ARBA MINCH WATER TECHNOLOGY INSTITUTE

ARBA MINCH UNIVERSITY

FACULTY OF WATER SUPPLY AND ENVIRONMENTAL ENGINEERING

Course title: Water treatment
Academic year: 2020
Course code: WSEE-3142
Instructor: **Samrawit D.** & **Kindeneh B.**
Credit hours: 4
Target groups: G$_3$ WSEE (A, B & C)
Pre-requisite: Water supply
Semester: 2nd

Course objective

The major objective of this course is to introduce students to various methods of treating drinking water along with water quality analysis.

On successful completion of the course the students will be able to:

- ✓ Identify the various drinking water treatment methods,
- ✓ Analyze water quality

Course contents

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- ❖ **Students must attend 80% of the classes.**

Evaluation

- Continuous assessment……………………………………..50%
- Final exam……………………………………………….50%

References

1. Peavy, Environmental Engineering book. Terence J.McGhee. 1981. Water Supply and Sewerage. McGraw –Hill international Edition.
2. D Barnes, PJ Bliss, BW Gould & HR Vallentine. 1981. Water and Wastewater Engineering Systems. Pitman Press. Great Britain.
3. Roland L.DRoste. 1997. Theory and Practice of Water and Wastewater Treatment. Jonn Wiley and Sons. New York.
4. Howard S. Peavy et al. 1985. Environmental Engineering. McGraw –Hill international Edition

Chapter 1. Introduction

Common words

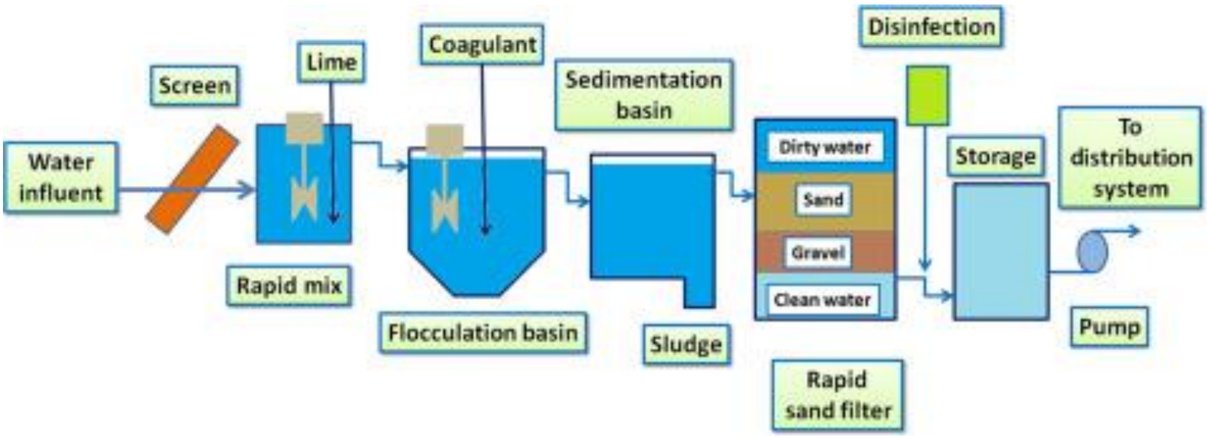
- Palatable- water having no unpleasant taste
- Safe- water free from pathogenic microorganisms & chemicals which could be a harmful for consumer
- Clean- water free from suspended solids and turbidity
- Colourless and odourless –water aesthetic to drink
- Reasonably soft – water allowing easy wash of cloths, dishes with less soap.
- Lower organic content- to avoid unwanted biological growth in pipes & smell
- Non corrosive- devoid of gases e.g. O_2,CO_2
- Potable- water suitable for drinking i.e. safe and pleasant to taste
- Whole same- potable water having sufficient minerals of natural origin which are must for life.
- Contaminated- water having microorganisms, chemicals or substances making the water unsafe and dirty.

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

The degree and methods of treatment depend upon nature of the source, quality of the source and purpose for which the water is supplied. The various treatment methods and the nature of source of impurities removed by employing them are given in the following table.

Table: Treatment methods

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



Treatment process selected will depend on the quality of water at the source and nature of water required. For example, in the case of water which taken from a surface source, generally the treatment unit required are plain sedimentation, coagulation, filtration and disinfection to make the fit for domestic use.

Objective of water treatment

The main objective of the treatment process is to remove the impurities of raw water and bring the quality of water to the required standard.

The objective may be summarized as follow:

(i) Preventing Disease Transmission

Organisms that cause disease must be removed or inactivated to make the water safe. Such organisms are small animals (invertebrates) and their eggs (ova), protozoa and their +cysts, bacteria which may form spores, and viruses.

(ii) Making the Water Acceptable

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

(iii) Protecting the distribution System

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

Location of treatment plant

The treatment plant should be located

- Near to the town to which water is to be supplied and near to the source of supply. This will prevent the water quality to depreciate after treatment.
- Away from any source of pollution.
- Away from the border of other countries and should be announced as a protected area. During war time, a neighbour country may play foul game by damaging the plant, poisoning the water.
- At higher elevation if the natural topography permit.

Treatment Schemes

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

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

c) **Treatment of Ground Water**:

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

Factors affecting the choice of treatment schemes

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

a. Limitation of capital

b. Availability of skilled and unskilled labour

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

d. Local codes, drinking water standards and material specifications

e. Local traditions, customs and cultural standards

f. National sanitation and pollution policies.

Consideration for treatment unities in developing country

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

Source of water pollution and water impurities

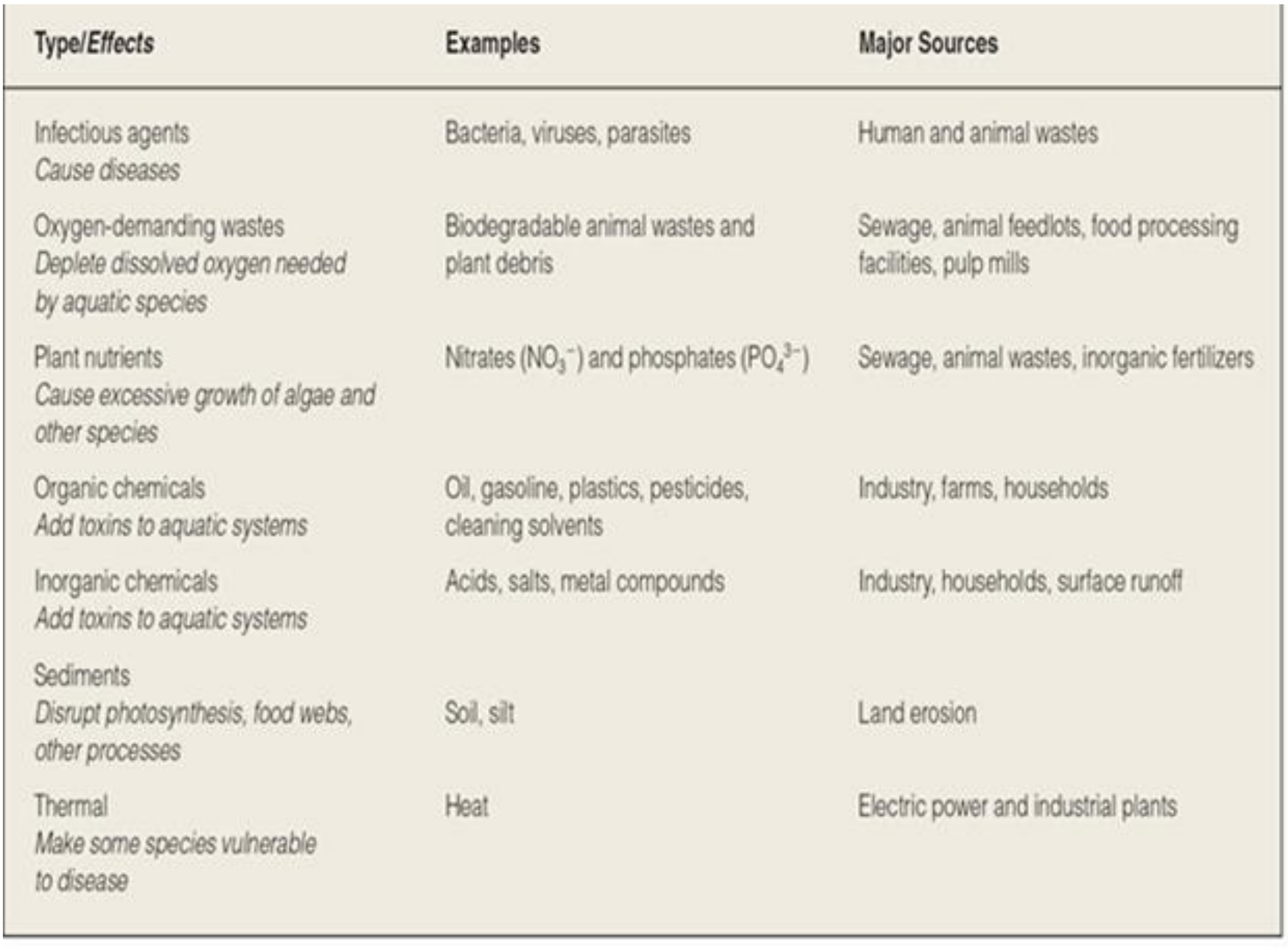
Water Pollution

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

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

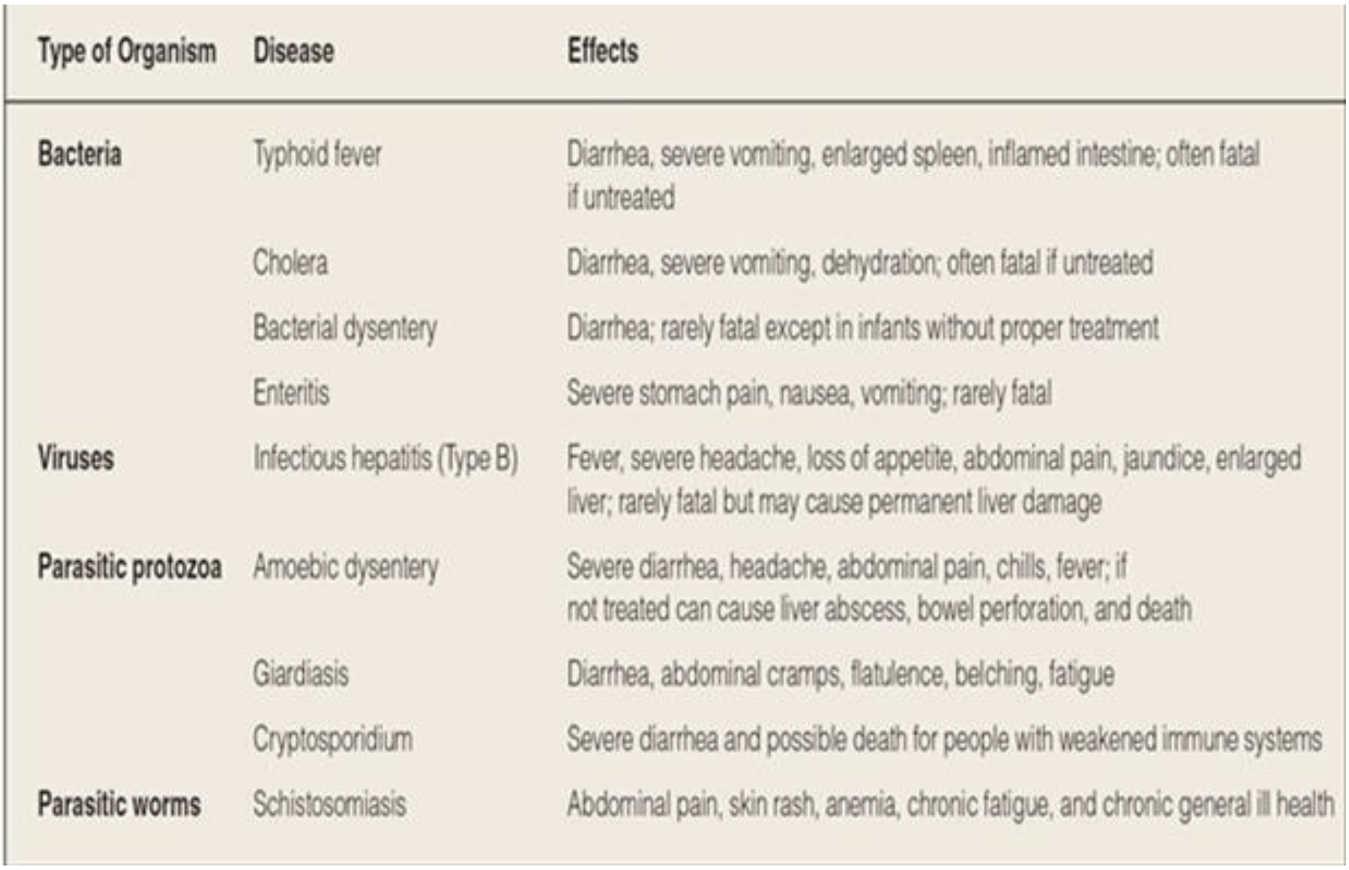
b) Non-point source: cannot be traced to a single site of discharge (atmospheric deposition, agricultural / industrial / residential runoff)

Table: Major water pollutants and their effect



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

Table: Common disease transmitted to humans through contaminated drinking water



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

Impurities in water

- The impurities which are present in water may be classified in the following three categories.

a) Suspended impurities

b) Colloidal impurities

c) Dissolved impurities

a) Suspended impurities

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

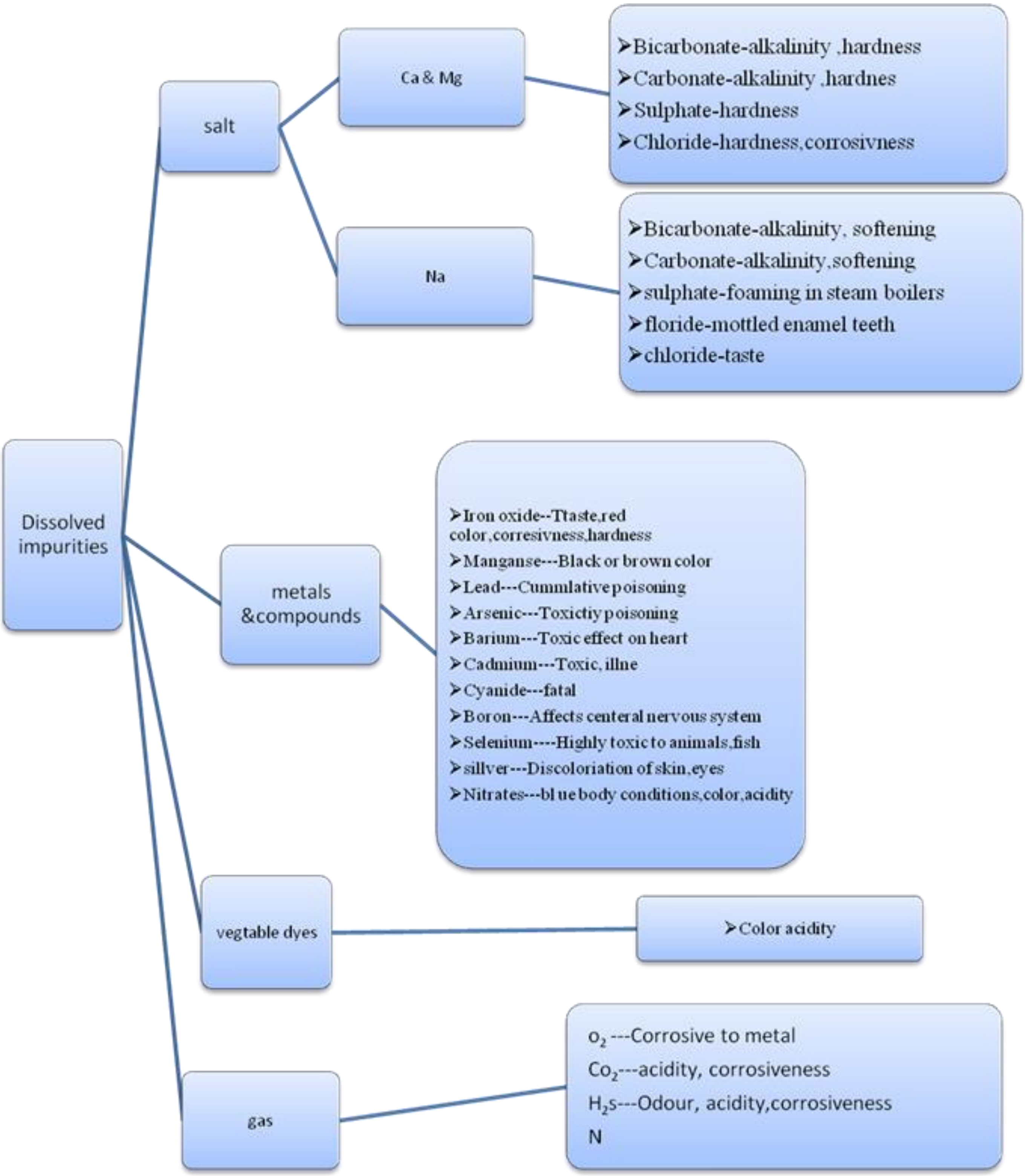
- Bacteria –some cause diseases
- Algae, protozoa –odor, turbidity
- Silt -muckiness or turbidity

b) Colloidal impurities

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

c) Dissolved impurities

- It includes organic compound, inorganic salt and gases etc.

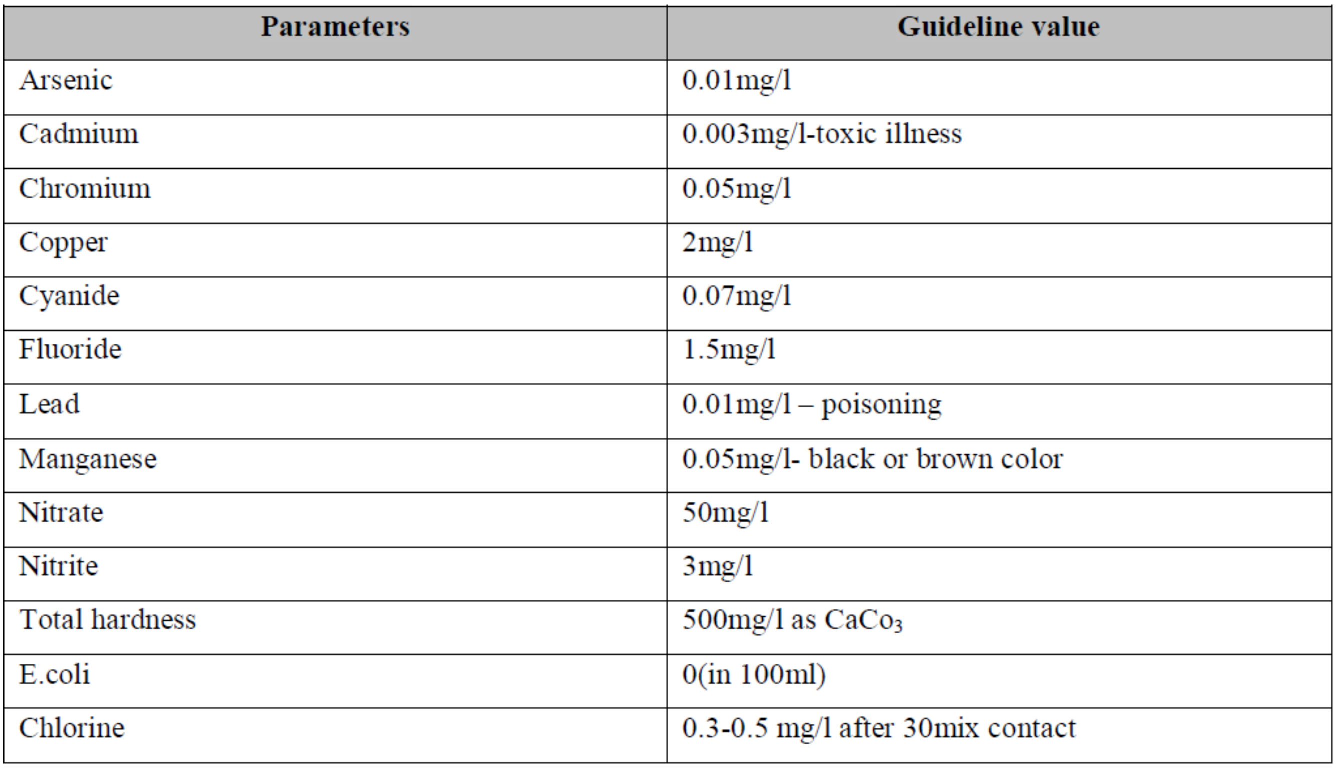


Drinking water quality standards

The treatment process should be designed such that, they should treat the water up to the desired standard for which it is to be used. A safe and potable drinking water should conform to the following water quality characteristics

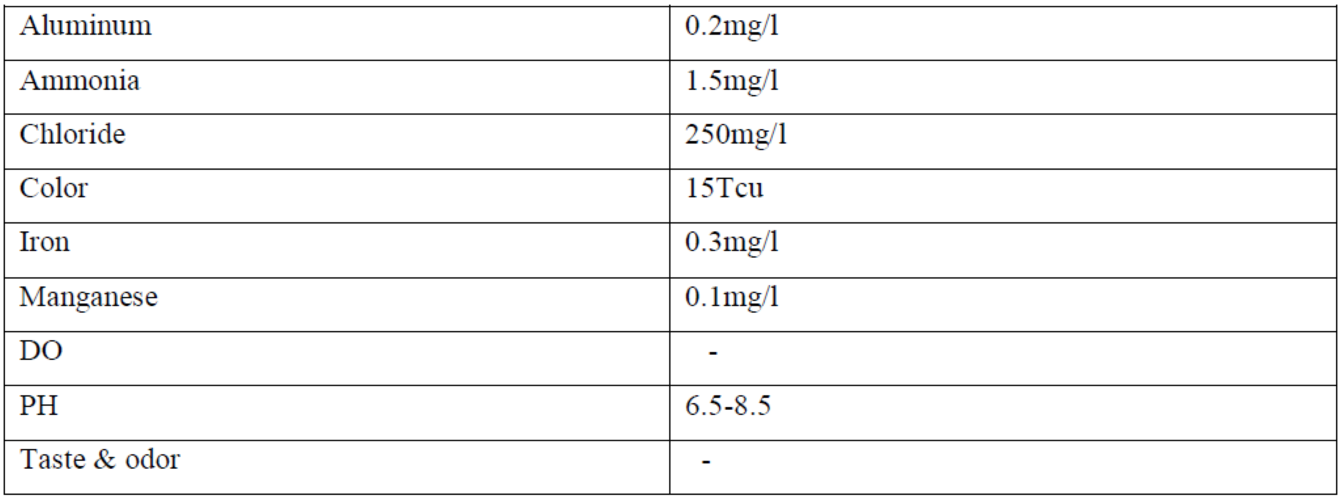
1. Free from disease causing microorganisms
2. Containing no compounds that have an adverse effect acute or in long term on human health.
3. Fairly clear (i.e. low turbidity, little colour)
4. No saline
5. Containing no compound that cause offensive taste or smell
6. No causing corrosion or encrustation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

Table : Standard for drinking water quality



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

Aesthetic quality value



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

Chapter 2. Preliminary treatment

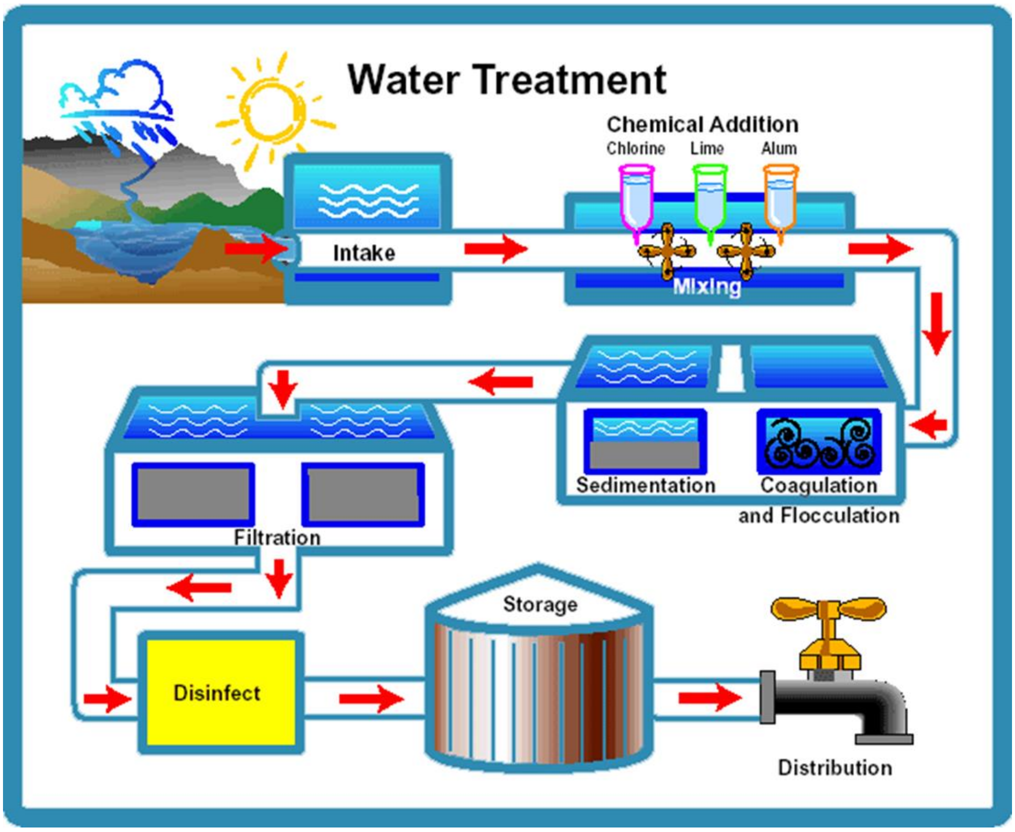
Pre-treatment

High turbidity water which may occur particularly during the rainy season requires pre-treatment in the form of sedimentation, storage or roughing filtration to reduce much of the suspended solids. This is an advantage otherwise a very large amount of chemicals may have to be employed for chemical coagulation which can be expensive. Pre-treatment provides cheaper treatment.

High turbidity waters require pre-treatment before slow sand filtration. There is, however, a form treatment known as dynamic filtration (although it is not very popular) which involves diverting a relatively large volume of water out of which a small percentage (10%) settles through the slow sand filter while the remaining is used to wash off the solids deposited on the sand top as the water moves horizontally via the sand bed.

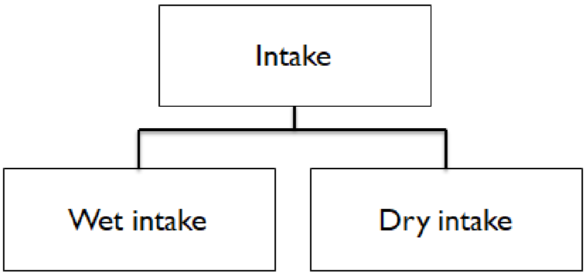
Types of preliminary treatment

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



Intakes

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



Wet Intake Structures

- It is a type of intake tower in which the water level is practically the same as the level of the sources of supply.
- It is sometimes known as Jack well and is most commonly used.

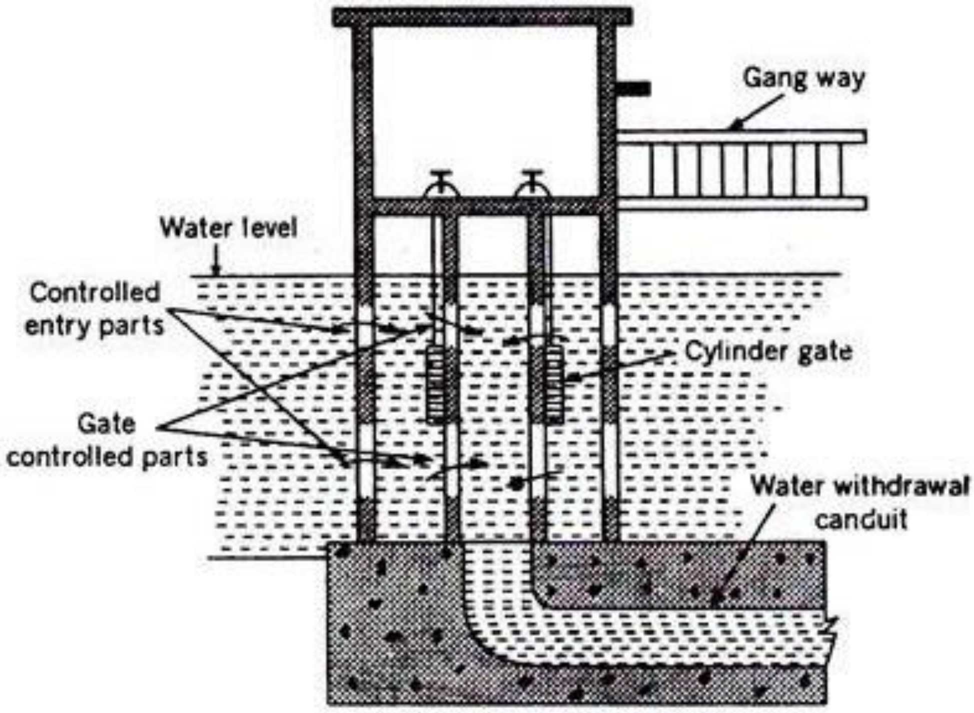
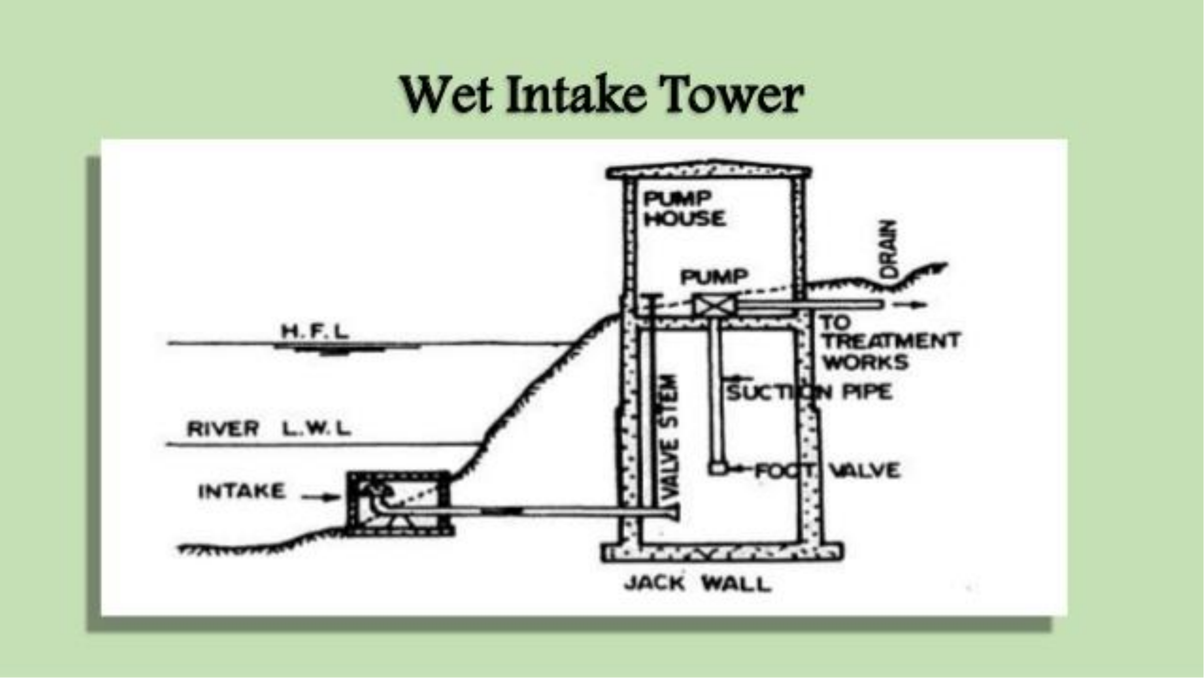


Fig. 7.4. Wet intake.

Wet intake towers

- It consist of a concrete circular shell filled with water up to the reservoir level and has a vertical inside shaft which is connected to the withdrawal pipe.
- The withdrawal pipe may lie over the bed of the rivers or may be in the form of tunnels below the river bed.
- Openings are made in to the outer concrete shell as well as, into the inside shaft.

- Gates are usually placed on the shaft, so as to control the flow of water into the shaft and the withdrawal conduit.



Wet Intake Tower

Dry Intake Structures

- In case of dry intake there is no water in the water tower.
- Water enters through entry port directly into the conveying pipes.
- It is simply used for the operation of valves etc.

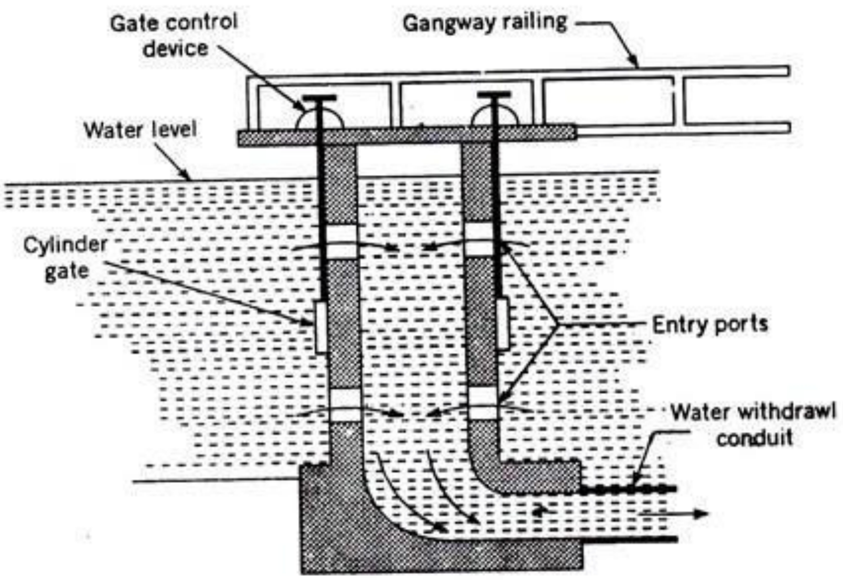
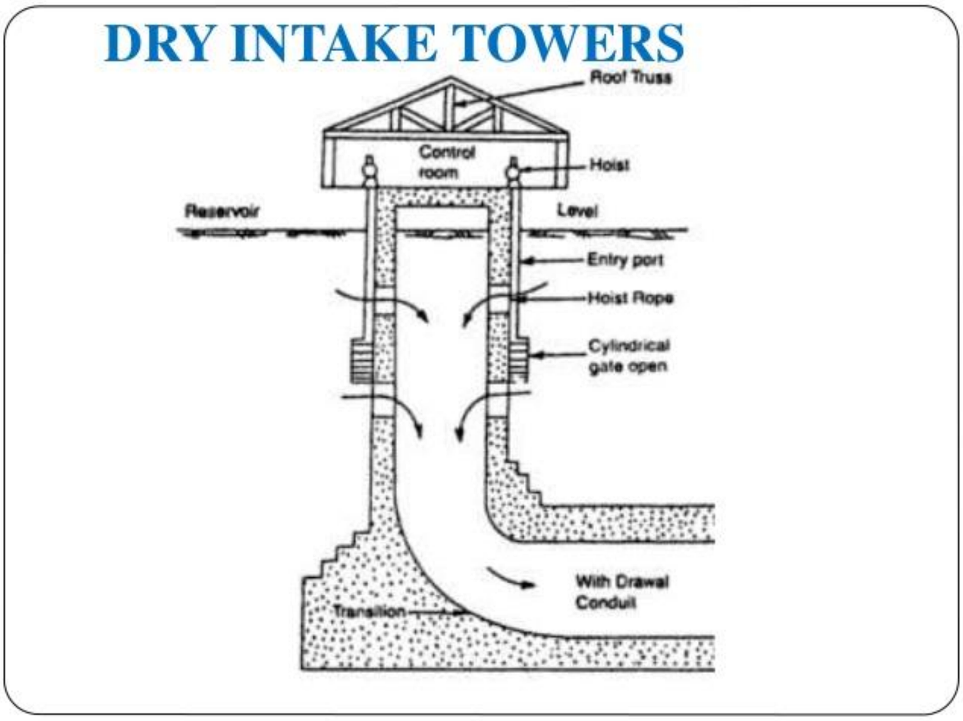
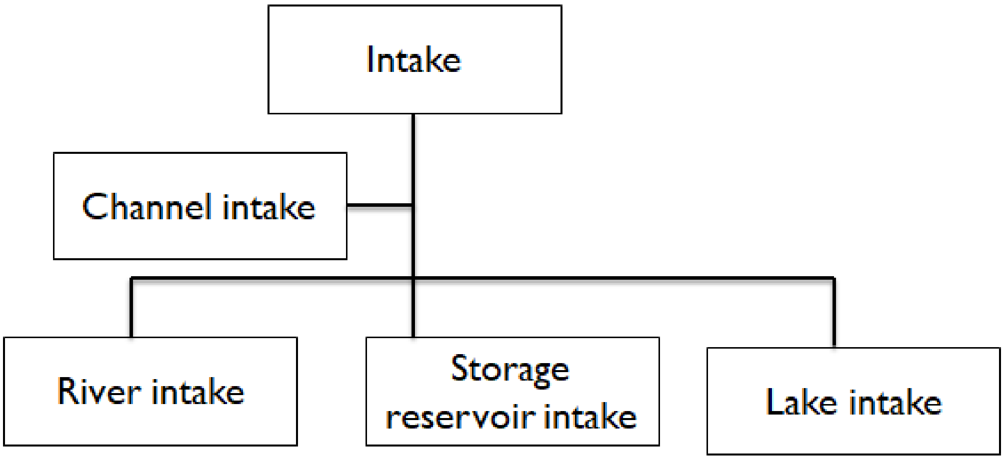


Fig. 7.5. Dry intake tower.



Types of intakes



Intake structures are used for collecting water from the surface sources such as river, lake, and reservoir and conveying it further to the water treatment plant. These structures are masonry or concrete structures and provides relatively clean water, free from pollution, sand and objectionable floating material.

River intake

- It is a type of **intake** which may either located sufficiently inside the **river** so that demands of water are met with in all the seasons of the year, or they may be located near the **river** bank where a sufficient depth of water is available.

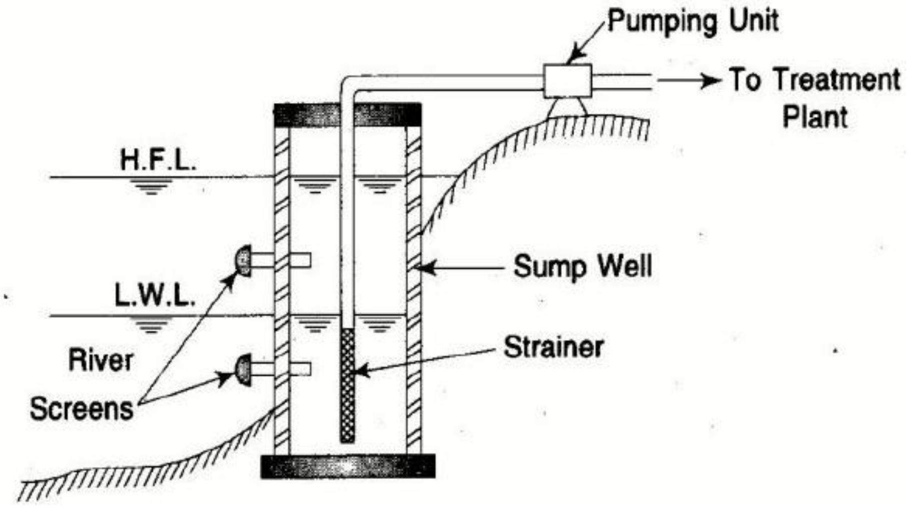


Figure: River Intake

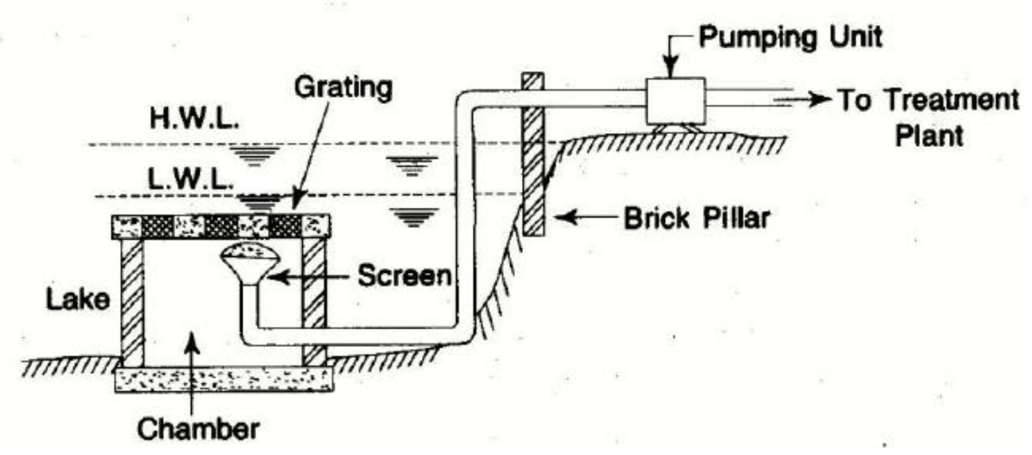


Figure : Lake Intake

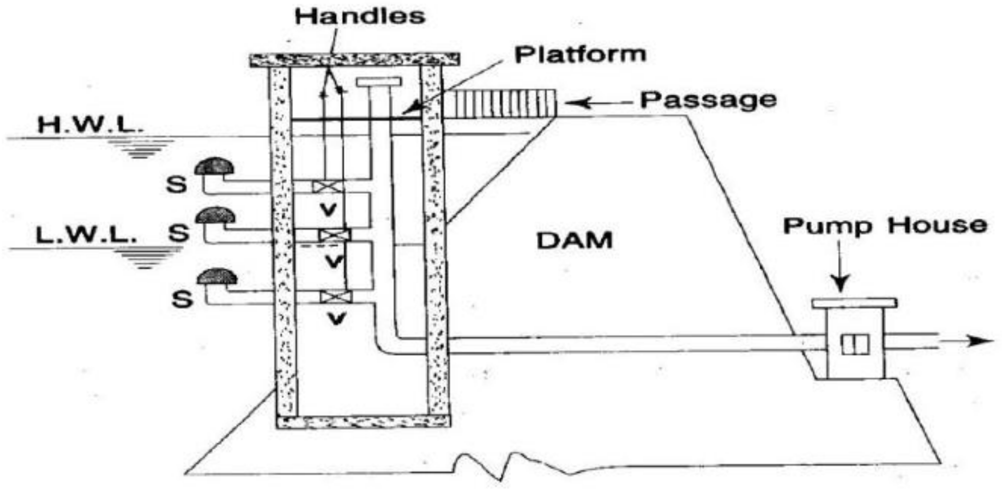


Figure : Reservoir Intake

Canal intake

- A **canal intake** consists of a pipe placed in a brick masonry chamber constructed partly in the **canal** bank.
- Opening of **structure** provided with coarse screen & end of pipe inside the chamber is provided with bell mouth fitted with hemispherical fine screen.

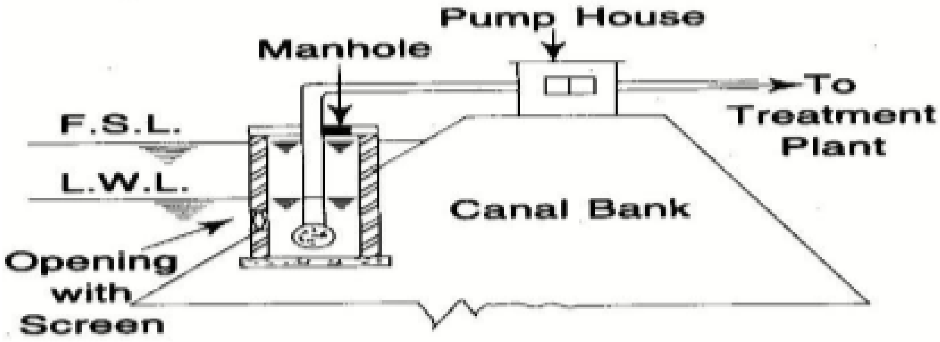


Figure : Canal Intake

Example 1

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

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

Solution:

1) Q through intake

Daily discharge = 150*80,000 = 12,000,000l/d

Since the channel runs only for 10 hrs

Intake load/hour = (12,000,000l/d))/(10hr/d)

$= 1200\text{m}^3/\text{hr} = 0.3333\text{m}^3/\text{s}$

2) Area of Coarse Screen in Front of intake

$$As = \left(\frac{Q}{V}\right) = \left(\frac{0.3333m\text{^}3/s}{0.16m/s}\right) = 2.083m\text{^}2$$

Let the area occupied by the solid bar be 30% of the total area

The actual area for the screen $\left(\frac{2.083m^2}{0.7}\right) = 2.98m^2$

Let assume the minimum water level @ 0.3m below the normal water level.

Let as keep the bottom of the screen @ 0.2m above the channel bed.

Available height of screen =1.8-0.3-0.2=1.3m

Required length of screen =2.98m/1.3m=2.29m ≈ 2.3m

Hence provide a screen size=1.3m*2.3m

3) Design a bell mouth entry

$$Ab = \left(\frac{Q}{V_b}\right) = \left(\frac{0.3333m^3/s}{0.32m/s}\right) = 1.042m^2$$

Diameter of bell mouth $Db = \sqrt{\left(\frac{1.042*4}{\pi}\right)} = 1.15m$

Hence provide a bell mouth of 1.2m

3) Design of intake conduit

Let us assume a velocity of 1.5m/s in the conduit

$$Di = \sqrt{\left(\frac{0.3333*4}{1.5*\pi}\right)} = 0..532m$$

Provide = 0.5m diameter intake conduit

Therefore the actual velocity V = $\left(\frac{0.3333*4}{\pi*0.5^2}\right) = 1.7m/s$

4) Head loss through the conduit

$V = 0.849CR^{0.63}S^{0.54}$

But C = 130 for cast iron

R = D/4 = 0.5/4 = 0.125

Substitute to the above Equation

$1.7 = 0.849*130*0.125^{0.63}S^{0.54}$

S = 4.98*10^{-3}

S = Hl/L

Hl = S*L

$= 4.98*10^{-3}*0.5*1000$

$= 2.49m$

Screens

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

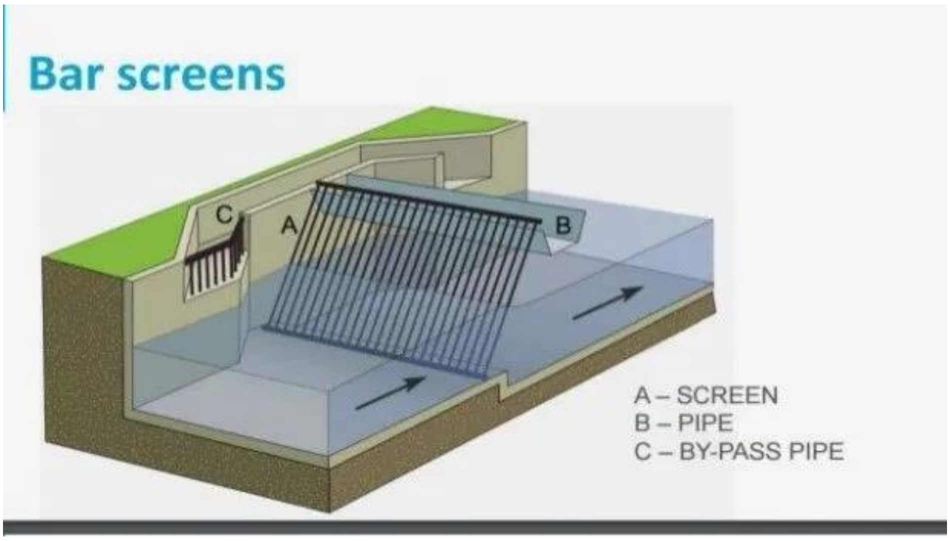
Purposes:

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

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

Bar screen spacing is typically between 0.5 and 5cm. Angle of inclination of bars is $60-75^0$ if screenings are very small and 30-450 if larger amount is retained over the screen bar. Velocity of flow should be low towards the screen bar (0.1-0.2m/sec.) it may be increased to 0.3-0.5 after the screen to prevent settling there.

Between the openings the velocity should be restricted to up to 0.7m/sec to avoid forcing through the suspended solids. If regular cleaning is done an allowance for loss of heads of up to 0.1 to 0.2m is made. However, to allow for delay and mechanical failures a loss of head allowance between 0.5 to 1.0m is made.



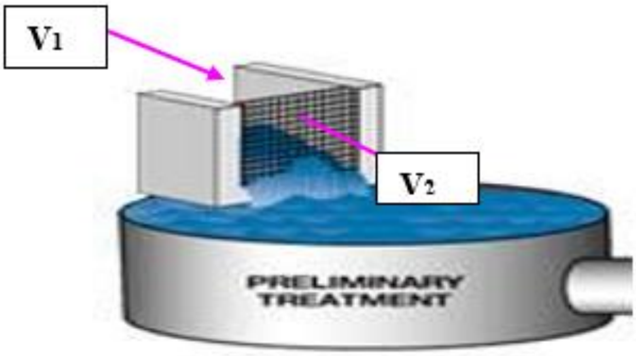


Figure: Screen

Head Loss Through Bar Screen

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

Where

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

V_2=velocity of flow through openings

V_1= approaching velocity of upstream channel

g= gravitational acceleration (9.81m/s^2)

Head Loss through Fine Screen

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

Where

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

g= gravitational acceleration (9.81m/s^2)

Q= discharge (m^3/s)

A=effective opening area of the screen, m^2

Example 2

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

Approach velocity = 0.6m/s

Velocity through a clean bar screen = 0.9m/s

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

Solution:

i. Compute the Headloss Through a clean Bar Screen

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

$$hl = \frac{1}{0.7} * \left(\frac{0.9^2 - 0.6^2}{2(9.81)} \right)$$

=**<u>0.0327m</u>**

ii. Compute the Headloss Through a clogged Bar Screen

$$Vi = \left(\frac{Q}{Ai} \right) \quad \text{And} \quad Vf = \left(\frac{Q}{Af} \right) \quad \text{but 50\% of Ai is only open}$$

Therefore, A_f =50%Ai

$$Af = \left(\frac{50Ai}{100} \right) = \frac{Ai}{2}$$

$$Vf = \left(\frac{Q}{Af} \right) = 2 * \frac{Q}{Ai} = 2Vi$$

From this the velocity through a clogged bar screen is doubled

V=0.9*2=1.8m/s

$$hl = \frac{1}{c} * \left(\frac{v_2{}^2 - v_1{}^2}{2g} \right) = \frac{1}{0.6} * \left(\frac{1.8^2 - 0.6^2}{2*9.81} \right) = 0.2$$

Example 3

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

Solution

i. Compute the Headloss Through a clogged Bar Screen

$Vi = \left(\frac{Q}{Ai}\right)$ And $Vf = \left(\frac{Q}{Af}\right)$ but 75% of Ai is only open

Therefore, $A_f = 75\% A_i$

$$Af = \left(\frac{75Ai}{100}\right) = \frac{3*Ai}{4}$$

$$Vf = \left(\frac{Q}{Af}\right) = \frac{4*Q}{3*Ai} = (4/3)Vi$$

From this the velocity through a clogged bar Screen is

V=0.9*(4/3) =1.2m/s

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

$$= \frac{1}{0.6}*\left(\frac{1.2^2 - 0.6^2}{2*9.81}\right) = 0.09\text{m}$$

Plain Sedimentation

Plain sedimentation is a form of pre-treatment that provides a low velocity of flow through a tank preferably excavated in the ground. The purpose is to settle some solids because of this low velocity by gravity sedimentation.

Plain sedimentation is favorable in a tropical climate due to the high load of suspended solids to be settled as a result of erosion by tropical high intensity rains, and also, because of the high temperature in the tropics and the associated low viscosity water in the sedimentation tank that provides less viscous resistance for the settlement of solids. Since plain sedimentation offers limited detention period for the water its effectiveness is restricted in that sense.

Significant reduction in solids is obtained for high turbidity waters, and, the feasibility of reducing the turbidity below 30 NTU- if the tank is provided as a pre-treatment unit for a slow sand filter must be tested through a settling column test. The settlement of solids is dependent on the nature of the suspension.

Table: Design parameter for plain sedimentation

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

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

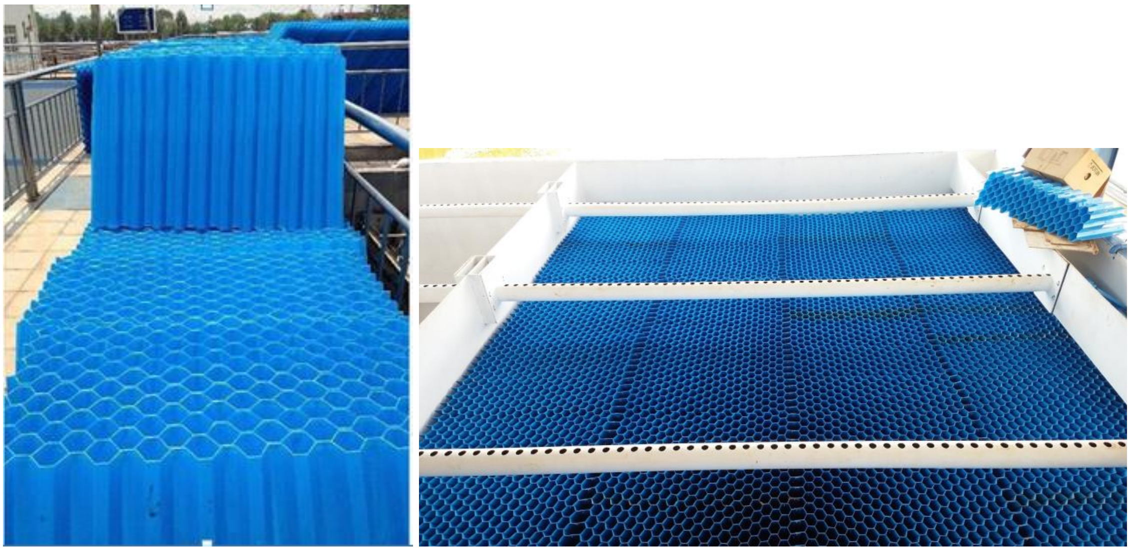
- For trapezoidal channels a thin Ferro-cement wall lining may be adequate.
- For waters laden with algae the outlet weir are arranged behind a deflecting baffle.
- At least two settling baffles are provided each designed for 3/4 of the design flow so that during cleaning of one of the tanks, the other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

Table : Turbidity removal

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

Tube Settlers

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



Storage

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

Advantages:

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

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

Example of treatment with Ponds (England)

Table : Storage effect on turbidity and E-Coli

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

- The costs of providing storage can be very high, however.
- Depths of ponds can be as high as 15m.

Roughing Filtration

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

Diameters:

- Rapid sand filters: 0.4 - 0.7mm
- Slow sand filters: 0.15-0.35mm
- Roughing filters: >2.0mm diam.

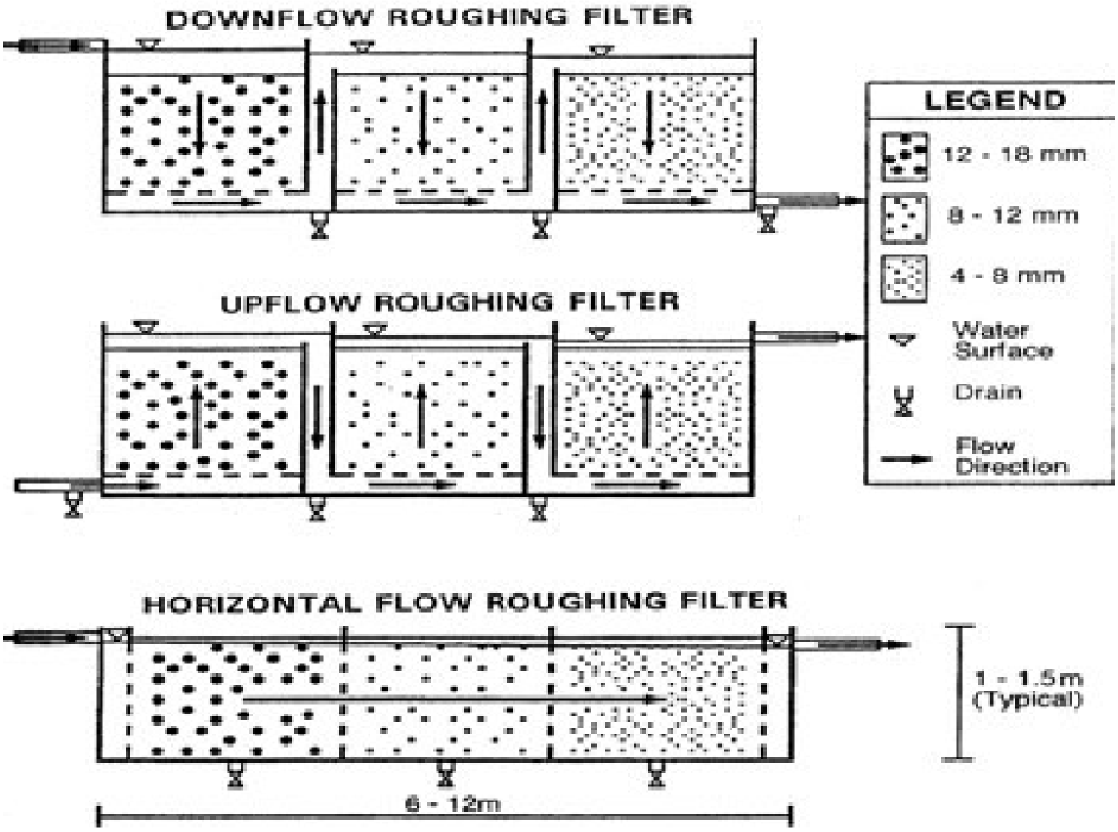
- Rate of filtration is variable depending on the nature of the media, turbidity of the incoming water, etc.

- Roughing filters if designed well, can serve as effective pre-treatment units prior to slow sand filtration for water with raw water turbidity of 20 to 150 NTU.

- For high turbidity waters, it may be difficult to clarify to below 30NTU without sedimentation or other pre-treatment such as storage.

- Apart from horizontal and vertical roughing filters, there is a form of roughing filtration known as up-flow down flow filtration that involves an up flow roughing filter together with chemical coagulation and flocculation followed by a down flow rapid filtration.

- This arrangement can result in reduction of cost almost by 50% compared with the conventional designs involving coagulation, sedimentation and filtration.



- Roughing filters are capable of reducing turbidity to as low as 5NTU from a raw water turbidity of 150NTU.

- Bacterial removal is between 60 and 90% without using any disinfectant.

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

a. Roughing filters do not require the addition of chemicals to reduce solids to the extent necessary for slow sand filters downstream.

b. They ensure long filter runs owing to their excellent solid storage capacity.

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

c. They do not require high skills to build them.

    - The filters can be filled with local materials screened on site.

d. Because of low head no substantial head is required.

    - The energy requirement is therefore reduced.
    - The water is beneath the gravel top hence contamination by man, animal, microorganisms and algae growths are avoided largely.

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

Filtration Theory

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

- Attachment or interception of the particle to be settled by the filter grain
- Transport of the particle to the filter surface by gravity

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

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

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

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

Design Guidelines:

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

ii. Three zones from course to fine media are provided with sizes from 4 to 40mm.

iii. The first course should be long:
    - First course: 4.5-6m

  - Second course: >4m
  - Last course: 1.5-2m

iv. Depth of 1.0-1.5m for easy manual desludging operations
v. There should be 10-20cm of gravel above the filter top
vi. Filter should slope in the flow direction to facilitate sludge withdrawal
vii. V-notch recommended at the outlet for measurement of flow.

Infiltration Gallery

- It is underground tunnel which have holes on its sides, used for tapping underground water near river, lakes and stream.
- This type of treatment is used when the water depth is very small and difficult to construct weir or dam to store water due to the seepage of the water.
- Sedimentation of silt is the main problem.

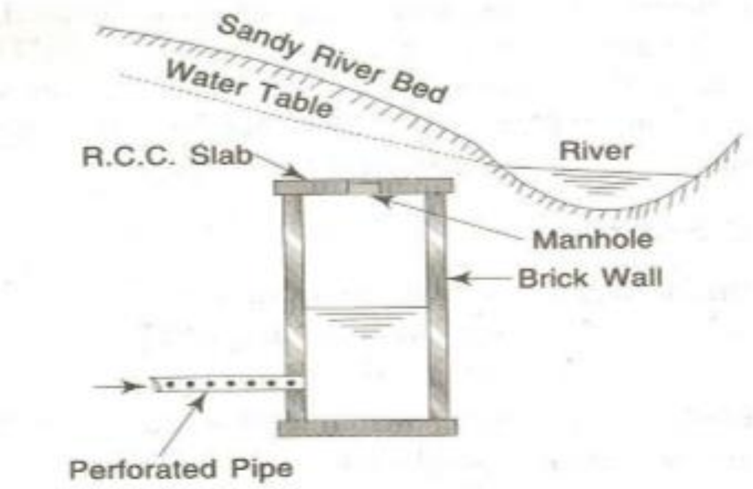
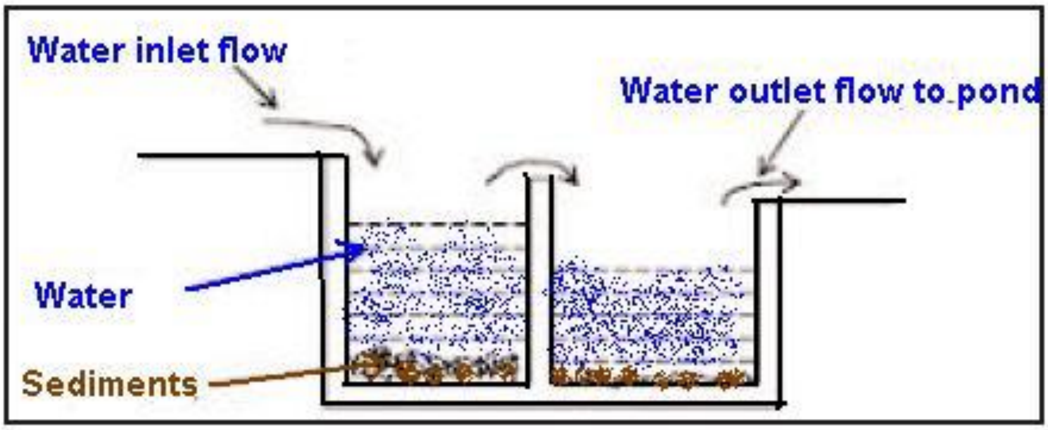


Figure : Infiltration Gallery

Silt trap

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



Silt trap on inlet channel (Image: International Water Management Institute)

Chapter 3. Sedimentation

Sedimentation is a treatment process in which the velocity of the water is lowered below the suspension velocity and the suspended particles settle out of the water due to gravity. The process is also known as settling or clarification.

- Most water treatment plants include sedimentation in their treatment processes.
- However, sedimentation may not be necessary in low turbidity water of less than 10 NTU.
- In this case, coagulation and flocculation are used to produce pinpoint (very small) floc which is removed from the water in the filters.
- The most common form of sedimentation follows coagulation and flocculation and precedes filtration.
- This type of sedimentation requires chemical addition (in the coagulation/flocculation step) and removes the resulting floc from the water.
- Sedimentation at this stage in the treatment process should remove 90% of the suspended particles from the water, including bacteria.
- The purpose of sedimentation here is to decrease the concentration of suspended particles in the water, reducing the load on the filters.
- Sedimentation can also occur as part of the pre-treatment process, where it is known as pre-sedimentation.
- Pre-sedimentation can also be called plain sedimentation because the process depends merely on gravity and includes no coagulation and flocculation.
- Without coagulation/flocculation, plain sedimentation can remove only coarse suspended matter (such as grit) which will settle rapidly out of the water without the addition of chemicals.
- This type of sedimentation typically takes place in a reservoir, grit basin, debris dam, or sand trap at the beginning of the treatment process.

- While sedimentation following coagulation/flocculation is meant to remove most of the suspended particles in the water before the water reaches the filters, pre-sedimentation removes most of the sediment in the water during the pre-treatment stage.
- So, pre-sedimentation will reduce the load on the coagulation/flocculation basin and on the sedimentation chamber, as well as reducing the volume of coagulant chemicals required to treat the water.
- In addition, pre-sedimentation basins are useful because raw water entering the plant from a reservoir is usually more uniform in quality than water entering the plant without such a holding basin.

Discrete particle settlement theory

- Any particle which does not alter its size, shape, and weight while rising or settling in any fluid is called discrete particle.
- All the particles having more specific gravity than the liquid will move vertically downward due to gravitation force.
- It will accelerate until the frictional resistance or drag become equal to the gravitational forces acting upon the particle.
- At such stage the particle will settle at uniform velocity.
- This uniform velocity is called Settling velocity.

Archimedes' principle states that the upward buoyant force that is exerted on a body immersed in a fluid, whether fully or partially submerged, is equal to the weight of the fluid that the body displaces.

In fluid dynamics, drag is a force acting opposite to the relative motion of any object moving with respect to a surrounding fluid. This can exist between two fluid layers or a fluid and a solid surface.

For gravity settling, this means that the particles will tend to fall to the bottom of the vessel, forming a slurry at the vessel base.

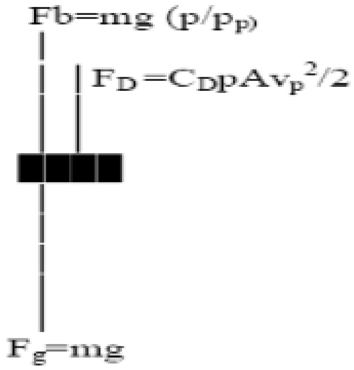


Figure: Forces which are subjected on settling discrete particle

- The impelling (gravitational) force at uniform settling velocity is equal to the effective weight of the particle in the fluid.

i.e. $F_I = (p_s – p) * g * V$ (1)

Where

F_I =Impelling force

p_s = mass density of the particle

p = mass density of the fluid

V=Volume of the particle

g=Acceleration due to gravity

From the Newton‟s law for frictional resistance or drag

$F_D = C_D pAvp^2/2$ (2)

Where

F_D = drag force

C_D = drag coefficient

A = projected area of the particle

v = relative velocity of particle and fluid

Equating equation 1 & 2 and putting the value of $\pi d^2/4$ for A , $\pi d^3/6$ for V.

$$\frac{mg}{\rho_p}(\rho_p - \rho) = C_D \rho \frac{v_p^{\,2}}{2} A_p$$

$$v_p^2 = \frac{2g}{C_D}\frac{(\rho_p - \rho)}{\rho}\frac{V}{A_p}$$

$$\frac{V}{A_p} = \pi\frac{d^3}{6} x \frac{4}{\pi d^2} = 2\frac{d}{3}$$

$$v_p = [\frac{4}{3}\frac{gd}{C_D}[(\frac{\rho_p - \rho}{\rho})]^{\frac{1}{2}}$$

C_d *is a function of the R'eynolds Number*

$$For\ R_N < 0.5 \quad C_D = \frac{24}{R_N}$$

$$v_p = \frac{gd^2}{18\nu}\frac{(\rho_p - \rho)}{\rho}$$

$$otherwise\ ;\ C_D = \frac{24}{R_N} + \frac{3}{\sqrt{R_N}} + 0.34$$

To avoid interpolation Camp‟s dimensionless graph can be used. The basis of the graph is

$$\frac{C_D}{R_N} = \frac{4}{3}\frac{(\rho_p - \rho)g\mu}{\rho^2 v_p^3}$$

$$C_D R_N^2 = \frac{4}{3}\frac{\rho(\rho_p - \rho)g\, d^3}{\mu^2}$$

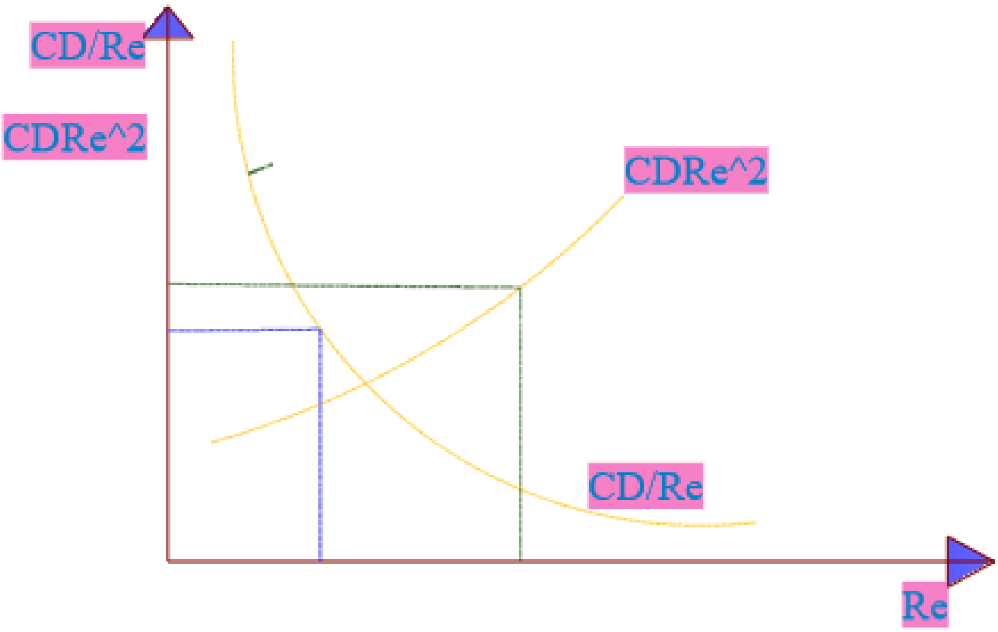


Figure: Camp's dimensionless graph

Example 1

Find the terminal settling Velocity of a spherical Particle with diameter of 0.5mm and a specific gravity of 2.65 settling through water (at 20^{0}C).

$\mu = 1.002*10^{-3} Ns/m^2, \rho w = 1000 Kg/m^3$

Solution

Given

D=0.5mm

Sg=2.65

$\mu = 1.002*10^{-3} Ns/m^2,$
$\rho w = 1000 Kg/m^3$

$$Sg = \frac{\rho p}{\rho w} \Rightarrow \rho p = Sg * \rho w = 998.2 * 2.65$$
$$\approx 2650 Kg/m^2$$

Assume the flow is Laminar

$$Vp = \frac{gd^2[\rho p - \rho w]}{18\mu} = \frac{9.81*[\ 0.5*10^{-3}\ ]^2[2650-998.2]}{18*1.002*10^{-3}}$$

= 0.22m/sec

Check the flow is laminar or not

$$CD = \frac{24}{\text{Re}} \quad , \quad \text{Re} = \frac{\rho w * Vp * d}{\mu} = \frac{998.2*0.22*0.5*10^{-3}}{1.002*10^{-3}} = 112$$

(Not!)

The flow is transitional flow

$$C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34 = \frac{24}{112} + \frac{3}{\sqrt{112}} + 0.34 = 0.84$$

$$Vp = \sqrt{\frac{4gd}{3CD} * \frac{(\rho p - \rho w)}{\rho w}}$$ =0.111m/sec

Check Re

$$\text{Re} = \frac{\rho w * Vp * d}{\mu} = \frac{998.2*0.11*0.5*10^{-3}}{1.002*10^{-3}} = 55$$

(Ok!)

Therefore Vp = 0.11m/sec

Example 2

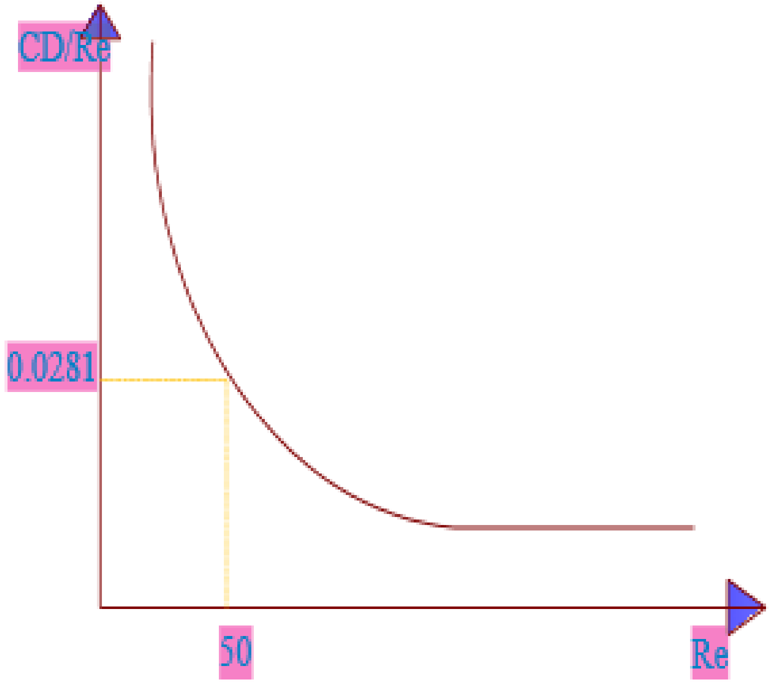
Find the diameter of the particle that has a settlement velocity of 0.1m/sec. Density of particle = 2650kg/m^3 and density of water = 1000kg/m^3.

$$\mu = 1.3X10^{-3}\ Ns/m^2$$

$$\frac{C_D}{R_N} = \frac{4}{3}\frac{1650}{10^6}\frac{9.81X1.3X10^{-3}}{10^{-3}} = 0.0281$$

From the curve R_N _ 50 for $\frac{C_D}{R_N} = 0.0281$

$$d = \frac{50X1.3x10^{-3}}{0.1x10^3} = 0.65x10^{-3} = 0.65mm$$



Allen's Law at intermediate Reynolds number

$$Vp = \frac{0.153 * g^{0.71} * d^{1.14}(\rho p - \rho w)}{\rho w^{0.29} \mu 0.43}$$

Example 3

Do Example 1 by this method (Ans. = 0.14m/s)

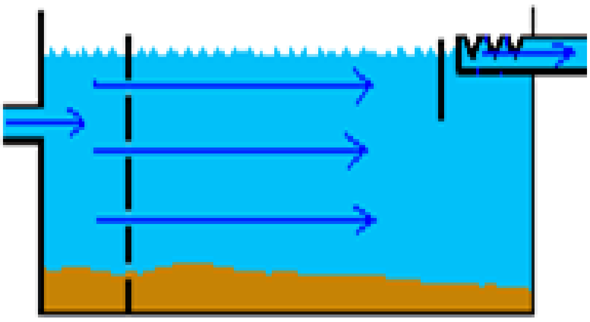
Sphericity Factor

V/Ap < 2d/3 for particles in water Phi = sphericity factor = ds/d (average) ds is the sphere diameter having the same settlement velocity as the particle of diameter da (The average size of the particular range can be found from the geometric mean size of sieves passing and retaining the sample.

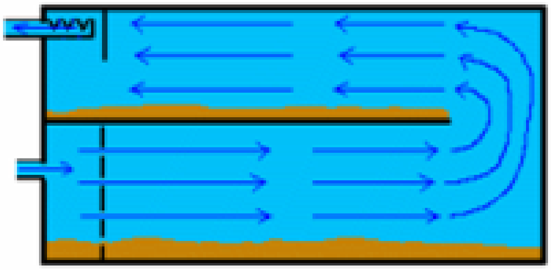
Sedimentation Tank

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

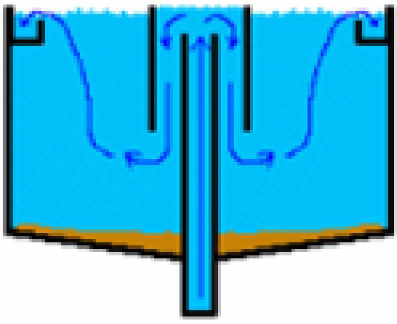
- The vertical flow type is generally used for sewage treatment.
- Three common types of sedimentation basins are shown below:



  - Rectangular basins are the simplest design, allowing water to flow horizontally through a long tank.
  - This type of basin is usually found in large-scale water treatment plants.
  - Rectangular basins have a variety of advantages - predictability, cost-effectiveness, and low maintenance.
  - In addition, rectangular basins are the least likely to short-circuit, especially if the length is at least twice the width.
  - A disadvantage of rectangular basins is the large amount of land area required.



  - Double-deck rectangular basins are essentially two rectangular sedimentation basins stacked one on top of the other.
  - This type of basin conserves land area, but has higher operation and maintenance costs than a one-level rectangular basin.



  - Square or circular sedimentation basins with horizontal flow are often known as **clarifiers**.
  - This type of basin is likely to have short-circuiting problems.

- A forth type of sedimentation is more complex.
- Solids-contact clarifiers also known as up flow solids-contact clarifiers up flow sludge-blanket clarifiers combine coagulation, flocculation and sedimentation within a single basin.

- Solids-contact clarifiers are often found in packaged plants and in cold climates.
- This type of clarifier is also often used in softening operation.
- All sedimentation basins have four zones - the inlet zone, the settling zone, the sludge zone, and the outlet zone (Fig. below).
- Each zone should provide a smooth transition between the zone before and the zone after.
- In addition, each zone has its own unique purpose.

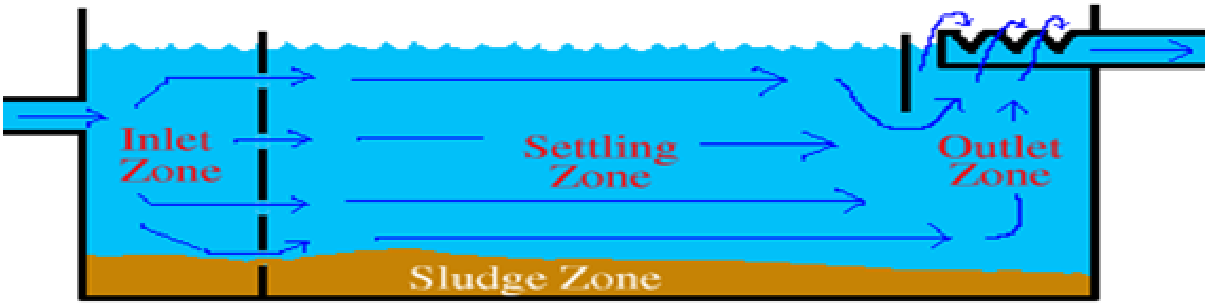


Figure: Zone of a settling tank

- In a clarifier, water typically enters the basin from the center rather than from one end and flows out to outlets located around the edges of the basin.
- But the four zones can still be found within the clarifier:

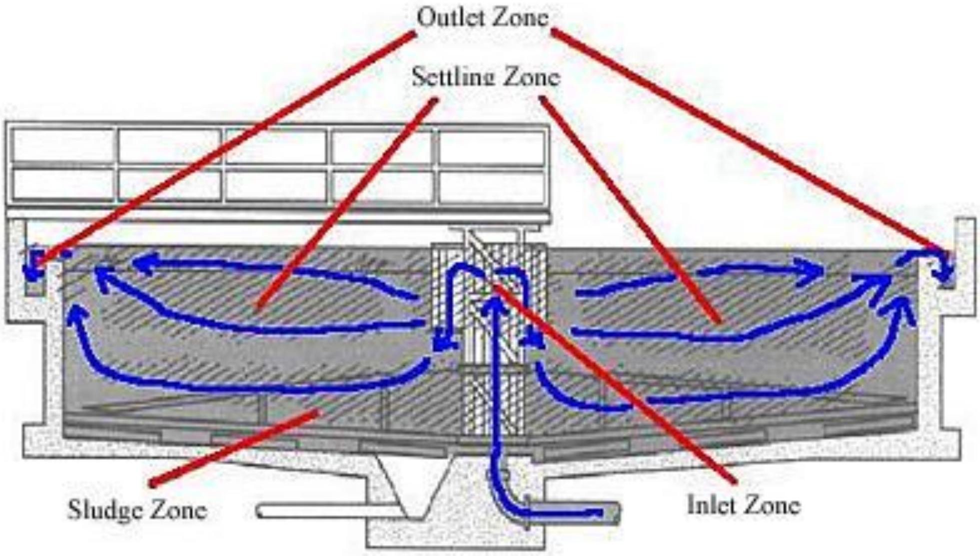


Figure: Clarifier

Inlet Zone

- The two primary purposes of the inlet zone of a sedimentation basin are to distribute the water and to control the water's velocity as it enters the basin.
- In addition, inlet devices act to prevent turbulence of the water.
- The incoming flow in a sedimentation basin must be evenly distributed across the width of the basin to prevent short-circuiting.
- Short-circuiting is a problematic circumstance in which water bypasses the normal flow path through the basin and reaches the outlet in less than the normal detention time.
- In addition to preventing short-circuiting, inlets control the velocity of the incoming flow.
- If the water velocity is greater than 0.5 ft/sec., then floc in the water will break up due to agitation of the water.
- Breakup of floc in the sedimentation basin will make settling much less efficient.
- Two types of inlets are shown below.
- The stilling wall, also known as a perforated baffle wall, spans the entire basin from top to bottom and from side to side.
- Water leaves the inlet and enters the settling zone of the sedimentation basin by flowing through the holes evenly spaced across the stilling wall.

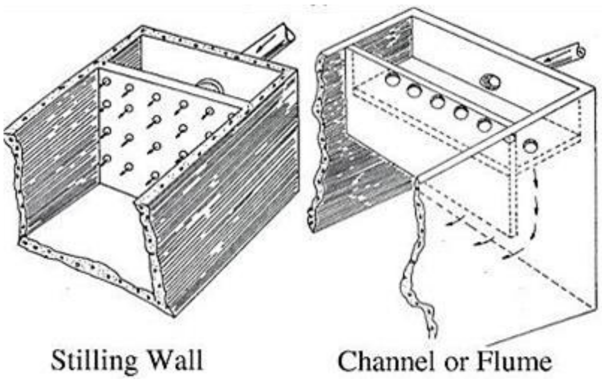


Figure: Types of Inlet
Source: Water Works Operator's Manual

- The second type of inlet allows water to enter the basin by first flowing through the holes evenly spaced across the bottom of the channel and then by flowing under the baffle in front of the channel.
- The combination of channel and baffle serves to evenly distribute the incoming water.

Settling Zone

- After passing through the inlet zone, water enters the settling zone where water velocity is greatly reduced.
- This is where the bulk of floc settling occurs and this zone will make up the largest volume of the sedimentation basin.
- For optimal performance, the settling zone requires a slow, even flow of water.
- The settling zone may be simply a large expanse of open water.
- But in some cases, tube settlers and lamella plates, such as those shown below, are included in the settling zone.

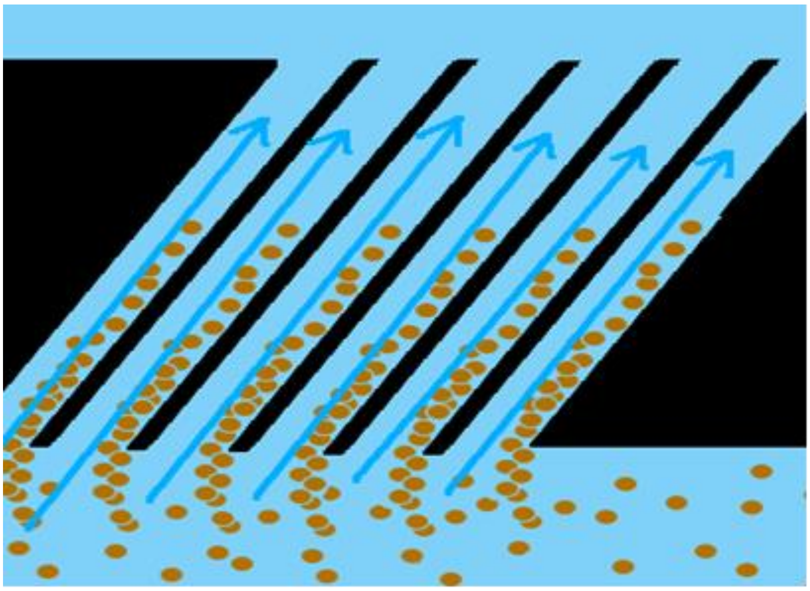


Figure: Settling zone

- In a tube settlers and lamella plates, water flows up through slanted tubes or along slanted plates.

- Floc settles out in the tubes or plates and drifts back down into the lower portions of the sedimentation basin.
- Clarified water passes through the tubes or between the plates and then flows out of the basin.
- Tube settlers and lamella plates increase the settling efficiency and speed in sedimentation basins.
- Each tube or plate functions as a miniature sedimentation basin, greatly increasing the settling area.
- Tube settlers and lamella plates are very useful in plants where site area is limited, in packaged plants, or to increase the capacity of shallow basins.

Determining the capacity of the settling zone

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

Let

L=Length of the settling zone

W=Width of the tank

H=depth of the tank

C=Capacity of the tank

T=time of horizontal flow(detention time)

V=Horizontal velocity of flow

Q=Discharge of flow

v= velocity of settlement of a particular particle

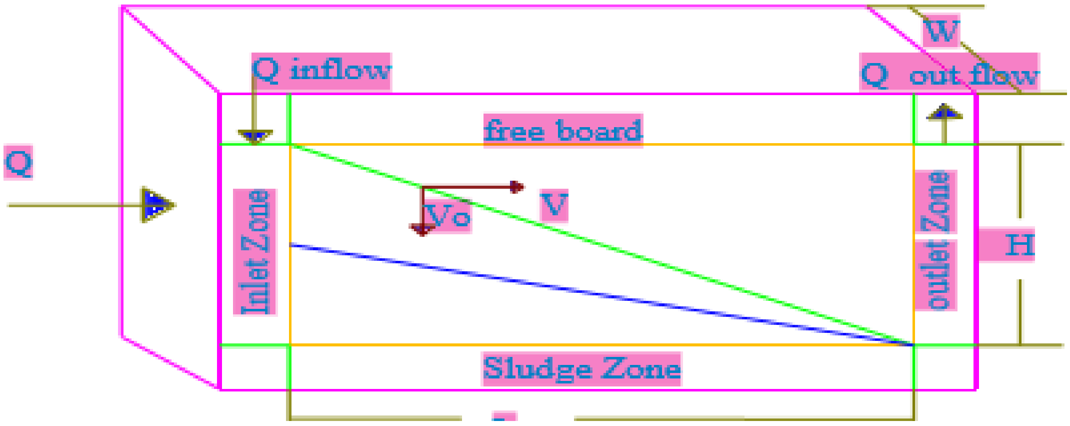


Figure: Horizontal flow settling tank with continuous flow

Detention time, T=L/V=Capacity/Q =L*W*H/Q (3)

But

T=Depth of the tank /velocity of the particle = H/v (4)

Equating (3) & (4), we get

L/V= =L*W*H/Q =H/v

v/V = H/L (5)

v =Q/L*W =Q/A (6)

Equation (6) shows that the velocity of settlement of the particle is independent on the depth of the tank, and it, inversely, varies as the surface area of the tank.

- This formula also reveals that all the particle whose velocity of settlement is either equal to or greater than V=Q/A will reach the sludge zone before the outlet end of the tank.
- All other particles with settlement velocity Vo will be removed in the proportion Vx/Vo.
- Assuming that Vx is distributed uniformly over the vertical distance H.

$$X_T = 1 - X_o + \int_{X_o}^{X} \frac{v_x}{v_o} dx$$

X_T = Total removal of particles in the tank Xo is the fraction of particles with velocity V < Vo.

A batch settlement process is used to determine the integral fraction.

Example

An experimental test was done to determine the total removal of particle in the tank by using a batch settlement process. The result of the batch settlement data is given in the following table.

Results of Batch Settlement

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

Weight fraction Xo	Weight Fraction dxi	Corresponding Velocity Vxi (mm/sec)	Product Vxidxi mm/sec.
0.55	0.05	0.63	0.315
	0.10	0.44	0.44
	0.1	0.26	0.026
	0.1	0.15	0.015
	0.1	0.06	0.006
	0.1	0.02	0.002
			$\sum Vxidxi = 0.125$

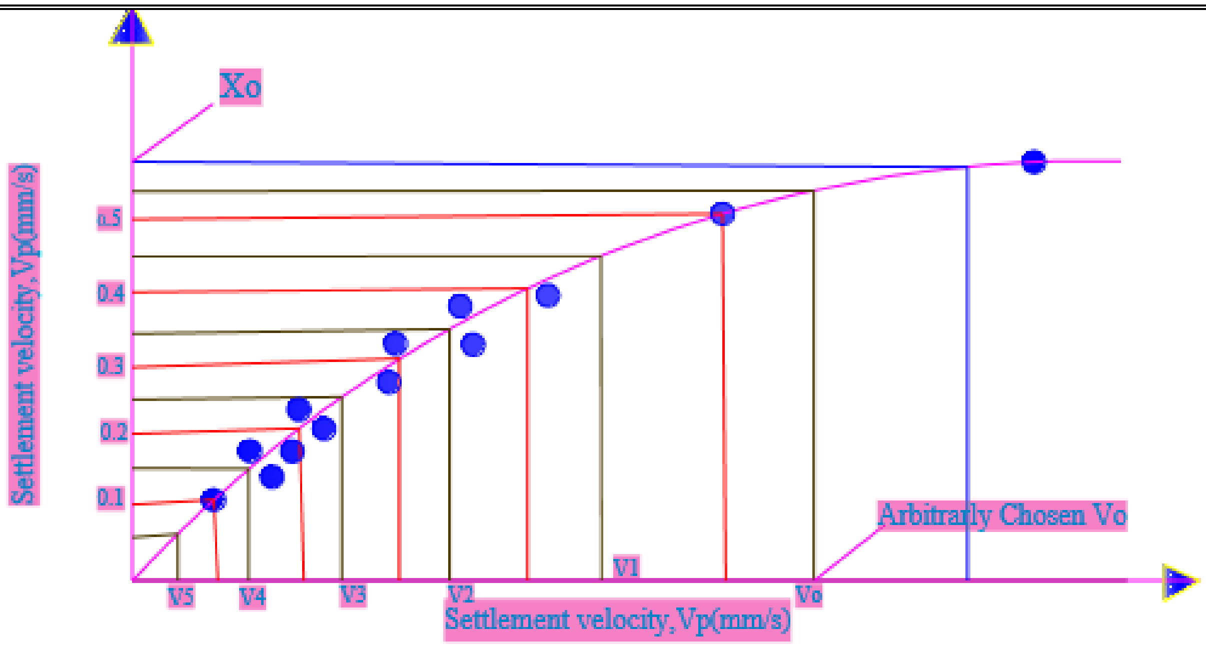


Figure: Fraction of particle Vs Settlement velocity

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

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

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

$$= 0.63$$

The predicted settlement is = 63%

Up flow Tanks

- The upward flow velocity is Q/A and therefore all particles with V<Vo will have a down ward resultant velocity.
- Hence the removal fraction will be: X_T = 1-Xo Where Xo is the reaction by weight with v less than or equal to Vo.

Flocculating Material

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

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

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

$$(\rho_f - \rho) = K_3 A_p^{-0.338}$$

r_f = Floc radius

η = Number of particles

A_p = The projected area of a spherical particle

Ω = Volume fraction of solids

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

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

Outlet Zone

- The outlet zone controls the water flowing out of the sedimentation basin - both the amount of water leaving the basin and the location in the basin from which the out flowing water is drawn.
- Like the inlet zone, the outlet zone is designed to prevent short-circuiting of water in the basin.
- In addition, a good outlet will ensure that only well-settled water leaves the basin and enters the filter.
- The outlet can also be used to control the water level in the basin.
- Outlets are designed to ensure that the water flowing out of the sedimentation basin has the minimum amount of floc suspended in it.
- The best quality water is usually found at the very top of the sedimentation basin, so outlets are usually designed to skim this water off the sedimentation basin.
- A typical outlet zone begins with a baffle in front of the effluent.
- This baffle prevents floating material from escaping the sedimentation basin and clogging the filters.
- After the baffle comes the effluent structure, which usually consists of a launder, weirs, and effluent piping.
- A typical effluent structure is shown below:

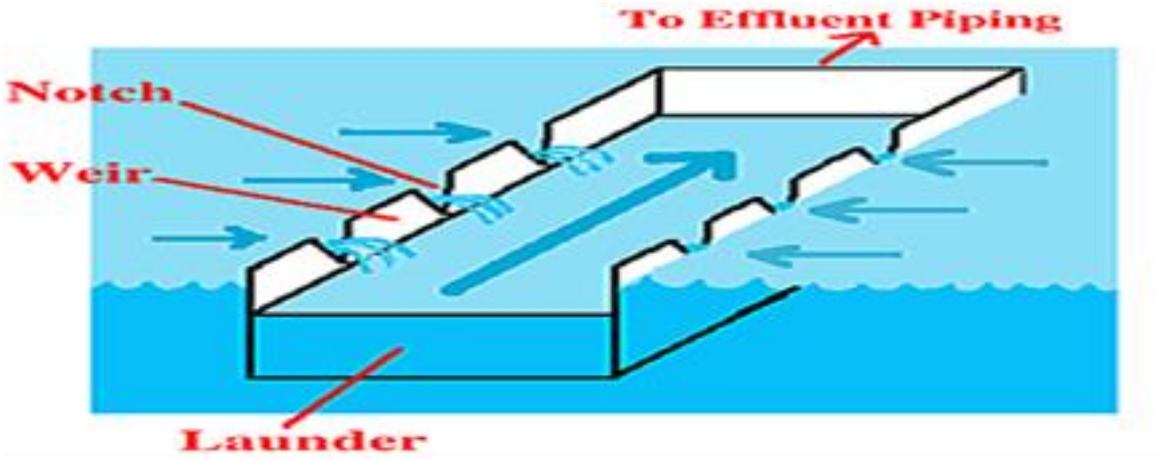


Figure: Outlet Zone

- The primary component of the effluent structure is the effluent launder, a trough which collects the water flowing out of the sedimentation basin and directs it to the effluent piping.
- The sides of a launder typically have weirs attached.
- Weirs are walls preventing water from flowing uncontrolled in to the launder.
- The weirs serve to skim the water evenly of the tank.
- A weir usually has notches, holes, or slits along its length.
- These holes allow water to flow in to the wire.
- The most common type of hole is the V-shaped notch shown on the picture above which allows only the top inch or so of water to flow to out of the sedimentation basin.
- Conversely, the weir may have slits cut vertically along its length, an arrangement which allows for more variation of operational water level in the sedimentation basin.
- Water flows over or through the holes in the weirs and in to the launder.
- Then the launder channels the water to out let, or effluent pipe.
- This pipe carries water away from the sedimentation basin and to the next step in the treatment process.
- The effluent structure may be located at the end of a rectangular sedimentation basin or around the edges of the circular clarifier.
- Alternatively, the effluent may consist of a finger weir, an arrangement of launders which extend out in to the settling basin as shown below.

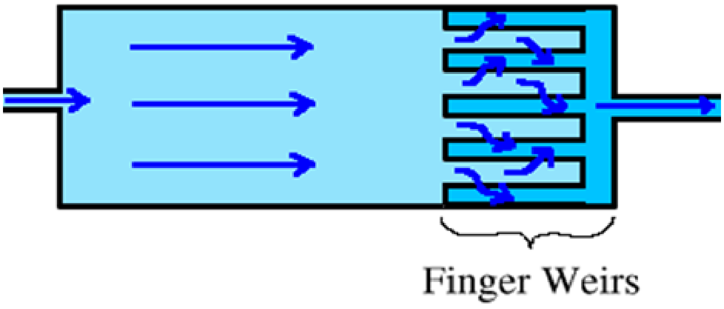


Figure: Finger Weir

Sludge Zone

- The sludge zone is found across the bottom of the sedimentation basin where the sludge collects temporarily.
- Velocity in this zone should be very slow to prevent re-suspension of sludge.
- A drain at the bottom of the basin allows the sludge to be easily removed from the tank.
- The tank bottom should slope toward the drains to further facilitate sludge removal.
- In some plants, sludge removal is achieved continuously using automated equipment.
- In other plants, sludge must be removed manually.
- If removed manually, the basin should be cleaned at least twice per year or more often if excessive sludge buildup occurs.
- It is best to clean the sedimentation basin when water demand is low.
- Many plants have at least two sedimentation basins so that water can continue to be treated while one basin is being cleaned, maintained, and inspected.
- If sludge is not removed from the sedimentation basin often enough, the effective (useable) volume of the tank will decrease, reducing the efficiency of sedimentation.
- In addition, the sludge built up on the bottom of the tank may become septic, meaning that it has begun to decay an aerobically.

- Septic sludge may result in taste and odor problems or may float to the top of the water and become scum.
- Sludge may also become re-suspended in the water and be carried over to the filters.

Sludge disposal

- The sludge which is found in the bottom of a sedimentation tank is primarily composed of water.
- The solids in the sludge are mainly excess coagulant, such as alum.
- Alum sludge has a solid concentration of only about 1% when automatically removed from the basin.
- Or about 2% if manually removed (the greater solid concentration of manually removed sludge is due to a small amount of gravity thickening).
- Many options exist for disposal of sedimentation sludge.
- Here we will discuss disposal in streams, sanitary sewers, lagoons and landfills.

Disposal in streams and sewers

- In the past, sludge and back wash water was typically released into streams and other bodies of water.
- However, this practice is becoming much less common and is now well regulated.
- Alternatively, sludge may be piped directly to the sanitary sewer. However, this disposal option also has its disadvantage.
- Sludge can cause sewer blockage.
- In addition, fees charges by the waste water treatment plant can be expensive.
- Releasing sludge in to the sewage lines in large batches makes treatment of the wastewater problematic, so sludge is typically released slowly over a long time period, which requires a large holding tank at the water treatment plant.

Thickening sludge

- Most of the other alternatives require transporting sludge away from the treatment plant.
- Sludge is typically dried before it is tucked away since the greater volume of wet sludge makes it much more expensive to transport.
- This drying process is known as dewatering or thickening.
- Alum sludge is difficult to thicken, but a Variety of devices have been developed to thicken, the sludge some of which are explained below.
- In many cases the sludge is treated by the addition of polymers to aid in the dewatering process, alternatively, the sludge can be heated or frozen and thawed to increase the solids concentration.
- Treating to aid thickening is known as conditioning the sludge.
- Once the sludge has been conditioned, it may be thickened in a lagoon, drying bed, or one of several devices.

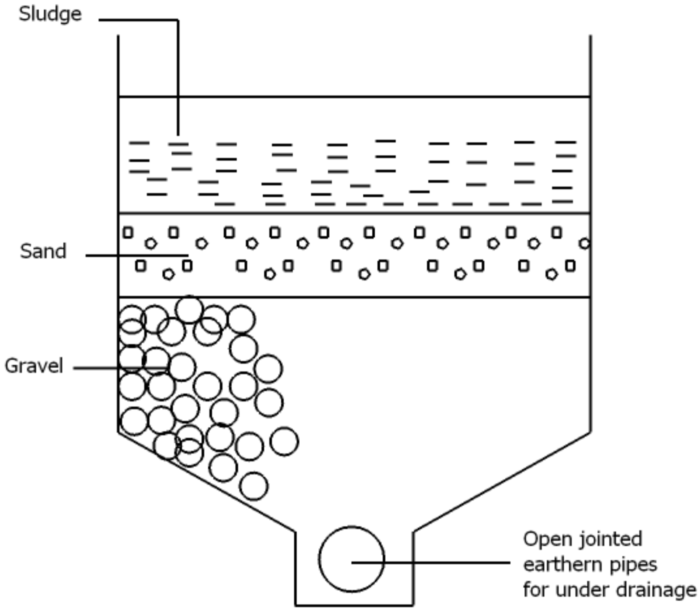
Lagoons

- Which are small-volume storage ponds, are the simplest device used to thicken sludge.
- These lagoons are filled with sludge and the solids are allowed to settle due to gravity to the bottom of the lagoon while the clear water is pumped off the top.

Figure: Lagoons

Drying beds

- Are often used to thicken sludge destined for a landfill more quickly than the sludge would be thickened in a lagoon.
- A drying bed is similar in design to a sand filter, with a layer of sand underlain by a layer of gravel.
- The sludge is applied to the top of the sand and the water percolates down through the sand and gravel and is drained away.



- When the sludge is sufficiently dry, it is carefully removed from the top of the sand and is trucked to a landfill.
- When sludge is preconditioned with chemicals, dewatering in a drying bed may take only a few days or weeks

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

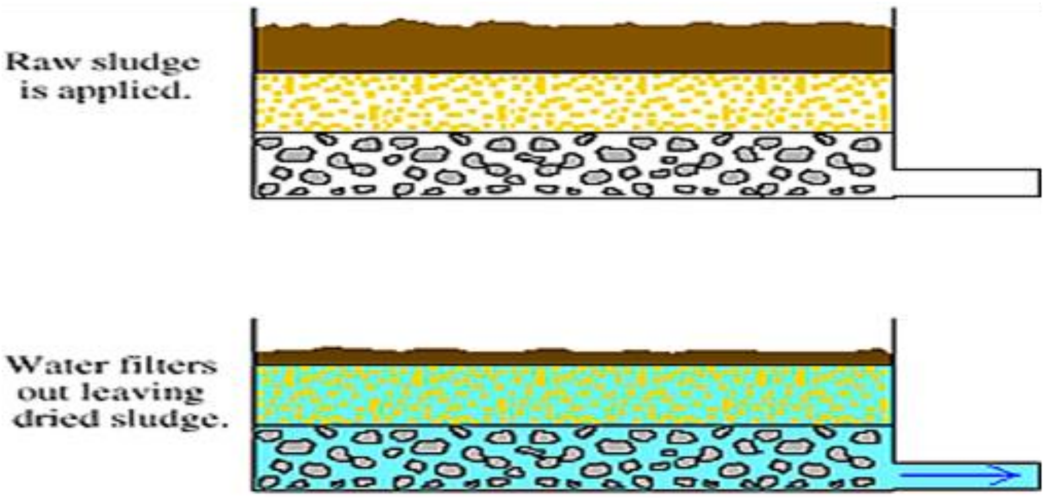


Figure: Drying bed

Factors Influencing Efficiency of the basin

Floc Characteristics

- To a large extent, a sedimentation basin's efficiency will depend on the efficiency of the preceding coagulation/flocculation process.
- The size, shape, and density of the floc entering the sedimentation basin will all influence how well the floc settles out of the water.
- Floc which is too small or too large, is irregularly shaped, or has a low density will not tend to settle out in the sedimentation basin.
- Even if the coagulation/flocculation process is very efficient, floc can disintegrate on its way to or in the sedimentation basin.
- Previously formed floc will disintegrate
    - ✓ If the water velocity is too high,
    - ✓ If there are sharp bends in the pipe at the inlet...

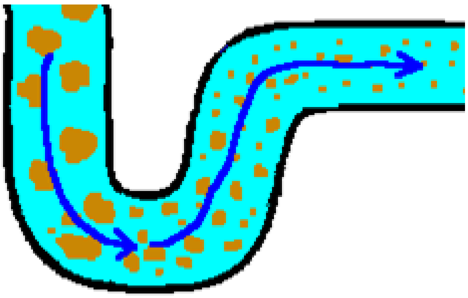


Figure: Sharp bends in the pipe at the inlet

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

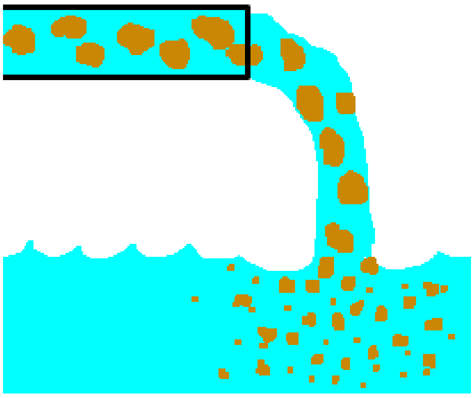


Figure: Water is discharged above the sedimentation basin

Short-circuiting

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

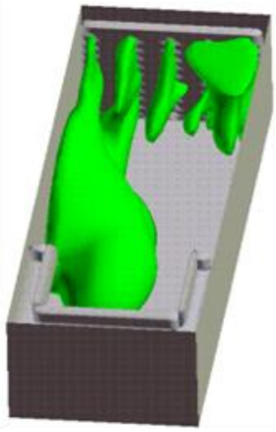


Figure: Short-circuiting

- When water in the sedimentation basin short-circuit, floc does not have enough time to settle out of the water, influencing the economy of the plant and the quality of the treated water.
- Short-circuiting in a sedimentation basin can be detected in a variety of ways.
- If areas of water in the basin do not appear to be circulating, or if sludge build up on the bottom of the basin is uneven, then tested may be called for.
- Floats or dyes can be released at the in-let of the basin to determine current.
- A variety of factors causes short-circuiting in a sedimentation basin. Basin shape and design along with the design of the inlet and out-let, can cause short-circuiting.
- A long, thin sedimentation basin is less likely to short-circuit than is a short broad one.
- Uneven distribution of flow either at the inlet or out-let can also cause short-circuiting.
- If the weir at the out-let is not level or if some of the notches clog, flow will be uneven and will cause short-circuiting.
- In addition to the design of the basin, characteristic of the water can also cause short-circuiting.
- Differences of temperature can cause stratification of the water-separation of water in to bands of different temperature.

- In coming water will tend to flow through the band of water which corresponds to its own temperature, will not spread throughout the rest of the basin.
- We should note that temperature can cause other problems with sedimentation as well.
- Cold water prevents floc from settling, so that longer settling times or larger doses of coagulant chemicals are needed.

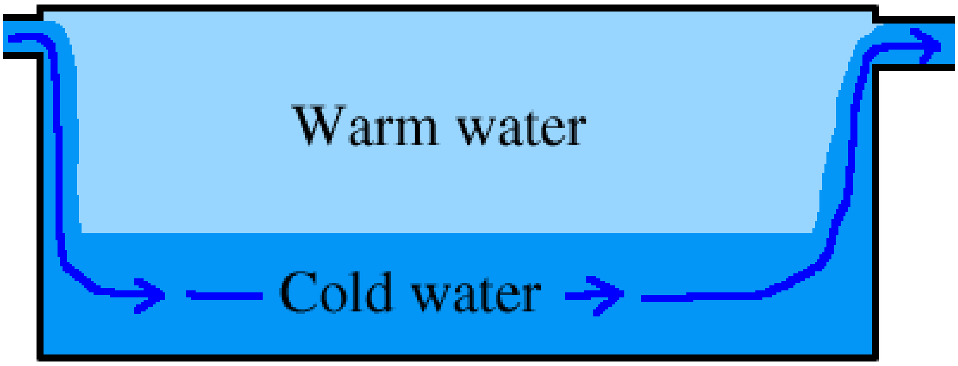


Figure: Effect of temperature

Other Problems

- Gases in the water may cause floating scum, which can carry over into the filters.
- Sprinkling water on the on the scum may cause the scum to settle, but it is usually a better practice to find and fix the source of the problem.
- Gases in the sedimentation basin are usually caused by water being introduced in the pump or by a leak in the raw water line.
- Another sedimentation basin problem is algal growth.
- If sedimentation basins have sufficient sunlight, algae will grow on the walls of the basin.
- These algae can break loose and clog the filter.
- Algae are best treated with shock chlorination, a method of feeding 5-10 ppm of chlorine into the raw water or of sprinkling HTH (High Test Hypochlorite) around the basin walls just before the plant is shut down for a few hours.

- The chlorine will kill the algae while the chlorinated water sits in the tank.
- A few other factors can also influence sedimentation basin efficiency.
- Intermittent operation of the basin can cause settling problems.
- Also, design problems such as excessive surface loading or weir loading can cause problems.

Designing a Rectangular Sedimentation Tank

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

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

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

Design of Horizontal Flow Tanks

- A depth of at least 3m is provided to allow for sludge storage.
- For a given flow rate, the remaining variable to determine will be either the plan area or the detention time.
- It is possible to run settling column tests to determine the expected percentage removal for a given detention period.

- If a well-defined zone of clear water is established, then there will not be a need for Flocculation.

- A factor of 1.5 to 2 should be applied (the detention time increased by 1.5 to 2) to allow for short-circuiting caused by wind temperature differences, etc.

- The following design guidelines are recommended.

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

Inlet Arrangement

- The flow should be distributed as uniformly as possible beyond the inlet zone.

- Weirs, Orifices as well as perforated baffles can be provided to induce uniform distribution. (Diagram below).

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

1. The velocity through the ports should be 4 times higher than any approaching velocities in order to equalize flows.
2. To avoid braking up flocs, the velocity gradient should be held to a value close to or a little higher than that in the last flocculator.
3. The maximum feasible number of ports should be provided to minimize the length of the turbulent entry zone.

4. The port configuration should be such as to assure that the discharge jets will direct the flow towards the basin outlet.

Outlet Arrangement

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

$$\boxed{L = 0.2Q / (H.Vs)}$$

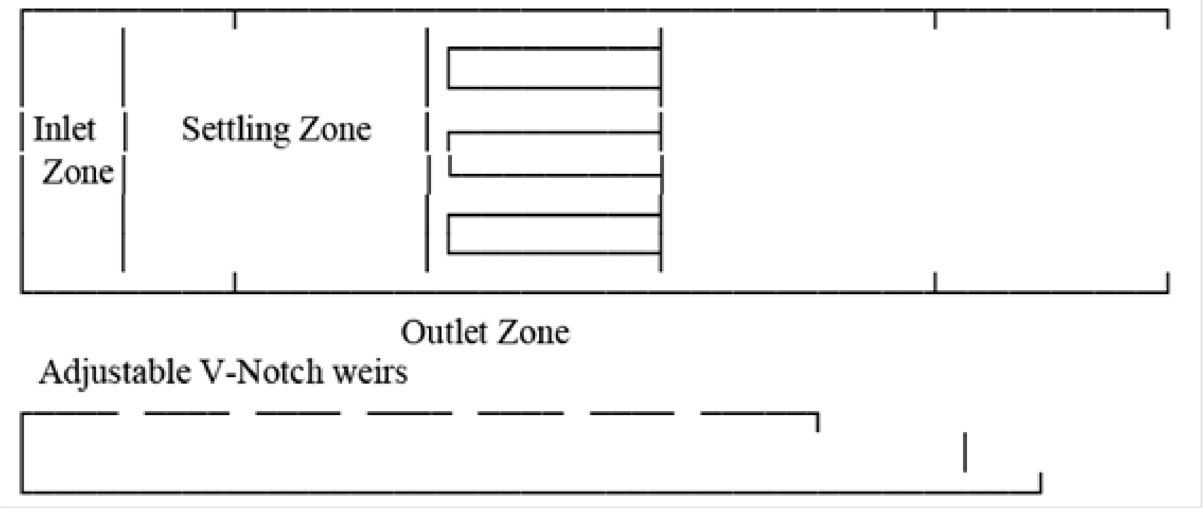
Where:

H = Depth of the tank

Vs = Settling velocity (m/day)

Q = Flow rate

L = Combined weir length



Specifications

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

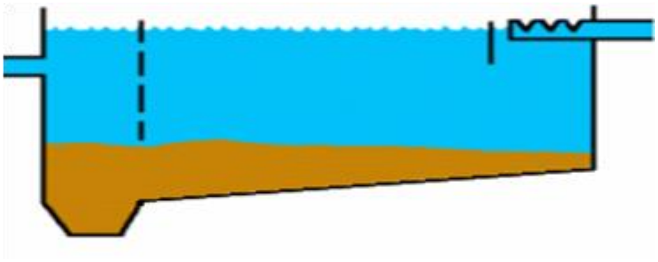


Figure: A rectangular sedimentation basin

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

Over view of calculations

- We will determine the surface area, dimensions, and volume of the sedimentation tank as well as the weir length.
- The calculations are as follows:
  1. Divide flow in to at least two tanks.
  2. Calculate the required surface area.
  3. Calculate the required volume.
  4. Calculate the tank depth.
  5. Calculate the tank width and length.
  6. Check flow through velocity.
  7. If velocity is too high, repeat calculations with more tanks.
  8. Calculate the weir length.

i. Divide the flow

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

$$\boxed{Q_C = \frac{Q}{n}}$$

Where:

Qc = flow in one tank

Q = total flow

n = number of tanks

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

QC=1.5MGD/3

QC=0.5MGD

ii. Surface area

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

A=Qc/O.R

Where:

A=surface area,ft^2 Qc=flow, gal/day

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

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

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

A=1,000ft^2

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

iii. Volume

- The tank volume is calculated just as it was for flocculation basins and flash mix chambers, by multiplying flow by detention time.
- The optimal detention time for sedimentation basins depends on whether sludge removal is automatic or manual.
- When sludge removal is manual, detention time should be 6 hours.
- We will consider a tank with automatic sludge removal, so the detention time should 4 hours.
- The volume of one our tanks is calculated as follows:

$$\boxed{V=Qt}$$

$V=(500{,}000\text{gal/day})(4\text{hr})(1\text{day}/24\text{hr})(1\text{ft}^3/7.48\text{gal})$ $V=11{,}141\text{ft}^3$

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

iv. Depth

- The tank's depth is calculated as follows:

$$\boxed{d=V/A}$$

Where:

d=depth, ft

V=volume,ft^3

A= surface area, ft^2

- For our example, the depth is calculated to be:

$d=(11{,}141\text{ft}^3)/(1{,}000\text{ft}^2)$

$d = 11.1$ ft

- The specifications note that the depth should be between 7 and 16 feet.

- Our calculated depth is within the recommended range.
- If the depth was too large, we would begin our calculations again, using a larger number of tanks.
- If the depth is too shallow, we would use a smaller number of tanks.

v. Width and Length

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

$$\boxed{V = LWd}$$

Where:

V=volume

L= length

W=width

d=depth

- For our tank, the length has been defined as follows: L= 4W
- Combining these two formulas, we get the following formula used to calculate the width of our tank: in the case of our example, the tank width is calculated as follows:

W=15.8ft

- The length is calculated as:

L = 4(15.8ft)

L= 63.2ft

6 and 7.

Flow through velocity

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

$$\boxed{A_x = Wd}$$

$Ax = (18.5ft)(11.1ft)$

$Ax = 175.4ft^2$

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

$$\boxed{V=Qc/Ax}$$

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

$V=0.26ft/min$

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

Weir length

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

$$\boxed{Lw = Qc/W.L}$$

Where:

L_w: weir length, ft

Q_c: flow in one tank, gal/day

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

So, in our example, the weir length is calculated as follows:

$L_w = (500{,}000gal/day)/(15{,}000gal/day\text{-}ft)$

$L_w = 33.3ft$

The weir length should be 33.3ft.

Design Elements

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

$$\boxed{T_c = \frac{V}{Q} = \frac{LBH}{Q}}$$

- Flow through periods (td)- the average time required for a batch of water to pass the settling tank.
- Always $T_d <$ To because of short-circuiting.
- Over flow rate or surface loading rate: the quantity of water passes per unit time horizontal area.

$$\boxed{V_s = Q/A}$$

- Basin dimension
    - ➢ Surface area = Volume/surface loading
    - ➢ Length to width ratio minimum of 2:1
        - Preferable 3:1 to 5:1
- ➢ Depth 2.5-5m (inclusive of allowance)
    - ▫ Preferred value = 3m

Example 1

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

- Volume of treated water $=3*10^6$l/s
- td=4hr =240min
- velocity of flow = 10cm/min

Solution

Length of tank = velocity of flow*td

= 0.1*240=24m

Volume of water in 4hr= $\frac{3*10^6*4}{10^3*24}$ = 500m^3

- X sectional area (B*H)=V/L=500/24=20.8m^2
- Assuming the working depth = 3m

  Width of tank = 20.8/3 = 7m

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

  Total depth = 3+1.5 =4.5m

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

  Total length = 24+6 = 30m

- Provide the Sedimentation tank =30*7*4.5

Example 2

Design a sedimentation basin for the following data

i. Flow rate
    - maximum flow rate =10,000m^3/day
    - average flow rate 5790m^3/day
ii. Design parameters
    - td = 4hr
    - L/W=2
    - SOR=10$m^3/m^2/d$
    - Design flow rate to be 25% of maximum flow rate

Solution

- Determine the number of tanks

No. of tanks=100/25 = 4

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

- Determine the area of the sedimentation tank

Area = maximum flow rate/SOR

= 2500/10

= 250m^2

A = 1*w = $2w^2$

W = 11.18m = 11m

L = A/W =23m

- Determine the effective height (H)

H =SOR*td

= 1.67m

- Provide the sludge and the free board zone of 0.5m

Ht=2+0.5+0.5=3m

- Assume the inlet and out-let zone to be to be equal to the effective depth of the tank.
- Total length= 23+2+2= 27m
- Provide the sedimentation tank=27m+11m+3m
- This tank will have a surface area of

A=LxW= 23mx11m= 253m^2

This tank will have a volume of

V=LxWxD= 23x11x3= 759m^3

The flow through velocity will be:

$V=Qc/Ax =2500m^3/d/250m^2 = 10m/d$

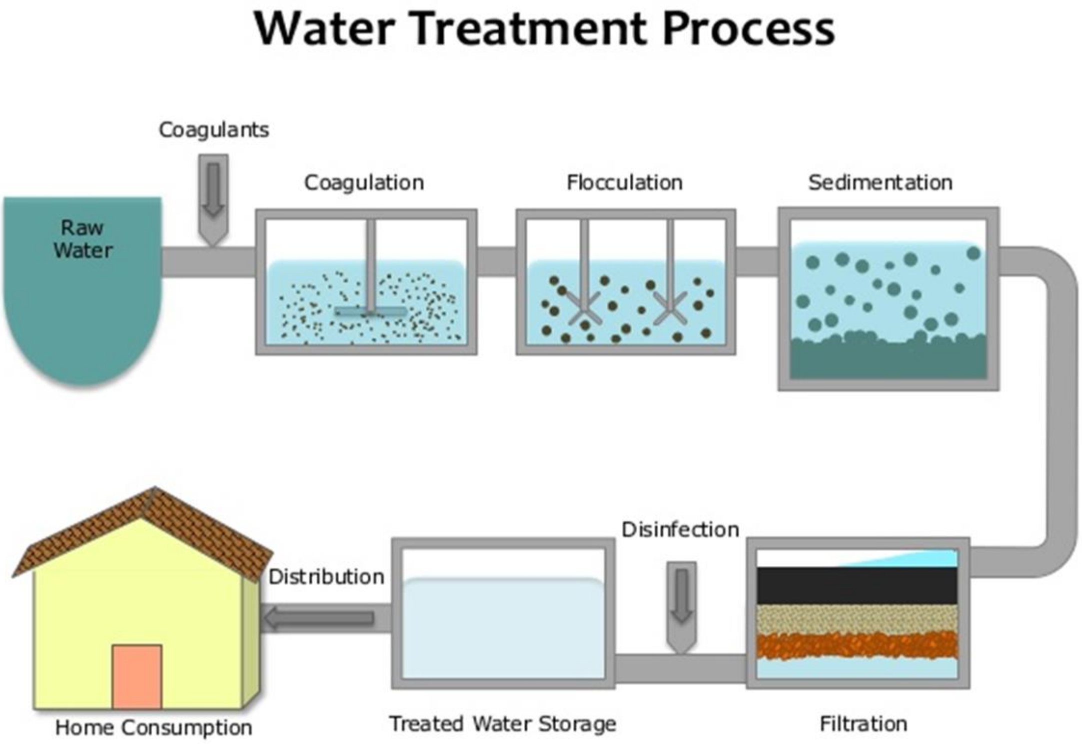
- The weir length will be; Lw = Qc/W.L

New Formulas Used

- To calculate tank surface area: A=Qc/O.R.
- To calculate tank depth: d=V/A
- To calculate width of a rectangular tank where length is four times the width:
- To calculate length of a rectangular tank where length is four times the width: L=4W
- To calculate flow through velocity: V=Qc/Ax
- To calculate weir length; Lw = Qc/W.L

Chapter 4.

4. Coagulation and flocculation



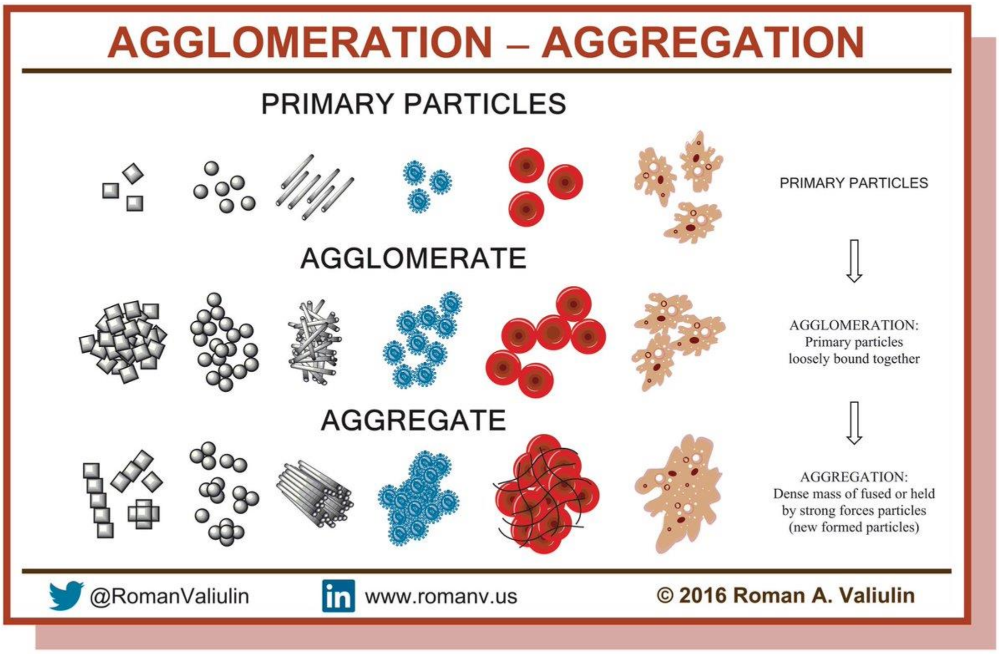
Purpose

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

Figure: Appearance of turbidity

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

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



Additional Benefits of Coagulation

1. The addition of some common coagulants to a wastewater not only produces coagulation of colloids but also typically results in the precipitation of soluble compounds, such as phosphates, that can be present in the wastewater.
    - In addition, coagulation can also produce the removal of particles larger that colloidal particles due to the entrapment of such particles in the flocs formed during coagulation. Coagulation vs. Flocculation
2. Although the words "coagulation" and "flocculation" are often used interchangeably they refer to two distinct processes

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

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

Location in the Treatment Plant

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

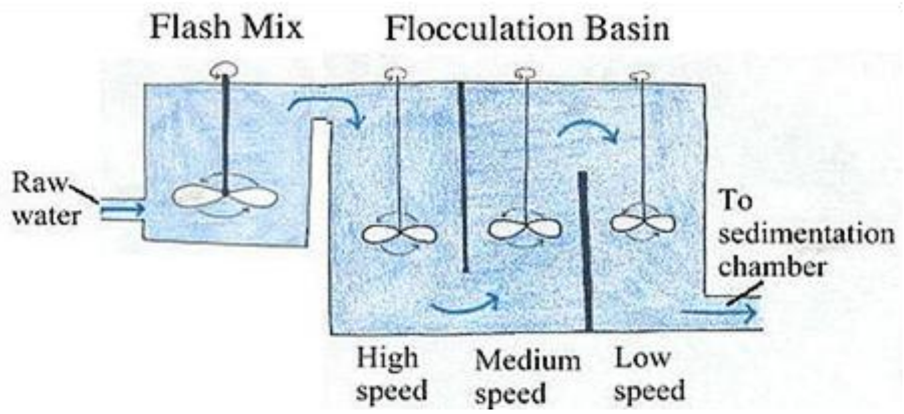


Figure: Flash Mixer and Flocculation Basin

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

Principle of coagulation

- It has been found that when certain chemicals are added to water an insoluble, gelatinous, flocculent is formed.
- This gelatinous precipitate during its formation and descent through the water absorb and entangle very fine suspended matter and colloidal impurities.

- The gelatinous precipitate therefore has the property of removing fine and colloidal particle quickly and completely than by plain sedimentation.
- These coagulants have further the advantage of removing colour, odor and taste from the water.
- These coagulants if properly applied are harmless to the public.
- First the coagulant are mixed in the water to produce the required precipitate, then the water is sent in the sedimentation basins where sedimentation of fine and colloidal particle takes place through the precipitate.

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

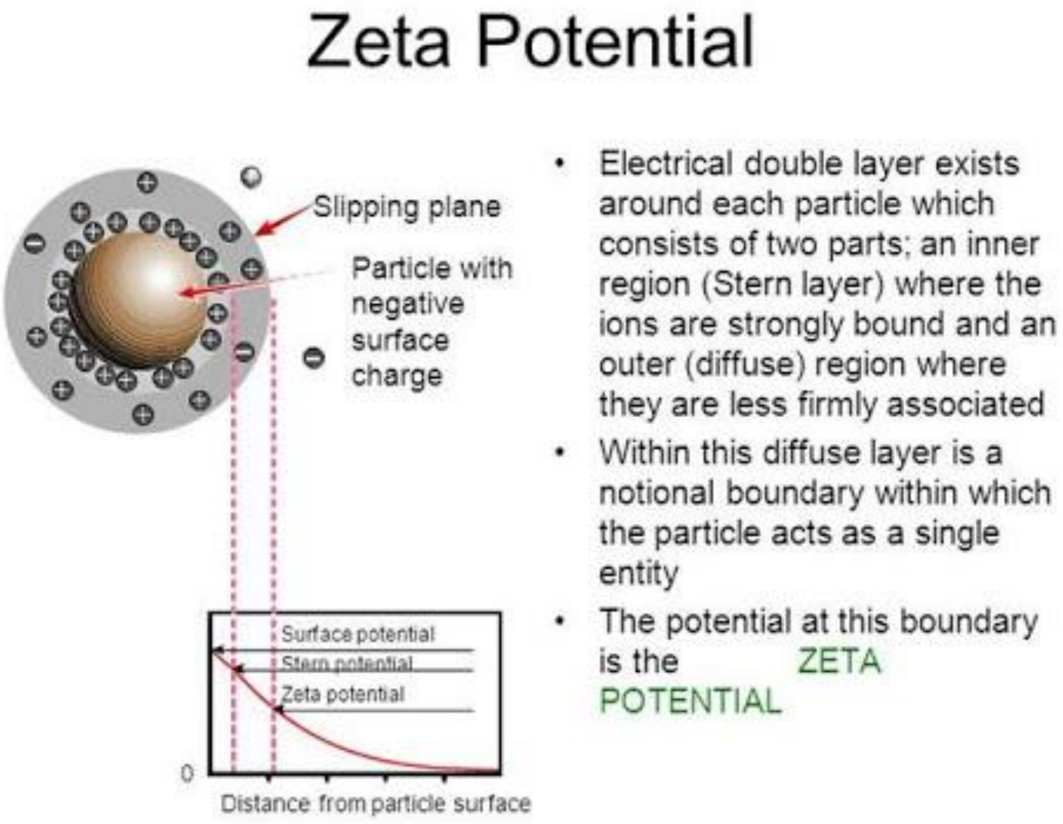
1. Floc formation

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

2. Electric charges

- Most particles dissolved in water have a negative charge, so they tend to repel each other.
- As a result, they stay dispersed and dissolved or colloidal in the water.
- The purpose of most coagulant chemicals is to neutralize the negative charges on the turbidity particles to prevent those particles from repelling each other.
- The amount of coagulant which should be added to the water will depend on the zeta potential, a measurement of the magnitude of electrical charge surrounding the colloidal particles.
- You can think of the zeta potential as the amount of repulsive force which keeps the particles in the water.

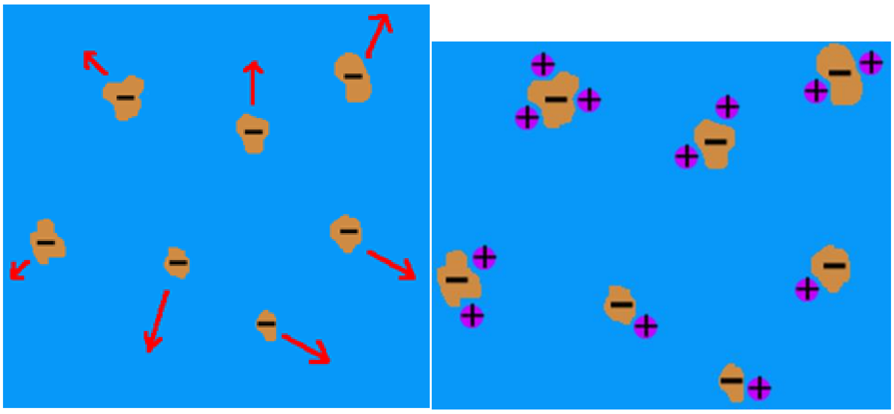
- If the zeta potential is large, then more coagulants will be needed.



Zeta Potential

- Electrical double layer exists around each particle which consists of two parts; an inner region (Stern layer) where the ions are strongly bound and an outer (diffuse) region where they are less firmly associated
- Within this diffuse layer is a notional boundary within which the particle acts as a single entity
- The potential at this boundary is the ZETA POTENTIAL

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



Negatively charged particles repel each other due to electricity.

Positively charged coagulants attract to negatively charged particles each other due to electricity

Figure: Negatively charged particle and positively charged coagulants

- The combination of positive and negative charge results in a neutral.
- As a result, the particles no longer repel each other.
- The next force which will affect the particles is known as van der Waal's forces.
- Van der Waal's forces refer to the tendency of particles in nature to attract each other weakly if they have no charge.

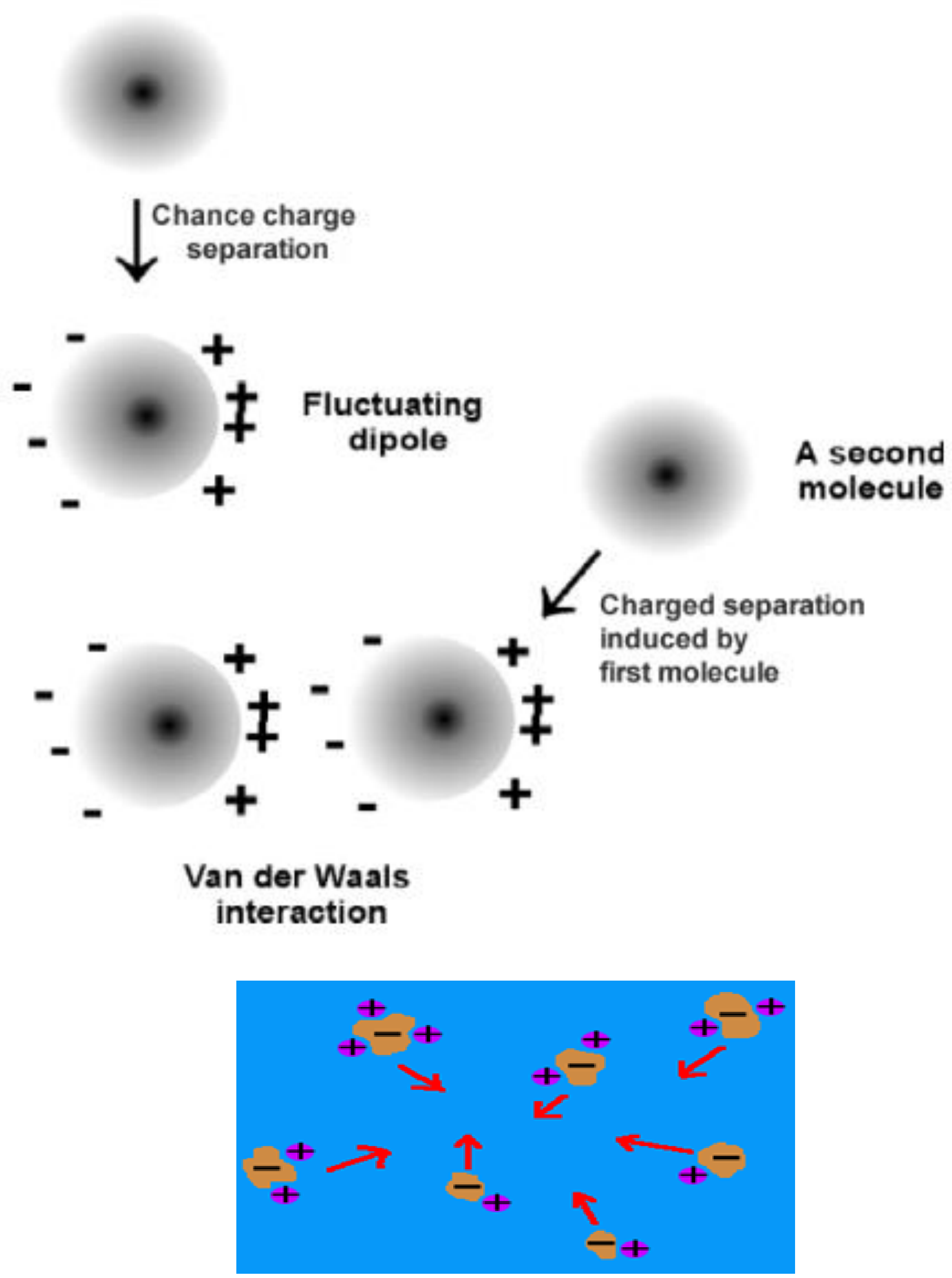


Figure: Neutrally charged particles attract due to van der Waal's forces.

- Once the particles in water are not repelling each other, van der Waal's forces make the particles drift toward each other and join together into a group.

- When enough particles have joined together, they become floc and will settle out of the water.

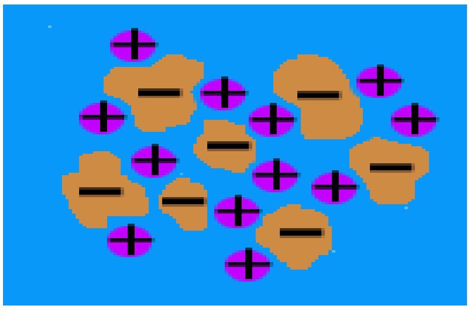
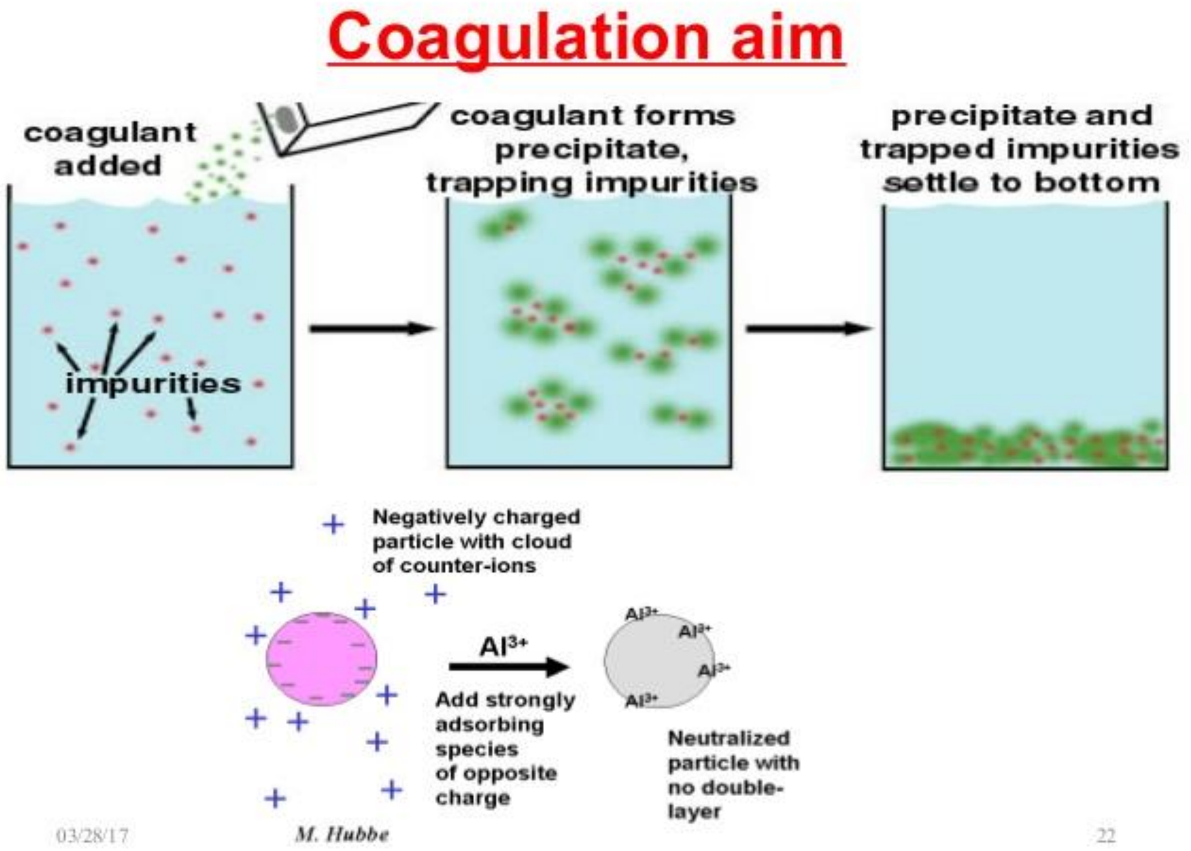


Figure: Particles and coagulants join together into floc



Coagulation aim

Factors affecting coagulation

1. Type of coagulant

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

Common Coagulants

- Coagulant chemicals come in two main types - primary coagulants and coagulant aids.
- Primary coagulants neutralize the electrical charges of particles in the water which causes the particles to clump together.
- Coagulant aids add density to slow-settling flocs and add toughness to the flocs so that they will not break up during the mixing and settling processes.
- In water treatment plants, the following are the coagulants most commonly used:

i. Aluminum sulfate [$Al_2(SO_4)_3.18H_2O$].

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

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

ii. $Al_2(SO_4)_3.18H_2O + 3Ca(OH)_2 \rightarrow 2Al(OH)_3\downarrow + 3CaSO_4 + 18H_2O$

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

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

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

ii. Sodium aluminates ($Na_2Al_2O_4$)

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

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

ii) $Na_2Al_2O_4 + CaSO_4 \rightarrow CaAl_2O_4\downarrow + Na_2SO_4$

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

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

iii. Chlorinated Copperas

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

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

- Ferric sulphate and Ferric chloride each is an effective floc and so also their combination.
- Both Ferric sulphate and Ferric chloride can be used independently with lime as a coagulant
- If alkalinity is insufficient, lime is added.
- Chemical reaction taking place
- $2FeCl_3 + 3Ca(OH)_2 \rightarrow 2Fe(OH)_3\downarrow + CaCl_2$
- $Fe_2(SO_4)_3 + 3Ca(OH)_2 \rightarrow 2Fe(OH)_3\downarrow + 3CaSO_4$
- Ferric chloride effective pH range 3.5 – 6.5 or above 8.5 and Ferric sulphate is effective with pH range of 4 – 7 or above 9.

iv. Poly electrolytes

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

Example

Find out the quantity of alum required to treat 18million liters of water per day. The dosage of alum is 14mg/lit. Also work out the amount of CO2 released per liter of treated water.

Solution

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

- The chemical reaction as follow:

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

- Molecular weight of alum:

  =2*26.97+3*32.066+36*1.008+30*16 =666

- Molecular weight of CO_2=(1*12.0)+2*16) = 44

- Thus 666mg of alum release 6*44mg of CO_2 14mg of alum will releases

  = (14*6*44)/666 =5.55mg of CO_2

Exercise

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

$Fe\,SO_4 .7H_2O + Ca\,(OH)_2 \longrightarrow Fe(OH_2) + CaSO_4 + 7H_2O + Ca(OH)_2 + CaO + H_2O$

Feeding of coagulant

- In order to feed chemicals to the water regularly and accurately, some type of feeding equipment must be used.
- Coagulants may be put in raw water either in powder form or in solution form.

i. Dry-feed type

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

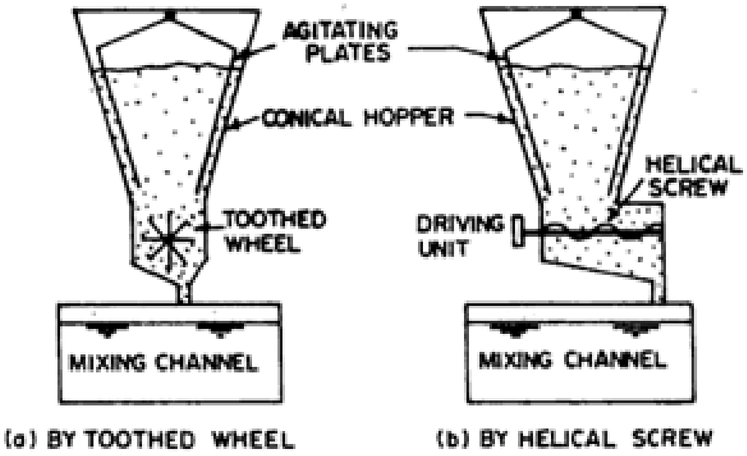


Figure: Dry feeding devices

II. Wet feeding type

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

Mixing devices

- The process of floc formation greatly depends upon the effective mixing (rapid mixing) of coagulant with the raw water.
- Rapid mixing of the mixture of coagulant and raw water is used to:
    - Disperse chemicals uniformly throughout the mixing basin
    - Allow adequate contact between the coagulant and particles
    - Formation of microflocs
- The mixing is done by mixing device.

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

4. Mixing channels

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

5. Mixing basin with baffle wall

6. Mechanical mixing basins

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

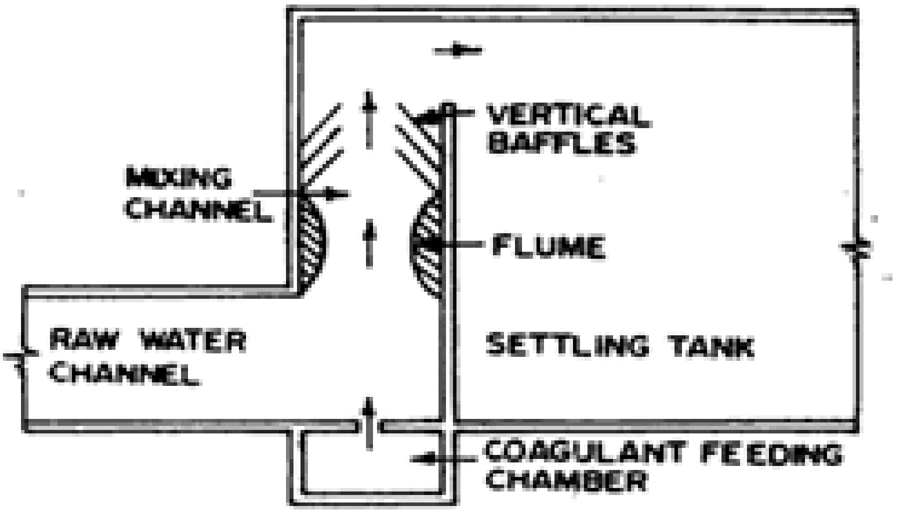


Figure: Mixing channel

A. Flash mixer

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

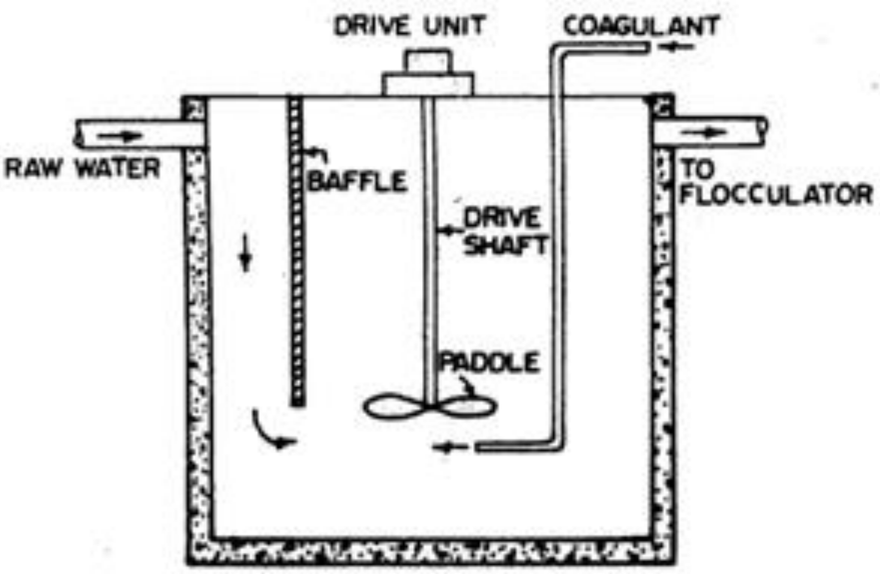


Figure: Flash mixer

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

B. Deflector plate mixer

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

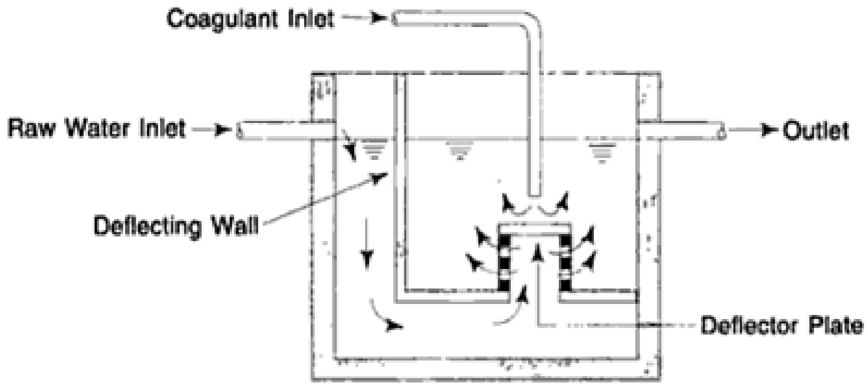


Figure: Deflector plate mixer

Flocculation

- After adding the coagulant to the raw water, rapid agitation is developed in the mixture to obtain a thorough mixing.
- Next to rapid mixing, mixture is kept slowly agitated for about 30 to 60min.
- Slow mixing process in which particles are brought into contact in order to promote their agglomeration is called flocculation.
- The tank or basin in which flocculation process is carried out is called flocculation chamber.
- The velocity of flow in the chamber is kept between 12 – 18cm/sec.
- Activated carbon in powder form can be used to speed up the flocculation.
- The rate of agglomeration or flocculation is dependent upon
    - Type and concentration of turbidity
    - Type of coagulant and its dose
    - Temporal mean velocity gradient – G in the basin
- The mean velocity gradient is the rate of change of velocity per unit distance normal to the section - (meter per second per meter) (T^{-1}).
- The value of G can be computed in terms of power input by the following equation

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

Where

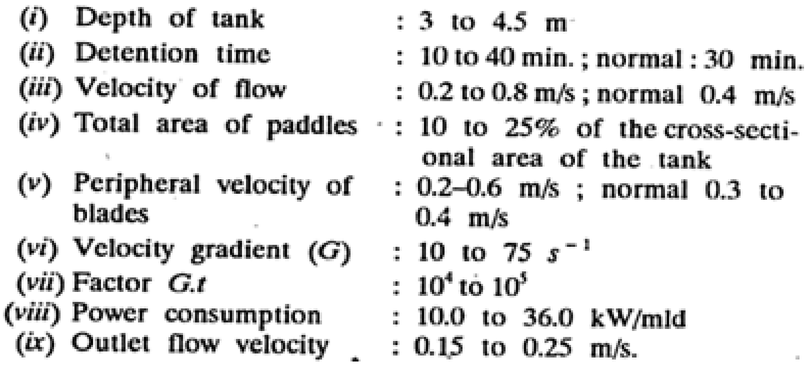
P – Power dissipated (watt)

μ - absolute viscosity (Ns/m^2)

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

G - Temporal mean velocity gradient (s^{-1})

- The flocculation technique most commonly used involves mechanical agitation with rotating paddle wheels or vertical mounted turbines (fig.).
- The design criteria of a horizontal continuous flow rectangular basin flocculator:



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

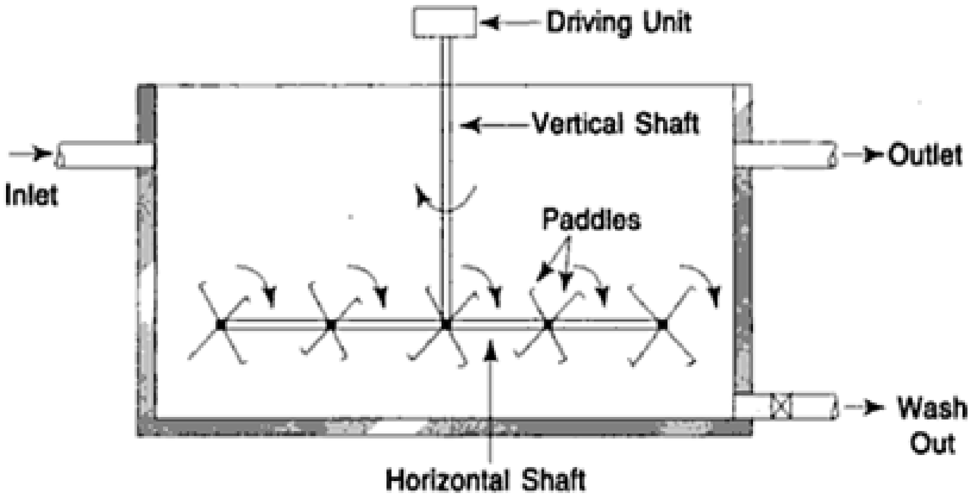


Figure: Flocculator

Clarifier (Secondary Sedimentation)

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

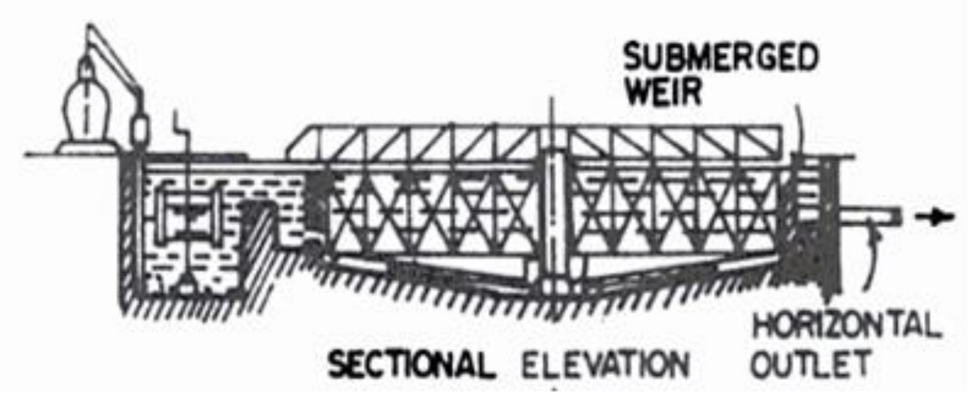


Figure: Section of secondary sedimentation unit

Flocculent settling

- Flocculent particles resulting from coagulation will agglomerate while settling with a resultant increase in particle size.
- The density of the composite particle will decrease due inclusion of water, however, the settling velocity will increase. (0.1 to 3mm best floc size)
- The clarification of dilute suspensions of flocculating particle is a function of:
    - Settling property of the particles
    - Flocculating characteristic of the suspension

Example1

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

Detention time (t): 20 – 60 s

Ratio of tank height (H) to diameter (D): (1:1 to 1:3)

Ratio of impeller diameter (DI) to tank diameter (D): (0.2:1 to 0.4:1)

Velocity gradient (G): >300 /s

Gt: 10000 – 20000

Tank diameter (D): < 3 m

Paddle tip speed (vp): 1.75 – 2.0 m/s

Velocity of paddle relative to water (v): 0.75 x paddle tip speed

Paddle area (Ap)/Tank section area (AT): 10:100 – 20:100

Coefficient of drag on impeller blade (CD): 1.8

Maximum length of each impeller blade (L): 0.25 x impeller diameter

Maximum width of impeller blade (B): 0.20 x impeller diameter

Impeller height from bottom (HB): 1.0 x impeller diameter

Kinematic viscosity (ν): 1.003 x 10-6 m2/s

Dynamic viscosity of water (μ): 1.002 x 10-3 N.s/m2

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

Solution:

Let the detention time (t) be 40 s.

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

Let the tank diameter (D) be 2 m

Tank cross-sectional area (Acs) = 3.14 m2

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

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

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

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

Let velocity gradient, G = 400 s-1

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

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

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

$G = \left\{\frac{C_d . A_p . v^3}{2 . \nu . V}\right\}^{1/2}$, or, $400 = \left\{\frac{1.8 . (A_p) . (1.35)^3}{2 . (1.003 \times 10^{-6}) . (4.63)}\right\}^{1/2}$

or, Ap = 0.335 m2

Tank sectional area = (D).(H) = (1.47).(2) = 2.94 m2

$\frac{\text{Paddle Area } (A_p)}{\text{Tank Sectional Area } (D.L)} = \frac{0.335}{2.94} = 0.1139$, which is around 0.15, hence okay.

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

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

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

Area of each blade = (L.B) = (0.2).(0.15) = 0.03 m2

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

Clearance of the paddles from the tank bottom = 0.8 m

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

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

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

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

Exercise:

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

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

Detention time (t): 10 – 30 minutes

Velocity gradient (G): 20 – 75 /s

Gt: 2 x 104 – 6 x 104

Tank Depth (D): < 5 m

Paddle tip speed (vp): 0.25 – 0.75 m/s

Velocity of paddle relative to water (v): 0.75 x paddle tip speed

Paddle area (Ap)/Tank section area (AT): 10:100 to 20:100

Coefficient of drag on paddle blade (CD): 1.8

Maximum length of each paddle (l): 5.0 m

Maximum width of each paddle (b): 0.50 m

Kinematic viscosity : 1.003 x 10-6 m2/s

Dynamic viscosity of water : 1.002 x 10-3 N.s/m2

Freeboard: 0.50 m

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

Chapter 5. Filtration

5.1.Theory of Filtration

- The effluent obtained after coagulation does not satisfy the drinking water standard and is not safe.
- So it requires further treatments.
- Filtration is one of the water purification process in which water is allowed to pass through a porous medium to remove remaining flocs or suspended solids from the previous treatment processes.
- Filtration process assist significantly by reducing the load on the disinfections process, increasing disinfection efficiency.
- Filtration consists of passing water through a thick layer of sand.
- During the passage of water through sand, the following effects take place.

i) Suspended matter and colloidal matter are removed

ii) Chemical characteristic of water get changed

iii) Number of bacteria considerably reduced.

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

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

II. Sedimentation and adsorption

- The interstices between the sand grains act as sedimentation basins in which the suspended particles smaller than the voids in the filter-bed settle upon the sides of the sand grains.

The particles stick on the grains because of the physical attraction between the two particles of matter and the presence of the gelatinous coating formed on the sand grains by the previously deposited bacteria and colloidal matter.

iii. Electrolytic action

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

iv. Biological action

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

Types of filters

Two types of filter:

1. Gravity filter system
    i. Slow Sand Filter (SSF)
    ii. Rapid Sand Filter (RSF)
2. Pressure filter system

Slow Sand Filters

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

- The sand used for the filtration is specified by the effective size and uniformity coefficient.
- The effective size, D_{10}, which is the sieve in millimeters that permits 10% sand by weight to pass.
- The uniformity coefficient is calculated by the ratio of D_{60} and D_{10}.

Construction

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

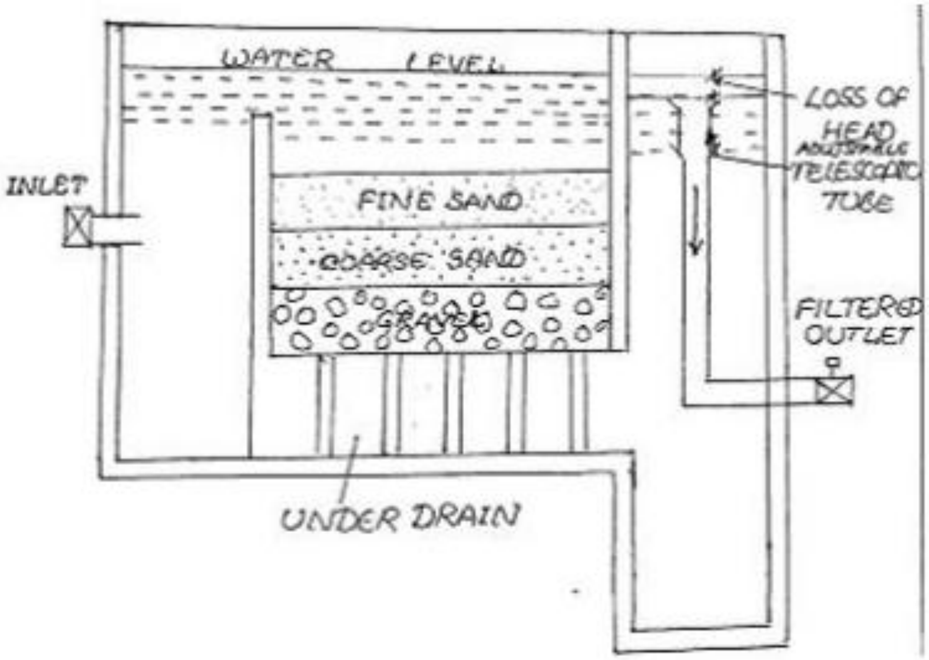


Figure: Slow sand filter

Operation

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

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

Uses

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

Maintenance

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

Rapid Sand Filter

- The rapid sand filter differs from the slow sand filter in a variety of ways, the most important of which are the much greater filtration rate ranging from 100 to 150m^3/m^2/day, the ability to clean automatically using backwashing and require small filter area.

- The mechanism of particle removal also differs in the two types of filters - rapid sand filters do not use biological filtration and depend primarily on adsorption and some straining.

The main features of rapid sand filter are as follows

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

Operation

- The water from coagulation sedimentation tank enters the filter unit through inlet pipe and uniformly distributed on the whole sand bed.
- Water after passing through the sand bed is collected through the under drainage system in the filtered water well.

- The outlet chamber in this filter is also equipped with filter rate controller. In the beginning the loss of head is very small.
- But as the bed gets clogged, the loss of head increases and the rate of filtration become very low.
- Therefore the filter bed requires its washing.

Washing of Filter

- Washing of filter is done by the back flow of water through the sand bed.
- First the value ‘’V1‟ is closed and the water is drained out from the filter leaving a few centimeter depth of water on the top of sand bed.
- Keeping all values closed the compressed air is passed through the separate pipe system for 2-3 minutes, which agitates the sand bed and stirrer it well causing the loosening of dirt, clay etc. inside the sand bed.
- Now value ‘’V4‟ and ‘’V5‟ are opened gradually, the wash water tank, rises through the laterals, the strainers gravel and sand bed.
- Due to back flow of water the sand expands and all the impurities are carried away with the wash water to the drains through the channels, which are kept for this purpose.

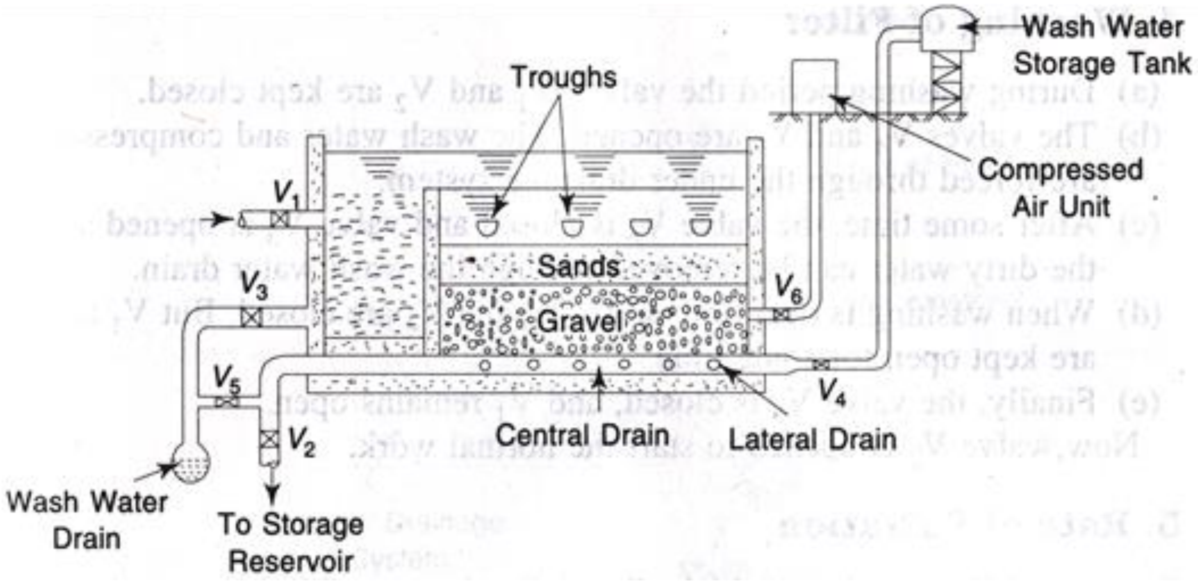


Figure: Rapid sand filter

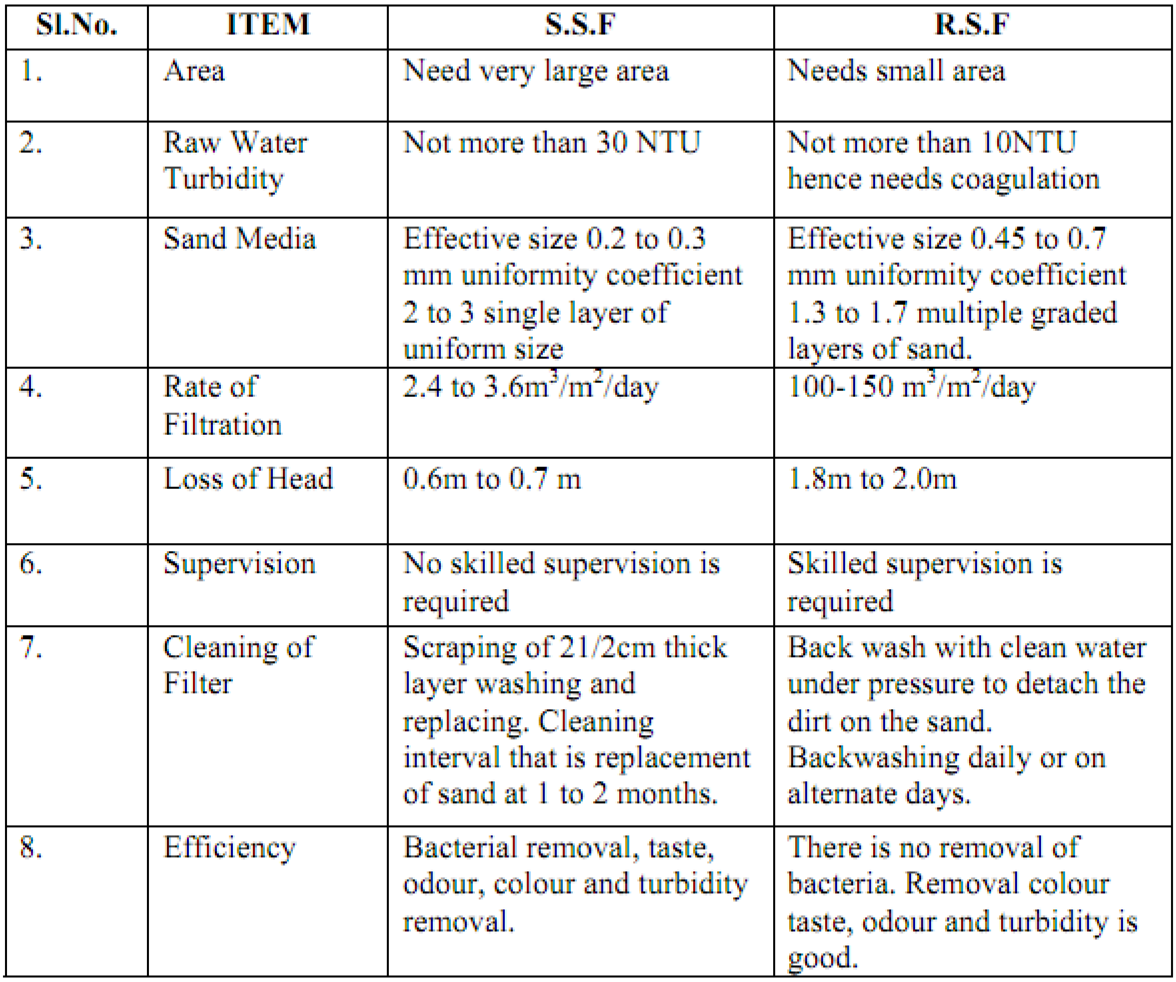
Construction

Rapid sand filter consists of the following five parts:

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

- Washing process is continued till the sand bed appears clearly.
- The washing of filter is done generally after 24 - 48hours and it takes 10 minutes and during back washing the sand bed expands by about 50%.
- Rapid sand filter bring down the turbidity of water to 1 N.T.U.
- This filter needs constant and skilled supervision to maintain the filter gauge, expansion gauge and rate of flow controller and periodical backwash.

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



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

Pressure Filter

- Pressure filter is type of rapid sand filter in closed water tight cylinder through which the water passes through the sand bed under pressure.
- All the operation of the filter is similar to rapid gravity filter; expect that the coagulated water is directly applied to the filter without mixing and flocculation.
- These filters are used for industrial plants but these are not economical on large scale.
- Pressure filters may be vertical pressure filter and horizontal pressure filter.
- Backwash is carried by reversing the flow with values.
- The rate of flow is 120 to 300m^3/m^2/day.

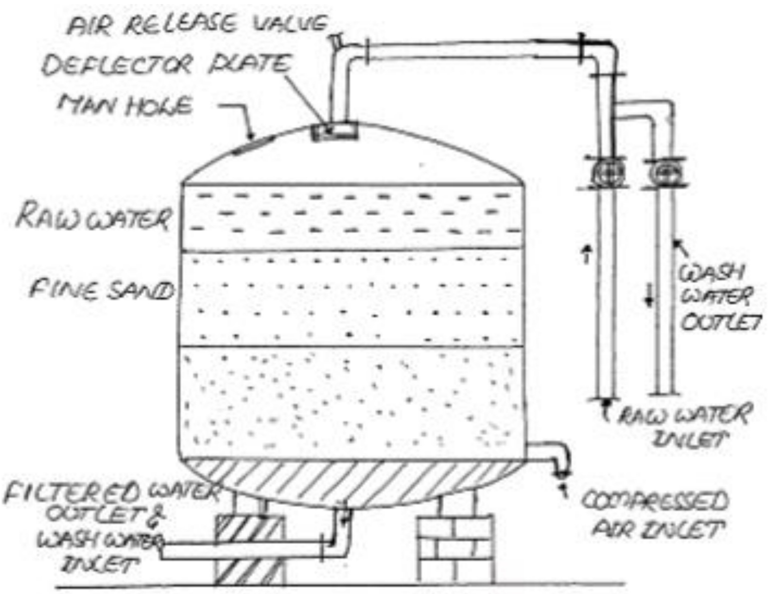


Figure: Vertical Pressure filters

Example1

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

Solution:

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

Filter is off-line for 1.5 hours every 24 hours

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

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

Let two filters be provided for this purpose

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

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

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

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

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

Example2

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

Solution:

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

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

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

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

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

Assume depth of sand = 50 to 75 cm.

<u>Under drainage system:</u>

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

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

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

Area of manifold = 2 (Area of laterals)

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

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

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

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

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

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

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

Number of holes per lateral = 650/50 = 13

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

Spacing of holes = 2.545/13 = 19.5 cm.

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

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

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

Rising washwater velocity in bed = 50 cm/min.

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

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

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

Washwater gutter

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

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

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

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

Assume b =0.3 m

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

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

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

Clear water reservoir for backwashing

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

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

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

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

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

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

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

Exercise

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

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

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

Flow, Q_f = 300 L/s
Rate of filtration, V_{of} = 170m^3/m^2/day
Time of washing = 10 min
Rate of washing, V_{oBW} = 15 mm/s

Chapter 6

6.1. Disinfection

- Before water treatment became common, waterborne diseases could spread quickly through a population, killing or harming hundreds of people.

- The table below shows some common, water-transmitted diseases as well as the organisms (pathogens) which cause each disease.

Pathogen	**Disease Caused**
Bacteria:	
Anthrax	anthrax
Escherichia coli	*E. coli* infection
Myobacterium tuberculosis	tuberculosis
Salmonella	salmonellosis, paratyphoid
Vibrio cholerae	cholera
Viruses:	
Hepatitis Virus	Hepatitis A
Polio Virus	polio
Parasites:	
Cryptosporidium	cryptosporidiosis
Giardia lamblia	giardiasis

- The primary goal of water treatment is to ensure that the water is safe to drink and does not contain any disease-causing microorganisms.

- The best way to ensure pathogen-free drinking water is to make sure that the pathogens never enter the water in the first place.

- However, this may be a difficult matter in a surface water supply which is fed by a large watershed.
- Most treatments plants choose to remove or kill pathogens in water rather than to ensure that the entire watershed is free of pathogens.
- Pathogens can be removed from water through physical or chemical processes.
- Storage can also kill a portion of the disease-causing bacteria in water.
- Disinfection is different from sterilization, which is the complete destruction of all organisms found in water and which is usually expensive and unnecessary.
- Disinfection is a required part of the water treatment process while sterilization is not.

Location in the treatment process

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

Requirements of good disinfectant

1) Destroy bacteria/pathogens within a practicable period of time, over an expected range of water temperature.
2) Effective at variable compositions, concentration and conditions of water treated.
3) Neither toxic to humans and domestic animals nor unpalatable or otherwise objectionable in required concentration.
4) Not change water properties

5) Have residual in a sufficient concentration to provide protection against recontamination
6) Can be determined easily, quickly, and preferably automatically.
7) Dispensable at reasonable cost
8) Safe and easy to store, transport, handle and supply
9) Not form toxic by-products due to their reactions with any naturally occurring materials in water.

Methods of Disinfection

- The disinfection of water can be done by one of the following methods:

a) Boiling of water
b) Ultra–Violate rays
c) Iodine and bromine
d) Ozone O_3
e) Excess lime
f) Potassium permanganate [$KMnO_4$]
g) Chlorine

In most situations chlorine is selected as one of the disinfecting agent that most closely satisfies the requirement of chemical disinfectants namely:

(i) Quick and effective at killing micro-organisms
(ii) Readily soluble at the concentration needed for disinfection
(iii) Tasteless and odourless at the concentration required
(iv) Non-toxic to human life at the concentration required
(v) Easy to handle, transport and apply
(vi) Easy to detect and concentration easy to measure
(vii) Capable of providing protection against later contamination.

(viii) Readily available

(ix) Cheap

The most common method of disinfection is the use of chlorine i.e. chlorination. The various chlorine compounds which are available in the market and used as disinfectants are:

1. Calcium hypo chlorite [$Ca(OCl)_2$] – powder form
2. Sodium hypo chlorite [NaOCl] –liquid form
3. Free chlorine Cl_2- Gaseous form

Chlorination

- Chlorination is the application of chlorine to water to accomplish some definite purpose.
- Chlorination is currently the most frequently used form of disinfection in the water treatment field.
- However, other disinfection processes have been developed.

Chlorination chemistry

- When chlorine is added to water, a variety of chemical processes take place.
- The chlorine reacts with compounds in the water and with the water itself.
- Some of the results of these reactions (known as the chlorine residual) are able to kill microorganisms in the water.

Chlorine Demand

- When chlorine enters water, it immediately begins to react with compounds found in the water.
- The chlorine will react with organic compounds and form trihalomethanes.
- It will also react with reducing agents such as hydrogen sulfide, ferrous ions, manganous ions, and nitrite ions.

- Let's consider one example, in which chlorine reacts with hydrogen sulfide in water.
- Two different reactions can occur:
- Hydrogen Sulfide + Chlorine + Oxygen Ion Elemental Sulfur + Water + Chloride Ions

$$H_2S + Cl_2 + O^{2-} \rightarrow S + H_2O + 2Cl^-$$

- Hydrogen Sulfide + Chlorine + Water Sulfuric Acid + Hydrochloric Acid

$$H_2S + 4Cl_2 + 4H_2O \rightarrow H_2SO_4 + 8HCl$$

- In the first reaction, hydrogen sulfide reacts with chlorine and oxygen to create elemental sulfur, water, and chloride ions.
- The elemental sulfur precipitates out of the water and can cause odor problems.
- In the second reaction, hydrogen sulfide reacts with chlorine and water to create sulfuric acid and hydrochloric acid.
- Each of these reactions uses up the chlorine in the water, producing chloride ions or hydrochloric acid which have no disinfecting properties.
- The total amount of chlorine which is used up in reactions with compounds in the water is known as the chlorine demand.
- A sufficient quantity of chlorine must be added to the water so that, after the chlorine demand is met, there is still some chlorine left to kill microorganisms in the water.

Reactions of chlorine gas with water

- At the same time that chlorine is being used up by compounds in the water, some of the chlorine reacts with the water itself.
- The reaction depends on the type of chlorine added to the water as well as on the pH of the water itself.
- Chlorine gas is compressed into a liquid and stored in metal cylinders.
- The gas is difficult to handle since it is toxic, heavy, corrosive, and an irritant.
- At high concentrations, chlorine gas can even be fatal.

- When chlorine gas enters the water, the following reaction occurs:
- Chlorine + Water $\longrightarrow$ Hypochlorous Acid + Hydrochloric Acid

$$\mathbf{Cl_2 + H_2O \longrightarrow HOCl + HCl}$$

- The chlorine reacts with water and breaks down into **hypochlorous acid** and hydrochloric acid.
- Hypochlorous acid may further break down, depending on pH:
- Hypochlorous Acid ↔ Hydrogen Ion + Hypochlorite Ion

$$\mathbf{HOCl \leftrightarrow H^+ + OCl^-}$$

- The concentration of hypochlorous acid and hypochlorite ions in chlorinated water will depend on the water's pH.
- A higher pH facilitates the formation of more hypochlorite ions and results in less hypochlorous acid in the water.
- This is an important reaction to understand because hypochlorous acid is the most effective form of free chlorine residual, meaning that it is chlorine available to kill microorganisms in the water.
- Hypochlorite ions are much less efficient disinfectants.
- So disinfection is more efficient at a low pH (with large quantities of hypochlorous acid in the water) than at a high pH (with large quantities of hypochlorite ions in the water.)

Hypochlorites

- Instead of using chlorine gas, some plants apply chlorine to water as a hypochlorite, also known as a bleach.
- Hypochlorites are less pure than chlorine gas, which means that they are also less dangerous.
- However, they have the major disadvantage that they decompose in strength over time while in storage.

- Temperature, light, and physical energy can all break down hypochlorites before they are able to react with pathogens in water.

- There are three types of hypochlorites - sodium hypochlorite, calcium hypochlorite, and commercial bleach: Hypochlorites and bleaches work in the same general manner as chlorine gas.

- They react with water and form the disinfectant hypochlorous acid.

- The reactions of sodium hypochlorite and calcium hypochlorite with water are shown below:

- Calcium hypochlorite + Water ⟶ Hypochlorous Acid + Calcium Hydroxide

$$\mathbf{Ca(OCl)_2 + 2H_2O \longrightarrow 2HOCl + Ca(OH)_2}$$

- Sodium hypochlorite + Water ⟶ Hypochlorous Acid + Sodium Hydroxide

$$\mathbf{NaOCl + H_2O \longrightarrow HOCl + NaOH}$$

Chloramines

- Some plants use chloramines rather than hypochlorous acid to disinfect the water.

- To produce chloramines, first chlorine gas or hypochlorite is added to the water to produce hypochlorous acid.

- Then ammonia is added to the water to react with the hypochlorous acid and produce a chloramine.

- Three types of chloramines can be formed in water - monochloramine, dichloramine, and trichloramine.

- Monochloramine is formed from the reaction of hypochlorous acid with ammonia:

- Ammonia + Hypochlorous Acid ⟶ Monochloramine + Water

$$\mathbf{NH_3 + HOCl \longrightarrow NH_2Cl + H_2O}$$

- Monochloramine may then react with more hypochlorous acid to form a dichloramine:

- Monochloramine + Hypochlorous Acid ⟶ Dichloramine + Water

$$\mathbf{NH_2Cl + HOCl \longrightarrow NHCl_2 + H_2O}$$

- Finally, the dichloramine may react with hypochlorous acid to form a trichloramine:
- Dichloramine + Hypochlorous Acid ⟶ Trichloramine + Water

$$\mathbf{NHCl_2 + HOCl \longrightarrow NCl_3 + H_2O}$$

- The number of these reactions which will take place in any given situation depends on the pH of the water.
- In most cases, both monochloramines and dichloramines are formed.
- Monochloramines and dichloramines can both be used as a disinfecting agent, called combined chlorine residual because the chlorine is combined with nitrogen.
- This is in contrast to the free chlorine residual of hypochlorous acid which is used in other types of chlorination.
- Chloramines are weaker than chlorine, but are more stable, so they are often used as the disinfectant in the distribution lines of water treatment systems.
- Despite their stability, chloramines can be broken down by bacteria, heat, and light.
- Chloramines are effective at killing bacteria and will also kill some protozoans, but they are very ineffective at killing viruses.

Dosage of Chlorine

- The amount of chlorine needed to disinfect water will vary from source to source. Waters should be clear, and free from organic matters and suspended solids.
- When chlorine is added to water some is used immediately to oxidize any organic matter and to kill bacteria in the water.
- Any that is not used straight away remains in solution in water as residual chlorine and protects against any possible future contamination of the water.
- There is no purpose in adding to little chlorine to a water, because the chlorine will be used up in oxidation of the organic material, and there may be insufficient chlorine to kill bacteria present.

- Disinfection is performed by any chlorine that remains after organic matter has been oxidized, and therefore is carried out by the last, and not the first, few parts per million of chlorine in the water.
- A sufficient quantity of chlorine should be added to leave an adequate chlorine residual in the water.
- The residual chlorine level should not, however, be so great as to cause consumers to complain about the taste of chlorine in the water.
- Applied chlorine dosage rates are usually in the range of 0.5 to 2.0mg/l, depending up on the initial quality of the water. a suitable contact time following addition of chorine should be provided to allow the water to be fully disinfected before it reaches the consumers.
- Contact times are between 10 and 30 minutes, with a time of 30 minutes being recommended.
- After this contact time the water should have a chlorine residual of 0.2 to 0.5mg/l.
- Higher chlorine dosage may be necessary for disinfecting new water mains, new wells or reservoirs, or other items that may have been contaminated.
- Samples of water should be taken immediately after disinfection to ensure that no coliforms are present.
- New water mains need to be disinfected prior to commissioning, and older mains should be disinfected after repairs or after any major cleaning program if they are found to be contaminated.
- First of all, the pipe lines should be flushed out with clean water and a foam swab passed through the pipe to remove slime, dirt and deposits.
- Pipe lines may then be disinfected by filling them with water containing chlorine at a concentration of 20mg/l (50mg/l is frequently used) and leaving this in the pipe line for not less than 24 hours for draining the chlorine solution away.
- Wells, reservoirs and storage tanks should also be disinfected before being put in to service following construction, repairs, cleaning or maintenance.

- The walls should brushed down as thoroughly as possible using a strong chlorine solution containing between 50 and 100mg/l of chlorine.
- Prior to this, reservoirs and storage tanks may be washed down using high pressure water jets.
- Following cleaning the well, reservoir or tank should be filled with water containing at least 20mg/l of chlorine and left to stand for not less than 24 hours before the contents are drained away to waste.
- The tank should then be refilled with fresh water to remove any chlorine that may create undesirable taste in water.

(A) Plain Chlorination

- Plain chlorination is the process of addition of chlorine only when the surface water with no other treatment is required.
- The water of lakes and springs is pure and can be used after plain chlorination.
- A rate of 0.8mg/lit/hour at 15N/cm^2 pressure is the normal dosage so as to maintain in residual chlorine of 0.2 mg/lit.

(B) Super Chlorination

- Super chlorination is defined as administration of a dose considerably in excess of that necessary for the adequate bacterial purification of water.
- About 10 to 15 mg/lit is applied with a contact time of 10 to 30 minutes under the circumstances such as during epidemic breakout water is to be dechlorinated before supply to the distribution system.

(C) Brake Point Chlorination

- The graph below shows what happens when chlorine (either chlorine gas or a hypochlorite) is added to water.
- First (between points 1 and 2), the water reacts with reducing compounds in the water, such as hydrogen sulfide.

- These compounds use up the chlorine, producing no chlorine residual.

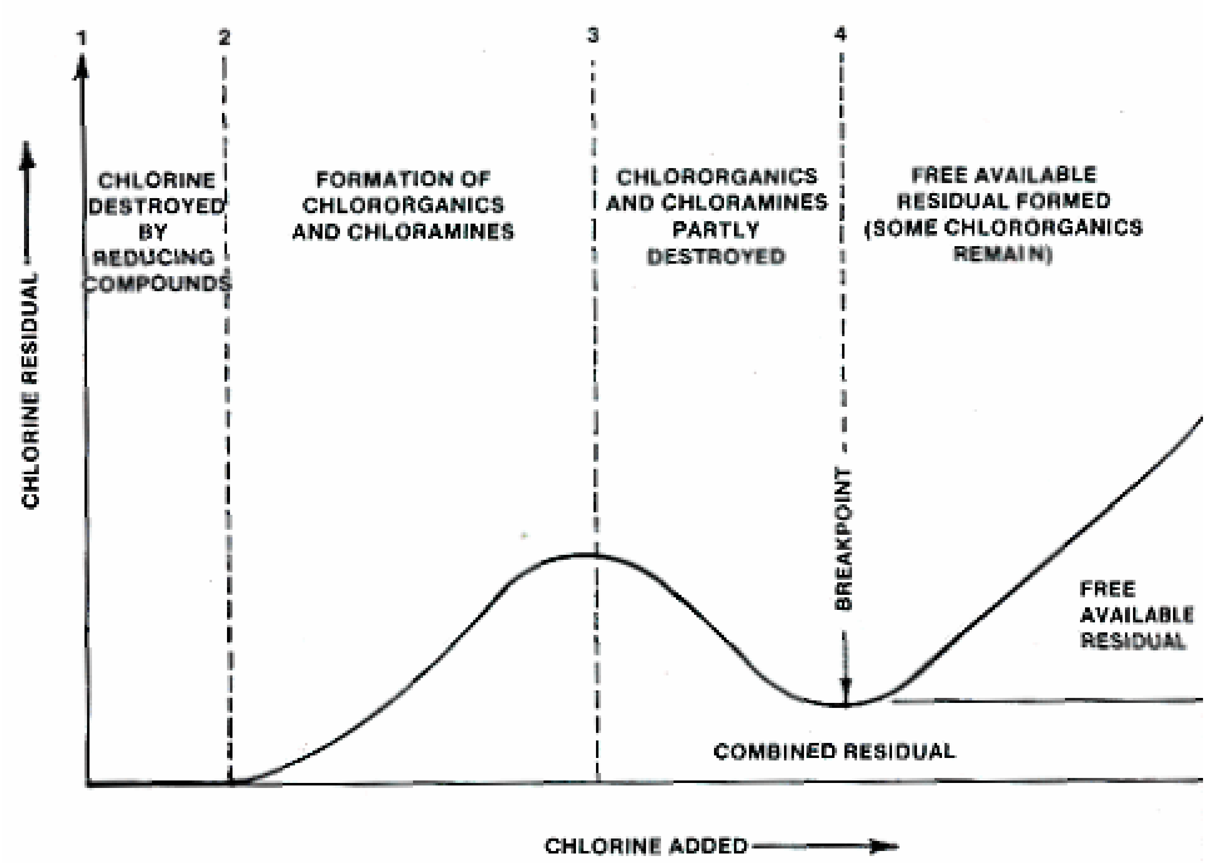


Figure: Break point chlorination

- Next, between points 2 and 3, the chlorine reacts with organics and ammonia which is naturally found in the water.
- Some combined chlorine residual is formed - chloramines.
- Note that if chloramines were to be used as the disinfecting agent, more ammonia would be added to the water to react with the chlorine.
- The process would be stopped at point 3.
- In contrast, if hypochlorous acid is to be used as the chlorine residual, then chlorine will be added past point 3.
- Between points 3 and 4, the chlorine will break down most of the chloramines in the water, actually lowering the chlorine residual.
- Finally, the water reaches the breakpoint, shown at point 4.

- The breakpoint is the point at which the chlorine demand has been totally satisfied - the chlorine has reacted with all reducing agents, organics, and ammonia in the water.
- When more chlorine is added past the breakpoint, the chlorine reacts with water and forms hypochlorous acid in direct proportion to the amount of chlorine added.
- This process, known as breakpoint chlorination, is the most common form of chlorination, in which enough chlorine is added to the water to bring it past the breakpoint and to create some free chlorine residual.

(D) De-chlorination

- Removal of excess chlorine resulting from super chlorination in part or completely is called „Dechlorination".
- Excess chlorine in water gives pungent smell and corrodes the pipe lines.
- Hence excess chlorine is to be removed before supply. Physical methods like aeration, heating and absorption on charcoal may be adopted.
- Chemical methods like sulphur dioxide (SO_2), Sodium Bi-sulphate ($NaHSO_3$), Sodium Thiosulphate ($Na_2S_2O_8$) are used.

Points of Chlorination

- Chlorine applied at various stages of treatment and distribution accordingly they are known as pre, post and re-chlorination.

a) Pre-Chlorination

- Chlorine applied prior to the sedimentation and filtration process is known as Pre-chlorination.
- This is practiced when the water is heavily polluted and to remove taste, odour, colour and growth of algae on treatment units.
- Pre-chlorination improves coagulation and post chlorination dosage may be reduced.

- The residual chlorine is useful in several stages of the treatment process - aiding in coagulation, controlling algae problems in basins, reducing odor problems, and controlling mud ball formation.
- In addition, the chlorine has a much longer contact time when added at the beginning of the treatment process, so pre chlorination increases safety in disinfecting heavily contaminated water.

b) Post Chlorination

- Post chlorination is the application of chlorine after water has been treated but before the water reaches the distribution system.
- At this stage, chlorination is meant to kill pathogens and to provide a chlorine residual in the distribution system.
- Postchlorination is nearly always part of the treatment process, either used in combination with prechlorination or used as the sole disinfection process.

c) Re-Chlorination

- In long distribution systems, chlorine residual may fall tendering the water unsafe.
- Application of excess chlorine to compensate for this may lead to unpleasant smell to consumers at the points nearer to treatment point in such cases chlorine is applied again that is re-chlorinated at intermediate points generally at service reservoirs and booster pumping stations.

Example

Consider water from a polluted river having BOD5 = 5 mg/L, TKN = 1 mg/l (as N), and MPN: 106 organisms / mL. This water will be treated in a conventional water treatment plant and supplied for potable purposes. Compute the chlorine dose (in mg/L as Cl2) required per liter of this water (consider both pre and post-chlorination) such that after treatment BOD5, TKN, NH3N are negligible and MPN < 1organism/mL.

Assumptions:

- Assume that 1 mg/L (as Cl_2) chlorine is required to destroy 1 mg/L of BOD5.

- Assume TKN is completely converted to NH_3-N during pre-chlorination.
- The average time between post-chlorination and water consumption by the end users is 1 hour
- The product of disinfectant dose (C in mg/L) and the contact time (t in minutes) for 5 and 6 log-kills using free residual chlorine as disinfectant is 96 and 120 respectively.

Assume 2 log-kill of microorganism during water treatment up to just before the post chlorination step.

Solution

- Chlorine dose required during pre-chlorination for destruction of BOD5 = 5 mg/L as Cl_2
- All TKN in water is converted to NH_3-N during this process.
- Hence ammonia concentration in water before post-chlorination = 1 mg/L (as N)
- Breakpoint chlorination has to be performed to destroy ammonia in water.
- Relevant equation:

$$2.NH_3 + 3.HOCl \rightarrow N_2 + 3.H_2O + 3.HCl$$

- Ammonia concentration in water = $\frac{1}{14} = 0.0714$ mmoles/L
- Chlorine required for destruction of ammonia = 1.5.(0.0714) = 0.1071 mmoles/L,
- Therefore, breakpoint chlorination dose = 71.(0.1071) = 7.6 mg/L
- Initial microorganism concentration = 106 /mL
- Removal during water treatment up to post-chlorination = 2 Log
- Hence microorganism concentration just before post chlorination = 104 /mL
- To get this concentration below 1 /mL, 5 log kills are required
- “C.t” for 5 log kills = 96

- Contact time = 1 hour = 60 minutes
- Therefore required free chlorine residual dose = $\frac{96}{60} = 1.6$ mg/L as Cl_2
- Therefore, total chlorine dose required = 5+7.6+1.6 = 14.2 mg/L as Cl

Chlorine Gas

- Chlorine gas is a yellowish green gas that is denser than air.
- It may be purchased in cylinders containing liquified gas at high pressure (containing about 30 Kg of liquid chlorine) or drums (containing about 860 kg of liquid chlorine).
- Chlorine gas may also be produced on site by passing an electric current through a solution of salt in water.
- Chlorine should never be injected in to water supply directly from cylinders or drums.
- Direct injection is a dangerous practice and it is not possible either to control or know with any accuracy how much chlorine is being applied.
- In all usual situations it is necessary to use chlorinating equipment that controls and measures the doses of chlorine applied.
- The chlorine dose is usually applied to the water to be disinfected by being sucked in to the water at a narrowing of the pipe, known as venturi constriction.
- The narrowing of the pipe causes the speed of the water to be increased and the pressure to be reduced, so allowing chlorine to be sucked in.
- Following the addition of chlorine, thorough mixing is necessary to ensure that the chlorine is uniformly distributed through the water.
- For small supplies salts of hpochlorous acid may be used to provide chlorination for swift disinfection of water, sodium hypochlorite solution known as Javel water contains about 5-16% available chlorine by weight.
- Bleaching powder (or chloride of lime) contains 20-35% available chlorine, and high test hypochlorite (HTH) contains 60-70%.

- The chlorine content of these various compounds and solutions made from them cam be expressed in various ways.
- By the percentage (by weight) of chlorine in parts per million (ppm), or milligrams of chlorine per litter. (Only for dilute solutions.)
- In order to convert from one unit to another, a 1% solution contains 10 grams of chlorine pet liter (1000 parts per million)
- Javel water, bleaching powder and high test hypochlorite are the most commonly used chlorine compounds, the choice of chemical being influenced by the reactive costs and availabilties, and by the ease with which they can be transported.
- In composition bleaching powder contains a mixture of calcium hypo chlorite, calcium chloride and calcium hydroxide.
- The costs of transport and storage are high because bleaching powder contains excess lime that serves no purpose in disinfection.
- Lime is also insoluble in water, and solutions made from bleaching powder should be allowed to stand after mixing and the solids allowed to sink.
- The solution on top should then be decanted in to a storage tank.
- Any lumps of cemented bleaching powder should then be broken up prior to mixing, and the bleaching powder added to water NOT the water added to then bleaching powder.
- the sediment, lime residue resulting from preparation of chlorine solution from bleaching powder is of no value to the disinfection process and should be thrown away,
- Using a safe disposal system. Failure to remove insoluble material results in the blocking of valves and pipes.
- High test hypochlorite consists of Calcium hypochlorite, and allows chlorine to be released slowly.

- It has certain advantages over bleaching powder, being more stable, less likely to deteriorate during storage even in tropical climates, and having good solubility so that relative clear solutions may be prepared.
- Storage and handling conditions should be as for bleaching powder.
- The stability of High Test Hypochlorite should not be overestimated, however.
- The high percentage of available chlorine and the good solubility in water results in low transport and storage costs than for bleaching powder.
- High test hypochlorite may be supplied in powder, tablet or granular form, and chlorine solution should be prepared using the same method as for bleaching powder.
- Javel water (Sodium Hypochlorite Solution) is convenient because it is supplied as a clear solution which is unlikely to cause blockage of dosing equipment.
- On the other hand because the solution contains a large proportion (by weight) of water, the handling and transport costs are high.

Storage of Chlorine Compounds

- Bleaching powder can be handled easily, but it is bulky and unstable, losing some of the available chlorine during storage.
- Loss of chlorine results from exposure to the atmosphere, moisture, light or heat.
- The powder, and solution prepared from it, should therefore be stored in cross - resistant containers in cool, dry, dark locations.
- Sealed drums of bleaching powder should be used within two years of productions, and once opened should be used within three weeks.
- High test hypochlorite is more stable than Bleaching powder, but the same storage requirements apply to both components.
- Javel water should also be stored in sealed containers and in dark conditions.
- Once containers of Javel water are opened their contents should be used within a week.

- The rate at which chlorine is lost from chlorine compounds during storage can not be estimated reliably.
- Before a concentrated solution should be made, and the strength of this solution measured.
- A solution should then be prepared of the strength and volume required for dosing purposes.
- For a sample of bleaching power containing 30% available chlorine, 33.3 grams of bleaching powder in 1 liter of water will produce a 1% chlorine solution (10 grams of chlorine per liter) which could be used to dose 10,000 liters (10 cubic meters) of water at a dosage of 1mg/l.
- For high test hypochlorite containing 65% available chlorine, only 15.3 grams of the sample would be needed to produce an equivalent solution.
- Unfortunately, in practice it is not usually possible to know what is the percentage of available chlorine in a sample of bleaching powder or high test hypochlorite prior to making a chlorine solution.
- The following tables indicate the quantities of various chlorine compounds that should be used to produce chlorine solutions and to disinfect water supplies.

Table: Weights in grams of chlorine compounds needed to produce 1% chlorine solutions for dosing purposes

Chlorine content in the compound	**5%**	**10%**	**15%**	**20%**	**30%**	**50%**	**60%**	**70%**
To make 1 liter	200	100	67	50	33	20	17	14
To make 5 liter	1000	500	333	250	167	100	83	71

To make 10 liter	2000	1000	667	500	333	200	167	143
To make 20 liter	4000	2000	1333	1000	667	400	333	286
To make 50 liter	10000	5000	3333	2500	1667	1000	833	714

- To make X liters of chlorine solution of Y% strength from a compound containing Z% chlorine by weight. The amount of compound to be used is:

X x Y x 10 x 100 b grams of chlorine compound

Z

Table: Volumes (litre) of 1% chlorine solution required for disinfection of water supplies.

Chlorine dosage (mg/l)	**0.5**	**1.0**	**1.5**	**2.0**
Volume of water to be Chlorinated (cubic m.)				
5	0.5	0.25	0.75	1.0
10	0.50	1.0	1.5	2.0
25	1.25	2.5	3.75	5.0
50	2.5	5.0	7.5	10.0
100	5.0	10.0	15.0	20.0
250	12.5	25.0	37.5	50.0

500	25.0	50.0	75.0	100.0

- The volume of A% chlorine solution needed to chlorinate B cubic meters of water at a dosage rate of c mg/l is:

$$\frac{B \times C}{10 \times A} \text{ Liters}$$

Sample Constant Rate Systems for Dosing Chlorine Solution

- A variety of constant rate chlorine dosing systems have been designed, and the designs can often be copied or adapted to make use of locally available materials.
- Two of the more reliable devices are the constant head syphon and the floating bowl solution feeder.
- Despite its reliability, the outlets of these devices may become partially obstructed by sediment and scale. Especially at low flow rates. Chlorine solutions should be added to a flow of water, ensuring that the chlorine solution added can be adjusted so that the required level of residual chlorine is maintained after the necessary contact time.
- For the constant head syphon flow rates remain constant because the head causing flow remains steady.
- The driving head is the difference in level between the base of the air inlet tube (where pressure is atmospheric) and the siphon outlet.
- The syphon device is very reliable giving constant feed rates irrespective of the level of water in the reservoir.
- Construction, operation and maintenance is simple, although care should be taken to ensure that the reservoir is well sealed and that air cannot enter the siphon tube from the air inlet pipe.

- Flow control can be achieved by moving the air inlet tube, changing the height of the siphon with respect to the air inlet tube, or by use of a tap on the siphon outlet.
- The air inlet tube should not be obstructed, and flow control by use of a tap on the air inlet is unsatisfactory.
- The floating bowl solution feeder may be of two possible designs; either with the inlet above the outlet, or with the inlet below the outlet.
- Either design may be used, but the bowl may require some ballast (such as gravel) and a vertical guide wire should be provided to prevent the bowl from tilting over and to keep it floating in a controlled manner as the bowl descends during dosing.
- Flow control by altering the sizes of the tubes or by adjusting the levels of the tubes in the bowl.

Chlorination of Wells

- Water obtained from wells and boreholes is very difficult to disinfect with any reliability, and under-dosing or overdosing is almost inevitable.
- The volume of water contained in a well or borehole is seldom known accurately and rates of extraction will vary from day to day and during each day.
- Simple disinfection may be used to provide residual chlorine in waters of high initial quality, but the most reliable form of ensuring accurate dosing is to store the water obtained from a well or borehole in a reservoir, and to disinfect the water, whose volume and chlorine requirement can be measured, in the storage reservoir.
- The practice of adding chlorine solution directly to a well or borehole is not recommended, as the only factor that can be controlled is the residual relative to the original dose.
- The chlorine solution may not mix thoroughly with the water in the well, and any metal lings or fittings may become corroded.
- It has been reported that water in wells may be disinfected by the use of chlorination pots, containing a mixture of sand and either bleaching powder or HTH.

- The problems associated with this technique are not so widely reported.
- Chlorination pots are not widely used, although certain publications would lead one to believe otherwise.
- The operating principle for these chlorination pots assumes that water soaks into the chlorine compound used, forms a chlorine solution which then mixes with the remainder of the water in the well.
- In reality, the outer layer of the chlorine compound adsorbs water and then sets hard, like cement, forming a hard shell that effectively prevents water from reaching the interior of the mixture.
- Tests have shown that by adding a small quantity (about 5% by weight) of sodium Hexametaphosphate to the bleaching powder or HTH, the mixture remains soft, but the difficulty of controlling the dosing rate remains.
- A practical problem associated with chlorination pots is related to patterns of water use.
- During the night, when little or no water is taken, the water in contact with the pot may develop such a high chlorine content that it becomes unacceptable for drinking purposes.
- Through the day, as water is extracted and stocks of water are replenished, the chlorine level decreases, reaching a minimum level by evening.

Chlorination Techniques

- Various chlorination techniques may be used.
- The type of chlorination used in a particular situation depends on the quality of the water to be disinfected, but for most situations chlorine is added following other water treatment processes (post - chlorination).
- In conclusion, various disinfection methods can be used in order to ensure that water is of good bacteriological quality.

- Chlorine and chlorine based chemicals are the most commonly used products, and over many years and in variety different conditions these have proved to be simple and reliable to use.
- Disinfection should not be considered to be a complete water treatment process in itself, but should be used in conjunction with other treatment operations to produce waters that are consistently safe and of good quality and appearance.

Chapter 7

7.1. Miscellaneous water treatment

Removal of Taste and Odor Problem

- The sense of odour is closely related to that of taste.
- In fact it is normally correct to suggest that most 'tastes' in water are really a sensation of smell.
- There are only four sensation of taste (1) sour, sweet, salty and bitter-all other sensations are of odour although not necessarily noticed until the water is in the mouth (2).
- Some tastes in water are unrelated to odour and these include the brackishness associated with relatively high concentrations of salts such as sodium chloride or magnesium sulphate.
- Iron and manganese in water often produce an astringent taste.
- Chalk derived waters are frequently described as possessing a 'sharpness' _ possibly due to CO_2 or bicarbonate _while surface waters deficient in dissolved oxygen are often 'flat' to the taste.
- Consumers identify and accept water as being pure as a result of its lack of colour, its clarity and its lack of taste and odour.
- Hence it is not only necessary that water is pure but that it also appears to be pure.
- Some substances, such as many inorganic salts do not create sensations of odour or taste until present in a concentration of several hundreds or even thousands of mg/1.
- Others such as commercial grade parathion (0.003 mg/1) and sodium fluoride (2, 4 mg/1) become noticeable at much lower concentrations.
- People also vary in their sensitivity to odour and taste and some people are perhaps as much as eleven times as sensitive as others.
- In addition many causes of odour and taste in water possess additive, synergistic or antagonistic effect, so that the mixing of two sources of water, each possessing only a limited odour can result in a combined water with a pronounced odour problem.

- Conversely, mixing odoriferous water can produce a non-offensive supply.

Causes of Odor and Taste

- Concentrations of inorganic salts (brackishness)
- Hydrogen sulphide
- Contact with painted surfaces
- Industrial discharges - pesticides, phenols etc
- Metabolites of actinomycete, algae etc
- Dead and decaying organic material including sewage and algae
- Chlorination

Some of these may be prevented by various means while others must be cured. Hydrogen sulphide is commonly found in ground waters from deep wells. Formation of H_2S on other water supplies should be prevented by maintaining the water always in an aerobic condition. Chlorination should be carefully controlled to prevent excessive odor and taste in the treated water.

Prevention of odours and tastes from microorganisms

- Bacteria - Some bacteria reduce sulphate to sulphide (H_2S) and these are controlled by aeration and chlorination.
- Actinomycete (mold-like bacteria) - grow frequently in mains as a slime on the inside of the pipes.
    - They may be controlled by treating reservoirs with copper sulphate or by ensuring there is residual chlorine throughout the mains, or by biological control employing Bacillus cereus to reduce the obnoxious metabolites produced.
- Iron bacteria- grow in water containing appreciable concentrations of iron and in which the DO is limited.

    - They produce objectionable odours and tastes as well as leading to the precipitation of iron. The remedies are removal of iron, chlorination or dosing with copper sulphate.

- Plankton (algae, protozoa)- these either produce taste or odour as a result of normal metabolic activity or as a result of death and decay.

    - Growth may be controlled by limiting nutrients, desertification of reservoir, mechanical removal of plants, and chemical control.

- In addition the levels of water containing the troublesome microorganisms may be avoided by varying the level of the water intake.

- Chemical control is nearly invariably by the addition of small quantities of copper sulphate.

Recommended method to remove test and odour problem

1. *Aeration*

- This is of limited value, particularly with the only slightly volatile odors resulting from the decomposition of vegetation.

- However, concentrations of odoriferous metabolites emanating from living microorganisms can be decreased, thus reducing the amount of activated carbon required at a later stage.

2. Chlorine

- Marginal chlorination is not sufficient to remove odoriferous compounds and also may add a noticeable smell of chlorine.

- In the presence of phenols a pronounced smell of chlorinated phenols will become apparent.

- Super chlorination with more than a breakpoint dosage will destroy most malodorous compounds.

- Ammonia/chlorine treatment can be useful in preventing chlorophenolic, odours.

3. Ozone

- Usually deals effectively with odors that are only accentuated by chlorine.
- Modest doses of 1 to 2 mg/1 will completely destroy phenols at any likely pH value.
- Ozone reduces H_2S odors and tastes and also the odors from decaying vegetable matter.
- Algae must be removed prior to the addition of ozone.
- Partial breakdown by the addition of zone of previously non-biodegradable organic material can result in the production of soluble substrate to encourage growth of obnoxious microorganisms in the distribution system.

4. Hydrogen peroxide

- This is occasionally used for odor control.
- It works by releasing 'nascent' oxygen but is generally too costly.

5. Chlorine Dioxide

- ClO_2 is particularly effective at removing phenolic odors from water.
- It does not react with ammonia and hence its treatment strength is not dissipated.
- It is most effective when mixed with an excess of chlorine.

6. Potassium Permanganate

- It is one of the most effective chemical reagents for odour taste control.
- It is normally applied as a solution and as such is easy to control.
- In the USA it is widely used and is most effective between pH8.0 and pH8.3.
- The benzene rings of both phenol and aniline are broken by permanganate.
- Although it is about three times as costly as activated Carbon it has been claimed to be up to five times as effective.

7. Activated Carbon

- Relies on its large surface area and physical adsorption for its effect and tends to remove larger, rather than smaller organic molecules and non-polar rather than polar compounds.
- It is used either as powdered activated carbon or as granular activated carbon.
- Powdered activated carbon is a fine black powder which is formed into slurry before use and applied either to the raw water, or the mixing basin, on the settling basin or directly before the filters.
- It is not recoverable.
- A normal dose might be 2.0 mg/1 to clarified water although it has been applied at rates up to 125 mg/1 to raw water.
- Granular activated carbon consists of relatively large grains and is held in filter towers through which the water is continually passed.
- Periodically the carbon beds are cleaned by back washing and when approaching exhaustion the carbon is regenerated in a furnace in carefully regulated oxidizing conditions at about 800^0c.
- At low levels of taste and odour powdered activated carbon is suggested as being more economic than the granular variety but the reverse is true once higher dosages are required.
- The pH of the water to which the carbon is added is of importance, as the adsorptive properties appear to be about four times as great at pH5 to pH6 than between pH9 and pH10.
- Chlorine is effectively removed by activated carbon treatment and hence post-chlorination is necessary.

Removal of Hydrogen sulphide

- The permissible concentration of H_2S in water, which are required for domestic purpose, is 0.5 mg/1 of H_2S and it is offensive when its concentration greater than 1.0 mg/1.

- The odours are less noticeable as the pH rises due to the formation of alkaline sulphides (s'' & Hs'), although these sulphides add appreciably to the chlorine demand of the water.
- Hydrogen sulphide is removed by aeration and chlorination.
- Aerators are normally of the natural draft cascade-type but aeration has only a limited effect and it is necessary to establish a cost balance between aeration and chlorination.
- Removal by chlorination is highly effective but as much as 10 kg of chlorine may be required to remove 1.0 kg of hydrogen sulphide.

Threshold odour Number

- The T.O.N is the number of times a sample must be diluted with odor-free water so that the odor is the least definitely perceptible.
- Since there is little consistency in sensitivity of at least five persons, and preferably of ten, is employed.
- The sample is progressively diluted with odor-free water which is normally obtained by passing tap-water through a column of activated carbon.
- Most tap-waters are suitable for this purpose so long as they do not contain unusual salt concentrations and do not vary appreciably from a normal pH.
- All glassware used must be clean and odor-free, having been washed with a non-odorous soap, rinsed with an acid cleaning solution and finally rinsed with odor-free water.
- The testers must be free from colds, be non-smoking, should not use cosmetics, perfumes or shaving lotions and should not have recently eaten a small.
- All extraneous odors such as from fresh paint must be excluded.

Removal of Iron and Manganese

- Before assuming that iron has come from the other source one should check that it is not coming from corroding steel well screens or pipes.

- Iron and manganese can be dissolved from sandy soils, laterite, shale, sandstone and other rocks by acid water which contains no dissolved oxygen. (such water may have- or have had- an organic content.)
- Decomposition consumes the oxygen and produces carbon dioxide, which is acidic.) the iron is usually in the form of ferrous bicarbonate and the manganese in the form of manganese oxide or bicarbonate.
- These forms are soluble.
- If the water is now exposed to air so that oxygen can enter into solution and carbon dioxide leaves, the ferrous from is oxidized to ferric, and the manganese to manganic.
- These latter forms are insoluble and so visible precipitates are formed.
- Therefore iron bearing water may be clear when pumped up from a well, but may, on standing, turn brown and turbid. (For this reason samples of groundwater should be analyzed before they have a chance to lose carbon dioxide and gain oxygen).
- The formation of a precipitate may take longer with manganese, or when the metals are combined in organic compounds.
- Sometimes the evidence of a manganese problem in a distribution system may take years to appear because the reactions are slow until catalyzed by deposits.
- Iron and manganese problems most commonly occur with groundwater, but may also appear in water drawn from the bottom of a stratified reservoir or in acid peat waters.
- Manganese may occur in significant quantities in river water, sometimes coming from acid mine drainage.

Causes of Iron and Manganese

These metals in potable water can cause

a) Colour in the water

b) Staining of plumbing fixtures and clothing (Fe -brown. Mn - black)

c) Astringent, metallic or medicinal taste in the water

d) Growth of iron bacteria (mainly crenothrix and Leptohrix) which feed on iron when the dissolved oxygen content is low and form slimes which clog well screens and pipes and water meters.

- They also produce tastes and odors in the water.
- Manganese bacteria may produce long black streamers of slime which emerge from consumers' taps.
- When the water contains sulphate, sulphate reducing bacteria may work together to produce black deposits of iron sulphate and H_2S in dead ends.

e) Brown stains on cooked vegetables and discoloured etc.

Method of Treatment

- Groundwater containing iron and/or manganese needs only treatment to remove these metals, and disinfection.
- Surface water containing iron or manganese will usually also require removal of the turbidity of clay colloids.
- Small quantities of iron can be removed by normal conventional treatment.
- If a higher pH is needed to remove iron or manganese then alum may not be suitable and ferric coagulants may be preferred if removal of turbidity and iron or manganese is effected simultaneously.
- In general the aim is to form insoluble compounds so that precipitates can be separated by sedimentation and filtration.
- The variety of forms in which the iron or manganese exist requires a variety of treatment processes, as summarized in the attached table.
- The only satisfactory method of choosing the most economical appropriate method is a program of pilot tests.
- Usually manganese is more difficult to remove than iron.

1. Aeration

- It is commonly used, but some processes require exclusion of air at the early stages.

- Aeration raises the pH by removing carbon dioxide, and adds oxygen.
- Only 0.14 mg/1 of dissolved oxygen is needed to oxidize 1 mg/1 of iron.

2. Sedimentation

- It is useful where large particles are formed in a short time, but for some waters it achieves nothing.

3. Filtration

- It is required in most processes as a means of capturing small suspended particles, but also often to promote oxidation by the catalytic effect of deposits on the sand grains.
- Because the deposits are useful the filter will take time to mature and should not be back washed too thoroughly.
- Coarser grain sizes may be used (compared with normal filters- between 0.7 and 1.0mm E.S) and higher filtration rates are possible (up to 10 m/hr or over 50 m/hr.)

4. Contact oxidation

- It provides a catalytic effect, speeding up the oxidation.
- This is often achieved using layers of coke or gravel on tray aerators, or by providing a bed up to 1.8 m deep with ventilation holes.
- Deposits on the grains assist oxidation.
- Basic rocks such as limestone or dolomite may be used to raise the pH.
- Iron or copper sulphate solutions catalyze oxidation of iron, and pyrolusite (a manganese or potassium permanganate are used for manganese).
- Hopper bottomed clarifiers have been used to oxidize iron and manganese in the sludge blanket, using potassium permanganate solution as a catalyst.
- One case when aeration at first must be prevented is process 5 in the table.

- Here soluble manganous hydroxide is formed first, and then this is oxidized to insoluble manganic hydroxide by aeration. (If aeration were used first the precipitate would be so fine that it would be very difficult to separate.)
- This is achieved by upward flow through a bed of pyrolusite before aeration.
- Contact beds can be cleaned by rapidly draining down or hosing.

5. Chlorination

- It is simple and cheap, but slow for manganese below pH 8.5 and in some cases may form very fine precipitates.
- Chlorination before addition of potassium permanganate may reduce the consumption of the latter.
- 1.25 mg/1 of free chlorine is needed for each mg/1 of iron or manganese.
- Zeolite Manganese ('greensand') is effective in removing manganese but it needs periodic regeneration.
- One cubic meter of manganese zeolite oxidizes 1.63 kg manganese and is regenerated by 3.26 kg potassium permanganate.

6. Lime

- Raising the pH will cause oxidation, but this technique is not suitable for hard waters because it will cause deposition of calcium carbonate on the filter sand.

7. Iron exchange

- When used for softening water will also remove soluble iron and manganese, it should therefore be done in the absence of air to prevent formation of the insoluble forms.

8. Sequestration

- Iron and manganese may be held in solution by dosing with twice the concentration of sodium hex meta phosphate (Calgon).
- This method is not suitable if the water is to be boiled.