Chapter 6

WATER TREATMENT PROCESS

6.1 Introduction

Water treatment describes those processes used to make water more acceptable for a desired end use. These can include use as drinking water, industrial processes, medical and many other uses. The goal of all water treatment process is to remove existing contaminants in the water, or reduce the concentration of such contaminants so the water becomes fit for its desired end-use. One such use is returning water that has been used back into the natural environment without adverse ecological impact. Absolute pure water is not found in nature. In fact absolute pure water (tasteless) is not always desired. Pure water is a relative term according to its use. As condensed waterfalls, its sweeps up other material from the air, and becomes still more contaminated on reaching the ground.

The objective of water treatment is to produce an adequate and continuous supply of water that is chemically, bacteriologically and aesthetically pleasing. More specifically, water treatment must produce water that is:

- **Palatable:** that is, has no unpleasant test;
- **Safe:** it should not contain any pathogenic organism or chemical that could be harmful to the consumer;
- **Clear:** be free from suspended matter and turbidity;
- **Colorless and odorless:** be aesthetic to drink;
- **Reasonably soft:** to allow consumers to wash clothes, dishes and themselves without excessive use of detergents or soaps;
- **Non-corrosive:** water should be corrosive to pipe work or encourage leaching of metals from pipes or tanks;
- **Low in organic content:** a high organic content will encourage unwanted biological growth in pipe or storage tanks, which can affect the quality of the water supplied.

6.1.1 Potable and palatable water

- **Potable water:** It means the water which is physically, chemically, and bacteriologically acceptable to use. Therefore, it is
  - Safe to drink as it is free from disease producing organisms and harmful chemicals.
  - Pleasant to taste as it is free from physical and some chemical impurities.
  - Sustainable for domestic uses like cooking and washing as it is free from hardness and iron.

- **Palatable water:** It is free from turbidity, color, taste, odor, and moderate temperature. The palatable water is physically acceptable but not necessary to be acceptable as chemically or bacteriologically.

The differences between potable and palatable water in the Table 6.1

Table 6.1: **Difference between potable and palatable water**

Potable Water	Palatable Water
• Free from disease producing organism and harmful chemicals.	• Free from turbidity, color, taste, odor and of moderate temperature.
• Free from physical and chemical impurities.	• Free from physical impurities.
• It is suitable for domestic use like cooking, drinking, washing etc.	• It is suitable for public use like clearing streets, gardening etc.
• It is physically, chemically and bacterologically safe.	• It is physically acceptable

6.2 Classification of Impurities

Impurities in water may be classified according to the source of origin and form of presence (Table 6.2):

- **Type and origin**
  - Inorganic and mineral origin
  - Organic origin
  - Living impurities
  - Radioactive impurities.
- **From of presence**
  - Suspended
  - Colloidal
  - Dissolved impurities including dissolved gases.

6.3 Physical Impurities

6.3.1 Turbidity

Turbidity is the cloudiness or haziness of water caused by individual particles

Table 6.2: **Impurities in water**

Form	Inorganic	Organic	Living
Suspension: (> 1μ)	Silt, clay (color, turbidity)	Fats, oils, leaves, animals & plant, organic coloring materials (color turbidity, odor, help the growth of pathogen)	Algae, fungi etc (color, turbidity, odor)
In colloidal: viruses suspension (1μ-$10^{-3}\mu$)	Silica, alumina, iron Oxidized etc. (color turbidity)	Animal excretion and decomposable Organic matter (color, odor, help the growth of pathogens)	Bacteria, protozoa, (some produce diseases)
In Solution: ($<10^{-3}$)	Many organic matters like organic acid, pesticides etc. remain in solution. They produce color, taste, odor and toxicity. Most of the inorganic minerals remain in solution.		

(suspended solids) that are generally invisible to the naked eye, similar to smoke in air. The measurement of turbidity is a key test of water quality. It is a measure of presence of suspended solid materials. It decreases the clarity of water and occurs in the most surface waters as a result of suspended clay, silt, and soil particles and other finely divided organic and inorganic colloidal matter, plankton (algae) and micro-organisms. The turbidity test measures an optical property of water sample which result from the scattering and absorbing of light by the particulate matter present, which depends, in a complex manner on such factors as the number, size shape and relative index of foreign material. Turbidity is expressed as Jackson (JTU) Turbidity unit (1 mg/l of S_2O_2 = 1 unit of turbidity) or Nephelometric Turbidity Unit (NTU) that measures the intensity of light scattered 90° to the path of incident light. The 90° receptor is a silicon photo detector aligned to receive the light scattering from the suspended particles. The amount of light received by the photo detector is linearly proportional to the turbidity of the sample (for samples with turbidity value of less than 50 NTU). The design of the cuvet holder and cuvet cap minimize the amount of stray light in this system. A suspended of Formazine (formed by the infraction under specified conditions of 50mg/l of Hydrazine Sulfate with 500 mg/l of Hexamethylamine Tetramine), has a defined turbidity of 40 NTU. These are also known as Formazine Turbidity Unit (FTU).

The Infrared Method (ISO 7027) of measuring turbidity indicates that the use of HI93703 and LP2000 Hanna turbidity meters operate by passing a beam of infrared light through a vial containing the sample to be tested. The light source is a High Emission Infrared LED. A sensor, positioned at 90° with respect to the direction of the light, detects the amount of light scattered by the undissolved particles present in the sample (Figure 6.1 a). A microprocessor converts such readings into FTU (NTU) values.

Moreover, employing the USEPA accepted nephelometric principle; a pure green LED is used as a light source.

The turbidity can be measured using Spectrophotometer (Figure 6.1 b) as well. The levels of turbidity is shown in the Figure 6.1 (c).

Sanitary significance

- **Aesthetic:** Turbidity in excess of 5 NTU is generally detectable to consumer. Any turbidity is associated will pollution.
- **Filterability:** Filtration is more difficult and costly.
- **Disinfection:** Interfere in disinfection due to lack of contact between disinfecting agent and bacteria. Turbidity can be removed by sedimentation, filtration and coagulation.

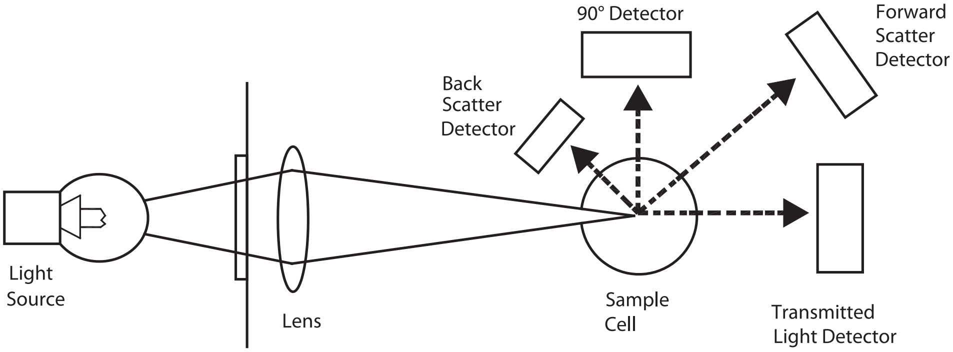


Figure 6.1 (a): Turbidimrter

6.3.2 Color

True color is caused by materials in solution and should be distinguished from turbidity (materials in suspension) which may cause an apparent color. The

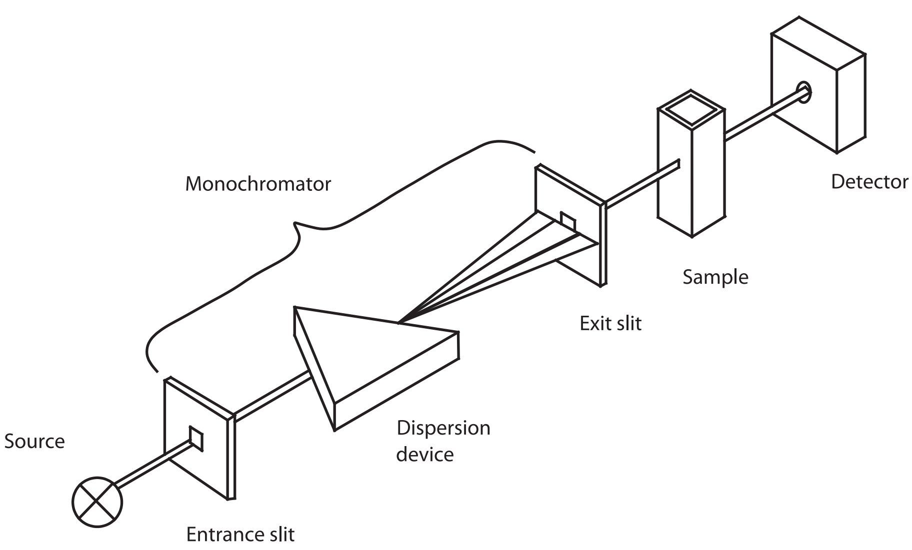


Figure 6.1 (b): Mechanism in Spectrophotometer to measure turbidity

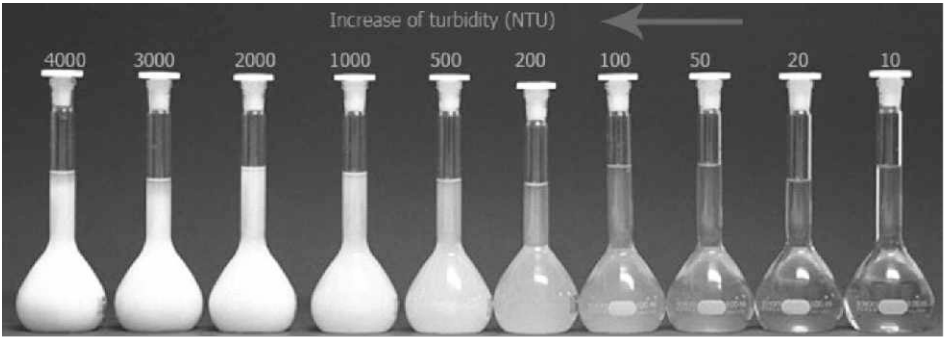


Figure 6.1 (c): Levels of turbidity in NTU/FTU

color in tea is an example of organic colloidal color. True color may be due to presence of dyes derived from decomposing vegetation, other organic wastes and highly colored industrial wastes. Colored water is mainly undesirable for its appearance and may discolor clothing's and adversely affect industrial processes. Moreover chlorinated organic compounds may be formed during chlorination of colored water. Color is expressed in color units and is measured by comparison to platinum Cobalt Standard One TCU (true color unit) correspond to the amount of color exhibited under specific test conditions by containing 1 mg/l of Platinum in the form of chloroplatinate ion. A color of 15

TCU can be detected in glass of water by most consumers.

The differences between color and turbidity are presented in the Table 6.3

Table 6.3: Differences between Color and Turbidity.

Color	Turbidity
1. Caused by materials in solution or in a colloidal state.	1. Caused by suspended matter like clay, salts, soil particles, Finely divided organic and colloidal matter, plankton and micro organism.
2. Caused true color.	2. Caused apparent color.
3. Measured/expressed in true color units (TCU). A value of more than 15 TCU is objectionable to the consumer	3. Expressed in terms of Nephelometric turbidity (NTU). A turbidity in excess of 5 NTU is generally objectionable to the consumer.
4. TCU is measured by measuring the wavelength of the incident light exhibited by the solution.	4. Turbidity test measures property of the water sample resulting from absorbing and scattering of incident light.
5. Colored water is undesirable for its appearance, may discolor clothing and aduersely affect industrial process.	5. Turbidity decreases the clarity of water.

6.3.3 Alkalinity

The alkalinity of water is a measure of its capacity to neutralize acids. The carbonates and bicarbonates of Ca, Na, and Mg are the common impurities that cause alkalinity. Because the alkaline salts are very common in the ground. Most of the sources of waters are more or less alkaline. Caustic alkalinity is due to free hydroxyl ion and does not occur naturally in potable waters. Caustic alkalinities are accompanied by high pH values, 9.5 or higher. Alkalinity is expressed in mg/l in terms of equivalent $CaCO_3$. Excessive or insufficient alkalinities interfere with water treatment (coagulation).

6.3.4 Acidity

The most common cause of acidity in water is CO_2 (present as H_2CO_3). Acidity is measured in terms of the calcium carbonate (mg/l) needed to neutralize the H_2CO_3.

6.3.5 pH

It is a measure of the concentration of free hydrogen ion in water. Increasing acidity leads to higher values of (H^+), thus to lower values of pH. pH is important in the control of a number of water treatment processes.

6.3.6 Odor and Taste

Tastes in water, except those caused by salts, are practically indistinguishable from odor. Odor in water are caused by

- dissolved gasses (like H_2S)
- organic matter derived from algae and other micro-organisms
- decomposing organic matter and volatile chemicals
- industrial wastes (like phenol)
- Cl, Fe, Mn etc.

The odor intensity of water is usually measured in terms of its Threshold Odor Number (TON), which is defined as the geometric mean of the dilution ratios with odor-free water, the odor of which is just detectable by a panel of judges under very carefully controlled test condition. Water must be in offensive to most consumers. Maximum recommended TON =3.

$$\text{TON} = \frac{(A+B)}{A}$$

where,

A = volume of odorous water (ml) and

B = volume of odor-free water (ml) to produce a 200 ml mixture.

The analogy between taste and odor are presented in the Table 6.4

Table 6.4: Analogy between Taste and Odor

Taste	Odor
• It is caused due to pollution by acid and industrial waste water and by the presence of salts.	• It is caused by dissolved organic matter, industrial waste, chlorine, iron etc.
• Also originates in the treatment process like chlorination.	• Measured by threshold odor number (TON).

6.3.7 Temperature

The most desirable range of temperatures for a public water supply is between 40°F and 50°F.

6.3.8 Specific conductivity

It is used a measure of the quality of the water. The conductivity of water is determined by measuring its electric resistance between two electrodes specific conductance of pure at 25°C is in the neighborhood of 5.5×10^{-3} mhos/cm. For most waters the concentration of dissolved solids in mg/l is equal to 0.55 to 0.7 times the conductivity in micro-siemens per centimeter (μS/cm = μ mohos/cm) at 25°C.

6.3.9 Total dissolved solid (TDS)

Bangladesh standard guideline value is 1000 mg/l. TDS comprise of inorganic salt and small amount of organic matter. There is no evidence of deleterious physiological reactions occurring in persons consuming drinking water supplies that have TDS level in excess of 1000 mg/l affect faster. The level of presence of TDS is :

- Excellent: <300 mg/l (Extremely low TDS may also be unacceptable)
- Good: 300 - 600 mg/l
- Fair: 900 mg/l
- Poor: 900-1200 mg1
- Unacceptable: > 1200 mg/l

6.3.10 Chloride

High concentrations of chloride give an undesirable taste to water & beverages. Taste thresholds chlorides (as Na, K or Ca) are in the range of 200-300 mg/l. High chloride concentration are corrosive to metal.

6.3.11 Fluoride

It naturally presents in some food stuffs as well as in water are desirable at a minimum limit of about 0.6 mg/l to prevent dental caries. At levels above 1.5 mg/l mottling (discolored) of teeth has been reported very occasionally, and at 3.0 to 6.0 mg/l keletal fluorosis may be observed.

6.3.12 Nitrate

NO_3 is toxic when present in excessive amounts in drinking water, which may then cause methamoglobinaemia in bottle fed infants (blue babies).The Nitrate are reduced in the body to Nitrites (NO_2) which react with the oxygen receptor sites on the Hemoglobin fraction of the blood and impair its oxygen carrying capacity. For this reason a value of 10 mg/l is recommended for Nitrate concentration. There is a possibility that certain forms of cancer might be associated with very high nitrate concentration.

6.3.13 Sulfate

Ingestion of water containing high concentration of sulfate can have laxative effect, which is enhanced when the sulfate is consumed in combination with magnesium. Metal corrosion may be increased by high sulfate levels. Sulfates generally have less effect on taste than chloride & carbonates.

6.3.14 Lead poisoning

The presence of lead is undesirable in water because its tendency to accumulate in the body resulting in 'plumbisrn'. Its presence in natural water is unusual, but it may be found in water supplies that have come in contact with lead containers such as lead pipes, lead lined tanks and lead paints.

6.3.15 Mercury

Mercury is a toxic element & has no beneficial function in man. Elevated mercury (organic) level has been found in fresh water taken from areas with suspected mercury contamination. Lead & mercury are usually associated with chronic effects on nervous system.

6.3.16 Arsenic

Most of the arsenic found in ground water . Acute poisoning by arsenic involves central nervous system, leading to coma and for doses of 70-180 mg, to death. WHO Guide line value is 0.01 mg/l (1996). Bangladesh Standard is 0.05 mg/l.

6.3.17 Hydrogen sulfide

In ground water it is produced by reduction of sulfates, iron pyrites, or decomposition of organic matter and is likely to be found in water that has percolated through beds of lignite or other organic remains. Its disagreeable rotten egg odor makes it highly objectionable (maximum limit 1mg/l).

6.3.18 Dissolved oxygen

Its content in surface water is dependent upon the amount and character of the unstable organic matter in the water and is an important factor in self-purification capacity of polluted streams. Free oxygen in solution is an agent of corrosion, particularly when CO_2 is also present.

6.3.19 Carbon dioxide

It is dissolved by water from the atmosphere, from decomposing organic matter at the earth's surface or from underground sources. It unites with water to form H_2CO_3 that permits the solution of $CaCO_3$ or $MgCO_3$ into the bicarbonates of those elements. It causes corrosion in metal in presence of free O_2.

6.4 Microbiological Quality of Water

Many bacteria are found in water. Most of them are of no sanitary significance, some are indicator of pollution but are harmless; others, few in numbers, are pathogenic (produce diseases). Viruses are parasitic agents smaller in size than Bacteria, requiring a particular host cell which serves as a source of raw material for viral reproduction. Microscopic organisms other than Bacteria and Virus e.g. Algae, Fungi, Protozoa, Worms etc. often have undesirable effects upon water. Some of them produce color, taste, earthy and fishy odor in water. The most common and widespread danger associated with domestic water is contarnination. Either directly or indirectly, by sewage, by other wastes, or by human or animal excrement. Such contamination of water may introduce a variety of intestinal pathogens. These include (i) Bacteria causing typhoid fever Salmonella Typhi: bacillary dysentery Shigella cholera Vibrio Cholerae, (ii) Virus causing infection hepatitis; poliomyelitis and gasroenteritis, (iii) Protozoa causing amoebic dysentery E. Histolytica. The use of such water may result further cases of infection. Ground water normally does not contain any Bacteria since the effect of filtration, unfavorable environment and will eliminate most of them. Soils or aquifers having holes cracks or crevices may allow contaminated water to enter wells or springs. Surface waters usually contain many Bacteria.

6.5 Indicator Organism

It is impracticable to monitor drinking water for every possible microbial pathogen that might occur with contamination. A more logical approach is the detection of organisms normally present in the faces of man as indicators of excremental pollution. The coliform group (specifically Escherichia Coli) normally inhibits the intestinal track of man and other warm blooded animal and is excreted in large number (500 million per gram in a ratio of 1 typhoid organism to 1 million coliform) with the faeces. It is considered non pathogenic, but may cause infections of the genitor urinary tract. The presence of such organisms indicates the presence of faecal material, and thus that intestinal pathogens could be present. Conversely the absence of faecal commensal organisms indicates that pathogens are probably also absent. Thus the coliform group is of great importance in the microbiological quality analysis of water and includes a number of organisms. Some coliform species Aerobacter Aerogenes and Aerobacter Cloaceas are normally found on plant, soil and as well as in the faeces are termed as “total coliform”(including Escherichia Coli) which ferment lactose at 35°C or 37°C with the production of acid, gas and aldehyde within 24 or 48 hours. Excherichia Coli which is exclusively of faecal origin is known as “Faecal Coliform” tolerates and grows at the higher selective temperature range of 44 to 44.5°C.

Methods of Analysis: Two basic methods have been developed for the detection of indicator bacteria it waters-

- **Multiple Tube Method (MPN):** Different amount of water are added to tubes containing a suitable culture media (lactose broth). After a specified incubation times at given temperature each tube showing gas formation is regarded as ‘presumptive positive’ and indicates the possible presence of coliforms. From the number of tubes with positive reaction, the most probable number (MPN) of bacteria present in the original water sample can then be statistically determined.
- **Membrane Filter Technique (MF):** In contrast to the MPN method, the membrane-filter method gives a direct count of the total colifroms and faecal colifroms. The method is based on the filtration of a known volume of water through membrane filter (pore dia = 0.45μm). The bacteria are retained on the surface of the filter which is incubated with a culture media at a selective temperature for specific period of time. Colonies of coliforms develop on the filter can be counted directly.

6.6 Radioactivity in Water Supplies

Radioactive materials are introduced into the environment from a number of sources; naturally occurring and man-made. The naturally occurring sources include those substances produced by cosmic rays which may find their way to water courses with rainfall and run off and those present in the rocks and soil. The man-made radionuclides are those resulting from fallout from nuclear tests, nuclear power production, and medical and other uses of radioactive materials. Various body tissues have different sensivities to radiation exposure. The detrimental health effects of exposure to radiation are either "Somatic" i.e. those that become manifest in the exposed individual or "Hereditary" i.e. those that affect the individuals descendants. The recommended Guide Line Values are 0.1bq/liter for gross alpha activity and 1bq/liter for gross beta activity. (bq= Becquerel the SI unit of radionuclide activity); 1 Becquerel is equivalent to 1 spontaneous nuclear transformation per second and corresponds approximately to 27 picocuries.

6.7 Organic Contaminants

Both naturally occurring & artificial organics (like pesticides) are found in water. The former may be associated with color, taste or odor while the latter in sonic cases may be toxic or careinogenic (Turk 1974). The organic materials in water may be altered by treatment process, which some time makes them more dangerous (like trihalomethanes). Water quality standards are shown in Table 6.5.

6.8 Process Selection Factors

The design of treatment facilities will be determined by feasibility studies, considering all engineering, economic, energy and environmental factors. All legitimate alternatives will be identified and evaluated by life cycle cost analyses. Additionally, energy use between candidate processes will be considered. For the purpose of energy consumption, only the energy purchased or procured will be included in the usage evaluation. All treatment process systems will be compared with a basic treatment process system, which is that treatment process system accomplishing the required treatment at the lowest first cost. Pilot or laboratory analysis will be used in conjunction with published design data of similar existing plants to assure the optimal treatment.

Table 6.5: Water Quality Standards

Constituents/ Characteristics	WHO Guide Line Value	EPA Drinking Water Stands	Bangladesh Standards
pH value	6.5-8.5	6.5-8.5	6.5-8.5
Turbidity (NTU)	5	5	10 (JTU)
Color (TCU)	15	15	15 (Hazen Unit)
Hardness (as CaCO$_3$)	500	---	200-500
Iron mg/l	0.3	0.3	0.3-1(5)*
Manganese mg/l	0.1	0.05	0.1
Chloride mg/l	250	250	150-600 (1000)**
Sulfate mg/l	400	250	400
Odor (TON)	Not offensive	3	Odor less
Suspended solid mg/l	---	---	10
TDS mg/l	1000	500	1000
Ammonia mg/l	---	---	0.
Zinc mg/l	5	3	5
Fluoride mg/l	0.6-1.5	0.7-2.4	1
Nitrate mg/l	10	10	10
Alkalinity mg/l	130	---	130
Lead mg/l	0.05	0.05	0.05
Mercury mg/l	0.001	0.002	0.001
Arsenic mg/l	0.01	0.05	0.05
Sodium mg/l	200	---	200
Aluminum mg/l	0.05-0.2	0.2	0.2
Boron mg/l	2.4	---	1.0
Cadmium mg/l	0.003	0.005	0.01
Chlorine (residual) mg/l	---	---	0.2
COD mg/l	---	---	4
BOD$_5$ mg/l	---	---	0.2
T.Colifrom/100ml	n/100ml	0	0
T.Colifrom/100ml	n/100ml	0	0

*The values are for small diameter hand pump tube wells.

**Coastal area

n = water in the distribution system & unpiped water supplies may show 3 and 10 coliforms respectively in occasional but not consecutive samples.

6.9 Preliminary Treatment

Surface waters contain fish and debris which can clog or damage pumps, clog pipes and cause problems in water treatment. Streams can contain high concentrations of suspended sediment. Preliminary treatment processes are employed for removal of debris and part of the sediment load.

6.9.1 Screens

- **Coarse screens or racks**: Coarse screens, often termed bar screens or racks, must be provided to intercept large, suspended or floating material. Such screens or racks are made of ½ inch to ¾ inch metal bars spaced to provide 1 to 3 inch openings.
- **Fine screens**: 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 travelling type, which are cleaned by water jets. Fine-screen, clear openings should be approximately ⅜ inch. The velocity of the water in the screen openings should be less than 2 feet per second at maximum design flow through the screen and minimum screen submergence.
- **Ice clogging**: In northern areas screens may be clogged by frazil or anchor ice. Exposure of racks or screens to cold air favours ice formation on submerged parts and should be avoided to the maximum practicable extent. Steam or electric heating, diffusion aeration and flow reversal have been used to overcome ice problems.
- **Disposal of screenings**: Project planning must include provision for the disposal of debris removed by coarse and fine screens.

6.9.2 Flow measurement

Water treatment processes, e.g., chemical application, are related to the rate of flow of raw water; therefore, it is essential that accurate flow-rate measurement equipment is provided. Pressure differential producers of the Venturi type are commonly used for measurement of flow in pressure conduits. An alternative selection for pressure conduits is a magnetic flow meter if the minimum velocity through the meter will be 5 feet per second or more. A Parshall flume can be used for metering in open channels. Flow signals from the metering device selected should be transmitted to the treatment plant control center.

6.9.3 Flow division

While not a treatment process, flow division (flow splitting) is an important treatment plant feature that must be considered at an early stage of design. To insure continuity of operation during major maintenance, plants are frequently designed with parallel, identical, chemical mixing and sedimentation facilities. No rigid rules can be given for the extent of duplication required because a multiplicity of factors influences the decision. Normally, aerators are not provided in duplicate. Pre-sedimentation basins may not require duplication if maintenance can be scheduled during periods of relatively low raw water sediment load or if the following plant units can tolerate a temporary sediment overload. If it is determined that pre-sedimentation at all times is essential for reliable plant operation, then the flow division should be made ahead of the pre-sedimentation basins by means of identical splitting weirs arranged so that flow over either weir may be stopped when necessary.

During normal operation, the weirs would accomplish a precise equal division of raw water, regardless of flow rate, to parallel subsequent units; rapid-mix, slow-mix and sedimentation. The water would then be combined and distributed to the filters. If pre-sedimentation units are not provided, then the flow is commonly split ahead of the rapid-mix units. If a single treatment train is to be provided initially with the expectation of adding parallel units in the future, then the flow-split-ting facilities should be provided as part of the original design, with provision for Mocking flow over the weir which is to serve future units.

6.9.4 Sand traps

Sand traps are not normally required at surface water treatment plants. Their principal application is for the removal of fine sand from well water. The presence of sand in well water is usually a sign of improper well construction or development. If sand pumping cannot be stopped by reworking the well, the sand must be removed. Otherwise, it will create serious problems in the distribution system by clogging service pipes, meters, and plumbing. Centrifugal sand separators are an effective means of sand removal. These cyclone-separator devices are available assembled from manufacturers and require no power other than that supplied by the flowing water. They operate under system pressure; therefore, re-pumping is not necessary. Water from the well pump enters tangentially into the upper section of the cone and centrifugal force moves the sand particles to the wall of the cone. They then pass down water into the outlet chamber. Sand is periodically drained to waste from this chamber through a valve that can be manually or automatically operated. The clarified water is discharged from the top of the cone, These units are available in diameters of 6, 12, 18, 24, and 30 inches. providing a capacity range from 15 to 4500 gallons per minute (gpm) and are suitable for operation up to 150 pounds per square inch (psi). Pressure drop through the unit ranges from 3 to 25 psi, depending on unit size and flow rate. These separators will remove up to 99 percent of plus 150 mesh sand and about 90 percent of plus 200 mesh. The units are rubber lined for protection against sand erosion.

6.9.5 Plain sedimentation

Plain sedimentation, also termed “pre-sedimentation” is accomplished without the use of coagulating chemicals. Whether plain sedimentation is essential is a judgment decision influenced by the experience of plants treating water from the same source. Water derived from lakes or impounding reservoirs rarely requires pre-sedimentation treatment. On the other hand, water obtained from notably sediment-laden streams, requires pre-sedimentation facilities for removal of gross sediment load prior to additional treatment. Pre-sedimentation treatment should receive serious consideration for water obtained from rivers whose turbidity value frequently exceeds 1,000 units.

- **Plain sedimentation basins:** Plain sedimentation or pre-sedimentation basins may be square, circular, or rectangular and are invariably equipped with sludge removal mechanisms.
- **Design criteria:** Detention time should be approximately 3 hours. Basin depth should be in the approximate range of 10 to 15 feet, corresponding to up flow rates of 600 to 900 gallons per day (gpd) per square foot for a detention period of 3 hours. Short circuiting can be minimized by careful attention to design of inlet and outlet arrangements. Weir loading rates should not exceed approximately 20,000 gpd per foot. Where pre-sedimentation treatment is continuously required, duplicate basins should be provided. Basin bypasses and overflows should also be included.

6.10 Aeration

The term “aeration” refers to the processes in which water is brought into contact with air for the purpose of transferring volatile substances to or from water. These volatile substances include oxygen, carbon dioxide, hydrogen sulfide, methane and volatile organic compounds responsible for tastes and odor.

Aeration is frequently employed at plants treating ground water for iron and manganese removal.

6.10.1 Purpose of aeration

The principle objectives of aeration are:

- Addition of oxygen to ground water for the oxidation of iron and manganese. Ground waters are normally devoid of dissolved oxygen. The oxygen added by aeration oxidizes dissolved iron and manganese to insoluble forms which can then be removed by sedimentation and filtration.

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

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

- Partial removal of carbon dioxide/H_2S and other gases to reduce the cost of water softening by precipitation with lime and to increase pH.
- Reduction of the concentration of taste-and-odor producing substances, such as hydrogen sulfides and volatile organic compounds.
- Removal of volatile organic compounds which are suspected carcinogens.

6.10.2 Types of aerators

Three types of aerators are commonly employed (see Figure 6.2). These are: waterfall aerators exemplified by spray nozzle, cascade, and multiple tray units; diffusion or bubble aerators which involve passage of bubbles of compressed air through the water; and mechanical aerators employing motor-driven impellers alone or in combination with air injection devices. Of the three types, waterfall aerators, employing multiply trays, are the most frequently used in water treatment. The efficiency of multiple-tray aerators can be increased by the use of enclosures and blowers to provide counter flow ventilation.

6.10.3 Design criteria

- **Cascade aerators:** This type of aerators consist a series of waterfalls that drop into small pools. In this case, water is not dispersed as droplets but is exposed to atmosphere in thin sheets as it moves down each step.

  Removal rates: Remove 20 - 45% CO_2
  Design criteria: Area: 85 – 105 $m^2/m^2.s$
  Each step height: 0.3m; 10 steps can be used.

- **Multiple-tray tower aerators:** Multiple-tray aerators are constructed of a series of trays, usually three to nine, with perforated, slot or mesh bottoms. The water first enters a distributor tray and then falls from tray to tray, finally entering a collection basin at the base. The following design considerations are important in multiple tray aerators.

- **Design criteria:**
  - The vertical opening between trays usually ranges from 12 inches to 30 inches. Good distribution of the water over the entire area of each tray is essential.

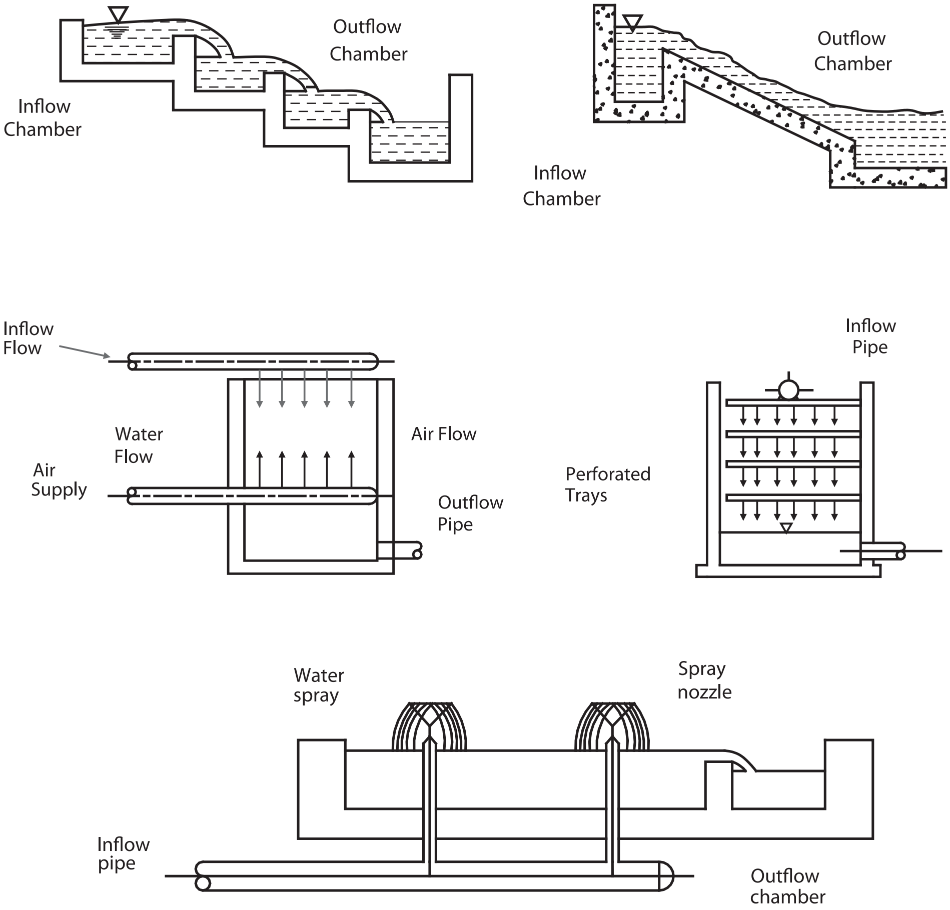


Figure 6.2 (a): Different types of aerators

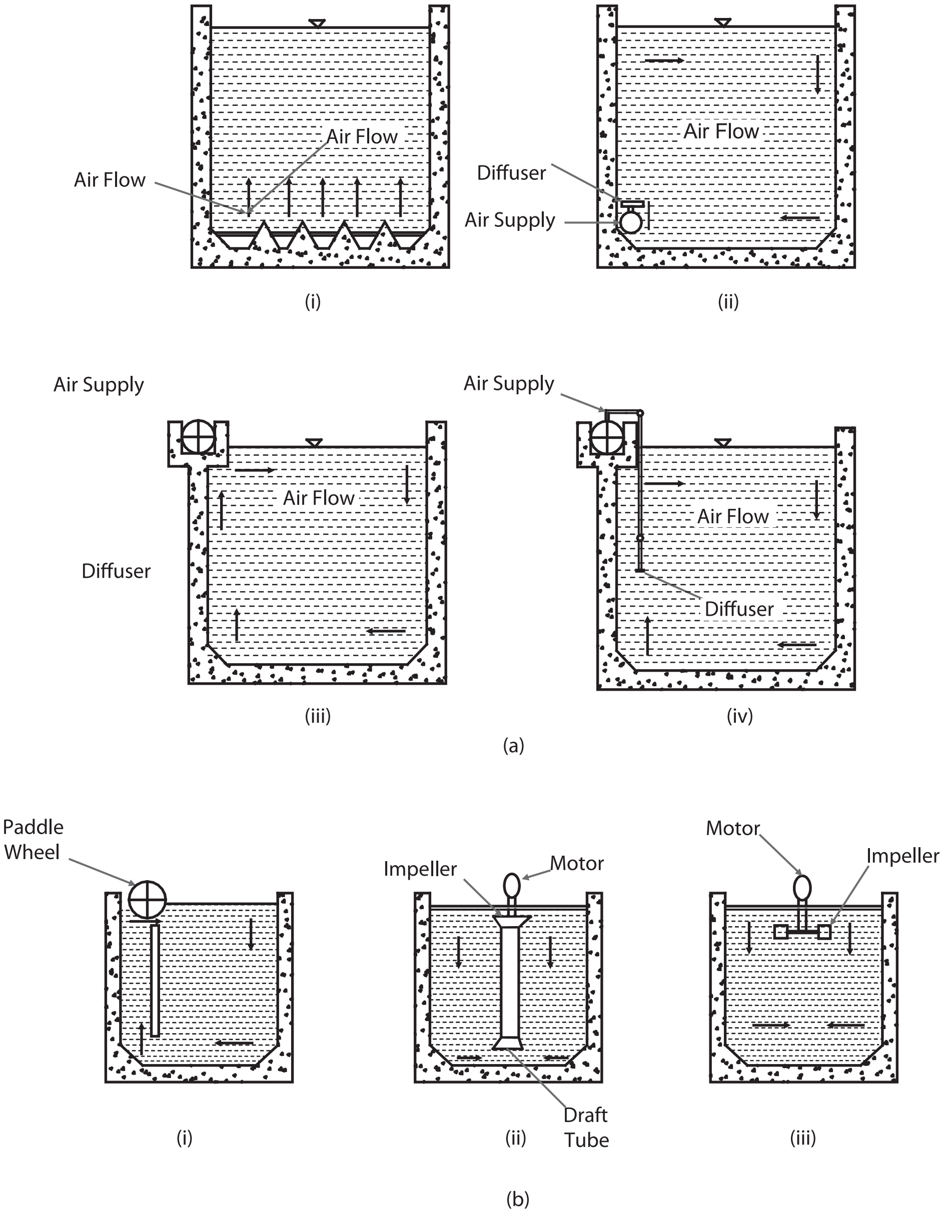


Figure 6.2 (b): **Different types of aerators**

- Perforated distributors should be designed to pro-vide a small amount of head, approximately 2 inches on all holes, in order to insure uniform flow.

  - In aerators with no provision for forced ventilation, the trays are usually filled with 2 to 6 inch media, such as coke, stone, or ceramic balls to improve water distribution and gas transfer and to take advantage of the catalytic oxidation effect of manganese oxide deposits in the media.
  - The water loading on aerator trays should be in the range of 10 to 20 gpm per square foot. Good, natural ventilation is a requirement for high efficiency.
  - The efficiency of removal of CO_2 is more than 90% and the following empirical equation can be used to estimate carbon dioxide (CO_2) removal:

    Where,

    $C_r = C_o(10^{-kn})$

    C_r = mg/l CO_2 remaining after aeration

    C_o = mg/l CO_2 present in water in distribution tray

    n = number of trays, including distribution trays

    k = 0.11 to 0.16 depending on temperature, turbulence, ventilation etc.
  - Air requirement of $7.5\ \frac{m^3}{m^3}$ of water is usually required.
  - Where icing is a problem and the aerator must be housed, artificial ventilation by fans or blowers is necessary. An enclosed induced or positive-draft aerator requires approximately 3.5 to 6 standard cubic feet of ventilating air per gallon of water aerated. Thus, for an enclosed aerator operating at a rate of 1.5 million gallons per day (mgd), air requirements will be in the range of 3600 to 6200 standard cubic feet of air per minute. Positive-draft aerators employing the higher air-flow rates exhibit the highest efficiency for the addition and removal of dissolved gases and oxidation of iron, manganese, and sulfide.
  - Power requirements for a natural draft, multiple-tray aerator having an overall height of 10 feet will be approximately 1.7 kilowatts per mgd of aeration capacity. Power demands for forced draft units will be greater.

- **Counter-current packed column aerators:** A counter-current parked column aerator tower is similar to operation to counter-current multiple tray aerators, but are particularly efficient at the removal of volatile organic compounds (VOCs) through air-strip- ping. Packed column aerators consist typically of a long thin tower filled with either a random dumped media (Rasching rings, Ber) saddles, Pall rings) or corrugated sheet media, held by a packing support plate. Water is pumped to the top of the tower over a distribution plate and allowed to fall through the media. Air is blown up through the tower by a fan counter to the falling water. Redistributor plates are used through out the column to prevent channelling of the water or air stream. Efficiency of the tower is dependent on the extent of contact between the air and water.

- **Diffusion aerators:** Compressed air is injected into the water as it flows through a rectangular basin. A variety of air injection devices may be employed including perforated pipes, porous plates or tubes and various patented sparger devices.
  - Basin size is determined by desired detention time, which commonly ranges from 10 to 30 minutes.
  - Tank depth is usually from 10 to 15 feet.
  - Air requirements, supplied by a compressor, generally range from 0.1 to 0.2 standard cubic foot per gallon of water aerated.
  - 80% VOC can be removed
  - Major advantages of a diffusion aeration system include practically no head loss and freedom from cold-weather operating problems.
  - An additional advantage is that a diffusion aerator may also be used to provide chemical mixing.
  - Power requirements are those associated with air compression and range from 1.0 to 2.0 kilowatts per mgd of aerator capacity.
  - Aeration efficiency in terms of addition of oxygen or removal of carbon dioxide is generally similar to that provided by multiple-tray aerators employing natural ventilation.
- **Mechanical aerators:** Mechanical aerators typically consist of an open impeller operating on the water's surface.
  - Basin size is determined by detention time required.
  - Basin depth can vary from 5 to 17 feet with the average depth being 10 feet.
  - Major advantages of mechanical aerators are practically no head loss and the ability to provide mixing.
  - 50-80% CO_2 can be removed.
  - Mechanical aerators are generally not as efficient as aeration towers or diffused aerators and longer detention times are required.

6.10.4 Criteria for installation of aerators

Aeration is a gas transfer process which is not needed at all water treatment plants. A decision as to whether to aerate or not requires assessment of the economic and water quality benefits achieved by its use.

- **Addition of oxygen:** Aeration processes are commonly used in adding oxygen to ground waters and to oxidize iron, manganese, hydrogen sulfide and to a limited extent, organic matter. Ground waters are usually deficient in oxygen and aeration is an effective means of adding it. Oxygen addition is normally required if iron and manganese removal is a treatment objective. Aeration will also help oxidize hydrogen sulfide and some organic matter.
- **Partial removal of volatile substances:** Aeration is a useful method of removing volatile substances from water. Ground waters while being deficient in oxygen can contain objectionable levels of carbon dioxide. An efficient aerator will result in near saturation with oxygen and about 90 percent reduction of the carbon dioxide content of groundwater. At lime-soda water softening plants, any carbon dioxide dissolved in the water at the point of lime application will consume lime without accompanying softening. For high (>50 mg/L) carbon dioxide concentrations, as encountered in some ground waters, aeration for its removal is probably justified. For concentrations on the order of 10 mg/L, or less, aeration is probably not economically valid. Before deciding to aerate for carbon dioxide removal, the cost of purchasing, maintaining and operating the aerator should be compared to the value of the lime saved. At softening plants, each mg/L of carbon dioxide removed will effect a saving of about 1.3 mg/L quicklime (95 percent calcium oxide). It will also reduce the quantity of softening sludge produced proportionately.
- **Reduction of hydrogen sulfide:** Aeration is also used for removing hydrogen sulfide from well water. It may be sufficient in itself if the hydrogen sulfide concentration is not more than about 1.0 or 2.0 mg/L. Otherwise, it may be used in conjunction with chlorine to oxidize the hydrogen sulfide not removed by aeration.
- **Reduction of Volatile Organic Compounds (VOCs):** Recent studies have shown that aeration can be successfully employed to reduce volatile organic compounds (VOCs) such as total Trihalomethane (TTHM) concentration in chlorinated water to meet standard regulations limiting

TTHM concentrations. Aeration by diffused air or multiple-tray aerators can reduce TTHM concentration at low cost, with cost increasing with higher concentrations of Trihalomethane (THM). Counter-current packed tower aeration is most efficient in achieving mass transfer of VOC.

Moreover, where icing is a problem and the aerator must be housed, artificial ventilation by fans or blowers is necessary. An enclosed induced or positive draft aerator requires approximately 3.5 to 6 standard cubic feet of ventilating air per gallon of water aerated. Thus, for an enclosed aerator operating at a rate of 1.5 mgd, air requirements will be in the range of 3600-6200 standard cubic feet of air per minute. Positive-draft aerators employing the higher air-flow rates exhibit the highest efficiency for the addition and removal of dissolved gases and oxidation of iron, manganese, and sulfide. Counter-current packed column aeration is particularly efficient to remove volatile organic compounds, Requirements for a natural draft, multiple-tray aerator having an overall height of 10 feet will be approximately 1.7 kilowatts per mgd of aeration capacity, and Power demands for forced draft units will be greater. In general, aeration is worthy of consideration in connection with the treatment of groundwater supplies in conjunction with lime softening and for the removal of some VOCs. Surface waters usually exhibit low concentrations of carbon dioxide, no hydrogen sulfide and fairly high dissolved oxygen. As a consequence, aeration is not required for the removal or addition of these gases. However, surfaces waters contain higher levels of THM precursors than ground waters and therefore a need for aeration may arise to reduce TTHM following chlorination. Water high in the bromine-containing THMs is difficult to treat by aeration (McGauhey 1968) and other methods of removal should be used, such as coagulation and flocculation or contact with granular activated carbon.

6.11 Coagulation and Flocculation

Chemical conditioning of the colloids is known as coagulation and involves addition of chemicals that modify physical properties of the colloids to enhance their removal. Physical conditioning or joining together of small particles into larger, settleable and filterable particles is known as flocculation. This process involves gently mixing the suspension to accelerate interparticle contact, thus promoting agglomeration of colloidal particles into larger flock for enhanced settling. Thus, coagulation precedes flocculation and the two processes must be considered conjunctively.

6.11.1 Purposes of coagulation and flocculation

Raw water supplies especially surface water supplies; often contain a wide range of suspended matter, including suspended minerals, clay, silt, organic debris and microscopic organisms ranging in size from about 0.001 to 1.0 micrometer. Small particles in this size range are often referred to as "colloidal" particles. Larger particles, such as sand and silt, readily settle out of water during plain sedimentation, but the settling rate of colloidal particles is so low that removal of colloidal particles by plain sedimentation is not practicable. Chemical coagulation and flocculation processes are required to aggregate these smaller particles to form larger particles which will readily settle in sedimentation basins. The coagulation-flocculation processes are accomplished step-wise by short-time rapid mixing to disperse the chemical coagulant followed by a longer period of slow mixing (flocculation) to promote particle growth.

6.11.2 Forces controlling colloids behavior

The colloidal particles in natural water are stable because of their surface charge (usually –ve), The stationary charged layer on the surface is surrounded by a bound layer of water in which ions of opposite charge (counter ion i,e +ve) drawn from the bulk solution with greater concentration the vicinity. This arrangement produces a net charge that is strongest at the bound layer and decreases exponentially with distance from the colloid. When two colloids come in close proximity there are two forces acting on them:

- **Electrostatic forces:** It is the principal force contributing to the stability of the colloid suspensions. Most colloids are electrically charged. It depends on the nature of colloids. For example, metallic oxides are positively charged and non metallic oxides and metallic sulfides are negatively charged. Colloids of similar charge will repel each other. Negatively charged colloids usually dominate in natural water. The surface charge on the colloids attracts ions of opposite charge known as counter ions. These ions (Hydrogen and other cations) form a dense layer adjacent to the particle known as stern layer. A second layer, known as diffused layer is also attracted to the colloid. In this layer, ions of both electrical charges are attracted, but counter ions predominate. These two layers are termed as double layer. In diffused layer, water molecules are sufficiently bound to create shear surface. Water molecules inside this shear surface will behave as if attached to the colloid. Water molecules outside the shear surface will behave as if independent of the colloid.

- **Van der Waals forces:** A force of attraction exists between any two masses. This force is inversely proportional to the sixth power of the distance between the particles and also decays exponentially with distance. It decreases more rapidly than electrostatic potential, but is a stronger force at close distances. The sum of two forces as they relate to one colloid in close proximity to another is repulsive at greater distances and becomes attractive only after passing through a maximum net repulsive force, called the 'energy barrier' at some distance between colloids. Once the force becomes attractive, contact between the particles takes place.

In some instances, Brownian motion can cause two colloids to move sufficiently close for Vander Waals forces to predominate over electrostatic repulsion. If this occurs, the particles could agglomerate. The action of the forces is described in the next section 6.11.3 with theory of coagulation.

6.11.3 Theory of Coagulation

- **Double layer theory:** The negative ions shown in the figure 6.3 below are often referred to as 'co-ions' due to them having the same charge as the colloid. Likewise the positive ions are often referred to as 'counter-ions' due to them having an opposite charge to the colloid. The figure 6.3 below shows the double layer theory which proposes that a fixed layer of positively charged ions are attracted to the colloid surface due to its net negative charge. This layer of counter-ions is referred to as the Stern layer. Outside of the Stern layer in the bulk, positively charged ions are still attracted to the net negative colloid particle but are repelled by the positive ions of the Stern layer, as well as other positive ions attracted to the colloid. The result is a dynamic equilibrium which forms the diffuse layer of counter-ions which has a high concentration of positive ions near the stern layer which gradually decreases with distance until it reaches equilibrium with the normal counter-ion concentration in the bulk. Similarly, there is a lack of negative ions on the boundary between the Stern and diffuse layer due to repulsion from the net negative colloid. However as distance from the colloid increases so does the negative ion concentration until it also reaches equilibrium with the normal co-ion concentration in the bulk.

  The thickness of the double layer is dependent upon the concentration of ions in solution. A higher level of ions will result in a greater availability of

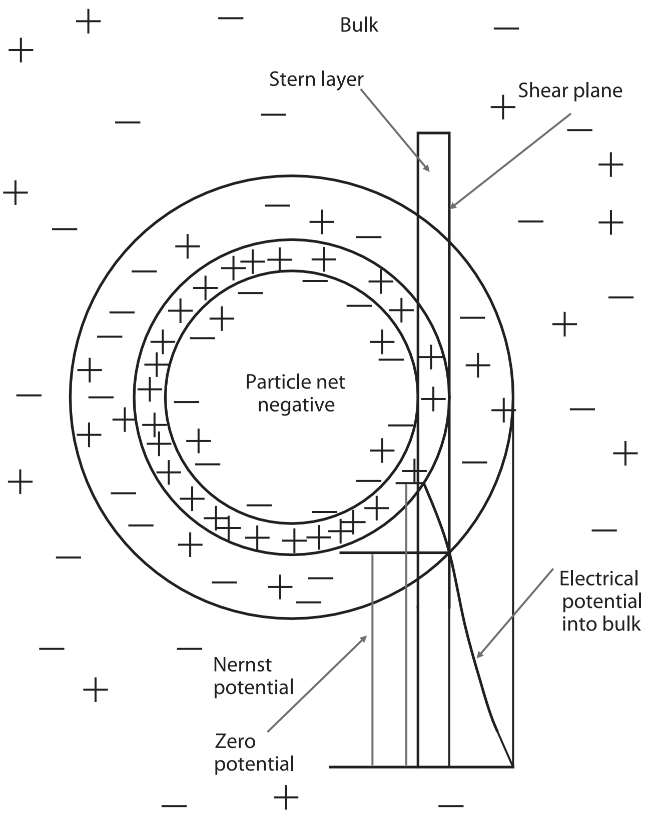


Figure 6.3: **The colloidal model showing zeta potential.**

positive ions to neutralise the colloid and thus a reduction in the thickness of the double layer. The type of positive counter-ion is also important. An equal concentration of Aluminium^{3+} ions will have a much greater effect than Sodium^{1+} ions would in neutralising the colloidal charge and would therefore cause a greater degree of double layer compression (J.M.G, Lelyveld et al. 1981). Due to the presence and concentration of counter-ions and co-ions around the colloid an electrical potential is formed from a maximum at the surface to zero at the outer edge of the diffuse layer. Although it is not possible to measure the surface potential it is relatively simple to find at the zeta potential, which is the electrical potential at the boundary between the Stern and diffuse layer. The zeta potential can be an effective tool for monitoring coagulation control and dosing where the technology is available as it will indicate changes in the repulsive forces between the colloids.

- **DLVO theory:** DLVO Theory, named after Derjaguin, Landau, Verwery and Overbeek, explains how colloidal particles interact based on the balance between Van der Waal's attraction and electrostatic repulsion.
  - **Repulsion:** The net negative charge and electrical potential of the colloids causes them to repel each other and prevents effective agglomeration and flocculation, thus creating stability due to electrostatic repulsion. In order for the particles to come together sufficient energy is required as to overcome electrostatic repulsive energy. (Figure 6.4)

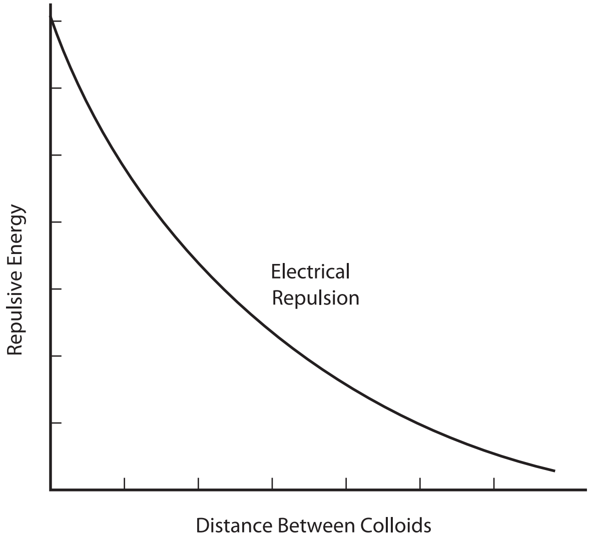


Figure 6.4: Electrostatic repulsion.

  - **Attraction:** Attraction between two colloids is due to London-Van der Waal's forces, the weakest of all intermolecular forces. The overall force is due to the cumulative interaction of the individual molecules in one colloid with each molecule from the other. The Figure 6.5 above shows how the attractive energy varies with distance between the colloids.
- **The energy barrier:** Combining the curves of electrostatic repulsion and Van der Waal's attraction enables a net interaction energy curve to be determined. At each given distance the smaller force is subtracted from the larger force and if the result is repulsive it is plotted above the x-axis and vice versa to produce the curve as shown in the Figure 6.6. When the net

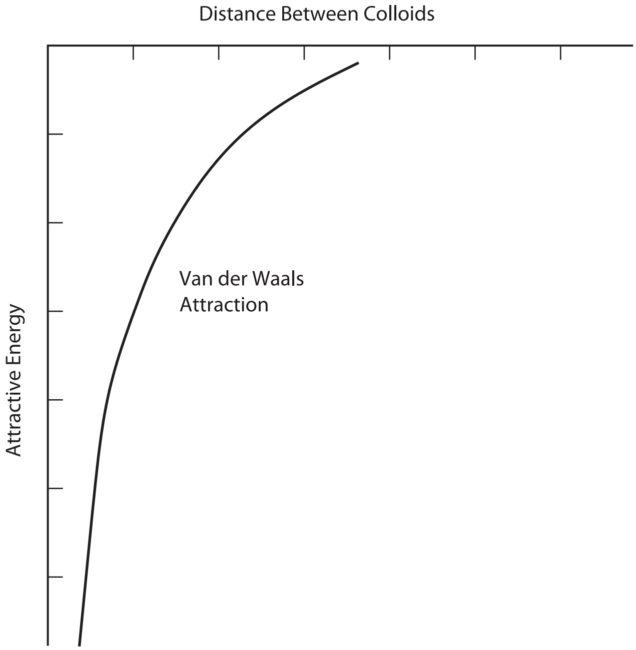


Figure 6.5: Van der Waals attraction

interaction energy is a repulsive force it represents an energy barrier, with the maximum height of the curve indicating its resistance to flocculation. In order to flocculate, particles must have kinetic energy (due to speed and mass) greater than the maximum energy barrier. When this occurs, the particles become sufficiently close and the net interaction energy is all attractive. As such the particles bond together due to the overpowering effect of the Van der Waal's forces.

For coagulation to be most effective, the energy barrier needs to be lowered or removed completely. The latter case would result in the net interaction being attractive at all distances between the colloids. However this is not necessary as the barrier is only required to be lowered to such a point where the kinetic energy (from mixing velocity and mass) is sufficient to overpower it. The two main mechanisms for lowering the energy barrier are;

- **Charge neutralisation:** The addition of a coagulant such as aluminium sulphate or ferric chloride will reduce the surface charge

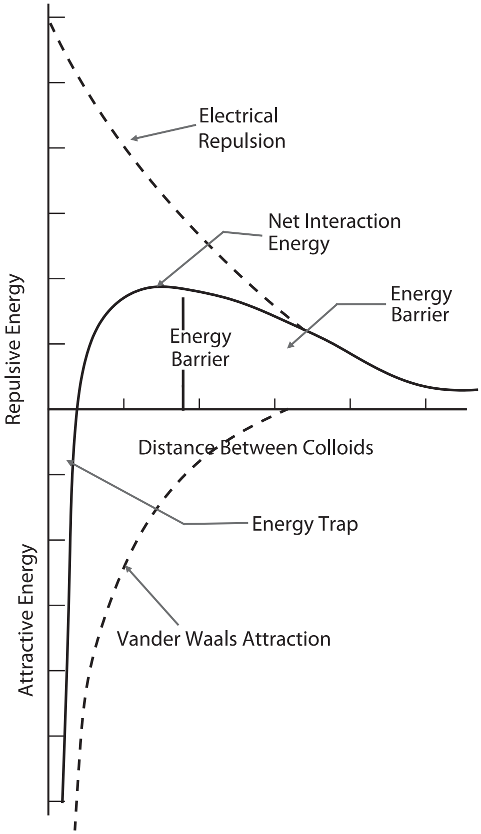


Figure 6.6: **The energy barrier from DVLO theory**

and thus also the zeta potential by charge neutralisation. This works by adsorption of positively charged coagulant ions on the surface of the colloid, which neutralises the net negative charge of the colloid. The outcome is a lowering of the electrical repulsion curve and an almost negligible net interaction energy barrier (Figure 6.7). This causes the colloids to become destabilised.

- **Double layer compression:** As previously mentioned compression of the double layer can occur due to an increase in the number of counter ions in solution. Practically this can be done by the addition of salts to the water until the energy barrier is substantially lowered due to significant compression of the repulsion curve. Adding salts to water is not a suitable means of coagulation for drinking water however due to

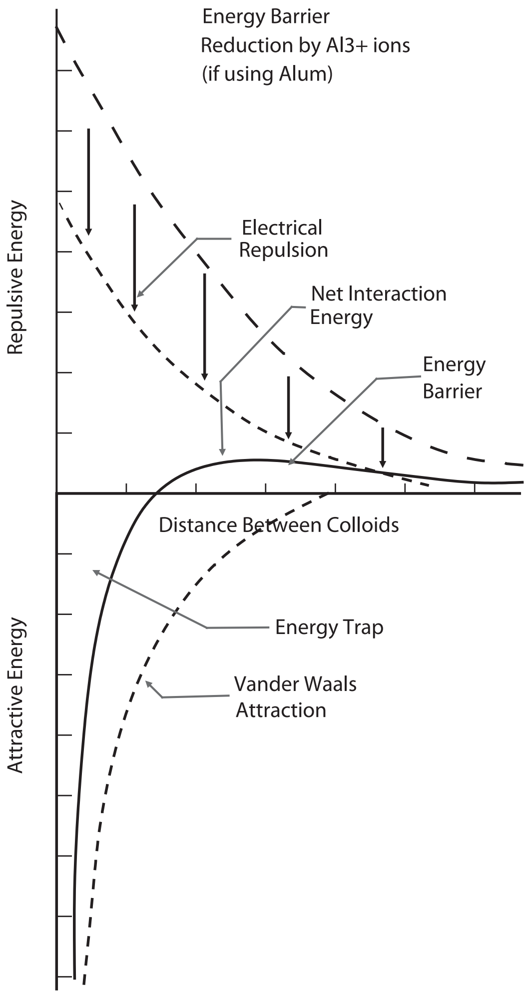


Figure 6.7: **Lowering of the colloid surface charge.**

the large amount undesirable residuals. Positive ions present in solution due to the addition of a coagulant would contribute to the double layer compression especially in the case of overdosing.

6.11.4 Chemical coagulant

The most frequently used chemical coagulant is aluminium sulphate ($Al_2(SO_4)_3.14H_2O$). This aluminium coagulant is also called "alum" or "filter alum," and dissociates in water for form $SO_4^{=}$, Al^{3+} ions and various aluminium hydroxide complexes.

Alum has an advantage in addition to its high positive charge: some fraction of the aluminum ions may form aluminum oxide and hydroxide by the reaction

$$Al^{3+} + 3OH^{-} \rightarrow Al(OH)_3$$

These complexes are sticky and heavy and will greatly assist in the clarification of the water in the settling tank if the unstable colloidal particles can be made to come in contact with the floc. This process is enhanced through flocculation process.

Other aluminium compounds used as coagulants are potash alum ($AlK(SO_4)_2.12H_2O$) and sodium aluminate (Na_2Al_2O), principally the latter. Iron coagulants include ferric sulphate ($Fe_2(SO_4)_3.9H_2O$), ferrous sulphate ($FeSO_4.7H_2O$), also available as $FeSO_4.5H_2O$ and ferric chloride ($FeCl_3.6H_2O$). Magnesium hydroxide ($Mg(OH)_2$), is also an effective coagulant, Organic polyelectrolyte compounds, applied in low dosages alone or in combination with the metal coagulant, are also employed, Polyelectrolytes are high-molecular-weight polymers that dissociate in water to give large highly charged ions, The polyelectrolyte's and dissociated ions destabilize the colloids and promote their settling, These polymers can be classified an anionic, cationic or nonionic according to their dissociated polymeric ions being negatively charged, positively charged or both negatively and positively charged.

Flocculation

A flocculator introduces velocity gradients into the water so that the particles in a fast-moving stream can catch up and collide with slow-moving particles. Such velocity gradients are usually introduced by rotating paddles, as shown in Figure 6.8. The power required for moving a paddle through the water is

$$P = \frac{C_D A \rho v^2}{2}$$

where,

P = power (N/s or ft-lb/s),

A = paddle area (m^2 or ft^2)

ρ = fluid density (kg/m^3 or $lb\text{-}s^2/ft^3$), and

C_D = drag coefficient.

v = velocity (m/s or ft/s)

Typical velocity criteria in the flocculation unit are:

- Typical velocity in conduits or flume from rapid mixing unit to the flocculation basins is from 0.45-0.9 m/s.
- The flocculation basins should be designed to have a velocity through the basin between 0.15-0.45 m/min.
- Baffles are designed to have a velocity of 0.3-0.45 m/s.
- A velocity in the range from 0.15-0.45 m/s is used in the design of a pipe to transfer coagulated water to the sedimentation tank.

6.11.5 Design criteria for mixing

When a coagulant is added to water it needs to be fully mixed to ensure even distribution. Bratby (1980) highlights its importance by saying that the rapid mixing stage is possibly the most important operation in the physico-chemical process as it is when destabilisation occurs and primary floc particles are formed.

Criteria for rapid and slow-mix processes have been developed on the basis of detention time, power input, velocity gradient (G) and the product (Gt) of velocity gradient and detention time. The values of G and Gt are computed from:

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

where

G = velocity gradient (fps/ft; of $\sec^{-1}$)
P = the power dissipated in the water (ft-lb/sec)
μ = water viscosity (lb-sec/ft^2); (standard value 2.73×10^{-5} lb-sec/ft^2)
v = volume of mixing basin (cubic feet)
t = mixer detention time (seconds)

6.11.6 Rapid mixing

For rapid mix units, detention periods usually range from 10 to 30 seconds with installed mixer power approximately 0.25 to 1.0 hp per mgd. Power and detention time should be matched so that values of G will be in the approximate range: 500-1000 $\sec^{-1}$. A rapid-mix unit having efficiency of 80 percent, a water temperature of 50°F, a power input of 1.0 hp per mgd and a detention time of 10 seconds, yield a G value of about 1000 $\sec^{-1}$ and a Gt value of 10,000. Similarly, a 30-second detention time gives a G value of about 600^{-1} and a Gt value of 18,000.

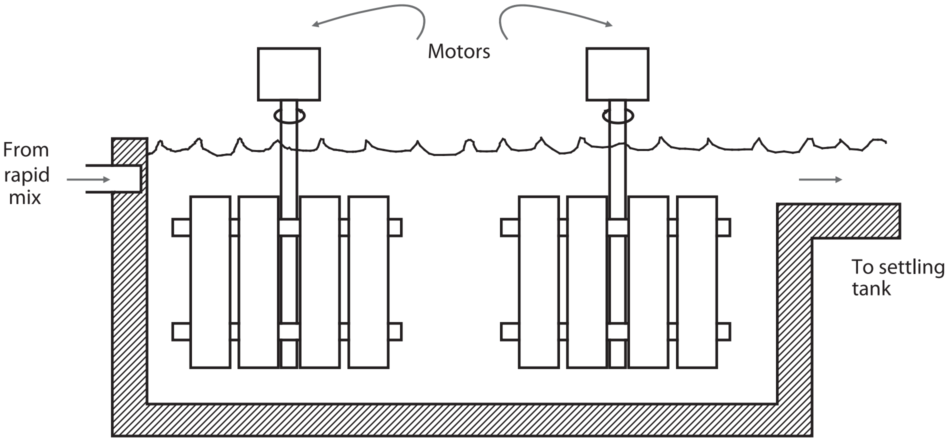


Figure 6.8: Flocculator used in water treatment.

Long detention period for rapid-mix basins should be avoided because of higher power requirements and inferior coagulation results. The rapid-mix basin should be designed to minimize short circuiting.

6.11.7 Slow mix

For slow-mix (flocculating) units, detention periods should range from 30 minutes to 60 minutes, with installed mixer power of approximately 0.1 to 3.5 hp per mgd. G values in the range of 20 sec^{-1} to 100 sec^{-1} are commonly employed, corresponding Gt values will, therefore, be in the range of 36,000 to 360,000. Tapered, slow mixing with G decreasing from a maximum of about 90 sec^{-1} down to 50 sec^{-1} and then to 30 sec^{-1} can be used and will generally produce some improvement in flocculation. Somewhat higher G values, up to 200 sec^{-1}, are employed in some water softening plants. For normal flocculation, using alum or iron salts, the maximum peripheral speed of the mixing units should not exceed about 2.0 fps and provision should be made for speed variation. To control short circuiting, two to three compartments are usually provided. Compartmentation can be achieved by the use of baffles. Turbulence following flocculation must be avoided, Conduits carrying flocculated water to sedimentation basins should be designed to provide velocities of not less than 0.5 fps and not more than 1.5 fps, and Weirs produce considerable turbulence and should not be used immediately following flocculation.

Generally accepted design standards require G to be between 30 and 60 sec^{-1}. Time is also an important variable in flocculation, and the term Gt is often used in design, where t is the hydraulic retention time in the flocculation basin. Gt values are typically between 10^4 and 10^5.

Example 6.1: A water treatment plant is designed for 30 million gallons per day (mgd). The flocculator dimensions are length = 100 ft, width = 50 ft, depth = 16 ft. Revolving paddles attached to four horizontal shafts rotate at 1.7 rpm. Each shaft supports four paddles that are 6 in. wide and 48 in. long. Paddles are centered 6 ft from the shaft. Assume C_D = 1.9, and the mean velocity of water is 35% of the paddle velocity. Find the velocity differential between the paddles and the water. At 50°F, the density of water is 1.94 lb-s^2/ft^3 and the viscosity is 2.73×10^{-5} lb-s/ft. Calculate the value of G and the time of flocculation (hydraulic retention time).

The rotational velocity is

$$v_t = \frac{2\pi rn}{60}$$

Where

r =radius in feet and n= rpm, so that,

$$v_t = \frac{(2\pi)(6)(1.7)}{60} = 1.07\ ft/s$$

The velocity differential between paddles fluid is assumed to be 65% of

$$v_t = 0.65 v_t = (0.65)(1.7) = (0.70\ ft/s)$$

Total power input is

$$P = \frac{C_D A\ \rho v^2}{2}$$

$$P = \frac{(1.9)(16)(0.5\ ft)(48\ ft)(1.94\ lb - s^2/ft^3)(0.70\ ft/s)^3}{2} = 243\ ft - lb/s$$

And from the velocity gradient is

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

$$G = \sqrt{\frac{243}{(100)(50)(16)(2.73\times 10^{-5})}} = 10.5\frac{ft/s}{ft}$$

Which is little low, the time of flocculation is (where, V= Volume of basen)

$$v_t = \frac{V}{Q} = \frac{(100)(50)(16)(7.48)(2.48)(60)}{(30)10^5} = 28.7\text{min}$$

So that the Gt value is 1.8×10^4. This is within the accepted range.

Example 6.2: The following criteria are used in the design example: Design the treatment unit.

1. flow rates:
   (a) Maximum flow rate=113,500m^3/d
   (b) Average flow rate=57,900m^3/d
2. Raw water quality:
   (a) Turbidity: 3-17 NTU
   (b) Seasonal Fe and Mn concentration: 0.2-0.7 mg/L and 0.05-0.4 mg/L
   (c) pH = 7.9-8.2
   (d) Total alkalinity: 80-110mg/L as $CaCO_3$
   (e) Total hardness: 100-120mg/L as $CaCO_3$
3. Chemicals:
   (a) Coagulant: ferric sulfate, optimum dosage 25mg/L, maximum feed rate 60 mg/L. Application point is rapid mix basin.
   (b) pH adjustment: $Ca(OH)_2$ is required to adjust pH 8 to 9 units. 15mg/L of $Ca(OH)_2$ is sufficient to provide optimum pH.
   (c) $KMnO_4$: Needed for Fe, Mn precipitation.
4. Rapid Mix basin design parameters:
   (a) Number of units= 4 basin
   (b) Detention time = 20-30s
   (c) G = 950/s

5. Flocculation basin design parameters:
   (a) Number of basins=4
   (b) Number of staged=3
   (c) Detention time =30min total, 10min in each stage
   (d) G=60/s→1st stage
   30/s→2nd stage
   15/s→3rd stage.

Solution:

Step A: Rapid Mix basin design:

1. Unit dimension:
   (a) Calculate design flow each process train:
   Max^m design flow=113,500m^3/d
   Each process train receives ¼ of max^m design flow.

$$\text{Design flow for each process train} = \frac{113500\ \text{m}^3/\text{d}}{4}$$
$$= 28375\text{m}^3/\text{d}$$
$$= 0.328\ \text{m}^3/\text{s}$$

   (b) Calculate basin volume for 30sec detention time:

$$v = Q \times t = 0.328\ \text{m}^3/\text{s} \times 30\text{s}$$
$$= 9.84\text{m}^3$$

   Use a square basin with depth to width ratio 1.5.

$$V = w \times l \times d$$
$$= w \times w \times 1.5w \ \ = 1.5\, w^3$$

   Now, $1.5\, w^3 = 9.84$

$$\therefore\ w = 1.9\text{m}$$

$$\text{Water depth} = 1.5w \times 1.9$$
$$= 2.85\text{m}$$

   Provide four basins of 1.9m×1.9m×2.9m with V=10.5m^3

2. Equipment Design:
   (a) Rapid mixer:
      (i) Power Requirements:

Design each mixer for a velocity gradient G = 950/s at a flow rate $0.328 m^3/s$

(ii) Mixer power:

$P = G^2 V\mu$

$\mu = 1.518 \times 10^{-3}$ N-S/m^2 at 5°C

V = basin volume = $10.5 m^3$

$P = (950/s)^2 \times 10.5 m^3 \times 1.518 \times 10^{-3}$ N-S/m^2

$= 1.44 \times 10^4$ N-m/s

= 14.4 KW

P is the power imparted to water. The power of the driver (P') is calculated by dividing P by gearbox efficiency (90%)

$$P' = \frac{P}{0.9} = \frac{14.4KW}{0.9} = 16KW$$

Step B: Flocculation basin design:

1. Unit dimension:

(a) Calculate required volume:

The required design flow for each basin is ¼th of the maximum day flow, and total detention time of 30min is provided for 3 stage flocculation basin.

Volume = $QGT = 0.328 m^3/s \times 60 s/min \times 30 min$

$= 590 m^3$

Volume of each stage of flocculation basin $= 590 m^3/3 = 197 m^3$

(b) Calculate basin dimension:

A flocculator axis perpendicular to the flow direction is provided in the flocculation basin. The width of the sedimentation basin is parallel to the flocculator axis is equal to the width of the sedimentation basin (18.4 m). The length of each stage perpendicular to the flocculator axis is equal to the water depth. There are three stages. So, total length of 3 stages=*3d*

Volume of each flocculator stage (v)$=18.4 \times d \times d$

$$d = \left(\frac{197 m^3}{18.4 m}\right)^{½}$$

= 3.27m

Water depth, d = 3.27m.

Dimension $= 3.3\text{m long} \times 18.4\text{m wide} \times 3.27$ deep and $V = 199\text{m}^3$

Step C: Chemical requirements:

Storage should be for 30days. Assume that bulk liquid ferric sulfate contains 50% by weight $Fe_2(SO_4)_3$. Use optimum feed rate.

Assuming optimum feed rate $= 25\text{mg/L}$

$= 0.025\text{kg/m}^3$ as $Fe_2(SO_4)_3$.

Maximum flow rate $= 113500\text{m}^3/\text{d}$

Maximum daily feed rate $= 0.025\text{kg/m}^3 \times 113{,}500\ \text{m}^3/\text{d}$

$= 2840\text{kg/d}$ of $Fe_2(SO_4)_3$.

for 30 day's storage $= 2840\text{kg/d} \times 30$ days

$= 85{,}200\text{kg}$ of $Fe_2(SO_4)_3$.

50% by weight of liquid ferric sulfate is $Fe_2(SO_4)_3$, so mass of liquid ferric sulfate for 30 days storage $= 85{,}200\text{kg} \div \frac{50}{100}$

$= 170{,}400\text{kg}$ liquid

Assume, bulk ferric sulfate liquid has a mass density of $1122\ \text{kg/m}^3$ then the total volume for storage $= \frac{170{,}000\text{Kg}}{1122\text{Kg}/m^3}$

$= 152\ \text{m}^3$

6.12 Sedimentation Basin

Sedimentation follows flocculation. The most common types of sedimentation basins in general use are shown in Figures 6.9 and 6.10. A recent innovation in clarifiers is a helical flow solids contact reactor, consisting of a above ground steel conical basin as shown in Figure 6.11. However, these above ground basins require a high head and additional pumps may be required. A minimum of two basins should be provided to allow one unit to be out of service for repair or maintenance. The design must include arrangements that permit use of a single basin when necessary.

Particle settling in water is a process causing the organic or inorganic particles heavier than water to settle by retaining water in a tank or basin. These particles are held in suspension in natural water mainly by turbulence or current and when the current is retarded, the suspended particles settle at the bottom of the basin.

A particle having specific gravity of more than 1, i.e. heavier than water, tends to move downward in relatively quiescent water by the force of gravity, accelerating until the frictional resistance ('drag') of the water equals the gravitational force acting upon the particle. Thereafter the particle travels with a constant vertical velocity called the 'terminal velocity' or 'settling velocity' of the particle.

The settling velocity of the particle depends upon:

- horizontal flow velocity of water
- shape and size of the particle
- specific gravity of the particle
- viscosity of water
- density of water
- temperature of water.

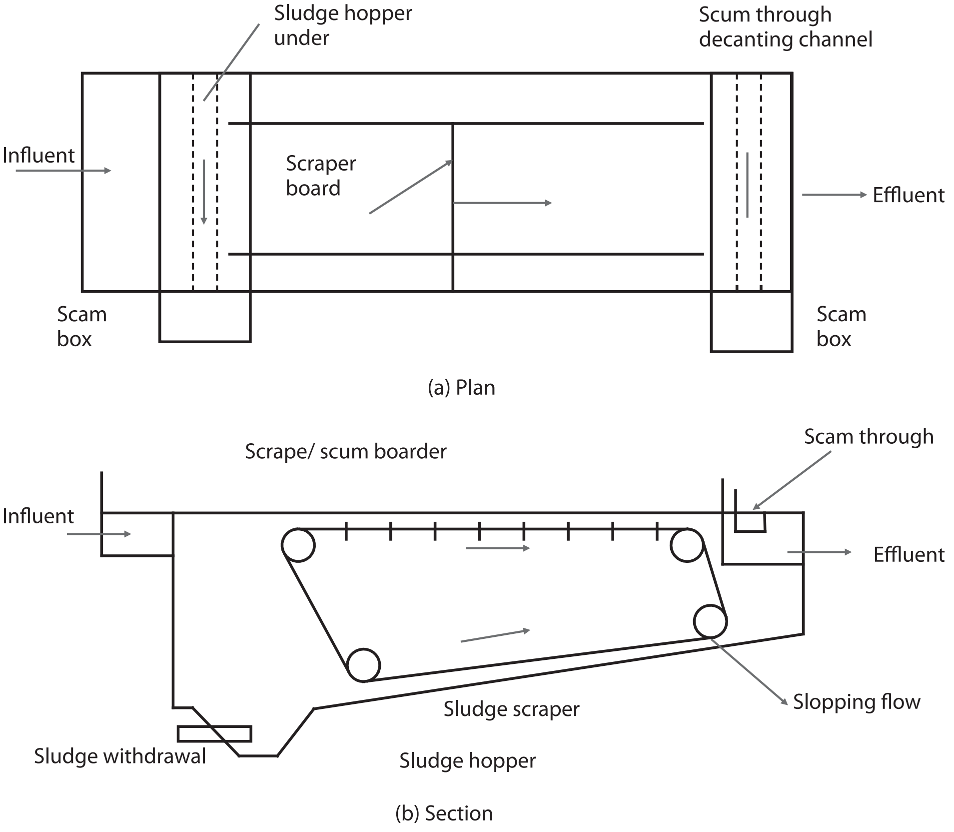
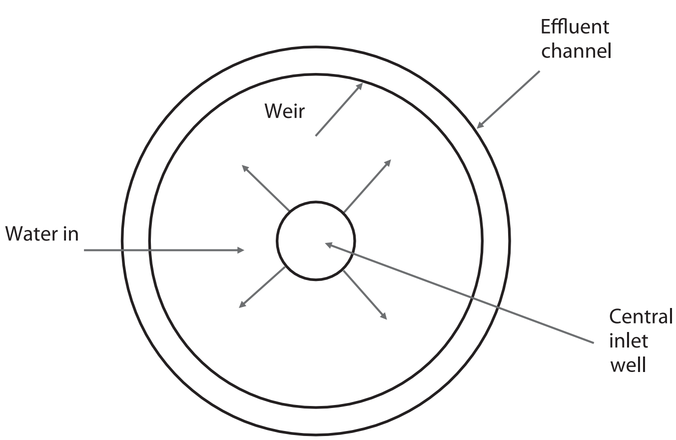
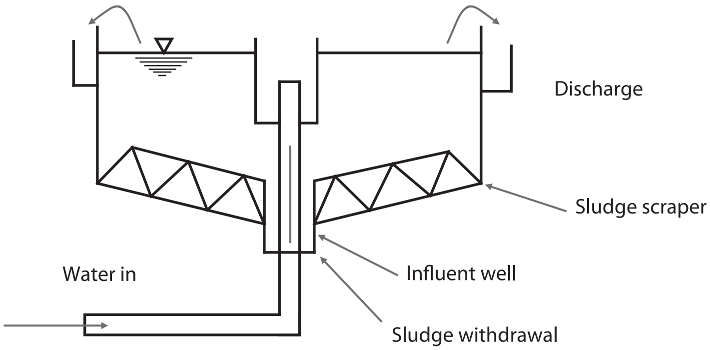


Figure 6.9: **Schematic of rectangular setting tank**



(a) Plan



(b) Section

Figure 6.10: Schematic of circular setting tank

6.12.1 Types of sedimentation

Sedimentation by definition is ‘the solid—liquid separation using gravity settling to remove suspended solids. In water treatment, sedimentation processes used are:

Type I: To settle out discrete non-flocculent particles in a dilute suspension. This may arise due to the plain settling of surface waters prior to treatment by sand filtration.

Type II: To settle out flocculent particles in a dilute suspension. This may arise after chemical coagulation and flocculation where the non-discrete particles are chemically assisted to coagulate.

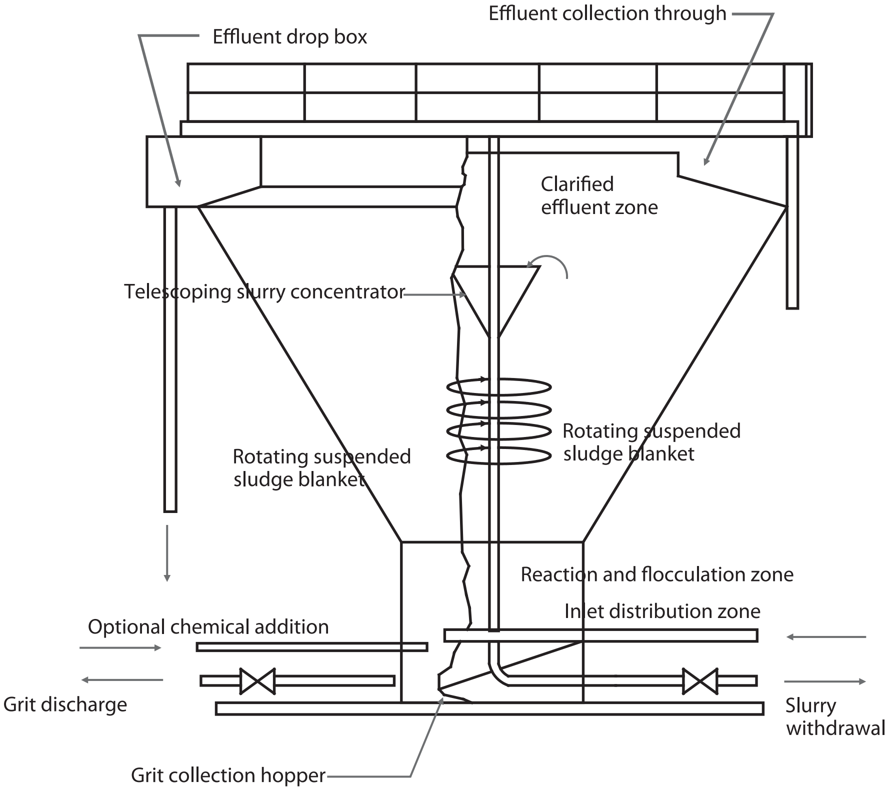


Figure 6.11: Conical sedimentation basin

Other types of settling are combinations of I and II.

Sedimentation of discrete particles—type I: Settling tanks are of two types: rectangular and circular. A rectangular settling tank is shown in Figure 6.9. They tend to have a length—width ratio of about 2 and a depth of the order of 1.5 to 6m. A sludge draw-off well is located at the upstream base, and the sludge is drawn to this by a travelling scraper board.

Figure 6.10 shows a circular settling tank. Dimensions typically are 10 to 50m in diameter and 2.5 to 6m in depth. Water enters to the central well either at the top or up through a central pipe. As the influent water settles, it spreads out and a sludge scraper moves the sludge towards a central sludge withdrawal hopper at bed level. The clarified water exits over a weir along the perimeter of the tank at surface level.

The key parameters and typical values in the design of settling tanks are:

- Surface overflow rate – 20–35 $m^3/day/m^2$
- Detention times 2—8 h
- Weir overflow rate 150—300m1/day/m^2

The above values vary depending on whether the water treated is raw water for potable treatment or coagulated raw water. Similar settling tanks are used in wastewater treatment.

In type I settling, the particles settle out individually and it is assumed that there is no flocculation or coagulation between the particles. Such sedimentation may typically be the first physical process of surface waters where discrete organic and grit particles are given adequate time to settle. This type of settling is somewhat similar to grit channel settling (for wastewater) described.

The design of such tanks is based on knowledge of setting velocity as described in the following text. In type I setting, a particle will accelerate vertically downwards unit the drag force F_D equals impelling force F_I and thereafter the particle setting at a constant velocity known as stokes' velocity V_S:

The impelling force

$$F_1 = (\gamma_s - \gamma_w)V_{ol} \qquad 6.1$$

Where

γ_s= weight density of solid particles ρsg
γ_w = weight density of water ρwg
V_{ol} = volume of particle

The drag force

$$F_D = C_D A_s \rho_w \left(\frac{V_s^2}{2}\right) \qquad 6.2$$

Where,

C_D = drag coefficient ≈0.4 for spheres

$$C_D = \frac{24\mu}{V_s d} \text{ for laminar flow for Re} < 100 \qquad 6.3$$

A_s = sphere section area orthogonal to velocity vector
V_s = settling or stokes' settling velocity
μ = kinematic viscosity

Equating

$$F_1 = F_D \tag{6.4}$$

$$(\gamma_s - \gamma_w)\frac{\pi}{6}d^3 = \frac{24\mu}{V_s d}\frac{\pi}{4}d^2\rho_w\left(\frac{V_s^2}{2}\right) \tag{6.5}$$

Solving

$$V_s = \frac{g}{18\mu}(S_p - 1)d^2 \tag{6.6}$$

Where

S_p = specific gravity of particles

or

$$V_s = \frac{g}{18\mu}(\rho_s - \rho_w)d^2 \tag{6.7}$$

This is known as Stokes' law for settling velocity of discrete particles and applies for *Re* < 0.5. A tank for preliminary settling of raw waters is assumed to behave as follows:

1. Type I settling applies.
2. The flow entering and leaving the tank is uniform.
3. There are three zones within the tank (Figure 6.12):
   (a) an inlet zone,
   (b) an outlet zone,
   (c) a sludge zone.
4. The particle distribution throughout is uniform.
5. The particles, on entering the sludge zone, stay there until scraped off the bottom.

Such an arrangement is shown in Figure 6.12.
For a rectangular tank:

Retention time $t = \frac{H}{V_s} = \frac{L}{V}$ 6.8

Note that $L \geq 2W$ and $L >> H$

Horizontal velocity $V = \frac{Q}{WH}$ 6.9

$$t = \frac{WHL}{Q} = \frac{V_{ol}}{Q} \qquad 6.10$$

Therefore

$$\frac{V_{ol}}{Q} = \frac{H}{V_s} \qquad 6.11$$

$$V_s = \frac{Q}{LW} = \frac{Q}{A_p} \qquad 6.12$$

However

$$\frac{Q}{A_p} = \text{surface overflow rate} \qquad 6.13$$

Where

A_p = plan area

Therefore, the settling velocity, V_s, is equal to the surface overflow rate for a rectangular tank. The same is true for a circular tank.

The question remains as to what percentage of discrete particles are removed. From the Figure 6.12, it is seen that a particle entering the tank at point a, if it settles at V_1, then leaves the tank at point e. Similarly, a particle entering at point b, settling at V_1, leaves the tank at point *f*. It is also seen (in a vector sense) that $V_1 < Q/A_p$. The percentage of particles (settling with V_1) removed is then:

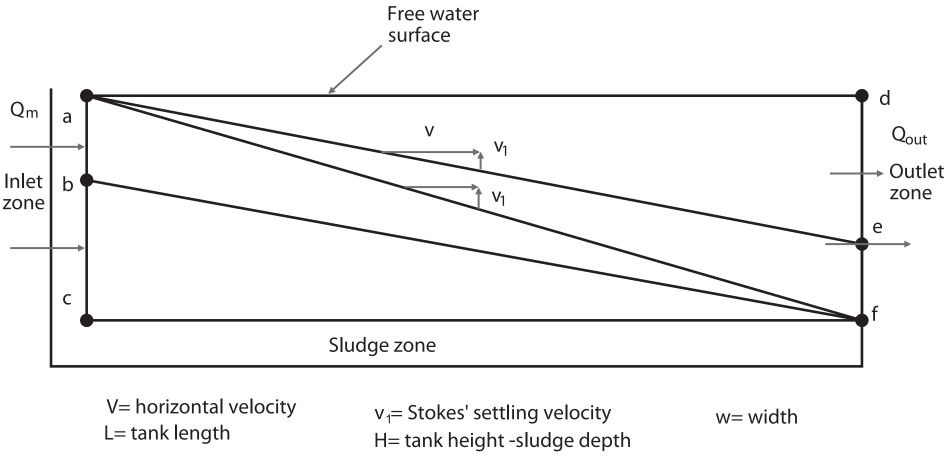


Figure 6.12: Elevation of type I setting tank

$$X_1 = \frac{b-c}{a-c} = \frac{(V_1/V)L}{(V_s/V)L} = \frac{V_1}{V_s} \qquad 6.14$$

Note that a particle entering at point a (larger than the particle of the previous paragraph), if settling at Q/A_p, will exit the tank at point *f* So all particles with a settling velocity greater than V_1 will settle out. Therefore, if all the particles were of one size (all entering at point a) and settling at k then theoretically 100 per cent settling could be removed. However, water usually has a range of particle sizes and proper design of settling tanks requires a particle size distribution analysis or a settling column test. Thus, Q/A_p or the overflow rate for any basin is a measure of the effective removal of particles in this basin.

Typically a cumulative distribution of particle settling velocity is computed for a water sample. A schematic of such a curve is shown in Figure 6.13. In any settling tank, all particles with a settling velocity > Q/A_p will settle, plus an additional as yet unknown fraction of those smaller particles with a settling velocity < V. The total fraction that settles is then

$$X_r = (1 - X_s) + \int_0^{X_s} \frac{V}{V_s} dx \qquad 6.15$$

Where

$(1 - X_s)$ is the fraction of particles with settling velocity greater than V_s and

$\int_0^{X_s} \frac{V}{V_s} dx =$ fraction of particles removed with velocity less than V_s The discrete settling equation becomes

$$X_r = (1 - X_s) + \frac{1}{V_s} \sum V \Delta X \qquad 6.16$$

Example 6.3: Size a square type I settling tank to treat 36400m^3/day of raw water, with a surface overflow rate of 12 m^3/day/m^2 and a detention time of 6 h. If the particle size distribution is given below, determine the overall removal when the specific gravity is 1.15.

Particle size mm	0.1	0.08	0.07	0.06	0.04	0.02	0.01
Weight fraction %	10	15	35	65	90	98	100
V mm/s	0.81	0.52	0.40	0.30	0.13	0.03	0.008
Re	0.08	0.042	0.028	0.018	0.005	0.0006	0.00008

Solution:

$$\text{Surface area required} = \frac{Q}{overflowrate}$$

$$A_p = \frac{36400}{12} = 3033m^2$$

$$L = W = 55\text{m, say } 60 \times 60$$

$$\text{Depth } H = V_s t$$

However, the settling rate equals the surface overflow rate (SOR):

$$\text{Actual } SOR = \frac{36400}{60 \times 60} = 10.1$$

Therefore

$$H = 10.1 \times \frac{6}{24} = 2.57\,m$$

Say

$$H = 2.6m$$

Check the weir overflow rate (WOR):

$$WOR = \frac{Q}{W}$$

$$= \frac{36400}{60}$$

$$= 606\,m^3/day/m$$

To stay within a WOR of < 300 m^3/day/m the width should be close to 120m.

Recall $Re = \frac{V_s d}{\mu}$ where μ = kinematic viscosity.

Particle size, mm	0.1	0.08	0.07	0.06	0.04	0.02	0.01
Weight fraction greater than, %	10	15	35	65	90	98	100

The settling velocity from Stoke's law, with particles of specific gravity of 1.15 is:

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

$$= \frac{9.81}{18 \times 1.002 \times 10^{-3}}(1.15 - 1.0)d^2$$

$$= 81.6d^2$$

As $Re \ll 0.5$, Stokes' law applies. All particles will be removed that have a settling velocity greater than the actual surface overflow rate (*SOR*) (plus another fraction):

$$SOR = 10.1\ m^3/day/m^3$$
$$= 10.1\ \text{m/day}$$
$$= 0.12\ \text{mm/s}$$

From the above table it is seen that slightly more than 90 per cent of the particles or greater than about 0.04mm in size are removed. To determine the precise number, a cumulative particle settling curve can be prepared (Figure 6.13)

Sedimentation of flocculent particles- type II: As defined type II settling is settling of flocculent groups of particles. Flocculent particles arc those particles that are chemically assisted to come together and produce large particles, and thus settle. Coagulation is the first process of adding the coagulated chemical which changes the particles' electric charge and is then amenable to aggregation. Flocculation is the second process of getting the 'coagulated mix' to form larger flocs. While the particles are settling, they are also flocculating and so increase in size and mass during the settling process. This phenomenon occurs in settling of chemically coagulated potable waters and wastewaters. Primary settling of wastewaters is also type II settling. Because the mass/size increases with depth, the process does not lend itself to direct analyses as was the case for type I settling. Instead, to determine the settling rates, laboratory tests are carried out using a hatch settling column. This column has openings at different points, where samples are withdrawn at regular time intervals and the suspended solids concentration is determined. The column is typically 100 to 200mm in diameter with a height equal to the proposed settling tank height (1 to 3m). The column is filled initially with a well- mixed sample of known total solids concentration, such that initially the same solids concentration exists for the full depth. For the samples taken during the settling process, the percentage removal (of solids) is computed and plotted, with equi-percentage removal curves, R_A, R_B, etc. The overflow rates (equal to the settling rates) are

$$V_s = \frac{H}{t_x} \times \text{Scale up factors} \qquad 6.17$$

Stoke's Law is valid for computation of settling velocity of discrete particles. Discrete particles are those which do not change size, shape and mass during settling and which do not influence each other by being too close. Particle settling under this condition is called discrete settling. In case of closely packed particles, the water displaced by the particles may cause additional friction and the settling velocity is reduced. This is termed as hindered settling. Hindered settling becomes noticeable when the concentration of suspended solids is greater than 2,000 mg/l. This situation of high concentration of suspended solids may happen in river water during high flooding and heavy rainfall. Sometimes settling particles may adhere to each other and grow in size and thus deviate from the settling characteristics represented by Stoke's Law. This may occur in settling of algae or freshly formed floc by the process of flocculation with coagulant. These particles/flocs tend to stick together and form new bigger particles which settle at a faster rate. This type of settling is called flocculent settling. Discrete, hindered and flocculent settling are shown in Figure 6.13

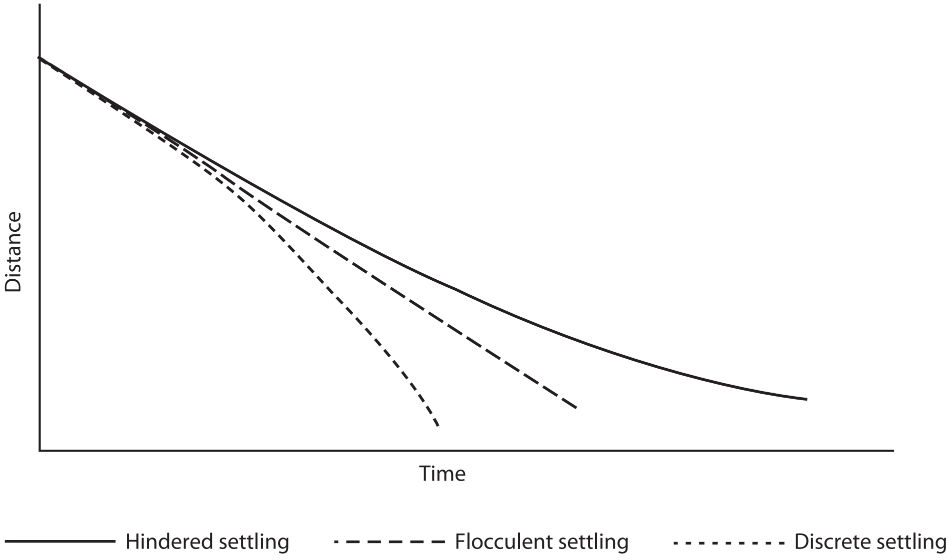


Figure 6.13: Settling of different types of particles in water.

6.12.2 Design criteria

The design of a sedimentation tank is based on the criterion as listed in Table 6.6; the sedimentation basins should have adequate capacity to handle peak flow conditions and to prevent excessive deteriorated effluent water qualities. The above design data represent common conditions, higher overflow rates may be used at lime softening plants and at some plants employing up flow clarification units as indicated in the tables of Water Treatment Plant Design by ASCE, AWWA, CSSE. Unusual conditions may dictate deviation from these general criteria. Detention time is in the range of 8 to 12 hours, or more provided in several stages, maybe necessary for treating highly turbid water. On the other hand, conical clarifiers are more efficient in softening and/or turbidity removal and require a detention time of one hour or less. The design data shall be examined by laboratory analysis or pilot plant studies especially for larger plants. Jar test for coagulant dosage in conjunction with settling column analysis to determine the optimal design criteria will be beneficial for design. If the space available for sedimentation basins is limited, multiple-story basins, in which the water flows horizontally along one level and then passes upward and then flows horizontally along another level, may be utilized. All basins must be provided with drains and overflows. Basin covers, usually in the form of a super structure, will be required in northern localities where ice is a problem.

Table 6.6: Design criteria for sedimentation tank

Design Element	Criteria
Tank depth (ft)`	10-16
Length/ width Ratio (Rectangular)	3:1 to 5:1
Diameter (circular) (ft)	10-100
Flow-Through velocity (fpm)	0.5-3
Entrance Velocity of inlet ports (fps)	0.5-2
Detention Time (hour)	2-4
Surface Overflow Rate (conventional coagulation Plant) (gpm/ ft^2)	0.35-1.5
Weir overflow Rate (gpm/ ft^2)	<15

Flocculation-sedimentation basins:

Units of this type, usually circular, combine the functions of flocculation, sedimentation and sludge removal, Flocculation is accomplished in a circular center well, Sedimentation occurs in the annular space between the flocculation section and the perimeter effluent weir. Design criteria are generally similar to those applicable to separate units.

Suspended solids contact basins:
Basins of this type combine rapid-mixing, flocculation, sedimentation, and sludge removal in a single unit. Coagulation and flocculation take place in the presence of slurry of previously formed precipitates which are cycled back to the mixing and reaction zone. Upflow rates at the point of slurry separation should not exceed about 1.0 gpm per square foot for units used as clarifiers following coagulation and approximately 1.5-1.75 gpm per square foot for units used in conjunction with lime softening.

Sedimentation with coagulation: type II:
A large portion of the suspended particles in water are sufficiently small ($<50\mu$) that their removal in a sedimentation tank is impossible at reasonable surface over flow rates and detention time.

The objective of coagulation and subsequent flocculation is to turn the smaller particles of color, turbidity and bacteria into larger flocs (increased effective size) either as precipitates or suspended particles to increase their settling velocity. The details of the sedimentation with coagulation theory were described in the section 6.11.3.

6.13 Filtration

Filtration of water is defined as the separation of colloidal and larger particles from water by passage through a porous medium, usually sand, granular coal, or granular activated carbon. The suspended particles removal during filtration range in diameter is from about 0.001 to 50 microns and larger. Several different types of medium arrangement and rates of flow through filters can be used. The filtration process most commonly used is gravity filtration, but pressure filters and diameter filters are used at smaller installations. Recently high rate filters have been developed which require less space and have higher solids loading capacity than conventional filters.

6.13.1 Theory of Filtration

The phenomenon of filtration though quite complex, is generally explained on the basis of the following four actions.

- **Mechanical straining:** This is responsible for removing such particles of suspended matter as are too large to pass through the interstices between the sand grains.
- **Sedimentation and adsorption** account for the removal of colloids, suspended and bacterial particles. The interstices between the sand grains act as minute sedimentation basins in which the suspended particles smaller than the voids in the filter-bed settle upon the sides of the sand grains. The particles adhere to the grains because of the physical attraction between the two particles of matter and because of the presence of the gelatinous coating formed on the sand grains by the previously deposited bacteria and colloidal matter.
- **Bological metabolism** is the growth and life process of the living cells. The surface layer gets coated with a zoogleal film (slime layer) in which the bacterial activities are the highest and which feed on the organic impurities convening them by a complex biochemical action into sample, harmless compounds, resulting in the purification of water.
- **Electrolytic action:** A certain amount of dissolved and suspended mailer in water is ionized i.e., carries charge of one polarity and the particles of sand in filter which are also ionized, possess electrical charges of opposite polarity. These neutralize each other and while so doing, change the chemical character of water.

6.13.2 Types of Filter media

Rapid sand filters

- **Filter Design:** Table 6.7 summarizes significant facts in filtration that are reasonably well documented and have a major impact on filter design. These concepts are of major importance in design of direct filtration plants, which are commonly the most economic treatment scheme for low-turbidity raw water (<50 TU), such as those from lakes needing low-alum coagulation dosages.
- **Filtration rate:** Rapid sand filters are those filters which commonly operate at rates between approximately 2 and 8 gpm per square foot. The usual rate is about 3 to 5 gpm per square foot. The rate of filtration to be employed at a specific plant can be determined only after careful consideration, by the designer, or raw water quality and the probable efficiency of pre-treatment that will be consistently provided. Good

Table 6.7: **Variable Affecting Filter Operation and Design**

Item	Effect
1. Pre-treatment, filtration rate and water quality	With adequate pre treatment, filtration rates from 2-6 gpm/ft^2 (.08-.24 m^3/min/m^2) produce the same water quality. Inadequate pre treatment may produce poor quality effluent even at filtration rates less than 2 gpm/ft^2.
2. Filtration rate changes, water quality rate control for design	Any rate change during filtration causes significant deterioration of the effluent quality. Degradation in quality can be qualitatively correlated with magnitude of rate change and inversely with the time for the rate change.
3. Filter medium, run length and backwashing	The finer the medium, the better the quality, but headloss increases which reduces run length. Coarse medium implies a balance between effluent quality degration and headloss buildup, but requires higher backwash rates.
4. Filtration and polyelectrolytes	Polyelectrolytes can assist in improving effluent quality when added in small amounts as a filter aid (.008-.001) mg/L just prior to filtration. Addition of polyelectrolytes makes backwash more difficult due to the higher attachment forces and auxiliaries like surface wash or air scour are necessary.

quality water is not ensured by low filtration rates. Adequate pre-treatment and filter design will allow application rates of up to 6 gpm per square foot with little difference in water quality. It is emphasized that if high rates are to be used in design, great care must be taken to insure that all prefiltration treatment process including coagulation, flocculation, and sedimentation will perform satisfactorily and consistently. High rate filter operation definitely requires excellence in prefiltration treatment, especially in the case of surface waters. It is recommended that data from laboratory or pilot studies be utilized whenever possible rather than an arbitrary selection of criteria.

- **Filter medium**
  - **Sand:** Silica sand is most commonly used filter medium and its depth should be at least 24 inches and not more than 30 inches. When sand is

employed, its effective size should be fall in the range 0.35 to 0.5 millimeter (mm) with a uniformly coefficient of 1.3 to 1.7 filter sand usually ranges in size from that passing a 16 mesh sieve to that retailed on a 50-mesh (U.S. Series) sieve. Approximately 100 percent by weight, of the sand should pass the 16-mesh sieve and 90 to 100 percent be retained on a 50-mesh sieve. Filter sand should be clean silica sand having a specific gravity of not less than 2.5. The hydrochloric acid volubility of the sand should be less than 5 percent.

- **Anthracite:** Anthracite is an alternative medium consisting of hard anthracite coal particles. The effective size commonly ranges from about 0.45 mm to 0.6 mm with a uniformity coefficient not to exceed 1.7. The hardness should not be less than 2.7 on the Moh scale and the specific gravity not below 1.4. Also, the anthracite should be visibly free of clay, shale, and dirt.

- **Multimedia:** Multimedia filters employ two or three layers of media of different size and specific gravity. A common arrangement, the dual media filter, is 20 inches of anthracite overlaying a sand layer of approximately 8 to 12 inches. The anthracite layer has size range of about 0.8 to 2.0 mm; the sand layer, about 0.4 to 1.0 mm. Tri-media filters employ an 18-inch anthracite layer, an 8-inch sand layer, and an underlying 4-inch layer of garnet or limonite having a size range of 0.2 to 0.4 mm. Garnet has a specific gravity of about 4, and limonite about 4.5.

- **Filter gravel and underdrains**

  The filter media is commonly supported by a 10 to 18 inch layer of coarse sand and graded gravel. The gravel depth may range from 6 inches to 24 inches, depending on the filter underdrain system chosen. The gravel should consist of hard, rounded stones having a specific gravity of at least 2.5 and an acid volubility of less than 5 percent. A 3 to 4 inch transition layer of coarse (torpedo) sand, having a size range of about 1.2 to 2.4 mm, is placed on top of the filter gravel. Gravel size usually ranges from about 0.1 inch to about 2.5 inches. Filter underdrains may be constructed of perforated pipe grids or various proprietary underdrain systems. A variety of the latter are available. Design details for pipe underdrains are given in numerous texts and handbooks. Manufacturers will furnish design and installation criteria for proprietary systems.

- **Sand, anthracite, gravel specifications**
  Detailed specifications for filter sand, anthracite and gravel are contained in AWWA B100.

- **Number of filters**
  Not less than two filters should be installed regardless of plant size. For large plants, rough guidance as to the number of filters to be provided may be obtained from:

  Where

  $N = 2.7\sqrt{Q}$
  N = number filter units
  Q = design capacity in mgd

  Thus, a nine mgd plant would require eight filters.

- **Size of filter units**
  The maximum filter size is related to wash water flow rate and distribution. Normally individual filters sizes do not exceed about 2100 square feet corresponding to a capacity of about 6 mgd at a flow rate of 2.0 gpm per square foot. A unit of this size would require a maximum backwash water rate of about 60 mgd, which is excessive. Consequently, it should be divided into two parts of equal size arranged for separate backwashing. Total filter depth should be at least 9 feet.

- **Filter backwash**
  Backwash facilities should be capable of expanding the filter media 50 percent. This will require wash rates in the range of 10 to 20 gpm per square foot. Backwash water can be supplied by a backwash pump or from elevated storage provided specifically for this purpose. Filter down time during wash periods commonly average 10 to 20 minutes including a 5- to 15-minute wash period. For a 15 minute backwash of a single unit, at maximum rate, the wash water volume will be 300 gallons per square foot of filtration area in that unit. In addition to backwashing, auxiliary scour is commonly provided. This aids in cleaning the filter and is commonly accomplished by rotary or fixed surface-wash equipment located near the top of the bed. It is operated for a time period equal to that of the backwash, Water pressures of 40-100 psi are required for surface-wash operation at a rate of 0.5 gpm per square foot. Air scour may also be employed but is not

generally used. If an independent washwater storage tank is used, it must refill between washes. Tank capacity should be at least 1.5 times the volume required for a single wash.

- **Wash water troughs**

  Wash water troughs equalize the flow of wash water and provide a conduit for removal of used water. Two or more troughs are usually provided. The elevation of the trough bottoms should be above that of the expended bed. The clear horizontal distance between troughs should not exceed 5 to 6 feet, and the top of the troughs not more than 30 inches above the top of the bed.

- **Filter piping and equipment**

  Essential filter control valves, etc., are shown schematically in Figure 6.14. Each filter should be equipped with a rate-of-flow controller plus associated equipment for automatic filter water-level control. The latter senses the water level in the main influent conduit and transmits a signal to the flow controllers. The controllers, in response to this signal, adjust filtration rates to match the inflow from the sedimentation basins. Thus, within practical limits, total filter outflow always equals total inflow and the filter water level remains virtually constant. A device that will sense

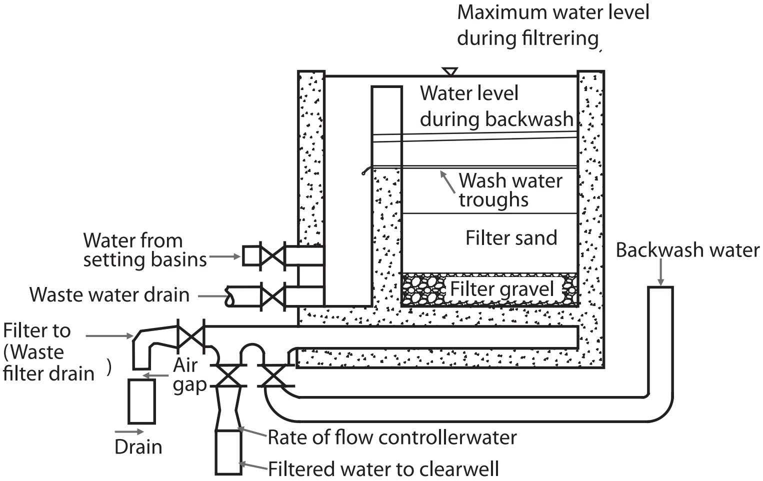


Figure 6.14: **Schematic of rapid sand operational controls**

maximum permissible clearwell level should also be provided. This should be arranged so that at maximum allowable clearwell water level, a shut-off signal will be transmitted to all filter controllers and also to an audible alarm. Other designs, not involving rate controllers, such as “influent flow splitting” and “variable declining rate” have been developed and may be employed at the discretion of the designer.

In general, each filter must have five operating valves: influent, wash water, drain, surface wash, and filter-to-waste. It is emphasized that the filter-to-waste piping must not be directly connected to a plant drain or sewer. An effluent sampling tap must be provided for each filter. Valves can be manually, electrically, hydraulically, or pneumatically operated. Butterfly type valves are recommended for filter service. Design velocities commonly employed for major filter conduits are as follows:

Conduit	Design velocity, ft/sec
Influent	1-4
Effluent	3-6
Wash water	5-10
Drain	3-8
Filter to waste	6-12

The effluent conduit must be trapped to present back flow of air and provide a seal for the rate controllers. The filter pipe gallery should have ample room and good drainage, ventilation, and lighting. Dehumidification equipment for the gallery should receive careful consideration. Filters should be covered by a superstructure except under favorable climatic conditions. Drainage from the operating floor into the filter should be prevented by a curb. Access to the entire bed should be provided by a walkway at operating floor level around the filter. Filters may be manually or automatically controlled from local or remote locations. Facilities permitting local, manual control are recommended irrespective of other control features. Used backwash water should be discharged to a wash water recovery basin or to a waste disposal facility. Regulatory agencies generally view filter wash water as a pollutant and forbid its direct discharge to the natural drainage.

- **Essential instrumentation**

  Minimum essential instrumentation for each filter will be provided as follows: rate of flow indicator; loss of head indicator; effluent turbidity indicator; wash water rate-of-flow indicating and totalizing meter. If a wash water storage tank is provided, it must be equipped with a water level indicator. While not absolutely required, a turbidity indicator on the main filter influent is desirable.

- **Diatomite filters:** Filtration is accomplished by a layer of diatomaceous earth supported by a filter element termed a septum, This layer of diatomaceous earth is about ⅛ inch thick at the beginning of filtration and must be maintained during filtration by a constant feed of diatomaceous earth (body feed) to the influent water. At the conclusion of a filter run, the layer of diatomaceous earth will have increased in thickness to about ½ inch. Filtration rates generally vary from 0.5 to 2.0 gpm per square foot. The principal use of diatomite filters has been for swimming pool waters, but some have been installed for the treatment of potable water.

- **Pressure filters:** Pressure filters are similar in construction and operating characteristics to rapid sand filters. However, in a pressure filter the media, gravel bed, and under drains are enclosed in a steel shell. There are a variety of new pressure filters in use today. The most common of these are the conventional down flow filter, the high-rate down flow filter and the up flow filter. An advantage of any pressure filter is that any pressure in waterlines leading to the filter is not lost, as in the case of gravity filters, but can be used for distribution of the filter effluent. Between 3 and 10 feet of pressure head are lost through the filter. The primary disadvantage of a pressure filter is that, due to the filter being enclosed in a steel shell, access to the filter bed for normal observation and maintenance is restricted. Also, the steel shells require careful periodic maintenance to prevent both internal and external corrosion. The use of pressure filters is not advantageous in most systems. However, if the pressure requirements and conditions in a particular system are such that re-pumping of filtered water can be eliminated, cost savings will be realized,
- **Conventional downflow filters:** Conventional downflow pressure filters consist of a bed of granular media or multimedia and are good in removing suspended solids comprised of floc. The advantages over gravity filters include lower installation cost and adaptability to different piping systems. Hydraulic loadings range from 1 to 4 gpm/sq. ft.
- **High-rate downflow filters:** High-rate down- flow filters have

filtration rates of 10-20 gpm/sq. ft. The higher downflow velocities require coarser media which allow suspended solids to penetrate deeper into the medium. As a result, more solids can be stored in the filter bed before backwashing is required, many units exhibit 1-4 lbs/sq. ft. solids-loading capacity, The higher filtration rates also allow smaller or fewer filters to be used over conventional filters. However, the high solids-loading capacity of this filter requires higher backwashing flow rates and hence larger back-washing water storage tanks.

- **Upflow filters:** Upflow multimedia filters allow filtration of high solids-loaded liquids in concentration up to 1,000 mg/L. The advantage of upflow multi-media filters is that the coarser material at the inlet collects the heavier particles, while the finer material collects the smaller particles, thus efficiency of the filter is increased.
- **Upflow continuous backwash sand filters:** Upflow continuous backwash sand filters continuously clean the filter medial by recycling the sand internally through an air lift pipe and sand washer. The regenerated sand is then redistributed to the top of the sand bed. Once the sand migrates down to the bottom of the bed it is again airlifted and repeats the cycle. Upflow continuous backwash sand filters require no backwash valves, storage tanks, or backwash pumps, therefore their operation is greatly simplified.

Slow sand filters (SSF)

In slow sand filtration, water is allowed to pass through a bed of fine sand which retains most of the impurities present in water. It is suitable for the development of a surface water-based water supply system in developing countries. The main purposes of slow sand filters are to:

- reduce the number of micro-organisms present in the water;
- retain fine organic and inorganic solid matters;
- oxidize organic compounds dissolved in water.

The effluent of a well designed, properly operated and well maintained slow sand filter should be clear and virtually free from all pathogenic micro¬organisms, in particular the bacteria, viruses, protozoa and helminths responsible for spreading of water-related diseases. Treatment of a lightly polluted surface water by slow sand filtration will produce a bacteriologically safe water while heavily polluted waters treated by slow sand filtration may require disinfection for complete destruction of pathogens.

- **Characteristics of SSF**

  The important characteristics of slow sand filters are as follows:

  - rate of filtration is low, 0.1-0.3 m^3 per m^2 per hr;
  - very high removal of turbidity and colour, (80-85%) and bacteria (95-99.9%);
  - cleaning of filter bed by scraping and removal of a top layer of sand;
  - no pre-treatment is generally required;
  - not suitable for water having turbidity greater than 30 NTU;
  - not very effective in removing colloidal matters;
  - low-cost of operation and maintenance.

  Slow sand filtration is a suitable method for complete treatment of water in developing countries. It does not require any pre-treatment of water if the turbidity of raw water is not very high. Slow sand filters do not require complex mechanical and electrical equipment or coagulating chemicals commonly required for most water treatment processes. The skills for operation and maintenance can be easily acquired by training. Cleaning may call for unskilled labour which is available in developing countries.

- **Description:** A slow sand filter consists of an open tank containing a sand bed of approximately 0.5-0.7 m in thickness. The bed is composed of relatively fine sand, usually ungraded and free from impurities. The upper part of the tank is filled with water to a depth of some 1 m above the filter bed. It flows by gravity through the filter bed. A free board of about 0.1 m is required above the maximum water level. The filtered water is collected by an underdrain system and conveyed through flow control devices to the clear water tank. The raw water level and filtration rate are regulated by valves and control devices. The purification process is mainly located in a thin layer located at the surface of the sand bed in the so called 'Schmutzdecke'.

- **Design:** The main features of a slow sand filter consist of an inlet, the filter box, and an outlet control box as shown in Figure 6.15. The shape of filter box has no influence on the filter performance and can be rectangular, square, or circular. The box has a total height of some 2 m in order to provide space for the underdrain system, sand bed, the supernatant water and a free board. The filtration rate should be in the order of 0.1 to 0.2m/hr.

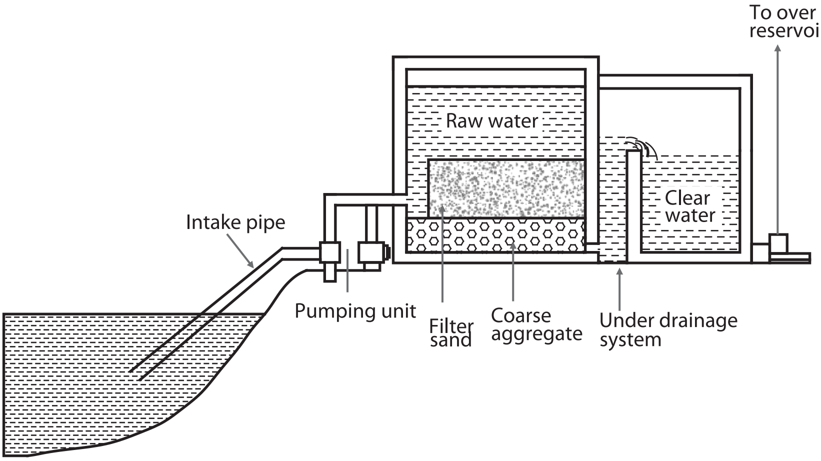


Figure 6.15: Slow sand filtration unit

Clean sand free from clay, silt and organic matter is used as filter material. The sand should not be too fine to avoid high initial head loss. The sand used for the filter bed should have an effective size, d_{10}, between 0.1 and 0.3 mm and a uniformity coefficient d_{60}/d_{10} below 3. The sand may require sieving to discard fine and coarse fractions. The underdrain system is usually composed of a gravel layer with a total height of 0.3 to 0.5 m which supports sand and provides enough space for a perforated pipe system to evenly collect filtered water. The installation of a false floor made of concrete blocks or brick can be used as an alternative to perforated pipes.

In community water supply based on slow sand filters, at least two filters are required to ensure safe and continuous operations and to allow one of the beds to be cleaned while the other one is in operation. In large plants, the number of units can be increased with a little additional cost to ensure greater flexibility in operation and maintenance. The number of rectangular filters required for a plant can be obtained from the equation:

$$n=0.5\sqrt[3]{A}$$

Where n = total number of rectangular units, A = total surface area in m^2 computed on the basis of iltration rate and design water demand.

- **Operation:** Slow sand filters perform best under continuous operation and constant flow conditions. A 24 hour operation makes maximum use of the plant. The biological activities must first be allowed to develop in a newly installed filter bed. This ripening period will take two to four weeks after first installation. Later on, cleaned filters will regain their full biological activities within two to three days provided the cleaning procedure is of few hours duration.

The flow through the filter bed is controlled by inlet and outlet structures. The inlet-controlled slow sand filter is easier to operate. The flow rate is adjusted in the inlet valve at the start of the filtration operation. The water depth on the filter bed will gradually increase with the running of the filter and development of head loss.

- **Cleaning:** Filter cleaning is required once the supernatant water reaches the highest permissible level. The proven method of cleaning a slow sand filter is by scraping off the sand surface with hand shovels to remove the top 1.5-2 cm of dirty sand. The scraped-off mixture of sand and impurities may be discarded and replaced by new sand or washed for re-use if it is cheaper than buying new sand. The thoroughly washed, scraped sand may be applied during the next cleaning operation. The cleaning frequency for well operated slow sand filters is approximately one to three months. After removal of the top layer the filter operation is immediately started in order to minimize interference with the biological activity within the filter bed.
- **Limitations:** Slow sand filtration can only treat relatively clear water. For successful operation of slow sand filters, the turbidity of water should be below 30 NTU. Higher turbidity will rapidly clog the filter bed, interfere with biological process and reduce the filter runs between filter cleanings. The presence of small numbers of algae is beneficial to the treatment process but algal bloom will result in filter failure. In case of excessive turbidity and algal growth, pre-treatment of raw water prior to slow sand filtration is generally necessary.

6.13.3 Difference of action between rapid and slow sand filters

In a slow sand filter, the action takes place principally at the surface of the sand-bed, though it's also continued for some distance below. The surface gets coated with a skin or layer formed due to the bacterial action of finely suspended matter, plankton and other organic matter present in raw water, with the algae, bacteria etc., previously coated on the surface of the sand-bed by the raw water itself. The layer is called schmutzdecke. The successful operation of a slow sand filter is principally dependent upon the existence of this layer. Below this layer and up to

30 cm. or so in depth are present other bacterial zones. Here the actions involved are to completely oxidize the organic matter, destroy most of the bacteria present and let only simple and unobjectionable inorganic salts to pass through the filter bed into the effluent.

In the case of a rapid sand filter, there is no such schmutzdecke acting as a surface straining-mat. The straining action in this case proceeds through out the depth of the filter sand rather than on the surface or to a small distance below this. Other actions involving complex biological and chemical changes as water passes through the sand-bed are similar to those in a slow-sand filter. Ii is believed that sedimentation and adsorption are largely responsible in causing the suspended load to settle on the sand grains. It is then acted upon by the previously-coated material and gets converted. The existence of the former i.e., the previously-coated material is important in this case. This material is largely fed by the pie-treatment or coagulation of water. That means that it is necessary for waters, to be coagulated before they are allowed to pass through the rapid sand filters. Other considerations for successful operation of these filters are proper thickness of sand-bed, and size of the sand grains. The differences between Filter and Characteristic are shown in Table 6.8.

Table 6.8: Types of filter and characteristic difference

Characteristics	Slow sand Filter (SSF)	Rapid sand Filter (RSF)
Suitability	Low turbid water (40-50 NTU)	High turbid water
Pre-treatment	Not required	Coagulation & Flocculation
Rate of filtration	<0.4 m/h [0.16gal/min/ft^2]	5-10 m/h [2-4 gal/min/ft^2]
Ripening	0.25day-30days (Biological slimes)	5-10 min [coagulated floc of $Al(OH)_3$]
d_{10}	0.1-0.3 mm	0.45-0.55mm
Thickness	1m-1.4m (sand)	0.3 m to several meters
Under drain	Perforated pipe grid	Gravel & perforated, tiles
Cleaning procedure	Surface scraping	Back washing
Filter run	40-60 days	1-3 days
Area	As large as 4.5 acre	Small size

6.14 Problems Caused by Deficiencies in Washing

Poor filter bed condition has often been due to neglect, but in some instances the neglect had a plausible rationale in the belief that sand coated with a gelatinous film is more effective than clean sand in removing impurities from water. This concept was a hangover from the early days of slow- sand filtration. Until about 1935, filter designers sought gentle rather than vigorous agitation of the sand during the washing process in order not to disturb the gelatinous coating around the sand grains. It is now known that such a coating has little value, and that it is readily removed with good backwashing. A thin coating does, however, assist in removal of iron or manganese. Except for problems caused by uneven distribution of wash water and improper grading of the filtering materials, nearly all filter bed trouble is attributable to failure of the washing process to remove from the filter the material that has been filtered from the water.

- **Formation of mud balls**
  With deficient washing, the compacted material accumulates in the bed and grows increasingly compact. The particles grow to larger size by adhering to each other and to sand grains at the surface of the bed. Newly formed mud balls remain at the filter surface when the wash water is cut off, as shown in Figure 6.16.

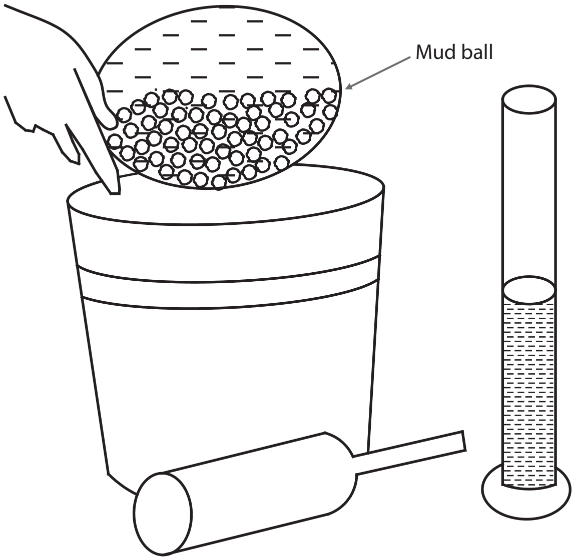


Figure 6.16: Equipment for making mud ball volume measurements

The mud balls grow increasingly compact and eventually reach a density great enough to cause them to sink into the sand during the washing period. While they are forming and being compacted they are also being worn away gradually by abrasion by the sand during the washing periods.

Some of them clump together in the bed and form mud deposits at the bottom of the sand bed or against the sidewall of the filter. If the rate of wearing away is as great as the rate at which the mud balls are built up, the volume of mud balls in the filter does not increase. (Figure 6.17)

Figure 6.17: **Mud balls on filter surface**

The rate of wearing away of clogged masses and mud balls is fairly uniform throughout the year and from plant to plant, but the rate of mud ball formation varies greatly from place to place and from time to time. In many plants, the concentration of mud balls in the filters diminishes in winter.

After a filter run it may be noted that some of the material filtered front the water is not removed from the filter at the usual rate of backwashing.

Part of the mud or coagulated material has become compacted during filtration. While the density of the compacted particles is less than that of the sand grains, the particles are much larger, and this gives them a settling rate through water nearly as great as that of sand. If the mud particles are not broken up by an agitating force much greater than that produced by backwash alone, these particles are not removed.

- **Bed shrinkage during filtration**
  When the sand grains in a filler are clean, they rest directly against other grains in the bed, and unless there is some vibration of the filter, there will be little compacting of the bed as the loss of head increases. If the sand

grains are covered with a soft coating, the bed compresses as the loss of head increases. Cracks may form as shown in Figure 6.18.

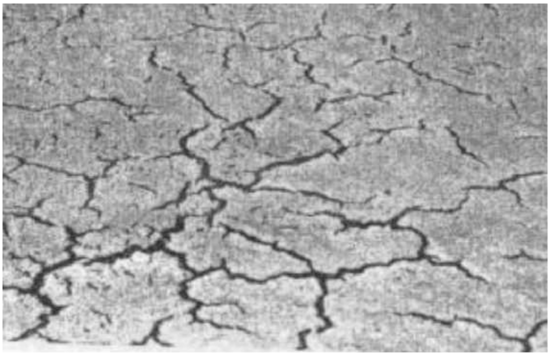


Figure 6.18: Cracks in filter beds

Measurements of vertical compaction have been made in a number of filter beds. The measurements were made with a light vertical rod supported by a flat base on the sand surface, placed after each wash. Movement of the rod was periodically measured from a reference point at the top of the filter. The curves show the progress of bed compaction as the head loss increases.

The upper line in Figure 6.19 shows what happened when the sand grains had a fairly thick coating. The middle curve shows the settlement of the

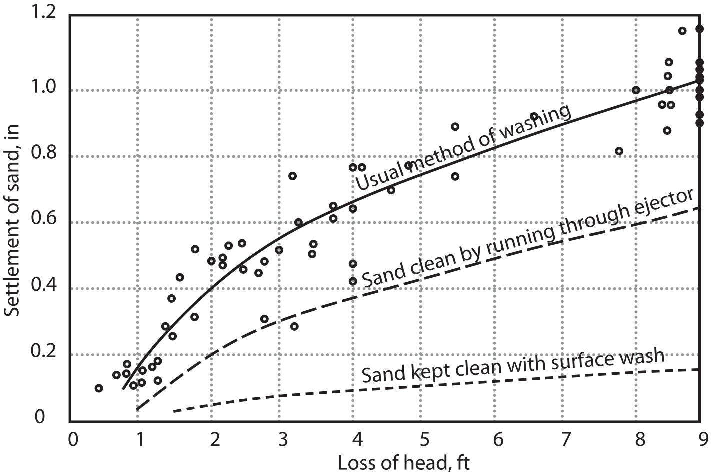


Figure 6.19: Relation between the loss of head and settlement of sand in filters

same sand after all of the mud balls and part of the coating were removed by running the sand through a hydraulic injector. The settlement was reduced, but it was still substantial. The lower line represents the settlement in a filter bed kept clean by a surface wash. The measurements on the surface-washed filter show that if the sand grains are kept reasonably free from coating there is little compacting of the bed while the filter is in service.

- **Pulling away from sidewalls**

  Settlement of the beds contributes to certain other filter bed troubles, for it allows beds to pull away from the sidewalls and at times permits cracks to open as shown in Figures 6.20 and 6.21. Filter beds pull away from the sidewalls only when there is substantial contraction or settlement of the bed. Horizontal filtration begins to take place through the openings along

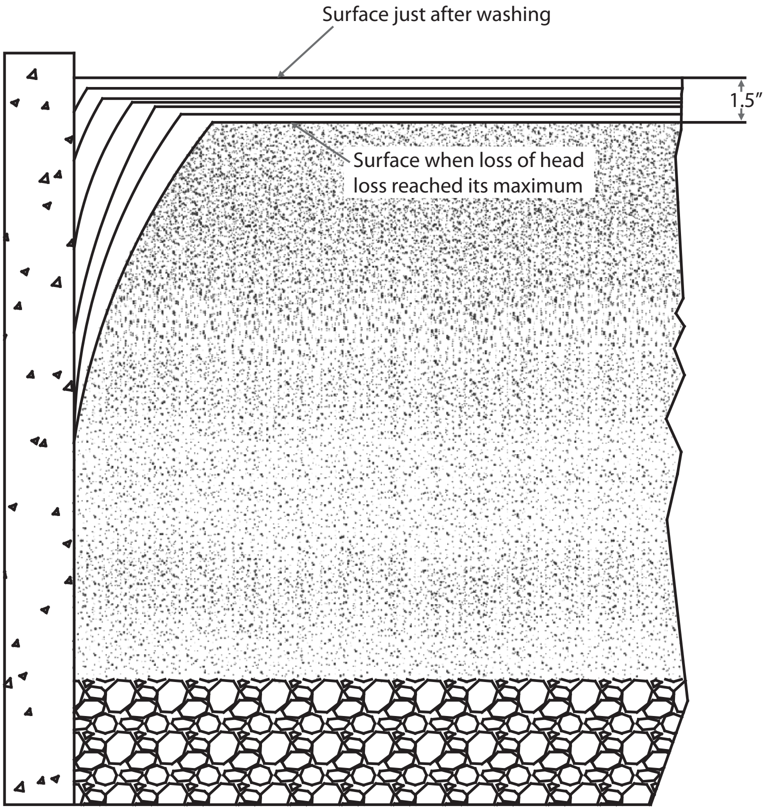


Figure 6.20: Cracks along sidewalls of filters

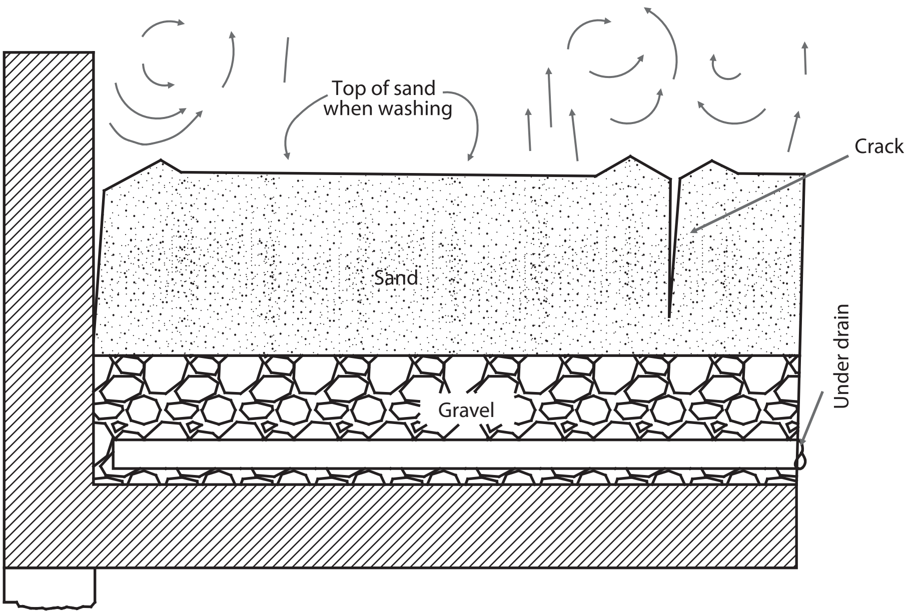


Figure 6.21: Sketch showing typical surface cracks in top 3 in of filter beds

the sidewalls, and it compresses the bed laterally. If the sand grains are so clean that the vertical compressibility of the bed is small, the bed cannot be compressed laterally. Cracks perhaps in wide may occur at the surf ace, and 2 to 4 in. below the surface with fairly clean sand, but no sidewall cracks will form.

Cracks along the sidewalls of filters are objectionable because:

- finer media at the top of the bed, which is the most effective part of the bed, are bypassed; and
- clogged areas are caused to form along the sidewalls.

- **Clogged areas**

  For the formation of clogged areas there must first be an accumulation of mud or mud balls in the filter. This is succeeded by shrinkage of the bed as the loss of head increases.

  There is a tendency for the mud balls near the surface of a filter to dump together during the washing process, especially where the expansion of the bed is not more than 20 to 30 percent. This is more pronounced near the sidewalls than in other parts of the bed. If a dump of mud balls that are drifting about in the filter during washing moves over to the sidewall, it frequently remains there and grows by accretion until it becomes a clogged mass as shown in Figure 6.22. Additional clogged masses can form in other parts of the bed, as indicated in Figure 6.22.

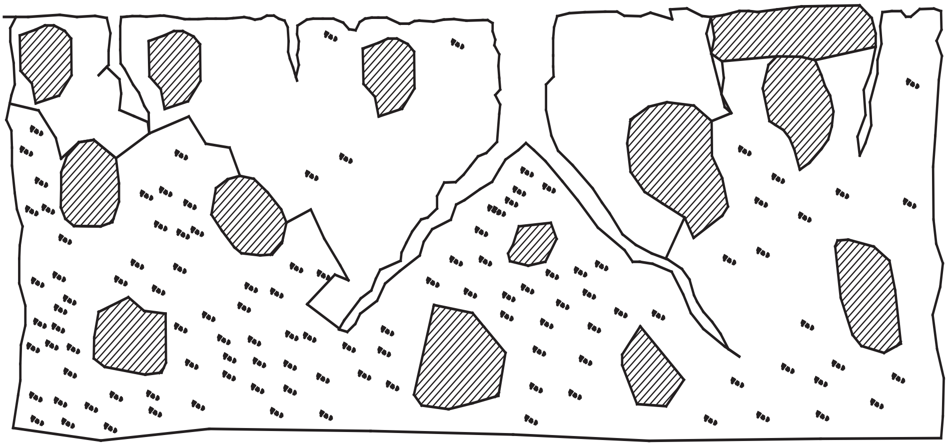


Figure 6.22: Clogged areas in a filter bed

Figure 6.23 is a sketch illustrating both typical sidewalls clogging and clogging in another part of the bed. The sidewall-clogged mass rests upon the gravel and does not adhere to the filter sidewall. It Leans away from the sidewall as the loss of head increases, sometimes opening a crack all the way to the gravel. There is some side filtration through the clogged mass, especially in the upper part where the sand usually is not badly clogged, even though the mass is not agitated much during the washing because of the impermeable mass beneath it. There may be a very gradual displacement of this sand. The clogged mass moves against the sidewall during the washing period and is pushed away as the loss of head increases on the filter. In this condition the filter fails to do a satisfactory straining job.

Jet action helps pile up the mud balls at the gravel surface. Such pileups in turn accentuate the jet action. Prevention of mud ball formation by use of a surface wash at the top of the sand bed precludes the accumulation of mud balls at the gravel surface and thereby reduces the tendency of jet action to disrupt the filter bed.

Figure 6.23: Sand ridged by clogged places in the filter bed

The accumulated sediment which can be observed in the clear water reservoirs of many filtration plants is proof that suspended matter is getting past the filters. This is often due to shrinkage which leads to cracks along the sidewalk of the filters or cracks within the bed and to clogged masses in the beds. In some instances, such a condition constitutes a menace to health. In any event, the passage of any coagulated matter through the filter bed is objectionable and should be avoided even at considerable cost.

- **Pressure sand filters**

  Pressure filters are based on the same principle as gravity-type rapid sand filters, but the under drains, gravel, and sand are placed in a cylindrical tank, and the water is passed through the filter under pressure. In pressure filters the tank axis may be either vertical or horizontal. Filters of this type are intended for use where raw water is supplied under pressure and where it is desired to filter and deliver the water without further pumping. A cross section of a pressure filters is shown in Figure 6.24.

  Finished water from a pressure filter requires as careful pre-treatment and as good a quality of applied water as would be necessary for finished water from free-surface filters. The extent and character of pre-treatment are, of course, governed by the quality of the effluent desired.

  Pressure filters are subject to the same operating difficulties as afflict

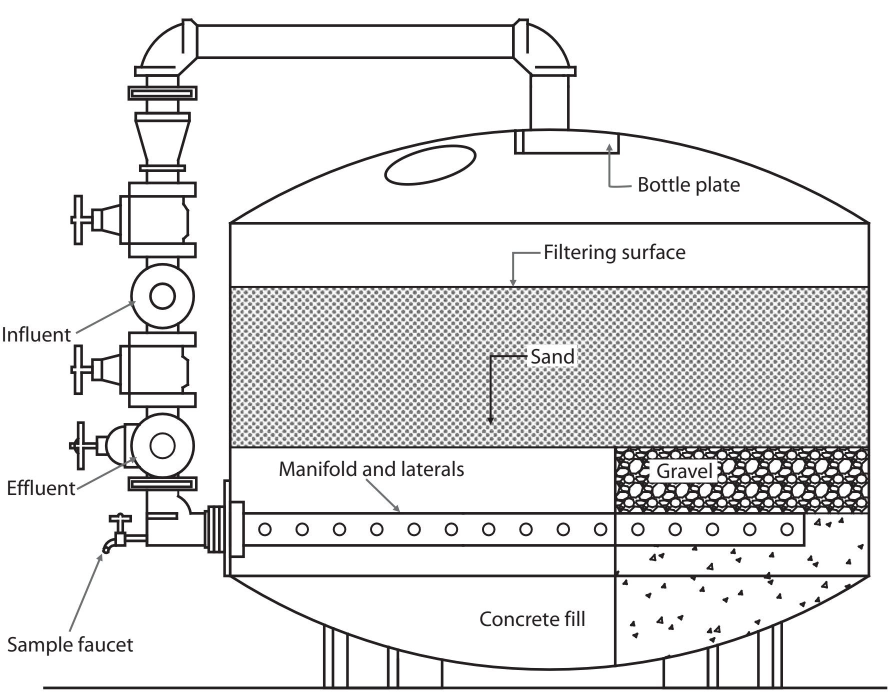


Figure 6.24: Cross section of typical pressure filter

standard rapid filters, except that air binding rarely occurs. Most of the troubles encountered are a result of careless operation or improper application of the water by the operator, both inherent in the nature of these filters. Their use for the production of a domestic water supply is not approved by some authorities.

- **Filters in softening plants**

  The conventional rapid-sand filter can be used for the clarification of lime-softened water as well as coagulated water, and the design details of filters for both services are substantially the same. In softening plants the function of the filter is to remove fine particles of calcium carbonate which have not been removed by the settling basin. The load on filters can be much heavier in softening plants, and more frequent backwash is sometimes needed. On the other hand, in some plants the precipitates produced are very dense and settle rapidly, and very long filter runs result.

  There is a tendency for calcium carbonate to precipitate on the sand grains and to increase their size. In time, this process may increase the grain size to a point where backwashing is no longer effective, and the sand must be replaced. Recarbonation of the water ahead of the filters is frequently employed to prevent this difficulty. Some operators feed polyphosphate to the settled water to prevent this incrustation.

6.15 Disinfection

Disinfection involves destruction or inactivation of organisms which may be objectionable from the stand point of either health or aesthetics. In as much as the health of water consumers is of principal concern to those responsible for supplying water, design of facilities for disinfection must necessarily be carefully executed.

6.15.1 Chlorination

The application of chlorine for disinfection of drinking water goes as far back as the nineteenth century. By the year 1800 sewage had already been treated with chlorinated lime in England and France. The large-scale technical application of chlorination to drinking water was first performed in the United States at the beginning of this century. It replaced the slow sand filtration in use at that time. During the past 50 years, it has developed into the most widely used procedure for the treatment of surface water. In many countries, it is still performed as a 'breakpoint' chlorination following sedimentation and filtration.

Under natural conditions, chlorine (Cl_2) is a yellow-greenish gas showing high toxicity to humans and animals. Due to its high reactivity, it does not occur naturally as Cl_2 but forms many compounds found on earth such as the well-known NaCI (common salt). Most chlorine is produced on a large-scale industrial process known as the chlor-alkali electrolysis. Chlorine can be liquefied at room temperature. Therefore, it can be stored and transported. It is a very efficient oxidizing, bleaching and disinfecting agent. In water, chlorine reacts as follows:

$$Cl_2 + H_2O = H^+ + Cl^- + HOCl \qquad 6.18$$

HOCl is hypochioric acid. Depending on the pH value of the water, HOCI can dissociate to hypochiorite:

$$HOCl = H^+ + OCI^- \qquad 6.19$$

Hypochloric acid and hypochlorite ions represent the 'free chlorine residual' which is the primary disinfectant employed. Experiments have shown that the undissociated HOCl molecule is the most effective compound for the disinfecting process. Its efficiency is considered to be 80 times as high as that of the hypochiorite ion (OCl^-). Figure 6.25(a) shows the dissociation of HOCl as a function of the pH value. At a pH > 5, the reaction equilibrium is shifted to the left, i.e. the lower the pH value the greater the quantity of Cl_2 occurring in the water. At a pH > 5, the concentration of OCl increases until it reaches 100 per cent at a pH of 10. The optimum pH range for disinfectant application lies in the range 6 to 8.

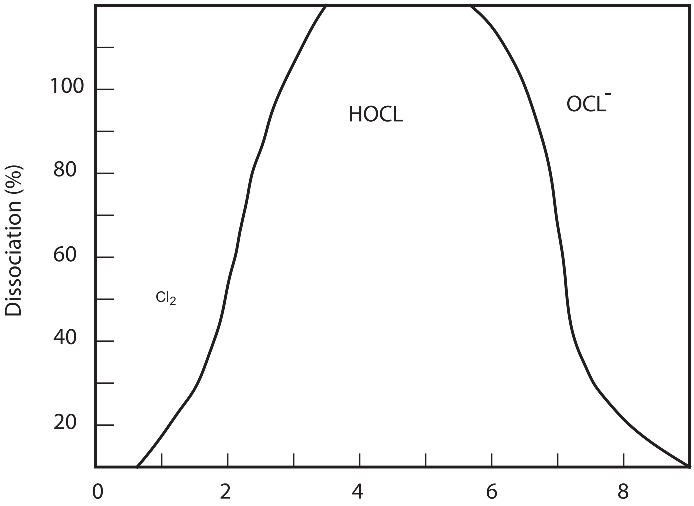


Figure 6.25(a): **Dissociation of HOCl**

Mechanism of pathogen kill by chlorination:

- A penetration into the cell of micro-organisms with subsequent blocking of an essential enzyme
- Destruction of cell walls

Factors affecting the process:

- Chemical form of chlorine
- pH
- Concentration
- Contact time
- Type of organisms

- Suspended solids
- Temperature

Measurement of chlorine: Most of the chlorine feed systems in use are gas-to-solution systems, implemented only for indirect chlorination. Indirect chlorination means the preparation of a chlorine solution from Cl_2 gas and water on site, which then serves as the disinfectant Instead of adding Cl_2 gas to the water it is also possible to use chlorine in an ionized state, e.g. as the compounds $Ca(ClO)_2$ or NaClO. This procedure is chosen particularly when small amounts of water are to be treated with a relatively small expenditure on technical equipment. Another consideration for choosing hypochlorite is for safety reasons. Chlorine gas is very toxic and its handling requires extreme care. In spite of the relatively safe injections systems, chlorine gas cylinders have to be transported, stored and changed when empty. Since chlorine gas is heavier than air, it spreads slowly at ground level. Therefore, in highly populated areas, the use of hypochlorite may be advantageous. Hypochlorite solutions in water are alkaline, which is a disadvantage as the disinfection efficiency is less at higher pH values. There is some difficulty involved in dissolving $Ca(ClO)_2$. This compound contains approximately 70 to 80 per cent available chlorine. It is often used on a short-term basis in the form of 'Chlorine' tablets at special locations, i.e. water storage containers or after some minor works on the distribution system. $NaClO_2$ solutions are more commonly used. The solution contains only 15 to 17 per cent chlorine and is not very stable. It degrades with time, particularly when exposed to light. The application of both hypochlorites is much more expensive than the application of chlorine gas.

The accurate measurement of chlorine is a very important control tool for efficient but careful dosage of chlorine, in the German Drinking Water Directive, a residual of at least 0.1 mg/L must be detectable after the disinfection step. The maximum level must not exceed 0.3 mg/L (except for extraordinarily occurring events: 0.6 mg/L are allowed over a short period). These relatively low levels take into account the disadvantages associated with chlorination and seek to minimize these. In the case of some groundwater supplies, the water is naturally free from pathogens. Either there is no chlorination at all or the addition of chlorine is performed on a very low level (about 0.02 mg/L detectable). This procedure is not disinfection, only a 'conservation precaution' to ensure the microbiological quality of the water during the distribution step.

Minimizing chlorine by Redox potential: A method used for the minimization of the chlorine application is the measurement of the redox potential. The target is to use the smallest amount possible to fully disinfect, while avoiding side reactions, i.e. associated with unpleasant taste and odour of chlorine and the formation of undesirable compounds. The redox potential is determined by the Nernst equation:

$$E = E_0 + RT/nF \, 1\text{n}(\text{Ox})/(\text{Re}d) \qquad 6.20$$

Where,

E = redox potential
E_o = potential against normal hydrogen electrode
R = gas constant
F = Faraday constant
n = number of electrons transferred
Ox = concentration of oxidized compounds
Red = concentration of reduced compound
T = absolute temperature, K

The equation (6.20) shows the dependence of the redox potential on the concentrations of redox pairs. This means that if strong oxidants like oxygen. chlorine, etc., are present, the redox potential is high and positive. Groundwater containing no oxygen would have a negative value. The advantage of determining the redox potential lies in the fact that it gives information about the disincentive ability of water. There may be waters with a high concentration of the disinfectant. However, this quantity might not be available for the disinfection process, because of the presence of chlorine using impurities. All the chlorine would be used for oxidation rather than disinfection. Due to the fact that the redox potential measures the relation of both oxidizing and reducing compounds it gives a reliable statement about the still available disinfection power within water (see also redox potential).

Positive chlorine by analysis: However, studies of the continuously measured redox potential gave information on the microbiological quality at any time and helped the setting of the chlorine dose. Later, when no chlorination was performed at all, the level of the redox potential remained greater than 600 mV. This assured the good microbiologial quality of the water. It was empirically derived that at a redox potential value of 600 mV bacteria cannot survive. The pH value and temperature influences must be considered.

Due to its reactivity, chlorine reacts with a multitude of inorganic and organic materials present in the water. In some cases, it is used as an oxidizing agent first to break down these materials. This type of chlorination is called 'breakpoint' chlorination. Due to the high dosages of chlorine normally employed, this procedure is no longer performed in Germany, but is still widely used in the United States and other countries that predominantly use surface water as the potable water source.

Figure 6.25 (b) shows a generalized curve of chlorine added versus chlorine residual, obtained during breakpoint chlorination. The oxidizable materials (even small amounts of BOD) consume chlorine before it has a chance to react as a disinfectant. At the breakpoint, these reactions are complete and it is only then that disinfection can take place. Continued addition of chlorine past the breakpoint will result in a directly proportional increase in the free available chlorine (unreacted hypochlonte).

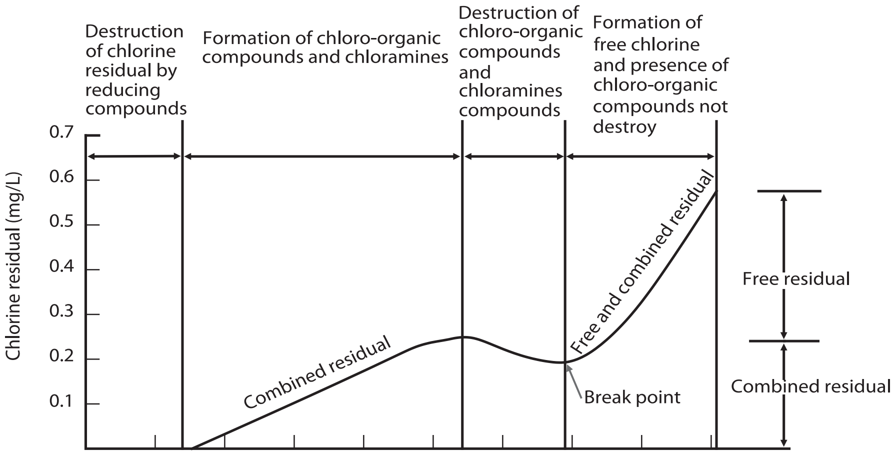


Figure 6.25(b): Generalized curve of chlorine

The by-products of organics oxidized by chlorine are often undesirable. Minute quantities of phenolic compounds react with chlorine to form severe taste and odour problems, due to the formation of chiorophenols ('medicinal' smell and taste). A concentration as low as 1:20000000 can still be detected by the human nose. Materials (e.g. glue) used during work in wells or pipes can sometimes contain phenolic compounds. To avoid customer complaints, it is recommended that the materials used in pipeline repairs be tested in advance or that the repair areas be thoroughly cleaned afterwards by rinsing with unpolluted water. Another important reaction is the formation of halogenated hydrocarbons, including the trihalomethanes (THM).

The most important classes of by-products are:

- Trihalomethanes
- Chlorinated phenols
- Halogenated methanes, ethanes and ethenes
- Halogenated polynuclear aromatic hydrocarbons
- Chlorinated aldehydes and ketones

6.15.2 Terminology of chlorination process

Terms frequently used in connection with chlorination practice are defined as follows:

- **Chlorine demand:** The difference between the concentrations of chlorine added to the water and the concentration of chlorine remaining at the end of a specified contact period. Chlorine demand varies with the concentration of chlorine applied, time of contact, temperature, and water quality.
- **Chlorine residual:** The total concentration of chlorine remaining in the water at the end of a specified contact period,
- **Combined available residual chlorine:** Any chlorine in water which has combined with nitrogen. The most common source of nitrogen is ammonia, and compounds formed by the reactions between chlorine and ammonia are known as chloramines. The disinfecting power of combined available chlorine is about 25 to 100 times less than that of free available chlorine.
- **Free available residual chlorine:** That part of the chlorine residual which has not combined with nitrogen.

6.15.3 Chlorination practice

- Combined residual chlorination. Combined residual chlorination entails the application of sufficient quantities of chlorine and ammonia, if ammonia is not present in the raw water, to produce the desired amount of combined available chlorine (chloramine) in water. If enough ammonia is present in raw water to form combined chlorine residual, only chlorine need be added to the water. Combined residual chlorination is generally used only when maintaining an adequate free chlorine residual in the distribution system is difficult or when objectionably high levels of total trihalogen methanes TTHMs would be formed as a result of free residual chlorination. Due consideration of other TTHM control alternatives should be made before using chloramines.

- **Breakpoint chlorination:** If water contains ammonia or certain nitrogenous organic matter which reacts with chlorine, the addition of chlorine causes the formation of chloramines until the ratio of elemental chlorine to ammonia compounds is about 5 to 1. Further addition of chlorine results in the oxidation of chloramines to gaseous nitrogen and nitrogen oxides, which decreases the quantity of chloramines present. After all of the chloramines have been oxidized, additional chlorine added to the water forms only free available chlorine. The point at which all of the chloramines have been oxidized and only free chlorine is formed is called the “breakpoint. “If no ammonia is present in the water, there will be no breakpoint. The chlorine required to reach the breakpoint is usually about 10 times the ammonia nitrogen content of the water. However, in certain waters, because of the presence of other chlorine consuming substances, as much as 25 times the ammonia nitrogen concentration may be required. Enough chlorine should be added past the breakpoint to ensure adequate free chlorine residual.
- **Marginal chlorination:** Marginal chlorination involves the application of chlorine to produce a de- sired level of total chlorine residual regardless of the relative concentrations of free or combined chlorine present. In marginal chlorination the initial chlorine demand has been satisfied but some oxidizable substances remain.

6.15.4 Other effects of chlorination

In addition to the disinfection achieved with chlorination, other beneficial effects should be noted. Since the oxidizing power of chlorine is high, in the presence of free chlorine, hydrogen sulphide is oxidized, nitrites are oxidized to nitrates, and soluble iron and manganese are oxidized to their insoluble oxides. Free chlorine also re- acts with naturally occurring taste, odor and color producing organic substances to form chloro-organic compounds, e.g., trihalomethanes.). The US EPA, after much discussion over costs/benefits, has chosen a maximum contaminant level for THM of 100 micrograms per liter (μg/L) for plants serving above 10,000 person and has indicated a future goal of 10-25 μg/L. The present maximum contaminant level of 100 μg/L was a compromise with the water treatment industry to avoid costly modifications to existing plants. To reach the US EPA's future maximum contaminant level for THM's, more significant changes in disinfection practices will be required.

6.15.5 Application of chlorine

Chlorine may be applied to water of two forms: As gaseous elemental chlorine or as hypochlorite salts. Gaseous elemental chlorine shall be used for water disinfection at all fixed installations. The cost of hypochlorite salts is prohibitive in all plants larger than 0.5 mgd. For remote sites at fixed installations, some well sources require 5 gpm or less. These sources with small demands can use hypochlorite for disinfection.

- **Point of application:** Chlorine may be applied to water in a variety of locations in the water treatment plant, storage facilities, or distribution sys- tem. It is absolutely essential that the chlorine applied to the water be quickly and thoroughly mixed with the water undergoing treatment. If required, special chlorine mixing facilities should be provided. In conventional water treatment plants, chlorine may be applied prior to any other treatment process (prechlorination), following one or more of the unit treatment process (postchlorination), and again in the more distant points of the distribution system (dechlorination).
    - Prechlorination has often been used so the water would maintain chlorine residual for the entire treatment period, thus lengthening the contact time. The coagulation, flocculation, and filtration processes were thought to be improved by prechlorination of the water, and nuisance algae growths in settling basins were reduced. In prechlorination, the chlorine was usually injected into the raw water at or near the raw water intake. Prechlorination was the most accepted practice of disinfection in the past. However, since many surface waters contain THM precursors that will combine with the free chlorine during prechlorination and form potentially carcinogenic THMs, such as chloroform, the point of application has been shifted further down the treatment process to take advantage of precursor removal during treatment.
    - Postchlorination generally involves the application of chlorine immediately after filtration and ahead of the clear well. The design and construction of water treatment plants for military installations will include the necessary provisions for changing the locations of chlorine applications as may later be desirable for improving treatment or disinfection processes.

  - Dechlorination is the practice of adding chlorine to water in the distribution system to maintain minimum chlorine residual through out the system.
- **Chlorination equipment:** Hypochlorite salts must be applied to the water in solution form. Hypo-chlorite solutions are pumped by a diaphragm pump through an injection system into the water to be chlorinated. If elemental chlorine is used for disinfection, it shall be injected by solution-type chlorinators. Since chlorine solutions are acidic, many components of a chlorination system must be constructed of corrosion resistant materials such as glass, silver, rubber, or plastics. Maintaining the chlorination apparatus in a trouble-free state is essential, Key spare parts and repair kits for chlorination systems must be kept on hand. Critical components of the chlorination system shall be installed in duplicate.
- **Automatic control:** If automatic chlorination control is utilized, the chlorine feed rate should be controlled primarily by the rate of flow of water, with a signal from a downstream residual chlorine analyzer used to trim the feed rate. Provision for manual control during emergency situations must be included.
- **Superchlorination and dechlorination:** Super chlorination may be necessary if there are large variations in chlorine demand or if available contact time is brief. Water which has been superchlorinated generally requires dechlorination before discharge to the distribution system. Dechlorination may be achieved through the application of sulfur dioxide, sodium bi-sulfite, or sodium sulfite, or by passing the water through granular activated carbon filters. The dechlorination process (and subsequent dechlorination, if necessary) shall be controlled so that the free residual chlorine remaining in the water is at least 0.2 mg/L. Careful monitoring must be practiced to assure that potentially harmful levels of THMs are not exceeded. A summary of THM regulations in the US presented in Table 6.9.

6.15.6 Alternate disinfectants

If the use of chlorine as a disinfectant causes unacceptably large concentrations of chlorinated organic compounds, and if all other methods for reducing THM's have been exhausted, such as moving the point of chlorination, aeration, and special coagulant (as shown in Table 6.10 for chloroform which is the main constituent of THMs in many cases) and if an alternate raw water source, such as a ground water source, is not available, an alternative disinfectant must be considered. Any alternate disinfectant system installed as the primary means of water disinfection shall have chlorination facilities available and operative for stand-by use. Four alternative disinfectants are discussed below; ozone, chlorine dioxide, chloramines and ultraviolet (UV) radiation, while chlorine is the least costly disinfectant, considering dosage and energy consumption basis. However alternate disinfectants are not significantly more expensive as well.

Table 6.9: Characteristics of private/public management options

Maximum Contaminant Level (MCL)	0.10 mg/l (100 microorgams per liter) Total Trihalomethans
Applicability	Community water systems that add disinfectant to the treatment process (ground and surface)
Effective	System>75,000: November 29, 1981 System 10-75,000: November 29, 1983 System<10,000: State discretion
Monitoring requirements	Running annual average of a minimum of 4 samples per quarter per plant taken on same day. System using multiple wells drawing raw water from a single aquifer may, with state approval, be considered one treatment plant for termining the required number of samples

Table 6.10: Effectiveness of various unit processes for reducing chloroform formation potential

Process	Chloroform formation potential μg/l	Chloroform formed μg/l	Remarks
Aeration followed by chloroform	66[a]	66[a]	Diffused air aeration with air to water rations up to 20:1 did not reduce cholorofrom formation potential (10 min contact time)
Coagulation, sedimentation and Dual-Media filtration followed by Chlorination	48[b]	13[b]	

Process	Chloroform formation potential µg/l	Chloroform formed µg/l	Remarks
Coagulation sedimentation		≪ 1	GAC would be effective for 3 weeks
Filtration adsorption by Granular Activated Carbon	48	≪ 10	GAC would be effective for 8 weeks
Powdered activated carbon added after coagulation and settling followed by chlorination	27^{a}	20^{a}	At PAC dosage = 8 mg/l
Ozone only	48	None found	O_3 neither forma trihalomethanes. Nor removes precursors at disinfection doses
Ozone followed by chlorination	48	48	Disinfection doses (≪ 1 mg/l)
Chlorine Dioxide only Coagulation, sedimentation and Filtration followed by:	74	< 1	ClO_2 does not from trihalomethanes
1. Chlorination	-	17^{c}	1.3 mg/l ClO_2 and 1.5 mg/l Cl_2
2. Chlorine dioxide	-	3d	Trihalomethane with chlorine formation decreases as the ratio of ClO_2 to Cl_2 increases

All tests performed on Ohio River water Chlorofrom formation Potential is the amount of chloroform formed when raw water is chlorinated past break point and stored at 25°C for a specified contact time.

a – chlorine contact time = 48 hr
b – chlorine contact time = 96 hr
c – chlorine contact time = 22 hr
d – contact time for combination of chlorine dioxide with chlorine = 22 hr

Source: Turk (1974)

- **Ozone:** Ozone is an extremely powerful disinfectant that has been used in Europe either as a sole disinfectant, or in conjunction with postchlorination to impart a persistent chlorine residual in the water distribution system. United States potable water plants have in the past used ozone to control taste and odor. Today ozonation is being increasingly used as a primary disinfectant prior to rapid mixing, flocculation and filtration. Ozonation does not produce THMs. It is reduced to oxygen and does not leave any residual disinfectant. Hence, the need for postchlorination. Ozone is generated electrically, as needed using the electric discharge gap (corona) technique. Air or oxygen stream, a cooling water stream and alternating electric current are required. Efficient cooling is essential to reduce thermal decomposition of ozone. Bubble diffusers appear to be the most economic ozone contractors available.
- **Chlorine Dioxide:** Chlorine dioxide is a highly effective disinfectant producing minimal THMs in the presence of their precursors. Chlorine dioxide uses in the United States have been limited to taste and odor control although it has been used elsewhere as a primary disinfectant and is presently receiving more attention in the United States. The common method of chlorine dioxide production is to react chlorine gas from a conventional chlorinator with a sodium chlorite solution. Following the mixing of the chlorine and sodium chlorite streams and prior to introduction into the main stream the mixed stream is passed through a packed column contactor to maximize chlorine dioxide production. A major disadvantage of chlorine dioxide is the formation of chlorate and chlorite which are potentially toxic.
- **Chloramines:** The use of chloramines as a disinfectant fell into disuse after the introduction of breakpoint chlorination. To achieve the same disinfection ability of chlorine, 10 to 15 times the amount of chloramines are needed or longer contact time is required. More chloramines are needed if high concentrations of organic material are found in the influent water, Chloramines are easy to generate, feed, and produce a persistant residual that will remain through the water distribution system. Chloramines may be produced by introducing ammonia to the water stream prior to the addition of free chlorine. This process can be optimized for minimum THM production and maximum disinfection. Recently however there has been some concern over chloramine toxicity.
- **UV radiation:** Ultraviolet disinfection of water consists of a purely

physical, chemical-free process. UV-C radiation in particular, with a wavelength in the 240nm to 280 nanometers range, attacks the vital DNA of the bacteria directly. The radiation initiates a photochemical reaction that destroys the genetic information contained in the DNA. The bacteria lose their reproductive capability and are destroyed. Even parasites such as Cryptosporidia or Giardia, which are extremely resistant to chemical disinfectants, are efficiently reduced. UV can also be used to remove chlorine and chlormaine species from water; this process is called photolysis, and requires a higher dose than normal disinfection. UV disinfection leaves no taint, chemicals or residues in the treated water. Disinfection using UV light is quick and clean and no bacteria, viruses or moulds are immune to UV.

6.15.7 Household disinfection methods

There are many situations where individuals or families would need to resort to simple and effective methods for drinking-water disinfection. These include the following:

- catastrophic conditions leading to displacement (earthquakes, floods, hurricanes, wars, or civil disturbances);
- emergencies arising from flourishing waterborne diseases; and
- resident populations and foreigners at risk in endemic areas with unsafe water supplies.

Physical methods (boiling or the use of ceramic filters), chemical methods (chlorine compounds in solution or tablet form, e.g., sodium hypochlorite solutions, calcium hypochlorite tablets, organic chlorine compounds, iodine solution, and organic iodine compounds) can be used in process of disinfection. None of these methods is entirely free from practical problems that could induce users to revert to untreated water. Fuelwood, for instance, for boiling is no longer a tenable practice, particularly in areas where it is absent or being depleted. Besides, the flat taste of boiled water discourages some consumers. The diverse types of ceramic filters have a wide range of pore sizes and present difficulties in selection. They suffer frequent clogging of the ceramic candles and often leak through disguised fine cracks. Proprietary halogen preparations frequently lead to consumer complaints and rejection because of the undesirable tastes and odours imparted to the water. It is especially so if high doses are applied inadvertently or as required in cases of heavily polluted waters. Relief agencies are often trapped in a dilemma by the requirements for importing and distributing, in addition to shortages, cost acceptability, and expiry dates. These issues encourage attempts to resolve them through the development of practical and effective techniques, simple enough to be applied by individuals or households.

Solar water disinfection, also known as SODIS is a method of disinfecting water using only sunlight and plastic PET bottles. SODIS is a free and effective method for decentralized water treatment, usually applied at the household level and is recommended by the World Health Organization as a viable method for household water treatment and safe storage.

Exposure to sunlight has been placed to deactivate diarrhea-causing organisms in polluted drinking water. Three effects of solar radiation are believed to contribute to the inactivation of pathogenic organisms:

- UV-A interferes directly with the metabolism and destroys cell structures of bacteria.
- UV-A (wavelength 320-400 nm) reacts with oxygen dissolved in the water and produces highly reactive forms of oxygen (oxygen free radicals and hydrogen peroxides), that are believed to also damage pathogens.
- Cumulative solar energy (including the infrared radiation component) heats the water. If the water temperatures rises above 50°C, the disinfection process is three times faster.

At a water temperature of about 30°C (86°F), a threshold solar radiation intensity of at least 500 W/m^2 (all spectral light) is required for about 5 hours for SODIS to be efficient. This dose contains energy of 555 Wh/m^2 in the range of UV-A and violet light, 350 nm-450 nm, corresponding to about 6 hours of mid-latitude (European) midday summer sunshine. At water temperatures higher than 45°C (113°F), synergistic effects of UV radiation and temperature further enhance the disinfection efficiency.

Therefore, SODIS is an effective method for treating water where fuel or cookers are unavailable or prohibitively expensive. Even where fuel is available, SODIS is a more economical and environmentally friendly option. The application of SODIS is limited if enough bottles are not available, or if the water is highly turbid. In fact, if the water is highly turbid, SODIS can not be used alone, additional filtering is then necessary.

The summary of conventional water treatment unit processes is tabulated in the Table 6.11.

Example 6.4: To obtain a free residual of chlorine of 0.5 mg/L. a dose of 10mg/L is added to a 36400 m /day water treatment plant. Calculate the amount of Cl_2 and NH_3 required achieving this.

Solution

$$Cl_2 \text{ required} = 36400 \times 10 \times 10^{3} \times 10^{-6} = 364 \text{ kg/day} \qquad 6.21$$

$$\text{Stoichiometrically: } NH_3 + HOCl \rightarrow NH_2CI + H_2O$$

1 mole of NH_3 reacts with 1 mole of HOCl to produce NH_2Cl (monochloramine). However, if this dose is applied 1:1, then the reaction also proceeds to form dichloramines (NH_2Cl) as in

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \qquad 6.22$$

Dichloramines arc undesirable in taste and odour. It is more common to apply Cl_2/NH_3 dose ratios of about 3:1 or 4:1 but not 1:1 to prevent the formation of dichloramines. Therefore,

$$NH_3 \text{ required} \approx 0.5 \text{ mg/L} \times 0.25 = 0.125 \text{ mg/L}$$

$$= 36\,400 \times 0.125 \times 10^{-3} = 4.6 \text{ kg/day}$$

Table 6.11: **Conventional water treatment unit processes**

Process/Step	Purpose
Screening	Removes large debris (leaves, sticks, fish) that can foul or damage plant equipment
Chemical pretreatment	Conditions the water for removal of algae and other aquatic nuisances
Presedimentation	Removes gravel, sand, silt, and other gritty materials
Micro straining	Removes algae, aquatic plants, and small debris
Chemical feed and rapid mix	Adds chemicals (coagulants, p1-I, adjusters, etc.) to water
Coagulation/flocculation	Converts non settleable or settable particles
Sedimentation	Removes settle able particles
Softening	Removes hardness-causing chemicals from water
Filtration	Removes particles of solid matter which can include biological contamination and turbidity
Disinfection	Kills disease-causing organisms
Adsorption using granular activated carbon	Removes radon and many organic chemicals such as pesticides, solvents, and trihalomethanes

Process/Step	Purpose
Aeration	Removes volatile organic chemicals, radon H_2S, and other dissolved gases; oxidizes iron and manganese
Corrosion control	Prevents scaling and corrosion
Reverse osmosis, electro dialysis	Removes nearly all inorganic contaminants
Ion exchange	Removes some inorganic contaminants including hardness-causing chemicals
Activated alumina	Removes some inorganic contamination
Oxidation filtration	Removes some inorganic contaminants (e.g., iron, manganese, radium)

6.16 Fluoridation

6.16.1 Health effects

An excessive fluoride concentration will damage the teeth of children using the water for extended periods. On the other hand, moderate concentrations, 0.7-1.2 mg/L, are beneficial to children's teeth. Most natural waters contain less than the optimum concentration of fluoride. Upward adjustment of the fluoride concentration can be achieved by application of a measured amount of a fluoride chemical to the water. For installations where it is desirable and feasible to add fluoride, control limits and optimum concentrations are as follows (as USEPA):

Maximum Daily Air Temperatures °F*	Control Limits Fluoride Concentration mg/l		
	Lower	Optimum	Upper
50.0-53.7	0.9	1.2	1.7
53.8-58.3	0.8	1.1	1.5
58.4-63.8	0.8	1.0	1.3
63.9-70.6	0.7	0.9	1.2
70.7-79.2	0.7	0.8	1.0
79.9-90.5	0.6	0.7	0.8

6.16.2 Fluoridation chemicals

Chemicals most frequently used for fluoridation are: Sodium silicofluoride (Na_2SiF_6), sodium fluride (NaF), and fluosilicic acid (H_2SiH_6). For a particular installation, the choice of chemical will depend principally on delivered cost and availability.

- **Sodium fluoride:** This chemical is commercially available in international market as a white crystalline powder having a purity of 95 to 98 percent. (Sometimes it is artificially colorednile blue.) Volubility is approximately 4 percent at 770°F. The pH of a saturated solution is 6.6. The 100 percent pure material contains 45.25 percent fluoride. It is available in 100-pound bags, 125 to 400 pound drums, and bulk.
- **Sodium silicofluoride:** This compound is commercially available as a white powder with a purity of 98 to 99 percent. Volubility is only about 0.76 percent at 770°F. The pH of a saturated solution is 3.5. The 100 percent material contains 60.7 percent fluoride. It is available in 100 pound bags, 125 to 400 pound drums, and bulk.
- **Fluosilicic acid:** This chemical is commercially available in international market as a liquid containing 22 to 30 percent by weight of fluosilicic acid. It is sold in 13 gallon carbons, 55 gallon drums, and in bulk. The 100 percent pure acid contains 79.2 percent fluoride. The pH of a 1 percent solution is 1.2, and the use of fluosilicic acid as a fluoridation agent in a water of low alkalinity will significantly reduce the pH of the water. It should not be used for fluoride adjustment of waters of this type unless pH adjustment is also provided.

6.16.3 Point of application

It is essential that all water pass the point of injection of the fluoridation chemical and that the flow rate past this point be known with reasonable accuracy. At a water treatment plant, the preferred application point is usually the combined effluent of all filters. The fluoride chemical can be fed at an earlier stage of treatment, for example, the combined filter influent, but part of the fluoride applied will be removed by the filtration process. Coagulation and lime softening will also remove a small amount of the applied fluoride. A larger dose is required to offset treatment process losses. If ground water is the supply source, the fluoride chemical should be injected into the discharge pipe of the well pump. Where the supply is from several wells, each pumping independently to the distribution system, it will be necessary to provide an injection point at each well. If flow past the injection point is variable, automatic equipment that will feed fluoride chemical at a rate proportional to flow is a requirement.

6.16.4 Fluoride feeders

Volumetric or gravimetric dry feeders equipped with dissolvers are suitable for sodium fluoride or sodium silicofluoride. Feeders should be equipped with weighing devices that will accurately measure the weight of chemical fed each day and the feed equipment should be designed to minimize the possibility of free flow (flooding) of chemical through the feeder. Normally, the feed machine's supply hopper should hold no more than 100 to 200 pounds of chemical. Large extension hoppers holding much greater quantities of dry fluoride chemical increase the danger of flooding and overfeeding and are not recommended for most installations. Solutions of sodium silico fluoride are acidic and corrosion-resistant dissolvers and solution piping must be provided where this chemical is employed. If fluosilicic acid is used, it can be applied by means of a small metering pump into an open channel or a pressure pipe. Storage tanks, feeders, and piping for fluosilicic acid must be made of corrosion-resistant material. The acid is slightly volatile and the feed system should be enclosed. If not enclosed, special exhaust ventilation should be provided to protect personnel from fluoride fumes.

6.16.5 Fluoride removal

Fluoride removal can be accomplished by passage of the water through beds of activated alumina, bone char, or tricalcium phosphate. When the capacity of the bed to remove fluoride is exhausted, it can be regenerated by treatment with a caustic soda solution followed by rinsing and acid neutralization of the residual caustic soda. Other methods of fluoride removal include electrodialysis, reverse osmosis and ion exchange. Some fluoride reduction can be obtained by water softening using excess lime treatment. Fluoride reduction by this method is associated with magnesium precipitation and the extent of fluoride removal is a function of the amount of magnesium precipitated from the water. All removal processes produce liquid wastes and suitable provision must be made for their disposal. An example of guideline of as to the fluoride removal process to be employed can be obtained from laboratory studies of process effectiveness and fluoride removal capacity, using samples of the water that is to be treated is shown in (Figure 6.26).

6.17 Advanced Water Treatment Processes

The purposes of advanced water treatment processes are:

- To take a water treated by standard processes and to improve it to an exceptionally high quality as often required by particular industries, e.g. beverage, pharmaceutical

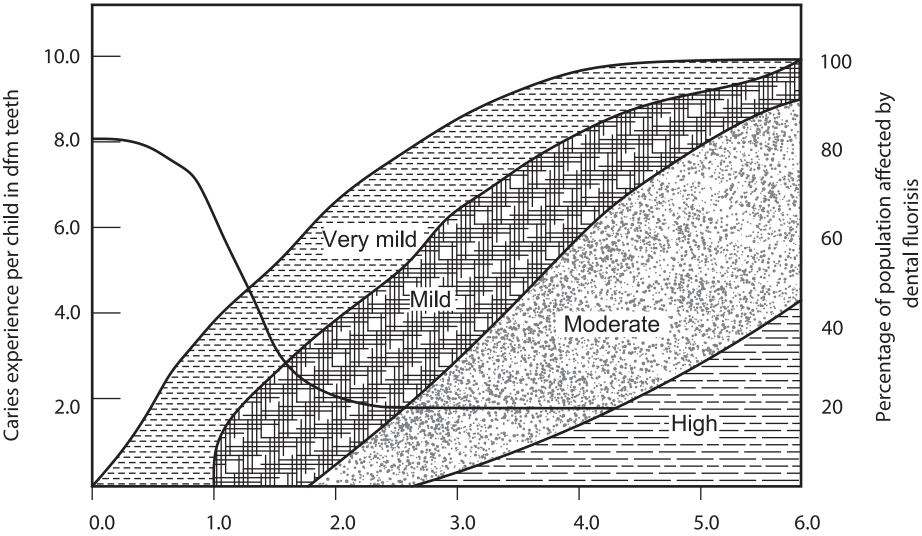


Figure 6.26: Fluoride levels

- To treat a water containing specific chemical or microbiological ‘contaminants’ to an acceptable standard, e.g. the removal of iron and manganese, the removal of blue-green algae, the removal of specific organics
- There are wide ranges of physical, chemical and microbiological techniques that can be utilized to achieve the above objectives. Some of these include:
  - iron and manganese removal (Article 6.17.1)
  - Ion exchange and inorganic absorption (Article 6.19.3)
  - Adsorption of organics (Article 6.20)
  - Oxidation including chemical oxidation (Article 6.21)
  - Membrane processes including reverse osmosis (Article 6.22)

6.17.1 Iron and manganese removal

Traces of iron and manganese are found in many waters. Quantities can range from g/L to mg/L. Both metals are often found to occur together. But there are also waters with only iron and more rarely just manganese. The natural occurrence of iron on earth is large: 4.7 per cent of the earth crust consists of iron, making it the fourth most common element. Manganese occurs as commonly as carbon, taking a share of 0.08 per cent of the earth crustal solids. Therefore in water the concentration of iron is usually higher than that of manganese. Both elements are essential trace elements for all living creatures. Iron, for example, plays an important role in the formation of the red blood colour haem (together with protein it is called haemoglobin). Manganese in animals is important for growth and healthy functioning of the nervous system. In spite of this fact, excessive quantities of both elements must be removed from water, as otherwise disturbance and damage within the distribution system would occur.

To understand how Fe and Mn manage to get into the water the occurrence of micro-organisms in the ground is examined. Bacteria use organic material as food, converting it ultimately to CO_2 and H_2O. For that purpose, they need oxygen. The easiest way to obtain oxygen is to take it out of water. Therefore, a raindrop, for example, gradually loses its oxygen on its way through the soil towards the groundwater table. After having used all oxygen in the water, the micro-organisms look for another source of oxygen. This is obtained by scavenging the oxygen from commonly occurring metal oxides, like Fe_2O_3 and MnO_2. Iron and manganese are therefore reduced to ferrous and manganous (Fe^{2+} and Mn^{2+}) compounds, which dissolve in the water. Groundwaters that do not contain any oxygen are called 'reduced' groundwaters. The properties of this kind of water are:

- Clear, colourless
- Metallic smell and/or taste
- Free of oxygen
- Sometimes a smell of H_2S

When'reduced' water comes in contact with air, it is aerated easily and the Fe is oxidized after a short time. This is seen clearly in the formation of brown flocs which then precipitate. Manganese is not oxidized purely by aeration alone, in the normally occurring pH range.

- **Forms of iron and manganese in groundwater:** When reduced to their +2 state, Fe and Mn can form ionized compounds like hydrogen carbonates (bicarbonates), chlorides, suiphates, etc. These are the forms which are easily oxidized (Fe) by oxygen. In the presence of humic acids, Fe can also form complexes. Complexes are stable, geometrically arranged, large molecules with a positive charged centre atom, here Fe^{2+} or Fe^{3+}, which is surrounded by electronegative ligands. The stability of complexes is often very high.

- **Forms of iron and manganese in surface water:** For reasons discussed above, Fe^{2+} compounds are not found in natural surface waters. The iron (Fe^{3+}) there can be dissolved by the formation of colloids consisting of Fe^{3+} hydroxide molecule groups.
- **Undesirable properties of iron and manganese:** Iron is not harmful, but undesirable on aesthetic such deposits can give rise to iron bacteria, which in turn cause further deterioration in the quality of the water by producing slimes with objectionable odours. Also, other types of bacteria use these deposits to adsorb and start breeding, which then leads to a regrowth in the distribution system.

6.17.2 Iron removal

The principal method of removing iron from these waters is to oxidize the ferrous ion to ferric iron, which can be done by aeration, and to provide a suitable filter or other method to remove the precipitated iron compound.

It has been shown that oxidation of Fe^{2+} by oxygen is slow and is therefore the rate determining step. The whole process consists of several reactions but can be satisfactorily described by the following equation:

$$2Fe^{2+} + 0.5O_2 + (x+2)\,H_2O \rightarrow Fe_2O_3.\,x\,H_2O + 4H^+ \qquad 6.23$$

- **Removal procedures for low concentrations of iron:** If the amount of iron in a water is fairly low (<1 mg/L), the removal of iron is often achieved in a pressure filter containing the usual bed of sand, but sometimes incorporating a layer of proprietary matenal such as 'Polarite' or an iron core which acts as a catalyst. As the amount of oxygen required is small, it is provided merely by introducing air into a space of the filter shell. Much faster rates can be used for this type of filtration than are possible for waters where chemical coagulation precedes sand filters: rates > 50 m/h have been successfully used. As an alternative to sand filtration, it is possible to use semi-calcined dolomitic limestone ('Akdolit') as the filter medium (contains $Ca/MgCO_3$ and oxides). This material can be effective for the removal of low concentrations of soluble iron by precipitating ferric hydroxide at its alkaline surface and the material can help render the water less aggressive. Furthermore, it can increase the amount of magnesium in the water which is believed to have beneficial effects on health.
- **Removal procedures for high concentrations of iron:** Where the amount of iron is large (> 10 mg/L), the traditional practice was to treat

the water as a surface water (where the iron is dissolved as a colloid) providing a coagulation step before rapid sand filtration. That is, however, not always necessary and should be avoided because of the high demand in equipment and personnel. It is now possible to decrease filtration rates to approximately 5 m/h or use a sedimentation tank to allow a large proportion of the femc hydroxide to settle out. Multimedia filtration, involving the use of a filter containing several layers of different materials, has proven to be highly effective. Raw water with about 15 mg/L Fe and high concentrations of humic acids can be treated at filtration rates of 15 m/h.

- **Poorly buffered waters:** Waters which are described as being difficult in terms of removal of iron are often poorly buffered; i.e. the amount of hydrogen carbonate is low. There is the formation of protons during the oxidation process. In well-buffered waters these H^+ do not cause problems, but poorly buffered waters may suffer acidification. During aeration of water, a lot of carbon dioxide is removed, the pH rises and iron begins to be oxidized. The H^+ produced reacts with HCO_3 to give more CO_2. In poorly buffered waters the pH will finally reach a low point where the reaction stops (self-inhibition). Because there is still a large amount of Fe^{2+} left, it was thought necessary to complete the reaction by adding oxidizing agents. This was later identified to be wrong, because after that it was no longer possible to destabilize the Fe^{3+} during the following filtration. The reason lies in the formation of stable colloidal solutions in which destabilization is possible when there is still enough Fe^{2+} left positive or negative charges. The stability is often so high that destabilization by filtration succeeds only in a very thin layer on the filtration material.

6.17.3 Manganese removal

Manganese occurs in natural waters as Mn^{2+}. There is no way to oxidize it by aeration in the normal pH range even if the amount is 10mg/L or higher. The pH has to be 9 or 10 before oxidation by oxygen takes place. This can be achieved only with strongly alkaline filter materials. The processes going on during the oxidation are even more complicated than for iron, as Mn can occur in different oxidation states (II, III, IV occur naturally, V, VI, VII can be produced artificially). For the removal of Mn_2 in practice it has proven favourable to add a $KMnO_4$ solution as a conditioning agent prior to the filtration step without changing the pH. The stoichiometrically evaluated value of 1.9 kg $KMnO_4$ per kg Mn is often found to be too high (or too low), depending on simultaneously occurring reduction and oxidation processes. The addition of $KMnO_4$ has to be carried out for several days (3 days to 1 month) before the process works on its own.

Figure 6.27 is a schematic of a groundwater treatment for the removal of iron and manganese. Here, the iron is removed by multimedia pressure filters. The manganese is removed by multimedia gravity sand filters.

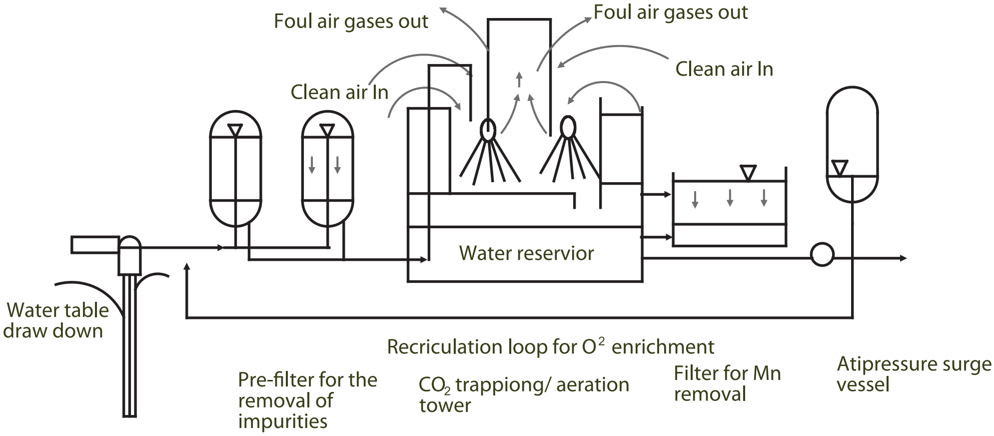


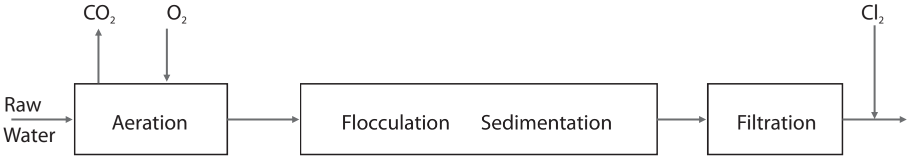
Figure 6.27: Schematic of a groundwater treatment plant

6.17.4 Mechanisms behind removal of iron and manganese

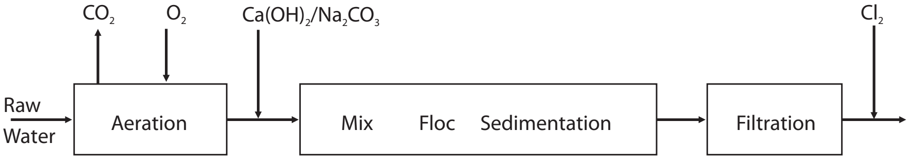
- Iron (Fe^{2+}) Manganese (Mn^{2+}) are chemically reduced (soluble form) in absence of DO & at low pH in ground water & anaerobic reservoir water(at the bottom) iron alone in ground waters which contain little or no organic material; when well acraied, CO_2 & H_2S arc released raising the pH & soluble iron(Fe^{+2}) oxidized to insoluble iron(Fe^{+3}).The rate of oxidation depends up on the type & concentration of oxidizing agent pH (>7.5) and alkalinity (should be > 103mg/l) organic content & presents of catalyst (Cu, Si).

  $$4Fe(HCO_3)_2 + O_3 + H_2O \rightarrow 4Fe(OH)_3 + 8CO_2$$

- Iron alone in ground waters which contains little or no organic matter with reasonable alkalinity when aerated CO_2 and H_2S arc released raising the pH and oxidized to insoluble iron.



- Low alkalinity water (<130 mg/l as $CaCO_3$) need some chemical additive to raise both pH / and alkalinity like lime [$Ca(OH)_2$], soda ash [Na_2CO_3] etc. If the water is softened by addition of line additional benefits include removal of iron. Aeration prior to 1ime addition reduces the chemicals through CO_2 reduction.



The insoluble precipitate can be removed through flocculation, sedimentation and filtration, if both iron and manganese arc present or in the water contains organic material (humic and fulvic acid), strong oxidizing agents (Cl_2,O_3,ClO_2 or $KMnO_4$) are required. Lime may be used for both as oxidizing agent and to rise the pH value for Low alkalinity water with organic content. Manganese and iron can be removed from solution by adsorption (filtration through manganese zeolite).

- If the water contains organic matter such as humic or fulvic acid and if the Alkalinity is low, aeration is sufficiently rapid only if it is catalyzed by accumulation of oxidation products (Fe_2O_3) on a porous bed (aeration tower containing trays with coarse media). Iron is removed from solution by adsorption on the bed. Ionic material interferes with removal by forming soluble complexes.

6.18 Taste and Odor Control

Most taste and odors in surface water are caused by low concentrations of organic substances derived from decomposing vegetation, microscopic organisms, sewage and industrial waste pollution, etc. Treatment for taste and odor removal involves destruction of the odorous substance by chemical oxidation or its removal by aeration or adsorption or activated carbon.

- **Chemical oxidation:** Chemical oxidizing agents which have been found effective and which can be used in the treatment of potable water are chlorine, chlorine dioxide, potassium permanganate, and ozone. No single chemical is completely effective under all operating conditions.
- **Aeration:** Aeration is helpful in eliminating odor caused by hydrogen sulfide, but is ineffective in significantly reducing odor associated with dissolved organics.
- **Absorption:** Powdered activated carbon is commonly used for removal of tastes, odor and color by ad- sorption. The carbon can be applied to the water at any point in the treatment plant prior to filtration, but it is usually advisable to apply it early in the treatment process to prolong contact. For maximum effectiveness, carbon should be applied well ahead of chlorine, and preferably in advance of lime softening, the influent to a pre-sedimentation basin is normally an effective carbon application point. Powdered carbon dosages usually range from 5 to 10 mg/L, but as much as 50 mg/L may be required. The use of powdered activated carbon adds more suspended solids and increases the amount of sludge, thereby creating a sludge disposal problem. Powder activated carbon is marginally effective in reducing THMs. Granular activated carbon (GAC) has also been used for taste and odor removal. It has been employed as a separate treatment step in the form of carbon columns and as a substitute for sand in the filtration process. Used in this way, the granular carbon serves in a dual capacity as a filtration medium and for taste and odor removal. Granular activated carbon is also excellent at reducing THMs. Granular activated carbon must be reactivated on a regular basis to keep its absorptive abilities. Because of the cost of reactivation of GAC, other methods of taste-and-odor control and reduction of THMs should be considered. Aeration is generally more cost effective than GAC contractors.

6.19 Softening

Whether water softening is provided will depend entirely on the type of project and the uses to be made of the water. Two general types of processes are used for softening: The “lime-soda ash” process and the “cation ion exchange” or "zeolite" process. However, the hardness can be classified in to four categories which are shown in the Table 6.12.

Table 6.12: Classification of Hardness

Hardness (mg/l as $CaCO_3$)	Description
0-75	Soft
75-150	Moderately hard
150-300	Hard
>300	Very hard

6.19.1 Applications

- **Permanent posts or bases:** Softening of the entire supply for a permanent post or base may be considered if the hardness exceeds 200 mg/l, with hardness expressed as equivalent $CaCO_2$. Softening of a post water supply to a total hardness of less than 100 mg/L is not required, however, softening to less than this amount is justified for the special purposes like laundries, boiler water and for dining facilities.
- **Nonpermanent bases:** For temporary construction and for bases not in the permanent category, the entire supply will not be softened unless the total hardness exceeds 300 mg/L. However, when a treatment plant is constructed for the removal of turbidity or iron, the plant may also be designed to accomplish partial softening.
- **Laundries:** Water for laundries shall have a hardness of 50 mg/l or less. Installation of cation ion exchange water softeners to reduce the hardness to zero is recommended.
- **Boiler water:** Boiler water for power plants and heating plants may require softening, but satisfactory results can often be obtained by application of corrosion and scale inhibitors. Depending on the pressure at which the boiler is to operate, partial water de-mineralization may also be necessary,
- **Dining facilities:** The installation of softeners for small dining facilities, latrines and bathhouses is not recommended. However, water softeners to reduce hardness to 50 mg/L maybe justified for large central dining facilities to protect equipment and to insure satisfactory washing of dishes. Each such instance will be justified separately.
- **Hospitals:** When the water supplied to a hospital has a hardness of 170 mg/L or more, the water will be softened to approximately 50 mg/L.

Where critical equipment requires water having a hardness of less than 50 mg/L, as special study will be made to determine the most feasible means of obtaining water of the necessary hardness. Zero hardness water may be piped from the main softener or maybe supplied from small individual softeners, whichever is the more feasible. The sodium content of the treated water must be taken into account when selecting a softening method for hospitals.

6.19.2 Mechanism of softening

Hardness consists principally of Calcium and Magnesium salts but includes all cation. These while not undesirable for a health standpoint, may make the water less suitable for some non potable uses. Benefits of softening to domestic users include reduction in soap use, longer life for water heaters, and less incrustation of pipes. If the water is softened by addition of lime, additional benefits include removal of iron and manganese, co-precipitation of humic and fulvic acid, and reduction of suspended solids including bacteria and viruses. Industrial users of municipal water benefit through the lower cost of producing process and boiler waters from softened water, here is evidence that consumption of very soft water may contribute to heart disease and accumulation of certain heavy metal in the liver. The two basic methods used for removal of hardness are the (a) Chemical Precipitation process and (b) Ion-Exchange Process.

Theory of softening: The different species of hardness have different solubility limits. The least soluble forms are $CaCO_3$ and $Mg(OH)_2$. The objective of chemical softening is to convert all the calcium hardness to $CaCO_3$ and magnesium hardness to $Mg(OH)_2$.

- In order to precipitate $CaCO_3$, the pH must be raised above 8.3 (optimum 9.0-9.5) to convert bicarbonate alkalinity to carbonate alkalinity.If there is not sufficient natural occurring bicarbonate alkalinity (HCO_3^-) for the $CaCO_3$ precipitate to form (i.e. there is non-carbonate hardness), we must add CO_3^{2-} source (soda ash Na_2CO_3).
- To precipitate $Mg(OH)_2$, the pH must be raised above 10.6 to convert carbonate alkalinity to hydroxide alkalinity. Sources of OH^- ion are lime [$Ca(OH)_2$] and caustic soda [NaOH].

  The addition of chemicals $(CO_3)^{2-}$ and or OH^-) elevates the pH, which shifts the equilibrium of the carbonate acid system in favour of the $(CO_3)^{2-}$ ion, i.e. it changes the alkalinity from bicarbonate to the carbonate form.

This increases the concentration of the $(CO_3)^{2-}$ ion and shifts the equilibrium to the right.

$Ca_2+ + CO_3^{2-} \leftrightarrow CaCO_3$

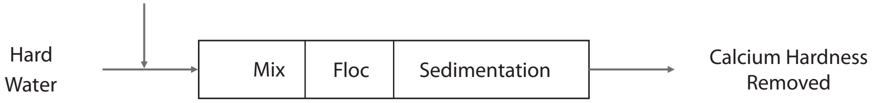
$Mg_2+ (aq) + 2OH- (aq) \leftrightarrow Mg(OH)_2(s)$

However, complete removal of hardness cannot be accomplished by chemical precipitation.

Softening Under Different Environmental Conditions:

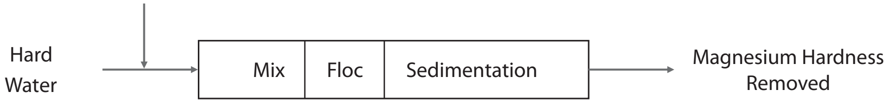
Case 1: When Alkalinity ≥ Hardness and pH < 8.3 (sufficient bicarbonate hardness is present)

- Addition of lime $[Ca(OH_2)]$ or caustic soda $[Na(OH)]$ to raise pH up to 9.4 will precipitate $CaCO_3$. $Ca(OH)_2 / NaOH$ [pH 9.4]



$$\left\{\begin{array}{l} Ca(HCO_3)_2 + CaO + H_2O \rightarrow CaCO_3 + H_2O \\ Ca(HCO_3)_2 + NaOH \rightarrow CaCO_3 + Na_2CO_3 + H_2O \end{array}\right.$$

- Addition of lime or caustic soda to raise pH> 10.6 will precipitate $Mg(OH)_2$.
  $Ca(OH)_2 / NaOH$ [pH >10.6]



$$\left\{\begin{array}{l} Mg(HCO_3)_2 + CaO + H_2O \rightarrow CaCO_3 + Mg(OH)_2 + H_2O \\ Mg(HCO_3)_2 + NaOH \rightarrow Mg(OH)_2 + Na_2CO_3 + H_2O \end{array}\right.$$

Case 2: When Alkalinity < Hardness (noncarbonate hardness, i.e. $(CI)^-$ and $(SO_4)^{2-}$ of Ca, Mg are present)

- Carbonate source like soda ash $[Na_2CO_3]$ should be added to precipitate $CaCO_3$ according to the following equation.

$$\begin{cases} CaSO_4 + Na_2CO_3 \rightarrow CaCO_3 + CaSO_4 \\ CaCl_2 + Na_2CO_3 \rightarrow CaCO_3 + NaCl \\ Ca(NO_3)_2 + Na_2CO_3 \rightarrow CaCO_3 + NaNO_3 \end{cases}$$

- Both lime and soda ash $[Na_2CO_3]$ should be added to raise pH above 10.6 for $Mg(OH)_2$ and $CaCO_3$ precipitate according to the following equations.

$$\begin{cases} MgSO_4 + CaO + H_2O \rightarrow CaSO_4 + Mg(OH)_2 \\ MgCl_2 + CaO + H_2O \rightarrow CaCl_2 + Mg(OH)_2 \\ Mg(NO_3)_2 + CaO + H_2O \rightarrow Ca(NO_3)_2 + Mg(OH)_2 \end{cases}$$

$$\begin{cases} CaSO_4 + Na_2CO_3 \rightarrow CaCO_3 + Na_2SO_4 \\ CaCl_2 + Na_2CO_3 \rightarrow CaCO_3 + NaCl \\ Ca(NO_3)_2 + Na_2CO_3 \rightarrow CaCO_3 + NaNO_3 \end{cases}$$

Case 3: Caustic soda [NaOH] alone is capable to remove noncarbonate magnesium hardness.

$$MgSO_4 + NaOH \rightarrow Mg(OH)_2 + Na_2SO_4$$

Calculation of hardness of water: There are two types of water hardness: Temporary and permanent hardness. Total permanent water hardness is calculated with the following formula:

$$\text{Total permanent hardness} = \text{calcium hardness} + \text{magnesium hardness}$$

The calcium and magnesium hardness is the concentration of calcium and magnesium ions expressed as equivalent of calcium carbonate. The molar mass of $CaCO_3$, Ca^{2+} and Mg^{2+} are respectively 100.1 g/mol, 40.1 g/mol and 24.3 g/mol.

The ratio of the molar masses are:

$$\frac{M_{CaCO_3}}{M_{Ca}} = \frac{100.1}{40.1} = 2.5 \quad \text{and} \quad \frac{M_{CaCO_3}}{M_{Mg}} = \frac{100.1}{24.3} = 4.1$$

Therefore, the total permanent water hardness expressed as equivalent of $CaCO_3$ can be calculated with the following formula:

$$CaCO_3 = 2.5\left[Ca^{2+}\right] + 4.1\left[Mg^{2+}\right]$$

6.19.3 Ion-exchange process

An ion-exchange unit resembles a sand filter in which the filtering medium is anion-exchange resin, 'Z' (either natural-zeolite or synthetic) rather than sand. As the hard water passes through the ion-exchange bed, there is an exchange of cation, the calcium and magnesium in water are exchanged for the sodium in the resin. When a significant portion of the sodium in the resin has been replaced by Ca & Mg, it is regenerated with a NaCl solution.

$$\text{Softening}\begin{cases} \mathrm{Ca(HCO_3)_2 + Na_2Z \rightarrow (HCO_3)_2 + CaZ} \\ \mathrm{CaSO_4 + Na_2Z \rightarrow Na_2SO_4 + CaZ} \\ \mathrm{MgCl_2 + Na_2Z \rightarrow NaCl_2 + MgZ} \end{cases}$$

$$\text{Regeneration}\begin{cases} \mathrm{CaZ + NaCl \rightarrow Na_2Z + CaCl_2} \\ \mathrm{MgZ + NaCl \rightarrow Na_2R + MgCl_2} \end{cases}$$

6.19.4 Recarbonation

Recarbonation involves the introduction of carbon dioxide and/or bicarbonate ion into softened water for the purpose of neutralizing excess hydroxide alkalinity and relieving calcium carbonate and magnesium hydroxide supersaturation.

Complete removal of hardness cannot be accomplished by chemical precipitation. Precipitation of the supersaturated solution of $CaCO_3$ will continue slowly, however, resulting in deposits in water lines and storage facilities. It is therefore necessary to stabilize the water by convening the supersaturated $CaCO_3$ back to the soluble form as $Ca(HCO_3)_2$. The most common practice however, is to make the conversion with CO_2. CO_2 also neutralize excess lime. This process is generally called recarbonation.

Carbon dioxide should either be purchased as liquefied carbon dioxide, which must be stored at the plant in a refrigerated storage tank, or generated at the water treatment plant by the combustion of coke, oil, or gas. Recarbonation can also be achieved by utilizing carbon dioxide and bicarbonate available in the raw water. This is the "split" treatment process.

- **Chemical reactions:** The following reactions illustrate the chemistry of the recarbonation process:
    - Neutralization of excess lime.

      $CO_2 + Ca(OH)_2 \rightarrow CaCO_3 \downarrow + H_2O$
    - Conversion of magnesium hydroxide to magnesium carbonate.

      $CO_2 + Mg(OH)_2 \rightarrow MgCO_3 + H_2O$
    - Conversion of carbonates to bicarbonates.

      $2CO_2 + CaCO_3 + H_2O \rightarrow Ca(HCO_3)_2$

      $2CO_2 + MgCO_3 + H_2O \rightarrow Mg(HCO_3)_2$
    - Use of bicarbonate derived from raw water.

      $2HCO_3^- + Ca(OH)_2 \rightarrow CaCO_3 \downarrow + CO_3^{2-} + 2H_2O$

The above reactions are accompanied by important changes in the pH of the softened water, and the pH value is used as a recarbonation control parameter.

- **Stages of Recarbonation:** Recarbonation can be practiced in a single-stage or two stage configuration.
    - If recarbonation is accomplished in two stages, the first stage is devoted to neutralization of most of the excess lime. This involves conversion of excess lime to calcium carbonate and a pH change from about 11 to approximately the 9.5-10 range. Following the first stage of recarbonation, the water must be flocculated and settled to remove excess calcium carbonate. Coagulant such as silica, starch, polymer or ferric sulfate may be employed to assist in coagulation and settling of the calcium carbonate particles.
    - The second stage of recarbonation, usually just ahead of filtration, serves principally as a “trim” stage in which final pH adjustments are made, as necessary.

  Single stage recarbonation should be followed by facilities for removal of precipitated calcium carbonate. A single stage of recarbonation provides no opportunity for further pH adjustment prior to filtration. Application of polyphosphate compound, such as sodium hexametaphosphate, may be desirable following recarbonation. Approximately 0.5 mg/L of a polyphosphate will inhibit scale formation and prevent instruction of the filter media with hard, crystalline deposits of calcium carbonate.
- **Requirement of Carbon dioxide:** Carbone dioxide requirements are estabilished by the amount of hydroxide alkalinity converted to carbonate

and the amount of by carbonate formed. For plant control purposes, alkalinity values are generally expressed in terms of calcium carbonate ($CaCO_3$). Total carbon dioxide requirements are calculated as follows, where (OH^-) is the hydroxide alkalinity in mg/L, as $CaCO_3$ in mg/L, as $CaCO_3$; and (HCO_3^-) is the desired bicarbonate alkalinity in mg/L as $CaCO_3$ in the finished water:

- Total mg/L $CO_2 = (.44)[(OH^-)+(HCO_3^-)]$
- lbs CO_2 per million gallons $= 3.67[(OH^-)+(HCO_3^-)]$
- The approximate are presented in the Table 6.13:

Table 6.13: **CO_2 yields of common fuels**

Fuel	Basis	CO_2 yield lbs
Natural gas	1000 cu ft	115
Coke	1 lb	3
Kerosene	1 gal	20
Fuel Oil	1 gal	23
Propane	1000 cu ft	141
Butane	1000 cu ft	142

Carbon dioxide injection equipment must be carefully designed to insure maximum absorption efficiency. This is especially important if pure (99.5 percent and) carbon dioxide is used. Equipment is available which will give essentially 100 percent absorption of pure CO_2 when injected as a gas. An alternative feed method for pure CO_2 is use of a chlorinator as a feeding device, with this feeding arrangement, the CO_2 is in solution at the feed point and the problems associated with interphase gas transfer are avoided.

6.19.5 Split treatment

Split treatment is accomplished by excess lime treatment of the raw water and then, after removal of most of the precipited magnesium hydroxide and calcium carbonate by sedimentation, adding the balance of the raw water and again mixing and setting. Employment of excess lime treatment in the first stage insures efficient magnesium removal. The excess lime is neutralized in the second stage by carbon dioxide and bicarbonate in the unsoftened, bypassed water. Conventional recarbonation facilities are not normally required when split treatment is used. Split treatment is often applicable to well waters where taste, odor, turbidity and color are not a problem. While the process can be employed with surface waters, it is generally preferable to follow conventional procedures in which all of the water receives two stage treatments. The principal operating problems associated with split treatment are correct adjustment of chemical feeds and bypass flow. The calcium carbonate equivalent of the HCO_3 and CO_2 content of bypassed water must equal the calcium carbonate equivalent of the OH^- content of the first stage softened water plus the desired bicarbonate alkalinity in the finished water, all expressed in consistent units. This is shown in the following equation:

$$QB(\mathrm{A}+2.27\mathrm{C})+(QR-QB)(H)+(QR)(F)$$

Where

QR = rate of flow of raw water

QB = rate of flow of bypass raw water

$QR-QB$ = rate of flow of softened water

P = % of raw water bypass =

A = bicarbonate alkalinity of raw water, mg/L as $CaCO_3$

C = carbon dioxide in raw water, mg/L as CO_2 (2.27C = CO_2 expressed as $CaCO_3$

H = hydroxide alkalinity of the first stage softened water, mg/L as $CaCO_3$

F = bicarbonate alkalinity desired in finished water, mg/L as $CaCO_3$

Solving this equation and recalling this P, the percent

Age of total raw water bypassed = $\frac{100Q_B}{Q_R}$ given the following:

$$P=\frac{100(H+F)}{H+A+2.27C}$$

The magnesium hardness of the finished water can be estimated from the following:

$$MgH=\frac{(P)(MgR)+(100-P)(MgS)}{100}$$

MgH = magnesium hardness of finished water in mg/L

MgS = magnesium hardness of the first stage softened water in mg/L

MgR = magnesium hardness of the raw water in mg/L

P = % bypass water

6.19.6 Incidental benefits of lime softening

- **Disinfection:** Excess lime provides excellent bactericidal treatment, especially at pH values above 10.5. Lime treatment, while not a substitute for chlorination, is an effective supplement.
- **Reduction of dissolved solids:** Removal of carbonate hardness by lime treatment results in reduction in the total dissolved solids content of the water. All reaction products of lime softening are relatively insoluble. The lime added to the water, as well as the carbonate hardness constituents in the water, are largely precipitated.
- **Iron and manganese removal:** Lime softening is also highly effective as a means of iron and manganese removal. The high pH achieved insured essentially complete precipitation of any iron and manganese present in the raw water.
- **Clarification:** Lime softening provides excellent coagulation and clarification as a result of the precipitation of magnesium hydroxide plus a large amount of calcium carbonate.

Example 6.5: A groundwater analysis indicated the following quality. Determine the lime dosage if straight lime softening is used.

pH = 7.5

Ca^{2+} = 150 mg/L as $CaCO_3$

Mg^{2+} = 20 mg/L as $CaCO_3$

Alkalinity = 200 mg/L as $CaCO_3$

Solution:

Lime dose = carbonic acid concentration + calcium carbonate hardness

Bicarbonate concentration:

$$\begin{aligned}[HCO_3^-] &= 200 \times \frac{60}{50} \times 10^{-3} \times \frac{1}{61} \\ &= 4.0 \times 10^{-3} \text{mol/L}\end{aligned}$$

The first dissociation constant for carbonic acid at 10°C for the dissociation into the HCO_3^- ion

$$K_1 = \frac{[H^+][HCO_3^-]}{[H_2CO_3^*]}$$

$$= 4.47 \times 10^{-7} \text{mol/L at STP}$$

The second dissociation content for the HCO_3-ion into CO_3^{2-}

$$K_2 = \frac{[H^+][CO_3^{2-}]}{[H_2CO_3^*]}$$

$$= 4.8 \times 10^{-11} \text{ mol/L at STP}$$

The total carbonic species concentration:

Where

$$\alpha 1 = \frac{1}{1 + [H^+]/K_1 + K_2/[H^+]}$$

$$= \frac{1}{1 + 10^{-7.5}/(4.47 \times 10^{-7}) + (4.8 \times 10^{-11})/10^{-7.5}} = 0.93$$

$$C_T = \frac{4 \times 10^{-3}}{0.93} = 4.29 \times 10^{-3} mol/L$$

$$[H_2CO_3^*] = C_T - [HCO_3^-] - [CO_3^{2-}]$$

$$= 4.29 \times 10^{-3} - 4.0 \times 10^{-3} = 0.29 \times 10^{-3} \text{mol/L}$$

$$= 29 \text{mg/L as } CaCO_3$$

Therefore

$$\text{Lime dose} = 29 + 150 = 179 \text{mg/L as } CaCO_3$$

$$= 179 \times \frac{37}{50} = 133 \text{mg/L as } Ca(OH)_2$$

Example 6.6: A synthetic zeolite with a capacity of 400 eq/m^3 and a 'filtration' rate of 4 L/s-m^2 is used to soften water with a flow rate of 10 L/s and a hardness of meq/L (250 mg/L as $CaCO_3$). The bed depth is 1.5 m and an 85 per cent exchange utility rate is achieved before 'breakpoint'. Determine the medium diameter and the volume of water passed through before regeneration is required. Also determine the regeneration time required.

Solution:

$$\text{Area} = \frac{Q}{V} = \frac{10}{4} = 2.5m^2$$

Vessel diameter Ø = 1.78m

Bed volume = 1.5m × 2.5m = 3.75m^3

$$\text{Volume of water passed before regeneration} = \frac{(3.75 \times 1000\text{L})\ 0.85}{2.5\ \text{m}^2}$$

$$= 1275\ \text{L/m}^2$$

$$\text{On line time between regenerations} = \frac{1275\ \text{L/m}^2}{4\ \text{L/s-m}^2} = 3195$$

$$= 5.31\ \text{min}$$

Ion exchange processes are also used for the removal of other undesirable cations including barium, strontium and radium and undesirable anions including fluoride, nitrate, humates. silicates, chromates, etc.

6.20 Adsorption

Some undesirable contaminants can be adsorbed on to solid adsorbents. Adsorption is both the physical and chemical process of accumulating a substance at an interface between the liquid and solid phases. Adsorbents used in the water treatment industry include:

- Activated carbon-Powered achivated carbon (PAC) or Granular achivated carbon (GAC)
- Activated alumina
- Clay colloids
- Hydroxides
- Adsorbent resins

Activated carbon is commonly used to adsorb organics that cause taste, odour, colour and microbiological problems. It is used to adsorb algae which can be the cause of undesirable tastes, colours and odours. PAC can be fed (in slurry foam) to the water stream either at coagulation stage or just prior to filtration. Dose rates range from 1 to 100 g/m^3. In this application, the PAC produces additional sludge and is not available for regeneration.

GAC is sometimes used as a final filter bed after sand filtration. It produces very high quality water and can reduce chlorine levels. Some industries receiving municipal water will treat it with GAC prior to use, e.g. pharmaceutical industry.

For PAC, the significant properties are bulk density and filterability, the latter meaning that they will eventually be filtered out in the sand filter backwash. Bulk density means that since mass is proportional to adsorption capacity, the higher bulk densities have higher adsorption capacities. For GAC the significant properties are hardness and particle size. The harder the particle, the less is lost by attrition. Also, the smaller the particle, the greater the availability of macropore space and also less head is required.

6.21 Chemical Oxidation

Chemical oxidation is the resultant reaction when two or more chemical species are added with the purpose of increasing the oxidation state of one. The processes of oxidation and reduction occur in the same reaction. For one chemical species to increase its oxidation state (lose electrons) the other chemical(s) in the reaction must be reduced (gain electrons). The following equation is where Fe^{2+} is oxidized by HOCl:

$$2Fe^{2+} + HOCl + 5H_2O \rightarrow 2Fe(OH)_3 + Cl^- + 5H^+ \qquad 6.24$$

The ferrous ion increased in oxidation state from Fe^{+2} to Fe^{+3}, i.e. it lost one electron. Chlorine was reduced from +1 to -1, i.e. it gained two electrons. For compatibility, two iron atoms are oxidized for each hypochloric acid atom reduced settling tanks. The oxidation process is essentially microbiological but may also be photochemical assisted. Iron and manganese can be oxidized as well as natural organics.

Chemical oxidation is a common practice within water treatment plants. Traditionally chlorine was used but in recent years attention is focused on chlorine alternatives, because of the production of trihalomethanes (THM) when chlorine reacts with natural organics. Oxidants are used for the following purposes:

- Oxidation of iron

- Oxidation of manganese
- Colour removal
- improvement of taste
- Improvement of odour
- Flocculent aid

The alternatives to chlorine as an oxidant are:

- Chloramines
- Ozone
- Potassium permanganate
- Chlorine dioxide

6.22 Membrane Processes Including Reverse Osmosis

Membrane separation techniques include:

- Microfiltration (MF)
- Ultrafiltration (UF)
- Reverse osmosis (RO)
- Electrodialysis (ED)

These are broadly sophisticated filtration techniques used to litter' out minute impurities. MU and UF separate molecules according to their size and molecular mass. Figure 6.28 is a schematic of the particle size and associated separation technique.

- **Microfiltration** allows macromolecules (10^{-4} to 10^{-3}mm) to flow through the membrane. Bacteria that are larger than 10^{-4}mm are prevented from passing through the membrane wall. The wall pores in these units are 10^{-3} to 10^{-2} mm. Generally, as in conventional filtration techniques, the retained particle size is approximately one order of magnitude smaller than the particle/pore size of the filter.
- **Normal ultrafiltration** the cut-off threshold lies in the range of 10^{-6} to 10^{-4}mm pore size.
- **Reverse osmosis** is very different to MF or UF. This is a solubilization diffusion technique that makes use of a semi-permeable membrane which acts as a barrier to dissolved salts and inorganic molecules. It also confines organics with molecular weights greater than 100. RO membranes do not have identifiable pores as in MF. RO has been used in desalination. The

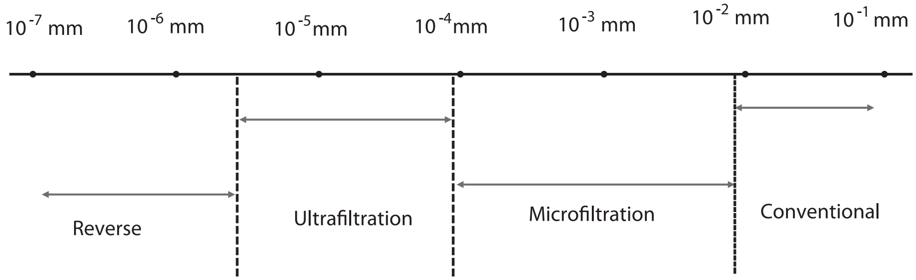


Figure 6.28: Schematic of the particle size

process is shown schematicajly in Figure 6.29. In Figure 6.29(c), the pressure that is applied exceeds the osmotic pressure of the saline solution against a semi permeable membrane, thus forcing pure water through and leaving only salts behind, i.e. all the ions are retained on the right side.

- **Electrodialysis** (ED) is an electrically charged membrane process where the ions are transferred through a membrane from a less concentrated to a more concentrated solution. The flow of pure water is tangential to the membrane while the flow of ions is perpendicular.

Membrane processes that traditionally used in desalination had limited uses. However, today, because of the vast array of impurities in waters, membrane processes are undergoing a revived interest. Some of the more recent uses

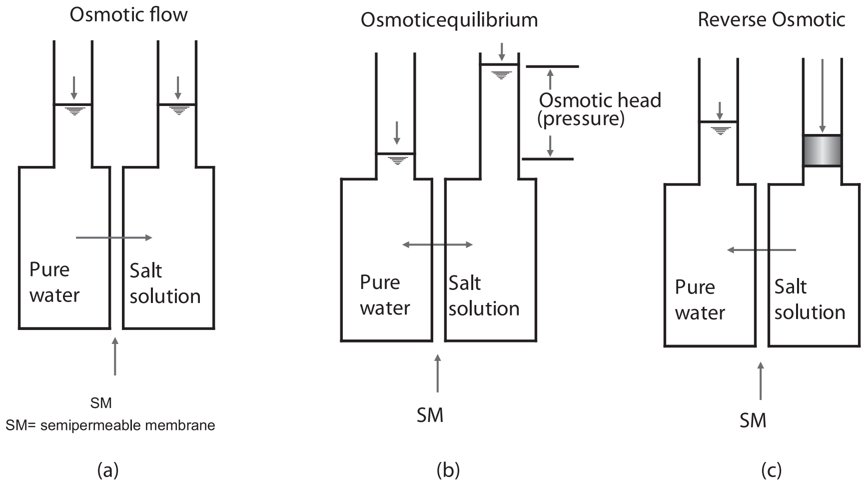


Figure 6.29: Schematic of reverse osmosis

include liquid or liquid effluents from industry where conventional treatment is deemed inadequate to satisfy standards with respect to complex organics and inorganics. Applications to solving contamination problems include heavy metals, oily waters, chlorinated hydrocarbons and also sludges.

6.23 Arsenic Removal

As (V) has been found to be more effectively removed than As (III). Because the As (III) usually occurs in non ionized form which is not easily removed by adsorption on the metal hydroxide flocs. Oxidation of As (III) to As (V) is thus required as pretreatment. This can be achieved by addition of oxidizing agents like chlorine products are potassium permanganate. As (III) removal, however, can be improved at higher pH value because of the increase n the fraction of charged As (III) species (H_3AsO_2) with increasing pH.

6.23.1 Methods of removal

Various technologies are available for removal of arsenic from ground water. The most commonly used technologies include, first coagulation and adsorption onto coagulated flocs and subsequently removed through co-precipitation adsorption onto activated alumina, activated carbon or other adsorption media while passing through the fixed bed of those media giving a low effluent arsenic concentration.

In the Alum coagulation process the dissolved aluminum sulphate reacts with natural alkalinity of water and $Al(OH)_3$ micro-flocs arc produced. During the stirring (flocculation) process all kinds of micro-particles and negatively charged arsenic ions arc removed by electrostatic attachment to the flocs. In coagulation with Iron salts freshly precipitated amorphous $Fe(OH)_3$, is formed upon addition of the coagulant. Arsenic is primarily removed by adsorption on the surface of $Fe(OH)_3$ flocs and subsequently co-precipitated. Iron coagulation seem to perform better than aluminum coagulants primarily because iron hydroxide is insoluble over a wide pH range and is less soluble than aluminum hydroxide. Moreover, iron coagulants form stronger and heavier floes. In the lime softening process precipitated $Ca(OH)_2$ acts as sorbing flocculant for arsenic. The highest removals are achieved at pH value 10.6-11.4.

The Table 6.14 shows the species of arsenic.

The following flow diagram (Figure 6.30) is presented to describe the process of arsenic treatement of ground water.

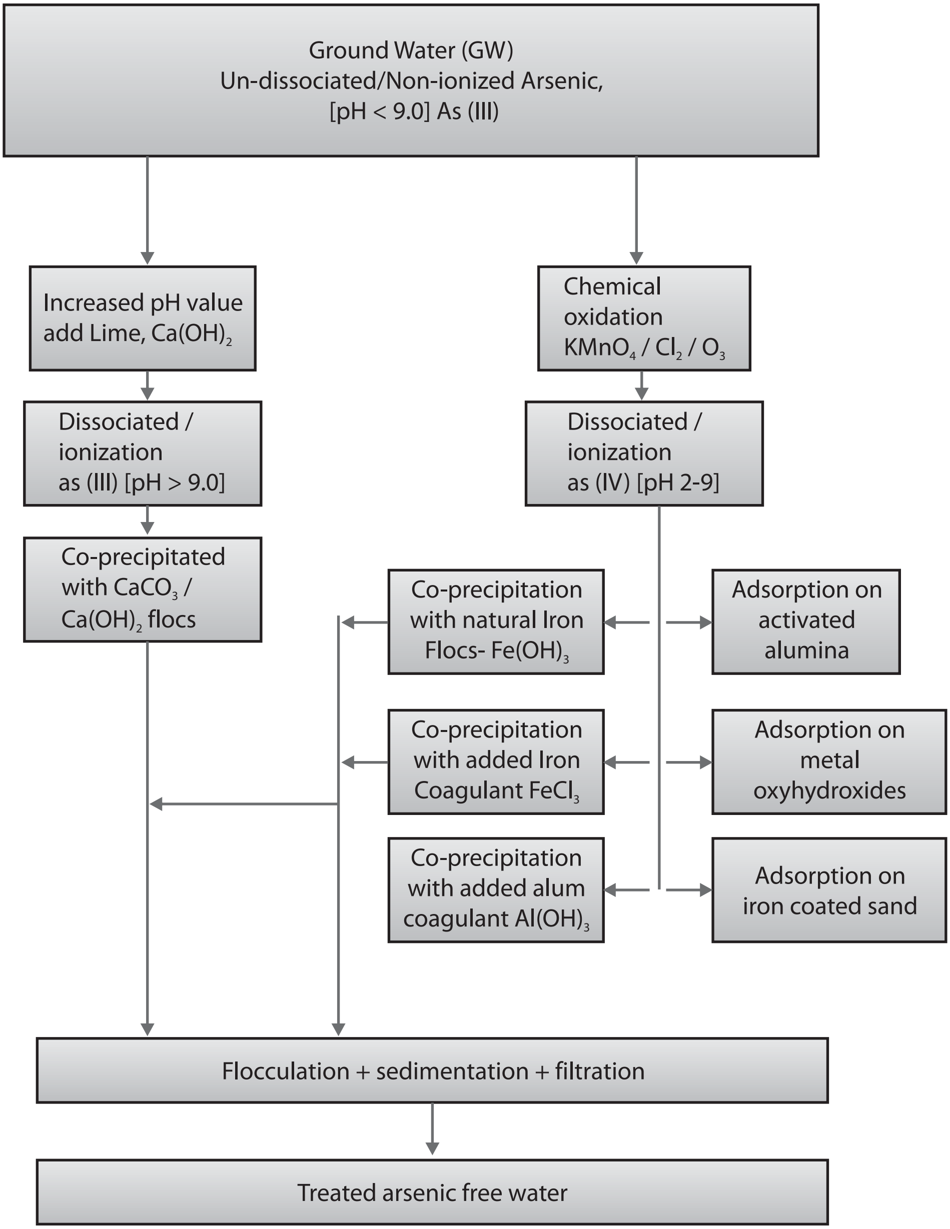


Figure 6.30: **Flow diagram of removal of arsenic from groundwater**

However, since As is a major problem in drinking water in pastoral areas of the Bangladesh, the practices of removal technologies of arsenic in details are described in the chapter 7.

Table 6.14: Species of arsenic

Type of Arsenic	Arsenic, As(III)	Arsenate As(V)
Occurrence	• Predominates in reducing conditions. • Occur as un-dissociated week acid, Arsenious acid (pH 2-9)	• Predominates in oxic conditions. • Occur as strong acid & dissociates in to different species of Arsenate ion depending on pH value
Species	• pH> 8.0 dissociate into $H_2AsO_3^-$ arsenite ion • pH> 11.0 dissociate into $HAsO_3^{2-}$ arsenite ion	• pH <7.0 dissociate into H_2AsO_4 arsenate ion • pH 7.0— 11.5 dissociate into $HAsO_4^{2-}$ arsenate ion • pH> 11.5 dissociate into AsO_4^{3-} arsenate ion

6.24 Design Consideration of Water Treatment Plant

The following considerations are important while designing a water treatment plant:

- Proximity to the source of raw water.
- Proximity to the area to be served.
- Potential for flooding of the site.
- Availability and reliability of electric power.
- Geology and topography of the site.
- Availability of transportation facilities.
- Size of the site, both for original and for anticipated expansions.
- Legal obligations or restrictions.
- Environmental effects.

6.25 Operating Considerations of a Water Treatment Plant

To simplify plant operations, the following guidelines should be observed during the design stage.

- Unnecessary equipment and operations should be eliminated.
- Operations requiring frequent attention from plant operators should be located reasonably close together. The most attention is generally required for operation of filters, flocculators, and chemical feeding equipment.
- Chemical handling and feeding should also be simplified as much as possible. Unloading and storage areas for chemicals should be easily maintained and readily accessible and be close to the point of application of chemicals.
- Plants treating river water must be arranged to provide the flexibility of treatment needed to cope with raw water quality changes.

However, safe and adequate quantity of water is essential to the health and welfare of a community. Therefore, the main objectives of water purification are to make water potable. The Table 6.15 shows the various methods of water purification processes in summary.

Table 6.15: **Selection of processes in potable water treatment system**

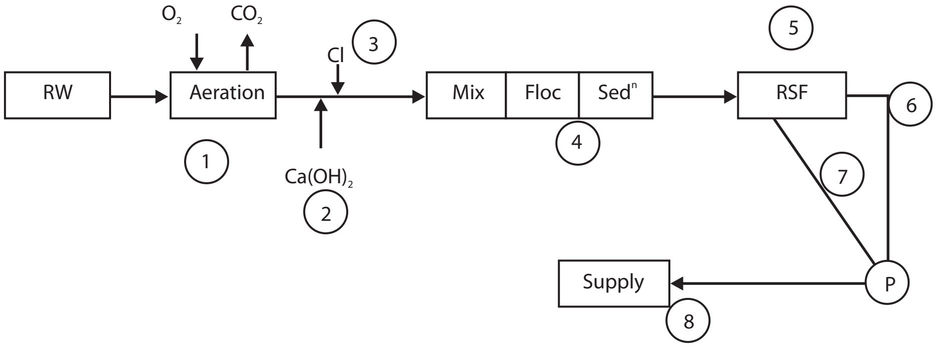
1. Plain Sedimentation, 2. Sedimentation with Coagulation, 3. Filtration, 4. Disinfection (Chlorination), 5.Softening, 6.Aeration, 7.Activated carbon application, 8.Desalination, 9.Demineralization, 10.Recarbonation, 11.Fluoridation, 12.Air stripping Special Treatment Methods: (a) Removal of Taste & Odor (b) Iron & Manganese Removal (c) Nitrate Removal (d) Radioisotopes Removal	
(A) To Make Water Safe to Drink:	
Treatment Methods	**Process**
Removal and Destruction of Pathogens	1,2,3,4 (indicated by the presence of coliform organisms)
Removal of excess Fluoride (≤1.5 mg/l)	2,4
Removal of Nitrate-N (≤10.0 mg/l)	1,2
Removal of Lead (≤0.05 mg/l)	9
(B) To Make Water Pleasant to Taste:	
Removal of Objectionable Gases	4,6,7 (H_2S, NH_3^- not detectable by consumers)
Removal and Destruction of Algae	2,3,4
Removal of Organic Matter and Industrial Waste	2,4,6,7

(C)To Make Water Suitable for Domestic Use:	
Removal of Turbidity (≤5 NTU)	1,2,3
Removal of Color (≤15 TCU)	2,3,4
Removal of Iron (≤0.3 mg/l) & Manganese (≤ 0.1 mg/l)	6,2,3
Removal of Objectionable Salt (Chloride ≤250 mg/l)	8
Removal of Objectionable Minerals (TDS ≤ 1000 mg/l)	9
Water and Impurities (Inorganic, Organic and Living)	
(I) Inorganic- $>10^{-3}$ mm size	(Solid transfer) Screening, if floating Plain sedimentation, if settleable suspended
10^{-3}-10^{-6} mm size Colloidal	Sedimentation with coagulation & flocculation Filtration
$<10^{-6}$ mm size Dissolved salts- Dissolved gases-	(Molecular transfer) Adsorption, Membrane filtration (RO) (Ion transfer) Ion exchange, Electro-dialysis (Chemical precipitation) Precipitation softening (Gas transfer) Aeration
(B) Organic	
Color, Taste and odor producing substances	Chemical oxidation (Chlorine and other oxidants) Adsorption (Activated carbon application)
(C) Living (Microbial)	
	Disinfection (Chlorination, UV radiation)

Example 6.7: Draw a flow diagram showing treatment for the raw water (W_1, W_2, W_3) having characteristics shown in the following table and also explain the steps of the flow diagram. The index show in the parenthesis of the parameter represents Bangladesh Standard (BS) value of that parameter. The following table also shows the comparison of the value of the parameters with BS so that the it can be easily identified the parameters which need to treat. '√' represents the parameter is within limit of Bangladesh Standard (BS) and 'X' represents the parameter is beyond the limit of BS which need to treat.

Parameter	W_1		W_2		W_3	
	Value	Check with BS	Check with BS		Check with BS	
pH	6.3		6.2		7.4	
Turbidity (25)	1.5	√	2	√	105	X
Color (30)	1	√	1	√	30	√
CO (1$_2$5)	75	X	80	X	12	√
Alkalinity, A	85	H<A, A is low	110	H>A	160	H<A
Hardness, H	70.83		308.33		45.83	
Fe (1)	4.5	X	0.8	√	0.1	√
Mn (0.1)	1	X	0.1	√	0.02	√
As (0.05)	0.03	√	0.28	X	0.01	√
BOD ($_5$10)	2	√	1	√	28	X

For the source W_1: The following flow diagram is provided for water source W_1. The steps are encircled in the diagram and its description is provided at below:



The following steps will be followed in the treatment of source W_1.

Step 1: First of all, aeration is done to release CO_2 from the raw water and therefore, intrusion of O_2 occurred. Here Fe^{++} and Mn^{++} oxidizes.

Step 2: As alkalinity is very low and pH= 6.3, therefore, to increase alkalinity, $Ca(OH)_2$ is added. In addition, pH also increases. Here, it is mentioned that for efficient removal of iron, pH should be raised to at least 7.5.

Step 3: Cl_2 is being used to oxidize

- Fe^{++} to Fe^{+++} $[Fe(OH)_3]$

- Mn^{++} to Mn^{++++} [Mn_2O_3]

Here, organic matter also diminishes, if any.

Step 4: Flocculation and Sedimentation to settle down the particles.

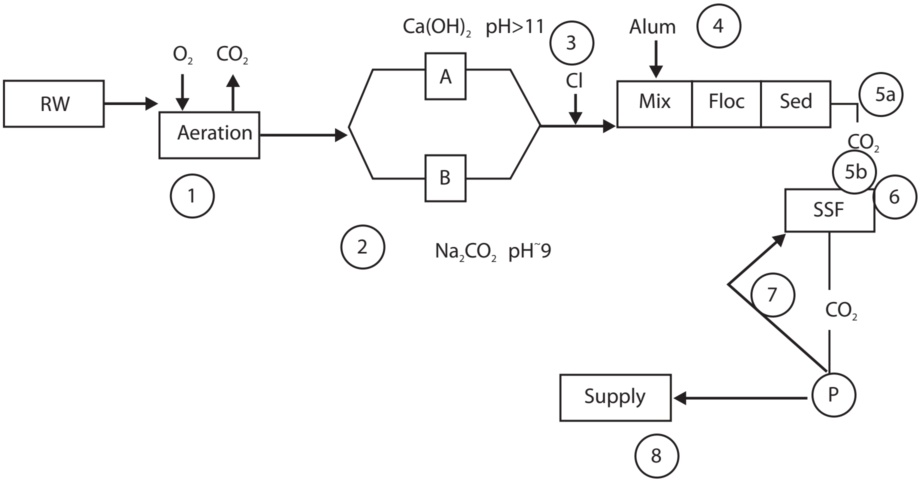
Step 5: Rapid Sand Filter (RSF) is used as turbidity increase by $Fe(OH)_3$.

Step 6: Post chlorination is done to remove iron bacteria.

Step 7: Backwashing of RSF

Step 8: Supply of treated water.

For the source W_2: The following flow diagram is provided for water source W_2. The steps are encircled in the diagram and its description is provided at below:

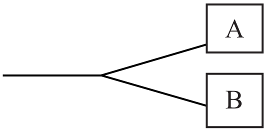


The following steps will be followed in the treatment of source W_2:

Step 1: First of all, aeration is done to release CO_2 from the raw water and therefore, intrusion of O_2 occurred.

Step 2: Hardness > Alkalinity; in this raw water source W_2. Therefore, non carbonate hardness present in this sample. It is preferred to use splitting technology to treat this raw water.

The main is divided into two



A= Lime is added to remove Mg hardness, when pH > 11.

B= Soda ash is added to remove Ca hardness, when pH ≈9.

Step 3: Cl_2 is added to oxidize As^{+3} to As^{+5} to co-precipitate.

Step 4: Because Fe and Mn are very low, so Alum is used as coagulant agent by which As^{+5} is co-precipitated through subsequent Mixing followed by Flocculation and sedimentation.

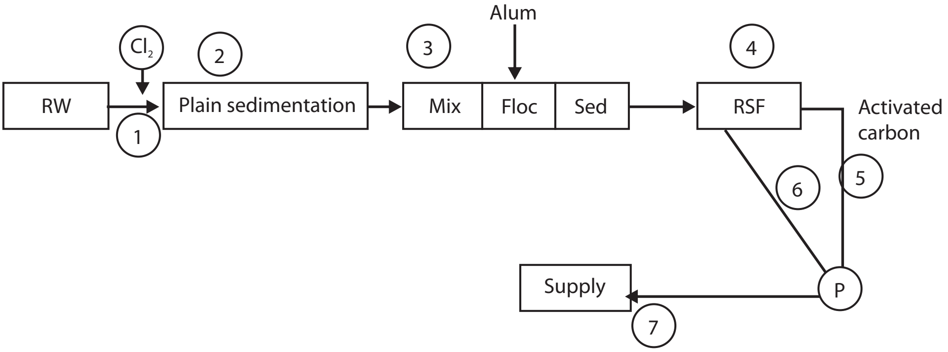
Step 5: Recarbonation is done to prevent clogging of bed $CaCO_3$.

Step 6: SSF is being used as turbidity is low and no possibility to increase turbidity by Fe.

Step 7: Back washing of SSF.

Step 8: Supply of treated water.

For the source W_3: The following flow diagram is provided for water source W_3. The steps are encircled in the diagram and its description is provided at below:



Step 1: As turbidity is very high, therefore there might be possibility of presence of organic matter in this raw water. Therefore, pre chlorination is done to oxidize the organic matter and hence reduction of the organic material occurred.

Step 2: Plain sedimentation is done to remove the turbid particles from the raw water.

Step 3: As color is 30 TCU i.e. at boundary limit, so adding Alum facilitates the coagulation of colloids and other suspended particles which impart color. Subsequently, flocculation and sedimentation were followed to settle.

Step 4: RSF is being used as turbidity is very high.

Step 5: Activated carbon is used to remove total organic matter.

Step 6: Back washing of RSF

Step 7: Supply of treated water.

Box 6.1

Important Notes on Water Treatment

Alcalinity: Presence of anion (-ve)

- HCO_3^- is present when pH<8.3
- $CO_3^=$ is present when pH 8.3~9.4
- OH^- is present when pH>10.6

Hardness: Presence of cation (+ve)

- Ca^{++} is commonly present
- Mg^{++} is commonly present
- Fe^{++}
- Sr^{++}

SSF:

- Filter bed is consisted of fine sand
- Suitable for low turbid water

RSF:

- Filter bed is consisted of coarser sand
- Suitable for high turbid water

While removing Fe in the treatment plant, RSF need to use (even for low turbidity), because turbidity increases due to precipitation of $Fe(OH)_3$.

Removal of Hardness: In case of removal of hardness, the common precipitated form is $CaCO_3/Mg(OH)_2$. Here, checking pH, hardness and alkalinity is imperative first to identify the salt responsible for hardness.

- **Option 1 (individually Ca/Mg hardness removal):** When hardness ≤alkalinity and pH<8.3, HCO_3^- salt of Ca^{++} & Mg^{++} is present in raw water. Therefore, $Ca(OH)_2$ needs to add to precipitate as $CaCO_3/Mg(OH)_2$.

$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow CaCO_3\downarrow + H_2O$ here, pH is about 9.4

But in order to remove Mg hardness, it is required to precipitate $Mg(OH)_2$ and therefore more lime is needed as pH>10.6 is required

$Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow Mg(OH)_2\downarrow + CaCO_3\downarrow + H_2O$ here, pH is about more than 10.6

- **Option 2 (individually Ca/Mg hardness removal):** When hardness > alkalinity, non carbonate source of hardness ($CaCl_2$, $CaSO_4$) is present. Therefore, carbonate source is needed to add to precipitate as $CaCO_3/Mg(OH)_2$.

$CaSO_4 + Na_2CO_3 \rightarrow CaCO_3 \downarrow + Na_2SO_4$

$CaCl_2 + Na_2CO_3 \rightarrow CaCO_3 \downarrow + Na_2Cl_2$ here, pH is about 9.4

$CaNO_3 + Na_2CO_3 \rightarrow CaCO_3 \downarrow + Na_2NO_3$

But to precipitate $Mg(OH)_2$, more NaOH is needed as pH>10.6 is required

$MgCl_2 + NaOH \rightarrow Mg(OH)_2 \downarrow + NaCl$ here, pH is about more than 10.6

- **Option 3 (Only Mg hardness removal):** When hardness ≤ alkalinity and pH 8.3~9.4 , $CO_3^{=}$ salt of Mg^{++} ($MgCO_3$)is present in raw water. Therefore, NaOH can be added to precipitate as $Mg(OH)_2$.

$MgCO_3 + NaOH \rightarrow Mg(OH)_2 \downarrow + Na_2CO_3$

Here, it is mentioned that, in order to remove Mg hardness only, it is better to use caustic soda (NaOH) instead of $Ca(OH)_2$ as using it will increase hardness further and recarbonation will be required.

- **Option 4 (Both Ca/Mg non carbonate hardness removal):** When hardness > alkalinity, and both Ca, Mg hardness (Ca/$MgCl_2$, Ca/$MgSO_4$) is present, non carbonate salt is needed as source of hardness. In order to remove Mg hardness, it is required to precipitate $Mg(OH)_2$ and therefore pH>10.6 is required. But on the other hand, in order to remove Ca hardness, it is required to precipitate $CaCO_3$ and therefore pH 8.3 ~9.4 is required. Therefore, once Mg hardness is removed by using high dose of $Ca(OH)_2$, carbonation is required to reduce pH at 8.3 ~9.4 for removal of Ca hardness by adding Na_2CO_3.

  Here, it is mentioned that, using CO_2 to reduce pH will form scaling in pipe and there is a possibility of clogging the filter media due to scale formation. And to break this scaling effect, two stage recarbonation is needed before and after filter media.

- **Option 5 (Both Ca/Mg bicarbonate hardness removal):** When alkalinity ≥ hardness, and both Ca, Mg bicarbonate hardness (Ca/Mg $(HCO_3)_2$, Ca/Mg$(HCO_3)_2$) is present. In order to remove Mg hardness, it is required to precipitate $Mg(OH)_2$ and therefore pH>10.6 is required. But on the other hand, in order to remove Ca hardness, it is required to precipitate $CaCO_3$ and therefore pH 8.3 ~9.4 is required. Therefore, once Mg hardness is removed by using high dose of $Ca(OH)_2$, carbonation is required to reduce pH at 8.3 ~9.4 for removal of Ca hardness by adding Na_2CO_3.

  Here, it is mentioned that, using CO_2 to reduce pH will form scaling in pipe and there is a possibility of clogging the filter media due to scale formation. And to break this scaling effect, two stage recarbonation is needed before and after filter media.

Aeration is needed when high concentration of CO_2 is present in raw water. Therefore, O_2 lets in removing CO_2 for which soluble Fe^{++} can be oxidized to insoluble Fe^{+++}.

Pre-chlorination is needed while high concentration of organic matter or high color or high concentration of BOD is present in raw water.

Fe removal

Iron is soluble in the range of $pH<7.5$. Therefore, in order to remove iron the pH should be 7.5 or higher. Adding lime will increase pH and also its alkalinity. However, if $pH \approx 7.5$ but alkalinity is low, then oxidizing agents (Cl_2, $KMnO_4$) are required to add. Here it is mentioned that, post chlorination is must after filtration bed due to iron bacteria.

Mn removal:

Mn (II) is soluble but Mn (IV) is insoluble. Therefore, oxidizing agent (e,g, Cl_2) is needed to use to make insoluble Mn (IV). Here, pH should be around or above 9.0.

Arsenic removal:

Arsenic is co-precipitated with iron/alum as both As (III) and As (V) are soluble. Using oxidizing agent (Cl_2 or $KMnO_4$) will oxidize As (III) to As (V) and this As+5 will co-precipitate with $FeCl_3$ or Alum (need to add).

Pre-sedimentation is done when turbidity is high. In this case, pr-chlorination is also done to prevent growth of algae.

6.26 Conclusion

Only a few of the most important mechanisms used in water contramination control have been discussed in this chapter. Hundreds of analytical procedures have been documented, many of which can be performed only with special equipment and skilled technicians, which may also vary with local condition. The next chapter will discuss specifically the arsenic treatment methodologies in Bangladesh.

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