CHAPTER THREE

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CHAPTER THREE

TABLES AND FIGURES

CHAPTER THREE – WATER TREATMENT

3.1 INTRODUCTION

3.1.1 GENERAL

This Chapter contains some additions and minor changes to that in the previous version of the Design Manual. Roughing Filters, Upflow Filters, Micro-straining, Lamella Plate Settlers and Reverse Osmosis are introduced and sections pertaining to Trihalomethane risk with pre-chlorination and Radon are also included. Disposal or re-use of water recovered from treatment works sludges are discussed and the 2004 edition of the WHO Guidelines for Drinking Water Quality is introduced.

3.1.2 WATER TREATMENT PROCESSES

A major consideration when designing a water supply scheme is the nature of the water source that is to be used. Questions that arise include:

- is it in reasonable proximity to the area to be supplied?
- is there a readily available and reliable power supply source
- is the flow sufficient, or will an impoundment be needed?
- how variable is the quality?
- will the quality of its waters pose special concerns for the efficacy of treatment? For example, might such variations cause occasional impairment of desired plant performance? Or excessive treatment costs?
- is the catchment or recharge area vulnerable to contamination (e.g. from geothermal areas, mining activities, urban and agricultural pollutants: faecal microbes, sediment, fertilisers and pesticides)?
- what management techniques are available to mitigate contamination and how might their efficacy vary with soil type and topography, for example?

A variety of sources are used for the purpose of water supply and range in size from those needed by single households to supplies needed for large cities. Each kind of supply can be characterised according to its raw water quality (Table 3.1) and there are two rules of thumb that can be applied with regard to the necessary levels of treatment for each source-type:

- except for small rural water supplies where stringent protective measures are taken, the widely-accepted minimum treatment for a groundwater source is disinfection
- the widely accepted minimum treatment for a surface water source is some form of filtration followed by disinfection. This minimum level should also be considered for a groundwater source that is under the direct influence of surface water

Treatment to produce safe drinking-water depends on the raw water source that is used. Some natural purification occurs in surface waters as a result of dilution, storage time, sunlight exposure, and associated physical and biological processes. With groundwater, natural purification may occur by infiltration of rainfall through soil and percolation through underlying porous materials such as sand, gravel and joints or fractures in bedrock. Effective treatment should be provided to ensure safety and consistency in the quality of drinking-water.

Rivers used for drinking-water supply are generally accessible to the public as well as having private lands draining to them. This means that water quality is variable and not easily controlled and that such waters are unprotected from illegal activities and major pollution events (e.g. spills from tanker accidents, discharges of urban, farm and factory wastes).

TABLE 3.1: SOURCE WATER QUALITY

RAW WATER SOURCE	MICROBIOLOGICAL QUALITY	CHEMICAL QUALITY	AESTHETIC QUALITY
Roof water	Often poor	Usually good, subject to air and roof paint contaminants	Soft/corrosive so could contain some metals
Unconfined aquifer	Often poor	Can be high in nitrate and ammonium	Variable, can be turbid, discoloured, soft/corrosive. Can be high in iron or manganese
Confined aquifer	Usually good	Usually good. Can be high in carbon dioxide and ammonium	Variable. Can be hard or soft/corrosive and high in iron or manganese. Usually low turbidity
River or stream. Controlled or few human/animal impacts	Good to poor	Usually good	Usually good but turbid and discoloured under flood conditions
River or stream. High human and/or animal impacts	Poor	Often poor	Good to poor. Turbid and discoloured under flood conditions
Lake/reservoir. Controlled or few human/animal impacts	Usually good	Usually good	Usually good. May have iron and manganese in deep water. May be coloured water from bush catchments
Lake/reservoir. High human and/or animal impacts	Often poor	Good to poor	Usually good, may not be good if prone to algae blooms. May have iron/manganese in deep water

Monitoring programmes are needed to determine when river sources may be unacceptable for treatment and to determine the quality of influent water prior to treatment. They should provide an understanding of average water quality, changing water quality conditions and the magnitude and frequency of extreme water quality occurrences. Monthly sampling is commonly chosen for river monitoring networks because it provides useful information about average or characteristic water quality, and changes in water quality. Results can be used for trend analysis after sufficient data have been collected (at least 5 years, or 50 - 100 data sets).

Table 3.2 on the following page indicates the percentage removal of faecal coliform bacteria as a result of the processes indicated. Care must be taken to see percentage removal in context where the actual numbers of bacteria may be up to 10^6/ml. Monitoring for microbiological quality is simply a check that barriers are working and should not be regarded as a replacement for removal of any of the barriers.

Faecal material from humans and animals is the most likely source of waterborne pathogens. Wherever possible humans and domestic animals should be excluded from water supply catchments, and particularly if the water treatment process does not include flocculation, sedimentation, filtration and disinfection.

Water treatment involves physical, chemical, bacteriological and biological transformation of raw water into potable quality. The quality of water for use is given by the Tanzania Temporary Standards and the World Health Organisations (WHO) standards, (see Section 3.13).

TABLE 3.2: ACHIEVABLE PERFORMANCE WITH EFFECTIVE CONTAMINATION BARRIERS

PROCESS	REMOVAL OF FAECAL INDICATOR BACTERIA
protection of catchment	variable
artificial impoundments (3 - 4 weeks' storage)	variable
coagulation and sedimentation	40 - 90 percent
filtration	99 - 99.9 percent
chemical disinfection [a]	>99% with sufficient Ct values [b]
UV disinfection	>99%, depends on dose [d]
a. Chlorine/chloramine/chlorine dioxide/ozone. *b. This does not necessarily apply to protozoan cysts.*	

Design of a water treatment plant requires the information given below.

- The water demand: The treatment works should be designed for the peak, day demand
- The raw water quality: It is required to enable a decision on the type of treatment.

The following methods of water treatment are considered to be suitable for Tanzania, used in different combinations as necessary to achieve the desired results:

- Screening or straining
- Roughing or pre-treatment filters
- Plain sedimentation
- Chemical coagulation, flocculation and settling
- Filtration either by Slow Sand or Rapid Gravity filtration
- Disinfection
- Control of algae
- Taste and odour control
- Softening
- Removal of iron and manganese
- De-fluoridation of water

Based on the life span of the different structures and components, the following recommendations are made:

- Pumps designed for a future demand and life expectancy of ten years.
- Steel tanks, M&E equipment, internal piping designed for a future demand of 15 years.
- All other structures designed for the future demand of 20 years ahead.

3.1.3 WATER TREATMENT OPERATION AND CONTROL

It is also important to identify the types of control networks and the equipment needed to ensure a smooth operation of both water intakes and treatment plants. Over sophistication should however be avoided, especially in remote areas where manual controls are often best.

Even where more sophisticated methods are deployed, all equipment should be provided with easily operable manual overrides wherever possible in case of breakdown or emergency. This is particular important for all the various types of valves that are installed even in the most sophisticated of plants.

3.2 SCREENING OR STRAINING

This is a physical, pre-treatment process of surface filtration used to remove weeds, grass, twigs, bilharzial snails and other freshwater crustacea as well as coarser particles, so that they do not enter the pumping, treatment, or supply system.

3.2.1 Screening

The following are typical of surface water screens:

- They should be easily accessible, at least during medium and low flows and inclined upwards away from the river or stream.
- Velocity entering the screen not should exceed 0.3 m/sec.
- Allow for a head loss 15 cm to 30 cm
- Size of the openings of the screen should be: 10^{-2} to 10^{-3} m
  (normally in the size of 25mm for coarse and 6 mm for fine screen)
- Small screens are made removable for cleaning, medium sized can be hand raked in-situ whilst large screens will need in-situ mechanically operated rakes.

Screening can then be followed by one of the following:

3.2.2 Desilting Chamber and Silt Removal

Desilting is a process of removing particles of sizes smaller than 10-^{2}, i.e. 10^{1} to 10^{-2} by 1owering the velocity through a chamber or basin usually with bottom entry and top exit. In such units the following criteria are common:

- Intake velocity is normally in the range of 1 to 1.5 m/sec
- Velocity entering the chamber is reduced to 0.5m/sec whereby kinetic energy is changed to potential energy
- Silt then settles when upward velocity is between 0.15 m/min - 0.90 m/min.
- A means of silt removal is provided and where possible this should be by flushing under gravity back to the water source

3.2.3 Roughing Filters

Roughing filters have their place as a form of pre-treatment, especially for turbid or highly changeable river water. This technique has been greatly under-utilised in Tanzania in the past.

It is used primarily to remove solids from high turbidity source waters prior to treatment with such things as slow sand filters, but has a useful part to play in many treatment processes.

Typically, roughing filters consist of a series of tanks filled with progressively smaller diameter media in the direction of the flow. Whilst the media is usually gravel, crushed coconut, rice husks, or a similar locally available material are alternatives. The flow direction in roughing filters can be either horizontal or vertical, and vertical roughing filters can be either upflow or downflow. The media in the tanks effectively reduces the vertical settling distance of particles to a distance of a few millimetres. As sediment builds on the media, it eventually sloughs off and begins to accumulate in the lower section, while simultaneously regenerating the upper portions of the filter. The filters require periodic cleaning to remove the collected silt.

Some research has reported that long filters (10 m) at low filtration rates (0.5 m/h) were capable of reducing high suspended solids concentrations (1000 mg/l TSS) down to less than 3 mg/l.

Other studies have suggested that roughing filters were capable of reducing peak turbidities by 80 to 90 percent, and faecal coliforms by 77 to 89 percent.

For detailed information, designers are recommended to download the comprehensive SANDTEC design report entitled 'Surface Water Treatment by Roughing Filters, A Design, Construction and Operation Manual' available at
http//sandec.ch/WaterTreatment/Documents/Surface%20Water%20Treatment.pdf

A schematic representation, abstracted from the above, showing different types of roughing filter is presented below:

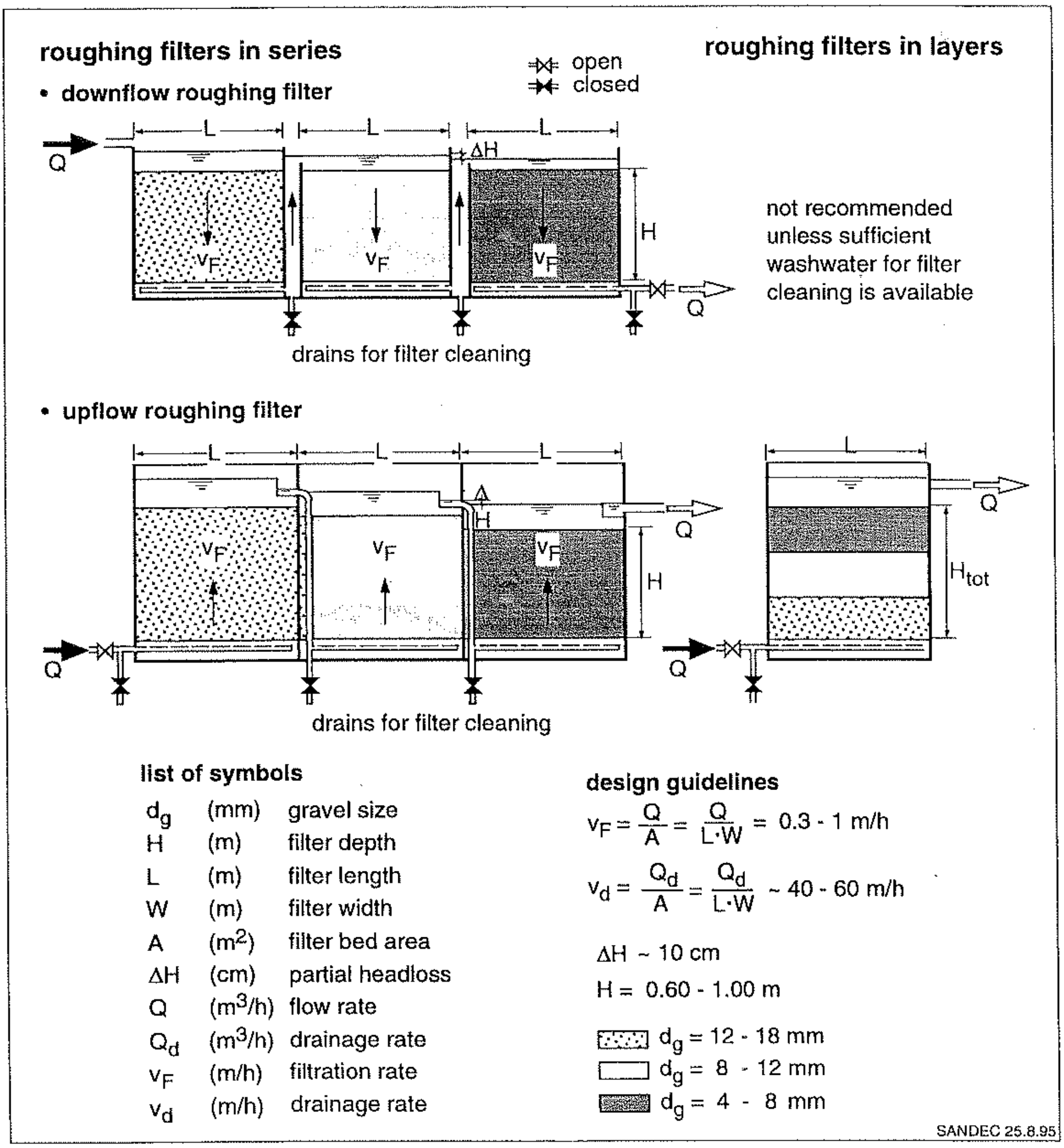


FIGURE 3.1: OUTLINE OF DIFFERENT TYPES OF ROUGHING FILTERS

3.2.4 Plain Sedimentation

Plain sedimentation before filtration is recommended where the silt content is 5-10 mg/l. Where turbidity is higher during the rainy season and especially if in excess of 50 mg/l desilting .chambers and/or roughing filters should be used to lower the concentration of silt and turbidity so that excessive use of chemicals can be limited. For water with turbidity less than 10 mg/l rapid sand filtration process can be applied without pre-treatment other than screening.

Plain sedimentation or roughing filtration, followed by slow sand filtration and if necessary, disinfection is applicable where suspended matters have sizes more than 10^{-3} - 10^{-7} cm diameter, organic content is low and where sufficient land area is available.

3.3 CHEMICAL COAGULATION AND FLOCCULATION

Coagulation and flocculation processes are intended to form particles large enough to be separated and removed by subsequent sedimentation, or alternative clarification processes.

Chemical coagulation is the normal way to eliminate turbidity and colour. Aluminium sulphate (Alum) is the most common coagulant used in Tanzania.

The following methods are being used in practice:-

- Twin tank system
- Flocculation/Settling/Rapid Sand filtration

3.3.1 Coagulation and Flocculation

Coagulation and Flocculation are aimed at removing a colloidal matter with sizes less than 10^{-4} to 10^{-7} cm diameter. The colloidal matters (clay, bacteria, and colour particles) are balanced by electrical force which is predominant and equal to the gravity force such that natural settlement does not occur.

3.3.1.1 Coagulation

Coagulation is the process of adding a chemical (coagulant) to the raw water containing colloidal matter to form small gelatinous precipitated masses, which can readily settle out in sedimentation tanks within the normal range of surface loading.

The coagulation stage occurs when a coagulant, such as alum, is added to the water to neutralise the charges on the colloidal particles in the raw water, thus bringing the particles closer together to allow a floc to begin to form. Rapid, high energy mixing (e.g. mechanical mixers, in-line static mixers, jet sparge mixing) is necessary to ensure the coagulant is fully mixed into the process flow to maximise its effectiveness. The coagulation process occurs very quickly, in a matter of fractions of a second. Poor mixing can result in a poorly developed floc.

In the process

- Particles sizes are in enlarged from 10^{-6} cm to 10^{-4} cm, and
- Creates micro flocs

(i) Character of Water

The character of water has a considerable influence on the pH value at which satisfactory flocs may be formed. Soft, coloured and acidic waters present many problems in coagulation and are difficult to clarify satisfactorily as the dosage of both coagulant and alkali are critical.

(ii) Optimum pH value

There is at least one pH range for any given water in which good flocculation occurs in the shortest time with a given dosage of coagulant or a given time with the minimum dose of coagulant. Coagulation should be carried out within this optimum range whenever possible.

This optimum pH range should be determined by conducting tests. In the case of coagulation with alum, the control over the alkalinity is very important. Not only should the water contain sufficient alkalinity to completely react with aluminium sulphate, there should also be sufficient residual alkalinity so that treated water is not corrosive.

The pH range of least solubility for the hydrolysis product of aluminium sulphate ranges from 5.5 to 7.8. Lime may be added to increase the pH value of given water and sulphuric acid may be added to reduce the pH value.

However, such adding processes are far from simple and the source of lime is especially important on the method of preparation and dosage.

(iii) Choice of Coagulant

In selecting the best coagulant for any specific treatment problem, a choice has to be made from various chemical depending on their merits. Alum (Aluminium sulphate $Al_2(S0_4)_3$; 18 H_20) is the most widely used chemical coagulant at water treatment plants.

The dose of coagulants is determined by a jar test in the laboratory and this requires routine checking, in some instances at least twice daily. The dose of alum to be added to the water must be adjusted accordingly so as to be as close to optimum as possible.

(iv) Formation of Coagulant Solution

Mixing is an operation whereby the coagulant is introduced into raw or pre-treated water. The coagulant can be mixed into the water in two forms.

a) In the powder form (dry feed)

The method is done for large plants either by screw feeder, conveyors or batching.

b) In solution form

This is a widely used method in small and medium water treatment plants. A solution of known strength, i.e. 2% - 10% of coagulant is introduced into the raw or pre-treated water. Flows of chemicals should be visible and easily measured.

c) Formation of Alum Solution

Alum solution can be formed in one of two ways:

(i) Parallel flow doses

Raw water is introduced into the alum tank where stirring is conducted. The solution of alum is introduced into the water from the other end of the pipe.

(ii) Separate method

Alum solution is thoroughly prepared before being introduced into the water. The method has the following advantage over the parallel dose method:-

  - Clay does not get inside
  - The concentration of alum can be easily calculated; depending on the fluctuation in the concentration of turbidity.

3.3.1.2 Rapid Mixing

Mixing is an operation in which two or more materials are quickly intermingled to attain the desired degree of uniformity. The purpose of rapid mixing is to ensure adequate, prompt dispersion of chemicals in the water being treated. For discharges of more than 300 m^3/hr and where the head loss is to be minimised mechanical mixing should be considered. Multiple units may be provided for large plants.

For smaller plants, mixing in a tank with paddles is often preferred and in this case the following design values can be used:

- mixing time - 0.2 to 2 minutes
- circle speed 0.6 to 2m/sec
- revolution per minute 5 to 20
- energy 1 to 4 wh/m^3
- G = velocity gradient 700 - 1000 (S^{-1})

The value of G can be obtained from the expression:

$$G = (W/u)^{1/2} \qquad 3.1$$

Where,

W = dissipation function = mixing power/volume unit (w/m^3), and
u = absolute viscosity of water (Ns/cm^2)

In addition, $W = (p \times g \times h)/T \qquad 3.2$

Where,

p = unit weight of fluid
g = acceleration due to gravity
h = head loss due to friction, and
T = detention time

Values of density and viscosity of water are given in the following table:

TABLE 3.3: ·VISCOSITY AND DENSITY OF WATER

TEMPERATURE	DENSITY (grams/cm^3)	ABSOLUTE VISCOSITY (centipoises)	KINEMATIC VISCOSITY (centistokes)
10°C	0.99973	1.3097	1.3101
12°C	0.99952	1.2390	1.2396
14°C	0.99927	1.1748	1.1756
16°C	0.99897	1.1156	1.1168
18°C	0.99862	1.0603	1.0618
20°C	0.99823	1.9987	1.0105
22°C	0.99780	0.9608	0.9629
24°C	0.99733	0.9161	0.9186
26°C	0.99681	0.8746	0.8774
28°C	0.99626	0.8363	0.8394
30°C	0.99568	0.8004	0.8039

In the table, the following conversions apply:

> One centipoise = (10^{-2} gramme-mass / cm) = 1 millipascal-second.. To convert to lb.force.sec / sq.ft multiply centipoise by 2.089×10^{-5}.
>
> One centistoke = (10^{-2} cm^2/sec) and is equal to the viscosity in poises divided by the density of the fluid in grams per cc. To convert to sq. ft/sec. multiply centistoke by 1.076×10^{-5}.

(i) Gravitational or hydraulic devices

In these devices the required turbulence is obtained from the flow of water under gravity or pressure. Some of more common devices are described below:-

(a) Hydraulic jump mixing

This is achieved by combination of a chute created super critical flow (velocity 2 to 4 m/s), the sill defining the location of the channel and the gently sloping channel induces the jump. Standing wave flumes specially contracted for' measurement of flow can also be used in which the hydraulic jump mixing takes place at the throat of the flume. In hydraulic jump mixing, loss of head is appreciable and the detention time is also brief.

(b) Static mixing

Alternatively, in-line pipe static mixers can be considered provided that the raw water is properly screened and does not contain leaf debris that might wrap around the elements of the static mixer. Static mixers are a relatively new devices for blending (mixing) two liquid materials. The device consists of mixer elements contained in a cylindrical (tube) or squared housing. The static mixer elements consist of a series of baffles that are made from metal or a variety of plastics. Typical materials of construction for the static mixer components included stainless steel, polypropylene, and teflon.

The overall system design incorporates a method for delivering two streams of liquids into the static mixer. As the streams move through the mixer, the "non-moving" elements continuously blend the materials. Complete mixing is dependent on many variables including mixer length, tube inner diameter, number of elements and the design of the elements.

(c) Other types of hydraulic mixing

The sudden drop in hydraulic level of water over a weir causes turbulence and chemicals can be added at this ‘’plunge’’ point with the aid of diffusers. Similarly in pressure conduits, the chemicals can be added at the throat of venture or just upstream of orifice located within the pipe.

Such methods are highly recommended as opposed to electric stirrers which spread the chemical across channel or water.

(ii) Mechanical devices

There are two types of mechanical devices, the usual one, being the rapid rotation of impellers or blades in water and the other being mixing with the aid of a jet or impingement over a plate.

(a) Rapid rotation of impeller type

These operate at high rotational speeds from 150 to 1500 rpm or more and are mounted on a vertical or inclined shaft. In the design of this type of flash mixer which are usually deep, circular or square, a detention time of 10 to 40 seconds is provided. The preferred velocity gradient is 300 m/s/metre depth or more.

Power required is 1 to 3 watts per m3/hr. flow. Impeller diameter to tank diameter Di/Dt should be 0.2 to 0.4.

Where Di is impeller diameter and Dt is tank diameter.

The shaft speed of the propeller should be greater than 100 rpm for imparting a tangential velocity greater than 3 mps. The preferred ratio of height to diameter of tank is 1:1 to 3:1. To reduce vortex and rotational movement of water about the impeller shaft, baffles projecting 1/10 to 1/12 of the tank diameter at least in four place along the wall of tank, should be provided. The mixing chamber can be placed below the chemical feed flow ensuring short chemical feed lines. However use of electric driven stirrers should be avoided where possible due to maintenance problems.

(b) Impingement types

Water is forced as a jet through a nozzle impinging on a plate where the chemical is added. An auxiliary pump is used to create the jet action.

3.3.1.3 Flocculation

Whilst the reaction· of coagulant with alkalinity of water is practically instantaneous, the precipitate first formed is of molecular size, and so materials in water must be brought together by some means of aggregation for particles growth to occur. This process is called flocculation. The rate of flocculation depends on parameters such as particle size, pH value, water temperature, electrolyte concentration, time of flocculation, size of mixing. basin and nature of mixing devices.

Hence, the flocculation process, following coagulation, allows smaller particles formed during the rapid coagulation stage to agglomerate into larger particles to form settleable and/or filterable floc particles. After coagulant addition, the process water is mixed slowly for a defined flocculation period, commonly 10 - 30 minutes. However the optimum flocculation time will vary depending on the raw water quality and downstream clarification process. Gentle mixing during this stage provides maximum particle contact for floc formation, whilst minimising turbulence and shear which may damage the flocs. Effectiveness of flocculation depends on the delay (or contact) time and mixing conditions prior to any flocculants being added, the rate of treatment, water temperature and the mixing conditions within the flocculation chamber.

Contact flocculation is a variation from conventional flocculation in which the flocculation takes place within the clarification process. The coagulation step remains the same, however the flocculation chamber contains a contact medium. This medium traps the flocculating particles, which will then attach to other particles, thereby continually increasing the size of the flocs until the build up of particles clogs the media. Backwashing is then required to remove the flocculated particles.

3.3.1.4 Slow Mixing

The intensity of mixing. depend upon the temporal mean velocity gradient (G). This is defined as rate of change of velocity per unit distance-normal to a section and has the dimension of t^{-1} and is generally expressed as $sec^{\cdot 1}$. G can be calculated in terms of power input by the following expression

$$G=1000 \times (P/(M \times V))^{1/2} \qquad 3.3$$

Where,

P is the power dissipated in the water (kilowatts)

V is the volume in m^3, of the tank or basin, contents to which power is applied,

M is the absolute viscosity of the water in centipoises, and

G is the mean velocity of gradient (l/sec).

The total number of particle collisions ·is proportional to G × T where T the detention period of the flocculation basin which is very significant since flocculation is a time rate process. The product, G × T is non-dimensional and a useful factor for design. The desirable values of G in flocculation vary from 20 to 75 sec^{-1} and G × T, from 2 to 6×10^4 for aluminium coagulation and 1 to 1.5×10^5 for ferric coagulants. The usual detention period varies from 10 minutes to 30 minutes.

A) Types of slow mixers

These can be categorized as:

(i) gravitational or hydraulic
(ii) mechanical and pneumatic

(a) Gravitational and Hydraulic Types

These type of mixers use the kinetic energy of water flowing through the baffles in the plant.

(b) Baffles channel or basins

A baffled or sinusoidal channel consists of series of baffles. around the ends of which the flowing water is reversed in direction, thus causing turbulence or agitation at each point of reversed flow. The same effects are obtained by arranging the baffle so that water flow over and under them. This system is recommended up. for a flow of up to 200 m^3/hr. as no mechanical equipment is needed. Such channels can either be horizontal or vertical.

B) Horizontally Baffled Flocculation (mixing) Basin

This should have a velocity of flow of 0.25 to 0 4 m/sec and a detention period of 20 to 30 minutes. The head loss should be 15 to 60 cm.

Higher velocities are preferred for highly turbid water. The loss of head in bends is computed as 2 to 3.5 times the velocity head for each 180^0 bend, (half cycle turn), for round-the-end type. The disadvantage of this type is that velocity gradients tend to be too large at the 180^0 degree bend and too small in the straight channels.

C) Velocity baffled flocculation (mixing)basin

This is quite similar in function with the same disadvantages as the horizontally baffled floc basin. For loss of head calculation each lower pass is considered as an orifice and that each upper pass as submerged weir for over and under type. The friction loss for straight channels, is worked out by assuming suitable friction coefficients. Other parameters remain the same as for horizontally baffled flocculation basin.

D) Pipe Flocculators (Mixers)

If a long stretch of raw water main is available, coagulant can be introduced at the far upstream end with a provision for a venturi throat and gradually diverging section of the down stream or by a static mixer. About 3 to 8 km pipeline is necessary to provide effective flocculation. High friction losses are likely to be encountered.

3.3.1.5 Recommendations

(i) for rapid mixing

Hydraulic jump mixing should be used in small plants (flow less than 300 m^3/hr) as this device is simple and free from moving parts. Standing wave flumes which are constructed for measurement of flows, can also be used for rapid mixing as hydraulic jumps takes place at the throat. Hydraulic jump mixing can be included for larger plant as a standby.

(ii) for slow mixing

The baffle system is recommended up to 200 m^3 hr, as no mechanical equipment is needed. The relative advantages are less chances for short circuiting and simplicity in construction. The disadvantages with this type of system are more loss of head and space requirement.

3.3.1.6 Design of Flocculation

Formation of flocs is done by one of two main methods:

(i) Manual operation or baffle basin, operation

The operational does not involve mechanical moving parts except only fixed staggered baffles to achieve large and settliable floes. The following are essential parameters:

1. Detention time for floes 40 to 60 minutes
2. Velocity of water 0.2 m/sec to 0.45 m/sec
3. Calculate the capacity to be treated
4. $V = m^3/day \times Time$
5. Select the required depth and calculate the basin area
6. Calculate the dimensions for the basin

Check for the right basin

$$W = (Q\ h) / V = h / T \qquad 3.4$$

Where,

W = dissipation function
Q = usual Weight of Fluid
h = head loss by friction
G = rate of flocs

$$G = (W / u)^{1/2} \quad \text{or} \quad G \times T = (G / u)^{1/2} \qquad 3.4$$

Range of $G \times T = 20 - 75\ sec^{-1}$,and
Range of $G \times T = 10 \text{ to } 10^5$
Where $G \times T$ = controlling parameter for flocculation

(ii) Mechanical Flocculation (mixing)

This is flocculation where mechanical moving parts are involved.

- Detention time = 30 - 60 minutes
- Velocity of agitation = 0.45 m/sec
- Paddle area = 10% - 20% of .area of flocculator
- Relative speed of paddles
  - at first 0.4 - 0.8 m/sec
  - at the end 0.1 - 0.3 m/sec
- Head loss in mechanical flocculation is generally of the order of 150mm.

3.3.2 Flocculation

Flocculation is an agitation process to form gelatinous cloudlike aggregations after adding coagulant or is to bring colloids together by physical phenomenon to achieve large sizes.

- Time of mixing in the flocculation chamber should not exceed 1 min.
- Detention time in flocculation tank should be 10 - 20 minutes
- Flocculation tanks should be equipped with baffles

3.3.2.1 Gravitational Flocculation

Baffle chambers should be designed for a channel velocity of 0.1 - 0.3 m/s and a slot velocity of 0.5 - 0.6 m/s in the first 2/3 and 0.3 - 0.4 m/s in the last 1/3 of the chamber.

3.3.2.2 Mechanical Flocculation

This type of flocculator should be used only in very exceptional cases. There should be a minimum of 2 chambers. The peripheral velocity should be 0.6 m/s in the first tank and 0.2m/s in the second tank. If more. than two chambers are used the peripheral velocity can vary from 0.9 m/s to 0.2m/s

3.3.2.3 Power Input and Detention Period

The following criteria apply to both flocculation methods. The detention period should be 15-20 min. and· the velocity gradient G is 30 to 60 sec^{-1}. The product G × t (where t is the detention period in seconds) should be 30,000 to 100,000. (Refer the formula for computing the velocity gradient on 3.4.1.6).

The power, P can be calculated as:

$$P = (q \times h)/102 \quad (\text{kW}) \qquad 3.5$$

Where,

q = flow through the flocculation tank in l/s

h = head loss in the flocculation tank in m

It may be assumed that all the velocity head is lost when the water passes through the baffle slot and that also velocity head is lost at all changes of direction of 90° and more. Thus the losses can be calculated using the following formula:

$$h = (n \times v_1^2 - m \times v_2^2) / (2 \times g) + \text{normal channel losses} \qquad 3.6$$

Where,

n is the number of direction changes

m is the number of baffles

v_1, is the velocity in the channel

v_2 is the velocity in the baffle slot.

Unless the floor at the flocculation channel is sloping with the same gradient as the hydraulic gradient of the water, v_l and v_2 will change along the channel due. to the decreasing water depth. The total head· losses in the flocculation tank usually amount to between 150 and·600 mm.

3.3.3 Settling

Settling of the flocs produced by the coagulation and flocculation process is termed clarification or sedimentation. This is distinct from pre-settling of highly turbid waters in detention ponds.

Historically, clarification involved the simple principle of particle settling to separate the floc particles. New technologies such as dissolved air flotation (DAF), and high rate clarification processes, such as lamella plates, tube settlers, and buoyant media clarification, have been developed. Lamella plates have been successfully used to uprate plain settling basins of different types. This clarification process is illustrated in Figures 3.3.and described below.

The surface loading rate is a key parameter in clarifier design, irrespective of the clarifier type. This is preferably expressed in $m^3/h.m^2$ (m/h). This is the flow (m^3/h) that occurs over the horizontal area (m^2) of the settling zone of the tank. Acceptable surface loading rates vary significantly for the different clarification types from 1 m/h for a hopper bottomed upflow clarifier, to 12 m/h for a Dissolved Air Flotation process and 20 m/h for purpose built lamella plate systems.

I Conventional Settling Basins

Conventional settling basins (or sedimentation tanks) may be classified on the basis of flow direction (horizontal, radial, or upflow), the presence or absence of a sludge blanket, and shape (circular, rectangular, or hopper/wedge bottomed). It is possible to recycle a fraction of the sludge in an effort to improve flocculation. Horizontal settling tanks are preferable for smaller plants providing there is sufficient space, whilst upflow clarifiers are suitable for larger plants and a large range of raw water turbidities. However they are sensitive to flow changes. Flocculation times for upflow tanks are generally less than for horizontal tanks (3 - 6 minutes) as further flocculation will continue to occur in the clarifier itself. Typical surface loading rates for conventional settling basins are 1 m/h for horizontal tanks and 2 m/h for upflow tanks. The latter can be increased to up to 5 m/h if polyelectrolytes are used.

Conventional settling tanks should be designed using weir overflow rates of 250 to 300 m^3/m^2/day and surface loading of 1.0 m^3/m^2/h (or 24m^3/m^2/day) calculated on net available surface area.

Effective depth of water should be a minimum of 2.0 m allowing for sludge deposit, with the total net depth at least 2.5 metres. To avoid destruction of flocs, the velocity of the flocculated water between flocculation tank and the settling tank should at no place exceed 0.2 m/sec.

The flocculated water should be distributed evenly into the settling tank through ports with a maximum individual distance of 0.5 m. In order to prevent inlet ports from being clogged by sludge the lowest ports should be positioned at least 0.3 m above the tank bottom.

II Lamella Settlers

Lamella settlers, Figure 3.2 illustrates the use of inclined plates or tubes to increase the effective surface area for settling (and hence are also known as plate or tube settlers), thereby increasing the efficiency of the clarification process. For a given throughput the footprint of a lamella settler will be considerably less than a conventional settling tank. Typical surface loading rates for uprated conventional settlers are 7 - 12 m/h, but purpose built units can attain 20 m/h or even higher rates. Lamella settlers are less vulnerable to flow fluctuations than conventional settling tanks but require a slightly higher level of operation and maintenance. The angle of inclination to the horizontal is usually 55° – 60°.

Because the retention period is considerably less than with conventional settlers, good flocculation is necessary.

The introduction of lamella plates is however an excellent way of uprating an existing settler. The use of new purpose built lamella settlers should however be limited to treatment works where management and operators can be expected to achieve efficient flocculation practices.

III Dissolved Air Flotation

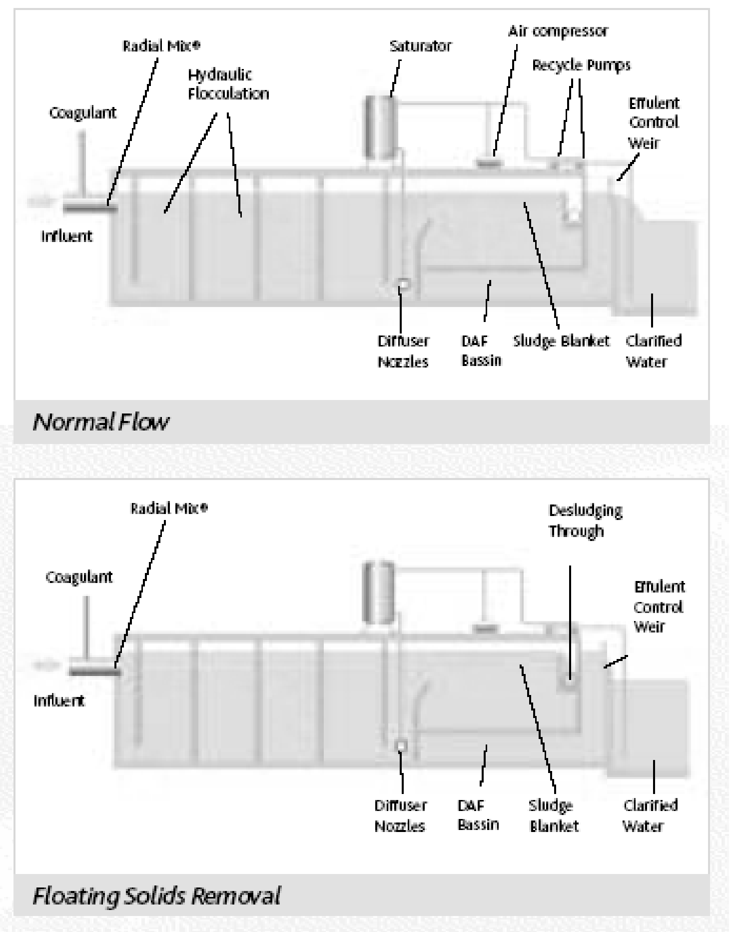
This is a relatively new solution for the clarification of surface and ground waters

The process as shown opposite is relatively simple and can be very effective. After flocculation, the produced floc attaches to micro-bubbles and rises to the water's surface. The floated solids are periodically evacuated either hydraulically or mechanically, depending on the sludge concentration required.

The dissolved air flotation process can be a good solution for the treatment of water with a high concentration of algae or other low density particles.

The system is said to have a number of advantages including:

- Reliable removal of algae, *cryptosporidium* and *giardia*
- Removal of colour and taste compounds
- Removal of low-density solids
- No polymer required
- Concentrated sludge
- Rapid start-up after shutdown
- Compact footprints
- Few mechanical components



Normal Flow

Floating Solids Removal

FIGURE 3.2: DISSOLVED AIR FLOTATION PROCESS

- Low operating costs

It does however require a relatively high level of competence to operate and maintain the micro-bubble production system although the remainder of the system is straightforward to operate and maintain.

3.3.3.1 Sedimentation in Settling Tanks

Sedimentation is the process whereby particles of sizes more than 10^{-3} or 10^{-4}/cm can easily settle with respect to' time. The basic equation is:

$$V_s = [g/(1.8 \times 10^4)] \times (r - 1) \times d^2 / \gamma \qquad 3.7$$

with Reynolds No. less or equal to 0.5

Where,

V_s = settling velocity (mm/s) (range 2-4 cm/min)
g = acceleration due to gravity = 9.81 cm/sec^2
r = relative density of the particle
d = diameter of the particle in mm
γ = kinematic viscosity of water in m^2/sec at the temperature of the fluid

3.3.3.2 Factors Influencing Sedimentation

The factors which influence sedimentation are:

a) Size, shape and weight of the particle
b) Viscosity and temperature of water
c) Surface overflow
d) Surface area
e) Velocity of flow of water
f) Inlet and outlet arrangement
g) Detention periods, and
h) Effective depth of basin.

3.3.3.3 Types of Tanks

In drinking water treatment, continuous flow tanks are generally used. They can be functionally divided into three categories

(i) Vertical flow tanks
(ii) Horizontal flow tanks
(iii) Combined flow tanks (vertical and horizontal)

The efficiency of vertical flow tanks is limited to those settling particles which have settling velocity greater than upward flow. For flocculent particles the efficiency is however higher than for discrete particles in vertical flow tanks because a blanket will be formed.

Horizontal flow tanks are more effective than vertical flow tanks and they are also cheaper to construct and therefore, generally, are used in small and medium sized drinking water treatment plants. Amongst the representative designs of horizontal flow tanks, the following maybe mentioned.

3.3.3.4 Rectangular tanks

These are usually manually cleaned or by using pressure hoses or cleaned under hydrostatic pressure using perforated pipes embedded in furrow ridge and furrow bottom.

3.3.3.5 Design of Conventional Horizontal Sedimentation Tank

The following procedure can be adopted:

(i) Assume a uniform velocity in the tank (surface loading Q/A = 20 to 33 $m^3/m^2/d$) for:

  (a) Horizontal flow settling tanks
    - plain settling less than 0.15 $m^3/m^2/d$
    - chemical – added, settling less than 24 m^3/m2/d

  (b) Vertical flow settling tanks less than 36 m3/m2/d

(ii) Max. Weir loading should be 245 $m^3/m/d$

(iii) Calculated the area of the Basins, A = flow capacity / surface loading (m2)

(iv) Calculate Dimension of the tanks, L/B, which should lie between 2 and 4.

(v) Common Depth is 3 to 4 metres, a large depth recommended where sludge consists of inorganic matter and can be stored for longer periods.

3.3.3.6 Vertical Flow Tanks

Solid particles can be moved in, a single vertical flow tank when the settling vertical is higher than the upflow velocity and the decanted water is removed from the top.

The solid contact units combine various pre-filtration operations of water treatment in a single (circular or square) basin when the upward vertical flow of water passes through a layer (blanket) of flocculated suspended matter.

3.3.3.7 Check for Detention Time

It is necessary to check for detention time, T = v/Q 3.8

Where,

T = time in hours
v = volume of the settling, basin (m^3)
Q = Flow capacity (m^3/hr)

The time should be:

- (3 - 4 hours) in chemical added tanks
- 6 -9 hrs in plain settling tanks, and
- or 1.5 - 2 hrs in vertical settling tanks.

It is advisable to conduct a settling analysis in the laboratory to arrive at the optimum surface loading for a particular situation.

3.3.3.8 Inlets and Outlets

The best conditions for sedimentation are secured with uniform velocity of flow and a satisfactory method of attaining this purpose is to pass the water through a diffuser wall. The inlet to the tanks may be over a submerged weir or through openings large enough to prevent breaking of floc and also to provide a reasonably good distribution. The diffuser wall should be built within 1.5 to 4.5 m of the inlet to the tanks and training walls provided around the tank to prevent irregularities in the flow.

In horizontal flow settling tanks, velocity of flow through the slots or perforations near the inlet should be between 15 and 25 cm/sec to ensure uniform distribution. It is desirable that the diameter of the slots does not exceed the thickness of the diffuse wall. In the sludge blanket type of vertical flow settling tanks the velocities of incoming water are kept at about 60 cm/sec to ensure proper mixing of chemical coagulants.

The average flow-through velocity, in horizontal flow rectangular tanks is normally kept between 0.5 and 1.5 cm/sec. Weirs, notches or orifices are used as outlets. There is a growing trend towards the use of outlet launders or troughs across a good part of the surface of the settling tanks. These are spaced at a distance of one tank depth between each trough.

3.3.3.9 Outlet Weir Loading

Weir length relative to surface area determines the strength of the outlet current. Weir loading up to 300 m^3/m/d are recommended. When weirs are very long in relation to the flow, it may be difficult to secure uniform discharge over their entire length, unless a saw toothed edge breaks up the weir into triangular notches with jets of adequate thickness. The outlet launders of the settling tank can be designed in a similar manner

3.3.3.10 Settling Tank Efficiency

The hydraulic efficiency of a settling tank is the flow through period, which can be expected to approach the critical detention period only in ideal basins. The efficiency is dependent upon eddy currents, concentration of particles, shape of tank and also inlet and outlet arrangement.

A well designed tank should be capable of having a volumetric efficiency of at least 70%. Settling tanks should be capable of giving settled water having a turbidity not exceeding 20 and preferably not exceeding 15 JTU.

3.3.3.11 Lamella Plate and Tube Settlers

Settling efficiency of a basin depends upon surface loading. Lamella plates and small diameter tubes having a large wetted perimeter, relative to wetted area providing laminar flow conditions and low surface loading rates have shown good results in terms of settling efficiency and economy in space as well as cost. Plate or tube configuration can be horizontal or steeply inclined. In inclined plates or tubes (55^o – 60^o) continuous gravity drainage of the settleable material onto the floor below can be achieved, without impairment of effluent quality. However, to work effectively an efficient flocculation stage is critical.

For the uprating of existing settling tanks a relatively cheap method is to use epoxy coated trapezoidal section roofing sheets, riveted together on the flat part to form tube packs that can then be fitted into a simple supporting frame erected in the settling basin.

In purpose built lamella plate settlers the water enters at the base of the lamella plates and travels upwards between the lamellas. Each space between the lamella plates tends to act as semi-independent settling modules with the lamella plates extending from near the base of the tank to about 125 mm above the top water level. The clarified water is collected by saw-toothed notched launders running along each side of the plate. Unless sludge is removed mechanically by a scraper, sufficient depth beneath the plates is required for access for cleaning, although this can be aided by pressurised water. A typical arrangement is illustrated below:

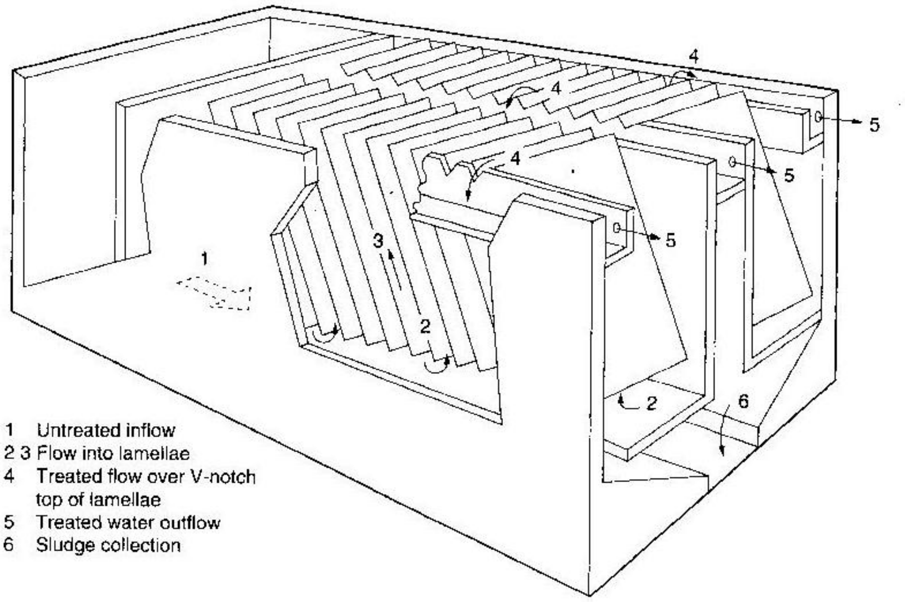


FIGURE 3.3: TYPICAL PURPOSE BUILT LAMELLA SETTLER

Plates are made of stainless steel or plastic with a width of 1.25 to 1.5 m and a length of 2.5 to 3.25 m including the length above water. Plate thickness is usually about 0.7 mm for stainless steel whilst the horizontal spacing between plates is varied according to nature of the raw water but within the range 50 – 80 mm.

Total settled area is then:

$$(n-1) \times L \times W \times \cos\theta \qquad 3.9$$

Where,

n = number of plates

L = plate length in water (m), less the transition length

W = the plate width (mm), and

θ = the angle of inclination of the plates to the horizontal ($55^{o} – 60^{o}$)

3.3.3.12 Settling Basin Sludge

In settling tanks, 80% of flocs settle down and should be frequently removed to allow for other incoming settleable particles. The flocs to be removed will be the dissolved and settleable solids from water sources and that from alum solution reaction. Clarifiers which rely on a floc blanket which is held in suspension by an upward flow of water should only be used where the water flow is continuous i.e. 24 hours, 7 days a week. Otherwise a horizontal flow sedimentation tank should be preferred.

The net calculated volume of settling tanks should be increased by 25% to accommodate sludge.

3.3.3.13 Sludge Removal (Design)

The design criteria recommended is as follows:

(1) Solid sludge is assumed to be 600 mg/litre of water

(2) Efficiency of settling units is 80% – 90%
(3) Solid removed = solid sludge / efficiency
(4) Add Alum dose to (3) above
(5) in every 100cc – 5mg of (ss) is removed.
Volume of sludge = (total ss removed × 100) / 5
% of sludge = (volume of sludge x 100) / Volume of waste
(6) Capacity of sludge in settling tanks = $\frac{(\text{Total water demand} \times \text{\% of sludge})}{10^6}$

3.4 FILTRATION

Filtration is the process in which organisms, bacteria and particles of size less than 10^{-8} cm are removed. There are four main types;

- slow sand filters
- rapid (gravity) sand filters
- pressure filters
- upflow sand filters

All methods are used in the water industry throughout the world; however the first two are the more common. The middle two require pre-treatment by the use of coagulation and flocculation normally followed by settling basins prior to filtration whilst slow sand filters can under the right conditions produce high quality water free from pathogens, taste and odour without the need for chemical aids. Upflow filters are the least common and usually require similar pre-treatment.

However, there is always some floc carry over from the settling process and it is necessary to pass this water through a rapid sand filter to capture and strain out the unsettled floc and the particles trapped within it reducing numbers of bacteria and removing most of the solids in the process. The medium of such filters is usually sand of varying grades.

Where taste and odour may be a problem (organoleptic impacts), the sand filter may include a layer of activated carbon to remove such taste and odour.

Sand filters become clogged with floc after a period in use and they are then backwashed or pressure washed to remove the floc. This backwash water is usually run into settling tanks so that the floc can settle out and it is then disposed of as waste material. The supernatant water is sometimes run back into the treatment process, although this can bring some problems with it, or disposed off as a waste-water stream.

Inadequate filter maintenance has been the cause of occasional drinking water contamination and it is therefore important that both management and operation is provided.

During the filtration, processes such as:

a) Screening,
b) Sedimentation,
c) Biological Action, and
d) Electrolytic Action,

are simultaneously taking place while flow passes between the sand particles.

Particles and organisms cling to sand by mass alteration because of the zigzag movement of water and get attached due to molecular attraction.

In horizontal flow, particles and organisms are more efficiently removed than in vertical flow.

Filtration also helps in removing colour, odour, iron and manganese from water.

The criteria for designing filters are:

(i) Rate
(ii) Size of Media
(iii) Depth of Media
(iv) Type of Media
(v) Arrangement of gradation of Media
(vi) Fluid characterises
(vii) Head loss
(viii) Length of run
(ix) Method of cleaning

3.4.1 Slow Sand Filtration

Slow sand filters operate in two ways, a deep sand bed, and a surface coating filter which processes the water biologically. In this way, treatment in slow sand filters includes the removal of pathogenic organisms, thereby reducing the risk of transferring water borne diseases.

They consist of a relatively deep sand bed over underdrains (typically 0.9 – 1.2 m deep on start-up and are cleaned by periodically removing the surface layer until the remaining bed depth is not less than 0.6 m before being re-sanded. The sand used is finer than the 1 – 2 mm range that is typical in rapid sand filters, having, typically, a mean particle size in the range of 0.15 – 0.4 mm. The surface of the sand ripens; that is, a microbiological layer, primarily of algae and bacteria, develops on it, adding a further, biological process to the sand filtering. This layer is called a schmutzdecke layer, a German term meaning dirt layer. It takes a day or so to develop and, until it does, the filter will not present a proper barrier to microbial pathogens. This layer does not develop on rapid sand filters because the water flows much faster, providing higher shear loads, and the sand is much coarser, allowing the algae cells to pass into the media and not grow.

There is no backwash system, so all solids captured build up on the surface, with a small amount of penetration into the sand. For protozoa compliance, the final water turbidity should be below 0.5 NTU, and some form of post-disinfection may be necessary in order to achieve bacterial acceptability.

For slow sand filtration, the turbidity in the in coming water should not exceed an average of 5 - 10 JTU and certain (filamentous) types of algae and organic load should be low, otherwise there is danger that the schmutzdecke will become a gelatinous layer greatly reducing or even preventing through flow. This means that swamp water, water from rotting vegetation in natural forest areas and from reservoirs where trees and scrub have not been removed prior to filling may cause problems. Peak turbidity's above this value should be of short duration only. In cases of higher turbidity, preliminary treatment such as roughing filters is necessary.

To provide for the relatively long time required for cleaning and .restarting the filters, a minimum of two units should be built at anyone location, designed in such a way that the units can be operated independently.

For calculation of the total area of filter beds a working rate of 0.1 - 0.15 $m^3/m^2/hr$ is recommended. When one filter is not operational, the working rate of the remaining filter should not exceed 0.2 $m^3/m^2/h$.

The filter should be constructed with an inlet chamber. In the chamber one wall will be made as an adjustable weir to be regulated to fit the level of the filterbed after the top sand layer has been added to or scraped off. The chamber outlet should be made large enough to keep the entrance velocity into the slow sand filter below 0.1 m/sec.

Including allowance of progressively scraping off, the total filter sand thickness should be about 1.0 m. Below the filterbed allowance must also be made for the drainage system.

There should be ample room for the head of water above the filterbed, and about 1.5 is considered to be adequate. To avoid uncontrolled over-flow, the filter chamber should be provided with an overflow arrangement at the top water level.

The filter sand must be free from any clay or silt content and preferably of a well-rounded quartz material. Organic matter should be avoided. When choosing the filter sand, the grain size distribution should meet the effective size 0.15 to 0.35mm and the coefficient of uniformity should be less than 3 and preferably about 1.75. Sand washing and storage bays will be required on the site and all filter sand brought to the works should be thoroughly washed before use.

To prevent the water being filtered from short-circuiting along the chamber walls, these should have a rough surface in the filter zone.

The collection pipes beneath the filter zone should be placed at 1m intervals and be perforated with holes of 3.5mm dia., or by saw cuts placed on the underside of the pipe. The distance between holes or saw cuts should be about 100mm. Around the collection pipes and up to a level of about 300mm above, a bottom layer of broken stones (50 100mm) and coarse gravel (grain size 16 - 23mm) should be placed. Above this should be placed a layer of about 100mm finer gravel (grain size 4 - 5.6mm) and a layer of about 100mm coarse sand (grain size 1-4mm) on top of which is placed the actual filter sand.

Typical performance of slow sand filters is shown in Figure 3.4 and in the Table on the following page.

In summary, this type of filter should only be considered when turbidity .is less than 10 NTU (l-15mg/l), and there is ample flat or gently sloping land for both its construction as well as that for sand washing facilities and storage bays.

Where seasonal turbidity are higher, pre-filtration in roughing filters may render slow sand filters satisfactory provided the organic load of the raw water is not likely to be a problem.

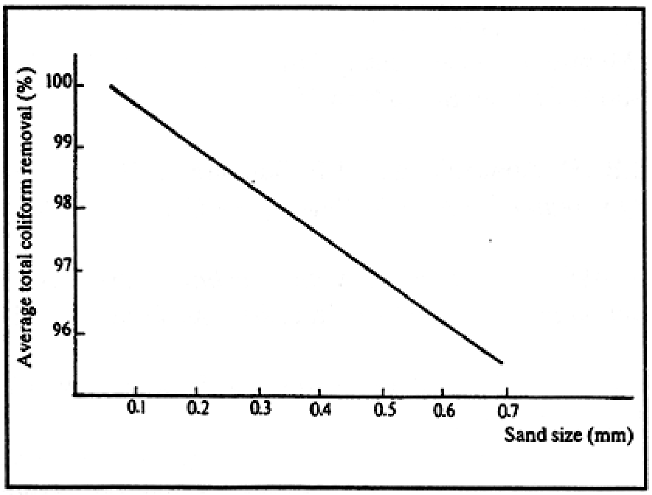


FIGURE 3.4: EFFECT OF SAND SIZE ON THE REMOVAL OF TOTAL COLIFORM BACTERIA

TABLE 3.4: TYPICAL PERFORMANCE OF SLOW SAND FILTERS

PARAMETER OF WATER QUALITY	PURIFICATION EFFECT OF SLOW SAND FILTRATION
Colour	30% to 100% reduction
Turbidity	Turbidity is generally reduced to less than 1 NTU
Faecal coliforms	95% to 100%, and often 99% to 100%, reduction in the level of faecal coliforms
Cercariae	Virtual removal of cercariae of schistosomes, cysts and ova
Viruses	Virtually complete removal
Organic matter	60% to 75% reduction in COD
Iron and manganese	Largely removed
Heavy metals	30% to 95% reduction

Slow Sand Filter - Design

For the design of slow sand filters, the following is required:

(A) Filter Units

(1) Rate of .filtration should be 0.15 - 0.3 $m^3/m^2/hr$.

(2) Determine flow capacity in the filter (water m^3/day.demand)

(3) Calculate filter surface area = flow capacity (m^3) / rate of filtration

(B) Main Water Under-drain

(4) Select flow capacity and flow velocity

(5) Calculate Diameter of the under drain = (22 × d2) / (28 × r)

(6) Area of holes or slots to be 1.5% of area of the filter.

Operation and Maintenance

In theory, the most attractive aspect of slow sand filtration is its simplicity of operation and control, and after a short training period, the operation of the system can be mastered by local operators. In practice however and because it is labour intensive, unless there is good management the routines required are not followed.

Hence the suitability of slow sand filtration needs careful attention to strict adherence to operation and maintenance procedures to be successful.

Filter cleaning, by scraping off a 25 mm surface layer of sand, is required at intervals, usually of between 30 and 100 days depending on the turbidity of the water being filtered. Filter surface fabrics have been developed for slow sand filters to improve and ease the cleaning process. However, because a small proportion of the sand is removed in each the cleaning process, the sand layer will need to be topped up from time to time. The depth of sand should not be allowed to become less than 600 mm.

Topping should be done using washed sand only, removed during previous filter cleanings, or by fresh washed sand. As pathogen removal is achieved within the biological slime layer (schmutzdecke) which forms on the surface of the sand and this takes a few days to re-form after each cleaning, there will be a slight decline in the performance of the filtration system during this time. Usually, this decline is not significant, however for other than small rural supplies it is usual that precautionary disinfection is used, especially during such periods.

3.4.2 Rapid Gravity Sand Filtration

In this filtration process, water flows onto the top of the filter media and is driven through it by gravity. In passing through the small spaces between the filter's sand grains, impurities are removed. The water continues its way through the support gravel, enters the under-drain system, and then flows to the reservoir. It is the filter media which actually removes the particles from the water. The filter media is routinely cleaned by means of a backwashing process.

Single media gravity filtration is inefficient in that within the grading range of the sand, the finer particles tend to desegregate to the top with the coarser ones below so that the water being filtered is presented with the smallest openings first.

Rapid sand filtration is a technique commonly used for treating large quantities of drinking water. It is a relatively sophisticated process usually requiring power-operated pumps for backwashing or cleaning the filter bed, and some designs require flow control of the filter outlet. A continuously operating filter will usually require backwashing about every two days or so when raw water is of relatively low turbidity and at least daily during periods of high turbidity. Pre-treatment of the raw water, using coagulation / flocculation agents in combination with settling tanks is necessary, especially where turbidity is high. Relatively large quantities of filter backwash water, as well as sludge from the settling process may be generated and require some form of treatment before discharge to the environment.

Because of the higher filtration rates, the area requirement for a rapid gravity filtration plant is about 20% of that required for slow sand filters.

Filters are constructed in banks of three or more (constant flow) and six or more (declining rate) and each filter unit should have an individual inlet that can be closed for maintenance and back washing. The inlet should be designed in such a way that flushing and velocities over 0.4m/sec do not occur.

Surface load should be between 4 and 7 $m^3/h.m^2$, and the filter structure should be designed with a minimum height between the top of the filter media and the bottom of the wash water channel of at least 30% of the height of the filter media as this expands during backwashing. It may be necessary to include for air-scour as well as backwashing, or for the two combined in a single operation.

There are two principle types of rapid sand filters, namely uniform rate and declining rate. The former require sophisticated rate control valves and are not recommended.

There should be a first - filtrate connection on the outlet pipe through which filtered water can be let to waste the first 10 minutes or so after backwashing.

Normally a sufficient distribution of the washwater will be achieved if:

- Ratio of area of orifice to area of bed served (1.5 to 5) × (10^3) : 1
- Ratio of area of main to laterals served (1.5 to 3) : 1
- Diameter of orifices 5 - 20 mm
- Spacing of orifices 100 - 300 mm centre to centre
- Spacing of laterals approximating to the spacing of orifices.

The filter bed should be approximately 1.0 thick and preferably consist of well rounded quartz sand with an effect size of 0.7 - 1.0 mm and uniformity coefficient in the range of 1.3 - 1. 5.

The available hydraulic head above the top of the filter bed should be 1.3 - 1.5 m.

The following stratification of the sub structure should be used to support a filter of an effective grain size as suggested earlier (finest strata at the top.

(i) 0.15 m of grain size 2 - 2.8 mm
(ii) 0.10 m of grain size 5.6 - 8 mm
(iii) 0.10 m of grain size 10 - 20 mm
(iv) 0.10 m of grain size 20 - 40 mm
(v) 0.10 m of grain size 40 - 60 mm

The washing velocity should be in the range of 35 – 55 m^3/m^2/hour, however care must be taken to ensure that sand carry over into the washwater channel does not occur and the actual washwater rate adjusted accordingly.

In order to achieve proper washing of the filter a storage volume sufficient for continuous washing for an 8 to 10 minute period should be available.

In order to achieve the best possible control of flow through the filter after cleaning, it is recommended that a weir is located on the clear water line after the filter, and on such a level that the top of the filter bed is always submerged during be normal operation. The weir may be replaced by a gooseneck where desired.

Rapid Sand Filter - Design

The following design guidelines are suggested:

(A) Filter Units

(1) Rate of .filtration should be 4 - 6 m^3/m^2/hr.
(2) Determine flow capacity in the filter (water m^3/day.demand)
(3) Calculate filter surface area = flow capacity (m^3) / rate of filtration

(B) Main Water Under-drain

(4) Select flow capacity and flow velocity
(5) Calculate area required = (low capacity)/ (low velocity) (m2)
(6) Calculate diameter of the under drain = $(\Pi \times d2) / 4 = A$
(7) Area of holes = 0.2 - 0.4 % of the area of filter
(8) Use five layers .of sand particles in the filter as indicated above:
(9) Water depth 1 – 1.2 m
(10) Sand depth 1 – 1.2 m

Backwashing

A major cause of poor performance by rapid gravity filters is a result of either inadequate or excessive backwashing rates. Backwashing is sometimes carried out by water alone but more often by air and water usually applied one after the other by reverse flow to the filter bed.

The first operation however is to allow the filter to drain down until the water lies a few centimetres above the top of the bed. Air is then introduced through the collector system at a rate of about 6.5 to 7.5 mm/s. The surface of the filter sand should show an even spread of bursting air bubbles coming through the sand. The water over the bed quickly becomes very dirty as the air-agitated sand breaks up, and as surface scum and dirties loosened from the sand grains. This is then followed by an upward flow of water at a very carefully selected and controlled velocity, sufficient to just expand and fluidise the bed by about 2 % but not too much so as to over expand the bed which reduces the scouring action or to cause sand to carry over into the washwater channel.

Getting this velocity correct is very important but all too often not achieved. A guide to washwater rates is given in the following table. However, plant managers must be prepared to adjust this based on specific experience, and if necessary to vary it as temperatures change throughout the year.

TABLE 3.5: WASHWATER RATES (mm/s) FOR 2% BED EXPANSION AT VARYING TEMPERATURES

SAND SIZE RANGE (mm)	EFFECTIVE SIZE (mm)	WATER TEMPERATURE (^oC)			
		15	20	25	30
0.50 – 1.00	0.55	4.0	4.5	5.0	5.5
0.60 - 1.18	0.75	5.6	6.3	6.9	7.5

Where air and water is applied separately, air scour normally lasts about 3 – 4 minutes and the water wash about 4 – 6 minutes. Where applied concurrently, air is first introduced and after about 1.5 – 2 minutes when it is fully established water is introduced and the combined backwash last for about 6 – 8 minutes. Air is stopped first and the water run for several more minutes to rinse the bed. Generally, total water consumption per wash amounts to about 2.5 bed volumes, but should normally not exceed 2% of the treated water output in well run plants..

Operation and Maintenance

Rapid sand filtration plants are complicated to operate, requiring good operator training in order for the plant to produce a product water of consistent quality and quantity. Good plant management is also essential

Problems with rapid sand filters are caused mainly by filter or media breakthrough which refers to water passing through the filter without adequate treatment. The turbidity of the filtered water is one of the best indicators of filter performance and should be checked regularly. Inadequate flocculation, filter clogging, mudball formation, and filter cracking can all lead to media breakthrough. In light of these problems, the filter top should frequently be visually inspected for loss in depth, mudball formation, surface cracks, and bacterial growth. The operator should also check that the backwash cycle is operating normally.

It is good practice to keep written records of all testing and work carried out on the treatment plant. This should include any maintenance tasks performed. This not only provides a record of the last check or maintenance task but also enables the operator to detect any changes in the operating characteristics of the plant. In addition, a spare parts inventory should be kept so that if a critical part breaks down, there is a replacement available. This is especially important in remote sites where the breakdown of a key part can result in the treatment plant being out of operation for an inconveniently long time while the replacement part is obtained.

3.4.3 Other Types of Filters

Two other types of filters are worth mentioning as under some conditions they may be preferred.

3.4.3.1 Pressure Filters

Pressure filters are circular pressurised vessels containing the filter media and usually designed for vertical flow. They work on the same principle as rapid gravity filters differing in that the filter medium is enclosed in a steel vessel and the water is forced through it under pressure.

Perforated pipes or a steel plate with nozzles are used to collect the filtered water and for the distribution of washwater. However, perforated pipes are recommended as they are far easier to replace than floor plates. The whole vessel is kept filled with water under pressure and at the highest point an air-release valve is fitted.

Manufacturing constraints keep the diameter to about 2.75 m.

The advantage of pressure filters is that the pressure in the mains is not lost in the filtration process, and such filters are worth considering for small schemes and flows, for medium turbidity ranges (suspended solid up to the concentration of approx. 100 mg/l) and where a single pumping system from source through to storage is considered appropriate. They also allow for progressive expansion by increasing the number of filters in parallel.

3.4.3.2 Upward Flow Filters

Upward flow filters are theoretically more efficient than gravity filters where the water to be filtered flows upwards through the naturally desegregated, progressively finer and finer media so that coarser particles are trapped first in the coarser bottom layers. This tends to extend the period between backwashing. Several sand grades, getting progressively finer upwards have also been used in which case some restraining means such as a grid is required often located about 0.1 m below the surface where upward backwashing is used.

Smaller units are also readily backwashed by reversing the flow of the filtered water and lend themselves to an automatic backwashing process. They may therefore be appropriate for small water supplies, and for use in emergencies. Patented continuous backwash upward flow filters are also manufactured.

Flow rates tend to be better than for rapid gravity filters but not usually markedly so.

3.5 SLUDGE DISPOSAL

Sludges can be classified according to the type of water treatment process adopted. Non-chemical sludges come from roughing filters, micro-strainers, and settlers unaided by chemicals, and from slow-sand filters. These sludges are for the most part innocuous and may often be disposed of to watercourses. Coagulant sludges result from treatment plants using coagulation and these often need to be pre-treated before disposal should the River Basin Authority require it as there may be insufficient onward flow in the river to provide an acceptable level of dilution. However, such sludges are difficult to treat for alternative disposal because of the relatively large volumes involved and the difficulties in de-watering due to their gelatinous nature.

In a well operated clarification and filtration plant using coagulation, most of the suspended matter is removed in the clarifiers, leaving about 2-5 mg/l of suspended solids to be removed by the filters. Therefore filter washwater is comparatively dilute and chemical free when compared to the clarifier sludge and if disposed of separately may be acceptable as return flow by the River basin Authority or otherwise relatively easy to dispose of, or to settle and return the supernatant water to the raw water entry point of the treatment plant. If this is done however, it should be done gradually and not as periodic shock loads and brings with it the risk of recycling micro-organisms. Additionally, the holding tanks for this washwater can also provide a breeding ground for some of the organisms and therefore has the potential to increase their number entering the treatment process.

Hence for filter washwater, the preferred method is either by return to the river or by settlement, in one of two tanks with gradual de-canting of the supernatant liquid and periodic removal and drying of the remainder and disposal as landfill or possibly as a soil conditioner.

The sludge water from the settling tanks is however the problem. As mechanical means such as sludge presses or centrifuging are not considered appropriate for Tanzania, it requires lagooning and then being allowed to dry assuming that the net evaporation rate exceeds the rainfall for most of the year. The dried sludge from the coagulated sludge is not suitable as a soil conditioner due to the ability of the aluminium to fix phosphorous in the soil thereby making it unavailable for plants.

An acceptable method may however be to use it to manufacture bricks and this and other methods are being actively researched, especially in Europe. Designers should therefore keep abreast of such developments where return to the water source or landfill disposal is not acceptable or practicable.

3.6 DISINFECTION

3.6.1 Introduction

The single most important requirement of drinking water is that it should be free from any micro-organisms that could transmit disease or illness to the consumer. Processes such as storage, sedimentation, coagulation and flocculation and rapid filtration reduce to varying degrees the bacterial contact of water. However these processes cannot assure that the water they produce is bacteriologically safe, and finally disinfection will be needed.

Disinfection is carried out observing the following criteria:

(i) The nature and number of organisms to be destroyed
(ii) The type and concentration of the disinfectant used
(iii) The temperature of water to be disinfected
(iv) The. time of contact
(v) The nature of water to be disinfected
(vi) The pH, acidity/alkalinity of the water

There are two principle ways of disinfecting water; one is physical and the other chemical.

In chemical disinfection in particular, the following points need to be followed:

- Good mixing between water and disinfection agent
- Sufficient dosage compared to water quality and types of micro-organisms that are to be filled.
- Sufficient contact time between water and disinfection agent.
- Suitable water quality with regard to turbidity and organic matter.

3.6.2 Physical Disinfection

The two principal physical disinfection methods are boiling of the water, and radiation with ultraviolet rays.

Boiling

Boiling is safe process which destroys pathogenic micro-organisms and is effective as a household treatment, but is not a feasible method for community water supplies.

Ultraviolet radiation

Light radiation is an effective disinfection method for clear water but its effectiveness is significantly reduced when the water is turbid and contains constituents such as nitrate, sulphate, and ferrous iron. In addition, this disinfection method does not produce any residual that would protect the water against any new contamination that could serve for control and monitoring purpose. Hence it is not recommended

3.6.3 Chemical Disinfection

A good chemical disinfectant should possess a number of important characteristics, including:

- Quick and effective in killing pathogenic micro-organisms present in water.
- Be readily soluble in water in concentrations required for the disinfection, and capable of providing a residual.
- Not impart taste, odour or colour to water at the concentrations used
- Be easy to detect and measure in water
- Be readily available at moderate cost.

The chemicals that have been successfully used for disinfection are: chlorine, and chlorine compounds and to a lesser extent iodine dosed in suitable form, ozone and other oxidants like potassium permanganate and hydrogen peroxide. Each one of these has its advantages and limitations.

(a) Chlorine and Chlorine Compounds

Their ability to destroy pathogens is fair and their widespread availability makes them well suited for disinfection. Their cost is moderate and they are for this reasons widely used as disinfectants throughout the world. Environmentally, the production of chlorine has negative health impacts however and certain by-products are of possible health concern such as trihalomethanes.

(b) Iodine

In spite of its attractive properties as a disinfectant, iodine 'has serious limitations. High doses (10-15 mg/l) are required to achieve satisfactory disinfection. It is not effective when the water to be disinfected is coloured or turbid. Hence it is not recommended.

(c) Potassium Permanganate

This is powerful oxidizing agent and has been found to be effective against cholera vibrio but not for other pathogens. It leaves stains in the container and hence it is not a very satisfactory disinfectant for community water supplies. It is destructive to aquaculture. Hence .it is not recommended .

(d) Ozone

Ozone is increasingly used for disinfection of drinking water supplies in industrialized countries as it is effective in eliminating compounds that give objectionable taste or colour in water. Like ultraviolet light, ozone normally leaves no measurable residual which could serve for monitoring the process. The absence of a residual also means that there is no protection against new contamination of the water after its disinfection. The high installation and operation costs and the need for: continuous power make the use of ozone relatively expensive. Hence it is not recommended.

3.6.4 Trihalomethanes (THMs)

Trihalomethanes can be formed when raw waters containing naturally occurring organic compounds such as humic and fulvic acids are chlorinated. They are also formed by the reaction of chlorine with some algal derivatives. Control is best achieved by avoiding pre-chlorination and only using post chlorination with the removal of as much of the organic precursors as possible before the chlorine is introduced into the water.

There is evidence that trihalomethanes pose a cancer risk and for this reason the WHO has set guideline values and it is recommended that total trihalomethanes (TTHM) in public water supplies be limited to 0.2 mg/l (200 µg/l), with any single trihalomethane limited to half of this.

However, the WHO Guidelines emphasise that the disinfection process must not be compromised, and that 'inadequate disinfection in order not to elevate the THM level is not acceptable'.

3.6.5 Choice of Chemical Disinfectant

As a result and despite a number of environmental health drawbacks in its production and sometimes in its use, disinfection is still overwhelmingly done using chlorination, and one of the following agents maybe used:

For large schemes:

- Gaseous chlorine

For medium sized schemes

- Sodium hypochlorite (NaOCL), especially for on-site production electrolytically using near-pure salt and electricity
- Calcium hypochlorite (HTH) with 65 – 70 % available chlorine

For small schemes:

- Chlorinate lime or bleaching powder $Ca0_2.Ca(OCl)_2$ with up to 39 % available chlorine

Great care must always be taken when using chlorine as in gaseous form it is extremely poisonous. Only qualified and authorised personnel should be involved in mixing and dosing and under no circumstances should members of the public be allowed unaccompanied into mixing and dosing facilities.

- Chlorine solutions should wherever possible be fed into the water by means of gravity or displacement dosers, and dosing pumps should preferably be confined to medium and large plants.
- A contact period of at least 30 minutes in the clear water tank and the transmission main should be allowed before the water reaches the first consumer.

- Normally the necessary disinfectant added should be in the-range of 0.5 - 2.0 mg/l (free chlorine)

If chlorine gas is used the cylinders or drums must be stored in a cool, well-ventilated place. Valves must never be left open after use as any residual gas then combines with moisture to form hydrochloric acid.

If bleaching powder is used separate mixing and dosing tanks must be provided so that solid deposits do not clog the dosing mechanism and the dose at the point of application be visible

Chlorine

As noted above and although a number of disinfectants have been tried and used in the past in a very few instances are they still used. Notwithstanding its disadvantages, it is still the most widely used water supply disinfectant, either in the form of a gas or one of its several compounds such as chlorine of lime or sodium hypochlorite. In all cases, the active disinfectant is chlorine. Because of cost; dependability efficiency .and relative ease of handling, provided this is done with care, chlorine or chlorine compounds are almost always used. As a matter 'of fact the term "chlorination" is generally used synonymously with disinfection in water works practice.

Chlorine may be applied either as a gas or as a solution, either alone or in conjunction with other chemicals. Regardless of· the form of application, the quantity or dosage is controlled by special apparatus called chlorinators or hypochlorinators.

Selection of the equipment depends on a particular installation.

Chlorine Water Reactions

Chlorine reacts with water to form hypochlorous acid (HOCL) and hydrochloric acid (HCl) according to the equation.

$$Cl_2 + H_20 \quad = \quad H^+ + Cl^-$$

This hydrolysis reaction is reversible. The hypochlorous acid disassociates into hydrogen ions (H^+) and hypochlorite ions (OCl-) according to the equation

$$HOCl \quad = \quad H^+ \; OCl^-$$

HOCl is about 100 times more powerful as disinfection than OCl^- ion. When the pH value of the chlorinated water is above 3, the hydrolysis reaction is almost complete and the chlorine exists entirely in the form of HOCl.

From a consideration of the second equation it is evident that as the pH increase more and more HOCl disassociates to form OCl ion. At pH values of 5.5 and below, it is practically 100% unionised HOCl while above a pH 9.5, it is all OCl ions. Between pH 6.0 to 8.5, there occurs a very sharp change from undissociated to completely disassociated hypochlorous acid with 96% to 100% of HOCl, with equal amounts of HOCl and OCl being present at pH 7.5. The addition of chlorine does not produce any significant change in the pH of the natural waters because of their buffering capacity. Free available chlorine may be defined as the chlorine existing in water as hypochlorous acid and hypochlorite ions.

Chlorinated lime (Bleaching Powder)

Before the advent of liquid chlorine, chlorination was· mostly accomplished by the lime and chlorine gas, with the approximate composition

CaCl2 Ca(OH)2. H20 + CaCl2 2Ca(HO)2·

Fresh, chlorinated lime has a chlorine content of 33 to 37percent. However, chlorinated lime is unstable and exposure to air, light and moisture causes the chlorine content fall rapidly. The compound should therefore be stored in dark, cool and dry place; in closed, corrosion resistant containers.

It also produces waste sludge which has to be disposed of.

High Test Hypochlorite (HTH)

These are not only twice as strong as chlorinated lime (60 to 70 percent available chlorine content) but retain their original strength for more than a year under normal storage conditions. HTH may be obtained in packages of 2-3 kg, and in cans of up to 50 kg, and are also available in granular or tablet form. It should therefore be used instead of chlorinated lime.

Sodium Hypochlorite

Sodium Hypochlorite is a solution, (NaOCl) produced electrolytically and usually contains 10 to 15 percent available chlorine in the commercial form. House hold bleach solutions of sodium hypochlorite usually contain only 3 to 5 percent available chlorine. It has a very short shelf life and should be produced on site as required.

Where a waterworks requires electricity and a nearly pure salt is available locally it is a preferred method.

3.6.6 Chlorination Practice

Chlorination practises may be grouped under two categories depending upon the desired level of residual chlorine and the point of application. When it is required to provide a residual and the time of contact is limited, it is common practice to provide for free available residual chlorine. If combined available residual chlorination is used, the chlorine is applied to water to produce, with natural or added ammonia, a combined residual, effect. pre-chlorination is the application of chlorine prior to any other treatment. This has been used for the purpose of controlling algae, taste and odour but should be avoided whenever possible because of trihalomethanes (THMs). Post chlorination refers to the application of chlorine after other treatment processes particularly after filtration and should be preferred.

Chlorine Demand

This is the difference between the amount of chlorine added to water and the amount of free or combined available chlorine remaining at the end of a specified contact period.

Residual Chlorine

Several methods are available to measure residual chlorine in water. Consult standard water treatment textbooks to select the most appropriate method for a particular situation.

3.6.7 Gaseous Chlorine Storage

Gaseous chlorine is heavier than air and lethal. Should a leak be detected, people nearby should be alerted and told to move upwind. Usually cylinders are used for chlorine storage. Their choice in terms of capacity depends upon' the chlorine requirement. Cylinders up to 67 kg capacity should be stored vertically so that a leaking container if found can be removed with the least possible handling of others. It is preferable to provide space for separate storage of full and empty cylinders. Care should be taken to prevent them from falling over or from being hit by moving objects. Dropping of containers is very dangerous. Containers should be stored in a cool ventilated area protected against external sources of heat like steam, electric heaters and away from inflammable goods

3.6.8 Chlorinators

A chlorinator is a device designed for feeding chlorine in to a water supply. Its functions are:

(a) to regulate the flow of gas from the chlorine container at the desired rate of flow.
(b) to indicate the flow rate of gas feeding
(c) to provide means or properly mixing the gas either with an auxiliary supply of water' or with the main body of the liquid to be disinfected.

The usual fittings and parts of a chlorination system are:

(i) Chlorine cylinder or drum supplied with its own main valve and filled with liquid and gaseous chlorine, under pressure.
(ii) Fusible plug, a safety device provided on all cylinders and containers designed to meet temperatures often between 70^0C to 75^0C
(iii) Reducing valve / vacuum regulator to bring the pressure of the gas down to between 70 to 30 kPa so that the pressure is below atmospheric (approx 100 kPa). This should be located in the storage room so that any leakage in the dosing room is into the feed pipes rather than into the room itself.
(iv) Pressure gauges one to indicate the cylinder pressure and the other the delivery pressure
(v) A measurement device consisting of an orifice to measure upstream or downstream pressure of gas with manometer containing liquid of carbon tetrachloride
(vi) A “desiccator valve” or non-return valve containing concentrated sulphuric acid or calcium chloride through which the chlorine must pass to free it from moisture so that any corrosive action of the moist chlorine on the fitting is prevented.

Typical arrangements for gaseous chlorination is shown below:

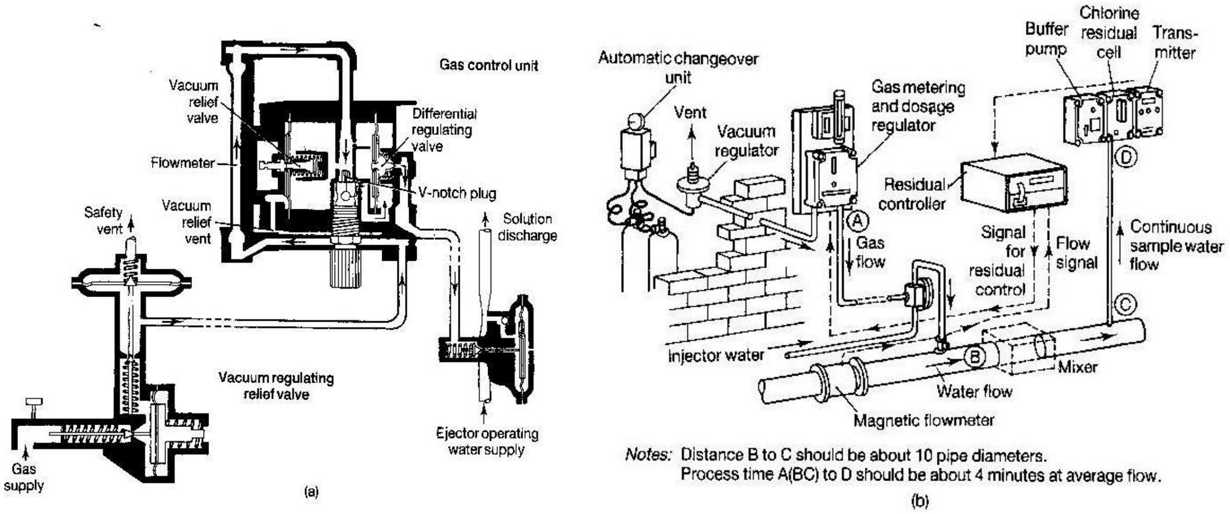


FIGURE 3.5: TYPICAL VACUUM-OPERATED GAS METERING SYSTEM FOR AUTOMATIC DOSAGE INJECTION

Access to storage and dosing rooms should separate and be from the open air and doors should always open outwards. External windows should be avoided where possible with artificial illumination being provided throughout. Both storage and dosing rooms should be provided with low level outlet venting fans that either come on automatically when the door is opened or are activated from outside the room so that any leakage is purged to the outside before entering such rooms. High level fresh air inlets should be provided, especially to the storage room.

3.6.9 Chlorine Piping Systems

Moist chlorine unlike dry gas or liquid chlorine is highly corrosive. Pipelines, valves and other fittings through which dry chlorine passes should be tightly closed when not in use to prevent ingress of moisture from the air.

Dry chlorine gas or liquid chlorine under pressure should be conveyed through extra heavy duty wrought iron or steel pipe or flexible annealed copper tubing. The discharge line from the chlorine container should be flexible and sloping upwards, especially when chlorine is discharging in a liquid state.

A long pipeline should be avoided. Hard rubber, silver or platinum tubing is necessary for conveyance of moist chlorine gas or aqueous chlorine solutions at low pressure.

To prevent condensation of gas, the piping system and control equipment should be at the same or a higher temperature than that of the chlorine container. Chlorine gas lines are preferably located along walls or overhead rather than along the floor, to take advantage of the warmer temperatures. For liquid chlorine piping systems, any conditions which might contribute to vaporization should be avoided

3.7 CONTROL AND REMOVAL OF ALGAE

3.7.1 General

These are small weed-like plants which are present in water exposed to the sun’s rays and cause turbidity and some colours, brown or green. The presence of algae and the death and decay of this, impart characteristic odours and taste to the water. Algae interfere with the process of sedimentation and also choke filters and as a result reduce the filter runs.

Algae can be killed by treating the water with suitable chemicals. However, prevention is better than cure, since adopting algicidal measures has a number of disadvantages, viz:

(i) the dose of chemical required is greater. than that needed if the treatment is adopted at the incipient stages of growth;

(ii) the dead algae decay and produce acute odour problems.

(iii) the dead algae provide pabulum for a second crop which are generally more prolific than the first and also more resistant to the action of algicides. It is, therefore, preferable to take all possible measures to prevent the growth of algae and to reserve the use of algicides as a final treatment.

Where removal is found necessary, two methods have been found practicable namely micro-straining or flotation.

Factors Causing Algal Growth

The following factors influence algal growth in water supplies:-

(a) Nutrients in Water

Water in contact with decaying vegetation as well as water polluted by sewage, favour algal growth because of the large amounts of organic matter they contain. Nutrients like nitrogen, phosphorus and potash compounds are generally washed into watercourse from fertilizer farming or from industrial waste.

These are particularly favourable to algal growth and algicidal treatment has limited effect in such circumstances because these conditions are repetitive and favour the growth of succeeding crops of algae.

(b) Sunlight.

Algae require sunlight for their life processes and hence growths are profuse in seasons of intense sunlight. Clear waters favour the growth of algae because they permit the penetration of light to greater depths.

(c) Characteristics of Reservoirs

Shallow reservoirs offer more favourable condition than do deep reservoirs because the decayed material closer to the surface may stimulate algal growth. Irregular margins and shallow areas encourage the growth of aquatic weeds which offer anchorage for the algae.

(d) Temperature Effects

The blue-green and the green algae increase their presence when the water temperature reaches 15° - 20°C.

3.7.2 Remedial Measures

3.7.2.1 Preventative Measures

Preventative measures should therefore be based on control of those factors such as reduction of food supply, change of the environment or exclusion of sunlight though they are not always practicable. Clear water reservoirs, service reservoir and wells may be covered to exclude sunlight. Of course, such preventive measures are inapplicable in the case of large raw water storage reservoirs.

3.7.2.2 Control Measures

A knowledge of both the types and the quantities of algae are necessary to decide upon control measures. The type of algae are recorded by a differential count. The density is expressed in terms of acral units which denote the number of organisms present in a square of side 20 microns.

Generally, the practice is to apply algicides when the total count reaches or exceeds 300 acral units. This should be taken as a general indication and each plant manager has to decide on the best time for treatment on the basis of local knowledge and conditions. However, algicidal treatment is indicated as soon as synura algae, a type which causes severe smell trouble, is encountered irrespective of the total count.

Copper Sulphate as an Algicide

The most commonly used algaecide is copper sulphate.

Its action is due to the copper iron which acts as a direct protoplasmic poison. Though doses up to 12 mg/l are not dangerous to water consumers dosage levels should be limited to prevent fish from being killed. Most fish can tolerate doses up to about 0.5 mg/l and practically none will be affected at 0.3 mg/l. The amount of copper sulphate required should be computed on the basis of only a 3 m depth of water and should be applied at intervals of two to four weeks during warmer periods of the year except when information is available to serve as a guide for more specific treatment.

Effective control throughout the year is facilitated by the continuous application of about 0.18 mg/l, to the water entering the reservoir. In that way micro-organisms are controlled before heavy growths occur to minimise the need for periodic treatment.

Continuous application of copper sulphate, however, reduces the available food supply for fish and should be restricted to reservoirs used exclusively as source of public water supply. Dosing most also be carefully controlled because over-dosing is lethal to fish.

As a guide the lethal dose of copper sulphate $CuS0_4.5H_{2O}$ to fish life is given in the following table:

TABLE 3.6: LETHAL DOSE LEVEL OF COPPER SULPHATE TO FISH

FISH	mg/l
Trent	0.14
Carp	0.30
Suckers	0.30
Catfish	0.40
Picerel	0.40
Goldfish	0.50
Perch	.75
Sunfish	1. 20
Black Bass	2.10

Periodic application of copper sulphate may be accomplished by any one of the following methods:

- by dragging bags of crystals behind a boat
- by dragging special screened boxes containing the copper sulphate attached to the side of boat.
- by the use of motor driven blowers discharging the copper sulphate in powder forms or
- by the use of convectional tree spraying equipment to distribute a solution of copper sulphate over the water's surface.

Chlorine is also an effective algicide, but as noted above, pre-chlorination brings with it the production of trihalomethanes so whenever used requires very careful monitoring and control.

Whereas copper sulphate is more commonly applied to water in reservoirs, chlorine is generally added. to the water as it passes the control point. Chlorine has specific toxic effect and causes death and disintegration of the algae.

The essential oils present in the algae are thus liberated and may combine with chlorine to form new or intensified odours and tastes. Experience has shown that concentration or free residual chlorine between 0.2 and 1.0 mg/1 are effective in destroying most micro--organisms. Chlorine may be applied either as a slurry of bleaching powder or as a strong solution of chlorine from a chlorinator. Algal growths in raw water conduits can be got rid of by heavy doses of chlorine.

Each plant manager where algae poses a problem should conduct experiments and decide on the type and dosage of the algicide on the basis of local conditions and possible trihalomethane production with samples submitted to a suitable laboratory for the necessary tests.

To a certain extent the method will depend upon the facilities available for dosing the water with chemicals, the general arrangement of the system as well as on the costs. There are however, certain special conditions where the use of copper sulphate is not possible and chlorination may appear to be the only solution. For example, when the point of application is too near the point of entry into a pipe, copper sulphate cannot be used as it will be thrown out of solution almost immediately. In case, where the supply cannot be shut off for periods, sufficient to cause a reduction of copper content to permissible limits, chlorination has to be preferred.

However, in all such cases, specialist advice should first be sought. In difficult cases where pre-chlorination is ruled out due to trihalomethanes it may be necessary to consider installing micro-strainers or to remove by flotation if settling is not very successful.

3.7.2.3 Removal of Algae

Settling

Settling after coagulation and flocculation can, in certain circumstances be a moderately successful way of reducing algal load. It does however require pilot trials to ensure that a reasonable success rate is achieved.

Microstraining

Microstrainers are revolving drums mounted horizontally in open tanks with a straining medium which is usually of stainless steel wire fabric of a very fine mesh, fitted to the periphery of the drum. The drum is submerged to about 75% of its diameter (66% of its area) and rotation speed is in the range 0.5 – 5 rpm giving a peripheral drum speed of the order of 3 – 50 m/min. Water to be filtered enters the drum axially under gravity and flows out radially through the fabric, depositing particulate matter. Cleaning is accomplished by a row of water jets mounted above and along the full length of the drum operated at about 2.5 bar pressure, with a collector hopper within the drum immediately below.

The intercepted algae and other particulate matter that has been intercepted by the fabric rotates to the top of the drum where it is backwashed into the hopper supported from the drum axle which is itself a pipe that leads to a point outside for discharge. Jetted backwash water usually amounts to 1 – 1.5 % of the total quantity of water strained.

Total headloss including inlets and outlets is in the range of 150 – 200 mm. Single units have capacities of between 10 – 4,000 m^3/h at which flow rate drum size is of the order of 3.2 m diameter and 5 m long.

Usually used for lake and reservoir waters only and located before settling and filtration, the woven fabric usually consists of 0.05 mm diameter wire with apertures of 23 to

25 µm. It is however important when used for algal removal which is normally in the range of 60 -80 % of the total, to either carry out pilot trials or at least laboratory tests to ensure that micro straining will be successful as zooplankton occurs in a wide variation of sizes and selecting the correct aperture opening is critical.

Flotation

The dissolved-air flotation process is particularly well suited for removal of blue-green algae because of the natural tendency for such algae to rise to the surface. It is however a sophisticated clarification process that follows flocculation and is unsuited to smaller, remote plants.

It operates on the principle of the transfer of floc and algae to the surface of the water through attachment to air bubbles. This then accumulates on the surface, known as the "float" and is then skimmed off as sludge.

The table below shows some typical results from the UK where treating algal laden waters by sedimentation, or by dissolved air flotation when compared to typical results for micro-straining.

TABLE 3.7: COMPARISON OF ALGAL CELLS IN RAW WATER AND AFTER MICRO-STRAINING, OR COAGULATION AND SEDIMENTATION OR FLOTATION

Algae	Raw Water	Sedimentation	Flotation	Micro-straining
Aphanizomenon	179,000	23,000	2,800	
Microcysts	102,000	24,000	2,000	
Stephanodiscus	53,000	21,900	9,100	
Chlorella	23,000	3,600	2,200	
Total	**357,000**	**71,600**	**16,100**	**143,00 – 71,500**
Method of controlling Phytoplankton, W. Parr, WRc, UK, 1991				*at 60 – 80%*

3.8 TASTE AND ODOUR CONTROL

3.8.1 Taste and Odour

Taste is the combination of sensations of four basic tastes, namely sweet, sour, bitter and salt together with that of smell. To some extent it is a subjective or qualitative matter and is best examined by smelling a slightly warm sample.

Taste and odour problems in water are common in surface water sources, as these are more subject to contamination by natural and man made wastes than is groundwater. Taste and odour are caused by dissolved gases such as hydrogen sulphide, mercaptans (a group of sulphur-containing organic chemical substances that smell like rotting cabbage), methane organic matter derived from certain dead or living microscopic organism (such as blue and green algae), decomposing organic matter, industrial liquid wastes containing phenols, cresols, ammonia, agricultural chemicals, high residual chlorines and chlorophenols. It is possible that some of these dissolved gases may also be found in well water

Biological organism are also one of the more common causes of taste and odour in water, *Diatomaceae* with *asterionella* and *synedra actinomycetes* and free swimming nematodes are the principal offenders causing earthy or musty odour. Odour is classified as aromatic, earthy, swampy *i* septic or chemical. Odour tests indicate that only a few mg/l of these materials are sufficient to produce detectable odour.

The most likely causes of unacceptable tastes and odours are:

- Decaying vegetation such as algae which mostly cause offensive odours as they die off, although some living algae like *Cyanophyceae* or blue-green algae cause taste and odour problems. Such odours are variously described as grassy, fishy, or musty.
- Iron and sulphur bacteria produce deposits which on decomposition release an offensive smell especially of bad eggs (hydrogen sulphide).
- Moulds and actinomycetes which can give rise to earthy, musty, or mouldy tastes sometimes wrongly attributed to algae. Particularly noticeable in warm stagnant waters such as in long lengths of pipeline used only occasionally and then noticeable in the first water drawn off.
- Iron above a concentration of about 0.3 mg/l will tend to impart a bitter taste to water.
- Chlorides and sulphates, if excessive, impart a brackish taste to water and thresholds for sodium and calcium chlorides and for sodium, calcium and magnesium sulphates are specified to guard against this.
- Many industrial wastes are a prolific source of taste and odour problems, and those produced by phenols are the most common especially in the presence of free chlorine as chlorophenal is formed giving a pronounced medicinal taste.
- Chlorine, either in excess or in reaction to a number of organic substances, giving rise to chlorinous tastes. Reaction of chlorine with ammonia is a particular problem in this regard.

Because of its subjectivity it is usual for guidelines to stipulate that the taste and odour of drinking water should be acceptable to consumers.

3.8.2 Control of Taste and Odour

For control of taste and odour, some preventive and corrective measures are needed. When control of micro-organism is needed where the source of water supply is a river, special treatment has to be given to the water in the treatment plant for the removal of such odours by aeration which only removes hydrogen sulphide or by oxidation by chemicals or absorption by activated carbon in powder or granular form.

3.8.2.1 Preventive Measures

Where a raw water imparts a taste or odour, prevention is always the better solution although often difficult to achieve in practice.

Reservoir management in order to control algal growth is fundamental for an effective tackling of many taste and odour problems. Such measures were discussed in the section on control of algae.

3.8.2.2 Corrective Measures

Before treatment can be started, the problem must be identified control procedures must be established which will determine the intensity and character, as well as changes in taste and odour in the influent and effluent.

For removing dissolved gases like hydrogen sulphide (H_2S) and volatile matter, aeration can be practiced at the start of the treatment process, and can be very effective for removing hydrogen sulphide from water with a $pH < 6$.

Free available residual chlorination at any pre-chlorination stage but preferably post-chlorination because of the risk of producing THM (see section 3.5.4) can usually bring about complete elimination of taste and odour. Inadequate chlorination will only intensify the odour of the water containing phenolic compounds, tanin or lignin, imparting a medicinal taste. Using ammonia with chlorine in combined residual chlorination can partly mask or delay chlorophenol tastes in waters.

The most frequently used procedure is the threshold odour determination outlined in the section on standard methods. How often this determination should be made will depend upon the variations in raw water quality. Other tests may be used in addition to or in place of the threshold odour determination test. Odour may be detected wherever there are large volumes of water in agitation for example at weirs, aeration units, or re-carbonation basins.

Copper sulphate and chlorine are used to control the growth of algae, failing which micro-straining or flotation techniques will be necessary. The two chemicals were briefly discussed in the chapter of corrective measures. An alternative is to use activated carbon.

Activated Carbon

Activated carbon is made from hydrocarbon or carbohydrate sources, the principal requirement being that the carbon residue left after destructive distillation has a porous structure. Odour producing substances which cannot be removed by oxidation are physically absorbed on the surface.

This treatment is either applied during filtration or prior to filtration. The contact time varies from 10 to 60 minutes. Activated carbon performs well at lower pH values. A bed of carbon or suspension kept in circulation can be used. The active surface must be preserved from coating by other chemicals. Application of carbon can be done before sedimentation if taste and odour is severe and frequent, and in certain cases after sedimentation.

The approximate dosage for routine, continuous application as a suspension is 2 - 8 mg/l. For intermittent application it is 5 - 20 mg/l; and for emergency treatment 20 - 100 mg/l. Carbon beds are generally 1.5m to 3m deep with the size between 0.2 - 0.4mm and with loadings of about 80 lpm/m^3 of bed. Filtration rate ranges from 120 to 250 lpm/m^2 with expected efficiencies of about 90%.

Such things as pH value, temperature, quantity and type of organic matter in the influent water and detention time have a marked effect on the efficiency of removal of odorous materials.

Chlorine dioxide

Chlorine dioxide, ($Cl0_2$) is a heavy, reddish yellow gas with a pungent disagreeable odour. It is a very active oxidising agent. While used primarily for taste and odour control, it has been found that satisfactory bacterial reductions can be obtained also. The method is at least as effective as chlorine on the normal range of taste and odour and more effective at the higher values.

Chlorine dioxide should be produced at the treatment plant at the time when it is to be used. It results from the reaction of sodium chlorite and strong chlorine solution at a pH of less than 4. As chlorine dioxide is generated as needed, the reaction must be essentially completed before· the final product is added to the water to be treated. Therefore, the solution of sodium chlorite and chlorine are passed into a reaction chamber which provides a sufficiently long detention period and agitation.

When the chlorine, solution is in the form of hydrochlorite, it is necessary to add an acid to the hypochlorite solution before it comes into contact with the sodium chlorite. This is essential in order to depress the pH value to 4 or less, as indicated above. The chemical reaction is:

$$2\ NaCl0_2 + Cl_2 \quad = \quad 2\ Cl0_2 + 2\ NaCl$$

Though the theoretical ratio of chlorinate sodium chlorite is 1:2.6, values between 1:2 and 1:1 are employed in practice.

Ozone at dosage of 1.0 mg/l has also produced good results.

Potassium permanganates

Potassium permanganate in solution may be used to control taste and odour in low concentration. In general, dosage of one to two mg/l has been found effective when applied with the pre-settling coagulant. As the addition of manganese in the finished water is often considerable, periodic testing to determine proper dosage is very important.

Aeration

Aeration is a relatively simple treatment for dealing with tastes and odour control but its use and effectiveness rate are rather limited. However, aeration usually reduces the load in other taste and odour control processes. Aeration methods are described in the section on removal of Iron and Manganese.

3.8.3 Distribution System Complaints

A treated water free of taste and odour does not always remain that way. Some taste and odour problem originate in the distribution system, and samples should be collected from taps of complaining consumers. Tests such as temperature, type of taste and odour, threshold odour number and pH, will establish whether or not the water quality has deteriorated.

Results of records kept over the long period can point to deficiencies in the distribution system that need correction and help in identifying sources of unusual taste and odour problems that may occur in the future

3.9 COLOUR

3.9.1 Introduction

The colour of water is usually expressed in Hazen units which are the same as true colour units (TCU) or mg/l on the platinum-cobalt (Pt-Co) scale. True colour can only be determined after filtration, usually through a 0.45 micron filter, as water will often appear coloured due to material in suspension. The Hazen scale is based on a standard solution which is made up of mixture of potassium chloro-platinic and cobalt chloride such that each ml of standard solution, when diluted to 100 ml with water, represent one degree of colour.

The level at which colour becomes unacceptable depends largely on consumer perception, however the WHO gives a guide value of 15 TCU, above which the colour is likely to be noticeable in a glass of water. However, the level has to be acceptable to consumers and show no abnormal changes.

3.9.2 Causes of Colour

Colour in water is usually due to colloidal organic matter which is generally harmless in itself. Water containing oxidised iron and manganese impart characteristic reddish or black· colour. Heavy growth of algae may also impart colour to the water. Discharge of industrial waste or heavy sewage pollution may also bring about colour. Colour occurs in greatest degree in water from upland gathering grounds which carry beds of peat.

3.9.3 Colour Removal

The appropriate treatment for the removal of colour from water has to be determined for each individual case after a consideration of the causative factors. The following methods can be examined for this purpose.

(1) Colour due to Iron and Manganese

Iron and Manganese impart reddish or brown colour. For removal of colour due to iron and manganese, specific treatment is needed. The methods for the specific treatment are outlined in the section on Aeration.

(2) Colour due to Algae

A water which is coloured due to algae has to be treated to eliminate the algae. The methods which can check the growth of algae are outlined in the section on control of algae. Microstrainers can also be employed for removal of a large proportion of algal growth as described in section 3.6.2.3.

(3) Colour due to Industrial Waste

Colour due to industrial waste can often be removed by the use of bleaching powder or chlorine or activated carbon.

(4) Colour due to Colloidal Organic Matter

Coagulation of low pH value water by ferric. salts can be used. for removing colour due to colloidal organic matter. Ferric salts are generally superior to alum. After removal of the colour colloids, the pH value of the water has to be corrected by treatment with lime.

(5) Oxidation of Colour

In cases where other methods have not been effective in removing colour, it was often the practice to apply heavy doses of chlorine, or in case of failure of this too, to apply chlorine dioxide. However due to the formation of THMs this can no longer be recommended, and alternatives such as the use of ozone for oxidation in conjunction with chemical coagulation, sedimentation and filtration; activated carbon; or as a last resort membrane filtration may be necessary.

(6) Treatment with Activated Carbon

In most cases of persistent colour, activated carbon will be effective to reduce the colour. This chemical is discussed in the section on Taste and Odour.

3.10 SOFTENING

3.10.1 General

The hardness or soap consuming power of water is due to presence of bicarbonates, carbonates, sulphates, chlorides, and nitrates of calcium and magnesium. The dissolved compounds have the following negative effects:

(a) Soap destroying or increased soap consumption in laundries,
(b) Deposition of scale in boilers and engine jackets,
(c) Corrosion and incrustation of pipelines, joints valves and plumbing fixtures; and
(d) Serious difficulties and detrimental affects in the manufacturing processes, e.g. textile finishing, dyeing, canning, paper making, ice manufacturing, tanning etc.

However, softening of public water supplies due to hardness is not recommended due to the apparent statistical relationship between the softness of water and the incidence of cardiovascular disease, even though the reason for this remains unclear.

Hence and where a relatively hard water is available and supplied it should be left to the individual industry to soften as necessary for their specific purpose.

3.10.2 Hardness and Its Measurement

Hardness can be described as temporary or permanent as shown in the following table:

TABLE 3.8: COMPOUNDS PRODUCING TEMPORARY AND PERMANENT HARDNESS

CAUSING TEMPORARY HARDNESS (CARBONATE HARDNESS)	CAUSING PERMANENT HARDNESS (NON-CARBONATE HARDNESS)
Calcium bicarbonate [$Ca(HCO_3)_2$] Magnesium bicarbonate [$Mg(HCO_3)_2$]	Calcium sulphate [$CaSO_4$} Magnesium sulphate [$MgSO_4$} Calcium chloride [$CaCl_2$] Magnesium chloride [$MgCl_2$]

Hardness is expressed in terms of mg/l by weight in terms of calcium carbonate. Water having hardness not exceeding 70 mg/l is termed ‘soft’ and above that 'hard'. In public water supplies, it used to be customary to reduce carbonate hardness to 35-40 mg/l and total hardness to between 50 and 100 mg/l. However as indicated above this is no longer recommended unless hardness exceeds about 130 mg/l, but should still be practised for strictly industrial supplies of hard water.

The scale of hardness is shown below:

TABLE 3.9: SCALE OF HARDNESS

HARDNESS in mg/l	SCALE DESCRIPTION
15	Extremely soft
30	Very soft
45	Soft
90	Moderately soft
110	Moderately soft
130	Hard
170	Very hard
230	Excessively hard
250	Too hard for use

3.10.3 Methods of Softening

The methods ordinarily used for removal of calcium and/or magnesium and hence achieve softening are:

- chemical precipitation, and
- ion exchange

3.10.3.1 Chemical Precipitation

Softening by chemical precipitation is accomplished by adding lime or lime and soda ash. Softening with these chemical is used particularly for water with high initial hardness greater than 500 mg/l and suitable for water containing turbidity, colour, and iron salts because these have a tendency to inactivate the ion exchange bed, by a coating on the granules. Lime-soda softening cannot, however, reduce the hardness to values less than 40 mg/l and this should not be attempted.

Ion-exchange softening can produce a zero-hardness water but such water should always be blended with water to leave a residual hardness of not less than 70 mg/l because apart from the risk of cardiovascular problems, very soft drinking water may be corrosive and result in feelings of sickness.

(a) Lime Treatment

Addition of lime to hard water only removes the carbonate hardness. Insoluble carbonates of calcium and magnesium are precipitated out and removed in sedimentation tanks. (An overdose of lime is usually used and the excess lime is neutralized by re-carbonation before filtration. This treatment is good when the bulk of the hardness is due to calcium, and magnesium is insignificant. When the water contains more than 40 mg/l of magnesium warranting its removal, excess lime treatment has to be resorted to.

(b) Lime –soda Treatment

In lime treatment only the carbonate hardness is removed but by addition of soda, the non-carbonate hardness is also removed, thus the removal of both carbonate as well as non-carbonate hardness is possible in the lime-soda process. For calculating the theoretical amount of lime and soda required for softening, an analysis of the following constituents in the water is necessary.

(i) free carbon dioxide dissolved in the water bicarbonate (total alkalinity)
(ii) total hardness
(iii) total magnesium
(iv) Chemical requirement (mg/lm) are computed by the sum of the following factors

Lime requirements as Ca (100% purity)

56/44	of concentration of $C0_2$ (mg/l as CO_2)
56/24	of concentration of Mg (mg/l as Mg)
56/100	of concentration of alkalinity (mg/l as $CaC0_3$)

Additional lime required for raising the pH to the range of 10 to 10.5 for precipitation of $Mg(OH)_2$ is about 30 - 50 mg/l as CaO.

Soda requirements as Na_2CO_3

106/100 of difference between total hardness and bicarbonate alkalinity both expressed as $CaCO_3$.

For neutralizing excess lime at 30 mg/l, additional soda required is (30/56) x 106 mg/l as Na_2CO_3.

Plant conditions like temperature; time of detention and agitation, influence the completeness of reactions and dosage of chemicals may have to be increased to provide for the inadequacies.

Alternatively caustic soda can be used instead of lime. Liquid caustic soda should be used since it can be handled and fed easily. The amount calcium carbonate sludge formed in this case is theoretically half that formed by use of lime. However, using soda is costlier than soda ash which is more expensive than lime.

(c) Excess lime Treatment

When the water contains more than 40 mg/l of magnesium, excess lime treatment has to be resorted to since magnesium has to be removed as magnesium hydroxide whose solubility decreases with. increasing pH values. The water treated thus is highly caustic and has to be neutralised after precipitation either by re-carbonation or by split treatment.

In split treatment, the total flow is divided into two parts, one part being treated with excess lime and the settled effluent then mixed with unsoftened water. The final residual hardness in the water will depend on the percentage flow by-passed and the levels of or hardness in both the portions (treated and by passed).

(d) Hot lime-soda Treatment

This process is used for boiler feed water treatment. It is similar to the cold process already discussed except that the raw water is heated to about 95° - 100°C before being taken' to the reaction tank. Reactions take place rapidly due to decreased viscosity hastening the settling of the precipitates. A greater degree of softening is accomplished than that in the conventional cold processes.

A lime or soda ash mixing and dosing plant is illustrated here:

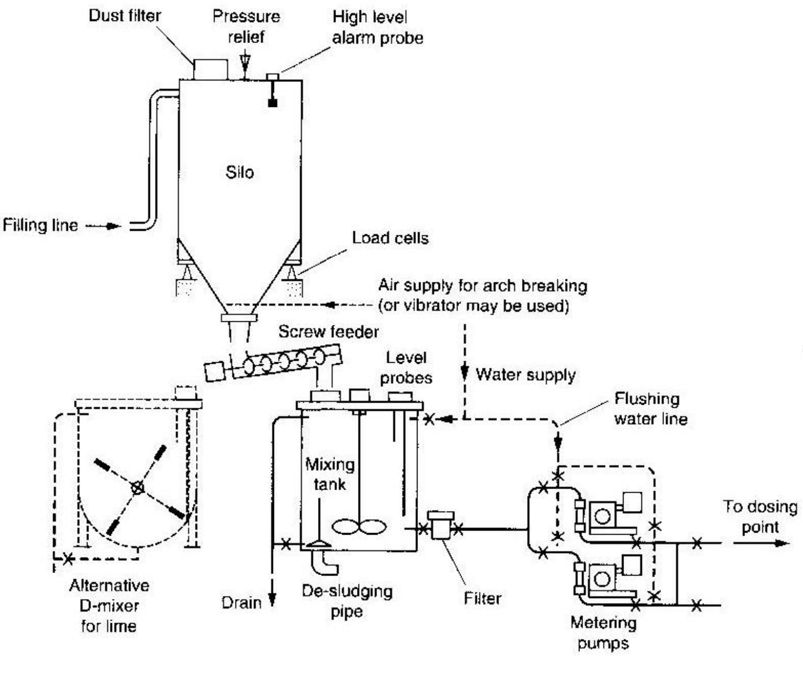


Figure 3.6: Chemical Plant for Lime or Soda Ash Mixing and Dosing

3.10.3.2 Ion-exchange

The ion-exchange process is the reversible inter-change of ions between a solid ion exchange medium and a solution and is used extensively in industrial water/softening. The hardness producing ions preferentially replace the cations in the exchangers and hence this process is also known as base or cation exchange softening.

The ion exchange works on the hydrogen or sodium cycle, the hydrogen ions being released into the water in the former case and the sodium ions in the latter. There is only a temporary change in the structure of the exchange material. The exchange material can be re-generated using an acid and sodium chloride respectively.

In general, ion exchange materials used in softening, also called zeolites, are hydrated silicates of sodium and aluminium. There are inorganic and organic zeolites.

(a) Inorganic Zeolites

Natural inorganic zeolite is available as 'green sand' while synthetic or gel type is obtained by the reaction of either sodium aluminate or aluminium and is graded to suitable sizes by the reaction of either sodium aluminate or aluminium sulphate with sodium silicate which after drying is graded to suitable sizes by screening. For regeneration, 3.5 to 7 kg of salt is required for every kilogram of hardness removed.

(b) Organic Zeolites

These consist of sulphonated carbonaceous materials and sulphonated stayrone type resins which have excellent cation exchange properties, requiring for regeneration, 2-4 kg salt for every kilogram of hardness removed.

These are resistant to attack by acid solutions and hence can be regenerated by acid also. They can be used for waters with a wide pH range, whilst the loss due to attrition is negligible compared to the synthetic inorganic zeolites.

(c) Raw water characteristics

Raw water to be treated by ion-exchange should be relatively free from turbidity otherwise the exchange material gets a coating which affects the exchange capacity of the bed. The desirability of using filters prior to zeolite beds or resorting to more frequent regeneration would depend upon the level of turbidity. Metal ions like iron and manganese, if present, are likely to be oxidised and can coat zeolites, thus deteriorating the exchange capacity steadily since the regeneration can not remove the coats.

Oxidising chemicals like chlorine and carbon dioxide as well as low pH in the water will have a tendency to attack the exchange materials particularly the inorganic type, the effect being more pronounced on the synthetic inorganic zeolites. Waters low in silica inorganic zeolites, are to be avoided in boiler feed water.

The organic zeolites, operating on a brine regeneration cycle do not add any silica to the water and consequently are ideally suited for boiler feed water.

(d) Recommendation

The ion exchange process is both costly and delicate and should not be adopted without advice from a competent authority. In case the need arises for using this type of process for softening then the details of the process design should be obtained from a standard textbook or plant manufacturer.

3.11 RADON

3.11.1 Introduction

No modern day design manual would be complete without at least a brief review of radon.

Radon is a radioactive gas, and its origin or ultimate source is uranium. All rocks contain some uranium, although most contain just a small amount - between 1 and 3 mg/l of uranium. In general, the uranium content of a soil will be about the same as the uranium content of the rock from which the soil was derived. Some types of rocks have higher than average uranium contents. These include light-coloured volcanic rocks, granites, dark shales, sedimentary rocks that contain phosphate, and metamorphic rocks derived from these rocks. These rocks and their soils may contain as much as 100 mg/l uranium. Just as uranium is present in all rocks and soils, so are radon and radium because they are products formed by the radioactive decay of uranium.

A radium atom decays to radon by releasing an alpha particle, containing two neutrons and two protons, from its nucleus. Because radon is a gas, it has much greater mobility than uranium and radium, which are fixed in the solid matter in rocks and soils. Radon can more easily leave the rocks and soils by escaping into fractures and openings in rocks and into the pore spaces between grains of soil.

Radon can move through cracks in rocks and through pore spaces in soils. Radon moves more rapidly through permeable soils, such as coarse sand and gravel, than through impermeable soils, such as clays. Fractures in any soil or rock allow radon to move more quickly.

Radon in water moves slower than radon in air. The distance that radon moves before most of it decays is less than 3 cm in water-saturated rocks or soils, but it can be more than 2 m, and sometimes tens of metres, through dry rocks or soils. Because water also tends to flow much more slowly through soil pores and rock fractures than does air, radon travels shorter distances in wet soils than in dry soils before it decays.

Radon is soluble in water and thus may accumulate in water as it passes over rocks and through soil. As radon is a gas it is easily lost from water when it is agitated for example when river or lake water flows over rocks or is moved by wind. In this way surface water (lake and river water), normally show very low levels of radon unless the water has flowed slowly over areas releasing radon gas shortly before abstraction.

In water supplies that aerate the water, this allows radon to escape, but all surface water treatment plants delay the use of water until most of the remaining radon has decayed.

Ground water (from wells and bore holes) on the other hand, does not tend to be agitated before reaching the domestic tap, so if there is radon present it tends to remain in the water. When the domestic tap is turned on some of the radon will be released into the air due to the agitation of the water as it is dispensed from the tap, but some will remain dissolved in the water and may be consumed. The areas most likely to have problems with radon in ground water are areas that have high levels of uranium in the underlying rocks, for example, granites.

However in countries where studies have been undertaken, no direct relationship between the amount of uranium in the underlying rocks and soil and the concentration of radon in the water supply has been established, nor has any direct relationship been established between radon in indoor air and radon in water. Therefore it is not possible to predict if any areas of Tanzania are more at risk than others.

3.11.2 Health implications of radon in drinking water

Ingestion of dissolved radon will result in a radiation dose to the lining of the stomach. However, a definitive link between consumption of radon-bearing water and cancer has not been established. Inhalation of radon gas that has been released from tap water will contribute to the radon content of indoor air and, if inhaled, will result in a radiation dose to the lung. Long-term exposure to high concentrations of radon in indoor air increases the risk of lung cancer.

Globally, the radiation dose from radon in drinking water is low compared with that from the inhalation of radon gas. However it is prudent to regard both of these sources of radon with similar concern and reasonable measures should be taken to minimise exposure to radon in drinking water. For this reason the Ministry of Water advises that domestic water supplies of groundwater or partial groundwater origin should be tested for radon, and if the recommended standards are exceeded the supply should be aerated before supply.

3.11.3 Recommended Standards

Guidance on levels of radon and its decay products in drinking water is covered in EC Recommendation (2001/928/Euratom) on the protection of the public against exposure to radon in drinking water supplies. Ministry of Water recommendation proposes 1000 becquerels per litre (Bq/l) as an action level for both public and private water supplies and can be summarised as follows:

- For public water supplies, if the radon concentration exceeds 1000 Bq/l, remediation is considered justified.
- Where the radon level is above 500 Bq/l but below 1000 Bq/l, the water authority must consider whether this poses a risk to human health.
- Where the radon level is above 100 Bq/l but below 500 Bq/l, the water authority is advised to consider whether this poses a risk to human health.

If it is concluded that such a risk exists, then remedial action such as aeration should be undertaken.

3.12 IRON AND MANGANESE AND ITS REMOVAL

3.12.1 Sources and Nature

The presence of iron and manganese in water is objectionable owing to the production of colour, turbidity, deposit and taste. Water containing iron and manganese is undesirable for culinary use, causing brown coloured deposition during washing and cooking. Even small traces of iron and manganese in water lead to the accumulation of appreciable deposits in distribution mains and reservoirs.

Under such conditions, growth of iron bacteria often develop in the distribution system causing blockage of mains, meters, pipes, etc. and complaints of coloured, turbid and unpalatable water become more serious. Sulphide of iron and hydrogen sulphide may also be produced.

Iron occurs in the silicate minerals of igneous rocks. Pyroxenes, amphiboles and some mica generally contain such things as magnetites (Fe_30_4), hematite (Fe_20_3), and Limonite ($2Fe_20_3$ $3H_20$). The sulphide and carbonate minerals are also important sources of iron. They include pyrite (FeS_2) and siderite ($FeCO_3$). Manganese is found in greatest abundance in metamorphic and sedimentary rocks. Only small amounts are found igneous rocks. The fact that manganese bearing minerals are less abundant than iron bearing minerals accounts for the fact that iron is found more frequently than manganese in groundwater. The hydroxides and carbonates of Mn^{++} are, however, somewhat more soluble than those of Fe^{++}.

The concentrations of iron and manganese found in water do not often exceed 10 mg/l and 2 mg/l respectively. Iron and manganese in water supply may originate from two sources. Iron and manganese present in the raw water or iron dissolved from inside surfaces of the pipe network. Trouble due to iron and manganese are therefore avoided by:

- treatment of the water before distribution, if it contains more than the allowable amounts of iron and manganese or if it is corrosive to metals.
- only using acceptable thermoplastic or lining of ferrous pipes which should for reasons of preventing corrosion be mandatory anyway
- avoidance of dead ends in pipe distribution networks
- periodical flushing of mains

3.12.2 Removal Methods

Chemical analysis of water alone may not always provide a clue as to the removal methods to be adopted. Hence it is advisable to carry out laboratory and pilot plant studies before any particular method is proposed. For surface waters with an organic content, pre-treatment oxidation, preferably by aeration, potassium permanganate or ozone followed by filtration alone or by settling and filtration is usually necessary. Alternatively, it can be achieved by post-treatment use of chemicals such as chlorine or chlorine dioxide after the removal of most organic matter because of the risk of producing THMs.

A guide to the more suitable method or methods of oxidation is given in the following table:

TABLE 3.10: OXIDATION OF IRON AND MANGANESE

METAL	OXIDANT	STOICHIOMETRIC QUANTITY OF OXIDANT (mg/mg Fe or Mn)	REDUCTION IN ALKALINITY (mg $CaCO_3$/mg Fe or Mn)	OPTIMUM pH
Fe(II)	Oxygen	0.14	1.80	>7.5
Mn (II)	Oxygen	0.29	1.80	>10.0 (a)
Fe (II)	Chlorine	0.63	2.70	>7.0
Mn (II)	Chlorine	1.29	3.64	>9.0 (a)
Fe(II)	Potassium permanganate	0.94	1.49	>7.0 (a)
Mn (II)	Potassium permanganate	1.92	1.21	>7.0
Fe(II)	Chlorine dioxide	0.24	1.96	>7.0
Mn (II)	Chlorine dioxide	2.45 0.49	3.64 2.18	≤7.0 ≤7.5
Fe(II)	Ozone	0.43	1.80	(c)
Mn (II)	Ozone	0.87	1.80	(c)

(a) The use of a catalytic filter medium may reduce the pH to 7.5 – 8.5

(b) Reaction is known to proceed at pH ≥ 5.5

(c) pH value at which the reaction occurs is less dependant than for other oxidants. Low pH values are preferred as ozone performs better under acidic conditions.

More specifically the treatment processes include:

(i) Precipitation and filtration

(ii)
  (a) Aeration, detention and filtration (with supplementary chlorination and/or addition of lime
  (b) Oxidation by ozone or potassium permanganate, followed by filtration
  (c) Oxidation by chlorine or chlorine dioxide (not for surface waters containing organic matter) followed by filtration
  (d) Calcined magnesite diatomaeous earth filtration

(iii) Zeolites

(iv) Stabilization and polyphosphates

3.12.3 Precipitation and Filtration

The great majority of iron manganese removal plants employ aeration, sedimentation and filtration, and this should always be considered first. The basic theory depends upon the fact that solubility of F^{+++} is very low compared to that of Fe^{++}. Also the higher oxidation states of manganese are far less soluble than Mn++

The first step in the treatment process is the oxidation of ferrous to ferric, i.e. aeration. The rate of oxidation of ferrous ions by aeration is slow under low pH, increasing 100% for every unit rise of pH.

Aeration has a large number of uses in water treatment, including:

(i) to increase dissolved oxygen content in the water;
(ii) to reduce tastes and odours caused by dissolved gases in the water, such as hydrogen sulphide, which are then released; and also to oxidise and remove organic matter;
(iii) to decrease carbon dioxide content of water and thereby reduce its corrosiveness and raise its pH value;
(iv) to oxidise iron and manganese from their soluble states to their insoluble states and thereby cause them to precipitate so that they may be removed by clarification and filtration processes;
(v) reduction of radon; and
(vi) to remove certain volatile organic compounds.

Aerators fall into two categories:

- Falling water aerators
- Injection aerators

In the falling water aerators, water is dropped through air and in the second group; air is introduced in to the water as small bubbles. Falling water aerators can be divided into:

(i) spray aerators
(ii) multiple tray aerators
(iii) cascade aerators

3.12.3.1 Spray Aerators

Water is sprayed through nozzles upward into the atmosphere and broken up into either a mist or droplets. Water is directed vertically or at a slight inclination to the vertical. The installation consists of trays and fixed nozzles on a pipe grid with necessary outlet arrangement.

Nozzles usually have diameters varying from 10 to 40 mm, spaced along the pipe at intervals of 0.5 to 1m or more. Special (patented) types of corrosion resistant nozzles and sometimes plain openings in pipes, serving as orifices are used. The pressure required at the nozzle head is usually 7 m of water but in practice, varies from 2 – 9 m and the discharge rating per nozzle varies form 30 - 600 l/min. Usually aerator areas are 30 – 90 m^2 per 1,000 m^3/hr

The ‘Dresden’ type of nozzle has given very good results in removing CO_2 and in adding O2 but poor for radon removal.

3.12.3.2 Multiple Tray Aerators

These aerators consist of a series of trays with perforated bottoms. The trays are filled with coke, stone or ceramic balls having a catalytic effect on iron removal. Normally 3 - 5 trays are used at the intervals of 0.3 - 0.7 m which means that the head needed is 1.5 – 3 m. The area required is 15 45 m^2 per 1,000 m^3/hr. These aerators have good CO_2 removal and good 0_2 increase.

However, disadvantages are:

- risk of clogging
- difficult cleaning and breeding places for worms

3.12.3.3 Cascade Aerators

In cascade aerators water is allowed to flow progressively downward either after spreading over inclined sheets or over a series of free overflowing weirs. Turbulence is secured by allowing the water to pass through a series of steps or baffles. An illustration of a cascade aerator is presented below:

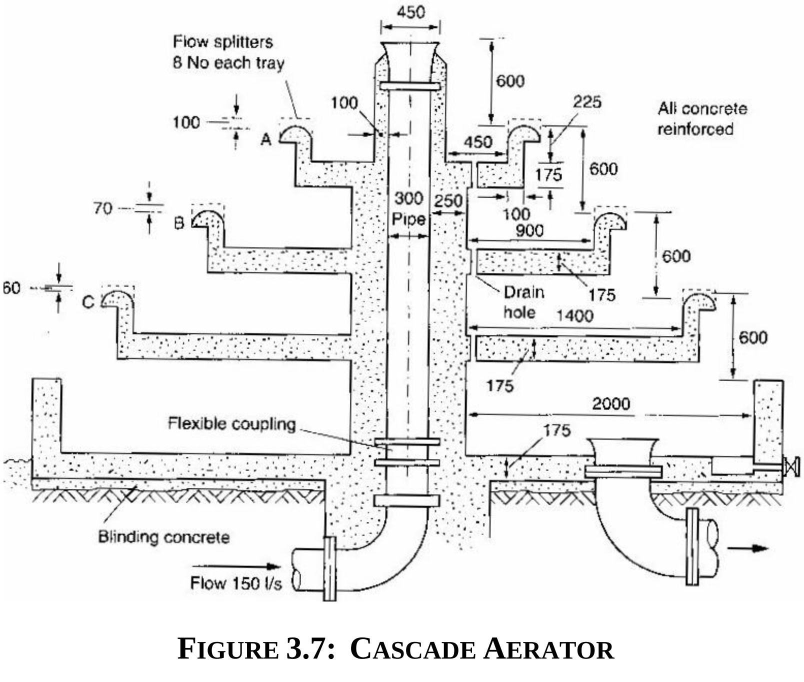


FIGURE 3.7: CASCADE AERATOR

These are the simplest type of free-fall aerators and will take large quantities of water in a comparatively small area and at low head. They are simple to keep clean and can be made of robust durable material such as reinforced concrete and are best in the open air. Exposure time can be increased by increasing the number of steps which is normally between 3 and 10. The fall in each step is usually between 0.15 - 0.6 m. The area required is about 40 m^2 per 1,000 m^3/hr.

The efficiency for raising the content of 0_2 is good and can reach 2.5 kg O_2/kWh, but that for CO_2 removal rarely better than 60 – 70 %, whilst radon reduction can exceed 50%.

3.12.3.4 Injection Aerators

These aerators have a good efficiency in raising O_2 content but poor CO_2 removal. They can be categorised as:

- bubble aerators
- venturi aerators
- brush aerators
- Inka aerators

(a) Bubble Aerators

In these aerators, air is blown to the bottom of the tank through porous filters. The depth of the tank is 3 - 4.5 m and the width about 2 times the depth. Detention time in the tank is 10 - 20 minutes. Air required is 40 – 120 m^2 per 1,000 m^3/hr, being 40% to 80% of water capacity. The air bubbles should be as small as possible but the clogging of the filters interferes with that aspect. Good mixing in an aeration tank improves efficiency. Bubble aerators are often applied to existing treatment plants where no spare hydraulic head is available.

(b) Venturi Aerators

In venturi aerators air is not blown into the water but is drawn in by a venturi. Improvement in 0_2 is good but CO_2 removal is poor.

(c) Brush Aerator

This type of aerator consists of a revolving drum, diameter about 0.5 m, submerged about 0.4m of the diameter, and rotating about 100 rpm. However, this type is not commonly used for water treatment.

(d) Inka Aerators

This aerator consists of a perforated stainless steel plate under which air is blown. Water flows over the plate. The air water ratio is very high. It can be as high as 100:1. This amount of air causes heavy turbulence and so O_2 raising efficiency and CO_2 removal are good. Energy consumption is rather high corresponding to a hydraulic head of 7 m if air water ratio is 100. A disadvantage is the clogging of the perforated plate.

Conclusions

Aeration can be achieved by any of the above mentioned methods. Sometimes aeration is not effective in oxidation of iron and especially manganese. This is the case if iron and manganese are in the form of organic complexes. Strong oxidants are then employed to speed up the oxidation. Chlorine and chlorine dioxide should only be considered for water low in organic content due to the formation of THMs and in that case potassium permanganate should be considered as the oxidant but the dose must be carefully controlled to minimise any excess passing into the water supply and causing a pink coloration. In some instances, ozone may be the preferred choice. The rate of oxidation of Fe^{++} by oxygen is slow under conditions of low pH. In such cases, pH is increased by removal of CO_2 or addition of lime.

3.12.4 Zeolite

By filtering the water through an ion exchange material such as sodium cation exchange the cations of valence two or higher will be exchanged for sodium ions.

That means for example that manganese ions are retained by the medium and the amount of sodium in the filtered water is increased. The capacity of the exchange material is limited and after the capacity is exhausted, the filter is regenerated by NaCl solution. The deposited ion, manganese, etc. are removed from the filter as brine. Some difficulties may be experienced in using this method. Iron and manganese may form hard deposits, which may not be removed by generation, thereby reducing the exchange capacity of the material.

This method of treatment should only be considered for small water supplies in which concentrations of iron and manganese are low, generally less than 1.0 mg/l.

3.12.5 Stabilization of Iron and Manganese

Sodium hexametaphosphate has been suggested as a means of preventing the appearance of iron deposits. It may be added to well water before the oxygen of air has had an opportunity to convert ferrous ions to their ferric state. However , relatively large concentrations of sodium hexametaphosphate may be required, up to the four times the iron content of water and it's use should be limited to water having an iron content of less than 1mg/l.

Such treatment if introduced subsequently to the initial scheme must be applied with caution, because overdosing will detach pre-existing deposits and cause serious blocking of mains, meter boxes, etc. than would have been experienced with any rusty water prior to the introduction of treatment.

3.13 DESALINATION OF WATER BY REVERSE OSMOSIS

3.13.1 General

Of the several techniques available for desalination and in the absence of large quantities of low grade heat from power stations, the most promising method of desalination of both brackish and sea water remains reverse osmosis (RO) whether it be for brackish water or seawater.

Reverse osmosis is the process of forcing a solvent from a region of high solute concentration through a membrane to a region of low solute concentration by applying a pressure in excess of the osmotic pressure. This is the reverse of the normal osmosis process, which is the natural movement of solvent from an area of low solute concentration, through a membrane, to an area of high solute concentration when no external pressure is applied. The membrane here is semi-permeable, meaning it allows the passage of solvent but not of solute.

Membranes used for reverse osmosis have a dense polymer barrier layer in which separation takes place. In most cases the membrane is designed to allow only water to pass through this dense layer while preventing the passage of the solute (e.g. salt). This process requires that a high pressure be exerted on the high concentration side of the membrane, usually 2 - 14 bar for fresh and brackish water, and 40 - 70 bar for seawater, which has around a 24 bar natural osmotic pressure which must be overcome.

Package plants are available from small single membrane units producing 20 l/hr of potable water to large units producing 20m^3/hr and greater and providing there is a good and reliable electricity supply, and a competent workforce, a potential area where these can prove competitive is in coastal areas to deal with seasonal peaks such as occur in tourist beach areas and where the alternative is the duplication of pipelines just to meet such peak demands as such water can be readily blended with the base flow of water from a surface source.

3.13.2 The Process of Reverse Osmosis

Conventional systems typically include a number of stages, including:

- a chlorine disinfectant stage;
- a sediment filter to trap particles including rust and calcium carbonate;
- a second filter stage often using sand/anthracite pressure filters;
- an activated carbon filter to trap organic chemicals and residual chlorine;
- one or two stages of high pressure pumps with an energy recovery turbine after the first stage;
- one or two stages of a reverse osmosis (RO) filter with thin film composite membranes;
- optionally a second carbon filter to capture those chemicals not removed by the RO membrane; and
- disinfection of the remaining microbes.

3.13.3 RO Plants Manufactured for the African Market

Whilst capital costs are high per m^3 capacity, running costs continue to fall as techniques and membranes improve.

Should a designer require any further information, a possible source for this can be contacted at rsesales@icon.co.za.

For small systems particularly well suited to brackish borehole water there is a chemical free RO system patented under the name of ‘Silver Spring’ and further information can be obtained by e-mail from ‘ivan.dulic@zg.t-com.hr’.

3.14 DEFLUORIDATION OF WATER

3.14.1 Fluorides

Fluoride is the ionic form of fluorine. Fluorides are organic and inorganic compounds containing the element fluorine. As a halogen, fluorine forms a monovalent ion (−1 charge). Fluoride forms a binary compound with another element or radical. Examples of fluoride compounds include hydrofluoric acid (HF), sodium fluoride (NaF) and calcium fluoride (CaF_2), and uranium hexafluoride (UF_6).

Fluoride compounds, usually calcium fluoride, are naturally found, usually in low concentration in water. However, water from underground sources can have higher levels of fluoride to the level that it becomes a health hazard. Excessive fluorides in drinking water may cause mottling of teeth or dental fluorosis, a condition resulting in the coloration of the tooth enamel, with chipping of the teeth in severe cases, particularly in children. With even higher levels of fluorides, there are cases of fluorosis of the bony structure.

The chief sources of fluorides in nature are:

- fluorapatite (phosphate rock)
- fluorspar
- crylite and
- igneous rocks containing fluosilicates

Because defluoridation is both difficult and expensive, the Tanzanian Temporary Standards allows a higher level of fluoride in water than WHO. A designer in deciding on whether or not to include defluoridation in a water supply scheme should therefore consider both number of potential consumers, alternative sources, the financial consequences both in capital and running and whether or not there is a possibility to dilute the water containing the fluoride as a means of reducing the concentration.

3.14.2 Defluoridation

Defluoridation is necessary when the fluoride concentration is higher than acceptable limits. The following methods may be considered for attaining defluoridised water.

(i) Desalination
(ii) Additive methods
(iii) Absorption methods

3.14.2.1 Desalination

Desalination effectively removes all dissolved impurities from water. This can be accomplished in one of several ways, by freezing, by distillation, by electrolysis or by reverse osmosis. The cost of desalination is high although the costs of reverse osmosis have fallen considerably in recent years. Nevertheless, this method is more appropriate to deal with brackish or sea water although it should not be entirely ruled out where there a few if any alternative sources and there is a good supply of electricity.

3.14.2.2 Additive method

In this method, one or more chemicals are added to water. The fluoride is then absorbed and both the additive and the fluoride are consequently removed by using conventional treatment processes such as sedimentation and filtration. A wide variety of materials have been tried including lime, magnesium sulphate, magnesium oxide, calcium phosphate, aluminium sulphate, various natural earths, bauxite, sodium silicate and sodium aluminate.

Excessive lime treatment for softening affects removal of fluoride due to its absorption by the magnesium hydroxide floc. However, sizeable fluoride removal is possible only when magnesium is present in large quantities which may not always be the case and magnesium may have to be supplemented in the form of salts.

The initial cost and cost of chemicals is very high and resultant sludge is environmentally difficult to dispose of.

3.14.2.3 Absorption methods

Absorption methods employ a bed or filter of generally insoluble material through which the water is allowed to percolate periodically. As it becomes saturated, with the fluoride 'removed, the absorptive media is either replaced or appropriately regenerated.

Materials used have included charred bone, activated alumina, activated carbon, tricalcium phosphate, natural and synthetic ion exchange materials and aluminium sulphate. Studies elsewhere have revealed that activated carbon has a good capacity to remove fluoride where the concentration is less than 10 mg/l. and the water is low in salinity. An activated carbon for fluoride removal has been developed in India by carbonising paddy husk or saw dust, digesting under pressure with alkali and quenching it in a 2% alum solution. The spent material can be regenerated by soaking it in a 2% alum solution. Also a granular ion exchange material, 'Defluoron 2', which is sulphonated coal operating on the aluminium cycle has been developed in India.

The water treatment specialist M/S Degrémont recommended activated alumina in the case of the Arusha urban water supply project. However, defluoridation must be regarded as a sophisticated process and to determine suitability and quantity of chemical, pilot plant trials should be conducted first.

3.15 WATER QUALITY

3.15.1 Introduction

Domestic water supplied to a community should be free from particles and pathogens hazardous to human being and livestock; whilst taste, colour and odour should be kept at low limits acceptable to consumers. Water quality guidelines applicable in Tanzania are the WHO International Standards and the Tanzanian Temporary Standards.

3.15.2 The Water Quality Used in Tanzania

3.15.2.1 WHO Standards

These are International Standards for drinking water supply and are mostly applied to Urban Water Supplies and large rural water supplies These WHO Standards latest edition, ‘Guidelines for Drinking Water Quality, 2004, are obtainable without charge from: <http://www.who.int/water_sanitation_health/dwq/en/>.

3.15.2.2 Tanzania Temporary Standards – (T.T.S.)

The temporary water quality standards for domestic water supply in Tanzania are applicable to small rural water supplies and are as shown below.

TABLE 3.11: THE TANZANIA TEMPORARY STANDARDS FOR DOMESTIC WATER SUPPLY

NO	NAME OF CONSTIT-UENT	SYMBOL	UNITS	WHO STANDARD 3rd Ed., 2004 GUIDELINE VALUES	TANZANIA STANDARD (RURAL WATER SUPPLIES 1974)
	TOXIC				
1	Lead	Pb	mg/1	0.01	0.01
2	Arsenic	As	mg/1	0.01	0.05
3	Selenium	Se	mg/1	0.01	0.05
4	Chromium	Cr	mg/1	0.05	0.05
5	Cyanide	Cn	mg/1	0.07	0.20
6	Cadmium	Cd	mg/1	0.003	0.05
7	Barium	Ba	mg/1	0.7	1.00
8	Mercury	Hg	mg/1	0.001	Not mentioned
9	Silver	Ag	mg/1	Not mentioned	-do-

Cont’d

NO	NAME OF CONSTIT-UENT	SYMBOL	UNITS	WHO STANDARD 3rd Ed., 2004 GUIDELINE VALUES	TANZANIA STANDARD (RURAL WATER SUPPLIES 1974)
EFFECTING HUMAN HEALTH					
1	Fluoride	F	mg/1	1.50	8.00
2	Nitrate	NO_3	mg/1	50	(100)
3	Nitrite	NO_2	mg/l	3	Not mentioned
BEING ORGANO-SEPTIC					
1	Colour			15 TCU$^{(1)}$	50 mg/l
2	Turbidity			5 NTU$^{(3)}$	30 mg/l
3	Taste		-	Not objectionable$^{(1}$)	Not objectionable
4	Odour		-	Not objectionable$^{(1)}$	Not objectionable
SALINITY AND HARDNESS					
5	pH			6.5-8.5/9.5$^{(4)}$	6.5-9.2
6	Total Filterable Residue			1500$^{(1)}$	2000$^{(2)}$
7	Total Hardness	$CaCO_3$	mg/1	Not mentioned$^{(1)}$	600$^{(2)}$
8	Calcium			200$^{(1)}$	Not mentioned
9	Magnesium	Ca	mg/1	150$^{(1)}$	Not mentioned
10	Magnesium-sodium-sulphate	Mg Mg-Na_2- SO_4	mg/1 mg/1 -	-	Not mentioned
11	Sulphate	SO_4	mg/1	400	600$^{(2)}$
12	Chloride	CI	mg/1	600	800$^{(2)}$
NON TOXIC METALS					
13	Iron	Fe	mg/1	1.00$^{(1)}$	1.00$^{(2)}$
14	Manganese	Mn	mg/1	0.40	1.50$^{(2)}$
15	Copper	Cu	mg/1	2	3$^{(2)}$
16	Zinc	Zn	mg/1	15$^{(1)}$	15(2)
ORGANIC POLLUTION OF NATURAL ORIGIN					
17	BOD_5 (5 days)	O_2	mg/l	6.00$^{(1)}$	6.00
18	PV (oxygen abs. $KMNO_2$)	O_2	mg/l	10.00$^{(1)}$	20.00
19	Ammonium	NH_3	mg/l	0.50$^{(1)}$	Not mentioned
20	Total Nitrogen exc. Nitrate		mg/l	0.10$^{(1)}$	1.00

Cont'd

NO	NAME OF CONSTITUENT	SYMBOL	UNITS	WHO STANDARD 3rd Ed., 2004 GUIDELINE VALUES	TANZANIA STANDARD (RURAL WATER SUPPLIES 1974)
ORGANIC POLLUTION INTRODUCED ARTIFICIALLY					
21	Surfactants ABS (Akyl Benzyl Sulphonates)		mg/l	1.00(1)	2.00(2)
22	Organic matter as Carbon in chloroform extract		mg/l	0.20	0.50
23	Phenolic substances as phenol		mg/l	0.001(1)	0.002

(1) *Not included as Guideline values in WHO 2004 edition which shows significant changes in approach to recommended values from earlier editions*

(2) *Experimental and subject to change*

(3) Not included as a Guideline but ideally below 0.1 NTU

(4) 1984 edition 6.5 – 8.5. 2004 edition, no guideline value but proposes within 6.5 – 9.5

The latest WHO 2004 edition no longer has two types of criteria these being acceptable and allowable, but instead quotes Guideline values and leaves it to individual countries to have a greater say in what they consider best. In the table above, only WHO guideline and Tanzanian allowable values are shown. Otherwise ISO international standards should be applied to water distributed through water sources or systems serving cities, municipalities and townships. Furthermore, they apply to those water systems serving rural populations of more than 5,000 people - large scale rural water supplies; and lastly all those water systems which have a treatment system more complex than simple sedimentation or roughing filters and / or rapid sand filtration units. Tanzanian Maximum Permissible Values are as follows:

TABLE 3.12: MAXIMUM PERMISSIBLE VALUES

CHARACTERISTIC UNIT		CONCENTRATION		
	UNIT	CATEGORY I	CATEGORY II	CATEGORY III
1	2	3	4	5
A: TOXIC (Health significance)				
1. Lead (Pb)	mg/1	0.10	0.10	0.20
2. Arsenic (As)	mg/1	0.10	0.10	0.10
3. Selenium (Se)	mg/1	0.05	0.05	0.05
4. Chromium (Cr)	mg/1	0.05	0.10	0.20
5. Cyanide (Cn)	mg/1	0.05	0.05	0.10
6. Cadmium(Cd)	mg/1	0.05	0.10	0.20
7. Barium (Ba)	mg/1	1.00	1.00	1.00
8. Mercury (Hg)	mg/1	0.05	0.05	0.05

Table 3.12 Cont’d

CHARACTERISTIC UNIT		**CONCENTRATION**		
	UNIT	**CATEGORY I**	**CATEGORY II**	**CATEGORY III**
1	2	3	4	5
B: AFFECTING HUMAN HEALTH				
1. Fluoride (F)	mg/1	1.50	1.5	
2. Nitrate)SO$_3$)	mg/1	50	50	1.5
C: ORGANIC-SEPTIC				
1. Colour	mg/1	50	50	
2. Turbidity (SiO)	mg/1	25	30	
3. Taste	-			
4. Odour	-			
D: SALINITY AND HARDNESS				
1. pH		6.5-8.5	6.5-8.5	6.5-9.0
2. Total filtration residue	mg/1	1500	2000	
3. Total hardness (CaCO)	mg/1	-	600	
4. Calcium (Ca)	mg/1	200	300	500
5. Magnisum	mg/1		-	
6. Sodium sulphate	mg/1	150		
(SO)	mg/1	400	600	
7. Chloride (Cl)	mg/1	600		
E:NON-TOXIC METALS				
1. Iron (Fe)	mg/1	1.0	1.2	1.5
2.Magnesium(Mn)	mg/1	0.5	0.8	0.8
3. Copper (Cu)	mg/1	3.0	3.0	4.0
4. Zinc (Zn)	mg/1	0.2	0.2	0.1
F: ORGANIC POLLUTION OF NATURAL ORIGIN				
1. BOD$_5$ (5days)	mg/1	5	5	10
2. PV (oxygen abs) Mn0$_4$ (O$_2$)	mg/1	10	20	-
3. Ammonium	mg/1	0.5	2.0	-
4. Total Nitrogen Exclusive Nitrate	mg/1	0.1	1.0	-

Table 3.12 Cont’d

CHARACTERISTIC UNIT		**CONCENTRATION**		
	UNIT	**CATEGORY I**	**CATEGORY II**	**CATEGORY III**
1	2	3	4	5
ORGANIC POLLUTION INTRODUCED ARTIFICIALLY				
1. Surface ABS (Allkyl Benzy1 Surphanates)	mg/1	0.5	1.0	1.0
2. Organic matter as carbon in chloride contact	mg/1	0.5	0.5	-
3. Phenolic substances as phonol	mg/1	0.002	0.002	-

3.15.3 Examination of Water Quality

Bacteriological

It is essential to check the following periodically during the operation of a water supply.

TABLE 3.13- BACTERIOLOGICAL DRINKING WATER QUALITY

SUBSTANCE	**UNIT**	**EARLIER WHO GUIDELINES**		**STANDARDS FOR RURAL WATER SUPPLY IN TANZANIA**
		ACCEPT-ABLE	**ALLOW-ABLE**	
1. Coliform count per 100 ml at 37°C	Each	-	1-3	1-3
2. E. Coliform Count per 100 ml at 44°C	Each	NIL	NIL	NIL

Table 3.13 cont'd

1. *98% samples should not contain any coliform organism in 100 ml.*
2. *Up to 3 counts is allowed in Occasional Samples but not In consecutive samples.*
3. *Not occurring repeatedly of which improvement to sanitary protection should be sought, or alternative source should be found.*
4. *No sample should contain E. Coli in 100 ml.*

3.15.4 Bacteriological Examination and Organisms Indicative of Faecal Pollution

This should include colony counts of micro-organisms on non-selective media, and examination for faecal streptococci and possibly for *clostridium perfrigens* as well as for coliforms organisms and *Escherichia Coli*. Reporting forms for result on bacteriological examination is given in Appendix A and B.

Organisms used as indicators of pollution are E. Coli and the coliform group. E. Coli is of faecal origin. The coliform group may be of faecal origin and should be assumed that they are all of faecal origin unless a non faecal origin can be proved. Organisms of coliform group are all foreign to water and their presence should be an indicative of pollution in its wider sense.

Faecal streptococci (*streptococcus faecal*, can be a help when confirming the nature of pollution in water). Faecal streptococci occur in faeces and are smaller than that of E. Coli and their presence in a water supply indicates faecal nature of the pollution.

3.15.5 Anaerobic – Spore – forming organisms

The presence of spores of *Cl. Perftingens* in natural water suggests that faecal contamination has occurred.

Examination of faecal streptococci and anaerobic spore forming organisms should be a value determined when water samples are examined.

3.15.6 Virological Examination

Viruses, like enteroviruses, retroviruses and adenoviruses have been found in water, enteroviruses being resistant to chlorination. In the absence of the above viruses in chlorinated water, it may be assumed that the water is safe.

3.15.7 Biological Examination

Biological examination is used to determine tastes, odours and control remedial treatments. Biological examination is essential when water supply is to undergo conventional flocculation and filtration processes.

Chlorination is ineffective against certain parasites, including amoebic cysts, but the parasites can also be removed by filtration.

Amoebiasis can be conveyed by water completely free from enteric bacteria. Microscopic examination is the only safe method of identification and also with the same examination cercariae of schistosomiasis can be detected, although no other evidence suggests its existence through piped water supply.

Cyclops vector embryos of *Dracunculus Medinensis* which causes guinea worm disease or dracontiasis, also known as or related to dracunculiasis, are found in wells and simple physical improvements in the form of curbs, drainage and apron surrounds can exclude parasites in untreated wells.

3.15.8 Radiological Examination

The radioactivity levels are based on the recommendation of the International Commission on Radiological Protection (ICRP).

Gross alpha activity 3 p Ci/l, and gross beta activity 30 p Ci/l are the maximum permissible concentrations in water (in PCW). For occupational exposure to the respective nuclides multiply the figures for the gonadel or whole body exposure by a factor of 1/100 so as to make them applicable to consumers of drinking water belonging to total population.

The methods for analysis of gross alpha and gross beta activities should be carried out in the light of local conditions and in collaboration with the appropriate authorities

Radioactivity in drinking water should be kept to a minimum; radioactive wastes should not be discharged indiscriminately into sources that are to be used for supplies of drinking water. See also section 3.10 on radon.

3.15.9 Physical and Chemical Examination

Refer to Table 3.11 above.

3.15.10 Fluorides

Concentrations of fluoride in drinking water of up to 8 mg/l do not produce abnormal effects on bones in man but can in fact benefit the elderly from bone decalcification. So far nothing has been done on the defluoridation of water in a water supply in Tanzania at scheme level. What has been done first is at laboratory pilot plant scale, for this is to be· studied further and confirmed as far as Tanzanian conditions are concerned. Since the project is still on going it is not advisable to include specific recommendations in this design manual. However, whenever defluoridation is being considered the Ministry of Water should be consulted and the latest literature on the subject studied, to decide on and develop a method that suits the economic and climatic conditions and natural resources of Tanzania.

3.16 SAMPLING AND STORAGE OF SAMPLES

3.16.1 Bacteriological Sampling

The need for aseptic treatment must be strongly stressed, and the sample should be carefully handled to avoid pollution from hands, etc.

3.16.2 Bacteriological Sampling Procedure

The following procedure should be adopted:

- Clean the working
- Wash hands.
- Avoid dust and draughts in the working area
- Use only equipment known to be sterile
- Avoid touching any part of the container dish

3.16.3 Sampling Locations and Frequency

3.16.3.1 Bacteriological Samples

Samples should be collected from:

- Pumping stations Treatment plants
- Reservoirs
- Booster pumping stations Distribution system

3.16.3.2 Frequency of Sampling

This should depend on:

1. Quality of water source
2. Risks of contamination
3. Complexity of the system
4. Number of water sources
5. Length of distribution system
6. Dangers of epidemics arising

A guide to sampling requirements is given in the table below:

TABLE 3.14: SAMPLING REQUIREMENTS

POPULATION SERVED	MAX. INTERVAL BETWEEN SUCCESSIVE SAMPLES	MIN. NO. OF SAMPLES TO BE TAKEN FROM WHOLE DISTRIBUTION
Less than 20,000	1 Month}	1 sample/5,000 people/month
20,000 - 50,000	2 Weeks}	
50,000 – 100,000	4 Days}	
More than 100,000	1 Day	1 sample/10,000 people/month

3.16.4 Collection, Transport and Storage of sample for.

3.16.4.1 Bacteriological Examination

The following should be observed:

- Samples should be representative of the site where taken.
- Sodium thiosulphate should be added to the sampling bottles; if water contains traces of chlorine, chloramine or ozone:
- 0.1 ml of a 3% solution of crystalline sodium thiosulphate in a 170 ml bottle has no significant effect on the coliform or E. Coli contents of unchlorinated water up to 6 hours of storage. Sodium thiosulphate is sufficient to neutralize up to at least 5 mg/l of residual chlorine and therefore is recommended to be added to all bottles used for water samples.

3.16.4.2 Surface sampling

In rivers, lakes, stream and reservoirs, samples should be collected away from banks and at a depth approx. 30 cm below the surface.

3.16.4.3 Boreholes and Wells

Sampling should be taken after 15-30 minutes of pumping to avoid standing water. The pump outlet should be sterilized.

3.16.5 Sampling for Virological Examination

3.16.5.1 Frequency of Sampling

The frequency of examination for viruses depends on local circumstances.

It is not practicable for examination for viruses to be carried out as frequently as for bacteriological examination.

3.16.5.2 Collection, Transport and Storage of Sample

Sampling bottles for virological examination should be clean but not necessarily sterile. The sample size is normally 2 litres.

3.16.6 Sampling for Biological Examination

3.16.6.1 Collection, Transport and Storage of Sample

(a) Collecting Samples from Taps

The sample should pass through a membrane filter, then dry it;

The membrane filter is made transparent by treating it with immersion oil;

The deposit in the membrane filter can be 'examined both microscopically and macroscopically.

(b) Collecting Samples from Mains

Special nylon net or cotton bags can be attached to the outlet of a hydrant and the upstream section of the main washed through using a high water flow rate.

3.16.7 Sampling for Radiological Examination

The samples for radiological examination should take into account the fluctuation of observed activity levels of radionuclides in the water, the vicinity of any major sources of radiopollution, and the risk of contamination. The samples should be representative of the water that it is desired to examine as radionuclides are absorbed on the surface and solids. The samples should be collected in polythene bottles to reduce absorption on the walls of the containers.

3.16.8 Sampling for Physical and Chemical Examination

3.16.8.1 Sampling

Chemical examination should at least be carried out once a year. A shorter period routine examination should be adopted; when the supply is serving more than 50,000 people, when examination should be carried out once a month. For populations between 10,000 and 50,000, examination should be carried out four times a year, and more frequently for water works treatment processes and for new sources of supply.

3.16.8.2 Collection, Transport and Storage of Samples

A sample of 2 litres should be collected in a chemically clean bottle. The collection procedure for samples resembles that of bacteriological sampling except that the bottles should not necessarily be sterile. The samples should be transported to the laboratory within a maximum of 72 hours of collection & preferably within 24 hours.

APPENDIX A. Form Required for Examination of Water Quality

THE UNITED REPUBLIC OF TANZANIA WD. 7 HD. H26A
MINISTRY OF WATER (Revised 73)

Telegram: ………………………………………………………………………………… Laboratory
Telephone: …………………………………………………………………………………
In reply please quote: …………………………………………………………………
Laboratory No: ……………………… Date: ……………………………………………

WATER ANALYSIS REPORT

(1) ORIGIN OF THE SAMPLE

Analysis requested by …………………… Ref. No………………… Date ………………..
Date received at the Laboratory ………. Date collected for analysis …….time ………..
Temp …………………………….. °C ………………………………………………………..
Sources ……………………………… Site ……………….. Sampling position …………..
Depth ……………………………….. Treatment ………………………………………….

(2) PHYSICAL EXAMINATION

Appearance ……………………………… Colour ……………………………………… mg Pt/l
Turbidity ……………………………… J.T.U ………………….. Odour …………………..
Sediments …………………………… ml/l ………………….... pH ……………………..
Conductivity at 25°c ……………………… micro-ohms/cm

(3) CHEMICAL EXAMINATION (in milligrams per litre)

Alkanity (as CaCo]) Total Nitrogen ……………………… Sodium …………………
Phenolphthalein ………………… Ammoniacal Nitrogen …………… Potassium ……………
Total …………………………… Albuminoid Nitrogen …………… Sulphate ………………
Hardness (as $CaCo_3$) Nitrite Nitrogen ……………….. Chloride ………………
Carbonate ……………………….. Nitrate Nitrogen …………………… Fluoride ………………
Non Carbonate ………………… Manganese …………….. Total filterable solids ………………
Total ……………………………… Zinc …………………… Total suspended solids ……………
Calcium ……………….
Magnesium ……………………….. Lead ……………………… Others: ……………………………
Permanganate value (10 minutes) Iron ……………………………………………………………...
boiling using N/8O $KMn0_4$ ………………………………… Copper ………………………………
BOD (5 days) ……………………………………………………………………………………

(4) BACTERIOLOGICAL EXAMINATION

Number of colonies per ml growing on Nutrient agar {a) In 1 day at 37^oC ………………..
(b) In 3 days at 25°C …………………… Coliform M.P.N. per 100 ml …………………...
Escherichia Coli, (faecal coli.) M.P.N. per 100 ml ………………… other test …………………
……………………… Class of water ……………………………………..
REMARKS: ………………………………………………………………………………………
…………………………………………………………………………………………………
…………………………………………………………………………………………………
RECOMMENDATIONS: ……………………………………………………………………
…………………………………………………………………………………………………..
…………………………………………………………………………………………………..
Date ………………………………… ……………………………………
Senior Research Officer

APPENDIX B THE UNITED REPUBLIC OF TANZANIA
MINISTRY OF WATER

Telegrams ……………………	SOIL AND WATER LABORATORY
Telephone ……………………	PROJECT PREPARATION DIVISION
In reply please quote:	P. O. BOX 35066
LABORATORY NO………..	DAR ES SALAAM

BACTERIOLOGICAL WATER ANALYSIS REPORT

Analysis requested by ……………………………………………………………………...
Ref. No………………………………………..Dated…………………………………...
Source:……………………………………………………………………………………...
Data collected for analysis………………………Date analysed…………………………..

Lab No and site of sampling	Total coliform bact. 100ml (35°C	Faecal Coli/100 ml (44-50 °C)	Residual Chlorine mg/l	Comment

Date:……………   ……………………...

G P DSM 13542/7-81/5m   Senior Research