Peter King

Has a BSc (Chem) and GDE (Civil) and has retired with more than 40 years' experience in water treatment, wastewater treatment and groundwater recharge with treated effluent.

He has lectured at the N Level, T level and has been external Examiner to 4th year Civil Engineering students

He is a retired Senior Fellow of the Water Institute of Southern Africa and Fellow of the Chartered Institution of Water and Environment and is a Chartered Water and Environmental Manager.

He was Editor of the Newsletter of the former Association of Water Treatment Personnel from 1985 to 1999,

He remains dedicated to the professionalization, education and upgrading of Process Controllers in both the water and wastewater field.

THE PROCESS CONTROLLER's GUIDE TO

WATER SOURCES and WATER TREATMENT

This is number 2 in the Process Controller Guide series of documents

Number 1	Pollution Control.
Number 2	**Water Sources and Water Treatment.**

These notes were prepared for delivery to persons studying for the Diploma in Water Care Technology offered by the then Cape Technicon, in the early 1980's.

Since the time of writing, new Acts of Parliament, Guidelines and Regulations have been implemented. These have NOT been incorporated into this document. Person interested in the latest Acts, Guidelines and Regulations need to look elsewhere for these.

It is intended that this document be a useful reference and training manual guide to all persons involved in the Water and Wastewater Industry.

When originally prepared, the series of documents consisted of:

1. Pollution Control
2. Watercare Technology 1 (both Water and Wastewater)
3. Watercare Technology 2 (both Water and Wastewater).

The series has been arranged so that Pollution Control remains as volume 1, while the Water documents have been combined into one document, as Volume 2. Similarly, the Wastewater documents have been combined into one document as Volume 3.

Where photographs and images are taken from published sources, credit is not explicitly made here. However, the original producers of such material are thanked for their information that will contribute to this document being of use to all in the water and wastewater industry.

These documents are dedicated to the thousands of men and women (both present and past) who are involved in the life critical profession of Water and Wastewater Treatment.

THE PROCESS CONTROLLER's GUIDE TO WATER SOURCES AND WATER TREATMENT

CONTENTS

PART 1

WATER SOURCES

PART 2

WATER QUALITY STANDARDS

PART 3

PRE-TREATMENT

PART 4

COAGULATION AND FLOCCULATION

PART 5

SEDIMENTATION

PART 6

FILTRATION

PART 7

CHLORINATION AND DISINFECTION PROCESSES

PART 8

ADVANCED WATER TREATMENT

PART 9

IRON AND MANGANESE REMOVAL.

PART 10

DEMINERALIZATION OF WATER

PART 11

SPECIAL TREATMENT PROCESSES

PART 12

CONTROL OF WATER TREATMENT WORKS

PART 13

THE CONTROL OF DISTRIBUTION SYSTEMS

PART 14

THE TREATMENT AND DISPOSAL POF WATERWORKS SLUDGES

WATER SOURCES and WATER TREATMENT.

PART 1.

WATER SOURCES.

1. INTRODUCTION.

In Volume 1 "Pollution Control", the reasons for the need to treat water before consumption were given. In this Volume, the methods and processes employed will be discussed. Firstly the simpler and more common processes will be covered, and later the more sophisticated processes will be covered. The treatment of sludges will also be covered.

There are 3 main sources of water for domestic and industrial purposes. These are surface waters, groundwater and the direct collection of rainwater.

1.1 SURFACE WATERS.

There are three main sources from which surface water may be drawn. These are:

1. Rivers, streams and lakes from which water is drawn continuously at the required rate. This is possible only if the stream flow or lake capacity is high enough at all seasons of the year to furnish the required water volume;
2. When the total annual flow in a river is sufficient, but the minimum flow of the river is less than the minimum water requirement, it may be possible to construct a wall across the river in order to store the excess water from the high river flow period for use when the river flow is low. One may not be able to totally stop the flow in the river for storage purposes as persons or communities living below the site of the dam may have a legal right to a certain quantity of water;
3. When it is not possible to build a wall across a river to form a dam either because the valley is too flat or when the river is used as a mode as transport, then impoundments may be constructed on the edge of the river.

The operating procedure in this case would be to draw water from the river, during periods of high flow, in quantities in excess of the domestic requirements and store the excess in the specially constructed areas. When the river flow dropped below the minimum rate, one would draw the additional water from the impoundments.

1.1.1 Characteristics of Surface Waters.

Most surface waters tend to have the following characteristics.

1. a significant suspended solids content. However this can vary from less than 5 mg/L when the water is stored in a reservoir, to more than 2 000 mg/L when a river is in flood;
2. a fairly high dissolved oxygen content. During certain times of the year, the water in the lower levels of a reservoir could have diminished oxygen content. Water in a river could have a fairly low dissolved oxygen content, if an oxygen sag was present as a result of the discharge of some waste upstream;
3. a fairly low dissolved free carbon dioxide content, as the pH of the surface water is usually above 7 and the carbon dioxide is present as the bicarbonate ion;
4. where the water has runoff an area containing certain types of vegetation the pH may be low as a result of the dissolution of humic acid or as a result of pollution;
5. the colour of the water is often significant, particularly if humic acids are present;

6. the dissolved solids content is usually low. The dissolved solids content of river water increases in passing from the source to the mouth of the river due to the discharge of wastewater and as a result of surface runoff;

7. the iron and manganese content is usually low. However during storage in a reservoir, sudden increases in iron and manganese content can occur;

8. the calcium and magnesium contents are usually low i.e. the water is soft;

9. the quality of the surface waters tend to change fairly rapidly with time, both in the short term between rainy and dry periods and in the long term when development takes place in the area above which the water is abstracted. The development of a forest area into a residential, industrial or agricultural area will influence the quality and quantity of the surface water runoff and this will in turn affect the quality and quantity of water in the river;

10. Depending on the human activity in the catchment area the coliform count may be significant.

1.1.2 The Effects of Storage of Surface Water.

The storage of water in impounding reservoirs creates favourable conditions for self-purification of the stored water and also certain conditions for unfavourable changes in water quality.

The benefits of storage are due to the effects of sedimentation, the influence of environmental conditions unfavourable to bacteria associated with prior domestic wastewater pollution of the water, and the chemical action of the oxygen dissolved in the water. Unfavourable conditions created by storage include those associated with growths of micro-organisms, as well as the reduction and dissolution of iron and manganese from flooded soils and rocks containing these minerals.

It will be noted that both oxidation and reduction phenomena are involved. This apparent paradox is due to the stratification of the stored waters when surface waters are warmer and less dense that the cooler, deeper waters. When the surface waters are cooled to a temperature of 4°C (the temperature of maximum density), they sink to the bottom and the replaced warmer waters are in turn cooled to this temperature and sink. This continues until the whole volume of stored water has a temperature of 4°C. As water of lower temperature is less dense, it will remain near the surface; when ice forms it will float on the surface, because the density of the ice is less than that of water.

After the spring overturn, the waters near the surface having a temperature of 4°C will gradually be warmed by the air and sunshine. This will continue until a surface layer 5 to 10m deep has become warmed, while the lower waters remain cool. There is then a marked difference between the two layers, characterized by rapid fall in temperature at the zone of separation. The pronounced temperature gradient in this narrow zone is known as the thermocline, where temperature changes can be up to 1°C or more per metre of increased depth. The depth of the level where the thermocline is most pronounced depends on climatic conditions, the depth and size of the reservoir, and the opportunity for surface agitation and current formation due to wind action.

When water is stored in reservoirs located where water temperatures never reach as low as 4°C, the surface waters become heated by the air and sunlight. These warmer waters become mixed with lower, cooler water through currents induced by wind action, but the zone below the influence of these currents comprises permanently stratified cooler waters. Under these circumstances the thermocline persists throughout the year, but its depth below the surface varies with the season and meteorological conditions. The waters in deep reservoirs consist of three layers or zones:

1. the top or circulation zone;

2. the middle or transition zone;

3. the bottom or stagnant zone.

See figure 1 below:

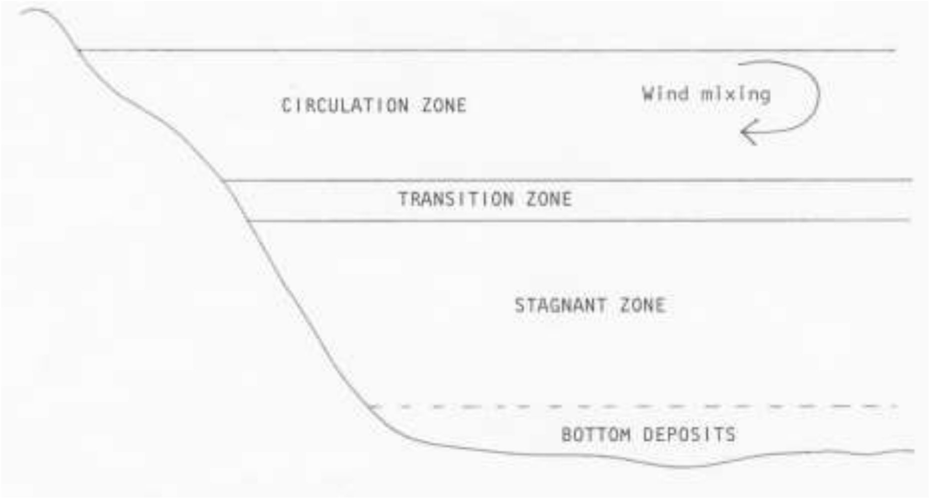


Figure 1 – ZONES IN A STRATIFIED WATER BODY.

Since the warmer water near the surface in the top zone is subject to re-aeration and mixing by wind action, it contains dissolved oxygen. Any dissolved iron or manganese that may reach the surface waters with convection currents will therefore be oxidized in time, and the resulting insoluble ferric and manganic compounds will precipitate and settle back to the lower portion of the stored water, where the minerals dissolve. Generally speaking, the surface waters do not contain significant concentrations of iron and manganese except during the periods when the spring and autumn overturns occur.

The deeper water below the thermocline in the stagnation zone will contain little or no dissolved oxygen, because of its removal by the processes of oxidation of organic matter. This in turn leads to the production of carbon dioxide and to the resulting lowering of the pH of the water; this is favourable to the solution of iron and manganese from the flooded soil or rock (and also to increase solubility of phosphates). The absence of dissolved oxygen leads to the reduction of sulphates to odour-producing sulphides such as hydrogen sulphide and ferrous sulphide, and also to the decomposition of organic matter, with the production of colour and substances having tastes and odours. The quiescent, stagnant waters below the thermocline are characterized by the absence of dissolved oxygen, the higher dissolved carbon dioxide content, the lower pH value, the presence of soluble iron, manganese and sulphide and the presence of taste and odour producing substances.

The surface waters above the thermocline are favourable for the growth of algae or microscopic plants that need sunlight for the photosynthesis process. These liberate oxygen and utilize carbon dioxide, thus raising the pH value and creating conditions still more favourable to the oxidation of iron and manganese. Conversely, the floating organisms of the animal kingdom, having no photosynthetic activity, are independent of sunlight and hence migrate to the level where their food is most abundant. Algae serve as food for the animal kingdom in the upper waters. On or near the bottom of reservoirs, decaying dead algae and other organic matter are available as food.

The phenomena mentioned above are more pronounced in new reservoirs, where flooded vegetation and the organic content of topsoil are subject to active decomposition. The area to be flooded by a new reservoir should therefore be cleared of vegetation, which should be burned or preferably hauled off the area. Usually the best practice is to remove the peat like deposits of any swampy area to be flooded, but such soil stripping ordinarily is not practised over the whole reservoir site because of the costs involved. New reservoirs gradually become stabilized in three to five years.

Storage provides conditions favourable to the sedimentation of particles responsible for turbidity. Coarse suspended solids settle rapidly and finer material progressively more slowly. The remaining very fine material is in the colloidal state and will not settle unless conditions favour natural coagulation, or until the water is coagulated at a treatment works. Reservoirs will also equalize highly turbid floodwaters through mixing with previously settled water in the reservoir. Turbidity of less than 30 N.T.U. (Nephelometric Turbidity Units) is usually produced by sedimentation in reservoirs.

The effect of storage on the colour of waters is more complex, because colloidal colouring matter does not settle unless conditions are favourable for its natural coagulation. Colouring matter in the upper portion of reservoirs is subject to the bleaching action of sunlight. Overall the reduction in colour is usually much less than the reduction in turbidity.

Another favourable influence of storage is the effect on bacterial content. This is due mainly to the dilution of more highly polluted floodwaters by the previously settled waters in reservoirs and to the influence of sedimentation. The effects are those associated with:

1. sedimentation;
2. chemical changes;
3. environmental conditions unfavourable for bacteria associated with pollution, such as water temperature and absence of food;
4. the germicidal action of sunlight in the upper 3m of waters that are of low turbidity;
5. the consumption of bacteria by predatory organisms such as the free swimming microscopic protozoa.

All this requires time, so the period of storage is the controlling factor in these self-purification processes. It is for these reasons that many large lakes yield clear waters with a low degree of bacteriological pollution.

1.1.3 Abstraction from Dams.

Because of the phenomena outlined above, reservoirs should be equipped with multiple level draw offs, to permit water to be withdrawn from the most favourable level. A problem arises here as to which level to use, because although the upper waters are usually free of iron, manganese, sulphides and the colour, tastes and odours produced locally by decomposition and reduction processes, they have the maximum concentration of algae.

The choice may be algae control to permit the use of surface waters or iron and manganese removal processes to permit the use of the lower waters.

1.1.4 Seasonal Variations in Quality.

Perhaps the most significant effect on the seasonal variation in the quality of the water in a river is that of rainfall with the resulting runoff. Heavy rains in the catchment area can increase the suspended solids content of the water by a factor of 100 or more. Usually the dissolved solids content will decrease when the flow rate increases.

The runoff from seasonal rains falling on agricultural areas can result in high nitrate and phosphate concentrations in the river. The first rains after a dry spell can result in the stormwater from industrial and residential areas carrying a large pollution load into a river.

The seasonal variation in the quality of water in dams and reservoirs is not so dependent on the rainfall or the associated runoff. The interplay of temperature, density and wind during the different seasons of the year produces a sequence of characteristic patterns of thermal stratification in lakes and reservoirs.

Figure 2 (A) shows a series of temperature gradients of water in an area where ice can occur in winter.

During the winter the water just below the ice will have a temperature of nearly 0°C, even though the ice may be much colder. At the same time the temperature at the bottom of the lake or reservoir will be nearly 4°C i.e. the temperature of maximum density. The water is in comparatively stable equilibrium and is inversely stratified in terms of temperature but directly in terms of density. This is the condition of winter stagnation. Ice cover shuts out wind disturbances of the underlying waters, and vertical and horizontal movements are suppressed.

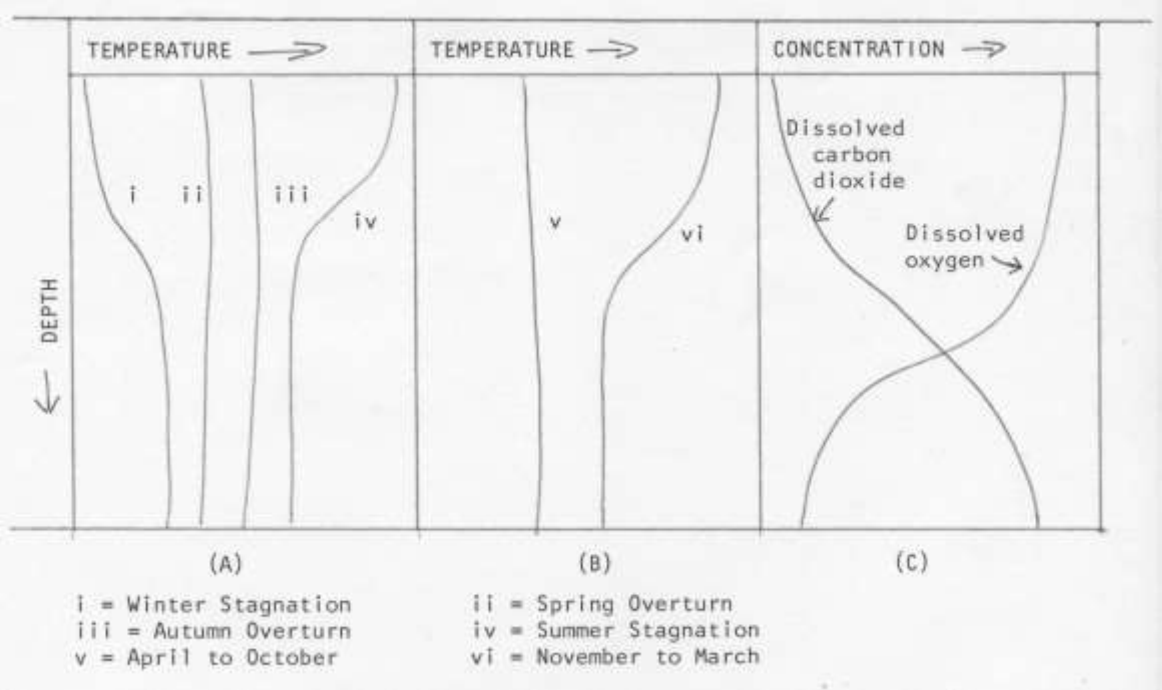


Figure 2: VARIOUS TEMPERATURE PROFILES IN A WATER BODY.

When the ice breaks up in the spring, the waters near the surface begin to warm up. Until the temperature of maximum density is reached, they also become denser and tend to sink. Equilibrium is upset by diurnal fluctuations in temperature, and vertical circulation is aided by wind. When the water temperature is practically uniform at all depths and close to the temperature of maximum density, circulation becomes especially pronounced. This is the spring circulation or spring overturning. It may last several weeks, and it varies in length during different years.

As spring turns into summer, the surface water becomes progressively warmer. Soon lighter water overlies denser water once again and, as temperature differences increase, circulation is confined more and more to the upper waters. A second period of stable equilibrium is established, and the water becomes directly stratified in terms of both temperature and density. This is the condition of summer stagnation, which extends from October to May in the southern hemisphere. The water below 8m is nearly stagnant, and the bottom temperature remains almost constant and not far from 4°C.

When autumn comes, surface layers cool and sink. Once again equilibrium is upset. The water is stirred to greater and greater depths, and the temperature gradient eventually becomes substantially vertical. The major or autumn overturning then takes place, because the waters are easily put into circulation by autumn winds. When the surface water freezes, the condition of winter stagnation is re-established.

When the temperature never falls below 4°C at any depth, the lake or reservoir will have only one circulation each year and will directly stratify during the summer. For example, stratification may occur between November and March and may circulate continuously between April and October. This is shown diagrammatically in figure 2 (B).

The dissolved oxygen and dissolved carbon dioxide gradients during the stratification stage are shown in figure 2 (C).

1.1.5 Protection of Surface Water Supplies.

Water treatment works owners usually have little or no control over the degree of pollution of sources of water supply, except where catchment areas are sufficiently small to permit their ownership and their exclusive use for water-supply purposes or when wells and springs are utilized, and even then seepage from sources of chemical pollution may flow from areas beyond the well sites. Generally speaking it is necessary to rely on official agencies for control of water pollution. This is of prime importance in conserving water resources for economical and effective treatment.

The current trend is to place the duty of protecting water resources under a government department concerned with all uses of the resources, including that of water supply. This permits the establishment of policies of relating to:

1. existing sources of pollution;

2. the degree of treatment of wastewaters needed to restore polluted waters to a reasonable degree of purity or to prevent significant pollution;

3. the existing or anticipated legitimate uses of waters for various purposes.

The ownership of catchment areas is feasible only when their area is restricted and their value permits purchase. Such ownership makes possible complete control over the areas and the elimination of sources of pollution. Soil erosion may be prevented by re-afforestation practice and swamps may be drained.

Many catchment areas are too large to warrant their ownership by a water authority yet small enough to permit convenient inspection. In these circumstances, organized inspection should be undertaken with the co-operation of the officials having jurisdiction. The endeavour should be to locate and record all occupied properties and the related actual and potential sources of pollution, especially the more serious sources such as any sewer discharges into tributary.

The results of sanitary surveys will help to establish the significance of any given degree of pollution of the raw waters by coliform organisms, widely used as indicators of domestic wastewater pollution, but also present in animal wastes and soil drainage.

Routine inspections of private properties on catchment areas require the co-operation of the property owners, unless the officials are authorized by law to make such inspections and to require corrective action to be taken by the property owners to abate sources of pollution. A friendly approach will usually establish an awareness in property owners of their duty to maintain sanitary conditions on their property, for the protection of their families and neighbours and also of the public served by the water supply involved. Of major importance is the elimination of direct domestic wastewater discharges, overflowing septic tanks and soakaways. The operators of large treatment works should develop routine inspection programmes of the catchment area and examination of such samples collected at significant points. Bacteriological examination of such samples will disclose the degree of pollution as well as the most likely source. Such inspections must include industrial establishments.

It was seen in Volume 1 "Pollution Control", that the introduction of nutrients into water that is stored in an impoundment - be it a natural lake or a man-made reservoir will increase the quantity of organic matter present in the water. This can make the effects of the overturn more drastic as large quantities of nutrients can suddenly be made available causing an algal bloom that can introduce tastes and odours into the water and affect the length of the runs on the sand filter. In certain "sensitive" areas, the Department of Water Affairs has introduced a strict limit of 1 mg/L of ortho-phosphate for treated effluents to try to reduce the rate of eutrophication. There is no limit for nitrate since when stratification occurs, denitrification can take in the lower levels of the reservoir and the nitrogen gas will either dissolve in the water or will pass to the surface of the water and be lost to the atmosphere.

1.2 GROUNDWATER SOURCES.

There are 3 main sources from which groundwater may be taken. These are:

1. Springs - these occur when the ground surface drops sharply below the normal groundwater table or when a geological obstruction impounds water behind it and forces it to the surface or when a fault in an impervious stratum allows artesian water to escape to the surface. These effects are seen in figure 3;

2. Wells - these are holes dug into the ground in order to provide access to the groundwater. They are usually of a relatively large diameter i.e. greater than 1 metre. See figure 4;

3. Boreholes - these are usually drilled into the ground using a rotary or percussion drill and have a sieve at the bottom to prevent sand etc. from entering the shaft. Borehole

diameters are usually in the range of 100 - 300 mm. Boreholes differ from wells in that there is no storage volume within the excavated portion. See figure 5.

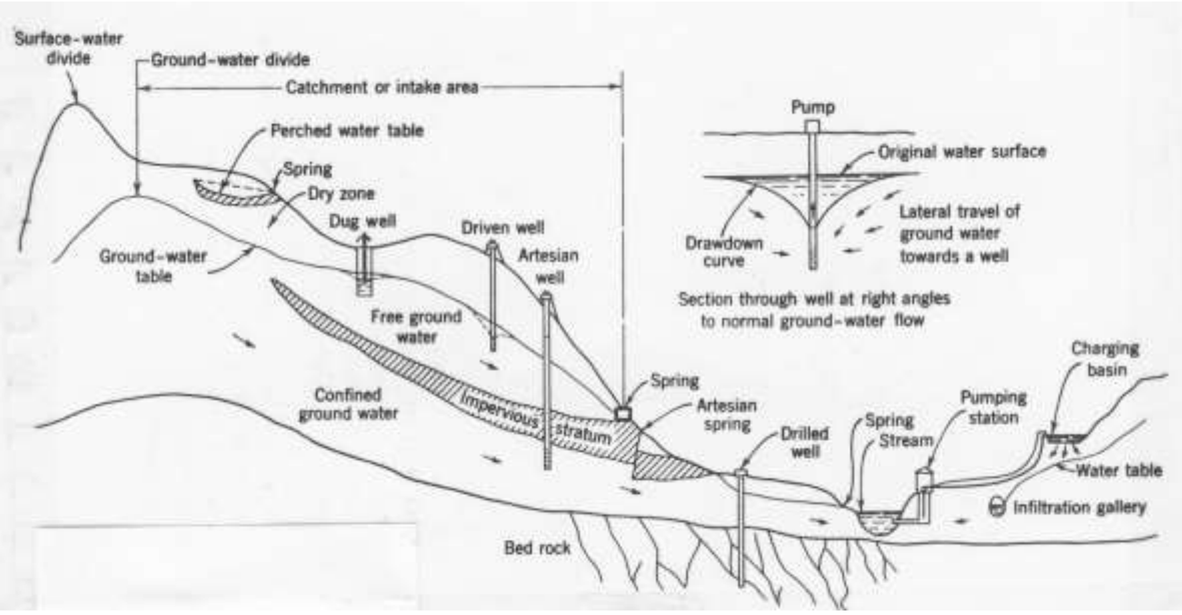
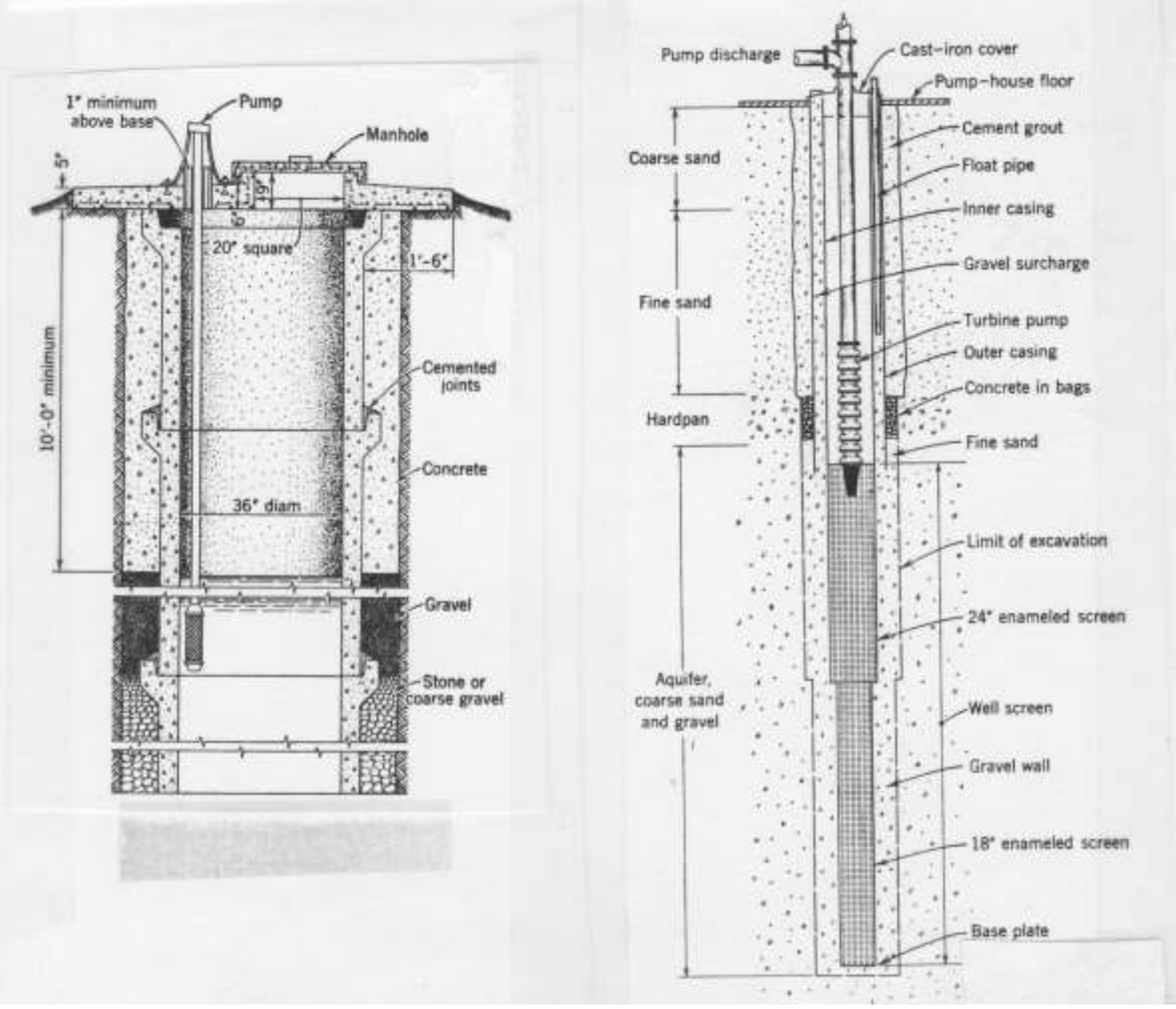


Figure 3 – GROUNDWATER FEATURES



Figures 4 & 5 - TYPICAL LAYOUT OF A WELL AND A BOREHOLE

1.2.1 Characteristics of Groundwater

Most groundwater tends to have the following characteristics:

1. A low suspended solids content. However during the development of the water source, the finer particles will pass out with the water. The suspended solids content could increase for a short time after abstracting from a source that had been out of use for some time or when suddenly increasing the rate of abstraction;

2. A fairly low dissolved oxygen content. When the surface water moves down through the earth to the water table, the biological activity in the soil will deplete the dissolved oxygen content. As there is little or no free oxygen available in the lower levels of the earth, the oxygen deficit cannot be made up. The actual oxygen deficit will depend on many factors amongst them, the quality of the surface water, the biological activity in the soil through which the surface water passes, the presence of any chemical reactions etc.;

3. A fairly high dissolved carbon dioxide content. In the biological processes mentioned above, the dissolved oxygen is used up by the bacteria and carbon dioxide is produced as one of the end products;

4. The pH is generally in the range pH 5 - 7, due to the presence of dissolved carbon dioxide. If the pH is less than about pH 4.5, one should suspect contamination of the water;

5. The water generally has no colour. However water containing iron will stain concrete etc. due to the deposition of ferrous and ferric oxides;

6. The dissolved solids content is generally higher than that of surface water as on passing down from the surface the water will dissolve certain minerals. In some areas, such as dried up prehistoric lakes, and seas, the dissolved solids content may be very high. The organic solids content of the water is generally low;

7. The iron and manganese content can be fairly high. In most underground water sources, the low pH value and the presence of a reducing medium i.e. nil dissolved oxygen content, iron and manganese can be leached out of rocks;

8. The water will vary in its degree of hardness i.e. its calcium and magnesium content. Water with a high dissolved carbon dioxide content and a low pH value will dissolve calcium and magnesium from carbonate rocks containing these minerals. These minerals will be present as the bicarbonate. Water found in sandstone areas will generally have a low calcium and magnesium content but will have a relatively high sodium content;

9. The quality of the groundwater in a particular area tends to remain fairly constant. Quality changes can occur when the water table is lowered so far that water is pulled in from another area. Once the groundwater is polluted it will usually take many years to work out the pollution;

10. The bacterial population of the water is generally low unless there is a nearby source of surface or sub-surface water pollution or if the abstraction point is in a fractured rock stratum where the filtering action of the rock layer may be faulty.

1.2.2 Pollution of Groundwater by Surface Water.

There are 2 main considerations here:

1. Pollution on the macro-scale where a large area is affected, or when large quantities of water are involved and;

2. Pollution on a micro-scale where the pollution occurs via the point of abstraction of the groundwater.

On the macro-scale, the pollution of groundwater by surface waters may be caused by leachate from landfills, tip sites etc., the downward movement of water from maturation ponds, oxidation ponds or sludge disposal areas at wastewater treatment works and from irrigated farmlands.

Among the pollutants that may be found in groundwater polluted by surface water are: nitrates, pesticides, radio-active materials, and carbon-chloroform extractable materials.

On the micro-scale, the major concern is the prevention of water passing down the borehole or the well into the groundwater. Various techniques for preventing this movement of water are shown in figures 6 and 7. Pollution of a lower stratum of water can take place when water passes down the side of the well or borehole. The poor water stratum should be sealed off as is seen in figure 6.

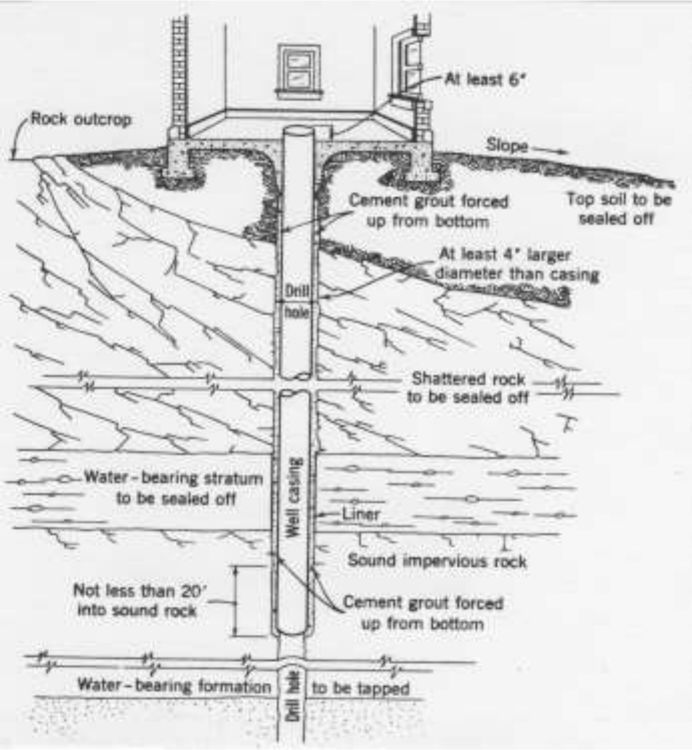


Figure 6 - PREVENTION OF POLLUTION OF GROUNDWATER AT A BOREHOLE.

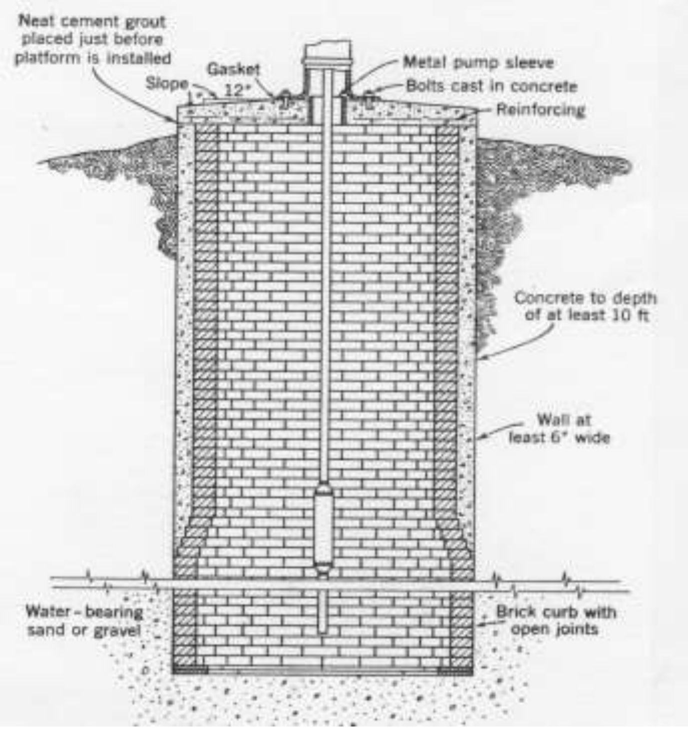
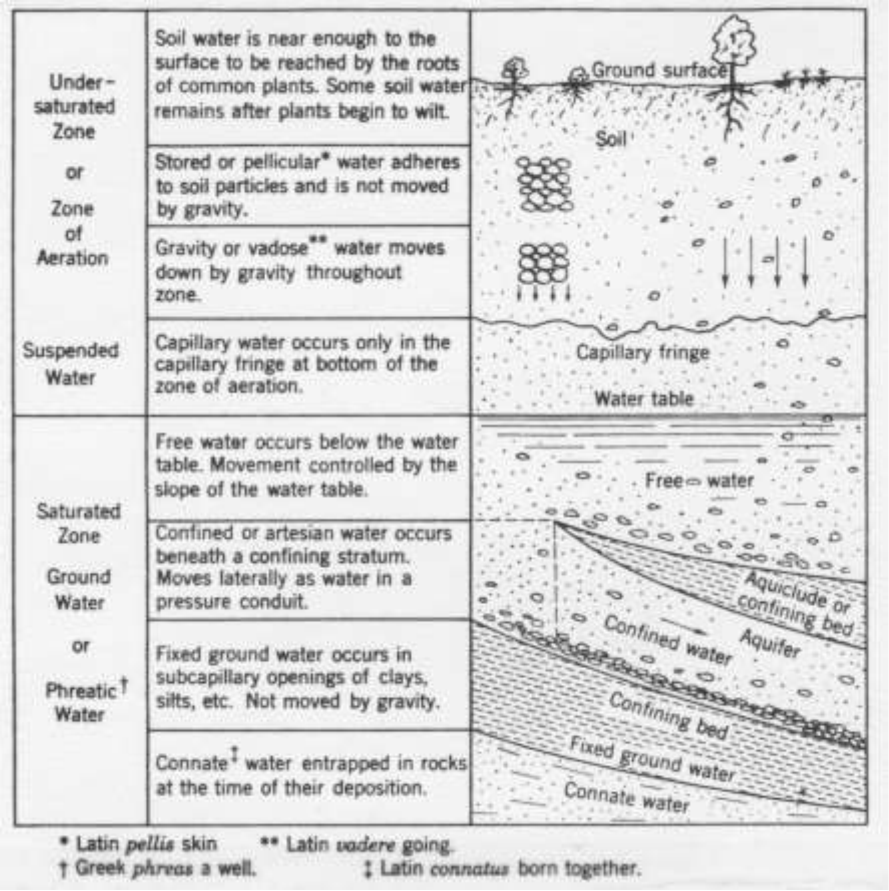


Figure 7 – PREVENTION OF POLLUTION OF GROUNDWATER AT A WELL.

1.2.3 The Movement of Groundwater.

Water is present in the ground in a number of forms, some of which can be removed by mechanical means and some of which cannot. The various water bearing zones are indicated in figure 8.



Under-saturated Zone or Zone of Aeration	Soil water is near enough to the surface to be reached by the roots of common plants. Some soil water remains after plants begin to wilt.	
	Stored or pellicular* water adheres to soil particles and is not moved by gravity.	
	Gravity or vadose** water moves down by gravity throughout zone.	
Suspended Water	Capillary water occurs only in the capillary fringe at bottom of the zone of aeration.	
Saturated Zone Ground Water or Phreatic† Water	Free water occurs below the water table. Movement controlled by the slope of the water table.	
	Confined or artesian water occurs beneath a confining stratum. Moves laterally as water in a pressure conduit.	
	Fixed ground water occurs in subcapillary openings of clays, silts, etc. Not moved by gravity.	
	Connate‡ water entrapped in rocks at the time of their deposition.	

* Latin *pellis* skin ** Latin *vadere* going.
† Greek *phreas* a well. ‡ Latin *connatus* born together.

Figure 8 – THE VARIOUS WATER BEARING ZONES.

It is seen that although there is water present in the undersaturated or aeration zone, this water cannot be removed by the usual mechanical means. In talking of the movement of groundwater one is meaning the movement of the water in the saturated zone.

The movement of water in the undersaturated zone is of interest when dealing with:

1. the downward movement of polluted waters;

2. the development of leachate;

3. the rate of recharge of an aquifer, both natural recharge and artificially aided recharge.

The movement of groundwater is determined primarily by the geology of the area. The rate of movement of water through the ground is slow, varying from about 2 metres per day to a few metres per year. The rate at which water moves through the ground is the permeability, the more permeable the soil/rock the faster the water will move through it.

Groundwater will move towards a lower point if this is possible. This lower point may be a river, lake, sea or point of abstraction. This may be seen in figure 3 - earlier.

A stratum that contains water that can be removed at a quantity that permits its development is called an aquifer - examples are sand and gravel. An aquiclude is a stratum that contains water but is incapable of transmitting it in significant amounts - an example is clay. An aquifuge can neither contain nor transmit water - an example is solid granite.

Aquifers may be unconfined when a free water surface or water table exists. This is the area of free groundwater as shown in figure 3, or they may be confined between two aquicludes or aquifuges.

1.2.4 The Abstraction of Groundwater.

Groundwater may be abstracted in a number of ways:

1. from a spring. Here the water appears at the surface by the force of gravity. The water may be collected and led or pumped away. To prevent the contamination of the water, a collection unit such as shown in figure 9 is essential;

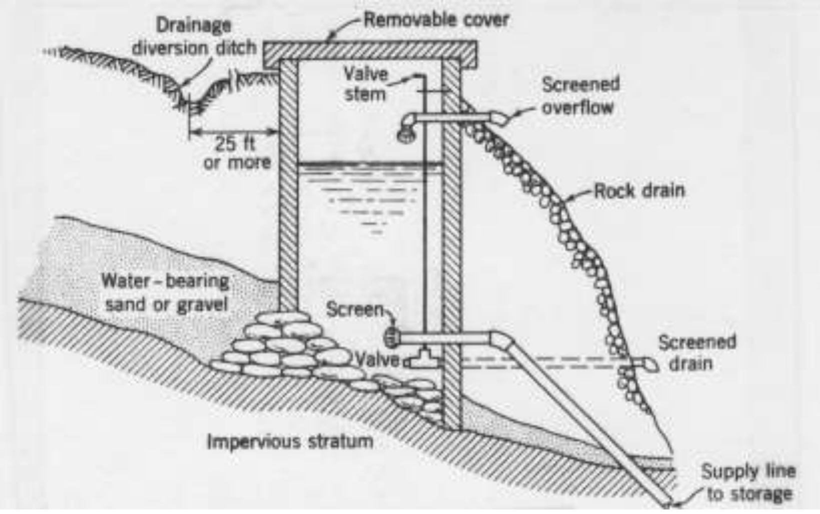


Figure 9 – PROTECTION OF A SPRING.

2. From an artisan well. Here a well or borehole is constructed from the surface down through the upper confining layer of the aquifer and into the aquifer itself. As the aquifer is confined, the water is under pressure and will rise in the well or borehole. If the pressure is high enough, the water may reach the surface or may even spray into the air. See figure 10.

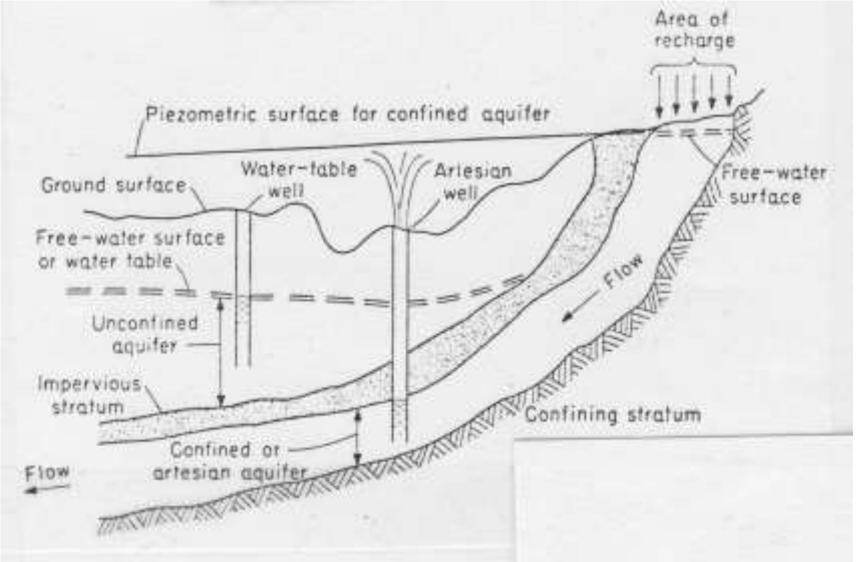


Figure 10 – AN ARTESIAN WELL.

3. From a well or borehole. Here the well or borehole is dug or drilled down to the water table. The water will rise to a certain distance up the well or borehole, but the water will have to be pumped out to reach the surface. If the water surface is always less than about 6 - 7 metres below the surface the pump may be placed on the ground next to the well or borehole and the suction pipe lowered down the shaft. If the water surface is more than 6 - 7 metres below the ground the surface mounted pump will not be able to lift the water and

the pump will have to be lowered down the shaft. The pump may be driven from the top by means of a rotating shaft or the pump/motor assembly may be submerged. Naturally a special type of electric motor is required.

Since a well has storage capacity, small quantities of water may be removed by lowering a suitable container on the end of a piece of rope. When a borehole is used for a small abstraction rate a hand operated pump may be installed. A lift pump may be used for up to about 5 metres lift and a force pump will be needed for a greater lift. Wind power could also be used as in a windmill.

The rate at which water may be abstracted depends on the rate of replenishment of the water at the point of abstraction. The water table will be depressed at the point of abstraction - the so called cone of depression. See figure 11.

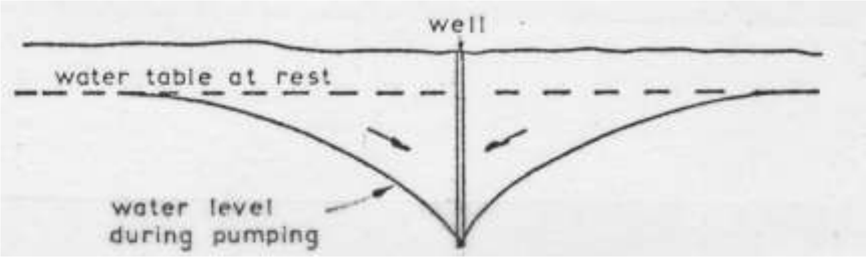


Figure 11 – CONE OF DEPRESSION DURING ABSTRACTION.

If the rate of abstraction is excessive and exceeds the rate of recharge then the water level will drop and mining of the water has occurred. If this occurs near the sea then salt water intrusion can occur and this can ruin the water source. See figures 12 and 13.

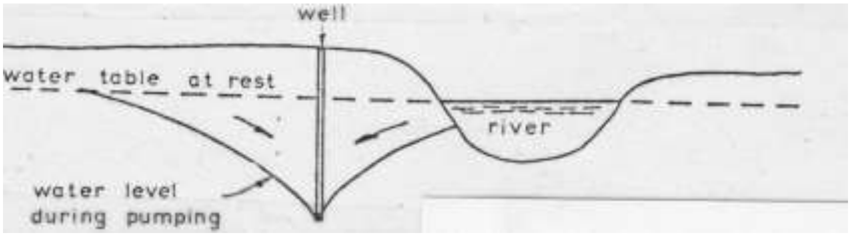


Figure 12 – DRAWDOWN ALLOWING WATER FROM RIVER INTO AQUIFER.

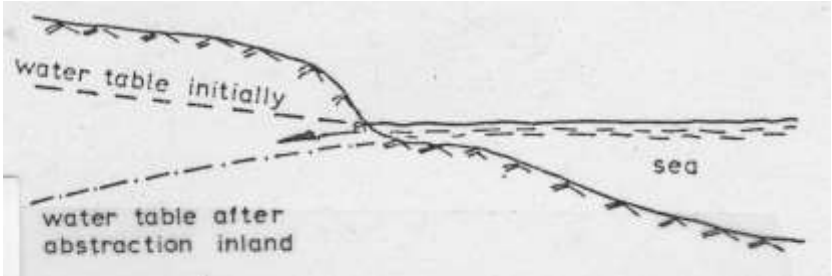


Figure 13 – OVER ABSTRACTION ALLOWING SEA WATER INFILTRATION.

The volume of groundwater that is replaced through natural means is relatively small because of the slow rate of movement of the groundwater and the limited opportunity for surface waters to penetrate the earth's surface. The latter opportunity has been made worse by the construction of impervious surfaces such as houses and roads, and by the removal of forests and their replacement by fields. All these changes increase the rate of runoff and so reduce the opportunity for the water to infiltrate the ground.

As man has to import water from further afield to satisfy this demand for water of an acceptable quality, the replenishment of groundwater supplies by artificial recharge is beginning to receive more attention. In certain areas, mining of groundwater has resulted in the water table dropping. At one place in the United States of America, the groundwater has dropped by 100 metres over a 30 year period.

There are advantages in recharging aquifers, amongst these are:

1. the water table can be raised, thereby increasing the total quantity of water stored. This can be particularly useful in areas where the land is flat and it is not possible to build a reservoir or where the evaporation is so high that significant losses of water would occur, with the resultant increase in the dissolved solids content;

2. the water recharged to the aquifer may be used to prevent saline or poor quality water intrusion;

3. the recharge of an aquifer with treated high quality wastewater may delay or even render unnecessary the development of an expensive surface water source;

4. provision for the concurrent development for surface and groundwater supplies. (See also later in this section). The water used for recharge purposes may be excess surface water, certain industrial effluents such as cooling water and treated wastewater.

There are several methods of groundwater recharge depending on the quality of the water being recharged and whether any other benefits are obtained at the same time.

1. The utilization of holding basins. Here high quality water is run into earth wall basins and allowed to seep into the ground. See figure 14.

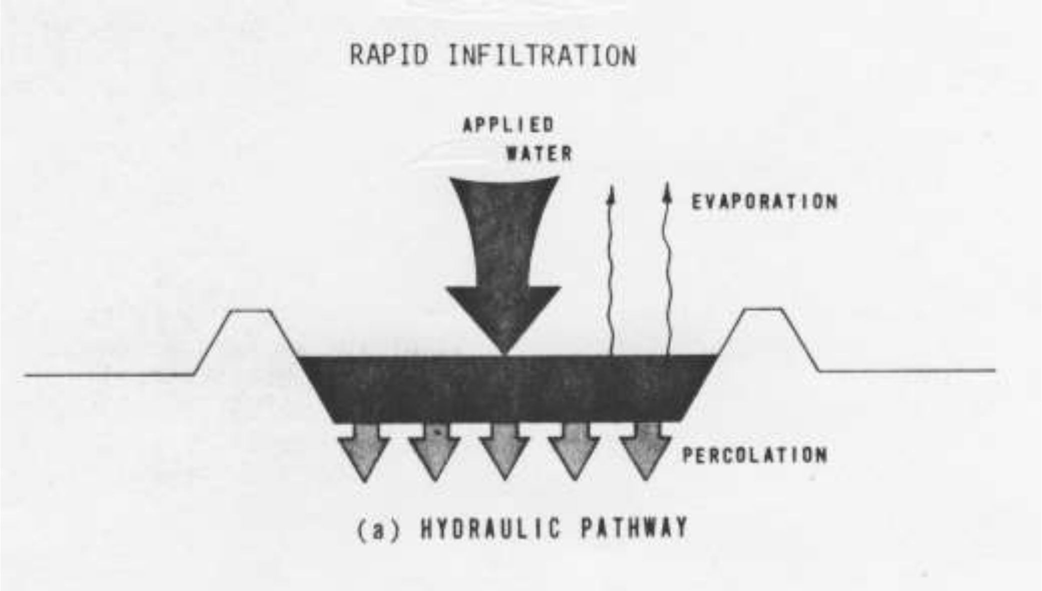


Figure 14 – RECHARGING OF GROUNDWATER BY RAPID INFILTRATION.

2020 note: This is the method used in Atlantis in the Western Cape where reed bed polished domestic wastewater effluent and stormwater is used to recharge the aquifer ***above*** *the water abstraction boreholes. Treated effluent from the industrial wastewater treatment works together with stormwater from the noxious trade area and the acid effluent from the water softening plant is used to recharge the aquifer* ***below*** *the water abstraction boreholes. This is to act as a hydraulic barrier and to prevent seawater intrusion into the aquifer. The writer was a major role player in this concept and the operation of the system.*

Depending on the quality of the water and of the soil, infiltration rates of up to 90 metres per year are possible i.e. 900 m^3 per hectare per year. Accumulated fine solids trapped on the surface of the soil will have to be removed periodically to maintain the rate of infiltration.

The higher the suspended solids content of the feed water, the shorter the runs between renovating the soil surface. High quality water only should be fed onto these areas because nitrogen in the ammonia form can travel readily through the ground. The dissolved solid content of the water being recharged must be low. If intermittent infiltration is practised then aerobic conditions should exist in the soil during the dry period and oxidation of the ammonia to nitrate should occur. During the subsequent flooding period some denitrification should occur but much of the nitrate will pass further downward.

Ideally the water used for rapid infiltration should have a low total nitrogen content. The limitation on a low phosphorus content is not so severe, as phosphate can be locked up in the soil in an insoluble form.

2. Water may be returned to the ground by mean of injection wells or boreholes that are operated in reverse. Only very high quality water can be injected to prevent blocking on the soil. In this case the renovating effect of the water passing through the soil, as in the above case, is lost. Injection wells are most suitably used to prevent salt water intrusion in areas close to the sea.

Both the above methods use fairly small areas of land and do not provide any benefit other than the return of the water to the ground. Pre-treatment costs are also fairly high.

There are two methods available where a lower grade of water may be used and where a side benefit is obtained:

1. Slow rate land infiltration. Here water is spread over suitable ground by means of sprinklers or shallow surface channels. See figure 15. Crops may be grown at the same time. The type of crop to be grown depends on:

    1. the quality of the water being handled;
    2. the climate of the area;
    3. the type of soil of the area;
    4. the natural rainfall of the area;
    5. the rate of application of the water.

    Here the crop being grown will remove some of the nutrients from the water being applied, the rate of application of the water being an important factor. The application of water to the land is seasonal, as the crops do not require water throughout the year. However, by planting various crops or by sacrificing the collection of a crop the system may be used all year round.

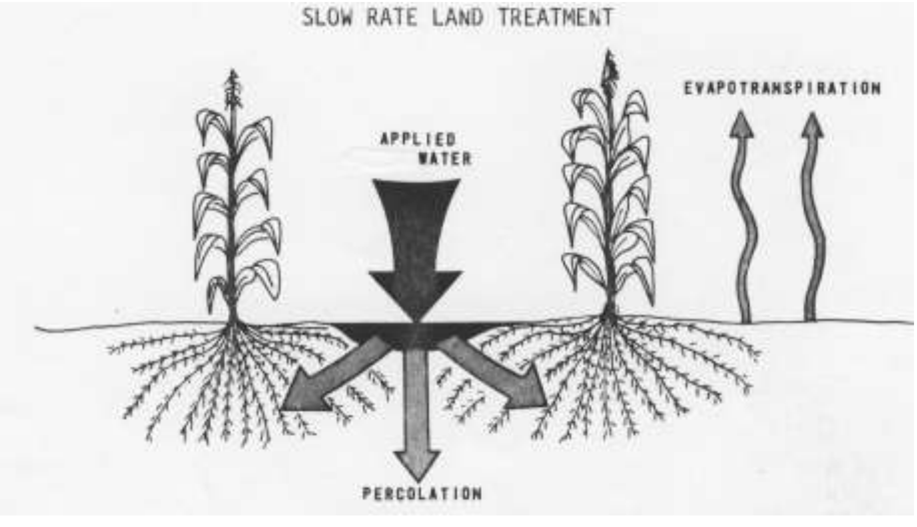


Figure 15 – RECHARGING OF GROUNDWATER BY SLOW INFILTRATION.

2. Overland Flow is another method of groundwater recharge, although this system is usually used as a treatment process rather than as a groundwater recharge system. See figure 16.

   The water collected in the run off collection ditch could be used for rapid infiltration.

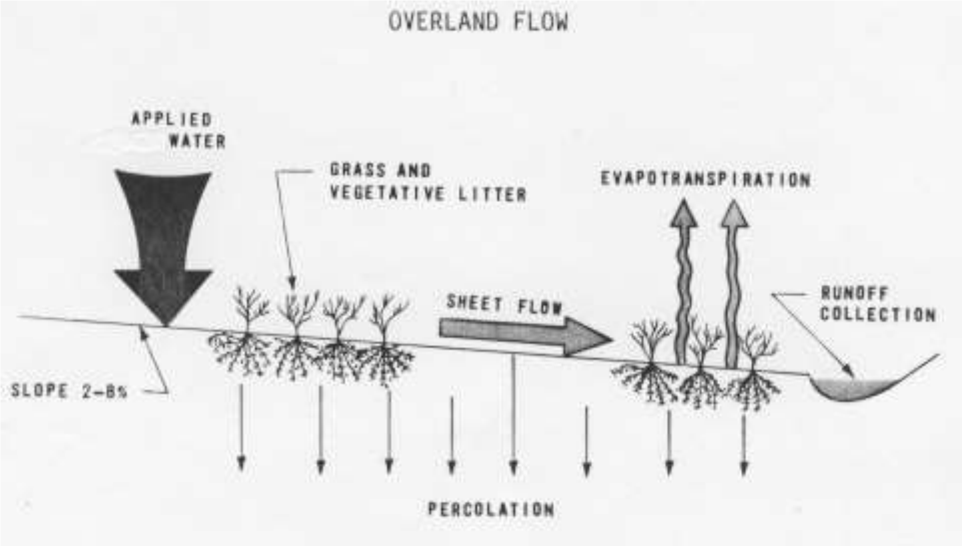


Figure 16 – RECHARGING GROUNDWATER BY OVERLAND FLOW.

In certain circumstances, it may be possible to develop a combined surface water - groundwater scheme where during the dry months the water requirements additional to the surface water availability is drawn from the groundwater supply; while during the wet months, the excess surface water is used to recharge the groundwater supply.

1.3 THE COLLECTION OF RAINWATER.

In this section, one is referring to the direct collection and storage of rainfall water falling on man-made impervious surface. Catchment areas of reservoirs are excluded.

The collection of rainwater is the water source most prone to fluctuation. The only storage capacity is in the tank being used to hold the water. Usually the first flush of water collected on the surface is of a poor quality as this water will contain dust, leaves, general dirt, and even salt in areas near the sea. It is advisable to discharge this water using one of the patented devices available.

Only in rare instances are systems larger than that intended for one family, installed.

WATER SOURCES and WATER TREATMENT.

PART 2

STANDARDS FOR POTABLE WATER.

2.1 INTRODUCTION.

Various organisations have set standards to which the quality of water supplied for drinking purposes must or should comply. Three such sets of standards are discussed below.

Quality standards for water treated to drinking water standard may be divided into 4 sections:

1. substances affecting the potability of the water;
2. substances that may affect health;
3. substances that are toxic;
4. bacteriological quality standard.

2.1.1 The SABS Standard.

The recommended standard used in South Africa is SABS 241 of 1971 ***[since replaced by a later issue]*** in which a "Recommended" and a "Maximum Allowable" limit has been laid down.

2.1.2 The World Health Organisation.

This organisation published a set of standards for drinking water under the title "International Standards for Drinking Water Third Edition 1971".

2.2 DISCUSSION OF THE WATER QUALITY STANDARDS.

2.2.1 Substances affecting the potability of the water.

The table below gives the recommended and maximum allowable concentrations as listed in SABS 241/1971; and the maximum acceptable and the maximum allowable as listed by the W H O. The standards issued by the W H O for the developing nations and relate to a quality that is lower than that which would be demanded by richer and more sophisticated communities.

TABLE 1 – RECOMMENDED AND ALLOWABLE CONCENTRATION IN POTABLE WATER.

PARAMETER	SABS Rec.	SABS Max	WHO 1971 Max Acceptable	WHO 1971 Max. Allowable
Total Dissolved Solids mg/L	500	2000	500	1500
Colour on Hazen Scale	20	N.S.	5	50
Turbidity (NTU)		1	5	10
Taste	(a)	(a)	(a)	(a)
Odour		(b)	(b)	(b)
Chloride as Cl mg/L		250	200	600
Sulphate as SO_4 mg/L		250	200	400
Calcium as Ca mg/L	N.S.		75	N.S.
Magnesium as Mg mg/L		100	50	150
Total Hardness as $CaCO_3$ mg/L	200 (c)	1000	N.S.	N.S.
Iron as Fe mg/L		0.3	0.3	0.7
Manganese as Mn mg/L		0.1	0.1	0.4
Copper as Cu mg/L		1.0	1.0	1.5
Zinc as Zn mg/L		5	5	15
Mg + Na + SO_4 mg/L	N.S.	N.S.	N.S.	500
Phenol as C_6H_6OH mg/L	0.001	0.002		0.001
pH value (range)	7.0 - 8.5	5.5 - 9.0		7.0-8.5

Notes:
N.S. - not specified.
(a) - unobjectionable
(b) - essentially nil.
(c) - there is a recommended minimum hardness limit of 20 mg/L as $CaCO_3$.

The physical characteristics of turbidity, colour, taste and odour are limited primarily for aesthetic reasons.

Chloride, sulphate and dissolved solids have taste and laxative properties. The combination of sodium sulphate with magnesium sulphate is particularly noticeable. The limit for copper is to prevent a copperous taste. Like copper, zinc is also an essential element in the human diet; however the excess amount acts as a gastro-intestinal irritant. Iron and manganese are objectionable because of brownish-coloured stains imparted to laundry and porcelain and the bittersweet (astringent) taste attributed to iron.

The taste due to phenols is enhanced significantly by chlorination, due to the formation of chloro-phenols.

2.2.2 Substances that may affect health.

TABLE 2 – LIMITS FOR FLUORIDE AND NITRATE.

Fluoride as F.	SABS Recommended Limit	1.5 mg/L
	S A Criteria	not stated
	WHO Maximum Acceptable	1.5 mg/L
Nitrate as N	SABS Recommended Limit	10 mg/L
	S A Criteria	10 mg/L
	WHO Maximum Allowable	10 mg/L

NOTE: 10 mg/L as N is equal to 45 mg/L as NO_3.

Fluorides cause mottling of the enamel of the teeth when present in water in concentrations in excess of 1.5 mg/L

Nitrates present in water consumed directly or indirectly in prepared food by infants may be reduced to nitrites in the intestinal tract, leading to infantile methaemoglobinaemia. This disease does not usually occur in older children or adults.

Radioactivity also falls into the classification of substances that may affect health. The WHO has tentatively established the following limits as the maximum acceptable.

TABLE 3 - TENTATIVE LIMIT FOR RADIO-ACTIVITY.

PARAMETER	TENTATIVE LIMIT
alpha - emitters	1 µµC/L
beta - emitters	10 µµC/L

Radium 226 is an alpha emitter and strontium 90 is a beta emitter. When gamma emitters and strontium 90 are absent, the total concentration of beta emitters may not exceed 1 000 µµC/l.

Strontium and radium having similar atomic structure to calcium, are concentrated in the bones of the body from where the radiation can damage the body.

2.2.3 Substances that are toxic.

TABLE 4 – MAXIMUM LIMITS FOR TOXIC SUBSTANCES.

ation in µg/l	Recommended Maximum Limit - SABS 241/1971	Maximum Allowable Limit - WHO 1971
Arsenic as As	50	50
Barium as Ba	N.S.	1000
Cadmium as Cd	50	10
Chromium as Cr	50	50
Cyanide as CN	10	200
Lead as Pb	50	50
Mercury as Hg	N.S.	1
Selenium as Se	N.S.	10

N.S. - Not Specified.

With the exception of cyanide and selenium, the above are cumulative poisons. Arsenic in the inorganic state was used in insecticides but has since been superseded by organic poisons. Although human blood contains traces of arsenic (about 100 µg/l), it does not appear to have any beneficial effects. Some authorities consider arsenic to be carcinogenic. Arsenic is very slowly eliminated from the body.

Barium has a toxic effect on the heart, blood vessels and nerves. The limit appears to be an arbitrary one.

Cadmium has a high toxic potential, its tolerable level is not known.

Chromium also has a high toxic potential and its tolerable level is also not known. One reference states that the 50 µg/L limit was the lowest level that was analytically determinable at the time that the standards were drawn up.

Cyanide is usually oxidised to thiocyanate in the body or by the residual chlorine in the water. One reference states that the cyanide limit appears to have been based on its toxicity for fish and not for man.

Lead poisoning was a problem when soft water was reticulated in lead pipes. This problem has been overcome by the elimination of lead pipes and in many cases by stabilizing the water to reduce its plumbo-solvency. Although the lead intake via the water has reduced, the lead intake via the air and via food has increased and hence the need to set the lead limit at a low value.

Mercury is a serious cumulative poison, hence the very low allowable level.

Selenium is toxic to man and has been cited as a cause of dental caries and said to be carcinogenic.

2.2.4. Bacteriological quality standards.

Experience has established that water in which the number of coliform organisms is below a certain range of values will not contain pathogenic bacteria. This range is specified in drinking-water standards such as the International Standards for Drinking-Water. The number of coliform organisms found to be present in samples of water is expressed statistically as the "*most probable number*" of coliform organisms per 100 mL of sample, termed "MPN index". For treated water, the International Standards sets the following requirements:

In 90 %, of the samples examined throughout any year, coliform bacteria shall not be detected or the MPN index of coliform micro-organisms shall be less than 1.0. None of these samples shall have an MPN index of coliform bacteria in excess of 10.

An MPN index of 8 - 10 should not occur in consecutive samples. With the examination of five 10 ml portions of a sample, this would preclude three of the five 10 ml portions (an MPN index of 9.2) being positive in consecutive samples.

In any instance in which two consecutive samples show an MPN index of coliform bacteria in excess of 8, an additional sample or samples from the same sampling point should be examined without delay. This is the minimum action that should be taken. It may also be desirable to examine samples from several points in the distribution system and to supplement these with samples collected from sources, reservoirs, pumping stations and treatment points. In addition, the operation of all treatment processes should be investigated immediately.

The SABS standard is nil MPN coliforms per 100 ml.

2.3 TREATABILITY TO POTABLE STANDARD.

Although it is possible to treat almost any water to the potable standard, the cost of treating certain water may be excessive. Without resorting to very sophisticated treatment processes, a guide can be given to the quality of the raw water that can usually be treated to the potable standard.

2.3.1 Physical quality.

The limiting value for colour should be set at 300 units on the Hazen scale, on the basis that a value of less than 300 units indicates an acceptable quality for treatment and anything over 300 units indicates that special treatment may be needed to provide water meeting the drinking water standards. Colour produced by certain industrial wastes may have to be oxidized or adsorbed by activated carbon.

The limiting value for turbidity or suspended solids is usually about 1 000 mg/L. Where higher values are obtained a pre-settlement tank is indicated.

The limiting value for odour and taste cannot be set as the nature of the odour or taste has a bearing on the acceptable limit. Certain odours can be removed by aeration while others may have to be removed by oxidation or activated carbon treatment.

2.3.2 Chemical quality.

Certain substances will pass through a conventional water treatment works essentially unaltered. In these cases the limiting value in the raw water is set as the same as the limiting value in the treated water.

Total dissolved solids - 1 500 mg/L -When coagulants are added, the total dissolved solids will increase, the actual increase being dependent on the dosage used.

Iron as Fe - 50 mg/L - Any iron present in the raw water must be oxidised to an oxide or hydroxide of iron for removal. Excessive concentrations of iron would need excessive doses of oxidant.

Manganese as Mn - 5 mg/L - The same applies here. Special processes are available to remove iron and manganese.

TABLE 5 – CHEMICAL QUALITY LIMITS FOR WATER SUPPLY SOURCE

Parameter	Limit mg/L
Copper as Cu	1.5
Zinc as Zn	5
Barium as Ba	1.0
Chromium as Cr	0.05
Lead as Pb	0.05
Chloride as Cl	250
Sulphate as SO_4	250
.Fluoride as F	1.5
Nitrate as N	10
Phenols as C_6H_50H	0.002

The following parameters may be used as chemical indicators of pollution, where the value indicated in the minimum indicative of pollution.

TABLE 6 – PARAMETERS INDICATING POLLUTION OF WATER SOURCE.

Parameter	Value mg/L
Chemical Oxygen Demand	10
Ammonia as N	0.5
Total Nitrogen excl Nitrate as N	1.0
Grease	1.0
Carbon-chloroform Extract	0.5

Excess phosphate concentrations in raw water are associated with eutrophication problems such as excess algal growth and with coagulation difficulties attributed to complex phosphates which have a dispersant action.

It is difficult to set a maximum value for hardness as this depends on so many factors. Hardness can be removed by special processes that are covered later.

2.3.3 Bacteriological quality.

TABLE 7 – CATERGORIZING WATER SOURCE BY BACTERIAL CONTENT.

CATEGORY	MPN/100mL
Bacterial quality applicable to disinfection treatment only	0 - 50
Bacterial quality requiring conventional methods of treatment (coagulation, filtration, disinfection).	50 – 5 000
Heavy pollution requiring extensive types of treatment.	5 000 – 50 000
Very heavy pollution, unacceptable unless special treatment designed for such water is used, source to be used as last resort only	> 50 000

Where more than 40 % of the bacteria are of the faecal coliform group, the water source should be demoted into the next category with respect to the treatment required.

XXXXXXXXXXXX

THE FOLLOWING PARTS DEAL WITH THE VARIOUS STAGES IN WATER TREATMENT (UNIT OPERATIONS)

The potable water standards and the suggested limits for the raw water have been discussed above in Part 2. Parts 3 to 7 will cover the so-called conventional treatment processes. Parts 8 to 11 will cover the more sophisticated process for the removal of certain substances.

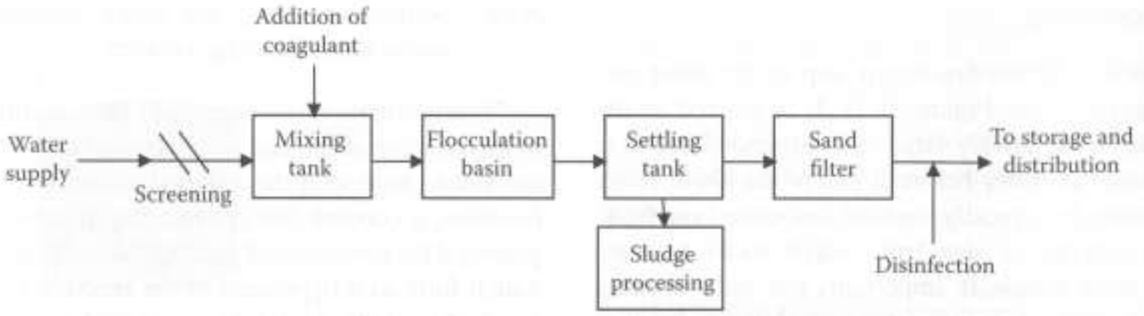


Figure 17 – TYPICAL WATER TREATMENT WORKS FLOW DIAGRAM.

WATER SOURCES and WATER TREATMENT.

PART 3.

PRE-TREATMENT.

3.1 AERATION.

3.1.1 The purpose of aeration.

Water devoid of dissolved air and hence oxygen has a "flat" taste, which is replaced by a "fresh" taste on aeration of the water. Taste and odour producing substances such as hydrogen sulphide or some of the volatile substances liberated by algae growths or incidental to the decomposition of organic matter, are released from water by aeration. Aeration is utilized primarily to provide oxygen from the atmosphere for the oxidation of iron and manganese and to liberate hydrogen sulphide and carbon dioxide from the water - hence it serves as one aspect of corrosion prevention.

3.1.2 Limitation.

Not all odour and taste producing substances will be removed by aeration. These compounds that are only partially volatile will be removed only partially by aeration. Even fully volatile substances such as carbon dioxide will not be removed fully, because no aeration system is 100% efficient and also because there will be a certain concentration of the substances in the water that is in equilibrium with a certain concentration of the substance in the air.

3.1.3 Theory.

Gases are dissolved by or liberated from aerated water until equilibrium is reached between the content of each specific gas in the atmosphere and its content in the water.

The concentration of each gas in aerated water at equilibrium is directly proportional to that portion of the atmospheric pressure on the water surface due to that specific gas. This is known as Henry's Law of partial pressure. The concentration of gases dissolved in water is inversely proportional to the temperature of the water.

The diffusion of a gas into water is very slow unless large water surfaces are exposed, and unless the water is agitated so as to expose various portions of the water to the gas, (usually the atmosphere).

Gases will be absorbed by water when their concentration in the water is below that at equilibrium. Conversely gases will be released when the concentration of the gases is higher than that at equilibrium. The gases of interest in water treatment are oxygen and carbon dioxide in the atmosphere, and these gases plus hydrogen sulphide and volatile substances dissolved in water.

The equilibrium oxygen concentration in fresh water at a pressure of 760 mm Hg and at 20°C is 9.17 mg/L; at 15°C is 10.15 mg/L and at 10°C is 11.33 mg/L. These values are above those found in stored waters below the thermocline, so aeration will increase the oxygen content of such waters. On the other hand, growth of algae may cause the dissolved oxygen content of the water in the circulating zone of reservoirs to be above saturation, in which case aeration will reduce the dissolved oxygen content.

Since the normal atmosphere contains no hydrogen sulphide, aeration will remove this volatile substance, provided the atmosphere contacting the water is replaced by suitable air movement to eliminate the released hydrogen sulphide from the vicinity of the aerator.

The situation is more complex when carbon dioxide is involved, because both physical and chemical equilibrium are involved. Physical equilibrium between the carbon dioxide in the atmosphere and the carbon dioxide dissolved in distilled water of zero alkalinity, would be about 0.5 mg/L. Aeration of water, seldom reduces the carbon dioxide content below about 4.5 mg/L and a minimum of 10.0 mg/L is more common. As will be seen later under corrosion control, a carbon dioxide content of 4.5 mg/L is in chemical equilibrium with alkaline bicarbonates when the alkalinity is 100 mg/L. That portion of the carbon dioxide content below 4.5 mg/L is non-corrosive at alkalinity 100 mg/L. Aeration effective enough to secure a final concentration of 4.5 mg/L will then prevent corrosion only with water having alkalinities in excess of 100 mg/L. Less alkaline waters may have a high carbon dioxide content, and aeration will remove enough carbon dioxide to reduce materially the dose of calcium hydroxide needed to reduce the carbon dioxide content further for full corrosion prevention. The above aeration process will increase the dissolved oxygen content of the water. This will reduce somewhat the benefits of carbon dioxide removal as far as reducing the corrosivity of the water is concerned.

Many taste and odour producing substances are of low volatility, so aeration is not so effective in eliminating them. Aeration alone cannot completely control tastes and odours, but it serves to reduce the doses of chlorine or activated carbon needed for complete taste and odour removal.

Aerators should be designed to provide the maximum area for the surface separating the atmosphere and the water, to provide agitation, and to ensure free flow of the atmosphere to remove gases and volatile substances liberated from the water. Two types of aerators have been developed:

1. those forming drops of water or thin sheets of water exposed to the atmosphere;

2. those forming small bubbles of air which rise in the water.

3.1.4 Operation and maintenance of aeration devices.

The problems of operation consist mainly in maintaining the equipment in a clean condition and controlling the flow of water so that maximum water surface and agitation are provided.

Slime and algae growths on the surfaces of aerators will require periodic removal.

The diffusers used with diffused air aerators may become partly clogged, either from dust in the compressed air or from the collection of sediment on the outside surfaces during periods when the aerator is not in use.

Cleaning methods vary depending on the construction of the diffuser. Among the methods used are:

1. burning in a furnace;

2. washing in 35% nitric acid;

3. washing in "chromic" acid - 1g Sodium Dichromate / 50mL H_2SO_4;

4. washing in detergent solution.

3.1.5 Types of aeration devices.

1. Spray aerators - here special nozzles are used to produce a fine spray;

2. Waterfall aerators - here water falls down a number of steps or trays. A useful improvement is to fill the trays with coke and perforate the base of the tray, so that the water passes through as many small streams;

3. Cascade aerators - here water runs down an incline in a thin sheet onto numerous small baffles or obstructions that divert the flow into many criss-crossing currents;

4. Diffused air aerators - similar to those used for wastewater treatment.

3.1.6 Effectiveness of aeration.

The effectiveness of aeration is measured by the increase in dissolved oxygen content or by the decrease in hydrogen sulphide or carbon dioxide content, and also by the removal of taste and odour producing compounds.

Generally speaking the minimum dissolved oxygen content should be 50% saturation after aeration in order to oxidize iron and manganese compounds completely. Hydrogen sulphide should be eliminated completely.

3.2. CONTROL OF TASTES AND ODOURS.

Consumers object to potable waters having noticeable tastes or odours, both because they are objectionable and because many tastes and odours are interpreted as evidence of pollution or of unwholesomeness of a water. The control of tastes and odours should not be neglected; otherwise the consumers will lose confidence in the quality of an actually safe supply and will use any available substitute as long as it is attractive. Thus tastes and odours in potable waters can present a public health problem. The only real solution is the delivery of potable water that is both safe and attractive.

The control of tastes and odours involves a number of unrelated procedures and processes which must be considered separately. They may be characterized as either preventive or corrective measures:

1. Preventive - includes the prevention of the growth of taste and odour producing organisms and the prevention of chemical changes that detract from the quality of a water;

2. Corrective - includes aeration, the use of chlorine as an oxidizing agent, the use of ozone as an oxidizing agent, and the use of activated carbon to adsorb taste and odour producing substances.

These procedures may be used in combination and also as part of a broader application of treatment processes. For example, pre-chlorination may be used to improve disinfection and coagulation, and at the same time to oxidize tastes and odours and to suppress the biochemical decomposition of organic matter in sludge in sedimentation tanks.

Usually the most complicated group of processes are those involved with taste and odour control and these must be balanced and manipulated by trial and error, as the only ultimate guide is the subjective attitude towards the taste and odour of the treated water. Even this appraisal is not final, because a satisfactory treatment works effluent may deteriorate in distribution systems or become less satisfactory, as when odourless cold water is heated and hidden odours are liberated.

The natural sources of tastes and odours are:

1. the growth of micro-organisms in waters that release taste and odour producing substances;

2. the decomposition products of these same organisms when killed, or of leaves, grasses and aquatic vegetation in reservoirs;
3. the growth of slime organisms, moulds and fungi;

4. the reduction of sulphates to sulphides in water devoid of oxygen.

In addition to the above sources may be added the taste and odour producing compounds the domestic and industrial wastewaters discharged into natural waters.

3.3 THE CONTROL OF TASTES AND ODOURS BY PREVENTATIVE MEASURES.

3.3.1 Water pollution control.

Increasing attention is being given to the treatment of domestic and industrial wastewaters to restore polluted water resources or to protect those waters not yet significantly polluted. The object of water pollution control agencies is to maintain water resources so that the normal and economic treatment of the water drawn from them will produce safe and wholesome potable water. This may require drastic action, such as limiting phenolic wastes so that the receiving waters will not have a phenol content of greater than 0.005 mg/L, because these compounds react with chlorine to produce chlorophenolic compounds having an objectionable taste even at these low concentrations. This residual concentration of phenols may be oxidised by free-residual chlorine or by chlorine dioxide, ClO_2, treatment. Paper mill wastes produce a high chlorine demand and also lead to "woody" tastes. These tastes are not removed by aeration or easily oxidised by free chlorine, so they must be removed by activated carbon treatment.

3.3.2 Prevention of chlorinous tastes.

Prior to the use of free-residual chlorination, chlorine-ammonia treatment was used to secure disinfection without chlorinous tastes, since adequate disinfection with doses of chlorine alone often caused objectionable chlorinous tastes. The chloramines produced by chlorine-ammonia treatment are poor disinfectants, so taste control was accompanied by less effective bactericidal action.

The chlorinous tastes produced by small doses are not actually tastes of chlorine itself but rather of chloro-organic compounds formed by the reaction between chlorine and organic matter. Thus the remedy is not to reduce the chlorine dose but to increase it so that these compounds will be oxidized or altered, leaving free-residual chlorine. This is covered in greater detail later.

The attitude of the individual to the taste and odour of free-residual chlorine depends on their subjective reactions. It is generally held that they are less objectionable than chlorinous tastes. In any case, free-residual chlorine will produce a different degree of tastes and odours in waters of different characteristics, being somewhat more noticeable as an odour in soft waters of lower pH value. No hard-and-fast guides are available, except that residuals of 0.3 to 0.5 mg/L may usually be present in delivered water without serious consumer complaints.

These concentrations of free-residual chlorine are more than adequate for post chlorination, provided pre-chlorination has been controlled to achieve its purpose. One should endeavour to produce safe waters that do not have an objectionable taste of free-residual chlorine.

3.3.3 The suppression of growths of organisms.

1. Bacteria.

Many bacteria in water have no relationship to sanitary quality criteria, some may be beneficial such as those that utilize organic matter in domestic wastewaters or they may be objectionable but still of no sanitary significance such as the sulphate reducing bacteria. The suppression of growths of bacteria in distribution systems is by ensuring that a suitable free-residual chlorine content is maintained at all times.

The so-called iron bacteria "Crenothrix" and "Leptothrix" may develop in gelatinous masses around the screens of shallow wells yielding iron-bearing water, or in distribution systems where the dissolved oxygen content of the water is low. Their growth will create objectionable odours and lead to the presence of precipitated iron in the water. The growth of these organisms in the distribution system should be prevented by iron removal treatment and by maintaining a satisfactory free-residual chlorine content.

2. Rooted aquatic vegetation.

Such vegetation is of three types:

1. emergent plants i.e. with roots in the soil and leaves sticking out of the water;
2. floating leaf plants;
3. submerged rooted plants.

This vegetation is of significance as it contributes organic matter and colour to stored water when it dies and it clogs screens of the intakes. Emergent plants and floating leaf plants are difficult if not impossible to control. The use of the hormone regulation chemicals such as 2.4D (2,4-dichloro-phenoxy-acetic acid) has been found to be reasonably effective when dosed by 1 kg per hectare. 2.4D is water and oil soluble. It is used in water diluted 1:750 and in oil diluted 1:100. The oil solution seems to be preferable because the droplets of oil float and become attached to the exposed plants rather than being diluted in the water.

3. Marginal Weeds.

Reservoirs with steep bank and flooded areas do not usually support troublesome growths of marginal weeds, whereas those with gentle slopes and only seasonal fluctuations in water level will support weeds. These marginal weeds can be removed by cutting or burning. 2.4 D may be used against broad-leaved plants but is not effective against grasses.

4. Plankton or free floating organisms.

These include algae, protozoa and crustaceans. These are important to waterworks operators because they are prevalent in stored waters and create nuisances from the tastes and odours they produce, or lead to short filter runs because the floc in which they are enmeshed does not settle readily and a portion remains in settled waters leading to the filters.

For the most part the algae will be found in the upper, circulating zone. Many protozoa are found more widely distributed because they are independent of the influence of light. Higher forms of organisms are not taste and odour-producing but are of significance because they may be so prevalent as to shorten filter runs. Certain of these organisms are just visible to the naked eye: crustaceans, worms and insect larvae, hence their presence in unfiltered surface supplies will be objectionable to consumers, even though they are harmless.

Plankton must be controlled as a routine aspect of taste and odour prevention. The only way of preventing worms and insect larvae in distribution systems served by unfiltered supplies is not to use intakes at or near the bottom of reservoirs; these organisms congregate in the bottom deposits and are not killed by the usual dose of chlorine when stored water is treated as it enters the distribution system.

The growth of plankton in reservoirs may be controlled by copper sulphate treatment, generally satisfactory results are obtained. The dose required to kill the various organisms varies between 0.1 and 0.3mg/L $CuSO_4.5H_2O$. By dosing at 0.3mg/L, one can be fairly certain that nearly all the various organisms will be killed. The required dose is influenced by temperature, alkalinity and carbon dioxide content of the water. Low temperatures reduce the effectiveness of copper sulphate. High alkalinity and low carbon dioxide content of the water being treated decrease the effectiveness of copper sulphate, because insoluble copper carbonate is precipitated and thus is not absorbed into the cells of the organisms.

The dose of copper sulphate does not have to be computed from the total capacity of the reservoir, because plankton usually congregate in the upper circulating zone, it is usual to treat the top 4.5 m of water. The volume of water to be treated is then 45 000 m^3 per hectare of surface area. In certain cases only the water in shallow coves may need to be treated, especially when the areas are fed by streams containing much carbon dioxide and other plant food.

When protozoa are involved, or when spring and autumn overturns distribute the water throughout a reservoir depth, the whole reservoir content will have to be treated. This is difficult when waters have alkalinities much over 50 mg/L because much copper is precipitated as copper carbonate before the copper sulphate can diffuse to lower depths. For alkaline waters, the dose should therefore be based on surface area rather than on the volume of water. The difficulty in the treatment of alkaline waters is reduced when fine granular copper sulphate is scattered by a blower over the water surface so that the granulate dissolve and liberate copper sulphate as they sink through the lower depths. In general a dose of 5 kg per hectare is used.

There is a limitation to the use of copper sulphate - it is toxic to fish, however only trout is seriously affected by doses of 0.3 mg/L. Experience has shown that fish-kills have been due to careless distribution of the chemical which has resulted in localized high doses. Furthermore, immediate causes of fish-kills have been the clogging of the gills of the fish by dead algae or the reduction of the dissolved oxygen content of the water due to the decomposition of dead algae. These influences may be minimized by starting the application of copper sulphate at the wall of the reservoir and proceeding towards the points where the streams enter, so that the fish may avoid the chemical by swimming towards the untreated entering waters.

The need for copper sulphate treatment of a reservoir should preferably be determined by periodic microscopic examinations. In the absence of laboratory control, the odour test should be used and treatment provided when the odour has an intensity of 3 i.e. "distinct" by the simple procedure or a threshold odour number of 3 to 5 by the threshold procedure.

3.3.4 The dosing of copper sulphate to reservoirs.

1. The burlap method.

   Here the required mass of copper sulphate is placed in a bag made of burlap (a type of coarse canvas) and is towed behind a boat, preferably motor powered to help mix the solution.

2. The wooden box method.

   This is an improvement on the above method, special boxes are built onto a boat to hold the copper sulphate, the sides of the box are covered with a copper mesh screen that allows the water to pass through the box and dissolve the copper sulphate. One advantage of this method is that the box may be topped up with solid copper sulphate at any time.

3. The spray method.

   Here a concentrated solution of copper sulphate is sprayed over the water from a moving boat.

4. The blower method.

   Here finely ground copper sulphate powder is blown onto the water from a moving boat.

The effective control of micro-organisms throughout the year is facilitated by the continuous application of copper sulphate to the water entering the reservoir. The micro-organisms are thus controlled before heavy growths can develop; this has the advantage of removing the possibility of dangerous an oxygen sag taken place after the treatment with copper sulphate. The continuous application of copper sulphate reduces the availability of food for fish, so continuous treatment should be restricted to those reservoirs used exclusively as sources of public water supply.

3.4 THE CONTROL OF TASTES AND ODOURS BY CORRECTIVE MEASURES.

3.4.1 Aeration.

Aeration is not usually very effective in reducing tastes and odours because of the low volatility of many of the taste and odour components; however sulphides are removed by aeration. Aeration can have the advantage of releasing some of the more volatile odours and this can reduce the chlorine or activated carbon doses needed to remove the remaining odours.

3.4.2 Free-residual chlorination for taste and odour control.

A distinction must be made between free-residual chlorination to prevent chlorinous tastes and the procedures used to destroy tastes and odours in waters under treatment. The destruction takes while the various reactions are taking place, so it will be completed when only free-residual chlorine remains. In other words, chlorine doses designed to secure rapid and effective disinfection should be adequate for taste and odour control. Experience has shown that a reaction period of at least 2 hours may be needed for the destruction of odorous substances, i.e. the Cl_2 dose must be sufficient for these oxidation reactions so that only free residual chlorine persists thereafter.

For example, water may have free-residual chlorine of 0.5 mg/L after 30 mins and 0.2 mg/L free residual chlorine with 0.2 mg/L combined residual chlorine after 2 hours; the latter is due to reactions that have happened in the additional 1.5 hours. The combined-residual chlorine portion may have an objectionable taste if it is in the form of chloro-organic compounds rather than chloramines. The dose of chlorine should therefore be increased so that only free-residual chlorine is present at the 2 hours mark.

This pre-chlorination, as it is called, providing only free-residual chlorine in the effluent of sedimentation tanks, will ensure ample opportunity for the destruction of tastes and odours, as well as the other benefits mentioned later.

Some tastes and odours cannot be altered or removed by this method, and one of the alternative methods must then be used. See later.

3.4.3 Chlorine - Ammonia treatment.

Chlorine reacts with ammonia to yield a group of compounds known as chloramines.

1. $NH_4^+ + HOCl \rightarrow NH_2Cl + H_20 + H^+$ monochloramine;
2. $NH_2Cl + HOCl \rightarrow NHCl_2 + H_20$ dichloramine;
3. $NHCl_2 + HOCl \rightarrow NCl_3 + H_20$ trichloramine (nitrogen trichloride).

Usually only enough ammonia is added to produce monochloramines and some dichloramines.

This treatment procedure was developed many years ago to produce chloramines which are much more stable and less chemically active than free chlorine. This lesser activity reduces the rate of disinfection, but also lessens the reactions between chlorine and organic matter, thus minimizing chlorinous tastes and odours and the chlorine residual lasts for a longer period. This may be required where there is a very long residence time in a water pipe. The chlorine-ammonia treatment prevents or minimizes tastes and odours incidental to chlorination, whereas free-residual chlorination destroys taste and odour producing substances.

3.4.4 Chlorine Dioxide Treatment.

Chlorine dioxide (ClO_2) rapidly and completely oxidizes phenol compounds and many other pollutants from industrial wastes, but does not eliminate tastes incidental to decay of vegetation. Most economical results are often obtained by pre-chlorination to free-residual chlorine and post chlorination with chlorine dioxide.

3.4.5 Activated Carbon Treatment.

This treatment process is the most certain means of removing tastes and odours, provided that money is available for the purchase of the large doses occasionally needed. Activated carbon is more effective that chlorine in removing woody, earthy or mouldy tastes or odours due to actinomycetes or geosmin.

The action of the carbon is due to the very large active or adsorptive surface present. The carbon will usually be added as a slurry prior to filtration, or may sometimes be added directly to the reservoir.

3.4.6 Ozone Treatment.

Ozone is very effective in the destruction of tastes and odours and gives more uniform, predictable results. Since ozone is a very active oxidizing agent, it alters taste and odour producing compounds into substances that do not affect the senses of taste and smell. Ozone doses are determined on the basis of elimination of tastes and odours.

WATER SOURCES and WATER TREATMENT.

PART 4.

COAGULATION AND FLOCCULATION.

4.1 THE PHYSICAL STATE OF IMPURITIES IN WATER.

There are three forms of impurities in water, dissolved, suspended and colloidal.

4.1.1 Dissolved impurities.

In this case the divided material or solute is homogeneously and molecularly (or ionically) dispersed in the solvent to form a true solution. The particle diameters range from those of single atoms about 0.2 nm up to about 1 nm. This is approximately the resolution limit of the electron microscope. The separation of substances in true solution can be achieved by phase change such as distillation, by chemical reaction leading to phase separation such as precipitation, or by selective permeation through membranes such as electro-dialysis or reverse osmosis.

4.1.2 Suspended impurities.

This may be conveniently split into 2 states; the settleable solids and the non-settleable solids. The distinction between the two is not sharp as the time available for settling is important and inter-particle reactions may play a part. The lower limit of the size range for this class of material is about 0.5 µm (500 nm); this varies somewhat with the shape and density of the particles. This is about the size of bacteria and the wavelength of light and so represents also the lower limit of microscopic visibility.

4.1.3 Colloidal impurities.

In between the upper particle size of true solutions and the lower particle size of suspensions lies the colloidal range i.e. about 1 to 500 nm. In colloidal dispersions, the particles:

1. cannot be removed from water by ordinary filtration but can be removed by ultra-filtration or dialysis through special membranes;

2. cannot be seen directly but can be visualized as specks of light using the Tyndall effect;

3. can be seen and photographed by the electron microscope;

4. will not settle by gravity because of their Brownian motion, but will settle in an ultra-centrifuge.

Colloidal impurities may be responsible for either the turbidity or the colour in the water.

4.2 THE NEED FOR COAGULATION.

The essential properties of colloidal dispersions can be ascribed to their very large surface area to volume ratio of the colloidal particles. In a true solution the system consists of one phase only, and there is no true surface of separation between the molecular particles of solute and solvent. Colloidal dispersions are two phase systems and for each particle there is a definite surface of separation between it and the solvent. At this interface characteristic properties, such as adsorption and electrical potential are evident. When the total surface area is relatively small such as occurs in suspensions where the particles are relatively large, the effects of these properties are not marked. In colloidal systems, the surface area for a given volume of material is so large, that these surface effects play an important role.

The total surface area of 1 cm^3 of material in the form of a cube of 1 cm edge is 6 cm^2. If the material is broken up into cubes of 10 nm edge, then the total surface area of all the cubes is 6 000

000 cm^2. This is a million fold increase in surface area for the same mass of material.

The surface effects that are important are those of adsorption and the tendency to develop an electric charge relative to the surrounding medium.

Colloids are classified into hydrophobic or water-hating or hydrophilic or water-loving groups.

4.2.1 Hydrophobic Colloids.

These colloidal particles are normally charged with respect to the surrounding medium, the sign and the magnitude of the charge being characteristic of the colloidal material and the composition of the medium. Colloidal materials such as clay, the tea-like organic colour of some waters are negatively charged when present in waters with pH values near 7, while the hydrous oxides of iron and aluminium are usually positively charged. Since the colloidal particles have an electric charge on their surface, they cannot approach each other very closely as the like electric charges will repel each other. It is this repulsion of like charges that prevents the colloidal particles from coalescing and growing in size. There is also a natural force of attraction between any two masses, the Van der Waals force. While the forces of repulsion due to the like electric charges exceed the forces of attraction, the colloidal suspension will remain stable and will not settle.

4.2.2 Hydrophilic Colloids.

These colloidal materials are stable because their attraction for water molecules causes them to be encased by a "shield" of bound water that prevents the individual particles from approaching close enough that the forces of attraction can predominate and cause the particles to coalesce, and become large enough to settle. The particles may have an electric charge but this is of lesser importance that is the case with hydrophobic colloids. Soap, starch and detergents are examples of hydrophilic colloidal materials. Certain clays and hydrous ferric and aluminium oxides may show properties of both hydrophobic and hydrophilic colloids.

As long as the colloidal dispersion is stable, there will be no coalescence of the particles and no settlement will take place and the turbidity or colour of the water will remain. The purpose of coagulation is to destabilize the dispersion to allow the particles to come together and form insoluble particles large enough that the gravitational forces will predominate and cause the particles to settle.

4.3 THE MECHANISM OF COAGULATION.

In the case of hydrophobic colloids, this destabilization is brought about by adding a substance whose highly charged metal ion will reduce the effective charge of the colloidal particle so that the particles may approach each other close enough that the Van der Waals forces of attraction may predominate and cause the particles to coalesce. The more highly charged the metal ion the more effective the process. This is why the triple charged ions of iron and aluminium are most commonly used. Organic polymers may be used to help to bring the destabilized particles together by attaching themselves to many particles at once.

In the case of hydrophilic colloids, the addition of a coagulant destabilizes the dispersion by compressing the layer of bound water surrounding the colloidal particles and allowing the particles to approach each other more closely, so that the forces of attraction may predominate and allow the particles to coalesce.

It must be stressed that the process of coagulation is only the process of destabilizing the dispersion and allowing the destabilized particles to coalesce. The process of aggregation of the destabilized particles into large settleable flocs is called flocculation, it follows as a matter of course that coagulation must precede flocculation.

4.4 COAGULANTS.

Brief details of the most commonly used coagulants are given below.

4.4.1 Aluminium Sulphate.

$Al_2(SO_4)_3.14H_2O$. This is also known as alum or filter alum. It is available as lumps, granules, powder or as a liquid containing about 7% Al_2O_3. A solution of alum is acidic and is thus corrosive to most metals. Suitable storage vessels may be made of lead, rubber, PVC or polyethylene.

The reaction equation of aluminium sulphate in water is usually written as

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

In practice, however, the reaction is much more complicated as ionic forms such as $Al(OH)_2^{+}$ and $Al(OH)^{++}$ are also present.

The OH^{-} group is obtained from the alkalinity of the water. If the water is poorly buffered, the pH value will drop on the addition of the aluminium sulphate and the concentration of OH^{-} may be too low for the reaction to go to completion. In these cases, it will be necessary to increase the alkalinity by adding calcium hydroxide or sodium carbonate.

When highly buffered coloured waters are to be treated, the pH value may have to be lowered using sulphuric acid.

4.4.2 Sodium Aluminate.

$Na_2Al_2O_4$ or $Al(ONa)O$. This may be thought of as a compound of aluminium oxide and sodium hydroxide. This coagulant is not used alone but with alum to secure some special results. The failure of some very cold waters to coagulate with alum alone may be corrected by adding about 3 mg/L sodium aluminate with the alum. Another important use of this coagulant is in connection with "double coagulation" of highly coloured water in which alum alone or with sulphuric acid is added to the first stage coagulation and settlement. This results in rather high concentrations of soluble aluminium remaining in the second stage. Sodium aluminate, with lime if necessary, is added to raise the pH to precipitate the aluminium hydroxide that is allowed to settle in a second sedimentation tank. A third use of sodium aluminate is in conjunction with lime-soda softening of water in which the insoluble $CaCO_3$ and $Mg(OH)_2$ needs to be coagulated for quick sedimentation.

Sodium aluminate is available as crystals or as a solution containing about 32% $NaAlO_2$ ($Na_2Al_2O_4$). Being alkaline the solution may be stored in iron, polyethylene or rubber vessels. It is much more expensive than alum but smaller doses are used.

4.4.3 Ferrous Sulphate.

$FeSO_4.7H_2O$ also known as copperas. This is a granular powder that has an acidic reaction. In natural waters, ferrous hydroxide may be formed, but as this is appreciable soluble, it is necessary to oxidise it to ferric hydroxide which is insoluble. However this oxidation does not take place below pH 8.5 - 9.0. It is necessary to add lime with the ferrous sulphate to ensure coagulation. For this reason, ferrous sulphate is not used in the coagulation of highly coloured waters, which coagulate best at pH values less than 6.0.

Ferrous sulphate and lime make a very effective coagulant in the lime-soda softening process and in the coagulation of iron and manganese.

It is usually dosed as a liquid, as the crystals are hygroscopic and tend to cause "arching" in the hoppers of dry chemical feeders.

4.4.4 Chlorinated Copperas.

It was seen above that ferrous ions are oxidised by the dissolved oxygen of the water above about pH 8.5 - 9.0. Chlorine may be used to oxidise the ferrous ion irrespective of the pH value, ferric sulphate and ferric chloride are formed and these in turn act as coagulants. Usually 1 kg chlorine is reacted with about 5 kg ferrous sulphate.

$$6\ FeSO_4 + 3\ Cl_2 \rightarrow 2\ Fe_2(SO_4)_3 + 2\ FeCl_3$$

This chlorine dose is in excess of the stoichiometric quantity to ensure complete reaction but this has the advantage of helping to disinfect the water being treated. This system may be coupled with prechlorination.

4.4.5 Ferric Sulphate.

$Fe_2(SO_4)_3.9\ H_2O$. This is available as a powder; it is very soluble in water. The solution is acidic and must be stored in polyethylene, rubber or stainless steel vessels. The reaction is similar to ferrous sulphate, but it is already oxidised.

4.4.6 Ferric Chloride.

It may be stored in polyethylene, rubber or glass, but not in stainless steel because of the effect of high concentrations of chloride on the stainless steel. As with all iron salts, ferric hydroxide is the coagulant formed.

4.4.7 pH Adjustment.

It will be seen in the next section that the pH of the water being coagulated may have a large effect on the chemical dosage needed and the effectiveness of the process. All the coagulants covered above, with the exception of sodium aluminate have an acidic reaction in water. In some cases fairly high chemical dose is needed to destabilize the dispersion and in poorly buffered waters the pH value may drop to a point where the effectiveness of the process is reduced. In these cases an alkaline chemical must be added. See section 4.6.5.

If the pH need be lowered then sulphuric acid is usually used.

4.5 FACTORS AFFECTING COAGULATION.

Among the factors that affect the effectiveness of the coagulation process are:

4.5.1 The pH of the water.

There is always an optimum pH value at which the coagulation process is most effective. This can vary from lower than about pH 5 when colour is to be removed up to about pH 8 when turbidity is to be removed. When the sulphate content of the water is high, the optimum pH value tends to be lowered whereas when the calcium content is high the optimum pH value tends to be raised.

4.5.2 The coagulant used.

The most effective coagulant to be used depends on the quality and the nature of the water. This can usually only be found by experimentation.

4.5.3 The dose of coagulant to be used.

This again depends on the quality and nature of the water, and is found by experimentation. In many cases, it is found that when coagulation is performed at pH values other than the optimum value, then increased coagulant dosage is required.

4.5.4 The rate at which the flash mixing is done.

In order to ensure effective coagulation, it is essential that all portions of the water to be coagulated receive the exact dose of coagulant at the same time. As this is not practically possible, every effort must be made to ensure that the mixing is as rapid and as uniform as possible. Certain coagulants pass through several ionic forms before precipitating and usually not all forms have the same destabilizing efficiency

.

By not having effective rapid mixing, some of coagulant may pass into a less effective form before it has had a chance to destabilize some colloidal material with the result that some of the colloidal material will remain dispersed.

4.5.5 The temperature of the water.

This affects the rate of destabilization and the rate at which the coagulant passes through its ionic forms. As the temperature of the water is usually beyond the control of the waterworks operator, he may have to alter the coagulation process by one or more of the following methods:

1. change the pH value;
2. change the coagulant type;
3. change the coagulant dosage;
4. use a coagulant aid (see next section).

4.6 COAGULANT AIDS.

It will be seen later that it is essential that effective coagulation takes place as even with the best flocculation and sedimentation processes being used, the result will not be the desired one. In other words one cannot make up for ineffective coagulation.

Difficulties with coagulation often occur because of slow settling or fragile flocs that are easily broken up by the hydraulic forces in the water. Coagulant aids benefit flocculation by improving settling and toughness of flocs. The most widely used coagulant aids are given below.

4.6.1 Polyelectrolytes.

Synthetic polymers are long-chain, high molecular-mass, organic chemicals commercially available under a wide variety of trade names. Polyelectrolytes are classified according to the type of charge on the polymer chain. Those possessing negative charges are called **anionic**, those positively charged are called **cationic**; and those carrying no electric charge are **non-ionic**. Anionic or non-ionic are often used with metal coagulants to provide bridging between colloids to develop larger and tougher floc growth the dosage required as a coagulant aid is generally in the order of 0.1 to 1.0 mg/L. In the coagulation of some water, polymers can promote satisfactory flocculation at significantly reduced alum dosages. The potential advantages of polymer substitution are in reducing the quantity of waste sludge produced in alum coagulation, and in changing the character of the sludge that it can be more easily dewatered.

Cationic polymers have been used successfully in some waters as primary coagulants for clarification. Although the unit cost of cationic polymers is about 10 to 15 times higher than the cost of alum, the reduced dosages required may nearly offset the increased cost of chemicals. Furthermore, unlike the gelatinous and voluminous aluminium hydroxide sludges, polymer sludges are relatively dense and easier to dewater for subsequent handling and disposal.

Sometimes cationic and non-ionic polymers can be used together to provide an adequate floc, the former being the primary coagulant and the latter a coagulant aid. Although significant strides have been made in the application of polyelectrolytes in water treatment, their main application is still as an aid rather than as a primary coagulant. Many waters cannot be treated by using polymers alone but require aluminium or iron salts. Jar tests and actual plant operation must be used to determine the effectiveness of a particular proprietary polyelectrolyte in flocculation of a given water.

4.6.2 Activated silica.

This is sodium silicate that has been treated with sulphuric acid, aluminium sulphate, carbon dioxide, or chlorine. A dilute solution is partially neutralized by an acid and then is allowed to age for a period of time up to 2 hours. Additional aging runs the risk of solidification of the entire solution by gelation. The aged silicate solution is generally further diluted before being applied to the water. As a coagulant aid, it offers the advantages of increased rate of chemical reaction, reduced coagulant dose, extended optimum pH range, and production of a faster settling, tougher floc. One of the disadvantages of activated silica, relative to polyelectrolytes, is the precise control required in preparation and feeding. Activated silica is normally used with aluminium coagulants at a dosage between 7 and 11 percent of the alum dose, expressed as milligrams per litre of SiO_2.

4.6.3 Bentonitic clay.

This may be used in treating waters containing high colour, low turbidity, and low mineral content. The iron or aluminium floc produced under these conditions is frequently too light to settle readily. Addition of clay results in a weighting action that improves settleability. Clay particles may also adsorb organic compounds, improving treatment. Although exact dosage must be determined by testing, 10 to 15 mg/L frequently results in formation of a good floc. Some of the clays absorb taste- and odour-producing substances especially oily materials, so their use may assist in the treatment of certain polluted waters.

4.6.4 Oxidants.

Problems in clarification of surface waters and colour removal can often be minimized by applying oxidants. The most common practice is breakpoint chlorination of raw water. In this procedure, sufficient chlorine is added to oxidize interfering organic compounds. Other, less widely used oxidants are potassium permanganate, ozone and chlorine dioxide.

4.6.5 Alkaline substances.

Although not strictly coagulant aids, these substances are included for completeness. It was shown above, that in poorly buffered waters, the pH may be reduced below the optimum by the addition of the coagulant, and it may be necessary to increase the pH and the alkalinity of the water by the addition of an alkaline substances. Amongst those used are: calcium hydroxide, (known as slaked lime, hydrated lime or simply lime); calcium oxide (known as unslaked lime or quicklime), sodium carbonate (known as soda ash); or rarely because of its cost, sodium hydrogen carbonate (sodium bicarbonate).

4.7 THE REMOVAL OF COLOUR.

Organic colour in natural waters appears to be related to humic substances that occur in soil and peat. Chemically these are polymeric compounds with carboxylic and phenolic functional groups. They may exist as large molecules or as colloids.

The removal of colour by coagulation is most effective at low pH values, sometimes less than pH 5. This plus other substantive evidence indicates that the colour is removed as an aluminium colour complex with the colour molecule occupying the coordinate sites of the metal ion. The latter usually being aluminium, or occasionally iron.

After colour removal, it will be necessary to raise the pH to between 7 and 9 depending on the quality of the water in order to render it stable with respect to calcium carbonate to prevent it being corrosive. If turbidity is present as well, it may be necessary to remove the colour by coagulation and sedimentation and then to coagulate the turbidity in a separate stage.

4.8 FLOCCULATION.

In the coagulation stage, the colloidal material has been destabilized and a fine floc will have been formed. As this floc is very fine, it will have a very low settling velocity and since the rapid mixing stage will not have been 100% efficient some colloidal material will remain dispersed.

In the flocculation stage, the fine flocs are encouraged to come together and to form larger flocs that have a much higher settling velocity that will enable them to settle in the 2 - 4 hours usually allowed for this stage. By gently mixing the flocs with the water, use is made of the ability of the flocs to absorb any remaining unstabilized colloids present in the water.

The rapid mixing stage that immediately precedes the flocculation stage, to distribute the coagulant throughout the water being treated is usually called "Flash Mixing". This rapid agitation may be provided in special tanks with capacities equivalent to about one minute of flow, in which small propellers are driven by electric motors. Sometimes the hydraulic jump or standing wave is used for flash mixing, being provided by a channel with sloping and widening sections. The coagulant is added just before the water flows down the channel at high velocity to enter a level portion, where the energy of rapid flow is suddenly transformed into static head of deeper water, turbulence being produced at the wave front of the deeper water.

Flocculation tanks are of various types, some using patented equipment. Good results have been reported with very simple types consisting of a weir for flash mixing, with provision for subsequent gentle agitation in the channel leading from the weirs to the sedimentation tanks. Early flocculation tanks were fitted with a series of baffles around the ends of which the flowing water was reversed in direction, thus causing more gentle turbulence in the channels formed between the baffles, but more violent agitation at each point of reversed flow. The same effects were secured by arranging the baffles so that the water flowed over and under them alternately. Such tanks are cheap to build, as the baffles may be of ordinary timber placed in concrete tanks.

They are only moderately successful because the degree of agitation is determined by the space between the baffles, the total length of the channels so produced, and the volume of flow. While the spacing of the baffles may be altered, this is a major operation and usually is restricted to the correction of initially faulty design. Furthermore, as the degree of agitation decreases with the volume of flow, the water is less effectively flocculated during the period of low flow, and the serious loss of head results from turbulence at each point of change of direction together with friction due to the area of the baffles and of the bottom of each channel between the baffles in contact with the flowing water.

For example, a tank of this type with an average velocity of 0.3 m/s would require a channel length of 18 m for each minute of flocculation period. This period of only 5 minutes flocculation provided by a 90 m total length of channel would entail serious loss of head, especially at the smaller treatment works where the friction losses are proportionally higher due to a large surface ("wetted perimeter") per unit volume of water. This type of flocculation tank is seldom large enough to provide the flocculation periods of 15 - 30 minutes that are favoured by modern practice.

Baffled flocculation tanks are being superseded by those fitted with mechanical agitators or paddles, or using either diffused air or the jet action of the flowing water to secure controlled agitation.

Mechanical agitators, or flocculators, consist of revolving paddles with horizontal or vertical shafts, or of paddles suspended from horizontal oscillating beams and moving up and down as the beams are driven by a crank attached to a speed-reducing unit, in turn driven by an electric motor. The total area of the paddles is usually 10 - 25% of the area swept by their movement, and the paddles move at about twice the speed imparted to the water; this causes eddies to form around the edges of the paddles resulting in agitation. The speed-reducing units should preferably provide for variable speed, so that the degree of agitation may be regulated to secure the best results. The degree of agitation can then be made independent of the rate of flow of water. The maximum peripheral speed should be about 0.6 m/s so as to provide an adequate range of speeds below that value.

As the size of the flocs increases, they become easier to break up by shear. In order to prevent this happening it is preferable to decrease the intensity of the mixing gradually through the flocculation stage. This may be done in the "round the end" or the “under and over" tanks by increasing the spacing between the baffles along the length of the tank. However the previously mentioned problem of the inability of this type of tank to maintain a constant degree of agitation with varying flow rate is still applicable.

The most suitable arrangement for flocculation is a series of flocculation basins (three or four) in which the water passes from one basin into the next with each basin having a decreasing degree of agitation. This allows the floc size to increase as the water passes through the basin and has additional advantage of reducing the possibility of short circuiting and allowing unflocculated coagulated material to pass onto the next treatment stage.

In the flocculation stage, it is important to aim for a strong compact floc even though these may not be the largest floc particles. A large feathery floc will be delicate and will not compact well as there will be a large volume of water trapped within the floc. A dense firm floc will also settle more rapidly than a large feathery floc and this will lessen the possibility of floc carry over from the sedimentation tank during periods of high flow

4.9 PHYSICAL FACTORS AFFECTING THE COAGULATION AND FLOCCULATION PROCESSES.

There are several factors that affect the coagulation and flocculation processes, some within the control of the Process Controller and some beyond their control either by the constraints of the treatment works or by extraneous factors.

4.9.1 The degree of agitation.

It has been seen that flash mixing requires pronounced agitation for a brief period to mix the coagulant and water and to favour the initial chemical reactions associated with coagulation. Subsequent flocculation, however, should be aided by controlled agitation represented by velocities of flow of 0.2 - 0.6 m/s. Velocities below 0.1 m/s permit sedimentation of the floc when it should be in suspension aiding in the adsorption of smaller floc, whereas velocities over 0.6 m/s will prevent the growth of the larger, readily settling floc.

In general, best results are secured with intermediate degrees of flocculation with velocities of about 0.3 - 0.4 m/s. Higher velocities may be used for turbid waters and lower velocities for coloured waters or those of low turbidity.

The most effective layout is a series of tanks with decreasing degree of agitation. In an existing plant with only one flocculating tank, it may be advantageous to install a variable speed gearbox, in order that the degree of agitation may be varied until the most suitable degree is formed if this is not possible, structural modifications may have to be made.

4.9.2 The retention period.

Flocculation should be continued for 10 - 30 minutes. In general, the longer periods permit lower coagulant doses and the use of more gentle agitation to ensure the formation of larger, readily settling floc. Conversely, when detention time is short, the degree of agitation should be increased. The longer detention periods require larger and more costly tanks, the cost of the larger tanks must be compared with the lower chemical costs.

4.9.3 The temperature of the water.

The reduced reaction rate at lower water temperatures requires that the flocculating period be longer or that the coagulant dose be increased. Any given velocity of flow or speed of paddles will cause greater agitation of colder waters because of their higher viscosity. Usually the water usage is less during winter so that the flocculating period is automatically lengthened.

4.9.4 A change in the quality of the water.

A change in the quality of the water may require a change in the dose of the coagulant, or even a change in the coagulant.

4.9.5 A change in the flow rate through the works.

If the demand for water increases and it is not possible to enlarge the works, then any increase in flow automatically reduces the retention time in all sections of the works. Increases in flow may also cause an increase in turbulence in parts of the treatment works and this may damage the floc and result in floc being lost over the sedimentation tank weirs onto the sand filters.

This may necessary a change in the dosage or type of coagulant or the introduction of a coagulant aid to strengthen the floc and to increase its settleability.

4.10 THE MONITORING OF COAGULATION AND FLOCCULATION EFFICIENCY.

It has been seen above that if the coagulation process is not efficient that stabilized colloidal material may pass through the next stages of treatment and could result in the final water being turbid or coloured or both. If the coagulation stage is efficient but the flocculation stage is not, then fine floc may be lost over the sedimentation tank weir and will be caught in the sand filter. This will result in an increase in the solids loading on the filter and if excessive can drastically reduce the length of the filter runs.

A well coagulated and flocculated water will be clear and colourless and containing a compact rapidly settling floc. The turbidity of the water after settling for one hour should be preferably less than 5 NTU, and definitely less than 20 NTU.

Laboratory scale tests determine coagulant type and dose may be performed using the jar test. This uses a special stirring unit that has variable speed output. The unit usually has 4 or 6 stirrers so that different doses may be compared under identical conditions. The test portion giving the best flocculation and reduction of colour and turbidity by sedimentation and filtration will indicate the optimum doses of coagulant(s) and period of flocculation.

On the treatment works, the routine supervision of flocculation is aided by a few simple procedures which supplement the more technical procedures:

1. The appearance of the floc being flocculated and settled. If a white glazed tile is fixed some 200 - 300 mm below the surface, the floc may be seen clearly. Experience will tell if the floc is of good quality;

2. The degree of agitation in flocculating tanks may be estimated by using handfuls of confetti which are thrown into the water to be carried by the eddy currents under observation;

3. Dyes may be used to check on detention times to see if short circuiting is taking place (Similar to the method covered in section 5.6).

The final check on the effectiveness of the flocculation is the quality of the settled water passing to the filters and the filtered water.

WATER SOURCES and WATER TREATMENT.

PART 5.

SEDIMENTATION.

5. The purpose of sedimentation is to allow the settleable solids to settle out and to thus reduce the concentration of suspended solids that must be removed by the filters. The solids that have to be removed may be present in the raw water or may result from one of the chemical treatment processes employed or may be a mixture of the two.

5.1 SETTLING CHARACTERISTICS OF DISCRETE AND FLOCCULENT PARTICLES.

The sedimentation operations may be classified approximately as falling into 4 separate categories. These are:

1. Class 1;
2. Class 2;
3. Zone Settling;
4. Compression Zone.

Classes 1 and 2 relate to low concentrations of suspended solids and are dealt with below. Zone and compression sedimentation relate to higher concentrations of suspended solids and are dealt with later. See figure 18.

The class 1 sedimentation referred to above relates to discrete particles and class 2 to flocculent particles.

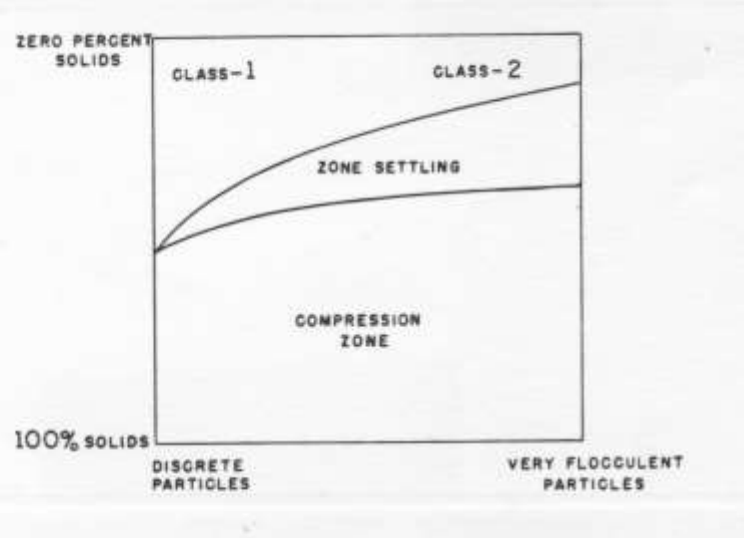


Figure 18 – CLASSIFICATION OF SEDIMENTATION PROPERTIES

5.1.1 The settling of discrete particles.

As one is dealing with low concentrations of suspended solids (i.e. less than about 2 000 mg/L), the settling of the particles is unhindered and is a function only of the fluid properties and the characteristics of the particle. A discrete particle is one that in settling does not alter its size, shape or mass. In falling through quiescent water, the particle accelerates until the drag force equals the gravitational acceleration force, the particle will then fall at a constant speed, called the terminal velocity. This terminal velocity depends on the viscosity of the water and the shape, size and the relative density of the particle. It can be shown mathematically that the larger the particle, the most dense the particle with respect to water and the more spherical the particle; the higher will be the terminal settling velocity. This will explain that when a sedimentation tank is overloaded, it is the fine particles that are lost first over the effluent weir.

Class 1 sedimentation will occur in pre-settlement tanks where silt etc., is allowed to settle out on its own accord before chemicals are added to coagulate the remaining suspended solids.

5.1.2 The settling characteristics of flocculating particles.

The settling properties of dilute suspensions of flocculating particles differ from those of non-flocculating particles in that the flocculating properties of the suspension must be considered along with the settling characteristics of the particles. In this case, the heavier particles having a greater settling velocity overtake and coalesce with the smaller and lighter particles to form even larger particles with an increased rate of settling. The opportunity for particle-particle contact increases as the depth of the sedimentation tank increases. As a result, the removal of suspended matter depends not only on the settling rate but on depth as well.

By comparison in class 1 settling, the depth of the sedimentation tank does not play a part. The terminal settling velocity is much more difficult to determine as the strength of the floc is important in determining what the largest floc size possible is before the floc starts to break up again.

5.2 FACTORS AFFECTING SEDIMENTATION.

Among the factors affecting sedimentation are:

5.2.1 The size, shape and relative density of the floc.

The larger the floc for a given shape and relative density, the higher the sedimentation rate. The more spherical the shape, the less the drag as the particle passes through the water and hence, all other factors being equal, the greater the terminal settling velocity. The higher the relative density of the particle; the higher the sedimentation rate. This is the reason why a dense floc albeit of smaller particle size is preferred. A large feathery floc will have a relative density closer to that of the water and will also have a poor shape factor with a high drag coefficient.

5.2.2 The viscosity of the water.

The viscosity of water at 5°C is much higher than at 25°C and would exert a much higher drag force on an identical particle at the lower temperature. This is one of the reasons why activated silica is sometimes used as a coagulant aid to produce a denser floc in cold water.

5.3.3 Temperature Effects.

If the influent water is warmer than the bulk of the water in tank, it being less dense will tend to ride over the more dense colder water and vice versa. This can lead to thermal current effects that can upset the sedimentation process.

5.2.4 The effective depth of the tank.

This applies in the case of class 2 sedimentation only, provided that there is enough sludge storage volume in a tank where it is known that only class 1 sedimentation takes place. A pure class 1 sedimentation is rare.

5.2.5 The surface area of the tank.

The surface area loading rate of a sedimentation tank is the most important in determining the efficiency of the removal of suspended solids when class 1 sedimentation is present. The surface area loading is usually expressed as m^3/m^2-h, it is seen that this is dimensionally equivalent to m/h, which is a velocity. The surface loading rate may thus be equalled to the effective upward velocity of the water. If the upward velocity is too high, then the effective downward sedimentation rate, relative to a fixed point on the wall of the tank, may be so low that solids are lost out of the tank. In this case both class 1 and class 2 sedimentation processes may be affected.

5.2.6 The average retention period for sedimentation.

This is calculated by dividing the volume of the tank (usually in m^3), by the flow rate (usually in m^3/h), to give the retention period in hours. However, some of the water will pass through the tank in less than this theoretical retention period while some of the water will take more than this time. Later in section 5.6, a method will be described of how to determine the effective retention time. If the tank is of poor design, short circuiting may take place, which the result that some of the water may have a very short retention period and this can reduce the efficiency of removal of suspended solids.

5.2.7 The weir overflow rate.

If the weir overflow rate (usually measured in $m^3/m\text{-}d$, i.e. m^3 of flow per day per metre of weir length) is too high, then excessive upward velocities can occur near the overflow weir which, if they exceed the sedimentation rate of the floc, draw floc up and over the weir.

5.2.8 The inlet and outlet design.

In order to have the maximum removal of suspended solids, there must be as few eddies and stray currents as possible. These eddies can resuspend settled solids, and reduce sedimentation rates by breaking up flocs. It is important that the water entering the tank be evenly distributed to reduce short-circuiting.

5.2.9 The effect of wind.

When strong winds may be present or when tanks of length or diameter greater than about 30 m are used, the wave action caused by the wind may cause surging and resuspension of settled floc. It may be necessary to build windbreaks or even to cover over the tanks.

5.2.10 Sudden increases in flow rate.

These can cause eddies and resuspension of settled floc. Careful and well planned operation of the treatment works should be able to reduce this problem.

5.3 SEDIMENTATION TANKS.

These come in 3 main formats: horizontal flow, radial flow and vertical flow. The direction of flow is not limited to any one direction, since there will be some elements of vertical flow in a horizontal flow tank although the main direction of flow will be in the horizontal direction. The three common types are shown in figure 19.

5.3.1 Horizontal Flow Sedimentation Tank.

These are usually rectangular in plan, occasionally square tanks are used, with the floor sloping towards a sludge draw-off at the inlet end. Although these tanks are called horizontal flow, the wastewater usually enters at a lower level than the invert of the outlet weir and during passage through the tank the flow is affected by eddies and currents, although its general direction is from one end of the tank to the other with the heavier solids being deposited at the inlet end and progressively lighter solids being deposited as the water flows from the inlet end to the outlet.

5.3.2 Radial Flow Sedimentation Tank.

These are circular in plan, with the floor sloping at 5 to 10% to a central sump. The inlet is at the centre and since the level at which the water enters the body of the tank is usually much lower than that of the peripheral weir, the flow has an upward as well as a radial component. Also, during its passage from the central inlet to the peripheral outlet the velocity of the water is constantly decreasing, reaching a minimum at the outlet weir.

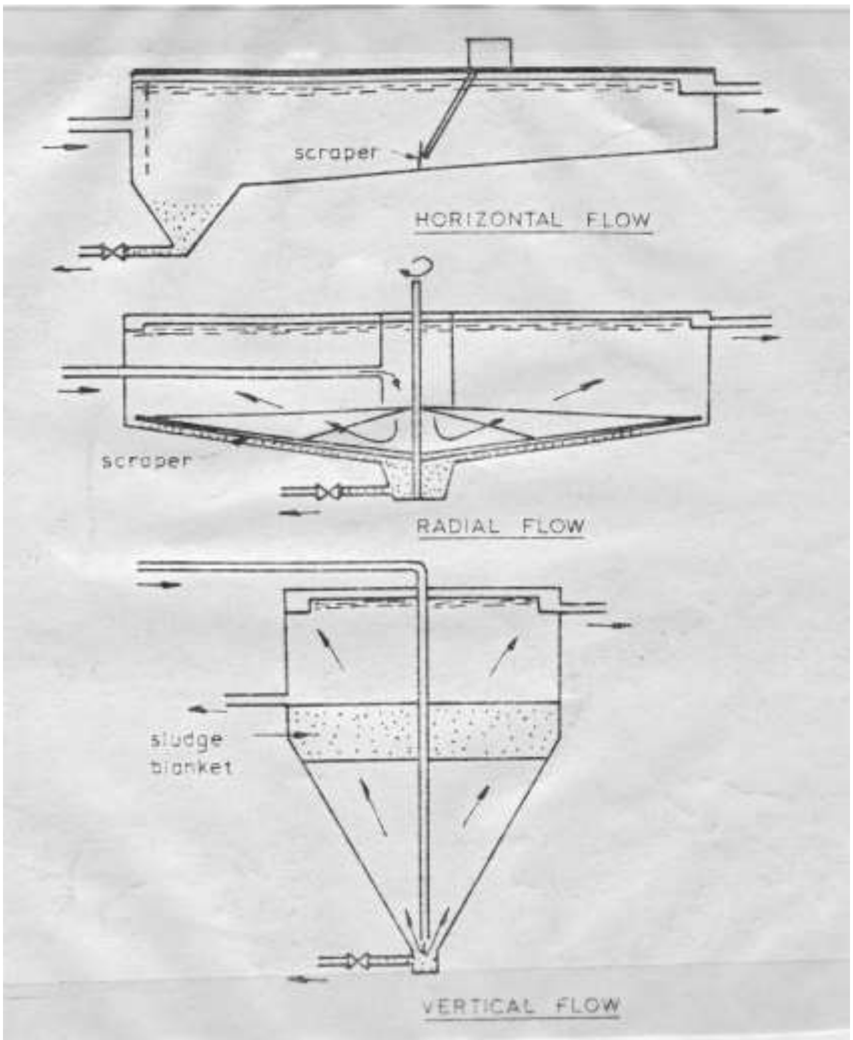


Figure 19 – THREE COMMON FORMS OF SEDIMENTATION TANK.

5.3.3 Upward Flow Sedimentation Tank.

These may be square or circular in plan and are basically an inverted pyramid or cone, with the floor sloping sharply to a central sump. The inlet is at the centre and the direction of movement of the water when it enters the body of the tank is downward, it then flows radially out ward and upward towards a peripheral outlet weir. As the distance below the surface of the zone in which the direction of flow changes from downward to outward and upward is relatively great compared with the horizontal distance from the central stilling box to the peripheral outlet weir, the direction of flow in the tank, although partly radial, is mainly upward, hence the term "upward flow" tank. With this type of tank, movement of the water is against the motion of the falling particles of solid matter. Consequently a greater opportunity is afforded for particles with consequent increased efficiency of settlement and a higher quality effluent.

5.4 SLUDGE REMOVAL FROM SEDIMENTATION TANKS.

In order to prevent the possible resuspension of settled floc when the depth of sludge becomes too great on the bottom of the tank, or to prevent the decomposition of organic sludges, it is necessary to frequently remove the sludge from the system and dispose of it in a safe manner. This will be covered later.

5.4.1 Sludge collection and withdrawal in mechanically desludged units.

Many years ago, sedimentation tanks were operated on a fill and draw basis. The tank was put into use for several hours, and then the flow was diverted to another tank. The supernatant was withdrawn until only the sludge layer remained. The sludge was manually pushed towards the outlet end from where it was manually removed. This very inefficient method has been replaced with continuously operated sedimentation tanks where the sludge is removed at intervals while the tank remains in use.

The sludge removal can be by manual operation of valves etc., or the process can be automated.

5.4.2 Mechanical Sludge Scraping and Withdrawal in Horizontal Flow Tanks.

Here two main types of scrapers are used:

1. Travelling bridge scrapers, where a power driven bridge spans the tank from which are supported the sludge scrapers. The speed of scraping is usually about 0.6 m/min. On the return journey the scrapers are either lifted or tilted. Formerly the bridges were rope hauled but problems occurred with crabbing. Among the modern methods used are rack and pinion drives.

2. Flight scrapers, where blades span the tank and are attached to endless chains, one on each side of the tank. The chains run on sprocket wheels, supported from the side walls of the tank, two near the floor and two just below the water surface, and the two chains are driven from a common shaft by an electric motor mounted on the tank wall at the inlet end. The scrapers move along the tank floor at about 0.6m/min and sweep the sludge into a hopper or hoppers at the inlet end. As the scraper returns along the surface it sweeps the scum to a collection point at the outlet end of the tank. The fact that scum is swept to the outlet end of the tank can be a disadvantage.

In the horizontal flow tank, sludge is withdrawn under pressure due to a differential head. When hoppers are provided, there are usually two or more each with its own sludge draw-off.

5.4.3 Mechanical sludge scraping and withdrawal in Radial Flow Tanks.

Two main types of scrapers are used:

1. Fixed bridge scrapers, where the bridge spans the tank and supports a walkway with hand railing providing access to the drive unit. The drive unit is mounted at the centre of the tank and consists of an electric motor driving a vertical shaft through a reduction gearbox. The shaft supports the two scraper arms and has a special watertight bearing at floor level.

   The feed pipe is usually laid beneath the floor to the centre of the tank and then turns upwards to discharge into a deflector drum. The electricity supply cable for the motor is carried along the bridge and this arrangement is much simpler than the power supply to a rotating-bridge scraper. With tanks up to 15 m diameter, the bridge structure us usually fabricated from channels or I-beams (RSJ's) suitably braced, whilst the lattice design is often used for tanks of larger diameter.

2. Rotating bridge scrapers, here there are two sub-types:

   1. Half bridge, extending over the radius of the tank;

   2. Full bridge, extending over the diameter of the tank.

A full-bridge scraper is usually centrally supported and one end of the bridge carries the drive motor, the other end being supported on an idler wheel. A half-bridge scraper is pivoted at the centre and either driven from the centre or by a motor on the outer end of the bridge.

The electrical supply system is more complicated than with a fixed-bridge scraper, the cable having to be laid in a duct in or under the tank floor and carried up the side of the feed pipe or in the central column. With smaller tanks, the cable may be carried overhead, being supported by a catenary wire. In either case there is a slip-ring assembly at the tank centre, with a cable to the drive motor. In the earlier design of scraper a continuous helical blade was used but this was unable to automatically adjust itself to the profile of the floor if the screed finish was inaccurate. Blade arrangements now normally used in sedimentation tanks are either:

1. a series of blades arranged in echelon at an angle of about 45° to a rotating radial arm, the

number depending on the diameter of the tank. Shown in figure 20;

2. a series of straight blade sections arranged to form a continuous volute. Shown in figure 21.

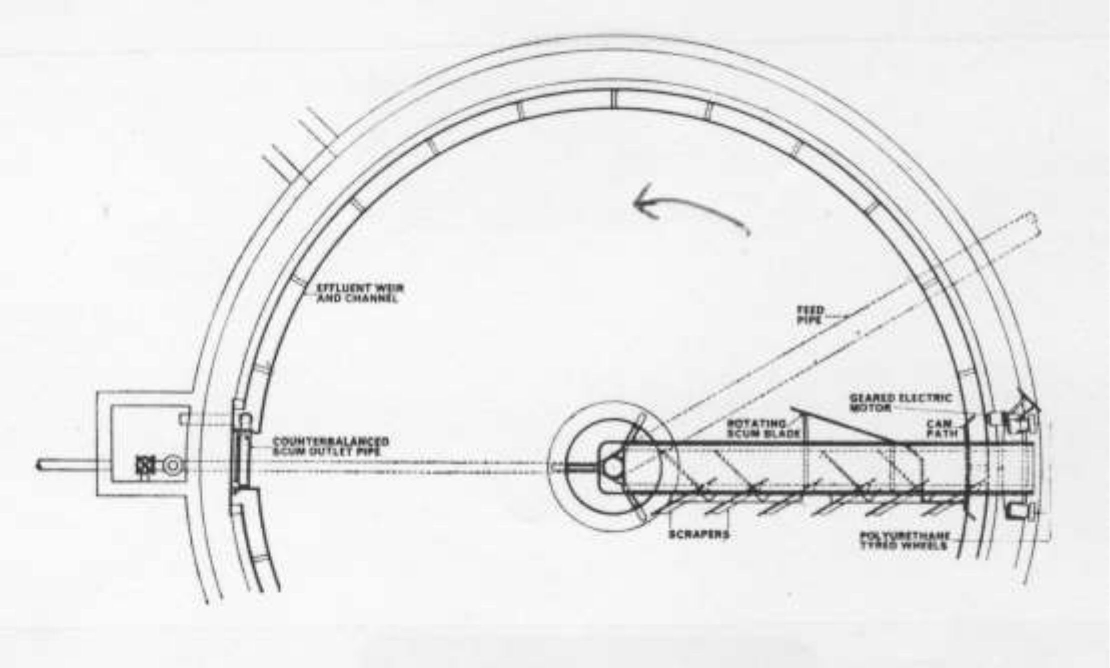


Figure 20 – SLUDGE SCRAPERS IN ECHELON LAYOUT.

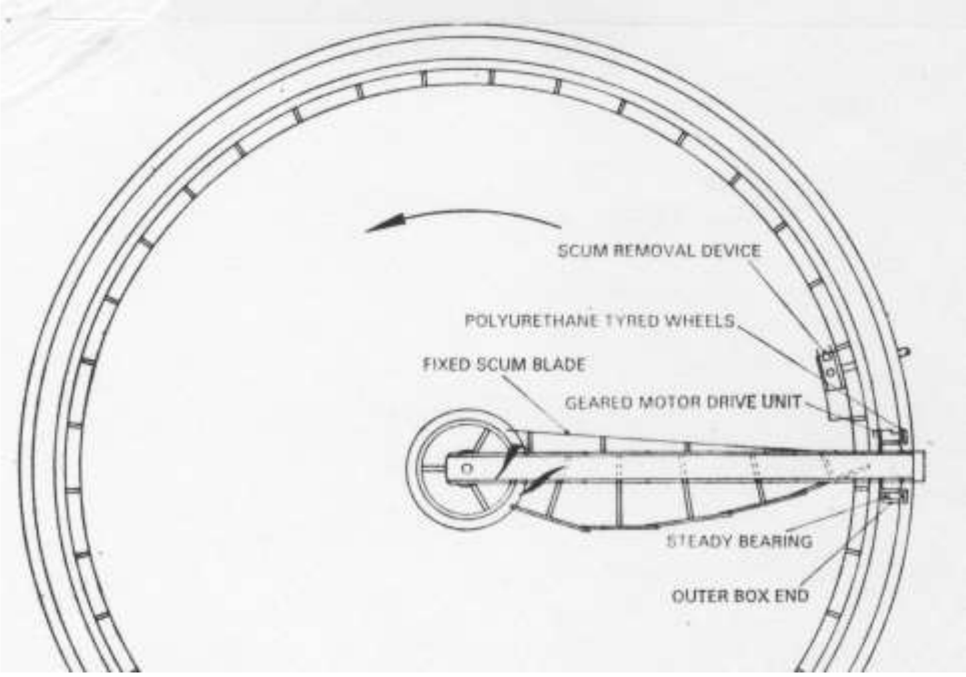


Figure 21 – SLUDGE SCRAPER AS A CONTINUOUS VOLUTE.

With blades arranged in echelon the sweeping process is more gradual since, unless the blades overlap, the unit must make as many revolutions as there are blades to convey an annulus of sludge from the periphery of the tank to the central hopper, although less load is imposed on the scraper mechanism.

Scraper blades are often attached to hinged arms and are trailing, relying on their weight or on a spring-loaded action to prevent their being lifted from the floor by an accumulation of sludge. When on a hinged arm the blade automatically adjusts itself to the profile of the tank floor.

Blades are edged with renewable rubber or rubber substitute wearing strips, some have a metallic edge. However, the weight of the trailing portion should not be such as to produce undue wear. Scraper blades are supported from a frame work carried from the bridge and if freely attached to the bridge structure they can be lifted for examination without having to empty the tank. The peripheral speed of the scraper is usually within the range of 1.8 to 2.5 m/min. An additional scraper is sometimes attached to the rotating framework carried by the bridge to maintain the cleanliness of the vertical side wall of the tank.

In the radial flow tank, sludge is withdrawn from the central hopper through a pipe running under the floor of the tank to an external inspection chamber where the rate of withdrawal can be controlled by a telescopic screw-down bell mouth weir or an adjustable straight weir.

In both horizontal flow and radial flow tanks, the sludge control valves may be hand operated; power operated by remote control or may even be partially or fully automatic.

5.4.4 Mechanical Sludge Scraping and Withdrawal in Vertical Flow Tanks.

It is usually not necessary to have scrapers in vertical flow tanks, since the sludge will slide down the steeply sloping sides, and will collect at the bottom. Sludge is withdrawn from the bottom through a pipe located inside the tank and terminating a short distance above water level. At least 1.2m, and preferably 1.5m below water level, the pipe has a horizontal branch which discharges into an inspection chamber at the side of the tank. Sludge is withdrawn through this branch, whilst the vertical extension is used as a rodding "eye". Sludge is withdrawn under pressure due to a differential head and discharges over a telescopic screw-down bell mouth weir or a swinging arm. The mouth of the sludge pipe should be 1.5 pipe diameters above the bottom of the tank to prevent blockage at this point, and if it is constructed so that it is in a vertical plane, this will facilitate both withdrawal of sludge and rodding, if this becomes necessary. The sludge pipe should be at least 150 mm diameter.

5.5 LOADING RATES.

5.5.1 Surface Loading Rate m^3/m^2-h or Upward Flow Rate m/h.

A well-formed dense floc settles at about 3 m/h. The surface loading rate at peak flow rate should not exceed 50 % of this rate i.e. about 1.5 m/h. In order to provide for variations in coagulation and flocculation effectiveness, and for abnormal hydraulic conditions, it would be advisable to limit the upper surface loading rate to between 0.75 and 1.0 m^3/m^2-h. With coagulant aids, which help to produce a more rapidly settling floc, are used, the surface loading rate may be increased to about 1.5-1.75 m^3/m^2-h. Upward flow tanks are sometimes designed with a higher surface loading rates of up to 1.5 m^3/m^2-h without coagulant aids and up to 3.0 m^3/m^2-h with coagulant aids.

5.5.2 Retention Period.

The theoretical retention period of a tank is the time required to fill the tank at the normal rate of flow. For example a 20 m diameter tank with a depth of 3 m has a volume of 942 m^3; at a flow rate of 65 l/s, the tank would take 4 hours to fill - this is the theoretical retention period when the flow rate is 65 l/s. Theoretical retention periods of 4 hours are usually used. The effective retention time is always less than the theoretical retention time. The method used to determine the effective retention time will be covered in section 5.6.

5.5.3 Weir Overflow Rate.

This is measured in m^3/m-d, i.e. m^3 per meter of weir length per day. The weir overflow rate is usually in the range of 100-250 m^3/m-d. If the weir overflow rate is too high, excessive upward velocities in the region near the weir can lift settled floc and cause it to pass over the weir.

Where weir overflow rates would be excessive if a single weir was used, a double sided weir may be used. In rectangular horizontal flow tanks, the weirs may extend a short distance down each side of the tank.

5.6 DETERMINATION OF THE EFFECTIVE RETENTION TIME.

It was stated earlier that the effective retention time is always less than the theoretical retention time. It is important to know what the effective retention time is, both in time units and as a percentage of the theoretical retention time. Although there are many methods of determining the time they all use the same basic principle of adding some chemical to the water and noting the time when it passes over the weir.

Among the chemicals that may be used are:

1. Sodium chloride, measurements may be made to measure the sodium content, the chloride content or the conductivity;

2. Lithium chloride, here a flame photometer or atomic absorption spectrophotometer would be needed;

3. A dye such as Sodium Fluorescein or Rhodamine B. Here a photometer or colourimeter would be needed;

In all cases the disposal of the effluent could be a problem. Usually the sodium chloride method is preferred, and this is the method that is described.

The principle of the test is based on the rapid addition of sodium chloride, to the influent of the sedimentation tank under study and the determination of the concentration of the salt in the effluent at frequent intervals. The first appearance of salt in the effluent represents the time of arrival of a portion of the salt-treated influent. This may occur in 10 - 20 minutes, but a portion of the salt-treated influent will be found to be leaving the tank long after the lapse of the theoretical detention period. Fluctuations in the concentration of salt in the effluent disclose erratic flow or short-circuiting portions of the influent. The salt concentration is determined by measuring the chloride or the sodium content of the influent and effluent.

In short, the procedure is adding enough salt to give the effluent a chloride content of 300 mg/L Cl^- for 1.0 minutes. If the flow is 20 L/s, then in 1 minute 20 * 60 or 1 200 litres will pass the influent reference point. The chloride content must be 300 mg/L as chloride. Using sodium chloride which contains 60% chloride, the mass of sodium chloride needed is 360/0.6 = 600 g sodium chloride. The sodium chloride must be dissolved in 20 litres of water and this solution must be poured into the effluent at a point where good mixing is obtained.

Add the 20 litres as quickly as possible. Immediately start taking samples of influent and effluent, the first influent sample must be taken at the instant that the salt is added. Calling the first sample of both influent and effluent; 0 mins, the samples should be taken at 1, 2, 5, 10, 15, 20, 30, 40, 60, 80, 100, 120, 150, 180, 210, 240, 300, 360, 480 minutes. Determine the chloride or sodium content of each sample and then plot the chloride or sodium content vs. time on linear/linear paper. Plot chloride or sodium on y-axis and time on x-axis. Draw the curve through the points representing the influent samples and the curve through the points representing the effluent samples. See figure 23.

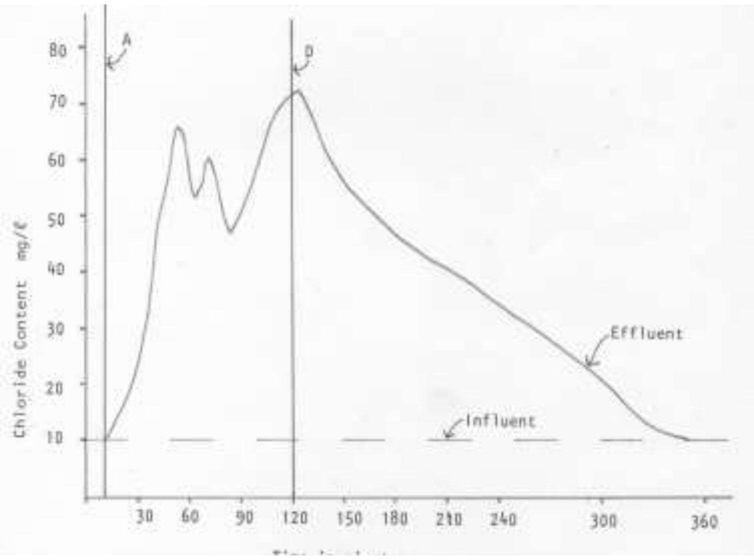
In this case the sodium chloride was added some distance upstream of the influent sampling point, this is shown by the fact that the sudden short-term increase in influent chloride took place some minutes after the sodium chloride had been added. The area between the influent and effluent curves is cut out with a pair of scissors and the centre of gravity of this area is determined by carefully balancing the piece of paper on a level knife edge so that the knife edge is parallel to the Y-axis. In this case, line D passes through the centre of gravity.

This line represents the time at which exactly 50% of the mass of sodium chloride had passed through and out of the tank. The effective detention time is then the time interval between the time when the high chloride water entered the tank (line A), and time when 50% of the chloride had left the tank (line D). Assuming that the theoretical detention time was 200 minutes, line A represented 10 minutes and line D representing 120 minutes.

$$\text{The efficiency of displacement} = \frac{\text{effective detention time} * 100}{\text{theoretical detention time}}$$

$$\text{In this example} = \frac{120-10}{200} * 100 = 55\ \%$$

Values of	5 - 30 %	represent poor tanks
	30 - 50 %	represent good tanks
	50 - 60 %	represent very good tanks
	> 60 %	represent excellent tanks.



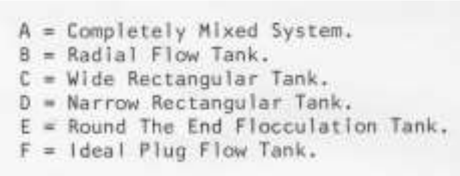

A = Completely Mixed System.
B = Radial Flow Tank.
C = Wide Rectangular Tank.
D = Narrow Rectangular Tank.
E = Round The End Flocculation Tank.
F = Ideal Plug Flow Tank.

Figure 22 – RESULTS FROM THE ABOVE TEST.

Figure 23 shows the shape of the curve that would be obtained in a perfect tank.

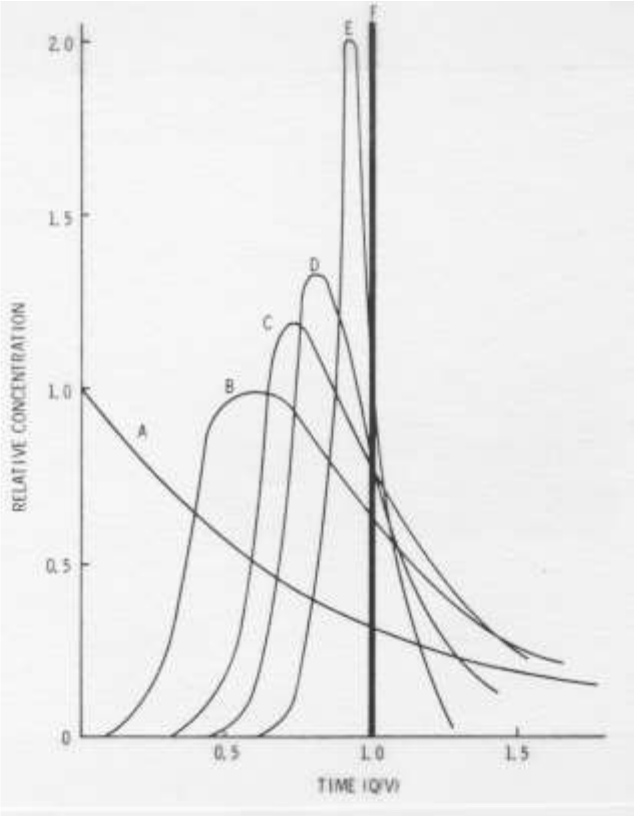


Figure 23: SHOWING COMPARATIVE RETENTION TIMES vs PERFECT TANK.

5.7 MONITORING THE EFFECTIVENESS OF A SEDIMENTATION TANK.

The effectiveness of the sedimentation process and hence of the sedimentation tank itself may be checked by determining the colour and the turbidity of the water leaving the sedimentation tank. The effluent should have no apparent or true colour and should have a turbidity of less than 10 NTU and preferably less than 5 NTU.

If the turbidity is more than 10 NTU, the sample of effluent should be allowed to stand for about 30 mins and the turbidity of the upper portion determined again. This is to determine whether the turbidity is due to resuspended material that has not been coagulated properly in the first place. The result of this determination will tell one where to look for the problem. If still turbid, then the coagulation and/or flocculation stage is not operating optimally.

WATER SOURCES and WATER TREATMENT.

PART 6

6. FILTRATION.

In the sedimentation stage, the bulk of the impurities that were rendered settleable by the coagulation and flocculation stages will have been removed, but some suspended matter still remains. The turbidity of the settled water should be between 1 and 10 NTU and at worst usually not exceeding 20 NTU. The more efficient the sedimentation process, the less the quantity of solids that has to be removed by the filtration process.

6.1 TYPES OF FILTERS.

6.1.1 Slow Sand Filters.

Slow sand filters were first used in London in 1820. They are of a more simple construction than rapid sand filters.

Among the advantages of slow sand filters are:

1. there is no need for coagulation facilities;
2. the equipment is simple;
3. suitable sand is usually readily available;
4. supervision is simple;
5. they give effective bacterial removal.

The disadvantages when compared with rapid sand filters are:

1. A large area is required;
2. they have less flexibility in operation;
3. they are not economical with raw waters having a turbidity greater than about 30 NTU, for extended periods, unless plain sedimentation can reduce the turbidity to that value;
4. they are less effective in removing colour;
5. they give poor results with water of high algal content, unless pre-treatment is used.

In general, the advantages of these filters justify their use for small plants not under technical supervision, where relatively clear surface water is to be treated.

Pre-chlorination of the water to be filtered, usually results in longer runs, this reduces the quantity of slime organisms and algal growths on the surface of the sand. Usually at least two filters are used and three are desirable when short filter runs require relatively frequent scraping.

Filtration rates are usually 2 - 5 m/d i.e. 2 - 5 m^3/m^2-d. Each filter should be fitted with a loss-of-head gauge. Automatic rate controllers provide convenient control, but the loss of head builds up so slowly with these filters that manual control is feasible, provided that a flow metering device is installed.

6.1.2 Rapid Sand Filters.

The filtration rate in rapid sand filters is between 60 and 240 metres per day with the common value of about 100 m/d (100 m^3/m^2 - d).

Among the advantages of the rapid sand filters are:

1. the much smaller area of needed to handle a given flow rate;
2. the filters can be backwashed quickly and easily when the head of loss across the sand exceeds a predetermined value;
3. as the area of sand filters is usually sufficiently small to allow the enclosing of the sand filter in a building, the problems of algae growing on the filter are reduced;
4. a multi-medium sand filter may be used with anthracite, silicon sand and garnet sand. As the anthracite has the lowest relative density (R.D. = 1.5) particles of 1.5 to 2.5 mm are used, compared with 0.5 to 1.0 mm for the silica (R.D. = 2.6) and 0.4 to 0.7mm for the garnet sand (R.D. = 4.0). Anthracite on top, silica sand next and garnet sand at bottom.

Among the disadvantages of rapid sand filters are:

1. they require frequent backwashing to remove trapped particles, usually once per day compared with about once per month for the slow sand filter;
2. there is deep penetration of the sand by the suspended material. This requires effective backwashing;
3. up to 4% of water filtered is needed for backwashing This is often wasted, but should be recovered by being returned to the head of the works;
4. the turbidity of the water fed onto the filter should not exceed 20 NTU. A maximum of 10 NTU is preferred to ensure that the filter run is more than about 18 hours. If the raw water has a turbidity of more than 20 NTU then coagulation, flocculation and sedimentation will be needed to reduce the feed water turbidity. By comparison a slow sand filter will handle a feed water of up to 30 NTU;
4. although the rapid sand filter uses coarser sand, the coefficient of uniformity should not exceed 1.5 whereas with slow sand filters the coefficient of uniformity may be up to 2.5. This means that the sand used in the rapid sand filter should have more similar sized particles. See also 6.2 below.

6.1.3 Pressure Filters.

1. Based on sand.

These are identical in principle to gravity filters, but their underdrain system, stone and sand bed are placed in horizontal or vertical steel cylinders that are designed to withstand water pressures up to about 1 000 kPa (1 MPa). The main advantage, of this system is that that filtered water is under pressure, this avoids double pumping. Chemical dosing pumps pumping coagulants into the system under pressure are needed.

Vertical pressure filters are inherently more satisfactory than horizontal units, because the area of the space above the sand is the same as that of the sand, stone and underdrains. One important disadvantage of the pressure filters is that inspection of the sand and the backwash water is not possible during backwashing.

2. Diatomaceous earth filters.

These consist of a thin coating of diatomaceous earth on a strainer element that has very fine openings to retain powder. Pressure units may be operated at pressures of 200-350 kPa.

This type of filter is used in the clarification of waters of low turbidity, when suspended solids and micro-organisms such as algae may be removed without coagulation.

These filters need to be precoated before being used. To do this about 0.75 kg of diatomaceous earth per square metre of element area is made into a slurry which is passed through the unit and recycled until the filtrate is clear. When clear the filtrate is suitable for use. Once the unit is on stream, diatomaceous earth must be added continuously to the raw water, this is called the body feed. Feed rates depend on the filtration rate and the quantity and type of turbidity, microorganisms etc. being removed. feed rates are usually in the range of 60 - 100 g/h-m^2.

When the filtration rate drops below a certain value, the precoat must be washed off and discarded. The unit is then precoated as above and the next cycle of operation is then started.

Filtration must not be interrupted; otherwise the precoat will fall off.

Among the advantages of these filters are:

1. there is a large filter area per unit of overall dimensions;

2. their mass is much less than that of a pressure sand filter of the same capacity - useful in portable units;

3. waters of less than about 30 NTU turbidity may be clarified without coagulation;

4. bacteria and cyst removals are very good.

Among the disadvantages are:

1. units are effective only if precoat is properly applied and the flow is continuous;

2. the loss of head is much higher than with sand filters;

3. diatomaceous earth is not always readily available;

4. filter runs are short if coagulated and settled water is applied because of clogging by residual floc.

A new development is the use of micro-strainers to remove microorganisms and to prolong filter runs. Strainers made of stainless steel with openings of less than 0.05 mm (50 micron) are available.

6.2 SAND GRADING.

The sand bed is the heart of the filtration process and thus the sand must be selected with great care. The efficiency of filtration, both with regard to loading rate and clarity of the filtrate depends to a great extent on the size of the grains. Although the finer sands are more effective in the filtration of water, they have a higher frictional resistance to the passage of the water and thus cannot be used economically except in slow sand filters, where the rate of filtration per unit area is about one fortieth of that obtained in the rapid sand filter. Moderately fine sand beds may be used in rapid sand filters but must be backwashed more frequently than coarse sand beds, and even then they are more difficult to maintain free from mud balls and clogged areas.

The various sizes of sand grains in a given sample are determined by passing a weighed quantity of sand through graded sieves of successively smaller mesh sizes and then weighing the portions of the original sample which pass through each successive sieve. It is usual to record the cumulative percentage passing sieve size and to plot this against sieve size on log - log paper.

The effective size of the sand is that size of grain such that 10% of the grains by mass are smaller in size and 90% by mass are larger. This may be seen on the log - log plot. This is the so-called d10 size.

Another characteristic of the sand which is very important is the uniformity of the size of the grains. The more uniform the size of the grains the better. The uniformity is indicated by the coefficient of uniformity. This is defined as the ratio of the sieve size passing 60% of the sand (the d60 size) to the sieve size passing 10% of the sand (the d10 size).

The d10 and d60 sizes need not correspond to actual sieve sizes of the British Standard, United

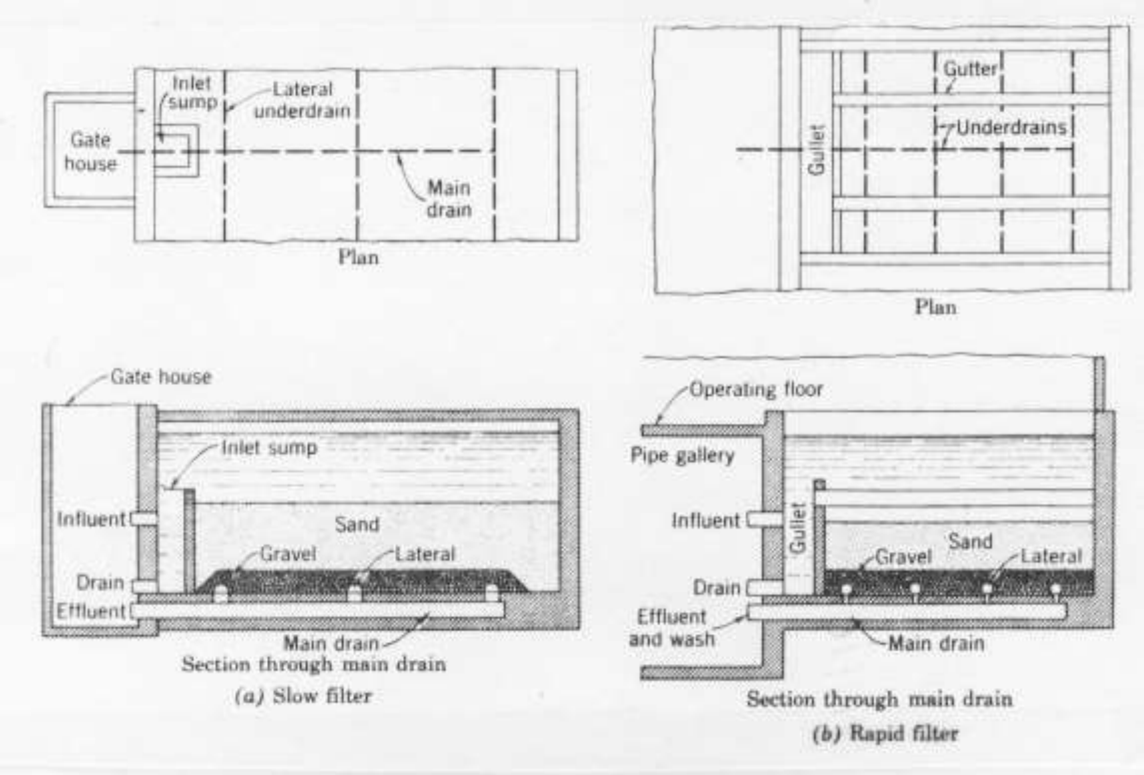
States Standard or Tyler Services Standard but are the sizes that the sieve would have to be made to pass either 10% or 60% of the sand as the case may be.

In slow sand filters, the most commonly used effective size is 0.3 mm with a range of 0.25 to 0.35 mm. In rapid sand filters, the range of effective sizes used is from 0.4 mm to 0.9 mm. With the larger effective sizes, very high flow rates will be achieved, but very close control will be required to ensure that floc-breakthrough does not occur. In slow sand filters, the coefficient of uniformity may be up to 3.0 although 2.5 is a preferred maximum value. For rapid sand filters a more uniform grain size is required, the maximum value of the coefficient of uniformity is usually set at 1.8, although 1.5 is often stated as the maximum value.

There are many areas where suitable sand cannot be purchased except by importing at great cost. In many cases, it is possible to screen out the largest particles and to remove a large proportion of the fine particles by backwashing the sand at a very high rate and either spading off the topmost stratum or deliberately allowing the fines to overflow and be lost.

6.3 CONSTRUCTION OF SAND FILTERS.

These are more easily explained by referring to drawings. Refer to figures 24 and 25.



(a) Slow filter

(b) Rapid filter

Figure 24 – PLAN AND CROSS-SECTION OF SLOW AND RAPID SAND FILTER.

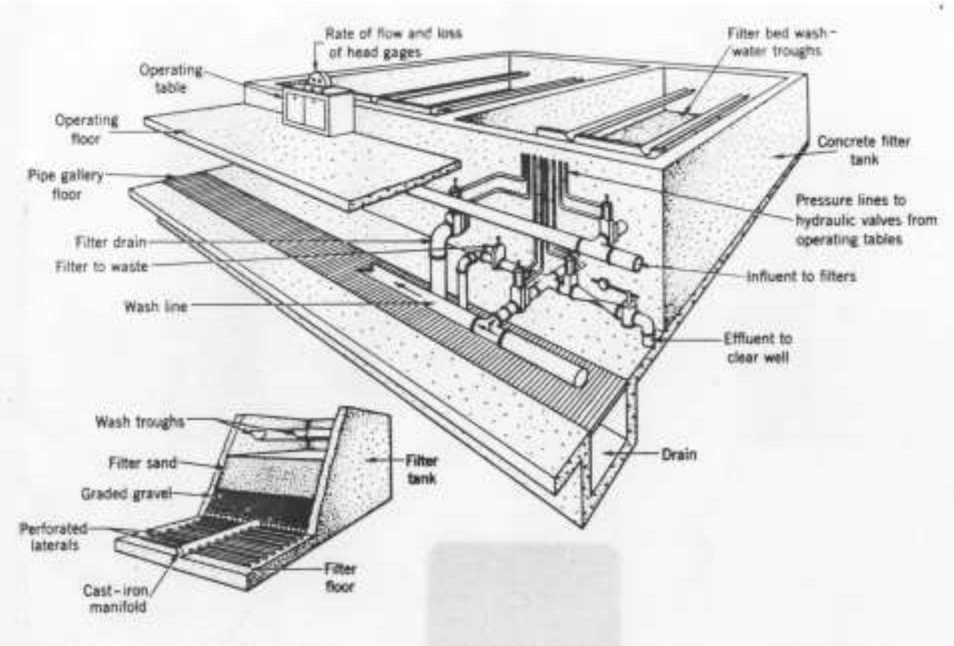


Figure 25– ISOMETRIC VIEW OF A RAPID SAND FILTER.

6.3.1 Slow sand Filters.

The depth of sand in a slow sand filter is usually in the range of 0.6 to 1.0 m. This sand is supported on layers of gravel or crushed stone. This support bed must be so graded as to prevent the penetration of sand and yet provide for the free flow of water towards the underdrains. Usually 3 or 4 layers of different sized stone are used, with the smallest stone on the top next to the sand. A typical grading is:

Bottom layer - 150 mm stone with min. size 10 mm, max. size 75 mm;

2nd layer - 50 mm stone with min. size 10 mm, max. size 25 mm;

3rd layer - 50 mm stone with min. size 5 mm, max. size 10 mm.

Sometimes a fourth layer of very coarse sand, about 2 mm grain size, is laid on top.

Each filter should be equipped with a main drain and enough lateral underdrains to collect the filtered water. Tile pipes with open joints are often used. The drains are usually laid 2 - 3 m apart, but these should not be any drain within 0.6 m of the side walls. This is to prevent water from flowing downwards in the spaces between the sand and the concrete wall and thence into the gravel without passing through the sand.

Each filter should be fitted with a loss-of-head gauge. For an illustration of the term loss-of-head, see figure 27. As the loss of head builds up slowly over a matter of days, manual control is feasible.

6.3.2 Rapid Sand Filters.

The underdrainage systems of rapid sand filters perform two primary functions:

1. they collect the filtrate - as in the case with slow sand filters;

2. they distribute the wash water to the bed during the backwash cycle.

Because the backwashing rate is much higher than the filtration rate, the size and number of underdrains is determined by the backwash rate.

The depth of sand is usually about 0.6 m. This is seen to be less than for slow sand filters. This is because sand is not lost during the cleaning cycle and because of the need to remove the impurities that have deeply penetrated the sand, during the backwash cycle.

The sand is again supported on gravel, usually several layers are used, and the total depth of the gravel is 0.4 to 0.6 m. A typical grading of the supporting gravel is:

Bottom layer	100 mm stone with min. size 38 mm, max. size 62 mm;
2nd layer	100 mm stone with min. size 20 mm, max. size 38 mm;
3rd layer	100 mm stone with min. size 12 mm, max. size 20 mm;
4th layer	75 mm stone with min. size 5 mm, max. size 10 mm.

It is possible to support the sand directly on porous filter plates or specially made nozzles. As with slow sand filters, underdrains are not installed closer than about 0.6 m from the walls, to prevent short-circuiting of the water.

It is important that most of the head loss occurs across the gravel and the openings in the underdrains and not in the underdrains themselves. This is to ensure an even filtration rate throughout the filter area and to ensure an even flow of backwash water. When air scour is used to assist the backwashing process, the air may be carried in the same conduits as the backwash water or a separate system may be used.

Loss of head indicators are important as the loss of head increases rapidly during the filtration cycle. This may be seen on a pair of piezometer tubes or may be shown by a revolving needle on a scale. The rate of filtration must be controlled especially just after backwashing. otherwise excessive filtration rates may occur and floc breakthrough could occur.

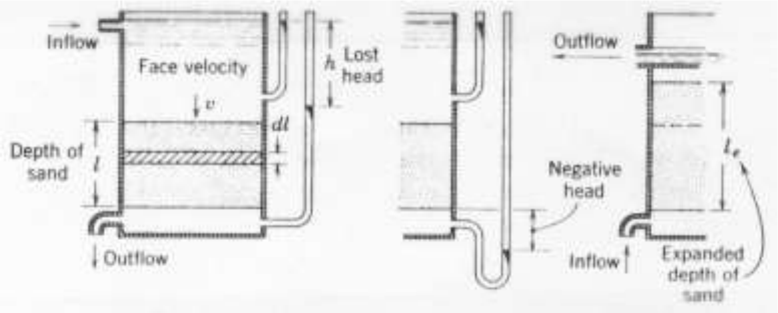


Figure 26 – HEAD LOSS IN FILTRATION AND EXPANSION IN BACKWASHING.

A typical flow rate controller is shown in figure 27.

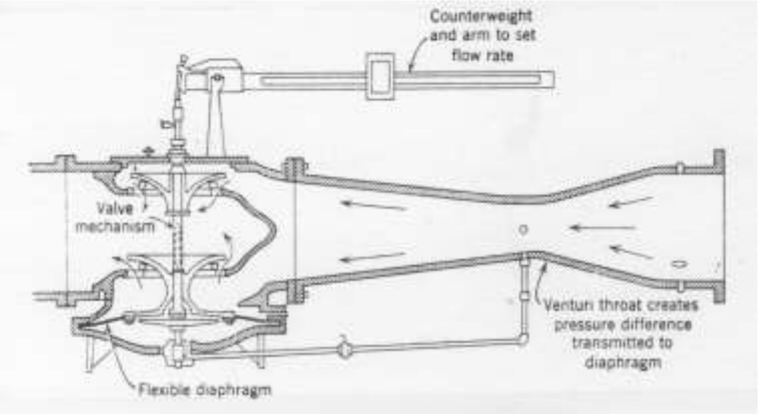


Figure 27 – A TYPICAL FLOW RATE CONTROLLER.

6.4 OPERATION OF SAND FILTERS.

6.4.1 The Slow Sand Filter.

After a slow sand filter is being returned to use, after cleaning, re-sanding or resting, filtered water should be allowed to pass into the filter via the underdrains until the water has risen to a depth of about 150 mm above the sand. The raw water may then be turned onto the filter without danger of scouring the surface by the incoming water. This is important because the slow sand filter is basically a surface filter whereas the rapid sand filter is a depth filter. If a slow sand filter is scoured then material trapped on the surface may be driven into the body of the filter, and as the filter is not backwashed, the solids may become trapped in the sand and hence reduce the flow rate or they may decompose and pass through the filter and appear in the filtered water.

The filter is then operated to filter water at low rates until a film has collected on the surface of the sand. This requires several days, so the best procedure is to increase the rate of filtration gradually during a period of four to seven days before the maximum rate is reached this conditioning period needs to be lengthened when filters containing sand are first placed in use. The filtrate from beds being conditional for the first time should be wasted or passed through a "matured" filter.

The maximum rate of filtration during normal operation should be maintained until the loss of head approaches the depth of water on the filter; this is usually 1.2 - 2 m. This loss of head should not be exceeded; otherwise the filter will be operated under a "negative head" or vacuum. This is undesirable because the film on the surface of the sand is likely to the ruptured, or the filter may become air bound through the release of gases dissolved in the water. See figure 27.

Generally speaking, the effectiveness of filtration is proportional to the depth and fineness of the filter sand and inversely proportional to the rate of filtration. With a sand depth of 0.9 m, the rate of filtration should not exceed about 3.7 m/d. (i.e. 3.7 m^3/m^2-d) otherwise some of the bacteria will pass through the filter and will be present in the effluent. The best criteria to determine the permissible flow rate, then additional filters must be brought into use in order to produce a treated water of safe sanitary quality.

When the build-up of material in the sand, causes the head loss to reach the permitted maximum, the filter must be stopped to allow the surface to be scraped. Usually 5 - 25 mm of the surface sand is removed. When this is complete, the filter may be returned to service as described above. Successive layers of sand are removed until the remaining depth of sand is about 0.8 m, after which about 150 mm of washed sand is replaced in the filters to restore the normal depth. The sand removed is either discarded or washed in special machines and re-used. When the sand is washed, it is important that the finer particles are not removed as this has the effect of increasing the effective size of the sand.

6.4.2 The Rapid Sand Filter.

The rate of filtration should be selected to provide the required volume of water with the lowest possible rate of filtration. This should be accomplished by adjusting the rate controllers of all the filters, rather than by shutting off one or more filters, when the consumption decreases.

Rate controllers should be given protective maintenance so that they function as intended. Certain types are fitted with diaphragms, which respond to pressures from Venturi meters and in turn control valves. These diaphragms develop leaks at times and then must be replaced. Follow the instruction manual accompanying the equipment.

If for some reason rate controllers are not available or cannot be made to function properly, three courses are open to the operator:

1. float-operated butterfly valves may be installed, provided that the rate of flow of the influent is constant and the flow can be equalized to all the filters;

2. the rate of flow of the influent may be equalised to all the filters, but without the use of float-operated butterfly valves. In this way, the rate of filtration equals the rate of entrance of water, but, inasmuch as a recently cleaned bed will filter at a rate in excess of the inflow, the water level drops to a point where the lowered head is equal to the loss of head. As the latter increases through partial clogging of a filter, the head of water will increase likewise, until it reaches the maximum level for the plant. Thereafter the rate of

filtration will decrease as the filter becomes more clogged, until washing is necessary;

3. an alternative procedure may be followed, allowing the rate of filtration to be controlled solely by the loss of head, without any restrictions on the rate of flow of filter influents. This means that filters will operate at a high rate when first washed, and that the rate will gradually decrease as the loss of head increases. Rates of filtration will vary between 12 m/h and less than 5 m/h.

This has the advantage of giving maximum rates higher than would be desirable at many plants, but correspondingly higher average rates. The important consideration with this third procedure is that flocculation must be satisfactory at all times, If it is, then floc penetration during high rates of filtration will be only into the upper portions of clean filters, and progressively lower rates will prevail as the penetration increases, The filters must be washed before floc "break-through" occurs. This procedure can be used only under those conditions where high-rate filtration is feasible.

If flocculation is not sufficiently effective and reliable to justify the third procedure, a compromise between the last two procedures is feasible. The influent valve is used to throttle the rate of flow of the influent to a filter, as guided by the rate of filtration indicator, so as to avoid excessive rates of filtration through recently washed filters. Eventually the influent valve will be fully open, and then the third procedure is followed. This compromise has the weakness of requiring detailed attention by Process Controllers, which may not be practicable when many filters are involved.

The second procedure is recommended when rate controllers are not available, provided that Process Controllers are able to install influent weirs to equalize flow to all filters when the total flow is reasonably uniform, as when low-lift pumps are used. One procedure is to install a 90º bend on influent pipes to serve as circular weirs, so that the level of water on the several filters may differ without diverting more water to the filters with lower levels.

In general, the use of rate controllers is advocated, as this permits complete control of the rates, within the hydraulic limitations of a plant.

The rate of filtration may be measured by closing the influent valve temporarily and then measuring the distance the water surface falls in 1 min. This measurement can be converted into the volume of water filtered in the selected time per square metre of filter area.

6.5 THE BACKWASHING OF SAND FILTERS.

It has been seen above that as the period of filtration progresses, the rate of filtration decreases and the loss of head across the filter increases. When the rate drops to too low a value or the loss of head becomes excessive, then the sand filter must be backwashed to remove the entrapped impurities.

This applies only to rapid sand filters. The earliest procedures for washing rapid sand filters consisted of one of two methods. In the first, mechanically revolving rakes were used in circular filters to agitate the sand while wash water was forced upward through the filter to separate the floc and sediment from the sand grains. In the second, compressed air was used to secure agitation. Later high velocity wash systems were developed followed by surface wash equipment.

In many plants, the old practice of backwashing at rates of 36 m/h, without air, continues to be used. The use of this "standardised" rate does not take the following factors into account:

1. the effective size of the sand;

2. the uniformity coefficient of the sand;

3. the temperature of the water.

Difficulties have occurred with mud balls, clogged areas and cracking, this has led to the use of the amount of sand expansion during backwashing as the measure of the rate of backwashing.

The current practice is to use air-water wash systems or to base the hydraulics of the tank and pipework on 50% sand expansion, so that there will be a factor of safety. On this basis, backwash rates of up to 56 m/h are used. Backwash water may be drawn from the clear well by a suitable pump as needed, or the water may be stored in a backwash water tank situated at a higher elevation that the sand filter, so that the backwashing is by gravity. In either case, only filtered water should be used for backwashing.

The air-wash system for backwashing uses compressed air to secure effective scrubbing action with a lower volume of wash water. Air may be forced into the underdrains and will then pass through the strainers with the backwash water or preferably, the air may be introduced via a separate piping system laid above the stone layers. In both cases about 1-1.5 m^3 of free air per minute is used per square metre of filter area. In both cases, backwash water rates of 20 - 50 m/h are used.

Other methods of washing filters are:

1. The surface wash system.

   This consists of many vertical pipes descending from a grid of pipe work to within 100 mm of the sand bed. Measured when the sand is at rest! The bottom ends of the vertical pipes are perforated with holes of about 2.5 mm diameter at an angle 30º below the horizontal. The vertical pipes are spaced at 600 - 750 mm centres. Water pressures of 70 - 200 kPa are used. One advantage of this system is that it may be added to an existing system to improve defective washing facilities.

2. The filter bed agitator.

   This consists of a revolving pipe, suspended at its centre by a suitable bearing, to which a number of nozzles are attached. One or more of these units are located in filter beds about 25 mm above the normal position of the sand. Wash water at a pressure of at least 270 kPa is forced through the nozzles during the washing process. The nozzles are directed downwards about 30° from the horizontal, the resulting jet action causes the arms to rotate. This system also has the advantage that it may be added to an existing system.

3. Mechanical rakes.

   These are occasionally used in circular filters. These consist of arms that rotate in a horizontal plane through the sand while the filter is being backwashed. There are effective in preventing clogged areas, mud balls etc., even when low backwash rates of 24 m/h are used.

   Filters should usually be backwashed when the loss of head reaches 2.5 - 3.0 m. Longer filter runs may be obtained when washing is delayed until the rate of filtration begins to decrease, showing that the rate controller is fully open and that the loss of head has reached a point where it is decreasing the flow below the normal rate.

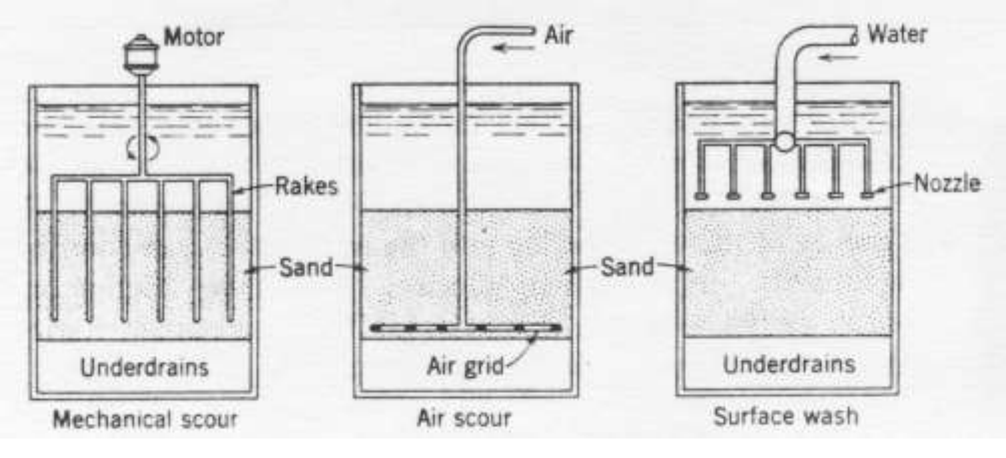


Figure 28 – ADDITIONAL METHODS TO ASSIST IN RAPID SAND FILTER CLEANING.

The procedure for backwashing a filter is as follows:

1. Close the influent valve but continue the filter in operation until the water level drops to a point about 150 mm above the sand. This reduces the quantity of settled water that is wasted and permits inspection of the filter sand;
2. Close the effluent valve;
3. Inspect the filter for mud balls, cracks, mounds or evidence that the sand is drawing away from the side walls;
4. Open the drain valve; this may be done any time after the water on the filter is below the washwater gutters;
5. Open the washwater valve <u>SLOWLY</u>; open it slowly during a period of 45 to 60 seconds to the point where sand expansion is about 15%, equivalent to a flow rate of about 24 m/h. This prevents the washwater from entering so fast that the compacted surface sand is lifted as a body until a portion cracks and causes high horizontal flow towards the crack, this flow carrying the cleaner bottom sand and the finer stone with it;
6. Wash at this low rate for about 3 minutes, to provide ample time for the sand grains to impinge on one another and dislodge the floc;
7. Open the wash water valve to the point that gives maximum sand expansion desired, usually about 40% and continue for about 1 minute or until the washwater appears to be relatively clear and the agitated sand can be seen;
8. Close the washwater valve;
9. Close the drain valve;
10. Open the influent valve slowly, to avoid turbulence and disturbing the sand;
11. When the water level reaches the normal level, open the influent valve fully;
12. Open the effluent valve.

When effective pre-treatment and good filter operation are practised, the washwater usage should amount to between 1.0 and 2.5% of the total amount of water filtered per day. When usages of more than 2.5% are found, the reason for this must be investigated.

Sand expansion is the increase in the depth of the sand bed resulting from a passage of water upward through the sand. Experience has shown that effective washing occurs when this expansion is about 40% of the depth of the quiescent bed. A normal sand depth of 0.75 m is equivalent to a depth of 1.05 - 1.12 m during the backwashing operation. Excessive expansion results in decreased scrubbing action. To determine the extent of the expansion the following simple test may be used. Seven small metal cups about 25 mm in diameter and 6 mm deep are attached to a rod at spacing of 50 mm. A cross piece is attached at right angles so that when the cross piece rests on the top of a wash water trough, the top edge of the bottom cup is 300 mm above the level of the quiescent sand. While the washing is in progress the rod is lowered until the cross piece rests on the wash water trough. After about 10 seconds, the rod is carefully withdrawn to observe the highest cup in which sand has collected. Knowing the height of this cup above the sand at rest enables one to calculate the sand expansion.

The rate of flow of wash water may be estimated, in the absence of meters, by timing the rate of rise of the wash water from a point about 150 mm below the gutter to the gutter.

6.6 <u>OPERATIONAL DIFFICULTIES.</u>

Provided the works is operated at realistic filtration rates and the turbidity or suspended solids load on the filter is not in excess of the design values, most operational difficulties can be attributed to poor backwashing of the filter.

The following details must be borne in mind in the care and supervision of the beds.

1. Note the initial loss of head when a washed filter is first placed in operation;
2. The distribution and appearance of the wash water should be observed during the washing period to see if washing is effective throughout the total area of each bed;
3. Look out for mud balls during washing;
4. Look out for mounds and craters in sand beds;
5. Look out for quiescent areas during washing;
6. Never allow water to drop from the wash water troughs directly on to the sand surface; always recharge the filter from the bottom;
7. Never allow floc breakthrough to occur;
8. Never allow a filter to operate under a "negative head" – a vacuum.

The presence of mud balls in the sand indicates incomplete washing. This may be due to insufficient rate of flow of washwater or to the use of too short a washing period. The mud balls may be removed by passing a wire mesh strainer through the sand during washing. Mesh of about 6 mm is used. Unchecked formation of mud balls will eventually result in large balls sinking through the agitated sand during the washing process and then collecting between the sand and the stone.

6.7 THE BAYLISS METHOD OF MEASURING THE VOLUME OF MUD BALLS.

A wire sieve having meshes 2.5 mm square is used. A sampling device consisting of a tube 75 mm in diameter and 150 mm, whose bottom end may be closed by a flap. After the filter has been washed in the normal manner, the water level is allowed to slowly drop to a point about 300 mm below the sand surface. Four samples of sand from different portions of the filter are taken in the top 150 mm of sand. Each sample of sand is transferred from the samples onto the sieve. The sieve is slowly agitated until water, so that only sand grains are washed through the mesh. The volume of the mud balls is determined by transferring them carefully to a partially filled graduated cylinder and noting the increase in water volume, due to the addition of the mud balls.

It has been found that most of the smaller mud balls remain near the surface of the sand bed, so if only a small percentage of mud balls is found in the top 150 mm, then the percentage of mud balls in the rest of the sand bed will probably be lower. Filter beds tested by this method have been classified by Bayliss as follows:

TABLE 8 – BAYLISS CLASSIFICATION OF FILTER BED CONDITION.

% of mud balls by volume	Condition of filter bed
0.0 to 0.1	Excellent
0.1 to 0.2	Very Good
0.2 to 0.5	Good
0.5 to 1.0	Fair
1.0 to 2.5	Fairly Bad
2.5 to 5.0	Bad
Greater than 5.0	Very Bad

Example:

A 75 mm diameter tube, 150 mm long was used to take the sample of filter sand. The volume of water in the measuring cylinder was 60 ml before the mud balls were added and 75 ml after. Determine the percentage of mud balls and the condition of the bed according to the Bayliss classification.

The volume of the sand sample taken was $\frac{D^2 \pi h}{4}$

$$= 75 * 75 * 3.142 * \frac{150}{4}$$

$$= 690 \text{ ml.}$$

The percentage volume of mud balls = $\frac{15 * 100}{690}$ = 2.17 %

According to the Bayliss classification, the condition of the filter bed is fairly bad. Cracks appear in most sand beds when the water is lowered below the surface of the sand. The existence of cracks in a sand bed under water indicates that the sand grains are being cemented together by some material in the water. These cracks are most likely to occur at the filter walls, where the sand is drawn away from the walls. The rate of flow of water increases through such cracks, and thus a heavier deposition of solids occurs at these points; this in turn intensifies the forces compacting the sand until a dense mass is formed. The degree of agitation of these dense masses during washing may be very limited, and dead areas may develop in the filter. These dead areas may be broken up by hand or raking or by draining the bed and removing the clogged sand.

Filter sand may need to be treated with chemicals at intervals to remove troublesome organic material and gelatinous floc. A 1 - 2 % sodium hydroxide solution left in the bed for 6 - 12 hours is usually effective. Fats and oils are saponified and removed as soaps; aluminium floc is dissolved and may be flushed out. Chlorine solutions containing 20 - 50 mg/L chlorine left in the bed for 12 - 24 hours are effective in removing the slime producing organisms and algae. Sodium chloride solutions of 1 - 2 % left in the bed for 24 h may be used to remove slime growths, but this method does not remove mineral deposits or floc from the sand. In all cases, the sand must be thoroughly backwashed to remove the chemical used and any organic material etc. released from the sand.

6.8 THE DETECTION OF FLOC BREAK-THROUGH.

Filtered water may be clear and yet contain discreet particles of floc that pass through the sand i.e. floc break-through has occurred. A floc detector may be used to determine if there is floc in the treated water and the Bayliss cotton plug filter may be used to determine how much floc has passed through.

A home-made floc detector is shown in figure 29.

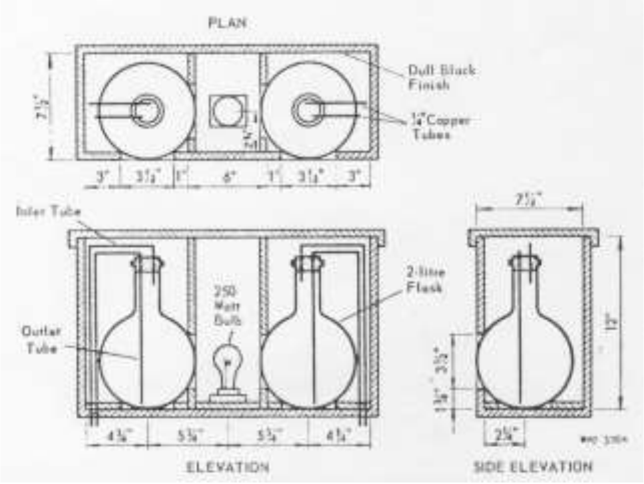


Figure 29– A HOME MADE FLOC DETECTOR.

The box is painted dull black on the inside to reduce reflected light. Floc is detected by the Tyndall effect. *(The scattering of light as a light beam passes with a medium with small particles in suspension. Also seen with dust in air.)*

The cotton-plug filter was developed by Bayliss as a means of collecting floc particles that have passed through a filter, so that their weight can be determined and expressed as turbidity units in terms of the volume of filtered water allowed to pass through the cotton plug during a specific time. At the end of the time, the plug is removed and ashed, the mass of ash represents the mass of inorganic material passing through the filter.

A sketch of the Bayliss cotton plug filter is shown in figure 30.

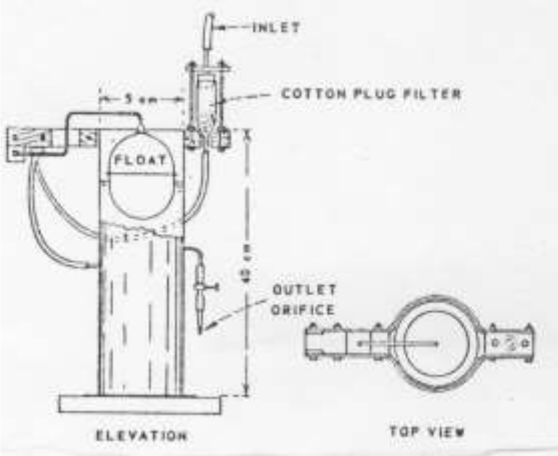


Figure 30 – THE BAYLISS COTTON PLUG FILTER.

The usual period used is 7 days and the usual flow rate used is 40 ml/min. The total volume of water passed is then 403.2 litres (may be taken as 400 litres). The floc content of the water is determined as mg/L. For low values 1 mg/L floc = 1.0 units of turbidity. The values for turbidity determined by this method should be less than 1.0 units and preferably less than 0.5 units.

WATER SOURCES and WATER TREATMENT.

PART 7.

CHLORINATION AND DISINFECTION PROCESSES.

7.1 PRINCIPLES OF CHLORINATION.

The disinfection of potable water is almost universally accomplished by the use of gaseous chlorine, or chlorine compounds, because of the limitation of other procedures.

The object of chlorination of potable waters is the destruction of bacteria through the germicidal effects of chlorine. There are several important secondary uses for chlorination, such as the oxidation of iron, manganese and hydrogen sulphide, the destruction of some taste and odour-producing compounds, the control of algae and slime organisms and as an aid to coagulation.

The problem of effective chlorination is to ensure:

1. uniform application of chlorine to all portions of the water being treated;
2. uninterrupted application of chlorine;
3. the selection of the dose of chlorine to meet the current needs of the specific water being treated.

Natural waters consist of a complex solution of many substances, some of which influence chlorination amongst these are:

1. suspended solids may shield bacteria from the action of the chlorine;
2. organic matter may react with chlorine so that it has only weak disinfecting properties;
3. ammonia reacts with chlorine to form chloramines or combined residual chlorine which have much lower disinfecting properties than free residual chlorine;
4. water having low alkalinity and pH values of less than 7.2 is more easily disinfected than water with a high pH value;
5. nitrites react with chlorine and may also produce a false colour with the reagent normally used to detect residual chlorine namely ortho-toluidine. The ortho-tolidine-arsenite test may be used overcome this interference;
6. Manganese interferes with the ortho-tolidine test but the ortho-tolidine - arsenite test may be used, ferric iron also interferes.

The rapidity of disinfection with chlorine is proportional to the temperature of the water, but at lower temperatures chlorine is more stable and will remain in the water for a longer period. The two effects compensate to some degree.

The period available for the interaction between chlorine and the constituents of the water is most important. The absolute minimum period should be 15 minutes but preferably several hours is needed so that effective disinfection may be ensured without as undesirably high concentration of residual chlorine reaching the consumer.

7.2 THE REACTION OF CHLORINE WITH VARIOUS CHEMICALS IN THE WATER.

Chlorine is a powerful oxidizing agent and as such will oxidize a wide variety of both inorganic and

organic substances in water. The degree of oxidation will depend on many factors such as temperature, concentration of chlorine, contact period etc.
When chlorine dissolves in water it forms hypochlorous acid (or hypo-chlorite ion) depending on the pH of the solution and hydrochloric acid:

$$Cl_2 + H_2O \rightarrow HClO + H^+ + Cl^-$$

The hypochlorous acid is unstable and breaks up to yield hydrochloric acid and oxygen. It is this oxygen that is responsible for most oxidation reactions. In the disinfection process, it is the hypochlorous acid that is the agent that kills the bacteria. The hypochlorous ion is less effective.

Chlorine reacts with ammonia and ammonium compounds in water to form firstly monochloramines, then dichloramines and finally trichloramine or nitrogen trichloride.

$NH_4^+ + HOCl$	$\rightarrow$	$NH_2Cl + H^+ + H_20$	monochloramine
$NH_2Cl + HOCl$	$\rightarrow$	$NHCl_2 + H_20$	dichloramine
$NHCl_2 + HOCl$	$\rightarrow$	$NCl_3 + H_20$	trichloramine (nitrogen trichloride)

The relationship between the amounts of the three types of chloramine depends on the pH and the ammonia concentration of the water. As trichloramine can only form at very low pH values, the other two prevail in water treatment; dichloramine is a more powerful bactericide than monochloramine. However, it is much less powerful than either hypochlorous acid or the hypochlorous ion.

7.3 RESIDUAL CHLORINE AND BREAK POINT CHLORINATION.

It has been found that small doses of chlorine used for disinfection resulted in the formation of combined residual chlorine i.e. the chlorine combined with substances such as ammonia to form chloramines, which have a slow disinfecting action. However, the combined-residual chlorine was found to be quite stable and did not react with some taste- and odour- producing compounds. Some authorities deliberately added ammonia to the water to ensure the production of chloramines. It has been found that the poor disinfection property of chloramines is a serious disadvantage, unless very long retention times were available and unless waters of low alkalinity were being treated. The modern trend has been to use higher doses of chlorine to maintain a residual chlorine content for longer periods. The term "super chlorination" was formerly used to signify heavier than usual chlorine doses, but this has now been replaced by the terms "break-point chlorination" or "free-residual chlorination".

The principle of free-residual chlorination is to add sufficient chlorine to oxidize the organic matter, iron, and manganese in the water being treated and also to oxidize free ammonia in the raw water, so that the remaining residual chlorine will be present as the active free-residual chlorine rather than the less active combined residual chlorine or chloramines. Refer to figure 31 for a graphical representation of the following discussion.

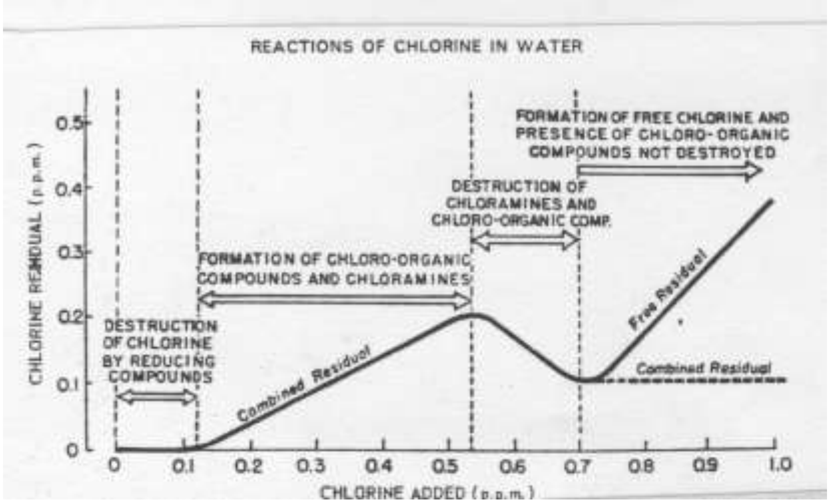


Figure 31 – REACTIONS OF CHLORINE IN WATER

It will be noted that there are four zones distinguishing four ranges in chlorine doses in which different conditions prevail and different reactions occur. The chlorine doses given here are for illustration only, and present possible doses in a high quality water.

1. The first zone, corresponding to chlorine doses of NIL to about 0.12 mg/L represents a condition in which all the chlorine is removed by reaction with organic matter so that no residual chlorine remains. No disinfection would occur in the absence of residual chlorine;

2. The second zone, corresponding to chlorine doses of about 0.12 mg/L to about 0.53 mg/L, represents a condition in which chlorine has reacted with ammonia to form chloramines and with certain organic compounds to form chloro-organic compounds. This zone represents earlier chlorination practice, in which residual chlorine contents of 0.1 to 0.2 mg/L were considered sufficient, regardless of whether the residual was combined chlorine or free chlorine;

3. The third zone, with chlorine doses of about 0.53 mg/L to about 0.7 mg/L, represents the break point at which these higher doses of chlorine lead to increased chemical activity, which in turn leads to the oxidation of the chloramines and the chloro-organic compounds. This activity utilizes the chlorine to produce compounds that do not react with ortho-tolidine reagent. This means that the higher chlorine doses lead to a greater degree of utilization of chlorine to oxidize the compounds; this in turn increases the removal of chlorine, so that higher doses of chlorine produce lower residual chlorine contents;

4. The fourth zone represents the conditions that prevail when the reactions in the third zone have been completed; any further increase in chlorine dose produces an increase in free-residual chlorine. A small concentration of combined-residual chlorine may remain as certain chloro-organic compounds are not destroyed by free-residual chlorine.

In the above example, it may be seen that a chlorine dose of 0.53 mg/L is needed to produce a combined-residual chlorine content of 0.2 mg/L whereas a chlorine dose of 0.8 mg/L is needed to produce a free-residual chlorine content of 0.2 mg/L.

Free-residual chlorine may be present in various forms depending on the pH value.

1. at very low pH values, i.e. less than about pH 4, some of the free residual chlorine is present as dissolved molecular chlorine;

2. At about pH 6 about 95 % of the free residual chlorine is present as HOCl (hypochlorous acid);

3. At about pH 8, some 85 % of the free-residual chlorine is present as OCl^{-} (hypochlorous ion) also known as hypochlorite ion.

This may be seen in graphical form in figure 32.

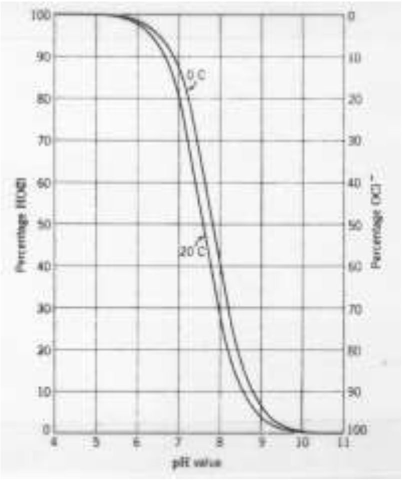


Figure 32 – CHLORINE SPECIES AS A FUNCTION OF pH VALUE.

The reactions are:

1. $Cl_2 + H_2O \rightarrow Cl_2$ aqueous

2. $Cl_2 + H_2O \rightarrow HOCl + HCl$

3. $Cl_2 + H_2O \rightarrow H^+ + OCl^- + HCl$

In the pH range usually found in treated waters, the free-residual chlorine is present as a mixture of hypochlorous acid and hypochlorous ion.

It is very important to note that hypochlorous acid is a more active disinfectant than the hypochlorous ion. The higher the pH value of the water being treated, the lower the percentage of hypochlorous acid present, with the result that higher chlorine residuals or longer retention times are needed to secure equivalent disinfection. This is highlighted in Table 9 below.

TABLE 9 - RECOMMENDED MINIMUM CONCENTRATIONS OF FREE-RESIDUAL CHLORINE OR COMBINED-RESIDUAL CHLORINE TO ENSURE EFFECTIVE DISINFECTION.

pH value	Free-residual Chlorine (Contact time 10 mins)	Combined-residual Chlorine (Contact time 60 mins)
Up to 7.0	0.2 mg/L	1.0 mg/L
7.0 to 8.0	0.2 mg/L	1.5 mg/L
8.0 to 9.0	0.4 mg/L	1.8 mg/L
9.0 to 10.0	0.8 mg/L	Not recommended
Greater than 10.0	0.8 mg/L	Not recommended

The limitations of treatment by chlorination alone should be considered, to avoid undue reliance on this process alone when other types of treatment also are required. Generally speaking, treatment by chlorination without filtration is effective and adequate only when:

1. the degree of bacteriological pollution is moderate and reasonably uniform, and the bacteria to be destroyed are not shielded from the chlorine by being bedded in suspended solids or within the bodies of worms, etc.;

2. the turbidity of the water does not exceed 5 - 10 units;

3. the colour of the water does not exceed 5 - 10 units;

4. the content of iron and/or manganese in the water does not exceed 0.3 mg/L;

5. the chlorine demand of the water does not fluctuate so rapidly as to prevent proper adjustment of the chlorine dose;

6. the taste and odour producing substances are absent or do not interfere with the selection of adequate chlorine doses through the production of chlorine tastes;

7. there is a contact period of at least 15 minutes between the point of chlorination and the first draw off point for domestic consumption.

7.4 CHLORINE DEMAND.

The chlorine demand of water is the amount of chlorine which is needed to react with the impurities in the water. It is the difference between the amount of chlorine added and the amount of residual chlorine remaining after a given time. The demand of water varies with the amount of chlorine added, the contact time, the temperature and to a lesser extent the pH value of the water. As larger amounts of chlorine are added, the chlorine demand of the water increases because the chemical reactions become more active and a wide number of reactions become possible.

There is no standardized test for chlorine demand. A residual of 0.2 mg/L after a 20 minute contact time has been offered as a standard.

7.5 THE VARIOUS CHEMICALS USED FOR CHLORINATION.

7.5.1 Liquefied Gas Chlorine.

This is gaseous chlorine that has been liquefied under pressure and is stored in pressure cylinders. The pressure at which chlorine liquefies varies from 266 kPa at 0°C to 4 100 kPa at 100°C. The cylinders are filled to a level so that at 65°C the liquid chlorine occupies 80% of the volume of the cylinder and gaseous chlorine 20%. The cylinders are fitted with fuse plugs for the release of gas when the temperatures exceed 65°C. It is thus very important that chlorine cylinders are not stored near steam pipes, boilers etc.

Dry chlorine gas is not corrosive and may be transported in iron cylinders or piped in iron pipes. When moist, chlorine gas becomes very corrosive due to the formation of hydrochloric acid and hypochlorous acid.

CHLORINE GAS IS VERY POISONOUS! It is 2.5 times as dense as air and will sink to the floor when leakage occurs. The rooms used for storing chlorine and the chlorinator must have sealed walls and an outward opening door. The door must have air ducts near the floor level and air vents near the top. It is recommended that at the larger installation a fan be installed to blow air into the room near the ceiling, with the usual vent at the bottom of the door.

Gas chlorinators consist of pressure-reducing valves actuated by metal diaphragms or hydraulically operated floats and orifices for measuring the rate of flow of gas after it has been reduced to a uniformly low pressure, and a device for injecting the gas into the water. Chlorine gas may be piped in tubes through a non-return valve directly into the water in which the tubing is led either to a perforated solid silver diffuser inserted into the water pipe or to a carborundum diffuser inserted into the pipe. These are the so called "dry feeders" which are designed for use where high pressure water is not available; they can be used at points where the water pressure in the pipes is less than 175 kPa.

During cold weather "chlorine ice" may form at the diffuser where the gas comes in contact with the water. Heating the diffuser may overcome this problem.

In general, the solution feed chlorinator is preferable. This also has a pressure reducing valve and a gas flow measuring device, often a rotameter, and has an injector that dissolves the chlorine and injects the solution into the water pipe.

This system needs water at a pressure of 3 times that of the water being chlorinated; often a booster pump is used.

The usual gas cylinders used in this country are 68 kg or 900 kg. The maximum draw off rate from a 68 kg cylinder is about 20 kg per day. A higher draw off rate will cause an excessive drop in temperature.

Corrosion of the chlorinator may be prevented by covering all exterior parts with a suitable grease such as petroleum jelly. The location of a leak can be readily detected by removing the stopper of a bottle of ammonia solution and holding the neck of the bottle near the various parts of the apparatus - if chlorine is escaping it combines with the ammonia fumes to provide a white clearly visible smoke of ammonium chloride:

$$Cl_2 + H_2O + NH_3 \rightarrow NH_4Cl + HOCl$$

The supply of gas to a chlorinator must be shut off at the cylinder and the equipment purged of chlorine through the vent before repairs are attempted. **Each time a cylinder is connected up a new lead washer must be used.**

Chlorine should be applied to the water at a point where all the water being treated comes in contact with the chlorine. Chlorine gas should not be applied at a depth of less than 1.2 m in open conduits and flumes, as it will not be completely absorbed.

As individual chlorinators vary, the manufacturer's instruction manual must be studied before the unit is operated.

7.5.2 Sodium Hypochlorite solution.

Sodium hypochlorite solution usually contains 3 - 15% available chlorine by mass. This solution stored in a dark, cool place is reasonably stable. Its main advantage is that it is free from suspended solids, but does have a limited shelf life. The concentrated solution is usually diluted as needed, to a strength of 1% (10 000 mg/L) available chlorine for more convenient application with solution-feed equipment.

7.5.3 Calcium Hypochlorite Granules.

This is a modern product considerably superior to chloride of lime. It usually contains about 90% available chlorine. It deteriorates more slowly than chloride of lime, particularly in hot, humid conditions. It is reasonably soluble in water leaving some suspended matter.

7.5.4 Chloride of Lime.

This has generally been replaced by Calcium Hypochlorite granules. The powder is unstable and decomposes when moist liberating chlorine. The fresh powder contains 25 - 37% available chlorine by mass, but is usually taken as 30%. Chloride of lime contains excess lime which is insoluble and thus solutions of this material contain suspended solids. It is usual to mix up the required solution in one container, allow the solids to settle and then to transfer the clear supernatant to the solution feed storage tank. Again a solution containing about 1% m/v available chlorine is usually used.

Carbon dioxide in the air reacts with calcium hypochlorite or chloride of lime solutions to precipitate the calcium as calcium carbonate. This may be avoided by deliberately precipitating all the calcium by adding a stoichiometric quantity of sodium carbonate.

$$Ca(OCl_2) + Na_2CO_3 \rightarrow CaCO_3 + 2\ NaClO$$

A method of avoiding a deposit of sludge when preparing solutions of calcium hypochlorite with hard waters is to add a sequestering agent to prevent the precipitation of calcium compounds. Tetra-sodium pyrophosphate, $Na_4P_2O_7$, is often used for this purpose. The dose is 660 mg of $Na_4P_2O_7$ per 100 litres of solution for each mg/L of hardness as $CaCO_3$.

7.6 OTHER DISINFECTION PROCESSES.

7.6.1 Ozonation.

Ozone (O_3) which is an allotrope of oxygen is a very powerful disinfectant.

Among the advantages of this method are:

1. no taste is imparted to the water, as a result of reaction with chemicals present in the water;
2. some organo chlorinated chemicals are oxidised;
3. viruses are inactivated.

Among the disadvantages of this method are:

1. ozone cannot be stored and has to be produced as required;
2. Ii is a costly process requiring skilled operation and maintenance;
3. the ozone does not have any residual effect;
4. ozone is very corrosive and attacks most metals, only glass and certain grades of stainless steel may be used in contact with the gas.

7.6.2 Chloride Dioxide ClO_2.

This is a very powerful oxidizing agent. It is prepared by reacting chlorine with sodium chlorite.

Among the advantages of this method are:

1. no taste is imparted to the water as a result of reaction with chemicals in the water;
2. phenols are oxidized completely.

Among the disadvantages of this method are:

1. the chlorine dioxide cannot be stored and has to be prepared as required;
2. only low concentrations of chlorine dioxide can be used since chlorine will remain in the reaction mixture and will pass into the water being treated;
3. the residual sodium chlorite has no disinfecting effect.

7.6.3 Ultra Violet Rays.

Ultra violet rays are an effective disinfecting agent and also kill large numbers of viruses, provided they are applied to a thin stream of water of the order of millimetres and at sufficient power, and provided also that the lamps are changed about every year as their output diminishes with time. The water must be clear, colourless and free from turbidity; it must contain no iron, organic colloids or planktonic microorganisms likely to form deposits on the pipes, with a consequent considerable reduction in radiation.

If these conditions are fulfilled, all living cells exposed to the UV rays die or at least are no longer capable of reproducing or acting on the surrounding medium.

To guarantee reliability, the unit must be large, well-monitored and well-maintained, and operated with water of constant quality throughout the year.

A major disadvantage of this process is the lack of a residual disinfection effect and also any contamination of the water downstream will allow bacteria to grow again.

7.6.4 Irradiation with Atomic Particles.

Investigations have been carried out into the use of high energy electric beams for the disinfection of water. Some work has also been done into these of gamma rays. While these processes are very effective, they have the disadvantage of being expensive to install and need sophisticated control.

WATER SOURCES and WATER TREATMENT.

PART 1.

ADVANCED WATER TREATMENT.

8.1. WATER SOFTENING.

Hardness may be defined as the property of certain waters which effects the lathering of soap. The less soap it takes to produce a lather, the softer the water while conversely the harder the water, the more soap it takes to produce a lather. The salts of calcium and magnesium react with the soap and until they have been neutralised, a lather cannot form. Hardness can also be caused by salts of iron, aluminium, strontium and manganese, but these elements are present usually in low concentrations by comparison.

There are two kinds of hardness - temporary and permanent. Temporary hardness is caused by the carbonate $CO_3^{=}$ and bicarbonate (or hydrogen carbonate) HCO_3^{-} salts of the hardness causing minerals mentioned above. This is also called carbonate hardness. Calcium and magnesium carbonate are only very slightly soluble in water, but calcium and magnesium bicarbonate are very much more soluble.

When water containing either or both of the latter compounds is heated, carbon dioxide is given off and the carbonate precipitates out:

$$M(HCO_3)_2 \rightarrow CO_2 + MCO_3 + H_2O$$

M = Calcium or Magnesium

This precipitate (scale) is the one that is seen inside kettles, etc.

The permanent or non-carbonate hardness is caused by the sulphates and chlorides of calcium or magnesium. This form of hardness cannot be removed by boiling. Total hardness is the sum of the carbonate and non-carbonate hardness.

The terms hard water and soft water are relative. Persons used to water with 35 mg/L hardness, will think that water with 125 mg/L hardness is very hard; while those used to 250 mg/L hardness would consider the 125 mg/L hardness water as relatively soft. In general, water having up to 150 mg/L of total hardness is not sufficiently hard to interfere seriously with most domestic uses. Hardness of more than 150 mg/L will be noticed by most consumers. Both the SABS and WHO have a recommended maximum hardness of 200 mg/L as calcium carbonate.

In general, one can say that water having a hardness of less than 50 mg/L is soft; 50 - 150 mg/L is moderately hard; 150 - 300 mg/L is hard; more than 300 mg/L is very hard.

In all cases hardness is expressed as mg/L as calcium carbonate regardless of what form the calcium or magnesium is present. This is done for convenience and in calculations - see later.

The following conversion factors may be used to express the concentrations of hardness producing compounds.

TABLE 10 – EXPRESSING VARIOUS CHEMICALS AS CALCIUM CARBONATE EQUIIVALENT.

1 mg/L of calcium bicarbonate	0.62 mg/L as $CaCO_3$
1 mg/L of calcium sulphate	0.74 mg/L as $CaCO_3$
1 mg/L of calcium chloride	0.90 mg/L as $CaCO_3$
1 mg/L of calcium ion	2.40 mg/L as $CaCO_3$
1 mg/L of magnesium carbonate	1.19 mg/L as $CaCO_3$
1 mg/L of magnesium bicarbonate	0.68 mg/L as $CaCO_3$
1 mg/L of magnesium sulphate	0.83 mg/L as $CaCO_3$
1 mg/L of magnesium chloride	1.05 mg/L as $CaCO_3$
1 mg/L of magnesium ion	4.17 mg/L as $CaCO_3$

Since the wider use of detergents which contain anti-deposition agents, the disadvantages of hard water for domestic washing have become less important. For certain industrial processes, a soft water is required. Softening may also be required when the only available source of water is excessively hard.

There are two basic methods of softening water are by precipitation using the so called lime-soda softening process and by ion exchange.

8.2 LIME-SODA SOFTENING.

8.2.1 Introduction.

This process of softening depends on the use of calcium hydroxide (lime), if necessary together with sodium carbonate (soda ash) to change soluble calcium and magnesium compounds into the sparingly soluble calcium and magnesium salts, which are allowed to settle out.

The actual reactions are complex but may be illustrated by the following:

1. $CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O$
2. $Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2\ CaCO_3 + 2H_2O$
3. $Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow CaCO_3 + MgCO_3 + 2H_2O$
4. $MgCO_3 + Ca(OH)_2 \rightarrow CaCO_3 + Mg(OH)_2$
5. $MgSO_4 + Ca(OH)_2 \rightarrow Mg(OH)_2 + CaSO_4$
6. $CaSO_4 + Na_2CO_3 \rightarrow CaCO_3 + NaSO_4$

NOTE:

1. In reaction 1, carbon dioxide is not hardness forming, but is removed by the lime and thus has an influence on the lime dose required.
2. The magnesium carbonate produced in reaction 3 is not sufficiently insoluble for effective removal and must be reacted with additional lime, as in reaction 4 to form magnesium hydroxide.
3. The calcium sulphate produced in reaction 5 is reasonably soluble and must be changed into calcium carbonate using sodium carbonate as in reaction 6.
4. Calcium sulphate in raw water is removed as per reaction 6.

It may be seen that temporary or carbonate hardness may be removed by using lime and that permanent or non-carbonate hardness may be removed using soda ash.

Although calcium carbonate is theoretically soluble to the extent of 17 mg/L at pH 9.4; treatment with lime and soda ash will not result in the precipitation of all but 17 mg/L of calcium carbonate. Usually about 35 mg/L of calcium carbonate remains. Precipitation to this level can be delayed at times.

If the magnesium content and non-carbonate hardness is low enough, then only carbonate hardness has to be removed. In this case, only lime would be added to raise the pH to the optimum value of pH 9.4. How to determine the required dosage will be covered later. Soda ash can be dispensed with also when only calcium and magnesium bicarbonates are present, but in this case lime is added to produce a pH of about 10.6 (depending on the residual magnesium content desired).

Lime-soda softening is a complex process that needs constant monitoring particularly if the quality of the input water varies widely or if the flow rate through the treatment works varies widely.

8.2.2 Determining the Chemical Dosage.

There are two methods of determining the chemical dosage required to reduce the hardness to the desired value;

1. the trial and error method;
2. the scientific method using the Caldwell - Lawrence conditioning diagram.

8.2.3 The Trial and Error Method.

This method uses factors to estimate the dose of the lime and soda ash to reduce or remove certain parameters. In order to soften a water using the trial-and-error method, the following parameters must be known of the water to be softened:

1. the free carbon dioxide content;
2. the pH value;
3. the bicarbonate alkalinity;
4. the total hardness;
5. the non-carbonate hardness;
6. the magnesium content.

The methods for determining these parameters will be covered later. It will be seen that the required reduction in bicarbonate alkalinity, the carbon dioxide content and the magnesium content determines the lime dose and the required reduction in non-carbonate hardness determines the soda ash dose. The dose of carbon dioxide needed for recarbonation is determined from the results obtained from the marble test for calcium carbonate equilibrium.

It has been seen that either hydrated lime $Ca(OH)_2$ or quicklime CaO may be used in water softening. The hydrated lime usually consists of 95% $Ca(OH)_2$ and the quicklime 80% CaO. On this basis the following factors are used to compute the doses of hydrated lime or quicklime to remove 1 mg/L of the following constituents:

TABLE 11 – CHEMICAL DOSAGES FOR REMOVAL OF SELECTED CHEMICAL.

Parameter to be removed 1 mg/L	95% Hydrated Lime	80% Quicklime
Free carbon dioxide as CO2	1.44 mg/L	1.77 mg/L
Bicarbonate alkalinity as $CaCO_3$	0.64 mg/L	0.78 mg/L
Magnesium as Mg	2.61 mg/L	3.20 mg/L

For lime of different analysis, a different factor is used, e.g. to remove 1 mg/L magnesium as Mg using 90% hydrated lime, the following factor is employed:

$$2.61 * \frac{0.95}{0.90} = 2.76 \text{ mg/L}$$

As is expected a greater quantity of less pure chemical is needed.

It is usual to increase the lime dose by about 30 mg/L to ensure complete reaction.

Soda ash is usually available as better than 99% purity Sodium Carbonate. The factor to reduce non.carbonate hardness is 1.06 mg/L soda ash to remove 1 mg/L of non-carbonate hardness as $CaCO_3$.

The dose of carbon dioxide required to change 1 mg/L of calcium carbonate into calcium bicarbonate and to change 1 mg/L magnesium hydroxide into magnesium bicarbonate is computed by one factor since alkalinity is always expressed in terms of calcium carbonate. The basic factor is 0.44 but as it is usual to add an excess of 25% to ensure complete reaction, the factor used is 0.55. This means that the carbon dioxide dose as CO_2 is 0.55 times the sum of the reduction in carbonate alkalinity and in hydroxide alkalinity.

It is usual to try to reduce the magnesium content to between 3 and 5 mg/L as Mg. Larger excess of lime are needed to remove the remaining magnesium. One problem is to decide to what total hardness the water should be treated. This depends on several factors:

1. the total hardness of the raw water;

2. the cost of the treated water;

3. the consumer's requirements;

4. whether the softened water is to be blended with other waters.

The lower the final total hardness, the higher must be the pH of the treated water.

The table below gives a **VERY ROUGH ESTIMATE** of the pH required for a water with the given hardness to have a Calcium Carbonate Precipitation Potential of about 2 mg/L.

TABLE 12 – A ROUGH INDICATION OF pH REQUIRED FOR STABILITY AT VARIOUS HARDNESS LEVELS.

HARDNESS as $CaCO_3$	APPROXIMATE pH at 20º
40	9.15
60	8.91
80	8.66
100	8.42
120	8.20

It is usual to soften water to a total hardness of about 80 mg/L as $CaCO_3$; as is seen above the final pH will be about 8.7.

The method of computing the doses of the various chemicals is shown below.

The raw water has the following analysis:

TABLE 13 – INITIAL WATER QUALITY OF WATER TO BE SOFTENED.

PARAMETER	CONCENTRATION
Free carbon dioxide	25 mg/L as CO_2
pH Value	7.5
Phenolphthalein alkalinity	Nil
Methyl orange alkalinity	250 mg/L as $CaCO_3$
Total Hardness	300 mg/L as $CaCO_3$
Magnesium	40 mg/L as Mg

It is required to soften this water to a total hardness of 70 mg/L as $CaCO_3$, a magnesium content of 3 mg/L as Mg. The marble test indicates that the final treated water should have a phenolphthalein alkalinity of 15 mg/L as $CaCO_3$ and a methyl orange alkalinity of 35 mg/L as $CaCO_3$

It is necessary to estimate the quality of the water after lime-soda addition and sedimentation but before recarbonation. This estimation is made on the basis of experience or by trial-and-error. Any error made here will result in the final quality of the water being different from that required. On the basis of the differences found, adjustments are made to the chemical doses, and hence to the quality of the intermediate water.

The quality of the water after lime and soda ash addition and sedimentation and before recarbonation is estimated to be:

TABLE 14 – WATER QUALITY AFTER FIRST STAGE OF TREATMENT.

PARAMETER	CONCENTRATION
Free carbon dioxide	Nil
pH Value	10.6
Phenolphthalein alkalinity	57.5 mg/L as $CaCO_3$
Methyl orange alkalinity	75 mg/L as $CaCO_3$
Total Hardness	110 mg/L as $CaCO_3$
Magnesium	3 mg/L as $CaCO_3$

The changes in the constituents of the water at the various stages are tabulated below:

TABLE 15 – SUMMARY OF THE WATER QUALITY AT EACH STAGE OF TREATMENT.

Parameter	Raw Water	After lime-soda & before recarbonation		After recarbonation & filtration	
	Content	Content	Change	Content	Change
Free carbon dioxide	25	0	-25	0	0
pH value	7.5	10.6	+3.1	8.8	-1.8
Phenolphthalein Alk	0	57.5	+57.5	15	-42.5
Methyl Orange Alk.	250	75	-175	35	-40
Bicarbonate Alk.	250	0	-250	5	+5
Carbonate Akl.	0	35	+35	30	-5
Hydroxide Alk.	0	40	+40	0	-40
Total Hardness	300	110	-190	70	-40
Magnesium	40	3	-37	3	0
Non-Carbonate Hardness.	50	35	-15	35	0

Notes:

1. Free carbon dioxide is expressed as mg/L $CO_{2;}$
2. Magnesium is expressed as mg/L Mg;
3. With the exception of pH value, all other constituents are expressed as mg/L $CaCO_{3;}$
4. How these values were obtained will be covered later.

8.2.3.1 Estimating the Lime Dose.

From the change in the content of free carbon dioxide, bicarbonate alkalinity and magnesium, it is possible to estimate the lime dose needed. From the change in the content of non-carbonate hardness it is possible to estimate the soda ash dose needed. The calculation of lime dose based on 95% hydrated lime:

TABLE 16 – CALCULATION OF LIME DOSE.

Parameter	Reduction in content mg/L	Factor	Chemical dose mg/L
Carbon Dioxide	25	1.77	44
Bicarbonate Alk.	250	0.78	195
Magnesium	37	3.20	119
TOTAL			357

Add 30 mg/L to ensure complete reaction.

Therefore the dose of 95% hydrated lime is 387 mg/L.

8.2.3.2 Estimating the Soda Ash Dose.

The calculation of soda ash dose based on 100% Sodium Carbonate:

TABLE 17 – CALCULATION OF SODA ASH DOSE.

Parameter	Reduction in content mg/L	Factor	Chemical dose mg/L
Non-carbonate Hardness	15	1.06	16

Therefore the dose of soda ash is 16 mg/L

8.2.3.3 Estimating the Carbon Dioxide Dose.

The calculation of carbon dioxide dose for recarbonation:

TABLE 18 – CALCULATION OF CARBON DIOXIDE DOSE.

Parameter	Reduction in content mg/L	Factor	Chemical dose mg/L
Hydroxide Alkalinity	40	0.55	22
Carbonate Alkalinity	5	0.55	2.8
TOTAL			24.8

Therefore the dose of carbon dioxide is 24.8 mg/e as CO_2.

8.2.3.4 Determining the Error.

It was mentioned above that it was necessary to estimate the quality of the intermediate water in order to estimate the chemical dose and that if the quality of the treated water was different from that required; it would be possible to modify the dose of chemicals in the light of information received from the analysis of the treated water. The results of the analyses are tabulated below:

TABLE 19 – DIFFERENCE BETWEEN INTENDED RESULT AND TEST RESULT.

Parameter	Intended Result	Test Result	Error
Total Alkalinity	35	60	25 too high
Phenolphthalein Alkalinity	15	25	10 too high
Carbon Dioxide	Nil	Nil	Nil
Total Hardness	70	100	30 too high
Non-Carbonate Hardness	35	25	10 too low
Magnesium	3	25	22 too high

Higher total hardness and magnesium values are higher than intended. On the other hand, the non-carbonate hardness is lower than intended; this is as a result of too much soda ash being used. The additional lime dose cannot be calculated but must be estimated. The reduction in the dose of soda ash is:

10 mg/L * 1.06 (conversion factor) = 10.6 mg/L of Soda Ash

This inability to calculate the doses accurately is the one major disadvantage of the trial-and-error method.

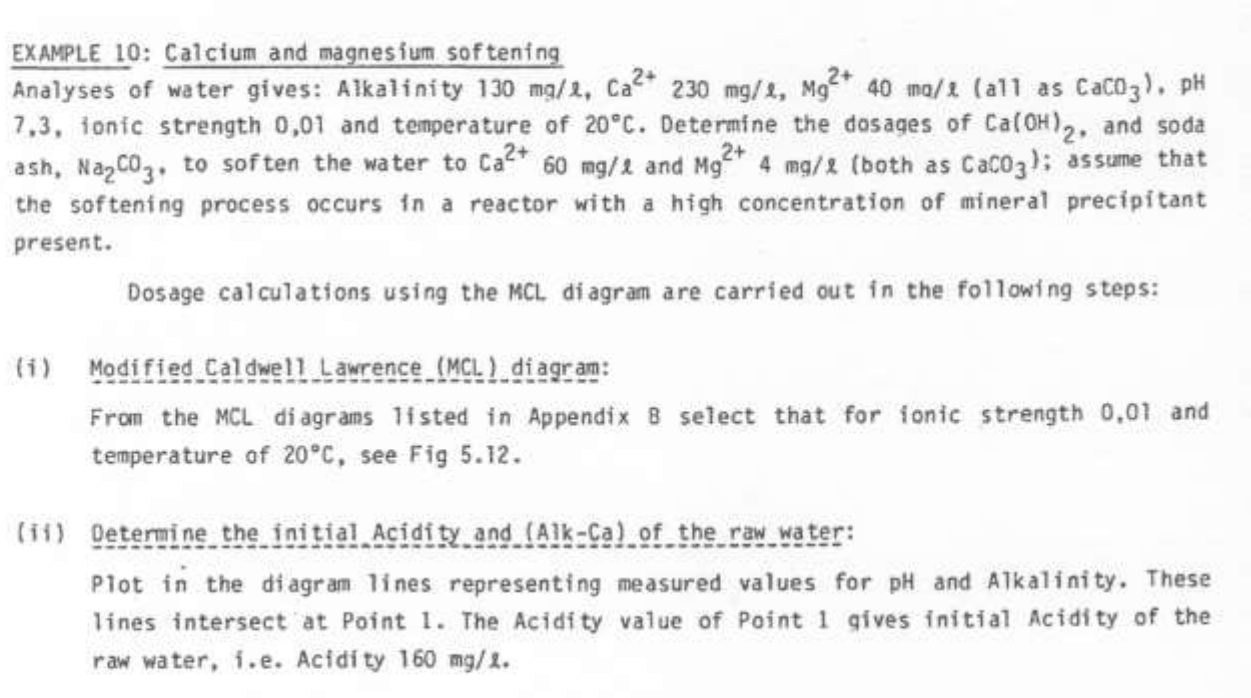
8.2.4 The Scientific Method Based on the Caldwell - Lawrence- Diagram.

The Caldwell - Lawrence diagram consists of a system of curves indicating the relationship between pH Value, total alkalinity and calcium for water in equilibrium between dissolved carbonic species (i.e. dissolved CO_2 , HCO_3^- and $CO_3^=$) and solid calcium carbonate. The 3 phase equilibrium line showing equilibrium between carbon dioxide in the air, dissolved carbonic species and solid calcium carbonate is also plotted.

THIS SECTION HAS BEEN UPDATED IN 2020.

This part is reproduced from the Water Research Commission Report TT24/86, Softening and Stabilization of Municipal Waters by R E Lowenthal, H N S Wiechers and G vR Marais.

They are acknowledged as the authors of this report.



EXAMPLE 10: Calcium and magnesium softening

Analyses of water gives: Alkalinity 130 mg/ℓ, Ca^{2+} 230 mg/ℓ, Mg^{2+} 40 mg/ℓ (all as $CaCO_3$). pH 7,3, ionic strength 0,01 and temperature of 20°C. Determine the dosages of $Ca(OH)_2$, and soda ash, Na_2CO_3, to soften the water to Ca^{2+} 60 mg/ℓ and Mg^{2+} 4 mg/ℓ (both as $CaCO_3$); assume that the softening process occurs in a reactor with a high concentration of mineral precipitant present.

Dosage calculations using the MCL diagram are carried out in the following steps:

(i) Modified Caldwell Lawrence (MCL) diagram:

From the MCL diagrams listed in Appendix B select that for ionic strength 0,01 and temperature of 20°C, see Fig 5.12.

(ii) Determine the initial Acidity and (Alk-Ca) of the raw water:

Plot in the diagram lines representing measured values for pH and Alkalinity. These lines intersect at Point 1. The Acidity value of Point 1 gives initial Acidity of the raw water, i.e. Acidity 160 mg/ℓ.

Initial (Alk-Ca) = (130 - 230) = -100 mg/ℓ.

(iii) Determine initial saturated state point:

Plot in the diagram the lines for initial Acidity (160 mg/ℓ) and (Alk-Ca) = -100 mg/ℓ. Intersection of these lines occurs at Point 2. The initial saturated state which the water would attain with time is given by the values of lines for Alkalinity, Ca^{2+} and pH through Point 2.

(iv) Determine the pH value such that the water is saturated with respect to $Mg(OH)_2$ with Mg^{2+} = 4 mg/ℓ:

Referring to the monogram of Mg_s^{2+} versus pH in Fig 5.12, at pH 10,8 the water is just saturated with 40 mg/ℓ Mg^{2+} (as mg/ℓ $CaCO_3$). Increasing pH above 10,8 causes precipitation of $Mg(OH)_2$.

Final stipulated magnesium concentration is 4 mg/ℓ (as $CaCO_3$). The Mg_s^{2+} - pH nomogram in Fig 5.12 shows that at $Mg(OH)_2$ saturation with pH 11,3 the maximum concentration of Mg^{2+} in the water is 4 mg/ℓ. Thus, in this problem, after precipitating (40-4) = 36 mg/ℓ $Mg(OH)_2$ (as $CaCO_3$) we require a pH of 11,3.

(v) Determine the saturated state point in the diagram after precipitation of 36 mg/ℓ $Mg(OH)_2$:

In the conditioning diagram draw the line representing the pH value at which the water is just saturated with the final desired Mg^{2+} concentration, at pH 11,3 the water is just saturated with 4 mg/ℓ Mg^{2+} (as $CaCO_3$).

Precipitation of 36 mg/ℓ $Mg(OH)_2$ (as $CaCO_3$) causes the (Alk-Ca) ordinate value to change by -36 mg/ℓ, thus,

(Alk-Ca) after $Mg(OH)_2$ precipitation
= (Alk-Ca) initial - $Mg(OH)_2$ precipitated
= (-100) - 36 = -136 mg/ℓ.

Intersection of lines for (Alk-Ca) = -136 and pH 11,3 occurs at Point 3. This is the saturated state point after $Ca(OH)_2$ dosing and $Mg(OH)_2$ precipitation with Acidity = 76 mg/ℓ and Alkalinity = 78 mg/ℓ (both as $CaCO_3$). The condition just prior to $Mg(OH)_2$ precipitation is given by 45° line from Point 3 to the intersection of (Alk-Ca) = -100, viz Point 3'.

(vi) Calculate $Ca(OH)_2$ dosage to effect the $Mg(OH)_2$ precipitation:

The $Ca(OH)_2$ dosage to effect precipitation of 36 mg/ℓ $Mg(OH)_2$ is given by the sum of the changes in Acidity due to dosing and $Mg(OH)_2$ precipitation. That is, the change in Acidity between Points 1 and 3 results from two effects, (a) $Ca(OH)_2$ dosing which decreases the initial Acidity, and (b) $Mg(OH)_2$ precipitation which increases Acidity by the mass concentration of $Mg(OH)_2$ precipitated, i.e.

Acidity change = - $Ca(OH)_2$ dosage + $Mg(OH)_2$ precipitated

i.e.

Acidity(3) - Acidity(1) = - $Ca(OH)_2$ dosage + $Mg(OH)_2$ precipitated

i.e.

$Ca(OH)_2$ dosage = (Acidity(1) - Acidity(3)) + $Mg(OH)_2$ precipitated
= (160 - (-76)) + 36 = 272 mg/ℓ (as $CaCO_3$).

At Point 3 the Ca^{2+} concentration now is 214 mg/ℓ as $CaCO_3$. The total carbonate species concentration, C_T, is obtained from Eq 4.21 i.e.

C_T = (Alkalinity + Acidity)/2
= [78 + (-76)]/2
= 1 mg/ℓ as $CaCO_3$.

Compare the C_T with the initial saturated C_T value (Point 2), which gives (140 + 160)/2 = 150 mg/ℓ as $CaCO_3$, and it is clear that after Mg^{2+} removal virtually all the C_T has been removed and water is virtually a slaked $Ca(OH)_2$ solution but with a higher Ca^{2+} concentration due to OH^- removal during Mg^{2+} precipitation.

(vii) Sodium carbonate dosage to effect Ca^{2+} softening:

Ca^{2+} concentration (given by Ca^{2+} value of Point 3, Fig 5.11) is now 214 mg/ℓ, a final Ca^{2+} value of 60 mg/ℓ is specified. Noting that Na_2CO_3 dosing does not affect Acidity, the final saturated condition of the water after Ca^{2+} precipitation is given by the intersection point of lines for Acidity = -76 mg/ℓ and Ca^{2+} = 60 mg/ℓ, i.e. Point 4.

The required Na_2CO_3 dosage is thus the difference in (Alk-Ca) between Points 3 and 4,

Na_2CO_3 dosage = $(Alk\text{-}Ca)_4 - (Alk\text{-}Ca)_3$
= 20 - (-136) = 156 mg/ℓ as $CaCO_3$.

The corresponding C_T value remains virtually the same as that at Point 4 i.e. 4 mg/ℓ as $CaCO_3$.

In the example above, after softening the pH of the water is 11,3. Clearly this value is too high for general municipal distribution, and the water must be stabilized. This aspect is considered in the following section.

An alternate way of visualizing the softening process shown in Fig 5.12 is as follows:

Addition of $Ca(OH)_2$ moves the saturated equilibrium Point 2 vertically (decreases Acidity) along a constant (Alk-Ca) path.

Precipitation of $Mg(OH)_2$ causes equal increase in Acidity and decrease (Alk-Ca) by an amount equal to the mass concentration of $Mg(OH)_2$ precipitated.

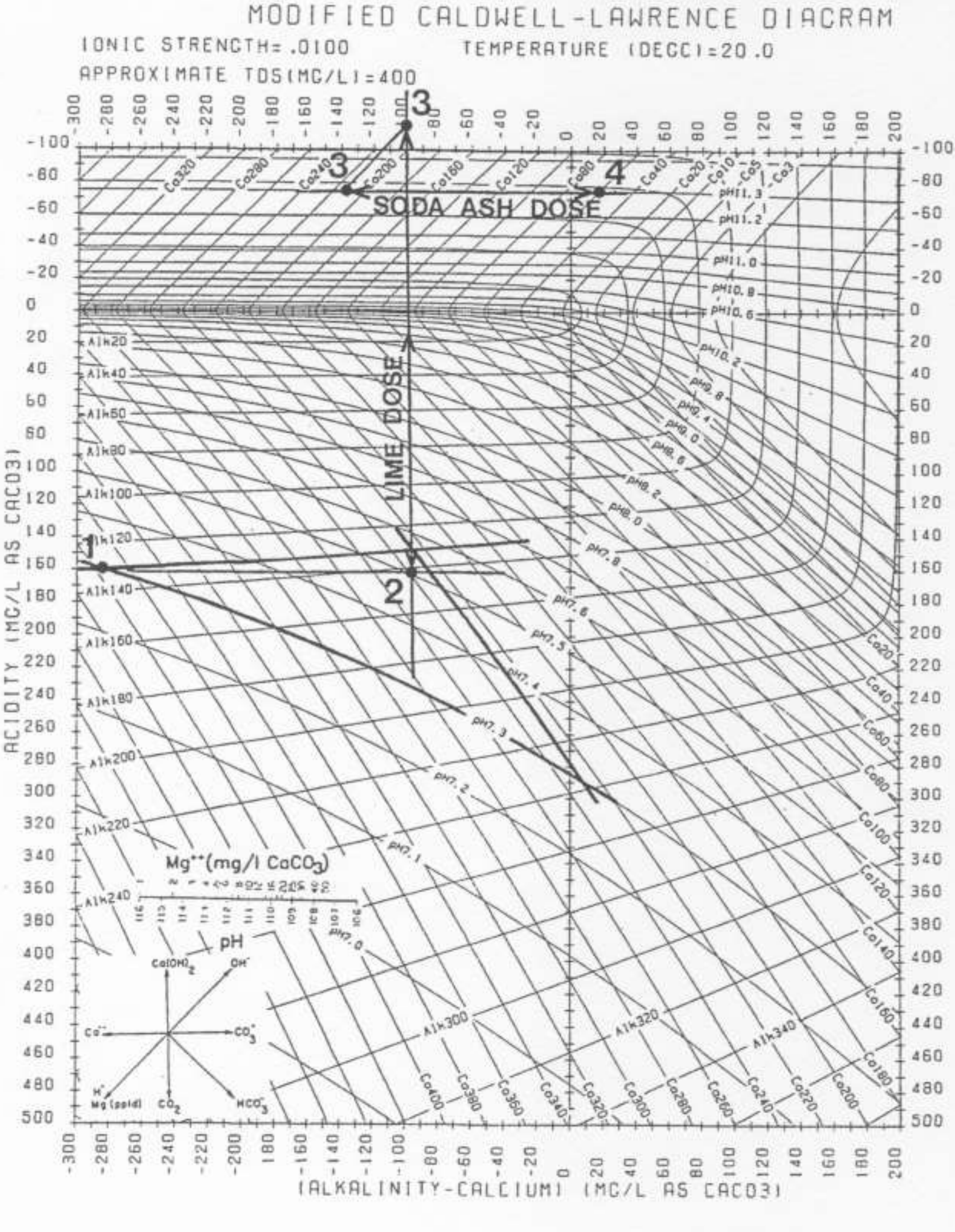
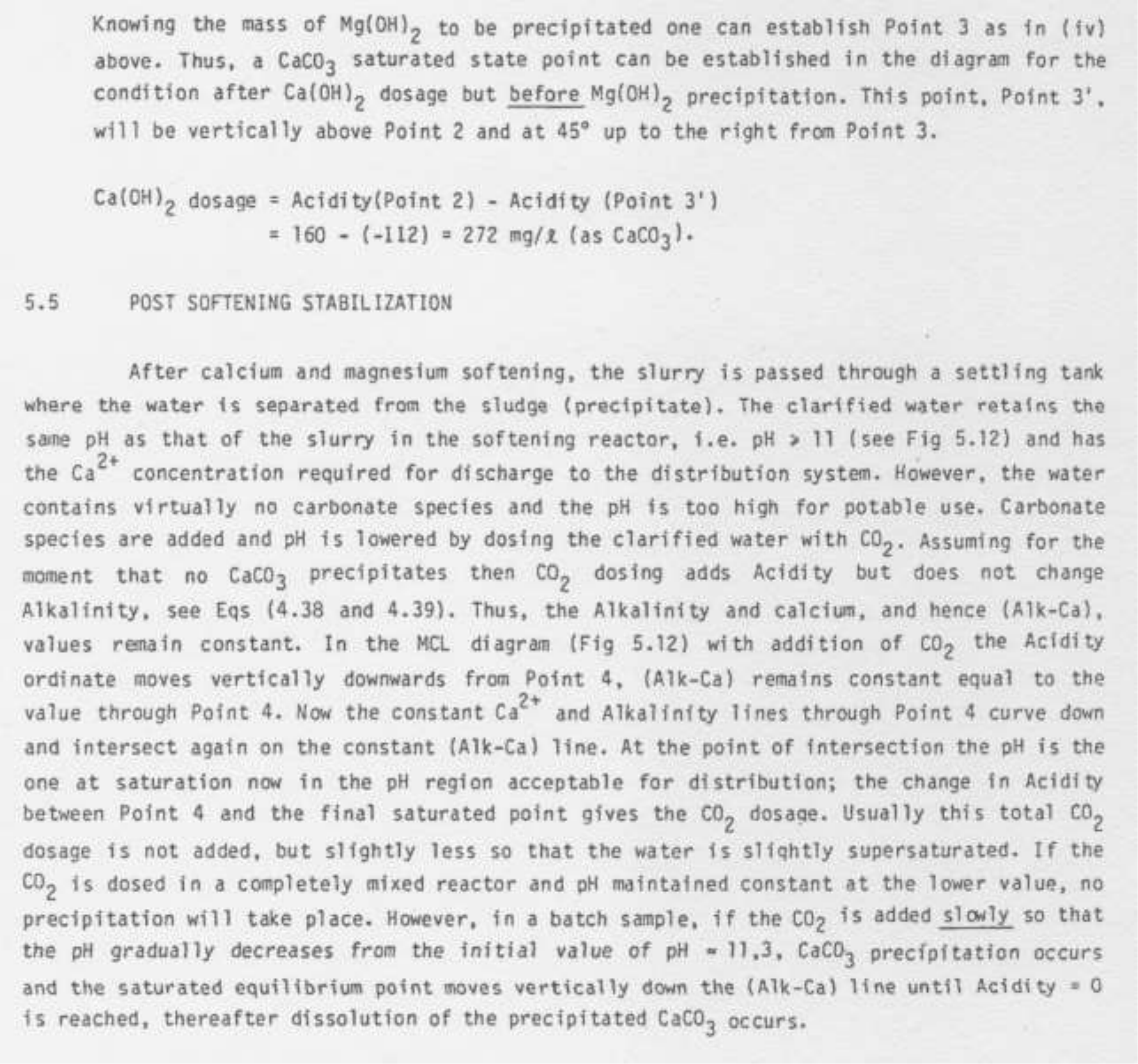


Fig 5.12 Example 10: Calcium and magnesium softening using lime and soda ash.



Knowing the mass of $Mg(OH)_2$ to be precipitated one can establish Point 3 as in (iv) above. Thus, a $CaCO_3$ saturated state point can be established in the diagram for the condition after $Ca(OH)_2$ dosage but before $Mg(OH)_2$ precipitation. This point, Point 3', will be vertically above Point 2 and at 45° up to the right from Point 3.

$Ca(OH)_2$ dosage = Acidity(Point 2) - Acidity (Point 3')
= 160 - (-112) = 272 mg/ℓ (as $CaCO_3$).

5.5 POST SOFTENING STABILIZATION

After calcium and magnesium softening, the slurry is passed through a settling tank where the water is separated from the sludge (precipitate). The clarified water retains the same pH as that of the slurry in the softening reactor, i.e. pH > 11 (see Fig 5.12) and has the Ca^{2+} concentration required for discharge to the distribution system. However, the water contains virtually no carbonate species and the pH is too high for potable use. Carbonate species are added and pH is lowered by dosing the clarified water with CO_2. Assuming for the moment that no $CaCO_3$ precipitates then CO_2 dosing adds Acidity but does not change Alkalinity, see Eqs (4.38 and 4.39). Thus, the Alkalinity and calcium, and hence (Alk-Ca), values remain constant. In the MCL diagram (Fig 5.12) with addition of CO_2 the Acidity ordinate moves vertically downwards from Point 4, (Alk-Ca) remains constant equal to the value through Point 4. Now the constant Ca^{2+} and Alkalinity lines through Point 4 curve down and intersect again on the constant (Alk-Ca) line. At the point of intersection the pH is the one at saturation now in the pH region acceptable for distribution; the change in Acidity between Point 4 and the final saturated point gives the CO_2 dosage. Usually this total CO_2 dosage is not added, but slightly less so that the water is slightly supersaturated. If the CO_2 is dosed in a completely mixed reactor and pH maintained constant at the lower value, no precipitation will take place. However, in a batch sample, if the CO_2 is added slowly so that the pH gradually decreases from the initial value of pH ≈ 11,3, $CaCO_3$ precipitation occurs and the saturated equilibrium point moves vertically down the (Alk-Ca) line until Acidity = 0 is reached, thereafter dissolution of the precipitated $CaCO_3$ occurs.

END OF 2020 UPDATE

XXXXXXXXXXXXXXXXXXX

8.2.3 The Softening Process.

After the lime followed by the soda ash if necessary has been added to the water and thoroughly mixed, the precipitates are allowed to settle out in a sedimentation tank. The effluent from this tank will have a pH value of between 9.4 and 11.0 depending on the actual requirements of the treatment process. The pH value of the water is too high for reticulation and has to be reduced using one of the methods indicated below. At the same time, the settled water will contain some calcium carbonate, magnesium hydroxide and possibly some magnesium carbonate in suspension. It is quite likely that some of the suspended particles may coagulate within the sand filtration system that usually follows the sedimentation process. If this happens in the bed or the sand filter, the precipitate may encrust the sand making it virtually impossible to remove. To prevent this happening, one can add a dispersant such as sodium hexametaphosphate that prevents the residual calcium carbonate from precipitating in a scale forming manner.

It is more usual however, to reduce the pH value of the treated water to re-dissolve the residual suspended material. This removes the problem of the possible coating of the filter sand and at the same time makes the water acceptable for reticulation.

Among the methods available for reducing the pH value are:

1. recarbonation;
2. blending with unsoftened water;
3. addition of a strong acid.

In recarbonation, carbon dioxide is added in order to reduce the pH value and also to dissolve any suspended calcium carbonate, magnesium hydroxide or residual calcium hydroxide in the following reactions.

$$CO_2 + H_2O \rightarrow H_2CO_3$$

$$CaCO_3 + H_2CO_3 \rightarrow Ca(HCO_3)_2$$

$$Mg\ (OH)_2 + 2H_2CO_3 \rightarrow Mg(HCO_3)_2 + 2H_2O$$

$$Ca(OH)_2 + 2H_2CO3 \rightarrow Ca(HCO_3)_2 + 2H_2O$$

The addition of carbon dioxide does not change the total alkalinity, but converts the phenolphthalein alkalinity into methyl orange alkalinity. Carbon dioxide may be purchased as the liquefied gas under pressure, solid carbon dioxide or produced on site from the combustion of fuel oil, diesel, coal or gas. With the latter processes, the gas must be cleaned to remove unburnt fuel and sulphur compounds.

It was in the calculation of the lime dose in the trial and error system, that the dissolved carbon dioxide present in the raw water had a lime demand. If a part of the water is softened and then blended with the raw water, the dissolved carbon dioxide in the raw water will react with the calcium carbonate, magnesium hydroxide and calcium hydroxide as shown above. The carbon dioxide content of the raw water may not be high enough to reduce the pH sufficiently without blending such a large proportion of the raw water that the hardness of the final water is too high. In this case, recarbonation is needed and all the blending is doing is reducing the amount of additional carbon dioxide that must be added.

If a strong acid is added to reduce the pH, the alkalinity is reduced and the water may become aggressive.

8.3 ION-EXCHANGE SOFTENING.

8.3.1 Introduction.

This water treatment procedure derives its name from the property that certain materials have to exchange their ions for those in water coming in contact with them. The best-known are the base exchange zeolites, which will exchange the sodium cation for the calcium and magnesium ions in the water. Although only the ions are exchanged, the net result is that calcium and magnesium bicarbonates are changed to sodium bicarbonates and the calcium and magnesium sulphates and chlorides are changed to sodium sulphate and chloride. This is known as base-exchange softening. When the natural occurring green sand is used, it is called zeolite softening.

2020 UPDATE

Atlantis uses the weak base ion exchange process where the Calcium and Magnesium are exchanged for hydrogen from Sulphuric Acid. This has the advantage of reducing the Total Dissolved Solids of the water.

The reactions involved in this process are shown below, the letter R is used to indicate the anionic portion of the material which does not enter into the reaction. Na_2R means the base-exchange material in the so-called sodium form, whereas CaR and MgR represents the same material after removal of these ions from the water and the release of an equivalent amount of sodium into the water.

The ionic reaction is:

$$Na_2R + Ca^{++} \rightarrow CaR + 2Na^+.$$

In terms of hardness producing compounds, the following reactions occur:

$$Ca(HCO_3)_2 + 2NaR \rightarrow CaR + 2NaHCO_3$$

$$CaSO_4 + Na_2R \rightarrow CaR + Na_2SO_4$$

$$MgCl_2 + Na2R \rightarrow MgR + 2NaCl$$

After the sodium has been removed from the material by base exchange, the material must be regenerated. A solution of common salt, sodium chloride, is used to reverse the process:

$$CaR + 2NaCl \rightarrow CaCl_2 + Na_2R$$

$$MgR + 2NaCl \rightarrow MgCl_2 + Na_2R$$

The calcium and magnesium chlorides are discharged to waste with the excess sodium chloride. The disposal of these wastes can be a problem inland.

Since water of zero hardness is corrosive, a portion of the raw water must be mixed with the softened water to provide the desired hardness.

8.3.2 Ion Exchange Softening Practice.

The capacity of an ion-exchange material to remove hardness has traditionally been expressed in grains per cubic foot. A more correct expression is mg of hardness as $CaCO_3$ per litre of resin.

1 grain/cubic foot = 2.29mg/L

The selection of the ion-exchange material to be used should be based on the consideration of local conditions, including availability, costs, hardness of the water, the quantity of material needed, the length of the operating cycle, etc. The number of milligrams of hardness that can be removed per litre of material varies from product to product. The mass of hardness that can be removed per litre of material is approximately:

6 000 to 12 000 mg/L for natural zeolites

20 000 to 30 000 mg/L for synthetic inorganic zeolites

About 15 000 mg/L for phenolic organic materials

About 60 000 mg/L for non-phenolic organic materials.

Capacities are sometimes given in milli-equivalents/l (meq/L) where 1 meq = 50 mg $CaCO_3$. Divide above figures by 50 to get meq CaCO3.

The amount of salt needed for regeneration is about 2g of salt per g of hardness when organic materials are used up to 4g of salt per g of hardness when inorganic zeolites are used.

Either pressure or gravity flow units may be used with either upward flow or downward flow. Downward flow is more usual as it allows a higher liquid loading rate without disturbing the bed. The usual maximum loading rate is 15 m^3/m^2-h. The minimum depth of material on the unit is usually about 750 mm. Regeneration liquid flow rates are usually about the same as the normal operating rate. Each unit should be fitted with a rate of flow indicator.

Bypass pipework should be incorporated in the softening units to permit the blending of raw water with the water softened to zero hardness, so as to secure the desired hardness of the blended water. The bypass system should be fitted with a metering device so that the proportion of hard water that is bypassed can be controlled. Consideration should be given to the removal of iron and/or manganese from the bypassed water if the concentration of these minerals in the blended effluent exceeds the acceptable levels of 0.3 mg/L iron as Fe and 0.1 mg/L manganese as Mn.

The size of the treatment works will dictate whether the salt is purchased in bags or in bulk. Often a large volume of saturated sodium chloride is made when required, the solution is pumped out and automatically diluted to the working strength of 5 - 10% m/v. Saturated sodium chloride solution contains 26% NaCl by mass or 331 g/l. In practice the strength is usually taken as 25% by mass or 300 g/l.

Disposal of the waste brine can be a serious problem. If the general standard of the Department of Environmental Affairs is applied, then the Total Dissolved Solids of the discharged effluent may not be more than 500 mg/L above the value of the intake water. The sodium concentration may not be more than 50 mg/L above the value of the intake water. In extreme cases, the brine may have to be evaporated to dryness and the solids disposed of in a suitable tipping site that will not cause secondary pollution.

8.3.3 Operation of an Ion-Exchange Softening Unit.

The softening cycle is continued until hardness appears in the effluent. This is tested using the standard soap test or by using the titrimetric method. When hardness appears, this indicates that the exchange capacity of the unit is exhausted, the unit must be closed down and regeneration started.

The regeneration cycle is started by pumping salt solution through the unit at a rate of 5 m/h. The cycle usually lasts about 30 minutes. The salt solution is usually a 5 - 10% m/v solution.

A rinsing cycle is necessary to remove the rest of the hardness and the regenerating solution. Rinsing is continued, using raw water, until the hardness is about 1 mg/L as $CaCO_3$ and the chloride is about 5 - 10 mg/L higher than that of the raw water. This usually takes about 20 minutes with zeolites and 30 minutes with organic resins, using a rate of 5 m/h. When the rinsing is complete another softening cycle may be started.

The water produced by a softening plant and blended with raw water to the required hardness may not be stabilised with respect to $CaCO_3$. It may be aggressive or scale-forming. The stability of the blended water may be improved by blending to a different hardness or by adding chemicals to adjust the pH value. It may be found that although 75 mg/L hardness was required, a water of hardness 85 mg/L is more stable; it would most likely be advisable to aim for the higher hardness. Usually soda ash, sodium hydroxide or sodium silicate is used to raise the pH as these chemicals do not increase the hardness. Lime would of course increase the hardness. Hydrochloric or sulphuric acid would be used to lower the pH if needed, but this is very rare.

It has been seen that in base exchange softening, calcium and magnesium are replaced by sodium. When very hard waters are being softened, the sodium content of the treated water may become unacceptably high. In this case some of the water would be passed through a base exchange unit where hydrochloric or sulphuric acid is used for regeneration. This hydrogen cycle method has the disadvantage that calcium, magnesium and sodium sulphates are converted into sulphuric acid (actually Ca^{++} is replaced by $2H^+$ etc.) and this will cause the pH to drop. This pH drop is neutralised to some extent by blending this water with raw water. Where this pH value cannot be corrected sufficiently, it may be necessary to replace the sulphate in the water with hydroxide which will neutralise the H^+ ions produced. This will be covered in ion-exchange demineralization later. The amount of sulphuric acid needed for regeneration is about 3g H_2SO_4 per g hardness as $CaCO_3$ removed.

8.4 COMPARISON OF LIME-SODA SOFTENING WITH ION-EXCHANGE SOFTENING.

1. In lime-soda softening, the dissolved solids content of the water is reduced by the amount equivalent to the hardness removed, whereas in ion exchange softening using sodium chloride as a regenerant, there is no such reduction. In weak base ion-exchange using sulphuric acid as a regenerant, the dissolved solids content of the water is reduced;

2. In lime-soda softening, there will be a slight increase in the sodium content when soda-ash is added, whereas in ion exchange softening using sodium chloride as a regenerant, there is a large increase in sodium content. Each mg of calcium removed results in the addition of 1.15 mg of sodium in the output water. When using sulphuric acid as a regenerant, there is no increase in sodium content unless sodium salts are used for final pH correction;

3. In lime-soda softening, the sodium carbonate addition may be eliminated if the permanent hardness content is low enough, whereas in ion exchange softening carbonate and non-carbonate are removed at the same time. Omitting the sodium carbonate addition reduces costs and prevents the build-up of sodium ion in the water;

4. In lime-soda softening, the pH need not be raised so high if the magnesium content is low enough; this reduces the lime dose and the recarbonation dose. In ion exchange softening, calcium and magnesium are removed together;

5. Lime-soda softening is a continuous process whereas ion exchange is a batch-wise process. In the latter process, it is necessary to have a number of ion exchange columns in order to have a continuous output from the plant;

6. In lime-soda softening removals of extraneous suspended solids will occur in the sedimentation stage. By comparison, the suspended solids content of the water fed into the ion exchange columns must be very low to prevent fouling of the resins;

7. Lime-soda softening produces a sludge that must be disposed of whereas ion exchange softening produces a brine, or an acidic waste that must be disposed of;

8. The presence of free residual or combined residual chlorine will not affect the lime soda softening process whereas some ion exchange resins are damaged by the chlorine.

WATER SOURCES and WATER TREATMENT.

PART 9.

IRON AND MANGANESE REMOVAL.

9 1 Effects of iron and manganese in potable waters.

.Iron and manganese, either separately or together, are objectionable in treated waters for the following reasons:

1. they stain clothing and plumbing fixtures;
2. they spoil the appearance of the water when the iron precipitates out;
3. they impart an astringent taste to the water.

The World Health Organisation standards are shown in Table 20:

TABLE 20 – WORLD HEALTH ORGANIOSATION LIMITS FOR IRON AND MANGANESE IN DRINKING WATER.

Standard	Iron as Fe	Manganese as Mn
Maximum acceptable level	0.3 mg/L	0.1 mg/L
Maximum allowable level	1.0 mg/L	0.5 mg/L

The recommended limits for high class water are: Iron as Fe - 0.l mg/L; Manganese as Mn - 0.02 mg/L.

In certain industries such as textiles, dyeing, beverages and the manufacture of white paper, the water should contain less than 0.05 mg/L of iron as Fe.

9.2 THE OCCURRENCE OF IRON AND MANGANESE IN NATURAL WATERS.

It was seen in Pollution Control that there are two main types of natural waters: surface waters and underground waters. The surface waters tend to have higher pH values, higher dissolved oxygen contents and lower dissolved carbon dioxide contents. By comparison, the underground waters tend to have lower pH values, lower dissolved oxygen contents and higher dissolved carbon dioxide contents. Under the latter conditions, waters passing through iron or manganese bearing rocks and soils will tend to dissolve the iron and manganese. Conditions similar to those found underground may also be found at the bottom of deep reservoirs. If iron or manganese bearing rock or soils are present, dissolution of these minerals may take place. Under these conditions, iron and manganese contents in excess of 50 mg/L may be found. In surface waters, the relatively high pH and the significant dissolved oxygen content do not allow the dissolution of iron and manganese. This results in the contents of these minerals being low.

9.3 FORM OF IRON AND MANGANESE IN VARIOUS WATERS.

Certain components of ground waters are often in a reduced state; i.e. iron is found in the ferrous form and manganese is found in the manganous form. These forms tend to be more soluble in the presence of dissolved carbon dioxide than their oxidized forms i.e. ferric and manganic respectively. Iron is usually present as ferrous bicarbonate or ferrous hydrogen carbonate $Fe(HCO_3)_2$, however, it may also be found as ferrous sulphide FeS or probably more correctly $Fe(HS)_2$. It may also be found as a loosely bound organic complex.

9.4 METHODS FOR THE REMOVAL OF IRON AND MANGANESE.

There are various methods available depending on the relative concentrations of iron and manganese and on the form in which they are present. The basic principle of iron and manganese removal is to convert the soluble form of the iron and manganese into an insoluble form so that it may be removed in a sedimentation tank or on a sand filter. The processes listed below may be combined with the normal coagulation and flocculation processes used.

In general, waters will be found to fall into 5 types:

1. waters containing only iron and with a pH or 7.0 or more after aeration, these waters will oxidize the iron upon aeration without need of further pH adjustment or of catalytic action;

2. waters containing iron and also a limited amount of manganese, the oxidation requires catalytic action in contact beds or chlorination or pH adjustment, prior to filtration, Potassium permanganate oxidation may be used;

3. waters containing iron bound with organic matter where pH adjustment to values of 8.5 to 9.0 is required to secure oxidation and precipitation. With manganese present and bound with organic matter, the pH may have to be as high as pH 10.0;

4. waters having a high hardness, if lime-soda or zeolite softening is performed, iron and manganese removal will result from this process. However, iron and manganese can be irreversible bound onto the resin and thus reducing its ion-exchange capacity. Some of the bound iron can be removed using inhibited hydrochloric acid as a regenerant. This should not be dome more than once per year to prevent damage to the resin;

5. waters containing manganese only or proportionally more manganese than usual in combination with the iron.

The various operations in iron and manganese removal are given below:

9.4.1 Oxidation using Aeration.

This may be combined with the process intended to remove carbon dioxide and taste and odour producing substances. Iron and manganese removal is most successful when the water is aerated while trickling down through beds of crushed stone, the coating on which acts as a primary contact bed for catalytic action. The stone is placed on 4 to 6 trays each about 300mm deep, and separated by about 150 to 250 mm. The trays are perforated to distribute the water. Rates of flow of 15 - 25 m/h are usually used. Aeration need not be excessive as only 0.14 mg/L oxygen is needed to oxidise 1 mg/L iron.

9.4.2 Oxidation using Contact Beds.

The purpose of contact beds is to facilitate oxidation of iron and manganese through the catalytic action of previously precipitated oxides of these minerals on the stone. Very good results are often obtained using the manganese ore - pyrolusite. Usually upward flow beds are used, so that the oxides may be flushed out easily. Upward velocities of up to l0 m/h are used. Bed depths of up to 2 m are usually sufficient i.e. a theoretical contact period of 5 minutes. Provision must be made for the rapid draining of the beds, so as to wash excess oxides from the stone and so that a high pressure hose may be used on the stone. The beds are regenerated by backwashing with a potassium permanganate solution, 10 g $KMnO_4$/L, when potassium permanganate is not dosed continuously to the raw water.

Contact beds of pyrolusite ore, used for manganese removal without lime or potassium permanganate treatment, must be closed in units to prevent the entrance of air. This prevents the formation of a colloidal form of manganese that precipitates very slowly. Upward flow rates of about 5m/h are usually used with bed depths of 2 m giving a theoretical detention period of about 10 minutes based on a void volume of 40%. After passing through the contact beds, the water must be aerated usually in a downward flow contact bed aerator followed by filtration to remove the precipitated solids.

Manganese zeolite, formed by treating sodium zeolite with potassium permanganate is an effective contact material that will remove up to 1.6 kg manganese per cubic meter of zeolite per cycle. Regeneration of the zeolite is carried out by using 3.3 kg $KMnO_4$ per cubic metre of zeolite.

Knowing the manganese content of the water, one can calculate the volume that may be treated before manganese bleed-through occurs.

9.4.3 Oxidation by Chlorination.

Before adopting this process solely for the purpose of oxidizing the iron and manganese, tests must be carried out to determine if chlorine in the absence of dissolved oxygen will be effective in oxidizing the iron and the manganese in the water being treated. It is also necessary to determine whether the cost of chlorination equipment and chlorine will be less than the cost of aeration and double pumping that will otherwise be required.

If pre-chlorination is intended, for one or more of the reasons mentioned before, then it is most likely that chlorination would be used rather than aeration. All that one would do is to increase the chlorine dose, pH correction may be required.

If the treatment works is built on a sloping site and pre-chlorination is not intended, then aeration would be used because double pumping is not necessary.

Manganese is oxidised by free-residual chlorine, but the reaction rate is slow below about pH 9. To ensure complete oxidation each mg/L of manganese requires about 1.25 mg/L free chlorine. Iron is more readily oxidised by chlorine and over a wider pH range. The ratio of iron to chlorine is about 1 : 1.25 for complete oxidation. In certain cases the rate of oxidation is increased by the addition of 0.2 mg/L $CuSO_4$ copper sulphate, with the cupric ion acting as a catalyst.

9.4.4 Oxidation using potassium permanganate.

Potassium permanganate provides much more effective oxidation of iron and manganese than does chlorine. The reaction is independent of the pH of the water above pH 7.0. This has the advantage that manganese may be oxidised without lime treatment to raise the pH, except with soft waters. The reaction is rapid, except when reduced substances such as organic matter, hydrogen sulphide, etc., are present in high concentrations; in this case reaction periods of up to 20 minutes should be provided prior to filtration. If permanganate treatment is used prior to passing the water through a manganese zeolite bed, then residual permanganate will pass into the zeolite bed and this will prevent the oxidising power of the beds being depleted. This means that the beds will not need regeneration.

The dose of potassium permanganate is determined as that needed to produce a faint pink colour in the treated water prior to filtration. Usually the dose required for iron removal is numerically about 2/3 of the iron content in the raw water expressed as $KMnO_4$, and as Fe mg/L respectively. For manganese removal, the dose is numerically about twice that of the manganese content in the raw water expressed as $KMnO_4$ and as Mn mg/L respectively. The exact dose is not critical but it must be in at least slight excess. Any larger excess will be removed in the contact beds or in the sand filter.

Potassium permanganate may be dosed as the crystals or as a solution. The solution is harmful to rubber compounds, but has no effect on iron, steel or stainless steel.

9.5 SEDIMENTATION.

A thorough investigation of the treatment processes required for a particular water is necessary to determine exactly which process is needed and which may be left out. Usually when lime treatment is needed or when the iron content of the raw water exceeds 2 mg/L as Fe, it is advantageous to have sedimentation tanks before the sand filters. With the sedimentation tanks in operation, longer runs with the sand filters will be possible.

A plant designed to remove iron in the absence of oxygen requires flocculation, sedimentation and filtration under pressure to avoid exposing the water to the atmosphere. A cylindrical flocculation tank of a size providing about 5 minutes detention may be used, the inlet and outlet may be located so as to produce spiral flow to provide agitation. Sedimentation under pressure may be secured in a larger cylindrical tank providing a minimum detention period of one hour.

9.6 SAND FILTRATION.

Filters may serve adequately as contact beds following aeration, provided that iron alone is involved. This should be determined for a specific water by pilot-plant studies, because of the economy of eliminating a separate contact bed.

Sand filters used to remove iron or manganese remaining in the effluents of aeration units, contact beds or sedimentation tanks will generally have an effective size of 0.7 - 1.0 mm. This is much coarser than for filters intended for bacteria removal. Rates of filtration of up to l0 m/h are used.

9.7 LON EXCHANGE SOFTENERS FOR IRON AND MANGANESE REMOVAL.

Conventional ion exchange softeners may be considered for hard water containing iron and manganese. The raw water must be devoid of oxygen, because an ion-exchange process removes only the soluble iron and manganese together with the calcium and magnesium. This process should not be used when the iron and manganese content together exceeds 0.5 mg/L for each 17 mg/L of hardness as $CaCO_3$. In any case, the sum of iron and manganese should not exceed 10 mg/L. Thus for an iron content of 10 mg/L the lower limit of hardness is 340 mg/L as $CaCO_3$. This means that waters with 10 mg/L of iron and a hardness of less than 340 mg/L as $CaCO_3$ should not be treated by this method and waters of a hardness of more than 340 mg/L as $CaCO_3$ may be treated by this method.

9.8 OPERATION AND CONTROL OF IRON AND MANGANESE REMOVAL PROCESSES.

The supervision and operation of iron and manganese removal processes should be considered in two categories:

1. those followed at conventional filtration plants with coagulation, flocculation, sedimentation and sand filtration,

2. those followed at special iron and manganese removal plants.

9.8.1 Conventional Treatment Works.

Raw surface waters containing iron and manganese in limited quantities may be coagulated with aluminium sulphate, aided by pre-chlorination especially when colour is present as complex compounds of iron, manganese and organic matter. If the iron and manganese content is high and turbidity and colour removal are also needed, oxidation must be by aeration and pH correction. This increase in pH value using lime creates conditions under which aluminium sulphate is not so effective as a coagulant. In these cases ferric sulphate, ferric chloride, chlorinated copperas or copperas with additional lime will be used. Chlorinated copperas has the advantage of combining pre-chlorination with coagulation.

The floc derived from iron compounds is usually heavier than the floc derived from aluminium compounds. Unless flocculation is effective there is a tendency for a feathery, fragile floc to form. Coagulation must also be effective, because any uncoagulated iron might pass out in the treated water. The soluble iron content of the sand filter effluent should be less than 0.3 mg/L at all times. The soluble aluminium content may be up to 1 mg/L without creating an objectionable effluent. It is important not to confuse soluble aluminium with alum floc that may pass through a filter. The first is due to incomplete coagulation whereas the second is due to faulty filtration. Iron floc is also likely to coat the filter sand and be more difficult to remove by the back-washing process than alum floc. Some coating of iron floc is advantageous as it acts as a catalyst in the further oxidation and removal of any iron remaining in the applied water.

Manganese is more difficult to remove than iron. The lime dose with ferric coagulants should be higher as coagulation occurs at pH 8.5 to 9.0. Pre-chlorination may alter the situation so that manganese may be coagulated at lower pH values. Only laboratory tests will disclose the best process to use. Manganese removal is especially aided by contact with previously oxidised manganese on filter sand. The backwashing of sand filters should be controlled so as to remove the bulk of the floc without actually cleaning the sand. Observation of the length of the filter run and the rate of increase in loss of head will serve as a guide to the degree of backwashing needed.

9.8.2 Iron and Manganese Removal Plants.

The operation of these plants differs with the process used and so is not covered here. One would need to refer to the operating manual for the plant operating instructions.

9.9 LABORATORY CONTROL.

At the special treatment plants, tests for iron, manganese, residual chlorine, pH value and alkalinity are needed. At conventional treatment works, the above tests as well as those for turbidity and colour are needed.

9.10 IRON AND MANGANESE BACTERIA.

The presence of iron in waters allows the development of the so-called iron bacteria "Crenothrix" and Leptothrix". These organisms utilize iron. They develop in gelatinous masses around the screens of shallow wells yielding iron-bearing water and in distribution systems where the content of dissolved oxygen is insufficient. The above bacteria form slimes. The iron bacterium "Gallionessa Ferriginea" forms a system of complex stalks. All these organisms lead to the development of objectionable tastes and odours on their death; as well as to the presence of precipitated iron in the water.

The growth of these organisms is inhibited by the presence of free residual chlorine. In spite of the inhibition of the growth by chlorine, it is still necessary to practise iron removal when the iron content is high as the presence of significant numbers of the iron bacteria will exert a significant chlorine demand that may reduce the residual chlorine content to nil.

Similar effects are shown by the manganese utilizing bacteria.

WATER SOURCES and WATER TREATMENT.

PART 10.

DEMINERALISATION.

10.1 DISCUSSION.

With the increase in the level of pollution of many water sources, particularly mineralisation, supplies of water with a total dissolved solids of less than 1 500 mg/L are becoming rarer. 1 500 mg/L is the World Health Organisation's maximum allowable limit.

Waters may be grouped in various classes depending on their dissolved solids content:

1.	Brackish Waters	1 000 - 2 000 mg/L dissolved solids,
2.	Moderately Saline Waters	2 000 - 3 000 mg/L dissolved solids,
3.	Severely Saline Waters	10 000 - 30 000 mg/L dissolved solids,
4.	Sea Water	30 000 - 36 000 mg/L dissolved solids

Where the available water supplies with low dissolved solids are fully utilized, it may be possible to blend in a percentage of water with a higher dissolved solids content.

The maximum percentage that can be blended depends on the dissolved solids content of each of the waters and the maximum value acceptable in the final water. It has been seen earlier that softening using the lime-soda process does result in a decrease in dissolved solids content. In this case the dissolved solids removed are calcium and magnesium bicarbonate. If softening is not required or if the excess dissolved solids are due to sodium salts, as is usually the case; then alternative methods of dissolved solids reduction are required.

In some industrial process, waters of low or even essentially nil dissolved solids content are required e.g. boiler feed water, chemicals manufacture, etc.

There are two main groups of processes for dissolved solids reduction or demineralisation:

1. the removal of the salts from the water;
2. the removal of the water from the salt solution.

10.2 THE REMOVAL OF DISSOLVED SOLIDS FROM WATER BY ION EXCHANGE.

This is a two stage process. In one stage a cation exchange resin is used and cations such as sodium, calcium, magnesium, etc., are removed from the water and replaced by the hydrogen ion. It has been seen that the pH of the water will drop due to the production of H^+ ions. This partially treated water is then passed into an anion exchange resin where sulphate, chloride are removed and replaced by hydroxide ions (OH^-). The hydrogen ions and hydroxide ions combine to form water and the pH will tend to move towards neutrality. Carbonates will be removed by strongly basic resins but not by weakly basic resins.

Ion exchange columns are either mixed bed where the cation exchange resin is mixed with the anion exchange resin or kept in separate units. When being regenerated, the resins in the mixed bed unit must be separated so that they may be treated with the separate chemical solutions. Usually the resins have differing densities so that they may be separated by backwashing.

The cation exchange resin is regenerated using sulphuric acid and the anion exchange resin using sodium hydroxide. Both cationic and anionic resins are available with capacities of up to 60g hardness as $CaCO_3$ per litre i.e. about 1.2 eq/L.

As before the amount of sulphuric acid is needed for regeneration is about 3 g H_2SO_4 per g hardness removed as $CaCO_3$. The amount of sodium hydroxide needed is also about 3 g NaOH per g hardness removed as $CaCO_3$. Usually 10% solutions are used.

The reactions are as follows:

$Na^{++} + R_cH \rightarrow R_cNa + H^+$ $\quad R_c$ = cation exchange resin

$Cl^{-} + R_aOH \rightarrow R_aCl + OH^{-}$ $\quad R_a$ = anion exchange resin

The regeneration reactions are as follows:

$H^+ + R_cNa \rightarrow R_cH + Na^+$ $\quad R_c$ = cation exchange resin

$OH^{-} + R_aCl \rightarrow R_aOH + Cl^{-}$ $\quad R_a$ = anion exchange resin

The brine formed from the sodium chloride and other salts from the regeneration needs to be disposed of without causing pollution.

Demineralized and raw water are blended to give the required hardness as with base-exchange treated water.

Disposal of brine wastes can be a problem. The cost of this process at present usually exceeds that of distillation when the total solids content is greater than about 3 000 - 5 000 mg/L.

The effluent from the unit in the case of the mixed bed system or from the anion exchange unit in the case of the separate bed system is checked for electrical conductivity at intervals. When the conductivity starts to rise then breakthrough is occurring because the ion exchange capacity is exhausted and regeneration is needed.

This process is operated on a batch wise basis, this is a disadvantage. A recently developed process is almost continuous flow where a small quantity of resin is removed from the bottom of the ion exchange column during a short shutdown period. The resin is then regenerated in a separate column while the main column is producing demineralised water. It is called the counter current continuous flow ion exchange system CCIX. Because the anion exchange resin is regenerated in a separate container, it is possible to use calcium hydroxide slurry as a regenerant. With the use of sulphuric acid as a regenerant for the cation exchange resin, a precipitate of calcium sulphate is formed which may be removed by sedimentation.

The reactions are as follows, using calcium as an example:

$Ca^{++} + 2\ R_cH \rightarrow (R_c)_2Ca + H^+$ $\quad R_c$ = cation exchange resin

$Cl^{-} + R_aOH \rightarrow R_aCl + OH^{-}$ $\quad R_a$ = anion exchange resin

On regeneration using sulphuric acid for regenerating the cation exchange resin and sodium hydroxide to regenerate the anion exchange resin, the reactions are:

$(R_c)_2Ca + H_2SO_4 \rightarrow 2R_cH + CaSO4$

$2\ R_aCl + Ca(OH)_2 \rightarrow 2\ R_cOH + CaCl_2$

As calcium sulphate is soluble to about 2 000 mg/L, the excess will precipitate and may be removed by sedimentation.

Among the advantages of the counter current continuous method over the older batch wise method are:

1. that the process is almost continuous;
2. a much smaller excess of regenerant is required;
3. lime may be used instead of sodium hydroxide, this is cheaper;

4. some of the removed dissolved solids is precipitated and may be removed as sludge;

5. if ammonia and nitric acid are used as regenerants, then a waste ammonium nitrate solution containing up to 17% NH_4NO_3 can be produced - this has commercial value;

6. the columns are less prone to block as they are counter current washed;

7. the water recovery rate is higher.

10.3 REMOVAL OF DISSOLVED SOLIDS FROM WATER BY ELECTRO-DIALYSIS.

Special membranes are available that have the property of permitting the passage of ions without allowing water to pass through. One type of membrane permits the passage only of positive ions or cations; a second type permits negative ions, or anions to pass through.

The electro-dialysis unit consists of a series of chambers including alternating anion-permeable and cation permeable membranes, together with inert electrodes in the outermost compartments. Water introduced into alternate chambers is demineralized by passage of a direct current through the battery of compartments. The applied voltage drives anions toward the inert anode (the positively charged electrode) and out of the cells with anion permeable membranes on the anode side. However, the anions are trapped in the adjacent cell because it has a cation permeable (anion impermeable) membrane on the side facing the anode. In this way, cells from which anions migrate toward the anode also lose cations towards the cathode (the negatively charged electrode), while alternate cells retain both anions and cations. As the water to be treated is fed into the ion-losing compartments and the concentrated brine is bled from the ion-gaining compartments, the water can be de-mineralized electro-chemically in continuous flow. The overvoltage at the electrodes and the ohmic resistance of the membranes reduce the electrical efficiency of the system. The power loss may be reduced by partially demineralising the water to a T.D.S. of about 500 mg/L only. The higher the salinity the higher the electrical costs. Usually water of more than 5 000 to 10 000 mg/L T.D.S. is best demineralized by some other method.

Electro-dialysis has the following disadvantages:

1. the feed water must be free of turbidity;

2. the iron content of the feed water must be well below 1 mg/L;

3. problems with precipitation of calcium carbonate can occur in hard waters;

4. the final water has a relatively high dissolved solids of about 500 mg/L;

5. for dissolved solids of greater than about 5 000 mg/L, it may be necessary to operate two units in series.

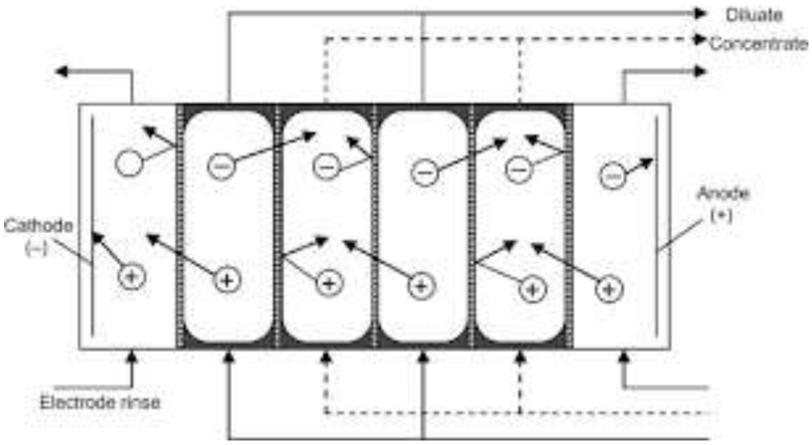


Figure 33 – TYPICAL ELECTRO-DIALYSIS UNIT.

10.4 THE SEPARATION OF WATER FROM THE SALT SOLUTION BY REVERSE OSMOSIS.

Osmosis is defined as the spontaneous transport of a solvent (water) from a dilute solution to a concentrated solution across an ideal semi-permeable membrane which impedes the passage of the solute (usually salts) but allows solvent flow. As a result of this flow of solvent a pressure called the osmotic pressure, is built up on the concentrated solution side of the membrane. If an external pressure, of magnitude greater than the osmotic pressure, is exerted on the concentrated solution side then the solvent will pass through the membrane to the dilute side. It can be seen that a **reverse** osmosis process has taken place. As the concentration increases, so will the osmotic pressure until it equals the externally applied pressure after which the amount of solvent passing each way through the membrane will be equal.

As with all membrane processes the feed water has to be essentially free of turbidity. After pre-treatment, the feed water passes through a high pressure pump into the membrane modules. The concentrated reject water will pass through a pressure reducing valve to waste while the product water passes out of a second set of pipes. It may be possible to pass the reject water directly into a second module in series and only then to pass it through a pressure reducing valve, in order to improve the recovery rate. The pre-treatment stage may include pH adjustment by means of acid addition, to prolong the life of the membranes and to prevent the precipitation of calcium and iron salts.

Small molecules such as sodium and chloride do pass through the membrane to a limited extent. The rejection rate of these two ions is about 80 - 85%. The larger ions such as calcium and sulphate may be rejected to the extent of greater than 99%. Organic molecules of molecular mass greater than about 200 are almost entirely rejected. For a typical brackish water, the overall salt rejection is about 90%.

Fouling of the membranes is a problem, both from the cleaning aspect and the reduction in rate of yield.

Reverse osmosis may be used for waters of up to about 10 000 mg/L dissolved solids. The product water dissolved solids is usually of the order of 500 mg/L with an 80% water recovery. At lower rates of water recovery, a higher quality water may be obtained.

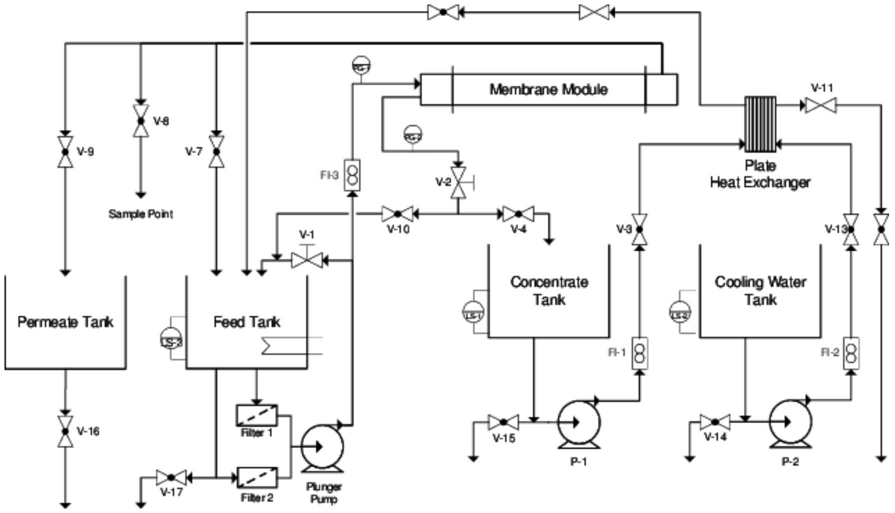


Figure 34 – TYPICAL LAYOUT OF REVERSE OSMOSIS UNIT.

10.5 THE SEPARATION OF THE WATER FROM THE SALT SOLUTION BY ULTRAFILTRATION.

Ultrafiltration is similar to reverse osmosis in that it depends on a pressure driving force and a membrane that is permeable to some components in the water and impermeable or only slightly permeable to others. The main difference is that ultra-filtration operates at relatively low pressures of the order of 500 kPa. Ultrafiltration is mainly used for removal of organic materials with molecular masses of greater than about 500.

The rejection of low molecular substances such as sulphates is only about 10%.

10.6 THE SEPARATION OF THE WATER FROM THE SALT SOLUTION BY DISTILLATION.

In distillation, water is removed from the solution to be treated, by converting it into steam and then condensing the steam back into water. In the two membrane processes covered above, there are dissolved solids remaining in the treated water, whereas in distillation, the treated water is essentially free of dissolved material.

There are 3 main types of evaporators in use:

1. the multiple effect evaporators. These are relatively efficient. Each component unit of a multiple effect evaporator is maintained in sequence at a slightly lower pressure and temperature in order to permit the steam produced in one effect to become the source of heat in the next. Kilogram for kilogram, the amount of product water then approximates the number of effects;

2. the multistage flash evaporator. Here, also, consecutively lower temperatures and pressures are used. The incoming water is warmed by the heat of condensation and only a small amount of heat energy is required to flash the pre-heated water in the reduced pressure stage into steam;

3. the vapour-compression still. This process relies on the mechanical compression of the vapour to boost its temperature high enough to supply through its own condensation the heat necessary to evaporate the feed water. Once started, this process does not draw upon further heat energy, only upon mechanical energy.

In the distillation process, the salt content of the water to be distilled has little effect upon the cost of the energy needed. However, corrosion can be a serious problem.

10.7 THE SEPARATION OF THE WATER FROM THE SALT SOLUTION BY FREEZING.

The amount of energy needed to freeze unit mass of water is much less than that needed to boil unit mass.

Although the ice crystals that are formed are essentially pure water, the yield of product water is decreased because some of the water must be used to wash the salt from the ice surfaces. Heat energy is also required to melt the ice. The use of a counter-current system, i.e. the incoming water melts the ice via a heat exchange, does help to improve the efficiency.

The recovery of certain elements such as bromine and magnesium from sea water would help to offset the cost of obtaining pure water by this method.

10.8 ECONOMICS.

The economic advantage of demineralising otherwise unsuitable waters must be appraised in the light of broad social and industrial factors underlying the well-being and advancement of communities in areas where fresh water resources are very limited.

At times the very existence of an industry has depended on the distillation of sea water, e.g. the oil industry in parts of the Middle East. In other cases, the comparison of costs between demineralizing a local brackish water supply and the piping of fresh water from distant points is quite complex and would require detailed engineering analysis.

Among the factors which suggest that one process has a greater advantage over another are:

1. the availability of water with dissolved solids of 2 000 - 3 000 mg/L would favour the reverse osmosis or ion exchange processes;

2. the availability of waste heat - which would tend to favour the distillation process;

3. nearness to the sea into which the concentrated brine may be discharged would favour the reverse osmosis or ion exchange processes.

WATER SOURCES and WATER TREATMENT.

PART 11.

SPECIAL TREATMENT PROCESSES.

There are many cases where the quality of the water to be reticulated is not high enough even after using some or even all of the processes covered up to now. Four additional specialized processes will be covered.

11.1 ACTIVATED CARBON TREATMENT.

Certain materials have the property of being able to adsorb certain substances onto themselves. The adsorption phenomenon is the interface accumulation of a substance. Absorption by comparison is a process in which one substance penetrates into another. For simplicity one may consider adsorption as a surface effect and absorption as a body effect. Activated carbon is one material that illustrates the first effect, while the dissolving of oxygen in water, as in the aeration process, illustrates the second effect.

The activated carbon used in water treatment is usually prepared from wood, coal, bone, petroleum residues or nut shells. The raw material is generally activated in an oxygen deficient atmosphere at a temperature of between 300°C and 1 000°C.

In part the adsorption properties of activated carbon are due to its extremely large relative surface area, 200 to 1000 m^2 per gram. Activated carbon has a preference for organic compounds and, because of this selectivity, is particularly effective in removing organic compounds from aqueous solution. When the activated carbon has been exhausted, it may be regenerated and re-used.

Activated carbon has four main uses:

11.1.1 Removal of Colour.

Certain colours imparted to water by various organic materials cannot be removed easily by coagulation. Chlorination or some other oxidation reaction may also not be feasible or desirable. Often activated carbon may be used to adsorb the residual colour.

11.1.2 Removal of Tastes and Odours.

Some tastes and odours cannot be removed entirely by aeration and chlorination may enhance the taste. This is the case with phenol where the chlorinated derivatives of phenol impart a stronger, more unpleasant taste than the original phenol itself. Activated carbon may be used to adsorb the residual taste and odour.

11.1.3 Removal of Residual Organic Material.

Certain organic materials such as volatile halogenated hydrocarbons (VHH), and pesticides are present in small amounts in some waters, or may be formed during the chlorination process. These materials cannot be removed by the conventional treatment processes. Although not much is known about the long term effects of ingesting these organic materials, they are considered undesirable. Activated carbon is effective in removing them.

11.1.4 Removal of Residual Chlorine or De-chlorination.

Activated carbon is effective in removing all forms of residual chlorine. Activated carbon may be used for any or all of the above uses simultaneously.

11.2 OZONE TREATMENT.

It was seen earlier that ozone may be used as a disinfectant either alone or in combination with chlorination. The effectiveness of ozone as an oxidizing agent goes further than just as a disinfectant.

Ozone can oxidize certain materials that are resistant to oxidation with chlorine. It does not have the disadvantage of chlorine of forming volatile halogenated hydrocarbons, actually ozone will oxidise some organo-chlorinated compounds. Ozone will oxidise phenols to tasteless and odourless compounds. Ozone destroys all polycyclic hydrocarbons such as 3:4 benzpyrene, which are suspected of being carcinogenic. Ozone also decomposes a proportion of the non-biodegradable fraction of detergents.

Ozone must be manufactured as required as it cannot be stored. It is a gas that is slightly soluble in water. It is made by passing dry air between two electrodes with a potential difference of between l0 kV and 20 kV AC. The treated air contains about l0 to 20 g ozone per cubic metre of air. Because of the low solubility of ozone in water, careful design of the ozone contact tank is required.

11.3 BREAKPOINT CHLORINATION AND DE-CHLORINATION.

Certain tastes and odours are enhanced by the addition of chlorine. It is necessary to increase the concentration of chlorine in order to oxidise the material and to destroy the taste or the odour. In the oxidation of ammonia, using chlorine, there is a chlorine dose above which the residual chlorine starts increasing. This chlorine dose corresponds to the breakpoint dose. In figure 32 a breakpoint occurs at a chlorine dose of 3.6 mg/L and that the chlorine residual at this point is 0.6 mg/L. This is rather high for reticulation. De-chlorination may be used to reduce the concentration of residue chlorine. Amongst the reducing agents that are used are sulphur dioxide, sodium hydrogen sulphite, sodium sulphite or sodium thiosulphate.

11.4 FLUORIDATION AND DE-FLUORIDATION.

The fluoridation of public water supplies is designed to supplement the natural fluoride content of potable waters so as to provide the proper amount of this element in the water consumed by children, thus aiding in the development of sound teeth with a high resistance to decay.

The optimum daily intake of fluoride is 1.6 mg based on drinking and using in cooking 1.6 litre of water dosed at 1 mg/L F^- or 2.3 mg/L NaF. The use of water dosed at 1.5 mg/L F^- for periods of more than 3 months can cause mottling of the teeth.

The removal of excess fluorides from water is usually expensive. Fluoride may be removed as magnesium fluoride MgF_2 during the water softening process. Fluoride may also be removed by passing the water through a bed of tricalcium phosphate $Ca_3(PO_4)_2$. The beds are regenerated using 1% Sodium Hydroxide solution and are then washed with dilute hydrochloric acid to neutralise the sodium hydroxide. The special plants are expensive to operate and are rarely used.

Among the chemicals used in fluoridation are:

1. Sodium fluoride, NaF - 95% pure powder containing 43% F;
2. Sodium Silicofluoride, Na_2SiF_6 - 99% pure powder containing 60% F;
3. Fluosilic acid, H_2SiF_6 - 30% m/m solution containing 23.7% F;
4. Ammonium Silicofluoride, (NH_4) SiF_6 - powder. The latter is very rarely used.

Both sodium salts are poorly soluble: Sodium fluoride 44 g/L, sodium silicofluoride 6.2 g/L. The two powders may be dosed as a liquid or as a powder. The acid must be used as is or diluted at least 20 times with water. Intermediate dilutions result in the formation of a precipitate.

Strict dust control must be enforced when handling the powders.

As the per capita usage of water varies with the temperature, the chemical dose of fluoride must be varied seasonally in order to keep the daily dose of fluoride constant at 1.5 mg per day as shown in Table 21.

TABLE 21 – RECOMMENDED FLUORIDE CONTENT OF WATER TO MAINTAIN UNIFORM DAILY UPTAKE.

Mean seasonal temp °C	Daily water consumption - litre	Dose of Fluoride ion – mg/L as F
10	1.3	1.2
12	1.4	1.1
14	1.6	1.0
17	1.8	0.9
21	2.0	0.8
26	2.3	0.7

The dose mentioned is the total fluoride content, i.e. the sum of the natural fluoride content, the fluoride content of the coagulants used and the fluoride dose added. Dry chemical feeders are often calibrated for aluminium sulphate or lime before despatch from the factory; they must therefore be calibrated for use with whichever fluoride chemical is used BEFORE they are put into service.

Since fluorides are removed to some extent by coagulation and to a marked extent by lime-soda softening, the fluoride dose must be calculated on the fluoride content of the filtered water in the clearwell and additional fluoride must be dosed at this point
.

WATER SOURCES and WATER TREATMENT.

PART 12.

CONTROL OF A WATER TREATMENT WORKS.

In order to exercise proper control over a water treatment works and to ensure that the best possible water is produced at the lowest unit cost, it is necessary that certain parameters be determined. One of the most important is the flow through the plant, upon this depends the feed rate of the chemicals, i.e. the dosage control and this aids the quality assurance.

12.1 FLOW MEASUREMENT.

It is important to know the daily volume of water produced by the treatment works as well as the actual flow rate. To do this one needs a measuring flume of one kind or another, with an indicating and integrating flow meter reading the level of the water in the flume. It is useful if there is a paper chart recording function as well. Both flow rate and total flow figures are needed because a treatment works may be designed to treat say 8 640 m^3 per day i.e. a mean rate of 100 L/s, whereas peak flows of say 200 L/s may be encountered during the day. When sand filter backwash water is returned to the head of the works, this flow should also be metered as it may contribute to temporary overloading of the works. Also, if the quantity of backwash water is increasing, this may indicate operational problems in the cleaning of the sand filters.

Flow measurement is vital if automatic or semi-automatic operation of the works is planned. When water is abstracted from the ground it is important to know the quantity of water withdrawn and to equate this to the level of the groundwater. This is to determine if mining of the water is occurring i.e. the permanent lowering of the water table. These figures may be used to estimate the ultimate capacity and the useful life of the particular water source.

12.2 FEED RATE OF CHEMICALS.

Usually before a treatment works is designed and constructed, laboratory tests have been done to determine the type and dosage of treatment chemicals to be used. On the treatment plant, dosages may not be the same or different chemicals may produce superior results from those found in the laboratory. This may be due to the physical size of the units and the degree of mixing, turbulence, etc. It is not possible to reproduce these exactly in the laboratory. The quality of the raw water may have changed since the initial tests as the result of less or more rainfall, a change in land usage in the catchment area, etc.

It is necessary to be able to change the feed rate of the chemicals as the quality of the raw water changes. In very cold climates, the temperature of the water may drop so low that sedimentation is affected and coagulation aids may be required.

12.3 DOSAGE CONTROL.

In 12.2, the necessity of being able to change the feed rate of the chemicals to match the change in the quality of the water was seen. It must also be necessary to be able to change the feed rate of the chemicals to match the change in the quantity of the water, i.e. the dosage control.

Dosage control may be either manual, automatic or automatic closed loop control.

12.3.1 Manual Control of Feed Rate.

Here the Process Controller has to watch the raw water flow indicator and when the flow rate increases they has to increase the chemical feed rate and vice-versa. This is not an efficient method of dosage control as a Process Controller will be tied up continuously doing the one job when their duties include backwashing sand filters or doing laboratory tests, etc. The Process

Controller may be busy when the increased flow enters the works and will be delayed in increasing the chemical feed rate. One may even miss a period of high flow.

12.3.2 Automatic control of feed rate.

Here the rate of dosing is made proportional to flow, i.e. a 20% increase in flow rate means a 20% increase in chemical dosing rate. This control may be achieved in a number of ways. This is an improvement over the manually controlled system particularly if the flow rate is very variable.

The control system does not take into account any variation in the quality of the water.

12.3.3 Automatic Closed Loop Control.

This is the most sophisticated type of control system where the monitoring unit measures the desired parameter, e.g. pH and compares it with the preset value and alters the dosing rate in order that the desired parameter remains at, or at least very close to the preset value.

Certain parameters such as pH and residual chlorine are simple to measure. The development of a system to vary the coagulant close to ensure optimum efficiency could be difficult.

12.4 QUALITY ASSURANCE.

It is important that routine monitoring of raw, in-process and treated waters be undertaken. The routine analysis of the raw water will detect any change in the quality of the water possibly as the result of contamination or the drawing in of a different groundwater as a result of lowering the water table. Tests and experiments can help one decide whether to draw from near the surface or deep in a dam.

Monitoring of in-process waters for coagulation and sedimentation efficiency can warn one of impending problems in the sand filtration stage.

Monitoring of the treated water is necessary to ensure that the water meets the required standard and to ensure that it is neither aggressive or excessively scale forming (See Section 13.9).

12.5 THE KEEPING OF RECORDS.

The keeping of records has little purpose unless they are available for ready reference and record results and occurrences in a form which can be used as a guide to solving of future problems. Records represent condensed information; they should be systematically kept and regularly filed, to be referred to as needed.

The type and complexity of record forms required depends on the size and character of the treatment works. Simple forms may be adequate for a chlorination plant treating a well supply, they would provide space for the time of starting and stopping of pumps, quantity of electricity or fuel used, volume of water treated, chlorine dose, residual chlorine and a remarks column for recording pertinent events or observations.

By contrast, very extensive records must be kept at large lime-soda softening plants where fluoridation is carried out and where laboratory control is available. Under these circumstances shift Process Controllers should keep individual records of the various chemical doses, volume of water treated, etc., at hourly intervals on forms suitable for an 8 hour shift or for a 24 hour period. The information recorded on these working forms should be transcribed daily onto daily summarising forms. A separate form would be used

The daily summary of all the relevant data should be on forms having lines for the 31 days, a total and an average.

The actual parameters that are monitored depend on the situation. Water temperature and meteorological conditions will be relevant for certain treatment works only.

A very important aspect of records that is often ignored is the detailing of information on the routine and non-routine maintenance and repair of equipment. Protective maintenance must be organised according to the instruction manuals for each piece of equipment. For example, a chemical feeder should be lubricated and checked for functioning characteristics each day, shut down for cleaning and adjusting once a month, and given an overhaul once or twice per year. The records of such protective maintenance will serve as a reminder of the need for such periodic checking to prevent the failure of equipment.

One of the more important pieces of information that is usually lacking at a treatment works is a set of "as built" mechanical and civil drawings. These are essential to the economical operation and maintenance of any plant, especially the more elaborate equipment at large filtration and softening plants. It is important that the Process Controllers study these so that they know the location, characteristics and functions of all pieces of equipment. Colour coding should be used to indicate the nature of piping, conduits, electric wiring, etc. in the treatment works, for guidance in repairs and maintenance.

12.6 THE MONITORING OF A WATER TREATMENT WORKS

Laboratory examinations of water are probably most frequently undertaken to aid in forming an opinion of the suitability of a water supply for public use. This involves consideration of several factors: whether the water is safe for human consumption as shown by the presence or absence of pollution, whether it is corrosive to metal pipes or will form scale in cold or hot water systems, whether it is of attractive appearance and taste, whether it is satisfactory for use in home laundry and washing, and whether it can be used for industrial purposes. Regular laboratory analyses are essential for the control of water treatment processes and to ensure a satisfactory effluent at all times. The various tests are tools that supplement and extend the human senses. Thus the gradual deterioration of coagulation and filtration processes can be detected by laboratory determinations before it becomes evident to visual observation: microscopic organisms can be magnified and counted, variation in dissolved constituents of water can be detected and the reasons for changes in plant operation discovered so that corrective measures can be adopted.

Unless the significance of the tests is well understood, however, they are of little value to Process Controllers. The results of these tests should be interpreted in the light of sanitary surveys of the sources of water supply, and conversely, used for additional information as to the character of the water being treated. Many of the tests are small-scale replicas of plant operation by means of which chemical doses or methods of treatment can be tried before application in the treatment plant. Only the tests concerned with the control of water treatment processes will be considered.

Laboratory examinations may be divided into 4 sections:

1. physical tests that measure and record the properties detectable by the senses;

2. chemical tests that determine the amounts of mineral and organic material present that affect the quality of the water and provide data as to past pollution and show variations due to treatment. The latter is a necessary requirement in the control of water treatment processes;

3. bacteriological examinations indicate the presence of bacteria characteristic of pollution and hence the safety of the water for consumption;

4. microscopic examinations provide information concerning growths in water that are often the cause of unpleasant tastes and odours or of clogging of filters;

The publication "Standard Methods for the Examination of Water and Wastewater" jointly published by A.P.H.A., A.W.W.A. and W.P.C.F. is the generally accepted guide when laboratory facilities are available to chemists. It is important that these standard procedures are followed when facilities and trained personnel are available.

For the colourimetric tests, various makes of colourimeter and/or comparator may be used. Usually a portable comparator is used. These have a disc or slide containing a number of colour standards each with its respective concentration or value.

The value of any laboratory test depends on the method of sampling. The samples must be representative of the water to be examined or the results will have no significance. The frequency of collection of samples for chemical analysis depends on the uniformity in the quality of the raw water, the types of treatment processes used, etc.

12.7 QUALITY CONTROL TESTS FOR WATER.

To ensure that the water treatment plant is effective in treating the raw water and to ensure that the water supplied to the consumer is of the required quality a number of physical, chemical, bacteriological tests are required. These are detailed below.

12.8 COLOUR.

12.8.1 Discussion.

The colour of water is commonly caused by the extraction of colouring material from the humus of forests or the deposit of vegetable matter in swamps and low-lying areas. This colouring matter is composed of humus and tannic acid compounds which cause the yellowish/brown tea colour of certain surface waters. The colour of water is of two types: "true-colour" that is present in the water after the suspended matter has been removed, and "apparent colour", that is true colour plus any other colour produced by substances in suspension. In certain cases colour may be imparted to water by dissolved iron or by the discharge of certain industrial wastes. Generally colour has little relation to pollution except as indicating that surface water has reached ground water supplies. The attractiveness of water is markedly dependent on the colour to which the public of any locality has become accustomed. The removal of colour is a function of water treatment, and so decrease in colour of the water may be taken as a measure of plant efficiency. Colour is measured on the Hazen Scale in units. One unit on this scale is the colour produced by 1 mg/L of platinum as Pt present as potassium chloroplatinate K_2PtCl_6 dissolved in 2 mg/L cobaltous chloride solution $CoCl_2.6H_2O$.

12.8.2 Sampling.

Samples of both raw and treated water should be taken. Since the biological activity may change the colour characteristics of a sample, the analysis should be made as soon as possible. If necessary a sample may be stored at 4°C for 24 hours.

12.8.3 Equipment.

The Taylor water analyser with slide number 1300A or the Lovibond comparator with the Nessler attachment with the disc number NSA may be used. With the latter instrument approximate results in the higher colour range may be obtained by using disc NSX.

12.8.4 Reagents needed.

The standard chloroplatinate solution is prepared by dissolving 1.246 g potassium chloroplatinate and 1 g cobaltous chloride $6H_2O$ in a solution containing 100 ml concentrated hydrochloric acid and making up to 1 litre. This standard solution is equivalent to 500 units. Standard dilutions of 5 to 70 units are prepared. The colours on the slide or disc may then be checked. If the colours correspond then the disc may be used in place of the standard solutions.

12.8.5 Procedure.

The sample tubes are filled to the mark with sample and distilled water as reference. The slide is moved or the disc rotated until the two areas of colour are the closest match. The colour is read off on the instrument. This colour is the apparent colour; if the sample is crystal clear then this is also the true colour. Any turbidity present may be removed by centrifuging or by filtration. Reading the colour of the clarified sample gives the true colour. If the sample has a colour exceeding 70 units, then the sample must be diluted with distilled water. With the Lovibond unit, the colour may be estimated with the high range disc.

12.8.6 Reporting of results.

Colour is reported as colour on the Hazen scale as shown in Table 22:

TABLE 22 – REPORTING OF COLOUR ON THE HAZEN SCALE.

Range	Report to nearest number of units
1 to 50	1
51 to 100	5
101 to 250	10
Greater than 250	20

12.8.7 Interpretation.

In general, consumer complaints will be received if the colour is great than 10 - 15 units. A colour of less than 5 units is desirable. Coagulation and filtration should reduce the colour to less than 5 units. Slow sand filters generally remove 50% of the colour of the raw water. Gradual increase in colour of the filtered water is an indication of impaired efficiency of either coagulation or filtration. Increase in the colour of a supply as it flows through the distribution system may be caused by corrosion or growths in the pipes.

12.9 CHARACTERIZATION OF ODOURS.

12.9.1 Discussion.

Odours in water are caused by extremely small concentrations of volatile compounds. Some of these compounds are produced when organic matter decomposes and are, therefore, likely to be present in surface waters because of the presence of organic matter from surface wash. Their intensity and offensiveness vary with the type, some being earthy and musty while others are putrefactive. Some odours of surface water are produced by pollution from industrial wastes typical of certain industries, such as phenol or oil waste. For the most part, objectionable odours in surface waters are caused by plankton, i.e. free-floating microscopic organisms. These organisms liberate minute traces of volatile essential oils, which impart various objectionable odours according to the type and concentration of the oils liberated. Some are aromatic, others are sweetish and flowery, and still others are fishy.

While organisms are increasing in concentration the odours are not as strong as when they are decreasing and the dead organisms are decomposing. Other various types of odours are produced by the addition of chlorine to water. In some cases odours are accentuated by marginal chlorination. In other cases chlorinous odours are formed that were not noticeable before the chlorine was added. In still other cases, the type of odour is changed from an unobjectionable one to one that may cause complaints. On the other hand, free-residual chlorination frequently destroys odour-producing substances.

Because odour-producing substances are usually present in extremely small concentration, and are usually complex mixtures, analytical procedures are unsatisfactory for their measurement and reliance is placed on the sense of smell. This varies with individuals, and therefore the results will vary somewhat. The sense of smell is easily fatigued; the ability to detect slight odours is rapidly lost when used for any length of time or if strong odours are encountered. It is important that the test be performed in an odour-free room and with odour-free equipment.

12.9.2 Sampling.

The odour is generally determined only on the raw and treated water. It is recommended that samples be taken from points in the distribution system at intervals. Samples must be taken in spotlessly clean and odour-free glass bottles, plastic bottles must not be used. The samples should be taken with as little aeration as possible or odour could be lost. When hydrogen sulphide is present the odour determination must be made immediately after sampling, as hydrogen sulphide is liberated by exposure to the air, or is modified by dissolved oxygen. It is usual to fill the bottle about 2/3 full.

12.9.3 Procedure.

If needed, warm the sample bottle and its contents to room temperature, shake the bottle, remove the stopper and sniff the odour at the mouth of the bottle.

12.9.4 Reporting of results.

The odour should be indicated or described according to the following code given in Table 23:
The intensity of the odour maybe graded into the following 5 grades.

12.9.5 Interpretation of Results

The character and intensity of odour often discloses the nature of pollution or the prevalence of micro-organisms and hence the type of treatment or preventative measures needed. Consumers will judge a supply by its taste and odour; while many consumers will become accustomed to a prevailing odour of low intensity, the object should be the delivery of water having an intensity rating of not more than 1 on the above scale.

The above procedure is not precise enough when control of taste and odour presents problems and costly treatment processes are involved. In such cases the Threshold Odour test is recommended.

TABLE 23 – ODOUR INTENSITY GRADE AND DESCRIPTION.

INTENSITY GRADE	DESCRIPTION
0	NONE - no odour perceptible,
1.	VERY FAINT - an odour that would not be detected ordinarily by the average consumer but could be detected in the laboratory by an experienced observer;
2	FAINT - an odour that the consumer might detect if his attention were called to it, but that would not attract attention otherwise,
3	DISTINCT - an odour that would be detected readily and that might cause the water to be regarded with disfavour,
4	DECIDED - an odour that would force itself upon the attention and that might make the water unpalatable
5	VERY STRONG - an odour of such intensity that the water would be absolutely unfit to drink. (This rating is used in extreme cases only.)

TABLE 24 – TYPES OF ODOURS AND DESCRIPTION THEREOF.

CODE	NATURE OF ODOUR	DESCRIPTION OR POSSIBLE CAUSE OF ODOUR
A	AROMATIC OR SPICY	LIKE CAMPHOR, CLOVES OR LEMON
Ac	Cucumber	May be caused by Protozoan Synura
B	BALSAMIC OR FLOWERY	Like geranium, violets or vanilla
Bg	Geranium	May be caused by the alga Asterionella
Bn	Nasturtium	May be caused by alga Aphonizomaenon
Bs	Sweetish	May be caused by alga Coelosphaerium
Bv	Violets	May be caused by protozoan Mallomonas
C	CHEMICAL	like odour of industrial waste
Cc	Chlorinous	like odour of free chlorine
Ch	Hydrocarbon	like odour of oil refinery wastes
Cm	Medicinal	like odour of phenol
Cs	Sulphuretted	like odour of hydrogen sulphide
D	DISAGREEABLE	PRONOUNCED UNPLEASANT ODOUR
Df	Fishy	may be caused by protozoan Dinobyron
Dp	Pigpen	may be caused by algae Anabaena
Ds	Septic	like odour of stale domestic W.W
E	EARTHY	like odour of damp earth
Ep	Peaty	like odour of peat
G	GRASSY	like odour of crushed grass
M	Musty	like odour of decomposing straw
Mm	Mouldy	like odour of damp cellar
V	Vegetable	like odour of root vegetables.

12.10. SEMI-QUANTITATIVE DETERMINATION OF ODOUR LEVEL BY THE THRESHOLD-ODOUR TEST.

12.10.1 Discussion.

This test is based on the dilution of portions of a sample of water with odour-free water until the mixture has only a barely discernible odour. The "threshold odour number" is the number of times the sample must be diluted with odour-free water for the odour to be just detectable. e.g. if 50 ml must be diluted to 200 ml for the odour to be just detectable then the threshold number is 4.

12.10.2 Sampling.

The same procedure as for the simple odour test is used.

12.10.3. Equipment.

Erlenmeyer flasks - glass stoppered 500 mL; thermometer up to 50°C; measuring cylinders 10, 50 and 200 mL; pipettes 10 mL graduated in 0.1 mL, 1 ml graduated in 0.1 mL; glass stoppered sample bottles 500 mL and 2 L; hotplate and an odour free water generator, 1 L beaker.

12.10.4 Reagents needed.

Odour-free water.

12.10.5 A Brief Explanation of the Procedure.

1. Heat the sample to 40°C, place bottle in water in a 1 L beaker, place clean thermometer in sample. Use 200 mL in 500 mL bottle;

2 When at 40°C remove thermometer and replace stopper, shake the bottle;

3. Sniff the odour by removing the stopper and holding the mouth of the bottle to the nose;

4. Record the type and intensity of the odour as indicated under 12.9.4. above. If the odour of this sample is only just discernible then the threshold odour number is 1. If the intensity is higher then proceed with step 5;

5. Place 200, 50, 17 and 4 mL of fresh sample respectively in 4 flasks and dilute the last 3 to 200 ml with odour-free water to determine the approximate range of the threshold odour number. Add 200 mL of odour-free water to a fifth flask;

6. Heat the samples and the odour-free water to 40°C before testing;

7. Shake the flask containing the odour-free water, remove the stopper and sniff the odour. **(Always start comparing odours with the flask containing odour-free water, followed by the flask containing the most diluted sample**;

8. Compare this with the odour of the mixture containing 4 ml of sample diluted to 200 ml, (i.e. 50 X dilution);

9 If the mixture has no odour, then compare the odour-free water with the mixture containing 17 ml sample and so on until a definite odour is noticed from the mixture in one of the flasks;

10 If the most diluted sample still had an odour, then a more dilute sample containing 2 ml diluted to 200 ml must be prepared;

11. Having established the approximate range of the threshold odour number, a series of 5 dilutions must be made, starting with the dilution that had the definite but faint odour;

12. Determine which sample has the just discernible odour and then refer to Table 25.

The dilutions for the various odour intensities are also given in Table 25:

12.10.5 Interpretation of Results.

The character and intensity of odour often discloses the nature of pollution or the prevalence of micro-organisms and hence the type of treatment or preventative measures needed. Consumers will judge a supply by its taste and odour; while many consumers will become accustomed to a prevailing odour of low intensity, the object should be the delivery of water having an intensity rating of not more than 1 on the above scale.

TABLE 25 – THRESHOLD ODOUR NUMBER CALCULATED FROM VOLUME OF SAMPLE USED FOR DILUTION.

Sample volume of water to be diluted to 200 mL	**Odour-free dilution water to make up to 200mL**	**Threshold Odour Number (TON)**
200	0	1.0
140	160	1.4
100	100	2.0
70	130	2.9
50	150	4.0
35	165	5.7
24	176	8.0
17	183	11.8
12	188	16.7
9	191	22.2
6	194	33.3
4	196	50.0
3	197	66.7
2	198	100
1	199	200

Example:

If the test showed that a mixture of 24 mL sample and 176 mL odour-free water had a just discernible odour, then the threshold odour number would be 8.

12.11 TASTE.

12.11.1 Discussion,

Taste in water is generally closely related to odour and is caused by the same conditions; however dissolved mineral matter may impart taste but no odour to the water. The presence of copper, zinc or iron in the water may cause a metallic taste. The chlorination of water containing phenol type compounds may have a distinct taste at concentrations below that detectable as odour.

12.11.2 Sampling.

Tests are made for taste if the water is safe to drink; the same samples are generally used as were collected for the odour determination. In order to check the taste of a chlorinated treated water, the residual chlorine must be removed. The residual chlorine content is determined as described later and a stoichiometric quantity of sodium thiosulphate is added. 1mg/L of residual chlorine requires 2.22 mg/L of sodium thiosulphate to remove the residual chlorine.

12.11.3 Procedure.

1. Warm sample to room temperature.

2. Pour a small quantity of sample into a clean beaker.

3. Taste.

12.11.4 Results.

Record the characteristics and intensity of the taste according to the same code as per Table 26.

TABLE 26 - TASTE INTENSITY GRADE AND DESCRIPTION

INTENSITY GRADE	DESCRIPTION
0	NONE - no taste perceptible,
1	VERY FAINT – a taste that would not be detected ordinarily by the average consumer but could be detected in the laboratory by an experienced observer;
2	FAINT – a taste that the consumer might detect if his attention were called to it, but that would not attract attention otherwise,
3	DISTINCT – a taste that would be detected readily and that might cause the water to be regarded with disfavour,
4	DECIDED – a taste that would force itself upon the attention and that might make the water unpalatable
5	VERY STRONG – a taste of such intensity that the water would be absolutely unfit to drink. (This rating is used in extreme cases only.)

It is possible to describe the taste in a similar manner as to that used in odour determination. Table 24 (above) could be used as a guide.

12.11.5 Interpretation.

Taste is a measure of consumer acceptance of water, although as with odour, a change in character may cause complaints if consumers have become accustomed to one type of taste. Generally, finished water should have a taste intensity of 2 or less. A metallic salty taste may indicate pollution in the water supply.

12.12 TURBIDITY.

12.12.1 Discussion.

Turbidity is the optical effect caused by dispersion of and interference with light rays passing through water containing small particles in suspension. It may be caused by silt extracted from soil, surface wash containing suspended organic and mineral matter, precipitated calcium carbonate in all waters, aluminium hydroxide floc in treated water, precipitated iron oxide in corrosive waters, microscopic organisms, etc. Measurement of turbidity is important because it is one of the visual factors affecting the consumer acceptance of water. In well water, it may indicate the entrance of surface wash and thus potential contamination. The turbidity of raw waters influences the doses of coagulants required for their clarification and gives an indication of the effectiveness of coagulation. If alum floc is passing the filters, or if floc is being formed in the clear well instead of in the coagulation basin, this will show up as turbidity.

The degree of turbidity depends on the concentration, fineness or dispersion of the suspended matter and the light-absorption properties of the suspension. Turbidity is measured on a numerical scale. The original scale was the Jackson Turbidity Unit Scale J.T.U.; this has been superseded by the Formazin Turbidity Unit's Scale F.T.U. The former uses fuller earth as a standard - 1 mg/L fuller's earth = 1 J.T.U. The latter uses Formazin as a standard. The particle size of the fuller's earth varies and so 1 mg/L of fuller's earth does not necessarily produce the same degree of turbidity as the same concentration of finer or coarser material. The FTU has in turn been replaced by the Nephelometric Turbidity Unit (NTU).

12.12.2 Sampling

Samples of raw water should be taken to determine the fluctuation in raw water quality and the amount of turbidity to be removed by treatment. Samples of coagulated and filtered effluent should be taken to show the effectiveness of coagulation and filtration. Samples from the distributing system will allow comparison with filter effluent to determine whether turbidity is being increased in the distribution system. Samples from the distribution system should be collected at predetermined points selected to be representative of the entire volume of water, not affected by dead ends or sudden variations in flow.

12.12.3 Equipment.

The turbidity of water may be determined by a photo-electric turbidimeter.

Photo-electric turbidimeters are available for use in the range 0.1 to 1 000 N.T.U. Specially prepared stabilized Formazin standards are used to calibrate the instrument each time that it is used. They are simple to use and are reliable. They may be used for continuous monitoring and the results may be recorded on paper. The instruments may be equipped with a suitable alarm system that is triggered by the turbidity exceeding a pre-set value.

The earlier turbidimeters were based on the dispersion of light on passing directly up the sample tube. Absorption of light may also occur; this has the effect of artificially increasing the turbidity.

An improvement on the turbidimeter is the nephelometer, this uses the principle that light is passed up the tube and is reflected by the particles causing turbidity, a photocell is placed at right angles to the original path of the light. This photocell can only record light that has been reflected by the particles.

Nephelometers are calibrated in Nephelometric Turbidity Units. N.T.U.

In the low ranges 1 J.T.U. = 1 F.T.U. = 1 N.T.U. Formazin standards are used.

The method of turbidity standards consists of a series of fresh standards in a set of identically sized bottles as the samples and estimating the turbidity of the sample by comparing it with the standards.

12.12.4 Interpretation.

If the turbidity of the water leaving the sedimentation tank is high then the runs on the sand filter will be shortened. Turbidity of in-process water may be due to poor coagulation or poor flocculation or even both.

12.13 HARDNESS (Total, using the soap test)

12.13.1 Discussion.

Water is a universal solvent and dissolves varying amounts of different mineral substances. Those producing hardness do not affect the sanitary quality but are of importance in the domestic use of water, particularly for laundry and boiler purposes. Calcium and magnesium salts, the principal mineral constituents, consume soap and precipitate as insoluble compounds or soap curds. Until all the calcium and magnesium is precipitated, no lather or washing action is obtained from the soap. The soap-consuming power of water is therefore a measure of its hardness.

Calcium and Magnesium generally are present at the soluble bicarbonate which will change on heating to the less soluble carbonate with precipitates and is one source of scale in distributing systems and hot-water heaters. Hard waters are usually less corrosive than soft waters.

12.13.2 Sampling.

Where softening of the water is not undertaken, the hardness is generally determined only on the finished water, i.e. from the clear well and from selected points in the distribution system.

Where softening processes are used, it will be usual to have a more sophisticated laboratory and the total hardness will be determined on the raw and in-process water as well as by using a titrimetric method (E.D.T.A.).

12.13.3 Equipment.

Burette 50 mL; pipette 50 mL; glass stoppered bottle 250 mL; standard soap solution.

12.13.4 Procedure.

1. Fill the burette to zero mark with standard soap solution. This may be bought as Wanklyn's Standard Soap Solution. This can be made with 5 grams of soap dissolved in 50mL warm water and then diluted to 500mL with 50% alcohol mixture;
2. Measure out 50 mL of sample into glass stoppered bottle;
3. Add standard soap solution from the burette in small quantities (0.5 mL) to sample in the bottle. Shake vigorously 25 times after each addition of standard soap solution. Add smaller quantities as the end point is reached;
4. Lay the bottle on its side and observe the lather;
5. The end point is reached when the lather persists for at least 5 mins;
6. Read the burette;
7. To ensure that the end point is the correct one, add an additional 0.5 ml of soap solution, shake and observe lather. A lather that then does not persist for 5 minutes indicates that the tentative end point was a false end point and the titration must be continued until the true end point is reached;
8. The lather factor must be subtracted from the titration volume to obtain the corrected volume. (This is the volume of soap solution that must be added to 50 mL of distilled water to give the standard lather, it is about 0.3 mL);
9. If the titration volume is greater than 7 mL then a smaller sample volume must be used, which must be diluted to 50 mL with distilled water.

12.13.5 Results.

Hardness is reported as mg/L as CaCO3

$$\text{Hardness as mg/L } CaCO_3 = \frac{\text{(mL soap solution - lather factor) x 1 000}}{50 \textit{ (being the original sample volume)}}$$

12.13.6 Interpretation.

Water for domestic and laundry use should preferably contain less than 100 mg/L hardness.

12.14 ALKALINITY.

12.14.1 Discussion.

Alkalinity may be thought of as a measure of the resistance of a water to the change in pH following the addition of an acid or an alkali. In natural waters, it is as a mixture of the Carbonate ion ($CaCO_3^{=}$); the Bicarbonate ion HCO_3^{-}) and the so-called Carbonic Acid (H_2CO_3). The letter is mainly a solution of dissolved carbon dioxide gas in the water. Alkalinity can exist as the hydroxide (OH^{-}). The various forms of alkalinity present at various pH values are shown in Table 26 and figure 36.

Sufficient alkalinity is necessary in the coagulation phase of water treatment. This is because, with the exception of Sodium Aluminate, the other commonly used coagulants are acidic. Without sufficient alkalinity, the addition of the coagulant could result in the pH dropping below the optimum point for the coagulation process. In poorly buffered water waters, it may be necessary to use sodium aluminate in conjunction with, say, aluminium sulphate, so that the alkaline nature of the former balances the acidity of the latter.

Other alkaline chemicals such as sodium carbonate or lime may be used - but these both have disadvantages.

TABLE 26 – CARBONIC SPECIES AT VARIOUS pH VALUES

pH	9.4	OH^-	$CO_3^=$
		HCO_3^-	$CO_3^=$
	8.3	HCO_3^-	CO_2
	4.5	CO2 plus free mineral acidity	

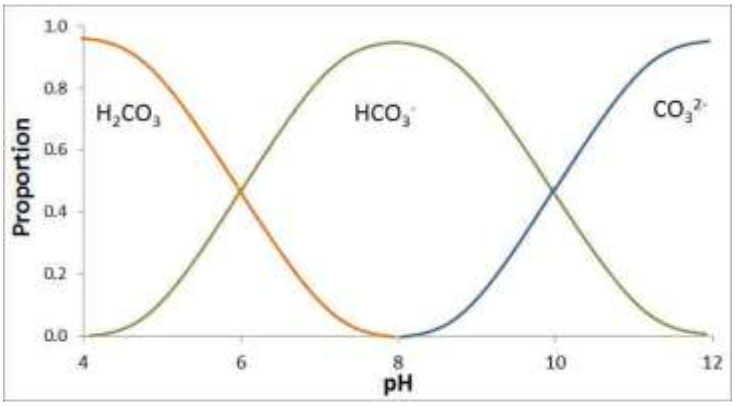


Figure 35 – SHOWING PROPORTION OF EACH CARBONIC SPECIES AT VARIOUS pH VALUES.

12.14.2 Sampling.

Samples of raw and treated waters would be analysed.

12.14.3 Equipment.

Burette, 0.20 N H_2SO_4; phenolphthalein solution; methyl orange indicator; Erlenmeyer flask; 100mL measuring cylinder.

12.14.4 Procedure.

1. 100 mL of sample is poured into a 250 mL Erlenmeyer Flask;
2. Add 2 drops phenolphthalein indicator, if it turns pink then pH is above 8.3, then titrate with 0.2 N H_2SO_4 till just colourless pH 8.3 - note titration volume;
3. The titration volume times 10 is the phenolphthalein alkalinity in mg/L as $CaCO_3$;
4. If the solution on adding phenolphthalein remains colourless or the pH is 8.3 or less, then the phenolphthalein alkalinity is zero;
5. The phenolphthalein alkalinity is also known as the P. alkalinity;
6. Using the same solution as above, add 2 drops of methyl orange indicator;
7. Titrate to pH 4.5 or to the first permanent faint pink colour. Screened methyl orange may be used with a grey end-point - note titration volume;
8. The titration volume times 10 is the methyl orange alkalinity in mg/L as $CaCO_3$;
9. The methyl orange alkalinity is also known as the T. alkalinity.

12.14.5 Interpretation.

The relative proportions of the carbonic species have been seen in figure 34. The so-called P alkalinity (phenolphthalein) and the so-called T alkalinity (total) have been determined by titration in 12.14.4, one can now draw certain conclusions.

If the sample is alkaline to phenolphthalein that is the pH is greater than pH 8.3 with a pink colour, then two cases arise:

1. where the P. alkalinity is less than half the T. alkalinity;
2. where the P. alkalinity is more than half the T. alkalinity.

In the first case, the pH will lie between 8.3 and 9.4 and in the second case the pH will be greater than 9.4. In the first case, bicarbonate and carbonate alkalinity will be present and in the second case, carbonate and hydroxide alkalinities will be present.

If the P. alkalinity is equal to the T alkalinity then only hydroxide alkalinity occurs and we say that there are no carbonic species present, this is unlikely to occur in drinking water.

If the sample is acid to phenolphthalein (that is colourless on adding phenolphthalein) and alkaline to methyl orange, i.e. pH is in the range of 4.5 to 8.3; the alkalinity will be in the bicarbonate form.

If a sample is acid to methyl orange, i.e. pH less than 4.5; then mineral acids may be assumed to be present. This will mean that the water is polluted by industrial or mining wastes as natural waters do not contain mineral acidity.

It is important to note that the P. alkalinity CANNOT be greater than the T. alkalinity.

12.15 THE DETERMINATION OF CARBONATE AND NON-CARBONATE HARDNESS.

The tests for total hardness and total alkalinity give results which can be used in the control of the lime-soda softening process where the non-carbonate hardness determines the soda ash dose and the carbonate hardness in part determines the lime dose needed to secure the desired degree of softening. Non-carbonate hardness does not contribute alkalinity, as it is due to neutral salts of calcium and magnesium, viz. chlorides and sulphates.

12.16 THE DETERMINATION OF OTHER ALKALINE SUBSTANCES.

The sodium alkalinity is the difference between the total alkalinity and the total hardness where the total alkalinity is greater than the total hardness.

The determination of alkalinity provides an estimate of the basic constituents. If there were present only as salts of calcium and magnesium, the alkalinity will be equal the hardness - as both are expressed as mg/L as $CaCO_3$.

Two conditions will arise here:

1. If the alkalinity is greater than the hardness then there are basic salts other than those of calcium and magnesium - usually sodium and potassium salts;
2. If the alkalinity is less than the hardness, then neutral salts of magnesium and calcium must be present - usually sulphates and chlorides.

Coagulation generally needs an alkalinity numerically equal to half the alum dose used to provide a good floc. That is 20 mg/L alum needs l0 mg/L alkalinity to ensure the formation of a good floc.

As the use of alum, ferric sulphate, etc. reduces the alkalinity, the water becomes more corrosive. If the alkalinity of the raw water was not enough, then extra alkalinity as soda ash or lime must be added.

12.17 THE DETERMINATION OF CARBON DIOXIDE CONTENT.

12.17.1 Discussion.

The content of the carbon dioxide in water is directly related to the type and degree of alkalinity and the pH value. The carbon dioxide content has an effect on coagulation, corrosion prevention, softening and iron removal.

12.17.5 Sampling.

Raw and treated waters will be analysed for carbon dioxide content. The accuracy of the determination is affected by aeration, it is essential that the sample be collected with as little agitation as possible. It is preferable that the pH be determined immediately.

12.17.6 Procedure.

1. Determine the pH of the sample;
2. Determine the T. alkalinity. (Methyl Orange Alk.);
3. Read off the Carbon Dioxide content on a suitable nomogram such as figure 36.

For more accurate work determine the temperature of the water, the total dissolved solids content and use a nomogram such as that in Standard Methods.

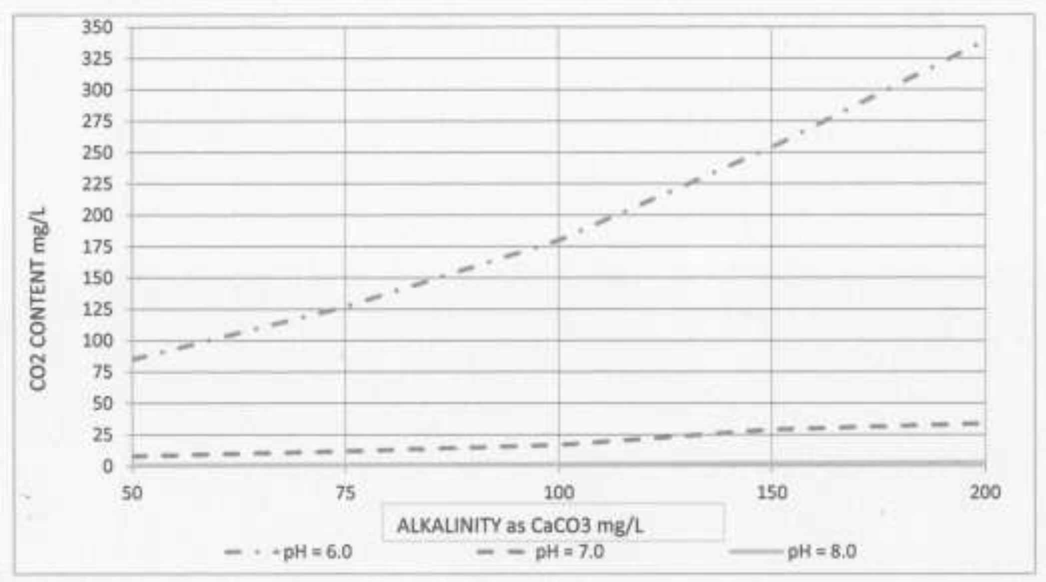


Figure 36 – NOMOGRAM FOR ESTIMATING CARBON DIOXIDE CONTENT BASED ON pH AND ALKALINITY VALUES.

12.18 THE DETERMINATION OF PH VALUE.

12.18.1 Discussion.

The pH value measures that intensity of the acid or alkaline reaction of the water. It is important to note that it does not measure the total concentration or mass of acid or alkali present. Water that has a neutral reaction is termed pH 7. Acids reduce the pH value and alkalis raise the pH value.

12.18.2 Sampling.

Raw, in-process and treated waters will be analysed. pH determinations should be done as soon as possible.

12.18.3 Procedure.

1. Using pH meter - follow the maker's instructions.
2. Colourimetric Method - pH indicators are used. Usually some or all of the following are used as listed in table 27.

TABLE 27 – INDICATORS USED FOR pH ESTIMATION.

INDICATOR	pH RANGE FOR COLOUR CHANGE	COLOUR CHANGE
Chlorphenol Red	pH 5.2 to 6.8	Yellow to Red
Bromthymol Blue	pH 6.0 to 7.6	Yellow to Blue
Phenol Red	pH 6.8 to 8.4	Yellow to Red
Thymol Blue	pH 8.0 to 9.6	Yellow to Blue
Methyl Orange	pH 3.1 to 4.6	Red to Yellow
Methyl Red	pH 4.2 to 6.2	Red to Yellow

The Taylor Water Analyser or Lovibond Comparator may be used.

It is seen that indicators with overlapping ranges are used. This is because if a sample showed pH 6.8 with phenol red, the pH could in fact be lower as a lower pH would still show exactly the same colour as pH 6.8. One would repeat the determination using Bromthymol Blue.

It is worth having standardised buffers whose pH values are accurately known and may be used to check that the indicator colours are still true.

12.18.4 Interpretation.

The pH value of a water has an influence on virtually every aspect of water treatment from aeration to chlorination.

12.19. THE JAR TEST.

12.19.1 Discussion.

The coagulation and flocculation of water is influenced by a number of inter-related variable factors such as temperature, turbidity, colour, pH, alkalinity, the period and degree of agitation during flocculation, and the characteristics of the coagulant or coagulants in use. The optimum dose of coagulant cannot be determined from the results of water analysis but must be based on experience with any given water. If an unknown water is to be treated, then the jar test will be used to determine the most suitable conditions to be used.

It is usual to start with aluminium sulphate and lime and later if necessary to experiment with ferrous and ferric salts.

12.19.2 Sampling.

A large sample of the raw water under study should be collected so that all portions tested will be from the same sample. At least 25 litres is required and preferably 50 litres.

12.19.3 Equipment.

4 or 6 place laboratory flocculator; 1 litre beakers; equipment for determining colour; turbidity; pH; alkalinity; stock solutions of various coagulants containing 1 g/L.

12.19.3 Procedure.

1. Determine the colour, turbidity, pH alkalinity, of the raw water;

2. Measure 1 litre portions into each beaker and place on laboratory flocculator;

3. Switch on and run at full speed;

4. Add predetermined doses of coagulants;

5. Allow to flash mix for about 1 minute;

6. Reduce paddle speed to about 20 rpm, and flocculate for about 20 minutes;

7. Note and compare type and rate of formation of floc;

8. Stop paddles and allow floc to settle for 1 hour;

9. Withdraw about 200 mL of supernatant and determine colour, turbidity, pH and alkalinity;

10. If one or more samples has a colour of less than about 20 or a turbidity of less than about l0, filter and aliquot through filter paper 540 and determine the colour, turbidity, and alkalinity. The filter paper can influence the pH of the water, so the pH determined on the settled water will be used. (This last step will simulate the sand filter);

11. If no samples give suitable results, try different concentrations of coagulant, different pH values or coagulation, different coagulant or mixtures of coagulants.

12.19.4 Interpretation.

The test portion giving the best flocculation and reduction of colour and turbidity by sedimentation and filtration will indicate the optimum doses of coagulants for the specific sample under test. Other tests varying the period of flash mixing and the period and intensity of flocculation should also be done to determine the most suitable parameters.

12.19.5 Softening reactions test.

This is a modification of the coagulation-flocculation jar test and is used to check the dosages of the various chemicals used in lime-soda softening. Basically the same procedure is followed.

12.20 RESIDUAL CHLORINE.

12.20.1 Discussion.

Chlorine when added to water reacts with organic matter which destroys its disinfecting power. It is thus necessary to add sufficient chlorine to react with all the various substances present and still leave a residual, if bacteria are to be destroyed. This residual chlorine may be present in the free state, which has a very rapid disinfecting power, it may be combined with ammonia to form the less active chloramines, or it may be absorbed by organic matter to form relatively inactive chloro-organic compounds with little or no disinfecting power. A residual of free available chlorine may be secured by the use of sufficient amount of chlorine to oxidize the organic matter or by the addition of relatively small amounts of chlorine to water free from appreciable quantities of ammonia or organic matter. In the control of chlorination, it is important to know whether the residual chlorine is present as free chlorine or as the less active combined chlorine.

12.20.2 Sampling.

Samples must be collected in clean bottles, and the tubes used in the test kit must be clean. Any dirt on the glassware will react with the chlorine giving low results. Samples must be analysed immediately.

12.20.3 Procedure using Ortho tolidine.

When ortho tolidine solution is added to water containing chlorine, a yellow colour develops, the density of which is proportional to the concentration of chlorine present. At higher values the colour becomes brown. Free-residual chlorine reacts with orthotolidine almost instantaneously, requiring less than 15 seconds for full colour development. Combined residual chlorine reacts relatively slowly requiring 5 minutes for full colour development at 20ºC.

1. Use Taylor water analyser or Lovibond comparator;

2. Fill tubes with sample to the mark;
.
3. Add a few drop of ortho tolidine solution;
.
4. Read off residual chlorine value, this is free-residual chlorine;

5. Wait 5 minutes then read off residual chlorine value - this is total residual chlorine. The difference is combined residual chlorine.

12.20.3 Procedure using the Ortho tolidine-Arsenite Method

Orthotolidine reacts with nitrites and magnesium to form a colour similar to that due to residual chlorine. To overcome this, the Arsenite modification is used.

1. Three tubes marked OT, A and B are used;

2 To tube OT, fill to mark with sample, add ortho tolidine;

3. To tube A, fill to mark with sample, add ortho tolidine and immediately add 1 ml sodium arsenite;

4. To tube B, fill to mark with sample, add 1 ml sodium arsenite and then add ortho tolidine;

5. Wait 5 minutes and then read off residual chlorine value;

6. Tube OT gives total-residual chlorine. Tube A gives free-residual chlorine. Tube B gives colour due to interfering substances, as the chlorine was removed before ortho tolidine was added;

7. The value of tube OT minus the value of tube A gives combined residual chlorine;

8 The value of B (if applicable) must be subtracted from the value of 'OT' and of 'A'.

12.20.4 <u>Procedure using the Diethyl - p - Phenylene Diamine (DPD) Method.</u>

Some authorities consider that ortho olidine is carcinogenic and should not be used. DPD may be used in its place. A similar method is followed and a pink colour is obtained.

12.20.5 <u>Interpretation.</u>

The disinfecting power of chlorine depends on the form of residual chlorine present, the contact time, the temperature and the pH value of the water. If the pH value is less than pH 8; 0.2 mg/L of free-residual chlorine will destroy bacteria in a l0 minute contact period.

To accomplish the same results with combined chlorine, a residual of 1 mg/L, at pH 6, 1.5 mg/L at pH 7 and 1.8 mg/L at pH 8 is required with a contact period of 60 minutes.

For high residual chlorine contents, greater than about 3 mg/L a volumetric method using sodium thiosulphate may be used.

12.21 CHLORINE DEMAND.

12.21.1 Discussion.

One proposed standard for the chlorine demand has been the dose of chlorine in mg/L required to leave a free-residual of 0.2 mg/L after a 20 minute contact time.

The chlorine demand of a water is the amount of chlorine which is needed to react with organic and other substances in the water.

This was covered in section 7.3 earlier.

12.21.2 Procedure.

1. Prepare a 100 mg/L chlorine solution. (a) Dilute 2 mL Jik to 1 litre with distilled water; or (b) dissolve 60 mg HTH in 1 litre distilled water;

2. Standardise the solution using sodium thiosulphate and dilute to 100 mg/L Cl_2 if needed;

3. Measure 100 mL portions of the sample into each of six clean bottles;

4. Add 0.5; 1.0; 1.5; 2.0; 2.5; 3.0 mL of chlorine solution, stopper and mix;

5. Let stand 20 minutes and then determine free residual and total residual chlorine;

6. If no bottle gives a 0.2 mg/L free residual after 20 minutes, repeat with higher doses of chlorine.

12.21.3 Interpretation.

The chlorine demand test indicates the amount of chlorine that will be needed to produce the 0.2 mg/L free-residual after 20 minutes. Since organic matter in the water absorbs chlorine, water having colour or turbidity will have a higher chlorine demand.

12.22 MISCELLANEOUS OTHER TESTS.

Where necessary, these could include Iron, Manganese, Fluorlde, Nitrite, Nitrate and Chloride.

12.23 BACTERIOLOGICAL EXAMINATIONS.

These are used to estimate the types and numbers of bacteria present in the raw and treated waters. This matter needs to be covered as a separate item and so is not included here.

12.24 MICROSCOPIC EXAMINATIONS.

These are of assistance in identifying the causative agents when certain odours and tastes are found in water. If done routinely, it may be possible to predict problems by identifying algal blooms, etc. This matter needs to be covered as a separate item and so is not included here.

WATER SOURCES and WATER TREATMENT.

PART 13.

THE CONTROL OF DISTRIBUTION SYSTEMS.

13.1 Discussion.

The quality of the water reaching the consumer depends on the supervision, maintenance, operation and control of the whole water supply system from the borehole, river or dam, through the treatment works and the distribution system. It is essential that each stage be operated at peak efficiency to ensure that the required quality of water reaches the consumer.

The protection of the water supply sources and the control of reservoirs has been covered earlier. The description of the tests needed to monitor the quality of the water has also been covered earlier.

The poor operation of a distribution system can cause a deterioration in the quality of the water.

Among the changes that can occur in the quality of the water are:

1. the reduction of sulphate to sulphide by certain bacteria which are present in water when there is no dissolved oxygen or residual chlorine present, this gives rise to an unpleasant smell and taste in the water;

2. if parts of the distribution system are allowed to operate under a negative head, i.e. a vacuum, at any time, then it is likely that groundwater could be drawn into the system at fittings and joints. Depending on the quality of the groundwater, the results of the contamination can vary between being not noticed and rendering the water dangerous for use;

3. the corrosion of iron pipes can result in noticeable quantities of iron being present in the water, this iron imparts an astringent taste to the water and can cause marks on clothes being washed;

4. the maintaining of too low a residual chlorine content in the water can result in the growth of slime, etc., in slow moving sections of the distribution system. This can also result in bacterial action causing the pH to drop; this can result in the protective coating of calcium carbonate being dissolved and allowing corrosion of the iron pipe.

The need for a continuous supply of high quality water cannot be overemphasised. When consumers find that the piped supply is not wholesome or reliable then they may turn to other possibly unsafe sources of water. This happened at Mamre some years ago when some of the inhabitants switched from the piped supply that had a high iron content to a spring that tasted much more pleasant because of the low iron and chloride contents. The latter supply became contaminated with Typhoid and an epidemic resulted. The original water supply was later treated for iron removal and chlorinated and made bacteriologically safe.

When the water supply is interrupted, people drawing water at lower levels will tend to cause a negative pressure or vacuum to occur in higher lying areas. (See 2 above). In older plumbing systems anti-syphoning fixtures were not used and it was possible that a person drawing water at a low level could draw water from a basin, etc., into the pipes.

In areas of limited economic resources, a choice often has to be made between continuous service to a restricted number of people and an intermittent service to a larger number of people. The first alternative is to be preferred.

No direct connections should be made between the piping of distribution systems and non-potable water supplies, sewers or industrial plant tanks. Scour valves situated at low points in the distribution system should terminate above the level of any nearby surface waters. They must never be connected to sewers.

Many distribution systems have extensions around the periphery, usually not connected to other extensions. These dead ends prevent the free circulation of the water, which in turn imposes hydraulic restrictions to the flow of water. Without free circulation of water, residual chlorine decreases to zero and conditions favourable for the reduction of sulphates and for the corrosion of the pipes to occur. The policy should therefore be to avoid dead-end mains where possible and to make a habit of flushing all such existing mains where possible and to make a habit of flushing all such existing mains through hydrants at regular intervals.

It is usually not economically feasible to cover all equalising reservoirs, because of their size, but all tanks should be covered to prevent the entrance of dust, insects and sunlight. The latter is to prevent the growth of algae. Ventilation holes must be covered with a fine gauze. It is essential that additions and repairs to the distribution system are properly disinfected before being put back into use.

The disinfection of the inner surfaces of new and repaired mains is much more difficult than the disinfection of polluted water, because the chlorine must penetrate through the organic matter coating the surfaces. Cleaning and flushing of the mains before disinfection are essential for good results. Chlorine doses of 50 mg/L and contact periods of 30 mins - 60 mins are usually used. The water must be flushed through all valves, hydrants before the system is brought into use.

Once a section of distribution system has been brought into use, it is usual to maintain a free-residual chlorine content of 0.2 - 0.4 mg/L. Even after pre-disinfection of the system with high chlorine doses, three transition periods with respect to the residual chlorine will be encountered:

1. no or very low residual chlorine;

2. while the initial chlorine demand of organic matter has been satisfied and while combined-residual chlorine is present owing to reactions with free ammonia;

3. after all organic matter and ammonia has been oxidised by the chlorine, leaving free-residual chlorine as desired.

13.2 RE-CHLORINATION.

Due to a number of reasons, one being that there is always some biological activity in water mains and reservoir, the residual chlorine tends to decrease. If the residence time in the distribution system is too long, the residual chlorine content may decrease to zero. It is undesirable to reticulate water with a nil residual chlorine. Among the reasons for this are:

1. increased biological activity can result in the production of carbon dioxide as a respiration by product, and this can result in a depression of the pH value of the water which can render the water aggressive;

2. increased biological activity can result in dissolved oxygen depletion thus allowing the sulphate reducing bacteria to increase in activity.

Re-chlorination is usually done at the feed to or discharge from a service reservoir. It is useful to be able to chlorinate at both points. Ideally chlorination at a service reservoir discharge should be proportional to flow. This is expensive but it ensures that neither excessive residual chlorine is produced, which will give rise to complaints, or too low which will mean that the desired protection is not being obtained.

13.3 CORROSION CONTROL.

Corrosive or aggressive water may be defined as water that has the property of dissolving metals to an appreciable degree. Corrosion in this context refers to corrosion of piping by the surrounding moist soil and by the water in the pipe but does not include the corrosion by electric currents known as electrolysis.

Corrosion is due to the solution of the exposed metal by a physicochemical action related to the character of the water and the metal. The rate of solution of the metal is inversely proportional to the alkalinity, the hardness, and the hardness of the water and is directly proportional to the dissolved oxygen content, the dissolved carbon dioxide content, the temperature and velocity of flow of the water. There are many other factors not all completely understood.

There are many cases where the straight treatment of water makes it more aggressive, e.g. the brown Cape waters and the softening process. Corrosion is aggravated when sulphate reducing organisms are active.

The metal of the pipe dissolves in the form of positive metal ions, the electrons are accepted by the hydrogen ions whose concentration is a function of the pH value. This action leads to the deposition of a polarizing film of gaseous hydrogen, thus stopping electrochemical action unless the hydrogen is removed by the scouring action of the water or is oxidised by the dissolved oxygen of the water.

$$\mathrm{Fe} + 2\,\mathrm{H}^+ \rightarrow \mathrm{Fe}^+ + \mathrm{H}_2$$

$$2\,\mathrm{H}_2 + \mathrm{O}_2 \rightarrow 2\,\mathrm{H}_2\mathrm{O}$$

This is an over-simplification of the reaction but it gives a practical basis for the treatment procedures needed.

The corrosion of exposed metal may be prevented by:

1. coating the pipe surface with protective paints or cement;
2. making the water non-corrosive by treatment;
3. treating the water to make it deposit a protective film onto the pipe wall.

With dissolved oxygen present, the corrosion of metals carrying water of low pH value containing carbon dioxide takes place as follows:

1. $\mathrm{Fe} + 2\mathrm{H}^+ + 2\mathrm{OH}^- \rightarrow \mathrm{Fe(OH)}_2 + \mathrm{H}_2$
2. $\mathrm{Fe(OH)}_2 \rightarrow \mathrm{FeO} + \mathrm{H}_2\mathrm{O}$
3. $4\,\mathrm{FeO} + \mathrm{O}_2 \rightarrow 2\,\mathrm{Fe}_2\mathrm{O}_3$

Both ferrous oxide and ferric oxide are insoluble, they form a film on the pipe wall, but this film does not fully protect the pipe unless calcium carbonate is precipitated at the same time and forms part of the film.

Serious corrosion of distribution systems is indicated by "red Water" or rust in the water. This corrosion may be detected by :

1. a decrease in the dissolved oxygen content of the water in the distribution system;
2. an increase in the pH value;
3. an increase in the iron content.

Perhaps the best measure of the degree of corrosion is the increase in the frictional loss in pipelines caused by the corrosion of the pipe surface, and the accumulation of tubercles. Corrosion will seriously decrease the capacity of ferrous water mains even when the rate of corrosion is less than that required to produce appreciable concentrations of iron in the water, this is due to the development of a rough, pitted surface on the inside of the mains. When the water contains proportionally too much calcium carbonate this will tend to precipitate out on the inside of the pipe and over the years this will reduce the available diameter of the pipe.

The principle methods of controlling corrosion are:

1. the removal of dissolved oxygen;

2. the removal of free carbon dioxide;

3. the deposition of a protective coating on the inside of the pipe;

4 chlorination to prevent biochemical action that influences corrosion;

5 the use of paint, enamel, tar compounds, or cement linings on the inside of the pipe;

6. cathodic protection.

13.4 THE REMOVAL OF DISSOLVED OXYGEN.

Oxygen as well as nitrogen and carbon dioxide may be removed by exposing the water to a partial vacuum. This is usually used only for industrial purposes such as boiler feedwater. The water is exposed to a vacuum of about 725 mm Hg, which removes all but 5% of the dissolved gases. Sufficient sodium sulphite, Na_2SO_3, is added to chemically remove the rest of the dissolved oxygen:

$$2\ Na_2SO_3 + O2 \quad \rightarrow \quad 2\ Na_2SO_4$$

This treatment is effective only above pH 7, it will be necessary to raise the pH above this value sometimes, using sodium carbonate or lime.

13.5 THE REMOVAL OF CARBON DIOXIDE.

The deposition of a protective mineral film on pipes, requires the removal of aggressive carbon dioxide, which otherwise would prevent the deposition, or would dissolve the mineral film previously formed and expose the pipe to continued corrosion.

Carbon dioxide in the atmosphere is in equilibrium with about 0.5 mg/L carbon dioxide in water, theoretically aeration of water containing more than this content, should reduce the carbon dioxide content of 0.5 mg/L. However, aeration does not lower the concentration to less than 3 - 5 mg/L. This concentration of carbon dioxide is aggressive, i.e. it will dissolve calcium carbonate at alkalinities less than about 80 mg/L as $CaCO_{3.}$

While aerators are the most economical means of eliminating aggressive carbon dioxide above 5 mg/L, they have the disadvantage of increasing the dissolved oxygen content. Well water devoid of dissolved oxygen but having a high carbon dioxide content is best treated by raising the pH value so that the carbon dioxide is transformed into the bicarbonate ion HCO_3 and some into the carbonate ion CO_3:

13.6 PROTECTIVE COATINGS ON THE INSIDE OF THE PIPE

These may be subdivided into

1. coatings deposited by special treatment of the water;

2. coatings deposited at the time of manufacture;

3. coatings deposited by special treatment of the water may involve the deposition of a metaphosphate film, a silicate film or a calcium carbonate film.

   (i) The metaphosphate film - Calgon, i.e. Sodium hexa meta phosphate and Nalco, i.e. Sodium pyrophosphate have been used.

   Doses of 5 - 10 mg/L are used for the first 4 - 8 weeks, the dose is then reduced to 1 - 2 mg/L.

   (ii) The silicate film - Sodium silicate is dosed at 28 mg/L. A hard, dense calcium silicate film is formed with waters of adequate calcium content or when a limited dose of lime is added with the sodium silicate.

(iii) The calcium carbonate film - Calcium carbonate is deposited when the pH or alkalinity is above the calcium carbonate equilibrium point.

If the alkalinity is less than about 30 mg/L, lime must be added to increase the calcium content and the alkalinity. Sodium carbonate may be used if the calcium content is high enough, but the alkalinity is not high enough.

Lime reacts with dissolved carbon dioxide

$Ca(OH)_2 + H_2CO_3 \rightarrow Ca(HCO_3)_2 + H_2O$

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

If too much calcium is added than an undue amount of scale is deposited and the pipe may become constricted. The calcium carbonate stability test used to predict the corrosiveness of the water will be covered later in section in 13.10.5

4. The coatings deposited at the time of manufacture may be paint, enamel or bitumen. The latter can cause tastes. Iron pipes may be cement lined or asbestos cement pipes may be used. In general, it is wise to treat the water for corrosion prevention even though internal protective coatings are used. If paint or a similar coating is used, it is essential to ensure that taste problems do not occur

13.7 CHLORINATION TO PREVENT BIOCHEMICAL ACTION ON THE PIPES.

Biological reaction in dead ends of pipes can cause a drop in pH value which can change the water from being stabilised to being aggressive. Chlorination is used to destroy the organisms responsible for the decomposition of organic matter and thus prevent aggressive conditions developing.

13.8 CATHODIC PROTECTION.

The corrosion of a metal pipe takes place at the anode where the electrons are removed from the water whereas no corrosion takes place at the cathode where electrons pass into the water. There are two basic methods of cathodic protection for a pipe (or structure) in contact with water.

13.8.1 Galvanic Cathodic Protection.

Here a more reactive metal than that of which the pipeline is constructed is placed in electrical contact with the pipe. For steel, cast iron or copper pipes, the metal used is magnesium, zinc or aluminium. The latter metal is corroded preferentially and is called the sacrificial anode. This may be seen in figure 37 below.

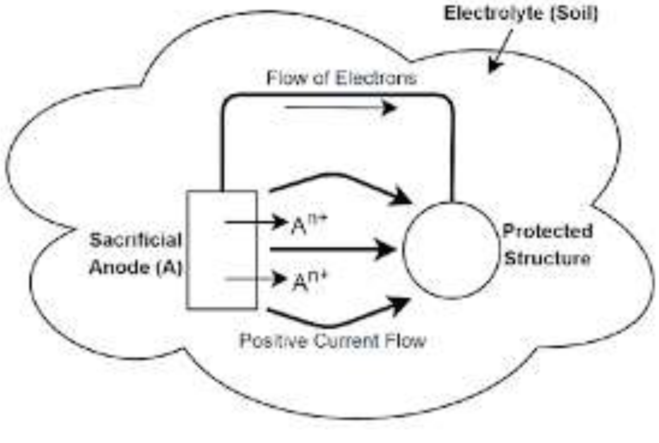


Figure 37 – SCHEMATIC OF SACRIFICAL ANODE PROTECTION OF A PIPELINE.

13.8.2 Impressed Current Cathodic Protection

Here electrons are fed into the material of the pipe line in order to render it cathodic to a separate auxiliary anode made of scrap steel or graphite. See Figure 38. Below:

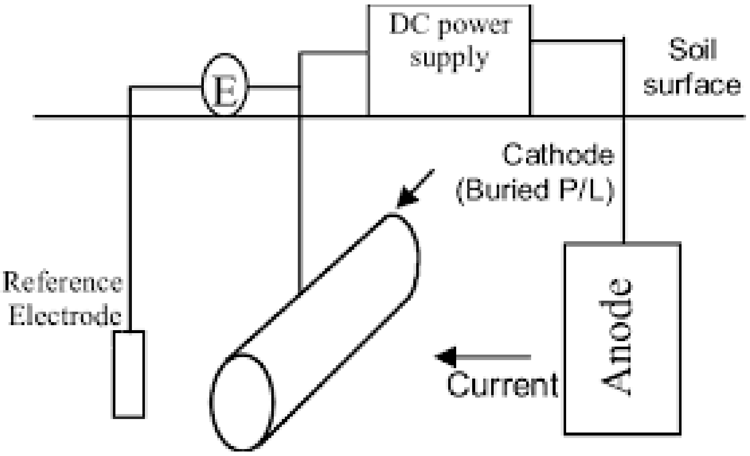


Figure 38 – LAYOUT OF IMPRESSED CURRENT CATHODIC PROTECTION SYSTEM.

13.9 THE DETERMINATION OF WHETHER A PARTICULAR WATER IS SCALE-FORMING OR CORROSIVE.

There is no single method that will give 100% true determination of whether a particular water is scale forming or corrosive. Below are some of the more commonly used methods.

13.9.1 The Langelier Saturation Index.

This was the first and is probably the best known index used to determine the condition of the water with regard to the precipitation or dissolution of calcium carbonate. After determining the pH value, calcium, alkalinity total dissolved solids and temperature, the saturation pH value or pHs can be determined by using one of the methods described below.

The saturation index S.I. = actual pH - pHs. A negative value, i.e. actual pH value lower than the saturation pH value indicates that the water is corrosive and can dissolve calcium carbonate. The index is only a **qualitative** indication of the corrosiveness or scale forming property of the water.

For a particular water the greater the positive value the more the degree of super saturation with respect to calcium carbonate of the water. However, one cannot compare two different waters and say that the one with the smaller positive value is less scale forming. The index gives no indication of the ability of the water to withstand a change in pH value. If a particular water is treated to a saturation index of +0.2 (this is a common value), and the decomposition of some organic matter on the pipe surface releases some carbon dioxide, the pH of the water will drop.

The actual drop in pH depends on the resistance of the water to the drop in pH; this is called the buffer capacity (See 13.11). If the pH of the above water drops by 0.3 units; the saturation index will become -0.1 and the water will be corrosive.

13.9.2 The Ryznar Stability Index.

There are cases where the water has a positive Langelier Saturation Index and yet the water is corrosive. Ryznar has proposed the stability index = 2 pHs - pHa where pHs is the saturation pH value, determined below and the pHa is the actual pH of the water. This value is always positive, and may be interpreted as seen in table 28.

TABLE 28 – PROPERTY OF WATER RELATIVE TO RYZNAR STABILITY INDEX.

pH value	PROPERTY
Less than 6.5	Scale forming
6.5 to 7.5	Fairly stable with respect to calcium carbonate
Greater than 7.5	Corrosive / aggressive

13.9.3 Corrosivity Ratio.

Even the stability index does not give the complete picture concerning the scale-forming or corrosive properties of the water because other factors also influence the process. The presence of chloride and sulphate play a part as they reduce the denseness of the calcium carbonate film and reduce its bond to the pipe material

The corrosivity ratio has been defined as:

$$\frac{\text{Chloride in m mol/L} + 0.5\ (\text{Sulphate in m mol/L})}{0.5\ (\text{alkalinity as } CaCO_3 \text{ in m mol/L})}$$

The corrosivity ratio applies only to water in the pH range 7 to 8. Values below 0.2 indicate that the water does not have corrosive properties. Higher ratios are indicative of increasing corrosive tendencies.

13.9.4 The Caldwell - Lawrence Conditioning Diagram.

If the intention is to only stabilise the water rather than to soften it, then the calculation is much simpler than the one described earlier.

The main advantage of this method over the above indices is that it gives an indication of the Calcium Carbonate Precipitation Potential (CCPC). This is the quantity of calcium carbonate likely to be deposited if the water is scale forming or the deficiency of calcium carbonate if the water is aggressive. Each particular water should have its own conditioning diagram, however, for convenience they are sometimes produced for increasing ionic strength in steps of 0.005. Approximately a dissolved solids of 400 mg/L is equivalent to an ionic strength of 0.010

The principle of the chart is that on dissolving or precipitating calcium carbonate, the term (alkalinity – calcium) does not change. With the method, the amount by which the water is super saturated with respect to calcium carbonate may be estimated and this is the Calcium Carbonate Precipitation Potential. A value to aim for is 2 - 4 mg/L.

EXAMPLE OF DETERMINING THE CALCIUM CARBONATE PRECIPITATION POTENTIAL.

Initial quality of the water: pH 8.6; Calcium as $CaCO_3$ = 100 mg/L; Alkalinity as $CaCO_3$ = 80 mg/L
Therefore (alkalinity – calcium) = 80 m- 100 = -20 mg/L

The various steps are (referring to figures 39 and 40) are:

.

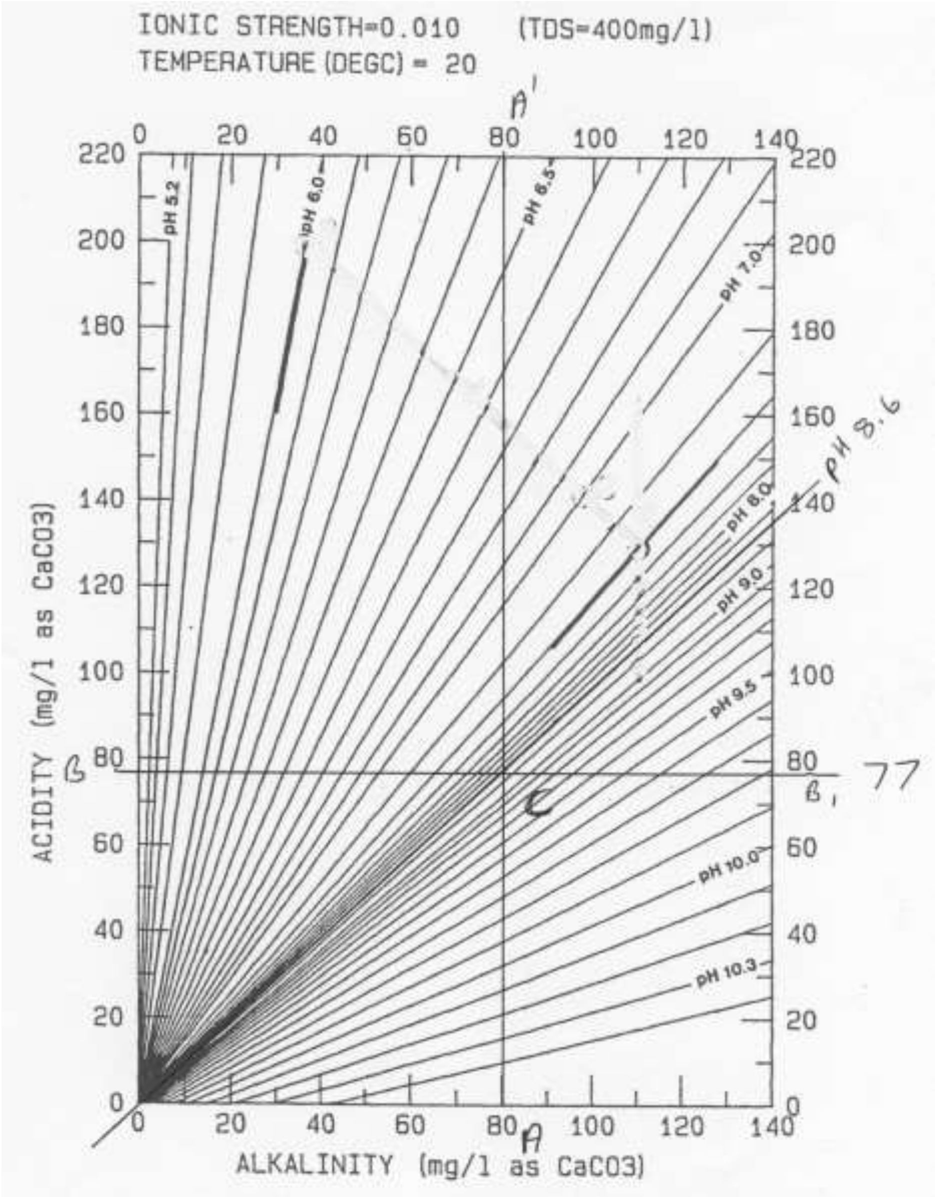


Figure 39 – ACIDITY / ALKALINITY / pH CONVERSION CHART

1. Highlight the pH 8.6 line;
2. Draw in the Alkalinity = 80 line pon the horizontal axis as A – A$^{1;}$
3. Mark point C where Alkalinity line intersects the pH8.6 line;
4. Draw a horizontal line through point C – line B- B^{1};
5. Read off the Acidity on the vertical axis = 77 mg/L as $CaCO_3$;

The Acidity = 77 mg/L value in used in figure 40.

.

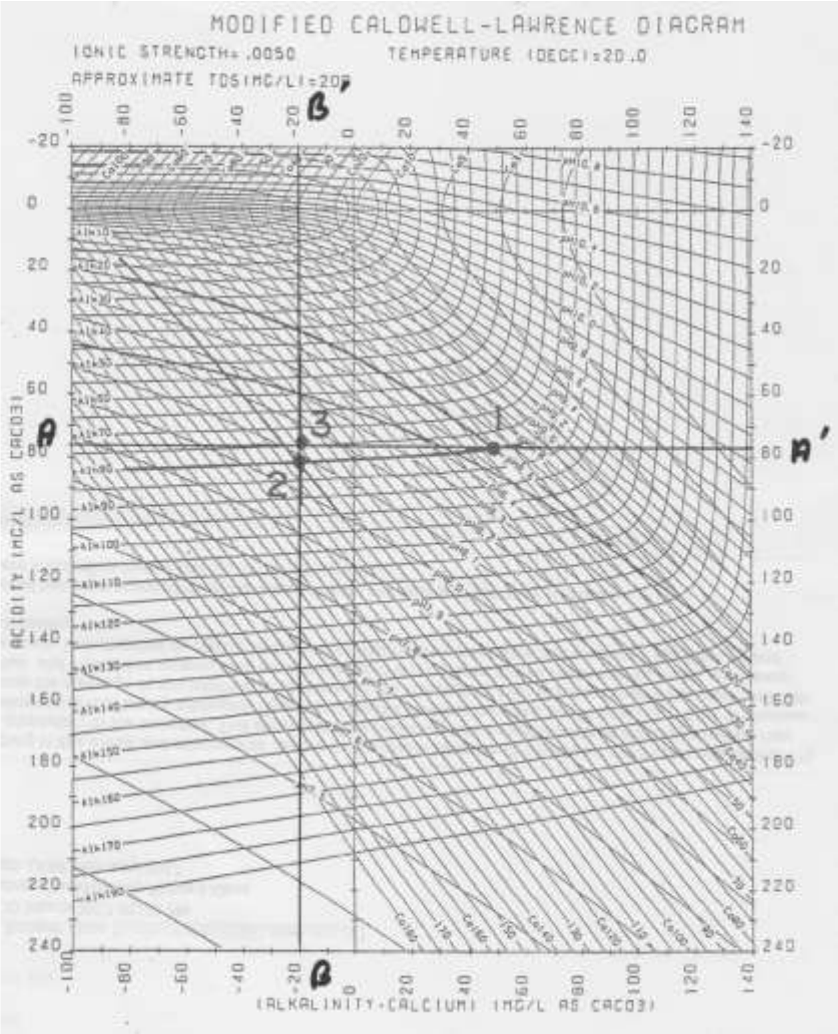


Figure 40 – CALDWELL – LAWRENCE DIAGRAM FOR THIS EXAMPLE.

6. Draw in the Acidity = 77 mg/L on the vertical axis as line A – A^1.;

7. Note where it intersects the pH8.6 line and mark this point as 1;.

8. Draw in the (Alkalinity- Calcium value) = -20 on the horizontal axis as line B – B^1.;

9. One can also draw line B – B^1 through point 2 where the Alkalinity = 80 mg/L and the Calcium = 100 mg/L intersect;

10. The equilibrium point is where lines A – A^1 and B – B^1 intersect. This is point 3;

11. The properties of this point are: pH 8.13 (This is the pHs value); Alkalinity = 75 mg/L and Calcium = 95 mg/L.

The Calcium Carbonate Precipitation Potential is:

(Initial Calcium value – Final Calcium value)

= 100 – 95 = **5 mg/L as $CaCO_3$.**

Note that the reduction in Alkalinity is also 5 mg/L as $CaCO_3$.

13.9.5 Other Methods of Determining the Stability of water with Respect to Calcium Carbonate.

There are older tests that may also be used. These are described below.

13.9.6. The so-called Marble Test.

1. Determine, the pH and Alkalinity of the water to be tested;
2. Add 250 mg of pure calcium carbonate powder to a 250 mL Narrow Mount bottle;
3. Carefully fill the bottle with a fresh sample of water without entraining air, fill right to the top, replace stopper;
4. Shake sample every 30 minutes for 3 hours or preferably continuously for 3 hours.
4. Allow the sample to settle overnight;
6. Carefully remove about 150 mL of supernatant with a pipette without disturbing the solids;
7. Determine the pH using about 20 mL of supernatant;
8. Filter the rest of the supernatant through an acid-washed filter paper, e.g. 540;
9. Discard the first few mL of filtrate;
10. Determine the total alkalinity of the filtrate.

As seen in section 13.9.4, the change in Alkalinity will be numerically the same as the change in Calcium value. Using the values from above, the Calcium Carbonate Precipitation Potential would be (Initial Alkalinity – Final Alkalinity) = 80 – 75 = 5 mg/L as $CaCO_3$.

If the water was corrosive / aggressive, there would be an increase in the Alkalinity and therefore an increase in Calcium value. This would give an indication of the aggressiveness of the water.

The pH as determined in step 7 is the pHs value used in the Langelier Saturation Index.

13.9.7 The Enslow Stability Indicator.

This is effectively a continuous flow version of the Marble Test. Water is passed through the unit at such a rate that the total retention time is about 2 hours. The influent and effluent are analysed for pH and total alkalinity and the result calculated in the same manner.

If the pH and the alkalinity of the water increase on passing through the unit, then it shows that the water is aggressive and it is dissolving $CaCO_3$. If the pH and the alkalinity decrease then it shows that the water was scale-forming and it is precipitating $CaCO_3$. There should be no change or a slight decrease in pH and alkalinity. If the water is aggressive the magnitude of the increase in alkalinity gives an indication of the increase in lime or soda ash dose needed to increase the alkalinity of the water.

13.10 THE BUFFER CAPACITY.

Buffer capacity quantifies the ability of a solution to resist changes in pH by either absorbing or desorbing H^+ and OH^- ions. When an acid or alkali is added to a buffer system, the effect on pH change can be large or small, depending on both the initial pH and the capacity of the water to resist change in pH. Buffer capacity (β) is defined as the moles of an acid or base necessary to change the pH of the sample by 1 unit divided by the pH change and the volume of buffer in litres; it is a unitless number.

$$\text{Buffer Capacity} = \frac{\text{Moles of acid or alkali to change pH by 1.0 unit}}{\text{Volume of sample}}$$

As long as the buffer has not been completely reacted, the pH will not change drastically. The pH change will increase (or decrease) more drastically as the buffer is depleted: it becomes less resistant to change.

In water, the buffering action in provided by the Carbonic species – Carbonate, Bi-Carbonate and the so-called Carbonic Acid. It is measured by titration with an acid or alkali and noting how much is required to change the pH by 1 unit.

The moles of acid or alkali care converted to the equivalent mg/L of Calcium Carbonate.

For Hydrochloric Acid the conversion is 73 mg HCl = 100 mg $CaCO_3$

$$2\ HCl + CaCO_3 \rightarrow H_2CO_3 + CaCl_2$$

2 x 36.5 mg HCl reacts with 100 mg $CaCO_3$

A value to aim for is at least10 mg/L calcium carbonate per unit pH - note the units.

WATER SOURCES and WATER TREATMENT.

PART 14.

THE TREATMENT AND DISPOSAL OF WATER TREATMENT SLUDGES.

14.1 Discussion.

Only very rarely would it be permitted to discharge water treatment sludges into a river or the sea. The usual disposal route is via some form of treatment before being disposed of on the land. The quantity and nature of sludges produced at a water treatment works will vary very widely. The volatile or organic matter content will also vary but will generally be very much lower than that found in wastewater treatment. Because of this and because of the high iron or aluminium content (when either of these is used as a coagulant), anaerobic digestion of the sludge is not possible.

The normal goals of sludge treatment are to reduce the volume of sludge requiring disposal and/or to alter the nature of the sludge to render it inoffensive. The solids content of sludge drawn from a sedimentation tank is usuallv in the range of 0.5% to 1.0% for normal water treatment

Most sludges produced when iron or aluminium salts are used as coagulants are light and feathery and although settle fairly quickly, they do not compact well. Polyelectrolytes may be used to improve the ability of the sludge to compact and thus reduce the volume.

14.2 THICKENING BY AIR FLOTATION.

To accomplish thickening by flotation, fine bubbles are produced which become associated with the sludge solids, increase their buoyancy, and cause them to rise to the surface where they are removed.

A major difference between gravity thickening and flotation thickening is in the nature of the sludge which can be effectively thickened. The effectiveness of thickening by flotation depends on how well sludge particles can be agglomerated with gas bubbles to cause them to rise. Surface properties of sludge solids are hence of great importance in flotation thickening. Particle surface characteristics also influence the behaviour of sludges in gravity thickeners, but sludges comprised of flocculent particles of near neutral density may be concentrated by flotation more readily than by gravity.

In the thickening process, the percentage solids content is increased from 0.5 – 1.0% to 3 - 5%. Increasing the solids content from 1% to 5% reduces the volume of the sludge to 20% of the original value. However, the sludge still contains 95% water. In order to reduce the volume further, dewatering is necessary.

14.3 DEWATERING.

In this step, the solids content will be increased to 10 - 30% depending on the nature of the sludge and the equipment used. Among the processes that may be used are: centrifuging, vacuum filtration, pressure filtration and drying beds.

14.3.1 Centrifuging.

The solid bowl centrifuge is the most commonly used type. Sludge is introduced through the hollow shaft. High-speed rotation causes the sludge to occupy an annular space at the outside of the bowl. Depth of the liquid pool is controlled by adjustable ports or weirs from which clarified effluent is removed. Centrifugal force causes sludge solids to consolidate at the outer centrifuge wall. The solids are removed to discharge ports by a helical screw or scroll, which rotates at a speed slightly different than the bowl. An important feature of the centrifuge is the conical portion which forms a beach or drainage deck up which solids are conveyed by the screw to accomplish further removal of water. Scrolling of solids up the beach of a solid bowl centrifuge is a critical part of the centrifugation process, for high shearing forces caused by agitation from the scroll and by fluid drag from escaping liquid can carry solids back into the pool.

14.3.2 Vacuum Filtration.

About 20 to 40% of the drum surface is submerged in sludge in the vat below the drum. Because the vacuum of 250 to 660 mm of mercury maintained inside the submerged part of the drum, water is drawn into the drum and solids are deposited on the media surrounding the drum. The cake grows in thickness and is lifted from the sludge by the sloh otation of the drum. A partial vacuum is maintained in the segments of the drum above the vat as well, and further moisture reduction may occur because of further removal of moisture and possibly because of drying by the air which rushes through the cake into the drum. The time during which a point on the drum circumference is submerged is called the form time, and the length of time which the cake remains on the drum above the vat is called the drying time. Cake is removed before the media re-enters the vat by a knife edge, roller, or by unaided gravity discharge.

14.3.3 Pressure Filtration.

The filter pressure is an effective means of dewatering conditioned sludges to a higher degree than normally is possible with vacuum filters. Filter presses consist of a series of parallel plates covered on both sides with filter cloths and shaped to permit drainage. Sludge is introduced under pressure into the spaces separating successive plates, and water is forced through the filter cloths, leaving behind the solids. After a period of time (the "pressing Time"), the press is opened and the accumulated dewatered solids are discharged.

The term "press" is a misnomer, for no mechanical squeeze is involved. Pressures up to several hundred kPa imparted by the sludge pump provide the gradient necessary to pass filtrate through the sludge cake. Pressing time is dependent on the nature of the sludge and may be less than an hour, or more than a day. Filter cake solids concentrations in the order of 50 to 60% may be achieved even with organic sludges.

Filter pressing is inherently a batch operation, and high labour costs have been associated with the periodic discharge of dewatered cake. Because of this disadvantage, there is a trend toward automation to make filter pressing a mechanized semi-continuous process.

14.3.4 Drying Beds.

Sludge drying beds usually consist of about 150 mm of sand overlying about 300 mm of stone in which open-jointed tiles are placed for removal of filtrate. Beds are surrounded by low walls to retain sludge and are segmented to permit independent operation of a number of beds. Beds commonly are filled with 105 mm to 300 mm of sludge. Initially most water loss is by filtration through the sludge and percolation into the sand, but after a day or two, evaporation becomes the prominent mechanism. The solids concentration may be up to 20 to 40%. After drying the sludge is removed, usually by manual means - and the bed is returned to service. In this way 5 to 10 applications per year can be made on each bed. To minimize odour problems and to improve dewaterability, organic sludges almost always have been treated prior to application to sand beds.

Sludge drying beds are by far the most common method of dewatering sludge at smaller water treatment plants. The major disadvantage of drying beds is the great amount of labour required to lift dried sludge from the beds. Other disadvantages include a large area requirement,the need for treatment of decomposable sludges, and dependence on climatic conditions. Equipment for mechanically lifting sludge from beds is available although it is not in common use. Future widespread application of drying beds in installations which are large enough to incur additional layout expenses for manual lifting sludge; would seem to be dependent on the use of such equipment. Some variations on conventional sludge drying bed practices include the use of covered beds and paved beds, and application of coagulant aids prior to sludge application. Covering of beds is done with greenhouse-like structures

Empirical design standards commonly provide for 20% to 40% reduction in required bed area when such enclosures are used.

14.4 RE-USE OF SLUDGES.

As the disposal of water treatment works sludges have always been a problem, numerous investigations have been carried out into developing a method that recovers material from the sludge that can be re-used as a coagulant. Very rarely has any method proved feasible.

One possible method is the use of magnesium carbonate as a coagulant. Here the water is treated with lime and magnesium carbonate. These two chemicals react and produce a floc:

$$Ca(OH)_2 + MgCO_3 \rightarrow CaCO_3 + Mg\,(OH)_2.$$

These products settle with the raw water turbidity and clarify the water. The sludge is removed from the sedimentation tank and carbonated. The magnesium hydroxide is dissolved preferentially forming magnesium bicarbonate. (With excess carbon dioxide, the calcium carbonate would also dissolve.)

$$Mg(OH)_2 + 2CO_2 \rightarrow Mg\,(HCO_3)_2 + H_2O$$

The magnesium hydroxide carbonate is removed by filtration and heated to about 45ºC in air, where the carbon dioxide is given off, but is recovered.

In this process and in others where lime is used, such as lime softening, it may be feasible to remove the calcium carbonate precipitate and after drying it, to heat it to a high temperature in a furnace. Here the material breaks up into calcium oxide and carbon dioxide.

$$CaCO3 \rightarrow CaO + CO_2$$

In water treatment, it is worth trying out the use of a sludge return in an attempt to reduce the chemical dosages. The sludge return provides a seeding effect that may aid precipitation of colloidal material.

14.5 DISPOSAL OF WATERWORKS SLUDGES.

Unlike certain wastewater treatment sludges, water treatment sludges have no value as soil conditioners. The disposal of these sludges is usually as landfill or in a tip site. As the metal coagulant sludges are generally gelatinous, it is preferable that they be disposed of in admixture with wastewater treatment sludges or with domestic refuse.

The subject of the disposal of solid wastes has been adequately covered in the "Pollution Control" notes.

****