CHAPTER ONE

1. Introduction to Sewerage System

With the growth of the cities, the more primitive methods of excreta disposal have necessarily given place to the water carriage system. It is the duty of a sanitary engineer to judge the feasibility of any new sewerage scheme for a town, its construction and maintenance during operation. This includes safe disposal of waste materials of the community and keep a good sanitary condition of the town. Sanitary engineers through proper control and management of sewage disposal, refuse sanitation, air pollution control etc keep a clean and healthy environment and prevent the transmission of diseases.

Drainage systems are needed in urban areas because of the interaction between human activity and the natural water cycle. This interaction has two main forms:

i. The abstraction of water from the natural cycle to provide a water supply for human life,

ii. The covering of land with impermeable surfaces that divert rainwater away from the local natural system of drainage.

These two types of interaction give rise to two types of water that require drainage. The first type, wastewater, is water that has been supplied to support life, maintain a standard of living and satisfy the needs of industry. After use, if not drained properly, it could cause pollution and create health risks. Wastewater contains dissolved material, fine and larger solids, originating from Water closet (toilet), from washing of various sorts, from industry and from other water uses. The second type of water requiring drainage, storm water, is rainwater (water resulting from any form of precipitation) that has fallen on a built-up area. If storm water were not drained properly, it would cause inconvenience, damage, flooding and further health risks. It also contains some pollutants, originating from rain, air or the catchment surface.

Urban drainage systems handle these two types of water with the aim of minimizing the problems caused to human life and the environment. Thus urban drainage has two major interfaces: with the public and with the environment.

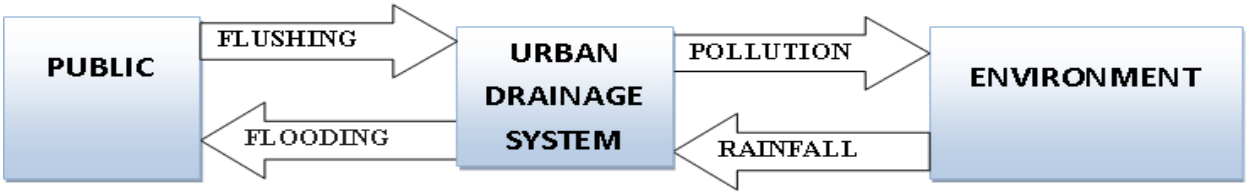


Figure 1.1 Interfaces with the public and the environment

The public is usually on the transmitting rather than receiving end of services from urban drainage („flush and forget"), and this may partly explain lack of public awareness and appreciation of a vital urban service. The sanitary engineer job became essential with the rapid increase of cities and other rural population concentrations. It is the branch of public health engineering dealing with collection, conveyance, treatment and disposal of wastes (garbage, sullage, sewage). The main purpose of sanitary engineering is to maintain such environment as will not affect the public health in general.

Definition of terms

- **Sewage:** It is a liquid waste or foul water of the community conveyed by sewer. Depending on the nature and sources sewage can be classified as:
- **Domestic Sewage**: The sewage from residential buildings, business centers, institutions also contains human body waste (faeces and urine) and also sullage.
- **Industrial Sewage:** The liquid wastes obtained from industrial process such as dying, papermaking etc are known as industrial sewage.
- **Sanitary Sewage**: is a combination of what is produced by domestic and industrial sewages. It contains organic matters and other pollutant that are harmful in nature.
- **Storm sewage**: It is that part of surface run-off which is flowing in sewer during or following a period of rainfall.
- **Sullage:** It is the waste water resulting from personal washing, bathing, laundry, food preparation and cleansing of utensils. It does not include discharge from hospital operation which has high content of organic matters. Sullage is not very foul and can be disposed in open drains without treatment.
- **Garbage:** It is the dry refuse of town containing organic, inorganic solids, semisolids, combustible, noncombustible, putresible and non putrescible substances. It includes sweeping from homes, streets market, public places, gardens, waste papers, leaves, grass parings of vegetables, decaying fruits etc with small amount of cinder, clay and gravel. It is collected separately from sewage and sullage and disposed of separately.
- **Infiltration: i**t is the water that enters the sewers from the ground through leaks from cracks or loose joints.
- **Exfiltration:** It is leakage from the sewer to the ground.
- **Inflow:** It is the water which enters the sewers through cracks in manholes, open cleanouts, perforated manhole covers and roof drains connected to the sewers during runoff.

- **Sewers:** it is a pipe or conduit carrying sewage. Sewers are usually not flow full (Gravity Flow). The full flowing sewers are called force main as the flow is under pressure. Based on the method (systems) of collection and conveyance, sewers are divided in to three:
  - *Sanitary sewer* is one, which carries sanitary sewage and excludes storm sewage as far as possible.
  - *Storm sewer* carries storm water only, including surface runoff and street wash.
  - *Combined sewer* is designed to carry all types of sewage, such as domestic sewage, industrial wastes, and storm water.

Based on the of the function of the sewers, the principal types found in most collection system are:
Building sewers, Trunk sewers, Lateral or branch sewers, Intercepting sewers and Main sewers.

1.1 Type Sewerage System

Sewerage is the science and art of collecting, treating and disposal of sewage. Sewerage system is the entire system used for collection, treatment and disposal of liquid waste. This includes pipes, manholes and all structures used for disposal of sewage. Generally there are three types of sewerage system.

1. **Separate sewerage system:** in this system the sanitary sewage and storm water are carried separately in two set of sewers. The sewage is conveyed to waste water treatment plant (WWTP) and the storm water is discharged into rivers without treatment. The separated system is suitable when separate outlet for storm water is available and the topography is such that storm water can be disposed into natural drains.

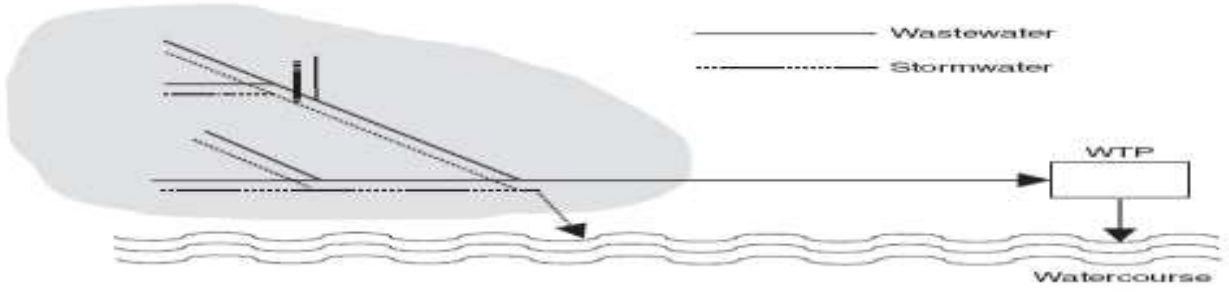


Figure1.2 schematic plan Separate system

Advantages of Separate Sewerage System:

- The load on treatment plant is less as only sewage is carried to the plant.
- The size of sewer is mall, thus economical
- When pumping is required, the system proves to be economical.
- Storm water is not unnecessarily polluted by sewage.

Disadvantages of Separate System:

- Cleaning of sewer is difficult due to their small size.
- Self-cleaning velocity is not available, due to small quantity of sewage; therefore flushing is required at various points.

The storm sewers come into operation in rainy season only. It may be chocked in dry season by garbage. As two sets of sewers are laid, therefore maintenance cost is more. lying of two sewers are difficult in busy lanes, which causes great inconvenience to traffic during repair. There is always a risk that storm water may enter the sanitary sewer and cause overflowing of sewer and heavy load on treatment plant.

2. **Combined sewerage system:** in this system the sewage and storm water are carried combine in only one set of sewers to the waste water treatment plant (WWTP) before disposal.

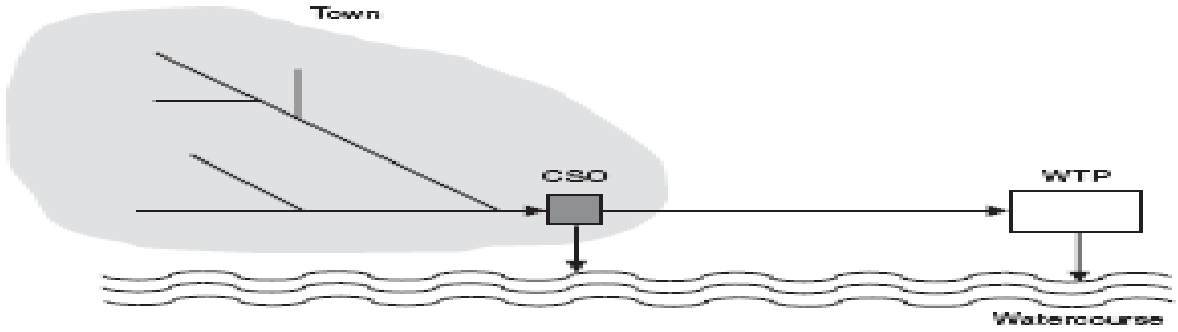


Figure 1.3 Schematic plan combined sewerage system

It is simply not economically feasible to provide capacity for this flow along the full length of the sewers.
It would also be unfeasible to provide this capacity at the treatment plant in the treatment processes. Therefore, the solution is to provide structures in the sewer system which, during medium or heavy rainfall, diverts flows above a certain level out of the sewer system and into a natural watercourse. These structures are called combined sewer overflows, or CSOs.

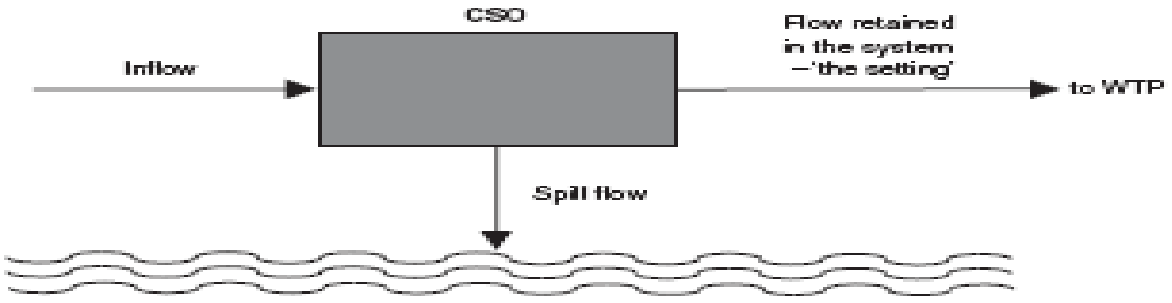


Figure 1.4 Combined sewer overflows

The basic function of a CSO is to receive inflow, during rainfall, which consists of storm water mixed with wastewater. Some flow is retained in the sewer system and continues to the treatment works. The amount of

this flow is an important characteristic of the CSO, and is referred to as the „setting". The remainder is overflowed to the watercourse – the overflow or „spill flow".

Advantages of Combined Sewerage System

- Easy cleaning because of larger diameter
- Reasonable maintenance cost
- Strength of sewage is reduced due to dilution of sewage by storm water
- This system requires only one set of sewer making it economical

Disadvantages of Combined Sewerage System

- During heavy rains the over flowing of sewers will endanger the pubic heath
- The combined sewer gets silted and becomes foul in dry season
- Load on treatment plant is more because storm water is also carried there
- The storm water gets polluted unnecessarily
- The system becomes uneconomical when pumping is needed

3. **Partially separate sewerage system***:* This system is the compromise between separate and combine system taking the advantages of both systems.

Advantages of Partially Separate Sewerage System:

- It combines the good features of both systems.
- The silting is avoided due to entry of storm water.
- The storm water from houses is easily disposed off.
- Sewers are of reasonable size.

Disadvantages of Partially Separate Sewerage System:

- There may be delayed storm discharge and the concerned area may remain flooded for a considerable time.
- Consequently sanitation of the city is affected.
- Not popular in many areas and seen as compromise solution.
- Not used to any considerable extent for new developments.

Sewerage system is the system of sewer including all appurtenances required for sewage disposal. A sewerage system will consist of mainly:

A. Collection work
B. Treatment work
C. Disposal work

A. Collection works: are provided for collecting sewage from different points of occurrence and conveying sewage to any desired points with the help of a sewer system.

- *Building sewers/connections* bring wastewater from the houses to the laterals/branch lines.
- *Laterals/ branch sewers* are used to collect wastewater from one or more building sewers and convey it to a main sewer.
- *Main sewers* are used to convey wastewater from one or more lateral to trunk sewers or intercepting sewers.
- *Trunk sewers* are large sewers that are used to convey wastewater from main sewers to treatment or other disposal facilities or large intercepting sewers.
- *Intercepting sewers* are large sewers that are used to intercept a number of main or trunk sewers and convey the wastewater to treatment or other disposal facilities.

B. Treatment work: sewerage treatment covers any process to which the sewage is subjected in order to remove or alter its objectionable constituents so as to render it less dangerous or offensive.

C. Disposal work: sewage disposal applies to the act of disposing of sewage by any method to a body of water or on land. It may be done with or without any previous treatment of the sewage.

1.2 Sanitation System

System of sanitation is classified on the basis of collection and conveyance of waste produced in a community. This is mainly related to excreta disposal. The methods of collection adopted are mainly the following two types:

1. Non-water carriage sanitation systems
2. Water carriage sanitation system.

1.2.1 Non-water carriage sanitation systems

This is the oldest system by which garbage or dry refuse of a town is collected in dustbins, placed along the roads, from where it is conveyed (trucks or covered carts), once or twice a day to the points of disposal.

1. All for filing, the non-combustible portions of garbage (sand, dust & clay) are used for filling the low-level areas to reclaim land for further development of town.
2. The combustible portion of garbage (dry levels, waste paper, broken furniture etc) is burnt.
3. The decayed fruits and vegetables, grass and other such things are first dried and then disposed by burring or in the manufacture of manure(substance especially dung as fertilizer).

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Human excreta or night soil is collected separately in privies or conservancy latrines. The liquid and semi-liquid wastes are collected in separate pans in the same latrine, from where trey are removed through human agency and night soil is taken outside the town in closed animal drawn carts, trucks and is buried in trenches. After 2 to 3 years the buried night soil is converted in to excellent manure, which can be used for growing crops.

Waste Disposal in Non-Sewered Areas

In case of poor countries, it is not possible to have the water carriage system in all towns, villages and cities in rural areas having a shortage of water supply due to which the quantity of wastewater is also small. Sewerage system (wastewater collection and treatment) is not available for the disposal of sanitary wastes in most towns and all rural areas of our country. Therefore, privies are in use for collection and disposal of night soils. Conservancy is chosen and in such system human excreta is collected in various types of privies. The types of privies that may be adopted are the following:

- Bore -hole latrines
- Pour- flush latrines
- Conventional unimproved pit latrines
- Ventilated improved pit latrines (VIP)
- Double tank latrines
- Septic tank latrines
- Aqua privy
- Chemical toilets
- Working type privy

The following are the merits of the conservancy system

- It is very cheap in initial cost
- Storm water is carried in usually surface drains, hence no problem of pumping the storm water.
- The quantity of waste liquid reaching the disposal point is less, hence it can be disposed of without any treatment
- As the storm water is carried in open drains, the sewer section will be small and will run full for the major portion of the year, due to which there will be no silting and deposits in the sewer lines.

Demerits of the conservancy system

- Due to foul smell from the latrines, they are to be constructed away from the living room, so buildings cannot be constructed as compact units.
- The transportation of night soil through streets or other crowded localities causes nuisance of smell and insanitation.
- As sewage is disposed of without any treatment, it may pollute the natural water courses
- It is unhygienic from point of view of removal of human excreta manually.
- Due to improper or careless disposal of the night soil, there is risk of outbreak of epidemics
- For burying human excreta more space of land is required
- In the presence of conservancy system, the aesthetic appearance of the city cannot be increased

1.2.2 Water Carriage System

In water carriage system, water is used as a media of conveying the sewage to the points of treatment and disposal. Spent up water for domestic use which otherwise goes to waste is employed carrying night soil, hence the name is given as water carriage system. In this system excremental matters are mixed up with large quantity of water, are takeout from the city through properly designed sewerage systems, and disposed after necessary treatment. Here no human agency is directly employed for collection and conveyance of night soil. Some additional water is required for flushing and collecting human excreta.

The disposal of sewage is done to a body of natural water or on land after doing the required treatment. Storm water may be carried in combined sewers or in separate sewerage system. If untreated wastes are accumulated or spread everywhere, the decomposition of organic materials lead to production of malodorous gases, pathogenic organisms, and toxic compounds and will pollute our environment (land, air, water).

Merits of the Water-carriage system

- All solid and liquid wastes have to be collected and conveyed to suitable places for treatment and are disposed safely.
- It is hygienic method because all the excremental matters are collected and conveyed by water only and no human agency is employed for it.
- There is no nuisance in the streets of the towns because all the sewage goes in closed sewers under the ground, the epidemics (spreading of disease) is reduced.
- Due to more quantity of sewage self cleaning velocity can be obtained even at less gradient.
- This system does not depend on manual labor at every time except when sewers get chocked.

Demerits of the Water-carriage system

- High initial cost
- Maintenance of this system is also costly
- Large volume of sewage is to be treated during monsoon (wet season) where as very small volume is to be treated in the remaining period of the year.

Comparison of water-carriage and non water-carriage system

Conservancy system	Water-carriage system
- Cheaper as it has no initial cost - As the waste liquid is less foul in nature and small in volume it needs no treatment. - Storm water carried in open drains, generally does not require pumping. - Sanitation is of lower degree as storm water may get polluted with organic load while conveyed through open drains - Excreta are removed after causing putrefaction and resulting nuisance of smell, fly and unhygienic condition. - The cities aesthetic appearance cannot be increased - Disposal of excremental matters by burial will require extensive areas - Possibility of foul liquid from latrines to pollute the ground water is high - Buildings cannot be designed as one compact unit due to smell	- It has high initial cost - The sewage is highly polluted in nature and huge in amount it needs treatment before disposal. So it is costlier in operation also - Pumping of sewage is frequently required - Sewage treatment before disposal gives a required degree of sanitation - Excreta are immediately removed, so no such problem arises. This system is much more hygienic. - Aesthetically the system is much more acceptable - Compact unit of modern sewage treatment work will require much less area - The chances of underground water getting polluted are less - Latrines can be attached to the living rooms and buildings can be designed as one compact unit

1.3 Planning a Sewerage Project

Prior to planning and designing a sewerage project certain survey and investigation works need to be conducted. On the basis of the findings decision has to be taken, whether sewerage scheme is required or not, if required what system is to be adopted, nature and extent of treatment to be provided etc. The following data need to be collected in preparing a scheme.

a. **Population:** this will be required for knowing the present and future population at the end of design period.
b. **Quantity of water:** Quantity of water supply from various sources in a city will help in determining the quantity of sewage flow.
c. **Industries:** the nature and number of various industries are to be determined. This helps in assessing the quantity of industrial wastes
d. **Rainfall data:** this will help in estimating the quantity of storm flow. It also helps in deciding which sewerage system to adopt, whether a combined or separate system.
e. **Surveying:** the area survey including contour map, location of pumping station, treatment plant, outfall etc. are determined with the help of survey report. Nature and layout of sewage system is also dependent on the topography of the area under consideration.
f. **Scope of expansion**: care should be taken in planning a sewerage scheme that there should be possibility for future expansion. On practical and economic considerations, the design period cannot be kept too long and the system may have to be extended after design period.
g. **Economy:** The scheme has to be made not only economical by proper planning, design and execution but also has to be adjusted within the available funds.

Under favorable geographical and topographical conditions, a separate system with minor modifications for existing natural drainage system for storm water disposal may be helpful. This may help in reducing the cost of smaller size sanitary sewers and sewage treatments as well.

CHAPTER TWO

2. Type and Quantity of Sewage

2.1 Introduction

Determining the rates wastewater flow is a fundamental step in the design of wastewater collection, treatment, and disposal facilities. Reliable data on existing and projected flows must be available if these facilities are to be designed properly and if the associated costs are to be minimized and also shared equitably when the facilities serve more than one community or district. In situations where wastewater flow rate data are limited or unavailable, wastewater-flow rate estimates have to be developed from water consumption records and other information.

The sewage collected from the municipal area consists of wastewaters from the residences, commercial centers, recreational activities, institutions and industrial wastewaters that are discharged in to sewer network. Before designing the sewer, it is necessary to know the discharge (i.e., quantity of sewage) which will flow into it after completion of the project. Accurate estimation of sewage discharge is necessary for hydraulic design of the sewers.

Actual measurement of the discharge is not possible when sewers do not exist, the capacity of the existing sewers is inadequate and need to be increased and there is unaccounted overflow and leakages that might be occurring in the existing system. Since sewers are designed to serve for some more future years, engineering skills have to be used to accurately estimate the sewage discharge. Determining the rates wastewater flow is a fundamental step in the design of wastewater collection, treatment, and disposal facilities.

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2.2 Types of sewage

The components that make up the wastewater flow from a community depend on the type of collection system used and may include the following:

1. **Domestic (also called sanitary) sewage:** wastewater discharged from residences and from commercial, institutional, and similar facilities. Sources of sanitary sewage could be:
   - Water supplied by water authority for domestic usage, after desired use is discharged in to sewers as sewage.

- Water supplied to different industries for various industrial processes by local authority.
- Some quantity of this water after use in different industrial applications is discharged as wastewater.
- 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 sewers as wastewater.
- Water drawn from wells by individuals to fulfill domestic demand. After uses this water is discharged in to sewers.
- The water drawn from individual water sources such as wells, tube wells, lake, river, etc for various industrial purposes.
- Fraction of this water is converted in to wastewater and discharged in to sewers.
- Infiltration of ground water into sewers through leaky joints.
- Entrance of rainwater in sewer lines during rainy season through faulty joints or cracks in sewers.

Most municipal wastewater treatment plants discharge their effluent into rivers, lakes, or oceans. Reduced oxygen levels in rivers can cause harm to the aquatic life, and increased nutrient levels stimulate the growth of algae, which consume oxygen.

Pathogenic microorganisms discharged into the ocean can infect human who come in contact with the ocean water in recreational areas such as beaches. Domestic wastewater discharged from septic tanks contains large numbers of pathogenic microorganisms, with viruses being of particular concern because of the ability of viruses to move considerable distances in ground water. Septic tanks represent the highest total volume of wastewater discharged directly to ground water and are the most common source of ground-water contamination.

Properly designed, operated, and maintained sanitary-sewer systems collect and transport all of the sewage to treatment works Occasionally, however, unintentional discharges of raw sewage from municipal sanitary-sewer occur in almost every system. These types of discharges called sanitary-sewer overflows (SSO), have a variety of causes including extreme weather, improper system operation and maintenance, and damage etc. The untreated sewage from SSOs can contaminate receiving waters and decrease in water quality reduces number and range of plants and fish. Nutrients and toxicants may cause algal blooms and harm wildlife. Algal blooms remove O2 from water, smothering aquatic life.

2. **Industrial sewage (Non-domestic):** wastewater in which industrial wastes predominate. Some industries provide pretreatment prior to discharging their wastewaters either directly into surface waters or into municipal sewer systems for further treatment. In many countries where industries are

permitted to discharge their wastewater without adequate pretreatment, the subsequent human and environmental impacts are usually noticeable.

3. **Storm water:** Runoff resulting from rainfall and snowmelt. Storm water discharges are generated by runoff from open land and impervious areas such as paved streets, parking lots, and building rooftops during rainfall and snow events. Although storm water runoff is commonly discharged through a single outfall pipe, such discharges are more accurately classified as nonpoint pollutant sources. Storm water runoffs usually contain pollutants in quantities that could affect water quality adversely. The primary method to control the quality and quantity of storm water discharges is through the use of best management practices.

4. **Infiltration /Inflow:** water that enters the sewer system through indirect and direct means. Infiltration is extraneous water that enters the sewer system through leaking joints, cracks and breaks, or porous walls. Inflow is storm water that enters the sewer system from storm drain connections (catch basins), roof leaders, foundation and basement drains, or through manhole covers.

2.3 Wastewater sources and flow rates

2.3.1 Domestic Wastewater Sources and Flow rates

The principal source of domestic wastewater in a community is the residential areas and commercial districts. Other important sources include institutional and recreational facilities. For areas now served with sewers, wastewater flow rates are commonly determined from existing records or by direct field measurements. For new developments, wastewater flow rates are derived from an analysis of population data and corresponding projected unit rates of water consumption or from estimates of per capita wastewater flow rates from similar communities.

1. **Residential Areas:** Water used by residential households consists of water for interior use such as showers and toilets and water for exterior use such as lawn watering and car washing. For small residential districts, wastewater flow rates are commonly determined on the basis of population density and average per capita contribution of wastewater. For large residential areas, it is advisable to develop flow rates on the basis of land use areas and anticipated population densities. Where possible, these rates should be based on actual flow data from selected similar residential areas, preferably in the same setting. In many cases, design flows are fixed by water and wastewater agencies. If these data are not available, an estimate of 70% to 80% of the domestic water consumption may be taken as wastewater.

Table 2.1 Typical wastewater flow rates from residential sources

Source	unit	Flow, Lit/unit. d	
		Range	Typical
Apartment			
High-rise	Person	135-285	180
Low-rise	Person	190-305	245
Hotel	Guest	115-210	170
Individual residence:	Person	170-340	265
Typical home	Person	230-380	305
Better home	Person	285-570	360
Luxury home	Person	115-230	170
Older home	Person	95-190	150
Motel			
With kitchen	Unit	340-680	375
Without kitchen	Unit	285-570	360

2. **Commercial Districts:** commercial wastewater flow rates are generally expressed in m3/ ha. d and are based on existing or anticipated future development or comparative data. Average unit-flow rate allowances for commercial developments normally range from 7.5 to 14 m^3/ ha. Day. Because unit flow rates can vary widely for commercial facilities, every effort should be made to obtain records from similar facilities.

Table 2.2 Typical wastewater flow rates from commercial districts

Flow, Lit/unit. day			
Source	**Unit**	**Range**	**Typical**
Airport	Passenger	7-15	11
Automobile service	Vehicle served	25-50	38
station	Employee	35-55	45
Bar	Customer	5-20	11
	Employee	40-60	49
Department store	Toilet room	1500-2270	1893
Hotel	Employee	30-45	38
	Guest	150-210	182

Industrial building	Employee	25-50	38
Laundry	Employee	25-60	49
	Machine	1705-2460	2082
	Wash	170-210	189
Office	Employee	25-60	49
Restaurant	Meal	10-15	11
Shopping center	Employee	25-50	38
	Parking space	5-10	8

3. **Institutional Facilities:** Actual records of similar institutions are the best source of flow data for design purpose. When records are not available, the flow from institutional facilities can be estimated with the data given below.

Table 2.3 Typical wastewater flow rates from institutional sources

Source	**Unit**	**Flow, Lit/unit. d**	
		Range	**Typical**
Hospital Medical	Bed	500-950	650
	Employee	20-60	40
Hospital Mental	Bed	300-500	400
	Employee	20-60	40
Prison	Inmate	300-600	450
	Employee	20-60	40
School with cafeteria, gymnasium and shower	Student	60-115	80
With cafeteria only	Student	40-80	60
School without cafeteria, gymnasium and shower	Student	20-65	40
School boarding	Student	200-400	280

4. **Recreational Facilities :** recreational facilities such as swimming pools, bowling alleys, resorts, and country clubs perform a wide range of functions involving water use.: Wastewater flow rates from many recreation facilities are highly seasonal.

Table 2.4 Typical wastewater flow rates from recreational sources

Facility	Unit	Flow, Lit/unit. d	
		Range	Typical
Apartment, resort	Person	190-265	265
Cabin, resort	Person	30-190	189
Cafeteria	Customer	5-15	11
	Employee	30-45	45
Campground	Person	75-155	151
Cocktail lounge	Seat	45-95	95
Coffee shop	Customer	15-30	30
	Employee	30-45	45
Country club	Member present	225-495	492
Day camp	Employee	40-60	57
Dinning hall	Person	40-60	57
Dormitory	Meal served	15-40	38
Hotel, resort	Person	75-190	189
Store resort	Person	150-230	227
	Customer	5-15	15
Swimming pool	Employee	30-45	45
	Customer	20-45	45
	Employee	30-45	45
Theatre	Seat	7-15	15
Visitor center	Visitor	15-30	30

2.3.2 Industrial wastewater

Wastewater flow rates from industrial sources vary with the type and size of the facility, the degree of water reuse, and onsite wastewater treatment method if any. Extremely high peak flow rates may be reduced by the use of detention tanks and equalization basins. For industries without internal recycling or reuse programs, it can be assumed that about 85 to 95% of the water used in the various operations and processes will become wastewater.

2.3.3 Infiltration/Inflow

water that enters the sewer system through indirect and direct means. Extraneous flows in sewers,

described as infiltration and inflow, are defined as follows:

- **Infiltration:** water entering a sewer system including sewer service connections, from the ground through defective pipes, pipe joints, connections, or manhole walls.
- **Inflow** is storm water that enters the sewer system from storm drain connections (catch basins), roof drains, foundation and basement drains, or through manhole covers.
- **Steady inflow:** water discharged from cellar and foundation drains, cooling water discharges, and drains from springs and swampy areas. This type of inflow is steady and is identified and measured along with infiltration.
- **Direct inflow:** this types of inflow has a direct storm water runoff connection to sanitary sewer and cause an immediate increase in wastewater flows. Possible sources are roof leaders, yard and areaway drains, manhole covers, cross connections from storm drains and catch basins, and combined sewers.
- **Delayed inflow:** storm water that may require several days to drain through the sewer system. This category can include the discharge of sump pumps from cellar drainage as well as the slowed entry of surface water through manholes in ponded areas.
- **Total inflow:** The sum of direct inflow at any point in the system plus any flow discharged from the system upstream through overflows, pumping station bypasses, and the like.
- **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 depend upon the rate of water supply, type of area served, economic conditions of the people, weather conditions and infiltration of ground water in sewers, if sewers are laid below ground water table.

The rate and quantity of infiltration depend on the length of sewers, the area served, the soil and topographic conditions, and to a certain extent the population density (which affects the number and total length of house connection). Infiltration and inflow should be reduced as much as possible. Otherwise there will be additional cost of collection and treatment of infiltration water. Construction specification for sewer projects now limits infiltration to 45 l/km. d [mm] (i.e. 45 l/d per kilometer of sewer length and mm of pipe diameter). If field measurements of wastewater flow rates are not possible and actual wastewater flow rate data are not available water supply records can be used as an aid to estimate these flow rates. Because wastewater consists primarily of used water, the portion of the water supplied that reaches the collection must be estimated.

2.4 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 makes the system uneconomical and hydraulically inefficient. Hence, before designing the sewerage system it is important to know the discharge/quantity of the sewage that will flow 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, the following quantities are considered while estimating the sewage quantity:

i. **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 use is converted into wastewater and ultimately discharged in to sewers. This quantity can be estimated by actual field observations.

ii. **Addition due to infiltration:** The additional quantity of water due to groundwater seepage in to sewers through faulty joints or cracks in the pipe. The quantity of water depends upon the height of the water table above the sewer invert level. If the water table is well below the sewer invert level, the infiltration can occur only after rain when water is moving down through soil. The quantity of the water entering sewers depends upon the permeability of the ground soil and it is very difficult to estimate.

Table 2.5 Suggested estimates for ground water infiltration for sewers lay below ground water 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.

iii. **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.

iv. **Subtraction due to water not entering the sewerage system:** The amount of water that is used for certain purpose like boiler feed water, water sprinkled over the roads, streets, lawns, and gardens, water consumed in industrial product, water used in air coolers, etc. may not generate sewage.

Theoretically speaking, the quantity of sewage (i.e. domestic sewage + industrial sewage) that is likely to

enter the municipal sewers under design should be equal to the quantity of water supplied to the contributing area, from the water works. But in actual practice, this is not the precise quantity which appears as sewage, but certain additions and subtractions do take place from it, as stated below:

i. Additions due to unaccounted private water supplies
ii. Additions due to infiltration
iii. Subtractions due to water losses
iv. Subtractions due to water not entering the sewerage system

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

This net value is vary with in 705 to 130% the accounted water supplied from the works. Generally, 75 to 80% of accounted water supplied is considered as quantity of sewage produced.

2.5 Variation in Sewage Flow

The quantity of sewage as calculated by the average per capita contribution will be purely hypothetical and cannot be used directly for the design purpose. This is because; practically the average sewage never flows in a sewer. It continuously varies from hour to hour (short term variation) and season to season (seasonal variation). The flows in these sanitary sewers, though fluctuates time after time, yet they are sometimes delayed and less pronounced as they are damped because of the storage space in the sewers and because of the time required for the sewage to reach the point of gauging. The typical hourly variation in the sewage flow is shown in the Figure below.

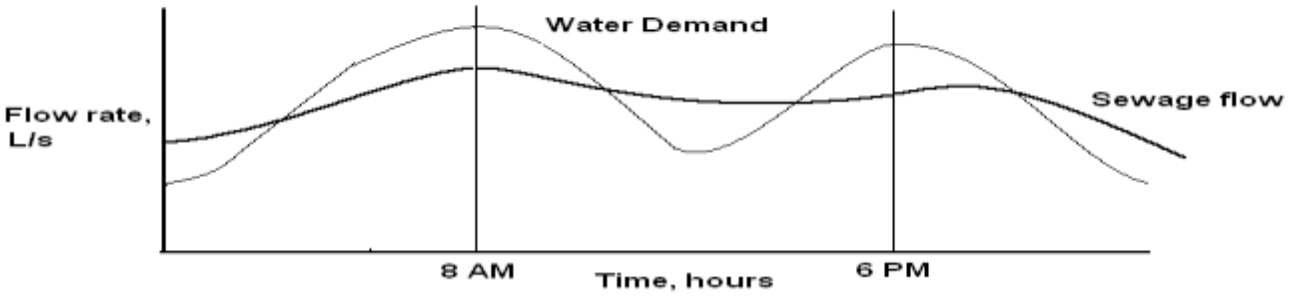


Figure 2.1 Typical hourly variations in sewage flow

In other words, the peaks are flattened, because it requires considerable sewage to fill the sewers to the high flow point, and the high flow from various sections will reach the gauging point after various times of

flow. Thus the time as to when the peak flow occurs will depend upon the flow time in sewers and the type of district served. Hence, if the sewage is gauged near its origin, the peak flow will be quite pronounced; whereas, if the sewage must travel a long distance before being gauged, the peak will be deferred. It, therefore, follows, that the peak flow (expressed as number of times of their average values) will be much greater for smaller lateral sewers, as compared to these for large trunk sewers.

The peak will put off 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 the time required in travelling. For smaller township the 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.

For estimating design discharge the following relation can be considered.

- Maximum daily flow = 2 times the average daily flow (representing seasonal variations). As the tributary area increases, peak hourly flow will decrease.
- For smaller population served (<50,000) 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.
- For the design of treatment facility, the peak factor is considered as 1.5 times the annual average daily flow.

Table 2.6 Hourly Variations in Sewerage Flow

Type of Sewer	Ratio of maximum flow to average flow
Trunk mains above 1.25 m in dia.	1.5
Mains upto 1 m dia.	2.0
Branches upto 0.5 m dia.	3.0
Laterals and small sewers upto 0.25 m dia.	4.0

The minimum flow passing through a sewer is also an important factor in the design of the particular sewer because at low flow, the velocity will be reduced considerably, which may cause silting. Hence, the slope at which the sewer is to be laid has to be decided in accordance with the requirement of minimum permissible velocity at minimum flow. The minimum flow passing through sewers is important to develop self cleansing velocity to avoid silting in sewers. This flow will be generated in the sewers during late night hours.

The effect of this flow is more pronounced on lateral sewers than the main sewers. For moderate areas, such as involved for branch sewers, the following minimum flows may be assumed.

- 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. sizes of the sewers can then be easily designed for carrying the computed maximum hourly flows, with sewers running ¾ full. The peak sewage flow has been related to population by certain empirical follows.

Table 2.7 Empirical relating peak sewage flow to population

Reference	*Method*	*Notes*
Harman (1918)	$1 + \frac{14}{4 + \sqrt{P}}$	1
Gifft (1945)	$\frac{5}{P^{1/6}}$	1
Babbitt (1952)	$\frac{5}{P^{1/5}}$	1
Fair & Geyer (1954)	$1 + \frac{18 + \sqrt{P}}{4 + \sqrt{P}}$	1
—	$4Q^{-0.154}$	2
Gaines (1989)a	$2.18Q^{-0.064}$	3
Gaines (1989)b	$5.16Q^{-0.060}$	3
BS EN 752—4	6	—

[1]Population P in 1000s, [2]Flow Q in 1000 m^3/d, [3]Flow Q in l/s

The sewers and the drains of a separate sewerage system should be designed to carry the maximum sewage discharge and the maximum rain runoff, respectively. On the other hand sewers of a combined sewerage system should be designed to carry the sewage discharge plus the rain runoff. The sewers of a combined system can also carry the low sewage discharge during non-monsoon periods, as dry weather flow, with minimum permissible velocities. The partially separate sewers may be designed for carrying the sewage discharge plus part of the storm drainage, particularly that coming from the roofs and courtyards.

In order to design sewers and drains properly, it is absolutely necessary to estimate the sewage discharge and the urban storm drainage discharge that are likely to enter the sewers or drains. Based upon these basic principles, the rational formula was evolved as:

Q = 2.78 CIA

Where: Q = Peak rate of runoff (l/s)

C=Coefficient of runoff

I= Rainfall intensity (mm/hr).

A = Catchment area (ha).

2.6 Types of Sewer Pipes in a Typical Separate Sanitary Collection System

- *Trunk Sewers* are large sewers that are used to convey wastewater from main sewers to treatment or other disposal facilities, or to large intercepting sewers.
- *Interceptor Sewers* are large sewers that are used to intercept a number of main or trunk sewers and convey the wastewater to treatment or other disposal facilities
- *A main or trunk sewer* collects flow from several sub-mains as well as laterals and house sewers.
- *A lateral sewer* is a common sewer collects flow from one or more laterals as well as house sewers.
- *A house sewer (waste pipe)* is a pipe conveying wastewater from an individual structure to a common sewer or other point of disposal.
- *Force mains* are pressurized sewer lines, which convey sewage from a pumping station to another main or to a point of treatment or disposal.
- *A relief sewer* is a sewer, which has been built to carry a portion of the flow in a system which otherwise inadequate capacity.
- *An outfall sewer* is a sewer, which carries the collected waste to a point of treatment or disposal.
- *Manholes* are provided in every sewer pipe at a suitable interval, so as to facilitate their cleaning and inspection.
- In the sewers, which carry the drainage discharge either solely or in combination with sewage, *inlets* called *catch basins* are provided to permit entrance of storm water from street gutters.

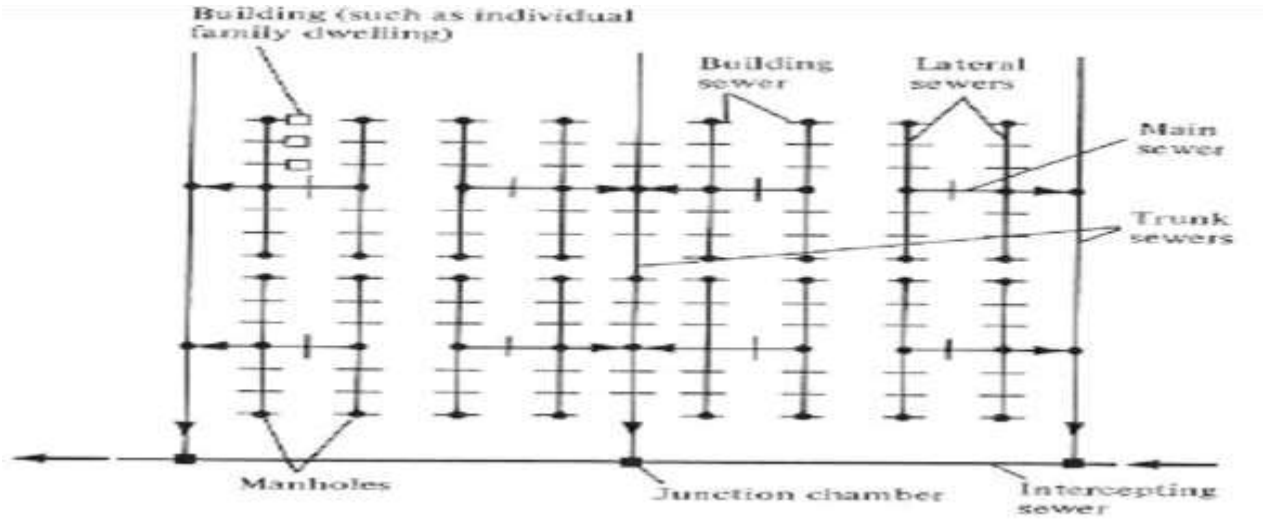


Figure 2.2 Sewer Pipe Types in a Collection System

2.7 Choice of Wastewater Collection System

1. **Pressure Flow:** is a flow condition in which the fluid moves through a closed conduit as a result of source of energy which is external to the conduit proper, such as the energy supplied by a pump or an external pressure head.

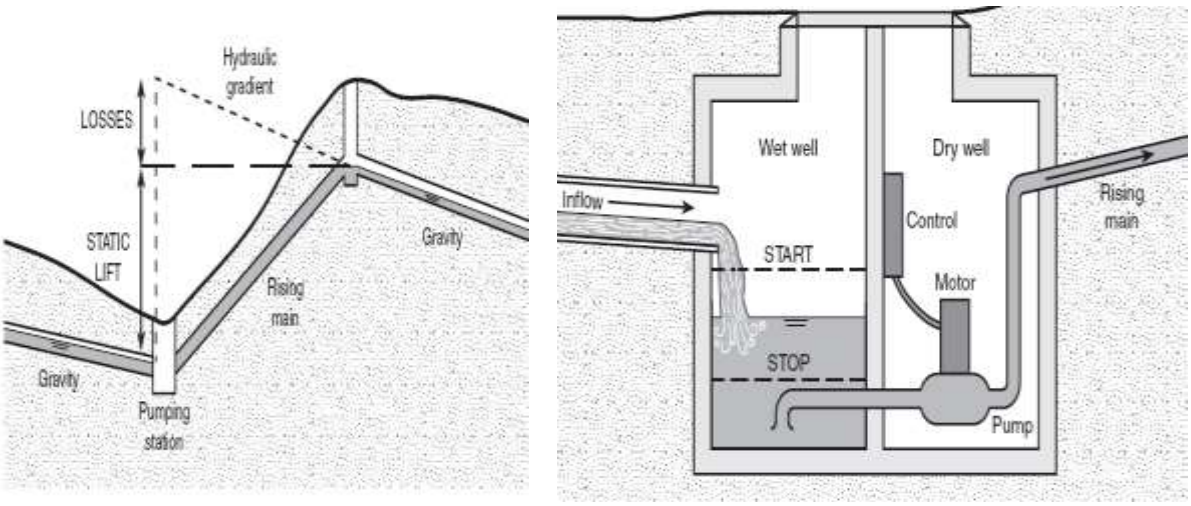


Figure 2.3 Typical sewer pumping arrangement and pumping station arrangement

2. **Gravity Flow:** is a flow condition in which flow takes place due to the energy within the conduit and flowing fluid namely, the force of gravity. Gravity flow can take place in any pipe of conduit, whereas, pressure flow can exist only in a closed conduit, flowing full. Wherever possible, use a gravity flow system. When the natural slopes are not sufficient to convey flow, a combination of gravity and pressure flow systems may be used. The gravity sewer transports flows to a collection point, such as a wet well. The wastewater is pumped from the wet well through a force main over some obstruction or hill to another gravity sewer, or directly to a wastewater treatment facility.

Chapter Three

3. Layout and Design of Sewers

3.1 Sewer Layout

For the collections and disposal of sewage a net work of sewers to be laid. The pattern of sewer layout system depends on the following factors.

a) The Sewerage system adopted whether combined, separate or partially separate system
b) The area to be drained
c) Location of disposal point and
d) The topography and hydrological feature of the area

The following are most common among the various patterns of collection system:

1. **Perpendicular Pattern:** in this pattern number of sewers are laid in such a way that each reaches to the respective outfall point through the shortest route. The sewers are useful for carrying storm water in separate system; otherwise the river will get polluted in several points. Some of the characteristics of perpendicular pattern sewer layout are:
   - Suitable for separate systems and partially separate system.
   - Not suitable for combined system.
   - Treatment is difficult due to many outlets
   - Pollutes the water of natural course.

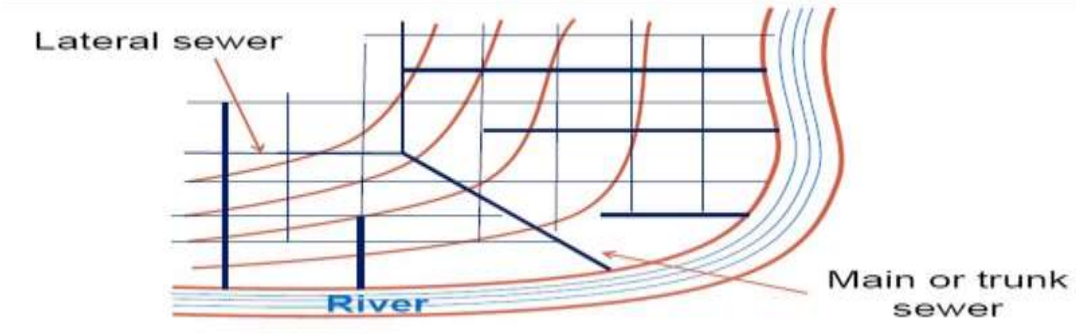


Figure 3.1 Perpendicular pattern sewer layout

2. **Interceptor Pattern:** in this system a number of main sewers are laid serving different zones. Again the mains at their outfall ends are connected by a large size-intercepting sewer, which leads the sewage to pump house or treatment plant. Intercepting sewer at the junctions of main sewers may be provided with over flows that will carry the excess and less polluted storm water to the river. Generally, sewers are intercepted by a large size sewer, which is laid to carry sewage to a common point, where it can be disposed with or without treatment.

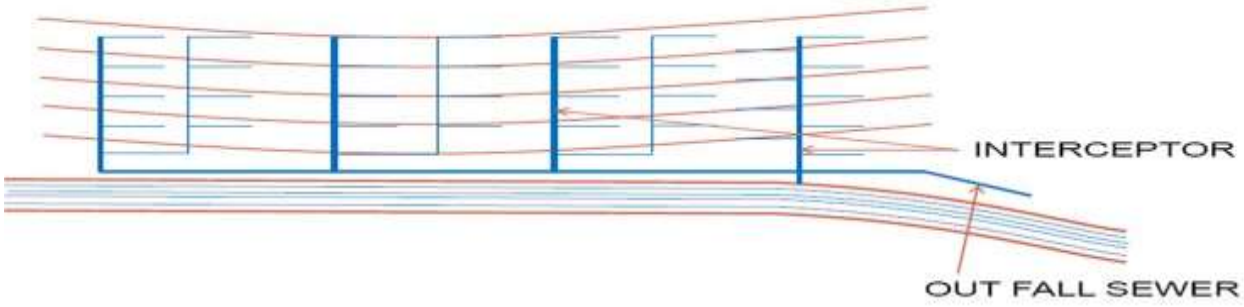


Figure 3.2 Interceptor Pattern Sewer layout

3. **Radial Pattern:** This pattern is used for sanitary sewage in a place where the central part of the area is high and going down all round towards the outskirts. This type of layout pattern can only be used where several disposal points are available. In this pattern large number of outlets is provided and sewers are laid radially outwards from center of the city. Suburbs can be served by relatively small and short lines of sewers, which make it economical. The disadvantage this pattern is it requires large number of disposal works.

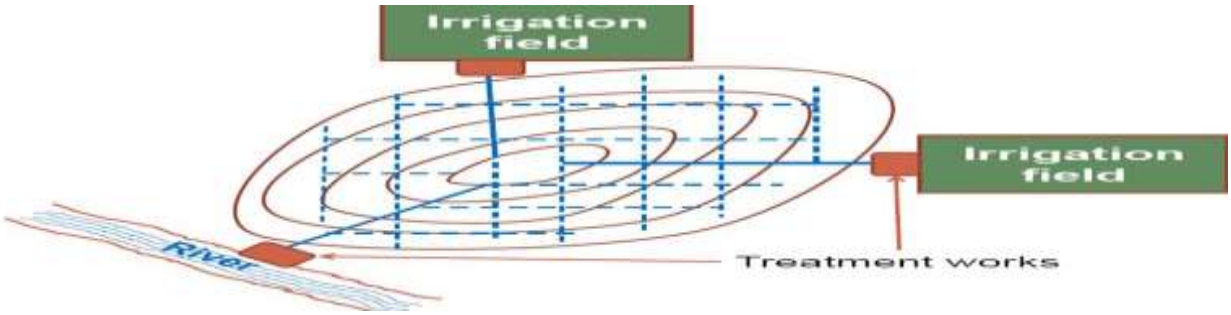


Figure 3.3 Radial Pattern Sewer layout

4. **Fan Pattern:** In this pattern sewers can be laid in such a way that the whole sewage flows to a common point (towards the outfall sewer) where one treatment plant is located. This type of system is used where natural topography is lower towards outfall point and there is short of valley formation along more or less central line Suitable for combined systems and partially separate system. Having a single unit treatment is its advantage. The main trunks are very large and development of suburbs is restricted due to increase of loads is the disadvantage. It may pollutes the water of natural course.

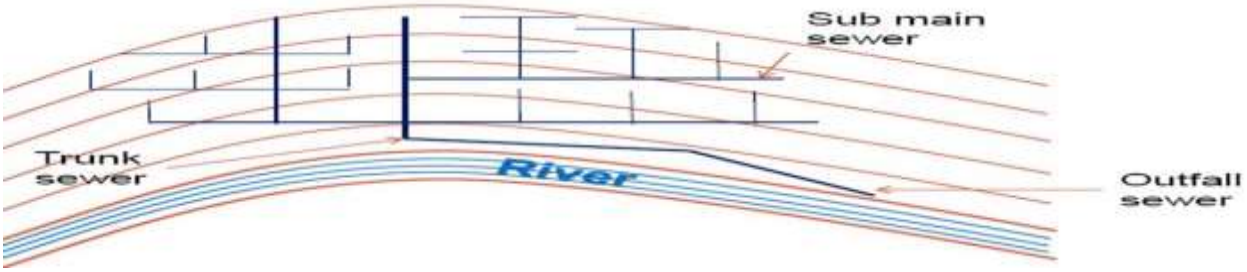


Figure 3.4 Fan Pattern Sewer layout

5. **Zone Pattern:** In the interceptor pattern only one single large size intercepting sewer is used to collect and convey the entire sewage, due to which it is over loaded. This over-loading can be removed by providing more number of interceptors for each zone. This pattern is used when there are different zones with different elevations exist. Here several intercepting sewers are laid according to elevations of the zone. Storm overflows are directly discharged to the river but sewage is brought to a common point for treatment. This type of system is useful in combined flow.

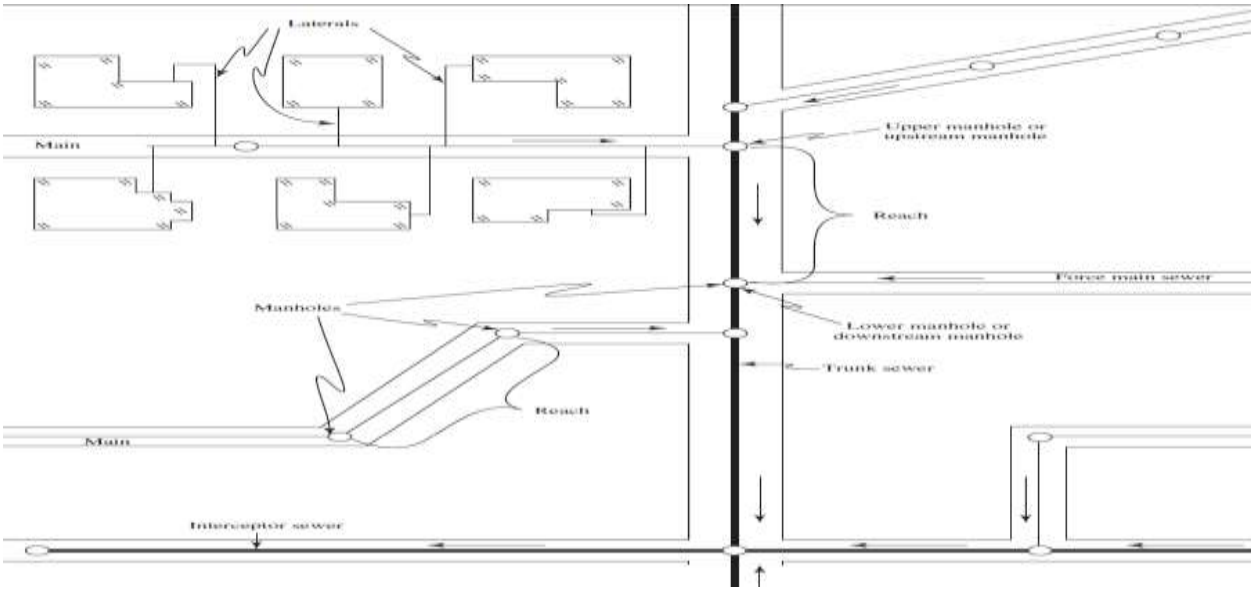


Figure 3.5 Zone Pattern Sewer layout

3.2 Hydraulic Design of Sewers

The design of a sewer system involves preliminary investigations, a detailed survey, the actual design, preparation of final drawings, and correction of plans to conform to changes made during construction. Preliminary investigation is required to arrive at an estimate of cost. A tentative layout is made by drawing lines along the streets or utility easements. Generally, Design of sewer system includes the following steps :

1. Preliminary investigations

- The anticipated population, its density, and its waste production must be estimated for the planning period which is normally 20 years or more.
- Estimate cost from official maps e.g. streets.
- Present & future water consumption.
- Prospective disposal sites are selected & evaluated.
- Cost estimates are made for the alternatives.
- Cost should be based, when possible, upon local bid tabulations.

2. Detailed survey

- Location of all existing underground structures.
- Location & basement elevations of all buildings.
- Profiles of all streets.
- Elevation of all streams, culverts & ditches & max. water elevations therein.
- Permanent benchmarks should be established for use in construction.
- The elevations of any existing structure, e.g. sewers, lift stations, outfalls, etc. which the new construction must meet are noted on the map.

3. Actual design

- A tentative layout is made by drawing lines along the streets.
- Sewers will follow natural surface drainage & street layout.
- In flat area a central location may be selected to which all lines will drain for pumping to gravity main or to treatment plant.

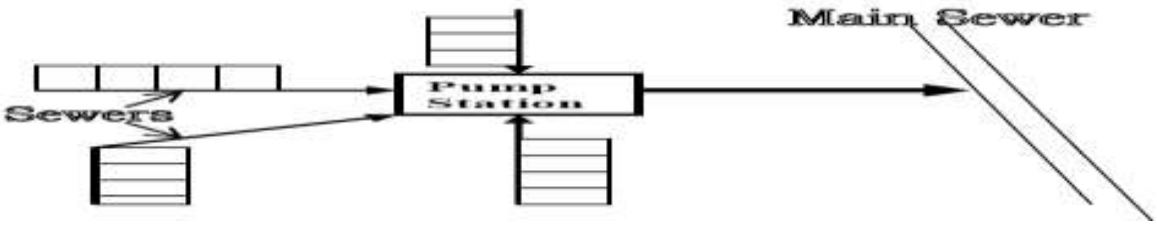


Figure 3.6 Simple layout of sewer System

4. Preparation of final drawings

5. Correction of plans to conform to changes made during construction (as built drawing)

3.2.1 Design periods for different components of a sewerage scheme

The design period should neither be too long nor it should be too short, and moreover, it should not exceed the useful life of the component structures. The design period is generally guided by the following considerations.

- Useful life of component structures, and the chances of their becoming old and obsolete.
- Ease of expansion.
- Amount of additional investment likely to be incurred.
- Anticipated rate of growth of population including shifts in communities, industries and commercial investments.
- Hydraulic constraints of the systems design.

The following design periods are often used in designing the different components of a

sewerage scheme.

Table 3.1 Design periods for different components of sewerage

Type of structure	Characteristics	Design period
Lateral sewers less than 30cm diameter	Easy to refurbish/replace	Full development
Mains and trunk sewers	Difficult and expensive to enlarge/replace	40-50
Treatment units	Difficult and expensive to enlarge/replace	15-20
Pumping plant	The additional pumps can be installed in future, very easily	5-10

3.2.2 Provision of Freeboard in Sewers and storm water Drains

Sanitary sewers are designed large enough to carry maximum sewage discharge while flowing half or two-third or three-fourth full. Generally, the sewer pipes of sizes less than 0.4m diameter are designed as running half-full at maximum discharge and the sewer pipes greater than 0.4 m in diameter are designed as running 2/3 or 3/4 th full at maximum discharge. The extra space, thus left, above the designed full supply line, will leave an ample margin, so as to act as a factor of safety to counter-act against the factors, such as:

1. Low estimates of average and maximum flow made due to some wrong data obtained regarding the water consumption by industries, or public, or about the quantity to be supplied from the water-works at the end of design period.
2. Large-scale infiltration of storm water due to wrong or illegal connections, and that of underground water through cracks or open joints in sewers.
3. Unpredicted increase in population or water consumption and the consequent increase in sewage production.

The storm water drains, on the other hand, are generally not provided with so much margins above their full supply lines (FSLs), because the over-flowing of sewers, mainly because sewage is highly polluted as compared to the storm water. The storm water drains are, therefore, provided with nominal provisions of freeboard above their designed FSLs, as given in the table below.

Table 3.2 Values of Freeboard to be adopted for the design of storm water Drains

Peak discharge in the drain which designed in cumecs	Freeboard to be left in meters
Below 0.3	0.3
0.3-1.0	0.4
1-5	0.5
5-10	0.6
10-30	0.75
30-150	0.90
More than 150	1.0

3.2.3 Hydraulic Formulas for determining Flow Velocities in Sewers and Drains

Sewers and drains are generally designed as open channels except when it is required to design them as flowing under pressure, as in the case of inverted siphons and discharge lines from sewage pumping stations, which always flow under pressure. Various empirical formulas have been suggested for determining the gradients necessary to obtain design velocities of flow in sewers.

1. **Chézy's (Kutter's) Equation**

   The Chézy equation in conjunction with Kutter"s equation is used in sanitary sewer design and analysis. The roughness component, C is a function of the hydraulic radius, friction slope and lining material of the channel.

   The Chézy equation is: $V = C\sqrt{RS}$

   Where: V = mean velocity (m/s, ft/s)

   C = roughness coefficient

   R = hydraulic radius (m, ft)

   S = friction slope (m/m, ft/ft)

The roughness coefficient, C is related to Kutter"s n through Kutter"s equation. Note that the n-values used in Kutter"s equation are actually the same as Manning"s n coefficients. The Chezy's constant, c depends upon:

- The size and shape of the channel,
- roughness of the channel surface and,
- hydraulic characteristics of the channel

According to Kutter's formula, the value of c to be used is given by:

$$C = \frac{23 + \frac{0.00155}{S} + \frac{1}{n}}{1 + \frac{\left(23 + \frac{0.00155}{S}\right)n}{\sqrt{R}}}$$

Where: C = roughness coefficient

n = Manning"s roughness value

R = hydraulic radius (m, ft)

S = friction slope (m/m, ft/ft)

This formula may be simplified by omitting the term 0.00155/(s) from the numerator as well as the denominator, since this will affect the results much less than an increase of 0.001 in the value of n.

Table 3. 3 Manning or Kutter's Rugosity Coefficients (n)

No	Pipe Material	Values of n at full depth for	
		Good interior surface Condition	**Fair interior surface condition**
1.	Salt glazed stoneware pipes	0.012	0.014
2.	Cement concrete pipes	0.013	0.015
3.	Cast iron pipes	0.012	0.013
4.	Brick, unglazed sewers/drains	0.013	0.015
5.	Asbestos cement	0.011	0.012
6.	Plastic (smooth) pipes	0.011	0.011

Area, Wetted Perimeter, and Hydraulic Radius:

- *Area* refers to the cross-sectional area of flow within the channel. When a channel has a consistent cross-sectional shape, slope, and roughness, it is called a prismatic channel. If the flow in a conveyance section is open to the atmosphere, such as in a culvert flowing partially full or in a river, it is said to be open-channel flow or free-surface flow. If a channel is flowing completely full, as with a water distribution pipe, it is said to be operating under full-flow conditions.
- A section"s *wetted perimeter* is the portion of the channel in contact with the flowing fluid.
- *Pressure flow* is a special type of full flow in which forces on the fluid cause it to push against the top of the channel as well as the bottom and sides.

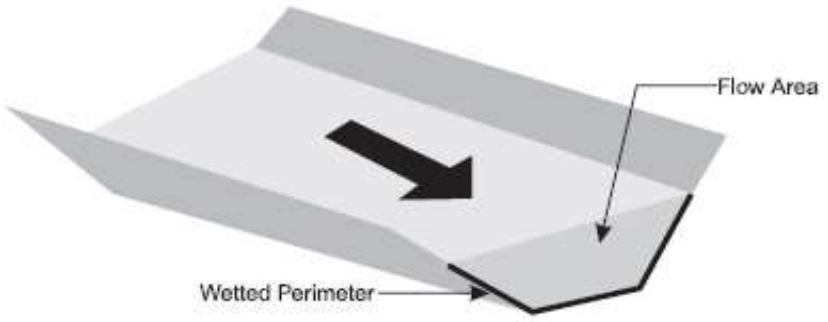


Figure 3.7 Flow Area and Wetted Perimeter

- *The hydraulic radius* of a section is not directly measurable characteristic but it is calculated. It is the area divided by the wetted perimeter and therefore has units of length. The hydraulic radius can be related directly to the geometric properties of the channel. The hydraulic radius of a full circular pipe (such as a pressure pipe) can be directly computed as:

$$R = \frac{A}{P_w} \quad \text{or} \quad R_{circular} = \frac{\pi \cdot D^2/4}{\pi \cdot D} = \frac{D}{4}$$

Where: R = hydraulic radius (m, ft)

A = cross-sectional area (m^2, ft^2)

P_w = wetted perimeter (m, ft)

D = pipe diameter (m, ft)

2. Manning's Equation

Manning's Equation for velocity and flows applicable for both pipe (closed-conduit) flow and open channel flow. Manning equation has found wide application in sewer design because it applies both to sewers flowing full and those flowing partially full. These n-values have been experimentally determined for various materials.

Manning"s equation is: $V = \frac{1}{n} R^{2/3} S^{1/2}$

Where: V = mean velocity (m/s, ft/s)

n = Manning"s roughness value

R = hydraulic radius (m, ft)

S = slope of the energy gradient (m/m, ft/ft)

Diameter and Hydraulic radius of circular pipe flowing full using Manning"s Equation for Velocity can be related to geometrical properties:

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

$$A = \frac{\pi}{4} D^2$$

$$P = \pi D$$

$$R = \frac{A}{P} = \frac{\frac{\pi}{4} D^2}{\pi D} = \frac{D}{4}$$

$$D = 1.548 \left[\frac{nQ}{S^{0.5}} \right]^{0.375}$$

Where: D = pipe diameter (m, ft)

R = hydraulic radius (area/wetted perimeter (m))

S = slope of pipe (m/m) in %

Manning"s Equation for Fluid Flow in SI Units: $$Q = \frac{AR^{2/3} S^{1/2}}{n}$$

Where: Q = Design flow (l/sec)

n = Manning's roughness coefficient

A= cross sectional area of flow (m^2)

For pipes flowing full or half-full, other useful forms of Manning"s equation are:

$$v = \frac{0.397}{n} (D)^{2/3} S^{1/2}$$

$$Q = \frac{0.312}{n} (D)^{8/3} S^{1/2}$$

$$S = \frac{10.3(n^2)(Q^2)}{(D^{16/3})}$$

Example1: Determine the velocity of 0.0081 m^3/s flow in a 200 mm diameter sewer at a slope of 0.0040 m/m and *n= 0.013.*

a. Estimate the velocity and flow rate for the sewer flowing full.
b. Calculate *Q/Q full*
c. The velocity flowing partially full

Solution:

a. $$v_{full} = \frac{0.397}{0.013} (0.200\ m)^{2/3} (0.0040\ m/m)^{1/2}$$
$$= (30.53)(0.3420)(0.0632) = \underline{0.66\ m/s}$$

b. $$Q_{full} = \frac{0.312}{0.013} (0.200\ m)^{8/3} (0.0040\ m/m)^{1/2}$$
$$= (24.00)(0.0137)(0.0632) = \underline{0.0208\ m^3/s}$$

$$\frac{Q}{Q_{full}} = \frac{0.0081\ m^3/s}{0.0208\ m^3/s} = \underline{0.39}$$

c. $v = 0.78\, v_{full} = (0.78)(0.66\,m/s) = \underline{0.51\,m/s}$

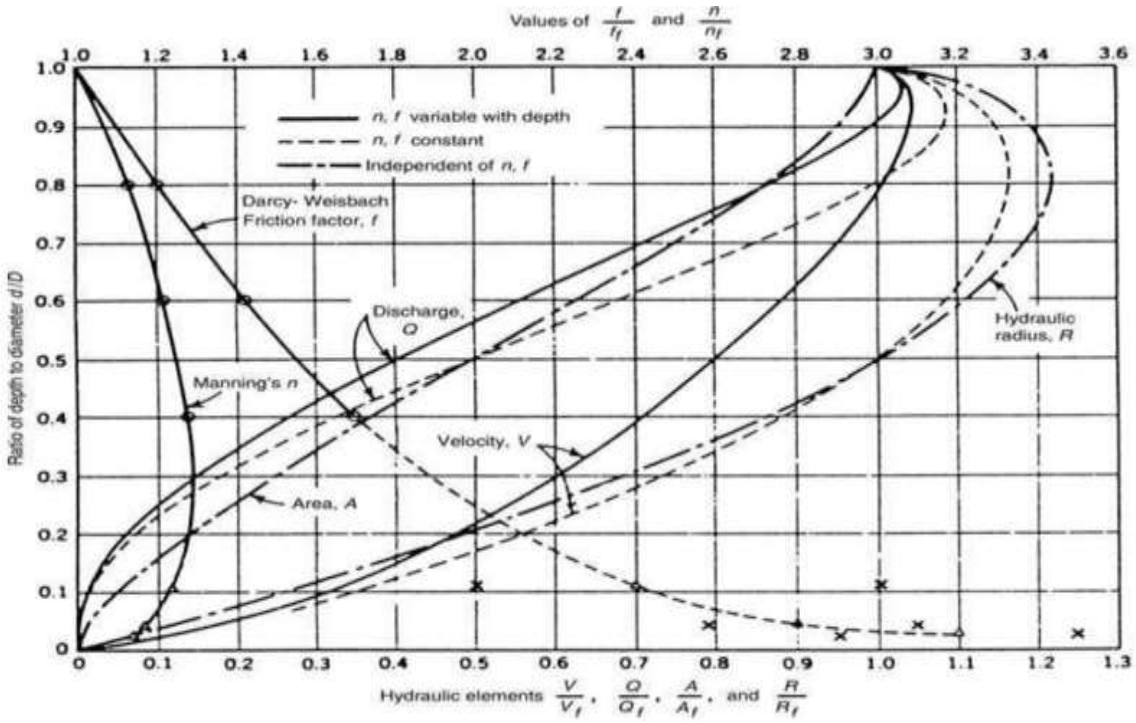


Figure 3.8 Hydraulic properties of circular sewers

3. Hazen-Williams Formula

The Hazen-Williams formula is developed for use with the flow of water and has been accepted for pipe flow problems. The formula is used exclusively for flows under pressure for designing water supply pipes and is seldom used for gravity flow.

Hazen-Williams equation:

$$V = 0.849 C R^{0.63} S^{0.54} \quad \text{and} \quad Q = 0.849 C A R^{0.63} S^{0.54}$$

Where: V= velocity of flow,

R = hydraulic radius, m

S = slope of the energy gradient

C = a roughness coefficient

The values of coefficient Hazen-Williams may be taken as shown in the table.

Table 3.4 Values of C for Hazen's-William formula

No	Type of pipe material	Value of C for	
		New pipes	Design purposes
1	Concrete and R.C.C. pipes	140	110

2	Cast iron pipes	130	100
3	Galvanized iron pipes	120	100
4	Steel pipes with welded joints	140	100
5	Steel pipes with riveted Joints	110	95
6	Steel pipes with welded joints lined with cement for bituminous enamel	140	110
7	Asbestos cement pipes	150	120
8	Plastic pipes	150	120

3.2.4 Effects of Flow Variation on Velocity in a Sewer

The sewage discharge flowing through a sewer does not remain constant all the time, but varies considerably from time to time. Due to this variation in discharge, the depth of flow varies hence the hydraulic mean depth (HMD) that is hydraulic radius(R) varies. Due to the change in hydraulic mean depth, the flow velocity gets affected from time to time. Since the velocity developed in sewer of a given section and grade will be lower as the flow reduces and sewer becomes less than half full. Therefore, it is necessary to check the sewer for maintaining a minimum velocity of about 0.45 m/s at the time of minimum flow (assumed at about 1/3 rd of average flow).

Moreover, care should be taken to see that the velocity generated does not exceed the scouring value during maximum flow. However, in flat countries sewers are designed to develop self-cleansing velocities of about 0.9 m/sec at maximum discharge and the condition of developing such a velocity at average flow is waived off. This permits flatter gradients, avoiding deep excavations.

Table 3.5 Non-scouring Limiting Velocities in Sewers and Drains

No	Sewer Material	Limiting velocity in m/sec
1	Vitrified tiles and glazed bricks	4.5-5.5
2	Cast iron sewers	3.5-4.5
3	Stone ware sewers	3.0-4.0
4	Cement concrete sewers	2.5-3.0
5	Ordinary brick-lined sewers	1.5-2.5
6	Earthen channels	0.6-1.2

On the other hand, if the available ground slopes are neither too steep nor too flat, the condition of developing velocities of about 0.9 m/sec. At average flow may be practically possible and economical, and hence may be insisted upon. In hilly areas permitting too high slopes, the sewers may be designed to develop such velocities

of 0.9 m/sec even at minimum discharge, and they may be checked for limiting non-scouring velocities at maximum discharge.

Diameter of sewer mm	Gradients 1 in….for developing self-cleansing velocity of		
	0.75 m/sec	0.90 m/sec	1.05 m/sec
10	90	75	60
150	150	105	78
225	265	180	135
300	385	270	195
375	520	355	265
450	660	465	340
525	820	570	415
600	970	680	500
675	1,100	790	580
750	1,300	910	670
900	1,700	1,200	850
1,050	2,100	1,450	1,050
1,200	2,500	1,700	1,250

3.2.5 Hydraulic Characteristics of Circular Sewer Sections running full or partially full

The circular sewer is most common and widely adopted for sewer pipes and drains when the flow is between full & half full. Sewers may, however, sometimes be of „egg shape" or „horse shoe shape' or „u- shape'. The circular sewers may sometimes run full or may run partially full.

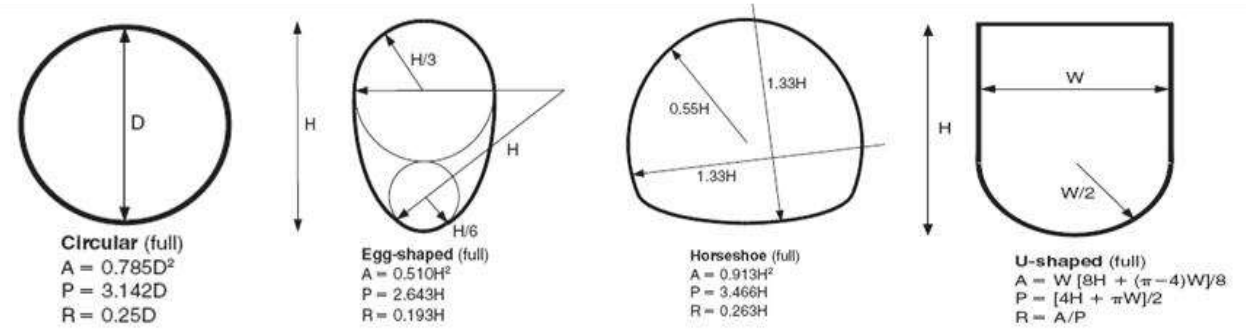
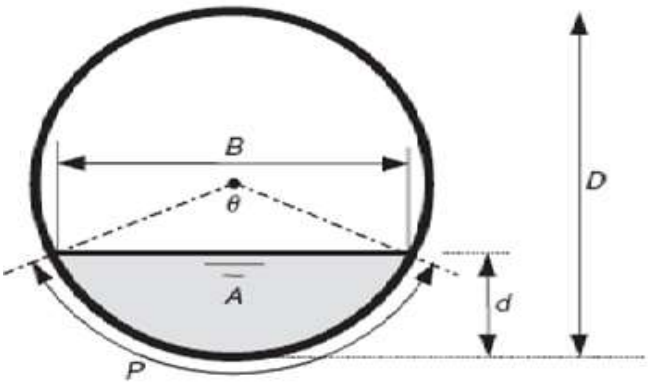


Figure 3.9 shape of sewer

For circular sewers running partially full the following relations are applied: The following figure shows the cross-section of a pipe of diameter D with flow of depth d. The angle subtended at the pipe centre by the free surface is Θ. From geometrical considerations, Θ is related to proportional depth of flow d/D as follows:

$$\theta = 2\cos^{-1}\left[1 - \frac{2d}{D}\right]$$



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

$$\frac{Ap}{Af} = \left(\frac{\theta}{360} - \frac{Sin\ \theta}{2\pi}\right)$$

$$\frac{Rp}{Rf} = \left(1 - \frac{360\ Sin\ \theta}{2\pi\theta}\right)$$

$$\frac{V_p}{V_f} = \left(\frac{R_p}{R_f}\right)^{\frac{2}{3}}$$

$$\frac{Q_p}{Q_f} = \left(\frac{A_p V_p}{A_f V_f}\right)$$

Where: d= Partial flow depth

R= Hydraulic radius (p= partial, f= full)

Θ= flow angle in degree

Maximum capacity of the pipe when d/D= 0.95

A= Flow area

Maximum velocity in the pipe occurs at d/D = 0.81

3. Calculate Q_f:

$$Q_f = \frac{Q_P}{0.789} = \frac{0.186}{0.789} = 0.2355\ m^3/s$$

4. Find the diameter of the pipe (D):

$$Q_f = \frac{0.312}{n} D^{\frac{8}{3}} S^{\frac{1}{2}} = \frac{0.312}{0.013} D^{\frac{8}{3}} (0.004)^{\frac{1}{2}} = 0.2355$$

$D = (0.15517)^{3/8} = 0.497$? 0.50m ? 20" (design pipe diameter)

5. Find the partial flow velocity (V_p):

$$V_f = \frac{1}{n} R_f^{\frac{2}{3}} S^{\frac{1}{2}} = V_f = \frac{1}{0.013}\left(\frac{0.497}{4}\right)^{\frac{2}{3}} (0.004)^{\frac{1}{2}} = 1.203\ m/s$$

$$A_f = \frac{Q_f}{V_f} = \frac{0.2355}{1.203} = 0.196\ m^2 \text{ (or } A_f = \frac{\pi D^2}{4}\text{)}$$

$$A_p = A_f * 0.712 = 0.196 * 0.712 = 0.139\ m^2$$

$$V_p = V_f * 1.109 = 1.203 * 1.109 = 1.33\ m/s$$

Example 2:

Find the diameter of the pipe required to carry a design flow of 0.186 m3/s when flowing partially, d/D = 0.67, slope = 0.4%, n = 0.013 use the relations of partial flow.

Solution

1. Find the flow angle θ

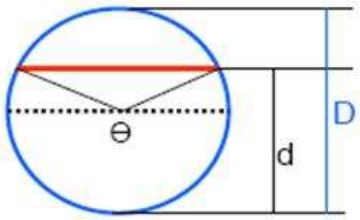
$$\frac{d}{D} = \frac{1}{2}\left(1 - \cos\frac{\theta}{2}\right) = 0.67$$

From this relation $\theta = 219.75^{\circ}$

2. Find Q_p/Q_f :

$$\frac{Rp}{Rf} = \left(1 - \frac{360\ Sin\ \theta}{2\pi\theta}\right) = \left(1 - \frac{360\ Sin\ 219.75}{2\pi(219.75)}\right) = 1.167$$

$$\frac{Ap}{Af} = \left(\frac{\theta}{360} - \frac{Sin\ \theta}{2\pi}\right) = \left(\frac{219.75}{360} - \frac{Sin\ 219.75}{2\pi}\right) = 0.712$$



$$\frac{V_p}{V_f} = \left(\frac{R_p}{R_f}\right)^{\frac{2}{3}} = (1.167)^{2/3} = 1.1088$$

$$\frac{Q_p}{Q_f} = \left(\frac{A_p V_p}{A_f V_f}\right) = 0.712 * 1.1088 = 0.789$$

3. Calculate Q_f:

$$Q_f = \frac{Q_P}{0.789} = \frac{0.186}{0.789} = 0.2355\ m^3/s$$

4. Find the diameter of the pipe (D):

$$Q_f = \frac{0.312}{n} D^{\frac{8}{3}} S^{\frac{1}{2}} = \frac{0.312}{0.013} D^{\frac{8}{3}} (0.004)^{\frac{1}{2}} = 0.2355$$

$D = (0.15517)^{3/8} = 0.497$? 0.50m ? 20" (design pipe diameter)

5. Find the partial flow velocity (V_p):

$$V_f = \frac{1}{n} R_f^{\frac{2}{3}} S^{\frac{1}{2}} = V_f = \frac{1}{0.013}\left(\frac{0.497}{4}\right)^{\frac{2}{3}} (0.004)^{\frac{1}{2}} = 1.203\ m/s$$

$$A_f = \frac{Q_f}{V_f} = \frac{0.2355}{1.203} = 0.196\ m^2 \left(or\ A_f = \frac{\pi D^2}{4}\right)$$

$A_p = A_f * 0.712 = 0.196 * 0.712 = 0.139\ m^2$

$V_p = V_f * 1.109 = 1.203 * 1.109 = 1.33\ m/s$

Example 3:

A trunk sewer is to be sized for a 25 km^2 city. It will be 60% residential, 30% commercial, and 10% industrial. The residential area will have 40% large lots, 55% small single family lots, and 5% multi-story apartments. The average domestic wastewater flow rate is 800 L/d/capita ($9.26x10^{-6}m^3$/sec/person), the average commercial flow rate is 25,000L/D/ha ($2.89x10^{-4}m^3$/sec/ha), and the average industrial flowrate is 40,000 L/d/ha ($4.63x10^{-4}m^3$/sec/ha). I&I is 1,000 L/d/ha for the entire area. Estimate the peak and minimum flows to be handled by the trunk.

The saturation densities for the residential areas are given in the table below

Type of Area	Density (persons/ha)
Large lots	5-7
Small lots, single family	75
Small lots, two family	125
Multistory apartments	2500

Solution:

i. The residential area will be 60% of 2,500 ha = 1,500 ha. The flowrates for each residential area will be:

Type	Area (ha)	Density (persons/ha)	Population	Flow (m^3/s)
Large lots	0.40(1500) = 600	6	3,600	0.03
Small single-family lots	0.55(1500) = 825	75	61,875	0.57
Multistory apartments	0.05(1500) = 75	2,500	187,500	1.74
Total			252,975	2.34

ii. The commercial area will be 30% of 2,500 ha = 750 ha, with a flow rate of $2.89x10^{-4}$ m^3/sec /ha, the average flow for commercial areas will therefore be **0.22 m^3/sec.**

iii. The industrial area covers 10% of 2,500 ha = 250 ha, with a flow rate of $4.63x10^{-4}m^3$/sec/ha, the average flow for industrial areas will therefore be **0.12 m^3/sec.**

iv. The I&I for the entire area is: (1,000 L/ha)(2500ha) =$2.5x10^6$L/day = **0.03 m^3/sec**

The total city flow, excluding I&I will be: 2.34+ 0.22+0.12= **2.68m^3/sec.** The total city population will be252, 975 (or 252.975 thousand people). The peak and minimum flow rate can be estimated:

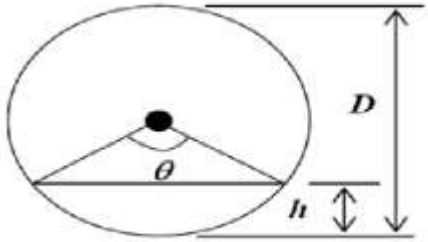
$$\frac{Q_{peak}}{Q_{ave}} = \frac{5.5}{(P/1000)^{0.18}} = \frac{5.5}{(252.975)^{0.18}} = 2.0$$

$$\frac{Q_{min}}{Q_{ave}} = 0.2(P/1000)^{0.16} = 0.2(252.975)^{0.16} = 0.48$$

The peak flow is therefore estimated to be: 2.0 (2.68 m^3/sec) + 0.03m^3/sec = **5.39 m^3/sec**

The minimum flow is estimated to be: 0.48 (2.68 m^3/sec) + 0.03 m^3/sec = **1.32 m^3/sec**

Here is another relation from simple geometric relationships, for pipes flowing partially full, this can be shown to be:



$$h = \frac{D}{2}\left[1-\cos\left(\frac{\theta}{2}\right)\right],\quad A = D^2\left[\frac{\theta-\sin\theta}{8}\right],\quad P = \frac{D\theta}{2}$$

$$Q = \frac{A}{n}\left[\frac{A}{P}\right]^{2/3} S_0^{1/2}$$

Combining these equations we get

$$\frac{(\theta-\sin\theta)^{5/3}}{\theta^{2/3}} - \frac{20.16nQ}{D^{8/3}S_0^{1/2}} = 0$$

After solving it by trials we can compute A and then, $V = \frac{Q}{A}$

$V < V_{min}$, reduce the diameter to achieve $V \geq V_{min}$

If diameter is the minimum (150 mm), put $V = V_{min}$ and find the slope of the pipe from the Manning's equation.

Example 4:

Determine the average velocity in a trunk sewer made of concrete (n=0.015) having 1500 mm diameter laid on a slope of 1%. The peak discharge through the pipe is estimated to be 4000L/s.

Given

$$n = 0.015,\quad Q = 4m^3/s,\quad S_0 = 0.01,\quad D = 1.5m$$

$$\frac{(\theta-\sin\theta)^{5/3}}{\theta^{2/3}} - \frac{20.16(0.015)(4)}{(1.5)^{8/3}(0.01)^{1/2}} = 0$$

By trials, *θ= 3.5 radian.* Therefore,

$$h = \frac{D}{2}\left[1-\cos\left(\frac{\theta}{2}\right)\right] = \frac{1.5}{2}\left[1-\cos\left(\frac{3.5}{2}\right)\right] = \underline{0.88m}$$

$$A = D^2\left[\frac{\theta-\sin\theta}{8}\right] = 1.5^2\left[\frac{3.5-\sin 3.5}{8}\right] = \underline{1.08m^2}$$

The average flow velocity in the sewer is given by:

$$V = \frac{Q}{A} = \frac{4}{1.08} = \underline{3.7m/s}$$

3.3 Storm water collection

In urban areas, storm water runoff generally occurs as sheet flow to the streets, conveyed as gutter flow along the streets to drain inlets; collected by storm sewers in buried conduits; and discharged into streams, lakes, or ponds. The storm water collection system is a combination of streets and storm sewers. The lots are graded so that the runoff flows toward the street, and the street conveys the runoff to the storm sewer drain inlets. The storm sewers are generally designed for a 5, 10 or 25-year return period storm. Some street flooding can be expected during a major flood event. Because the storm water collection system consists of both the streets and storm sewers, the system is designed for at least a 100-year return period storm.

The streets must be able to carry flood flow that is in excess of the storm sewer capacity. The system of streets and storm sewers used to collect storm water runoff is often referred to as the minor urban drainage system, and the rational method is commonly used to estimate the peak discharge rates. The rational equation is expressed as:

$Q = CIA$

Where: Q is the peak discharge rate in (m^3/s) at the point of design in the collection system.

C is the coefficient of runoff representing the ratio of runoff rate to rainfall rate.

I is the average rainfall intensity in (m/s) during a time period equal to the time of concentration.

A is the drainage area in (m) contributing to the flow at the point of design.

The time of concentration *(t_c)* is the time required for water to flow from the most hydraulically remote point in the drainage area to the point of design. t_c can also be defined as the time for the runoff from the drainage area to reach equilibrium under a steady rainfall rate. Although the total storm duration may be considerably longer than $t_{c,\ the}$ rainfall intensity (I) is the maximum average rainfall intensity that occurs in the storm for a period of time equal to t_c. The rainfall intensity for the design of the storm water collection system can be given by the intensity-duration frequency (IDF) equation such as:

I = ———

Where: I is rainfall intensity in mm/hr, t_c is time of concentration in minutes, a, b, and c are constants for the location and storm frequency.

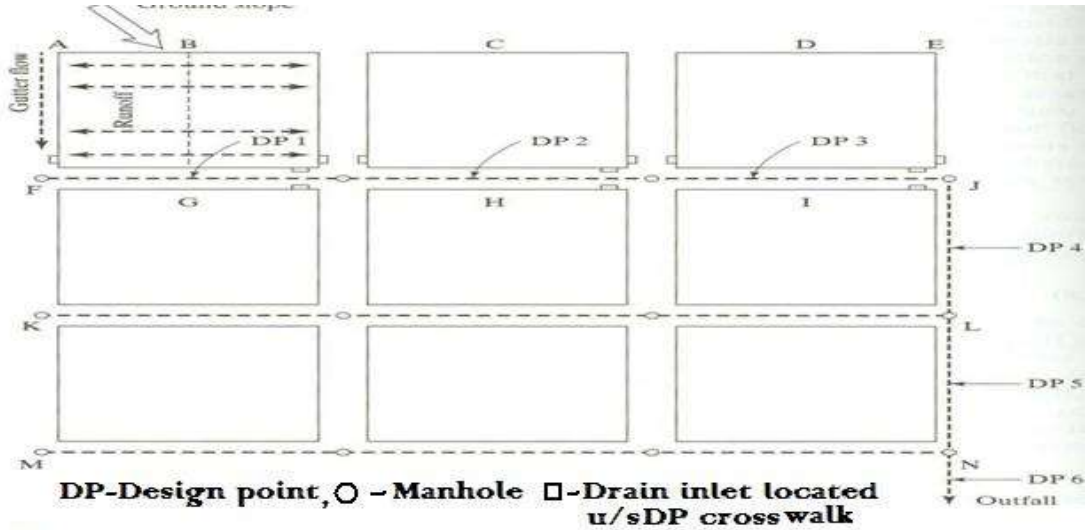


Figure 3.10 Typical layout of a storm sewer

The area contributing to the flow at design point 1 in Figure 3.10 is ABGF, whereas the area contributing to the flow at design point 5 is AELK. As the drainage area becomes larger, the time of concentration increases and the design rainfall intensity decreases. The time of concentration for the storm sewer system consists of overland flow time plus gutter flow time plus conduit flow time. The gutter flow time and the conduit flow time

can be estimated by dividing the channel length by the average water velocity. The overland flow can be estimated based on the kinematics wave equation:

CI= — — Using Manning's equation: q =

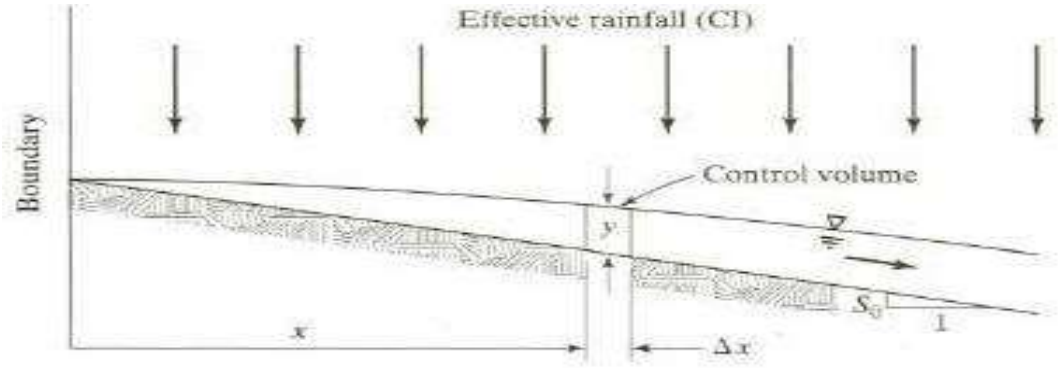


Figure 3. 11 Kinematics wave overland flow travel time

Where = ——— and m= – for turbulent flow

Where: S_o=slope of the ground , C_m=1.49 (1.0) , N=Manning's roughness coefficient for overland flow,

L=Length of overland flow (m), C=runoff coefficient and I=average rainfall intensity (mps)

Combining the kinematics wave and the Manning's equations gives: CI= — — (a)

The control volume in figure 13.11 is moving at a velocity of —. If *t* is the time for the control volume to travel from the boundary to a distance *x*, it follows that: —— = CI= — —— (b)

Comparing (a) and (b) gives: — (c)

Integrating (b) gives: = CI (d)

Combining (c) and (d) and integrating gives: (e)

The travel time (t_c) for $x = L$ is: = ——— (f)

Where: = time of concentration in seconds, Smaller values of runoff coefficients are to be used with storm recurrence intervals of less than 25 years, and the larger values are to be used for storm recurrence intervals of 25 years or more.

Example 4:

Determine the time of concentration and peak discharge rate for design point 1 in Figure 3.10. The storm water collection system is for a high-density residential area with hydrologic soil group C. The lots have been graded to drain toward the street at a slope of 0.01 and have a Manning "n" value for overland flow of 0.2. The street gutter (A-F) has a longitudinal slope of 0.015, a Manning "n" value of 0.017, and a transverse side slope of 30:1. The blocks in Figure 3.10 are 115.2 square meters, which includes a 15.2-m wide street. The storm sewers are to be designed for a 25-year return period storm. The design IDF (intensity-duration- frequency) equation for the system is I = ————— , where I is rainfall intensity in cm/hr.

Solution

- The residential area contributing to the flow is: A_1= 100 x 50 = **5000m^2** with a runoff coefficient of 0.60
- The area of the street contributing to the flow is: A_2 = .6 · 7.6 = **817.6 m^2** with a runoff coefficient of 0.95
- Equation (f) is used to compute the overland flow travel time. The time of concentration *(t_c)* is:

$t_C = t_{overland} + t_{gutter}$

I = ————— in meter per hour, hence t_C = —————

Using an assumed value of 1.5 minutes for gutter flow time, the above equation Becomes Q peak at design point 1 = CIA = (0.60 x 1.03 + 0.95 X 0.19)7.7 = **6.1 cfs** .Check gutter travel time based on an average discharge rate of **3.1 cfs**.

3.4 Design criteria

The design capacity of the combined street and storm sewer collection system must equal or exceed the runoff rate for the l00-year return period storm. During a major flood event, street flooding may occur, but residential, public, commercial, and industrial buildings should not flood. The capacity of the storm sewer collection system is generally selected to handle a 5, 10, or 25 year return period storm. Streets and open channels are required to carry the flood flows that are in excess of the storm sewer capacity.

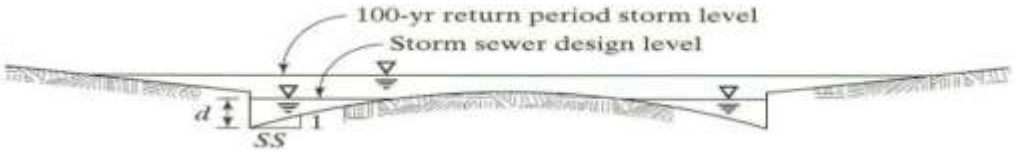


Figure 3.12 Water levels in street

The design of the storm water collection system is based on the principles of steady flow in open channels, and Manning's equation is typically used to compute the water velocity and depth in gutters and the velocity and pipe diameter in the storm sewers.

3.4.1 Street Gutters

The discharge rate in a triangular curb gutter based on Manning's equation is:

$$= \frac{}{}$$

Where: Q is the discharge rate in cums, C m is 1.49 for British Gravitational (BG) i units and 1.0 for the International System (SI) of units, n is Manning roughness co-efficient, SS is the street cross slope (ranges from 10 to 50), d is the normal depth in ft (m), and So is the longitudinal slope of the gutter. A Manning "n" value of 0.017 is recommended for gutter flow. Because of the shallow depth of flow, the recommended "n" value is greater than that normally used for smooth concrete or asphalt pavement. The curb height should be at least 0.5 ft (0.15 m). Overtopping of the curb is not allowed for the design storm. For the safety of children, the drain inlets should be spaced so that the maximum water depth in the gutter does not exceed 0.5 ft (0.15 m), and the water velocity does not exceed 10 fps (3.0 mps). A minimum longitudinal slope of 0.004 will generally ensure a low flow cleansing velocity in the gutter.

The top width of flow (T) is the spread of water on the street and is equal to, T = SS X d. For local streets, the water can spread to the crown of the street during the design storm. Collector streets are usually designed so that one traffic lane is free of water, and arterial streets are usually designed so that one traffic lane in each direction is free of water during the design storm. Because of the potential for hydroplaning, no encroachment of water is allowed on any traffic lane for high speed roads.

3.4.2 Drain Inlets

Inlets are used in the drainage system to intercept the surface runoff (primarily in the street gutter) and direct the flow into the storm sewer. They are often located at street intersections upstream of the crosswalks. Inlets can be curb, grate, combination, or slotted. At design capacity, the flow into the inlet is by weir flow. During a major flood event, ponding over the inlet may occur and flow into the inlet will be by orifice flow. Curb inlets are the most common as they provide the least interference with bicycle traffic and have the least tendency for blockage with debris. The height of the curb opening should not exceed 0.5 ft (0.15 m) because of the risk to small children.

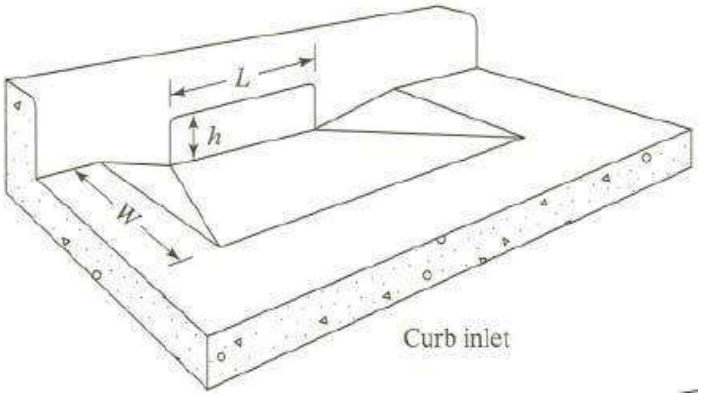


Figure 3.13 Curb inlet

To increase the capacity of the inlet, the invert of the inlet is depressed below the invert of the street gutter. Typically the inlet is depressed 0.25-0.50 ft (0.07-0.15 m) with W ranging from 1.0-2.0 ft (0.3-0.6 m). The weir flow equation gives the discharge into the curb inlet on a mild slope *(QI).*

$QI = Cw(L + 1.8W)d^{3/2}$, Where Cw is 2.3 for BG units and 1.25 for SI units, Land Ware defined in Fig. 10.5, and d is the normal depth of the water in the approach gutter. Because some flow may bypass the curb inlet located on a gutter with a steep slope, drainage manuals should be consulted for sizing drain inlets located on a gutter with a steep slope. Inlets in sumps are located at the low point in a roadway (such as the sag in a vertical curve) and are located to prevent ponding of water on the roadway. If clogging by debris is not anticipated, grate inlets work well when located in a sump.

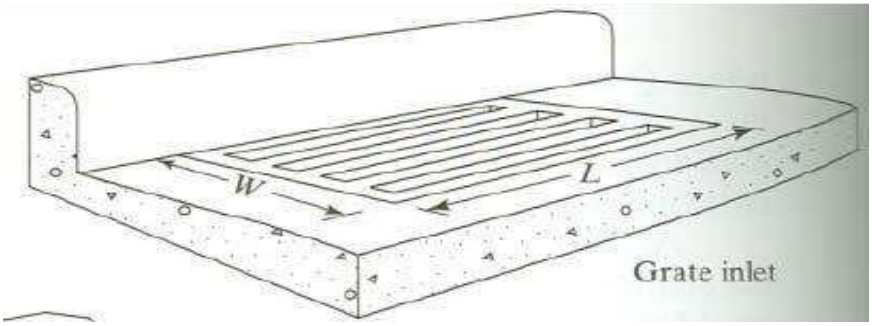


Figure 3.14 Grate inlet

A grate inlet will operate as a weir at shallow water depths *(d)* less than 0.5 ft (0.12 m) and the grate inlet capacity *(QI)* is: $QI = CwPd^{3/2}$, where Cw is 3.0 for BG units and 1.7 for SI units, *P* is the perimeter of the grate opening in ft (m), not including the side against the curb, and *d* is the depth of flow in ft (m).



Figure 3.15 Combination inlet

A slotted drain inlet will function as a weir at shallow depths $d < 0.2$ ft (0.06 m) and operate as an orifice when submerged with a depth> 0.4 ft (0.12 m).

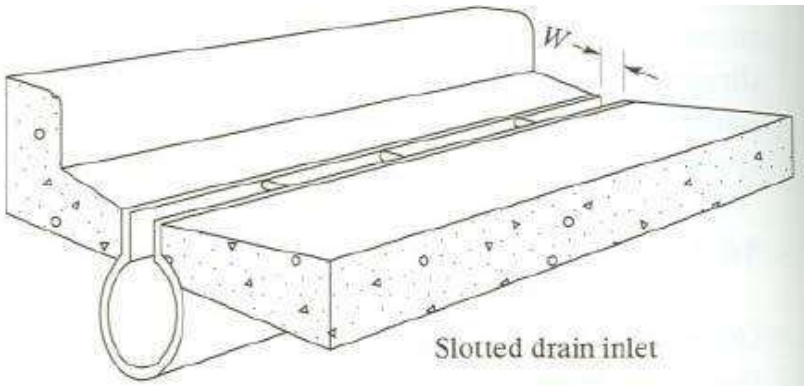


Figure 3.16 Slotted drain inlet

The capacity *(QI)* of a slotted drain inlet with a width *(W)* > 0.15 ft (0.04 m) is: $QI = CwLd^{3/2}$**.** *I* for weir flow and *QI = 0.6A V2id***,** for orifice flow Cw is equal to 2.3 for BG units and 1.25 for SI units, and *A* is the area of the open space in the grate.

3.4.3 Design of Storm Sewers

The design of storm sewers is based on flow in open channels under steady flow conditions. Some general guidelines are as follows:

1. The storm sewer pipe is assumed to flow full, and the pipe size is computed using the Manning equation.
2. The minimum pipe size considered for storm sewers ranges from 0.30 to 0.46 m in diameter.
3. To prevent sedimentation, the minimum water velocity for the pipe flowing full is (0.75 mps).
4. To avoid clogging with debris in the storm sewer, the pipe size should not decrease downstream.
5. Pipe grades are typically specified in terms of elevation of the invert at manholes.
6. At changes in pipe size, the crown of the two pipes is at the same elevation.
7. Pipe slopes should conform with the ground slope where possible, and there should be at least 3.0 ft (0.9 m) of cover over the pipe to prevent excessive wheel loading on the pipe.

Manholes are used for maintenance access and are located where two or more pipes join, where there is a change in pipe size or grade, and at sharp bends. The maximum spacing between manholes depends on the pipe size and may range from 150 m to over 300 m. Head losses in manholes are computed as minor losses and are included in the design by lowering the invert elevation of outlet pipe at the manhole. The typical form of the minor loss equation is:

= — , The minor loss includes junction loss, bend loss, and transition loss. For inflow and outflow pipes aligned with each other, K is between 0.1 and 0.3. As the amount 0: turbulence in the manhole increases, the value for K will also increase.

Chapter Four

4. Sewers and Appurtenances

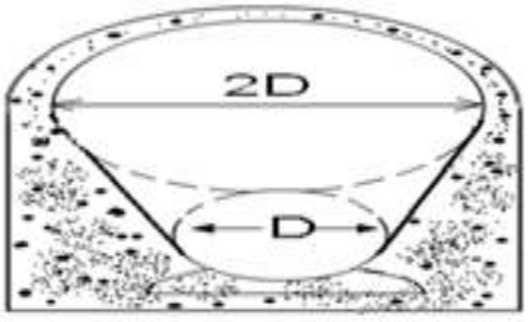
4.1 Sewer Shapes

Sewer systems require a variety of appurtenances to insure proper operation. These include manholes, inlets, inverted siphons, pumping stations, and regulators. The shape of the sewer depends upon hydraulic considerations, construction conditions, and available space. The lower surface is generally curved to concentrate low flows and maintain self-cleansing velocities. Semi elliptical, circular, inverted egg, horseshoe, or rectangular box with inclined lower surface shapes may be used according to the prevailing site conditions.

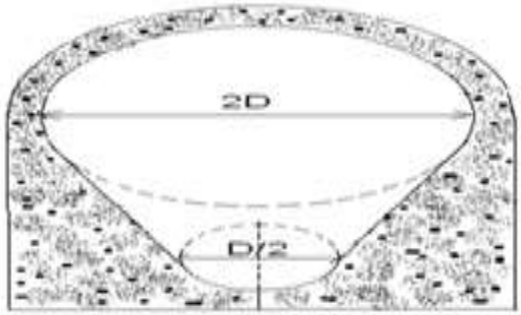
Sewers are generally circular pipes laid below ground level and slopping continuously towards the outfall. These are designed to flow under gravity. Circular sewers are easily manufactured. A circular sewer gives the maximum area for a given perimeter, thus the greatest HMD is ensured when running full or half full. It is therefore the most efficient section at these flow conditions. It is the most economical section since it utilizes minimum quantities of the material. Circular section has uniform curvature all around, that's why it offers less opportunity for deposits.

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. Its main advantage is that it gives higher velocity during low flow than a circular sewer of the same capacity.

Horse shoe shaped sewers, semi-circular and U-shaped sections are used for large sewers with heavy discharge such as trunk and outfall sewers. Rectangular section is used for conveying storm water. Other sections of sewers have become obsolete due to difficulty in their construction and non availability of these shapes in the market.



(a) Standard Egg Shaped Sewer



(b) New/ Modified Egg shaped Sewer

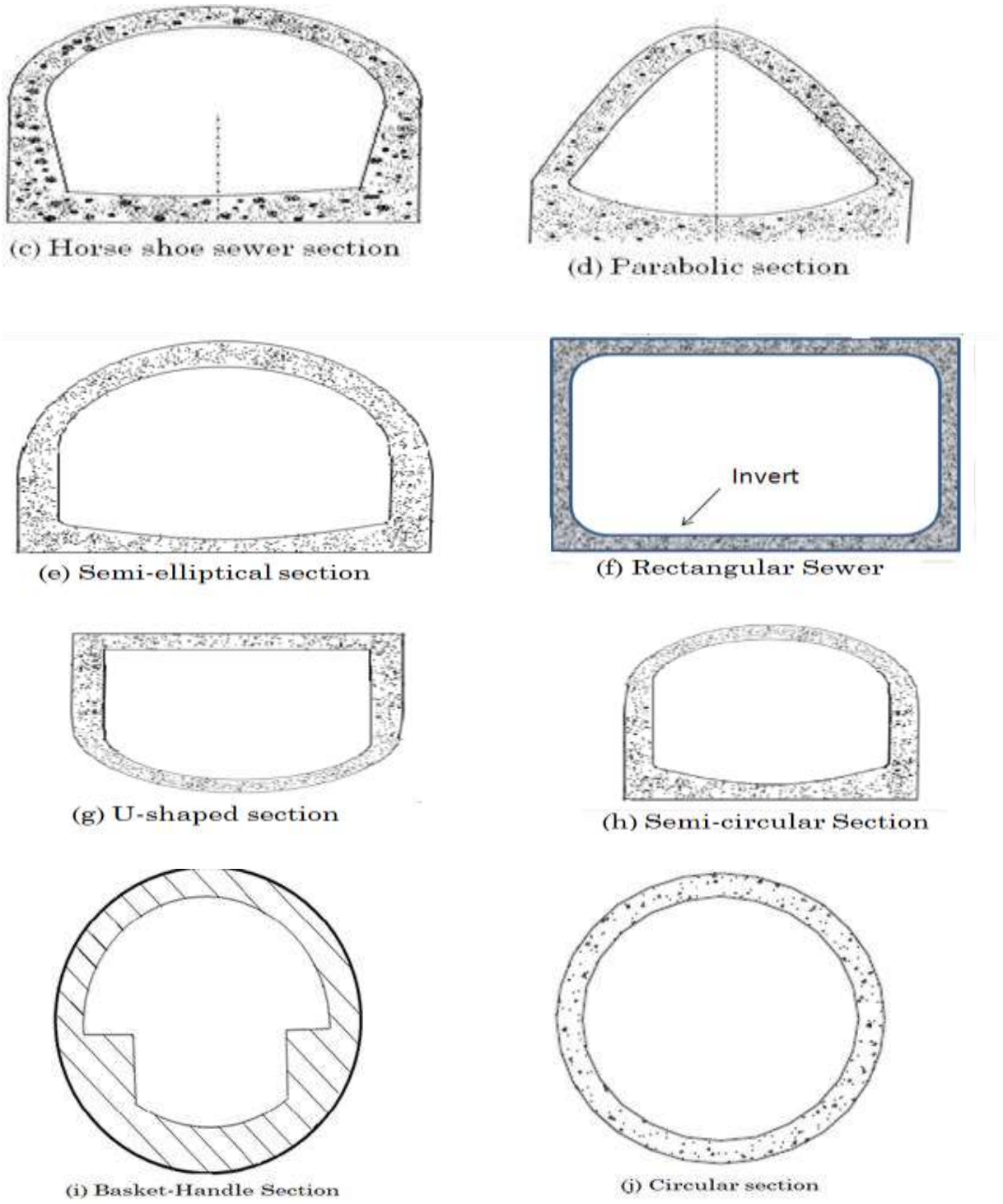


Figure 4.1 Different shape Sewers

4.2 Sewer materials

Different materials are available to be used for sewer construction. Sewer materials may be either pre-cast or

cast-in-situ. Pre-cast sewer materials include vitrified clay, plain concrete sewer, reinforced concrete sewer, asbestos cement pipe and plastic truss pipe. Factors influencing the selection of material for sewer construction are availability, weight of the material, resistance to corrosion, resistance to abrasion, strength and durability, Imperviousness, hydraulic efficiency, economy and cost. No single material will meet all the conditions that may be encountered in sewer design. Selection should be made for particular application and different materials may be selected for parts of a single project.

a. Clay sewer pipes

Clay pipe is made of clay or shale, which has been ground, wet, molded, dried, and burned in a kiln. Near the end of the burning process, sodium chloride is added to the kiln and vaporizes to form a hard waterproof glaze by reacting with the pipe surface. The burning itself produces a fusion, or verification, of clay, making it very hard and dense. Vitrified glazed clay pipe is not subject to mineral or bacterial corrosion. Clay pipe is manufactured with integral bell and spigot ends fitted with polymeric rings of various designs on the spigot end. Clay pipe is manufactured to standard specification of the ASTM in diameters from 100 to 1070mm.

b. Plain concrete sewer pipe

Pre-cast concrete pipe may be used for small storm drains and sanitary sewers in locations where grades, temperatures or sewage characteristics prevent corrosion. This pipe is not suitable where the temperature is high and flat slope is not available. Joints for plain concrete sewer should be constructed using rubber gaskets.

c. Reinforced concrete sewer pipe

Pre-cast concrete pipe in diameters larger than 610mm is reinforced. The pipe is manufactured in sizes from 305 to 4570mm. Joints are either ball and spigot or tongue and groove from 305 to 760mm and tongue and groove above that size. Joints are made by mortaring the cleaned and wetted tongue and groove before assembly or by mechanical or O-ring joints. Special fittings are seldom used with reinforced concrete pipe. In large sizes curves are made by deflecting the joints and then filling the resulting opening with poured concrete. Connections are made by cutting an opening and mortaring. If reinforced pipe is used to convey untreated sanitary sewage it may be subject to corrosion in the same fashion as plain concrete pipe. Large concrete sewers may be lined in place with corrosion-resistant material.

d. Asbestos cement Pipe

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 75mm to 500mm in diameter and 3 to 4 meters in length. These pipes are used for vertical transport of water namely: Transport of rainwater from roofs in multistoried buildings, transport of sewage to grounds, and transport of less foul sullage (i.e.,

wastewater from kitchen and bathroom). Asbestos cement is subject to the corrosion mechanism common to sanitary sewers and may be undesirable for that reason.

Advantages

- These pipes are light in weight and hence, easy to carry and transport.
- They can be easily cut and assemble without skilled labor.
- The Inside surface is relatively smooth hence, they are hydraulically efficient.

Disadvantages

- They are brittle.
- They are weak against impact forces. Hence careless handling may result in their breakage.
- They have less structural strength against heavy external forces.
- They are susceptible for sulphide corrosion when bacteria produce H_2S, in the presence moisture, H_2SO_4 can be formed.

e. Vitrified Clay or Stoneware Sewers

These pipes are manufactured from clays and shale's of special qualities and grades, using hot process. These pipes are used for house connections as well as lateral sewers. These pipes are rarely manufactured for diameter greater than 90mm. These are joined by bell and spigot flexible compression joints.

Advantages

- For many years the most widely used pipe for gravity sewers.
- They are cheap and easily available, hence widely used in small and medium sizes

Advantages

- The pipes are highly impervious.
- Possess high compressive strength
- Possess enough resistant to erosion due to grit and silt.
- They are highly resistant to hydrogen sulfide corrosion
- These pipes are durable and economical for small diameters.
- Heavy, bulky and brittle, hence their handling, laying and transportation is difficult .
- 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

f. Brick Sewers

This material is used for construction of large size combined sewer or particularly for storm water drains.

The pipes are plastered from outside to avoid entry of tree roots and ground water through brick joints. These are lined from inside with stoneware or ceramic block to make them smooth and hydraulically efficient. Lining also make the pipe resistant to corrosion.

g. Cast Iron Sewers

These pipes are stronger and capable to withstand greater tensile, compressive, as well as bending stresses. However, these are costly. They are available in diameter ranging from 150mm to 750mm and lengths up to 3 to 3.5m. Cast iron pipes are used for outfall sewers, rising mains of pumping stations and inverted siphons where pipes are running under pressure. They are suitable at places where heavy external loads on sewers exist, such as sewer line below railway lines, roads, foundation walls, high overburden of soil etc.

h. Cast iron pipe

They are preferred when sewage flows under pressure i.e. conveying sewage through pumping stations and treatment works (high internal pressure). They are used at low locations when sewers are to be carried over pipes while crossing low laying areas. Cast iron pipes are 100% leak proof, therefore the pipes are used where there is danger of contamination of ground water due to leakage of sewage through walls of sewers. Because of their imperviousness they are suitable for wet ground conditions with fear of infiltration. 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.

i. Steel pipes

These pipes are used at locations where high external and internal pressure is encountered. These are used for main sewers under water crossing, bridge crossing, railway crossings, necessary connections for pumping stations, outfall and trunk sewers having large diameters. They can withstand internal pressure, impact load and vibrations much better than CI pipes. They are more ductile and withstand water hammer pressure better (i.e. easily welded). They are light in weight. Because of their flexibility they can absorb vibrations and shock loads. They are susceptible to corrosion and are not generally used for partially flowing sewers. They are made corrosion resistant either by heavy galvanization or bituminous coatings.

j. Ductile Iron Pipes

Ductile iron has proven to be a better pipe material than cast iron but they are costly. Available sizes:4 - 54 in (100-1350mm). The predominant wall material is ductile iron. Internally these pipes are coated with cement mortar lining or polyethylene/plastic bagging to inhibit corrosion and various types of external coating are used to inhibit corrosion from the environment. Ductile iron is still believed to be stronger and more fracture resistant material like most ferrous materials. 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. Often used for river crossings where the pipe must support unusual high loads. Useful where unusually leak proof sewer is required. Generally should not be used where groundwater is brackish. It is susceptible to acid corrosion and hydrogen sulfide attack.

k. Corrugated metal pipe

Sometimes used for storm sewers. It may be galvanized, coated with bituminous material or coated with asbestos. Corrugated metal pipe sections are available in many thicknesses and cross sections. Their primary use is roadway drainage.

l. Plastic sewers (PVC pipes)

Plastic is recent material used for sewer pipes. These are used for internal drainage works in house. It is highly resistant to corrosion. These pipes are tough, economical in laying, jointing and maintenance. Light weight, handling, transportation and fabrication of these pipes are easy. Used as an alternative to asbestos cement and vitrified clay pipe.

m. Plastic sewers (PVC pipes)

High Density Polyethylene (HDPE) Pipe: Use of these pipes for sewers is recent development. They are not brittle like AC pipes and other pipes. They can be joined by welding or with detachable joints up to 630 mm diameter. These are commonly used for conveyance of industrial wastewater. They offer all the advantages offered by PVC pipes.

4.3 Sewer appurtenances

Sewer appurtenances are those structures which are constructed at suitable interval along a sewerage system to insure proper operation and maintenance. They include structures and devices such as manholes, cleanouts (lamp holes), traps, intercepting chambers, flush tanks, ventilation shafts, catch basins, street inlets, regulators, siphons, sewer crossings and pumping stations and out fall structures.

a. Manholes

A manhole is an opening constructed in the alignment of a sewer for facilitating means of access into a sewer system for the purpose of inspecting, cleaning and repairs (removal of obstruction of the sewer line). They are placed at intervals of 90 to 150m and at points where there is a change in direction, change in pipe size, and change in grade and elevation and at junctions of two or more sewers. For larger sewers, these changes may be made without installing a manhole or requiring fewer manholes.

Manhole Spacing

- Sewers = 375mm or less, Place manholes at intervals not greater than 120m(400 ft)
- Sewers = 450 to 750 mm or less, Place manholes at intervals not greater than 150m or at intervals of 180m with adequate cleaning equipment.
- Sewers = 825 to 1,200 mm, Place manholes at intervals of 180m

- Sewers = 1,200 mm or greater , Manholes may be placed at intervals of 460m depending on local conditions like breaks in grade, location of street intersections etc.

Manhole design has become largely standardized, and most of the larger cities have standard plans, which they have adopted for general use.

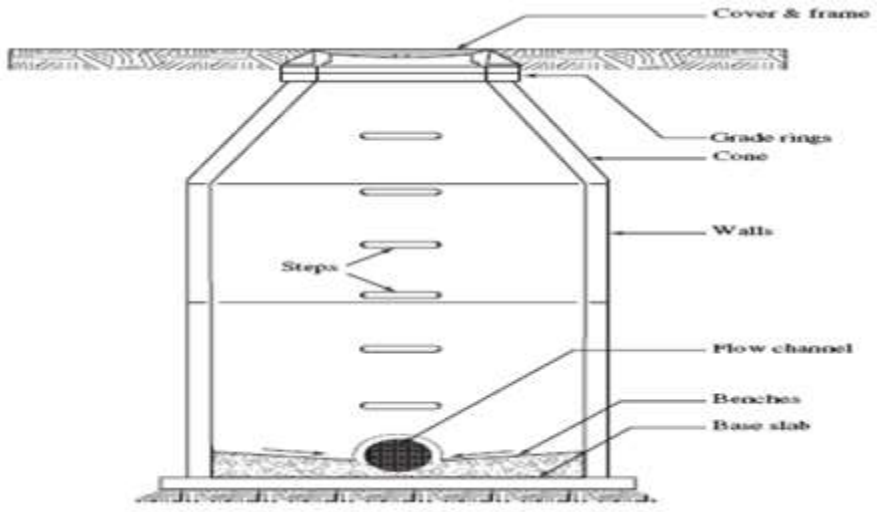


Figure 4.2 Manhole design

The manhole has a cast-iron frame and cover with a 500 to 600mm clear opening. Manholes are of rectangular, arch type and circular type. Circular manholes are stronger than rectangular and arch type manhole and hence circular manhole is preferred over other two types. The circular manholes can be provided for all depths, starting from 0.9 meters. Depending on the depth of manhole, diameter of manhole change.

DROP Manholes

Drop manholes are used when the inflow and outflow sewers differ in elevation by more then 0.6 m. They may also be used to reduce the slope when the velocities exceed erosive velocities (2.5–3.0 m/s). This provides an acceptable workspace for maintenance and repair, instead of allowing sewage to cascade down from a large height.

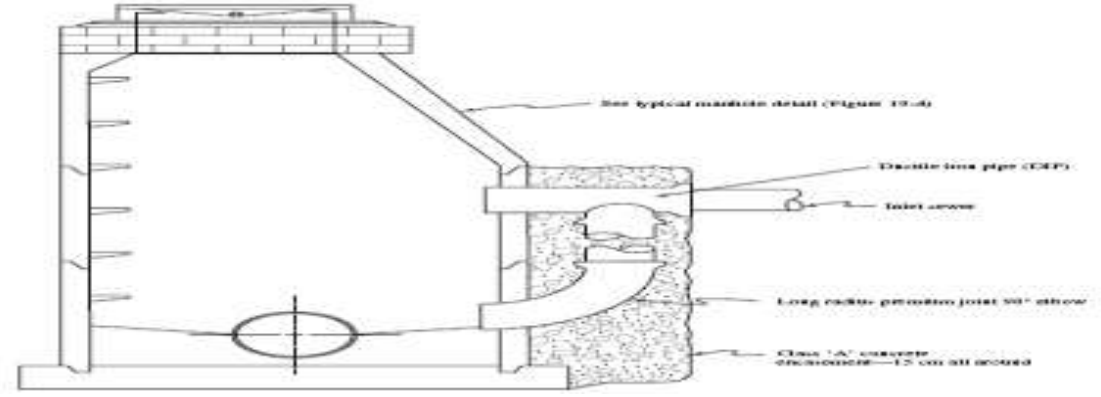


Figure 4.3 Drop manhole detail

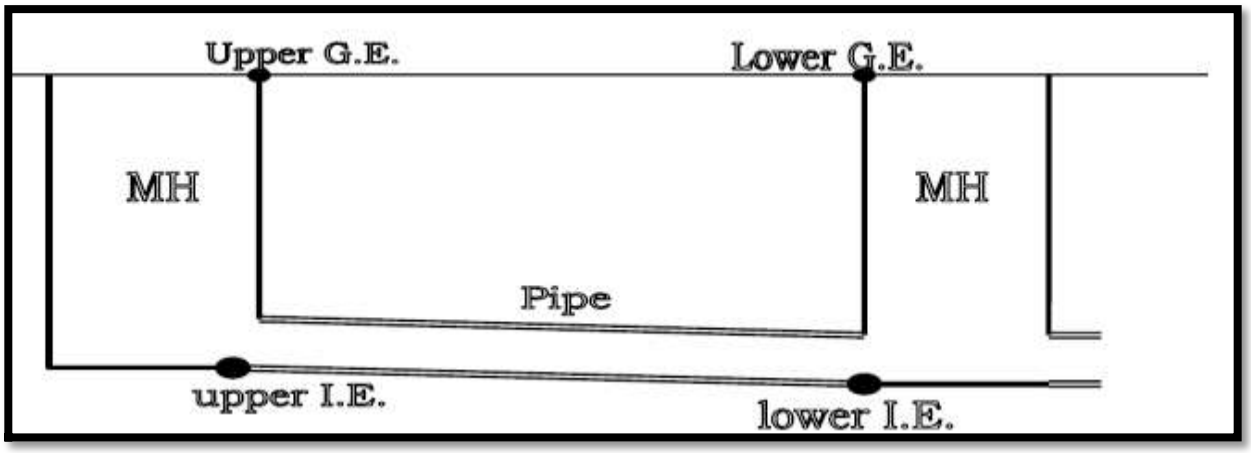


Figure 4.4 Pipes, manholes, & elevations

Manhole size

- *Manhole covers:* A minimum clear opening of 60 cm is recommended.
- *Floor slab of manhole:* RCC 150 mm thick to withstand uplift.
- The width/diameter of the manhole should not be less than the internal diameter of sewer plus 150mm benching on both sides (300mm).
- Manholes in small sewers are usually about 4 feet (1.2m) in diameter when the sewers have circular cross sections
- In large sewers, larger manholes may be required to accommodate larger cleaning devices
- Manhole should be large enough to provide easy access to the sewer
- There should be a room for a worker to handle a shovel
- Bottoms are usually concrete, sloping toward an open channel which is an extension of the lowest sewer.
- Bottom should provide footing for a person working in the manhole.

Street Inlets

An inlet is an opening into a storm or combined sewer for entrance of storm runoff. It is placed at intersections and at intervals of 20 to 100m but occasionally at midpoints of the blocks, if they are more than 150m long. Placing of inlets at the corners not only requires the pedestrians to step across flooded gutters but also subjects the inlets to considerable traffic wear and damage.

Catch basins

Catch basins are included in storm sewer system designs in order to remove solids such as gravel, sand and organic material carried by storm water. Catch basins also contain elevated concentrations of metals (attached to the solids) from street runoff or drainage from industrial, commercial and residential properties. To maintain

the storm sewer systems effectiveness, catch basins must be periodically cleaned out. The outlet pipe is usually trapped in order to prevent escape of odors from the sewer, a condition that causes retention of floating matter. The most economical method of cleaning catch basins is pumping the basin contents into trucks by means of a specially designed portable centrifugal pump.

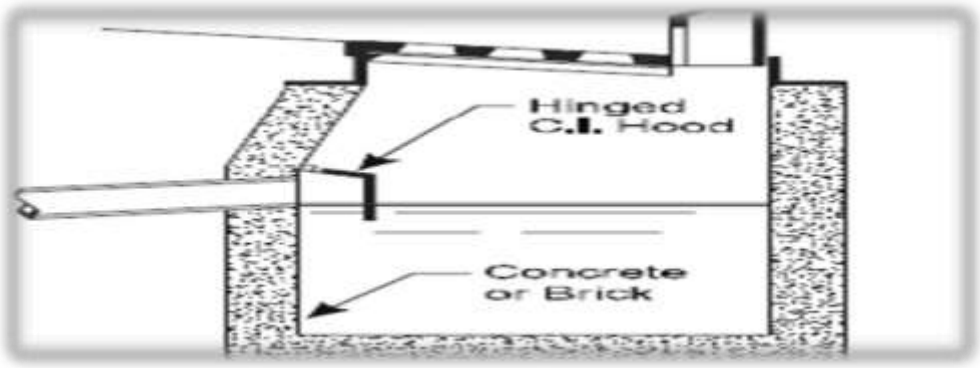


Figure 4.5 Catch basin

Sand, Grease, and Oil traps

The sewage from kitchens of hotels and restaurants contains grease, which tends to accumulate on sewer walls and cause clogging. Some cities, therefore, require grease traps in the waste lines from the sinks of such kitchens. Sewage from garages particularly from the floor drains and wash racks contains oil, mud and sand. Such a trap would also prevent gasoline from entering the sewers and causing an explosion hazard. It is important; however, that the traps should be regularly cleaned otherwise they are of no value. Regular inspections are necessary to insure this.

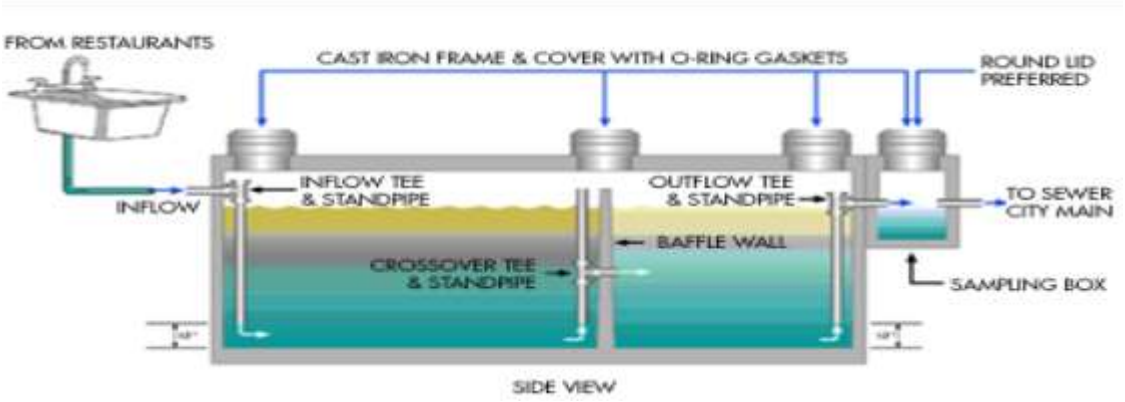


Figure 4.6 Sand, Grease, and Oil traps

Inverted siphons

Inverted siphons are constructed to convey sewage flows across obstructions (like depressed highways, under a canal, under a river, or across a valley floor) where such crossings cannot be attained by a sewer placed on a continuous grade. This pipe drops sharply, then run horizontally under obstruction and finally rises to the

desired elevation. Siphons are usually made of circular concrete pipe or PVC connecting two canal reaches in series. Some siphons have rectangular cross-sections. When going across a depression, siphon should be completely buried, usually with a minimum of about 1m of cover. Siphons are like culverts, but instead of sloping down from inlet to outlet, they slope down, then back up to the outlet.

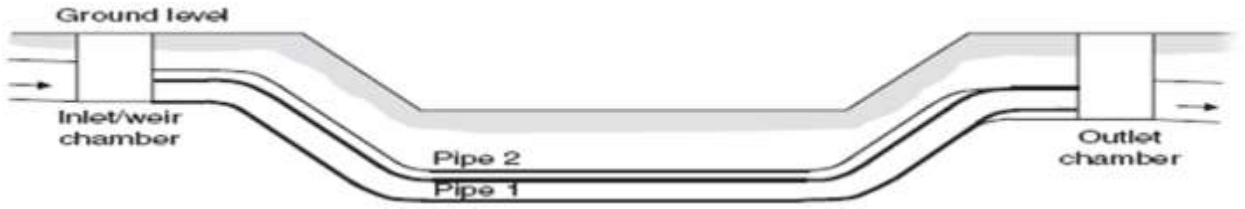


Figure 4.7 Inverted siphons

Cleanouts

A sewer cleanout is a capped pipe which provides access to a sewer line, allowing people to cleanout blockages in the sewer. In many regions, sewer cleanouts can be found along the lateral sewer line, the sewer line which connects a home's plumbing to the municipal plumbing. Lateral clogs and the sewer cleanout are the responsibility of the property owner. Larger sewer cleanouts are also located at intervals along the municipal sewer line, so that water and sewer employees can clear blockages in the part of the sewer system which is the municipal utility districts responsibility.

Sewer crossings

It is frequently possible to cross depressions or small watercourses without resorting to inverted siphons. Occasionally an existing bridge may be used to suspend the pipe from a truss. Vibration will occur, however, and cast or ductile iron pipe should be used with special couplings to avoid leakage. A more common method of crossing shallow depressions is to support the pipe on piers or bents of concrete. If the exposed pipe is long, provision should be made for thermal expansion.

Flushing Tank

Flushing is a technique in which short duration waves of liquid are introduced so as to clean the sediment into suspension and, hence, transport it downstream. Located at the head of a sewer. They are designed for 10 minutes flow as a self cleansing velocity of 0.6 m/sec. The capacity of these tanks is usually 1/10 of the cubic capacity of sewer length to be flushed.

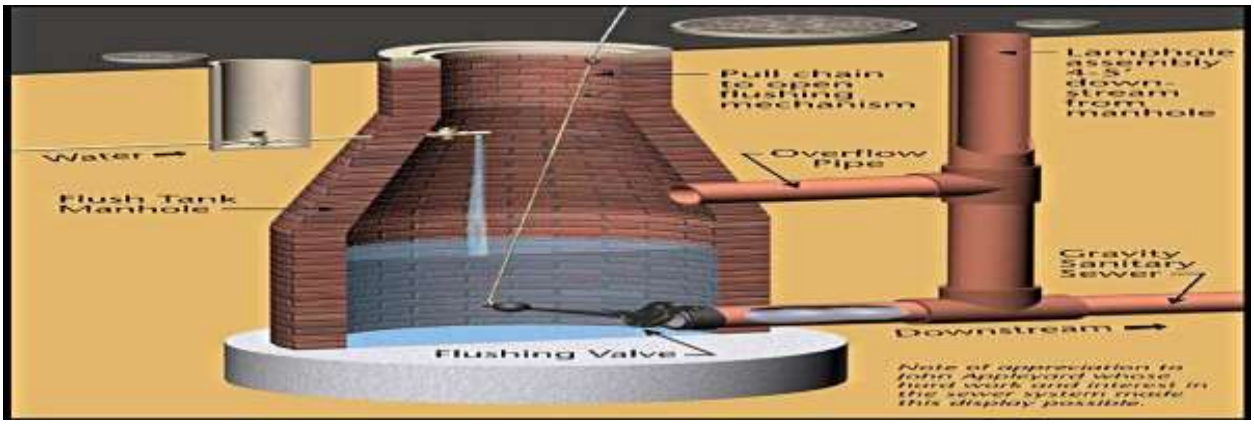


Figure 4.7 Flushing tank

Pumping stations

They are used to move wastewater to higher elevations. In general, the need for sewer pumping stations arises when: The existing topography and required minimum sewer grades create deep sewers that have high construction costs. The sewage is raised and then conveyed by gravity. Basements are too low to discharge sewage to the main sewer. Sewage must be conveyed over a ridge. The sewage must be raised to get head for gravity flow through a treatment plant. Discharge outlets are below the level of the receiving body of water. An existing gravity system is not yet available. A pumping station will enable development and growth in accordance with the applicable community plan. Sewage is fed into and stored in an underground pit (wet well). When the level rises to a predetermined point, a pump will be started to lift the sewage upward through a pressurized pipe system (sewer force main or rising main). From there it is discharged into a gravity manhole again.

Chapter Five

5. Sewer construction

Most sewers are constructed underground. This is achieved by one of three general methods. These methods of construction of sewer are:

a. embankment condition
b. trench condition and
c. tunnel condition

Open-trench construction consists of excavating vertically along the line of the sewer to form a trench, laying pipes in the trench and backfilling. It is suitable for a wide range of pipe sizes and depths and is the common method for small- to medium-scale sewer construction.

5.1 Trenching and Excavation

Trench width must be great enough to provide room to join pipe sections and install required fittings and clearance of about 150 mm (6 inches) on either side normally adequate for. In rock excavations, the trench is typically cut at least 6 inches (150 mm) below the final grade of the pipe and sand or clean fill is placed between the rock and the pipe. The ability to withstand the loads is derived from the strength of the pipe itself and from the nature of the bedding on which it is laid.

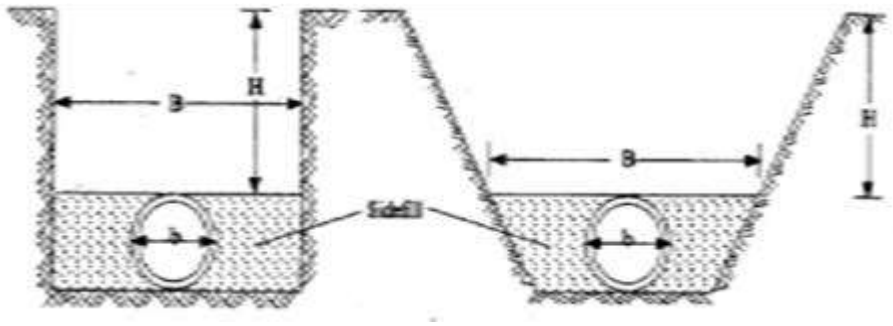


Figure 5.1 Excavation of trench for sewer pipe

Different calculation procedures are used depending on whether the pipe is considered to be rigid or flexible.

- Rigid: asbestos-cement, cast iron, concrete, vitrified clay
- Flexible: ductile iron, steel, corrugated aluminum, plastic (UPVC, PVC).

With increasing depth of cover, the load from the soil increases and the load from traffic decreases. Generally Sewers are laid in trenches by excavation of earth and refilling to the original ground level. Loads on sewer lines affected by conditions of flow, groundwater, adjacent earth, and superimposed situation.

Loads include hydraulic loads, earth loads, groundwater loads, and superimposed loads. Therefore, crushing strength of sewer material, type of bedding and backfill load are all important. Hence types of loads in trench condition are discussed below:

5.2 Type of loads

In a buried sewer, stresses are induced by external loads and internal pressure in case of a pressure main. The external loads are of two categories viz. load due to backfill material known as backfill load and superimposed load which again is of two type‟s viz. concentrated load and distributed load. Moving loads may be considered to be equivalent as uniformly distributed load. Sewer lines are mostly constructed of stoneware, which are considered as rigid pipes. The flexibility affects the load imposed on the pipe and the stresses induced in it.

5.2.1 Loads on conduits due to backfill

The resultant load on the horizontal plane at the top of the pipe within the trench is equal to the weight of the backfill minus upward shearing forces.

Computation of loads:

The load on rigid conduits in trench condition is given by the Marston‟s formula in the form:

$$W_c = C_d \, w \, B^2_d$$

Where: W_c = the load on the pipe in kg per linear meter

w = the unit weight of backfill soil in kg/m^3

B_d= the width of trench at the top of the pipe in m and

C_d= the load coefficient which is a function of a ratio of height of fill to width of trench (H/ B_d)

H = Depth of refilling of soil from top of pipe to the ground level in meters.

Weights of common filling materials (w) and values of Cd for common soil conditions encountered are given in Table 1 and 2 respectively.

Table 5.1 weights of common filling materials

Materials	Weight (kg/m^3)
Dry sand	1600
Ordinary (Damp sand)	1840
Wet sand	1920
Damp clay	1920
Saturated clay	2080
Saturated top soil	1840
Sand and Damp soil	1600

Table 5.2 Values of C_d for calculating loads on pipes in trenches ($W_c=C_dWB^2d$)

Ratio H/B	Safe working values of C_d				
	Minimum possible without cohesion	Maximum for ordinary sand	Completely saturated Top Soil	Ordinary maximum for clay	Extreme maximum for clay
0.5	0.455	0.461	0.464	0.469	0.474
1.0	0.830	0.852	0.864	0.881	0.898
1.5	1.140	1.183	1.208	1.242	1.278
2.0	1.395	1.464	1.504	1.560	1.618
2.5	1.606	1.702	1.764	1.838	1.923
3.0	1.780	1.904	1.978	2.083	2.196
3.5	1.923	2.075	2.167	2.298	2.441
4.0	2.041	2.221	2.329	2.487	2.660
4.5	2.136	2.344	2.469	2.650	2.856
5.0	2.219	2.448	2.590	2.798	3.032
5.5	2.286	2.537	2.693	2.926	3.190
6.0	2.340	2.612	2.782	3.038	3.331
6.5	2.386	2.675	2.859	3.137	3.458
7.0	2.423	2.729	2.925	3.223	3.571
7.5	2.454	2.775	2.982	3.223	3.571
8.0	2.479	2.814	3.031	3.366	3.764
8.5	2.500	2.847	3.073	3.424	3.845
9.0	2.518	2.875	3.109	3.476	3.918
9.5	2.532	2.898	3.141	3.521	3.983
10.0	2.543	2.918	3.167	3.560	4.042
11.0	2.561	2.950	3.210	3.626	4.141
12.0	2.573	2.972	3.242	3.676	4.221
13.0	2.581	2.989	3.266	3.715	4.285
14.0	2.587	3.000	3.283	3.745	4.336
15.0	2.591	3.009	3.296	3.768	4.378
Very Great	2.599	3.030	3.333	3.846	4.548

5.2.2 Load on conduit due to super imposed loads

The type of super imposed loads which generally encountered in buried conduits may be:

a. concentrated load and
b. distributed load

a. Concentrated Load: The formula for load due to super imposed concentrated load such as a truck wheel is given in the following form by Holl"s integration of Boussinesq"s formula.

$$W_{sc} = C_s\,(PF/L)$$

Where: W_{sc}= the load on the conduit in kg/m

P = the concentrated load in kg acting on the surface

F = the impact factor (1.0 for air field runways, 1.5 for highway traffic and air field taxi ways, 1.75 for railways traffic) and

Cs= the load coefficient which is a function of $\frac{B_c}{2H}$ and $\frac{L}{2H}$

Where: H = the height of the top of the conduit to ground surface in m

Bc= the outside width of conduit in m and

L= the effective length of the conduit to which the load is transmitted in m and Values of Cs for various values of (Bc/2H) and (L/2H) are obtained from Table 5.3

b. Distributed load : Superimposed loads, the formula for load on conduit is given by:

$$W_{sd} = C_s\, p\, F\, B_c$$

Where: W_{sd}= the load on the conduit in kg/m

p = the intensity of the distributed load in kg/m2

F = the impact factor

B_c= The width of the conduit in m

C_s = the load coefficient, a function of D/2H and L/2H from Table 3

H = the height of the top of conduit to the ground surface in m and D and L are width and length in m respectively of the area over which the distributed load

Table 5.3 Load factors for different classes of bedding

CLASS OF BEDDING	CONDITION	LOAD FACTOR
A	a. concrete cradle plain concrete and lightly tamped backfill	2.2
A	b. Concrete cradle plain concrete with carefully tampled backfill	2.8
A	c. Concrete cradle – RCC with P-0.4 %	Upto 3.4
A	d. Arch type plain concrete	2.8
	RCC with P-0.4%	Upto 3.4
	RCC with P-1.0% (P is the ratio of the area of steel to the area of concrete at the crown)	Upto 4.8
B	Shaped bottom or compacted granular bedding with carefully compacted backfill	1.9
C	Shaped bottom or compacted granular bedding with lightly compacted backfill	1.5
D	Flat bottom trench	1.1

C type of bedding is normally adopted.

5.2.3 Relationship between the different element in structural Design

The basic design relationships between the different design elements for rigid pipes are as follows:

$$\text{Safe working strength} = \frac{\text{Ultimate three edge bearing strength}}{\text{Factor of safety}}$$

$$\text{Safe field supporting strength} = \text{safe working strength} \times \text{load factor}$$

Note: The factor of safety recommended is 1.5.

Example1: Determine the structural requirement of 200 mm diameter stone ware pipe laid in a trench to a width of 0.8 m in depth of 1.30 meter in ordinary soil and wheel load of 6.25 tones.

Solution:

Pipe thickness, t = 16 mm for 200 mm dia

i. Back fill load:

BC = D + 2t = 200 + 2x16 = 232mm

B_d=0.8m

H=1.30-0.232=1.068 m

H/B_d=1.068/0.8=1.335

C_d= 1.05 (From Table 5.2) W= 1840 (From Table 5.1)

$W_c = C_d\, w\, B^2_d$ =1.05 x 1840 x 0.82 = **1237 kg/m**

ii. Concentrated load:

L = 0.60 (normal length of Stoneware pipe)

H= 1.068 m

L/2H=0.60/2 x 1.068 = 0.28

BC/2H= 0.232/2x1.068 = 0.11

From Table 5. 3 L/2H = 0.28 and BC/2H = -11 Cs=0.0498

$W_{sc} = C_s\,(PF/L)$ = 0.0498 x 1.5 /0.60 = **778 kg/m**

iii. Internal load ie Water Load at 75 % flow

$$\text{Water load} = \frac{22}{7} \times \frac{2}{10} \times \frac{2}{10} \times \frac{1}{4} \times 1000 \times -0.6 \times \frac{75}{100} = 14 \text{ kg/m}$$

Total load WLo= 1237 + 778+14 = 2029 kg/m

Safe supporting strength of 200 mm stone ware pipe with `C" clean bedding= 1650 x 1.5/1.5

= 1650 kg/m

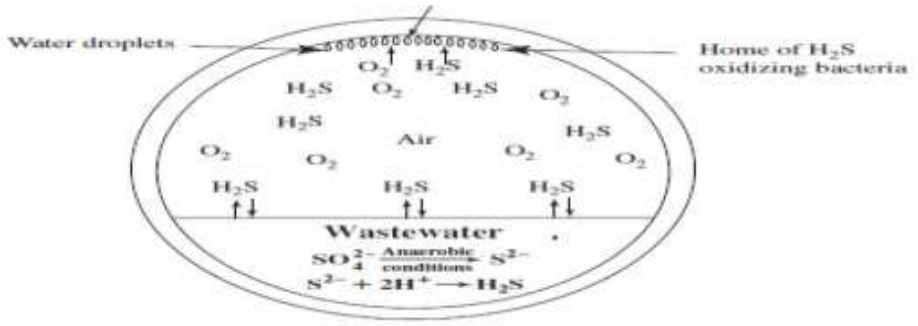
5.3 Sewer Corrosion

Sewer Corrosion is an Important Cause of sewerage Failure. Sewer corrosion takes place when low pH waste, usually industrial wastewater comes in contact with the concrete sewer structure. Corrosion is easily identifiable due to its propensity of attacking below the wastewater level or anywhere the wastewater contacts the cement structure on a regular basis.

1. Sewer Corrosion – Hydrogen Sulfide

Sulfide attack, hydrogen sulfide corrosion or simply sulfide corrosion is extremely costly problem. It is closely related to acid attack in that they both involve sulfuric acid attacking the structures. However, hydrogen sulfide corrosion can be found above the wastewater surface, usually in the crown of the pipe and is caused mainly by biological processes. Slow moving sewage allows anaerobic bacteria to reduce sulfate ions to sulfide ions. Corrosion occurs when the produced H_2S gas condenses on the sewer crown. Condensate oxidized by aerobic bacterium (sulphur bacteria (Thiobacillus) into H_2SO4. The strong acid reacts with the lime in the concrete to form calcium sulphate, which weakens the pipe leading to eventual collapse.

$$H_2S + O2 \xrightarrow{\text{Thiobacillus}} H_2SO_4 + \text{energy}$$



H_2S reacts with metals in the fittings and electronic equipment and all forms of iron and steel. If sewer pipes are made of these materials, they can be attacked from both sides. These bacteria reduce the sulfate in the groundwater to precipitate iron (II) sulfides, which are key in the corrosion of the exterior of the pipes. This type of corrosion is not exclusive to sewer pipes. This corrosion can affect any type of pipe exposed to soils containing sulfate. Sulfide Generation based on Z values Pomeroy and Parkhurst (1977) have suggested a formula that produces an index Z that is broadly indicative of the conditions under which sulphide may be formed in gravity sewers under 600mm in diameter. The „Z formula" gives:

$$Z = \frac{3(EBOD)}{S_o^{\frac{1}{2}} Q^{\frac{1}{4}}} \frac{P}{B}$$

Where: EBOD = Effective biochemical oxygen demand, BOD5 $= BOD_5 \times 1.07^{(T-20)}$ (mg/l)

BOD5 = 5-day, 20°C biochemical oxygen demand of the wastewater, mg/l

T = wastewater temperature (°C)

Q = flow-rate (l/s)

So = sewer gradient (m/100m)

P = wetted perimeter (m)

B= flow width (m).

Table5. 4: Likelihood of sulphide development

Z	Prevalence of sulphide
<5000	Rarely present.
~7500	Low concentrations likely.
~10 000	May cause odour and corrosion problems.
~15 000	Frequent problems with odour and significant corrosion problems.

Example 2: A 500mm diameter concrete sewer (n=0.012) has been laid at a gradient of 0.1%. At average flow, the pipe runs half full of wastewater. If the wastewater has a BOD5 of 500 mg/l and the summer temperature is 30 °C, estimate the likelihood that hydrogen sulphide will be generated.

Solution

Geometric properties of flow:

$$\text{Area of flow, } A = \pi D^2/8 = 0.098\,\text{m}^2$$

$$\text{Wetted perimeter, } P = \pi D/2 = 0.785\,\text{m}$$

$$\text{Flow width, } B = D = 0.5\,\text{m}$$

Flow rate,

$$Q = \frac{A}{n} R^{\frac{2}{3}} S_o^{\frac{1}{2}} = \frac{0.098}{0.012}\left(\frac{0.098}{0.785}\right)^{\frac{2}{3}} 0.001^{\frac{1}{2}} = 64.6\,\text{l/s}$$

$$EBOD = BOD_5 \times 1.07^{(T-20)} = 500 \times 1.07^{10} = 984\,\text{mg/l}$$

$$Z = \frac{3(EBOD)}{S_o^{\frac{1}{2}} Q^{\frac{1}{3}}} \frac{P}{B} = \frac{3 \times 984}{0.1^{\frac{1}{2}} 64.6^{\frac{1}{3}}} \frac{0.785}{0.5} = 3653$$

This value of Z implies sulphide is unlikely to be present.

Sulfide Generation based on Z values:

$$Z = (0.308)\left(\frac{EBOD}{(S_o^{0.50})(Q^{0.33})}\right)\left(\frac{P}{B}\right)$$

- EBOD is the effective BOD defined by: EBOD = BOD $(1.07)^{T-20}$
- P/B is the ratio of the wetted perimeter to the top width of the flow

Table 5.4 Sulfide Generation based on Z values

Z Values	Sulfide condition
$Z < 5{,}000$	sulfide rarely generated
$5{,}000 \leq Z \leq 10{,}000$	marginal condition for sulfide generation
$Z > 10{,}000$	sulfide generation common

Example 3: Determine the potential of sulphide generation in a trunk sewer made of concrete (n=0.015) having 1500 mm diameter laid on a slope of 1% and its peak discharge through the pipe is estimated to be 4000L/s. If 5-day BOD of the sewage is measured as 1600mg/l and the ambient temperature in the sewer is 30^{o}c.

Solution

The following equation can be solved by iteration to obtain θ:

$$\theta^{-2/3}(\theta-\sin\theta)^{5/3} - 20.16nQD^{-8/3}S_o^{-1/2} = 0$$

$$n = 0.015,\quad Q = 4m^3/s,\quad S_0 = 0.01,\quad D = 1.5m$$

$$\frac{(\theta-\sin\theta)^{5/3}}{\theta^{2/3}} - \frac{20.16(0.015)(4)}{(1.5)^{8/3}(0.01)^{1/2}} = 0$$

By trials, **θ= 3.5**

$$EBOD = BOD \times 1.07^{T-20} = 1600 \times 1.07^{10} = 3147.44$$

$$P = \frac{D\theta}{2} = \frac{1.5 \times 3.5}{2} = 2.625m$$

$$B = D\sin\frac{\theta}{2} = 1.5\sin\left(\frac{3.5}{2}\right) = 1.475$$

$$Z = 0.308\frac{EBOD}{S_0^{0.5}Q^{0.33}} \times \frac{P}{B} = 0.308\frac{3147.44}{0.01^{0.5}4^{0.33}} \times \frac{2.625}{1.475}$$

=10919>10,000 so, sulphide generation will be common in the sewer.

Waste Water Treatment

CHAPTER Six:

INTRODUCTION

1.1. General

Common wards:-

- **palatable**- water having no unpleasant taste
- **safe**- water free from pathogenic micro organisms & chemicals which could be a harmful for consumer
- **clean**- water free from suspended solids and turbidity
- **colourless and odourless** –water aesthetic to drink
- **Reasonably soft** – water allowing easy wash of cloths, dishes with less soap.
- **Lower organic content**-to avoid unwanted biological growth in pipes & smell
- **Non corrosive**- devoid of gases e.g. O_2,Co_2
- **Potable**- water suitable for drinking i.e. safe and pleasant to taste
- **Whole same**- potable water having sufficient minerals of natural origin which are must for life.
- **Contaminated**- water having microorganisms, chemicals or substances making the water unsafe and dirty.

Water available in various sources may contain various types of impurities and cannot be directly used by the public for various purposes, before removing the impurities. 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 from water is called *water treatment* and the treated water is called *wholesome water*.

The degree and methods of treatment depend upon nature of the source, quality of the source and purpose for which the water is supplied.

The various treatment methods and the nature of source of impurities removed by employing them are given in table1.

Table 1: Water Treatment

Process	Impurity Removal
Aeration	Taste and Odour removal, oxygen deficiency
Screening	Floating matter
Plain sedimentation	Large suspended solids
Coagulation	Fine particle
Filtration	Colloidal particles, microorganisms
Activated Carbon	Elements causing tastes and odour
Softening	Hardness
Disinfection	Living organism including pathogen

It is not that all the treatment process tabulated above will be required for a treatment plant. Treatment process selected will depend on the quality of water at the source and nature of water required. For example, in the case of water which taken from a surface source, generally the treatment unit required are plain sedimentation, coagulation, filtration and disinfection to make the fit for domestic use.

1.2. Objective of treatment

The main objective of the treatment process is to remove the impurities of raw water and bring the quality of water to the required standard. The objective may be summarized as follow:

(i) Preventing Disease Transmission

Organisms that cause disease must be removed or inactivated to make the water safe. Such organisms are small animals (invertebrates) and their eggs (ova), protozoa and their

+cysts, bacteria which may form spores, and viruses.

Chlorine is most commonly used to inactivate such pathogens, but the effectiveness of chlorine on some forms e.g. cysts and ova) is much less than on others, and suspended material in the water may shelter the pathogens from the chlorine.

(ii) Making the Water Acceptable

If the consumers regard the water as unsatisfactory they may use an alternative source which is hazardous. The taste, appearance and suitability for washing clothes shall all be considered.

(iii) Protecting the distribution System

- Corrosion of the system can be reduced by raising the PH of the water or adding chemicals. Corrosion reduces the life of the pipes, reduces their carrying capacity, and forms deposits which may colour the water. Harbour animals and interfere with valves.
- Depositions in pipes may result from unsatisfactory addition of chemicals, reactions within the system. or poor turbidity removal

1.3. Location of treatment plant

The treatment plant should be located

- Near to the town to which water is to be supplied and near to the source of supply. This will prevent the water quality to depreciate after treatment.
- Away from any source of pollution.
- Away from the border of other countries and should be announced as a protected area.

During war time, a neighbour country may play foul game by damaging the plant, poisoning the water.

- At higher elevation if the natural topography permit.

1.4. Treatment Schemes

a) Slow Sand Filters: water passes slowly through a bed of sand, and as it does suspend and dissolved organic impurities is removed by physical and biological processes. If the filters are well operated, chlorination is not essential. if the turbidity of the incoming water is greater than 30 NTU, the water should pass through a pre-treatment stage before reaching the slow sand filter.

b) Conventional Treatment**:** This process relies on a chemical called a coagulant, to coagulate (colloidal) suspended material in to flocs which are separated from water by gravity sedimentation and rapid filtration. Even when the rapid filters are well operated, disinfection by chlorine is essential.

c) Treatment of Ground Water: usually disinfection is not essential. Certain gases cause tastes

or odours when dissolved in water. Dissolved iron and manganese can be precipitated and removed from ground water by oxidation (often simple aeration), sedimentation and filtration. Other dissolved impurities may require more sophisticated and expensive treatment, such as reverse osmosis or ion exchange.

1.5. Factors Affecting the Choice of Treatment Schemes

The following factors influence the choice of treatment alternative discussed below:

a) Limitation of capital

b) Availability of skilled and unskilled labour

c) Availability of equipment, construction material, and water treatment chemicals

d) Local codes, drinking water standards and material specifications

e) Local traditions, customs and cultural standards

f) National sanitation and pollution policies.

Consideration for Treatment unities in developing country

- Use hydraulic devices instead of mechanical equipments e.g. for mixing of chemicals
- Use indigenous materials & manufacturing to reduce the cost
- Lower peak and per capital consumption
- Lower design period
- Organisational capacity to recruits and retrain
- Head lose should be conserved possible

1.6. Source of Water Pollution and Water Impurities

1.6.1. Water Pollution

Water pollution is any chemical, biological, or physical change in water quality that has a harmful effect on living organisms or makes water unsuitable for desired uses.

a) Point source: specific location (drain pipes, ditches, sewer lines).

b) Non point source: cannot be traced to a single site of discharge (atmospheric deposition, agricultural / industrial / residential runoff)

Table 2: Major Water Pollutants and Their Effect

Type/*Effects*	Examples	Major Sources
Infectious agents *Cause diseases*	Bacteria, viruses, parasites	Human and animal wastes
Oxygen-demanding wastes *Deplete dissolved oxygen needed by aquatic species*	Biodegradable animal wastes and plant debris	Sewage, animal feedlots, food processing facilities, pulp mills
Plant nutrients *Cause excessive growth of algae and other species*	Nitrates (NO_3^-) and phosphates (PO_4^{3-})	Sewage, animal wastes, inorganic fertilizers
Organic chemicals *Add toxins to aquatic systems*	Oil, gasoline, plastics, pesticides, cleaning solvents	Industry, farms, households
Inorganic chemicals *Add toxins to aquatic systems*	Acids, salts, metal compounds	Industry, households, surface runoff
Sediments *Disrupt photosynthesis, food webs, other processes*	Soil, silt	Land erosion
Thermal *Make some species vulnerable to disease*	Heat	Electric power and industrial plants

Table 3: Common Disease Transmitted to Humans Through Contaminated Drinking Water

Type of Organism	Disease	Effects
Bacteria	Typhoid fever	Diarrhea, severe vomiting, enlarged spleen, inflamed intestine; often fatal if untreated
	Cholera	Diarrhea, severe vomiting, dehydration; often fatal if untreated
	Bacterial dysentery	Diarrhea; rarely fatal except in infants without proper treatment
	Enteritis	Severe stomach pain, nausea, vomiting; rarely fatal
Viruses	Infectious hepatitis (Type B)	Fever, severe headache, loss of appetite, abdominal pain, jaundice, enlarged liver; rarely fatal but may cause permanent liver damage
Parasitic protozoa	Amoebic dysentery	Severe diarrhea, headache, abdominal pain, chills, fever; if not treated can cause liver abscess, bowel perforation, and death
	Giardiasis	Diarrhea, abdominal cramps, flatulence, belching, fatigue
	Cryptosporidium	Severe diarrhea and possible death for people with weakened immune systems
Parasitic worms	Schistosomiasis	Abdominal pain, skin rash, anemia, chronic fatigue, and chronic general ill health

1.6.2. Impurities in water

The impurities which are present in water may be classified in the following three categories. a) Suspended impurities

b) Colloidal impurities c) Dissolved impurities

a) Suspended impurities

These impurities are dispersion of solid particles that are large enough to be removed by filtration or sedimentation.

Suspended impurities are
- Bacteria –some cause diseases
- Algae, protozoa –odor, turbidity
- Silt -muckiness or turbidity

b) Colloidal impurities

The finely divided dispersion of solid particles which are not visible to the naked eye cannot be removed by ordinary filters are known as colloidal impurities. They are chief source of epidemic because they are associated with bacteria. It is removed by coagulation.

c) Dissolved impurities

It includes organic compound, inorganic salt and gases etc.

1.7. Drinking water quality standards

The treatment process should be designed such that, they should treat the water up to the desired standard for which it is to be used. Table 2 gives the standard of water quality for domestic water supplies.

A safe and potable drinking water should conform to the following water quality characteristics

1) Free from disease causing micro organisms
2) Containing no compounds that have an adverse effect acute or in long term on human health.
3) Fairly clear (i.e. low turbidity ,little colour)
4) No saline
5) Containing no compound that cause offensive taste or smell
6) No causing corrosion or encrustation of the water supply system, nor staining clothes

washing on it.

Table 4: Standard for Drinking Water Quality

Parameters	Guideline value
Arsenic	0.01mg/l
Cadmium	0.003mg/l-toxic illness
Chromium	0.05mg/l
Copper	2mg/l
Cyanide	0.07mg/l
Fluoride	1.5mg/l
Lead	0.01mg/l – poisoning
Manganese	0.05mg/l- black or brown color
Nitrate	50mg/l
Nitrite	3mg/l
Total hardness	500mg/l as $CaCo_3$
E.coli	0(in 100ml)
Chlorine	0.3-0.5 mg/l after 30mix contact

Aesthetic quality value

Aluminum	0.2mg/l
Ammonia	1.5mg/l
Chloride	250mg/l
Color	15Tcu
Iron	0.3mg/l
Manganese	0.1mg/l
DO	-
PH	6.5-8.5
Taste & odor	-

CHAPTER Seven: PRELIMINARY TREATMENT

2.1. Pre treatment

High turbidity water which may occur particularly during the rainy season requires pre treatment in the form of sedimentation, storage or roughing filtration to reduce much of the suspended solids. This is an advantage otherwise a very large amount of chemicals may have to be employed for chemical coagulation which can be expensive. Pre-treatment provides cheaper treatment. Slow Sand Filters require raw water turbidity below 30NTU. High turbidity waters require pre-treatment before slow sand filtration. There is, however, a form treatment known as dynamic filtration (although it is not very popular) which involves diverting a relatively large volume of water out of which a small percentage (10%) settles through the slow sand filter while the remaining is used to wash off the solids deposited on the sand top as the water moves horizontally via the sand bed.

2.2. Types of preliminary treatment

- Intakes
- Screens
- Plain sedimentation
- Storage
- Roughing filter
- Infiltration galleries
- Silt trap

2.2.1. Intakes

Proper design of the intake structure is one way of achieving preliminary treatment. The intakes should be located in such a way that rolling debris at the bottom is prevented from entering via the intake. Bar Screens are provided to screen out larger size floating and suspended materials. Sometimes two filters are provided successively for coarse and fine screening. A floating intake ensures intake from the top and relatively clean layer. Multiple level intakes (low le vel intake in the dry season and to avoid algae at the top and high level intake in the wet season to avoid HU

suspended solids at the bottom) are provided in lakes and reservoirs.

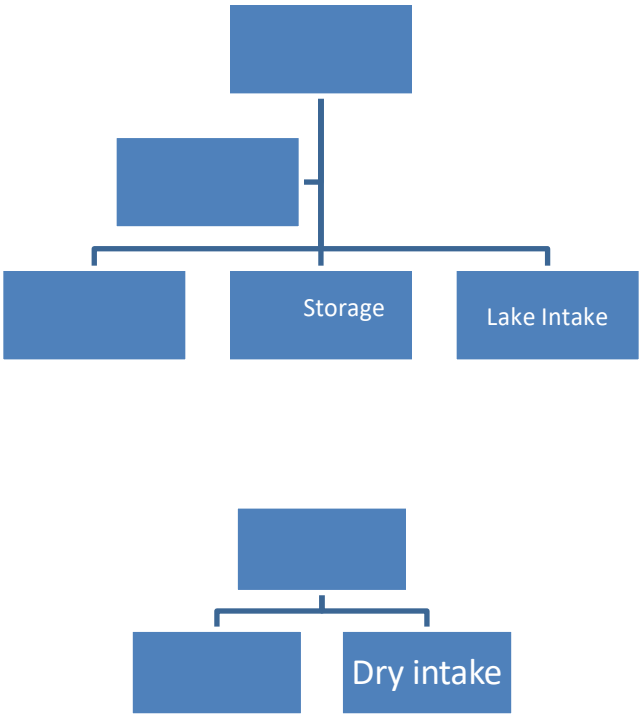


Figure 1 : Type of intake

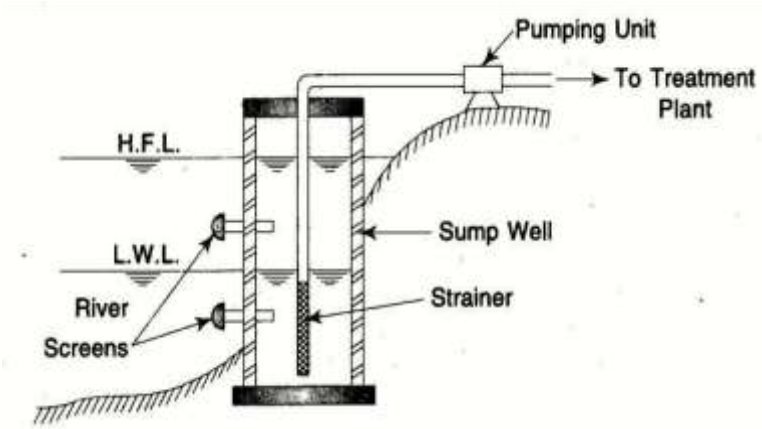


Figure 2 : River Intake

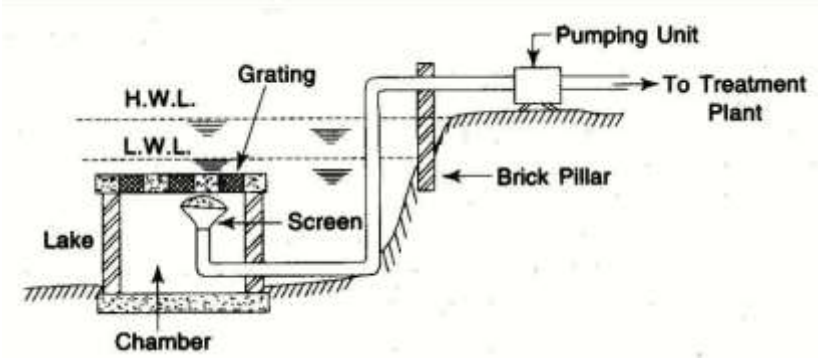


Figure 3 : Lake Intake

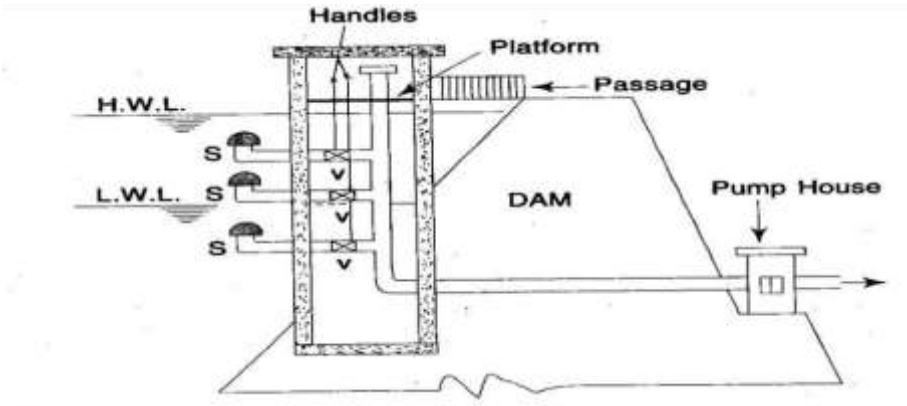


Figure 4: Reservoir Intake

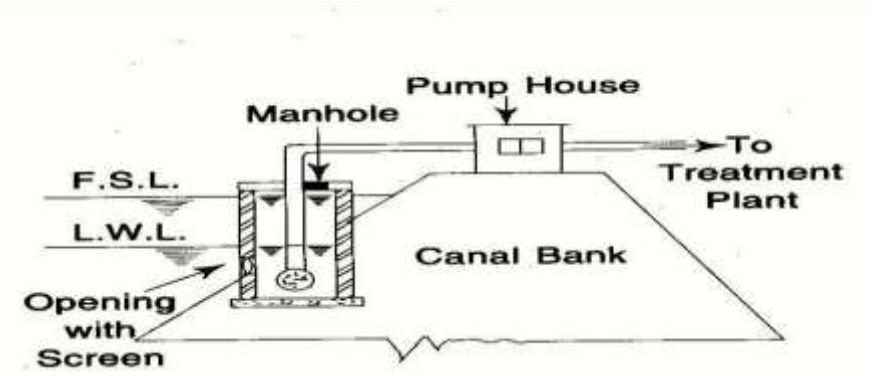


Figure 5: Channel Intake

Example 1

Design a bell mouth intake for a city of 80,000 persons drawing water from a channel which runs for 10hrs in a day with depth of 1.8m. Also calculate the head loss in intake conduit if the treatment plant is ½ km away .draw a net sketch of the canal intake assume average consumption per person =150l/d.

Assume the velocity through the screen & the bell mouth to be less than 16cm/s & 32cm/s respectively.

2.2.2. Screens

Screening of water which is one form of pre-treatment is done by passing the water through closely spaced bars, gratings or perforated plates. Screening does not change the chemical or bacteriological quality of the water. It serves to retain the coarse material and suspended matter that are larger than the screen openings.

Purposes:

(i) Removal of floating and suspended matter which clogs pipes, damages pumps, etc.

(ii) Clarification by removal of suspended matter to lighten the load on subsequent treatment processes.

Bar screen spacing is typically between 0.5 and 5cm. Angle of inclination of bars is $60\text{-}75^0$ if screening are very small and $30\text{-}45^0$ if larger amount is retained over the screen bar.

Velocity of flow should be low towards the screen bar (0.1-0.2m/sec). it may be increased to 0.3-0.5 after the screen to prevent settling there. Between the openings the velocity should be restricted to up to 0.7m/sec to avoid forcing through the suspended solids. If regular cleaning is done an allowance for loss of heads of up to 0.1 to 0.2m is made. However to allow for delay and mechanical failures a loss of head allowance between 0.5 to 1.0m is made.

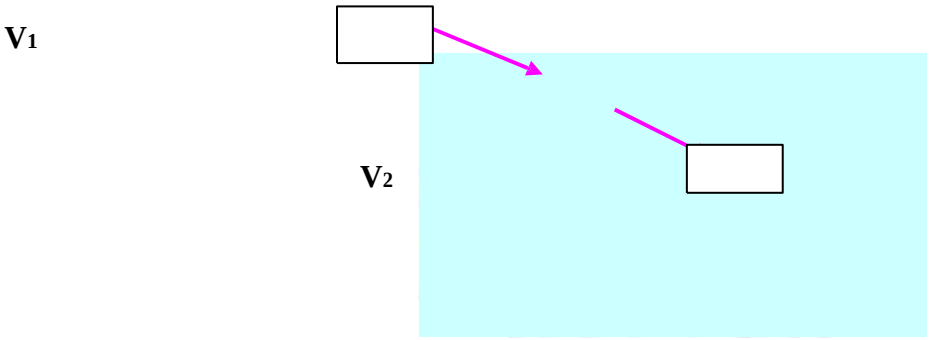


Figure 6: Screen

Example 2

Determine the building up of head loss through a bar screen .when 50% of the flow area is blocked off by the accumulation of coarse solides assume the following conditions are applied.

Approach velocity =0.6m/s

Velocity through a clean bar screen =0.9m/s

Open area for flow through clean bar screen =0.19m^2

Example 3

Determine the building up of head loss through a bar screen .when 25 % of the flow area is blocked off by the accumulation of coarse solids. Assume the Conditions of the above Example.

2.2.3. Plain Sedimentation

Plain sedimentation is a form of pretreatment that provides a low velocity of flow through a tank preferably excavated in the ground. The purpose is to settle some solids because of this low velocity by gravity sedimentation. Plain sedimentation is favouable in a tropical climate due to the high load of suspended solids to be settled as a result of erosion by tropical high intensity rains,

and also, because of the high temperature in the tropics and the associated low viscosity water in the sedimentation tank that provides less viscous resistance for the settlement of solids.

Since plain sedimentation offers limited detention period for the water its effectiveness is restricted in that sense. Significant reduction in solids is obtained for high turbidity waters, and, the feasibility of reducing the turbidity below 30 NTU - if the tank is provided as a pretreatment unit for a slow sand filter -must be tested through a settling column test.

The settlement of solids is dependent on the nature of the suspension. The table below shows the typical values used in practice for the parameters listed.

Table 5 : Design parameter for plain sedimentation

Parameter	Range of Values
Detention Time (Hrs.)	0.5 to 3.0
Surface loading (m/day)	20 to 80
Depth of the basin (m)	1.5 to 2.5
L/W Ratio	4:1 to 6:1
L/D Ratio	5:1 to 20:1

The tank may be rectangular, or, to minimize the need for thicker walls trapezoidal shape (which also facilitates settlement to the bottom) tank can be used. Baffle walls are provided at the inlet to dissipate the kinetic energy of the incoming water and provide quiescent settlement. Less importantly though, they are also provided at the outlet to prevent turbulence in the outlet zone.

For trapezoidal channels a thin Ferro-cement wall lining may be adequate. For waters laden with algae the outlet weir are arranged behind a deflecting baffle. At least two settling baffles are provided each designed for 3/4 of the design flow so that during cleaning of one of the tanks, the other takes the full load and will be overloaded by 33% only. It is possible though to design both tanks to handle the full flow without being overloaded. This is, however, an over design as cleaning is needed for a short while and usually after a long period of operation. Manual cleaning can be done. Fixed nozzles and fire hoses can be used to help with cleaning.

The table below indicates the turbidity removal to be anticipated with respect to varying raw water quality and provided with different detention times (Experiment done in Iraq.)

Table 6: Turbidity removal

Initial Turbidity	Turbidity Remaining (NTU)	
	After 2Hrs.	After 3Hrs
500	145	90
1200	620	120
1800	450	90
2500	610	120

Tube Settlers

Inclined plates or tubes are fixed in the tank and water is let to pass through this tubes/ plates. The surface area of settlement increases and hence the efficiency of settlement. Tube settlers are employed for upgrading the performance of existing tanks and for reducing the size of new tanks. Best angle of inclination is 40^{o}.

2.2.4. Storage

Storage is very effective for high turbidity water. The detention time is greater than for plain sedimentation tanks. For extremely turbid waters annual average turbidity > 1000NTU storage provides the best pre-treatment.

Advantages:

- Natural sedimentation
- Attenuation of sudden water quality fluctuation.
- Reduction in the number of pathogenic bacteria
- Improved reliability of supply
- Excessively turbid water can be diverted away.

The storage can be in the form of ponds or lagoons which are natural or excavated in the ground, or in the form of man-made earth dams. Capacity should make allowance for seepage and evaporation. The bottom of the pond should be covered with clay or other impermeable material. In order to minimize pollution from outside, access to outsiders should be restricted. Fences can be provided with vegetation (bush). This also acts as a wind breaker.
Example of treatment with Ponds (England)

Table 7: Storage effect on turbidity and E-Coli

	River Thames		**River Great Ouse**	
	Raw water	**Stored water**	**Raw water**	**Stored water**
Turbidity	14	3.2	10	1.5
E-Coli (MPN/1000ml)	20000	100	1700	10

The above table shows how storage can be effective in reducing turbidity and suspended solids significantly. The costs of providing storage can be very high, however. Depths of ponds can be as high as 15m.

2.2.5. Roughing Filtration

Roughing filtration allows a deep penetration of the filter layer and holds a large silt storage capacity.

Diameters: Rapid sand filters 0.4 - 0.7mm

Slow sand filters 0.15-0.35mm

Roughing filters >2.0mm diam.

Rate of filtration is variable depending on the nature of the media, turbidity of the incoming water, etc. Roughing filters if designed well, can serve as effective pre treat ment units prior to slow sand filtration for water with raw water turbidity of 20 to 150 NTU. For high turbidity

waters, it may be difficult to clarify to below 30NTU without sedimentation or other pretreatment such as storage.

Apart from horizontal and vertical roughing filters, there is a form of roughing filtration known as up-flow down flow filtration that involves an up flow roughing filter together with chemical coagulation and flocculation followed by a down flow rapid filtration. This arrangement can result in reduction of cost almost by 50% compared with the conventional designs involving coagulation, sedimentation and filtration.

Roughing filters are capable of reducing turbidity to as low as 5NTU from a raw water turbidity of 150NTU. Bacterial removal is between 60 and 90% without using any disinfectant.

The following reasons are given to advocate the use of roughing filters in developing countries:

a. In some of these regions the use of flocculent seems to be a very delicate matter due to inadequate handling of flocculation installations or inadequate supply. Roughing filters do not require the addition of chemicals to reduce solids to the extent necessary for slow sand filters downstream.

b. They ensure long filter runs owing to their excellent solid storage capacity. Filter runs can be extended to periods close to one year for raw water with periodically high solids load, especially when long horizontal filter layers are applied. They do not require a very high construction (sometimes simple ditches as done in some refugee camps in Sudan can be adequate.)

c. They do not require high skills to build them. The filters can be filled with local materials screened on site.

d. Because of low head no substantial head is required. The energy requirement is therefore reduced. The water is beneath the gravel top hence contamination by man, animal, micro - organisms and algae growths are avoided largely.

The efficiency of solids removal in roughing filters depends on the character (chemical, electrostatic, etc) of the particles to be removed, the fluid property (dissolved matters such as pumice and Ca^{++}, etc.), the process related characters such as the media size, flow rate, head, layer thickness, layer configuration, filtration rate, etc.

Filtration Theory

The mechanism of filtration in both the roughing and rapid rate filtration (slow sand filters additionally incorporate biological action) is the following:

- Attachment or interception of the particle to be settled by the filter grain
- Transport of the particle to the filter surface by gravity
- Transport of particle by Brownian diffusion being more important for very small diameter particles
- Transport as a result of the Vander Walls forces of attraction between the particles and filter
- Transport by electric double layer forces, due to surface charges of the particle and the filter

For roughing filters (with grain sizes being greater than 5mm in most cases) the predominant mode of filtration is by gravity separation and to a lesser extent by interception/straining. All the other processes have negligible effect owing to the larger grain size of the filter which implies greater separation distance between the particle to be attracted/diffused and the filter grain. These other forces become dominant when the media size is less than 3mm.

A variation of the upward flow roughing filtration is the so-called up-flow-down flow unit which combines upward flow roughing filter with a down ward flow rapid filters. For effective roughing filtration in line addition of coagulating chemical is recommended in the roughing filters. Significant construction cost reduction is achievable with this arrangement compared with the conventional coagulation, flocculation, sedimentation and filtration units. The operation cost is also reduced because of simpler operation and less need for coagulating chemicals.

Horizontal flow roughing filters have larger silt storage capacity because of the coarser size and longer length available. Filter runs can extend a number of years of operation. They hold much promise as pre-treatment units for slow sand filters. For more uniform solids deposition a grading from course to fine sizes is recommended.

Design Guidelines:

i. Filtration rate is between 0.5 - 4.0m/hr. An upper limit of 2.0m/hr for waters with high suspended solids.

ii. Three zones from course to fine media are provided with sizes from 4 to 40mm. iii. The first course should be long:

First course: 4.5-6m Second course: >4m

Last course: 1.5-2m

iv. Depth of 1.0-1.5m for easy manual desludging operations v.

There should be 10-20cm of gravel above the filter top

vi. Filter should slope in the flow direction to facilitate sludge withdrawal vii. V-notch recommended at the outlet for measurement of flow.

Very long lengths of filter having filtered run up to 5 years have been built in Germany, Dortmund, to minimize the need for cleaning, and hence, the labour cost. Such filter although producing the required quality is not recommended in developing countries.

Since labour is cheap and acquiring media for such great length may not be all easy. The walls of the filter can be made of brick lined with cement. The wall between the media is made of strong Mesh wire that is detachable. Partial recovery of the media capacity is possible by quickly draining the filter from the bottom in both types of filters. This will delay the time when the filter has to be deluged by taking the media out entirely.

2.2.6. Infiltration Gallery

It is underground tunnel which have holes on its sides, used for tapping underground water near river, lakes and stream.

This type of treatment is used when the water depth is very small and difficult to construct weir or dam to store water due to the seepage of the water. Sedimentation of silt is the main problem.

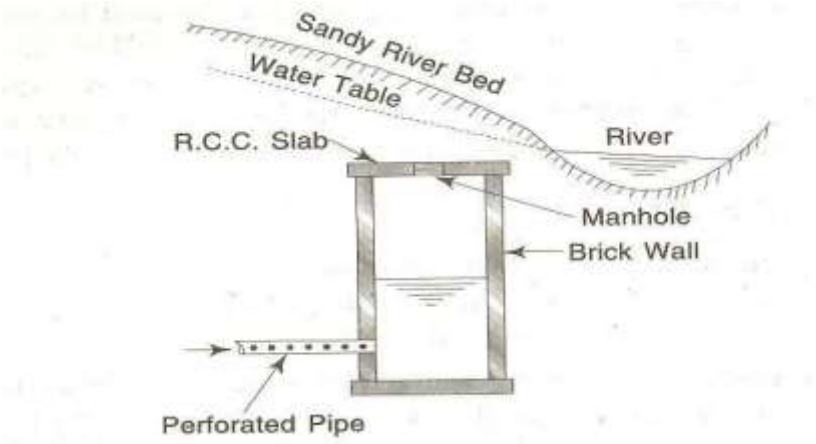


Figure 7: Infiltration Gallery

2.2.7. Silt trap

This is a box like structure to remove particles that have higher density than density of water .

Sampling Time (Hrs)	Sampling Depth (m)	Velocity (mm/sec)	% Remaining (w/w)
1	1	0.28	38
	2	0.56	50
	3	0.83	57
	4	1.11	60
3	1	0.09	17
	2	0.19	27
	3	0.27	35
	4	0.37	40
6	1	0.05	12
	2	0.09	18
	3	0.14	21
	4	0.19	32

Weight fraction Xo	Weight Fraction dxi	Corresponding Velocity Vxi (mm/sec)	Product Vxidxi mm/sec.

0.55	0.05 0.10 0.1 0.1 0.1 0.1	0.63 0.44 0.26 0.15 0.06 0.02	0.315 0.44 0.026 0.015 0.006 0.002
			$\sum Vxidxi = 0.125$

$$X_T = 1 - x_o + \sum \frac{V_x d_{xi}}{v_o}$$

$$\sum v_x d_{xi} = 0.125$$

$$= 0.45 + \frac{0.125}{0.7}$$

$$= 0.63$$

The predicted settlement is = 63%

Up flow Tanks

The upward flow velocity is Q/A and therefore all particles with V<Vo will have a down ward resultant velocity. Hence the removal fraction will be:

$X_T = 1-Xo$

Where Xo is the reaction by weight with v less than or equal to Vo.

Flocculating Material

Unlike discrete particles flocculating materials cannot be said to have fixed density. Discrete clay particles have a density of about 2640 Kg/m^3 whereas flocculated and mud particles have density down to 1030 Kg/m^3.

$$r_f = K_1 \eta^{0.429}$$

$$\Omega = K_2 \eta^{-0.29}$$

$$(\rho_f - \rho) = K_3 \bar{A}^{0.338\ p}$$

r_f = Floc radius

η = Number of particles

A_p = The projected area of a spherical particle

Ω = Volume fraction of solids

$(\rho_f - \rho)$ = Boyant density of floc

The above relationships demonstrate that increasing the floc radius will decrease the buoyant density. The net effect of increased radius and decreased density is to reduce the settlement velocity. Higher G values are therefore preferred since they reduce the floc radius and hence result in faster settlement. The decrease in buoyant density is due to the greater percentage of water entrained in the flocs.

The other disadvantage of increased floc radius is the difficulty associated with de-watering of the sludge.

Outlet Zone

The outlet zone controls the water flowing out of the sedimentation basin - both the amount of water leaving the basin and the location in the basin from which the out flowing water is drawn. Like the inlet zone, the outlet zone is designed to prevent short-circuiting of water in the basin. In addition, a good outlet will ensure that only well-settled water leaves the basin and enters the filter. The outlet can also be used to control the water level in the basin. Outlets are designed to ensure that the water flowing out of the sedimentation basin has the minimum amount of floc suspended in it. The best quality water is usually found at the very top of the sedimentation basin, so outlets are usually designed to skim this water off the sedimentation basin.

A typical outlet zone begins with a baffle in front of the effluent. This baffle prevents floating material from escaping the sedimentation basin and clogging the filters. After the baffle comes the effluent structure, which usually consists of a launder, weirs, and effluent piping. A typical effluent structure is shown below:

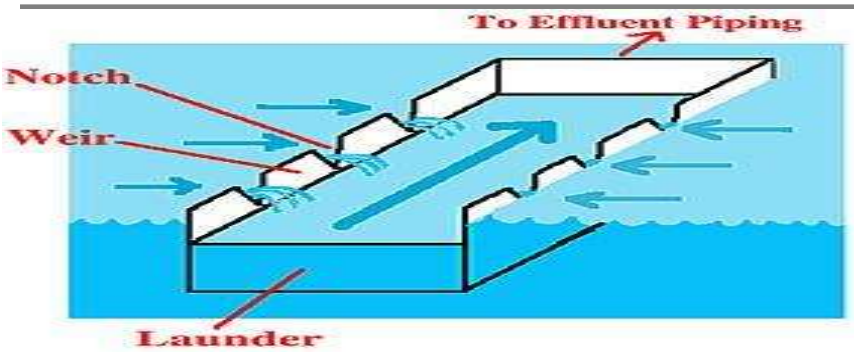


Figure 16: Outlet Zone

The primary component of the effluent structure is the effluent launder, a trough which collects the water flowing out of the sedimentation basin and directs it to the effluent piping. The sides of a launder typically have weirs attached. Weirs are walls preventing water from flowing uncontrolled in to the launder. The weirs serve to skim the water evenly of the tank.

A weir usually has notches, holes, or slits along its length. These holes allow water to flow in to the wire. The most common type of hole is the V-shaped notch Shown on the picture above which allows only the top inch or so of water to flow to out of the sedimentation basin. Conversely, the weir may have silts cut vertically along its length, an arrangement which allows for more variation of operational water level in the sedimentation basin.Water flows over or through the holes in the weirs and in to the launder. Then the launder channels the water to out let, or effluent, pipe. This pipe carries water away from the sedimentation basin and to the next step in the treatment process.

The effluent structure may be located at the end of a rectangular sedimentation basin or around the edges of the circular clarifier. Alternatively, the effluent may consist of a finger weir, an arrangement of launders which extend out in to the settling basin as shown below.

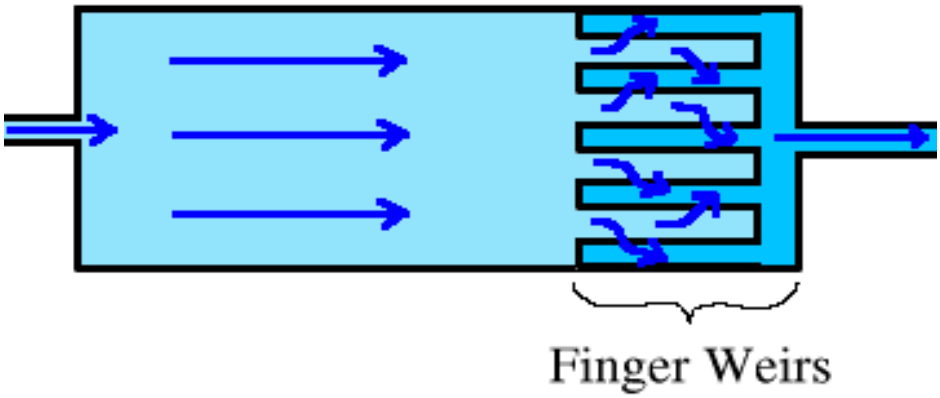


Figure 17: Finger Weir

Sludge Zone

The sludge zone is found across the bottom of the sedimentation basin where the sludge collects temporarily. Velocity in this zone should be very slow to prevent re-suspension of sludge.

A drain at the bottom of the basin allows the sludge to be easily removed from the tank. The tank bottom should slope toward the drains to further facilitate sludge removal. In some plants, sludge removal is achieved continuously using automated equipment. In other plants, sludge must be removed manually. If removed manually, the basin should be cleaned at least twice per year or more often if excessive sludge buildup occurs. It is best to clean the sedimentation basin when water demand is low, usually in April and October. Many plants have at least two sedimentation basins so that water can continue to be treated while one basin is being cleaned, maintained, and inspected.

If sludge is not removed from the sedimentation basin often enough, the **effective** (useable) volume of the tank will decrease, reducing the efficiency of sedimentation. In addition, the sludge built up on the bottom of the tank may become **septic**, meaning that it has begun to decay an aerobically. Septic sludge may result in taste and odor problems or may float to the top of the water and become scum. Sludge may also become re-suspended in the water and be carried over to the filters.

Sludge disposal

The sludge which is found in the bottom of a sedimentation tank is primarily composed of water. The solids in the sludge are mainly excess coagulant, such as alum. Alum sludge has a solid concentration of only about 1% when automatically removed from the basin. Or about 2% if manually removed (the greater solid concentration of manually removed sludge is due to a small amount of gravity thickening).

Many options exist for disposal of sedimentation sludge. Here we will discuss disposal in streams, sanitary sewers, lagoons and landfills.

Disposal in streams and sewers

In the past, sludge and back wash water was typically released into streams and other bodies of water. However, this practice is becoming much less common and is now well regulated. Back wash water and sludge can only be released in to streams if a discharge permit has been granted by the NPDES. The NPDES further requires extensive daily monitoring of the water quality when the sludge is being discharged.

Alternatively, sludge may be piped directly to the sanitary sewer. However, this disposal option also has its disadvantage. Sludge can cause sewer blockage. In addition, fees charges By the waste water treatment plant can be expensive. Releasing sludge in to the sewage lines in large batches makes treatment of the waste water problematic, so sludge is typically released slowly over a long time period, which requires a large holding tank at the water treatment plant.

Thickening sludge

Most of the other alternatives require transporting sludge away from the treatment plant. Sludge is typically dried before it is tucked away since the greater volume of wet sludge makes it much more expensive to transport. This drying process is known as **dewatering** or **thickening**. Alum sludge is difficult to thicken, but a Variety of devices have been developed to thicken, the sludge some of which are explained below.

In many cases the sludge is treated by the addition of polymers to aid in the dewatering process, alternatively, the sludge can be heated or frozen and thawed to increase the solids concentration. Treating to aid thickening is known as **conditioning** the sludge.

Once the sludge has been conditioned, it may be thickened in a lagoon, drying bed, or one of several devices.

Lagoons, which are small-volume storage ponds, are the simplest device used to thicken sludge. These lagoons are filled with sludge and the solids are allowed to settle due to gravity to the bottom of the lagoon while the clear water is pumped off the top. After a few months, gravity and evaporation will have reduced the sludge to a 30.

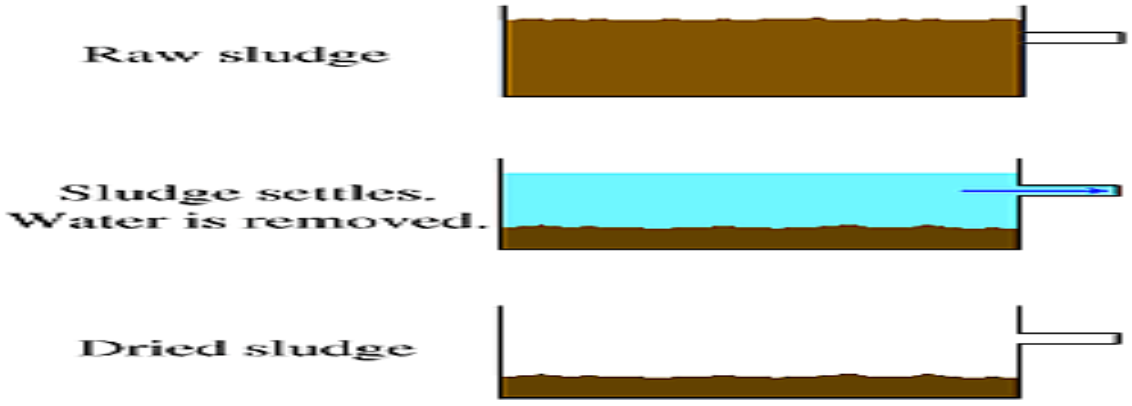
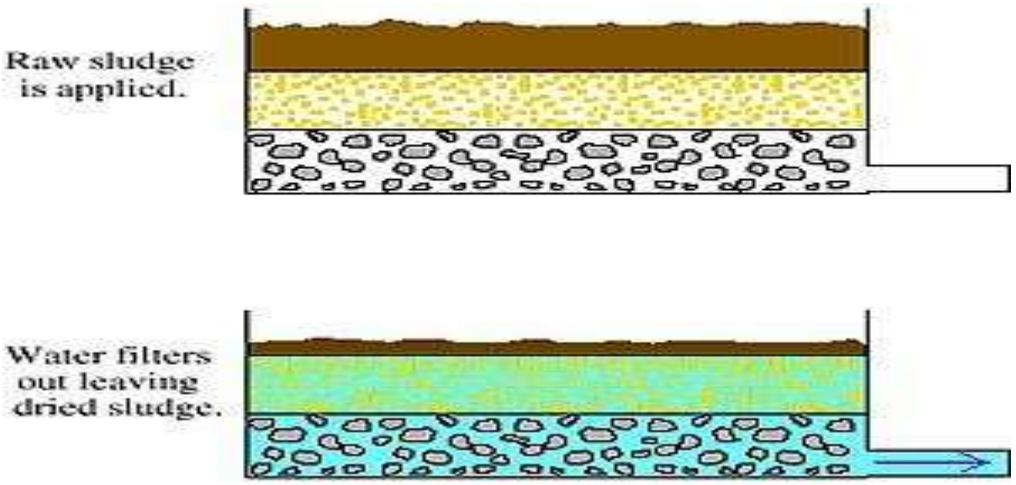


Figure 18: Lagoons

Drying beds are often used to thicken sludge destined for a landfill more quickly than the sludge would be thickened in a lagoon.

A **drying bed** is similar in design to a sand filter, with a layer of sand underlain by a layer of gravel. The sludge is applied to the top of the sand and the water percolates down through the sand and gravel and is drained away. When the sludge is sufficiently dry, it is carefully removed from the top of the sand and is trucked to a landfill. When sludge is preconditioned with chemicals, dewatering in a drying bed may take only a few days or weeks



.

Figure 19: Drying bed

Other processes used to thicken sludge include filter process, belt filter process, centrifuges, and vacuum filters. The processes result in sludge with a solids content ranging from 30-50%

3.4. Factors Influencing Efficiency of the basin

Floc Characteristics

To a large extent, a sedimentation basin's efficiency will depend on the efficiency of the preceding coagulation/flocculation process. The size, shape, and density of the floc entering the sedimentation basin will all influence how well the floc settles out of the water. Floc which is too small or too large, is irregularly shaped, or has a low density will not tend to settle out in the sedimentation basin.

Even if the coagulation/flocculation process is very efficient, floc can disintegrate on its way to or in the sedimentation basin.

Previously formed floc will disintegrate

- If the water velocity is too high,
- If there are sharp bends in the pipe at the inlet...

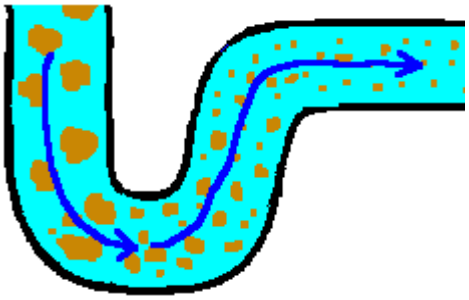


Figure 20: sharp bends in the pipe at the inlet

I. If water is discharged above the sedimentation basin water level or if throttle valves are used.

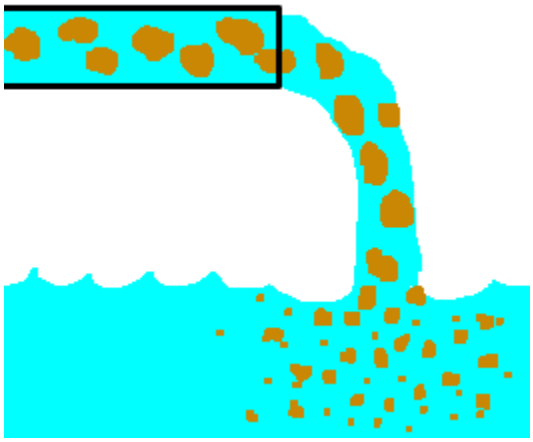


Figure 21: water is discharged above the sedimentation basin

Short-circuiting

Another major cause of inefficiency in the sedimentation basin is short-circuiting, which occurs when water bypasses the normal flow path through the basin and reaches the outlet in less than the normal detention time. The picture below shows a basin in which the water is flowing primarily through the left half of the basin. (Flowing water is shown as green blobs.) An efficient sedimentation basin would have water flowing through the entire basin, rather than through just one area.

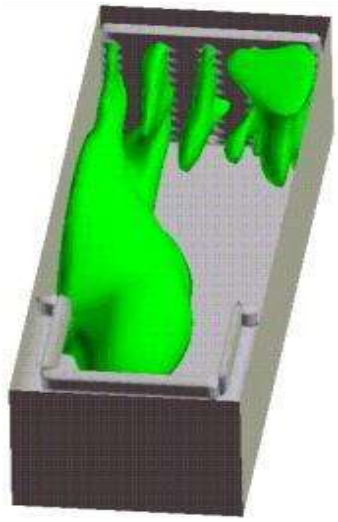


Figure 22:Short-circuiting

When water in the sedimentation basin short-circuit, floc does not have enough time to settle out of the water, influencing the economy of the plant and the quality of the treated water.

Short-circuiting in a sedimentation basin can be detected in a variety of ways. If areas of water in the basin do not appear to be circulating, or if sludge build up on the bottom of the basin is uneven, then tested may be called for. Floats or dyes can be released at the in-let of the basin to determine current. . A variety of factors causes short-circuiting in a sedimentation basin. Basin shape and design along with the design of the inlet and out-let, can cause short-circuiting. You may remember from the last lesson that a long, thin sedimentation basin is less likely to short-circuit than is a short broad one. Uneven distribution of flow either at the inlet or out -let can also cause short- circuiting. If the weir at the out-let is not level or if some of the notches clog, flow will be uneven and will cause short-circuiting.

In addition to the design of the basin, characteristic of the water can also cause short-circuiting. Differences of temperature can cause **stratification** of the water-separation of water in to bands of different temperature. In coming water will tend to flow through the band of water which corresponds to its own temperature, will not spread throughout the rest of the basin.

We should note that temperature can cause other problems with sedimentation as well. Cold water prevents floc from settling, so that longer settling times or larger doses of coagulant chemicals are needed.

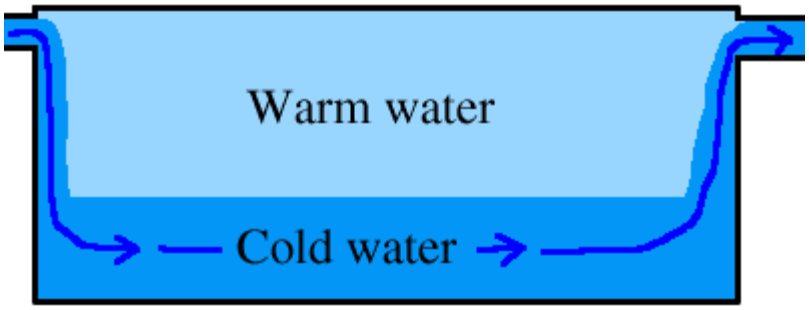


Figure 23: Effect of temperature

Other Problems

Gases in the water may cause floating scum, which can carry over into the filters. Sprinkling water on the on the scum may cause the scum to settle, but it is usually a better practice to find and fix the source of the problem. Gases in the sedimentation basin are usually caused by water being introduced in the pump or by a leak in the raw water line.

Another sedimentation basin problem is algal growth. If sedimentation basins have sufficient sunlight, algae will grow on the walls of the basin. These algae can break loose and clog the filter. Algae are best treated with **shock chlorination**, a method of feeding 5-10 ppm of chlorine into the raw water or of sprinkling HTH around the basin walls just before the plant is shut down for a few hours. The chlorine will kill the algae while the chlorinated water sits in the tank.

A few other factors can also influence sedimentation basin efficiency. Intermittent operation of the basin can cause settling problems. Also, design problems such as excessive surface loading or weir loading can cause problems. We will discuss surface and weir loading in the second half of this lesson.

Testing

In efficient operation of the sedimentation basin generally results in floc, scum, or algae carryover from the sedimentation basin to the filter. This carryover will result in clogging of filters and sometimes in degradation of the finished water quality. In order to prevent these problems, the operator should monitor the water entering, leaving, and in the sedimentation basin.

The turbidity of water entering and leaving the sedimentation basin should be tested regularly. This test is a direct measure of the efficiency of the sedimentation process in removing suspended particles from water .

The temperature of water entering the sedimentation basin should also be tested. Temperature changes, as you remember, can cause short-circuiting and can prevent floc settling. However, temperature changes are usually gradual and seldom cause sudden changes in the sedimentation process.

Finally, the operator should perform a visual survey of the sedimentation basin. He or she should note sludge and floc conditions, settling, and the clarity of the water in the outlet. If floc is being carried over the weirs, this condition is usually evident to the trained operator and can be corrected.

3.5. Designing a Rectangular Sedimentation Tank

Designing a rectangular sedimentation tank is similar in many ways to design a flocculation chamber. However, water in a sedimentation basin is not agitated, so the velocity gradient is not a factor in the calculation. Instead, two additional characteristics are important in designing a sedimentation basin.

The **overflow rate** (also known as **surface loading** or the **surface overflow rate**) is equal to the settling velocity of the smallest particle which the basin will remove. Surface loading is calculating by dividing the flow by the surface area of the tank. Overflow rate should usually be less than 1000gal/day ft^2.

The weir loading is another important factor in sedimentation basin efficiency. **Weir loading**, also known as **weir overflow rate**, is the number of gallons of water passing over a foot of weir per day. The standard weir overflow rate is 10,000 to 14,000gpd/ft and should be less than 20,000gpd/ft. Longer weirs allow more water to flow out of the sedimentation basin without exceeding the recommended water velocity.

Design of Horizontal Flow Tanks

A depth of at least 3m is provided to allow for sludge storage. For a given flow rate, the remaining variable to determine will be either the plan area or the detention time. It is possible to run settling column tests to determine the expected percentage removal for a given detention period. If a well-defined zone of clear water is established, then there will not be a need for Flocculation. A factor of 1.5 to 2 should be applied (the detention time increased by 1.5 to 2) to allow for short-circuiting caused by wind temperature differences, etc.

The following design guidelines are recommended.

Type	Description	Surface loading Rate (m/day)	Detention time (Hrs.)
A	Small installations with precarious operation	20-30	3-4
B	Installations planned with new technologies and reasonable operation	35-45	2 1/2 - 3 1/2
C	Installations planned with new technologies and good operation	35-45	2 to 3
D	Large installations with new technologies and excellent operations with provisions for adding coagulant aids whenever necessary	40-60	1 1/2 -2 1/2

Inlet Arrangement

The flow should be distributed as uniformly as possible beyond the inlet zone. Weirs, Orifices as well as perforated baffles can be provided to induce uniform distribution. (Diagram below).

Design of perforated baffles should try to satisfy the following criteria:

1. The velocity through the ports should be 4 times higher than any approaching velocities in order to equalize flows.
2. To avoid braking up flocs, the velocity gradient should be held to a value close to or a little higher than that in the last Flocculator.
3. The maximum feasible number of ports should be provided to minimize the length of the turbulent entry zone.
4. The port configuration should be such as to assure that the discharge jets will direct the flow towards the basin outlet.

Outlet Arrangement

The outlet weir length should be long enough to reduce the turbulence at outlet from travelling back to the settlement zone. The following formula can be used.

$L = 0.2Q / (H.Vs)$

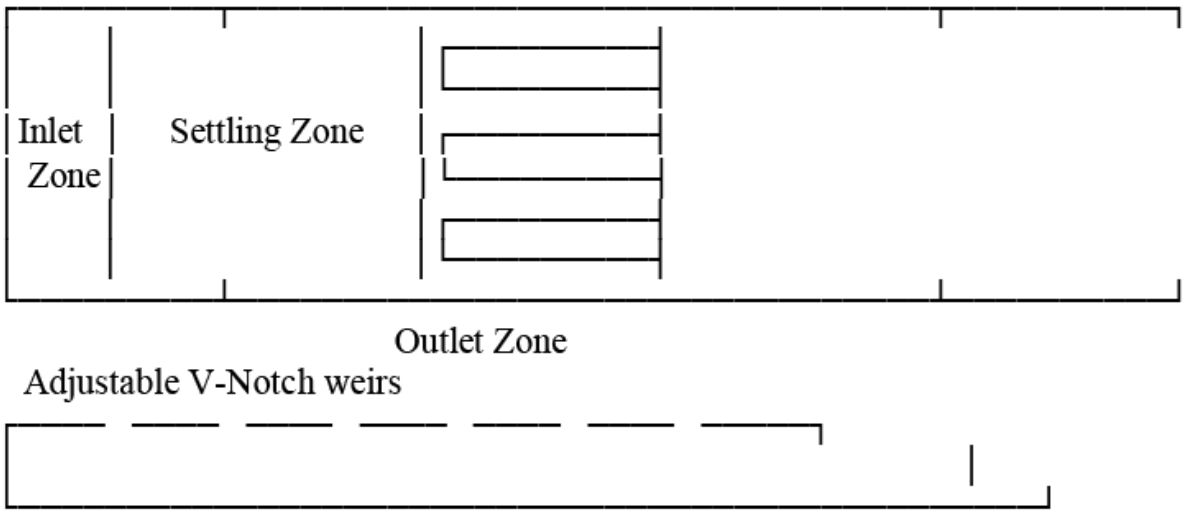
Where:

H = Depth of the tank

Q = Flow rate

Vs = Settling velocity (m/day)

L = Combined weir length



Specifications

The sedimentation basin we will design in this lesson will be a rectangular sedimentation basin with the following specifications:

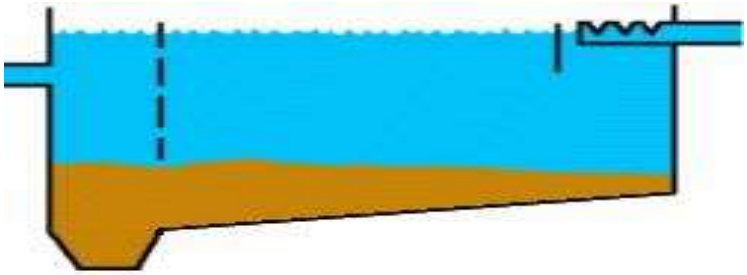


Figure 24: a rectangular sedimentation basin

- Rectangular basin
- Depth:7-16 ft
- Width:10-50ft
- Length 4mwidth
- Influent baffle to reduce the flow momentum
- Slope of bottom to ward sludge hopper >1%
- Continuous sludge removal with a scraper velocity <15ft/min
- Detention time:4-8 hr
- Flow through velocity :0.5ft/min
- Overflow rate:500-1,000gal/day-ft^2
- Weir loading:15,000-20,000gal/day-ft

Over view of calculations

We will determine the surface area, dimensions, and volume of the sedimentation tank as well as the weir length. The calculations are as follows:

1. Divide flow in to at least two tanks.
2. Calculate the required surface area.
3. Calculate the required volume.
4. Calculate the tank depth.

5. Calculate the tank width and length.

6. Check flow through velocity.

7. If velocity is too high, repeat calculations with more tanks.

8. Calculate the weir length.

I. Divide the flow

The flow should be divide in to at least two tanks and the flow through each tank should be calculated using the formula shown below:

Where:

Q_c=flow in one tank

Q=total flow

n = number of tanks

We will consider a treatment plant with a flow of 1.5MGD. We will divide the flow in to three tanks, so the flow in one tank will be:

$Q_{C=}$1.5MGD/3

$Q_{C=}$0.5MGD

II. Surface area

Next, the required tank surface area is calculated. We will base this surface area on an overflow rate of 500gal/day-ft^2 in order to design the most efficient sedimentation basin. The surface area is calculated using the following formula:

$A=Q_c/O.R.$

Where:

A=surface area,ft^2

Q_c=flow, gal/day

O.R. = overflow rate, gal/day-ft^2

In our example, the surface area of one tank is calculated as follows:

A = (500,000gal/day) (500gal/day-ft^2)

A=1,000ft^2

(Notice that we convert the flow from 0.5MGD to 500,000gal/day before beginning our calculations.)

III.volume

The tank volume is calculated just as it was for flocculation basins and flash mix chambers, by multiplying flow by detention time. The optimal detention time for sedimentation basins depends on whether sludge removal is automatic or manual. When sludge removal is manual, detention time should be 6 hours. .

We will consider a tank with automatic sludge removal, so the detention time should 4 hours. The volume of one our tanks is calculated as follows:

V=Qt

V=(500,000gal/day)(4hr)(1day/24hr)($1ft^3$/7.48gal)

V=11,141ft^3

(Notice the conversions between days and hours between cubic feet and gallons.)

IV. Depth

The tank‟s depth is calculated as follows:

d=V/A

where:

d=depth, ft

V=volume,ft^3

A= surface area, ft^2

For our example, the depth is calculated to be:

d=(11,141ft^3)/(1,000ft^2)

d = 11.1 ft

The specifications note that the depth should be between 7 and 16 feet. Our calculated depth is within the recommended range. If the depth was too large, we would begin our calculations again, using a larger number of tanks. If the depth is too shallo0w, we would use a smaller number of tanks.

V. Width and Length

You will remember that the volume of the rectangular solid is calculated as follows:

V= LWd

where: V=volume L= length W=width d=depth

For our tank, the length has been defined as follows: L= 4W

Combined these two formulas, we get the following formula used to calculate the width of our

tank: in the case of our example, the tank width

is calculated as follows:

W=15.8ft

The length is calculated as:

L = 4(15.8ft)

L= 63.2ft

6 and 7 .

Flow through velocity

Checking the flow through velocity is done just as it was for the flocculation basin. First, the cross sectional area of the tank is calculated: $A_x = Wd$

$A_x = (18.5ft)\ (11.1ft)$

$A_x = 175.4ft^2$

Then the flow through velocity of the tank is calculated (with a conversion from gallons to cubic feet and from days to minutes):

$V=Q_c/A_x$

$V = (0.0000928ft^3\text{-day/gal-min})(500{,}000gal/day)/175.4ft^2$

V=0.26ft/min

The velocity for our example is less than 0.5ft/min, so it is acceptable. As a result, we do not need to repeat our calculations.

Weir length

The final step is to calculate the required length of weir. We will assume a weir loading of 15,000gal/day-ft and use the following equation to calculate the weir length: $L_w = Q_c/W.L$

where:

L_w: weir length, ft

Q_c: flow in one tank, gal/day

W.L: weir loading, gal/day-ft

so, in our example, the weir length is calculated as follows: $L_w = (500{,}000gal/day)/(15{,}000gal/day\text{-}ft)$

$L_w = 33.3ft$

The weir length should be 33.3ft.

Design Elements

- Detention time: the time taken by the water particle to pass between entries and exist.

❖ Flow through periods (td)-the average time required for a batch of water to pass the settling tank.

Always T_d<to because of short-circuiting.

❖ Over flow rate or surface loading rate: the quantity of water passes per unit time horizontal area.

$V_s = Q/A$

❖ Basin dimension

➢ Surface area = Volume/surface loading

➢ Length to width ratio minimum of 2:1

Preferable 3:1 to 5:1

➢ Depth 2.5-5m (inclusive of allowance) Preferred value = 3m

Example 1

Find the dimension of a rectangular sedimentation basin for the following data

- Volume of treated water =$3*10^6$l/s
- velocity of flow = 10cm/min

td=4hr =240min

Solution

length of tank = velocity of flow*td

= 0.1*240=24m

X sectional area (B*H)=V/L=500/24=20.8m^2

Assuming the working depth = 3m

Width of tank = 20.8/3 = 7m

Provide 1m for sludge storage and 0.5m for free board

Total depth = 3+1.5 =4.5m

Provide the inlet zone = out-let zone =3m

Total length = 24+6 = 30m

Provide the Sedimentation tank =30*7*4.5

Example2

Design a sedimentation basin for the following data

1.1 flow rate

- maximum flow rate =10,000m^3/day
- average

flow rate 5790m^3/day

2.1 design parameters
- td

= 4hr

- L/W=2
- SOR=10m^3/m^2/d
- Design flow rate to be 25% of maximum flow rate

Solution

Determine the number of tanks

No. of tanks=100/25 = 4

Q single = 10,000/4 = 2500m^3/day

Determine the area of the sedimentation tank

Area = maximum flow rate/SOR

= 2500/10

= 250m^2

A = 1*w = 2w^2

W = 11.18m = 11m

L = A/W =23m

Determine the effective height (H) H

=SOR*td

= 1.67m

Provide the sludge and the free board zone of 0.5m

Ht=2+0.5+0.5=3m

Assume the inlet and out-let zone to be to be equal to the effective depth of the tank.

Our plant should build a sedimentation tank which is 11.1feet deep, 15.8feet wide, and 63.2feet long. This tank will have a surface area of 1,000ft^2 and a volume of 11,141ft^3. The flow through velocity will be 0.26ft/min. The weir length will be 33.3ft.

Review

Sedimentation basin efficiency is influenced by floc characteristics, water temperature, short-

circuiting, and gases in the water, algal growth on tank walls, intermittent tank operation, surface loading, and weir loading. To insure optimal performance, the operator should test turbidity and temperature of the water and should visually survey the basin

Design of a sedimentation basin involves the following steps:

- Divide flow in to at least two tanks.
- Calculate the required surface area.
- Calculate the required volume.
- Calculate the tank depth.
- Calculate the width and length.
- Check flow through velocity.
- If velocity is too high, repeat calculations with more tanks.
- Calculate the weir length.

New Formulas Used

To calculate tank surface area: $A=Q_c/O.R.$

To calculate tank depth: $d=V/A$

To calculate width of a rectangular tank where length is four times the width:

To calculate length of a rectangular tank where length is four times the width: $L=4W$ To calculate flow through velocity: $V=Q_c/A_x$

To calculate weir length; $L_w = Q_c/W.L$

COAGULATION AND FLOCCULATION In this lesson we will answer the following questions:

- How do coagulation and flocculation fit into the water treatment process?
- Which chemical principles influence coagulation and flocculation?
- Which chemicals are used in coagulation?
- What factors influence coagulation and flocculation?

4.1 Purpose

The primary purpose of the coagulation/flocculation process is the removal of turbidity from the water. Turbidity is a cloudy appearance of water caused by small particles suspended therein. Water with little or no turbidity will be clear.



Figure 25: Appearance of turbidity

Coagulation is the process by which colloidal particles and very fine solid suspensions initially present in a wastewater are combined into larger agglomerates that can be separated via sedimentation, flocculation, filtration, centrifugation or other separation methods·

Coagulation is commonly achieved by adding different types of chemicals (coagulants) to the waste water to promote destabilization of the colloid dispersion and agglomeration of the resulting individual colloidal particles.

Additional Benefits of Coagulation

1. The addition of some common coagulants to a wastewater not only produces coagulation of colloids but also typically results in the precipitation of soluble compounds, such as phosphates, that can be present in the wastewater.· In addition, coagulation can also produce the removal of particles larger that colloidal particles due to the entrapment of such particles in the flocs formed during coagulation. Coagulation vs. Flocculation

2. Although the words "coagulation" and "flocculation" are often used interchangeably they refer to two distinct processes

3. Coagulation indicates the process through which colloidal particles and very fine solid suspensions are *destabilized* so that they can begin to agglomerate if the conditions are appropriate

4. Flocculation refers to the process by which destabilized particles actually *conglomerate* into larger aggregates so that they can be separated from the wastewater.

4.2. Location in the Treatment Plant

After the source water has been screened and has passed through the optional steps of prechlorination and aeration, it is ready for coagulation and flocculation.

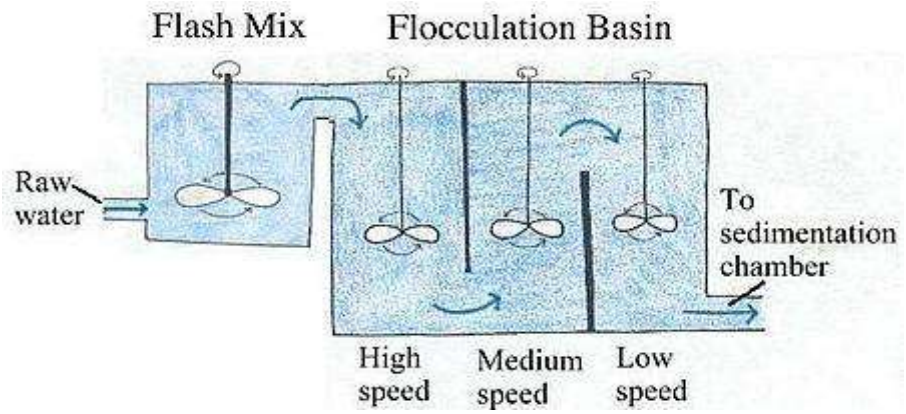


Figure 26: Flash Mixer and Flocculation Basin

In theory and at the chemical level, coagulation and flocculation is a three step process, consisting of flash mixing, coagulation, and flocculation. However, in practice in the treatment plant, there are only two steps in the coagulation/flocculation process - the water first flows into the flash mix chamber, and then enters the flocculation basin.

In this lesson, we will primarily be concerned with the theory behind coagulation /flocculation. In later lessons, we will consider the practice in more detail.

4.3. Principle of coagulation

It has been found that when certain chemicals are added to water an insoluble, gelatinous, flocculent is formed .this gelatinous precipitate during its formation and descent through the water absorb and entangle very fine suspended matter and colloidal impurities. The gelatinous precipitate therefore has the property of removing fine and colloidal particle quickly and completely than by plain sedimentation. These coagulants further the advantage of removing colour, odor and taste from the water. These coagulants if properly applied are harmless to the public.

First the coagulant are mixed in the water to produce the required precipitate ,then the water is sent in the sedimentation basins where sedimentation of fine and colloidal particle takes place through the precipitate.

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

I. Floc formation

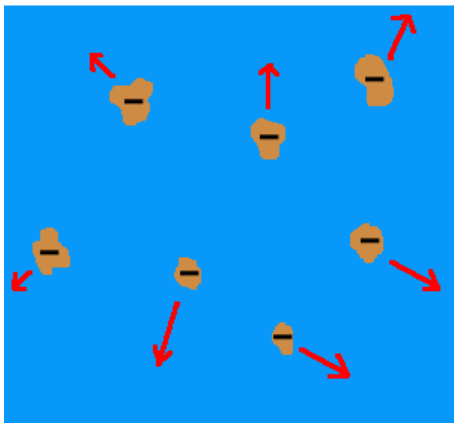
When coagulants (chemicals) are dissolved in water and thoroughly mixed with it, they produce a think gelatinous precipitate. This precipitate is known as **floc** and this floc has got the property of arresting suspended impurities in water during downward travel towards the bottom of tank. The gelatinous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

II. Electric charges

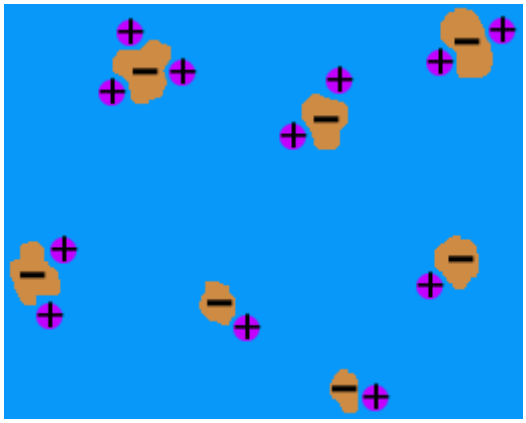
Most particles dissolved in water have a negative charge, so they tend to repel each other. As a result, they stay dispersed and dissolved or colloidal in the water.

The purpose of most coagulant chemicals is to neutralize the negative charges on the turbidity particles to prevent those particles from repelling each other. The amount of coagulant which should be added to the water will depend on the **zeta potential**, a measurement of the magnitude of electrical charge surrounding the colloidal particles. You can think of the zeta potential as the amount of repulsive force which keeps the particles in the wat er. If the zeta potential is large, then more coagulants will be needed.

Coagulants tend to be positively charged. Due to their positive charge, they are attracted to the negative particles in the water, as shown below.



charged particles repel each other due to electricity



Negatively positively charged coagulants attract to negatively charged particles due to electricity.

Figure 27: Negatively charged particle and positively charged coagulants

The combination of positive and negative charge results in a **neutral**. As a result, the particles no longer repel each other.

The next force which will affect the particles is known as van der Waal's forces. **Van der Waal's forces** refer to the tendency of particles in nature to attract each other weakly if they have no charge.

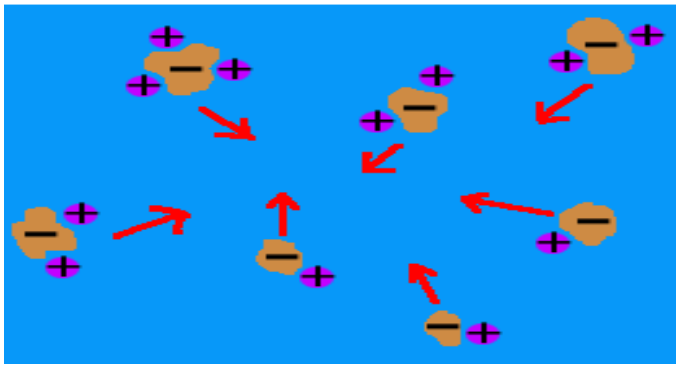


Figure 28: Neutrally charged particles attract due to van der Waal's forces.

Once the particles in water are not repelling each other, van der Waal's forces make the particles drift toward each other and join together into a group. When enough particles have joined together, they become floc and will settle out of the water.

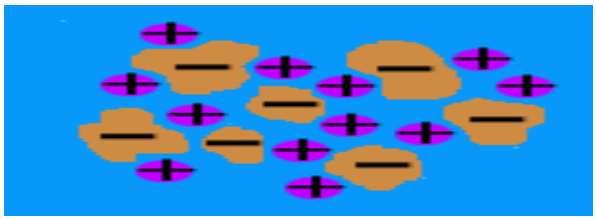


Figure 29: Particles and coagulants join together into floc.

4.3.1. Factors affecting coagulation:

1. Type of coagulant
2. Dose of coagulant
3. Characteristic of water
1. Type and quantity of suspended matter
2. Temperature of water
3. pH of water
4. Time and method of mixing

4.3.2. Common Coagulants

Coagulant chemicals come in two main types - primary coagulants and coagulant aids. **Primary coagulants** neutralize the electrical charges of particles in the water which causes the particles to clump together. **Coagulant aids** add density to slow-settling flocs and add toughness to the flocs so that they will not break up during the mixing and settling processes.

In water treatment plants, the following are the coagulants most commonly used:

i. Aluminum sulfate [$Al_2(SO_4)_3.18H2O$].

It is also called Alum. It is the most widely used chemical coagulant in water purification work. Alum reacts with water only in the presence of alkalinity. If natural alkalinity is not present, lime may be added to develop alkalinity. It reacts with alkaline water to form aluminum hydroxide (floc), calcium sulphate and carbon dioxide. Due to the formation of calcium sulphate, hardness and corrosiveness of water is slightly increased. .

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

The chemical is found to be most effective between pH range of 6.5 to 8.5. Its dose may vary from 5 to 30mg/lit, for normal water usually dose being 14mg/l. actually, dose of coagulant depends on various factors such as turbidity, colour, taste, pH value, temperature etc.

Due to the following reason, Alum is the most widely used chemical coagulant.

1. It is very cheap
2. It removes taste and color in addition to turbidity
3. It is very efficient
4. Flocs formed are more stable and heavy
5. It is not harmful to health
6. It is simple in working, doesn‟t require skilled supervision for dosing

ii. Sodium aluminates ($Na_2Al_2O_4$)

In the process of coagulation, it can remove carbonate and non-carbonate hardness. It reacts with calcium and magnesium salts to form flocculent aluminates of these elements.

Chemical reactions:

i) $Na_2Al_2O_4 + Ca(HCO_3)_2 \rightarrow CaAl_2O_4\downarrow + Na_2CO_3 + CO_2 + H_2O$

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

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

The pH should be within the range of 6 and 8.5.

iii. **Chlorinated Copperas**

Combination of Ferric sulphate and Ferric chloride. When solution of Ferrous Sulphate is mixed with chlorine, both Ferric sulphate and Ferric chloride are produced.

$6FeSO_4.7H_2O + 3Cl_2 \longrightarrow 2Fe_3(SO_4)_2 + 2FeCl_3 + 42H_2O$

Ferric sulphate and Ferric chloride each is an effective floc and so also their combination. Both Ferric sulphate and Ferric chloride can be used independently with lime as a coagulant If alkalinity is insufficient, lime is added.

Chemical reaction taking place

$2FeCl_3 + 3Ca(OH)_2 \rightarrow 2Fe(OH)_3\downarrow + CaCl_2$

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

Ferric chloride effective pH range 3.5 – 6.5 or above 8.5 and Ferric sulphate is effective with pH range of 4 – 7 or above 9.

iv. Poly electrolytes

They are special types of polymers. They may be anionic, cationic, and non-ionic depending upon the charge they carry. Out of these only cationic poly electrolytes can be used independently.

Example

Find out the quantity of alum required to treat 18million liters of water per day. The dosage of alum is 14mg/lit. Also work out the amount of CO_2 released per liter of treated water.

Solution

Quantity of alum per day=($14*18*10^6$)/ 10^6=252kg

The chemical reaction as follow:

$Al_2(SO_4)18H_2O+3Ca(HCO_3)_2 \longrightarrow 2Al(OH)_3+3CaSO_4+18H_2O+6CO_2$

Molecular weight of alum:

=2*26.97+3*32.066+36*1.008+30*16

=666

Molecular weight of CO_2=(1*12.0)+2*16)

= 44

Thus

666mg of alum release 6*44mg of CO_2

14mg of alum will releases=(14*6*44)/666

=5.55mg of CO_2

Exercise

A water treatment plan uses Ferrous sulphate and lime as a coagulant at the rate of 10mg/l of water .Find out the quantity of Ferrous sulphate and the required to treat 27 million liters of water.

$FeSO_4 7H_2O+Ca(OH)_2 \longrightarrow Fe(OH_2)+ CaSO_4+7H_2O+Ca(OH)_2+CaO+H_2O$

4.3.3. Feeding of coagulant

In order to feed chemicals to the water regularly and accurately, some type of feeding equipment must be used.

Coagulants may be put in raw water either in powder form or in solution form.

I. Dry-feed Type

Dry powder of coagulant is filled in the conical hopper. The hoppers are fitted with agitating plates which prevent the chemical from being stabilized. Agitating plates are used to prevent arching of chemicals. Feeding is regulated by the speed of toothed wheel or helical spring (fig 29). Activated carbon and lime are added to raw water in powder form.

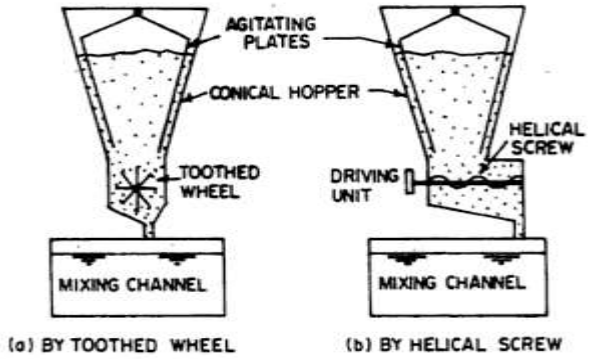


Figure 30: Dry feeding devices

II. Wet feeding type

First, solution of required strength of coagulant is prepared. The solution is filled in the tank and allowed to mix in the mixing channel in required proportion to the quantity of water. It can be easily controlled with automatic devices.

4.3.4. Mixing devices

The process of floc formation greatly depends upon the effective mixing (rapid mixing) of coagulant with the raw water.

Rapid mixing of the mixture of coagulant and raw water is used to:

- Disperse chemicals uniformly throughout the mixing basin
- Allow adequate contact between the coagulant and particles
- Formation of microflocs

The mixing is done by mixing device.

1. Hydraulic jump - flume with considerable slope is developed
2. Pump method - centrifugal pump is used to raise raw water
3. Compressed air method – compressed air is diffused from bottom of the mixing tank
4. Mixing channels

Mixing of raw water and coagulant is made to pass through the channel in which flume has been done. Vertical baffles are also fixed at the end of the flumed part on both sides of the channel (fig 6).

5. Mixing basin with baffle wall
6. Mechanical mixing basins

Mechanical means are used to agitate the mixture to achieve the objective of thorough mixing. Flash mixers and deflector plate mixers are used.

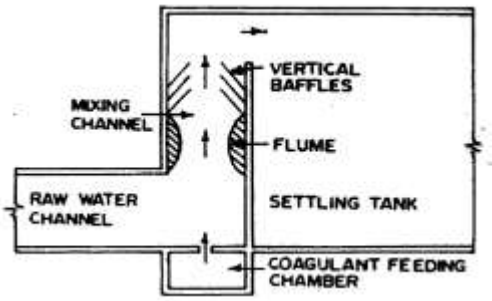


Figure 31: Mixing channel

A. Flash mixer

The mixing of coagulant in water is achieved by rotating vigorously fans fixed in the mixing basin. The deflecting wall avoids short circuiting and deflects the water flow towards the fan blades. Chemical pipe discharges the coagulant just near the rotating fan (fig 31).

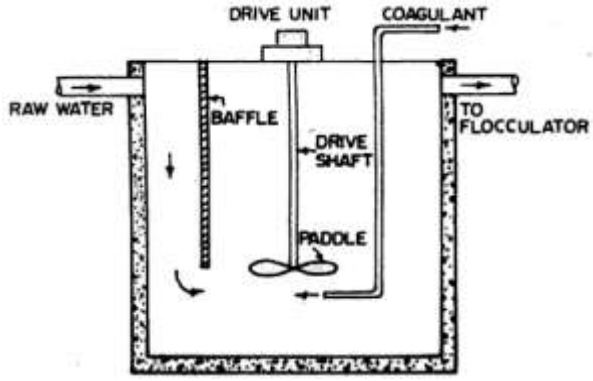


Figure 32: Flash mixer

Design criteria of flash mixer:

1. Detention period – 30 to 60 sec
2. Velocity of flow – 0.9m/sec
3. Depth – 1 to 3m
4. Rotation per minute of blade – 100
5. Power required – 0.041kW/1000m^3/day

B. Deflector plate mixer

Mixing is achieved by diffusing water through a deflection plate. Water enters from inlet pipe and comes out through a hole provided below the deflector plate where it gets agitated. Chemical pipe discharges the coagulant just near the deflector plate, where it gets thoroughly mixed with water (fig 32).

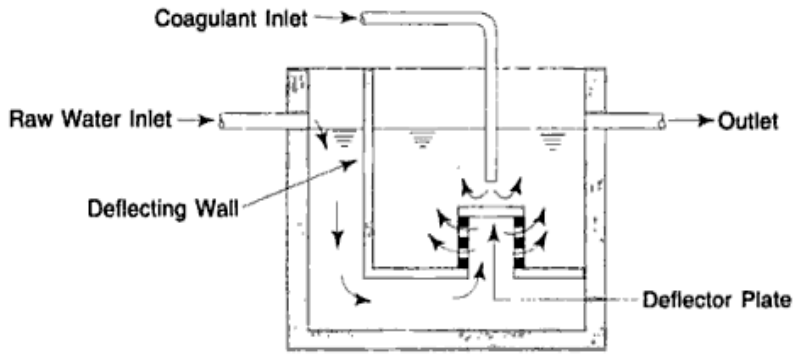


Figure 33 : Deflector plate mixer

4.4. Flocculation

After adding the coagulant to the raw water, rapid agitation is developed in the mixture to obtain a thorough mixing. Next to rapid mixing, mixture is kept slowly agitated for about 30 to 60min. Slow mixing process in which particles are brought into contact in order to promote their agglomeration is called *flocculation.* The tank or basin in which flocculation process is carried out is called *flocculation chamber***.** The velocity of flow in the chamber is kept between 12 – 18cm/sec. Activated carbon in powder form can be used to speed up the flocculation

The rate of agglomeration or flocculation is dependent upon

- Type and concentration of turbidity
- Type of coagulant and its dose
- Temporal mean velocity gradient – G in the basin

The mean velocity gradient is the rate of change of velocity per unit distance normal to the section - (meter per second per meter) (T^{-1}). The value of G can be computed in terms of power input by the following equation

(7)

Where P – power dissipated (watt)

μ - absolute viscosity (Ns/m^2)

V - the volume to which P is applied (m^3) G -

temporal mean velocity gradient (s^{-1})

The flocculation technique most commonly used involves mechanical agitation with rotating paddle wheels or vertical mounted turbines (fig 33).

The design criteria of a horizontal continuous flow rectangular basin flocculator:

(*i*)	Depth of tank	: 3 to 4.5 m
(*ii*)	Detention time	: 10 to 40 min. ; normal : 30 min.
(*iii*)	Velocity of flow	: 0.2 to 0.8 m/s ; normal 0.4 m/s
(*iv*)	Total area of paddles	: 10 to 25% of the cross-sectional area of the tank
(*v*)	Peripheral velocity of blades	: 0.2–0.6 m/s ; normal 0.3 to 0.4 m/s
(*vi*)	Velocity gradient (*G*)	: 10 to 75 s^{-1}
(*vii*)	Factor *G.t*	: 10^4 to 10^5
(*viii*)	Power consumption	: 10.0 to 36.0 kW/mld
(*ix*)	Outlet flow velocity	: 0.15 to 0.25 m/s.

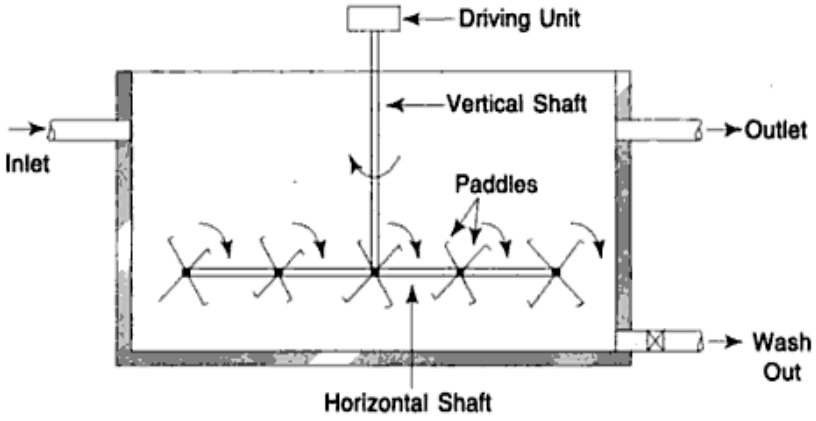


Figure 34: Flocculator

4.4.1. Clarifier (Secondary Sedimentation)

After flocculation, water enters the settling tank which is normally called a *clarifier*. Water is retained in the sedimentation tank for a sufficient period to permit the settlement of the floc to the bottom. The principle of design of clarifier is the same as for plain sedimentation basin except that its detention period is lower. The detention period commonly adopted is 2.5 to 3hrs with an overflow rate of 1 to 1.2 m/hr.

Section of sedimentation unit consisting of flash mixer, flocculator and settling basin (fig 34)

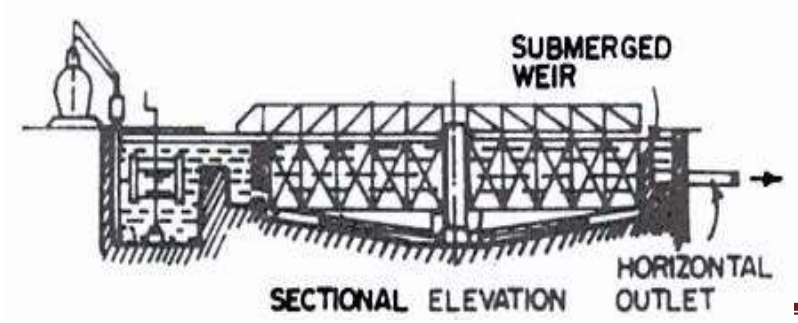


Figure 35: Section of secondary sedimentation unit

Flocculent settling

Flocculent particles resulting from coagulation will agglomerate while settling with a resultant increase in particle size. The density of the composite particle will decrease due inclusion of water, however, the settling velocity will increase. (0.1 to 3mm best floc size)

The clarification of dilute suspensions of flocculating particle is a function of:

- Settling property of the particles
- Flocculating characteristic of the suspension

Example1

Design a conventional vertical-shaft rapid mix tank unit for uniformly dispersing coagulant in 10 MLD of settled raw water as per design parameters given below:

Detention time (t):	20 – 60 s
Ratio of tank height (H) to diameter (D):	(1:1 to 1:3)
Ratio of impeller diameter (D_I) to tank diameter (D):	(0.2:1 to 0.4:1)
Velocity gradient (G):	>300 /s
Gt:	10000 – 20000
Tank diameter (D):	< 3 m
Paddle tip speed (v_p):	1.75 – 2.0 m/s
Velocity of paddle relative to water (v):	0.75 x paddle tip speed
Paddle area (A_p)/Tank section area (A_T):	10:100 – 20:100
Coefficient of drag on impeller blade (C_D):	1.8
Maximum length of each impeller blade (L):	0.25 x impeller diameter
Maximum width of impeller blade (B):	0.20 x impeller diameter
Impeller height from bottom (H_B):	1.0 x impeller diameter
Kinematic viscosity (ν):	1.003×10^{-6} m^2/s
Dynamic viscosity of water (μ):	1.002×10^{-3} $N.s/m^2$

Determine tank dimensions (provide a freeboard of 0.5 m), impeller diameter, paddle dimensions, number of paddles, clearance of the impeller from tank bottom, paddle rotation speed and power input requirement.

Solution:

Let the detention time (t) be 40 s.

Therefore, volume of tank (V) = $\frac{40}{60.60.24}.10000 = 4.63\text{m}^3$

Let the tank diameter (D) be 2 m

Tank cross-sectional area (A_{cs}) = 3.14 m^2

Tank height (H) = 1.47 m, provide freeboard of 0.5 m

Total height (H_T)= 1.97 m, say 2 m.

Tank height (H) to tank diameter (D) ratio = $\frac{1.47}{2} = 0.736$

(within the range of 0.33–1.0, hence okay)

Let velocity gradient, G = 400 s^{-1}

Therefore, G.t = (400).(40) = 16000 (within the limit of 10000-20000, hence okay)

Let the paddle tip speed (v_p) be 1.8 m/s

Hence, velocity of paddle relative to water (v) = 0.75.(1.8) = 1.35 m/s

$$G = \left\{\frac{C_d.A_p.v^3}{2.\nu.V}\right\}^{1/2}, \quad \text{or,} \quad 400 = \left\{\frac{1.8.(A_p).(1.35)^3}{2.(1.003x10^{-6}).(4.63)}\right\}^{1/2}$$

or, A_p = 0.335 m^2

Tank sectional area = (D).(H) = (1.47).(2) = 2.94 m^2

$$\frac{\text{Paddle Area } (A_p)}{\text{Tank Sectional Area } (D.L)} = \frac{0.335}{2.94} = 0.1139$$

, which is around 0.15, hence okay.

Let the impeller diameter be 0.8 m, i.e., $\left(\frac{D_I}{D} = \frac{0.8}{2} = 0.4\right)$, which is okay

Choose **length of each impeller blade (L) as 0.20m**, i.e., $\left(\frac{L}{D_I} = \frac{0.2}{0.8} = 0.25\right)$ (okay) Choose **breadth of each impeller blade (B) as 0.15m**, i.e., $\left(\frac{B}{D_I} = \frac{0.15}{0.8} = 0.187\right)$ (okay)

Area of each blade = (L.B) = (0.2).(0.15) = 0.03 m^2

Therefore, number of blades to be provided = $\frac{0.335}{0.03} = 11.17$ **, say 12**

Clearance of the paddles from the tank bottom = 0.8 m

Paddle rotation speed (w, radians/s) = $\frac{v_p.2}{D_I} = \frac{1.8.(2)}{0.8} = 4.5$ **radians/s,**

i.e., $\frac{4.5}{2.\pi}.60 = 43$ **revolutions per minutes**

Power requirement is given by, $G = \left[\frac{P}{V.\mu}\right]^{1/2}$ **, or,** $400 = \left[\frac{P}{4.63.(1.002x10^{-3})}\right]^{1/2}$ **or,**

P = 742 Watts, i.e., provide 1 KW motor for driving the impeller at 45 rpm.

Exercise:

1) Design a settling tank (coagulation–sedimentation) with continuous flow for treating water for a population of 48,000 persons with an average daily consumption of 135lit/head. Take detention period of 3hrs and maximum day factor of 1.8.

2) Design a conventional rectangular horizontal-shaft flocculation tank unit for 10 MLD of settled raw water after coagulant addition and rapid mixing as per design parameters given below:

Detention time (t):	10 – 30 minutes
Velocity gradient (G):	20 – 75 /s
Gt:	$2 \times 10^4 - 6 \times 10^4$
Tank Depth (D):	< 5 m
Paddle tip speed (v_p):	0.25 – 0.75 m/s
Velocity of paddle relative to water (v):	0.75 x paddle tip speed
Paddle area (A_p)/Tank section area (A_T):	10:100 to 20:100
Coefficient of drag on paddle blade (C_D):	1.8
Maximum length of each paddle (l):	5.0 m
Maximum width of each paddle (b):	0.50 m
Kinematic viscosity :	1.003×10^{-6} m^2/s
Dynamic viscosity of water :	1.002×10^{-3} $N.s/m^2$
Freeboard:	0.50 m

Draw a net sketch of the designed tank (top and front view) clearly showing tank dimensions, paddle shaft position, paddle blade dimensions, water level, etc. Also mention paddle rotation speed and power requirement.

FILTRATION

5.1 Theory of Filtration

The effluent obtained after coagulation does not satisfy the drinking water standard and is not safe. So it requires further treatments. Filtration is one of the water purification process in which water is allowed to pass through a porous medium to remove remaining flocs or suspended solids from the previous treatment processes.

Filtration process assist significantly by reducing the load on the disinfections process, increasing disinfection efficiency.

Filtration consists of passing water through a thick layer of sand. During the passage of water through sand, the following effects take place.

i) Suspended matter and colloidal matter are removed
ii) Chemical characteristic of water get changed
iii) Number of bacteria considerably reduced.

These phenomena can be explained on the basis of the following mechanisms of filtration.

I. Mechanical straining – Mechanical straining of suspended particles in the sand pores.

II. Sedimentation and Adsorption

- The interstices between the sand grains act as sedimentation basins in which the suspended particles smaller than the voids in the filter-bed settle upon the sides of the sand grains.
- The particles stick on the grains because of the physical attraction between the two particles of matter and the presence of the gelatinous coating formed on the sand grains by the previously deposited bacteria and colloidal matter.

III. Electrolytic action

Due to the friction between medium and suspended solids, certain amount of dissolved and suspended matter is ionized. Suspended matter in water is ionized, carries charge of one polarity and the particles of sand in filter which are also ionized, possess electrical charges of opposite polarity. These neutralize each other; change the chemical character of water.

IV. Biological Action

The growth and life process of the living cells, biological metabolism. The surface layer gets coated with a film in which the bacterial activities are the highest and which feed on the organic impurities. The bacteria convert organic impurities by a complex biochemical action into simple, harmless compounds – purification of water.

5.2. Types of filters

Two types of filter:

1. Gravity filter system

i. Slow Sand Filter (SSF)

ii. Rapid Sand Filter (RSF)

2. Pressure filter system

5.2.1. Slow Sand Filters

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

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

Construction

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

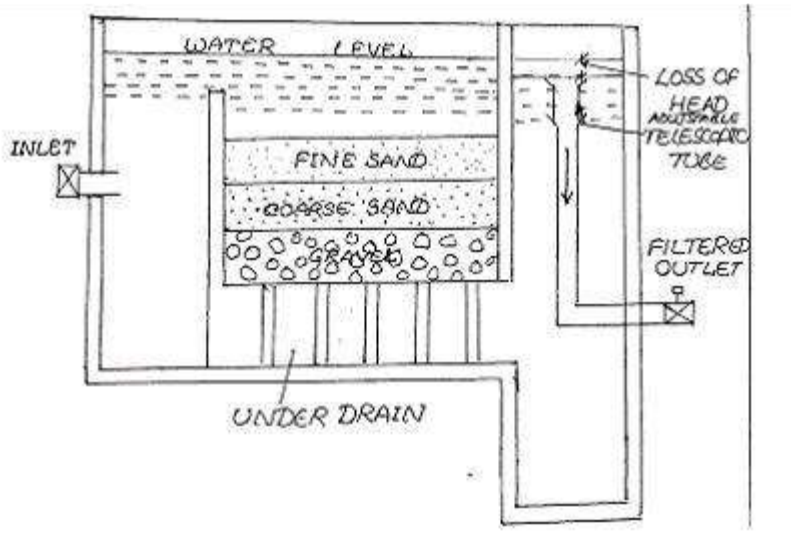


Figure 36: Slow sand filter

Operation

The water from sedimentation tanks enters the slow sand filter through a submersible inlet as shown in fig 11. This water is uniformly spread over a sand bed without causing any disturbances. The water passes through the filter media at an average rate of 2.4 to $3.6m^3/m^2/day$. This rate of filtration is continued until the difference between the water level on the filter and in the inlet chamber is slightly less than the depth of water above the sand. The difference of water above the sand bed and in the outlet chamber is called the loss of head. During filtration as the filter media gets clogged due to the impurities, which stay in the pores, the resistance to the passage of water and loss of head also increases. When the loss of head reaches 60cm, filtration is stopped and about 2 to 3cm from the top of bed is scrapped and replaced with clean sand before putting back into service to the filter.

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

Uses

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

Maintenance

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

5.2.2. Rapid Sand Filter

The rapid sand filter differs from the slow sand filter in a variety of ways, the most important of which are the much greater filtration rate ranging from 100 to 150m^3/m^2/day, the ability to clean automatically using backwashing and require small filter area. The mechanism of particle removal also differs in the two types of filters - rapid sand filters do not use biological filtration and depend primarily on adsorption and some straining.

The main features of rapid sand filter are as follows

Effective size of sand - 0.45 to 0.70mm
Uniformity coefficient of sand - 1.2 to 1.7
Depth of sand - 60 to 75cm
Filter gravel - 2 to 50mm size
(Increase size towards bottom) Depth of gravel - 45cm
Depth of water over sand
during filtration - 1 to 2m
Overall depth of filter
including 0.5m free board - 2.5m
Area of single filter unit - 100m^2 in two parts of each 50m^2
Loss of head - Max 1.8 to 2.0m
Turbidity of filtered water - 1 NTU

Operation

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

Washing of Filter

Washing of filter is done by the back flow of water through the sand bed as shown in Fig 12. First the value „V_1" is closed and the water is drained out from the filter leaving a few centimeter depth of water on the top of sand bed. Keeping all values closed the compressed air is passed through the separate pipe system for 2-3 minutes, which agitates the sand bed and stirrer it well causing the loosening of dirt, clay etc. inside the sand bed.

Now value „V_4" and „V_5" are opened gradually, the wash water tank, rises through the laterals, the strainers gravel and sand bed. Due to back flow of water the sand expands and all the impurities are carried away with the wash water to the drains through the channels, which are kept for this purpose.

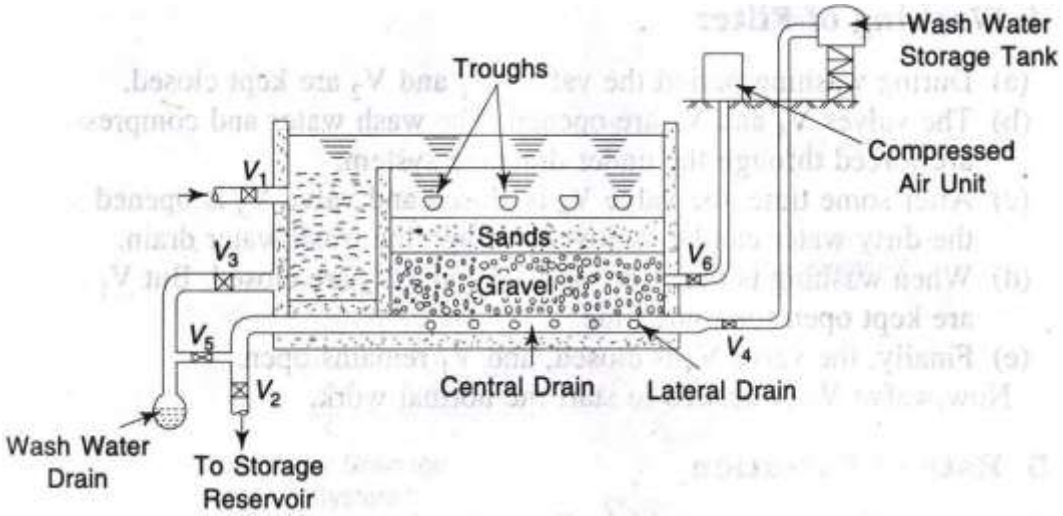


Figure 37: Rapid sand filter

Construction

Rapid sand filter consists of the following five parts:

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

Washing process is continued till the sand bed appears clearly. The washing of filter is done generally after 24 - 48hours and it takes 10 minutes and during back washing the sand bed expands by about 50%.

Rapid sand filter bring down the turbidity of water to 1 N.T.U. This filter needs constant and

skilled supervision to maintain the filter gauge, expansion gauge and rate of flow controller and periodical backwash.

Table 8: Comparison of slow sand filter and rapid sand filter

Sl.No.	ITEM	S.S.F	R.S.F
1.	Area	Need very large area	Needs small area
2.	Raw Water Turbidity	Not more than 30 NTU	Not more than 10NTU hence needs coagulation
3.	Sand Media	Effective size 0.2 to 0.3 mm uniformity coefficient 2 to 3 single layer of uniform size	Effective size 0.45 to 0.7 mm uniformity coefficient 1.3 to 1.7 multiple graded layers of sand.
4.	Rate of Filtration	2.4 to 3.6m^3/m^2/day	100-150 m^3/m^2/day
5.	Loss of Head	0.6m to 0.7 m	1.8m to 2.0m
6.	Supervision	No skilled supervision is required	Skilled supervision is required
7.	Cleaning of Filter	Scraping of 21/2cm thick layer washing and replacing. Cleaning interval that is replacement of sand at 1 to 2 months.	Back wash with clean water under pressure to detach the dirt on the sand. Backwashing daily or on alternate days.
8.	Efficiency	Bacterial removal, taste, odour, colour and turbidity removal.	There is no removal of bacteria. Removal colour taste, odour and turbidity is good.

i. Pressure Filter

Pressure filter is type of rapid sand filter in closed water tight cylinder through which the water passes through the sand bed under pressure. All the operation of the filter is similar to rapid gravity filter; expect that the coagulated water is directly applied to the filter without mixing and flocculation. These filters are used for industrial plants but these are not economical on large scale.

Pressure filters may be vertical pressure filter and horizontal pressure filter. Backwash is carried by reversing the flow with values. The rate of flow is 120 to 300m^3/m^2/day.

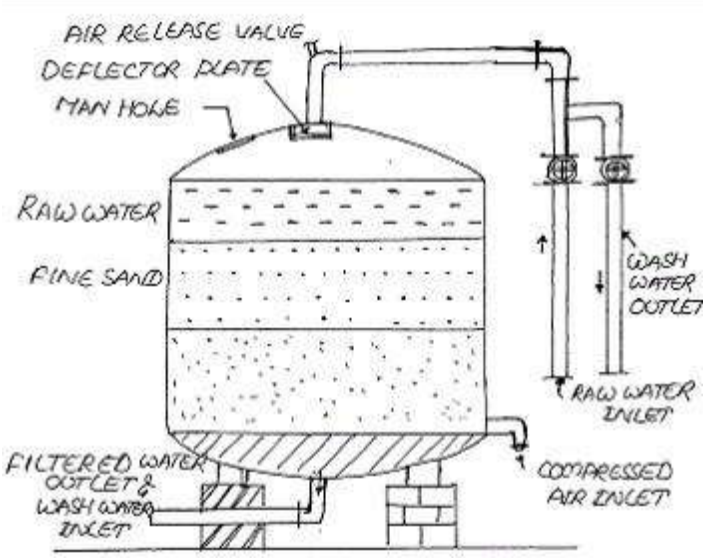


Figure 38: Vertical Pressure filters

Example1

10 MLD of water after secondary sedimentation (average turbidity: 10 NTU) is to be filtered through a battery of rapid sand filters to reduce water turbidity to < 2.5 NTU. Based on pilot plant studies, it was determined that 60 cm filter beds of sand (0.5 mm average sand diameter) were suitable for this purpose. It was further determined that such beds could be operated for 7.5 hours at a filtration rate of 10 $m^3/m^2/hr$ before the terminal head-loss of 3 m was reached. Filter backwashing rate was 1 $m^3/m^2/min$ and the backwash time was 5 minutes. A filter unit will be off-line for 30 minutes during each backwash operation. Based on this information, determine the numbers of filter units to be provided and dimensions of each unit. Determine how much filtered water is required for backwashing each day and hence determine the filtered water production per day.

Solution:

Nominal filtration rate: 10 $m^3/m^2/h$

Filter is off-line for 1.5 hours every 24 hours

Therefore, effective filtration rate = $\left(\frac{22.5}{24}\right).10 = 9.375$ $m^3/m^2/h$

Therefore, required filter cross-sectional area = $\frac{10.(1000)}{24.(9.375)} = 44.44$ m^2

Let two filters be provided for this purpose

Let the length of each filter be 5.5 m and width 4.1 m (length: width = 1.34)

Therefore total filter cross-section area provided = 2.(5.5).(4.1) = 45.1 m^2

Corrected actual filtration rate = $\frac{10.(1000)}{24.(45.1)} = 9.239$ $m^3/m^2/h$

Filtered water required for backwashing = 1.(5).(45.1) .3 = 676.5 m^3/d

Hence total filtered water production = 10 – 0.676 = 9.324 MLD

Example2

Design a rapid sand filter to treat 10 million litres of raw water per day allowing 0.5% of filtered water for backwashing. Half hour per day is used for bakwashing. Assume necessary data.

Solution:

Total filtered water = $\frac{10.05 \times 24 \times 10^6}{24 \times 23.5} = 0.42766$ Ml / h

Let the rate of filtration be 5000 l / h / m^2 of bed.

Area of filter = $\frac{10.05 \times 10^6}{23.5} \times \frac{1}{5000} = 85.5$ m^2

Provide two units. Each bed area 85.5/2 = 42.77. L/B = 1.3; $1.3B^2 = 42.77$

B = 5.75 m ; L = 5.75 x 1.3 = 7.5 m

Assume depth of sand = 50 to 75 cm.

Under drainage system:

Total area of holes = 0.2 to 0.5% of bed area.

Assume 0.2% of bed area = $\frac{0.2}{100} \times 42.77 = 0.086$ m^2

Area of lateral = 2 (Area of holes of lateral)

Area of manifold = 2 (Area of laterals)

So, area of manifold = 4 x area of holes = 4 x 0.086 = 0.344 = 0.35 m^2 .

Diameter of manifold = $(4 \times 0.35 / \pi)^{1/2}$ = 66 cm

Assume c/c of lateral = 30 cm. Total numbers = 7.5/ 0.3 = 25 on either side.

Length of lateral = 5.75/2 - 0.66/2 = 2.545 m.

C.S. area of lateral = 2 x area of perforations per lateral. Take dia of holes = 13 mm

Number of holes: $\frac{n \pi (1.3)^2}{4} = 0.086 \times 10^4 = 860$ cm^2

$\therefore n = \frac{4 \times 860}{\pi (1.3)^2} = 648$, say 650

Number of holes per lateral = 650/50 = 13

Area of perforations per lateral = 13 x $(1.3)^2 /4 = 17.24$ cm^2

Spacing of holes = 2.545/13 = 19.5 cm.

C.S. area of lateral = 2 x area of perforations per lateral = 2 x 17.24 = 34.5 cm^2.

$\therefore$ Diameter of lateral = $(4 \times 34.5/\pi)^{1/2}$ = 6.63 cm

Check: Length of lateral < 60 d = 60 x 6.63 = 3.98 m. l = 2.545 m (Hence acceptable).

Rising washwater velocity in bed = 50 cm/min.

Washwater discharge per bed = (0.5/60) x 5.75 x 7.5 = 0.36 m^3/s.

Velocity of flow through lateral = $\frac{0.36}{\text{Total lateral area}} = \frac{0.36 \times 10^4}{50 \times 34.5} = 2.08$ m/s (ok)

Manifold velocity = $\frac{0.36}{0.345} = 1.04$ m/s < 2.25 m/s (ok)

Washwater gutter

Discharge of washwater per bed = 0.36 m^3/s. Size of bed = 7.5 x 5.75 m.

Assume 3 troughs running lengthwise at 5.75/3 = 1.9 m c/c.

Discharge of each trough = Q/3 = 0.36/3 = 0.12 m^3/s.

$Q = 1.71 \times b \times h^{3/2}$

Assume b =0.3 m

$h^{3/2} = \frac{0.12}{1.71 \times 0.3} = 0.234$

$\therefore$ h = 0.378 m = 37.8 cm = 40 cm

= 40 + (free board) 5 cm = 45 cm; slope 1 in 40

Clear water reservoir for backwashing

For 4 h filter capacity, Capacity of tank = $\frac{4 \times 5000 \times 7.5 \times 5.75 \times 2}{1000} = 1725\ m^3$

Assume depth d = 5 m. Surface area = 1725/5 = 345 m^2

L/B = 2; $2B^2 = 345$; B = 13 m & L = 26 m.

Dia of inlet pipe coming from two filter = 50 cm.

Velocity <0.6 m/s. Diameter of washwater pipe to overhead tank = 67.5 cm.

Air compressor unit = 1000 l of air/ min/ m^2 bed area.

For 5 min, air required = 1000 x 5 x 7.5 x 5.77 x 2 = 4.32 m^3 of air.

Exercise

1) Design a slow sand filter for a community of 40,000 population. The per capita water demand of the water supply is 180lit/capita per day and the rate of filtration is 150lit/m^2/hr. MDF = 1.8.

2) Design a rapid sand filter to treat water for 240,000 population, the per capital water consumption of the town is 200lit/head/day. The filter works all the 24hrs. Assume rate of filtration as 100lit/m^2/min and maximum day factor is 1.8.

3) Determine the percentage of filtered water required for wash water based on the following criteria:

Flow, Q_f = 300 L/s

Rate of filtration, V_{of} = 170m^3/m^2/day

Time of washing = 10 min

Rate of washing, V_{oBW} = 15 mm/s

CHAPTER SIX: DISINFECTION

In this lesson we will answer the following questions:

- What disinfection requirements must be met in treating drinking water?
- How does chlorination fit into the water treatment process?
- How does chlorination work chemically?
- What factors influence the efficiency of chlorination?
- What equipment is used for chlorination?
- What other methods can be used to disinfect water?

6.1 Disinfection

Before water treatment became common, waterborne diseases could spread quickly through a population, killing or harming hundreds of people. The table below shows some common, water-transmitted diseases as well as the organisms (**pathogens**) which cause each disease.

Pathogen	**Disease Caused**
Bacteria:	
Anthrax	anthrax
Escherichia coli	*E. coli* infection
Myobacterium tuberculosis	tuberculosis
Salmonella	salmonellosis, paratyphoid
Vibrio cholerae	Cholera
Viruses:	
Hepatitis Virus	Hepatitis A
Polio Virus	Polio
Parasites:	
Cryptosporidium	Cryptosporidiosis

Giardia lamblia	giardiasis

The primary goal of water treatment is to ensure that the water is safe to drink and does not contain any disease-causing microorganisms. The best way to ensure pathogen-free drinking water is to make sure that the pathogens never enter the water in the first place. However, this may be a difficult matter in a surface water supply which is fed by a large watershed. Most treatments plants choose to remove or kill pathogens in water rather than to ensure that the entire watershed is free of pathogens.

Pathogens can be removed from water through physical or chemical processes. You may remember that some previously discussed treatment processes, notably sedimentation and filtration, can remove a large percentage of bacteria and other microorganisms from the water by physical means. Storage can also kill a portion of the disease-causing bacteria in water.

This lesson will be concerned with **disinfection**, which is the process of selectively destroying or inactivating pathogenic organisms in water, usually by chemical means. Disinfection is different from **sterilization**, which is the complete destruction of all organisms found in water and which is usually expensive and unnecessary. Disinfection is a required part of the water treatment process while sterilization is not.

6.2. Location in the Treatment Process

During prechlorination, chlorine is usually added to raw water after screening and before flash mixing. Postchlorination, in contrast, is often the last stage in the treatment process. After flowing through the filter, water is chlorinated and then pumped to the clear well to allow a sufficient contact time for the chlorine to act. From the clearwell, the water may be pumped into a large, outdoor storage tank such as the one shown below. Finally, the water is released to the customer.

6.3. Requirements of Good Disinfectant

1) Destroy bacteria/pathogens within a practicable period of time, over an expected range of water temperature.
2) Effective at variable compositions, concentration and conditions of water treated.

3) Neither toxic to humans and domestic animals nor unpalatable or otherwise objectionable in required concentration.

4) Not change water properties

5) Have residual in a sufficient concentration to provide protection against recontamination

6) Can be determined easily, quickly, and preferably automatically.

7) Dispensable at reasonable cost

8) Safe and easy to store, transport, handle and supply

9) Not form toxic by-products due to their reactions with any naturally occurring materials in water.

6.4. Methods of Disinfection

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

a) Boiling of water
b) Ultra–Violate rays c) Iodine and bromine d) Ozone O_3
e) Excess lime
f) Potassium permanganate [$KMnO_4$]
g) Chlorine

In most situations chlorine is selected as one of the disinfecting agent that most closely satisfies the requirement of chemical disinfectants namely:

(i) Quick and effective at killing micro-organisms

(ii) Readily soluble at the concentration needed for disinfection

(iii) Tasteless and odourless at the concentration required (iv) Non-toxic to human life at the concentration required (v) Easy to handle, transport and apply

(vi) Easy to detect and concentration easy to measure (vii)Capable of providing protection against later contamination. (viii) Readily available

(xi) Cheap

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

6.4.1. Chlorination

Chlorination is the application of chlorine to water to accomplish some definite purpose. In this lesson, we will be concerned with the application of chlorine for the purpose of disinfection, but you should be aware that chlorination can also be used for taste and o dor control, iron and manganese removal, and to remove some gases such as ammonia and hydrogen sulfide.

Chlorination is currently the most frequently used form of disinfection in the water treatment field. However, other disinfection processes have been developed. These alternatives will be discussed at the end of this lesson.

Chlorination Chemistry

When chlorine is added to water, a variety of chemical processes take place. The chlorine reacts with compounds in the water and with the water itself. Some of the results of these reactions (known as the **chlorine residual**) are able to kill microorganisms in the water. In the following sections, we will show the chemical reactions which occur when chlorine is added to water.

Chlorine Demand

When chlorine enters water, it immediately begins to react with compounds found in the water. The chlorine will react with organic compounds and form trihalomethanes. It will also react with reducing agents such as hydrogen sulfide, ferrous ions, manganous ions, and nitrite ions.

Let's consider one example, in which chlorine reacts with hydrogen sulfide in water. Two different reactions can occur:

Hydrogen Sulfide + Chlorine + Oxygen Ion Elemental ⟶ Sulfur + Water + Chloride Ions

$$\mathbf{H_2S + Cl_2 + O^{2-} \longrightarrow S + H_2O + 2Cl^-}$$

Hydrogen Sulfide + Chlorine + Water ⟶ Sulfuric Acid + Hydrochloric Acid

$$\mathbf{H_2S + 4Cl_2 + 4H_2O \longrightarrow H_2SO_4 + 8HCl}$$

In the first reaction, hydrogen sulfide reacts with chlorine and oxygen to create elemental sulfur, water, and chloride ions. The elemental sulfur precipitates out of the water and can cause odor problems. In the second reaction, hydrogen sulfide reacts with chlorine and water to create sulfuric acid and hydrochloric acid.

Each of these reactions uses up the chlorine in the water, producing chloride ions or hydrochloric acid which have no disinfecting properties. The total amount of chlorine which is used up in reactions with compounds in the water is known as the **chlorine demand**. A sufficient quantity of chlorine must be added to the water so that, after the chlorine demand is met, there is still some chlorine left to kill microorganisms in the water.

Reactions of Chlorine Gas with Water

At the same time that chlorine is being used up by compounds in the water, some of the chlorine reacts with the water itself. The reaction depends on the type of chlorine added to the water as well as on the pH of the water itself.

Chlorine gas is compressed into a liquid and stored in metal cylinders. The gas is difficult to handle since it is toxic, heavy, corrosive, and an irritant. At high concentrations, chlorine gas can even be fatal.

When chlorine gas enters the water, the following reaction occurs:

Chlorine + Water $\longrightarrow$ Hypochlorous Acid + Hydrochloric Acid

$Cl_2 + H_2O \longrightarrow HOCl + HCl$

The chlorine reacts with water and breaks down into **hypochlorous acid** and hydrochloric acid. Hypochlorous acid may further break down, depending on pH:

Hypochlorous Acid ↔ Hydrogen Ion + Hypochlorite Ion

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

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

Hypochlorites

Instead of using chlorine gas, some plants apply chlorine to water as a *hypochlorite*, also known as a *bleach*. Hypochlorites are less pure than chlorine gas, which means that they are also less dangerous. However, they have the major disadvantage that they decompose in strength over time while in storage.

Temperature, light, and physical energy can all break down hypochlorites before they are able to react with pathogens in water.

There are three types of hypochlorites - sodium hypochlorite, calcium hypochlorite, and commercial bleach:

Hypochlorites and bleaches work in the same general manner as chlorine gas. They react with water and form the disinfectant hypochlorous acid. The reactions of sodium hypochlorite and calcium hypochlorite with water are shown below:

Calcium hypochlorite + Water ⟶ Hypochlorous Acid + Calcium Hydroxide

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

Sodium hypochlorite + Water ⟶ Hypochlorous Acid + Sodium Hydroxide

$$NaOCl + H_2O \longrightarrow HOCl + NaOH$$

Chloramines

Some plants use **chloramines** rather than hypochlorous acid to disinfect the water. To produce chloramines, first chlorine gas or hypochlorite is added to the water to produce hypochlorous acid. Then ammonia is added to the water to react with the hypochlorous acid and produce a chloramine.

Three types of chloramines can be formed in water - monochloramine, dichloramine, and trichloramine. Monochloramine is formed from the reaction of hypochlorous acid with ammonia:

Ammonia + Hypochlorous Acid ⟶ Monochloramine + Water

$$NH_3 + HOCl \longrightarrow NH_2Cl + H_2O$$

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

Monochloramine + Hypochlorous Acid ⟶ Dichloramine + Water

$$NH_2Cl + HOCl \longrightarrow NHCl_2 + H_2O$$

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

Dichloramine + Hypochlorous Acid ⟶ Trichloramine + Water

$$NHCl_2 + HOCl \longrightarrow NCl_3 + H_2O$$

The number of these reactions which will take place in any given situation depends on the pH of the water. In most cases, both monochloramines and dichloramines are formed. Monochloramines and dichloramines can both be used as a disinfecting agent, called *combined chlorine residual* because the chlorine is combined with nitrogen. This is in contrast to the free chlorine residual of hypochlorous acid which is used in other types of chlorination.

Chloramines are weaker than chlorine, but are more stable, so they are often used as the disinfectant in the distribution lines of water treatment systems. Despite their stability,

chloramines can be broken down by bacteria, heat, and light. Chloramines are effective at killing bacteria and will also kill some protozoans, but they are very ineffective at killing viruses.

Dosage of Chlorine

The amount of chlorine needed to disinfect water will vary from source to source. waters should be clear, and free from organic matters and suspended solids.

When chlorine is added to water some is used immediately to oxidize any organic matter and to kill bacteria in the water. Any that is not used straight away remains in solution in water as residual chlorine and protects against any possible future contamination of the water. There is no purpose in adding to little chlorine to a water, because the chlorine will be used up in oxidation of the organic material, and there may be insufficient chlorine to kill bacteria present. Disinfection is performed by any chlorine that remains after organic matter has been oxidized, and therefore is carried out by the last, and not the first, few parts per million of chlorine in the water. A sufficient quantity of chlorine should be added to leave an adequate chlorine residual in the water. The residual chlorine level should not, however, be so great as to cause consumers to complain about the taste of chlorine in the water.

Applied chlorine dosage rates are usually in the range of 0.5 to 2.0mg/l, depending up on the initial quality of the water. a suitable contact time following addition of chorine should be provided to allow the water to be fully disinfected before it reaches the consumers. Contact times are between 10 and 30 minutes, with a time of 30 minutes being recommended. After this contact time the water should have a chlorine residual of 0.2 to 0.5mg/l.

Higher chlorine dosage may be necessary for disinfecting new water mains, new wells or reservoirs, or other items that may have been contaminated. Samples of water should be taken immediately after disinfection to ensure that no coliforms are present.

New water mains need to be disinfected prior to commissioning, and older mains should be disinfected after repairs or after any major cleaning program if they are found to be contaminated. First of all the pipe lines should be flushed out with clean water and a foam swab

passed through the pipe to remove slime, dirt and deposits. pipe lines may then be disinfected by filling them with water containing chlorine at a concentration of 20mg/l (50mg/l is frequently used) and leaving this in the pipe line for not less than 24 hours for draining the chlorine solution away.

Wells, reservoirs and storage tanks should also be disinfected before being put in to service following construction, repairs, cleaning or maintenance. The walls should brushed down as thoroughly as possible using a strong chlorine solution containing between 50 and 100mg/l of chlorine. Prior to this, reservoirs and storage tanks may be washed down using high pressure water jets. Following cleaning the well, reservoir or tank should be filled with water containing at least 20mg/l of chlorine and left to stand for not less than 24 hours before the contents are drained away to waste. The tank should then be refilled with fresh water to remove any chlorine that may create undesirable taste in water.

(A) Plain Chlorination

Plain chlorination is the process of addition of chlorine only when the surface water with no other treatment is required. The water of lakes and springs is pure and can be used after plain chlorination. A rate of 0.8mg/lit/hour at 15N/cm^2 pressure is the normal dosage so as to maintain in residual chlorine of 0.2 mg/lit.

(B) Super Chlorination

Super chlorination is defined as administration of a dose considerably in excess of that necessary for the adequate bacterial purification of water. About 10 to 15 mg/lit is applied with a contact time of 10 to 30 minutes under the circumstances such as during epidemic breakout water is to be dechlorinated before supply to the distribution system.

(C) Brake Point Chlorination

The graph below shows what happens when chlorine (either chlorine gas or a hypochlorite) is added to water. First (between points 1 and 2), the water reacts with reducing compounds in the water, such as hydrogen sulfide. These compounds use up the chlorine, producing no chlorine residual.

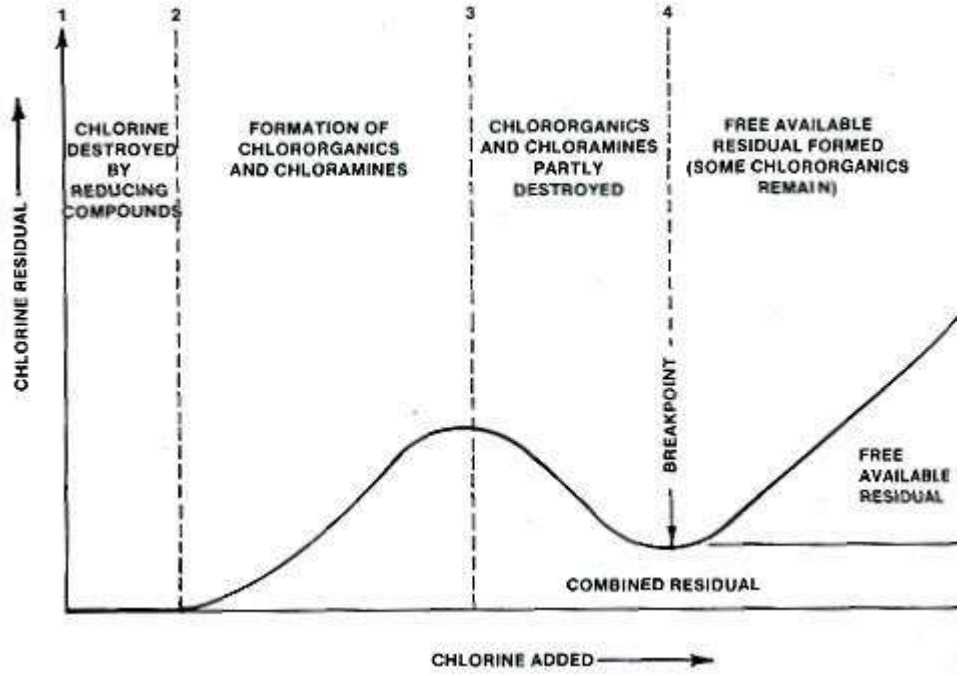


Figure 39: Break point chlorination

Next, between points 2 and 3, the chlorine reacts with organics and ammonia which is naturally found in the water. Some combined chlorine residual is formed - *chloramines*. Note that if chloramines were to be used as the disinfecting agent, more ammonia would be added to the water to react with the chlorine. The process would be stopped at point 3.

In contrast, if hypochlorous acid is to be used as the chlorine residual, then chlorine will be added past point 3. Between points 3 and 4, the chlorine will break down most of the chloramines in the water, actually lowering the chlorine residual.

Finally, the water reaches the breakpoint, shown at point 4. The **breakpoint** is the point at which the chlorine demand has been totally satisfied - the chlorine has reacted with all reducing agents, organics, and ammonia in the water. When more chlorine is added past the breakpoint, the chlorine reacts with water and forms hypochlorous acid in direct proportion to the amount of chlorine added. This process, known as ***breakpoint chlorination***, is the most common form of chlorination, in which enough chlorine is added to the water to bring it past the breakpoint and to create some free chlorine residual.

(D) De-chlorination

Removal of excess chlorine resulting from super chlorination in part or completely is called „De-chlorination". Excess chlorine in water gives pungent smell and corrodes the pipe lines. Hence

excess chlorine is to be removed before supply.

Physical methods like aeration, heating and absorption on charcoal may be adopted. Chemical methods like sulphur dioxide (SO_2), Sodium Bi-sulphate ($NaHSO_3$), Sodium Thiosulphate ($Na_2S_2O_8$) are used.

Points of Chlorination

Chlorine applied at various stages of treatment and distribution accordingly they are known as pre, post and re-chlorination.

a) Pre-Chlorination

Chlorine applied prior to the sedimentation and filtration process is known as *Pre-chlorination*. This is practiced when the water is heavily polluted and to remove taste, odour, colour and growth of algae on treatment units. Pre-chlorination improves coagulation and post chlorination dosage may be reduced.

The residual chlorine is useful in several stages of the treatment process - aiding in coagulation, controlling algae problems in basins, reducing odor problems, and controlling mud ball formation. In addition, the chlorine has a much longer contact time when added at the beginning of the treatment process, so pre chlorination increases safety in disinfecting heavily contaminated water.

b) Post Chlorination

Post chlorination is the application of chlorine after water has been treated but before the water reaches the distribution system. At this stage, chlorination is meant to kill pathogens and to provide a chlorine residual in the distribution system. Postchlorination is nearly always part of the treatment process, either used in combination with prechlorination or used as the sole disinfection process.

c) Re-Chlorination

In long distribution systems, chlorine residual may fall tendering the water unsafe. Application of excess chlorine to compensate for this may lead to unpleasant smell to consumers at the points nearer to treatment point in such cases chlorine is applied again that is re-chlorinated at intermediate points generally at service reservoirs and booster pumping stations.

Example

Consider water from a polluted river having BOD_5 = 5 mg/L, TKN = 1 mg/l (as N), and MPN: 10^6 organisms / mL. This water will be treated in a conventional water treatment plant and supplied for potable purposes. Compute the chlorine dose (in mg/L as Cl_2) required per liter of this water (consider both pre and post-chlorination) such that after treatment BOD_5, TKN, NH_3- N are negligible and MPN < 1organism/mL.

Assumptions:

- Assume that 1 mg/L (as Cl_2) chlorine is required to destroy 1 mg/L of BOD_5.
- Assume TKN is completely converted to NH_3-N during pre-chlorination.
- The average time between post-chlorination and water consumption by the end users is 1 hour
- The product of disinfectant dose (C in mg/L) and the contact time (t in minutes) for 5 and 6 log-kills using free residual chlorine as disinfectant is 96 and 120 respectively.
- Assume 2 log-kill of microorganism during water treatment up to just before the post-chlorination step.

Solution:

Chlorine dose required during pre-chlorination for destruction of BOD_5 = 5 mg/L as Cl_2

All TKN in water is converted to NH_3-N during this process.

Hence ammonia concentration in water before post-chlorination = 1 mg/L (as N)

Breakpoint chlorination has to be performed to destroy ammonia in water. Relevant

equation: $$2.NH_3 + 3.HOCl \rightarrow N_2 + 3.H_2O + 3.HCl$$

Ammonia concentration in water = $\frac{1}{14} = 0.0714$ mmoles/L

Chlorine required for destruction of ammonia = 1.5.(0.0714) = 0.1071 mmoles/L,

Therefore, breakpoint chlorination dose = 71.(0.1071) = 7.6 mg/L

Initial microorganism concentration = 10^6 /mL

Removal during water treatment up to post-chlorination = 2 Log

Hence microorganism concentration just before post chlorination = 10^4 /mL

To get this concentration below 1 /mL, 5 log kills are required

"C.t" for 5 log kills = 96

Contact time = 1 hour = 60 minutes

Therefore required free chlorine residual dose = $\frac{96}{60} = 1.6$ mg/L as Cl_2

Therefore, total chlorine dose required = 5 + 7.6 + 1.6 = 14.2 mg/L as Cl

Chlorine Gas

Chlorine gas is a yellowish green gas that is denser than air. It may be purchased in cylinders containing liquified gas at high pressure (containing about 30 Kg of liquid chlorine) or drums (containing about 860 kg o f liquid chlorine). Chlorine gas may also be produced on site by passing an electric current through a solution of salt in water.

Chlorine should never be injected in to water supply directly from cylinders or drums. Direct injection is a dangerous practice and it is not possible either to control or know with any accuracy how much chlorine is being applied. In all usual situations it is necessary to use chlorinating equipment that controls and measures the doses of chlorine applied.

The chlorine dose is usually applied to the water to be disinfected by being sucked in to the water at a narrowing of the pipe, known as venturi constriction. The narrowing of the pipe causes the speed of the water to be increased and the pressure to be reduced, so allowing chlorine to be sucked in. Following the addition of chlorine, thorough mixing is necessary to ensure that the chlorine is uniformly distributed through the water.

For small supplies salts of hpochlorous acid may be used to provide chlorination for swift disinfection of water, sodium hypochlorite solution known as Javel water contains about 5-16% available chlorine by weight. Bleaching powder (or chloride of lime) contains 20-35% available chlorine, and high test hypochlorite (HTH) contains 60-70%. The chlorine content of these various compounds and solutions made from them cam be expressed in various ways.
By the percentage (by weight) of chlorine in parts per million (ppm), or milligrams of chlorine per litter. (Only for dilute solutions.)

In order to convert from one unit to another, a 1% solution contains 10 grams of chlorine pet liter (1000 parts per million)

Javel water, bleaching powder and high test hypochlorite are the most commonly used chlorine compounds, the choice of chemical being influenced by the reactive costs and availabilties, and by the ease with which they can be transported.

In composition bleaching powder contains a mixture of calcium hypo chlorite, calcium chloride and calcium hydroxide. The costs of transport and storage are high because bleaching powder contains excess lime that serves no purpose in disinfection. Lime is also insoluble in water, and solutions made from bleaching powder should be allowed to stand after mixing and the solids allowed to sink. The solution on top should then be decanted in to a storage tank. Any lumps of cemented bleaching powder should then be broken up prior to mixing, and the bleaching powder added to water NOT the water added to then bleaching powder. the sediment, lime residue resulting from preparation of chlorine solution from bleaching powder is of no value to the disinfection process and should be thrown away, using a safe disposal system. failure to remove insoluble material results in the blocking of valves and pipes.

High test hypochlorite consists of Calcium hypochlorite, and allows chlorine to be released slowly. it has certain advantages over bleaching powder, being more stable, less likely to deteriorate during storage even in tropical climates, and having good solubility so that relative

clear solutions may be prepared. Storage and handling conditions should be as for bleaching powder. The stability of High Test Hypochlorite should not be overestimated, however. The high percentage of available chlorine and the good solubility in water results in low transport and storage costs than for bleaching powder. High test hypochlorite may be supplied in powder, tablet or granular form, and chlorine solution should be prepared using the same method as for bleaching powder.

Javel water (Sodium Hypochlorite Solution) is convenient because it is supplied as a clear solution which is unlikely to cause blockage of dosing equipment. On the other hand because the solution contains a large proportion (by weight) of water, the handling and transport costs are high.

Storage of Chlorine Compounds

Bleaching powder can be handled easily, but it is bulky and unstable, losing some of the available chlorine during storage. Loss of chlorine results from exposure to the atmosphere, moisture, light or heat. The powder, and solution prepared from it, should therefore be stored in cross - resistant containers in cool, dry, dark locations. Sealed drums of bleaching powder should be used within two years of productions, and once opened should be used within three weeks.

High test hypochlorite is more stable than Bleaching powder, but the same storage requirements apply to both components. javel water should also be stored in sealed containers and in dark conditions. Once containers of Javel water are opened their contents should be used within a week.

The rate at which chlorine is lost from chlorine compounds during storage can not be estimated reliably. Before a concentrated solution should be made, and the strength of this solution measured. A solution should then be prepared of the strength and volume required for dosing purposes.

For a sample of bleaching power containing 30% available chlorine, 33.3 grams of bleaching

powder in 1 liter of water will produce a 1% chlorine solution (10 grams of chlorine per liter) which could be used to dose 10,000 liters (10 cubic meters) of water at a dosage of 1mg/l. For high test hypochlorite containing 65% available chlorine, only 15.3 grams of the sample would be needed to produce an equivalent solution. Unfortunately, in practice it is not usually possible to know what is the percentage of available chlorine in a sample of bleaching powder or high test hypochlorite prior to making a chlorine solution.

The following tables indicate the quantities of various chlorine compounds that should be used to produce chlorine solutions and to disinfect water supplies.

Table 9 : weights in grams of chlorine compounds needed to produce a 1% chlorine solutions for dosing purposes.

Chlorine content in the compound	**5%**	**10%**	**15%**	**20%**	**30%**	**50%**	**60%**	**70%**
To make 1 liter	200	100	67	50	33	20	17	14
To make 5 liter	1000	500	333	250	167	100	83	71
To make 10 liter	2000	1000	667	500	333	200	167	143
To make 20 liter	4000	2000	1333	1000	667	400	333	286
To make 50 liter	10000	5000	3333	2500	1667	1000	833	714

to make X liters of chlorine solution of Y% strength from a compound containing Z% chlorine by weight. the amount of compound to be used is:

$$\frac{X \times Y \times 10 \times 100}{Z} \text{ b grams of chlorine compound}$$

Table 10: Volumes (liter) of 1% chlorine solution required for disinfection of water supplies.

Chlorine dosage (mg/l)	**0.5**	**1.0**	**1.5**	**2.0**
Volume of water to be Chlorinated (cubic m.)				
5	0.5	0.25	0.75	1.0
10	0.50	1.0	1.5	2.0
25	1.25	2.5	3.75	5.0
50	2.5	5.0	7.5	10.0
100	5.0	10.0	15.0	20.0
250	12.5	25.0	37.5	50.0
500	25.0	50.0	75.0	100.0

The volume of A% chlorine solution needed to chlorinate B cubic meters of water at a dosage rate of c mg/l is:

B X C

------- Litters

10 X A

Sample Constant Rate Systems for Dosing Chlorine Solution

A variety of constant rate chlorine dosing systems have been designed, and the designs can often be copied or adapted to make use of locally available materials. Two of the more reliable devices are the constant head syphon and the floating bowl solution feeder. Despite its reliability, the outlets of these devices may become partially obstructed by sediment and scale. especially at low flow rates. chlorine solutions should be added to a flow of water, ensuring that the chlorine solution added can be adjusted so that the required level of residual chlorine is maintained after the necessary contact time.

For the constant head syphon flow rates remain constant because the head causing flow remains steady. The driving head is the difference in level between the base of the air inlet tube (where

pressure is atmospheric) and the siphon outlet. The syphon device is very reliable. giving constant feed rates irrespective of the level of water in the reservoir. Construction, operation and maintenance is simple, although care should be taken to ensure that the reservoir is well sealed and that air cannot enter the siphon tube from the air inlet pipe.

Flow control can be achieved by moving the air inlet tube, changing the height of the siphon with respect to the air inlet tube, or by use of a tap on the siphon outlet. The air inlet tube should not be obstructed, and flow control by use of a tap on the air inlet is unsatisfactory.

The floating bowl solution feeder may be of two possible designs; either with the inlet above the outlet, or with the inlet below the outlet. Ether design may be used, but the bowl may require some ballast (such as gravel) and a vertical guide wire should be provided to prevent the bowl from tilting over and to keep it floating in a controlled manner as the bowl descends during dosing. Flow control by altering the sizes of the tubes or by adjusting the levels of the tubes in the bowl.

Chlorination of Wells

Water obtained from wells and boreholes is very difficult to disinfect with any reliability, and under-dosing or overdosing is almost inevitable. The volume of water contained in a well or borehole is seldom known accurately and rates of extraction will very from day to day and during each day.

Simple disinfection may be used to provide residual chlorine in waters of high initial quality, but the most reliable form of ensuring accurate dosing is to store the water obtained from a well or borehole in a reservoir, and to disinfect the water, whose volume and chlorine requirement can be measured, in the storage reservoir.

The practice of adding chlorine solution directly to a well or borehole is not recommended, as the only factor that can be controlled is the residual relative to the original dose. The chlorine

solution may not mix thoroughly with the water in the well, and any metal lings or fittings may become corroded.

It has been reported that water in wells may be disinfected by the use of chlorination pots, containing a mixture of sand and either bleaching powder or HTH. The problems associated with this technique are not so widely reported.

Chlorination pots are not widely used, although certain publications would lead one to believe otherwise. The operating principle for these chlorination pots assumes that water soaks into the chlorine compound used, forms a chlorine solution which then mixes with the remainder of the water in the well. In reality, the outer layer of the chlorine compound adsorbs water and then sets hard, like cement, forming a hard shell that effectively prevents water from reaching the interior of the mixture. Tests have shown that by adding a small quantity (about 5% by weight) of sodium Hexametaphosphate to the bleaching powder or HTH, the mixture remains soft, but the difficulty of controlling the dosing rate remains.

A practical problem associated with chlorination pots is related to patterns of water use. During the night, when little or no water is taken, the water in contact with the pot may develop such a high chlorine content that it becomes unacceptable for drinking purposes. Through the day, as water is extracted and stocks of water are replenished, the chlorine level decreases, reaching a minimum level by evening.

Chlorination Techniques

Various chlorination techniques may be used. The type of chlorination used in a particular situation depends on the quality of the water to be disinfected, but for most situations chlorine is added following other water treatment processes (post - chlorination).

In conclusion, various disinfection methods can be used in order to ensure that water is of good bacteriological quality. chlorine and chlorine based chemicals are the most commonly used products, and over many years and in a variety different conditions these have proved to be simple and reliable to use. Disinfection should not be considered to be a complete water

treatment process in itself, but should be used in conjunction with other treatment operations to produce waters that are consistently safe and of good quality and appearance.

CHAPTER SEVEN: MISCELLANEOUS WATER TREATMENT

7.1 Removal of Taste and Odor Problem

The sense of odour is closely related to that of taste. In fact it is normally correct to suggest that most 'tastes' in water are really a sensation of smell. There are only four sensation of taste (1) sour, sweet, salty and bitter-all other sensations are of odour although not necessarily noticed until the water is in the mouth (2). some tastes in water are unrelated to odour and these include the brackishness associated with relatively high concentrations of salts such as sodium chloride or magnesium sulphate. Iron and manganese in water often produce an astringent taste. Chalk derived waters are frequently described as possessing a 'sharpness' _ possibly due to Co2 or bicarbonate _while surface waters deficient in dissolved oxygen are often 'flat' to the taste.

Consumers identify and accept water as being pure as a result of its lack of colour, its clarity and its lack of taste and odour. Hence it is not only necessary that water is pure but that it also appears to be pure.

some substances, such as many inorganic salts do not create sensations of odour or taste until present in a concentration of several hundreds or even thousands of mg/1. Others such as commercial grade parathion (0.003 mg/1) and sodium fluoride (2,4 mg/1) become noticeable at much lower concentrations. People also vary in their sensitivity to odour and taste and some people are perhaps as much as eleven times as sensitive as others. In addition many causes of odour and taste in water possess additive, synergistic or antagonistic effect, so that the mixing of two sources of water, each possessing only a limited odour can result in a combined water with a pronounced odour problem. Conversely, mixing odoriferous water can produce a non-offensive supply.

7.1.1. Causes of Odor and Taste

- concentrations of inorganic salts (brackishness)
- Hydrogen sulphide
- contact with painted surfaces

- Industrial discharges _ pesticides, phenols etc
- Metabolites of actinomycete, algae etc
- dead and decaying organic material including sewage and algae
- chlorination

Some of these may be prevented by various means while others must be cured. Hydrogen sulphide is commonly found in ground waters from deep wells. Formation of H2s on other water supplies should be prevented by maintaining the water always in an aerobic condition. Chlorination should be carefully controlled to prevent excessive odor and taste in the treated water.

7.1.2. Prevention of odours and tastes from microorganisms

Bacteria - Some bacteria reduce sulphate to sulphide (H_2S) and these are controlled by aeration and chlorination.

Actinomycete (mold-like bacteria) - grow frequently in mains as a slime on the inside of the pipes. They may be controlled by treating reservoirs with copper sulphate or by ensuring there is residual chlorine throughout the mains, or by biological control employing Bacillus cereus to reduce the obnoxious metabolites produced.

Iron bacteria _ grow in water containing appreciable concentrations of iron and in which the d.o. is limited. They produce objectionable odours and tastes as well as leading to the precipitation of iron. The remedies are removal of iron, chlorination or dosing with copper sulphate.

Plankton (algae, protozoa) _ these either produce taste or odour as a result of normal metabolic activity or as a result of death and decay. Growth may be controlled by limiting nutrients, desertification of reservoir, mechanical removal of plants, and chemical control.

In addition the levels of water containing the troublesome microorganisms may be avoided by varying the level of the water intake. Chemical control is nearly invariably by the addition of small quantities of copper sulphate.

7.1.3. Recommended method to remove test and odour problem

1. Aeration

This is of limited value, particularly with the only slightly volatile odors resulting from the decomposition of vegetation. However, concentrations of odoriferous metabolites emanating from living microorganisms can be decreased, thus reducing the amount of activated carbon required at a later stage.

2. Chlorine

Marginal chlorination is not sufficient to remove odoriferous compounds and also may add a noticeable smell of chlorine. In the presence of phenols a pronounced smell of chlorinated phenols will become apparent. Super chlorination with more than a breakpoint dosage will destroy most malodorous compounds. Ammonia/chlorine treatment can be useful in preventing chlorophenolic, odours.

3. Ozone

Usually deals effectively with odors that are only accentuated by chlorine.Modest doses of 1 to 2 mg/1 will completely destroy phenols at any likely PH value. Ozone reduces H2s odors and tastes and also the odors from decaying vegetable matter. Algae must be removed prior to the addition of ozone. Partial breakdown by the addition of zone of previously non-biodegradable organic material can result in the production of soluble substrate to encourage growth of obnoxious microorganisms in the distribution system.

4. Hydrogen peroxide

This is occasionally used for odor control. It works by releasing 'nascent' oxygen but is generally too costly.

5. Chlorine Dioxide

$Cl0_2$ is particularly effective at removing phenolic odors from water. It does not react with ammonia and hence its treatment strength is not dissipated. It is most effective when mixed with an excess of chlorine.

6. Potassium Permanganate

It is one of the most effective chemical reagents for odour taste control. It is normally applied as a solution and as such is easy to control. In the USA it is widely used and is most effective between PH8.o and PH8.3 (3). The benzene rings of both pheno l and aniline are broken by permanganate. Although it is about three times as costly as activated Carbon it has been claimed to be up to five times as effective.

7. Activated Carbon

Relies on its large surface area and physical adsorption for its effect and tends to remove larger, rather than smaller organic molecules and non-polar rather than polar compounds. It is used either as powdered activated carbon or as granular activated carbon.

Powdered activated carbon is a fine black powder which is formed into slurry before use and applied either to the raw water, or the mixing basin, on the settling basin or directly before the filters. It is not recoverable. A normal dose might be 2.0 mg/1 to clarified water although it has been applied at rates up to 125 mg/1 to raw water (4)

Granular activated carbon consists of relatively large grains and is held in filter towers through which the water is continually passed. periodically the carbon beds are cleaned by back washing and when approaching exhaustion the carbon is regenerated in a furnace in carefully regulated oxidizing conditions at about 800oc.

At low levels of taste and odour powdered activated carbon is suggested as being more economic than the granular variety but the reverse is true once higher dosages are required (5). the PH of the water to which the carbon is added is of importance, as the adsorptive properties appear to be about four times as great at PH5 to PH6 than between PH9 and PH10. Chlorine is effectively

removed by activated carbon treatment and hence post-chlorination is necessary.

7.2. Removal of Hydrogen sulphide

The permissible concentration of H_2S in water, which are required for domestic purpose, is 0.5 mg/1 of H_2S and it is offensive when its concentration greater than 1.0 mg/1. The odours are less noticeable as the PH rises due to the formation of alkaline sulphides (s" & Hs'), although these sulphides add appreciably to the chlorine demand of the water.

Hydrogen sulphide is removed by aeration and chlorination. Aerators are normally of the natural-draft cascade-type but aeration has only a limited effect and it is necessary to establish a cost balance between aeration and chlorination. Removal by chlorination is highly effective but as much as 10 kg of chlorine may be required to remove 1.0 kg of hydrogen sulphide.

Threshold odour Number

The T.O.N is the number of times a sample must be diluted with odor-free water so that the odor is the least definitely perceptible (1). Since there is little consistency in sensitivity of at least five persons, and preferably of ten, is employed.

The sample is progressively diluted with odor-free water which is normally obtained by passing tap-water through a column of activated carbon. Most tap-waters are suitable for this purpose so long as they do not contain unusual salt concentrations and do not vary appreciably from a normal PH.

All glassware used must be clean and odor-free, having been washed with a non-odorous soap, rinsed with an acid cleaning solution and finally rinsed with odor-free water. The testers must be free from colds, be non-smoking, should not use cosmetics, perfumes or shaving lotions and should not have recently eaten a small. All extraneous odors such as from fresh paint must be excluded.

7.3. Removal of Iron and Manganese

Before assuming that iron has come from the other source one should check that it is not coming from corroding steel well screens or pipes.

Iron and manganese can be dissolved from sandy soils, laterite, shale, sandstone and other rocks by acid water which contains no dissolved oxygen. (such water may have _ or have had _ an organic content. Decomposition consumes the oxygen and produces carbon dioxide, which is acidic.) the iron is usually in the form of ferrous bicarbonate and the manganese in the form of manganese oxide or bicarbonate. These forms are soluble.

If the water is now exposed to air so that oxygen can enter into solution and carbon dioxide leaves, the ferrous from is oxidized to ferric, and the manganese to manganic. These latter forms are insoluble and so visible precipitates are formed. Therefore iron bearing water may be clear when pumped up from a well, but may, on standing, turn brown and turbid. (For this reason samples of groundwater should be analyzed before they have a chance to lose carbon dioxide and gain oxygen). The formation of a precipitate may take longer with manganese, or when the metals are combined in organic compounds. Sometimes the evidence of a manganese problem in a distribution system may take years to appear because the reactions are slow until catalyzed by deposits.

Iron and manganese problems most commonly occur with groundwater, but may also appear in water drawn from the bottom of a stratified reservoir or in acid peat waters. Manganese may occur in significant quantities in river water, sometimes coming from acid mine drainage.

7.3.1. Causes of Iron and Manganese

These metals in potable water can cause a)

Colour in the water

b) Staining of plumbing fixtures and clothing (Fe -brown. Mn - black)

c) Astringent, metallic or medicinal taste in the water

d) Growth of iron bacteria (mainly crenothrix and Leptohrix) which feed on iron when the dissolved oxygen content is low and form slimes which clog well screens and pipes and water meters.

They also produce tastes and odors in the water. Manganese bacteria may produce long black streamers of slime which emerge from consumers' taps. When the water contains sulphate, sulphate reducing bacteria may work together to produce black deposits of iron sulphate and H2S in dead ends.

e) Brown stains on cooked vegetables and discoloured etc.

7.3.2. Method of Treatment

Groundwater containing iron and/or manganese needs only treatment to remove these metals, and disinfection.

Surface water containing iron or manganese will usually also require removal of the turbidity of clay colloids. Small quantities of iron can be removed by normal conventional treatment. If a higher PH is needed to remove iron or manganese then alum may not be suitable and ferric coagulants may be preferred if removal of turbidity and iron or manganese is effected simultaneously.

In general the aim is to form insoluble compounds so that precipitates can be separated by sedimentation and filtration. The variety of forms in which the iron or manganese exist requires a variety of treatment processes, as summarized in the attached table. The only satisfactory method of choosing the most economical appropriate method is a program of pilot tests. Usually manganese is more difficult to remove than iron.

1. Aeration

It is commonly used, but some processes require exclusion of air at the early stages. Aeration raises the PH by removing carbon dioxide, and adds oxygen. only 0.14 mg/1 of dissolved oxygen is needed to oxidize 1 mg/1 of iron.

2. Sedimentation

It is useful where large particles are formed in a short time, but for some waters it achieves nothing.

3. Filtration

It is required in most processes as a means of capturing small suspended particles, but also often to promote oxidation by the catalytic effect of deposits on the sand grains. Because the deposits are useful the filter will take time to mature and should not be back washed too thoroughly. Coarser grain sizes may be used (compared with normal filters _ between 0.7 and 1.0mm E.S) and higher filtration rates are possible (up to 10 m/hr or over 50 m/hr.)

4. Contact oxidation

It provides a catalytic effect, speeding up the oxidation. This is often achieved using layers of coke or gravel on tray aerators, or by providing a bed up to 1.8 m deep with ventilation holes. Deposits on the grains assist oxidation. Basic rocks such as limestone or dolomite may be used to raise the pH. Iron or copper sulphate solutions catalyze oxidation of iron, and pyrolusite (a manganese or potassium permanganate are used for manganese).

Hopper bottomed clarifiers have been used to oxidize iron and manganese in the sludge blanket, using potassium permanganate solution as a catalyst.

One case when aeration at first must be prevented is process 5 in the table. Here soluble manganous hydroxide is formed first, and then this is oxidized to insoluble manganic hydroxide by aeration. (If aeration were used first the precipitate would be so fine that it would be very difficult to separate.) This is achieved by upward flow through a bed of pyrolusite before aeration. Contact beds can be cleaned by rapidly draining down or hosing.

5. Chlorination

It is simple and cheap, but slow for manganese below PH 8.5 and in some cases may form very fine precipitates. Chlorination before addition of potassium permanganate may reduce the consumption of the latter.1.25 mg/1 of free chlorine is needed for each mg/1 of iron or manganese.

Zeolite Manganese ('greensand') is effective in removing manganese but it needs periodic regeneration. One cubic meter of manganese zeolite oxidizes 1.63 kg manganese and is regenerated by 3.26 kg potassium permanganate.

6. Lime

Raising the PH will cause oxidation, but this technique is not suitable for hard waters because it will cause deposition of calcium carbonate on the filter sand.

7. Iron exchange

When used for softening water will also remove soluble iron and manganese, it should therefore be done in the absence of air to prevent formation of the insoluble forms.

8. Sequestration

Iron and manganese may be held in solution by dosing with twice the concentration of sodium hex meta phosphate (Calgon). This method is not suitable if the water is to be boiled