

4. Water treatment

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4.1 Introduction

Absolutely pure water is that which contains only hydrogen and oxygen, i.e., H₂O and is never found in nature. The water found in nature contains a number of impurities in varying amounts. Therefore, removing these impurities up to certain extent so that it may not be harmful to the public health is necessary. The process of removing the impurities is called water treatment and the treated water is called wholesome water. Water treatment plants must produce water of sufficient quality and quantity for all intended purposes. If the water is to be used for human consumption, it must be free of organisms or substances posing health hazards at all times.

The primary goal of water treatment is to ensure that the water is safe to drink and does not contain any disease-causing microorganisms. However, most times ground water does not require any other treatment. Nevertheless, treatment of water is not complete unless the microorganisms present in water are removed. The most objectionable and harmful constituent of water is pathogenic bacteria. An excess number of harmless bacteria are also not desirable in potable 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 surface sources generally contain large amount of impurities therefore they require sedimentation, filtration and chlorination as treatment. If the water contains algae or other

micro organisms, pre-chlorination has to be done. Tastes and odors, dissolved gases like CO₂, H₂S are removed by aeration. During the flood season, the turbidity of the surface water may be high and flocculation may become necessary to remove turbidity.

Groundwater which is usually clear may require only disinfection and chemical treatment for the removal of pathogens, iron removal, softening etc. Sometimes ground water contains dissolved gases like hydrogen sulphide (H₂S) carbon dioxide (CO₂), which gives very bad odour and requires its removal by aeration.

River quality is largely influenced by pollution from municipalities, industries, and agricultural practices. The characteristics of a river can be highly variable. During rains turbidity may increase substantially. In warm months, algal blooms frequently cause taste and odor problems. Reservoir and lake sources have much less day-to-day variation than rivers. Additionally, the quiescent conditions will reduce both the turbidity and, on occasion, the color. As in rivers, summer algal blooms can create taste and odor problems in lakes and reservoirs.

The amount of treatment required depends on the:

- (a) Quantity and quality of raw water, and
- (b) Required standards of purified water

Table 4.1 comparison between ground and surface water characteristics

Ground	Surface
Constant composition	Varying composition
High mineralization	Low mineralization
Little turbidity	High turbidity
Low or no color	Color, taste and odour
Bacteriologically safe	Microorganisms present
No dissolved oxygen	Dissolved oxygen
High hardness	Low hardness
H ₂ S, Fe, Mn	Possible chemical toxicity

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. Harbor animals and interfere with valves. Depositions in pipes may result from unsatisfactory addition of chemicals, reactions within the system. or poor turbidity removal.

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.
 - It must be 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 neighbor country may play foul game by damaging the plant, poisoning the water.
- At higher elevation if the natural topography permit.
 - Ability to locate intake upstream of wastewater discharges.

- Land availability and costs.
- Potential for flooding of the site.
- Availability and reliability of electric power.
- Geology and topography of the site.
- Availability of transportation facilities.
- Legal obligations or restrictions.

The selection and design of the water treatment processes to be used at a particular facility are dictated by practicability, reliability, flexibility, and overall economics. Meaning the design of treatment facilities should consider all engineering, labor, materials, equipment, and energy factors.

Engineers experienced in water treatment plant design are needed to determine the best treatment system for any particular situation. To protect the main units of a treatment plant and to aid in their efficient operation it is necessary to remove the large floating and suspended solids which are often present in the inflow. Surface waters typically contain fish and debris which can clog or damage pumps, clog pipes, and cause problems in water treatment. Surface waters may also contain high concentrations of suspended matter. Preliminary treatment processes like screening is employed for removal of debris and part of the suspended materials.

Factors Affecting the Choice of Treatment Schemes

The following factors influence the choice of treatment alternative discussed below

- ✓ Limitation of capital
- ✓ Availability of skilled and unskilled labor
- ✓ Availability of equipment, construction material, and water treatment chemicals
- ✓ Local codes, drinking water standards and material specifications
- ✓ Local traditions, customs and cultural standards
- ✓ 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
- ✓ Organizational capacity to recruits and retrain
- ✓ Head lose should be conserved possible

4.2 Treatment methods

The various components of a water treatment plant have been derived from unit operations. A water treatment unit process is defined as an engineered system that employs particular kinds of influences or actions to effect certain intended state changes for the water. Every unit operation aims at the removal or reduction of specific objectionable substances to the desired degree.

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

1. Aeration
2. Screening and grit removal
3. Plain sedimentation
4. Coagulation and flocculation
5. Secondary sedimentation
6. Filtration
7. Adsorption
8. Softening
9. Disinfections

The various treatment methods and the nature of source of impurities removed by employing them are given in table 4.2.

Table 4.2: 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.

The degree and methods of treatment depend upon

- nature of the source
- quality of the source
- purpose for which it supplied

4.3 Basic water treatment

Preliminary 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.

High turbidity water requires pre treatment in the form of sedimentation, storage or filtration to reduce the suspended solids.

- to reduce the chemical needed for coagulation
- Pre-treatment provides cheaper treatment.
- Slow Sand Filters require raw water turbidity below 30NTU.
- High turbidity waters require pre-treatment before slow sand filtration.

Purpose of preliminary treatment is to protect the main units of treatment plant and to aid in their efficient operation. It is necessary to remove the large floating and suspended solids which are often present in the inflow. These materials include leaves, twigs, paper, rags and other debris which could obstruct flow through a plant or damage equipment in the plant.

Types of preliminary treatment

- ✓ Intakes
- ✓ Screens
- ✓ Grit removal

- ✓ Plain sedimentation
- ✓ Storage
- ✓ Roughing filter
- ✓ Infiltration galleries
- ✓ Silt trap

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 level intake in the dry season and to avoid algae at the top and high level intake in the wet season to avoid suspended solids at the bottom) are provided in lakes and reservoirs.

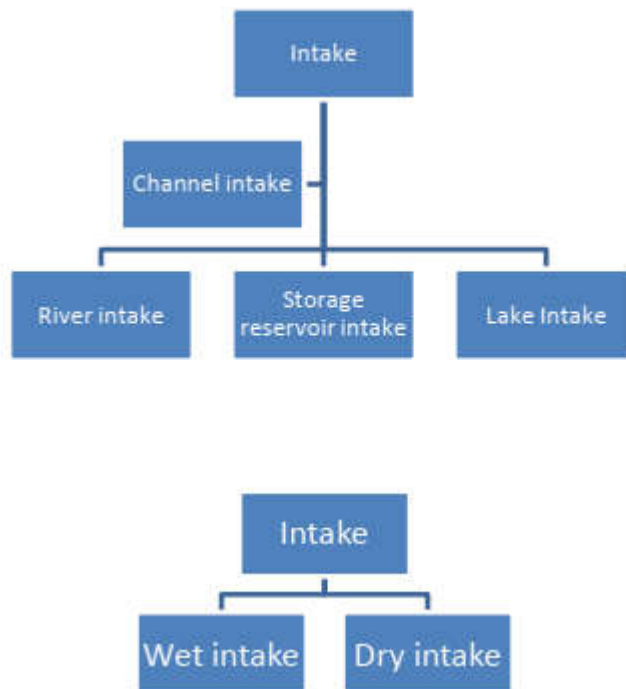


Fig1: Types of intake

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. It blocked and removed large particle e.g. plastic bags, rags, paper.

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.

The water usually flows through a bar screen, distance between single bars varies from 5 to 8 cm for course screen and 0.5 to 5.0cm for a fine screen. Solid matter is retained by the bar.

Spacing between screen bars:-

1. Fine screening, for spacing under 10 mm. Surface waters require screens or strainers for removal of material too small to be intercepted by the coarse rack. These may be basket-type, in-line strainers, manually or hydraulically cleaned by backwashing, or of the traveling type, which are cleaned by water jets. The velocity of the water in the screen openings should be less than 1.0m/s at maximum design flow through the screen and at minimum screen submergences which occur at minimum anticipated flows.
2. Medium screening, for spacing of 10 mm to 40 mm.
3. Course screening, for spacing of over 40mm: often termed bar screens or racks, must be provided to intercept large, suspended or floating material. Such screens or racks are made of 1/2- inch to 3/4-inch metal bars.

Are usually installed at the entrance of the water treatment plant

- ✓ to protect mechanical equipment,
- ✓ avoid interference with plant operations

- ✓ prevent objectionable floating materials such as rags or rubber from entering the primary settling tanks

Bar screen spacing is typically between 0.5 and 5cm. Angle of inclination of bars is 60-75° if screenings are very small and 30-45° 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.

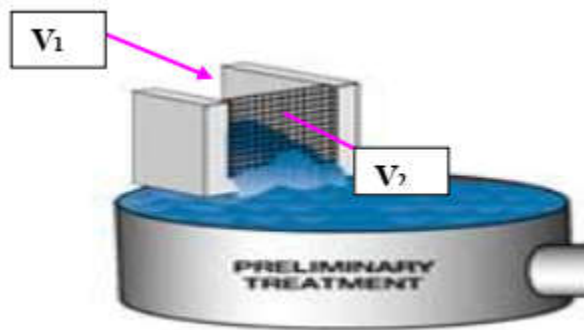


Figure 2: Screen

Head Loss Through Bar Screen

$$hl = \frac{1}{c} * \left(\frac{v_2^2 - v_1^2}{2g} \right)$$

Where

c =empirical discharge coefficient to account for turbulence and eddy motion. ($c=0.7$ for clean bar and 0.6 for clogged bar screen)

V_2 =velocity of flow through openings

V_1 = approaching velocity of upstream channel

g = gravitational acceleration (9.81m/s²)

Head Loss Through fine Screen

$$hl = \frac{1}{c(2g)} * \left(\frac{Q}{A}\right)^2$$

Where

c=empirical discharge coefficient to account for turbulence and eddy motion. (c=0.6)

g= gravitational acceleration (9.81m/s²)

Q= discharge (m³/s)

A=effective opening area of the screen

Disposal of screenings: - Project planning must include provisions for the disposal of debris removed by coarse and fine screens. Thus, screens must be cleaned regularly in order to prevent greater head loss which can reduce flow to the treatment plant or even damaging the screens. Cleaning is carried out by hand or with the help of a rake.

Grit Removal

Grit is composed of inorganic materials & non degradable organic material. Because the grit particles are relatively large, with a high density compared with the organic particles in untreated water, they are often removed using the principle of differential settling. By using a parabolic section channel it is possible to provide a constant horizontal velocity of around 0.3 m/s at all rates of flow. Under these conditions a channel of sufficient length to provide a retention time of 30-60 seconds will allow the grit particles to settle to the bottom whilst the remaining suspended solids are still transported by the flow. Sedimentation of grit is a form of pre-treatment that provides a low velocity of flow through a tank. It is favorable in a tropical climate due to high load of solid as a result of erosion by tropical high intensity rain and the high temperature of the tropics. Removal of grit may be accomplished in grit chambers. Grit chambers are designed to remove grit, consisting of sand, gravel, cinders, or other heavy solid materials that have subsiding velocities or specific gravities substantially greater than organic materials. Grit chamber is mostly located after the bar screens and before the primary sedimentation tanks.

Plain sedimentation

Plain sedimentation is a form of pre treatment 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 favorable 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 pre treatment 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.

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.)

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

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)

	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.

Roughing Filtration

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

	<u>Diameters:</u>
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 treatment 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 pre treatment 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, microorganisms 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 coarse to fine sizes is recommended.

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 labor cost. Such filter although producing the required quality is not recommended in developing countries. Since labor 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.

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.

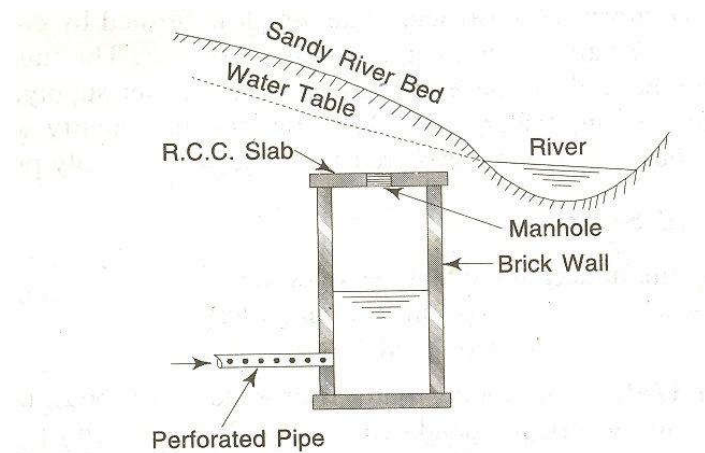


Figure 3: Infiltration Gallery

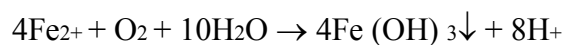
Silt trap

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

4.3.1 Aeration

It is the process of bringing water in intimate contact with air, while doing so water absorbs oxygen from the air. Aeration is the treatment process whereby water is brought in to intimate contact with air; A physical treatment process in which air promptly mixed with water; It is one of the important unit operations of gas transfer.

Aeration may be used to remove undesirable gases dissolved in water i.e. CO₂, H₂S, etc (degasification) or to add oxygen to water to convert undesirable substance i.e. Iron (Fe²⁺) & Manganese to more manageable form (oxidation). The Iron and Manganese may be removed as a precipitate after aeration. Chemically, these reactions may be written as follows:



The principal objectives of aeration are:

1. To increase the dissolved oxygen content of the water
2. To reduce tastes and odors caused by dissolved gases in the water, such as

hydrogen sulphide, which are then released; and also to oxidize and remove organic matter;

3. To reduce the carbon dioxide content of a water and thereby reduce its corrosiveness and raise its PH values;
4. To oxidize iron and manganese from their soluble states to their insoluble states and thereby cause them to precipitate so that they may be removed by clarification and filtration processes;
5. To remove certain volatile organic compounds

Application of aeration

The primary stage of water treatment that is aeration is applied for two reasons:

- ✓ For the treatment of ground water: Increasing oxygen and reducing carbon dioxide content
- ✓ For the treatment of surface water: When the water has a high content of organic matter

Different types of aerators are available

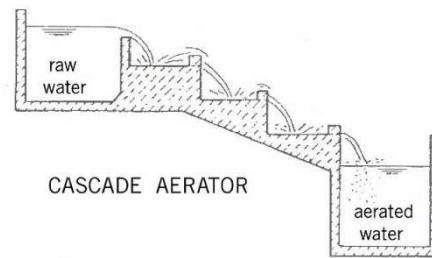
- ✓ Gravity Aerator
- ✓ Spray aerator
- ✓ Air diffuser
- ✓ Mechanical Aerator

i. Gravity aerators

Gravity aerators are primarily used in water purification plants for desorption of gases. Dispersing the water through the air in thin sheets or fine droplets i.e

a) Cascade towers

Cascade aerators are the simplest type of free falls aerators, by which the available difference of head is subdivided into several steps. Essentially this aerator consists of a height of 4 – 6 steps, each about 30 cm high with a capacity of about 0.01 m³/s per meter of width. They will take large quantities of water in comparatively small area at low head, they are efficient for raising the dissolved O₂ content but not for CO₂ content, which is usually in the range of 60-70%.



b) Inclined apron possibly shaded with plates

It uses the same principles to that of cascade aerator water aerator formed for full exposure of the water to the air.



Fig. 4 Gravity aerators

c) Tray aerator

Multiple tray aerators are generally constructed with three to nine trays & spacing of 30-76 cm b/n trays. Water application rates range from roughly 17 to 20 l/s/m². Multiple tray aerators are usually housed, particularly in colder climate. If there is no sufficient natural ventilation, artificial ventilation must be provided. Multiple tray aerators have excellent oxygen adsorption & CO₂ remover capacity.

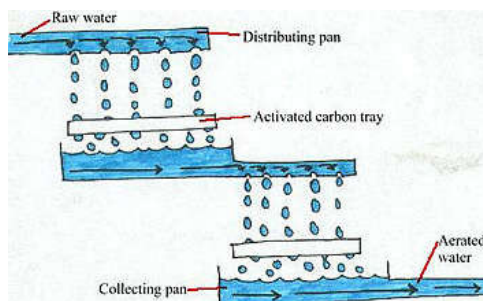


Fig. 5 Tray aerator

ii. **Spray aerators:** - By spray aerators the water is sprayed in the form of fine droplets into the air, thus creating a large gas-liquid interface for gas transfer. Consists of stationary nozzles connected to a distribution grid through which the water is sprayed into the surrounding air at velocity of 5 – 7 m/s. Spray aerators direct water up wards, vertically or at an inclined angle in a manner that causes water to be broken in to small drops. Installations commonly consist of fixed mobile or a pipe grid located over an open-top tank. The fine distribution of the water into the air is accomplished by pumping the water through orifices or nozzles mounted upon stationary pipes. Spray aerators are usually efficient with respect to gas transfer such as CO₂ removal or Oxygen addition. Provides a very simple and in expensive arrangement and it occupies little space.

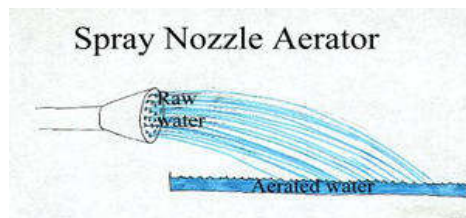


Fig. 6 Spray aerator

iii. **Air diffuser :-**By air diffusers compressed air is injected into water through orifices or nozzles in the air piping system to produce bubbles of various size and hence with different interfacial areas per m³ of air. By mixing the water with dispersed air E.g. Injection aerator, Diffused air bubble aerator .The amount of air required for bubble aeration of water is small, no more than 0.3 – 0.5 m³ of air per m³ of water, and these volumes can easily be obtained by sucking in of air. In diffused aeration systems, water is contained in basins. Compressed air is forced into this system through the diffusers. This air bubbles up through the water, mixing water and air and introducing oxygen into the water.

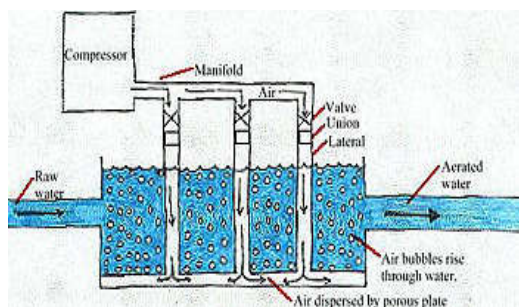


Fig. 7 Air diffusion aerator

iv. Mechanical Aerator

Mechanical aeration systems are fairly simple, but they are not among the most common purification techniques. These aerators work by vigorously agitating source water with mechanical mixers. As the waters churn, they become infused with purifying air. Mechanical aeration systems are able to remove most volatile contaminants, but they are limited to removals of 50 to 80 percent, depending on conditions.



Fig. 8 Mechanical aerator

4.3.1 Sedimentation

Waters contain a wide variety of particulate matter; the particles vary in size, shape and specific gravity. The removal of these impurities may require the provision of a combination of various unit operations and processes. The common unit operations provided for the separation of suspended and colloidal particles includes sedimentation, coagulation-flocculation and filtration

Sedimentation is a treatment process in which the velocity of the water is lowered below the suspension velocity and the suspended particles settle out of the water due to gravity. The process is also known as settling or clarification. Most water treatment plants include sedimentation in their treatment processes. However, sedimentation may not be necessary in low turbidity water of less than 10 NTU. In this case, coagulation and flocculation are used to produce pinpoint (very small) floc which is removed from the water in the filters.

The most common form of sedimentation follows coagulation and flocculation and precedes filtration. This type of sedimentation requires chemical addition (in the coagulation/flocculation

step) and removes the resulting floc from the water. Sedimentation at this stage in the treatment process should remove 90% of the suspended particles from the water, including bacteria. The purpose of sedimentation here is to decrease the concentration of suspended particles in the water, reducing the load on the filters. Sedimentation can also occur as part of the pretreatment process, where it is known as pre sedimentation. Pre sedimentation can also be called plain sedimentation because the process depends merely on gravity and includes no coagulation and flocculation. Without coagulation/flocculation, plain sedimentation can remove only coarse suspended matter (such as grit) which will settle rapidly out of the water without the addition of chemicals. This type of sedimentation typically takes place in a reservoir, grit basin, debris dam, or sand trap at the beginning of the treatment process.

While sedimentation following coagulation/flocculation is meant to remove most of the suspended particles in the water before the water reaches the filters, pre-sedimentation removes most of the sediment in the water during the pretreatment stage. So pre-sedimentation will reduce the load on the coagulation/flocculation basin and on the sedimentation chamber, as well as reducing the volume of coagulant chemicals required to treat the water. In addition, pre sedimentation basins are useful because raw water entering the plant from a reservoir is usually more uniform in quality than water entering the plant without such a holding basin. The rest of this lesson will be concerned with sedimentation following coagulation and flocculation. We will consider types of sedimentation basins and parts of a typical sedimentation basin, as well as the disposal of sludge. Then, in the next lesson, we will learn to design a sedimentation basin and will consider some problems which may affect sedimentation basins.

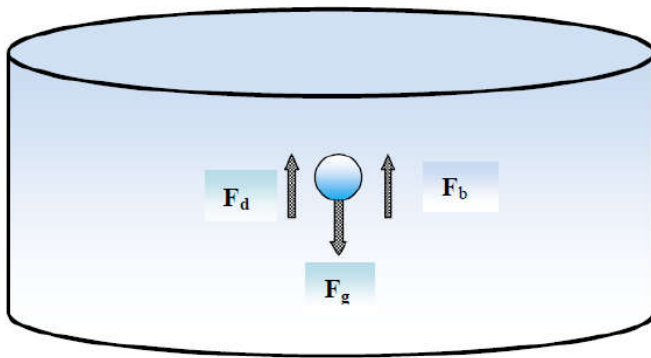
Before a basin to settle particle is designed, the settling velocity of the particles must be known. In the design of ideal sedimentation tank, first the settling velocity (V_s) of the particle to be removed has to be determined and then the overflow rate (V_o) has to be set at some value less than or equal to v_s . Physical characteristics of the particle determine its settling velocity. Some of the particles (discrete) have constant physical characteristics. Hence, in sedimentation it is necessary to differentiate between discrete particles which do not change in size, shape or mass during settling and flocculent particles which agglomerate during settling and thus do not have

constant characteristics. Based on the settling characteristics of particles, sedimentation of particles is classified in to four types:

- ✓ Type I Sedimentation (Discrete Particle Settling)
- ✓ Type II sedimentation (Flocculent Settling)
- ✓ Type III sedimentation (Zone Settling)
- ✓ Type IV sedimentation (Compression Settling)

Type I sedimentation

Type I sedimentation refers to discrete particle settling. A discrete particle is one that, in settling, is not altered in size, shape, and weight. They settle as individual particles and do not flocculate or stick to other particles during settling. Examples of these particles are sand and grit material. In falling freely through a quiescent fluid, such a particle accelerates until the frictional resistance or drag of the particle equals the weight of the particle in the suspending fluid. Thereafter, the vertical velocity of the particle with respect to the surrounding fluid will remain constant as v_s . Consider a spherical discrete particle is falling in a body of water in a quiescent condition.



The forces acting on the particle are the drag force, F_d , gravitational force, F_g and buoyant forces F_b . The effective gravitational force, F_1 , is the difference between the gravitational force, F_g , and the buoyant force F_b .

$$F_1 = F_g - F_b$$

$$= (\rho_s - \rho) g V_p \dots\dots\dots 1$$

Where

ρ_s is density of particle ρ_w is density of water V_p is volume of particle

The drag force (FD) is given by

$$F_d = \frac{C_D \rho_w V_s^2 A_s}{2} \dots\dots\dots 2$$

Where

CD: is Drag Coefficient (Dimensionless)

As: is projected area, in the direction of motion. Note that a projection of a spherical particle in any plane is circular.

Vs: is the settling velocity of the particle.

Equating F1 and FD and solving for Vs will give the settling velocity vs as:

$$V_s = \left(\frac{4g(\rho_s - \rho_w)d_s}{3C_d \rho_w} \right)^{1/2} \dots\dots\dots 3$$

$$V_s = \left(\frac{4g(S_s - 1)d_s}{3C_d} \right)^{1/2} \dots\dots\dots 4$$

Where S_s = specific gravity (relative density) of particle.

The drag coefficient CD, is not constant but varies with the Reynold's number Re, and the shape of the particle.

The Reynold number is given by

$$Re = \frac{\rho V_s d}{\mu}$$

Where:

μ is dynamic viscosity

d is diameter of particle,

CD is defined by

$$C_d = \frac{24}{Re} + \frac{3}{(Re)^{0.5}} + 0.34$$

For laminar flows with $Re < 1$, the last terms may be neglected to yield $C_D = \frac{24}{Re}$

Hence, for laminar flow range substituting the value of C_D into equation (4), gives

$$V_s = \frac{1}{18} \frac{g}{\mu} (\rho_s - \rho) d^2 \dots\dots\dots 5$$

This is called Stoke's law

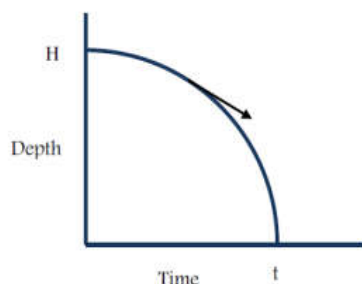
For turbulent flows with $Re > 1000$, C_D tends to a value of 0.4. Thus the settling velocity is given by:

$$V_s = (3.3gd (S_s - 1))^{1/2} \dots\dots\dots 6$$

Type II sedimentation

Under quiescent conditions suspended particles in many waters exhibit a natural tendency to agglomerate or the addition of chemical agents promotes this tendency. This phenomenon is known as flocculent or type II sedimentation. Type II sedimentation is characterized by particles that flocculate during sedimentation. Since they flocculate, their size and settling velocity will increase. In such cases the trajectory traced by a settling particle will be curvilinear because of the increase in its settling velocity. The instantaneous settling velocity is the tangent to the curve. The average settling velocity for the particle is below:

$$\bar{v} = \frac{H}{t} \dots\dots\dots 7$$



The average settling velocity distribution for the suspension is continually changing with time. In water treatment these types of particles occur in alum or iron coagulation. To design a basin for the flocculent settling, the average settling velocity distribution variation with time must be

found to calculate the total removal. There is no theoretical means of predicting the amount of flocculation and settling velocity distribution variation for a suspension i.e. no adequate mathematical relationship that can be used to describe Type II settling. The Stokes equation cannot be used because the flocculating particles are continually changing in size and shape. Laboratory tests with settling columns are used to develop design data. Settling column analysis is an important laboratory test to determine distribution of settling velocities for both discrete and flocculent sedimentation.

Type of sedimentation tank and their design

Sedimentation tanks are classified as continuous flow or intermittent flow. The continuous flow types are mostly used now days. Tanks also classified as horizontal flow when the liquid passes through in the horizontal direction and as vertical flow when the liquid enters near the bottom of the tank and is withdrawn at the surface. The vertical flow type is generally used for sewage treatment.

Sedimentation tanks are designed in different shapes including rectangular, square, or circular. They can be operated continuously or by batch, in a horizontal or vertical direction of flow. The most appropriate sedimentation tank is the rectangular, operated with horizontal flow and on a continuous basis. The flow condition in a circular tank is more complicated, and the operation of vertical-flow settling tanks is very difficult. The Intermittent tanks sometimes called quiescent type tanks are simple settling basins, which store the raw water for a certain period and keep it in complete rest. After giving it a rest of some specified time, the clear water is drawn off, and settled silt is cleaned off from the tank. The tank is again filled with the raw water to continue the next operation. This requires some period of time.

Such system necessitates the provision of more settling units and more time and labor is wasted. In a continuous flow type of sedimentation tank, the flow velocity is only reduced and the water is not brought to complete rest, as is done in an intermittent type. The working of such tank is simple, as the water enters from one end and comes out from the other end. The velocity is sufficiently reduced by providing sufficient length of travel.

Design of continuous flow type of sedimentation tank

Suppose we have a tank of length L , water depth H , width B and the inflow rate be Q as shown in the Figure 9.

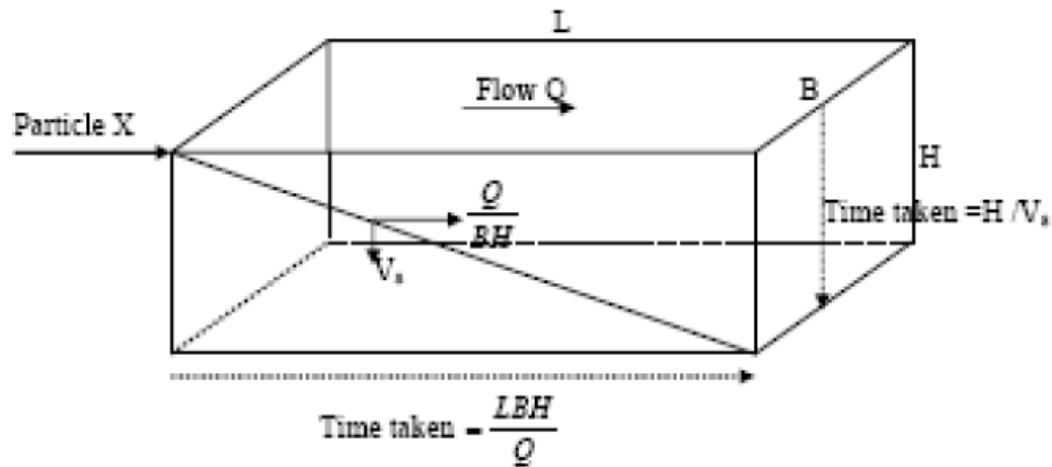


Figure 9: Theoretical flow through a rectangular sedimentation tank

It is assumed that the sediment is uniformly distributed as the water enters the basin. Let the water containing uniformly distributed sediment, enters a rectangular tank with a uniform flow velocity of V and the particle of silt entering the tank will have a vertical falling velocity of V_s . Now every discrete particle is moving with a horizontal velocity V and a down ward vertical velocity V_s .

The horizontal velocity is given by

$$V = \frac{Q}{BH}$$

And time of horizontal flow is given by

$$t_h = \frac{L}{V} = \frac{LBH}{Q}$$

The time for the falling distance is given by

$$t_v = \frac{H}{V_s}$$

For the particle to reach the bottom before the water leaves the tank the time of fall must equal the time of horizontal flow, i.e.

$$\frac{H}{V_s} = \frac{LBH}{Q}$$

From which

$$V_s = \frac{Q}{LB} = \frac{Q}{A}$$

Where A is the surface area of the tank

This is the limiting velocity of fall to enable the particle to reach the bottom of the tank. All particles with speed greater than Q/A will reach the bottom before the outlet end of the tank. In other words, no particles having a settling velocity more than or equal to Q/A will remain in suspended in such a tank. Particles with velocity less than Q/A will be removed in the same proportion with their velocity to Q/A . For example, if the speed of a particle is only half of Q/A then only fifty percent of the particles falling at this speed reach the bottom.

The quantity $Q/A = Q/BL$, i.e. the discharge per unit plan area (surface area) is a very important term for the design of continuous flow type settling tank; and is known as the overflow rate or the surface loading or the overflow velocity. Its normal value ranges between 500-750 liters/hr/m² for plain sedimentation, and between 1000-1250 liters/hr/m² for chemically aid sedimentation tank. Decreasing the overflow rate will lead to the settlement of particles which are having lower values of their settling velocities.

Another important term which is used in connection with the design of the sedimentation basin is the detention period or detention time. The detention time of a sedimentation basin may be defined as the average theoretical time required for the water to flow through the tank length. It is that time which would be required by the flow of water to fill the tank, if there were no outflows. It is given by the Equation:

$$t = \frac{LBH}{Q}$$

The detention time usually ranges between 4 to 8 hours for plain sedimentation, and from 2 to 4 hours when coagulants are used.

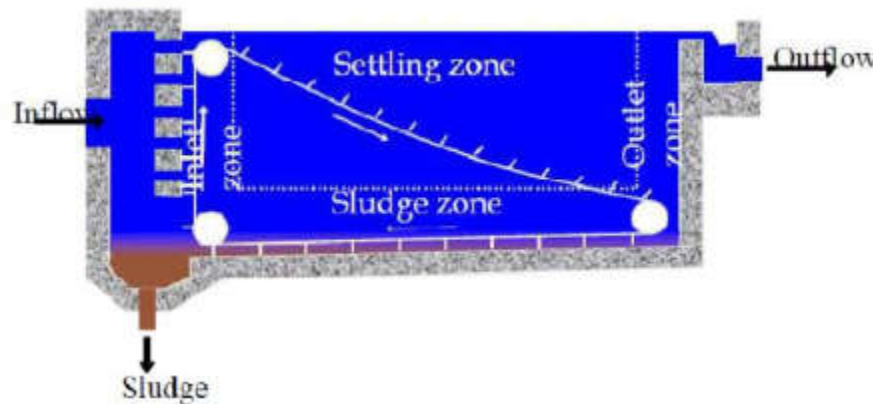


Fig 10: Layout and Design of Settling Tanks

The main features of a horizontal –flow rectangular sedimentation tanks are illustrated in Fig 3.4. The tank can be divided into different areas comprising an inlet, settling, outlet, and sludge accumulation zone. The inlet zone serves to provide even distribution over the full cross-section. The outlet zone collects the clarified water uniformly over the full tank width. Sludge is accumulated at the bottom, where it is stored and removed periodically. The most important area is the settling zone, where solid separation takes place.

Given a specified raw water quality, the efficiency of a sedimentation tank is mainly dependant on the surface loading, the tank depth, and detention time. Low surface loads should be applied for raw water with poor settling properties. Adequate weir loads will prevent formation of undesirable turbulence in the outlet zone. An appropriate length to width ratio is important to maintain laminar flow conditions and to prevent short circuits. Shallow sedimentation tanks with a depth of approximately 1.5 to 2.0m will ease manual removal of accumulated sludge. Although selection of appropriate values for the overflow rate and the detention time are important to determine the size of the basin, other parameters also have to be considered to achieve the desired removal efficiency. These include the horizontal velocity V_h and the weir overflow rate or weir loading q_w . Excessive horizontal velocity may move some of the previously settled particles toward the outlet zone. Horizontal flow velocity should not exceed 9.0m/h for light flocculent suspensions or about 36m/h for heavier discrete particle suspension.

Large weir overflow rates results in excessive velocities at the outlet. These velocities extend backward into the settling zone, causing particles and flocs which would otherwise be removed as sludge to be drawn into the outlet. Overflow rate ranging from 6m³/h per meter of weir for light flocs to about 14m³/h per meter of weir for heavier discrete particle suspension are commonly used.

Determining the Capacity of the Settling Zone

The capacity of the settling zone can be determined on the basis of over flow rate. It is assumed that the settlement of a particle at the bottom of the tank does not depend on the depth, but on the surface area of the tank.

Settling Velocity Distribution

The distribution of settling velocity for particles in a discrete suspension is determined experimentally in laboratory column test. Laboratory settling columns are usually Perspex tubes having diameter of at least 100mm and a length of about 2m, with provision for sampling at various points over the column height. Care is taken to ensure that the suspension is uniformly distributed at the start of a test.

Suppose that samples drawn off at a point 1m below the water surface at various times gave, on analysis, the results shown in table 3.1. from this set of results it can be seen that 95% of particulates have a settling velocity of less than 1m in 30min or 2m per hr and that 83% have a settling velocity less than 1m per hr, etc.

Table 4.3: Typical settling column analysis result

Time (min)	0	30	60	90	120	180	360
Concentration (mg/l)	42	40	35	29	22	10	2.5
Percentage of initial concentration	100	95	83	69	52	24	6

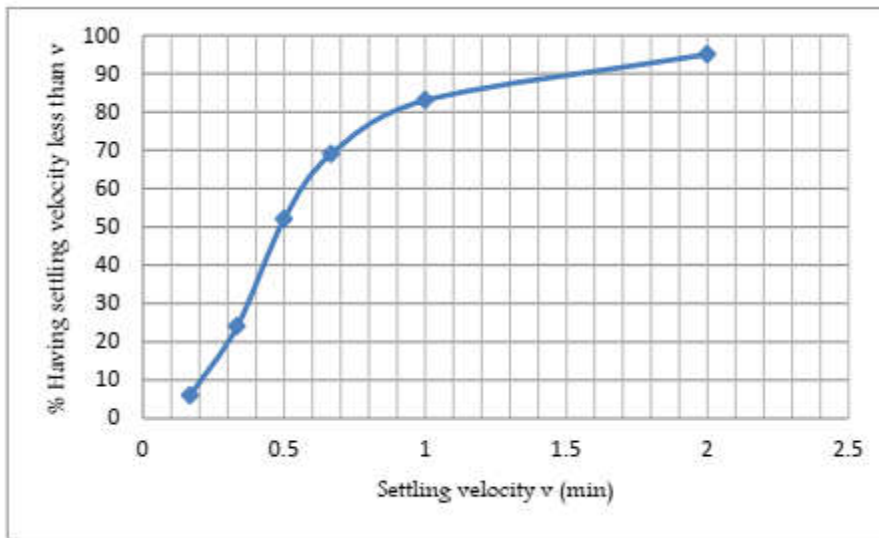


Figure 11: Settling velocity distribution for discrete particles

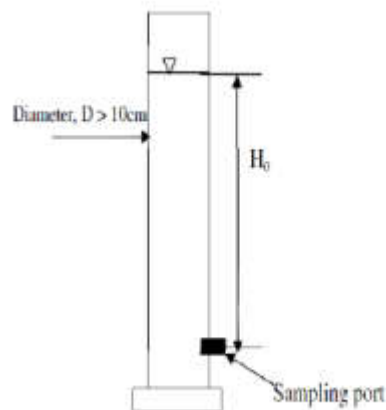


Figure 12: Settling column for analyzing type 1 suspension

For a given detention time t_0 , overall percent removal for a discrete particle can be obtained. All particles with settling velocity greater than $V_0 = H_0/t_0$ will be 100 percent removed. Thus $1 - p_0$ fraction of particles will be removed completely in time t_0 . The remaining will be removed to the ratio V_s/V_0 , corresponding to the shaded area in figure below

$$R = (1 - P_o) + \frac{1}{V_o} \int_0^{P_o} V_s dp$$

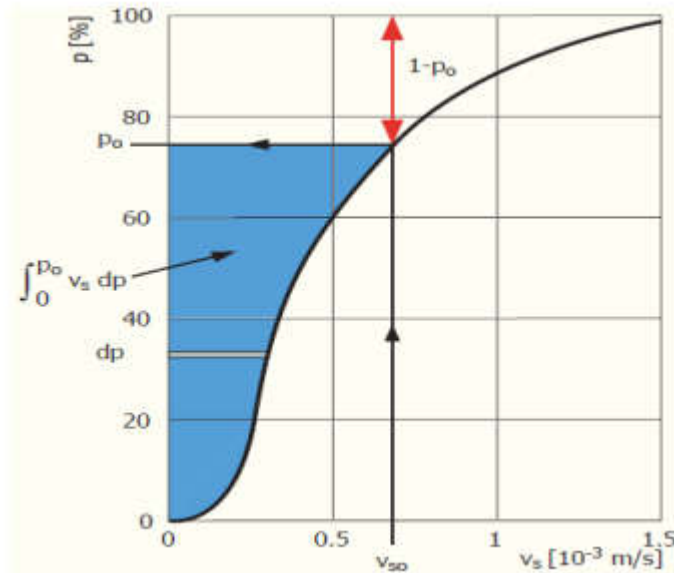


Figure 13: Particle settling velocity distribution

The cumulative distribution of settling velocity for this suspension is plotted in figure above. For truly discrete settling, the depth h of the sampling point will not affect the resultant distribution curves of the settling velocity. If the suspension is flocculent, however, a different velocity distribution will be found for each sampling point.

In many practical cases, even in relatively dilute suspensions, particles coalesce to form particle aggregates having increased settling velocities. The extent of such flocculation is a function of many variables including suspension type and concentration, the prevailing gradients, and time. The expected removal of a flocculent suspension in a sedimentation process can be estimated from laboratory settling test using a suspension column height equal to that used in the process. At various time intervals, samples are withdrawn from ports at different depths and analyzed for suspended solids. The percentage removal is computed for each sample and is plotted on a graph against time and depth. Curves of equal percentage removal are then interpolated between the plotted points, as illustrated in figure below. The resulting curves can be used to determine the overall removal of solids for any detention time and depth within the range of the data, bearing in

mind that the test conditions are quiescent. For example, for a detention period equal to t_2 and depth h_5 the removal is equal to

$$= \left[\frac{\Delta h_1}{h_5} * \left(\frac{R_1 + R_2}{2} \right) \right] + \left[\frac{\Delta h_2}{h_5} * \left(\frac{R_2 + R_3}{2} \right) \right] + \left[\frac{\Delta h_3}{h_5} * \left(\frac{R_3 + R_4}{2} \right) \right] + \left[\frac{\Delta h_4}{h_5} * \left(\frac{R_4 + R_5}{2} \right) \right]$$

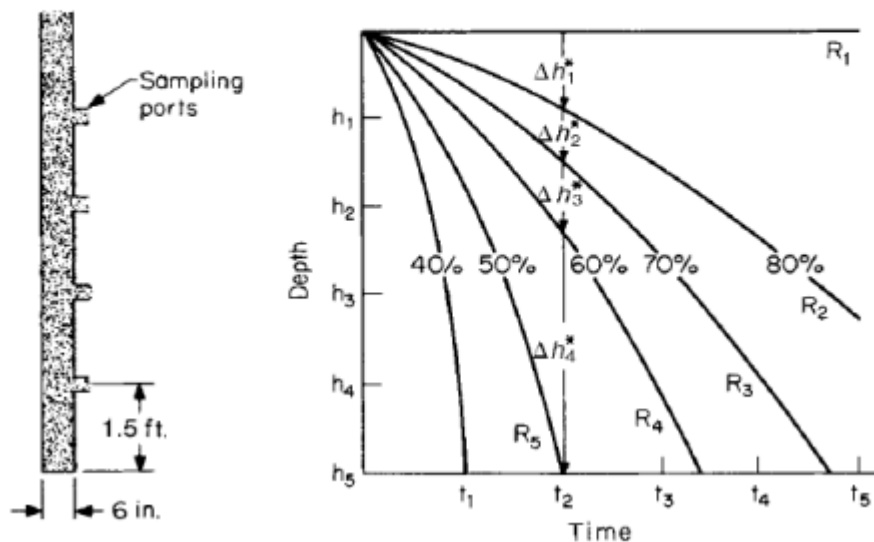


Figure 14: Settling column and isoremoval settling curves for flocculent particles

4.3.2 Coagulation and Flocculation

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.

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.

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 than colloidal particles due to the entrapment of such particles in the flocs formed during coagulation.

Although the words "coagulation" and "flocculation" are often used interchangeably they refer to two distinct processes. 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. Flocculation refers to the process by which destabilized particles actually *conglomerate* into larger aggregates so that they can be separated from the wastewater.

A large portion of the suspended particles in water are sufficiently small and their removal in a plain sedimentation alone is impossible at reasonable surface overflow rates and detention time. For example silt particle of 0.06mm size may require 10hrs to settle in 3m deep plain sedimentation tank and 0.002mm particle will require about 4 days for settling, which is impracticable because water cannot be detained for such a long time. In addition to the suspended solids, water also contains colloidal particles (colloids), which are midway in size between suspended solids and dissolved solids, which will be in continuous motion but never settle. In order to remove such fine particles from water, certain chemicals called coagulants are added and mixed in water. Colloids as a result of their small size have a very large ratio of surface area to volume.

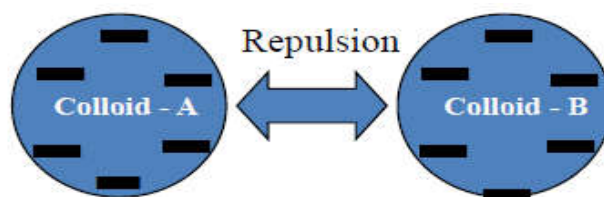


Figure 15: Colloidal stability due to electrostatic repulsion

Colloids are kept in suspension („stabilized“) by electrostatic repulsion and hydration. Electrostatic repulsion occurs because colloids usually have surface electric charge mostly negative. Colloids are continually involved in Brownian movement, which is merely random movement. Thus, the most important surface phenomena are the accumulation of electrical charges at the particles surface. This may involve the absorption of ions from solution onto the

colloidal surface and ionization of chemical groups on the surface produce net charges on the particles.

Since colloids are stable because of their surface charge, in order to destabilize the particles, we must neutralize this charge. Such neutralization can take place by the addition of an ion of opposite charge to the colloid. Coagulation essentially involves the reduction of these surface charges or destabilization of colloids by addition of positive ions. The purpose of coagulation is to alter the colloids so that they can adhere to each other. During coagulation a positive ion is added to water to reduce the surface charge to the point where the colloids are not repelled from each other. A coagulant is a substance (chemical) that is added to the water to accomplish coagulation.

Flocculation is a slow mixing process which promotes the agglomeration of destabilized particles. Agglomeration (bringing together) of particles into groups (flocs) increases the effective size and the settling velocities. There is a need to have gentle mixing so as to bring the flocs together in order to form large ones. The aggregation of these particles into large, more readily settleable aggregates is essential for successful separation by sedimentation followed by filtration. Substances that frequently are to be removed by coagulation and flocculation are those that cause turbidity and color.

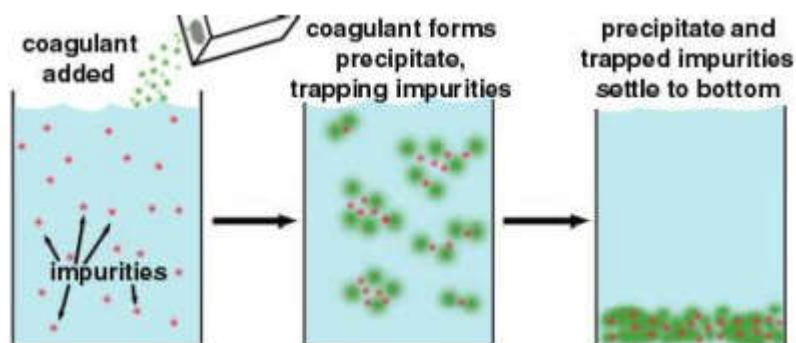
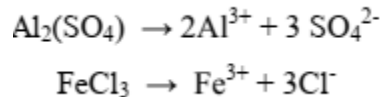


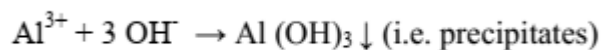
Figure 16: precipitation of colloidal impurities facilitated by addition of coagulant

The use of coagulants is generally necessary for clarifying raw waters containing turbidity and color causing materials in greater concentration. Chemicals with positive charges are used for this purpose: for example;

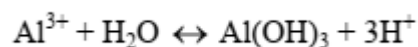


The positively charged radicals then attract the negatively charged colloids (chemical binding). The ability of an agent to coagulate water is related to its charge. The use of divalent and trivalent ions is more advantageous. One mole of a trivalent ion can reduce the charge as much as 30 to 50 moles of a divalent ion and as much as 1500 to 2500 moles of a monovalent ion.

Precipitation (Simplified hydrolysis equations): On dissolution in water



Precipitates form aggregates which enmesh (or agglomerate) the colloids. Therefore, rapid mixing is essential for immediate dispersal of the entire dose of chemicals through the mass of the raw water at this stage. Mixing has to be rapid because the hydrolysis of the coagulant is almost instantaneous (within a few seconds). The destabilization of colloids also takes very little time.



PH correction is also essential. Because due to 3H^+ ions, water will become acidic (due to decrease in PH) after coagulation and this can hinder the reaction of Al^{3+} ions if not controlled.

Factors affecting coagulation:

- ✓ Type of coagulant
- ✓ Dose of coagulant
- ✓ Characteristic of water
 - Type and quantity of suspended matter
 - Temperature of water
 - pH of water
- ✓ Time and method of mixing

Chemicals Used for Coagulation

There are three key properties of a coagulant:

1. Trivalent
2. Non-toxic
3. Insoluble in the neutral pH range.

The most common coagulants are alum (aluminum sulfate) and iron salts, with alum being the most extensively used agent. The multivalent characteristic of these cations strongly attracts them to charged colloidal particles and their relative insolubility ensures their removal to a high degree.

Generally these chemicals are most effective when water is slightly alkaline. In the absence of such an alkalinity in raw water external alkalis like sodium carbonate, or lime, etc. are added to the water, so as to make it slightly alkaline, and thus increase the effectiveness of the coagulants. Lime is usually the least expensive source of alkalinity.

The PH of coagulant is critical. Ferric salts work best in PH range of 4.5 to 5.5, whereas aluminum salts are most effective around a PH range of 5.5 to 6.3. The PH value should be attained after the coagulant is added.

For good coagulation, the optimal dose of coagulant should be fed into the water and properly mixed with it. The optimal dose will vary depending upon the nature of the raw water and its overall composition. A laboratory experiment called the „Jar test“ is generally used to determine the optimal dose. The „Jar test“ may be briefly described as follows:

- ✓ A series of samples of water are placed on a special multiple stirrer and the samples are dosed with a range of coagulant, e.g. 10, 20, 30, 40, and 50mg/l; they are stirred vigorously for about one minute.
- ✓ Then followed a gentle stirring (10 minutes), after which the samples are allowed to stand and settle for 30 to 60 minutes. The samples are then examined for colour and turbidity and the lowest of coagulant which gives satisfactory clarification of the water is noted.
- ✓ A second test involves the preparation of samples with the PH adjusted so that the samples cover a range (e.g. PH = 5; 6; 7; and 8). The coagulant dose determined

previously is added to each beaker. After this, the samples are examined and the optimum PH is determined.

- ✓ Aluminum sulfate the most commonly used coagulant in water purification, is most effective between PH ranges of 5.0 and 7.5. Ferric chloride, effective down to PH 4.5, and ferrous sulfate, effective only above PH 8.5



Figure 17: Jar test set-up

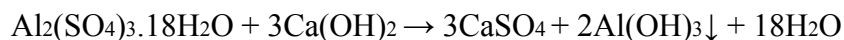
Use of Alum as Coagulant

Alum is the name given to the aluminum sulfate with its chemical formula as $\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O}$. Alum when added to raw water reacts with the bicarbonate alkalinities, which are generally present in raw supplies, so as to form a gelatinous precipitate (flock) of aluminum hydroxide. This flock attracts other fine particles and suspended matter, and thus grows in size, and finally settles down to the bottom of the tank. The chemical equation is



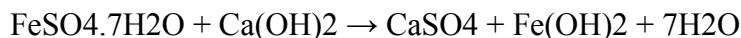
The amount of alum required for coagulation depends on the turbidity and colour of the raw water. The use of the optimum amount of a coagulant is indicated by the formation of large feathery flakes; and can be approximately determined by jar test. Alum dosage may range from 5mg/l to 50mg/l depending upon the turbidity and nature of the raw water.

If the raw water has no sufficient alkalinity, then external alkalis like lime ($\text{Ca}(\text{OH})_2$) or soda ash (i.e. Na_2CO_3) are generally added, and the following reactions take place:

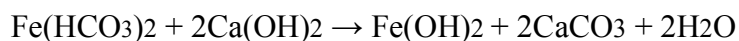
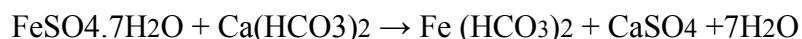


Use of Copperas as Coagulant

Copperas is the name given to ferrous sulphate with its chemical formula as $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$. Copperas is generally added to raw water in conjunction with lime. Either lime or copperas may be added first. When lime added first, the following reaction takes place



Similarly when copperas is added earlier to lime, the reaction that takes place is:



The ferrous hydroxide formed in either case, further gets oxidized, forming ferric hydroxide, as given below.



The ferric hydroxide forms the floc, and thus, helps in sedimentation. Copperas is extensively used as a coagulant for raw waters that are not colored. It is generally cheaper than alum, and functions effectively in the pH range of 8.5 and above.

Comparison of Alum and Iron Salts as coagulants

- The alum and iron salts are having their own advantages and disadvantages.
- Iron salts produce heavy flocs and can be, therefore, remove much more suspended matter than the alum.
- Iron salts being good oxidizing agents, can remove hydrogen sulphide and its corresponding tastes and odour from water.
- Iron salts can be used over a wider range of PH values.

- Iron salts cause staining and promote the growth of iron bacteria in the distribution system.
- Iron salts impart more corrosiveness to water than that which is imparted by alum.
- The handling and storing of iron salts require more skill

Mixing Mechanisms

The principal objective of mixing process as applied to water treatment is to obtain uniform dispersion of coagulant in the main flow of water and to enhance the agglomeration of destabilized colloids. The mixing process can be related to the coagulation and flocculation processes as rapid mixing and slow mixing.

Rapid Mixing

After the addition of the coagulant to the raw water, the mixture is thoroughly and vigorously mixed, so that the coagulant gets fully dispersed into the entire mass of water. The addition and mixing of chemicals to the main flow of water is a continuous process and is frequently described as either rapid or flash mixing. The methods used for mixing can be hydraulic or mechanical.

The design of mixers is often based upon the concept of velocity gradient and its value is used to express the degree of mixing at any point in the liquid system. The velocity gradient G is defined in terms of power input by the following relationship.

$$G = \left(\frac{P}{\mu V} \right)^{1/2}$$

Where, G = velocity gradient (S^{-1})

P = the useful power input (w) V = volume (m^3)

μ = absolute viscosity (NS/m^2)

V can be taken as the flow rate multiplied by the residence time in the mixer. Mixing efficiency is directly related to the local flow turbulence created and should give a high degree of chemical

in water homogeneity within a short time, with low absorption of power. Mixing design should aim to achieve the optimum combination of maximum turbulence and low power input.

Hydraulic Mixing: It makes use of the turbulent created due to the loss of head across an obstruction to flow such as bend, valves, or sudden drop in level. The best results are usually obtained by introducing a chemical immediately upstream of the point of turbulence. The usual power input for mixing is based upon a typical mixing time of 2 to 3 seconds, with a G value of about 800 to 1000s⁻¹. The power is related to the head loss by the equation:

$$P = \rho g Q h = \gamma Q h$$

Where: P = useful power input (W); ρ = density of fluid (kg/m³)

Q = flow rate (m³/s); g = acceleration due to gravity (m/s²)

h = head loss (m); γ = unit weight of water (N/m³)

In practice it is generally found that adequate mixing is obtained with a head loss of between 0.25 to 0.40m, in any case it should not be less than 0.20m. Hydraulic mixers are usually simple and particularly suitable where some head loss can be tolerated. They have the advantages of having no moving parts or direct power consumption, and maintenance is therefore negligible.

Mechanical Mixing: It is achieved in purpose built chambers equipped with mechanical rotary mixers such as turbines or propellers or paddles. Typical residence times are of the order of 20 to 30 seconds and the velocity gradient value G varies 500 to 600s⁻¹. Mechanical mixers have the advantage that they produce thorough mixing with minimal head loss and are not affected by flow variations.

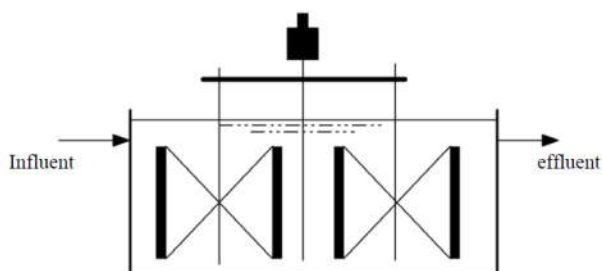


Figure 18: Mechanical mixer

Slow mixing

The process of agitating the agglomeration of destabilized colloids in chemical assisted sedimentation is done with the provision of prolonged, continuous, and gentle mixing mechanisms. The best floc will be formed when the coagulation process is followed by a relatively slow and gentle mixing to permit built up and agglomeration of the floc particles. Slow mixing is the hydrodynamic process which results in the formation of large and readily settleable flocs by bringing the finely divided matter into contact with the micro-flocs formed during rapid mixing. The operation of slow mixing is achieved in a basin commonly known as the flocculator. The rate of agglomeration or flocculation is dependent upon several factors such as type and concentration of turbidity, type of coagulant and its dose, and the temporal mean velocity gradient G in the basin. If the retention time in a flocculation chamber is t , then the extent of flocculation which takes place, or the number of particles collision which occur is a function of the dimensionless expression G_t which is given by the equation:

$$G_t = \frac{1}{Q} \left(\frac{PV}{\mu} \right)^{\frac{1}{2}}$$

Where, G_t = velocity gradient (dimensionless)

P = the useful power in put (W); V = volume (m^3)

μ = absolute viscosity (Ns/m^2); t = time (second)

Q = Flow rate (m^3/s)

For the common coagulants of aluminum and iron salts the value of G for the flocculation is usually in the range of 20 to 75 with retention time in flocculation chambers varying from about 10 to 60 minutes.

The agitation required for flocculation is usually provided by either hydraulic or mechanical means. The most common hydraulic flocculator is the baffled basin equipped with either horizontal or over and-under flow baffles.

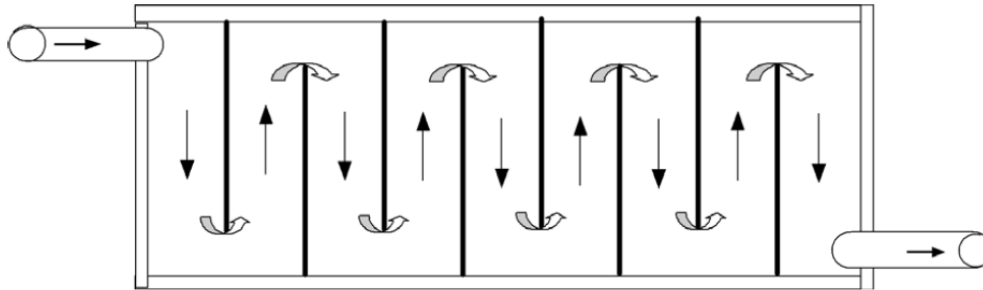


Figure 19 : Slow mixing

After determining the power input from an assumed Gt value the required head loss can be calculated from:

$$P = \rho g Q h = \gamma Q h$$

The advantage of the baffled basin is its simplicity because there is no mechanical or moving equipment. A disadvantage is that most head loss occurs at the 180° bends and therefore the value may be too high at bends and inadequate in straight channels for efficient flocculation. A second disadvantage is that the G value will vary as the flow Q varies.

Design criteria: - the velocity of flow in the channel between baffles is controlled to a value of 1.15 to 0.45 m/s. The detention period is normally kept between 20 to 50 minutes. To permit easy cleaning of the channels, the clear opening between the baffles should not be less than 45 cm. The clear opening between the end of each baffle and the tank wall (or roof or floor as the case may be) should be kept about 1.5 times the distance between the baffles, subject to a minimum value of 60 cm.

There are several types of mechanical device for flocculator, the most common being the paddle type which mounted either horizontally or vertically in the flocculating chamber. The power input is given by the equation:

$$P = F_D V = \frac{C_D \rho A V^3}{2}$$

Where, P = the power input (W); F_D = drag force (N); C_D = drag coefficient

A = submerged area of the paddles (m²)

V = Relative velocity of the paddle (m/s)

V can be approximated by 0.75 times the peripheral velocity of the paddle or equals $1.5Jrn$, where r is the effective radius of the paddle (m), n is the number of revolution per second. CD is about 1.8 for a paddle flocculator. The speed of rotation of the paddles varies from 2 to 15 revolutions per minute and the peripheral velocity of the paddles range 0.2 to 0.8m/s. Values of G from 10^4 to 10^5 are commonly used, with t ranging from 10 to 30 minutes. Large G values with short times tend to produce small dense flocs, while low G values and long times produce large, lighter flocs. Since large, dense flocs are easily removed in the settling basin. Thus, it may advantageous to vary the G values over the length of the flocculation basin. The small dense flocs produced at high G values subsequently combined into large flocs at lower G values.

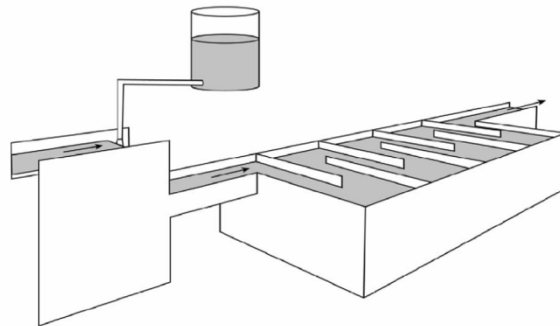


Figure 20 : Coagulant Dosing and Flocculator

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

$$G = \sqrt{\frac{P}{\mu \cdot V}}$$

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.

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.

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

4.3.3 Slow sand filter

The effluent obtained after coagulation-flocculation process does not satisfy the drinking water standards and is not safe or palatable. So it requires further treatment. Filtration is one of such a process that produces clear and sparkling water with negligible turbidity.

Filtration is the purification process whereby water to be treated is passed through a porous medium. Filtration is employed in water treatment, with or without pretreatment by coagulation and sedimentation for removal of solids present in water, precipitated hardness from lime softened water, and precipitated iron and manganese present in many well water supplies. During this passage, water quality improves by:

- Removal of suspended and dissolved solids content
- Removal of floating and colloidal matter
- Reduction of bacterial and other pathogenic microorganisms
- Change in its chemical constituents

The overall removal of impurities which is associated with the process of filtration is mostly brought by the following combination of phenomena:

- ✓ Mechanical straining
- ✓ Sedimentation
- ✓ Adsorption
- ✓ Chemical activities
- ✓ Biological activities

A) Mechanical straining

This is the process involving the removal of particles of the suspended matter that are too large to pass through the interstices. It takes place at the top surface of the filter bed and is generally independent of the filtration rate. During filtering or filter run mechanical straining becomes more effective due to depositions which reduce pore sizes until total clogging hinders any further

filtration. The process is not able to retain colloids which are in the size range of 0.01 to 10 μ m or bacteria.

B) Sedimentation

This process removes particulate suspended matter of finer sizes than the pore openings by precipitation upon the surface of the sand grain.

C) Adsorption

This is the most important purifying process in rapid sand filter (RSF), removing finely divided suspended matter as well as colloidal and molecular dissolved impurities. The forces of adsorption exert their influence over extremely short distances only. Thus purification by adsorption is only possible after another mechanism has brought the impurities to be removed in the immediate vicinity of the filter grain surface. The most important of such mechanisms are gravity, inertia; diffusion, hydrodynamic force, electrostatic attraction, and Vander Waal's force are important agents of adsorption.

D) Chemical activities

This is the process by which dissolved impurities are either broken down into simpler, less harmful substances or converted into insoluble compounds which can be removed by any of the previous three processes. This may include:

- Oxidation of organic matter to CO₂
- Oxidation of ammonia to nitrate
- Oxidation of Fe²⁺ and Fe³⁺
- Oxidation of Mn²⁺ and MnO₂

All these processes consume O₂ and take place on surfaces of the filter grains.

E) Biological activities

These are the most predominant in slow sand filter (SSF) which forms the living quarters for organisms at the top of the filter bed usually called schmutzdecke. The biochemical activities of

Microorganisms living here result into high improvements in bacteriological quality of water being filtered.

There are two types of filtration commonly used as a main treatment method for municipal water supply. These are slow sand filtration and rapid sand filtration. A third type of a rapid sand filter works under pressure and is known as a pressure filter. This type of filters are generally used for small plants, such as for individual industrial supplies, and are generally not adopted for treating large scale municipal supplies.

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

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.

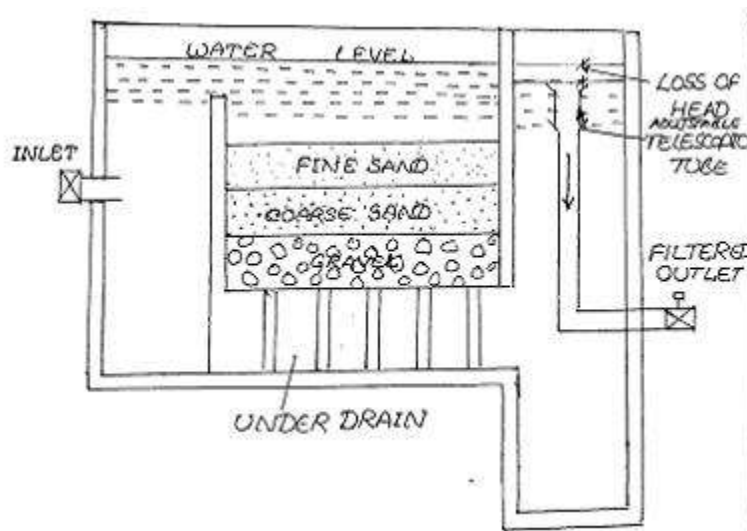


Figure 21 : Slow sand filter

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 3.6m³/m²/day. This rate of filtration is continued until the difference between the water level on the filter and in the inlet chamber is slightly less than the depth of water above the sand. The difference of water above the sand bed and in the outlet chamber is called the loss of head. During filtration as the filter media gets clogged due to the impurities, which stay in the pores, the resistance to the passage of water and loss of head also increases. When the loss of head reaches 60cm, filtration is stopped and about 2 to 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 odors, tastes and colors 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.

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 150 m³/m²/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 valve „V1“ is closed and the water is drained out from the filter leaving a few centimeter depth of water on the top of sand bed. Keeping all valves closed the compressed air is passed through the separate pipe system for 2-3 minutes, which agitates the sand bed and stirrer it well causing the loosening of dirt, clay etc. inside the sand bed. Now valve „V4“ and „V5“ 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.

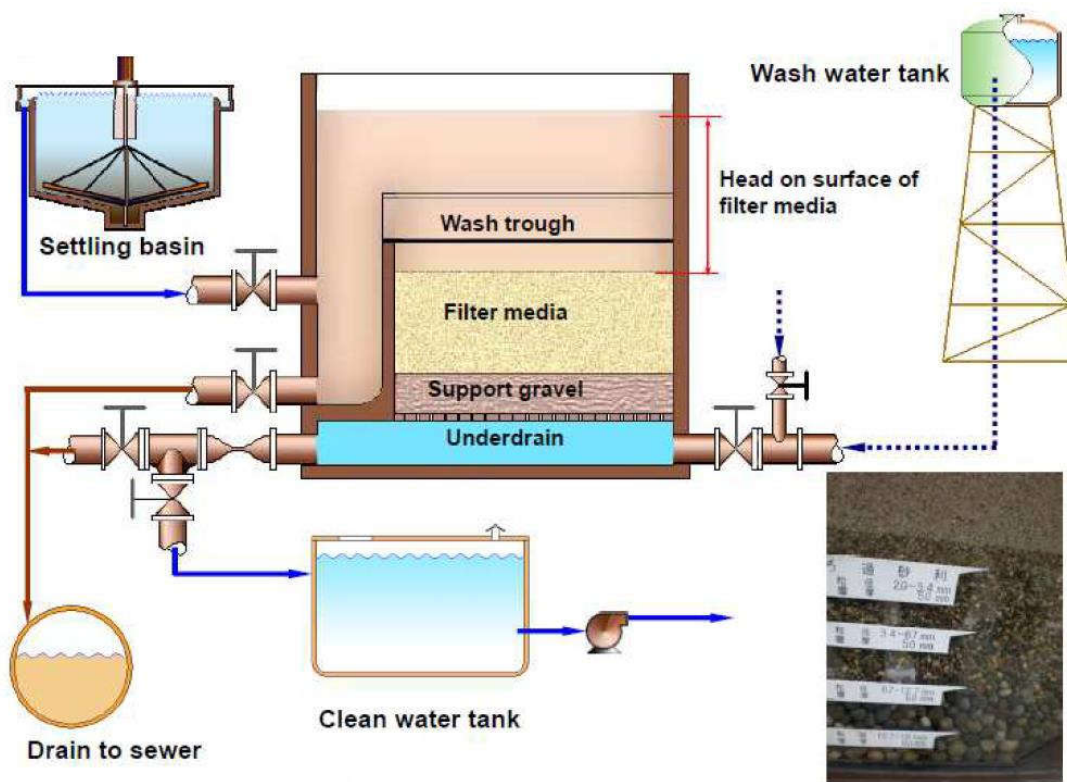


Figure 22: Rapid sand filter

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 - 48 hours and it takes 10 minutes and during back washing the sand bed expands by about 50%.

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

Comparisons of SSF and RSF

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 10 NTU 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 3.6 m 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.

4.3.6 Disinfections

Water contains many microorganisms, some of which cause disease. Water treatment process like coagulation, flocculation, sedimentation, and filtration assist in removing microorganisms. Typical bacterial reductions in coagulation flocculation sedimentation are 60 to 70 percent and addition of filtration process increases the overall removal to close to 99 percent. However, there are possibilities for some harmful bacteria to pass through the treatment plant or might enter the water after treatment. For this reason, public water supplies are treated with disinfectants to provide additional protection against transmission of disease.

Disinfection refers to the process of killing or inactivation of pathogenic organisms as opposed to sterilization which means killing all living organisms. In the process, coliform bacteria and other indicator species will be killed and the total bacterial count will be substantially reduced. Complete sterilization of water is not ordinarily sought nor achieved in disinfection processes. Disinfection is the final step in the treatment process and is necessary to provide a “bacteriological safe” drinking water for the public. Disinfection is now required for all public water supplies. Chlorination is the most common means of killing disease-causing bacteria in water supplies. There are two kinds of disinfection: primary disinfection achieves the desired level of microorganism kill or inactivation, while secondary disinfection maintains a disinfectant residual in the finished water that prevents the re-growth of microorganisms.

The purposes of disinfection are:

- To kill pathogens in the water, Germicidal effect
- To prevent any recontamination of the distribution system, Residual effect

A good disinfectant must be toxic to microorganisms at concentrations well below the toxic thresholds to humans and higher animals. Additionally, it should have a fast rate of kill and should be persistent enough to prevent re-growth of organisms in the distribution system. Moreover it should be easy to store, transport and handle.

Method of Disinfection

Primary methods of disinfection are heat treatment, chlorination, chloramines, ozone, and ultraviolet light. Other disinfection methods include chlorine dioxide, potassium permanganate, and nano-filtration.

1. **Heat Treatment (Boiling):** - applicable for household but not for centralized public water supplies.

2. **Ultra-violet (U.V) Radiation:** - suitable for application to relatively small amounts of rather pure water. It has the following disadvantages.

- Absorption capacity of water is limited (i.e. fit for thin or shallow layers only)
- Influence of other substances which can absorb U.V rays like organic matter and suspended solids if water is not pure.
- Does not have long lasting effect.

3. **Ozone treatment:** - Ozone gas (O_3), which has to be manufactured on site, and having 3 atoms to each molecule, is a powerful oxidizing and disinfecting agent. It is formed by passing dry air through a system of high voltage electrodes.

4. **Chemical method:-** These includes the application of oxidants like active chlorine in the form of:

- Chlorine gas (Cl_2)
- Solution, sodium hypochlorite, ($NaOCl$)
- Solid/granules as calcium hypochlorite, $Ca(OCl)_2$
- Chloramines

Other oxidants which can be used are:

- $KMnO_4$ (Potassium permanganate)
- Br_2 (bromine)
- I_2 (Iodine)

Disinfection by Chlorination

Chlorine is the most widely used disinfectant because it is readily available, easily applied, and cheaper than other oxidizing agents such as potassium permanganate ($KMnO_4$), chlorine dioxide

(ClO₂), or ozone (O₃). Chlorine is applied in one of three forms; chlorine gas, chlorine powder (HTH), or an aqueous solution like chlorine bleach.

Chlorine Gas

Chlorine gas (Cl₂) is compressed into a liquid for storage. It can be purchased in cylinders containing 150 or 2000 pounds of the liquefied gas. Chlorine gas is cheaper per pound than either of the other forms.

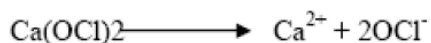
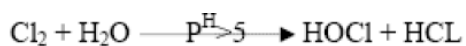
Chlorine Powder

Chlorine in its dry form is calcium hypochlorite [Ca(OCl)₂]. It is also most commonly known by the trade name HTH (High Test Hypochlorite). Only about 65 – 70% of the HTH is available as chlorine. The rest is calcium, which is not a disinfectant. Dry chlorine is 2-3 times more expensive, per pound of chlorine, than chlorine gas.

Chlorine Bleach

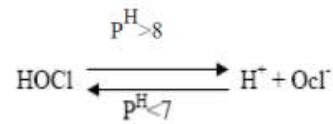
Chlorine bleach is a liquid solution of sodium hypochlorite (NaOCl). Bleach is usually 3 – 12% available chlorine and 88 – 97% water. Bleach is the most expensive form of chlorine and is normally used for disinfecting small wells and water lines. It is sometimes used for supply disinfection in very small water systems.

Chlorine may be applied to water in gaseous form (Cl₂) or as an ionized product of salts such as sodium or calcium hypochlorite. These will ionize in water to produce hypochlorous acid or hypochlorite ions as shown in the following chemical equations.



The hypochlorous acid (HOCl) and the hypochlorite ion (OCl⁻) in the above reactions are

Further related by:



The relationship is governed primarily by PH and temperature. The dissociation of hypochlorous acid into ions is more effective at high PH value and vice versa.

At low concentration, chlorine kills microorganisms by penetrating the cell and reacting with the enzymes and proptoplasm. At higher xconcentrations, oxidation of the cell wall will destroy the organism. Factors affecting the process are:

- Form of chlorine
- PH
- Concentration
- Contact time
- Type of organism
- Temperature

The total of the molecular chlorine, hypochlorous acid (HOCl), and hypochlorite ion (OCl) in water is called free available chlorine. Both hypochlorous acid and hypochlorite ion are disinfectants. But hypochlorous acid is more effective than the hypochlorite ion by approximately two orders of magnitude.

Chlorine is the very string oxidizing agent and, when added to water in the molecular form, will react with both organic and inorganic materials which are present in the water. On reaction with ammonia or amines, it will form chloramines which, while still providing reasonable disinfecting action. Chlorine in the form of chloramines is called combined available chlorine. The reaction of chlorine with ammonia occurs as follows:

Chlorine Dosage

The chlorine dosage is the amount of chlorine that is added to the water. The dosage can be determined from the number of kilograms of chlorine used and the number of millions of liters of water treated.

Chlorine Demand

Chlorine is a very reactive oxidizing agent. It will react with a certain substances that may be found in water. This list includes; iron, manganese, hydrogen sulfide, organic compounds and ammonia. When chlorine reacts with these substances, it loses its disinfecting properties. This is referred to as the chlorine demand. For chlorine to be effective as a disinfectant, the dosage must always exceed the demand that is present in the water. The chlorine demand may vary from day to day in a surface water supply. It is usually fairly constant in a ground water supply.

Chlorine Residual

The chlorine that remains in the water, after it has finished reacting with those substances that represent the demand, is known as the chlorine residual. The concentration of the residual is determined by subtracting the demand from the dosage.

Disinfection Requirements

Two factors must be taken into consideration when disinfecting drinking water. First, enough chlorine must be added to reach a predetermined concentration in the water. Then the bacteria must come in contact with the solution for a certain period of time. This is referred to as achieving the proper residual and contact time. Killing pathogenic bacteria requires a minimum of 0.2- 0.4 milligrams per liter (mg/l) of free chlorine residual and a contact time of 20 minutes. The contact time can be reduced if the residual is increased. Viruses, Giardia, and Cryptosporidium are harder to destroy than the other waterborne diseases. Free residuals of 1.5- 2.0 mg/l and much longer contact times may be required to destroy these organisms.

Effects of Temperature and PH

Changes in temperature and pH of the water can reduce the effectiveness of chlorine. Colder temperatures slow down reaction times requiring higher concentrations and longer contact times to achieve proper disinfection. A high pH impedes the formation of the hypochlorous acid and requires a higher dosage to obtain the proper residual.