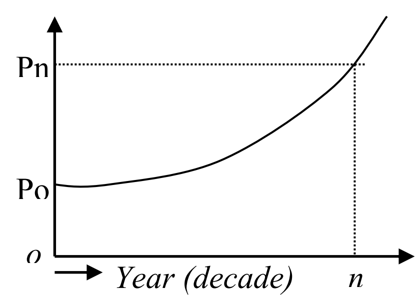
2. **Geometric increase method**

This method is based on the assumption that the percentage increase in population remains constant.

$P_1 = P_o + K P_o = P_o (1+K)$

$P_2 = P_1 (1+K) = P_o (1+K)(1+K)$

$P_3 = P_2 (1+K) = P_o (1+K) (1+K) (1+K)$



$P_n = Po (1+K)^n$

Where Po= initial population

Pn = population at n decades or years

n= decade or year

K= percentage (geometric) increase

This method is mostly applicable for growing towns and cities having vast scope of expansion.

Example: Forecast the population of example above by means of geometric increase method.

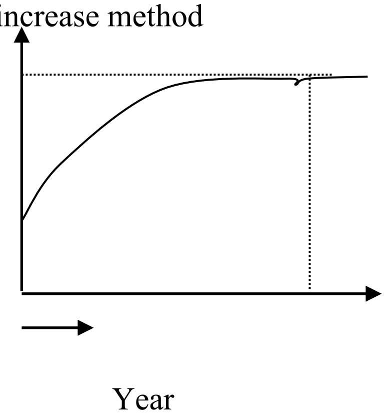
3. Incremental increase method

This method is improvement over the above two methods. The average increase is determined by the arithmetical method and to this is added the average of the net incremental increase.

Example: forecast the population of example above using incremental increase method

4. Decrease rate method

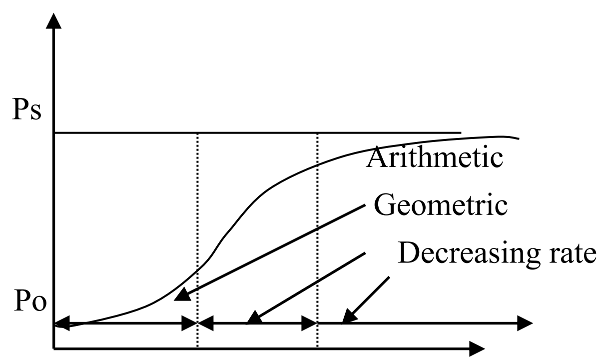
In this method, the average decrease in the percentage increase is Worked out and is subtracted from the latest percentage increase for successive period. This method is applicable only in such cases, where the rate of growth of population shown a down ward trend .see the following example.



Example 6. Forecast the population of example above by means of Decrease rate method.

5. Logistic curve method

If the population of a town is with plotted with respect to time, the curve so obtained under normal condition shall be S shaped logistic curve.



Mathematical solution of the logistic carver equation

$$P = \frac{p_5}{1 + me^{nt}}$$

Taking three points at equal time intervals

(t_1, p_1), (t_2, p_2), (t_3, p_3)

Where $t_2 = t1 + \Delta t$

$t_3 = t2 + \Delta t$

$$Ps = \frac{2p_1p_2p_3 - p_2^2(p1 + p2)}{p_1p_3 - p_2^2}$$

Example. *The following data have noted form the static's Authority.*

$P_{1980} = 40,000$

$P_{1990} = 100,000$

$P_{1990} = 130,000$

Determine the saturation population and the problem population in the year 2010.

Ans. $P_{2010} = 136,291$

6. Graphical method

In this method the population of last few year are correctly plotted to a suitable scale on the graph with respect to years. Then, the curve is smoothly extended to forecast the future population.

Example. *Solve Example above using graphical method*

Ans. $P_{1980} = 29.400$, $p_{1990} = 36,000$, $P_{2000} = 41,600$

7. Master plan method

In the method, the master plan of the city or town is used to determine the future expected population .The population densities for various zones of the town are fixed and hence the footer population of the city when fully developed can easily be worked out.

8.Ration and correlation method

In this method, the rate of population growth of a town is related to the rate of population growth of state or nation. Hence it is possible to estimate the population of a town under consideration by considering the rate of population growth of state or nation.

Example: Country, P_{1980}= 1, 000,000 P_{2004}= 1,5000,000

Town, P_{1980}= 10,000 P_{2004}= 15,000

9. Method used by Ethiopians statistic Authority (geometric increase method)

$$pn = poe^{kn}$$

Where, Pn =population at n decades or years

n= Decade or year

K= 3% = 0.03

Example. According to CA the population of certain town is 1564 in the year 1994. Determine the probable population in the year 2002.

Ans. 20, 040

CHAPTER TWO
WATER SOURCES

2.1 Types of water sources

The origin of all water is rainfall. Water can be collected as it falls as rain before it reaches the ground; or as surface water when it flows over the ground; or is pooled in lakes or ponds; or as ground water when it percolates in to the ground and flows or collects as ground water; from the sea (ocean) in to which it finally flows.

Therefore sources of water supply schemes can conveniently be classified as follows:

1. Rain and snow
2. Surface water:
    - Rivers
    - Lakes
    - Pond
    - Sea water
    - Impounding reservoirs
    - Wastewater reclamation
3. Underground sources
    - **Springs**
        - Depression springs
        - Contact springs
        - Artesian springs
        - Hot springs
    - **Wells**
        - Shallow wells
        - Deep wells
        - Infiltration galleries
        - Infiltration wells

Impounding reservoirs: Are artificial lakes formed by the construction of dams across a valley.

Wastewater reclamation: Sewage or other waste water may be used as source of water for cooling, flushing water closets (WCS), watering lawns, parks, etc. for fire fighting and for certain industrial purposes after giving the necessary treatment to suit the nature of the use.

Springs: Are formed when ground water appears at the ground surface for any reason as a current of flowing water.

Types of springs:

1. **Depression spring**: is a spring formed when the ground surface intersects the water table.

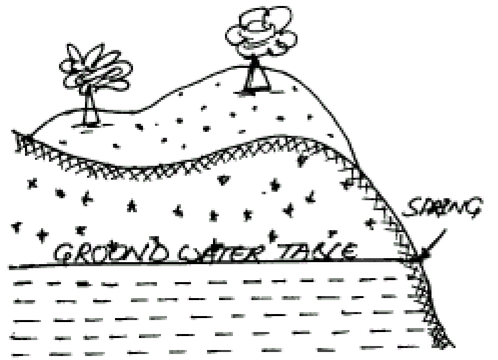


Fig. 2.1 Gravity/ Depression Spring

2. **Contact/Surface spring**: is a spring created by a water bearing formation overlying an impervious formation that intersects the ground surface.

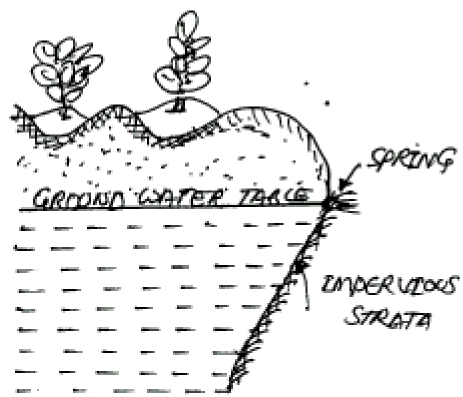


Fig. 2.2 Contact Spring

3. **Artesian spring**: is a spring that results from the release of water under pressure from confined water bearing formation through either a fault or fissure reaching the ground surface. It is also known as fracture spring.

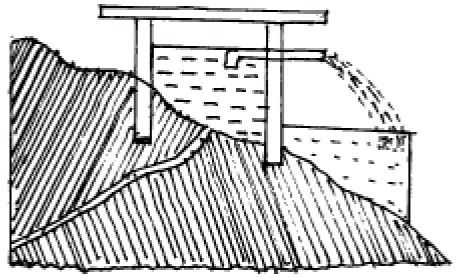


Fig. 2.3 Artesian springs

Wells: Are artificial holes or pits vertically excavated for bringing ground water to the surface.

Types of wells

1. Shallow wells

   Shallow wells may be large diameter hand dug wells (diameter 1-7m) and depth ≤ 20m. Or machine drilled wells of small diameter (diameter 8-60cm) and depth ≤ 60m.

2. **Deep wells**

   Deep wells are most large, deep, high-capacity wells constructed by drilling rig. Construction can be accomplished by cable tool method or rotary method. Drilling rigs are capable of drilling wells 8 to 60cm in diameter and depth ≤ 600m.

Infiltration Gallery

An infiltration gallery is a horizontal or nearly horizontal tunnel which is constructed through water bearing strata. It is sometimes referred as horizontal well. Infiltration gallery may be constructed with masonry or concrete with weep holes of 5cm by 10 cm.

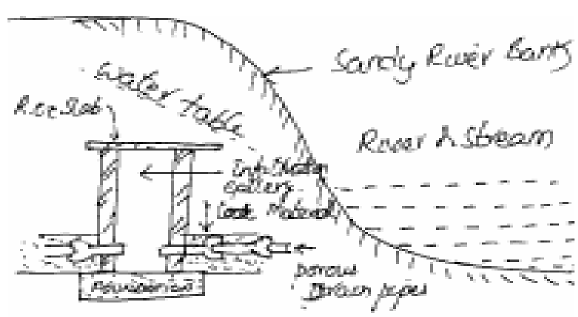
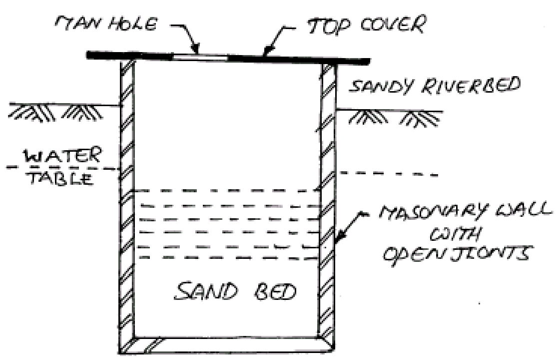


Fig. 2.4 Infiltration Gallery

Infiltration wells

In order to obtain large quantity of water, the infiltration wells are sunk in series in the blanks of river. The wells are closed at top and open at bottom. They are constructed by brick masonry with open joints as shown in fig. 2.5.

For the purpose of inspection of well, the manholes are provided in the top cover. The water filtrates through the bottom of such wells and as it has to pass through sand bed, it gets purified to some extent. The infiltration well in turn are connected by porous pipes to collecting sump called jack well and their water is pumped to purification plant for treatment.



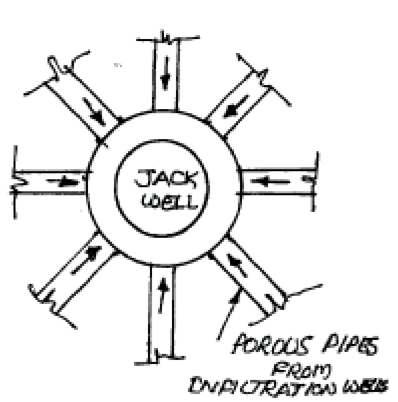


Fig.2.5: Infiltration well **Fig.2.6: Jack well**

2.2 Intake Structures

The main function of the intakes works is to collect water from the surface source and then discharge water so collected, by means of pumps or directly to the treatment water.

Intakes are structures which essentially consists of opening, grating or strainer through which the raw water from river, canal or reservoir enters and carried to the sump well by means of conducts water from the sump well is pumped through the rising mains to the treatment plant.

The following points should be kept in mind while selecting a site for intake works.

1. Where the best quality of water available so that water is purified economically in less time;
2. At site there should not be heavy current of water, which may damage the intake structure;

3. The intake can draw sufficient quantity of water even in the worest condition, when the discharge of the source is minimum;
4. The site of the work should be easily approachable without any obstruction;
5. The site should not be located in navigation channels;
6. As per as possible the intake should be near the treatment plant so that conveyance cost is reduced from source to the water works;
7. As per as possible the intake should not be located in the vicinity of the point of sewage disposal for avoiding the pollution of water.
8. At the site sufficient quantity should be available for the future expansion of the water-works.

Types of Intake structures:

Depending upon the source of water the intake works are classified as following

a. Lake Intake
b. Reservoir Intake
c. River Intake
d. Canal Intake

A. LAKE INTAKE:

For obtaining water from lakes mostly submersible intakes are used. These intakes are constructed in the bed of the lake below the water level; so as to draw water in dry season also. These intakes have so many advantages such as no obstruction to the navigation, no danger from the floating bodies and no trouble due to ice. As these intakes draw small quantity of water, these are not used in big water supply schemes or on rivers or reservoirs. The main reason being that they are not easily approachable for maintenance.

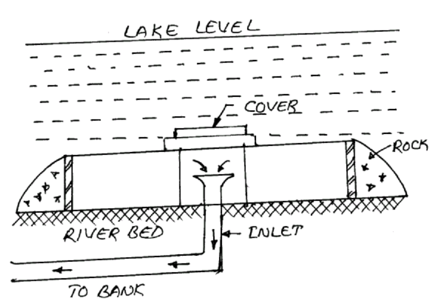


Fig. Lake Intake

B. RIVER INTAKE

Water from the rivers is always drawn from the upstream side, because it is free from the contamination caused by the disposal of sewage in it. It is circular masonry tower of 4 to 7 m in diameter constructed along the bank of the river at such place from where required quantity of water can be obtained even in the dry period. The water enters in the lower portion of the intake known as sump well from penstocks.

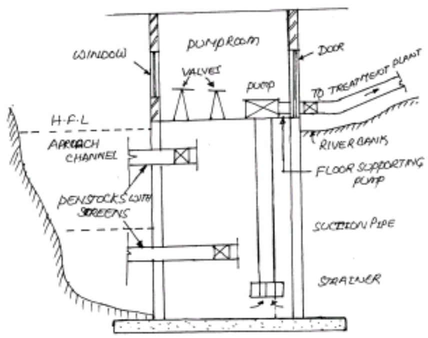


Fig. River Intake

C. RESERVOIR INTAKE

Fig below shows the details of reservoir intake. It consists of an intake well, which is placed near the dam and connected to the top of dam by foot bridge.

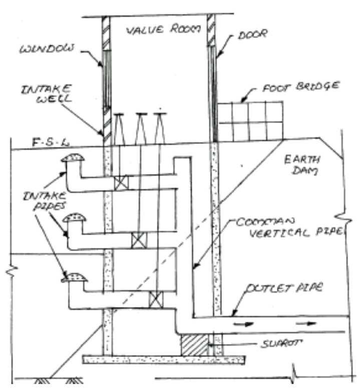


Fig. Reservoir intake

The intake pipes are located at different levels with common vertical pipe. The valves of intake pipes are operated from the top and they are installed in a valve room. Each intake pipe is provided with bell mouth entry with perforations of fine screen on its surface. The outlet pipe is taken out through the body of dam. The outlet pipe should be suitably supported. The location of intake pipes at different levels ensures supply of water from a level lower than the surface level of water.

When the valve of an intake pipe is opened the water is drawn off from the reservoir to the outlet pipe through the common vertical pipe. To reach up to the bottom of intake from the floor of valve room, the steps should be provided in Zigzag manner.

A. CANAL INTAKE

Fig 3.14 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.

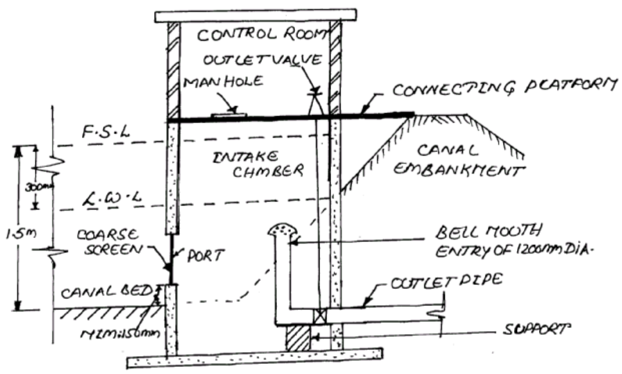


Fig. Canal Intake

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.

2.3 Water Sources Selection Criteria

The choice of water supply to a town or city depends on the following:

- Location
- Quantity of water
- Quality of water
- Cost
- Social and Environmental issues also considered

1. **Location**: The sources of water should be as near as to the town as possible.
2. **Quantity of water:** the source of water should have sufficient quantity of water to meet up all the water demand through out the design period.
3. **Quality of water**: The quality of water should be good so that it can be treated easily with affordable cost.
4. **Cost**: the initial cost, operation and maintenance cost of the water supply scheme have to be to the affordable level the community. When there are many alternative

water sources, in addition to the above criterion the source which demand list cost has priority for selection.

CHAPTER-THREE
WATER QUALITY AND POLLUTION

3.1 General Introduction

Absolutely pure water is never found in nature and contains number of impurities in varying amounts. The rainwater, which is originally pure, also absorbs various gases, dust and other impurities while falling. This water when moves on the ground further carries salt, organic and inorganic impurities. So this water before supplying to the public should be treated and purified for the safety of public health, economy and protection of various industrial process, it is most essential for the water work engineer to thoroughly check, analyze and do the treatment of the raw water obtained the from sources, before its distribution. The water supplied to the public should be strictly according to the standards laid down from time to time.

3.2 WATER QUALITY CHARACTERISTICS

For the purpose of classification, the impurities present in water may be divided into the following three categories.

3.2.1 Physical Characteristics

Physical characteristics include:

- Solids
- Turbidity,
- Color taste,
- Odor
- Temperature, and
- Foam

Obviously, these characteristics may be associated with ***chemical pollutants*** and may result from the discharge of chemicals in to the water body.

1. Color:

Color is caused by materials in solution or colloidal conditions and should be distinguished from turbidity, which may cause an apparent (not true) color.

True color is caused by dyes derived from decomposing vegetation. Colored water is not only undesirable because of consumer objections to its appearance but also it may discolor clothing and adversely affect industrial processes.

Before testing the color of water, total suspended solids should be removed by centrifugal force in a special apparatus. The color produced by one milligram of platinum in a liter of water has been fixed as the unit of color. The permissible color for domestic water is 20ppm on platinum cobalt scale.

2. Turbidity:

Turbidity is caused due to presence of suspended and colloidal solids. The suspended solids may be dead algae or other organisms. It is generally silt, clay rock fragments and metal oxides from soil.

The amount and character of turbidity will depend upon:

- The type of soil over which the water has run and
- The velocity of the water.

When the water becomes quite, the heavier and larger suspended particles settle quickly, while the lighter and more finely divided ones settle very slowly. Very finely divided clay may require months of complete quiescence for settlement. Ground waters are normally clear because, slow movement through the soil has filtered out the turbidity. Lake waters are clearer than stream waters, and streams in dry weather are clearer than streams in flood because of the smaller velocity and because dry-weather flow is mainly ground water seepage. Low inorganic turbidity (silt and clay) may result in a relatively high organic turbidity (color). The explanation of this is that low inorganic turbidity permits sunlight to penetrate freely into the water and stimulates a heavier growth of algae, and further, that organics tend to be absorbed upon soil fractions suspended in water.

Turbidly is measured by comparing the sample with a standard solution by optical means. Unit for measurement is **NTU** (**Nephelometry turbidity unit**). Sedimentation with or with out chemical coagulation and filtration are used remove it.

3. Temperature:

Temperature increase may affect the portability of water, and temperature above 15^0c is objectionable to drinking water. The temperature of surface waters governs to a large extent the biological species present and their of activity. Temperature has an effect on most chemical reactions that occur in natural water systems. It also has pronounced effect on the solubility of gases in water.

4. Solids:

Solids may be present in suspended and /or in solution and may be divided in to organic and inorganic matter.

Suspended solids (SS) are discrete particles which can measured by filtering a sample through a fine paper where as dissolved solids (DS) are due to soluble materials measured by evaporating a filtered sample of water and weighting the residue.

Total solids (TS) = SS+DS

5. Foam:

Foam form various industrial waste contributions and detergents is primarily objectionable from the aesthetic standpoint.

6. Tastes And Odor:

The terms taste and odor are themselves definitive of this parameter. Because the sensations of taste and smell are closely related and often confused, a wide variety of tastes and odors may be attributed to water by consumers. Substances that produce an odor in water will almost in variably impart a taste as well. The converse is not true, as there are many mineral substances that produce taste but no odor.

Many substances with which water comes into contact in nature or during human use may import perceptible taste and odor. These include minerals, metals, and salts from the soil, and products from biological reactions, and constituents of wastewater. Inorganic substances are more likely to produce tastes unaccompanied by odor. Alkaline material imports a bitter taste to water, while metallic salts may give salty or bitter taste.

Organic material, on the otter hand, is likely to produce both taste and odor. a multitude of organic chemicals may cause taste & odor problems in water with petroleum-based products being prime offenders. Biological decomposition of organics may also result in taste-and odor-producing liquids and gases in water. Principal among these are the reduced products of sulfur that impart a rotten egg taste and odor. Also certain species of algae secrete an oily substance that may result in both taste and odor.

Consumers find taste and odor aesthetically displeasing for obvious reasons. Because water is thought of as tasteless and odorless, the consumer associates taste and odor with contamination and may prefer to use a tasteless, odorless water that might actually pose more of a health threat.

Measurement

Quantitative tests employ the human taste smell can be used for this purpose. An example is the test for the **threshold odor number** (TON). Varying amounts of odorous water are poured into containers and diluted with odor free distilled water to make a 200-ml mixture. An assembled panel of five to ten "**noses**" is used to determine the mixture in which the odor is just barely detectable to the sense of smell. The ton of that sample is then calculated, using the formula

$$TON = \frac{A+B}{A}$$

Where A is the volume of odorous water (ml) and B is the volume of odor free water required to produce 200 ml mixture.

A similar test can used to quantify taste. EPA does not have a maximum standard for TON. A maximum of 3 has been recommended by the public health service and guideline rather that a legal standard.

7. Electrical Conductivity

The conductivity of a solution depends upon the quantity if dissolved alts present and for dilute solutions, it is approximately proportional to the TDS content.

Conductivity (us/m)*(0.6-0.8) =TDS (mg/1)

Conductivity measurements are therefore used to determine concentrations of dissolved salt or the concentration of ionized matter in general.

3.2.2 Chemical Characteristics

1. Dissolved Oxygen (DO)

Dissolved oxygen is present in variable quantities water. Its content in surface waters is dependent upon the amount and character of the unstable organic matter in the water. Clean surface waters are normally saturated with DO.

The amount of oxygen that water can hold is small and affected by the temperature. The higher the temperature, the smaller will be the DO.

Temperature (^{0}C)	0	10	20	30
DO (mg/1)	14.6	11.3	9.1	7.6

Oxygen saturated waters have pleasant taste and waters lacking in DO have an insipid tastes. Drinking water is thus aerated if necessary to ensure maximum DO. For boiler feed water DO is undesirable because its presence increases the risk of corrosion.

2. Oxygen Demand:

Organic compounds are generally unstable be oxidized biologically or chemically to stable, relatively inner end produce such as CO_2, H_20 & NO_3. Indicators used for estimation of the oxygen demanding substance in water are ***Biological Oxygen Demand (BOD), Chemical Oxygen Demand (COD), Total Oxygen Demand (TOD) and Total Organic Carbon (TOC).***

An indication of the organic content of water can be by measuring the amount of oxygen required for stabilization.

BOD is the quality of oxygen required for the biochemical oxidation of the decomposable matter at specified temperature within specified time. (20ºC and 5 day)

It depends on temperature and time t.

3. Nitrogen

The forms most important to water quality engineering include;

a) **Organic – nitrogen**: in the form of proton, amino acids and urea.
b) **Ammonia – nitrogen**: nitrogen as ammonium salts. E.g. (NH_4). CO_3
c) **Nitrate- nitrogen**: an intimidate oxidation stage. Not normally present in large quantity.
d) **Nitrate- nitrogen**: final oxidation product of nitrogen.
e) **Gaseous nitrogen (N_2)**

The presence of nitrogen compounds in surface waters usually indicate pollution excessive amount of ammonia and organic nitrogen may result form recent sewage discharges or runoff contamination by relatively fresh pollution. Therefore, waters congaing high org-N & ammonia –N levels are considered to be potentially dangerous. While waters in which most of nitrogen is in nitrate from are considered to some what stabilized to constitute prior pollution

4. Hydrogen Sulfide:

It is produced in ground water by reduction of sulfates, iron pyrites or decompositions of organic matter.

5. Acidity:

Most natural waters are buffered by a carbon dioxide – bicarbonate system. Acidity represent the amount of carbonic acid present .for all practical purposes if pH of the water is below 8.5 some acidity is present.

6. Alkalinity

It is defined as the quantity of ions in water that will react to neutralize hydrogen ions. Alkalinity is thus the measure of the ability of water to neutralize acids. By far the most constituents of alkalinity in natural waters are carbonate (CO_3^{2-}), bicarbonate (HCO_3^-) and hydroxide (OH). These compounds result from the dissolution of mineral substances in the soil atmosphere.

7. PH:

PH is a measure of the concentration of free hydrogen ion in water. It expresses the moral concentration of the hydrogen ion as its negative logarithm. Water, and other chemicals in solution therein, will ionize to a greater or lesser degree.Pure water is only weakly ionized.

The ionization reaction of water may be written:

$$HOH \leftrightarrow H^+ + OH^-$$

The reaction has an equilibrium defined by the equation:

$$(H)(OH)/\,(HOH) = K_w$$

In which HOH, H, OH is the chemical activities of the water hydrogen and hydroxyl ion respectively. Since water is solvent, its activity is defined as being unity. In dilute solution, molar concentrations are frequently substituted for activities yielding

$$(H)(OH) = K_w \ (10^{-14} \text{ at } 20^oC)$$

Taking negative logs of both sides, Log (H) + Log (OH) = -14

$$- \text{Log (H)} - \text{Log (OH)} = 14$$

Defining –Log = p; $pH + POH = 14$

In neutral solutions at equilibrium (OH) = (H), hence PH = POH = 7.

Mathematically it is expressed as; $PH = -\log [H^+] = \log \frac{1}{[H+]} = 7$

Increasing acidity leads to higher values of (H), thus to lower values of PH. Low PH is associated with high acidity, high PH with caustic alkalinity.

PH is important in the control of a number of water treatment and waste treatment processes and in control of corrosion. It may be readily measured potentially by use of a PH meter.

8. Hardness:

Hardness is caused by the sum of the alkali earth elements present in water although the major constituents are usually calcium and magnesium. These materials in water react with soap, causing precipitation which as scum or curd on the water surface. Until enough soap has been dissolved to react with all these material s, no lather can be formed. Water that behaves like this is said to be '**hard** '. The hardness compounds are temporary and permanent:

1. **Temporary hardness (carbonate hardness)**
   - Calcium bicarbonate ($Ca\ (HCO_3)_2$)
   - Magnesium bicarbonate ($Mg\ (HCO_3)_2$)
2. **Permanent hardness' (non- carbonate hardness)**
   - Calcium sulfate ($CaSO_4$)
   - Magnesium chloride ($MgSo_4$)
   - Calcium chloride ($CaCl_2$)
   - Magnesium chloride ($Mg\ cl_2$)

The most usual compounds causing alkalinity, calcium and magnesium bicarbonate, happen also to cause the temporally hardness. Hence, when the alkalinity and hardness are equal, all the hardness is temporary. If the total hardness is greater than the alkalinity, then the excess hardness represents permanent hardness. On the other hand, if the total hardness is less than the alkalinity, the difference indicates the presence of sodium bicarbonate, which adds to the alkalinity but doesn't increase the hardness.

A generally accepted classification of hardness is as follows:

Soft	<509 mg/1 as $CaCo_2$
Moderately hard	50 – 150 mg/1 as $CaCO_3$
Hard	150- 300mg/1 as $CaCO_3$
Very hard	>300 mg/1 as $CaCO_3$

9. Chloride:

Chloride may demonstrate an adverse physiological effect when present in concentration greater than 250 mg/1 and with people who are acclimated. However, a local population that is acclimated to the chloride content may not exhibit adverse effect from excessive chloride concentration. Because of high chloride content of urine, chlorides have sometimes been used as an indication of pollution.

10. Fluoride:

It is generally associated with a few types of sedimentary or igneous rocks; fluoride is seldom found in surface waters and appears in ground water in only few geographical regions. Fluoride is toxic to humans and other animals in large quantities, while small concentrations can beneficial.

Concentrations of approximately 1.0 mg/1 in drinking water help to prevent dental cavities in children. During formation of permanent teeth, fluoride combines chemically with tooth enamel, resulting in harder, stronger teeth that are more resistant to decay. Fluoride is often added to drinking water supplies if quantities for good dental formation are not naturally present.

Excessive intakes of fluoride can result in discoloration of teeth. Noticeable discoloration, called **mottling**, is relatively common when fluoride concentrations in drinking water exceed **2.0 mg/1**, but is rare when concentration is less that 1.5 mg/1.

Adult tooth are not affected by fluoride, although both the benefits and liabilities of fluoride during teeth formation years carry over into adulthood.

Excessive concentrations of grater than **5 mg/1** in drinking water can also result in bone **fluorisis** and other **skeletal abnormalities**.

11. Toxic Organic Chemicals

These include Polynuclear Aromatic Hydrocarbons (PAH); Pesticides. E.g. DDT; Phenol; e.t.c.

12. Lead, Arsenic and Other Metals:

These are toxic substance and are extremely dangerous from public health viewpoint. The presence of lead in excess of **0.1 mg/1** or arsenic in excess of **0.05 mg/1** or Selenium in excess of **0.05 mg/1** or hexavallent Chromium in excess of **0.05 mg/1** shall constitute grounds for rejection of the supply.

3.2.3 Biological Characteristics

A feature of most natural water is that they contain a wide variety of micro – organisms forming a balance ecological system. The types and numbers of the various groups of micro – organisms present are related to water quality and other environmental factors.

Microbiological indicators of water quality or pollution are therefore of particular concern because of their relationships s to human and animal health. Water polluted by pathogenic micro- organisms may penetrate into private and or public water supplies either before or after treatment.

1. Bacterium:

Many are found in water. Some bacteria are indicator of pollution but are harmless; other few in number are pathogenic. Bacterial –born diseases include: typhoid fever, cholera, and bacterial dysentery:

2. Viruses

These are group of infectious which are smaller that ordinary bacteria and that require susceptible host cells for multiplication and activity. Viral-born diseases include infectious hepatitis and poliomyelitis.

3. Algae:

These are small, Chlorophyll bearing generally one–celled plants of varying shapes and sizes which live in water. When present in large numbers they may cause turbidity in water and an apparent color. They cause trouble in water works by undue clogging of filters, but their most troublesome characteristics in the taste and odor that they may cause

4 protozoa:

They are the lowest and simplest forms of animal life. Protozoa–born diseases include giardiasis and amebic dysentery.

5 fungi:

These are non –chlorophyll bearing plants and they may therefore grow in the absence of light. Large numbers die and the decomposition of there will cause disagreeable tastes and odors.

6. Actinomycets:

These are related to both bacteria and fungi; and are responsible for earthy, muddy & misty color in water

3.3 EXAMINATION OF WATER QUALITY

Examination of water is made to help informing an opinion of the suitability of a water supply for public and other uses.

1. Sampling

Necessary to obtain a representation sample in a quantity sufficient for analysis complete preservation of sample is practically impossible; however, freezing or adding suitable preservatives may slow down changes in composition.

Plastic, glass or metal sample containers are able introduce contamination to sample. Normally plastics are used for chemical analysis (except for oil & grease) and glass for bacteriological analysis.

2. Standard Tests

i) Titration (volumetric) method

Using burettes, pipits, and other volumetric glass ware, standard solutions are prepared using analytical and distilled or doionized water.

APHA recommended determinations to be made by titration method are : Chloride (Cl^-), carbonates (CO_3^{2-}) , bicarbonates (HCO_3) , DO, BOD, COD , calcium (Ca^{++}), magnesium (Mg^{++}) , bromide (Br) , hydroxide (OH^-), sulfide(S^-) , sulfite(SO_3^2), acidity , alkalinity etc.

(ii). Colorimetric method (using color as the basis)

Measuring amount of color produced by mixing with reagents at fixed wavelength (using spectrophotometer) or comparison with colored standards or discs (comparator).

APHA recommended determinations made by colorimetric method are : color , turbidity , iron (Fe^{++}), manganese(Mn^{++}), chlorine (Cl_2) , flurried (F^-) , nitrate (NO_3^-) ,nitrite (NO_2) , phosphate (PO_4^{---}) , ammonia (NH_4^+) , arsenic , phenols , etc.

(iii). Gravimetric method (using weight ad the basis)

Using weight of insoluble precipitates or evaporated residues in glassware or metal and accurate analytical balance (+0.001 gm).

APHA recommended determinations made by gravimetric methods are: sulfate (SO4), Oil and grease, TDS, TSS, TS, etc.

(iv). Electrical method

Using probes to measure electrical potential in mill volts against standard cell voltage.

APHA recommended determinations made by electrical methods are: pH, Fluoride (F^-), DO, nitrate (NO_3), etc.

(v). **Flame spectra (emission & absorption) method**

At fixed wave length characteristics to ions being determined measuring intensity of emission or absorption of light produced by ions exited in flame or heated sources.

APHA recommended determinations made by flame spectra methods are ;sodium (Na^+), potassium (K^+) ,lithium (Li^+), etc.

3.4 Water Quality Standards

Public water supplies are obliged to provide a supply of wholesome water which is suitable and safe for drinking purposes.

Potable water is water which is satisfactory for drinking, culinary and domestic purposes. Water quality standards may be set regional, national, or international bodies. Guidelines for drinking water quality have established by the World Health Organization (WHO)as shown in table below.

Table WHO Guideline for drinking water quality

Parameter	Unit	Guideline value
Microbial quality		
Fecal coli forms	Number/ 100 ml	Zero*
Coli form organisms	Number /100 ml	Zero*
Inorganic constituents		
Arsenic	Mg/1	0.05
Cadmium	Mg/1	0.005
Chromium	Mg/1	0.05
Cyanide	Mg/1	0.1
Fluoride	Mg/1	1.5
Lead	Mg/1	0.05
Mercury	Mg/1	0.001
Nitrate	Mg/1	10
Selenium	Mg/1(N)	0.01
Aesthetic Quality		
Aluminum	Mg/1	0,2
Chloride	Mg/1	250
Color	Mg/1	15
Copper	Mg/1	1.0
Hardness	True color unit(TCU)	500
Iron	Mg/1	0.3
Manganese	Mg/1(as $caco_3$)	0.3
PH	Mg/1	6.5 to 8.5
Sodium	Mg/1	200
Total dissolved solids	Mg/1	1000
Sulfate	Mg/1	400
Taste and odor		inoffensive to most
Turbidity	NTU	consumers
Zinc	Mg/1	5
		5.0

* Treated water entering the distribution system

Source – Ethiopian building code standard, plumbing services of building (EBCS -9)

3.4 SOURCES OF WATER POLLUTION

Following are the main sources of water pollution

1. Domestic sewage:

If domestic sewage is not properly after it is produced or if the effluent received at the end of sewage treatment is not of adequate standard, there are chances of water pollution.

The indiscriminate way of hading domestic sewage may lead to the pollution of under ground sources of water supply such, as wells. Similarly if sewage or partly treated sewage is directly discharged into surface waters such as rivers, the waters of such rivers get contained.

2. Industrial wastes:

If industrial wastes are thrown into water bodies without proper treatments, they are likely to pollute the watercourses. The industrial wastes may carry harmful substances such as grease, oil, explosives, highly odorous substances, etc.

3. Catchment area:

Depending upon the characteristics of catchment area, water passing such area will be accordingly contained. The advances made in agricultural activities and extensive use of fertilizers and insecticides are main factors, which may cause serious pollution of surface waters.

4. Distribution system:

The water is delivered to the consumers through a distribution of pipes which are laid underground. If there are cracks in pipes or if joints are leaky, the following water gets contaminated by the surrounding substances around the pipes.

5. Oily wastes:

The discharge of oily wastes from ships and tankers using oil as fuel may lead to pollution.

6. Radioactive wastes:

The discharge of radioactive wastes from industries dealing with radioactive substance may seriously pollute the waters. It may be noted that radioactive substances may not have color, odour, turbidity or taste. They can only be detected by and measured by the use of special precise instruments.

7. Travel of water:

Depending upon the properties of ground through which water travels to reach the source of water supply; it is charged with the impurities. For instance, ground water passing through peaty land possesses brown color.

CHAPTER 4

INTRODUCTION TO WATER TREATMENT

4.1 – Introduction

Absolutely pure water is that which contains only hydrogen and oxygen i.e. H_2o and is never found in nature. The water found nature contains a number of impurities in varying amounts. Therefore, removing these impurities up to certain extent so that it may not be harmful to the public health is necessary, .the process of removing the impurities is called water treatment and the Treated water is called wholesome water.

The following are the requirement of wholesome water

- It should be free from bacteria which may cause diseases
- It should be colorless
- It should be tasty, odor free and cool.
- It should not corrode pipes.
- It should not free from all objectionable matter.
- It should have dissolved oxygen and free carbonic acid that it may remain fresh.

The common impurities found water and their effects as follows: -

4.2 METHODS OF WATER TREATMENT

The common methods of water treatment (water purification) are: -

1. Aeration
2. Screening
3. Plain sedimentation
4. Coagulation
5. Filtration
6. Adsorption
7. Softening
8. Disinfections

The degree and methods of treatment depend upon

- Nature of the source
- Quality of the source
- Purpose for which it supplied

1. Aeration

It is the process of bringing water in intimate contact with air, while doing so water absorbs oxygen from the air. Aeration may be used to remove undesirable gases dissolved in water i.e. Co_2, H_2S, etc (degasification) or to add oxygen to water to convert undesirable substance i.e. Iron (Fe^{2+}) & Manganese to more manageable form (oxidation).

The Iron and Manganese may be removed as a precipitate after aeration. Chemically, these reactions may be written as follows:

$$4\,Fe^{2+} + O_2 + 10\,H_2O \rightarrow 4\,Fe(Oh)_3\downarrow + 8H^+$$

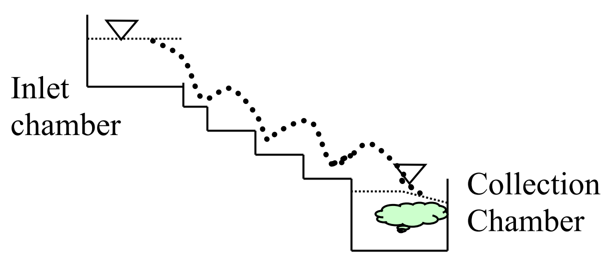
$$4\,Mn^{2+} + O_2 + 2\,H_2O \rightarrow 2\,MnO_2\downarrow + 4H^+$$

Different types of aerators are available

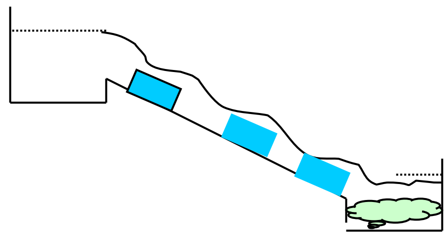
- Gravity Aerator
- Spray aerator
- Air diffuser
- Mechanical Aerator

1. Gravity aerators

a) Cascade towers



(b) Inclined apron possibly shaded with plates



c) **Tray aerator**

In tray aerator water falls through a series of trays perforated with small holes, 5-12 mm diameter and 25-75 mm spacing center to center. They are often built in stacks of 4-6 trays giving a total height of 1.2-3m. The trays may be filled with layers of coke or gravel of 50mm size to insure purification.

2 **spray aerators**: - spray droplets of water in to air from stationary or moving orifices or nozzles. Water is pumped through pressure nozzles to spray in the open air as in fountain to a height of about 2.5m.

3. **Air diffuser**

4. **Mechanical Aerator**

2. Screening

Screening usually involves a simple screening or straining operation to remove large solids and floating matter as leaves, dead animals, etc.

a) Bar screens- with openings of about 75 mm
b) Mesh screens –with opening of 5-20 mm

3. Plain Sedimentation

Sedimentation is the removal of particles (silt, sand, clay, etc.) through gravity setting in basins. No chemicals is to enhance the sedimentation process

Principle of plain sedimentation-Discrete particles

The sedimentation of discrete spherical particles may be described by Newton's Law, from which the terminal settling velocity is found to be:

$V_s =$ ……………………………………….(1)

Where V_s = the terminal settling velocity (L/T)

ρ_s = the mass density of the particle (M/L^3)

ρ = the mass density of the fluid (M/L^3)

g = the acceleration due to gravity (L/T^2)

d = the diameter of the particle (L)

C_d = is dimensionless drag coefficient defined by

$C_d =$ ………………………………..(2)

In which N_R is the Reynolds number

$N_R = V_s d * \rho / \mu$

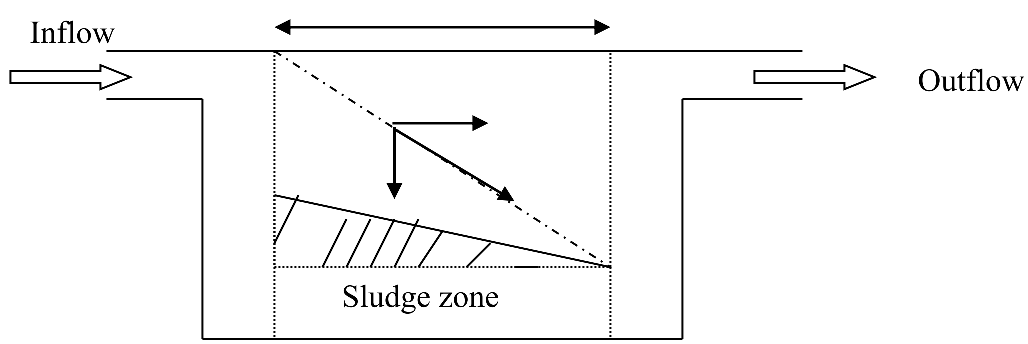
M = dynamic viscosity

When N_R is small (less than 0.5)

$C_d = 24/N_R = 24 \mu / V_s d \rho$ …………………………………………(3)

Substituting equation (3) into (1) yields

$V_s = g (\rho_s - \rho) d^2 / 18 \mu$…………………….***Stoke's Law.***



4. Coagulation (chemically assisted sedimentation)

Very fine suspend clay particles are not removed by plain sedimentation .in addition, water also contains electrically charged colloidal matter which are continuously in motion and never settle down due to gravitational forces. Such impurities i.e. fine clay and colloidal matter can be removed by the use of chemicals followed by sedimentation and such type of water treatment is called **Coagulation**.

The principle of coagulation can be explained from the following two conditions:

1. Floc formation

When coagulants (chemicals) are dissolved in water and thoroughly mixed with it, they produce a think gelatinous precipitate. This precipitate is known as **floc** and this floc has got the property of arresting suspended impurities in water during downward travel towards the bottom of tank.

2 Electric charges

The ions of floc are found to possess positive charge. Hence, they will attract the negatively charged colloidal particles and thus they cause the removal of such particles

3. Flocculation

Flocculation is used to denote the process of floc formation and thus follows addition of coagulants. Flocculators are slow stirring mechanisms, which form floc. They mostly consist of paddles, which are revolving at very slow speed about 2-3 rpm. The folcculators provide numbers of gentle contacts between the flocculating particles, which are necessary for the successful formation of floc. In this operation the floc, which has been formed above, is allowed to settle and is separated from the water by keeping the water in sedimentation tanks.

The following are the most commonly used coagulants:

1) Aluminum sulfate [$Al_2(SO_4)_3 18H2O$] .it is also called Alum

It reacts quite quickly giving excellent stable floc. It reacts with the natural alkalinity in water & if natural alkalinity is not sufficient, lime may be added and forms aluminum hydroxide floc.

Chemical Reaction Taking Place

i) $Al_2(SO_4)\ 3.18H2O + 3Ca\ (HCO_3)_2 \rightarrow 2Al(OH)_3\downarrow + 3CaSO_4 + 6CO_2 + 18\ H_2O$

ii) $Al\ (SO_4)_3.18H2O + 3Ca(OH)_2 \rightarrow 2Al(OH)_3\downarrow + 3CaSO_4 + 18H_2O$

iii) $Al_2\ (SO_4)_3 18H2O + 3Na_2CO_3 \rightarrow 2Al(OH)_3\downarrow + 3Na_2SO_4 + 3CO_2 + 18H_2O$

2) **Sodium aluminates ($Na_2Al_2O_4$)**

Chemical Reaction Taking Place

i) $Na_2Al_2O_4 + Ca\ (HCO_3)_2 \rightarrow CaAl_2O_4\downarrow + Na_2CO_3 + CO2 + H_2O$

ii) $Na_2Al_2O_4 + CaSO_4 \rightarrow CaAL_2O_4\downarrow + Na_2SO_4$

iii) $NaAl_2O_4 + CaCl_2 \rightarrow CaAl_2O_4\downarrow 2NaCl$

3) Ferric coagulants

… Ferric Chloride $Fe\ Cl_3$

… Ferric sulfate $Fe_2\ (SO_4)_3$

… Mixture of both ($Fe\ cl_3 = Fe_2\ (SO_4)_3$)

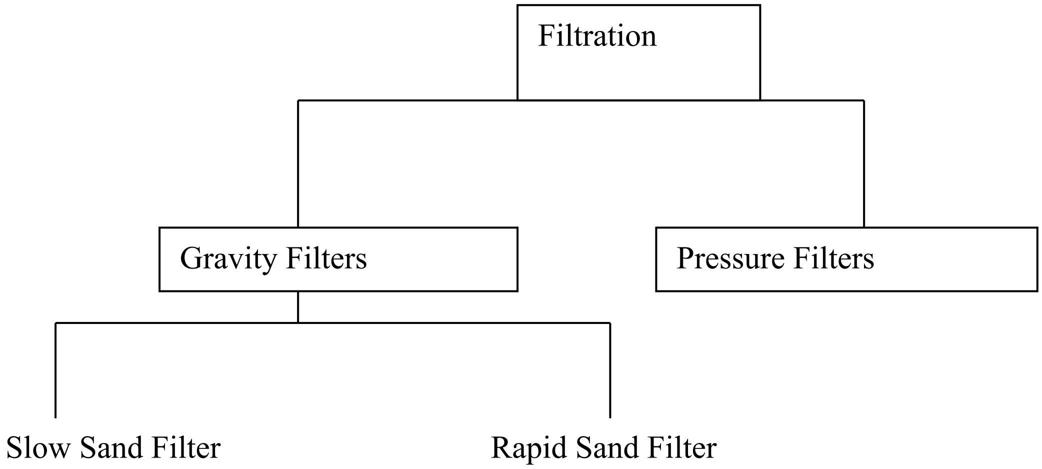
Chemical reaction taking place

$2Fe\ Cl_3 + 3Ca\ (OH)_2 \rightarrow 2Fe\ (OH)_3\downarrow + CaCl_2$

$Fe_2\ (SO_4)_3 + 3Ca\ (OH)_2 \rightarrow 2Fe\ (OH)_3\downarrow + 3CaSO_4$

5. Filtration

The effluent obtained after coagulation does not satisfy the drinking water standard and is not safe. So it requires further treatments. If water is allowed to pass through a bed of sand or fine granular material, the effluent obtained is clear and sparkling with negligible turbidity. This process is known as **filtration**. Filtration also removes bacteria, taste &odor.



1. SLOW SAND FILTERS

In slow filters a watertight tank is construct either in stone masonry or brick masonry and a layer of sand is placed over the gravel. The depth of sand varies from 60 to 90 cm. The depth of gravel varies from 30 to 60 cm.

The water is allowed to enter the filter thought the inlet chamber. It descends through the filter media and during this process it gets purified.
The purpose of cleaning, the top layer of sand is scrapped or removed through a depth of about 15-25 mm. The water is then admitted to the filter. When cleaning of filter had been done for a number of times, the effective depth of filter media of sand is reduced. In order to maintain the efficiency of filter, a fresh layer of about 15 cm depth of graded sand is then added to the filter.

The interval between two successive cleanings depends mainly on the nature of impurities present in the water to be treated. It usually varies from 1 to3 months.

The rate of filtration for a normal slow sand filter varies 100 to200 lit/hr/m2 of fitter area.

Example: find the area of slow sand filters for a town having a design population of 15000 with an average rate of demand as 160 l/p/d.

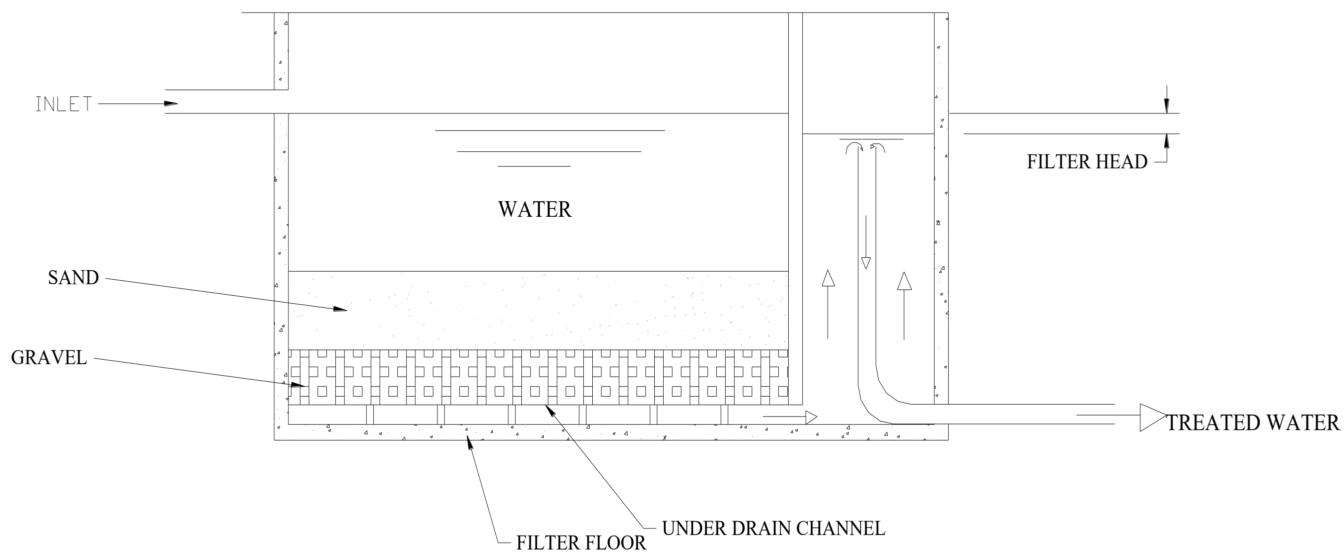


Fig 5. A Typical Section of Slow Sand Filter.

2. Rapid sand filters

The rapid filter is the most common type used in water treatment to remove non-settle able floc remaining after chemical coagulation and sedimentation. A typical rapid sand filter bed (figure) is placed in a concrete box with a depth of about 2.7 m. Graded gravel layer containing under drains supports the sand filter, about 0.6 m deep. During filtration, water passes down ward through the filter bed by a combination of water pressure form above and suction from the bottom. Filters are cleaned by backwashing (reversing the flow) up ward through the bed. Wash troughs suspended above the filter surface collect the back wash water and carry it out of the filter box.

The following descriptions of filter operation follow the valve numbering in figure. Initially valves 1 and 4 are opened, and 2, 3, and 5 are closed for filtration. Overflow from the setting basing supplied to the filer passes through the bed and under drain system to the clear well underneath. The depth of water above the filter surface is between 0.9 and 1.3m. The under drainpipe is trapped in the clear well to provide liquid connection to the water being filtered, thus preventing backflow of air into the under drain.

The maximum head available for filtration is equal to the difference between the elevation of the water surface above the filter and level in the clean; this is commonly 2.7 to 3.7 m.

The bed is cleaned by backwashing. Valves 1 and 4 closed (3 remaining closed) and 2 &5 are opened. Clear water flows into filter under drain and passes upward though the bed.

The sand layer expands hydraulically about 50 percent and the sand grains are scrubbed by rubbing against each other in the turbulent backwash flow. Dirty wash water is collected by troughs and conveyed to disposal.

The first few minutes of filtered water at the beginning of the next run generally wasted to flush the wash water remaining in the bed out through the drain. This is accomplished by opening valve 3 when valve 1 is opened to start filtration (valves 2, 4, and 5 are shut).

Finally, opening valve 4 closing 3 permits filtration to proceed again.

Example: Find the area of rapid sand filters for a town having a design population of 80000 with an average rate of demand as 200 l/c/d.

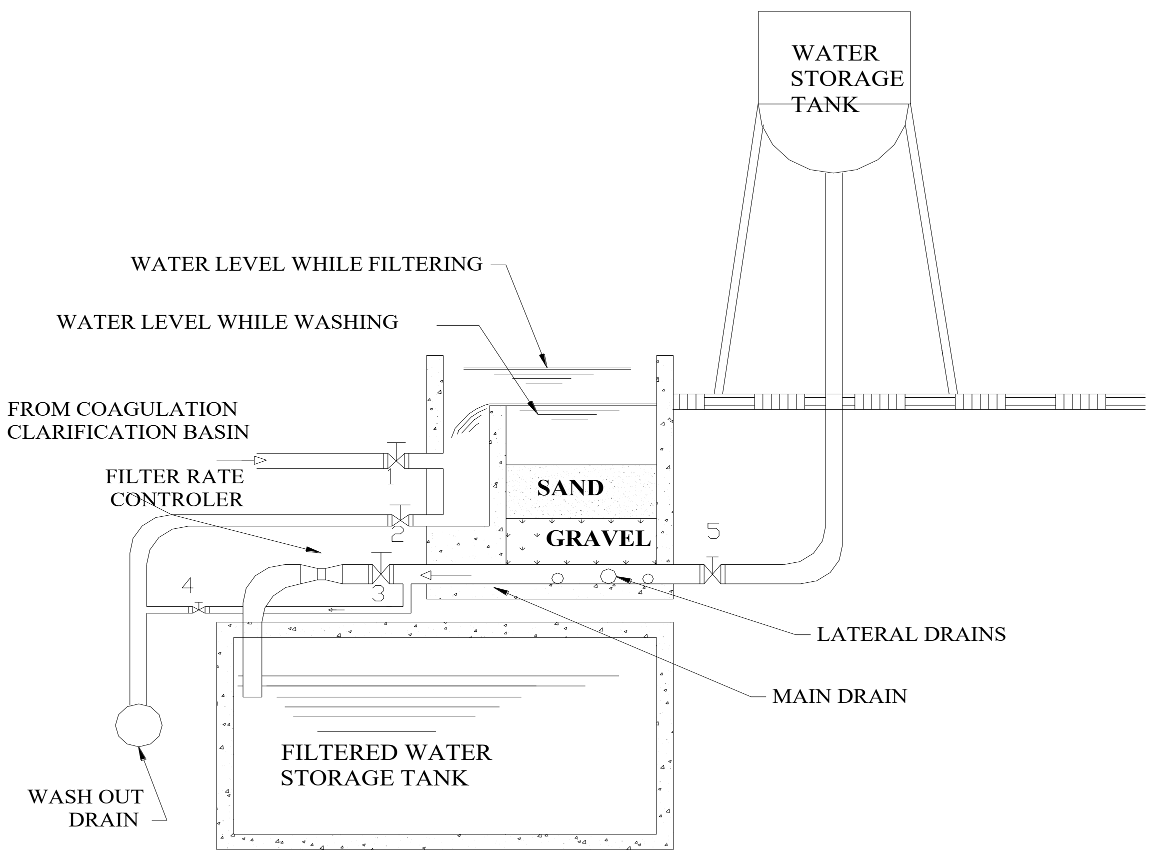


Fig 5… Rapid Sand Filter

Difference between Sand Filters and Rapid Sand Filters

Sr.No	Item	Slow sand filters	Rapid sand filters
1.	Coagulation	Not required	Essential
2	Compactness	Requires large area for its installation	Requires small area for its installation
3	Construction	Simple	Complicated
4	Economy	High initial cost both land & material	Cheap and quite economical
5	Method of cleaning	Scrapping of top layer of 15 to25 mm thickness. Long and laborious method	Agitation and back washing with or with out the help of compressed air short and speedy method
6	Period of cleaning	1 to3 months	2 to 3 days
7	Rate of filtration	100 to200 lit/hr/$m^2$of filter area	3000 to 6000 lit/hr/m^2
8	Skilled supervision	Not essential	Essential
9	Suitability	The filter can be constructed of local labor and material .it is suitable for small towns and villages where land is cheaply available.	It is suitable for big cites where land cost is high

3. Pressure Filters

Pressure filters have media and under drains contained in a steel tank. Media are similar to those used in gravity filters, and filtration rates from 1.4 to 2.7 lit/m^2/sec.
Water is pumped through the bed under pressure, and the unit is backwashed by reversing the flow, flushing out the impurities.
Pressure filters are not generally employed in lager treatment works; however they are popular in small municipal. Their most extensive application is for treating water for industrial purposes (e.g. pressure filters are in arbaminch textile factory)

6. Adsorption

Adsorption can be defined as the accumulation of substances at the interface between two phases. The material removed from the water is called **the adsorbent**, and the material providing the solid surfaces is called **adsorbent**.
The adsorbent most commonly used in water treatment is **activated carbon**. It is manufactured from carbonaceous material such as **wood, coal, petroleum residues** etc. The activated carbon is used for removal of taste and odors from water as it has excellent properties of attracting impurities such as gases, finely divided solid particles and other liquid impurities.

7. Softening

Softening is the removal or reduction of hardness from the water. Hard water causes the following troubles:

1. Requires more soap for washing clothes
2. The precipitate formed by the action of the soap spoils the cloth
3. Choking and clogging troubles of pipe lines due to precipitation of salt causing hardness
4. Formation of scales in boilers

For all these reasons removal of hardness or softening of water is required. There are two methods generally used for softening of water.

1) **The lime –Soda Process**

In the method hardness is removed by using lime [$Ca(OH)_2$] and soda ash [Na_2Co_3]. The following are the reactions taking place:-

1. $Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow CaCO_3\downarrow + 2H_2O$
2. $Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow Mg(OH)_2\downarrow + CaCO_3\downarrow + H_2O$
3. $MgSO_4 + Ca(OH)_2 + NaCO_3 \rightarrow Mg(OH)_2\downarrow + CaCO_3\downarrow + Na_2SO_4$
4. $MgCl_2 + Ca(OH)_2 + NaCO_3 \rightarrow Mg(OH)_2\downarrow + CaCO_3\downarrow + 2Nacl$

2) The ion exchange process

In this method hard water is passed thought a bed of **zeolite sand** (complex silicates of Al&Na), while passing through it Ca &Mg cations get replaced by sodium from the exchanger and the water becomes soft.

The sodium from the zeolite sand on getting exhausted and after some time it cannot remove the hardness of the water. But the sodium of the zeolite is generated by applying a solution of sodium chloride (called brine).

The reactions involved are:

(a) Softening

$$Na_2R + \boxed{\begin{array}{l} Ca\ (OHCO_3)_2 \\ CASO_4 \\ CACI_2 \\ Mg\ (HCO_3)_2 \\ MgSO_4 \\ MgCl_2 \end{array}} \longrightarrow \boxed{\begin{array}{l} Ca \\ Ca \\ Ca \\ Mg \\ Mg \\ Mg \end{array}} \boxed{R} + \boxed{\begin{array}{l} 2\ NaHCO_3 \\ Na_2SO_4 \\ 2NaCl \\ 2NaHCO_3 \\ Na_2SO_4 \\ 2NaCl \end{array}}$$

(b) Regeneration

$$CaR + 2NaCl \text{ — } Na2R + \begin{array}{l} CaCl_2 \\ MgCl_2 \end{array}$$

9. Disinfection

Disinfect ion is the process of killing pathogenic microorganisms in order to avoid water born diseases

The disinfection of water can be done by one of the following methods:

a) By the boiling of water
b) By ultra –violate rays
c) By the use of iodine and bromine
d) By the use of ozone O_3
e) By the use of excess lime
f) By the using potassium permanganate [$KMnO_4$]
g) By the use of chlorine

The most common method of disinfection is the use of chlorine i.e. **chlorination.** The various chlorine compounds which are available in the market and used as disinfectants are

1. Calcium hypo chlorite [$Ca\ (OCl)_2$] – **power from**
2. Sodium hypo chlorite [NaOCl] –**liquid from**
3. Free chlorine Cl_2- **Gaseous form**

4.3. Remarks in Water Treatment

1) The processes selected for the treatment of potable water depend on the quality of raw water supply.

2) Most ground waters are clear and pathogen-free and not contain significant amounts of organic materials. Such waters may often be used directly with a minimal does of chlorine to prevent contamination in the distribution system.

3) Other ground waters may contain large quantities of dissolved solids or gases. When these include excessive amounts of iron, manganese, or hardness, treatment methods described in section 4.2 may be required. Treatment systems commonly used to prepare potable water from such ground waters are shown in figures 5.1

4) Surface waters often contain a wide variety of contaminants than ground water, and treatment processes may be more complex. Most surface waters contain turbidity in excess of drinking water standards. Wide variety microorganisms, some of which may be pathogenic are common constituents of surface water. Treatment systems commonly used in treating surface waters are shown in figure 5.2

5) Ground waters like springs can directly be used after chlorination, as in the case of **ArbaMinch Water Supply**.

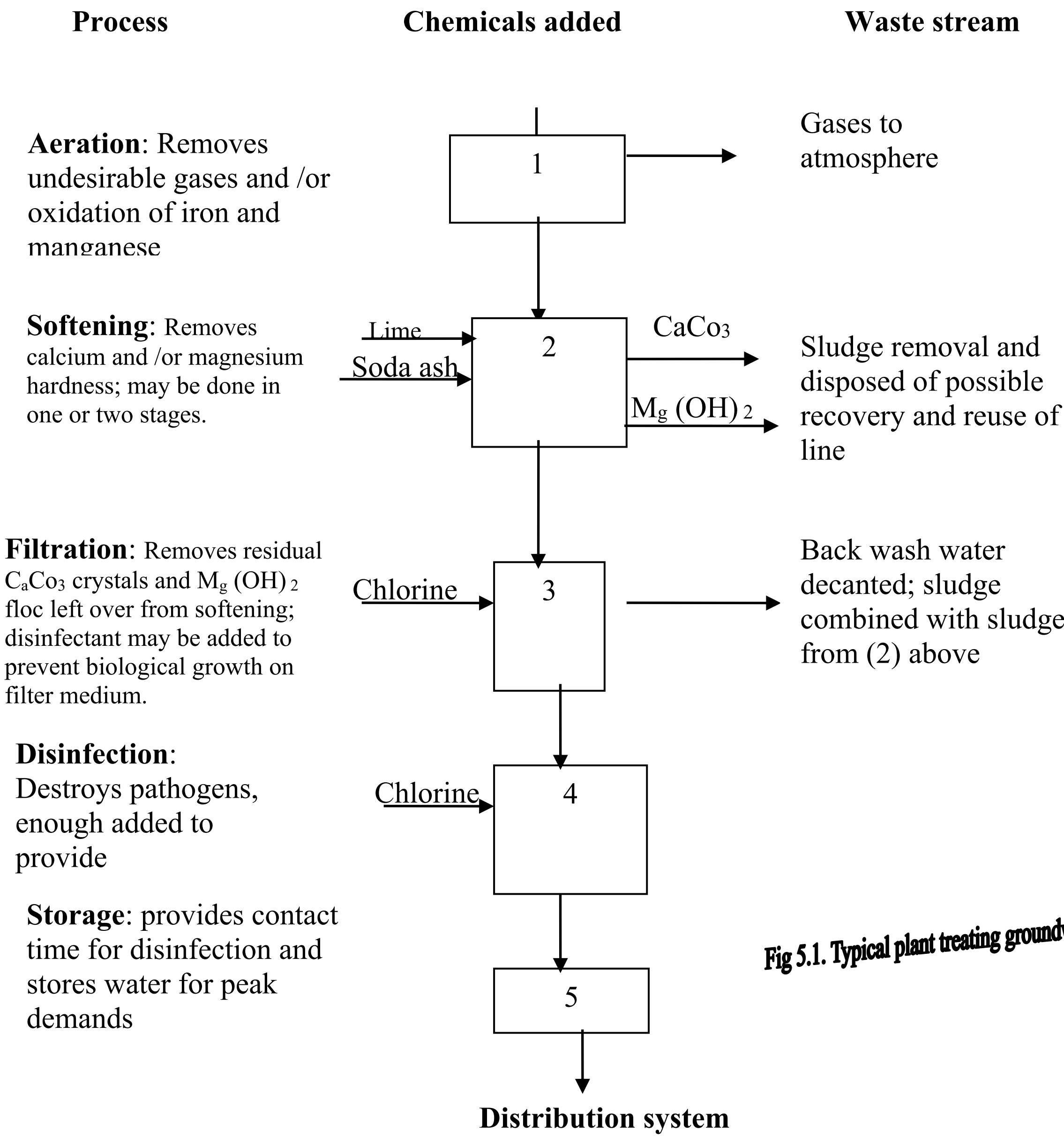


Fig 5.1. Typical plant treating groundwater

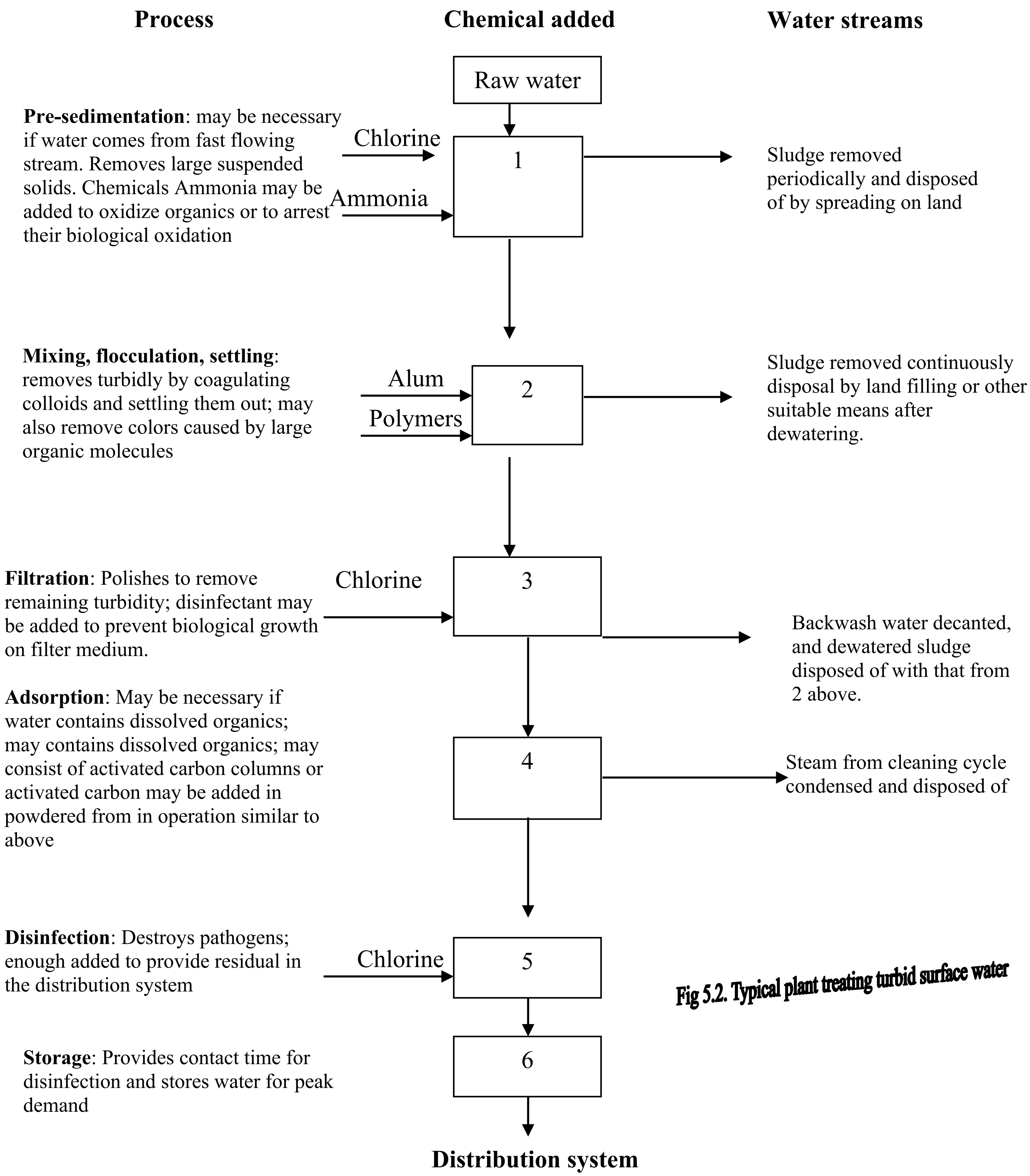


Fig 5.2. Typical plant treating turbid surface water

Urban Drainage part

Lecture 1: Introduction

1.1 BACKGROUND

Urbanization has encouraged the migration of people from villages to the urban areas. This has given rise to a number of environmental problems such as, water supply with desirable quality and quantity, wastewater generation and its collection, treatment and disposal. In urban areas for domestic and industrial uses the source of water is generally reservoir, river, lake, and wells. Out of this total water supplied, generally 60 to 80% contributes as a wastewater. In most of the cities, wastewater is let out partially treated or untreated and it either percolates into the ground and in turn contaminates the ground water or it is discharged into the natural drainage system causing pollution in downstream water bodies.

The importance of water quality as a factor constraining water use has often gone unacknowledged in the analyses of water scarcity. Water scarcity is a function not only of volumetric supply, but also of quality sufficient to meet the demand. The drinking water demand is perhaps the largest demand for high quality water apart from many industrial uses which also require high quality water. Agriculture, by far the largest consumer of water, also suffers when water supplies become saline. In India, water pollution comes from the main sources such as domestic sewage, industrial effluents, leachets from landfills, and run-off from solid waste dumps and agriculture land. Domestic sewage (black water) and sullage (grey water) is the main source of water pollution in India, especially in and around large urban centers. The regular monitoring of the water quality in the rivers and wells in the country revealed that the total coliform counts far exceeds the desired level in water to be fit for human consumption [CPCB, 1997].

In the past disposal of waste from water closets was carried out manually and wastewater generated from kitchen and bathrooms was allowed to flow along the open drains. This primitive method was modified and replace by a water carriage system, in which these wastes are mixed with sufficient quantity of water. This waste is carried through closed conduits under the conditions of gravity flow. This mixture of water and waste products is known as sewage.

The **advantages** offered by the water carriage system are:

- The carriage of wastes on head or carts is not required.
- Bad smell, which was unavoidable during open transport of sewage, is not occurring due to transport of this polluted water in closed conduits.
- The old system was posing the health hazards to sweepers and to the nearby residents, because of the possibilities of flies and insects transmitting disease germs from the accessible carts to the residents food eatables. This is avoided in water carriage system because of transport of night soil in close conduits.
- The human excreta is washed away as soon as it is produced in water carriage system, thus storing is not required as required in the old system of manual disposal. Thus, no bad smells are produced in closed conduit transport.
- In the old system, the wastewater generated from the kitchen and bathrooms was required to be carried through open roadside drains for disposal. This is avoided in sewerage system as the open drains could generate bad odours when used for disposal of organic wastes.
- The water carriage system does not occupy floor area, as the sewers are laid underground.
- In addition, the construction of toilets one above the other is possible in water carriage system and combining latrine and bathrooms together as water closets is possible. This is one of the important advantages of water carriage system.

However, this water carriage system also has certain **drawbacks** such as:

- A large network of pipes is required for collection of the sewage; hence, the capital cost for water carriage system is very high.
- In addition, the operation and maintenance of sewerage system is very expensive.
- Large wastewater volume is required to be treated before disposal.
- Assured water supply is essential for efficient operation of the water carriage system.

1.2 DEFINITIONS

Industrial wastewater: It is the wastewater generated from the industrial and commercial areas. This wastewater contains objectionable organic and inorganic compounds that may not be amenable to conventional treatment processes.

Night Soil: It is a term used to indicate the human and animal excreta.

Sanitary sewage: Sewage originated from the residential buildings comes under this category. This is very foul in nature. It is the wastewater generated from the lavatory basins, urinals and water closets of residential buildings, office building, theatre and other institutions. It is also referred as domestic wastewater.

Sewage: It indicates the liquid waste originating from the domestic uses of water. It includes sullage, discharge from toilets, urinals, wastewater generated from commercial establishments, institutions, industrial establishments and also the groundwater and stormwater that may enter into the sewers. Its decomposition produces large quantities of malodorous gases, and it contains numerous pathogenic or disease producing bacteria, along with high concentration of organic matter and suspended solids.

Sewage Treatment Plant is a facility designed to receive the waste from domestic, commercial and industrial sources and to remove materials that damage water quality and compromise public health and safety when discharged into water receiving systems or land. It is combination of unit operations and unit processes developed to treat the sewage to desirable standards to suit effluent norms defined by regulating authority.

Sewer: It is an underground conduit or drain through which sewage is carried to a point of discharge or disposal. There are three types of sewer systems that are commonly used for sewage collection. Separate sewers are those which carry the house hold and industrial wastes only. Storm water drains are those which carry rain water from the roofs and street surfaces. Combine sewers are those which carry both sewage and storm water together in the same conduit. House sewer (or drain) is used to discharge the sewage from a building to a street sewer. Lateral sewer is a sewer which collects sewage directly from the household buildings. Branch sewer or submain sewer is a sewer which receives sewage from a relatively small area. Main sewer or trunk sewer is a sewer that receives sewage from many tributary branches and sewers, serving as an outlet for a large territory. Depressed sewer is a section of sewer constructed lower than adjacent sections to pass beneath an obstacle or obstruction. It runs full under the force of gravity and at greater than atmospheric pressure. The sewage enters and leaves the depressed sewer at atmospheric pressure. Intercepting sewer is a sewer laid transversely to main sewer system to intercept the dry weather flow of sewage and additional surface and storm water as may be desirable. An intercepting sewer is usually a large sewer, flowing parallel to a natural drainage channel, into which a number of main or outfall sewers discharge. Outfall sewer receives entire sewage from the collection system and finally it is discharged to a common point. Relief sewer or overflow sewer is used to carry the flow in excess of the capacity of an existing sewer.

Sewerage: The term sewerage refers the infrastructure which includes device, equipment and appurtenances for the collection, transportation and pumping of sewage, but excluding works for the treatment of sewage. Basically it is a water carriage system designed and constructed for collecting and carrying of sewage through sewers.

Stormwater: It indicates the rain water of the locality.

Subsoil water: Groundwater that enters into the sewers through leakages is called subsoil water.

Sullage: This refers to the wastewater generated from bathrooms, kitchens, washing place and wash basins, etc. Composition of this waste does not involve higher concentration of organic matter and it is less polluted water as compared to sewage.

Wastewater: The term *wastewater* includes both organic and inorganic constituents, in soluble or suspended form, and mineral content of liquid waste carried through liquid media. Generally the organic portion of the wastewater undergoes biological decompositions and the mineral matter may combine with water to form dissolved solids.

1.3 SOURCES OF SEWAGE

The wastewater generated from the household activities contributes to the major part of the sewage. The wastewater generated from recreational activities, public utilities, commercial complexes, and institutions is also discharged in to sewers. The wastewater discharged from small and medium scale industries situated within the municipal limits and discharging partially treated or untreated wastewater in to the sewers also contributes for municipal wastewater.

1.4 SEWAGE DISCHARGE

The quality of sewage and its characteristics show a marked range of hourly variation and hence peak, average and minimum discharge are important considerations. The process loadings in the sewage treatment are based on the daily average characteristics as determined from a 24 hours weighted composite samples. In the absence of any data an average quantity of 150 LPCD may be adopted for design. The hydraulic design load varies from component to component of the treatment plant with all appurtenances, conduits, channels *etc.*, being designed for the maximum discharge, which may vary from 2.0 to 3.5 times the average discharge. Sedimentation tanks are designed on the basis of average discharge, while consideration of both maximum and minimum discharge is important in the design of screens and grit chambers. Secondary treatment is generally designed for average discharge, with sufficient safety margin to accommodate the peak discharge.

1.5 EFFECT OF UNTREATED WASTEWATER DISPOSAL

The daily activities of human beings produce both liquid and solid wastes. The liquid portion of the wastewater is necessarily the water supplied by the authority or through private water sources, after it has fouled by variety of uses. The sources of wastewater generation can be defined as a combination of the liquid or water-carried wastes removed from residences, institutions, and commercial and industrial establishments, together with groundwater, surface water, and storm water as may be present.

If the untreated wastewater is allowed to accumulate, it will lead to highly unhygienic conditions. The organic matter present in the wastewater will undergo decomposition with production of large quantities of malodorous gases. If the wastewater is discharged without treatment in the water body, this will result in the depletion of Dissolved Oxygen (DO) from the water bodies. Due to depletion of DO, the survival of aquatic life will become difficult, finally leading to anaerobic conditions in the receiving waters. The nutrients present in the wastewater can stimulate the growth of aquatic plants, leading to problems like eutrophication. In addition, the untreated domestic wastewater usually contains numerous pathogenic or disease causing microorganisms, that dwell in the human intestinal tract or it may be present in certain industrial wastewaters. Apart from this, the wastewater contains inorganic gritty materials. The continuous deposition of this inorganic material may reduce the capacity of water body considerably over a period.

Generally domestic sewage does not contain any inorganic matter or organic compounds in highly toxic concentration. However, depending upon the type of industries discharging into the public sewers and the dilution that is offered by sewage; the municipal wastewater may have these inorganic substances or toxic organic compounds with the concentration more than the discharge limits stipulated by the authorities. Certain compounds, such as sulphates, metals such as chromium, etc., if presents in higher concentration, may disturb the secondary treatment of the sewage.

1.6 OBJECTIVES OF SEWAGE COLLECTION AND DISPOSAL

The objective of sewage collection and disposal is to ensure that sewage discharged from communities is properly collected, transported, treated to the required degree so as not to cause danger to human health or unacceptable damage to the natural environment and finally disposed off without causing any health or environmental problems. Thus, efficient sewerage scheme can achieve the following:

- To provide a good sanitary environmental condition of city protecting public health.
- To dispose the human excreta to a safe place by a safe and protective means.
- To dispose of all liquid waste generated from community to a proper place to prevent a favorable condition for mosquito breeding, fly developing or bacteria growing.
- To treat the sewage, as per needs, so as not to endanger the body of water or groundwater or land to get polluted where it is finally disposed off. Thus, it protects the receiving environment from degradation or contamination.

1.7 WASTEWATER TREATMENT

The treatment and safe disposal of wastewater is necessary. This will facilitate protection of environment and environmental conservation, because the wastewater collected from cities and towns must ultimately be returned to receiving water body or to the land or reused to fulfill certain needs. The sewage treatment plants constructed near the end of nineteenth century were designed to remove suspended matter alone by the principal of simple gravity settling. It soon became apparent that primary treatment alone was insufficient to protect the water quality of the receiving water body. This was mainly due to the presence of organic material, in colloidal and dissolved form, in the sewage after settling. Thus, in the beginning of twentieth century several treatment systems, called secondary treatment, were developed with the objective of organic matter removal. For this secondary treatment, biological methods are generally used. The aerobic biological treatment processes were popularly used as a secondary treatment, and these processes are still at the first choice.

In the second half of twentieth century, it became clear that the discharge of effluents from even the most efficient secondary treatment plant could lead to the deterioration of the quality of receiving water body. This could be attributed partly to the discharge of ammonia in the effluent. In the receiving water body this discharge exerts an oxygen demand for the biological oxidation of ammonia to nitrate, a process called nitrification. However, even when nitrification is carried out at the treatment plant itself, the discharge of effluent can still be detrimental to the water quality due to introduction of nitrogen in the form of nitrate and phosphorus as phosphate. The tolerance limits of nitrates for the water when used as raw water for public water supplies and bathing ghats is 50 mg/L as NO_3. The availability of nitrogen and phosphorous tends to cause an excessive growth of aquatic life notably, autotrophic organisms such as algae, that can use carbon dioxide rather than organic material as a sources for cell synthesis. Thus, explosive development of biomass may occur when nitrogen and phosphorus are abundantly available. Although, this biomass may produce photosynthetic oxygen in the water during daytime, after sunset it will consume oxygen, so that the dissolved oxygen concentration will decrease and may reach to the levels that are too low to sustain the life of other (macro) organisms. This phenomenon of eutrophication has led to the development of tertiary treatment systems. In these, nitrogen and/or phosphorus are removed, along with solids and organic materials.

Once the minimum effluent quality has been specified, for maximum allowable concentrations of solids (both suspended and dissolved), organic matter, nutrients, and pathogens, the objective of the treatment is to attain reliably the set standards. The role of design engineer is to develop a treatment scheme that will guarantee the technical feasibility of the scheme, taking into consideration other factors such as construction and maintenance costs, the availability of construction materials and equipment, as well as specialized skilled personals for operation and maintenance of the treatment plant.

Primary treatment consists of screens (for removal of floating matter), grit chamber (for removal of inorganic suspended solids) and primary sedimentation tank (for removal residual settleable solids which are mostly organic). Skimming tanks may be used for removal of oils; however, in conventional treatment plant no separate skimming tank is used and oil removal is achieved by collecting the scum in primary sedimentation tank. This primary treatment alone will not produce an effluent with an acceptable residual organic material concentration. Almost invariably biological methods are used in the treatment systems to effect secondary treatment for removal of organic material. In biological treatment systems, the organic material is metabolized by bacteria. Depending upon the requirement for the final effluent quality, tertiary treatment methods and/or pathogen removal may be included.

Today majority of wastewater treatment plants uses aerobic metabolism for the removal of organic matter. The popularly used aerobic processes are the activated sludge process, oxidation ditch, trickling filter, and aerated lagoons. Stabilization ponds use both the aerobic and anaerobic mechanisms. In the recent years, due to increase in power cost and subsequent increase in operation cost of aerobic processes, more attention is being paid for the use of anaerobic treatment systems for the treatment of wastewater including sewage. Recently the high anaerobic process such as Upflow Anaerobic Sludge Blanket (UASB) reactor is used for sewage treatment at many places.

Depending on the mode of disposal the tertiary treatment may be given for killing pathogens, nutrient removal, suspended solids removal, etc. Generally secondary treatment followed by disinfection will meet the effluent standards for disposal into water bodies. When the treated sewage is disposed off on land for irrigation, the level of disinfection needs will depend on the type of secondary treatment and type of crops with restricted or unrestricted public access.

Questions

1. Describe advantages and disadvantages offered by the water carriage system.
2. What are the possible adverse effects when untreated or partially treated sewage is discharged to the environment?
3. Why it is necessary to treat wastewater before disposal? What is the objective of the sewerage works?
4. Define sewage, sullage, sewer, and sewerage.

Lecture 2: System of Sanitation

2.1 BACKGROUND

For safe disposal of the sewage generated from a locality efficient collection, conveyance, adequate treatment and proper disposal of treated sewage is necessary. To achieve this, following conditions should be satisfied:

1. Sewage should not pollute the drinking water source, either surface or groundwater, or water bodies that are used for bathing or recreational purposes.
2. The untreated sewage during conveyance should not be exposed so as to have access to human being or animals and should not give unsightly appearances or odour nuisance, and should not become a place for breeding flies.
3. It should not cause harm to public health and adversely affect the receiving environment.

The collection system is meant for collection of the sewage generated from individual houses and transporting it to a common point where it can be treated as per the needs before disposal. In olden days, waste generated from water closets was collected by conservancy methods and other liquid waste was transported through open drain to finally join natural drains. Since, the excreta was carried through carts, it was not hygienic method for transportation to the disposal point. Now, collection and conveyance of sewage is done in water carriage system, where it is transported in closed conduit using water as a medium.

2.2 TYPES OF SEWERAGE SYSTEM

The sewerage system can be of following three types:

Combined system: In combined system along with domestic sewage, the run-off resulting from storms is carried through the same conduit of sewerage system. In countries like India where actual rainy days are very few, this system will face the problem of maintaining self cleansing velocity in the sewers during dry season, as the sewage discharge may be far lower as compared to the design discharge after including storm water.

Separate System: In separate system, separate conduits are used; one carrying sewage and other carrying storm water run-off. The storm water collected can be directly discharged into the water body since the run-off is not as foul as sewage and no treatment is generally provided. Whereas, the sewage collected from the city is treated adequately before it is discharged into the water body or used for irrigation to meet desired standards. Separate system is advantageous and economical for big towns.

Partially separate system: In this system part of the storm water especially collected from roofs and paved courtyards of the buildings is admitted in the same drain along with sewage from residences and institutions, etc. The storm water from the other places is collected separately using separate storm water conduits.

2.2.1 Advantages and disadvantages of combined system

Advantages

- In an area where rainfall is spread throughout a year, there is no need of flushing of sewers, as self cleansing velocity will be developed due to more quantity because of addition of storm water.
- Only one set of pipe will be required for house plumbing.
- In congested areas it is easy to lay only one pipe rather than two pipes as required in other systems.

Disadvantages

- Not suitable for the area with small period of rainfall in a year, because dry weather flow will be small due to which self cleansing velocity may not develop in sewers, resulting in silting.
- Large flow is required to be treated at sewage treatment plant before disposal, hence resulting in higher capital and operating cost of the treatment plant.
- When pumping is required this system is uneconomical.
- During rains overflowing of sewers will spoil public hygiene.

2.2.2 Advantages and disadvantages of separate system

Advantages

- As sewage flows in separate pipe, hence the quantity to be treated at sewage treatment plant is small, resulting in economy of treatment.
- This system may be less costly as only sanitary sewage is transported in closed conduit and storm water can be collected and conveyed through open drains.
- When pumping is required during disposal, this system is economical due to less flow.

Disadvantages

- Self cleansing velocity may not developed at certain locations in sewers and hence flushing of sewers may be required.

- This system requires laying two sets of pipe, which may be difficult in congested area.
- This system will require maintenance of two sets of pipelines and hence maintenance cost is more.

2.2.3 Advantages and disadvantages of partially separate system

Advantages

- Economical and reasonable size sewers are required.
- Work of house plumbing is reduced as rain water from roofs, sullage from bathrooms and kitchen, etc. are combined with discharge from water closets.
- Flushing of sewers may not be required as small portion of storm water is allowed to enter in sanitary sewage.

Disadvantages

- Increased cost of pumping as compared to separate system at treatment plants and intermediate pumping station wherever required.
- In dry weather self-cleansing velocity may not develop in the sewers.

2.3 CONSIDERATIONS FOR THE TYPE OF SYSTEM

Following points are considered before finalizing the type of collection system.

- The separate system requires laying of two sets of conduits whereas in combined system only one bigger size conduit is required.
- Laying of two separate conduits may be difficult in the congested streets.
- In combined system sewers are liable for silting during non-monsoon season, hence they are required to be laid at *steeper gradients.* Steeper gradients for the sewers may *require more number of pumping stations*, particularly for flat terrain, which may make the system costly.
- Large quantity of wastewater is required to be treated before discharge in case of combined system. Hence, large capacity treatment plant is required.
- In separate system, only sewage is treated before it is discharged into natural water body or used for irrigation. No treatment is generally given to the rainwater collected before it is discharge in to natural water body.
- In case of separate system pumping is only required for sewage. Pumping can be avoided for storm water lines, as they are not very deep and normally laid along the natural slopes.

In combined system large capacity pumping station is required to safely handle the flow that is likely to be generated during highest design storm considered.

- Based on site conditions the economy of the system needs to be evaluated and selection is made accordingly.

2.4 PATTERNS OF COLLECTION SYSTEM

The network of sewers consists of house sewers discharging the sewage to laterals. The lateral discharges the sewage into branch sewers or sub-mains and sub-mains discharge it into main sewer or trunk sewer. The trunk sewer carries sewage to the common point where adequate treatment is given to the sewage and then it is discharged. The patterns of collection system depend upon:

1. The topographical and hydrological features of the area.
2. The location and methods of treatment and disposal works.
3. The type of sewerage system employed, and
4. Extent of area to be served.

Following patterns can be adopted for collection systems as per the suitability (Birdie, 1990).

a. Perpendicular pattern

- The shortest possible path is maintained for the rains carrying storm water and sewage (Figure 2.1).
- It is suitable for separate system and partially separate system for storm water drains.
- This pattern is not suitable for combined system, because treatment plant is required to be installed at many places; otherwise it will pollute the water body where the sewage is discharged.

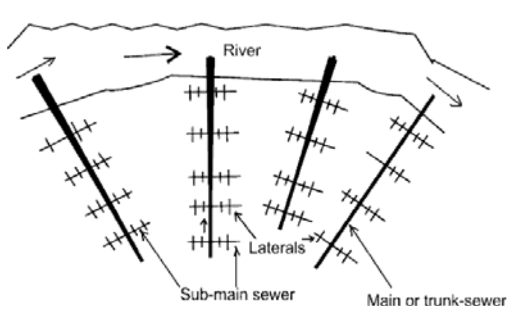


Figure 2.1 Perpendicular pattern of collection system

b. Interceptor pattern

- Sewers are intercepted with large size sewers (Figure 2.2).
- Interceptor carries sewage to a common point, where it can be disposed off with or without treatment.
- Overflows should be provided to handle very large flow.

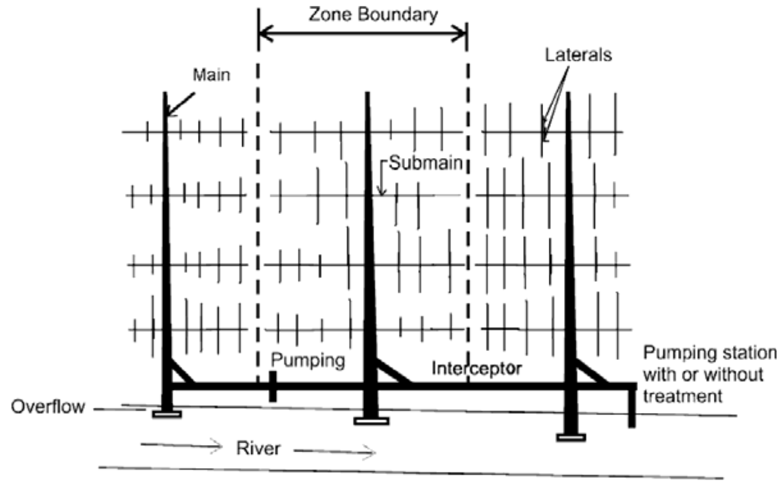


Figure 2.2 Interceptor pattern of collection system

c. Radial Pattern

- It is suitable for land disposal.
- In this pattern sewers are laid radialy outwards from the centre, hence this pattern is called as radial pattern (Figure 2.3).
- The drawback in this pattern is more number of disposal works are required.

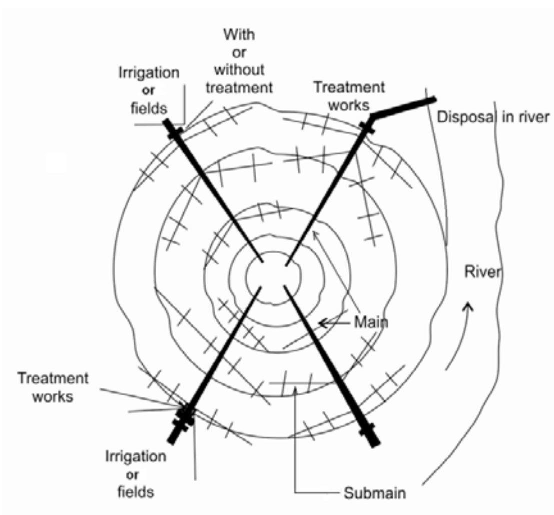


Figure 2.3 Radial pattern of collection system

d. Fan Pattern

- This pattern is suitable for a city situated at one side of the natural water body, such as river.
- The entire sewage flows to a common point where one treatment plant is located (Figure 2.4).
- In this number of converging main sewers and sub-mains are used forming a fan shape.
- Single treatment plant is required in this pattern.
- The drawback in this pattern is that larger diameter sewer is required near to the treatment plant as entire sewage is collected at a common point.
- In addition, with new development of the city the load on existing treatment plant increases.

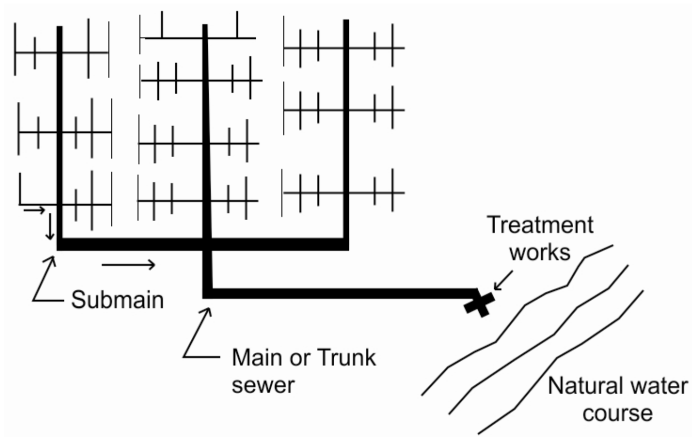


Figure 2.4 Fan pattern of collection system

e. Zone Pattern

- More numbers of interceptors are provided in this pattern (Figure 2.5).
- This pattern is suitable for sloping area than flat areas.

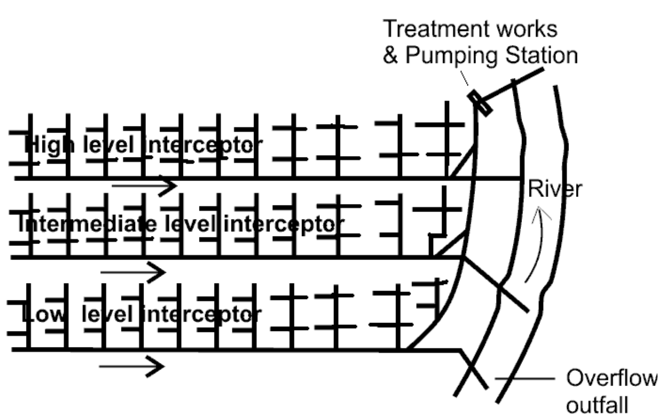


Figure 2.5 Zone pattern of collection system

Questions

1. Describe in brief various types of water carriage systems.
2. Describe merits and drawback of separate system, partially separate system and combined system.
3. What are the considerations while finalizing the type of sewerage system?
4. Write about various patterns of collection system.

Lecture 3: Sewer Material

3.1 Important Factors Considered for Selecting Material for Sewer

Following factors should be considered before selecting material for manufacturing sewer pipes:

a. Resistance to corrosion

Sewer carries wastewater that releases gases such as H_2S. This gas in contact with moisture can be converted into sulfuric acid. The formation of acids can lead to the corrosion of sewer pipe. Hence, selection of corrosion resistance material is must for long life of pipe.

b. Resistance to abrasion

Sewage contain considerable amount of suspended solids, part of which are inorganic solids such as sand or grit. These particles moving at high velocity can cause wear and tear of sewer pipe internally. This abrasion can reduce thickness of pipe and reduces hydraulic efficiency of the sewer by making the interior surface rough.

c. Strength and durability

The sewer pipe should have sufficient strength to withstand all the forces that are likely to come on them. Sewers are subjected to considerable external loads of backfill material and traffic load, if any. They are not subjected to internal pressure of water. To withstand external load safely without failure, sufficient wall thickness of pipe or reinforcement is essential. In addition, the material selected should be durable and should have sufficient resistance against natural weathering action to provide longer life to the pipe.

d. Weight of the material

The material selected for sewer should have less specific weight, which will make pipe light in weight. The lightweight pipes are easy for handling and transport.

e. Imperviousness

To eliminate chances of sewage seepage from sewer to surrounding, the material selected for pipe should be impervious.

f. Economy and cost

Sewer should be less costly to make the sewerage scheme economical.

g. Hydraulically efficient

The sewer shall have smooth interior surface to have less frictional coefficient.

3.2 Materials for Sewers

3.2.1 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’s 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 H_2S, in presence of water, H_2SO_4 can be formed leading to corrosion of pipe material.

3.2.2 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.

3.2.3 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.

3.2.4 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 groundwater through brick joints. These are lined from inside with stone ware or ceramic block to make them smooth and hydraulically efficient. Lining also makes the pipe resistant to corrosion.

3.2.5 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 groundwater 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.

3.2.6 Steel Pipes

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.

3.2.7 Ductile Iron Pipes

Ductile iron pipes can also be used for conveying the sewers. They demonstrate higher capacity to withstand water hammer. The specifications for DI pipes is provided in IS:12288-1987. The predominant wall material is ductile iron, a spheroidized graphite cast iron. Internally these pipes are coated with cement mortar lining or any other polyethylene or poly wrap or plastic bagging/ sleeve lining to inhibit corrosion from the wastewater being conveyed, and various types of external coating are used to inhibit corrosion from the environment. Ductile iron has proven to be a better pipe material than cast iron but they are costly. Ductile iron is still believed to be stronger and more fracture resistant material. However, like most ferrous materials it is susceptible to corrosion. A typical life expectancy of thicker walled pipe could be up to 75 years, however with the current thinner walled ductile pipe the life could be about 20 years in highly corrosive soils without a corrosion control program like cathodic protection.

3.2.8 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.

3.2.9 High Density Polythylene (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. PVC pipes offer very little flexibility and normally considered rigid; whereas, HDPE pipes are flexible hence best suited for laying in hilly and uneven terrain. Flexibility allows simple handling and installation of HDPE pipes. Because of low density, these pipes are very light in weight. Due to light in weight, they are easy for handling, this reduces transportation and installation cost. HDPE pipes are non corrosive and offer very smooth inside surface due to which pressure losses are minimal and also this material resist scale formation.

3.2.10 Glass Fiber Reinforced Plastic Pipes

This martial is widely used where corrosion resistant pipes are required. Glass fiber reinforced plastic (GRP) can be used as a lining material for conventional pipes to protect from internal or external corrosion. It is made from the composite matrix of glass fiber, polyester resin and fillers. These pipes have better strength, durability, high tensile strength, low density and high corrosion resistance. These are manufactured up to 2.4 m diameter and up to 18 m length (IS:12709-1989). Glass reinforced plastic pipes represent the ideal solution for transport of any kind of water, chemicals, effluent and sewage, because they combine the advantages of corrosion resistance with a mechanical strength which can be compared with the steel pipes. Typical properties that result in advantages in GRP pipes application can be summarized as follows:

- Light weight of pipes that allows for the use of light laying and transport means.
- Possibility of nesting of different diameters of pipe thus allowing additional saving in transport cost.
- Length of pipe is larger than other pipe materials.
- Easy installation procedures due to the kind of mechanical bell and spigot joint.
- Corrosion resistance material, hence no protections such as coating, painting or cathodic are then necessary.
- Smoothness of the internal wall that minimizes the head loss and avoids the formation of deposits.
- High mechanical resistance due to the glass reinforcement.
- Absolute impermeability of pipes and joints both from external to internal and vice-versa.
- Very long life of the material.

3.2.11 Lead Sewers

- They are smooth, soft and can take odd shapes.
- This pipe has an ability to resist sulphide corrosion.
- However, these pipes are very costly.
- These are used in house connection.

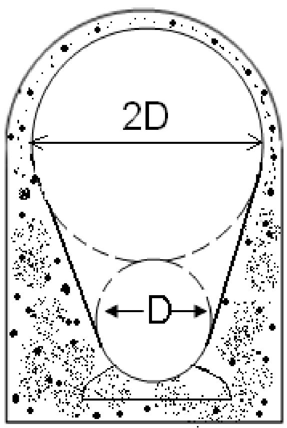
3.3 Shapes of Sewer Pipes

Sewers are generally circular pipes laid below ground level, slopping continuously towards the outfall. These are designed to flow under gravity. Shapes other than circular are also used.

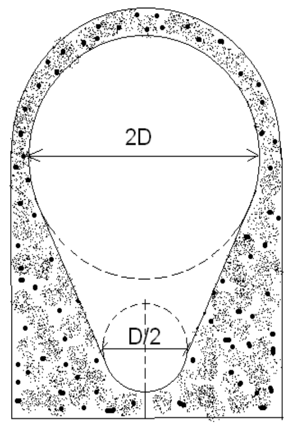
Other shapes used for sewers are (Figure 3.1 a through i):

a. Standard Egg-shaped sewer
b. New egg-shaped sewer
c. Horse shoe shaped sewer
d. Parabolic shaped sewer
e. Semi-elliptical section
f. Rectangular shape section
g. U-shaped section
h. Semi-circular shaped sewer
i. Basket handled shape sewer

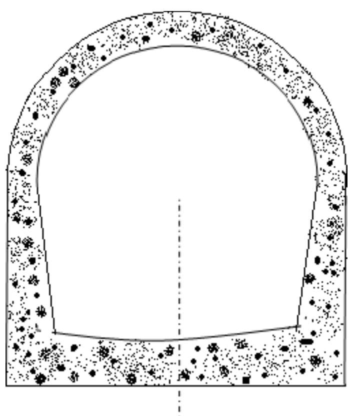
Standard egg-shaped sewers, also called as ovoid shaped sewer, and new or modified egg-shaped sewers are used in combined sewers. These sewers can generate self cleansing velocity during dry weather flow. Horse shoe shaped sewers and semi-circular sections are used for large sewers with heavy discharge such as trunk and outfall sewers. Rectangular or trapezoidal section is used for conveying storm water. U-shaped section is used for larger sewers and especially in open cuts. Other sections of the sewers have become absolute due to difficulty in construction on site and non availability of these shapes readily in market.



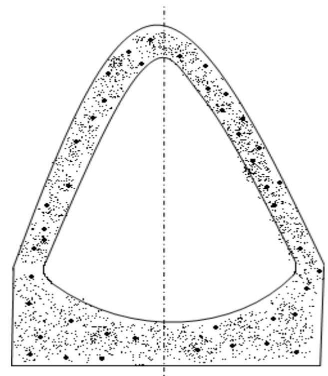
(a) Standard Egg Shaped Sewer



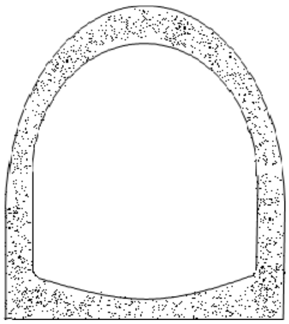
(b) New/ Modified Egg shaped Sewer



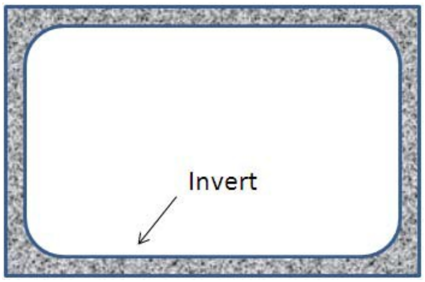
(c) Horse shoe sewer section



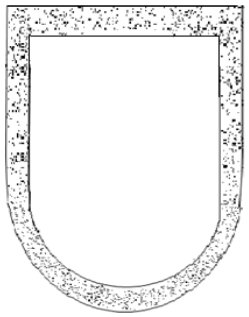
(d) Parabolic section



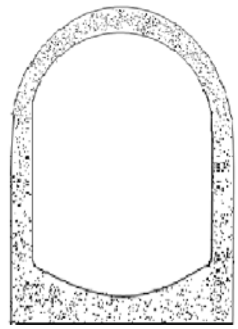
(e) Semi-elliptical section



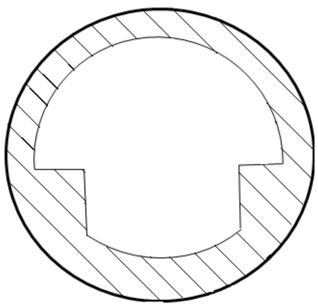
(f) Rectangular Sewer



(g) U-shaped section



(h) Semi-circular Section



(i) Basket-Handle Section

Figure 3.1: Different shapes used for construction of sewer other than circular

Questions

1. What should be properties of the material to be used for sewer construction?
2. Write a note on different materials used for sewer construction.
3. With schematic describe various shapes used for sewer section.
4. What are the advantages and drawback of the circular section sewers?

Lecture 4: Quantity Estimation of Sewage

4.1 Introduction

The sewage collected from the municipal area consists of wastewater generated from the residences, commercial centers, recreational activities, institutions and industrial wastewaters discharge into sewer network from the permissible industries located within the city limits. Before designing the sewer, it is necessary to know the discharge i.e., quantity of sewage, which will flow in it after completion of the project.

Accurate estimation of sewage discharge is necessary for hydraulic design of the sewers. Far lower estimation than reality will soon lead to inadequate sewer size after commissioning of the scheme or the sewers may not remain adequate for the entire design period. Similarly, very high discharge estimated will lead to larger sewer size affecting economy of the sewerage scheme, and the lower discharge actually flowing in the sewer may not meet the criteria of the self cleansing velocity and hence leading to deposition in the sewers. Actual measurement of the discharge is not possible if the sewers do not exist; and where the capacity of the existing sewers is inadequate and need to be increased, still actual present discharge measurement may not be accurate due to unaccounted overflow and leakages that might be occurring in the existing system. Since sewers are design to serve for some more future years, engineering skills have to be used to accurately estimate the sewage discharge.

4.2 Sources of Sanitary Sewage

1. Water supplied by water authority for domestic usage, after desired use it is discharged in to sewers as sewage.
2. Water supplied to the various industries for various industrial processes by local authority. Some quantity of this water after use in different industrial applications is discharged as wastewater.
3. The water supplied to the various public places such as, schools, cinema theaters, hotels, hospitals, and commercial complexes. Part of this water after desired use joins the sewers as wastewater.
4. Water drawn from wells by individuals to fulfill domestic demand. After uses this water is discharged in to sewers.
5. The water drawn for various purposes by industries, from individual water sources such as, wells, tube wells, lake, river, etc. Fraction of this water is converted into wastewater in different industrial processes or used for public utilities within the industry generating wastewater. This is discharged in to sewers.

6. Infiltration of groundwater into sewers through leaky joints.
7. Entrance of rainwater in sewers during rainy season through faulty joints or cracks in sewers.

4.3 Dry Weather Flow

Dry weather flow is the flow that occurs in sewers in separate sewerage system or the flow that occurs during dry seasons in combined system. This flow indicates the flow of sanitary sewage. This depends upon the rate of water supply, type of area served, economic conditions of the people, weather conditions and infiltration of groundwater in the sewers, if sewers are laid below groundwater table.

4.4 Evaluation of Sewage Discharge

Correct estimation of sewage discharge is necessary; otherwise sewers may prove inadequate resulting in overflow or may prove too large in diameter, which may make the system uneconomical and hydraulically inefficient. Hence, before designing the sewerage system it is important to know the discharge / quantity of the sewage, which will flow in it after completion of the project and at the end of design period.

Apart from *accounted water supplied* by water authority that will be converted to wastewater, following quantities are considered while estimating the sewage quantity:

a. Addition due to unaccounted private water supplies

People using water supply from private wells, tube wells, etc. contribute to the wastewater generation more than the water supplied by municipal authority. Similarly, certain industries utilize their own source of water. Part of this water, after desired uses, is converted into wastewater and ultimately discharged into sewers. This quantity can be estimated by actual field observations.

b. Addition due to infiltration

This is additional quantity due to groundwater seepage in to sewers through faulty joints or cracks formed in the pipes. The quantity of the water depends upon the height of the water table above the sewer invert level. If water table is well below the sewer invert level, the infiltration can occur only after rain when water is moving down through soil. Quantity of the water entering in sewers depends upon the permeability of the ground soil and it is very difficult to estimate. While estimating the design discharge, following suggested discharge can be considered (Table 4.1).

Table 4.1 Suggested estimates for groundwater infiltration for sewers laid below groundwater table (CPHEEO Manual, 1993)

Unit	Minimum	Maximum
L/ha.d	5000	50000
L/km.d	500	5000
L per day per manhole	250	500

Storm water drainage may also infiltrate into sewers. This inflow is difficult to calculate. Generally, no extra provision is made for this quantity. This extra quantity can be taken care of by extra empty space left at the top in the sewers, which are designed for running ¾ full at maximum design discharge.

c. Subtraction due to water losses

The water loss, through leakage in water distribution system and house connections, does not reach consumers and hence, not appear as sewage.

d. Subtraction due to water not entering the sewerage system

Certain amount of water is used for such purposes, which may not generate sewage, e.g. boiler feed water, water sprinkled over the roads, streets, lawns, and gardens, water consumed in industrial product, water used in air coolers, etc.

Net quantity of sewage: The net quantity of sewage production can be estimated by considering the addition and subtraction as discussed above over the accounted quantity of water supplied by water authority as below:

Net quantity of sewage = Accounted quantity of water supplied from the water works + Addition due to unaccounted private water supplies + Addition due to infiltration – Subtraction due to water losses – Subtraction due to water not entering the sewerage system

Generally, 75 to 80% of accounted water supplied is considered as quantity of sewage produced.

4.5 Variation in Sewage Flow

Variation occurs in the flow of sewage over annual average daily flow. Fluctuation in flow occurs from hour to hour and from season to season. The typical hourly variation in the sewage flow is shown in the Figure 4.1. If the flow is gauged near its origin, the peak flow will be quite pronounced. The peak will defer if the sewage has to travel long distance. This is because of the time required in collecting sufficient quantity of sewage required to fill the sewers and time required in travelling. As sewage flow in sewer lines, more and more sewage is mixed in it due to continuous increase in the area being served by the sewer line. This leads to reduction in the fluctuations in the sewage flow and the lag period goes on increasing. The magnitude of variation in the sewage quantity varies from place to place and it is very difficult to predict. For smaller township this variation will be more pronounced due to lower length and travel time before sewage reach to the main sewer and for large cities this variation will be less.

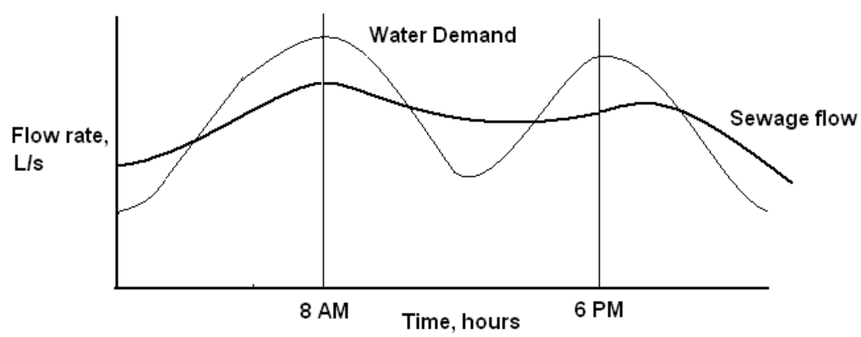


Figure 4.1 Typical hourly variations in sewage flow

For estimating design discharge following relation can be considered:

Maximum daily flow	=	Two times the annual average daily flow (representing seasonal variations)
Maximum hourly flow	=	1.5 times the maximum daily flow (accounting hourly variations)
	=	Three times the annual average daily flow

As the tributary area increases, peak hourly flow will decrease. For smaller population served (less than 50000) the peak factor can be 2.5, and as the population served increases its value reduces. For large cities it can be considered about 1.5 to 2.0. Therefore, for outfall sewer the peak flow can be considered as 1.5 times the annual average daily flow. Even for design of the treatment facility, the peak factor is considered as 1.5 times the annual average daily flow.

The minimum flow passing through sewers is also important to develop self cleansing velocity to avoid silting in sewers. This flow will generate in the sewers during late night hours. The effect of this flow is more pronounced on lateral sewers than the main sewers. Sewers must be checked for minimum velocity as follows:

Minimum daily flow = 2/3 Annual average daily flow

Minimum hourly flow = ½ minimum daily flow

= 1/3 Annual average daily flow

The overall variation between the maximum and minimum flow is more in the laterals and less in the main or trunk sewers. This ratio may be more than 6 for laterals and about 2 to 3 in case of main sewers.

4.6 Design Period

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

4.7 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 economical condition, changes in habit of people, and enhanced demand for public utilities.

Problem 4.1

A city has a projected population of 60,000 spread over area of 50 hectare. Find the design discharge for the separate sewer line by assuming rate of water supply of 250 LPCD and out of this total supply only 75 % reaches in sewer as wastewater. Make necessary assumption whenever necessary.

Solution:

Given data

Q = 250 lit/capita/day

Sewage flow = 75% of water supply

= 0.75* 250

= 187.5 LPCD

Total sewage generated = 187.5*60000/(24*3600)

= 130.21 lit/sec

= 0.13 m^3/s

Assume peak factor = 2

Total design discharge = 0.26 m^3/s.

Questions

1. What is dry weather flow?
2. Describe variation in sewage flow. How design of different component of sewerage scheme will be affected due to this variation?
3. What is design period? It depends on what parameters? Provide design period for different components of the sewerage scheme.

Lecture 6: Quantity Estimation of Storm Water

6.1 Factors Affecting the Quantity of Stormwater

The surface run-off resulting after precipitation contributes to the stormwater. The quantity of stormwater reaching to the sewers or drains is very large as compared with sanitary sewage. The factors affecting the quantity of stormwater flow are as below:

i. Area of the catchment
ii. Slope and shape of the catchment area
iii. Porosity of the soil
iv. Obstruction in the flow of water as trees, fields, gardens, etc.
v. Initial state of catchment area with respect to wetness.
vi. Intensity and duration of rainfall
vii. Atmospheric temperature and humidity
viii. Number and size of ditches present in the area

6.2 Measurement of Rainfall

The rainfall intensity could be measured by using rain gauges and recording the amount of rain falling in unit time. The rainfall intensity is usually expressed as mm/hour or cm/hour. The rain gauges used can be manual recording type or automatic recording rain gauges.

6.3 Methods for Estimation of Quantity of Storm Water

1. Rational Method
2. Empirical formulae method

In both the above methods, the quantity of storm water is considered as function of intensity of rainfall, coefficient of runoff and area of catchment.

Time of Concentration: The period after which the entire catchment area will start contributing to the runoff is called as the time of concentration.

- The rainfall with duration lesser than the time of concentration will not produce maximum discharge.
- The runoff may not be maximum even when the duration of the rain is more than the time of concentration. This is because in such cases the intensity of rain reduces with the increase in its duration.
- The runoff will be maximum when the duration of rainfall is equal to the time of concentration and is called as *critical rainfall duration*. The time of concentration is equal to sum of inlet time and time of travel.

Time of concentration = Inlet time + time of travel

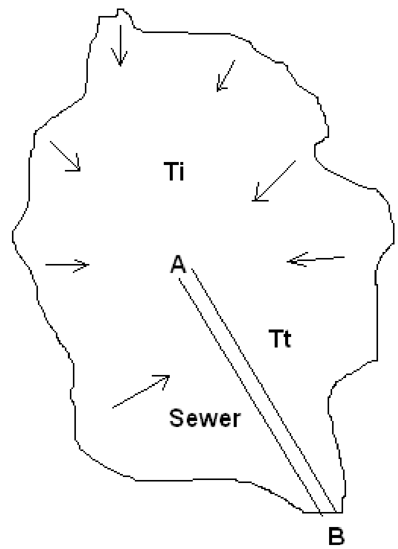


Figure 6.1 Runoff from a given catchment

Inlet Time: The time required for the rain in falling on the most remote point of the tributary area to flow across the ground surface along the natural drains or gutters up to inlet of sewer is called inlet time (Figure 6.1). The inlet time 'Ti' can be estimated using relationships similar to following. These coefficients will have different values for different catchments.

$$Ti = [0.885\ L^3/H]^{0.385} \qquad (1)$$

Where,

Ti = Time of inlet, minute

L = Length of overland flow in Kilometer from critical point to mouth of drain

H = Total fall of level from the critical point to mouth of drain, meter

Time of Travel: The time required by the water to flow in the drain channel from the mouth to the point under consideration or the point of concentration is called as time of travel.

Time of Travel (Tt) = Length of drain/ velocity in drain (2)

Runoff Coefficient: The total precipitation falling on any area is dispersed as percolation, evaporation, storage in ponds or reservoir and surface runoff. The runoff coefficient can be defined as a fraction, which is multiplied with the quantity of total rainfall to determine the quantity of rain water, which will reach the sewers. The runoff coefficient depends upon the porosity of soil cover, wetness and ground cover. The overall runoff coefficient for the catchment area can be worked out as follows:

Overall runoff coefficient, $C = [A_1.C_1 + A_2.C_2 + ...+ A_n.C_n] / [A_1 + A_2 + ...+ A_n]$ (3)

Where, A_1, A_2, ….A_n are types of area with C_1, C_2, …C_n as their coefficient of runoff, respectively.

The typical runoff coefficient for the different ground cover is provided in the Table 6.1.

Table 6.1 Runoff coefficient for different type of cover in catchment

Type of Cover	Coefficient of runoff
Business areas	0.70 – 0.90
Apartment areas	0.50 – 0.70
Single family area	0.30 – 0.50
Parks, Playgrounds, Lawns	0.10 – 0.25
Paved Streets	0.80 –0.90
Water tight roofs	0.70 – 0.95

6.3.1 Rational method

Storm water quantity can be estimated by rational method as below:

Storm water quantity, $Q = C.I.A / 360$ (4)

Where,

Q = Quantity of storm water, m^3/sec

C = Coefficient of runoff

I = intensity of rainfall (mm/hour) for the duration equal to time of concentration, and

A = Drainage area in hectares

OR

$$Q = 0.278\ C.I.A \quad (5)$$

Where, Q is m^3/sec; I is mm/hour, and A is area in square kilometer

6.3.2 Empirical Formulae

Empirical formulae are used for determination of runoff from very large area. Various empirical relationships are developed based on the past observations on specific site conditions suiting a particular region. These empirical formulae can be used for prediction of storm water runoff for that particular catchment.

A] Burkli – Zeiglar formula

$$Q = \frac{C.I.A}{141.58}\sqrt[4]{S/A}$$

(6)

B] Mc Math formula (used in USA)

$$Q = \frac{C.I.A}{148.35}\sqrt[5]{S/A}$$

(7)

C] Fuller’s formula

$$Q = \frac{C.M^{0.8}}{13.23}$$

(8)

(Where, S- Slope of the area in meter per thousand meter, M- drainage area in sq. km., A – drainage area in hectare)

6.3.3 Empirical formulae for rainfall intensities

These relationships between rainfall intensity and duration are developed based on long term experience in field (Figure 6.2). Under Indian conditions, intensity of rainfall in design is usually in the range 12 mm/h to 20 mm/h. In general, the empirical relationship has the following forms:

$I = a/(t + b)$ OR $I = b/t^n$ (9)

Where, a, b, and n are constants.

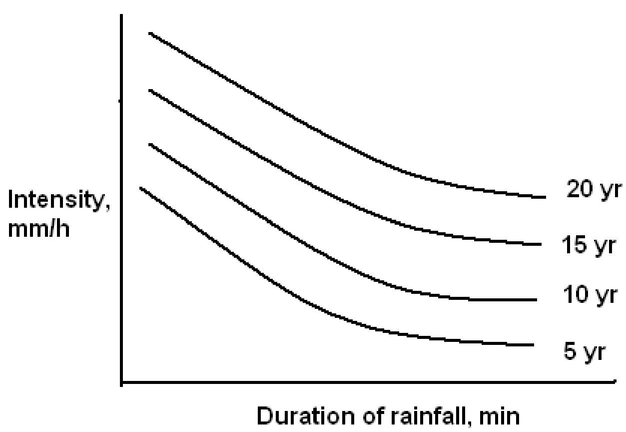


Figure 6.2 Relationship of rainfall duration and intensity

British Ministry of Health formula

$I = 760/(t + 10)$ (for storm duration of 5 to 20 minutes) (10)

$I = 1020/(t + 10)$ (for storm duration of 20 to 100 minutes) (11)

Where, I is intensity of rainfall, mm/h and t is duration of storm, minutes.

6.4 Examples

1. Determine designed discharge for a combined system serving population of 50000 with rate of water supply of 135 LPCD. The catchment area is 100 hectares and the average coefficient of runoff is 0.60. The time of concentration for the design rainfall is 30 min and the relation between intensity of rainfall and duration is I = 1000/(t + 20).

Solution

Estimation of sewage quantity

Considering 80% of the water supplied will result in wastewater generation,

The quantity of sanitary sewage = 50000 x 135 x 0.80 = 5400 m^3/day = 0.0625 m^3/sec

Considering peak factor of 2.5, the design discharge for sanitary sewage = 0.0625 x 2.5

= 0.156 m^3/sec

Estimation of storm water discharge

Intensity of rainfall, I = 1000/(t + 20)

Therefore, I = 1000/(30 + 20) = 20 mm/h

Hence, storm water runoff, Q = C.I.A/360

= 0.6 x 20 x 100/(360) = 3.33 m^3/sec

Therefore, design discharge for combined sewer = 3.33 + 0.156 = 3.49 m^3/sec

2. The catchment area is of 300 hectares. The surface cover in the catchment can be classified as given below:

Type of cover	Coefficient of runoff	Percentage
Roofs	0.90	15
Pavements and yards	0.80	15
Lawns and gardens	0.15	25
Roads	0.40	20
Open ground	0.10	15
Single family dwelling	0.50	10

Calculate the runoff coefficient and quantity of storm water runoff, if intensity of rainfall is 30 mm/h for rain with duration equal to time of concentration. If population density in the area is 350 persons per hectare and rate of water supply is 200 LPCD, calculate design discharge for separate system, partially separate system, and combined system.

Solution

Estimation of storm water discharge for storm water drain of separate system

Overall runoff coefficient $C = [A_1.C_1 + A_2.C_2 + \ldots + A_n.C_n] / [A_1 + A_2 + \ldots + A_n]$

$$= \frac{(0.15 \times 0.90 + 0.15 \times 0.80 + 0.25 \times 0.15 + 0.20 \times 0.4 + 0.15 \times 0.1 + 0.10 \times 0.5)}{0.15 + 0.15 + 0.25 + 0.20 + 0.15 + 0.10}$$

$= 0.44$

Therefore quantity of storm water, $Q = C.I.A/360$

$= 0.44 \times 30 \times 300/360$

$= 11\ m^3/sec$

Estimation of sewage discharge for sanitary sewer of separate system

Quantity of sanitary sewage = 300 x 350 x 200 x 0.80 = 16800 m^3/day = 0.194 m^3/sec

Considering peak factor of 2, the design discharge for sanitary sewers = 0.194 x 2

= 0.389 m^3/sec

Estimation of discharge for partially separate system

Storm water discharge falling on roofs and paved courtyards will be added to the sanitary sewer. This quantity can be estimated as:

Average coefficient of runoff = (0.90 x 45 + 0.80 x 45) / 90 = 0.85

Discharge = 0.85 x 30 x 90 / 360 = 6.375 m^3/sec

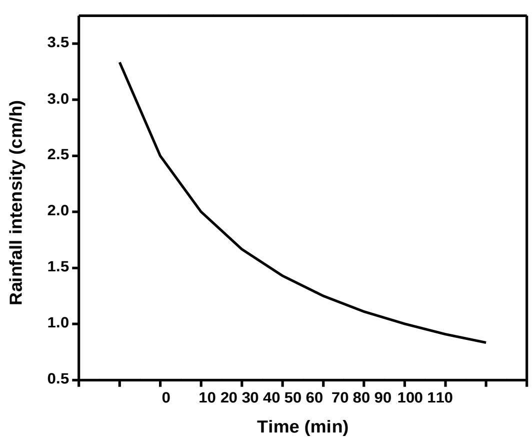
Therefore total discharge in the sanitary sewer of partially separate system = 6.375 + 0.389 = 6.764 m^3/sec, and the discharge in storm water drains = 11 – 6.375 = 4.625 m^3/sec

Questions

1. Explain the factors affecting the storm water discharge.
2. What is time of concentration? What is its role in determination of the storm water runoff?
3. Explain critical rainfall duration. Why rainfall of this duration will generate maximum runoff?
4. What is coefficient of runoff?
5. A catchment is having total area of 60 hectares. The rainfall intensity relation with duration for this catchment is given by the relation I = 100/(t+20), where I is in cm/h and t is duration of rain in min. (a) Draw the graph of rainfall intensity relation with duration at 10 min interval? (b) What will be the storm water runoff from this catchment if the average imperviousness factor is 0.63, and time of concentration is 35 min? (c) If population density of the area is 350 persons per hectare and water consumption is 170 LPCD, what will be the design discharge for separate system and combined system?

Q. 6. Solution:

(a)



b) 1.91 m^3/sec; c) design discharge for sanitary sewers of separate system = 0.0662 m^3/sec; and design discharge of combined system = 1.976 m^3/sec.

Lecture 7: Hydraulic Design of Sewers and Storm Water Drains

7.1 General Consideration

Generally, sewers are laid at steeper gradients falling towards the outfall point with circular pipe cross section. Storm water drains are separately constructed as surface drains at suitable gradient, either rectangular or trapezoidal section. Sewers are designed to carry the maximum quantity of sanitary sewage likely to be produced from the area contributing to the particular sewer. Storm water drains are designed to carry the maximum storm runoff that is likely to be produced by the contributing catchment area from a rain of design frequency and of duration equal to the time of concentration.

7.2 Requirements of Design and Planning of Sewerage System

The sewerage scheme is designed to remove entire sewage effectively and efficiently from the houses to the point of treatment and disposal. Following aspects should be considered while designing the system.

- The sewers provided should be adequate in size to avoid overflow and possible health hazards.
- For evaluating proper diameter of the sewer, correct estimation of sewage discharge is necessary.
- The flow velocity inside the sewer should neither be so large as to require heavy excavation and high lift pumping, nor should be so small causing deposition of the solid in the sewers.
- The sewers should be laid at least 2 to 3 m deep to carry sewage from basement.
- The sewage in sewer should flow under gravity with 0.5 to 0.8 full at designed discharge, i.e. at the maximum estimated discharge.
- The sewage is conveyed to the point usually located in low-lying area, where the treatment plant is located.
- Treatment plant should be designed taking into consideration the quality of raw sewage expected and to meet the discharge standards.

7.3 Difference Between Water Supply Pipes and Sewer Pipes

The major difference between the water distribution network and sewerage system is presented in the Table 7.1.

Table 7.1: Comparison between the water distribution network and sewage collection system

Water Supply Pipes	Sewer Pipes
• It carries pure water.	• It carries contaminated water containing organic or inorganic solids which may settle in the pipe. It can cause corrosion of the pipe material.
• Velocity higher than self-cleansing is not essential, because of solids are not present in suspension.	• To avoid deposition of solids in the pipes self-cleansing velocity is necessary at all possible discharge.
• It carries water under pressure. Hence, the pipe can be laid up and down the hills and the valleys within certain limits.	• It carries sewage under gravity. Therefore it is required to be laid at a continuous falling gradient in the downward direction towards outfall point.
• These pipes are flowing full under pressure.	• Sewers are design to run partial full at maximum discharge. This extra space ensures non-pressure gravity flow. This will minimize the leakage from sewer, from the faulty joints or crack, if any.

7.4 Provision of Freeboard in Sewers

7.4.1 Sanitary Sewers

Sewers with diameter less than 0.4 m are designed to run half full at maximum discharge, and sewers with diameter greater than 0.4 m are designed to flow 2/3 to ¾ full at maximum discharge. The extra space provided in the sewers provides factor of safety to counteract against the following factors:

1. Safeguard against lower estimation of the quantity of wastewater to be collected at the end of design period due to private water supply by industries and public. Thus, to ensure that sewers will never flow full eliminating pressure flow inside the sewer.
2. Large scale infiltration of storm water through wrong or illegal connection, through underground cracks or open joints in the sewers.
3. Unforeseen increase in population or water consumption and the consequent increase in sewage production.

7.4.2 Storm Water Drains

Storm water drains are provided with nominal freeboard, above their designed full supply line because the overflow from storm water drains is not much harmful. Minimum of 0.3 m free board is generally provided in storm water drains.

7.5 Hydraulic Formulae for Determining Flow Velocities

Sewers of any shape are hydraulically designed as open channels, except in the case of inverted siphons and discharge lines of pumping stations. Following formulae can be used for design of sewers.

1. Manning's Formula

This is most commonly used for design of sewers. The velocity of flow through sewers can be determined using Manning's formula as below:

$$v = \frac{1}{n} r^{2/3} s^{1/2} \qquad (1)$$

Where,

v = velocity of flow in the sewer, m/sec

r = Hydraulic mean depth of flow, m

= a/p

a = Cross section area of flow, m^2

p = Wetted perimeter, m

n = Rugosity coefficient, depends upon the type of the channel surface i.e., material and lies between 0.011 and 0.015. For brick sewer it could be 0.017 and 0.03 for stone facing sewers.

s = Hydraulic gradient, equal to invert slope for uniform flows.

2. Chezy's Formula

$$v = C\, r^{1/2} s^{1/2} \qquad (2)$$

Where, C is Chezy's constant and remaining variables are same as above equation.

3. Crimp and Burge's Formula

$$v = 83.5\, r^{2/3} s^{1/2} \qquad (3)$$

4. Hazen- Williams Formula

$$V = 0.849\, C\, R^{0.63} S^{0.54} \qquad (4)$$

The Hazen-Williams coefficient 'C' varies with life of the pipe and it has high value when the pipe is new and lower value for older pipes. For example for RCC new pipe it is 150 and the value recommended for design is 120, as the pipe interior may become rough with time. The design values of 'C; for AC pipes, Plastic pipes, CI pipes, and steel lined with cement are 120, 120, 100, and 120, respectively. Modified Hazen-William's equation is also used in practice.

7.6 Minimum Velocity: Self Cleansing Velocity

The velocity that would not permit the solids to settle down and even scour the deposited particles of a given size is called as self-cleansing velocity. This minimum velocity should at least develop once in a day so as not to allow any deposition in the sewers. Otherwise, if such deposition takes place, it will obstruct free flow causing further deposition and finally leading to the complete blocking of the sewers. This minimum velocity or self-cleansing velocity can be worked out as below:

$$Vs = \sqrt{\frac{8K}{f'}(Ss - 1)g.d'} \qquad (5)$$

Where,

K= constant, for clean inorganic solids = 0.04 and for organic solids = 0.06

f' = Darcy Weisbach friction factor (for sewers = 0.03)

Ss = Specific gravity of sediments

g = gravity acceleration

d' = diameter of grain, m

- Hence, for removing the impurities present in sewage i.e., sand up to 1 mm diameter with specific gravity 2.65 and organic particles up to 5 mm diameter with specific gravity of 1.2, it is necessary that a minimum velocity of about 0.45 m/sec and an average velocity of about 0.9 m/sec should be developed in sewers.
- Hence, while finalizing the sizes and gradients of the sewers, they must be checked for the minimum velocity that would be generated at minimum discharge, i.e., about 1/3 of the average discharge.
- While designing the sewers the flow velocity at full depth is generally kept at about 0.8 m/sec or so. Since, sewers are generally designed for ½ to ¾ full, the velocity at 'designed discharge' (i.e., ½ to ¾ full) will even be more than 0.8 m/sec. Thus, the minimum velocity generated in sewers will help in the following ways:

- Adequate transportation of suspended solids,
- Keeping the sewer size under control; and
- Preventing the sewage from decomposition by moving it faster, thereby preventing evolution of foul gases.

7.7 Maximum Velocity or Non-scouring Velocity

The interior surface of the sewer pipe gets scored due to the continuous abrasion caused by suspended solids present in sewage. The scoring is pronounced at higher velocity than what can be tolerated by the pipe materials. This wear and tear of the sewer pipes will reduce the life span of the pipe and their carrying capacity. In order to avoid this, it is necessary to limit the maximum velocity that will be produced in sewer pipe at any time. This limiting or non-scouring velocity mainly depends upon the material of sewer. The limiting velocity for different sewer material is provided in Table 7.2.

Table 7.2 Limiting or non-scouring velocity for different sewer material

Sewer Material	Limiting velocity, m/sec
Vitrified tiles	4.5 – 5.5
Cast iron sewer	3.5 – 4.5
Cement concrete	2.5 – 3.0
Stone ware sewer	3.0 – 4.5
Brick lined sewer	1.5 – 2.5

The problem of maximum or non-scouring velocity is severe in hilly areas where ground slope is very steep and this is overcome by constructing drop manholes at suitable places along the length of the sewer.

7.8 Effect of Flow Variations on Velocities in a Sewer

The discharge flowing through sewers varies considerably from time to time. Hence, there occur variation in depth of flow and thus, variation in Hydraulic Mean Depth (H.M.D.). Due to change in H.M.D. there occur changes in flow velocity, because it is proportional to $(H.M.D.)^{2/3}$. Therefore, it is necessary to check the sewer for minimum velocity of about 0.45 m/sec at the time of minimum flow (1/3 of average flow) and the velocity of about 0.9 to 1.2 m/sec should be developed at a time of average flow. The velocity should also be checked for limiting velocity i.e. non-scouring velocity at the maximum discharge.

For flat ground sewers are designed for self-cleansing velocity at maximum discharge. This will permit flatter gradient for sewers. For mild slopping ground, the condition of developing self-cleansing velocity at average flow may be economical. Whereas, in hilly areas, sewers can be designed for self-cleansing velocity at minimum discharge, but the design must be checked for non-scouring velocity at maximum discharge.

Example: 1

Design a sewer for a maximum discharge of 650 L/s running half full. Consider Manning's rugosity coefficient n = 0.012, and gradient of sewer S = 0.0001.

Solution

$Q \quad = A.V$

$0.65 \quad = (\pi D^2/8)\ (1/n)\ R^{2/3}\ S^{1/2}$

$R = A/P$

Solving for half full sewer, $R = D/4$

Substituting in above equation and solving we get D = 1.82 m.

Comments: If the pipe is partially full it is not easy to solve this equation and it is time consuming.

.9 Hydraulic Characteristics of Circular Sewer Running Full or Partially Full

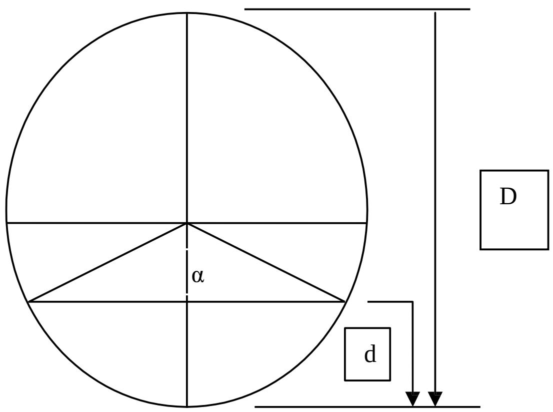


Figure 7.1 Section of a circular sewer running partially full

a) Depth at Partial flow

$$d = \left[\frac{D}{2} - \frac{D}{2} \cos\left(\frac{\alpha}{2} \right) \right] \tag{6}$$

b) Therefore proportionate depth

$$\frac{d}{D} = \frac{1}{2}\left[1 - \cos\left(\frac{\alpha}{2} \right) \right] \tag{7}$$

c) Proportionate area

$$\frac{a}{A} = \left[\frac{\alpha}{360} - \frac{Sin\alpha}{2\pi} \right] \tag{8}$$

d) Proportionate perimeter: $\frac{p}{P} = \frac{\alpha}{360}$ (9)

e) Proportionate Hydraulic Mean Depth

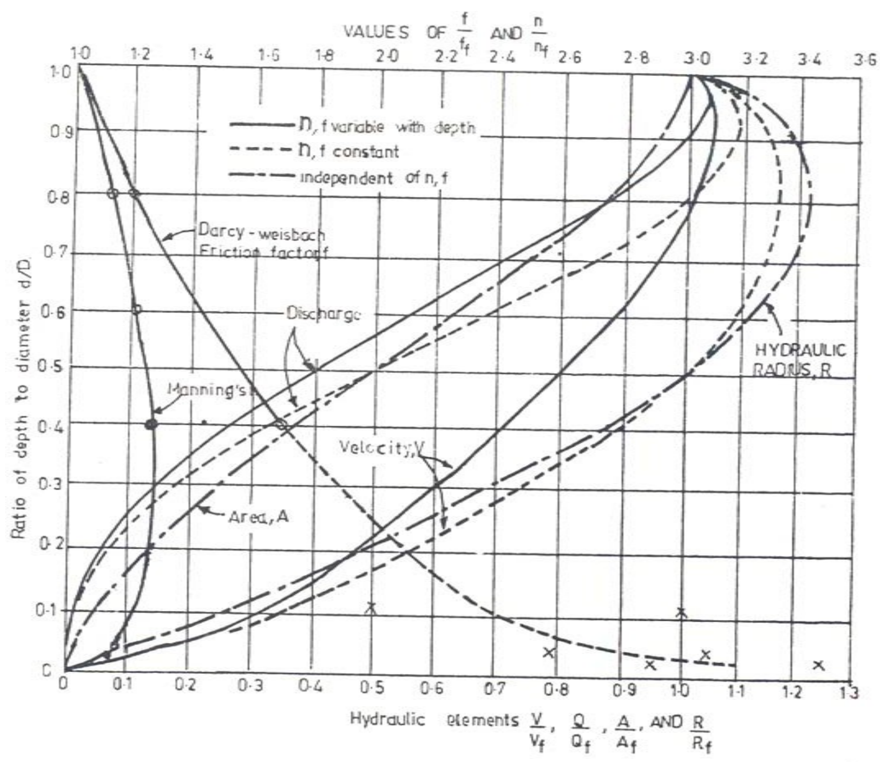
$$\frac{r}{R} = \left[1 - \frac{360 Sin\alpha}{2\pi\alpha} \right] \tag{10}$$

f) Proportionate velocity = $\frac{v}{V} = \frac{N}{n} \frac{r^{2/3}}{R^{2/3}}$ (11)

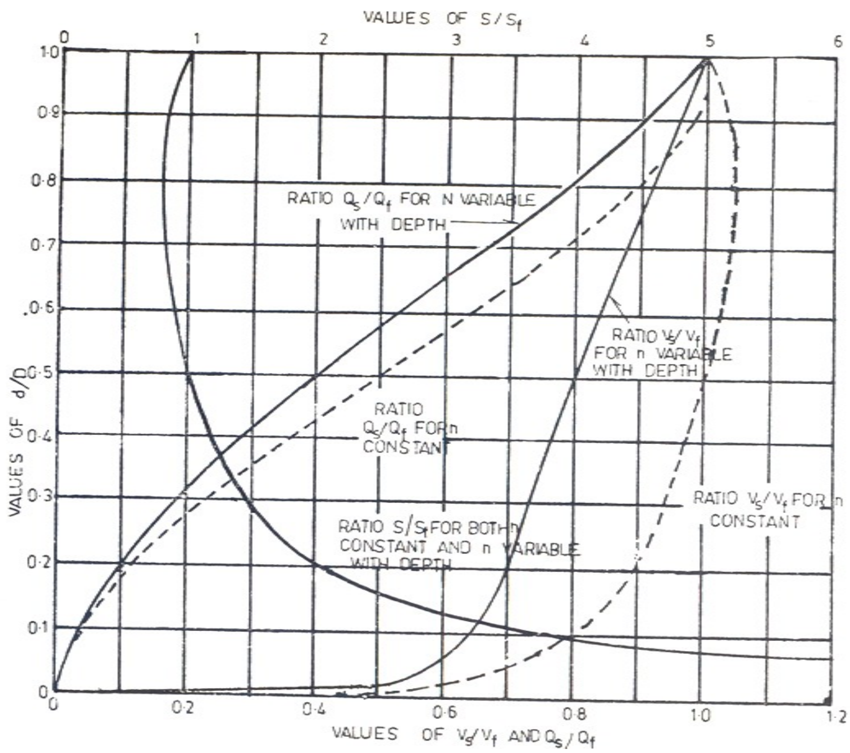
In all above equations except ‘α’ everything is constant (Figure 7.1). Hence, for different values of ‘α’, all the proportionate elements can be easily calculated. These values of the hydraulic elements can be obtained from the proportionate graph prepared for different values of d/D (Figure 7.2). The 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 v_s/V, q_s/Q and s_s/S can be computed as stated below:



(a) Hydraulic elements for circular sewer



(b) Hydraulic elements of circular sewer possessing equal selfcleansing properties at all depths

Figure 7.2 Proportionate graph for circular sewer section (CPHEEO Manual, 1993)

Consider a layer of sediment of unit length, unit width and thickness ‘t’, is deposited at the invert of the sewer (Figure 7.3). Let the slope of the sewer is θ degree with horizontal. The drag force or the intensity of tractive force (ι) exerted by the flowing water on a channel is given by:

$$\iota = \gamma_w \,.\, R.\, S \qquad (12)$$

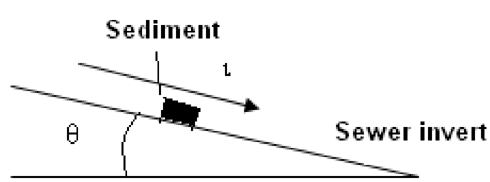


Figure 7.3 A sediment particle moving on the sewer invert

Where,

γ_w = unit weight of water

R = Hydraulic mean depth

S = slope of the invert of the sewer per unit length

With the assumption that the quantity of tractive force intensity at full flow and partial flow implies equality of cleansing, i.e., for sewers to be same self-cleansing at partial depth as full depth:

$$\iota = T$$

Therefore, $\gamma_w \,.\, r.\, s_s = \gamma_w .\, R.\, S$ (13)

Hence, $s_s = (R/r)\, S$

Or $$\frac{s_s}{S} = \frac{R}{r} \quad (14)$$

Therefore,

$$\frac{v_s}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{2/3}\left(\frac{s_s}{S}\right)^{1/2} \quad (15)$$

OR, by substituting $r/R = S/s_s$

$$\frac{v_s}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{1/6} \quad (16)$$

And

$$\frac{q_s}{Q} = \frac{N}{n}\frac{a}{A}\left(\frac{r}{R}\right)^{1/6} \quad (17)$$

Example: 2

A 300 mm diameter sewer is to flow at 0.3 depth on a grade ensuring a degree of self cleansing equivalent to that obtained at full depth at a velocity of 0.9 m/sec. Find the required grade and associated velocity and rate of discharge at this depth. Assume Manning's rugosity coefficient n = 0.013. The variation of n with depth may beneglected.

Solution:

Manning's formula for partial depth

$$v = \frac{1}{n} r^{2/3} s^{1/2}$$

For full depth

$$V = \frac{1}{N} R^{2/3} S^{1/2}$$

Using V = 0.90 m/sec, N = n = 0.013 and R = D/4 = 75 mm = 0.075 m

$$0.90 = \frac{1}{0.013} 0.075^{2/3} S^{1/2}$$

S = 0.0043

This is the gradient required for full depth.

and, Q = A.V = $\pi/4\ (0.3)^2 \times 0.90 = 0.064\ m^3/s$

At depth d = 0.3D, (i.e., for d/D = 0.3) we have a/A = 0.252 and r/R = 0.684 (neglecting variation of n)

Now for the sewer to be same self cleansing at 0.3 m depth as it will be at full depth, we have the gradient (s_s) required as $s_s = (R/r)S$

Therefore, $s_s = S / 0.684$

$= 0.0043 / 0.0684 = 0.0063$

Now, the velocity v_s generated at this gradient is given by

$$v_s = V \frac{N}{n}\left(\frac{r}{R}\right)^{1/6}$$

$= 1 \times (0.684)^{1/6} \times 0.9$

$= 0.846$ m/s

The discharge q_s is given by

$$q_s = Q \frac{N}{n}\frac{a}{A}\left(\frac{r}{R}\right)^{1/6}$$

$q_s = 1 \times (0.258) \times (0.939) \times (0.064)$

$= 0.015\ m^3/s$

Example: 3

A combined sewer was designed to serve an area of 60 sq. km with an average population density of 185 persons/hectare. The average rate of sewage flow is 350 L/Capita/day. The maximum flow is 50% in excess of the average sewage flow. The rainfall equivalent of 12 mm in 24 h can be considered for design, all of which is contributing to surface runoff. What will be the discharge in the sewer? Find the diameter of the sewer if running full at maximum discharge.

Solution:

Total population of the area = population density x area

$= 185 \times 60 \times 10^2$

$= 1110 \times 10^3$ persons

Average sewage flow = $350 \times 11.1 \times 10^5$ Liters/day

$= 388.5 \times 10^6$ L/day

= 4.5 m^3/sec

Storm water flow = 60 x 10^6 x (12/1000) x [1/(24 x 60 x 60)]

= 8.33 m^3/sec

Maximum sewage flow = 1.5 x average sewage flow

= 1.5 x 4.5 = 6.75 m^3/sec

Total flow of the combined sewer = sewage flow + storm flow

= 6.75 + 8.33 = 15.08 m^3/sec

Hence, the capacity of the sewer = 15.08 m^3/sec

Hence, diameter of the sewer required at the velocity of 0.9 m/s can be calculated as

$\pi/4\ (D)^2 \times 0.90 = 15.08\ m^3/s$

Hence, D = 4.62 m

Example: 4

Find the minimum velocity and gradient required to transport coarse sand through a sewer of 40 cm diameter with sand particles of 1.0 mm diameter and specific gravity 2.65, and organic matter of 5 mm average size with specific gravity 1.2. The friction factor for the sewer material may be assumed 0.03 and roughness coefficient of 0.012. Consider k = 0.04 for inorganic solids and 0.06 for organic solids.

Solution

Minimum velocity i.e. self cleansing velocity

$$Vs = \sqrt{\frac{8k}{f'}(Ss-1)gd'}$$

$$Vs = \sqrt{\frac{8x0.04}{0.03}(2.65-1)x9.81x0.001}$$

= 0.4155 m/sec say 0.42 m/sec

Similarly, for organic solids this velocity will be 0.396 m/sec

Therefore, the minimum velocity in sewer = 0.42 m/sec

Now, Diameter of the sewer D = 0.4 m

Hydraulic Mean Depth = D/4 = 0.4/4 = 0.1 m

Using Manning's formula:

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

$0.42 = (1/0.012) \times (0.1)^{2/3} \times S^{1/2}$

S = 1/1824.5

Therefore, gradient of the sewer required is 1 in 1824.5.

Example : 5

Design a sewer running 0.7 times full at maximum discharge for a town provided with the separate system, serving a population 80,000 persons. The water supplied from the water works to the town is at a rate of 190 LPCD. The manning's n = 0.013 for the pipe material and permissible slope is 1 in 600. Variation of n with depth may be neglected. Check for minimum and maximum velocity assuming minimum flow 1/3 of average flow and maximum flow as 3 times the average. (for d/D = 0.7, q/Q = 0.838, v/V = 1.12)

Solution

Average water supplied = 80000 x 190 x (1/24 x 60 x 60 x 1000) = 0.176 m^3/sec

Sewage production per day, (considering 80% of water supply) = 0.176 x 0.8 = 0.14 m^3/sec

Maximum sewage discharge = 3 x 0.14 = 0.42 m^3/sec

Now for d/D = 0.7, q/Q = 0.838, v/V = 1.12

Therefore, Q = 0.42/0.838 = 0.5 m^3/sec

Now

$$Q = \frac{1}{n}\frac{\pi D^2}{4}\left(\frac{D}{4}\right)^{2/3} S^{1/2}$$

$$Q = \frac{1}{0.013}\frac{\pi D^2}{4}\left(\frac{D}{4}\right)^{2/3}\left(\frac{1}{600}\right)^{1/2}$$

Therefore, D = 0.78 m

V = Q/A = 1.04 m/sec

Now, v/V = 1.12

Therefore v = 1.12 x 1.04 = 1.17 m/sec

This velocity is less than limiting velocity hence, OK

Check for minimum velocity

Now q_{min} = 0.14/3 = 0.047 m^3/sec

q_{min}/Q = 0.047/0.5 = 0.09

From proportional chart, for q/Q = 0.09, d/D = 0.23 and v/V = 0.65

Therefore, the velocity at minimum flow = 0.65 x 1.04 = 0.68 m/sec

This velocity is greater than self cleansing velocity, hence OK

d_{min} = 0.23 x 0.78 = 0.18 m

Comment: If the velocity at minimum flow is not satisfactory, increase the slope or try with reduction in depth of flow at maximum discharge or reduction in diameter of the sewer.

Assignment: Solve the above problem with population 100000 persons and pipe flowing 0.75 full at maximum discharge. The rate of water supply is 150 LPCD, n = 0.013, and permissible S = 1 in 600.

7.10 Design of Storm Water Drains for Separate System

Important points for design

Storm water is collected from streets into the link drains, which in turn discharge into main drains of open type. The main drain finally discharges the water into open water body. As far as possible gravity discharge is preferred, but when it is not possible, pumping can be employed. While designing, the alignment of link drains, major drains and sources of disposal are properly planned on contour maps. The maximum discharge expected in the drains is worked out. The longitudinal sections of the drains are prepared keeping in view the full supply level (FSL) so that at no place it should go above the natural surface level along the length. After deciding the FSL line, the bed line is fixed (i.e. depth of drain) based on following consideration.

a. The bed level should not go below the bed level of source into which storm water is discharged.
b. The depth in open drain should preferably be kept less than man height.
c. The depth is sometimes also decided based on available width.
d. The drain section should be economical and velocities generated should be non-silting and non-scouring in nature.

The drain section is finally designed using Manning's formula. Adequate free board is provided over the design water depth at maximum discharge.

7.11 Laying of Sewer Pipes

- Sewers are generally laid starting from their outfall ends towards their starting points. With this advantage of utilization of the tail sewers even during the initial periods of its construction is possible.
- It is common practice, to first locate the points where manholes are required to be constructed as per drawing, i.e., L-section of sewer, and then laying the sewer pipe straight between the two manholes.
- The central line of the sewer is marked on the ground and an offset line is also marked parallel to the central line at suitable distance, about half the trench width plus 0.6 m. This line can be drawn by fixing the pegs at 15 m intervals and can be used for finding out center line of the sewer simply by offsetting.
- The trench of suitable width is excavated between the two manholes and the sewer is laid between them. Further excavation is then carried out for laying the pipes between

the next consecutive manholes. Thus, the process is continued till the entire sewers are laid out.

- The width of the trench at the bottom is generally kept 15 cm more than the diameter of the sewer pipe, with minimum 60 cm width to facilitate joining of pipes.
- If the sewer pipes are not to be embedded in concrete, such as for firm grounds, then the bottom half portion of the trench is excavated to confirm the shape of the pipe itself. In ordinary or softer grounds, sewers are laid embedded in concrete.
- The trench is excavated up to a level of the bottom embedding concrete or up to the invert level of the sewer pipe plus pipe thickness if no embedding concrete is provided. The designed invert levels and desired slope as per the longitudinal section of the sewer should be precisely transferred to the trench bottom.
- After bedding concrete is laid in required alignment and levels. The sewer pipes are then lowered down into the trench either manually or with the help of machines for bigger pipe diameters.
- The sewer pipe lengths are usually laid from the lowest point with their sockets facing up the gradient, on desired bedding. Thus, the spigot end of new pipe can be easily inserted on the socket end of the already laid pipe.

7.12 Hydraulic Testing of Sewers

7.12.1 Test for Leakage or Water Test

The sewers are tested after giving sufficient time for the joints to set for no leakage. For this sewer pipe sections are tested between the manholes to manhole under a test pressure of about 1.5 m water head. To carry this, the downstream end of the sewer is plugged and water is filled in the manhole at upper end. The depth of water in manhole is maintained at about 1.5 m. The sewer line is inspected and the joints which leak are repaired.

7.12.2 Test for Straightness of alignment

This can be tested by placing a mirror at one end of the sewer line and a lamp at the other end. If the pipe line is straight, full circle of light will be observed.

Backfilling the trench: After the sewer line has been laid and tested, the trenches are back filled. The earth should be laid equally on either side with layer of 15 cm thickness. Each layer should be properly watered and rammed.

Questions

1. A 900 m long storm sewer collects water from a catchment area of 40 hectares, where 35% area is covered by roof (C=0.9), 20% area by pavements (C=0.8) and 45% area is covered by open plots (C=0.15). Determine the average intensity of rainfall and diameter of storm water drain. Assume the time of entry = 3 min; velocity at full flow = 1.45 m/sec; gradient of sewer = 0.001, and roughness coefficient = 0.013. The intensity of rainfall, cm/h = 75/(t + 5).
2. Explain the importance of considering minimum and maximum velocity while designing the sewers.
3. Explain 'Self-cleansing velocity'.
4. Explain important consideration while finalizing alignment and bed line of storm water drain.
5. Find the gradient required in sewer of 0.5 m diameter to maintain self cleansing velocity at flow full condition.
6. What hydraulic tests are conducted on the sewers?

Answers

Q. 1: Overall runoff coefficient = 0.5425; Average intensity of rainfall = 4.09 cm/h; Storm water quantity = 2.465 m^3/sec; and diameter of storm water drain = 1.556 m